STANDARDS NEW ZEALAND
PAEREWA AOTEAROA

NZS 3404:Parts 1 and 2:1997

Incorporating Amendment No. 1 and Amendment No. 2

Steel Structures Standard

Both Parts supersede NZS 3404:Parts 1 and 2:1992

NZS 3404:Parts 1 and 2:1997

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COMMITTEE REPRESENTATION

This Standard was prepared under the supervision of the Steel Structures Committee (P 3404) for the Standards Council established under the Standards Act 1988. The Committee consisted of:

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AMENDMENTS

No	*Date of issue*	*Description*	*Entered by, and date*
1	June 2001	Incorporates technical and editorial changes and includes items and references by way of clarification.	Incorporated in this reprint
2	October 2007	Aligns the Standard with the AS/NZS 1170 set and corrects an ambiguity in design action.	Incorporated in this reprint

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FOREWORD TO THE 1992 EDITION

The 1992 edition of this Standard was prepared by Standards New Zealand NZS 3404 Committee to supersede NZS 3404:1989, incorporating AS 1250:1981.

This Standard, broadly speaking, incorporates the provisions for bare steel member design from AS 4100:1990 *Steel Structures Standard* and the provisions for seismic and composite design from NZS 3404:Part 2:1989 *Steel Structures Code: Means of Compliance [With Part 1: New Zealand Amendments to AS 1250:1981]*.

This Standard is written in limit state format, for use with a limit state format Loadings Standard. While it is generally anticipated that the relevant Loadings Standard will be NZS 4203, other limit state format Loadings Standards may be used where indicated herein, provided the anticipated levels of member reliability against failure are maintained. The critical information on which the level of reliability for this Standard, when used in conjunction with NZS 4203, has been based, is presented in the Commentary to section 3.

The 1989 edition of this Standard was written in strength design format, however mostly by reference to working stress member design provisions with the factor of safety removed. In contrast, this edition is a formally calibrated limit state standard, with the appropriate functional states and corresponding performance limits presented in the format of design actions and corresponding design capacities.

The following brief outline gives an indication of the key changes between the 1989 and the 1992 editions, in addition to the change to limit state format already mentioned.

General application – where necessary, the requirements of this Standard have been broadened and modified to cover not only building structures but cranes and bridges.

The Standard is now suitable for bridge design utilizing steel or composite sections in conjunction with the Transit New Zealand Bridge Manual : Design and Evaluation (for road bridges) or the New Zealand Railnet Ltd : Railnet Code, Part 4, Code Supplements Bridges and Structures, Section 2 : Design (for railway bridges). Design engineers should note, however, that additional or more stringent provisions for some aspects of steel bridge design may still be required by the relevant authority. This Standard is not written for steel box girder bridge design, for which reference to an appropriate limit state Standard or design procedure is required.

Structural analysis – a new section (section 4) has been added, providing explicit guidance on the three methods of structural analysis covered by this Standard; namely elastic analysis, elastic analysis with redistribution or plastic analysis. It is envisaged that the first two methods will be the most commonly used. Redistribution may be applied to all structural systems. This section also provides explicit requirements for treatment of second-order effects; both P-Δ effects due to lateral movement of the structural system as a whole and P-δ effects on individual members, brought about by compression loading on these members.

The revised requirements for structural analysis have removed the inconsistency that arose in the 1989 edition of this Standard between treatment of an elastically stiff sway system and a braced system.

Other significant new inclusions or revisions

- new provisions for material selection to avoid brittle fracture have been added. These provisions offer a significant improvement in terms of design application to those contained in the 1989 edition of this Standard
- a section on design for fatigue has been added
- a section on design for fire has been added
- the requirements for fabrication and erection have been updated to reflect modern work practices and technology
- the section on seismic design has been substantially revised to improve its ease of application, expand its scope and remove some inconsistencies present in the 1989 edition of this Standard
- the section on composite design has been converted to limit state format and revised.

Major technical revisions – have been made in the design of members subject to bending, compression, tension or combined actions. These changes reflect recent advances in research into structural behaviour and computational methods of analysis. The basis for each technical provision is discussed in the Commentary together with selected references from the published technical literature.

Options for increasing efficiency of material use with increased designer effort – in a number of clauses throughout this Standard, there is more than one option available to designers to obtain the design answer. Where this applies, the trade-off between different design routes is greater material efficiency gained through increased designer effort. This applies especially to section 8 : Members Subject to Combined Actions, for sections with certain cross section properties.

This alternative option approach means that, in each section of this Standard, there is a design route that may be followed that will involve the designer in a similar amount of effort to that required from the corresponding section of the 1989 edition of this Standard and will result in a similar or more efficient use of material. In some sections there is also a more complex set of provisions that will allow still greater economy at the cost of more designer input.

To fully utilize the general provisions, designers should use published design load tables in the same manner as they have done with the 1989 edition of this Standard. To fully utilize the more complex sets of provisions and to efficiently undertake composite design, recourse to computer software is desirable (e.g. use of a spread-sheet program).

It must be stressed, however, that designers do not need computer software to effectively use this Standard.

Editorial changes – advantage has been taken of the current revision to rearrange the material contained herein to make it more readily useable by designers. This especially applies to the one-document format, rather than the two document AS 1250 and NZS 3404 format of the 1989 edition of the Standard. Further changes to improve useability have been made in the 1997 revision.

FOREWORD TO THE 1997 EDITION

NZS 3404:Parts 1 and 2:1992 have been in use now since January 1993. All sections of the Standard have been put through rigorous scrutiny and application since then, arising from use by practitioners and during the course of preparing 2 significant companion documents to the Standard (these being the Seismic Design Procedures for Steel Structures – HERA Report R4-76 and the Structural Steelwork Limit State Design Guides Volume 1 – HERA Report R4-80).

The process has shown up a number of technical changes that should be made, plus a few editorial changes to improve useability. These items form the first significant source of changes being recommended for this 1997 edition of NZS 3404.

The second significant source of amendments has been necessitated by the principal steel suppliers in New Zealand changing their grades of steel. BHP have replaced most of their Grade 250 steel sections with a new f_y = 300 MPa steel, while BHP New Zealand Steel have replaced their Grade 350 steel sections with a new f_y = 300 MPa grade steel. These changes have necessitated a considerable amount of research aimed at establishing the new grades seismic performance and subsequent changes to the seismic design provisions of the Standard.

The third and final significant source of amendments relates to the revision of the Concrete Structures Standard, NZS 3101, into limit state format. Rather than duplicate reinforcement design and detailing provisions for composite steel/concrete column design when writing the 1992 edition, where possible NZS 3404:1992 cross-referenced to the relevant provisions of the then current Concrete Structures Standard, NZS 3101:1982. Changes to the clause numbering, scope and content of the NZS 3101 provisions from the 1982 edition to the new (1995) Standard have necessitated changes to the provisions of NZS 3404 relating to the design of composite members and structures.

It was originally proposed to incorporate these changes via an amendment to the Standard. Because these 3 sources of change are largely independent of each other and involve different sections of the Standard, the result would have been an amendment of considerable length.

With support from the industry, the decision was made, in 1996, to prepare instead a first revision of the Standard. This presented the opportunity to widen the scope of editorial and technical changes, to the benefit of all users of this Standard.

The commentary (Part 2) has been updated in line with the changes to the Standard (Part 1).

Amendment No. 2 – October 2007

This amendment brings NZS 3404:Parts 1 and 2 into line with AS/NZS 1170 *Structural design actions*, and NZS 1170.5:2004 *Structural design actions – Earthquake actions*.

NEW ZEALAND STANDARD

STEEL STRUCTURES STANDARD

1 SCOPE AND GENERAL

1.1 SCOPE

1.1.1
This Standard sets out minimum requirements for the design, fabrication, erection, and modification of steelwork in structures in accordance with the limit state design method (sections 1-17, Appendices A-N) or in accordance with the alternative design method (Appendix P).

Designers using this Standard for the alternative design method must do so only as directed by Appendix P.

1.1.2
This Standard applies to buildings, structures and cranes constructed of steel or of composite steel and concrete members (in accordance with section 13).

This Standard applies to road, rail and pedestrian bridges. Derivation of loads for such bridges shall be to 3.2.1.

1.1.3
Where appropriate specific member design provisions are available for specialised steel structures, these provisions may be used in lieu of the member design provisions of this Standard, provided that the requirements of section 3 are complied with. The factor of safety against failure shall be consistent with that required for ultimate limit state design in accordance with this Standard and the Loadings Standard.

1.1.4
This Standard does not apply to the following structures and materials:

(a) Steel elements less than 3 mm thick, except for packers and square or rectangular hollow sections to 2.2.1(a), (b) or (c).

(b) Steel members for which the value of the yield stress used in design (f_y) exceeds 450 MPa. An exception is the use of quenched and tempered steel for which f_y = 690 MPa. This steel may be used as splice cover plates, in fully bolted connections only, within the scope of this Standard, provided the bearing stress is limited as specified in 9.3.2.4.2, and the appropriate grade for general structural use is selected (ASTM A514 or equivalent grade). Any welding of such steel shall be in accordance with AS/NZS 1554.4 with regard to any reduction in mechanical properties due to the fabrication process.

(c) Cold-formed members, other than those complying with the appropriate standard from 2.2.1, which shall be designed in accordance with AS/NZS 4600.

1.1.5 Use of this Standard as a means of compliance with the New Zealand Building Code

1.1.5.1
Part 1 of this Standard will be called up as a verification method for compliance with the New Zealand Building Code in Approved Documents B1: Structure-General, B2: Durability and C4: Structural Stability in Fire.

1.1.5.2
For use as a verification method in Approved Document B1, Part 1 of this Standard must be used in conjunction with NZS 4203.

1.1.5.3
Use of Part 1 of this Standard for design of structures which are outside the scope of AS/NZS 1170.0 clauses 1.1 and 1.2, such as those based on special studies or those relying on other loadings documents, comprises an alternative solution and compliance with the Building Code shall be demonstrated.

Amd 2 Oct. '07

1.1.5.4
The Commentary to this Standard, i.e. NZS 3404:Part 2:1997, does not contain requirements essential for compliance with this Standard but fulfils the following roles:

(a) It provides guidance on the use of Part 1;

(b) It summarizes the technical background to the provisions of Part 1;

(c) Wherever Part 1 specifies the use of appropriate or rational design procedures or member design provisions, Part 2 provides details which satisfy the intent of Part 1, either directly or by reference to relevant published documents.

1.1.5.5
Where Part 1 of this Standard contains provisions that are expressed in non-specific or unquantified terms (such as the required use of appropriate or rational design procedures or member design provisions) then these do not form part of the verification method and must be treated as an alternative solution.

In such instances, note the role of Part 2 in providing guidance (see 1.1.5.4(c)).

1.1.5.6
Notes in Part 1 have the same status as the clauses, tables or figures to which they refer.

1.2 REFERENCED DOCUMENTS

The documents referred to in this Standard are listed in Appendix A. Unless otherwise noted, a standard referred to in this Standard is the current edition thereof.

1.3 DEFINITIONS

For the purpose of this Standard, the following definitions apply:

ACTION. The internal force or bending moment.

NOTE – In AS/NZS 1170 this is defined as ACTION EFFECT.

Amd 2 Oct. '07

ASSOCIATED STRUCTURAL SYSTEM. A structural system which is not specifically designed for load combinations including earthquake loads but which is subjected to earthquake effects (principally rotations between beam and column members induced by lateral displacements) resulting from the deformation of the seismic-resisting system under its design loads (severe seismic level of loading unless stated otherwise).

AUTHORITY. A body having statutory powers to control the design and erection of a structure e.g. The Territorial Authority, for applications within the scope of 1.1.5.

BASE (of a structure). The average level at which design actions transfer into the foundations or the average level at which the structure as a lateral load-resisting system is externally laterally supported.

BEARING-TYPE CONNECTION. Connection effected using either snug-tight bolts, or high-strength bolts tightened to induce a specified minimum bolt tension, in which the design action is transferred by shear in the bolts and bearing on the connected parts at the ultimate limit state.

BRACED MEMBER. One for which the transverse displacement of one end of the member relative to the other is effectively prevented.

CAPACITY DESIGN. A part of design for the ultimate limit state used, where necessary, to direct inelastic demand under severe seismic forces into selected members of the seismic-resisting system.

Amd 2 Oct. '07 CAPACITY DESIGN ACTION. The design action derived from application of capacity design in accordance with 12.2.7.

CLAD (STRUCTURE). A clad steel structure is one with external cladding over a substantial part of its perimeter or with generally concrete or composite steel flooring systems integrally connected to the supporting structure. In the latter case the structure may be open or fully enclosed around its perimeter.

COMPACT SECTION. A section made up of individual elements with sufficiently low slenderness ratios to ensure that the plastic moment capacity of the section can be developed.

COMPLETE PENETRATION BUTT WELD. A butt weld in which fusion exists between the weld and parent metal throughout the complete depth of the joint.

CONCENTRICALLY BRACED FRAME SYSTEM. A braced frame in which the members are subjected primarily to axial forces.

CONCURRENT ACTION (seismic applications only). The occurrence of earthquake-induced deformation in directions not necessarily aligned to either principal direction of the structure's seismic-resisting systems and therefore causing simultaneous earthquake-induced actions in both principal directions.

CONNECTION. The entire assemblage of connection components and connectors at the intersection of two members.

CONNECTION COMPONENT. A fabricated item of a connection designed to transfer force from the member to the connector.

CONNECTOR. An element of a connection designed to transfer force from one member or connection component to another.

CONSTANT STRESS RANGE FATIGUE LIMIT. Highest constant stress range for each detail category at which fatigue cracks are not expected to propagate (see figure 10.6.1).

CONSTRUCTION REVIEWER. The person responsible for review of construction, as appointed through 1.6.3.1.

CRITICAL HEIGHT (for a structural system). Over 4 storeys (or over 5 storeys if the combined mass of the roof and top storey walls is less than 150 kg/m^2).

CUT-OFF LIMIT. For each detail category, the highest variable stress range which does not require consideration when carrying out cumulative damage calculations (see figures 10.6.1 and 10.6.2).

DESIGN. The use of rational computational or experimental methods in accordance with the established principles of structural mechanics, to analyse or predict structural performance.

DESIGN ACTIONS. The actions computed from the design loads or design forces.

DESIGN CAPACITY (alternatively known as the dependable capacity). The nominal capacity multiplied by the appropriate strength reduction (capacity) factor.

Amd 2 Oct. '07 NOTE – In AS/NZS 1170 this is defined as DESIGN STRENGTH.

Amd 2 Oct. '07 DESIGN ENGINEER. A person who shall be a chartered professional engineer who is registered under the CPEng of NZ Act 2002 and who is competent to design structural elements of the building under consideration to safely resist the design loads or effects likely to be imposed on the building.

DESIGN FORCES OR DESIGN LOADS. The combination of the nominal forces or loads and the load factors, as specified in the Loadings Standard. This applies to the ultimate limit state unless stated otherwise, except for Appendix P where it applies to design for strength and serviceability to the alternative design method.

DESIGN LIFE. Period over which a structure or structural element is required to perform its function without repair.

DESIGN RESISTANCE. The resistance computed from the loads and design capacities contributing towards the strength and stability of the structure in the ultimate limit state.

DESIGN SPECTRUM (fatigue). Sum of the stress spectra from all of the nominal loading events expected during the design life.

DETAIL CATEGORY. Designation given to a particular detail to indicate which of the S-N curves is to be used in the fatigue assessment.

DISCONTINUITY. An absence of material, causing a stress concentration.

DOUBLER PLATE. Additional side reinforcing plate, fixed to the web of a member, provided to increase the shear capacity of the web.

DUAL (SEISMIC-RESISTING) SYSTEM. A dual system consists of two separate seismic-resisting systems connected together and acting in unison to resist a given direction of earthquake loading.

DUCTILITY. The ability of a structure or member thereof to undergo repeated and reversing deflections beyond the yield deflection while maintaining a substantial proportion of its initial maximum load-carrying capacity. Global ductility relates to the structure as a whole, member ductility to an individual member or members.

DUCTILITY FACTOR OR DEMAND

MEMBER DISPLACEMENT DUCTILITY FACTOR OR DEMAND (μ). The ratio of maximum transverse displacement developed in a member to its yield displacement.

SECTION CURVATURE DUCTILITY FACTOR OR DEMAND. The ratio of maximum curvature developed in a member to its yield curvature.

STRUCTURE DISPLACEMENT DUCTILITY FACTOR OR DEMAND (μ). The ratio of maximum transverse displacement developed in a structure to its yield displacement.

ECCENTRICALLY BRACED FRAME SYSTEM (EBF). A braced frame in which at least one end of each brace frames only into a beam in such a way that at least one stable, deformable link beam is formed in each beam if the elastic limit of the frame is exceeded. In this event, energy is dissipated through shear and/or flexural yielding in the link beams (termed the active link regions) whereas the bracing members and columns shall remain essentially elastic.

EFFECT. A cause of stresses or deformations in a structure.

EFFECTIVE LENGTH FACTOR, k. The factor by which the actual column length is multiplied to obtain the effective length for use in design.

FASTENER. A prefabricated item which transfers load – i.e. bolt, rivet, turnbuckle.

FATIGUE. Damage caused by repeated fluctuations of stress leading to gradual cracking of a structural element.

FATIGUE LOADING. Set of nominal loading events described by the distribution of the loads, their magnitudes and the numbers of applications of each nominal loading event.

FATIGUE STRENGTH. The stress range defined in 10.6 for each detail category (see figures 10.6.1 and 10.6.2) varying with the number of stress cycles.

FIRE EXPOSURE CONDITION.

(a) Three-sided fire exposure condition – steel member incorporated into or in contact with a concrete or masonry floor or wall.

(b) Four-sided fire exposure condition – a steel member exposed to fire on all sides.

FIRE PROTECTION SYSTEM. The fire protection material and its method of attachment to the steel member.

FIRE-RESISTANCE RATING (FRR). The fire resistance grading period for structural adequacy only, in minutes, which is required to be attained in the standard fire test.

FIRST ORDER ANALYSIS. A (frame) analysis in which the derivation of member actions is based on the initial geometry of the structure and any reduction in the elastic stiffness of the members due to axial compressive forces is neglected.

FLANGE SLENDERNESS. The ratio of the critical unsupported width of flange to the average flange thickness.

FRICTION-TYPE CONNECTION. Connection effected using high-strength bolts tightened to induce a specified minimum bolt tension such that the resultant clamping action transfers the design shear forces at the serviceability limit state acting in the place of the common contact surfaces by the friction developed between the contact surfaces.

FULL TENSIONING. A method of installing and tensioning a bolt in accordance with 15.2.4 and 15.2.5.

GEOMETRICAL SLENDERNESS RATIO. The geometrical slenderness ratio (L_e/r), taken as the effective length (L_e), specified in 6.3.2, divided by the radius of gyration (r) computed for the gross section about the relevant axis.

IDEAL CAPACITY FACTOR. A factor used in the design of secondary seismic-resisting members or elements and which is set at (1.0/0.9).

INCOMPLETE PENETRATION BUTT WELD. A butt weld in which the depth of penetration is less than the complete depth of the joint.

IN-PLANE LOADING. Loading for which the design forces and bending moments are in the plane of the connection, such that the design actions induced in the connection components are shear forces only.

INSPECTOR. A person who, on the basis of experience or qualifications, is competent to carry out specific inspection duties stipulated by the design engineer or the requirements of this Standard or a referenced Standard.

INTERNAL FRAME. A frame (structural system) in which the tributary floor area for determination of gravity loading (e.g. dead, live, snow) on the compression members of the frame is approximately equal to the floor area supported laterally by the frame.

LATERAL BUCKLING. An instability phenomenon that reduces the major axis flexural strength of certain types of steel sections due to a combination of flexural action and twist.

LATERAL FORCE-RESISTING SYSTEM OR LATERAL SEISMIC-RESISTING SYSTEM. That part of a structural system assigned to resist lateral forces and suitably designed and detailed to achieve the anticipated level of ductility demand.

LENGTH (of a compression member). The actual length (L) of an axially loaded compression member, taken as the length centre-to-centre of intersections with restraints, or the cantilevered length in the case of a free-standing member.

LIMIT STATE. Any limiting condition beyond which the structure ceases to fulfil its intended function.

LOAD. An externally applied load or force.

NOTE – In AS/NZS 1170 this is defined as ACTION.

LOAD, DEAD. Is referred to in AS/NZS 1170 set as “permanent action”.

Amd 2 Oct. ’07 LOAD, LIVE. Is referred to in AS/NZS 1170 set as “imposed action”.

LOAD SET. A unique combination of limit state loads (ultimate limit state unless specified otherwise) used for deriving member actions.

LOADINGS STANDARD. A Code of practice or Standard written in limit state format for general structural design and design loadings for buildings and approved by the authority. For use of this Standard as a verification method in Approved Document B1, AS/NZS 1170 set shall be the Loadings Standard used. This is comprised of AS/NZS 1170.0, AS/NZS 1170.1, AS/NZS 1170.2, AS/NZS 1170.3, and NZS 1170.5. Amd 2 Oct. ’07

LOCAL BUCKLING. A local instability phenomenon which involves a change of shape of the member cross section along a relatively short length of member.

MEMBER. The length as defined in 5.3.1.3, 6.3.1, 8.1.2.1 or 8.1.2.2.

MINER’S SUMMATION. Cumulative damage calculation based on the Palmgren-Miner summation or equivalent.

MOMENT-RESISTING SYSTEM. A structural system of rigid or semi-rigid construction capable of resisting design loads or effects principally through the bending resistance of its members and connections.

NEGATIVE MOMENT (As applied to a composite beam; see 13.4). A moment which induces tension in the concrete slab reinforcement and compression in at least the bottom flange and lower region of the web of the supporting steel beam.

NEGATIVE MOMENT END (of member). In a member subject to varying bending moments under a combination of seismic-induced and gravity loading, the negative moment end of the member signifies that end of the member where the direction of member rotation caused by both seismic-induced and gravity loading is the same.

NOMINAL CAPACITY. The capacity of a member or connection computed excluding the strength reduction factors.

Amd 2 Oct. '07

NOTE – In NZS 1170.5 this is defined as NOMINAL STRENGTH.

NOMINAL EFFECT OR LOAD. An effect or load as specified in 3.2.1 or 3.2.2.

NOMINAL LOADING EVENT. The loading sequence for the structure or structural element.

NON-SLIP FASTENERS. Fasteners which do not allow slip to occur between connected plates or members at the serviceability limit state so that the original alignment and relative positions are maintained.

Amd 2 Oct. '07

(Text deleted)

OUT-OF-PLANE LOADING. Loading for which the design forces or bending moments result in design actions normal to the plane of the member or connection.

OVERSTRENGTH. The maximum strength that a member can generate, taking into account higher than specified steel yield stresses and an increase in strength due to strain hardening, where relevant.

Amd 2 Oct. '07

OWNER. The owner as defined in the Building Act 2004.

$P-\Delta$ (P – DELTA) EFFECT. Refers to the structural actions induced due to the gravity loads being displaced laterally by the action of the lateral effects or loads.

$P-\delta$ EFFECT. The second-order effect generated by the member axial compression load on the first-order design actions on an individual member.

PERIMETER FRAME. A frame (structural system) in which the tributary floor area for determination of gravity loading (e.g. dead, live, snow) on the compression members of the frame is significantly smaller than the floor area supported laterally by the frame.

PERIOD OF STRUCTURAL ADEQUACY (PSA) (fire). The time (t), in minutes, for the member to reach the limit state of structural adequacy in the standard fire test.

PIN. A fastener, manufactured out of round bar, which is intended to allow for some rotation in service.

PLASTIC HINGE. A yielded zone with significant inelastic moment or shear rotation which forms in a member when the plastic moment or shear strength is attained or exceeded. The member rotates as if hinged except that it is restrained by a moment or shear equal to or higher than the plastic moment or shear strength.

POSITIVE MOMENT (As applied to a composite beam; see 13.4). A moment which induces compression in at least the upper region of the concrete slab and tension in at least the bottom flange of the supporting steel beam.

POSITIVE MOMENT END (of member). In a member subject to varying bending moments under a combination of seismic-induced and gravity loading, the positive moment end of the member signifies that end of the member where the direction of member rotation caused by seismic-induced loading opposes the direction of member rotation caused by gravity loading.

PREQUALIFIED WELD PREPARATION. A joint preparation prequalified in terms of AS/NZS 1554.1 or AS/NZS 1554.5.

Amd 2 Oct. '07 PRIMARY SEISMIC-RESISTING ELEMENT. An element or member of a seismic-resisting system chosen and designed to be part of the main energy dissipating mechanism.

PRINCIPAL. The purchaser or owner of the structure being fabricated or erected or a nominated representative.

PROOF TESTING. The application of test loads to a structure, sub-structure, member or connection to ascertain the structural characteristics of only that one unit under test.

PROTOTYPE (fire). A test specimen representing a steel member and its fire protection system which is subjected to the standard fire test.

PROTOTYPE TESTING. The application of test loads to one or more structures, sub-structures, members or connections to ascertain the structural characteristics of that class of structures, sub-structures, members or connections which are nominally identical to the units tested.

PRYING FORCE. Additional tensile force developed in a fastener as a result of the flexing of a connection component in a connection subjected to tensile force.

RECTANGULAR FRAME. A frame comprising at least beam and column members in which the orientation of any beam member is sufficiently close to the horizontal such that axial forces in it from applied vertical loading are negligible and the orientation of the column member is not greater than 5° from the vertical.

RESTRAINT. (Of a member subject to bending). An element which effectively prevents deflection or twisting of a member out of the plane of bending (or transverse loading) on that member.

RESTRAINT. (Of a member subject to axial compression). An element which effectively prevents movement of all points of the cross section out of the plane of action of the compression load on the member.

NOTE – A restraint of a member subject to axial compression against buckling about the minor principal *y*-axis will also fully or partially restrain the cross section in major principal *x*-axis bending (see 5.4.2.1 or 5.4.2.2), if designed for the appropriate restraint forces from 5.4.3, for a member subject to combined bending and axial compression.

S-N CURVE. Curve defining the limiting relationship between the number of stress cycles and the stress range for a detail category.

Amd 2 Oct. '07

SECOND-ORDER ANALYSIS. A (frame) analysis (elastic, unless started otherwise) in which the derivation of member actions is based in the deformed shape of the structure and also includes, when required, the effects of reduction in elastic stiffness of the members due to axial compressive forces.

Amd 2 Oct. '07

SECONDARY SEISMIC-RESISTING ELEMENT OR MEMBER. An element or member of either a seismic-resisting system, or of an associated structural system which is subject to the capacity design process, which is chosen not to be part of the main energy dissipating mechanism.

SECTION FACTOR (fire). Either of:

(a) EXPOSED SURFACE AREA TO MASS RATIO. The ratio of the surface area exposed to the fire to the mass of the steel, or

(b) EXPOSED PERIMETER TO SURFACE AREA RATIO. The ratio of perimeter of the surface exposed to fire to the area of cross section of the steel.

SEGMENT (in a member subjected to bending). The length between adjacent cross sections which are fully, partially or laterally restrained, *or* the length between an unrestrained end and the adjacent cross section which is fully or partially restrained.

SERVICEABILITY LIMIT STATE. A limit state of acceptable in-service condition.

SEVERE EARTHQUAKE LOADS or SEVERE SEISMIC LOADS . The earthquake loads (resulting from application of earthquake-induced ground motion) applicable to the ultimate limit state as derived in accordance with the Loadings Standard.

SHALL. Implies that compliance with a requirement is mandatory for compliance with the Standard.

SHEAR WALL. A wall designed to resist lateral forces parallel to the plane of the wall.

SHOULD. Implies that compliance with a requirement is strongly recommended but is not mandatory for compliance with the Standard.

SNUG TIGHT. The tightness of a bolt achieved by a few impacts of an impact wrench or by the full effort of a person using a standard podger spanner.

SPECIAL STUDY. A procedure for the analysis and/or design of the structure, agreed between the authority and the design engineer, for use where directed by this Standard or where desired and initiated by the design engineer in order to enable some or all of the requirements of this Standard and the Loadings Standard to be modified or waived.

STABILITY (limit state). Part of the ultimate limit state corresponding to the loss of static equilibrium of a structure considered as a rigid body.

STANDARD FIRE TEST. The fire-resistance test specified in one of ISO 834, AS 1530.4 or BS 476:Parts 20-23.

STICKABILITY. The ability of the fire protection system to remain in place as the member deflects under load during a fire test.

STOREY. The part of a structural system between two logical consecutive horizontal divisions.

STRENGTH REDUCTION FACTOR (alternatively known as the CAPACITY FACTOR). A factor used to multiply the nominal capacity to obtain the design (dependable) capacity.

STRESS CYCLE (fatigue). One cycle of stress defined by stress cycle counting.

STRESS CYCLE COUNTING METHOD (fatigue). Any rational method used to identify individual stress cycles from the stress history.

STRESS RANGE (fatigue). Algebraic difference between two extremes of stress.

STRESS SPECTRUM (fatigue). Histogram of the stress cycles produced by a nominal loading event.

STRUCTURAL ADEQUACY (fire). The ability of the member exposed to the standard fire test to carry the test load specified in one of ISO 834, AS 1530.4 or BS 476:Parts 20-23.

STRUCTURAL PERFORMANCE FACTOR. A factor related to the structural form and materials of construction and which is used in the derivation of design earthquake loads in accordance with NZS 1170.5 of this Standard.

Amd 2 Oct. '07

STRUCTURAL SYSTEM. An assemblage of members and connections acting together to resist the actions due to the specified design loads.

SUPPORT. (Of a member subject to bending). An element which effectively prevents deflection of a member in the plane of bending (or transverse loading) on that member and which effectively prevents deflection or twisting of a member out of the plane of bending (or transverse loading) on that member (i.e. provides full or partial section restraint in accordance with clause 5.3.1.4).

SWAY MEMBER. One for which the transverse displacement of one end of the member relative to the other is not effectively prevented.

TENSILE STRENGTH. The minimum ultimate strength in tension specified for the grade of steel in the appropriate Standard as listed in 2.2.

TRANSVERSE LOAD. Load applied to a member or segment between supports or restraints and acting transverse to the longitudinal axis of the member.

ULTIMATE LIMIT STATE. A limit state of collapse of the structure or its components or a limit state of loss or structural integrity or loss of static equilibrium of a structure considered as a rigid body.

UNCLAD (structure). An unclad steel structure is one with an open floor system and no exterior covering. Steel grid panel flooring systems laid on steel beams do not constitute cladding.

WEB SLENDERNESS. The ratio of the unstiffened web depth, to the web thickness.

YIELD CURVATURE. The curvature which produces yield point stresses in the extreme fibres, multiplied by the shape factor for the section.

YIELD STRESS. The minimum yield stress in tension specified for the grade of steel in the appropriate Standard as listed in 2.2.

YIELDING REGION. That length of a member which is intended to yield under earthquake loads or due to redistribution of design actions or in which yield under earthquake loads or effects may occur.

1.4 NOTATION

Symbols used in this Standard are listed below.

Where non-dimensional ratios are involved, both the numerator and denominator are expressed in identical units.

The dimensional units for length and stress in all expressions or equations are to be taken as millimetres (mm) and megapascals (MPa) respectively, unless specifically noted otherwise.

A superscripted '*' placed after a symbol, denotes a design action due to the design load for the ultimate limit state, or for the serviceability limit state where expressly noted.

A = area of cross section

A_c = minor diameter area of a bolt, as defined in AS 1275; or
= effective area of concrete slab in a composite beam

A_{cs} = concrete slab cross section area under compression between shear planes (13.4.10)

A_{cv} = area of concrete in shear planes (13.4.10)

A_e = effective area of a cross section; or
= area enclosed by a hollow section

A_{ep} = area of an end plate

A_{fb} = flange area of incoming beam

A_{fc} = flange area at critical cross section

A_{fg} = gross area of a flange

A_{fm} = flange area at minimum cross section; or
= lesser of the flange effective areas

A_{fn} = net area of a flange

A_g = gross area of a cross section within the zone of intended yielding (section 12); or
= area of gross concrete section (13.8.2)

A_n = net area of a cross section at the connection (including allowing for loss of area due to the presence of bolt holes and subsequent area replacement in accordance with 12.9); or
= sum of the net areas of the flanges and the gross area of the web

A_o = plain shank area of a bolt

A_p = cross-sectional area of a pin

A_{rl} = area of slab longitudinal reinforcement within the concrete area A_{cs} (13.4.10)

A_{rs} = area of slab reinforcement within effective width of slab in a composite beam

Amd 1 June '01 A_{rt} = area of slab transverse reinforcement crossing shear planes (13.4.10)

A_s = tensile stress area of a bolt as defined in AS 1275; or
= area of a stiffener or stiffeners in contact with a flange and web; or
= area of an intermediate web stiffener; or
= area of encased steel member (13.8.2)

A_{sc} = shank area of stud

A_w = gross sectional area of a web; or
= effective shear area of a plug or slot weld

a = vertical distance from centre-line of bolt hole to edge of end plate as used in Appendix M

a_b = horizontal spacing between bolts through end plate at beam tension flange, as used in Appendix M

a_c = depth of equivalent rectangular stress block

a_e = minimum distance from the edge of a hole to the edge of a ply, measured in the direction of the component of a force, plus half the bolt diameter

a_o = length of unthreaded portion of the bolt shank contained within the grip

a_t = length of threaded portion of the bolt contained with the grip

a_0, a_1 = out-of-square dimensions of flanges

a_2, a_3 = diagonal dimensions of a box section

b = width; or
= lesser dimension of a web panel; or
= clear width of an element outstand from the face of a supporting plate element; or
= clear width of a supported element between faces of supporting plate elements; or
= distance from beam flange or weld face to centre line of bolts as used in Appendix M

b_b, b_{bf}, b_{bw}, b_o = bearing widths defined in 5.13

b_b = beam flange width at the active link of an eccentrically braced frame

b_{bp} = width of one backing plate

b_c = overall width of composite column member; or
= width of column flange in a joint panel zone

b_d = distance from the stiff bearing to the end of the member

b_e = effective width of a plate element

b_{ec} = concrete effective width in composite slab

b_{ep} = effective width of end plate

b_{es} = stiffener outstand from the face of a web

b_f = width of a flange

b_{fo} = distance from mid-plane of the web to the nearer edge of the flange; or
= half the clear distance between the webs

b_r = average width of concrete rib in a composite slab cast onto a profiled steel deck

b_s = stiff bearing length

b_w = web depth

b_1, b_2 = greater and lesser leg lengths of an angle section

C_d (T_1) = horizontal design action co-efficient, as determined from NZS 1170.5 for use in section 12

Amd 2 Oct. '07

(Text deleted)

C_1 = factor defined in 12.6.2.1; or
= factor defined in 12.9.4.3

C_2 = factor defined in 12.9.5.2

C_3, C_4, C_{4r} = factors defined by table H3 and clause H5

C_5, C_6 = factors defined in 13.4.3.2

c_b = vertical spacing between bolts through end plate at beam tension flange level, as used in Appendix M

(Text deleted)

Amd 2 Oct. '07

c_m = factor for unequal moments

d = depth of a section; or
= depth of preparation for partial penetration butt weld; or
= maximum cross-sectional dimension of a member

d_b = lateral distance between centroids of the welds or fasteners on battens; or
= depth of a beam

d_c = depth of a section at a critical cross section; or
= depth of a column at a joint panel zone

d_e = effective outside diameter of a circular hollow section; or
= factor defined in Appendix J

d_f = diameter of a fastener (bolt or pin); or
= distance between flange centroids

d_f' = diameter of bolt hole

d_{fb} = bolt shank diameter as used in Appendix M

d_m = depth of a section at the minimum cross section

d_o = overall section depth including out-of-square dimensions; or
= overall section depth of a segment; or
= outside diameter of a circular hollow section

d_p = clear transverse dimension of a web panel; or
= depth of deepest web panel in a length

d_{sc} = diameter of a stud shear connector

d_x, d_y = distances of the extreme fibres from the neutral axes

d_1 = clear depth between flanges ignoring fillets or welds

d_{1c} = twice the distance from the plastic neutral axis of a composite section to the inside face of the compression flange

d_2 = twice the clear distance from the neutral axis to the compression flange

d_3, d_4 = depths of preparation for incomplete penetration butt welds

d_5 = flat width of web

E = Young's modulus of elasticity, 205 x 10^3 MPa

E_c = short-term elastic modulus of concrete

E_n = inelastic modulus for steel (associated with strain hardening) = 0.025E

$E(T)$, $E(20)$ = E at T, 20 °C respectively

$(EI)_{cc}$ = flexural stiffness of an encased composite column (13.8.2)

e = eccentricity; or
= web off-centre dimension; or
= distance between an end plate and a load-bearing stiffener; or
= lever arm from centroid of steel area in compression block to centroid of steel area in tension (for a composite section); or
= clear length of active link in eccentrically braced frame

e' = lever arm from centroid of concrete compression block to centroid of steel area in tension (in a composite section)

Amd 1 June '01 | *(Text deleted)*

e_{min} = minimum length of active link = d_b (12.11.3.7)

F = action in general, force or load

F^* = total design load on a member between supports

F_n^* = design force normal to a web panel

F_p^* = design force parallel to a web panel

f_c = fatigue strength corrected for thickness of material

f'_c = specified concrete cylinder compression strength @ 28 days unless stated otherwise

Amd 2 Oct. '07 | f'_{cos} = the long term increase in concrete stress in the slab above the nominal 28 day strength, taken as 10 MPa

f_f = uncorrected fatigue strength

f_{rn} = detail category reference fatigue strength at n_r cycles – normal stress

f_{rnc} = corrected detail category reference fatigue strength – normal stress

f_{rs} = detail category reference fatigue strength at n_r cycles – shear stress

f_{rsc} = corrected detail category reference fatigue strength – shear stress

f_u = tensile strength, refer 2.1

f_{uf} = minimum tensile strength of a bolt

f_{up} = tensile strength of a ply

f_{uw} = nominal tensile strength of weld metal

f_y = yield stress, refer 2.1, of the element under consideration

$f_y(T)$, $f_y(20)$ = yield stresses of steel at T, 20 °C respectively

f_{ybp} = yield stress of a backing plate

f_{yep}	=	yield stress of an end plate
f_{yp}	=	yield stress of a pin used in design
f_{yr}	=	yield stress of tension reinforcement in the concrete slab of a composite member
f_{ys}	=	yield stress of a stiffener used in design
f_3	=	detail category fatigue strength at constant amplitude fatigue limit
f_{3c}	=	corrected detail category fatigue strength at constant amplitude fatigue limit
f_5	=	detail category fatigue strength at cut-off limit
f_{5c}	=	corrected detail category fatigue strength at cut-off limit
f^*	=	design stress range
f_i^*	=	design stress range for loading event *i*
f_{va}^*	=	average design shear stress in a web
f_{vm}^*	=	maximum design shear stress in a web
f_w^*	=	equivalent design stress on a web panel (Appendix J)
f_{yp}^*	=	design yield stress for joint panel zone
G	=	shear modulus of elasticity, 80 x 10^3 MPa; or
	=	dead load
H_p/A	=	exposed perimeter to surface area ratio (m^{-1})
h	=	rectangular centroidal axis for angle parallel to the loaded leg
h_b	=	vertical distance between tops of beams
h_e	=	effective thickness of fire protection material; or
	=	effective thickness of slab for determining the period of structural adequacy; or
	=	height to centre-line of rafter at the eave of a portal frame
h_i	=	thickness of fire protection material
h_{rc}	=	nominal height of steel deck rib
h_s	=	storey height
h_{sc}	=	length of stud connector after welding, mm, not to exceed the value (h_{rc} + 75) in calculations, although the actual length may be greater
I	=	second moment of area of a (bare steel member) cross section

I_{cy} = second moment of area of compression flange about the section minor principal y-axis

I_g = second moment of area of gross concrete section about its principal axis (13.8.2)

I_m = I of the member under consideration

I_{pc} = I of outer column in a portal frame, for use in 4.9.2.4

I_{pr} = I of outer rafter in a portal frame, for use in 4.9.2.4

I_r = I of a restraining member

I_s = I of a pair of stiffeners or a single stiffener; or
= second moment of area of an encased steel member about its principal axis (13.8.2)

I_{st} = second moment of area of steel beam alone

I_{tc} = second moment of area of composite beam transformed into equivalent steel section

I_w = warping constant for a cross section

I_x = I about the cross section major principal x-axis

I_y = I about the cross section minor principal y-axis

i = number of loading events

J = torsion constant for cross section

K = flexural-torsional buckling constant (Appendix H)

k = coefficient used in Appendix K

k_b = elastic buckling coefficient for a plate element

k_{bo} = basic value of k_b

k_e = member effective length factor

k_f = form factor for members subject to axial compression

k_h = factor for different hole types

k_l = load height effective length factor

k_p = factor for pin rotation

k_r = effective length factor for restraint against lateral rotation; or
= effective length factor for a restraining member; or
= reduction factor to account for the length of a bolted or welded lap splice connection

k_s = ratio used to calculate α_p and α_{pm}

k_{sm} = exposed surface area to mass ratio (m^2/tonne)

k_t = twist restraint effective length factor

k_{te} = correction factor for distribution of forces in a tension member

k_v = ratio of flat width of web (d_5) to thickness (t) of section

k_0–k_6 = regression coefficients (section 11)

L = span; or
= member length; or
= segment or sub-segment length

L_b = length between points of effective bracing or restraint

L_{bp} = length of backing plate

L_c = distance between adjacent column centres

L_e = effective length of a compression member; or
= effective length of a laterally unrestrained member

$\frac{L_e}{r}$ = geometrical slenderness ratio

$\left(\frac{L_e}{r}\right)_{bn}$ = slenderness ratio of a battened compression member about the axis normal to the plane of the battens

$\left(\frac{L_e}{r}\right)_{bp}$ = slenderness ratio of a battened member about the axis parallel to the plane of the battens

$\left(\frac{L_e}{r}\right)_{c}$ = slenderness ratio of the main component in a laced or battened compression member

$\left(\frac{L_e}{r}\right)_{m}$ = slenderness ratio of the whole battened compression member

L_j = length of a bolted lap splice connection

L_m = length of the member under consideration

L_r = length of a restraining member; or
= length of a segment over which the cross section is reduced

L_s = distance between points of effective lateral support

L_{sc} = length of channel shear connector

L_w = greatest internal dimension of an opening in a web; or
= length of a fillet weld in a welded lap splice connection

L_z = distance between partial or full torsional restraints

M_{bx} = nominal member moment capacity about the major principal x-axis

M_{bxo} = M_{bx} for a uniform distribution of moment

M_{cx} = lesser of M_{ix} and M_{ox}

M_f = nominal moment capacity of flanges alone

M_i = nominal in-plane member moment capacity

M_{ix} = M_i about major principal x-axis

M_{iy} = M_i about minor principal y-axis

M_o = nominal out-of-plane member moment capacity; or
= reference elastic buckling moment for a member subject to bending

M_{oa} = amended elastic buckling moment for a member subject to bending

M_{ob} = elastic buckling moment determined using an elastic buckling analysis

M_{obr} = M_{ob} decreased for elastic torsional end restraint

M_{oo} = reference elastic buckling moment obtained using $L_e = L$

M_{os} = M_{ob} for a segment, fully restrained at both ends, unrestrained against lateral rotation and loaded at shear centre

M_{ox} = nominal out-of-plane member moment capacity about major principal x-axis

M_p = nominal moment capacity of a pin

M_{pr} = nominal plastic moment capacity reduced by axial force

M_{prx} = M_{pr} about major principal x-axis

M_{pry} = M_{pr} about minor principal y-axis

M_r = nominal section moment capacity reduced by axial force

M_{rc} = nominal moment capacity of a composite section

M_{rx} = M_r about major principal x-axis

M_{ry} = M_r about minor principal y-axis

M_s = nominal section moment capacity

M_{sal} = nominal moment capacity of an active link in an eccentrically braced frame (12.11.3.5)

M_{sp} = nominal plastic moment capacity (of a compact section)

M_{sv} = nominal section moment capacity in the presence of shear

M_{sx} = M_s about major principal x-axis

M_{sy} = M_s about minor principal y-axis

M_{tx} = lesser of M_{rx} and M_{ox}

M_w = nominal section moment capacity of a web panel

M^* = design bending moment

M_e^* = second-order or amplified end bending moment

M_f^* = design end bending moment

M_{fb}^* = braced component of M_f^* obtained from a first-order elastic analysis of a frame with sway prevented

M_{fs}^* = sway component of M_f^* obtained from $(M_f^* - M_{fb}^*)$

M_h^* = design bending moment on an angle, acting about the rectangular h-axis parallel to the loaded leg

M_m^* = maximum calculated design bending moment along the length of a member or in a segment

M_r^* = controlling design bending moment for determination of length of yielding region, as used in 12.6

M_w^* = design bending moment acting on a web panel

M_x^* = design bending moment about major principal x-axis

M_y^* = design bending moment about minor principal y-axis

M_2^*, M_3^*, M_4^* = design bending moments at quarter and mid-points of a segment

Amd 2 Oct. '07 *(Text deleted)*

N_c = nominal member capacity in compression

N_{ch} = N_c for angle buckling about h-axis, parallel to the loaded leg

N_{cy} = N_c for member buckling about minor principal y-axis

Amd 2 Oct. '07 *(Text deleted)*

Amd 2 Oct. '07 *(Text deleted)*

N_{oL} = Euler buckling load = $\dfrac{\pi^2 EI}{L^2}$

N_{oLr} = Euler buckling load = $\dfrac{\pi^2 EI}{L_r^2}$

N_{om} = elastic flexural buckling load of a member

N_{omb} = N_{om} for a braced member

N_{oms} = N_{om} for a sway member

N_{on} = inelastic buckling load

N_{oz} = nominal elastic torsional buckling capacity of a member

N_p = bolt prying force

N_s = nominal section capacity of a compression member; or
= nominal section capacity for axial force

N_{sal} = nominal axial force capacity of an active link in an eccentrically braced frame (12.11.3.5)

N_t = nominal section capacity in tension

N_{tf} = nominal tension capacity of a bolt

N_{ti} = minimum bolt tension at installation; or
= tension induced in a bolt during installation

N_{wo} = nominal axial capacity of a web panel

N^* = design axial force, compressive or tensile

Amd 2 Oct. '07 N^*_{oc} = the capacity design derived design axial compression force on the column when capacity design is used and the column is a secondary element to 12.2.7.1 and 12.2.7.4

N^*_c = design axial compression force from elastic analysis, in outer column of a portal frame, for use in 4.9.2.4

N^*_{fb} = design axial force in beam flange generated by the design moment action plus axial force, if present, on the beam

N^*_{fbt} = design axial tension force in beam flange generated by the design moment action plus axial force, if present, on the beam.

N^*_g = design axial force generated by gravity loading alone (dead, live, snow loading)

N_r^* = design axial compression force from elastic analysis, in outer rafter of a portal frame, for use in 4.9.2.4; or
= design axial force in a restraining member

N_{tf}^* = design tensile force on a bolt

N_w^* = design axial force acting on a web panel

n = number of specimens tested; or
= number of shear connectors required between adjacent points of maximum and zero moment (n_n for a negative moment region, n_p for a positive moment region)

Amd 2 Oct. '07 *(Text deleted)*

n' = number of shear connectors required between a concentrated load and nearest adjacent point of zero moment

Amd 2 Oct. '07 *(Text deleted)*

n_b = number of parallel planes of battens

Amd 2 Oct. '07 *(Text deleted)*

n_{ei} = number of effective interfaces

n_i = number of cycles of nominal load event i

n_n = number of shear planes with threads intercepting the shear plane – bolted connections

n_r = reference number of stress cycles

n_{rc} = number of stud shear connectors on a beam in one rib, not to exceed 3 in calculations although more than 3 studs may be installed

n_s = number of shear planes

n_{sc} = number of stress cycles

n_w = number of webs

n_x = number of shear planes without threads intercepting the shear plane – bolted connections

(Text deleted)

Q = live load
= design transverse force

Amd 2 Oct. '07 *(Text deleted)*

q_r = nominal shear capacity of a shear connector

R_b = nominal bearing capacity of a web

R_{bb} = nominal bearing buckling capacity

R_{by} = nominal bearing yield capacity

R_{cc} = internal compression force from area of concrete in compression (in a composite section)

R_{sb} = nominal buckling capacity of a stiffened web

R_{sc} = internal compression force from area of steel section in compression (in a composite section)

R_{ss} = nominal capacity of all shear connectors between points of maximum and adjacent zero moment

R_{sy} = nominal yield capacity of a stiffened web

R_{tc} = internal tension force from area of steel section in tension (in a composite section)

R_u = nominal capacity

R^* = design bearing force; or
= design reaction

R_h^* = total horizontal shear to be resisted in a composite member

R_w^* = design bearing force or reaction on a web panel

r = radius of gyration; or
= transition radius; or
= the root radius of a section

r_{cc} = radius of gyration of an encased composite column (13.8.2)

r_{ext} = outside radius of section

r_f = ratio of design action on the member under design load for fire to the design capacity of the member at room temperature

r_r = ratio defined in 5.6.1.1.2

r_s = ratio defined in 5.6.1.1.2

r_y = radius of gyration about minor principal y-axis

S = plastic section modulus

/S = snug tight mode of bolt action as defined in 9.2, 9.3.1

S_p = structural performance factor (12.2.2.1)

S^* = design action

s = spacing of stiffeners; or
= width of a web panel

s_b = longitudinal centre-to-centre distance between battens

s_g = gauge of bolts

s_p = staggered pitch of bolts

s_r = centre-line length of rafter in a portal frame, for use in 4.9.2.4

T = steel temperature in degrees Celsius

/TB = tension bearing mode of bolt action as defined in 9.2, 9.3.1

/TF = tension friction mode of bolt as defined in 9.2, 9.3.1

T_l = limiting steel temperature in degrees Celsius (section 11)

T_1 = fundamental period of seismic-resisting system (12.2.9.2 and Appendix B)

t = thickness; or
= thickness of thinner part joined; or
= wall thickness of a circular hollow section; or
= thickness of an angle section; or
= effective thickness of a composite slab; or
= time

t_{bp} = thickness of a backing plate

t_{ep} = thickness of an end plate

t_f = thickness of a flange; or
= thickness of the critical flange

t_{fb} = thickness of a beam flange

t_{fc} = thickness of a column flange

t_n = thickness of a nut

t_p = thickness of a ply; or
= thickness of thinner ply connected; or
= thickness of a plate (including a doubler plate)

t_o = overall thickness of a composite slab (13.1.2.5)

t_r = thickness of a backing plate

t_s = thickness of a stiffener

t_t, t_{t1}, t_{t2} = design throat thickness of a weld

t_w = thickness of a web

t_{wb} = thickness of a beam web at the active link of an eccentrically braced frame

t_{wc} = thickness of a column web

t_{wf} = size of fillet weld to member flange

t_{ww} = size of fillet weld to member web

t_w, t_{w1}, t_{w2} = size of a fillet weld

V_b = nominal bearing capacity of a ply or a pin; or
= nominal shear buckling capacity of a web

V_c = nominal shear capacity of a joint panel zone

V_f = nominal shear capacity of a bolt – ultimate limit state

V_r = design longitudinal shear resistance of slab over composite beam (13.4.10)

V_{sf} = nominal shear capacity of a bolt – serviceability limit state

V_{si} = measured slip-load at the *i* th bolt

V_v = nominal shear capacity of a web

V_{val} = nominal shear capacity of an active link in an eccentrically braced frame (12.11.3.5)

V_{vm} = nominal web shear capacity in the presence of bending moment

V_{vn} = nominal shear capacity of a flat plate with non-uniform shear stress distribution (5.11.3)

V_{vu} = nominal shear capacity of a web with uniform shear stress distribution (5.11.2)

V_w = nominal shear yield capacity of a web; or
= nominal shear capacity of a plug or slot weld

V^* = design shear force; or
= design horizontal storey shear force at lower column end; or
= design transverse shear force

V_b^* = design bearing force on a ply at a bolt or pin location

V_f^* = design shear force on a bolt or a pin – ultimate limit state

V_l^* = design longitudinal shear force

V_p^* = design shear force for joint panel zone

V_{sf}^* = design shear force on a bolt – serviceability limit state

V_w^* = design shear force acting on a web panel; or
= design shear force on a plug or slot weld

Amd 2 Oct. '07 *(Text deleted)*

v_w = nominal capacity of a fillet weld per unit length

v_w^* = design force per unit length on a fillet weld

Amd 2 Oct. '07 *(Text deleted)*

x = major principal axis coordinate

y = minor principal axis coordinate

y_L = distance of the gravity loading below the centroid

y_o = coordinate of shear centre

Z = elastic section modulus

Z_c = Z_e for a compact section

Z_e = effective section modulus

Z_{we} = elastic section modulus of a web panel

α = angle between x- and h-axes for an angle section

α_a = compression member factor, as defined in 6.3.3

α_b = compression member section constant, as defined in 6.3.3; or
= factor defined in equation 13.4.3.2

α_{bc} = moment modification factor for bending and compression; or
= modification factor for limiting composite beam web slenderness ratio from that required for bare steel beam

α_c = compression member slenderness reduction factor

α_d = tension field coefficient for web shear buckling

α_{dc} = reduction factor for shear stud strength in a composite slab cast onto a profiled steel deck

α_f = flange restraint factor for web shear buckling

α_L, α_{Lc}, α_{mc} = factors for bending defined in H2 and H3

α_m = moment modification factor for bending

α_p = coefficient used to calculate the nominal bearing yield capacity (R_{by}) for square and rectangular hollow sections

α_{pm} = coefficient used to calculate α_p

α_{ry} = elastic stiffness of a flexural end restraint

α_{rz}	=	elastic stiffness of a torsional end restraint
α_s	=	slenderness reduction factor; or
	=	inverse of the slope of the S – N curve for fatigue
α_{sr}	=	stability function multiplier
α_{st}	=	reduction factor for members of varying cross section
α_T	=	coefficient of thermal expansion for steel, 11.7 x 10^{-6} per degree Celsius
α_t	=	factor for torsional end restraint defined in 5.14.5
α_v	=	shear buckling coefficient for a web
α_w	=	factor, as defined in Appendix J
ε_y	=	yield strain of steel
β_d	=	ratio of maximum design moment from design dead load (*G*) alone to maximum design moment from full design load, taken as positive (13.8.2)
β_e	=	modifying factor to account for conditions at the far ends of beam members
β_m	=	ratio of smaller to larger bending moment at the ends of a member, taken as positive when the member is bent in reverse (double) curvature; or
	=	ratio of end moment to fixed end moment
β_t	=	coefficient taking account of variation in bending moment along a braced member with transverse loading (4.4.3.2.4); or
	=	measure of elastic stiffness of torsional end restraint used in Appendix H
β_w	=	factor defined in Appendix J
β_x	=	monosymmetry section constant
γ	=	ratio used in 4.8.3.4.1; or
	=	index used in 8.3.4.2; or
	=	factor for transverse stiffener arrangement
ψ_f	=	factor defined in 4.9.2.4
γ_p	=	inelastic rotation angle between the active link and the adjacent beam in an eccentrically braced frame (12.11)
$\gamma, \gamma_1, \gamma_2$	=	ratios of compression member stiffness to end restraint stiffness used in 4.8.3.4
Δ	=	deflection; or
	=	deviation from nominated dimension; or
	=	measured total extension of a bolt when tightened

Δ_{ct} = mid-span deflection of a member resulting from transverse loading together with end bending moments

Δ_{cw} = mid-span deflection of a member resulting from transverse loading together with only those end bending moments which produce a mid-span deflection in the same direction as the transverse load

Δ_f = out-of-flatness of a flange plate

Δh_b = deviation from h_b

ΔL_c = deviation from L_c

Δ_s = translational displacement of the top relative to the bottom for a storey height

Δ_v = deviation from verticality of web at a support

Δ_w = out-of-flatness of a web

δ = standard deviation

δ_b = moment amplification factor for a braced member

δ_m = moment amplification factor, taken as the greater of δ_b and δ_s

δ_p = moment amplification factor for plastic design

δ_s = moment amplification factor for a sway member

ξ = compression member factor, as defined in 6.3.3

η = compression member imperfection factor, as defined in 6.3.3; or
= initial elastic (viscous) damping measured as a percentage value of critical damping (12.2.9); or
= a factor accounting for the influence of axial force on column panel zones in the design of connections (12.9.5.3.2)

θ = angle of preparation of a partial penetration butt weld

θ_p = inelastic rotation angle of frame drift in an eccentrically braced frame (assuming rigid column behaviour); or
= limiting plastic hinge rotation as specified in 4.7.2; and
= inelastic rotation demand, being the ratio of the rotation at a plastic hinge location or yielding region to the relative elastic rotation of the far end of the segment containing the plastic hinge or yielding region (13.4.3)

θ_p^* = plastic hinge rotation from structural analysis in accordance with 4.5.4.1

π = pi ($\approx$ 3.142)

λ = slenderness ratio; or
= elastic buckling load factor

λ_c = elastic buckling load factor

λ_e = plate element slenderness

λ_{ed} = plate element deformation slenderness limit

λ_{ep} = plate element plasticity slenderness limit

λ_{ey} = plate element yield slenderness limit

λ_m = elastic buckling load factor for a member

λ_{ms} = elastic buckling load factor for the storey under consideration

λ_n = modified compression member slenderness

λ_s = section slenderness parameter

λ_{sp} = section plasticity slenderness limit

λ_{sy} = section yield slenderness limit

μ = structural displacement ductility factor

μ_{act} = the structural displacement ductility factor associated with the actual ductility demand on the as-designed seismic-resisting system (see C12.3.2.3 in Part 2 of this Standard for a method of determining μ_{act})

μ_{des} = the structural displacement ductility factor as initially chosen by the designer from 12.2.3.1

μ_s = slip factor

μ_{sm} = mean value of the slip factor

υ = Poisson's ratio ≈ 0.3 for steel

ρ = the ratio of design axial force in a restraining member to the elastic buckling load for a member of length L (Appendix G); or
= I_{cy}/I_y

ϕ = strength reduction (capacity) factor

Amd 1 June '01 | ϕ_{fire} = strength reduction factor for fire emergency conditions (11.5)

ϕ_{om} = overstrength factor, incorporating only the statistical variation in yield stress component

ϕ_{oms} = overstrength factor, incorporating statistical variation in yield stress and a strain hardening component

ϕ_{os} = overstrength factor, incorporating only the strain hardening component

1.5 USE OF ALTERNATIVE MATERIALS OR METHODS

1.5.1 For new structures

Design using methods and/or materials not specifically referred to herein shall be permitted, provided that the requirements of section 3 are complied with and it is demonstrated, by one of the following methods, that the structural elements so designed have adequate performance at the serviceability limit state and at the ultimate limit state:

Amd 2 Oct. '07

(a) A special study, in accordance with clause 1.4.22 of AS/NZS 1170.0 or clause 1.4 of NZS 1170.5; or

(b) Experimental testing; or

(c) Rational design based on accepted engineering principles.

Such designs are outside the scope of this Standard as a Verification Method demonstrating compliance with the New Zealand Building Code and must be treated as alternative solutions under the Building Act 2004.

Amd 2 Oct. '07

1.5.2 For existing structures

Where the strength or serviceability of an existing structure is to be evaluated, the general principles of this Standard may be applied. The actual properties of the materials in the structure shall be used.

1.6 DESIGN AND CONSTRUCTION REVIEW

1.6.1 Design

The design of a structure or the part of a structure to which this standard is applied shall be the responsibility of the design engineer or his/her representative.

1.6.2 Design data and details

The drawings or specification, or both, for steel members and structures shall include, where relevant, the following:

(a) The corrosion protection requirements, if applicable;

(b) The steel grades used;

(c) The size and designation of each member;

(d) The sizes, types and categories of welds used in the connections, together with the level of NDE required;

(e) The sizes and categories of the bolts used in the connections;

(f) The sizes of the connection components;

(g) The locations and details of planned joints, connections and splices;

(h) Any constraint on construction assumed in the design;

(j) The camber of any members;

Amd 2 Oct. '07

(k) The fire protection requirements, if applicable.

NOTE –

Recommendations on matters to be considered in the contract documents relating to fabrication of steelwork are given in C14.6, Part 2 of this Standard; and relating to erection of steelwork in C15.7, Part 2 of this Standard.

1.6.3 Construction review

1.6.3.1
All stages of construction of a structure or the part of a structure to which this Standard is applied shall be adequately reviewed by a person who, on the basis of experience or qualifications, is competent to undertake the review. (This person is termed the construction reviewer).

1.6.3.2
The extent of review to be undertaken shall be nominated by the design engineer, taking into account those materials and workmanship factors which are likely to influence the ability of the finished construction to perform in the predicted manner.

NOTE –

Amd 2 Oct. '07

(1) A construction reviewer might be a Chartered Professional Engineer with suitable experience.

(2) Welding supervisors shall be qualified in accordance with clause 4.11.1 of AS/NZS 1554.1 or clause 4.11.1 of AS/NZS 1554.5, whichever is appropriate.

(3) Painting inspectors should hold Certified Coatings Inspector qualification from the Certification Board for Inspection Personnel or an equivalent qualification acceptable to the CBIP.

(4) Clause 4.1 of either AS/NZS 1554.1 or AS/NZS 1554.5 requires establishment of welding procedure sheets by the fabricator. Prior to commencement of welding, these shall be approved for the particular job by the design engineer, or by the construction reviewer, or by his/her nominated representative.

NOTES

2 MATERIALS AND BRITTLE FRACTURE

2.1 YIELD STRESS AND TENSILE STRENGTH USED IN DESIGN

2.1.1 Yield stress

The yield stress (f_y) used in design shall not exceed the specified minimum yield stress as given by the appropriate standard for material supply in accordance with 2.2.

2.1.2 Tensile strength

The tensile strength (f_u) used in design shall not exceed the specified minimum tensile strength as given by the appropriate standard for material supply in accordance with 2.2.

2.2 STRUCTURAL STEEL

2.2.1 Specification

All structural steel coming within the scope of these clauses shall, before fabrication, comply with the requirements (a), (b), (c) or (d) below. In addition, selection of material shall comply with 2.6.

Amd 2 Oct. '07

(a) Australian or Joint Australian/New Zealand Standards:

AS 1163	Structural steel hollow sections
AS 1594	Hot-rolled steel flat products
AS/NZS 3678	Structural steel – Hot-rolled plates, floorplates and slabs
AS/NZS 3679	Structural steel
Part 1	Hot-rolled bars and sections
Part 2	Welded I-sections

Amd 2 Oct. '07

(b) British Standards:

BS 4	Structural steel sections
Part 1	Specification for hot-rolled sections
BS 4848	Hot-rolled structural steel sections
Part 2	Specification for hot-finished hollow sections
Part 4	Equal and unequal angles
BS 7668	Weldable structural steels. Hot finished structural hollow sections in weather resistant steels. Specification
BS EN 10025	Hot rolled products of structural steels.
Part 1	General delivery conditions
Part 2	Technical delivery conditions for non-alloy structural steels.
Part 3	Technical delivery conditions for long products
Part 4	Technical delivery conditions for the thermomechanical rolled weldable fine grain steels
Part 5	Technical delivery conditions for structural steels with improved atmospheric corrosion resistance
Part 6	Technical delivery conditions for plates and wide flats of high yield strength structural steels in the quenched and tempered condition
BS EN 10029	Specification for tolerances on dimensions, shape and mass for hot rolled steel plates 3 mm thick or above

Amd 2 Oct. '07

BS EN 10210	Hot finished structural hollow sections of non-alloy and fine grain structural steels
Part 1	Technical delivery requirements

BS EN 10219	Cold formed welded structural hollow sections of non-alloy and fine grain steels
Part 2	Tolerances, dimensions and sectional properties

Amd 2 Oct. '07

(c) Japanese Standards:

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JIS G 3101	Rolled steel for general structure
JIS G 3106	Rolled steels for welded structure
JIS G 3114	Hot-rolled atmospheric corrosion resisting steels for welded structure
JIS G 3132	Hot-rolled carbon steel strip for pipes and tubes
JIS G 3136	Rolled steel for building structure
JIS G 3141	Cold reduced carbon steel sheets and strip
JIS G 3192	Dimensions, mass and permissible variations of hot rolled steel sections
JIS G 3193	Dimensions, mass and permissible variations of hot rolled steel plates, sheets and strip

Amd 2 Oct. '07

Amd 2 Oct. '07

(d) If structural steels or shapes other than those referred to in (a), (b) and (c) above are used, they shall comply with a Standard approved by the design engineer. Such an approval is outside the scope of this Standard as a Verification Method for the NZ Building Code.

Amd 2 Oct. '07

2.2.2 Acceptance of steels

Certified mill test reports, or test certificates issued by the mill, shall constitute sufficient evidence of compliance with the material supply standards referred to in this Standard.

2.2.3 Unidentified steel

If unidentified steel is used, it shall be free from surface imperfections, and shall be used only where the design engineer has determined that particular physical properties of the steel and its weldability will not adversely affect the strength and serviceability of the structure. Unless a full test in accordance with BS EN 10002-1 is made, the yield stress of the steel used in design (f_y) shall be taken as 170 MPa, and the tensile strength used in design (f_u) shall be taken as 300 MPa.

Amd 1 June '01

Unidentified steel shall not be used as elements in the seismic-resisting system. However when evaluating the performance of existing structures, unidentified steel may be used if shown by tests to meet the material requirements of 12.4.

Unidentified steel shall not be used in members of an associated structural system which are subject to inelastic demand (see clause 12.3.4.2) or in members which are subject to moment redistribution, ***unless*** it is shown by tests that the steel complies with the elongation requirements of 4.6.2(b)(iii).

Amd 2 Oct. '07

2.3 FASTENERS

2.3.1 Steel bolts, nuts and washers

Steel bolts, nuts and washers shall comply with the following Standards, as appropriate:

AS 1110	ISO metric hexagon bolts and screws – Product grades A and B
AS 1111	ISO metric hexagon bolts and screws – Product grade C
AS 1112:Part 1	ISO metric hexagon nuts – Style 1 – Product grades A and B
AS/NZS 1252	High strength steel bolts with associated nuts and washers for structural engineering
AS 1559	Fasteners – Bolts, nuts and washers for tower construction

Amd 2 Oct. '07

2.3.2 Equivalent high strength fasteners

The use of other high strength fasteners having special features in lieu of bolts to AS/NZS 1252 shall be permitted, provided that the design engineer provides evidence of their design equivalence to high strength bolts that comply with AS/NZS 1252 and are installed in accordance with this Standard.

Equivalent fasteners shall meet the following requirements:

(a) The mechanical properties of equivalent fasteners shall comply with AS/NZS 1252 for the relevant bolt, nut and washer components.

(b) The body diameter, head or nut bearing areas, or their equivalents, of equivalent fasteners shall not be less than those provided by a bolt and nut complying with AS/NZS 1252 of the same nominal dimensions. Equivalent fasteners may differ in other dimensions from those specified in AS/NZS 1252.

(c) The method of tensioning and the inspection procedure for equivalent fasteners may differ in detail from those specified in 15.2.5 and 15.4 respectively, provided that the minimum fastener tension is not less than the minimum bolt tension specified in table 15.2.5.1 and that the tensioning procedure is able to be checked.

2.3.3 Welds

All welding consumables and deposited weld metal shall comply with AS/NZS 1554.1, except that where required by 10.1.5, they shall comply with AS/NZS 1554.5.

2.3.4 Welded studs

All welded studs shall comply with, and shall be installed in accordance with, AS 1554.2.

2.3.5 Explosive fasteners

All explosive fasteners shall comply with, and shall be installed in accordance with, AS/NZS 1873.

2.3.6 Anchor bolts

Anchor bolts shall comply with either the bolt Standards of 2.3.1 or shall be manufactured from rods complying with the steel Standards of 2.2.1, provided that the threads comply with AS 1275.

2.4 STEEL CASTINGS

All steel castings shall comply with AS 2074.

2.5 CONCRETE

Amd 2 Oct. '07

Unless otherwise required by this Standard, all structural and fire protective concrete used in association with structural steel shall comply with NZS 3104 and NZS 3109.

2.6 MATERIAL SELECTION TO SUPPRESS BRITTLE FRACTURE

2.6.1 Methods

The steel grade shall be selected either by the notch-ductile range method as specified in 2.6.2, or by using a fracture assessment carried out as specified in 2.6.5.

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For steels used in seismic-resisting systems refer to table 12.4.

2.6.2 Notch-ductile range method

2.6.2.1

The steel shall be selected to operate in its notch-ductile temperature range.

2.6.2.2

The design service temperature for the steel shall be determined in accordance with 2.6.3. The appropriate steel type suitable for the design service temperature and material thickness shall be selected in accordance with 2.6.4.1, 2.6.4.2 and 2.6.4.3.

2.6.2.3
The steel grade shall be selected to match the required steel type in accordance with 2.6.4.4.

2.6.2.4
The welding consumables shall be selected in accordance with 2.6.4.5.

2.6.2.5
The bolts shall be selected in accordance with 2.6.4.6.

2.6.3 Design service temperature

2.6.3.1 Basic design temperature

2.6.3.1.1
The design service temperature shall be the estimated lowest metal temperature to be encountered in service or during erection, or testing, as determined by 2.6.3.1.2, 2.6.3.1.3 and 2.6.3.2.

2.6.3.1.2
The basic design temperature shall be the lowest one-day mean ambient temperature (LODMAT). LODMAT isotherms for New Zealand are given in figure 2.6.3.1.

2.6.3.1.3
The design service temperature shall be taken as the basic design temperature, except as modified by 2.6.3.2.

2.6.3.2 Modifications to the basic design temperature
The selection of the design service temperature shall also make allowance for the following conditions:

(a) For structures which may be subjected to especially low ambient temperatures, such as exposed bridges over inland rivers or structures located in alpine regions, a design service temperature 5 °C less than the basic design temperature shall be used.

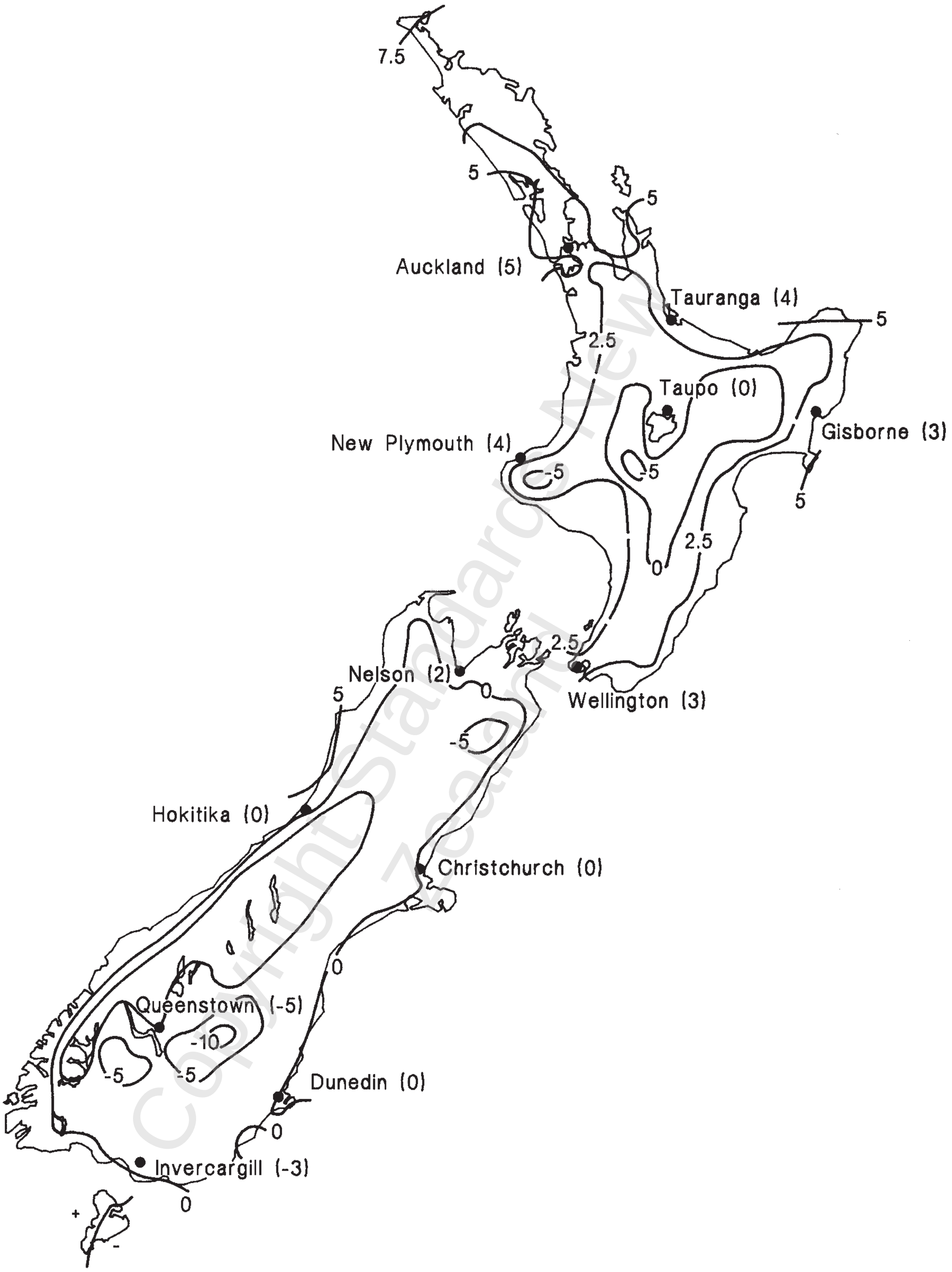
(b) For critical structures, records from the National Institute of Water and Atmosphere Research Ltd shall be consulted to ascertain whether abnormally low local ambient temperatures may occur at the particular site for sufficient time to further lower the design service temperature.

(c) For structures which are subject to artificial cooling, such as the structures of refrigerated buildings, the minimum expected metal temperature for the part concerned shall be used as the design service temperature.

NOTE – In special cases, metal temperatures lower than the LODMAT may occur when there is minimum insulation, minimum heat capacity and radiation shielding, and where abnormally low local temperatures may occur.

2.6.4 Material selection

2.6.4.1 Selection of steel type
The steel type for the material thickness shall be selected from table 2.6.4.1 so that the permissible service temperature listed in table 2.6.4.1 is lower than the design service temperature determined in accordance with 2.6.3. The permissible service temperatures listed in table 2.6.4.1 shall be subjected to the limitations and modifications specified in 2.6.4.2 and 2.6.4.3 respectively.



(1) The isotherms show the lowest one day mean ambient temperature in degrees Celsius (°C).
(2) Based on records from 1930 to 1990 supplied by the National Institute of Water and Atmosphere Research Ltd.
(3) Where site-specific LODMAT temperatures are available, these shall be used in lieu of temperatures from figure 2.6.3.1.

Figure 2.6.3.1 – LODMAT isotherms

2.6.4.2 Limitations

2.6.4.2.1
Table 2.6.4.1 shall only be used without modification for members and components which comply with the fabrication and erection provisions of sections 14 and 15, and with the provisions of AS/NZS 1554.1 or AS/NZS 1554.5 as appropriate.

2.6.4.2.2
Table 2.6.4.1 may be used without modification for welded members and connection components which are not subject to more than 1.0 % outer bend fibre strain during fabrication. Members and components subject to greater outer bend fibre strains shall be assessed using the provisions of 2.6.4.3.

NOTE – Local strain due to weld distortion shall be disregarded.

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Table 2.6.4.1 – Permissible service temperatures according to steel type and thickness

<table>
<tr><th rowspan="3">Steel type (see table 2.6.4.4)</th><th colspan="6">Permissible service temperatures (°C)</th></tr>
<tr><th colspan="6">Thickness (mm)</th></tr>
<tr><th colspan="6">0 5 10 20 30 40 50 ≥70</th></tr>
<tr><td>1</td><td>–20</td><td>–10</td><td colspan="3">0</td><td>+5</td></tr>
<tr><td>2</td><td>–30</td><td>–20</td><td colspan="2">–10</td><td colspan="2">0</td></tr>
<tr><td>3</td><td>–40</td><td>–30</td><td>–20</td><td colspan="3">–10</td></tr>
<tr><td>4</td><td>–10</td><td>0</td><td>0</td><td colspan="2">0</td><td>+5</td></tr>
<tr><td>5</td><td>–30</td><td>–20</td><td>–10</td><td colspan="3">0</td></tr>
<tr><td>6</td><td>–40</td><td>–30</td><td>–20</td><td colspan="3">–10</td></tr>
<tr><td>7A</td><td>–10</td><td colspan="5">0</td></tr>
<tr><td>7B</td><td>–30</td><td>–20</td><td>–10</td><td colspan="3">0</td></tr>
</table>

NOTE – Table 2.6.4.1 applies for:

(a) Elements of a member or connection component subject to tensile stress; and

(b) Plates and sections, the flange thickness being used for the latter.

2.6.4.3 Modification for certain applications

2.6.4.3.1 *Steel subject to greater than 1.0 % strain (non-seismic applications)*
Where a member or component is subjected to more than 1.0 % but less than 10.0 % outer bend fibre strain during fabrication, the permissible service temperatures for each steel type shall be increased by 20 °C above the value given in table 2.6.4.1.

NOTE – Local strain due to weld distortion shall be disregarded.

2.6.4.3.2 *Steel used in category 1, 2 or 3 members (seismic applications)*
For category 1 or 2 members, as selected in accordance with 12.2, the permissible service temperature for each steel type shall be increased by 10 °C above the value given in table 2.6.4.1. For category 3 members, table 2.6.4.1 shall be used without modification.

2.6.4.3.3 *Steel subject to greater than 10.0 % strain*
Where a member or component is subject to more than 10.0 % outer bend fibre strain during fabrication, the permissible service temperatures for each steel type shall be increased above the value given in table 2.6.4.1 by 20 °C plus 1 °C for every 1.0 % increase in outer bend fibre strain above 10 %.

NOTE – Local strain due to weld distortion shall be disregarded.

2.6.4.3.4 *Post weld heat treated members*
For members or components which have been welded or strained and which are subject to post weld heat treatment in excess of 500 °C but not exceeding 620 °C, no modification shall be made to the permissible service temperature given in table 2.6.4.1.

NOTE – Guidance on appropriate post weld heat treatment may be found in AS 1210.

2.6.4.4 Selection of steel grade
The steel grade shall be selected to match the required steel type given in table 2.6.4.4.

2.6.4.5 Selection of welding consumables

2.6.4.5.1
Welding consumable selection shall be in accordance with AS/NZS 1554.1 or AS/NZS 1554.5.

2.6.4.5.2
For welds subject to earthquake loads or effects (see 9.7.1.4.3(b)), the following shall apply:

(a) The welding consumables shall have a Ships Classification Societies Grade 3 approval as shown in table 4.6.1(A) of AS/NZS 1554.1:2004, as required for Steel Type 3 for Grade 300 steel, Steel Type 6 for Grade 350 steel and Steel Type 7C for Grade 450 steel.

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(b) The heat input in the deposited weld metal shall not exceed 2.5 kJ/mm run.

2.6.4.6 Selection of bolts
The impact resistance of a bolt must be not less than that for the grade of steel being joined.

2.6.5 Fracture assessment
A fracture assessment, if chosen under 2.6.1, shall be made using a fracture mechanics analysis coupled with fracture toughness measurements and non-destructive testing of the steel grade selected.

NOTE – For methods of fracture assessment, see BS 7910 Guide to methods for assessing the acceptability of flaws in metallic structures.

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An assessment made under this clause is outside the scope of this Standard as a Verification Method for the NZ Building Code.

Table 2.6.4.4 – Steel type relationship to steel grade
(For steels from 2.2.1)

Steel Type	Steel Grade					
	AS 1163	AS 1594	AS/NZS 3678 AS/NZS 3679.2	AS 3679.1	BS EN 10025	JIS G 3106 JIS G 3136
1	C250	HA200 HA250, HU250 HA300, HU300 HA300/1	200 250 300	250 300	S275 S275JR	SM 400A SN 400A
2	C250L0	–	–	250L0 300L0	S275JO	SM 400B SN 400B
3	–	–	250L15 300L15	250L15 300L15	S275J2G3/ S275J2G4	SM 400C
4	C350	HA350 HA400	350 WR350	350	S355 S355JR	SM 490YA
5	C350L0	–	WR350L0	350L0	S355JO	SM 490YB SM 520B SN 490B
6	–	–	350L15 400L15	350L15	S355J2G3/ S355J2G4	SM 520C
7A	C450	–	–	–	–	–
7B	C450L0	–	–	–	–	–
7C	–	–	450L15	–	–	–

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3 GENERAL DESIGN REQUIREMENTS

3.1 DESIGN

3.1.1 Aim

3.1.1.1
The aim of structural design is to provide a structure which has adequate stability and strength, is serviceable and durable.

3.1.1.2
A structure is stable if the probabilities of overturning, tilting or sliding throughout its intended life are acceptably low.

3.1.1.3
A structure has adequate strength and is serviceable if the probabilities of structural failure and of loss of serviceability throughout its intended life are acceptably low.

3.1.1.4
A structure is durable if it withstands the expected wear and deterioration throughout its intended life without the need for undue maintenance.

3.1.2 Requirements

The structure and its component members and connections shall satisfy the design requirements for stability, ductility, strength, serviceability, brittle fracture, fatigue, fire and earthquake in accordance with the procedures given in this Standard, as appropriate.

3.2 LOADS AND OTHER EFFECTS

3.2.1 Loads

The design of a structure for the ultimate and serviceability limit state shall account for the actions directly arising from the following loads:

(a) Dead, live, wind, earthquake, snow, rain, ice, soil and hydrostatic loads specified in the Loadings Standard;

Amd 2 Oct. '07 (b) For the design of cranes, any additional or alternative relevant loads specified in AS 1418 as appropriate;

Amd 2 Oct. '07 (c) For the design of fixed platforms, walkways, stairways and ladders, any additional or alternative relevant loads specified in NZS/AS 1657;

Amd 2 Oct. '07 (d) For the design of lifts, any relevant loads specified in NZS 4332;

(e) Other specific loads, as required.

NOTE –

Amd 2 Oct. '07 (1) For the design of bridges, loads specified in the Bridge Manual (SP/M/022) Second Edition, plus Amendments, Transit New Zealand (for road bridges) or in the New Zealand Rail Ltd : Railnet Code, Part 4, Code Supplements Bridges and Structures, Section 2 : Design (for rail bridges), as applicable, should be used.

(2) For multi-storey building structures and vertical cantilevers, see also 3.2.4.

(3) Only loads determined from (a) shall be used when this Standard is used as a Verification Method for Clause B1 of the New Zealand Building Code.

3.2.2 Other loads or effects

Any load or effect which may significantly affect the strength or serviceability of the structure, including the following, shall be taken into account:

(a) Foundation movements;

(b) Temperature changes and gradients;

(c) Axial shortening, both elastic and inelastic (under severe seismic forces);

(d) Dynamic effects, other than as already covered in 3.2.1;

(e) Construction loading.

3.2.3 Design load combinations

The design load combinations for the ultimate and serviceability limit state shall be those specified in the Loadings Standard.

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NOTE – For the design of bridges, load combinations specified in the Transit New Zealand document Bridge Manual (SP/M/022) Second Edition, plus Amendments, or in the Design Section of the Railnet Code, Part 4, as applicable, should be used.

3.2.4 Structural robustness

3.2.4.1 Minimum lateral resistance of the completed structure

Completed structures shall meet the minimum lateral resistance requirements of AS/NZS 1170.0 Clause 6.2.2.

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3.2.4.2 Construction stage

Structural robustness during construction shall comply with the requirements of AS 3828.

3.3 ULTIMATE LIMIT STATE

The structure and its component members shall be designed for the ultimate limit state as follows:

(a) The loads and effects shall be determined in accordance with 3.2.1 and 3.2.2, and the ultimate limit state design loads and effects shall be determined in accordance with 3.2.3 and 3.2.4.

(b) The design actions (S^*) resulting from the ultimate limit state design loads shall be determined by structural analysis in accordance with section 4 and, where appropriate, section 12.3.

(c) The design capacity (ϕR_u) shall be determined from the nominal capacity (R_u) determined from sections 5 to 9, 12 and 13 as appropriate, where the strength reduction factor (ϕ) shall not exceed the appropriate value given in table 3.3 or table 13.1.2 or clause 11.5.

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(d) All members and connections shall be proportioned so that the design capacity (ϕR_u) is not less than the design action (S^*), i.e.

$$S^* \leq \phi R_u$$

(e) For structures that are required to respond inelastically under severe earthquake loads, the level of ductility demand on the structure and parts thereof shall be determined from 12.2 and provided for in accordance with the design and detailing procedures given in section 12 of this Standard.

(f) The structure as a whole (and any part of it) shall be designed to prevent instability due to overturning, uplift or sliding in accordance with the Loadings Standard.

3.4 SERVICEABILITY LIMIT STATE

3.4.1 General

The structure and its components shall be designed for the serviceability limit state by controlling or limiting deflection, vibration, bolt slip and corrosion, as appropriate, in accordance with the relevant requirements of 3.4.2 to 3.4.6.

Table 3.3 (1) – Ultimate limit state strength reduction factors (ϕ) for bare steel members and connections

Design capacity at ultimate limit state for	Section or clause	Strength reduction factor (ϕ)	
Member subject to bending and shear	5	0.90	
Member subject to axial compression	6	0.90	
Member subject to axial tension	7	0.90	
Member subject to combined actions	8	0.90	
Connection component other than a bolt, pin or weld	9.1.9	0.90	
Bolted connection			
– bolt in shear	9.3.2.1	0.80	
– bolt in tension	9.3.2.2	0.80	
– bolt subject to combined shear and tension	9.3.2.3	0.80	
– ply in bearing	9.3.2.4	0.90	
– bolt group	9.4	0.80	
Pin connection			
– pin in shear	9.5.1	0.80	
– pin in bearing	9.5.2	0.80	
– pin in bending	9.5.3	0.80	
– ply in bearing	9.5.4	0.90	
		SP Category	**GP Category**
Welded connection			
– complete penetration butt weld	9.7.2.7	0.90	0.60
– longitudinal fillet weld in RHS ($t < 3$ mm)	9.7.3.10	0.70	–
– other fillet weld and incomplete penetration butt weld	9.7.3.10	0.80	0.60
– plug or slot weld	9.7.4	0.80	0.60
– weld group	9.8	0.80	0.60

Table 3.3 (2) – Serviceability limit state strength reduction factors (ϕ) for bare steel connections

Design capacity at serviceability limit state for	Clause	Strength reduction factor (ϕ)
Bolted connection		
– friction-type connection	9.3.3	0.7

3.4.2 Method

The structure and its components shall be designed for the serviceability limit state as follows:

(a) The loads and other effects shall be determined in accordance with 3.2.1 and 3.2.2, and the serviceability limit state design loads shall be determined from 3.2.3.

(b) Deflections due to the serviceability limit state design loads shall be determined by the first-order elastic analysis method of 4.4, with all amplification factors taken as unity (including the factor C_S from 12.12.3.3), except where limited moment redistribution is allowed by 4.5.5. Deflections shall comply with 3.4.3. Section properties shall comply with Appendix N.

(c) Vibration behaviour shall be assessed in accordance with 3.4.4.

(d) Bolt slip shall be limited, where required, in accordance with 3.4.5.

(e) Corrosion protection shall be provided in accordance with 3.4.6.

3.4.3 Deflection limits

The deflection limits for the serviceability limit state shall be appropriate to the structure and its intended use, the nature of the loading, and the elements supported by it.

3.4.4 Vibration of beams

3.4.4.1

Beams which support floors or machinery shall be checked to ensure that the vibrations induced by machinery, or vehicular or pedestrian traffic do not adversely affect the serviceability of the structure.

3.4.4.2

Where there is a likelihood of a structure being subjected to vibration from causes such as wind forces or machinery, appropriate measures shall be taken to prevent discomfort or alarm, damage to the structure, or interference with its proper function.

3.4.5 Bolt serviceability limit state

3.4.5.1

In a connection, where slip under the serviceability design loads must be avoided, the fasteners shall be selected in accordance with 9.1.6.

3.4.5.2

For a friction-type connection which is subject to shear force in the plane of the interfaces, and for which slip under serviceability loads must be avoided, the strength reduction factor (ϕ) shall be taken from table 3.3(2) and the bolts shall be designed in accordance with 9.3.3.

3.4.6 Corrosion protection

Where steelwork in a structure is to be exposed to a corrosive environment, appropriate corrosion protection shall be given to the steelwork. The degree of protection to be employed shall be determined after consideration has been given to the use of the structure, its intended life, its maintenance, and the climatic or other local conditions.

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NOTE – Protection against corrosion shall be to Appendix C. These provisions provide a means of compliance with the performance requirements of the New Zealand Building Code Clause B2: Durability, although this is not cited as a Verification Method for Clause B2 of the NZBC.

3.5 ULTIMATE AND SERVICEABILITY LIMIT STATES BY LOAD TESTING

Not withstanding the requirements of clause 3.3 or 3.4, a structure or a component member or connection may be designed for the ultimate or serviceability limit state, or both, by load-testing in accordance with section 17. If this alternative procedure is adopted, the requirements of 3.6 to 3.9, as appropriate, shall also apply.

3.6 BRITTLE FRACTURE

In order to avoid failure by brittle fracture, the selection of the parent material shall be made in accordance with 2.6.

3.7 FATIGUE

3.7.1

For structures and structural elements subject to loadings which could lead to fatigue, the fatigue strength shall be determined in accordance with section 10.

Amd 2 Oct. '07 *(Text deleted)*

3.8 FIRE

The structure, its component members and connections shall be designed in accordance with section 11.

3.9 EARTHQUAKE

The structure and its component members shall be designed in accordance with section 12.

4 STRUCTURAL ANALYSIS

4.1 METHODS OF STRUCTURAL ANALYSIS

4.1.1 General

The design actions in a structure and its members and connections shall be determined by structural analysis using the provisions of 4.2 and 4.3 and one of the following methods (a) to (c):

(a) Elastic analysis, in accordance with 4.4; or

(b) Elastic analysis with moment or shear redistribution, in accordance with 4.5; or

(c) Plastic analysis, in accordance with 4.6.

4.1.2 Braced or sway members

Members shall be defined as either braced or sway (see 1.3) for the application of this section. Members forming part of a seismic-resisting system which is being designed for load combinations that include earthquake loads shall be considered as sway members, except for members within a triangulated structure which forms part of the seismic-resisting system. Such members shall be considered as braced members, while the triangulated structure as a whole shall be considered as a sway member.

4.2 STRUCTURAL FORM

4.2.1 General

4.2.1.1

Modelling of the structure shall account for flexibility of its components, including its members, connections and foundations.

4.2.1.2

In determining the distribution of design actions on the members and connections of the structure, one or a combination of the following forms of construction shall be used, subject to the restrictions of 4.2.4 for structures used as seismic-resisting systems:

(a) Rigid;

(b) Semi-rigid;

(c) Simple.

4.2.2 Forms of construction

4.2.2.1 Rigid construction

For rigid construction, the connections shall be assumed to hold the original angles between the members effectively unchanged up to attainment of the nominal capacity of the weakest member.

4.2.2.2 Semi-rigid construction

4.2.2.2.1

For semi-rigid construction, the connections need not possess sufficient rigidity to hold the original angles between the members effectively unchanged up to attainment of the nominal capacity of the weakest member, but shall be required to furnish a dependable and known degree of flexural restraint under the design actions.

4.2.2.2.2
The relationship between the degree of flexural restraint and level of member action shall be established by a method of rational analysis which has been confirmed by test data.

4.2.2.3 Simple construction
For simple construction, the connections at the ends of members shall be assumed not to develop bending moments under the design actions, except as specified in 4.3.4.

4.2.3 Design of connections
The design of all connections shall be consistent with the form of construction, and the behaviour of the connections shall not adversely affect any other part of the structure beyond what is allowed for in design. Connections shall be designed in accordance with section 9 and with the appropriate provisions of sections 12 and 13.

4.2.4 Applicability of forms of construction for use in seismic-resisting systems

4.2.4.1
In a braced seismic-resisting system any of the three forms of construction from 4.2.2 may be used.

4.2.4.2
In a moment-resisting framed seismic-resisting system, either rigid or semi-rigid construction shall be used.

4.2.4.3
If semi-rigid construction is used in 4.2.4.2, the design engineer shall verify by special study that the behaviour of the seismic-resisting system meets the requirements of section 12 for the category of system as determined from 12.2.3.

4.3 ASSUMPTIONS AND APPROXIMATIONS FOR ANALYSIS

4.3.1 General

4.3.1.1
The structure shall be analysed in its entirety, or as a series of substructures, in accordance with the provisions of the Loadings Standard.

4.3.1.2
Regular building structures may be analysed as a series of parallel two-dimensional frames, the analysis being carried out in each of the two directions at right angles, except where there is significant load redistribution between the frames. For seismic and wind loading, the design actions on each element of the lateral load-resisting system shall be determined from the response of the structure as a whole to the applied loads.

4.3.1.3 Design actions in beams and columns of a braced multi-storey building under vertical loading

4.3.1.3.1
In order to determine the design actions in the floor beams, each level of beams thereof, together with the columns immediately above and below, may be analysed separately as a substructure. The columns shall be assumed fixed at the ends remote from the level under consideration.

4.3.1.3.2
Where floor beams in a multi-storey building structure are analysed separately as substructures, the bending moment at a support may be determined from the assumption that the floor is fixed at the support one span away, provided that the floor beam continues beyond that point.

4.3.1.3.3
Design actions in the columns shall be determined from the most adverse distribution of live loading to 4.3.3.1 on the beams framing into each end of the column.

4.3.2 Span length

The span length of a flexural member shall be taken as the distance centre-to-centre of the supports.

4.3.3 Arrangements of live loads for buildings

Amd 2 Oct. '07 Arrangement of live (imposed) loads shall comply with AS/NZS 1170.1 clause 3.3.

4.3.4 Simple construction

4.3.4.1
Bending members may be assumed to have their ends connected for shear only and to be free to rotate. In triangulated structures, axial forces may be determined by assuming that all members are pin connected.

4.3.4.2
A beam reaction or a similar load on a column shall be taken as acting at a minimum distance of 100 mm from the face of the column towards the span or at the centre of bearing, whichever gives the greater eccentricity, except that for a column cap, the load shall be taken as acting at the face of the column, or edge of packing if used, towards the span.

4.3.4.3
For a continuous column, the design bending moment (M^*) due to eccentricity of loading at any one floor or horizontal frame level shall be taken as:

(a) Divided between the column lengths above and below that floor or frame level in proportion to the value of I/L of the column lengths; and

(b) Having no effect at the floor or frame levels above and below that floor.

4.3.4.4
The design bending moment (M^*) calculated in accordance with 4.3.4 shall be used directly in design of members to section 8.

4.4 ELASTIC ANALYSIS

4.4.1 General requirement

Individual members shall be assumed to remain elastic for the purposes of analysis in accordance with the Loadings Standard. This shall apply to both the ultimate and serviceability limit states.

4.4.2 Second-order effects

4.4.2.1 General

Unless one of the criteria in 4.4.2.2 is satisfied, the analysis shall allow for the effects of the design loads acting on the structure and its members in their displaced and deformed configuration. These second-order effects shall be taken into account by using either:

(a) A second-order elastic analysis in accordance with Appendix E;

(b) A first-order elastic analysis with moment amplification in accordance with 4.4.3, provided that for members in statically indeterminate structures the moment amplification factors (δ_b) and (δ_s) are within the following limits:

(i) $\delta_b \le 1.4$ when calculated in accordance with 4.4.3.2.

(ii) $\delta_s \le 1.2$ when calculated in accordance with 4.4.3.3.

4.4.2.2 Exclusions from second-order effect consideration

4.4.2.2.1 *General exclusion*

Second-order effects may be neglected for any frame where the elastic buckling load factor (λ_c) of the frame, as determined in accordance with 4.9, is greater than 10.

NOTE –

(1) The value of λ_c depends on the braced/sway status of the members comprising the frame (see 4.1.2) and on the load set (see 1.3).

(2) The calculation of λ_c must account for both $P - \Delta$ and $P - \delta$ effects; see Appendix E.

4.4.2.2.2 *Specific exclusions*

Second-order effects do not need to be allowed for in analysis and design of the following structural systems when these are being designed for load combinations which do not include earthquake loads:

(a) Moment-resisting frames of rigid rectangular construction in which the height, measured from the base, does not exceed the critical height (see 1.3) and the following are satisfied:

$$\sum_{1}^{n} (N_g^* / \phi N_c) / n \le 0.4 \text{ for each storey; and}$$

$$N_g^* / \phi N_c \le 0.7 \text{ for all columns}$$

where

N_g^* = design compression action generated by vertical loading alone (dead, live, snow loading)

N_c = nominal member compression capacity calculated for buckling in the plane of the moment-resisting frame using an effective length factor k_e = 1.0

n = total number of columns in a storey

ϕ = strength reduction factor from table 3.3

and the summation $\sum_{1}^{n}$ is for all columns in a storey.

(b) Concentrically braced framed, eccentrically braced framed or triangulated structural systems in which the height, measured from the base, does not exceed the critical height (see 1.3) and the gravity design compression action (N_g^*) in all members satisfies the following:

$$(N_g^*) \leq 0.15\,\phi N_c$$

where

N_c = nominal member compression capacity.

NOTE – The exclusion check in (a) applies only in the plane of the moment-resisting frame. Out-of-plane second-order effects on any column of the moment-resisting frame must be considered separately.

4.4.3 First-order elastic analysis

4.4.3.1 General

In a first-order elastic analysis, changes in the geometry are not accounted for, and changes in the effective stiffnesses of the members due to axial force are neglected. The effects of these on the first-order bending moments shall be allowed for where required by using one of the methods of moment amplification of 4.4.3.2 or 4.4.3.3, except that where the moment amplification factor (δ_b) or (δ_s), calculated in accordance with 4.4.3.2 or 4.4.3.3 as appropriate, is greater than the limiting values given in 4.4.2.1(b), a second-order elastic analysis in accordance with Appendix E shall be carried out.

The maximum calculated bending moment (M_m^*) shall be taken as the maximum bending moment along the length of a member obtained by superposition of the simple beam bending moments, resulting from any transverse loading on the member, with the end bending moments as determined by the analysis.

4.4.3.2 Moment amplification for a braced member

4.4.3.2.1
For a braced member with zero axial force or a braced member subject to axial tension, the design bending moment (M^*) shall be calculated as follows:

$$M^* = M_m^*$$

4.4.3.2.2
For a braced bare steel member with a design axial compressive force (N^*) as determined by the analysis, the design bending moment (M^*) shall be calculated as follows:

$$M^* = \delta_b M_m^*$$

where δ_b is a moment amplification factor for a braced member calculated as follows:

$$\delta_b = \frac{c_m}{1 - \left[\dfrac{N^*}{N_{omb}}\right]} \geq 1$$

and N_{omb} is the elastic buckling load, determined in accordance with 4.8.2, for the braced member buckling about the same axis as that about which the design bending moment (M^*) is applied.

4.4.3.2.3
For a braced bare steel member subject to end bending moments only, the factor c_m shall be calculated from equation 4.4.3.2(1):

$$c_m = 0.6 - 0.4\beta_m \quad \text{(Eq. 4.4.3.2(1))}$$

where β_m is the ratio of the smaller to the larger bending moment at the ends of the member, taken as positive when the member is bent in reverse curvature, except that for column members being designed for load combinations which include earthquake loads, $\beta_m = 0$.

4.4.3.2.4
For a braced bare steel member with transverse load (see 1.3) applied to it, the factor c_m shall be calculated from equation 4.4.3.2(2):

$$c_m = 0.6 - 0.4\beta_t \quad \text{(Eq. 4.4.3.2(2))}$$

where β_t is determined by one of the following methods:

(a) $\beta_t = -1.0$; or

(b) β_t is approximated by the value obtained by matching the distribution of bending moment along the member with one of the case number distributions of bending moment shown in figure 4.4.3.2; or

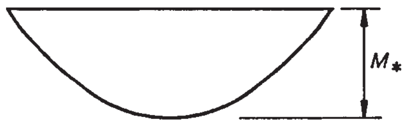
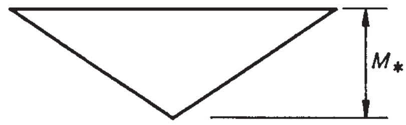
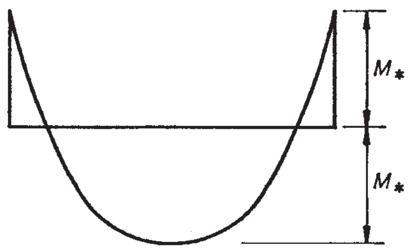
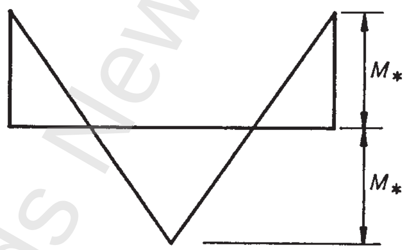
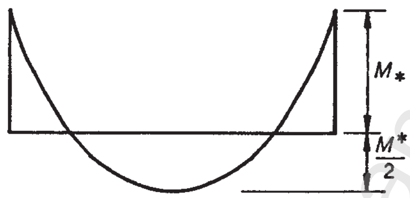
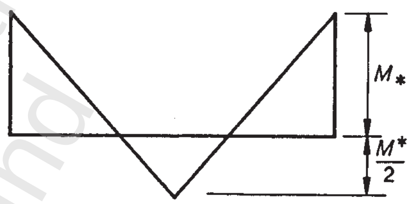
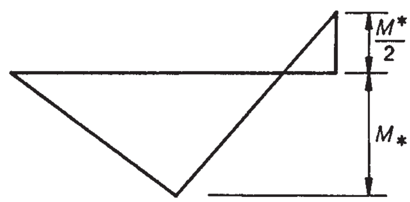
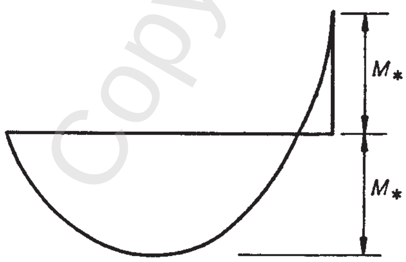
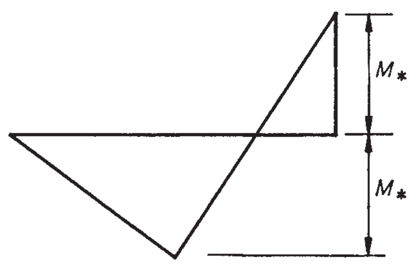
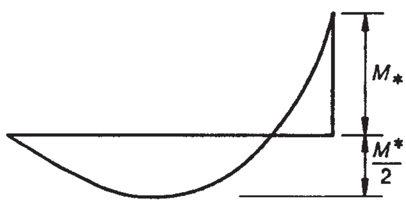
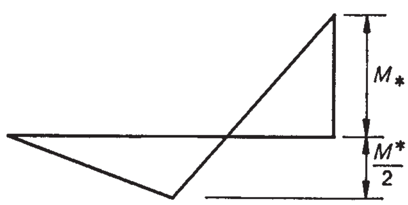
Case number	Moment distribution	β_t	Moment distribution	β_t	Case number
1		–1.0		–1.0	7
2		+0.2		+0.5	8
3		+0.6		+1.0	9
4		–0.5		–0.4	10
5		–0.2		0	11
6		+0.2		+0.5	12

Figure 4.4.3.2 (in part) – Values of β_t for various distributions of bending moment

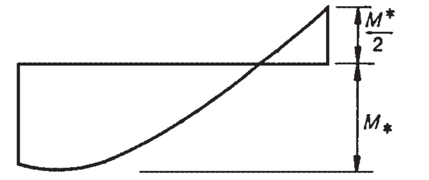
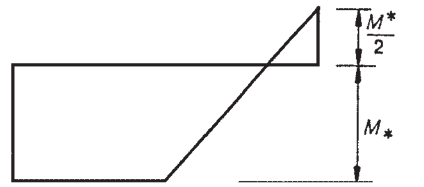
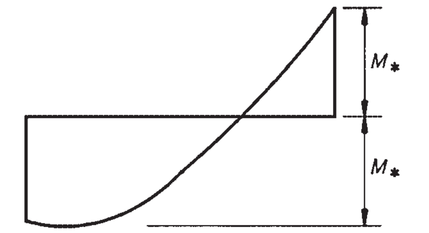
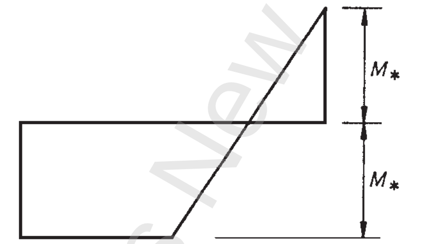
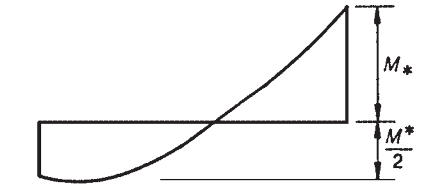
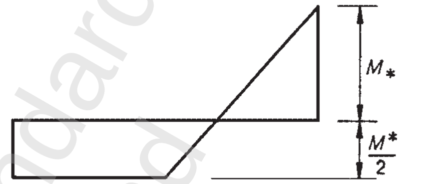
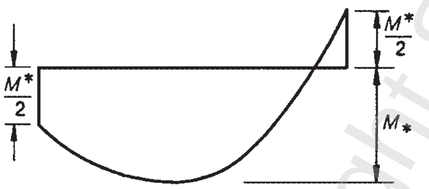
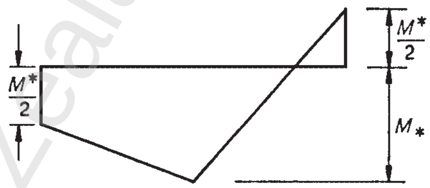
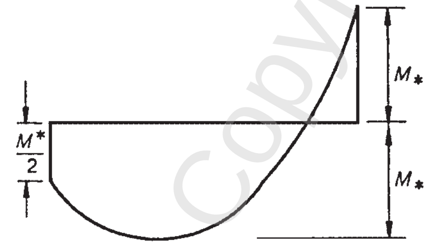
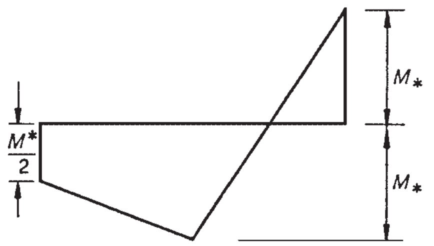
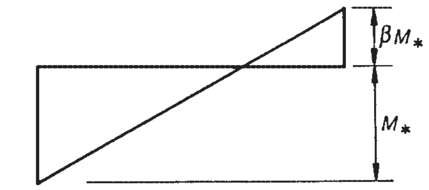
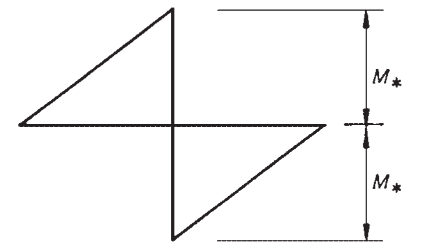
Case number	Moment distribution	β_t	Moment distribution	β_t	Case number
13		–0.4		–0.5	19
14		+0.1		–0.1	20
15		+0.7		+0.3	21
16		–0.5		–0.4	22
17		–0.2		–0.1	23
18		β		+1.0	24

Figure 4.4.3.2 (in part) – Values of β_t for various distributions of bending moment

(c) $\beta_t = 1 - \left[\dfrac{2\Delta_{ct}}{\Delta_{cw}}\right]$ with $-1.0 \le \beta_t \le 1.0$

where

Δ_{ct} = mid-span deflection of the member resulting from the transverse loading together with both end bending moments, if any, as determined by the analysis

Δ_{cw} = mid-span deflection of the member resulting from the transverse loading together with only those end bending moments which produce a mid-span deflection in the same direction as the transverse load.

4.4.3.2.5
For a braced composite column member refer to 13.8.2.2 or 13.8.3.4.

4.4.3.3 Moment amplification for a sway member

4.4.3.3.1
For a sway bare steel member, the design bending moment (M^*) shall be calculated using either the method given in 4.4.3.3.2, or the method given in Appendix F. For a sway composite column member, refer to 13.8.2.2 or 13.8.3.4.

The design bending moment (M^*) shall be calculated as follows:

$$M^* = \delta_m M_m^*$$

4.4.3.3.2
The moment amplification factor (δ_m) shall be taken as the greater of:

δ_b = the moment amplification factor for a braced member, determined in accordance with 4.4.3.2, and

δ_s = the moment amplification factor for a sway member, determined as follows:

(a) *Sway members in rectangular frames (see 1.3) being designed for load combinations which do not include earthquake loads* – For all sway columns in a storey of such a rectangular frame, the amplification factor (δ_s) shall be calculated using one of (i), (ii) or (iii) below:

(i) The relative sway displacement method, giving

$$\delta_s = \frac{0.95}{1 - \left[\dfrac{\Delta_s}{h_s} \dfrac{\sum N^*}{\sum V^*}\right]} \ge 1.0$$

where Δ_s is the translational displacement from elastic analysis of the top relative to the bottom in the storey of height (h_s), caused by the design horizontal storey shears (V^*) at the column ends, N^* is the design axial force in a column of the storey, and the summations include all the columns of the storey; or

(ii) The storey buckling load factor method, giving

$$\delta_s = \frac{0.95}{1 - \left[\dfrac{1}{\lambda_{ms}}\right]} \geq 1.0$$

where the elastic buckling load factor (λ_{ms}) for the storey under consideration is determined in accordance with 4.9.2.3.2.

(iii) The frame buckling load factor method, giving

$$\delta_s = \frac{0.95}{1 - \left[\dfrac{1}{\lambda_c}\right]} \geq 1.0$$

where the elastic buckling load factor (λ_c) is determined from a rational buckling analysis of the whole frame (see 4.9.2.3.3)

(b) *Sway members in non-rectangular frames (see 1.3) being designed for load combinations which do not include earthquake loads* – The amplification factor (δ_s) for each sway member shall be taken as the value for the frame calculated as follows:

$$\delta_s = \frac{0.95}{1 - \left[\dfrac{1}{\lambda_c}\right]} \geq 1.0$$

where the elastic buckling load factor (λ_c) is determined from a rational buckling analysis of the whole frame (see 4.9.2; typically 4.9.2.4).

(c) *Sway members in rectangular or non-rectangular frames (see 1.3) being designed for load combinations which include earthquake loads* – For all members in such a frame, any additional member actions generated by ($P - \Delta$) effects shall be determined in accordance with the Loadings Standard and the amplification factor (δ_s) shall be taken as 1.0.

4.5 ELASTIC ANALYSIS WITH MOMENT OR SHEAR REDISTRIBUTION

4.5.1 General requirements

Where applicable in accordance with 4.5.4, 4.5.5 or 4.5.6, the member capacity need not be matched to the actions determined from an elastic analysis.

4.5.2 Second-order effects

4.5.2.1

Second-order effects shall be considered in accordance with 4.4.2, except that the specific exclusions from second-order effect consideration given in 4.4.2.2.2 shall only apply to structural systems containing members in which moment redistribution does not exceed 20 % in any member.

4.5.2.2

Determination of second-order effects and resulting amplification of first-order design actions, where required by 4.5.2.1, shall be made subsequent to application of moment or shear redistribution and selection of member sizes.

4.5.2.3 Supression of snap-through instability
For portal frames with inclined rafters and with 3 or more bays per frame, the second-order effect of snap-through instability shall be suppressed by means of an appropriate design procedure.

4.5.3 Calculation of lateral deflection for determination of second-order effects in sway members

When second-order effects for sway members are calculated in accordance with the relative sway displacement method (4.4.3.3.2(a)(i)), the translational storey displacement Δ_s shall include increased deflection resulting from moment redistribution.

4.5.4 Moment redistribution in moment-resisting systems of rigid construction at the ultimate limit state

4.5.4.1 General application
The plastic hinge rotations (θ_p^*) in regions of members to which redistribution has been applied shall not exceed the limits given in 4.7.2 under the following ultimate limit state criterion:

(a) For design loads or effects not including earthquake loads, the application of the design loads or effects on the structural system;

(b) For design loads or effects including earthquake loads, the attainment of the total deflected shape of the structural system, calculated in accordance with the Loadings Standard. The total deflection shall include elastic plus inelastic components.

The category of member required for application of 4.5.7 shall be obtained by matching the design plastic hinge rotation with the appropriate limiting plastic hinge rotation from tables 4.7(1) to 4.7(4) and reading off the corresponding category of member required.

As an alternative to applying 4.5.4.1, for members subject to negligible axial force redistribution at the ultimate limit state may be applied in accordance with 4.5.4.2.

4.5.4.2 Alternative application

4.5.4.2.1
For members in which $N^* \leq 0.10\ \phi\ N_s$, redistribution of flexural actions from an elastic analysis may be carried out, with the maximum reduction in moment from an elastic analysis at any location along a member limited to that given in table 4.5.4.2.

Table 4.5.4.2 – Alternative application of moment redistribution

Member category (note 1)	Maximum percentage reduction in moment from elastic analysis
4	0
3	20
2	50
1	50

Note to table 4.5.4.2 – For application of 4.5.4.2, only the requirements of 12.4, 12.5 and 12.6 need to be satisfied for the given category of member.

4.5.4.2.2
The category of member required for application of 4.5.7 shall be obtained by matching the amount of moment redistribution applied with one of the limits in table 4.5.4.2 and selecting the appropriate category of member required.

4.5.4.3
The choice of member category required for moment redistribution is independent of the selection of member category for a seismic-resisting system or an associated structural system.

4.5.5 Moment redistribution in moment-resisting systems of rigid construction at the serviceability limit state

Redistribution of flexural actions from an elastic analysis may be carried out in accordance with the following provisions:

(a) Redistribution is only permitted for combinations of loads or effects including earthquake loads where departure from elastic behaviour is permitted for this limit state from the Loadings Standard.

(b) The maximum reduction in moment at any location along a member from an elastic analysis shall be 20 %. Such a member shall be classed as category 3 for application of 4.5.7, unless a more stringent classification applies from 12.3.3.

4.5.6 Shear or moment redistribution in eccentrically braced framed systems at the ultimate limit state

The forces obtained in the active links of an EBF under the design lateral loads may be redistributed in accordance with the following provisions:

(a) The amount of force (shear or moment as appropriate) that can be redistributed vertically between any two active links up an eccentrically braced bay shall not exceed 5 % of the maximum force derived for an active link in that bay from elastic analysis for the design lateral loads.

(b) The beam size chosen to form the active link at the lowest level should not be used for more than one quarter of the total height of the structure, for structures over 3 storeys in height.

(c) The redistributed design force at any given storey shall always be equal to or greater than that for the storey immediately above.

(d) The design shear capacity of the eccentrically braced bay, determined by the summation of the design shears at each active link, shall not be reduced by more than 5 % by shear or moment redistribution.

(e) The design overturning moment capacity of the eccentrically braced bay shall not be reduced by shear or moment redistribution.

(f) Redistribution of design shear capacity between eccentrically braced bays may be carried out in plan, provided the design base shear capacity of any braced bay is not altered by more than 30 % from the base shear capacity required by elastic analysis and the centre of rigidity of the structure is not significantly altered.

4.5.7 General design requirements

4.5.7.1 For members
For any member in which the design moment (M^*) has been reduced at a given location from that obtained from elastic analysis, the following provisions of section 12 shall apply:

(a) The material requirements for the member shall be as specified in 12.4 for the category of member;

(b) The appropriate section geometry requirements of 12.5 shall apply for the category of member;

(c) Restraint to the member at, and adjacent to, the given location shall be in accordance with 12.6, involving:

  (i) Each location along the member where the elastic analysis moment is reduced shall be classified as a yielding region for applying 12.6.

  (ii) The length of yielding region shall be determined in accordance with 12.6.2.1 as appropriate to the category of member.

(d) Webs of members shall comply with 12.7.2 or 12.8.5 as appropriate;

(e) If the member under consideration is a column member, 12.8.3 shall apply.

4.5.7.2 For connections

4.5.7.2.1 *Connection design moment*
The overstrength factor applicable for connection design shall be that associated with material variation only (ϕ_{om}), from 12.2.8, for the category of member from 4.5.4.

Connections in moment-resisting systems connecting members in which the design moment (M^*) at the connection has been reduced by moment redistribution shall be designed for the lesser of (a) and (b) below, except for panel zones where (b) only applies.

Amd 2 Oct. '07 (a) The material variation overstrength design moment of the member ($0.9\phi_{om}M_s$) at the connection for the category of member required from 4.5.4.

(b) The design moment from elastic analysis prior to redistribution.

The connection design moment shall also comply with the minimum design action requirements of 9.1.4.

4.5.7.2.2 *Design of connection components and connectors*
The following provisions shall apply to design of the connection components and connectors of moment-resisting connections to any member where the design moment (M^*) at the connection has been reduced from that obtained from an elastic analysis:

(a) Design actions from beams:

  (i) For determination of the design axial forces generated by the beam flanges, use the design moment (M^*) determined from 4.5.7.2.1.

  (ii) Use equation 12.9.5.2 (1) to determine the design shear force (V_p^*) on the panel zone with the out-of-balance moments determined from 4.5.7.2.1(b).

(b) Design of welded moment-resisting connections shall be in accordance with 12.9.5.3.1, 12.9.5.3.2 and 12.9.5.3.3, using the design actions from (a) above;

(c) Design of bolted moment-resisting connections shall be in accordance with 12.9.5.4, using the design actions from (a) above;

(d) Design of welds shall be in accordance with 9.7.

(e) Design of bolts shall be in accordance with 9.3.

4.6 PLASTIC ANALYSIS

4.6.1 Application

The design actions throughout all or part of a structure may be determined by a plastic analysis, provided that the limitations of 4.6.2 are observed.

4.6.2 Limitations

When a plastic method of analysis is used, all of the following conditions shall be satisfied in the members in which plastic hinges will form unless adequate ductility of the structure and plastic rotation capacity of its members and connections are established, in accordance with 1.5.1, for the design loading conditions:

(a) The minimum yield stress specified for the grade of the steel shall not exceed 350 MPa;

(b) The stress-strain characteristics of the steel shall not be significantly different from those obtained for steels complying with the material supply standards listed in 2.2.1(a) – (c), and shall be such as to provide for the required inelastic demand.

This requirement may be deemed to be satisfied if:

(i) The stress-strain diagram has a plateau at the yield stress extending for at least six times the yield strain; and

(ii) The ratio of the tensile strength to the yield stress specified for the grade of the steel (see table C2.1) is not less than 1.2; and

(iii) The elongation on a gauge length complying with AS 1391 or equivalent is not less than 15 %.

(c) The members used shall be hot-rolled or fabricated by welding from hot-formed plate;

(d) The members used shall be doubly symmetric sections;

(e) The geometry of the member section shall comply with the requirements specified for a compact section in 5.2.3;

(f) The members shall not be subject to fluctuating loading requiring a fatigue assessment (see section 10);

(g) Plastic analysis of seismic-resisting systems shall be limited to those systems listed in 12.3.2.1.1 (b) or (c).

4.6.3 Assumptions for analysis

4.6.3.1

The design actions shall be determined using a rigid plastic analysis.

4.6.3.2

For seismic-resisting systems designed using plastic analysis, all members required to form plastic hinges shall be designated primary seismic-resisting members for application of section 12. This designation shall also apply to members of the seismic-resisting system that are not required to form plastic hinges from analysis, unless these members are designed to resist the overstrength actions from the primary seismic-resisting members, using the principles of capacity design from 12.2.7.

4.6.3.3
It shall be permissible to assume full strength or partial strength connections, provided the capacities of these are used in the analysis, and provided that:

(a) For a full strength connection, for which the moment capacity of the connection shall be not less than that of the member being connected, the behaviour of the connection shall be such that the rotation capacity at none of the hinges in the collapse mechanism is exceeded; and

(b) For a partial strength connection, for which the moment capacity of the connection may be less than that of the member being connected, the behaviour of the connection shall be such as to allow all plastic hinges necessary for the collapse mechanism to develop, and shall be such that the rotation capacity at none of the plastic hinges is exceeded. Such connections used in a seismic-resisting system shall be shown to dependably provide for the anticipated ductility demand as required by 4.2.4.2.

4.6.4 Second-order effects

4.6.4.1
Any second-order effects of the loads acting on the structure in its deformed configuration may be neglected where the elastic buckling load factor (λ_c) (see 4.9) satisfies:

$$\lambda_c \geq 10$$

For $5 \leq \lambda_c < 10$, second-order effects may be neglected provided the design moments are amplified by a factor δ_p.

where

$$\delta_p = \frac{0.9}{1 - \left[\frac{1}{\lambda_c}\right]}$$

For $\lambda_c < 5$, a second-order plastic analysis shall be carried out.

4.6.4.2 Supression of snap-through instability
For portal frames with inclined rafters and with 3 or more bays per frame, the second-order effect of snap-through instability shall be suppressed by means of an appropriate design procedure.

4.7 LIMITS ON PLASTIC HINGE ROTATION IN YIELDING REGIONS OF MEMBERS

4.7.1 Application

4.7.1.1
The plastic hinge rotation in the yielding region of any member of a structural system analysed in accordance with 4.5.4.1 or 4.6 shall not exceed the limits specified in 4.7.2 for the appropriate level of axial force.

4.7.1.2
The limits given in 4.7.2 for design load combinations including earthquake loads apply to members of seismic-resisting systems and associated structural systems, designed in accordance with 12.3.3 or 12.3.4 respectively.

4.7.2 Plastic hinge rotation limits

(a) For members with negligible axial force ($N^* \leq 0.15\ \phi\ N_s$)

Table 4.7(1) – Limiting plastic hinge rotation (Radians x 10^{-3}) for load combinations

Category of member	Including earthquake loads or effects	Not including earthquake loads or effects
1	40	60
2	40	60
3	30	45

(b) For low axially loaded members ($0.15\ \phi\ N_s < N^* \leq 0.3\ \phi\ N_s$)

Table 4.7(2) – Limiting plastic hinge rotation (Radians x 10^{-3}) for load combinations

Category of member	Including earthquake loads or effects	Not including earthquake loads or effects
1	30	45
2	30	45
3	20	30

(c) For moderately axially loaded members ($0.3\ \phi\ N_s < N^* \leq 0.5\ \phi\ N_s$)

Table 4.7(3) – Limiting plastic hinge rotation (Radians x 10^{-3}) for load combinations

Category of member	Including earthquake loads or effects	Not including earthquake loads or effects
1	13	20
2	13	20
3	10	15

(d) For highly axially loaded members ($0.5\ \phi\ N_s < N^* \leq 0.8\ \phi\ N_s$)

Table 4.7(4) – Limiting plastic hinge rotation (Radians x 10^{-3}) for load combinations

Category of member	Including earthquake loads or effects	Not including earthquake loads or effects
1	8	15
2	8	15
3	5	10

Notes for tables 4.7(1) to 4.7(4)

(1) The category of member applies to 4.5.4.1, 4.6 or 12.3 as appropriate.

(2) The plastic hinge rotation as determined from structural analysis (θ_p^*) must be based on a plastic hinge length not exceeding the member depth.

(3) Where the analysis models the plastic hinge rotation as a spring, this shall be positioned at the centre of the plastic hinge length determined from NOTE (2), except for members with plastic hinges only at the member ends, the plastic hinge shall be positioned at the member ends.

(4) When the design axial force (N^*) includes earthquake loads, N^* shall be the mean axial force over one cycle of loading.

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(5) Plastic rotation limits for active links in eccentrically braced frames are given in 12.11.3.3

Notation for tables 4.7(1) to 4.7(4)

N^*	=	design axial load in accordance with Note (3)
ϕ	=	strength reduction factor from table 3.3(1)
N_s	=	nominal compression section capacity
θ_p	=	limiting plastic hinge rotation from tables 4.7
θ_p^*	=	plastic hinge rotation from structural analysis

4.8 MEMBER BUCKLING ANALYSIS

4.8.1 General

The elastic buckling load of a member (N_{om}) for the particular conditions of end restraint provided by the surrounding frame shall be determined in accordance with 4.8.2. The braced member buckling load (N_{omb}) is used in the determination of the moment amplification factor for a braced member (δ_b) in 4.4.3.2, and the sway member buckling load (N_{oms}) is used in the determination of the elastic buckling load factor (λ_{ms}) in 4.9.2.3 which is then used in the determination of the moment amplification factor for a sway member (δ_s) in 4.4.3.3.

4.8.2 Member elastic buckling load

The elastic buckling load of a member (N_{om}) shall be determined as follows:

$$N_{om} = \frac{\pi^2 EI}{(k_e L)^2}$$

where k_e is the member effective length factor, determined in accordance with 4.8.3 and L is the member length from centre to centre of its intersections with supporting members.

4.8.3 Member effective length factor

4.8.3.1 General

(a) For members being designed for load combinations which do not include earthquake loads, the member effective length factor (k_e) shall be determined in accordance with the following:

(i) Clause 4.8.3.2 for individual members with idealised end restraints. For a braced member, use k_e from 4.8.3.2 for second-order effect determination to 4.4.3.2 and for member design. For a sway member, use k_e from 4.8.3.2 for second-order effect determination to 4.4.3.3 and then use k_e = 1.0 for member design.

(ii) Clause 4.8.3.3 or Appendix G for determination of second-order effects to 4.4.3.2 or 4.5.2 in braced members of frames and for design of individual braced members.

(iii) Clause 4.8.3.3 for determination of second-order effects to 4.4.3.3, 4.5.2 or 4.6.4 in sway members of rigid-jointed frames with uniform or near-uniform gravity loading and negligible axial forces in the beams.

(iv) For design of individual sway members from (iii), k_e = 1.0 in the plane of the frame for which sway is considered.

(v) Clause 4.8.3.5 for members in triangulated structures.

(b) For members forming part of a seismic-resisting system and being designed for load combinations which include earthquake loads, the member effective length factor (k_e) shall be determined from 12.8.2.

4.8.3.2 Members with idealised end restraints

Values of the member effective length factor (k_e) which shall be used for idealized conditions of member end restraint are given in figure 4.8.3.2.

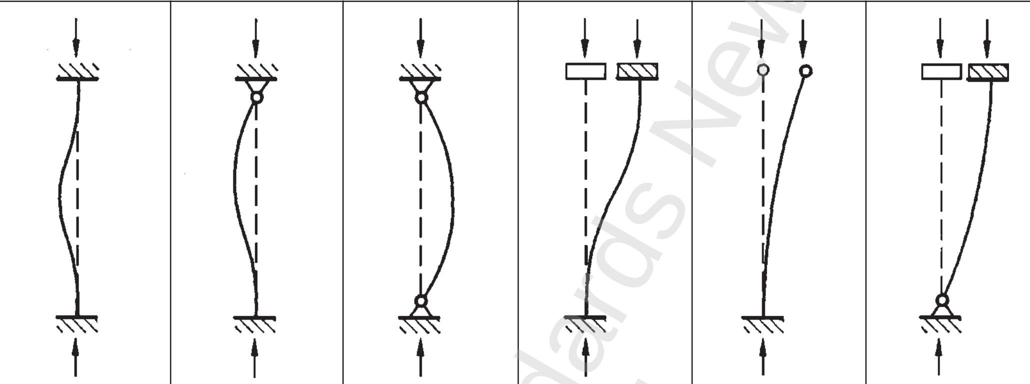
	Braced member			Sway member		
Buckled shape						
Effective length factor (k_e)	0.7	0.85	1.0	1.2	2.2	2.2
Case Number	1	2	3	4	5	6
Symbols for end restraint conditions	= Rotation fixed, translation fixed = Rotation free, translation fixed			= Rotation fixed, translation free = Rotation free, translation free		

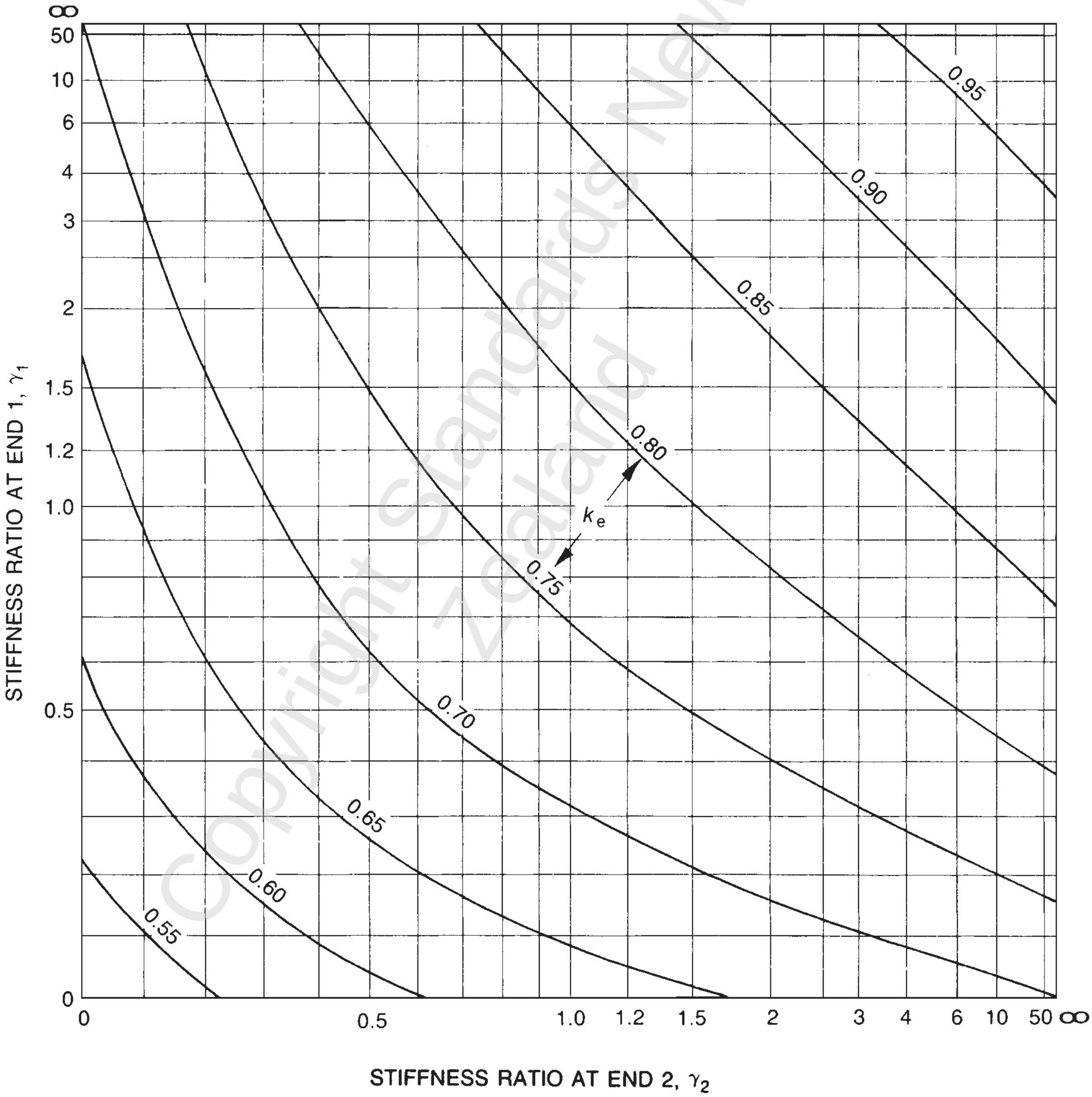
Figure 4.8.3.2 – Effective length factors for members with given conditions of end restraint

4.8.3.3 Members in frames

4.8.3.3.1

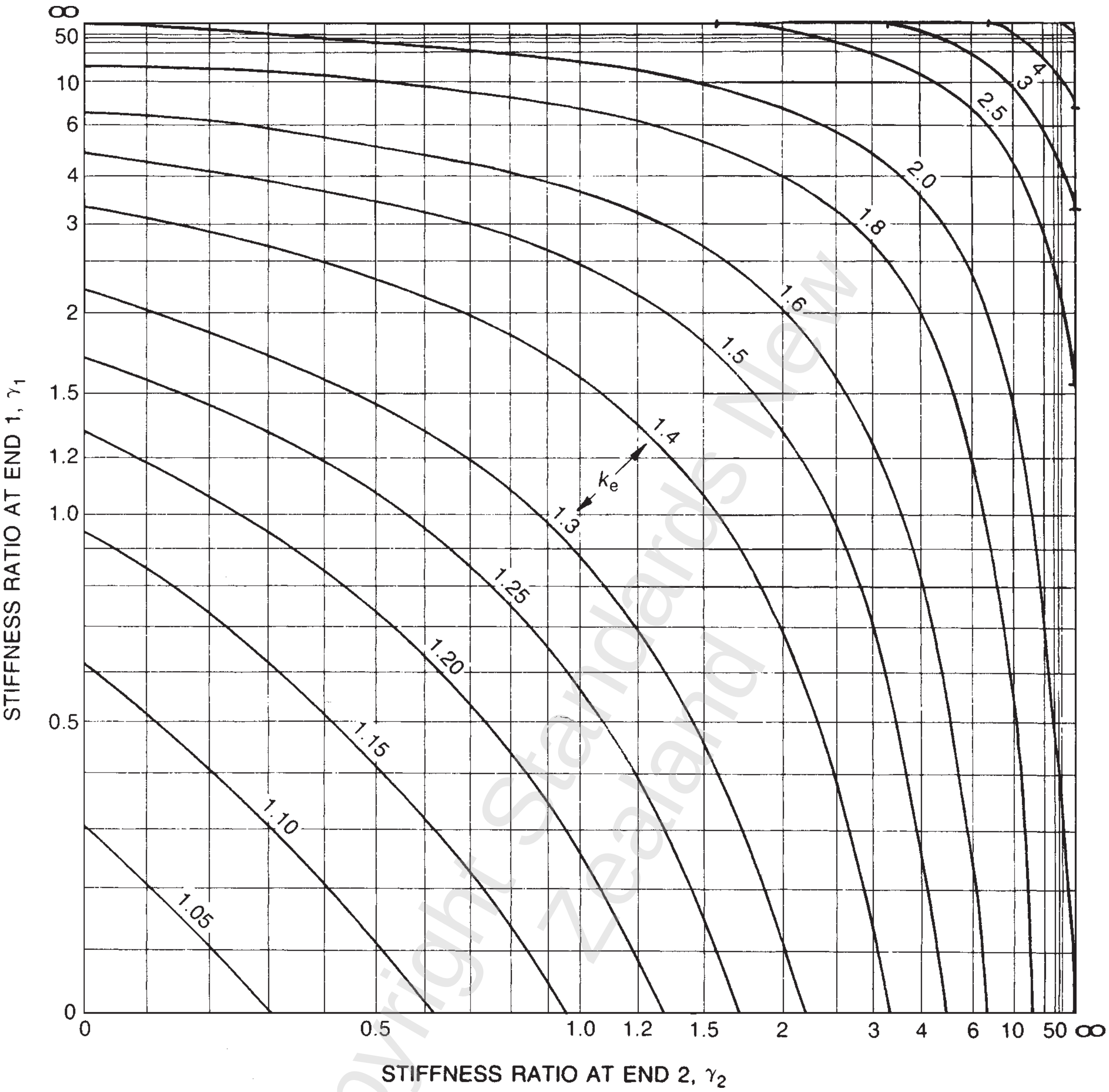
For a compression member which forms part of a frame of rigid construction being designed for load combinations which do not include earthquake loads, the member effective length factor (k_e) shall be obtained from figure 4.8.3.3 (a) for a braced member and from figure 4.8.3.3(b) for a sway member. The γ-values shall be determined in accordance with 4.8.3.4 or Appendix G, as appropriate.

4.8.3.3.2
For a compression member which forms part of a frame of rigid construction being designed for load combinations which include earthquake loads, the effective length factor shall be determined in accordance with 12.8.2.



(a) For braced members

Figure 4.8.3.3 (in part) – Effective length factors



(b) For sway members

Figure 4.8.3.3 (in part) – Effective length factors

4.8.3.4 Stiffness ratios in rectangular frames

4.8.3.4.1

The γ-value of a compression member in a rectangular frame with uniform or near-uniform gravity loading and negligible axial forces generated by gravity loading in the beams shall be calculated as follows:

$$\gamma = \frac{\sum\left[\frac{I}{L}\right]_c}{\sum\left[\frac{\beta_e I}{L}\right]_b} \qquad \text{(Eq. 4.8.3.4)}$$

except that:

(a) For a compression member whose base is not rigidly connected to a footing, the γ-value shall not be taken as less than 10 unless justified by a rational analysis; and

(b) For a compression member whose end is rigidly connected to a footing, the γ-value shall not be taken as less than 0.6 unless justified by a rational analysis.

4.8.3.4.2

The quantity $\sum(I/L)_c$ shall be calculated from the sum of the stiffnesses, in the plane of bending, of all the compression members rigidly connected at the end of the member under consideration, including the member itself.

4.8.3.4.3

The quantity $\sum(\beta_e I/L)_b$ shall be calculated from the sum of the stiffnesses, in the plane of bending, of all the beams rigidly connected at the end of the member under consideration. The contributions of any beams pin-connected to the member shall be neglected.

4.8.3.4.4

The modifying factor (β_e) which accounts for the conditions at the far ends of the beams, shall be determined from table 4.8.3.4.

Table 4.8.3.4 – Modifying factors (β_e)

Case number	Fixity conditions at far end of beam	Beam restraining a braced member	Beam restraining a sway member
1	Pinned	1.5	0.5
2	Rigidly connected to a column	1.0	1.0
3	Fixed	2.0	0.67

4.8.3.5 Members in triangulated structures

4.8.3.5.1 *General*

Except as given by 4.8.3.5.2, the effective length (L_e) of a member in a triangulated structure shall be determined from a rational elastic buckling analysis consistent with Appendix G. As an alternative, it may be taken as the length (L) from centre to centre of intersections with other members in the plane in which buckling is being considered.

4.8.3.5.2 *Discontinuous angle, channel and tee section compression members not requiring design for moment action*

For such members, which are designed in accordance with 6.6, the effective length L_e shall be calculated as required by 6.6.1 to 6.6.5.

4.9 FRAME BUCKLING ANALYSIS

4.9.1 General

The elastic buckling load factor (λ_c) shall be the ratio of the elastic buckling load set of the frame to the design load set for the frame, and shall be determined in accordance with 4.9.2.

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NOTE: $\lambda_c \geq 3.5$ is required when applying 4.9.2.

4.9.2 In-plane frame buckling

4.9.2.1

The elastic buckling load factor (λ_c) of a rigid-jointed frame shall be determined by using:

(a) A rational elastic buckling analysis of the whole frame; or

(b) One of the approximate methods of 4.9.2.2, 4.9.2.3 or 4.9.2.4.

NOTE – The value of λ_c depends on the load set (see 1.3).

4.9.2.2 Rectangular frames with all members braced

4.9.2.2.1

In a rectangular frame with uniform or near uniform gravity loading and negligible axial forces generated by gravity loading in the beams, the braced member buckling load (N_{omb}) for each column shall be determined in accordance with 4.8.2, 4.8.3.3 and 4.8.3.4.

4.9.2.2.2

The elastic buckling load factor (λ_{mb}) for each column shall be determined as follows:

$$\lambda_{mb} = \frac{N_{omb}}{N^*} \quad \text{(Eq. 4.9.2.1)}$$

4.9.2.2.3

The elastic buckling load factor (λ_c) for the whole frame shall be taken as the lowest of all the λ_{mb} values.

4.9.2.3 Rectangular frames with sway members

4.9.2.3.1

In a rectangular frame with uniform or near uniform gravity loading and negligible axial forces generated by gravity loading in the beams, the sway member buckling load (N_{oms}) for each column shall be determined in accordance with 4.8.2, 4.8.3.3 and 4.8.3.4.

4.9.2.3.2
The elastic buckling load factor (λ_{ms}) for each storey shall be determined as follows:

$$\lambda_{ms} = \frac{\sum\left[\frac{N_{oms}}{L}\right]}{\sum\left[\frac{N^*}{L}\right]} \quad \text{(Eq. 4.9.2.2)}$$

where

N^* = member design axial force, with tension taken as negative,

and the summation includes all columns within a storey.

4.9.2.3.3
The elastic buckling load factor (λ_c) for the whole frame shall be taken as the lowest of all the λ_{ms} values.

4.9.2.4 Portal frames of rigid construction
The elastic buckling load factor (λ_c) for a portal frame of rigid rectangular or non-rectangular construction, with pinned or fixed bases, shall be determined from (a) or (b) below, or from a rational elastic buckling analysis of the whole frame:

(a) *For pinned-base frames:*

$$\lambda_c = \frac{3EI_{pr}}{s_r\,[N_c^*\,h_e + 0.3\,N_r^*\,s_r]} \quad \text{(Eq. 4.9.2.3(1))}$$

(b) *For fixed-base frames:*

$$\lambda_c = \frac{5E(10+\psi_f)}{\left[\frac{5N_r^*\,s_r^2}{I_{pr}}\right] + \left[\frac{2\psi_f N_c^*\,h_e^2}{I_{pc}}\right]} \quad \text{(Eq. 4.9.2.3(2))}$$

where

$\psi_f = \frac{I_{pc}s_r}{I_{pr}h_e}$

N_c^* = maximum design axial compression force from elastic analysis, in outer column

N_r^* = design axial compression force from elastic analysis, in the outer rafter framing into the column carrying N_c^*

I_{pc} = second moment of area of outer column

I_{pr} = second moment of area of outer rafter

h_e = height to centre-line of rafter at the knee

s_r = centre-line length of rafter, measured from knee to apex and ignoring the presence of any haunches.

NOTES

5 MEMBERS SUBJECT TO BENDING AND SHEAR

5.1 DESIGN FOR BENDING MOMENT

5.1.1

A member bent about the section major principal *x*-axis and which is analysed by the elastic method (see 4.4) or the elastic method with redistribution (see 4.5) shall satisfy:

$M_x^* \leq \phi M_{sx}$, and

$M_x^* \leq \phi M_{bx}$

where

M_x^* = the design bending moment about the *x*-axis determined in accordance with 4.4 or 4.5

ϕ = the strength reduction factor (see table 3.3(1))

M_{sx} = the nominal section capacity in bending, as specified in 5.2, for bending about the *x*-axis

M_{bx} = the nominal member capacity in bending, as specified in 5.3 or 5.6, for bending about the *x*-axis.

5.1.2

A member bent about the section minor principal *y*-axis and which is analysed by the elastic method (see 4.4) or the elastic method with redistribution (see 4.5) shall satisfy:

$M_y^* \leq \phi M_{sy}$

where

M_y^* = the design bending moment about the *y*-axis determined in accordance with 4.4 or 4.5

M_{sy} = the nominal section capacity in bending, as specified in 5.2, for bending about the *y*-axis.

NOTE – For members subject to minor axis bending from laterally unrestrained loads applied above the shear centre, the destablizing effect of these loads shall be considered; refer to C5.1.2 for guidance.

5.1.3

A member which is analysed by the plastic method (see 4.6) shall be compact at all sections where plastic hinges may form (see 5.2.3), any segment containing a plastic hinge shall have full lateral restraint as specified in 5.3.2.1.2, and its web shall satisfy 5.10.6. The member shall satisfy:

$M^* \leq \phi M_s$

where

M^* = the design bending moment determined in accordance with 4.6, and

M_s = the nominal section moment capacity as specified in 5.2.1.

5.1.4
A member whose deflections are constrained to a non-principal plane shall be analysed as specified in 5.7.1, and shall satisfy 8.3.4.

5.1.5
A member which is bent about a non-principal axis and whose deflections are unconstrained shall be analysed as specified in 5.7.2, and shall satisfy 8.3.4 and 8.4.5.

5.1.6
A member subjected to combined bending and shear shall satisfy the requirements of 5.1 and 5.12.

5.1.7
A member subjected to combined bending and axial compression or tension shall satisfy section 8.

5.2 NOMINAL SECTION MOMENT CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

5.2.1 General
The nominal section moment capacity (M_s) shall be calculated as follows:

$$M_s = f_y Z_e \qquad \text{(Eq. 5.2.1)}$$

where

f_y is the value for the flanges

Z_e is as specified in 5.2.3, 5.2.4, or 5.2.5 as appropriate.

5.2.2 Section slenderness parameter

5.2.2.1
For a section with flat compression plate elements in either uniform or non-uniform compression, the section slenderness parameter (λ_s) shall be taken as the value of the compression plate element slenderness (λ_e) for the element of the cross section which has the greatest value of λ_e/λ_{ey}:

where

$$\lambda_e = \left(\frac{b}{t}\right)\sqrt{\left(\frac{f_y}{250}\right)}$$

λ_{ey} = the plate element yield slenderness limit (see table 5.2)

b = the clear width of the element outstand from the face of the supporting plate element or the clear width of the element between the faces of supporting plate elements

t = the element thickness.

5.2.2.2
The section plasticity and yield slenderness parameter limits (λ_{sp}) and (λ_{sy}) respectively shall be taken as the values of the element slenderness limits (λ_{ep}) and (λ_{ey}) respectively given in table 5.2 for the element of the cross section which has the greatest value of λ_e/λ_{ey}.

Table 5.2 – Values of plate element slenderness limits

Case number	Plate element type	Longitudinal edges supported	Residual stresses (see Notes)	Plasticity limit (λ_{ep})	Yield limit (λ_{ey})	Deformation limit (λ_{ed})
1	Flat (Uniform compression)	One	SR HR LW, CF HW	10 9 8 8	16 16 15 14	35 35 35 35
2	Flat (Maximum compression at unsupported edge, zero stress or tension at supported edge)	One	SR HR LW, CF HW	10 9 8 8	25 25 22 22	– – – –
3	Flat (Uniform compression)	Both	SR HR LW, CF HW	30 30 30 30	45 45 40 35	90 90 90 90
4	Flat (Compression at one edge, tension at the other)	Both	Any Any Any	45(6A) 82(6B) –	60(6A) 130(6B) 180(5)	–
5	Circular hollow sections		SR HR, CF LW HW	50 50 42 42	120 120 120 120	– – – –

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NOTE –

(1) SR – stress relieved
HR – hot-rolled or hot-finished
CF – cold formed
LW – lightly welded longitudinally
HW – heavily welded longitudinally

(2) Welded members whose compressive residual stresses are less than 40 MPa shall be considered to be lightly welded.

(3) For members subjected to design load combinations including earthquake loads, the plate element slenderness limits of table 12.5 also apply as appropriate to the member category.

(4) For composite concrete-filled structural hollow sections refer to 13.8.3.2.

(5) For the web of a doubly symmetric I-section in bending about the section's major principal *x*-axis, the yield limit is increased to 180.

(6a) These limits are applicable to the webs of rectangular and square hollow section members.

(6b) These limits are applicable to the webs of members other than rectangular and square hollow section members.

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5.2.2.3
For narrow rectangular sections without supported edges and bent about the major principal x-axis, $\lambda_s = \lambda_{sp}$.

5.2.2.4
For circular hollow sections, the section slenderness parameter (λ_s) shall be calculated as follows:

$$\lambda_s = \left(\frac{d_o}{t}\right)\left(\frac{f_y}{250}\right)$$

where d_o is the outside diameter of the section. The section plasticity and yield slenderness limits (λ_{sp}) and (λ_{sy}) respectively shall be taken as the values of the element slenderness limits (λ_{ep}) and (λ_{ey}) respectively given in table 5.2.

5.2.3 Effective section modulus for compact sections
For sections with $\lambda_s \leq \lambda_{sp}$, the effective section modulus (Z_e) shall be the lesser of S or $1.5Z$, where S and Z are the plastic and elastic section moduli respectively, determined in accordance with 5.2.7.

5.2.4 Effective section modulus for non-compact sections
For sections with $\lambda_{sp} < \lambda_s \leq \lambda_{sy}$, the effective section modulus (Z_e) shall be calculated as follows:

$$Z_e = Z + \left[\left(\frac{\lambda_{sy} - \lambda_s}{\lambda_{sy} - \lambda_{sp}}\right)(Z_c - Z)\right]$$

where Z_c is the effective section modulus (Z_e) for a compact section as specified in 5.2.3.

5.2.5 Effective section modulus for slender sections

5.2.5.1
For sections with flat plate elements in uniform compression for which $\lambda_s > \lambda_{sy}$, the effective section modulus (Z_e) shall be calculated either as follows:

$$Z_e = Z\left(\frac{\lambda_{sy}}{\lambda_s}\right)$$

or for the effective cross section determined by omitting from each flat compression element the width in excess of the width corresponding to λ_{sy}.

5.2.5.2
For a section whose slenderness is determined by the value calculated for a flat plate element with maximum compression at an unsupported edge and zero stress or tension at the other edge and for which $\lambda_s > \lambda_{sy}$, the effective section modulus (Z_e) shall be calculated as follows:

$$Z_e = Z\left(\frac{\lambda_{sy}}{\lambda_s}\right)^2$$

5.2.5.3
For circular hollow sections with $\lambda_s > \lambda_{sy}$, the effective section modulus shall be taken as the lesser of:

$$Z_e = Z\sqrt{\left(\frac{\lambda_{sy}}{\lambda_s}\right)} \quad \text{and}$$

$$Z_e = Z\left(\frac{2\lambda_{sy}}{\lambda_s}\right)^2$$

5.2.6 Deformation of flat plate elements at the serviceability limit state

For elements where $\lambda_e > \lambda_{ed}$ in which λ_{ed} is the deformation slenderness limit given in table 5.2, noticeable deformations may occur at the serviceability limit state.

5.2.7 Elastic and plastic section moduli

5.2.7.1
For sections without holes, or for sections with holes that reduce either of the flange areas by not more than 100 $\{1-[f_y/(0.85f_u)]\}$%, the elastic and plastic section moduli may be calculated using the gross section.

5.2.7.2
For sections with holes that reduce either of the flange areas by more than 100$\{1-[f_y/(0.85f_u)]\}$%, the elastic and plastic section moduli shall be calculated using either:

(a) (A_n/A_g) times the value for the gross section, in which A_n is the sum of the net areas of the flanges and the gross area of the web, and A_g is the gross area of the section, or

(b) The net section.

When net areas are calculated, any deductions for fastener holes shall be made in accordance with 9.1.10.

5.3 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS AND MEMBERS SUBJECT TO MAJOR PRINCIPAL X-AXIS BENDING AND WITH FULL LATERAL RESTRAINT

NOTE – The nominal member moment capacity of a member subject to minor principal *y*-axis bending shall be taken as the nominal section moment capacity in *y*-axis bending (see 5.1.2 and 5.2).

5.3.1 General

5.3.1.1 Role of a segment
The member moment capacity of a member subject to major principal *x*-axis bending shall be determined on a segment by segment basis.

For application of 5.3 (and 5.6) a member consists of one or more segments.

5.3.1.2 Definition of a segment
A segment in a member is defined as:

(a) The length between adjacent cross sections which are fully, partially or laterally restrained (see 5.4.2.1, 5.4.2.2 or 5.4.2.3); or

(b) The length between an unrestrained end (see 5.4.1.4) and the adjacent cross section which is fully or partially restrained (see 5.4.2.1 or 5.4.2.2).

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NOTE – Typically, 5.3.1.2(b) will only be applied to a free-standing cantilever member with one end unrestrained.

5.3.1.3 Definition of a member

A member is defined as:

(a) The length between adjacent supports (see 1.3); or

(b) The cantilevered length from the support, in the case of a free-standing member.

5.3.1.4 Restraint to a support

The cross section of a member at a support shall have full or partial restraint (see 5.4.2.1 or 5.4.2.2).

NOTE – The apex of a portal frame shall be considered as a support when the slope of the rafter framing into the apex exceeds 5^{o} from the horizontal for a symmetrical frame, or, for an asymmetrical frame, when the included angle is less than 170^{o}.

5.3.1.5 Segment with full lateral restraint; nominal member moment capacity

Where a segment has full lateral restraint (see 5.3.2.1), the nominal major principal *x*-axis member moment capacity (M_{bx}) of the segment shall be taken as the nominal major principal *x*-axis section moment capacity (M_{sx}) (see 5.2) of the critical section (see 5.3.3).

5.3.1.6 Member with full lateral restraint

5.3.1.6.1

When all segments along a member have full lateral restraint, the member shall be considered to have full lateral restraint.

5.3.1.6.2

Where a member has full lateral restraint, the nominal major principal *x*-axis member moment capacity (M_{bx}) of the member shall be taken as the nominal major principal *x*-axis section moment capacity of the critical section.

5.3.2 Segments or members with full lateral restraint

5.3.2.1 General requirements for full lateral restraint

5.3.2.1.1 *Full lateral restraint of segments or members not containing a yielding region*

The general requirements are:

(a) For a segment

A segment not containing a yielding region shall be considered to have full lateral restraint if, for that segment, $M_{bx} = M_{sx}$, where M_{bx} is calculated in accordance with 5.6 and M_{sx} is calculated in accordance with 5.2 at the critical section (see 5.3.3).

(b) For a member

A member not containing a yielding region shall be considered to have full lateral restraint if all the segments along the member have full lateral restraint, or, alternatively, if the member satisfies 5.3.2.2 or 5.3.2.3.

5.3.2.1.2 *Full lateral restraint of segments containing a yielding region*

A segment containing a yielding region is required to have full lateral restraint, in accordance with 12.6.2.2.

5.3.2.2 Members not containing a yielding region and with continuous lateral restraints
A member not containing a yielding region and with continuous lateral restraints may be considered to have full lateral restraint, provided that:

(a) The member supports are fully or partially restrained (see 5.4.2.1 or 5.4.2.2); and

(b) The continuous lateral restraints (see 5.4.2.3) act at the critical flange (see 5.5).

5.3.2.3 Members not containing a yielding region and with intermediate lateral restraints
A member not containing a yielding region and with intermediate lateral restraints (see 5.4.2.3) may be considered to have full lateral restraint, provided that:

(a) The member supports are fully or partially restrained (see 5.4.2.1 or 5.4.2.2); and

(b) The length (L) between adjacent restraints satisfies 5.3.2.4; and

(c) The lateral restraints (see 5.4.2.3) act at the critical flange (see 5.5).

5.3.2.4 Spacing between adjacent restraints
The maximum length (L) between adjacent restraints, for application of 5.3.2.3, must satisfy:

$$\frac{L}{r_y} \leq (80 + 50\beta_m)\sqrt{\left(\frac{250}{f_y}\right)}$$ if the member is an equal flanged I-section; or

$$\frac{L}{r_y} \leq (60 + 40\beta_m)\sqrt{\left(\frac{250}{f_y}\right)}$$ if the member is an equal flanged channel; or

$$\frac{L}{r_y} \leq (80 + 50\beta_m)\left[\sqrt{\left(\frac{2\rho A d_f}{2.5 Z_{ex}}\right)}\right]\sqrt{\left(\frac{250}{f_y}\right)}$$ if the member is an I-section with unequal flanges; or

$$\frac{L}{r_y} \leq (1800 + 1500\beta_m)\left(\frac{b_f}{b_w}\right)\left(\frac{250}{f_y}\right)$$ if the member is a rectangular or square hollow section; or

$$\frac{L}{t} \leq (210 + 175\beta_m)\left[\sqrt{\left(\frac{b_2}{b_1}\right)}\right]\left(\frac{250}{f_y}\right)$$ if the member is an angle section.

where

A = area of cross section

b_f, b_w = the flange width and web depth, respectively

b_1, b_2 = the greater and lesser leg lengths, respectively

d_f = the distance between flange centroids

I_{cy} = the second moment of area of the compression flange about the section minor y-axis

I_y = the second moment of area of the section about its minor y-axis

L = length of member between adjacent restraints

r_y = the radius of gyration about the y-axis

t = the thickness of an angle section

Z_e = the effective section modulus (see 5.2)

ρ = I_{cy}/I_y.

The ratio β_m shall be taken as one of the following, as appropriate:

(a) – 1.0 for any length (L); or

(b) – 0.8 for a length (L) which is subject to transverse load (see 1.3); or

(c) β_m for a length (L) which is not subject to transverse load, where β_m is the ratio of the smaller to the larger end bending moments over the length (L) and is taken as positive when this length is bent in reverse curvature and negative when this length is bent in single curvature.

5.3.3 Critical section

The critical section in a segment or member shall be taken as the cross section which has the largest value of the ratio of the design bending moment (M^*) to the nominal section capacity in bending (M_s) (see 5.2).

5.4 RESTRAINTS

5.4.1 General

5.4.1.1

A cross section may be considered to be fully, partially, laterally or rotationally restrained if its restraints satisfy the appropriate requirements of 5.4.2.

5.4.1.2

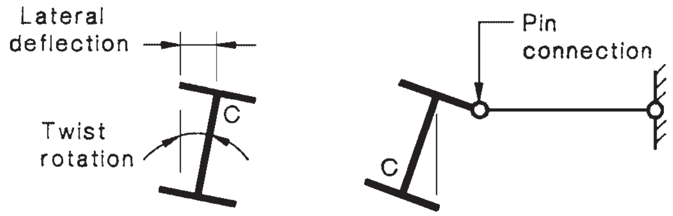
Restraints against lateral deflection, twist rotation, or lateral rotation may be considered to be effective if they satisfy the appropriate requirements of 5.4.3.

5.4.1.3

The members and connections of restraint systems shall be designed to transfer the appropriate forces and bending moments specified in 5.4.3, together with any other forces or bending moments which may act simultaneously, from the points where the forces or bending moments arise to anchorage or reaction points.

5.4.1.4

Any cross section of a member which does not satisfy any of clauses 5.4.2.1 to 5.4.2.3 shall be considered to be unrestrained, as for example in figure 5.4.1.



No critical flange lateral restraint, no twist restraint

Figure 5.4.1 – Unrestrained cross sections (U)

5.4.2 Restraints at a cross section

5.4.2.1 Full section restraint (F)

A cross section of a member may be considered to be fully restrained (F) if either:

(a) The restraint or support effectively prevents lateral deflection of the critical flange (see 5.5), and provides effective restraint against twist rotation of the section, as for example in figure 5.4.2.1 (a), or in figure 5.4.2.1 (b); or

(b) The restraint or support effectively prevents lateral deflection of some other point in the cross section, and effectively prevents twist rotation of the section, as for example in figure 5.4.2.1(c).

NOTE – When the bolt hole size is larger than the nominal diameter specified in 14.3.5.2.1, tensioned property class 8.8 bolts (category 8.8/*TB*) to 15.2.4 and 15.2.5 shall be used in any bolted connection providing full restraint between the restrained and restraining members, unless the effect of bolt slip on the restraint is accounted for by the use of Appendix H.

5.4.2.2 Partial section restraint (P)

A cross section of a member may be considered to be partially restrained (P) if the restraint or support effectively prevents lateral deflection of some point in the cross section other than the critical flange, and provides partial restraint against twist rotation of the section, as for example in figure 5.4.2.2.

A cross section of a member may also be considered to be partially restrained (P) if twist rotation of the cross section is effectively restrained and lateral deflection of the cross section is partially restrained.

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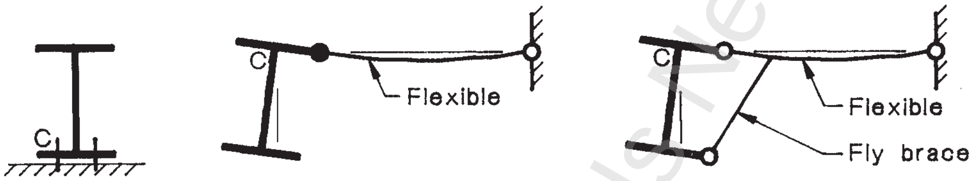
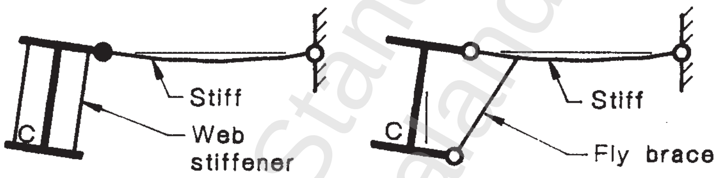
NOTE – When the bolt hole size is larger than the nominal diameter specified in 14.3.5.2.1, tensioned property class 8.8 bolts (category 8.8/TB) to 15.2.4 and 15.2.5 shall be used in any bolted connection providing partial restraint between the restrained and restraining members, unless the effect of bolt slip on the restraint is accounted for by the use of Appendix H.

NOTE – Either flange may be critical

(a) Critical flange lateral restraint, effective twist restraint



(b) Critical flange lateral restraint, effective twist restraint

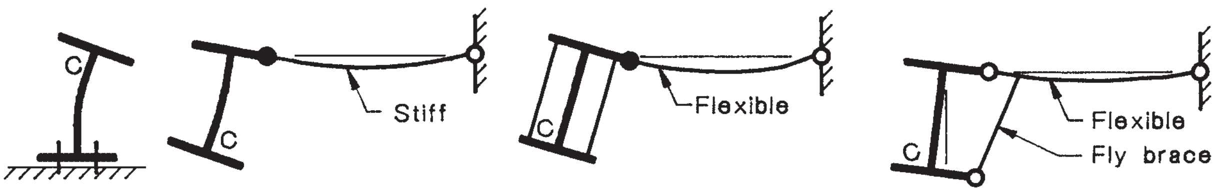


(c) Non-critical flange lateral restraint, effective twist restraint

LEGEND to figures 5.4.2.1 and 5.4.2.2

o : Pin connection
● : Moment connection
C : Critical flange

Figure 5.4.2.1 – Fully restrained cross sections (F)



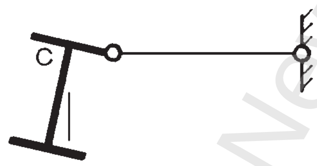
(a) **(b)** **(c)** **(d)**

Figure 5.4.2.2 – Partially restrained cross sections (P)

Non-critical flange lateral restraint, partial twist restraint

5.4.2.3 Lateral restraint to the critical flange (L)
The critical flange at a point of restraint may be considered to be laterally restrained (L) when the restraint effectively prevents lateral deflection of the critical flange (see 5.5) but is ineffective in preventing twist rotation of the cross section, as for example in figure 5.4.2.3.

NOTE – If one end of the segment under consideration is unrestrained (see 5.4.1.4), then the other end must be fully or partially restrained, rather than laterally restrained (see 5.3.1.2(b)).



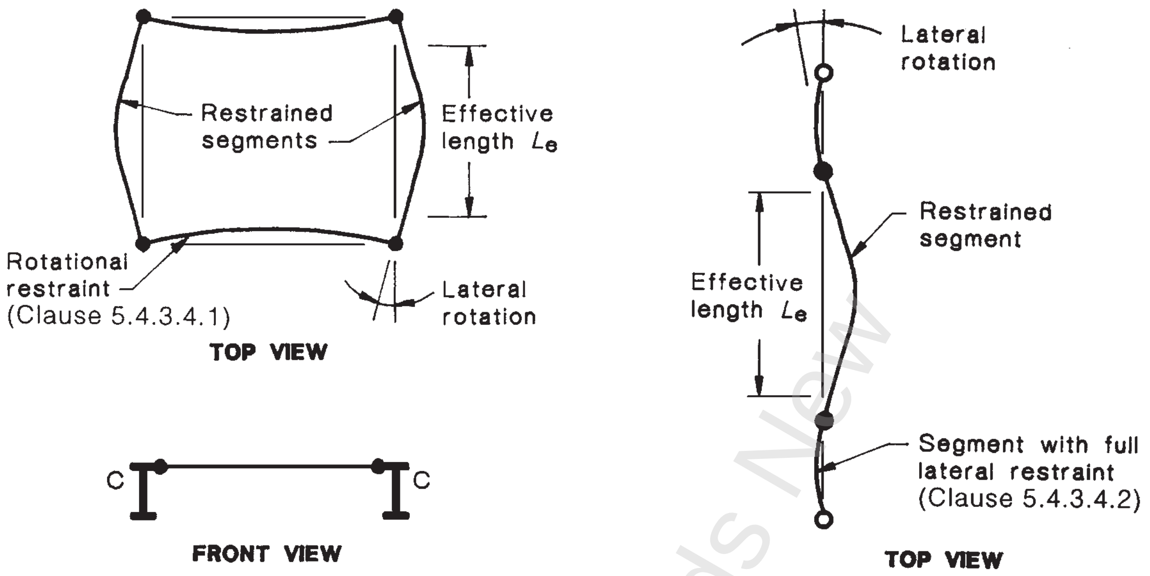
Critical flange lateral restraint, no twist restraint

LEGEND to figure 5.4.2.3

o	:	Pin connection
●	:	Moment connection
C	:	Critical flange

Figure 5.4.2.3 – Lateral restraint to the critical flange (L)

5.4.2.4 Rotational restraint in plan
A cross section of a member which is fully, partially or laterally restrained may also be considered to be rotationally restrained in plan when the restraint or support provides significant restraint against lateral rotation of the critical flange (see 5.5) about the minor principal y-axis of the cross section, as for example in figure 5.4.2.4. ➤



(a) Members with transverse restraint **(b) Continuous members**

LEGEND to figure 5.4.2.4

o	:	Pin connection
●	:	Moment connection
C	:	Critical flange

Figure 5.4.2.4 – Minor axis restraints for rotational restraint in plan

5.4.3 Restraining elements

5.4.3.1 Restraint against lateral deflection

5.4.3.1.1
The lateral restraint at any cross section considered to be fully, partially or laterally restrained in terms of 5.4.2 shall be designed to transfer to anchorage or reaction points a transverse force acting at the critical flange (see 5.5) equal to 0.025 times the maximum force in the critical flanges of the adjacent segments, except where the restraints are more closely spaced than is required to ensure that $M^* \le \phi M_{bx}$.

5.4.3.1.2
When the restraints are more closely spaced, then a lesser force may be designed for. The actual arrangement of restraints shall be assumed to be equivalent to a set of restraints which will ensure that $M^* \le \phi M_{bx}$. Each equivalent restraint shall correspond to an appropriate group of the actual restraints. This group shall then be designed as a whole to transfer to anchorage or reaction points the transverse force of 0.025 times the maximum force in the critical flanges of the equivalent adjacent segments.

5.4.3.2 Restraint against twist rotation

5.4.3.2.1

A restraint against twist rotation at a cross section provides effective restraint against twist rotation when it is designed to resist the moment generated by the following:

(a) A transverse force equal to 0.025 times the maximum force in the critical flange from the adjacent segments; acting at

(b) A lever arm equal to the distance from the centroid of that critical flange to the point of restraint against lateral movement of the cross section.

5.4.3.2.2

A restraint against twist torsion at a cross section may be deemed to provide partial restraint against twist rotation if it is able to provide an elastic restraint against twist rotation. Flexible elements such as unstiffened webs may form part of such a restraint, provided that they are connected in such a way as to prevent unacceptable rotational slip.

5.4.3.2.3

Any restraint at a cross section which permits rotational slip shall be deemed to be ineffective in restraining twist rotation, unless the extent of rotational slip possible is assessed by rational analysis to be sufficiently low to allow partial restraint to be provided.

NOTE – Guidance on the effects of the stiffness of a torsional restraint on the resistance to lateral buckling is given in H5.1 of Appendix H.

5.4.3.3 Parallel restrained members

When a series of parallel members is restrained by a line of restraints, each restraining element shall be designed to transfer to anchorage or reaction points a transverse force equal to the sum of 0.025 times the flange force from the connected member and 0.0125 times the sum of the flange forces in the connected members beyond, except that the contribution from no more than 7 members need be considered.

5.4.3.4 Restraint against minor principal y-axis rotation

5.4.3.4.1

A rotational restraint (see 5.4.2.4) at a cross section which is fully, partially or laterally restrained (see 5.4.2.1, 5.4.2.2 or 5.4.2.3) may be considered to provide restraint against lateral rotation of the critical flange about the minor principal *y*-axis of the cross section, providing that its flexural stiffness (EI/L) in the plane of rotation is comparable with the corresponding stiffness of the restrained member.

NOTE – Guidance on the effects of the stiffness of a rotational restraint on the resistance to lateral buckling is given in H5.2 of Appendix H.

5.4.3.4.2

A segment which has full lateral restraint (see 5.3.2.1) may be deemed to provide rotational restraint to an adjacent segment which is part of the same member (i.e. the member containing both segments is continuous through the point of restraint).

5.4.3.4.3

A segment which does not have full lateral restraint shall be assumed to be unable to provide rotational restraint to an adjacent segment which is part of the same member.

5.5 CRITICAL FLANGE

5.5.1 General

5.5.1.1
The critical flange at any cross section is the flange which, in the absence of any restraint at that section, would deflect the further during buckling.

5.5.1.2
The critical flange shall either be determined by an elastic buckling analysis (see 5.6.4) or as specified in 5.5.2 and 5.5.3.

5.5.2 Segments with both ends restrained

The critical flange at any section of a segment restrained at both ends shall be the compression flange.

5.5.3 Segments with one end unrestrained

5.5.3.1
When gravity loads are dominant, both flanges of a segment with one end unrestrained shall be considered critical.

5.5.3.2
When wind loads are dominant, the critical flange shall be the exterior flange in the case of external pressure or internal suction, and shall be the interior flange in the case of internal pressure or external suction.

5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING AND WITH OR WITHOUT FULL LATERAL RESTRAINT

5.6.1 Segments restrained at both ends

5.6.1.1 Open sections with equal flanges

5.6.1.1.1 *Segments of constant cross section*

(a) **The nominal member *x*-axis moment capacity (M_{bx}) shall be calculated as follows:**

$$M_{bx} = \alpha_m \alpha_s M_{sx} \leq M_{sx} \text{ (see Note below)} \quad \text{(Eq. 5.6.1.1(1))}$$

where

α_m = a moment modification factor
α_s = a slenderness reduction factor
M_{sx} = the nominal section moment capacity about the major principal *x*-axis determined in accordance with 5.2 for the gross section.

Note to equation 5.6.1.1(1)
Where this equation is applied through 12.6.2.2, $(\alpha_m \alpha_s) \geq 1.0$ is required for category 2 and 3 members and $(\alpha_m \alpha_s) \geq \phi_{oms}$ is required for category 1 members, where α_m and α_s are calculated in accordance with the provisions of this clause and ϕ_{oms} is given by 12.2.8.

(b) **The moment modification factor (α_m) shall be taken as one of the following:**

(i) 1.0; or

(ii) A value obtained from table 5.6.1; or

(iii) $$\alpha_m = \frac{17M_m^*}{\sqrt{\left[\left(M_2^*\right)^2 + \left(M_3^*\right)^2 + \left(M_4^*\right)^2\right]}} \le 2.5 \qquad \text{(Eq. 5.6.1.1(2))}$$

where

M_m^* = maximum design bending moment in the segment, taken as positive in sign

M_2^*, M_4^* = design bending moments at the quarter points of the segment

M_3^* = design bending moment at the mid-point of the segment; or

NOTE –
(1) Equation 5.6.1.1(2) gives a conservative answer for restrained cantilevers under uniform loading, for which α_m = 3.5 (see case 10 in table 5.6.1).

(2) When the segment contains a yielding region, the value of α_m used in design shall not exceed 1.75 (see 12.6.2.2).

(iv) A value obtained from an elastic buckling analysis in accordance with 5.6.4.

(c) **The slenderness reduction factor (α_s) shall be determined as follows:**

$$\alpha_s = 0.6\left\{\sqrt{\left[\left(\frac{M_{sx}}{M_{oa}}\right)^2 + 3\right]} - \left(\frac{M_{sx}}{M_{oa}}\right)\right\} \qquad \text{(Eq. 5.6.1.1(3))}$$

in which M_{oa} shall be taken as either:

(i) $M_{oa} = M_o$ from equation 5.6.1.1(4), where M_o is the reference buckling moment; or

(ii) The value determined from an elastic buckling analysis in accordance with 5.6.4.

(d) **The reference buckling moment (M_o) shall be determined as follows:**

$$M_o = \sqrt{\left\{\left(\frac{\pi^2 E I_y}{L_e^2}\right)\left[GJ + \left(\frac{\pi^2 E I_w}{L_e^2}\right)\right]\right\}} \qquad \text{(Eq. 5.6.1.1(4))}$$

where

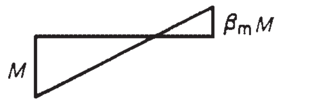
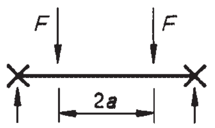
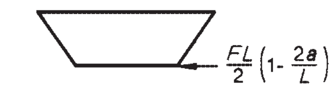
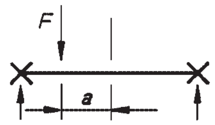
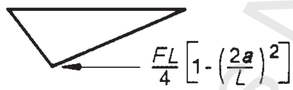
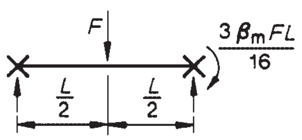
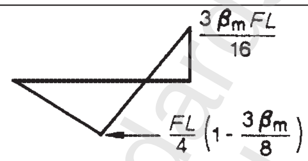
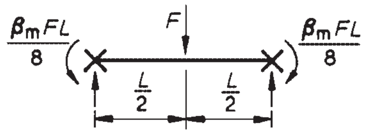
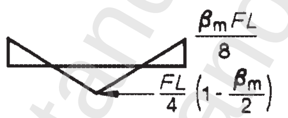
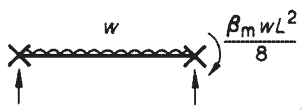
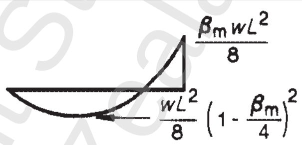
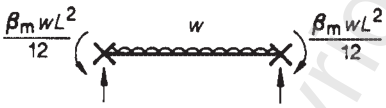
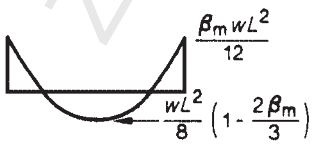
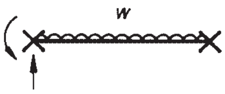
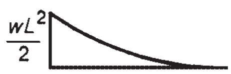
E, G = the elastic moduli (see 1.4)

I_y, J, and I_w = section constants (see 1.4)

L_e = the effective length determined in accordance with 5.6.3.

NOTE – Values of E and G and expressions for J and I_w are given in H4 of Appendix H.

Table 5.6.1 – Moment modification factors (α_m) for segments restrained at both ends

Case number	Member segment	Moment distribution	Moment modification factor (α_m)[3]	Range
1			$1.75+1.05\beta_m+0.3\beta_m^2$ 2.5	$-1 \leqslant \beta_m \leqslant 0.6$ $0.6 < \beta_m \leqslant 1$
2			$1.0+0.35\left(1-\frac{2a}{L}\right)^2$	$0 \leqslant \frac{2a}{L} \leqslant 1$
3			$1.35+0.4\left(\frac{2a}{L}\right)^2$	$0 \leqslant \frac{2a}{L} \leqslant 1$
4			$1.35+0.15\beta_m$ $-1.2+3.0\beta_m$	$0 \leqslant \beta_m < 0.9$ $0.9 \leqslant \beta_m \leqslant 1$
5			$1.35+0.36\beta_m$	$0 \leqslant \beta_m \leqslant 1$
6			$1.13+0.10\beta_m$ $-1.25+3.5\beta_m$	$0 \leqslant \beta_m \leqslant 0.7$ $0.7 \leqslant \beta_m \leqslant 1$
7			$1.13+0.12\beta_m$ $-2.38+4.8\beta_m$	$0 \leqslant \beta_m \leqslant 0.75$ $0.75 \leqslant \beta_m \leqslant 1$
8			1.00	
9			1.75	
10			3.50	

NOTE –

(1) X ≡ full, partial or lateral restraint

(2) β_m = for case number 1, the ratio of smaller to larger bending moment at the ends of the member, taken as positive when the member is bent in reverse (double) curvature; or
= for case numbers 4-7, the ratio of design end moment to fixed end moment, taken as positive;

(3) When the segment contains a yielding region, the value of α_m used in design shall not exceed 1.75 (see 12.6.2.2).

5.6.1.1 Open sections with equal flanges (continued)

5.6.1.1.1 *Segments of varying cross section*
The nominal member *x*-axis moment capacity (M_{bx}) at the critical cross section shall be determined in accordance with 5.6.1.1.1 and using either:

(a) The properties of the minimum cross section; or

(b) The properties of the critical cross section as specified in 5.3.3, provided that the value of M_{oa} determined in accordance with 5.6.1.1.1(c) is reduced, before it is used in equation 5.6.1.1(3), by multiplying it by the reduction factor (α_{st}) as follows:

$$\alpha_{st} = 1.0 - [1.2 r_r (1 - r_s)]$$

where

r_r = L_r / L for stepped members

= 0.5 for tapered members

$$r_s = \frac{A_{fm}}{A_{fc}} \left[0.6 + \left(\frac{0.4 d_m}{d_c} \right) \right]$$

A_{fm}, A_{fc} = the flange areas at the minimum and critical cross sections, respectively

d_m,d_c = the section depths at the minimum and critical cross sections, respectively

L_r = the length within the segment under consideration over which the cross section is reduced

L = the length of the segment; or

(c) The method of design by buckling analysis (see 5.6.4).

5.6.1.2 I-sections with unequal flanges

5.6.1.2.1
The nominal member *x*-axis moment capacity (M_{bx}) shall be determined in accordance with 5.6.1.1.1, except that the reference buckling moment (M_o) shall be determined by using either:

(a) $$M_o = \sqrt{\left(\frac{\pi^2 E I_y}{L_e^2}\right)} \left\{ \sqrt{\left[(GJ) + \left(\frac{\pi^2 E I_w}{L_e^2}\right) + \left(\frac{\beta_x^2}{4} \frac{\pi^2 E I_y}{L_e^2}\right) \right]} + \frac{\beta_x}{2} \sqrt{\left(\frac{\pi^2 E I_y}{L_e^2}\right)} \right\} \quad \text{.......(Eq. 5.6.1.2)}$$

or

(b) The method of designing by buckling analysis (see 5.6.4).

5.6.1.2.2
The monosymmetry section constant (β_x) for use in 5.6.1.2.1 shall be determined using either:

(a) $$\beta_x = 0.8 d_f \left[\left(\frac{2 I_{cy}}{I_y} \right) - 1 \right]$$

where

d_f = the distance between flange centroids

I_{cy} = the second moment of area of the compression flange about the section minor principal *y*-axis;

or

(b) $\beta_x = \frac{1}{I_x} \int (x^2 y + y^3)\, dA - 2y_o$

where y_o is the coordinate of the shear centre (see reference H6.11 of Appendix H).

5.6.1.2.3
The values of β_x from 5.6.1.2.2 are positive when the larger flange is in compression and negative when the smaller flange is in compression.

5.6.1.2.4
Expressions for I_w and J are given in H4, Appendix H.

5.6.1.3 Angle sections
The nominal member *x*-axis moment capacity (M_{bx}) of an angle section shall be determined in accordance with 5.6.1.1.1 using $I_w = 0$.

5.6.1.4 Hollow sections
The nominal member *x*-axis moment capacity (M_{bx}) of a rectangular or circular hollow section shall be determined in accordance with 5.6.1.1.1 using $I_w = 0$.

5.6.1.5 Narrow rectangular sections
The nominal member *x*-axis moment capacity (M_{bx}) of a narrow rectangular section shall be determined in accordance with 5.6.1.1.1 using $I_w = 0$.

5.6.1.6 Tee sections
The nominal member *x*-axis moment capacity (M_{bx}) of a tee section shall be determined in accordance with 5.6.1.2 using $I_w = 0$.

5.6.2 Segments unrestrained at one end

The nominal member *x*-axis moment capacity (M_{bx}) of a segment unrestrained at one end and at the other end both:

(a) Fully or partially restrained; and

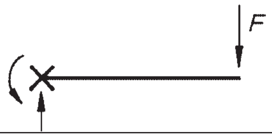
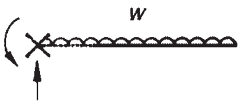
(b) Continuous past the restraint or with rotational restraint in plan (see 5.4.2.4)

shall be determined using either:

(i) Equations 5.6.1.1(1) and 5.6.1.1(3), with M_{oa} equal to the value of M_o obtained from equation 5.6.1.1(4), and the appropriate value of α_m given in table 5.6.2; or

(ii) The method of design by buckling analysis (see 5.6.4).

Table 5.6.2 – Moment modification factors (α_m) for segments unrestrained at one end

Case number	Member segment	Moment distribution	Moment modification factor (α_m)
1			0.25
2			1.25
3			2.25 (2)

NOTE –

(1) $X \equiv$ full or partial restraint.

(2) If the segment contains a yielding region, this value shall be taken as 1.75.

5.6.3 Effective length

5.6.3.1
The effective length (L_e) of a segment shall be determined as follows:

$$L_e = k_t k_l k_r L$$

where

k_t = a twist restraint factor given in table 5.6.3(1),

k_l = a load height factor given in table 5.6.3(2),

k_r = a rotation restraint factor, set = 1.0 or alternatively as given in table 5.6.3(3) and 5.6.3.2,

L = segment length, as given by 5.3.1.2.

5.6.3.2
The rotation restraint factor (k_r) shall only be taken as less than unity when effective rotational restraints, complying with 5.4.3.4, act at one or both ends of a segment which is restrained at both ends. The rotation restraint factor shall be taken as unity for all segments which are unrestrained at one end.

Table 5.6.3(1) – Twist restraint factors (k_t)

End restraint arrangement	Factor (k_t)
FF, *FL*, *LL*, *FU*	1.0
FP, *PL*, *PU*	$1 + \left[\left(\frac{d}{L}\right)\left(\frac{t_f}{2t_w}\right)^3\right]\frac{1}{n_w}$
PP	$1 + \left[2\left(\frac{d}{L}\right)\left(\frac{t_f}{2t_w}\right)^3\right]\frac{1}{n_w}$

In tables 5.6.3(1), 5.6.3(2) and, when applicable, table 5.6.3(3),

d = depth of the section

L = segment length

n_w = number of webs

t_f = thickness of critical flange

t_w = thickness of web

F ≡ fully restrained

L ≡ laterally restrained

P ≡ partially restrained

U ≡ unrestrained

and 2 of the symbols *F*, *L*, *P*, *U* are used to indicate the restraint conditions at the 2 ends e.g. *FF*.

Table 5.6.3(2) – Load height factors (k_l) for gravity loads

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Longitudinal position of the load	End restraint arrangement	Load height position	
		Shear centre (1)	Top flange (2)
Within segment	*FF, FP, FL, PP, PL, LL* *FU, PU*	1.0 1.0	1.4 2.0
At segment end	*FF, FP, FL, PP, PL, LL* *FU, PU*	1.0 1.0	1.0 2.0

NOTE –

(1) For singly-symmetric sections loaded through the centroid perpendicular to the axis of symmetry (e.g. channel sections subject to major axis bending), any reduction in member moment capacity arising from the offset of the shear centre from the centroid along the symmetry axis is accounted for through the load height factor, when the load itself or the structural system transferring the load to the segment is laterally restrained. When either the load or the transferring structural system is laterally unrestrained, then the member must be designed for the torsion arising from the offset of the shear centre and centroid, in addition to the direct bending.

(2) For loads applied to a section along a principal axis on which lie both the shear centre and the centroid of the section, the classification of load height applied through the top flange need be applied only when the load itself or the structural system transferring the load to the segment is laterally unrestrained.

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Table 5.6.3(3) – Rotation restraint factors (k_r)

End restraint arrangement	Ends with minor axis rotation restraints (see 5.4.3.4)	Factor (k_r)
FU, PU	Any	1.0
FF, FP, FL, PP, PL, LL	None	1.0
FF, FP, PP	One	0.85
FF, FP, PP	Both	0.70

5.6.4 Design by buckling analysis

When a member is designed by this method, the elastic buckling bending moment (M_{ob}) for the segment under consideration shall be determined by using the results of an elastic flexural-torsional buckling analysis. This analysis shall take account of the member support, segment restraint and loading conditions.

The value of M_{oa} to be used in 5.6.1 or 5.6.2 shall be taken as follows:

$$M_{oa} = \frac{M_{ob}}{\alpha_m}$$

The moment modification factor (α_m) shall be determined by using either (a) or (b) given below:

(a) Clause 5.6.1.1.1(b) or table 5.6.2; or

(b) The value given by $\alpha_m = \dfrac{M_{os}}{M_{oo}}$

where

M_{os} = the elastic buckling bending moment for a segment, fully restrained at both ends, which is unrestrained against rotation about the minor principal *y*-axis and loaded at the shear centre

M_{oo} = the reference elastic buckling bending moment given by equation 5.6.1.1(4) with $L_e = L$.

NOTE – Refer to Appendix H, especially H2, H3 and H5, when undertaking design by buckling analysis.

5.7 BENDING IN A NON-PRINCIPAL PLANE

5.7.1 Deflections constrained to a non-principal plane

5.7.1.1

When the deflection of a member is constrained to a non-principal plane by continuous lateral restraints which prevent lateral deflection, then the forces exerted by the restraints shall be determined by rational analysis and included in the calculation of the principal axis bending moments.

5.7.1.2
The calculated principal axis bending moments shall satisfy 8.3.4.

5.7.2 Deflections unconstrained

5.7.2.1
When the deflections of a member loaded in a non-principal plane are unconstrained, the principal axis bending moments arising from the loading shall be calculated.

5.7.2.2
The calculated principal axis bending moments shall satisfy 8.3.4 and 8.4.5.

5.8 SEPARATORS AND DIAPHRAGMS

If separators or diaphragms are used to permit two or more I-section members or channels placed side by side to act together as a unit in the distribution of external loads between them, the separators and diaphragms shall meet the following requirements:

(a) Separators, made up of spacers and through bolts, shall not be used to transmit forces between the members, other than those due to transverse forces (if any) and a design transverse force (Q^*), taken as not less than 0.025 times the maximum design force occurring in the most heavily loaded compression flange of any member forming the unit. The design transverse force (Q^*) shall be taken as shared equally between the separators.

(b) Diaphragms shall be used where external vertical as well as transverse forces are to be transmitted from one member to another. The diaphragms and their fastenings shall be proportioned to distribute the forces applied to them and, in addition, to resist the design transverse force (Q^*) specified above and resulting shear forces. The design transverse force (Q^*) shall be taken as shared equally between the diaphragms.

5.9 DESIGN OF WEBS

5.9.1 General

The geometry and arrangement of beam webs, including any transverse or longitudinal stiffeners, shall satisfy 5.10.

A web subject to shear force shall satisfy 5.11.

A web subject to shear force and bending moment shall satisfy 5.12.

A web subject to bearing load shall satisfy 5.13.

Load-bearing stiffeners and end posts shall satisfy 5.14.

Intermediate transverse stiffeners shall satisfy 5.15.

Longitudinal stiffeners shall satisfy 5.16.

Webs of yielding regions of members shall also satisfy 12.7.2 or 12.8.5 as appropriate.

5.9.2 Definition of web panel

A web panel of thickness (t_w) shall be considered to extend over an unstiffened area of a web plate length (longitudinal dimension) (s) and clear depth (transverse dimension) (d_p). The web panel may be bounded by flanges, transverse or longitudinal stiffeners, or free edges.

5.9.3 Maximum slenderness ratio of web panel

Unless a rational analysis would warrant a lesser value, the slenderness ratio of a web panel shall satisfy 5.10.1, 5.10.4, 5.10.5 and 5.10.6.

5.10 ARRANGEMENT OF WEBS

5.10.1 Unstiffened webs

5.10.1.1

The web slenderness ratio (d_1/t) of an unstiffened web, bounded on both longitudinal sides by flanges, shall comply with:

$$\left(\frac{d_1}{t}\right)\sqrt{\left(\frac{f_y}{250}\right)} \le 180$$

where

d_1 = the clear depth of the web between flanges, ignoring fillets or welds

t = the web thickness.

5.10.1.2

The web slenderness ratio (d_1/t) of an unstiffened web, bounded on one longitudinal side by a free edge, shall comply with:

$$\left(\frac{d_1}{t}\right)\sqrt{\left(\frac{f_y}{250}\right)} \le 90$$

where

d_1 = the clear depth of the web, ignoring fillets or welds

t = the web thickness.

5.10.2 Load bearing stiffeners

Load bearing stiffeners shall be provided in pairs where the design compressive bearing forces applied through a flange by loads or reactions exceed the design bearing capacity (ϕR_b) of the web alone specified in 5.13.2, or when required to form an end post (5.15.2.2).

5.10.3 Side reinforcing (doubler) plates

5.10.3.1

Additional side reinforcing plates (alternatively known as doubler plates) may be provided to augment the strength of the web. Proper account shall be taken of any lack of symmetry. The proportion of shear force assumed to be resisted by such plates shall be limited by the amount of horizontal shear which can be transmitted through the fasteners to the web and to the flanges.

5.10.3.2

For panel zones in moment-resisting connections subject to loads or effects including earthquake loads, refer to 12.9.5.2 (b) and 12.9.5.3.2.

5.10.4 Transversely stiffened webs

5.10.4.1
The web slenderness ratio of a web, transversely stiffened with stiffeners at spacing *s*, but without longitudinal stiffeners, shall comply with:

(a) When $1.0 < s/d_1 \leq 3.0$ $\quad \left(\frac{d_1}{t}\right)\sqrt{\left(\frac{f_y}{250}\right)} \leq 200$

(b) When $0.74 < s/d_1 \leq 1.0$ $\quad \left(\frac{s}{t}\right)\sqrt{\left(\frac{f_y}{250}\right)} \leq 200$

(c) When $s/d_1 \leq 0.74$ $\quad \left(\frac{d_1}{t}\right)\sqrt{\left(\frac{f_y}{250}\right)} \leq 270$

5.10.4.2
All web lengths for which $s/d_p > 3.0$ shall be considered to be unstiffened, where d_p is the greatest panel depth in the length, *s*.

5.10.5 Webs with longitudinal and transverse stiffeners

5.10.5.1
The web slenderness ratio of a web with a set of longitudinal stiffeners placed on one or both sides of the web, at a distance $0.2d_2$ from the compression flange, shall comply with:

(a) When $1.0 < s/d_1 \leq 2.4$ $\quad \left(\frac{d_1}{t}\right)\sqrt{\left(\frac{f_y}{250}\right)} \leq 250$

(b) When $0.74 < s/d_1 \leq 1.0$ $\quad \left(\frac{s}{t}\right)\sqrt{\left(\frac{f_y}{250}\right)} \leq 250$

(c) When $s/d_1 \leq 0.74$ $\quad \left(\frac{d_1}{t}\right)\sqrt{\left(\frac{f_y}{250}\right)} \leq 340$

5.10.5.2
The web slenderness ratio of a web with an additional set of longitudinal stiffeners placed on one or both sides of the web, at the neutral axis, shall comply with:

When $s/d_1 \leq 1.5$ $\quad \left(\frac{d_1}{t}\right)\sqrt{\left(\frac{f_y}{250}\right)} \leq 400$

5.10.6 Webs of members designed plastically

5.10.6.1
The web slenderness ratio of a member assumed to contain a plastic hinge shall comply with:

$$\left(\frac{d_1}{t}\right)\sqrt{\left(\frac{f_y}{250}\right)} \leq 82$$

5.10.6.2
Load bearing stiffeners shall be provided when a bearing load or shear force acts within $d_1/2$ of a plastic hinge location and the design bearing load or design shear force exceeds 0.1 times the design shear yield capacity (ϕV_w) of the member specified in 5.11.4.

5.10.6.3
These stiffeners shall be located within a distance $d_1/2$ on either side of the hinge location and shall be designed in accordance with 5.14 to carry the greater of the design bearing load or the design shear force considered as a bearing load.

5.10.6.4
If the stiffeners are flat plates, their slenderness parameter (λ_s), as defined in 5.2.2.1 using the stiffener yield stress (f_{ys}), shall be less than the plasticity limit (λ_{sp}) specified in 5.2.2.2.

5.10.7 Openings in webs

5.10.7.1
Except for a castellated member, an opening in a web may be unstiffened provided that the greatest internal dimension of the opening (L_w) satisfies either:

(a) $L_w/d_1 \leq 0.10$ for webs without longitudinal stiffeners, or

(b) $L_w/d_1 \leq 0.33$ for longitudinally stiffened webs,

provided that the longitudinal distance between boundaries of adjacent openings is at least three times the greatest internal dimension of the opening.

5.10.7.2
Unstiffened or stiffened openings with either the greatest internal dimension exceeding these limits or with more than one opening provided at any cross section shall be designed in accordance with an appropriate limit state design procedure.

5.10.7.3
The design of a castellated member shall be in accordance with an appropriate limit state design procedure.

5.11 NOMINAL SHEAR CAPACITY OF WEBS

5.11.1 Shear capacity
A web subject to a design shear force (V^*) shall satisfy:

$$V^* \leq \phi V_v$$

where

ϕ = the strength reduction factor (see table 3.3)

V_v = the nominal shear capacity of the web, determined from either 5.11.2 (V_{vu}) or 5.11.3 (V_{vn}).

When the design bending moment (M^*) associated with the design shear force (V^*) exceeds ($0.75\phi M_s$) for the gross cross section, interaction of shear and bending moment shall be considered in accordance with 5.12.

5.11.2 Shear capacity of a web with uniform shear stress distribution

5.11.2.1
The nominal shear capacity (V_{vu}) of a web with an effectively uniform shear stress distribution, such as:

(a) The web of an I-section or channel section subject to a design shear force parallel to the member web; or

(b) The webs of a rectangular or square hollow section; or

(c) A circular hollow section designed to 5.11.4.2;

shall be given by 5.11.2.2 for a stocky web or 5.11.2.3 for a slender web.

5.11.2.2 Shear capacity of a stocky web with uniform shear stress distribution
For a stocky web, in which the maximum web panel slenderness ratio (d_p/t_w) satisfies:

$$\frac{d_p}{t_w} \leq \frac{82}{\sqrt{\left(\frac{f_y}{250}\right)}}$$

the nominal shear capacity of the web (V_{vu}) shall be taken as:

$$V_{vu} = V_w$$

where the nominal shear yield capacity of the web (V_w) is specified in 5.11.4.

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5.11.2.3 Shear capacity of a slender web with uniform shear stress distribution
For a slender web, in which the maximum web panel slenderness ratio (d_p/t_w) satisfies:

$$\frac{d_p}{t_w} > \frac{82}{\sqrt{\left(\frac{f_y}{250}\right)}}$$

the nominal shear capacity (V_{vu}) of the web shall be taken as:

$$V_{vu} = V_b$$

where the nominal shear buckling capacity of the web (V_b) is specified in 5.11.5.

5.11.3 Shear capacity of a flat plate with non-uniform shear stress distribution

The nominal shear capacity (V_{vn}) of a flat plate with a non-uniform shear stress distribution, such as:

(a) A single element member with a rectangular cross section; or

(b) The flange of an I-section subject to a design shear force parallel to the member flange; or

(c) The web of a member with only one flange, such as a tee-section; or

(d) A web of an I-section containing holes larger than those specified in 5.10.7.1;

shall be calculated as follows:

$$V_{vn} = \frac{2V_{vu}}{0.9 + \left(\dfrac{f^*_{vm}}{f^*_{va}}\right)} \leq V_{vu}$$

where

V_{vu} = the nominal shear capacity of the flat plate calculated assuming a uniform shear stress distribution, from 5.11.2

f^*_{vm}, f^*_{va} = the maximum and average design shear stresses in the web determined by a rational elastic analysis.

5.11.4 Shear yield capacity

5.11.4.1
The nominal shear yield (V_w) of a flat plate web shall be calculated as follows:

$$V_w = 0.6 f_y A_w$$

where A_w is the gross sectional area of the web.

5.11.4.2
The nominal shear yield capacity (V_w) of a circular hollow section shall be calculated as follows:

$$V_w = 0.36 f_y A_e$$

where the effective sectional area (A_e) shall be taken as the gross area of the circular hollow section provided either that there are no holes larger than those required for fasteners, or that the net area is greater than 0.9 times the gross area, or otherwise shall be taken as the net area.

5.11.5 Shear buckling capacity

5.11.5.1 Unstiffened web
The nominal shear buckling capacity (V_b) for an unstiffened web or a web considered to be unstiffened (see 5.10.4.2) shall be calculated as follows:

$$V_b = \alpha_v V_w \leq V_w$$

where

$$\alpha_v = \left[\frac{82}{\left(\dfrac{d_p}{t_w}\right)\sqrt{\left(\dfrac{f_y}{250}\right)}}\right]^2$$

5.11.5.2 Stiffened web

The nominal shear buckling capacity (V_b) for a stiffened web with $s/d_p \leq 3.0$ shall be calculated as follows:

$$V_b = \alpha_v \alpha_d \alpha_f V_w \leq V_w$$

where

When $1.0 \leq s/d_p \leq 3.0$,
$$\alpha_v = \left[\frac{82}{\left(\frac{d_p}{t_w}\right)\sqrt{\left(\frac{f_y}{250}\right)}}\right]^2 \left[\frac{0.75}{\left(\frac{s}{d_p}\right)^2} + 1.0\right] \leq 1.0\text{; or}$$

When $s/d_p \leq 1.0$,
$$\alpha_v = \left[\frac{82}{\left(\frac{d_p}{t_w}\right)\sqrt{\left(\frac{f_y}{250}\right)}}\right]^2 \left[\frac{1}{\left(\frac{s}{d_p}\right)^2} + 0.75\right] \leq 1.0$$

$$\alpha_d = 1 + \frac{1 - \alpha_v}{115\alpha_v \sqrt{\left[1 + \left(\frac{s}{d_p}\right)^2\right]}}\text{; or}$$

α_d = 1.0 when required by 5.15.2.2, and

d_p = the depth of the deepest web panel.

Values of the product $\alpha_v \alpha_d$ are given in table 5.11.5.2.

The flange restraint factor (α_f) shall be taken as either:

(a) $\alpha_f = 1.0$; or

(b) $$\alpha_f = 1.6 - \frac{0.6}{\sqrt{\left[1 + \left(\frac{40 b_{fo} t_f^2}{d_1^2 t_w}\right)\right]}}$$ for webs without longitudinal stiffeners,

in which b_{fo} is the least of all the following:

(i) $\frac{12 t_f}{\sqrt{(f_y / 250)}}$;

(ii) The distance from the mid-plane of the web to the nearer edge of the flange (taken as zero if there is no flange outstand);

(iii) Half the clear distance between the webs if there are 2 or more webs.

NOTE – Guidance on the shear buckling capacity of a web which contains an axial force is given in Appendix J.

Table 5.11.5.2 – Values of $\alpha_v\alpha_d$

$\left(\frac{d_p}{t_w}\right)\sqrt{\left(\frac{f_y}{250}\right)}$	$\frac{s}{d_p}$										
	0.3	0.5	0.75	1.0	1.25	1.5	1.75	2.0	2.5	3.0	> 3.0
90	1.000	1.000	1.000	1.000	1.000	1.000	1.000	0.991	0.952	0.927	0.830
100	1.000	1.000	1.000	1.000	0.998	0.946	0.907	0.877	0.833	0.803	0.672
110	1.000	1.000	1.000	0.989	0.919	0.866	0.825	0.792	0.744	0.711	0.556
120	1.000	1.000	1.000	0.930	0.859	0.805	0.762	0.728	0.677	0.642	0.467
130	1.000	1.000	1.000	0.883	0.812	0.757	0.713	0.678	0.625	0.587	0.398
140	1.000	1.000	0.960	0.846	0.775	0.719	0.674	0.638	0.583	0.544	0.343
150	1.000	1.000	0.926	0.816	0.745	0.689	0.643	0.606	0.550	0.510	0.299
160	1.000	1.000	0.898	0.792	0.721	0.664	0.617	0.579	0.522	0.481	0.263
170	1.000	1.000	0.875	0.772	0.701	0.643	0.596	0.558	0.499	0.458	0.233
180	1.000	0.997	0.855	0.755	0.684	0.626	0.578	0.539	0.480	0.438	0.208
190	1.000	0.974	0.839	0.740	0.669	0.611	0.563	0.524	0.464	0.421	
200	1.000	0.955	0.825	0.728	0.657	0.598	0.550	0.511	0.450	0.407	
210	1.000	0.939	0.813	0.718							
220	1.000	0.924	0.803	0.709							
230	1.000	0.912	0.793	0.701							
240	1.000	0.901	0.785	0.694							
250	1.000	0.891	0.778	0.687							
260	1.000	0.883	0.772	0.682							
270	1.000	0.875	0.767	0.677							

5.12 INTERACTION OF SHEAR AND BENDING MOMENT

5.12.1 General

The nominal web shear capacity (V_{vm}) in the presence of bending moment shall be calculated using 5.12.2, except that:

(a) Interaction of shear and bending moment in the yielding regions of beams in a moment-resisting framed seismic-resisting system shall be to 12.10.3; and

(b) Interaction of shear and bending moment in the active links of an eccentrically braced frame need not be considered (see 12.11.3.5(d)).

5.12.2 Shear and bending moment interaction

Except as given by 12.10.3, the web design shear capacity in the presence of bending moment shall satisfy:

$$V^* \leq \phi V_{vm}$$

where

$$V_{vm} = V_v \quad \text{for } M^* \leq 0.75\phi M_s\text{; or}$$

$$V_{vm} = V_v \left[2.2 - \left(\frac{1.6M^*}{\phi M_s} \right) \right] \quad \text{for } 0.75\phi M_s \leq M^* \leq \phi M_s$$

where

V_v = the nominal shear capacity of a web in shear alone (see 5.11.1)

M_s = the nominal section moment capacity for the gross cross section, determined in accordance with 5.2.

NOTE –

(1) For *I* and channel cross sections with slender webs (see 5.11.2.3), with $M^* > 0.75\ \phi M_s$, and with the design shear force (V^*) parallel to the web, a less conservative method of considering the interaction of shear and bending moment is given in C5.12 of Part 2 of this Standard.

(2) Guidance on design of stiffened web panels required to resist bending moment, shear, axial force and transverse loading is given in Appendix J.

5.13 COMPRESSIVE BEARING ACTION ON THE EDGE OF A WEB

5.13.1 Dispersion of force to web

Where a force is applied to a flange either as a point load or through a stiff bearing of length (b_s), it shall be considered as dispersed uniformly through the flange at a slope of 1:2.5 to the surface of the flange, as shown in figure 5.13.1.1, or to the top of the flat portion of the web for rectangular and square hollow sections, as shown in figure 5.13.1.3. The stiff bearing length is that length which cannot deform appreciably in bending. The dispersion of load to the flange shall be taken at a slope of 1:1 through solid material, as shown in figure 5.13.1.2.

5.13.2 Bearing capacity

The design force (R^*) on a web shall satisfy:

$$R^* \leq \phi R_b$$

where

ϕ = the strength reduction factor (see table 3.3)

R_b = the nominal bearing capacity of the web under concentrated or patch loading, which shall be taken as the lesser of its nominal bearing yield capacity (R_{by}) defined in 5.13.3, and its nominal bearing buckling capacity (R_{bb}) defined in 5.13.4.

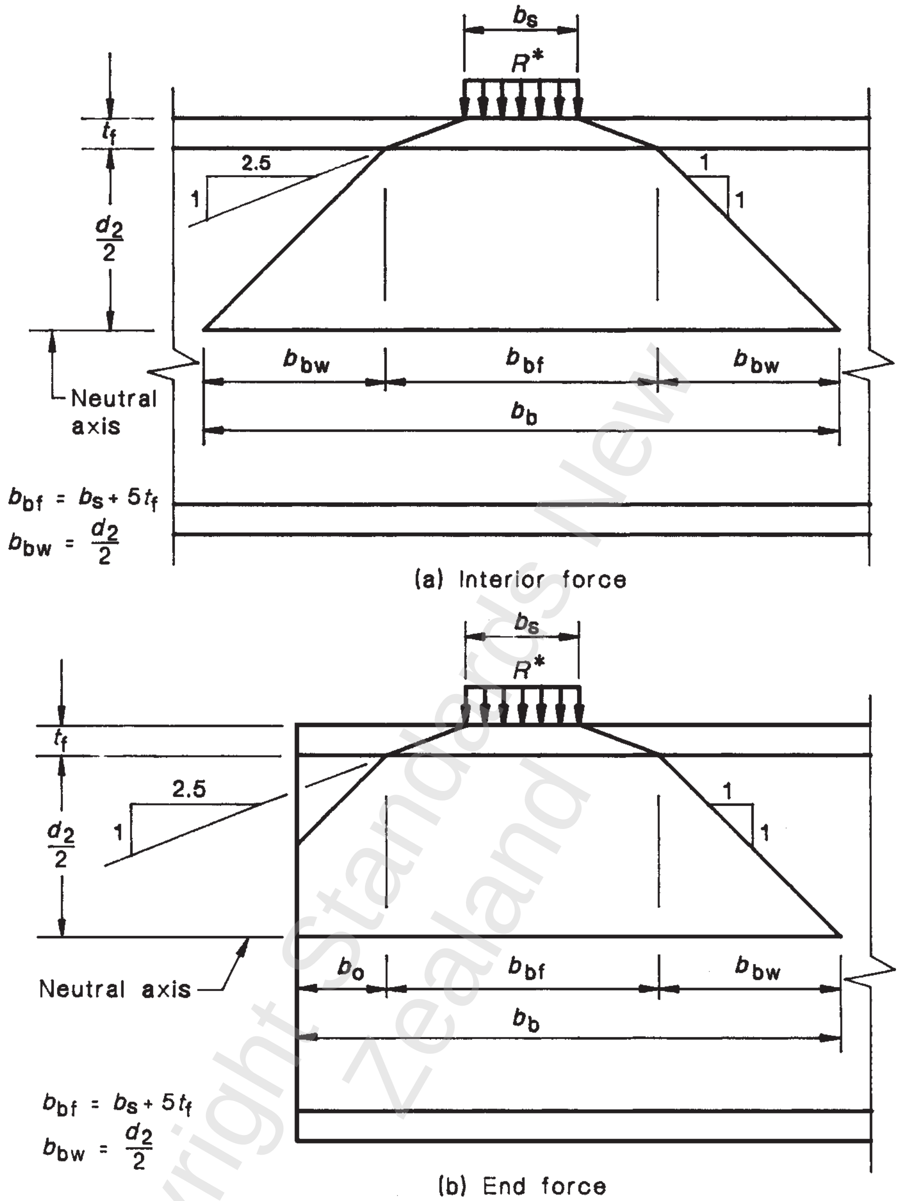

Figure 5.13.1.1 – Dispersions of force through flange and web

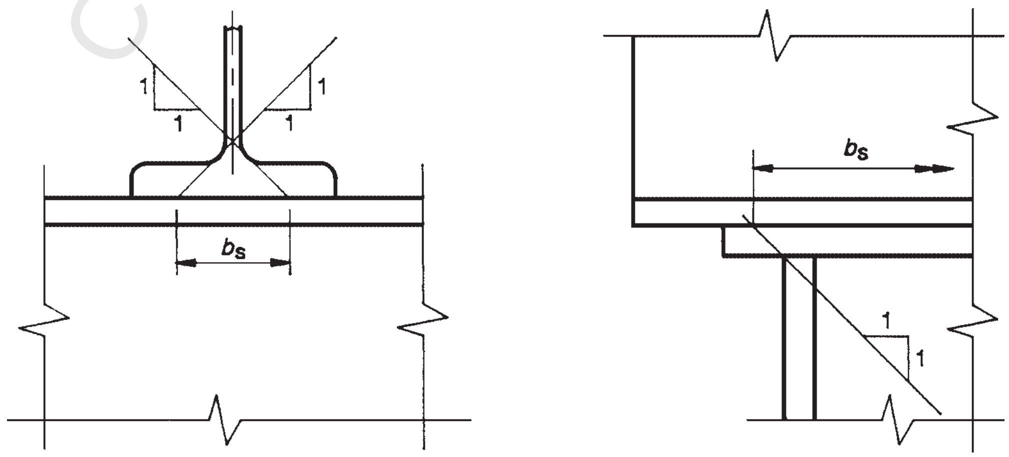

Figure 5.13.1.2 – Stiff bearing length on flange

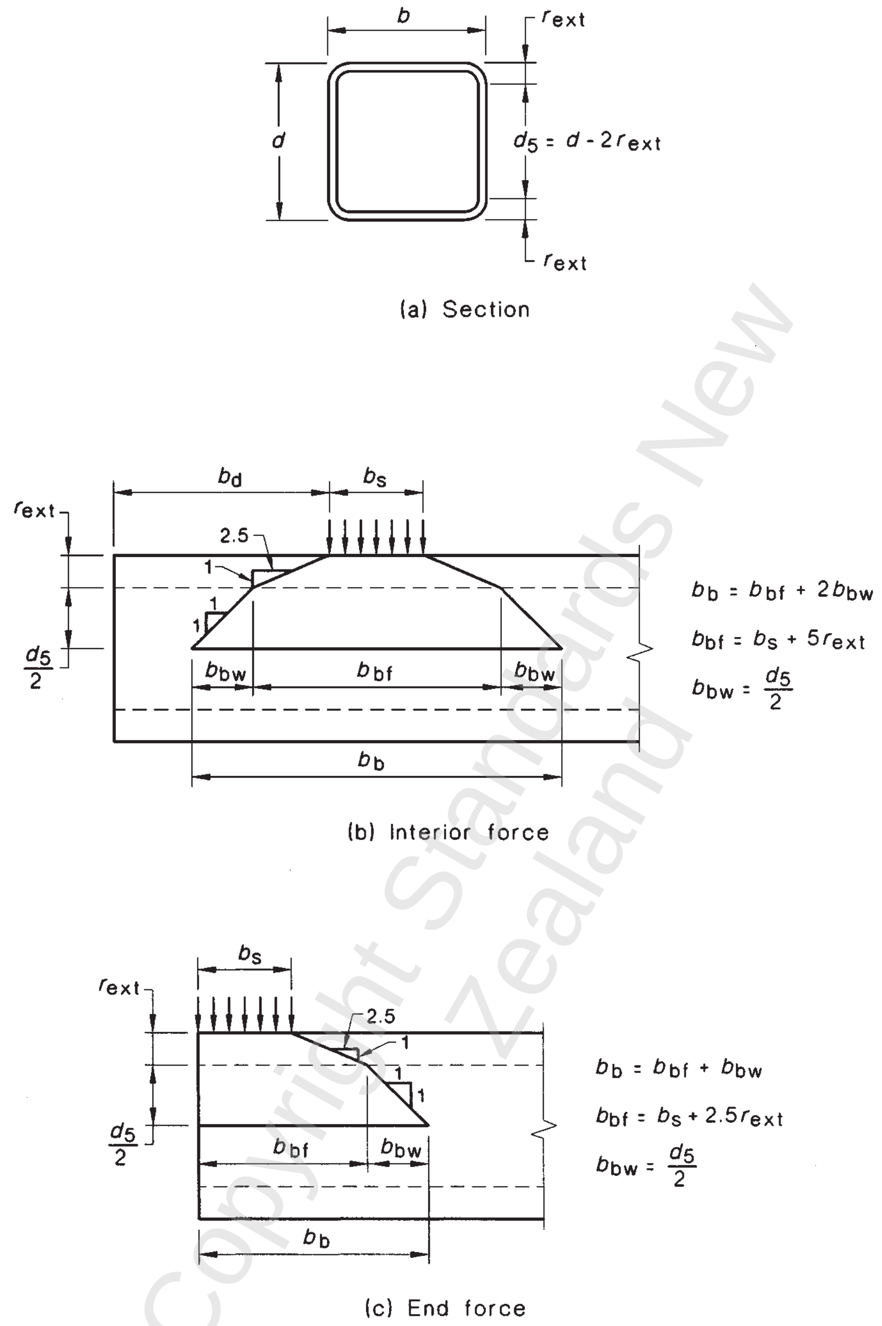

Figure 5.13.1.3– Rectangular and square hollow sections – Dispersion of force through flange, radius and web

5.13.3 Bearing yield capacity

5.13.3.1
The nominal bearing yield capacity (R_{by}) of a web shall be calculated as follows:

$$R_{by} = 1.25 b_{bf} t_w f_y \qquad \text{(Eq. 5.13.3(1))}$$

where b_{bf} is the bearing width shown in figure 5.13.1.1, except that for square and rectangular hollow sections, the nominal bearing yield capacity (R_{by}) of both webs shall be calculated as follows:

$$R_{by} = 2\, b_b\, t\, f_y\, \alpha_p \qquad \text{(Eq. 5.13.3(2))}$$

where

b_b = bearing width (see figures 5.13.1.3(b) and (c))

α_p = coefficient used to calculate the nominal bearing yield capacity (R_{by}) for square and rectangular hollow sections.

NOTE – Guidance on the nominal yield capacity of a stiffened web in bearing in the presence of bending moment and axial force is given in Appendix J.

5.13.3.2
The coefficient (α_p) shall be determined as follows:

(a) For interior bearing, where b_d is greater than or equal to $1.5 d_5$:

$$\alpha_p = \frac{0.5}{k_s}\left[1 + \left(1 - \alpha_{pm}^2\right)\left(1 + \frac{k_s}{k_v} - \left(1 - \alpha_{pm}^2\right)\frac{0.25}{k_v^2}\right)\right]$$

where

b_d = distance from the stiff bearing to the end of the member (see figure 5.13.1.3(b))

d_5 = flat width of web (see figure 5.13.1.3(a))

α_{pm} = coefficient used to calculate α_p

$$= \frac{1}{k_s} + \frac{0.5}{k_v}$$

k_s = ratio used to calculate α_p and α_{pm}

$$= \frac{2 r_{ext}}{t} - 1$$

k_v = ratio of flat width of web (d_5) to thickness (t) of section

$$= \frac{d_5}{t}$$

r_{ext} = outside radius of section (see figure 5.13.1.3(a)).

The bearing width (b_b) shall be calculated as follows:

$$b_b = b_s + 5 r_{ext} + d_5$$

(b) For end bearing, where b_d is less than $1.5d_5$:

$$\alpha_p = \sqrt{\left(2 + k_s^2\right)} - k_s$$

The bearing width (b_b) shall be calculated as follows:

$$b_b = b_s + 2.5r_{ext} + \frac{d_5}{2}$$

5.13.4 Bearing buckling capacity

The nominal bearing buckling capacity (R_{bb}) of a web without transverse stiffeners shall be taken as the axial load capacity, determined in accordance with section 6, using $\alpha_b = 0.5$ and $k_f = 1.0$, for a compression member of area $t_w b_b$ and slenderness ratio $L_e/r = 2.5d_1/t_w$,

where

b_b is the total bearing width obtained by dispersions at a slope of 1:1 from b_{bf} to the neutral axis, if available, as shown in figure 5.13.1.1,

except that,

for square and rectangular hollow sections, the slenderness ratio $L_e/r = 3.5d_5/t_w$ for interior bearing ($b_d \geq 1.5d_5$) and $L_e/r = 3.8d_5/t_w$ for end bearing ($b_d < 1.5d_5$), and b_b is as shown in figure 5.13.1.3.

NOTE – Guidance on the nominal bearing buckling capacity (R_{bb}) of a stiffened web with a bearing load between the stiffeners in the presence of bending moment and axial force is given in Appendix J.

5.13.5 Combined bending and bearing of rectangular and square hollow sections

Rectangular and square hollow sections subjected to combined bending moment and bearing force shall satisfy 5.2, 5.13.2, and either:

$$1.2\left(\frac{R^*}{\phi R_b}\right) + \left(\frac{M^*}{\phi M_s}\right) \leq 1.5 \text{ for } \frac{b_s}{b} \geq 1.0 \text{ and } \frac{d_5}{t_w} \leq 30$$

or

$$0.8\left(\frac{R^*}{\phi R_b}\right) + \left(\frac{M^*}{\phi M_s}\right) \leq 1.0 \text{ for } \frac{b_s}{b} < 1.0 \text{ and/ or } \frac{d_5}{t_w} > 30$$

where

ϕ = the strength reduction factor (see table 3.3)

R_b = nominal bearing capacity of a web specified in 5.13.2

M_s = nominal section moment capacity determined in accordance with 5.2

b_s = stiff bearing length

b = total width of section.

5.14 DESIGN OF LOAD BEARING STIFFENERS

5.14.1 Yield capacity

When a load bearing stiffener is required, it shall satisfy:

$$R^* \leq \phi R_{sy}$$

where

R^* = the design bearing force or design reaction, including the effects of any shear forces applied directly to the stiffener,

ϕ = the strength reduction factor (see table 3.3)

R_{sy} = the nominal yield capacity of the stiffened web
= $R_{by} + A_s f_{ys}$

R_{by} = the nominal bearing yield capacity (see 5.13.3.1)

R^* = the design bearing force or design reaction, including the effects of any shear forces applied directly to the stiffener,

A_s = the area of the stiffener in contact with the flange

f_{ys} = the yield stress of the stiffener.

5.14.2 Buckling capacity

5.14.2.1

When a load bearing stiffener is required, it shall satisfy:

$$R^* \leq \phi R_{sb}$$

where

ϕ = the strength reduction factor (see table 3.3)

R_{sb} = the nominal buckling capacity of the stiffened web, determined in accordance with section 6, using α_b = 0.5 and k_f = 1.0, for a compression member whose radius of gyration is taken about the axis parallel to the web.

5.14.2.2

The effective section of the compression member shall be taken as the area of the stiffener, together with a length of web on each side of the centre-line not greater than the lesser of:

$$\frac{17.5 t_w}{\sqrt{\left(\frac{f_y}{250}\right)}} \quad \text{and} \quad \frac{s}{2}\text{, if available}$$

5.14.2.3
The effective length (L_e) of the compression member used in calculating the buckling capacity (R_{sb}) shall be determined as either:

$$L_e = 0.7 d_1$$

where the flanges are restrained by other structural elements against rotation in the plane of the stiffener, or:

$$L_e = d_1$$

if either of the flanges is not so restrained.

5.14.3 Outstand of stiffeners

Unless the outer edge of a flat stiffener is continuously stiffened, the stiffener outstand from the face of a web (b_{es}) shall satisfy:

$$b_{es} \le \frac{15 t_s}{\sqrt{\left(\frac{f_{ys}}{250}\right)}}$$

where

t_s = the thickness of the stiffener

f_{ys} = the yield stress of the stiffener used in design.

5.14.4 Fitting of load bearing stiffeners

5.14.4.1
A load bearing stiffener shall be fitted to provide a tight and uniform bearing against the loaded flange, unless welds are provided between the flange and stiffener for the purpose of transmitting the concentrated force or reaction. Where a point of concentrated force is directly over a support, this provision shall apply to both flanges.

5.14.4.2
Load bearing stiffeners shall be provided with sufficient welds or bolts to transmit their share of the design bearing force or design reaction (R^*) to the web.

5.14.5 Design for torsional end restraint

When load bearing stiffeners are the sole means of providing torsional end restraint at the supports of a member, the second moment of area of a pair of stiffeners (I_s), about the centre-line of the web, shall be such that $I_s \ge I_{cr}$ for full section restraint (F), from 5.4.2.1, or $I_s \ge 0.2\, I_{cr}$ for partial section restraint (P), from 5.4.2.2, where I_{cr} is given by:

$$I_{cr} = \frac{\alpha_t}{1000}\left(\frac{d^3 t_f R^*}{F^*}\right)$$

where

$$\alpha_t = \left(\frac{230}{(L_e / r_y)} - 0.60\right) \quad \text{and} \quad 0 \le \alpha_t \le 4$$

R^* = the design reaction at the bearing point

F^* = the total design load on the member between supports

t_f = the thickness of the critical flange (see 5.5)

(L_e/r_y) = the load bearing stiffener slenderness ratio used in 5.14.2.

5.15 DESIGN OF INTERMEDIATE TRANSVERSE WEB STIFFENERS

5.15.1 General

Intermediate transverse web stiffeners shall extend between each flange and shall terminate no further from a flange than 4 times the web thickness.

NOTE – Intermediate stiffeners need only be provided on one side of a web.

5.15.2 Spacing

5.15.2.1 Interior panels

The spacing (s) of intermediate web stiffeners which define internal panels shall satisfy 5.10.4 or 5.10.5.

5.15.2.2 End panels

An end panel shall be provided with an end post which satisfies 5.15.9, unless the width (s) of the end panel is reduced so that its shear buckling capacity (V_b), calculated by using α_d = 1.0 in 5.11.5.2, satisfies 5.11.1 and 5.12.2.

5.15.3 Minimum area

An intermediate web stiffener not subject to external loads or moments shall have an area A_s which satisfies:

$$A_s \geq 0.5\gamma A_w(1-\alpha_v)\left\{\left(\frac{s}{d_p}\right)-\frac{\left(\frac{s}{d_p}\right)^2}{\sqrt{\left[1+\left(\frac{s}{d_p}\right)^2\right]}}\right\}\left[\frac{V^*}{\phi V_b}\right]$$

where

α_v = the value determined in accordance with 5.11.5.2;

γ = 1.0 for a pair of stiffeners;

= 1.8 for a single angle stiffener;

= 2.4 for a single plate stiffener;

V_b = nominal shear buckling capacity from 5.11.5.2.

5.15.4 Buckling capacity

5.15.4.1
An intermediate web stiffener shall satisfy:

$$V^* \leq \phi(R_{sb} + V_b)$$

where

ϕ = the strength reduction factor (see table 3.3)

V_b = nominal shear buckling capacity specified in 5.11.5.2 for a stiffened web using $\alpha_d = 1.0$ and $\alpha_f = 1.0$

R_{sb} = nominal buckling capacity of the intermediate stiffener determined in accordance with 5.14.2.

5.15.4.2
The effective length (L_e) of the compression member used in calculating R_{sb} shall be taken as:

$$L_e = d_1$$

5.15.5 Minimum stiffness

An intermediate web stiffener not subject to external loads or moments shall have a minimum second moment of area (I_s) about the centre-line of the web such that:

$$I_s \geq 0.75 d_1 t^3{}_w \qquad \text{for } \frac{s}{d_1} > \sqrt{2}\text{; and}$$

$$I_s \geq \frac{(1.5 d^3{}_1 t^3{}_w)}{s^2} \qquad \text{for } \frac{s}{d_1} > \sqrt{2}$$

5.15.6 Outstand of stiffeners

The outstand (b_{es}) of an intermediate web stiffener shall satisfy 5.14.3.

5.15.7 External forces

5.15.7.1 Increase in stiffness
Where an intermediate stiffener is used to transfer design forces (F_n^*) normal to the web or design moments $\left(M^* + F_p^* e\right)$ acting normal to the web (including moments $F_p^* e$ caused by any eccentric force F_p^* parallel to the web), the minimum value of I_s in 5.15.5 shall be increased by:

$$\frac{d_1^4\{2F_n^* + [(M^* + F_p^* e)/d_1]\}}{\phi E d_1 t_w}$$

5.15.7.2 Increase in strength

When an intermediate stiffener is required to carry a transverse load parallel to the web, it shall be designed as a load bearing stiffener in accordance with 5.14.

5.15.8 Connection of intermediate stiffeners to web

The web connections of intermediate transverse stiffeners not subject to external loading shall be designed to resist a design shear force per unit length, in kilonewtons per millimetre (kN/mm), of not less than:

$$\frac{0.0008(t_w)^2 f_{yw}}{b_{es}}$$

where b_{es} is the outstand width of the stiffener from the face of the web, in millimetres, t_w is the web thickness, in millimetres, and f_{yw} is the yield stress of the web in MPa (N/mm^2).

5.15.9 End posts

When an end post is required by 5.15.2.2, it shall be formed by a load bearing stiffener and a parallel end plate. The load bearing stiffener shall be designed in accordance with 5.14, and shall be no smaller than the end plate. The area of the end plate shall satisfy:

$$A_{ep} \geq \frac{d_1[(V^* / \phi) - \alpha_v V_w]}{8 e f_y}$$

where

α_v = is given in 5.11.5.2

V_w = is given in 5.11.4

e = the distance between the end plate and the load bearing stiffener.

5.16 DESIGN OF LONGITUDINAL WEB STIFFENERS

5.16.1 General

Longitudinal web stiffeners shall be continuous or shall extend between and be attached to transverse web stiffeners.

5.16.2 Minimum stiffness

5.16.2.1

When a longitudinal stiffener is required at a distance $0.2d_2$ from the compression flange, it shall have a second moment of area (I_s) about the face of the web such that:

$$I_s \geq 4 d_2 t_w^3 \left[1 + \frac{4A_s}{d_2 t_w} \left(1 + \frac{A_s}{d_2 t_w} \right) \right]$$

where A_s is the area of the stiffener.

5.16.2.2

When a second longitudinal stiffener is required at the neutral axis of the section, it shall have a second moment of area (I_s) about the face of the web such that:

$$I_s \geq d_2 t_w^3$$

NOTES

6 MEMBERS SUBJECT TO AXIAL COMPRESSION

6.1 DESIGN FOR AXIAL COMPRESSION

A concentrically loaded member subject to a design axial compression force (N^*) shall satisfy both:

$N^* \leq \phi N_s$, and

$N^* \leq \phi N_c$

where

ϕ = the strength reduction factor (see table 3.3 (1))

N_s = the nominal section capacity determined in accordance with 6.2.1.1 and, where appropriate, 6.2.1.2

N_c = the nominal member capacity determined in accordance with 6.3.

6.2 NOMINAL SECTION CAPACITY

6.2.1 General

6.2.1.1 Member not subject to inelastic action

The nominal section capacity (N_s) of a concentrically loaded compression member not subject to inelastic action shall be calculated as follows:

$$N_s = k_f A_n f_y \quad \text{(Eq. 6.2.1)}$$

where

k_f = the form factor given in 6.2.2

A_n = the net area of the cross section, except that for sections with penetrations or unfilled holes that reduce the section area by less than 100 $\{1 - [f_y / (0.85 f_u)]\}$%, the gross area may be used. Deductions for fastener holes shall be made in accordance with 9.1.10

f_y = the minimum value of yield stress from 2.1.1 for any element, or alternatively a weighted average value for all elements of the cross section.

6.2.1.2 Member subject to inelastic action

Equation 6.2.1 provides an upper limit for the nominal section capacity (N_s) of a concentrically loaded compression member. This limit may be lowered by the slenderness of the web, the category of the member, or the bending moment distribution, in accordance with:

(a) Clause 12.8.3.1 for a member subject to earthquake loads or effects or moment redistribution;

(b) Clause 8.4.3.3 for a member designed by the plastic method of design.

6.2.2 Form factor

The form factor (k_f) shall be calculated as follows:

$$k_f = \frac{A_e}{A_g}$$

where

A_e = the effective area

A_g = the gross area of the section.

The effective area (A_e) shall be calculated from the gross area by summing the effective areas of the individual elements, whose effective widths are specified in 6.2.4.

6.2.3 Plate element slenderness

6.2.3.1

The slenderness (λ_e) of a flat plate element shall be calculated as follows:

$$\lambda_e = \frac{b}{t}\sqrt{\left(\frac{f_y}{250}\right)}$$

where

b = the clear width of the element outstand from the face of the supporting plate element, or the clear width of the element between the faces of the supporting plate elements

t = the thickness of the plate.

6.2.3.2

For circular hollow sections, the element slenderness (λ_e) shall be calculated as follows:

$$\lambda_e = \left(\frac{d_o}{t}\right)\left(\frac{f_y}{250}\right)$$

where

d_o = the outside diameter of the section

t = the wall thickness of the section.

6.2.4 Effective width

6.2.4.1

The effective width (b_e) of a flat plate element of clear width (b), or the effective outside diameter (d_e) of a circular hollow section of outside diameter (d_o), shall be calculated from the value of the element slenderness (λ_e) given in 6.2.3 and the element yield slenderness limit (λ_{ey}) given in table 6.2.4.

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NOTE – For circular hollow sections, the effective inside diameter equals the effective outside diameter minus twice the actual wall thickness.

Table 6.2.4 – Values of plate element yield slenderness limit

Case number	Plate element type	Longitudinal edges supported	Residual stresses (see Notes)	Yield slenderness limit (λ_{ey})
1	Flat	One (Outstand)	SR HR LW, CF HW	16 16 15 14
	2		SR Both LW, CF HW	45 HR 45 40 35
3	Circular hollow sections		SR HR, CF LW HW	82 82 82 82

NOTE –

(1) SR – stress relieved
CF – cold-formed
HW – heavily welded longitudinally
HR – hot-rolled or hot-finished
LW – lightly welded longitudinally

(2) Welded members whose compressive residual stresses are less than 40 MPa shall be considered to be lightly welded.

6.2.4.2
The effective width (b_e) for a flat plate element shall be calculated as follows:

$$b_e = b\left(\frac{\lambda_{ey}}{\lambda_e}\right) \leq b$$

6.2.4.3
Alternatively, the effective width (b_e) for a flat plate element may be obtained from the following:

$$b_e = b\left(\frac{\lambda_{ey}}{\lambda_e}\right)\sqrt{\left(\frac{k_b}{k_{bo}}\right)} \leq b$$

where k_b is the elastic buckling coefficient for the element.

For a flat plate element supported along both longitudinal edges:

$$k_{bo} = 4.0$$

and for a flat plate element supported along one longitudinal edge (outstand):

$$k_{bo} = 0.425$$

The elastic buckling coefficient (k_b) for the flat plate element shall be determined from a rational elastic buckling analysis of the whole member as a flat plate assemblage.

6.2.4.4
The effective outside diameter (d_e) for a circular hollow section shall be the lesser of:

$$d_e = d_o\sqrt{\left(\frac{\lambda_{ey}}{\lambda_e}\right)} \leq d_o\text{, and}$$

$$d_e = d_o\left(\frac{3\lambda_{ey}}{\lambda_e}\right)^2$$

6.3 NOMINAL MEMBER CAPACITY

6.3.1 Definitions

For the purpose of this clause, the definitions below apply.

Geometrical slenderness ratio. The geometrical slenderness ratio (L_e/r), taken as the effective length (L_e), specified in 6.3.2, divided by the radius of gyration (r) calculated for the gross section about the relevant axis.

Length. The actual length (L) of an axially loaded compression member, taken as the length centre-to-centre of intersections with restraints, or the cantilevered length in the case of a free-standing member.

NOTE –

(1) Refer to 1.3 for definition of a restraint of a member subject to axial compression.

(2) A restraint may be effective about only one principal axis or about both principal axes.

6.3.2 Effective length

The effective length (L_e) of a compression member shall be determined as follows:

$$L_e = k_e L$$

where

k_e = the member effective length factor determined in accordance with 4.8.3

L = the length from 6.3.1.

6.3.3 Nominal capacity of a member of constant cross section

The nominal member capacity (N_c) of a member of constant cross section shall be determined as follows:

$$N_c = \alpha_c N_s$$

where

N_s = the nominal section capacity, determined in accordance with 6.2.1.1

$$\lambda_n = \left(\frac{L_e}{r}\right)\sqrt{(k_f)}\sqrt{\left(\frac{f_y}{250}\right)}$$

k_f = the form factor determined in accordance with 6.2.2

α_c = the member slenderness reduction factor.

Values of the member slenderness reduction factor (α_c) may be obtained directly from table 6.3.3(2), using the value of the modified member slenderness (λ_n) and the appropriate member section constant (α_b) given in table 6.3.3(1).

Alternatively, α_c may be calculated as follows:

$$\alpha_c = \xi \left\{ 1 - \sqrt{\left[1 - \left(\frac{90}{\xi\lambda} \right)^2 \right]} \right\}$$

$$\xi = \frac{\left(\frac{\lambda}{90}\right)^2 + 1 + \eta}{2\left(\frac{\lambda}{90}\right)^2}$$

$$\lambda = \lambda_n + \alpha_a \alpha_b$$

$$\eta = 0.00326\,(\lambda - 13.5) \geq 0$$

$$\alpha_a = \frac{2100(\lambda_n - 13.5)}{\lambda^2_n - 15.3\lambda_n + 2050}$$

α_b = the appropriate member section constant given in table 6.3.3(1).

6.3.4 Nominal capacity of a member of varying cross section

The nominal member capacity (N_c) of a member of varying cross section shall be determined using the provisions of 6.3.3, provided that the following are satisfied:

(a) The nominal section capacity (N_s) is the minimum value for all cross sections along the length of the member; and

(b) The modified member slenderness (λ_n), given in 6.3.3, is replaced by the following:

$$\lambda_n = 90\sqrt{\left(\frac{N_s}{N_{om}}\right)}$$

where N_{om} is the elastic buckling load of the member in axial compression determined using a rational elastic buckling analysis.

Table 6.3.3(1) – Values of compression member section constant

Section type	Form factor k_f	Compression member section constant (α_b)
UB and UC sections, hot-rolled (flange thickness up to 40 mm) Box sections, welded	$k_f \leq 1.0$	0
UB and UC sections, hot-rolled (flange thickness over 40 mm)	$k_f \leq 1.0$	1.0
RHS and CHS, hot-formed	$k_f = 1.0$	–1.0
RHS and CHS, cold-formed (stress relieved)	$k_f < 1.0$	–0.5
RHS and CHS, cold-formed (non-stress relieved)	$k_f \leq 1.0$	–0.5
H and I sections, welded from flame cut plate (flange thickness up to 40 mm)	$k_f = 1.0$ $k_f < 1.0$	0 0.5
H and I sections, welded from flame cut plate (flange thickness over 40 mm)	$k_f = 1.0$ $k_f < 1.0$	0 1.0
H and I sections, welded from as-rolled plate (flange thickness up to 40 mm)	$k_f \leq 1.0$	0.5
H and I sections, welded from as-rolled plate (flange thickness over 40 mm)	$k_f \leq 1.0$	1.0
Tee sections, flame cut from universal sections Angles Channels, hot-rolled	$k_f = 1.0$	0.5
Tee sections, flame cut from universal sections Angles Channels, hot-rolled	$k_f < 1.0$	1.0

Table 6.3.3(2) – Values of member slenderness reduction factor (α_c)

Modified member slenderness (λ_n)	Compression member section constant (α_b)				
	–1.0	**–0.5**	**0**	**0.5**	**1.0**
0	1.000	1.000	1.000	1.000	1.000
5	1.000	1.000	1.000	1.000	1.000
10	1.000	1.000	1.000	1.000	1.000
15	1.000	0.998	0.995	0.992	0.990
20	1.000	0.989	0.978	0.967	0.956
25	0.997	0.979	0.961	0.942	0.923
30	0.991	0.968	0.943	0.917	0.888
35	0.983	0.955	0.925	0.891	0.853
40	0.973	0.940	0.905	0.865	0.818
45	0.959	0.924	0.884	0.837	0.782
50	0.944	0.905	0.861	0.808	0.747
55	0.927	0.885	0.836	0.778	0.711
60	0.907	0.862	0.809	0.746	0.676
65	0.886	0.837	0.779	0.714	0.642
70	0.861	0.809	0.748	0.680	0.609
75	0.835	0.779	0.715	0.646	0.576
80	0.805	0.746	0.681	0.612	0.545
85	0.772	0.711	0.645	0.579	0.516
90	0.737	0.675	0.610	0.547	0.487
95	0.700	0.638	0.575	0.515	0.461
100	0.661	0.600	0.541	0.485	0.435
105	0.622	0.564	0.508	0.457	0.412
110	0.584	0.528	0.477	0.431	0.389
115	0.546	0.495	0.448	0.406	0.368
120	0.510	0.463	0.421	0.383	0.348
125	0.476	0.434	0.395	0.361	0.330
130	0.445	0.406	0.372	0.341	0.313
135	0.416	0.381	0.350	0.322	0.297
140	0.389	0.357	0.330	0.304	0.282
145	0.364	0.336	0.311	0.288	0.268
150	0.341	0.316	0.293	0.273	0.255
155	0.320	0.298	0.277	0.259	0.242
160	0.301	0.281	0.263	0.246	0.231
165	0.283	0.265	0.249	0.234	0.220
170	0.267	0.251	0.236	0.222	0.210
175	0.252	0.238	0.224	0.212	0.200

➤

Table 6.3.3(2) – Values of member slenderness reduction factor (α_c) (continued)

Modified member slenderness (λ_n)	Compression member section constant (α_b) –1.0	–0.5	0	0.5	1.0
180	0.239	0.225	0.213	0.202	0.192
185	0.226	0.214	0.203	0.193	0.183
190	0.214	0.203	0.193	0.184	0.175
195	0.204	0.194	0.185	0.176	0.168
200	0.194	0.185	0.176	0.168	0.161
205	0.184	0.176	0.168	0.161	0.154
210	0.176	0.168	0.161	0.154	0.148
215	0.167	0.161	0.154	0.148	0.142
220	0.160	0.154	0.148	0.142	0.137
225	0.153	0.147	0.142	0.137	0.132
230	0.146	0.141	0.136	0.131	0.127
235	0.140	0.135	0.131	0.126	0.122
240	0.134	0.130	0.126	0.122	0.118
245	0.129	0.125	0.121	0.117	0.114
250	0.124	0.120	0.116	0.113	0.110
255	0.119	0.116	0.112	0.109	0.106
260	0.115	0.111	0.108	0.105	0.102
265	0.110	0.107	0.104	0.102	0.099
270	0.106	0.103	0.101	0.098	0.096
275	0.102	0.100	0.097	0.095	0.092
280	0.099	0.096	0.094	0.092	0.089
285	0.095	0.093	0.091	0.089	0.087
290	0.092	0.090	0.088	0.086	0.084
295	0.089	0.087	0.085	0.083	0.081
300	0.086	0.084	0.082	0.081	0.079
305	0.083	0.082	0.080	0.078	0.077
310	0.081	0.079	0.077	0.076	0.074
315	0.078	0.077	0.075	0.074	0.072
320	0.076	0.074	0.073	0.071	0.070
340				0.064	
370				0.054	
400				0.047	
450				0.037	
500				0.031	
550				0.025	
600				0.021	

6.4 LACED AND BATTENED COMPRESSION MEMBERS

6.4.1 Design forces

If a compression member composed of 2 or more main components which are parallel is intended to act as a single member, the main components and their connections shall be proportioned to resist a design transverse shear force (V^*) applied at any point along the length of the member in the most unfavourable direction. The design transverse shear force (V^*) shall be calculated as follows:

$$(V^*) = \frac{\pi\left(\frac{N_s}{N_c} - 1\right)N^*}{\lambda_n} \geq 0.01N^*$$

where

N_s = the nominal section capacity of the compression member given by 6.2.1

N_c = the nominal member capacity of the compression member given by 6.3.3

N^* = the design axial force applied to the compression member

λ_n = the modified member slenderness.

The modified member slenderness (λ_n) of a battened compression member shall be determined using 6.4.3.2 and 6.3.3.

6.4.2 Laced compression members

6.4.2.1 Slenderness ratio of a main component

The maximum slenderness ratio $(L_e/r)_c$ of a main component, based on its minimum radius of gyration and the length between consecutive points where lacing is attached, shall not exceed the lesser of 50 or 0.6 times the slenderness ratio of the member as a whole.

6.4.2.2 Slenderness ratio of a laced compression member

The slenderness ratio shall be calculated by assuming that the main components act as an integral member but shall not be taken as less than 1.4 $(L_e/r)_c$.

6.4.2.3 Lacing angle

The angle of inclination of the lacing to the longitudinal axis of the member shall be within the following limits:

(a) 50° to 70° for single lacing;

(b) 40° to 50° for double lacing.

6.4.2.4 Effective length of lacing element

The effective length of a lacing element shall be taken as the distance between the inner welds or fasteners for single lacing, and 0.7 times this distance for double lacing which is connected by welds or fasteners.

6.4.2.5 Slenderness ratio limit of a lacing element

The slenderness ratio of a lacing element shall not exceed 140.

6.4.2.6 Mutually opposed lacing

6.4.2.6.1
Single lacing systems mutually opposed in direction on opposite sides of 2 main components shall not be used unless the resulting torsional effects are calculated and allowed for in the design.

6.4.2.6.2
Double lacing systems and single lacing systems mutually opposed in direction on opposite sides of 2 main components shall not be combined with members or diaphragms perpendicular to the longitudinal axis of the compression member, except for tie plates as specified in 6.4.2.7, unless all actions resulting from the deformation of the compression member are calculated and allowed for in the design.

6.4.2.7 Tie plates

6.4.2.7.1
Tie plates shall be provided at the ends of the lacing system, at points where the lacing system is interrupted, and at connections with other members. End tie plates shall have a width measured along the axis of the member of not less than the perpendicular distance between the centroids of their connections to the main components. Intermediate tie plates shall have a width of not less than three-quarters of this distance.

6.4.2.7.2
A tie plate and its connections shall be treated as battens for design purposes (see 6.4.3). The thickness of a tie plate shall not be less than 0.02 times the distance between the innermost lines of welds or fastenings, except where the tie plate is effectively stiffened at the free edges. In the latter case, the edge stiffeners shall have a slenderness ratio less than 170.

6.4.2.8 Members resisting load combinations including earthquake loads
The appropriate requirements of 12.9.8 shall also apply.

6.4.3 Battened compression members

6.4.3.1 Slenderness ratio of a main component
The maximum slenderness ratio $(L_e/r)_c$ of a main component, based on its minimum radius of gyration and the length between consecutive points where battens are attached, shall not exceed the lesser of 50, or 0.6 times the slenderness ratio of the member as a whole determined using 6.4.3.2.

6.4.3.2 Slenderness ratio of battened compression member
The slenderness ratio $(L_e/r)_{bn}$ of a battened compression member about the axis normal to the plane of the battens shall be calculated as follows:

$$\left(\frac{L_e}{r}\right)_{bn} = \sqrt{\left[\left(\frac{L_e}{r}\right)_m^2 + \left(\frac{L_e}{r}\right)_c^2\right]}$$

where

$\left(\frac{L_e}{r}\right)_m$ = the slenderness ratio of the whole member about the above axis calculated by assuming that the main components act as an integral member

$\left(\frac{L_e}{r}\right)_c$ = the maximum slenderness ratio of the main component, determined in accordance with 6.4.3.1.

The slenderness ratio $(L_e/r)_{bp}$ of a battened compression member about the axis parallel to the plane of the battens shall be taken as not less than 1.4 $(L_e/r)_c$.

6.4.3.3 Effective length of a batten
The effective length of an end batten shall be taken as the perpendicular distance between the centroids of the main components. The effective length of an intermediate batten shall be taken as 0.7 times the perpendicular distance between the centroids of the main components.

6.4.3.4 Maximum slenderness ratio of a batten
The slenderness ratio of a batten shall not exceed 180.

6.4.3.5 Width of a batten

6.4.3.5.1
The width of an end batten shall be not less than the greater of the distance between the centroids of the main components and twice the width of the narrower main component.

6.4.3.5.2
The width of an intermediate batten shall be not less than the greater of half the distance between the main components and twice the width of the narrower main component.

6.4.3.6 Thickness of a batten
The thickness of a batten shall be not less than 0.02 times the minimum distance between the innermost lines of welds or fasteners, except where the batten is effectively stiffened at the free edges. In this case, the edge stiffeners shall have a slenderness ratio of not greater than 170, where the radius of gyration is taken about the axis parallel to the member axis.

6.4.3.7 Loads on battens
The batten and its connections shall be designed to transmit simultaneously to the main components a design longitudinal shear force (V_1^*) calculated as follows:

$$V_1^* = \frac{V^* s_b}{n_b d_b}$$

and a design bending moment (M^*) calculated as follows:

$$M^* = \frac{V^* s_b}{2n_b}$$

where

V^* = the design transverse shear force specified in 6.4.1

s_b = the longitudinal centre-to-centre distance between the battens

n_b = the number of parallel planes of battens

d_b = the lateral distance between the centroids of the welds or fasteners.

6.4.3.8 Members resisting load combinations including earthquake loads
The requirements of 12.9.8 shall also apply.

6.5 COMPRESSION MEMBERS BACK TO BACK

6.5.1 Components separated

6.5.1.1
This clause applies to compression members composed of 2 angle, channel or tee-section components discontinuously separated back to back by a distance not exceeding that required for the end gusset connection. If such a member is designed as a single integral member, then it shall comply with 6.5.1.2 to 6.5.1.5.

Where such members are subject to load combinations including earthquake loads, the appropriate requirements of 12.9.8 shall also apply.

6.5.1.2 Configuration
The configuration of the main components shall be of sections with the same cross section arranged symmetrically with their corresponding rectangular axes aligned.

6.5.1.3 Slenderness
The slenderness of the compression member about the axis parallel to the connected surfaces shall be calculated in accordance with 6.4.3.2.

6.5.1.4 Connection
The main components shall be interconnected by fasteners. Where the components are connected together, the member shall be designed as a battened compression member in accordance with 6.4.3. The main components shall be connected at intervals so that the member is divided into at least 3 bays of approximately equal length. At the ends of the member, the main components shall be connected by not less than 2 fasteners in each line along the length of the member, or by welds of equivalent strength.

6.5.1.5 Design forces
The interconnecting fasteners shall be designed to transmit a design longitudinal shear force between the components induced by the transverse shear force (V^*) given in 6.4.1. The design longitudinal shear force (V_1^*) per connection shall be taken as follows:

$$V_1^* = 0.25V^* \left(\frac{L_e}{r}\right)_c$$

where

$(L_e/r)_c$ is the slenderness ratio of the main component between the interconnections.

6.5.2 Components in contact

6.5.2.1 Application
This clause applies to compression members composed of 2 angle, channel or tee-section components back-to-back or separated by continuous steel packing. If such a member is designed as a single integral member, then it shall comply with 6.5.2.2 to 6.5.2.5.

6.5.2.2 Configuration
The main components shall be of sections with the same cross section, arranged symmetrically with their corresponding rectangular axes aligned.

6.5.2.3 Slenderness

The slenderness of the compression member about the axis parallel to the connected surfaces shall be calculated in accordance with 6.4.3.2.

6.5.2.4 Connection

The main components shall be connected at intervals so that the member is divided into at least 3 bays of approximately equal length. At the ends of the member, the main components shall be interconnected by not less than 2 fasteners in each line along the length of the member, or by welds of equivalent strength.

6.5.2.5 Design forces

The interconnecting fasteners or welds shall be designed to transmit a longitudinal shear force between the components induced by the transverse shear force (V^*) determined from 6.4.1. The design longitudinal shear force (V_1^*) per connection shall be as specified in 6.5.1.5.

6.6 DISCONTINUOUS ANGLE, CHANNEL AND TEE SECTION COMPRESSION MEMBERS NOT REQUIRING DESIGN FOR MOMENT ACTION

6.6.1 Scope and general

6.6.1.1

Discontinuous compression members composed of angles, channels and tee sections which meet the conditions of 6.6.1 to 6.6.5 and are not subject to transverse loads may be designed for the design compression load (N^*) alone in accordance with 6.2 and 6.3.

6.6.1.2

In applying 6.6.2 to 6.6.5, the length L shall be taken as the distance between the intersection of centroidal axes or the intersections of the setting out lines of the bolts, and r is the radius of gyration about the relevant axis. Axes are defined in table 6.6.

6.6.1.3

Members which are subject to compression load alone and which do not meet the requirements of 6.6 shall be designed for the combined effects of the compression load and the moment induced by eccentric transfer of this compression load at the end connections.

6.6.2 Single angle compression members of moderate to high slenderness

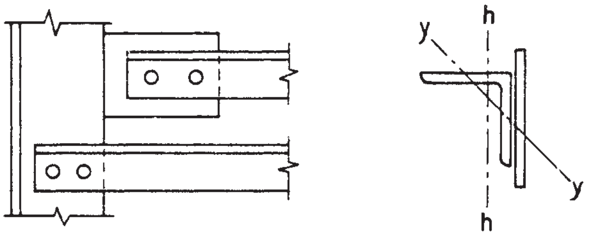
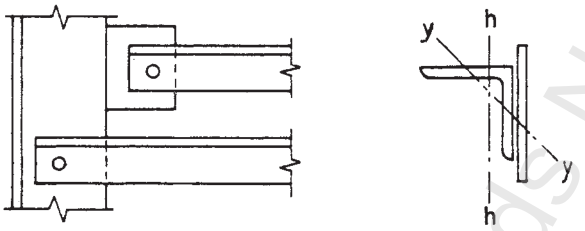
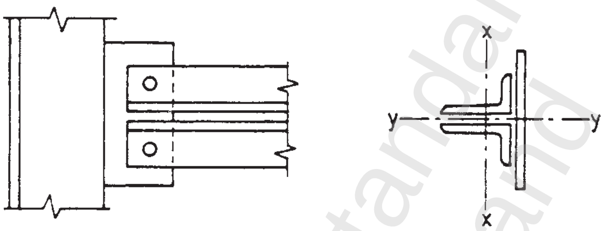
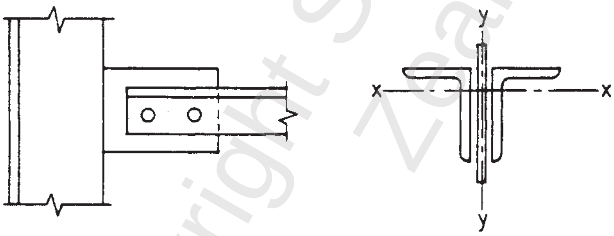
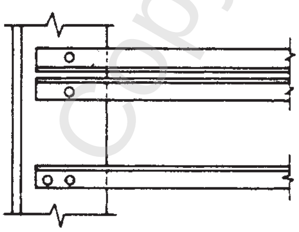
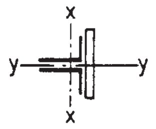
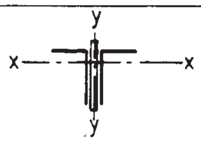
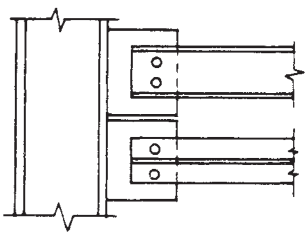
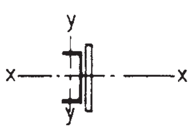
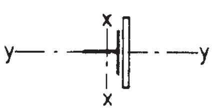
For single angle compression members with $L/r_y \geq 150$ and which are connected to a gusset plate or directly to another member at each end by:

(a) Two or more fasteners in line along the angle or by an equivalent welded connection, the geometrical slenderness ratio (L_e/r) for use in 6.3 shall be taken as not less than 0.85 L/r_y or 0.7 L/r_h + 30;

(b) A single fastener at each end, the geometrical slenderness ratio (L_e/r) for use in 6.3 shall be taken as not less than 1.0 L/r_y or 0.7 L/r_h + 30 and the design compression load (N^*) shall be not greater than 80 % of the design member compression capacity (ϕN_c) calculated in accordance with 6.3.

In 6.6.2, the rectangular h-axis is defined as parallel to the loaded leg.

Table 6.6 – Discontinuous angle, channel and tee section compression members

Clause	Condition	Slenderness ratio
6.6.2(a)		$0.85\,L/r_y$ $0.7\,L/r_h + 30$ only applicable for $L/r_y \geq 150$
6.6.2(b)		$1.0\,L/r_y$ $0.7\,L/r_h + 30$ only applicable for $L/r_y \geq 150$
6.6.3(a)		$0.85\,L/r_y$ $0.85\,L/r_x$ $0.7\,L/r_x + 30$ only applicable for $L/r_x \geq 150$
6.6.3(b)		$0.85\,L/r_y$ $0.85\,L/r_x$ $0.7\,L/r_x + 30$
6.6.3(c)	 	$1.0\,L/r_y$ $1.0\,L/r_x$ $0.7\,L/r_x + 30$ only applicable for $L/r_x \geq 150$
		$1.0\,L/r_y$ $1.0\,L/r_x$ $0.7\,L/r_x + 30$
6.6.4 6.6.5	 or connected directly to section 	$0.85\,L/r_y$ only applicable for $L/r_y \geq 150$
		$1.0\,L/r_y$ $1.0\,L/r_x$ $0.7\,L/r_x + 30$ only applicable for $L/r_x \geq 150$

6.6.3 Double angle compression members

For double angle compression members connected:

(a) To one side of a gusset plate by one or more fasteners in each angle or by an equivalent weld and with $L/r_x \geq 150$, the geometrical slenderness ratio (L_e/r) for use in 6.3 shall be taken as not less than 0.85 L/r about the minor principal axis or 0.7 L/r_x + 30.

(b) To both sides of a gusset plate by 2 or more fasteners in line along the angle or by an equivalent weld, the geometrical slenderness ratio (L_e/r) for use in 6.3 shall be taken as not less than 0.85 L/r about the minor principal axis or 0.7 L/r_x + 30.

(c) Directly to one or both sides of another member, the geometrical slenderness ratio (L_e/r) for use in 6.3 shall be taken as not less than 1.0 L/r about the minor principal axis or 0.7 L/r_x + 30. The moment induced by connection eccentricity shall be considered for a member connected to one side of the support when $L/r_x < 150$ for the member.

6.6.4 Single channel compression members

For a single channel compression member with $L/r_y \geq 150$ connected by its web only, the connection shall be by symmetrically placed fasteners or an equivalent weld and the geometrical slenderness ratio (L_e/r) for use in 6.3 shall be taken as not less than 0.85 L/r about its minor principal axis.

6.6.5 T-section compression members

For a T-section with $L/r_x \geq 150$ connected by its flange the connection shall be at least 2 symmetrically placed fasteners or an equivalent weld and the geometrical slenderness ratio (L_e/r) for use in 6.3 shall be not less than 1.0 L/r about its minor principal axis, or 0.7 L/r_x + 30.

6.7 RESTRAINING ELEMENTS

6.7.1 General requirements

The members and the connections of restraining systems required to brace compression members shall be determined by analysing the structure for its design loads, including any notional horizontal loads (see 3.2.4), from the points where the loads arise to anchorage or reaction points, and by designing the members and connections as specified in 6.7.2 and 6.7.3.

NOTE – When the bolt hole size is larger than the nominal diameter specified in 14.3.5.2.1, tensioned property class 8.8 bolts (category 8.8/TB) to 15.2.4 and 15.2.5 shall be used in bolted connections of restraining systems required to brace compression members.

6.7.2 Restraining members and connections

6.7.2.1

At each restrained cross section of a compression member, the restraining members and their connections which are required to brace the compression member shall be designed for the greater of:

(a) The restraining member forces specified in 6.7.1; and

(b) 0.025 times the maximum design axial compression force in the member at the position of the support,

except where the restraints are more closely spaced than is required to ensure that $N^* \leq \phi N_c$.

6.7.2.2

When the restraint spacing is less, then a lesser force may be designed for. The actual arrangement of restraints shall be assumed to be equivalent to a set of restraints which will ensure that $N^* \leq \phi N_c$. Each equivalent restraint shall correspond to an appropriate group of the actual restraints. This group shall then be designed as a whole to transfer to anchorage or reaction points the transverse force determined for the position of the equivalent restraint.

6.7.3 Parallel restrained compression members

When a series of parallel compression members is restrained by a line of restraints, each restraining element shall be designed to transfer to anchorage or reaction points the transverse force specified in 6.7.2, except that 0.025 times the axial compression force shall be replaced by the sum of 0.025 times the axial force in the connected compression member and 0.0125 times the sum of the axial forces in the connected compression members beyond, with no more than seven members considered in the summation.

7 MEMBERS SUBJECT TO AXIAL TENSION

7.1 DESIGN FOR AXIAL TENSION

A member subject to an axial tension force (N^*) shall satisfy:

$$N^* \leq \phi N_t$$

where

ϕ = the strength reduction factor (see table 3.3)

N_t = the nominal section capacity in tension determined in accordance with 7.2.

7.2 NOMINAL SECTION CAPACITY

7.2.1

The nominal section capacity of a tension member shall be taken as the lesser of:

$N_t = A_g f_y$; and .... (Eq. 7.2.1)

$N_t = 0.85 k_{te} A_n f_u$ .... (Eq. 7.2.2)

where

A_g = the gross area of the cross section

f_y = the minimum value of yield stress from 2.1.1 for any element, or alternatively a weighted average value for all elements of the cross section

k_{te} = the correction factor for distribution of forces determined in accordance with 7.3

A_n = the net area of the cross section, obtained by deducting from the gross area the sectional area of all penetrations and holes, including fastener holes. The deduction for all fastener holes shall be made in accordance with 9.1.10. For threaded rods, the net area shall be taken as the tensile stress area of the threaded portion, as defined in AS 1275.

f_u = the tensile strength from 2.1.2.

7.2.2

For yielding regions of tension members in seismic-resisting or associated structural systems, the ratio of net area of the cross section to gross area of the cross section shall comply with 12.9.4.2.

7.3 DISTRIBUTION OF FORCES

7.3.1 End connections providing uniform force distribution

Where for design purposes it is assumed that the tensile force is distributed uniformly to a tension member, the end connections shall satisfy the following:

(a) The connections shall be made to each part of the member and shall be symmetrically placed about the centroidal axis of the member, and

(b) Each part of the connection shall be proportioned to transmit at least the maximum design force carried by the connected part of the member. For members subject to design loads including earthquake loads, the overstrength force shall be used as required from 12.9.1.2.

For connections satisfying these requirements, the value of k_{te} shall be taken as 1.0.

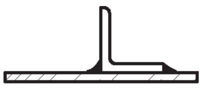
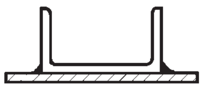
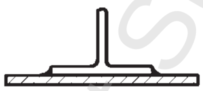
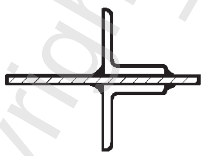
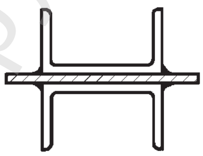
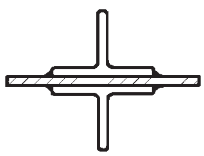
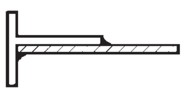
7.3.2 End connections providing non-uniform force distribution

If the end connections of a tension member do not satisfy the requirements of 7.3.1, then the member shall be designed to comply with section 8 using k_{te} = 1.0, except that 7.2 may be used for the following members:

(a) *Eccentrically-connected angles, channels and tees*

Eccentrically-connected angles, channels and tees may be designed in accordance with 7.2, using the appropriate value of k_{te} given in table 7.3.2. For yielding regions of category 1, 2 or 3 members in seismic-resisting or associated structural systems, only connection details with a correction factor (k_{te}) not less than 0.85 shall be used.

Table 7.3.2 – Correction factor (k_{te})

Case number	Configuration case	Correction factor (k_{te})
1		0.75 for unequal angles connected by the short leg 0.85 otherwise
2		0.85
3		0.90
4		1.0
5		1.0
6		1.0
7		0.75

NOTE – The plate to which the member is connected is shown hatched.

(b) *I-sections or channels connected by both flanges only*

A symmetrical rolled or built-up member of solid I-section or channel section connected by both flanges only may be designed in accordance with 7.2 using a value of k_{te} equal to 0.85, provided that:

(i) The length between the first and last rows of fasteners in the connection or, when the member is welded, the length of longitudinal weld provided to each side of the connected flanges shall be not less than the depth of the member; and

(ii) Each flange connection shall be proportioned to transmit at least half of the maximum design force carried by the connected member. For members subject to design loads including earthquake loads, the design actions shall be determined from 12.9.1.2.

7.4 TENSION MEMBERS WITH 2 OR MORE MAIN COMPONENTS

7.4.1 General

A tension member composed of 2 or more main components intended to act as a single member shall comply with 7.4.2 to 7.4.5 and with 12.9.8.

7.4.2 Design forces for connections

7.4.2.1

If a tension member is composed of 2 or more main components, the connections between the components shall be proportioned to resist the internal actions arising from the external design forces and bending moments (if any). The design forces for lacing bars, and the design forces and bending moment (if any) for battens, shall be considered as divided equally among the connection planes parallel to the direction of force.

7.4.2.2

The requirements of 12.9.8 shall also apply to members subject to design loads including earthquake loads.

7.4.3 Tension member composed of 2 components back-to-back

A tension member composed of 2 flats, angles, channels or tees, discontinuously connected back-to-back either in contact or separated by a distance not exceeding that required for the end gusset connection, shall comply with the following:

(a) *Where the components are separated.* They shall be connected either:

(i) Together at regular intervals along their length by welding, or bolting, so that the slenderness ratio of the individual components between connections does not exceed 300; or

(ii) By connections which comply with 6.5.1.4 and 6.5.1.5.

(b) *Where component members are in contact back-to-back.* They shall be connected together as required by 6.5.2.4 and 6.5.2.5.

The requirements of 12.9.8 shall also apply to members subject to design loads including earthquake loads.

7.4.4 Laced tension member

A tension member composed of 2 components connected by lacing shall comply with 6.4.2 except as follows:

(a) The slenderness ratio of the lacing elements shall not exceed 210;

(b) The slenderness ratio of a main component based on its minimum radius of gyration and the length between consecutive points where lacing is attached shall not exceed 300.

For tie plates, the requirements of 6.4.2.7 shall be satisfied except that the thickness of tie plates shall be not less than 0.017 times the distance between the innermost lines of connections.

The requirements of 12.9.8 shall also apply to members subject to design loads including earthquake loads.

7.4.5 Battened tension member

A tension member composed of 2 components connected by battens shall comply with 6.4.3 except as follows:

(a) The spacing of battens shall be such that the maximum slenderness ratio of each main component based on its minimum radius of gyration and the length between consecutive battens does not exceed 300;

(b) Battens attached by bolts shall be connected by not less than 2 bolts per connection and 6.4.3.7 shall not apply;

(c) Batten plates shall have a thickness of not less than 0.017 times the distance between the innermost lines of connections;

(d) Intermediate battens shall have a width of not less than half the effective width of end batten plates.

The appropriate requirements of 12.9.8 shall also apply to members subject to design loads including earthquake loads.

7.5 MEMBERS WITH PIN CONNECTIONS

The nominal capacity of a pin connection shall be determined in accordance with 9.5. A pin connection in a tension member shall comply with the following additional requirements:

(a) The thickness of an unstiffened element containing a hole for a pin connection shall be greater than or equal to 0.25 times the distance from the edge of the hole to the edge of the element measured at right angle to the axis of the member. This limit does not apply to the internal plies where the connected elements are clamped together by external nuts.

(b) The net area beyond a hole for a pin, parallel to or within 45° of the axis of the member, shall be greater than or equal to the net area required for the member;

(c) The sum of the areas of material each side of the hole for a pin, taken perpendicular to the axis of the member, shall be greater than or equal to 1.33 times the net area required for the member;

(d) Pin plates provided to increase the net area of a member or to increase the bearing capacity of a pin shall be arranged to avoid eccentricity and shall be proportioned to distribute the load from the pin into the member.

8 MEMBERS SUBJECT TO COMBINED ACTIONS

8.1 GENERAL

8.1.1

8.1.1.1
A member subject to uniaxial bending and axial actions need not be checked for combined actions to section 8 when the axial force is not significant, as defined by 8.1.4.

8.1.1.2
A member not satisfying 8.1.1.1 shall be proportioned so that its design actions specified in 8.2, in combination with the nominal section and member capacities (see sections 5, 6 and 7), satisfy 8.3 and 8.4, except where (a) or (b) is applicable:

(a) For category 1, 2 or 3 members which comply with equation 12.8.3.1 (clause 12.8.3.1(b)) and have full lateral restraint to 8.1.2.4, only the requirements of 8.3 need to be satisfied for the direction of loading in which equation 12.8.3.1 is complied with.

(b) For plastic design (see 4.6), only the requirements of 8.4.3 need to be satisfied.

8.1.2 Definition of a member

8.1.2.1 Member subject to combined bending and axial compression
A member subject to combined bending and axial compression is typically defined as:

(a) The length between adjacent points of support/restraint; or

(b) The length between a point of support/restraint and the adjacent point of support (for bending) or restraint (for compression or for both compression and bending), whichever is the closer; or

(c) The cantilever length from the point of support/restraint, in the case of a free-standing member.

NOTE – The cross section at a point of support/restraint is both supported for bending (see 1.3) and restrained for compression (see 1.3).

8.1.2.2 Member subject to combined bending and axial tension
A member subject to combined bending and axial tension is defined in 5.3.1.3.

8.1.2.3 Member with full lateral restraint
A member subject to combined bending and axial force (compression or tension) shall be considered to have full lateral restraint if it complies with 5.3.1.6.1, using the member length from 8.1.2.1 or 8.1.2.2, as appropriate.

8.1.2.4 Member containing one or more yielding regions
A member subject to combined bending and significant axial actions (see 8.1.4) and which contains one or more yielding regions shall have full lateral restraint.

NOTE – This means that all segments within the member must have full lateral restraint.

8.1.3 Restraining elements
Restraining elements at points of support (for bending) or points of restraint (for compression) shall be designed to 5.4.3 or 6.7, respectively.

Restraining elements at points of support/restraint and at points of restraint for both bending and compression shall be designed to 5.4.3 and 6.7.

8.1.4 Significant axial force

For application of section 8, the level of design axial force (N^*) shall be considered significant unless it complies with one of (a), (b) or (c) below:

(a) For a member subject to axial compression and either :

(i) Major principal x-axis bending with full lateral restraint in accordance with 8.1.2.3:

$N^* \leq 0.05\, \phi N_s$ if the member is an I-section or channel section

$N^* \leq 0.05\, \phi N_{cx}$ for all other sections

or

(ii) Bending in the major principal x-axis or the minor principal y-axis, for all sections except as covered by (c) :

$N^* \leq 0.05\, \phi N_{cy}$

(b) For a member subject to tension and bending in the major principal x-axis or the minor principal y-axis

$N^* \leq 0.05\, \phi N_t$

(c) For an angle to 8.4.6:

$N^* \leq 0.05 \phi N_{ch}$

where

N_{ch} is as defined in 8.4.6.1

N_{cx} is as defined in 8.4.2.2.1 for buckling about the major principal x-axis

N_{cy} is as defined in 8.4.4.1.1

N_s is as defined in 8.3.1

N_t is as defined in 8.3.1.

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8.1.5 Use of alternative design provisions

The alternative design provisions of 8.3.2.2 to 8.3.4.2, 8.4.2.2.2 and 8.4.4.1.2 may be used, provided all the following are satisfied:

(a) The member is either a doubly symmetric I-section or a rectangular or square hollow section; **and**

(b) Either:

(i) The cross section is compact, as determined from 5.2.3; or

(ii) The plate element slenderness of each flat plate element in uniform or non-uniform compression, calculated in accordance with 5.2.2.1, does not exceed the appropriate limit given in table 8.1;

and

(c) Either:

(i) The member is subject to design axial tension, or

(ii) The form factor (k_f) determined in accordance with 6.2.2, is unity, or

(iii) The design axial compression force (N^*) complies with equation 8.1.5;

$$\frac{N^*}{\phi N_s} \leq 1.9 - \frac{\left(\frac{d_1}{t}\sqrt{\frac{f_{yw}}{250}}\right)}{45}, \quad \text{subject to } 0 \leq \frac{N^*}{\phi N_s} \leq 1.0 \quad \text{.................................... (Eq. 8.1.5)}$$

NOTE –

(1) f_{yw} in equation 8.1.5 is that for the web.

(2) For values of (N^* / ϕN_s) ≤ 0.28, the less conservative equations 12.8.3.2(1) and 12.8.3.2(2) may be used, up to the web slenderness limit of 82, and substituting N^* for N_g^* therein.

and

(d) For 8.4.4.1.2 only, the member is not subject to transverse loads for which k_l > 1.0, where k_l is determined from table 5.6.3(2).

Table 8.1 – Values of plate element slenderness limits for accessing the alternative design provisions in section 8

Case number	Plate element type	Longitudinal edges supported	Residual stresses	Slenderness limit[(3)]
1	Flat (Uniform compression)	One	SR HR LW, CF, HW	11 10 9
2	Flat (Maximum compression at unsupported edge, zero stress or tension at supported edge)	One	SR HR LW, CF, HW	11 10 9
3	Flat (Uniform compression)	Both	Any	40
4	Flat (Either non-uniform compression or compression at one edge, tension at the other)	Both	Any	82

NOTE –

(1) SR – stress relieved
HR – hot-rolled or hot-finished
CF – cold formed
LW – lightly welded longitudinally
HW – heavily welded longitudinally

(2) Welded members whose compressive residual stresses are less than 40 MPa shall be considered to be lightly welded.

(3) The slenderness limits for case numbers 1, 2 and 3 correspond to the limits for a category 3 member from table 12.5.

8.1.6 Double bolted or welded single angles eccentrically loaded in compression

8.1.6.1
Eccentrically loaded single angles subject to design compression only and with slenderness ratios (L/r_y) < 150 shall be designed to 8.3 and 8.4.6.

8.1.6.2
Eccentrically loaded single angles subject to design compression only and with slenderness ratios (L/r_y) ≥ 150 shall be designed to 6.6.2.

8.2 DESIGN ACTIONS

8.2.1
For checking the section capacity at a section, the design axial force (N^*), which may be tension or compression, shall be the force at the section, and the design bending moments (M_x^*, M_y^*) shall be the bending moments at the section about the major *x*- and minor *y*- principal axes, respectively.

8.2.2
For checking the member capacity, the design axial force (N^*) shall be the maximum axial force in the member, and the design bending moments (M_x^*, M_y^*) shall be the maximum bending moments in the member.

8.2.3
M_x^*, M_y^* are the design bending moments resulting from frame action and transverse loading on the member, and include the second-order design bending moments resulting from the design loads acting on the structure and its members in their displaced and deformed configuration.

8.2.4
The design bending moments (M_x^*, M_y^*) shall be determined from one of the following methods of analysis:

(a) *First-order linear elastic analysis* – by modifying the first-order design bending moments, by using the appropriate moment amplification factors determined in accordance with 4.4.2 or 4.5.2 and 4.5.3.

(b) *Second-order elastic analysis* – in which the design bending moments (M^*) are obtained either directly, or by modifying the second-order end moments by using the moment amplification factors determined in accordance with Appendix E.

(c) *First-order plastic analysis* – in which the design bending moments (M^*) are obtained directly for frames where the elastic buckling load factor (λ_c) satisfies $\lambda_c \geq 5$ and the requirements of 4.6.4 are satisfied.

(d) *Second-order plastic analysis* – in which the design bending moments (M^*) are obtained directly for frames where the elastic buckling load factor (λ_c) satisfies $\lambda_c < 5$.

8.3 SECTION CAPACITY

8.3.1 General
The member shall satisfy 8.3.2, 8.3.3 and 8.3.4, as appropriate:

(a) For bending about the major principal *x*-axis only, sections at all points along the member shall have sufficient capacity to satisfy 8.3.2.

(b) For bending about the minor principal y-axis only, sections at all points along the member shall have sufficient capacity to satisfy 8.3.3.

(c) For bending about a non-principal axis, or bending about both principal axes, sections at all points along the member shall have sufficient capacity to satisfy 8.3.4.

In this section:

M_{sx}, M_{sy} = the nominal section moment capacities about the x- and y-axes respectively, determined in accordance with 5.2.

N_s = the nominal section axial capacity determined in accordance with 6.2 for axial compression, or 7.2 for axial tension (for which $N_s = N_t$).

8.3.2 Uniaxial bending about the major principal x-axis

8.3.2.1 General design provision

Where uniaxial bending occurs about the major principal x-axis, the following shall be satisfied:

$$M_x^* \leq \phi M_{rx}$$

where

ϕ = the strength reduction factor (see table 3.3)

M_{rx} = the nominal section moment capacity, reduced by axial force (tension or compression)

$$= M_{sx}\left(1 - \frac{N^*}{\phi N_s}\right)$$

8.3.2.2 Alternative design provision

Alternatively, for sections in which the cross section geometry and design axial force comply with 8.1.5, M_{rx} may be calculated as follows:

$$M_{rx} = 1.18 M_{sx}\left(1 - \sqrt{\frac{N^*}{\phi N_s}}\right) \leq M_{sx}$$

8.3.3 Uniaxial bending about the minor principal y-axis

8.3.3.1 General design provision

Where uniaxial bending occurs about the minor principal y-axis, the following shall be satisfied:

$$M_y^* \leq \phi M_{ry}$$

where

M_{ry} = the nominal section moment capacity, reduced by axial force (tension or compression)

$$M_{ry} = M_{sy}\left(1 - \frac{N^*}{\phi N_s}\right)$$

8.3.3.2 Alternative design provision

Alternatively, for sections in which the cross section geometry and design axial force comply with 8.1.5, M_{ry} may be calculated as follows:

$$M_{ry} = 1.19 M_{sy}\left(1 - \left(\frac{N^*}{\phi N_s}\right)^2\right) \leq M_{sy} \text{ for I – sections; or}$$

$$M_{ry} = 1.18 M_{sy}\left(1 - \frac{N^*}{\phi N_s}\right) \leq M_{sy} \text{ for hollow sections}$$

8.3.4 Biaxial bending

8.3.4.1 General design provision

Where biaxial bending occurs, the following shall be satisfied:

$$\frac{N^*}{\phi N_s} + \frac{M_x^*}{\phi M_{sx}} + \frac{M_y^*}{\phi M_{sy}} \leq 1.0$$

8.3.4.2 Alternative design provision

Alternatively, for sections in which the cross section geometry and design axial force comply with 8.1.5, sections at all points along the member shall satisfy the following:

$$\left(\frac{M_x^*}{\phi M_{rx}}\right)^{\gamma} + \left(\frac{M_y^*}{\phi M_{ry}}\right)^{\gamma} \leq 1.0$$

where

$$M_{rx} = 1.18 M_{sx}\left(1 - \frac{N^*}{\phi N_s}\right) \leq M_{sx} \text{ for I – sections or hollow sections}$$

$$M_{ry} = 1.19 M_{sy}\left(1 - \left(\frac{N^*}{\phi N_s}\right)^2\right) \leq M_{sy} \text{ for I – sections; or}$$

$$M_{ry} = 1.18 M_{sy}\left(1 - \frac{N^*}{\phi N_s}\right) \leq M_{sy} \text{ for hollow sections}$$

$$\gamma = 1.4 + \left(\frac{N^*}{\phi N_s} \right) \leq 2.0$$

8.4 MEMBER CAPACITY

8.4.1 General

The member shall satisfy 8.4.2, 8.4.3 and 8.4.4, as appropriate:

(a) For a member bent about the major principal *x*-axis only and which has full lateral restraint in accordance with 8.1.2.3, or for a member bent about the minor principal *y*-axis only, the member shall satisfy the in-plane requirements of 8.4.2 for a frame analysed elastically, or 8.4.3 for a frame analysed plastically.

(b) For a member bent about the major principal *x*-axis only and which does not have full lateral restraint in accordance with 8.1.2.3, the member shall satisfy both the in-plane requirements of 8.4.2 and the out-of-plane requirements of 8.4.4.

(c) For a member bent about a non-principal axis, or bent about both principal axes, the member shall satisfy the biaxial bending requirements of 8.4.5.

8.4.2 In-plane capacity–elastic analysis

8.4.2.1 Application

This clause applies to a member analysed using an elastic method in accordance with 4.4 or an elastic method with redistribution in accordance with 4.5, or to a member in a statically determinate structure.

8.4.2.2 Compression members

8.4.2.2.1 *General design provision*

A member bent about a principal axis shall have sufficient in-plane capacity to satisfy the following:

$$M^* \leq \phi M_i$$

where

M^* = the design bending moment about the principal axis

ϕ = the strength reduction factor (see table 3.3)

M_i = the nominal in-plane member moment capacity

$$= M_s \left(1 \pm \frac{N^*}{\phi N_c} \right)$$

M_s = the nominal section moment capacity determined in accordance with 5.2 for bending about the same principal axis as the design bending moment

N^* = the design axial compressive force

N_c = the nominal member capacity in axial compression determined in accordance with 6.3 for buckling about the same principal axis, with the effective length factor (k_e) taken as 1.0 for both braced and sway members, unless a lower value is calculated for braced members from 4.8.3.1.

8.4.2.2.2 *Alternative design provision*

Alternatively, for members in which the cross section geometry and design axial force comply with 8.1.5, M_i may be calculated as follows:

$$M_i = M_s \left\{ \left[1 - \left(\frac{1+\beta_m}{2} \right)^3 \right] \left(1 - \frac{N^*}{\phi N_c} \right) + 1.18 \left(\frac{1+\beta_m}{2} \right)^3 \sqrt{\left(1 - \frac{N^*}{\phi N_c} \right)} \right\} \leq M_{rx} \text{ or } M_{ry} \text{ as appropriate}$$

where

β_m = the ratio of the smaller to the larger end bending moment, taken as positive when the member is bent in reverse curvature, for members without transverse load; or

β_m = β_t from 4.4.3.2.4 for members with transverse load

M_{rx} or M_{ry} = the nominal section moment capacity about the appropriate principal axis determined in accordance with 8.3.

8.4.2.3 Tension members

A member subject to a design axial tensile force (N^*) and a design bending moment (M^*) shall satisfy 8.3.

8.4.3 In-plane capacity–plastic analysis

8.4.3.1 Application

8.4.3.1.1

This clause applies only to compact doubly symmetric I-section members. When the distribution of moments in a frame is determined using a plastic method of analysis in accordance with 4.6, then the design axial compressive force (N^*) in any member of the frame which is assumed to contain a plastic hinge shall satisfy the member slenderness requirements of 8.4.3.2, and the web slenderness requirements of 8.4.3.3.

NOTE – The restrictions of 8.4.3.2 and 8.4.3.3 do not apply to design axial tension force.

8.4.3.1.2

The design plastic moment capacity reduced by axial force (tension or compression) for compact doubly symmetric-sections shall be as specified in 8.4.3.4.

8.4.3.2 Member slenderness

8.4.3.2.1
The design axial compressive force (N^*) in every member assumed to contain a plastic hinge shall satisfy the following:

$$\frac{N^*}{\phi N_s} \le \left\{ \frac{0.263\left(\beta_m + 1\right)^{0.88}}{e^{\left(0.19/\left(\beta_m+1\right)\right)}} \right\}^{\lambda_{EYC}}$$

where

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$$\lambda_{EYC} = \sqrt{\frac{N_s}{N_{oL}}}$$

β_m = the ratio of the smaller to the larger end bending moment, taken as positive when the member is bent in reverse curvature

N_s = the nominal section capacity in axial compression determined in accordance with 6.2.

$$N_{oL} = \frac{\pi^2 EI}{L^2}$$

I = the second moment of area for the axis about which the design moment acts

L = the actual length of the member.

8.4.3.2.2
A member subject to design axial compression force which does not satisfy 8.4.3.2.1 shall not contain plastic hinges, although it shall be permissible to design the member as an elastic member in a plastically analysed structure to satisfy the requirements of 8.4.2.

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8.4.3.3 Web slenderness

The design axial compressive force (N^*) in every member assumed to contain a plastic hinge shall satisfy the following:

$$\frac{N^*}{\phi N_s} \le 0.60 - \left[\frac{d_1}{t}\frac{\sqrt{(f_y/250)}}{137}\right] \quad \text{for webs where } 45 \le \frac{d_1}{t}\sqrt{\left(\frac{f_y}{250}\right)} \le 82 \quad \text{..... (Eq. 8.4.3.3 (1))}$$

or

$$\frac{N^*}{\phi N_s} \le 1.91 - \left[\frac{d_1}{t}\frac{\sqrt{(f_y/250)}}{27.4}\right] \le 1.0 \text{ for webs where } 30 \le \frac{d_1}{t}\sqrt{\left(\frac{f_y}{250}\right)} < 45 \quad \text{.......... (Eq. 8.4.3.3 (2))}$$

or

$$\frac{N^*}{\phi N_s} \le 1.0 \quad \text{for webs where } 0 \le \frac{d_1}{t}\sqrt{\left(\frac{f_y}{250}\right)} \le 30 \quad \text{...... (Eq. 8.4.3.3(3))}$$

Members which have webs for which $(d_1/t)\sqrt{f_y/250}$ exceeds 82 shall not contain plastic hinges, although it shall be permissible to design such a member as an elastic member in a plastically analysed structure to satisfy the requirements of 8.4.2.

8.4.3.4 Plastic moment capacity

The design plastic moment capacity (ϕM_{pr}) reduced for axial force (tension or compression) shall be calculated as follows:

$$\phi M_{prx} = 1.18\phi M_{spx}\left(1-\frac{N^*}{\phi N_s}\right) \leq \phi M_{spx}$$

for members bent about the major principal *x*-axis, and

$$\phi M_{pry} = 1.19\phi M_{spy}\left[1-\left(\frac{N^*}{\phi N_s}\right)^2\right] \leq \phi M_{spy}$$

for members bent about the minor principal *y*-axis

where M_{spx} and M_{spy} are the nominal section moment capacities determined in accordance with 5.2.1.

8.4.4 Out-of-plane capacity

8.4.4.1 Compression members

8.4.4.1.1 *General design provision*

A member subject to a design axial compressive force (N^*) and a design bending moment (M_x^*) about its major principal *x*-axis, and which does not have full lateral restraint in accordance with 8.1.2.3, shall satisfy 8.4.2.2 and also the following:

$$M_x^* \leq \phi M_{ox}$$

where

ϕ = the strength reduction factor (see table 3.3)

M_{ox} = the nominal out-of-plane member *x*-axis moment capacity

$$= M_{bx}\left(1-\frac{N^*}{\phi N_{cy}}\right)$$

M_{bx} = the nominal member moment capacity of the segment or member without full lateral restraint and bent about the major principal *x*-axis, determined in accordance with 5.6 using a moment modification factor (α_m) appropriate to the distribution of design bending moments along the segment or member

N_{cy} = the nominal member capacity in axial compression, determined in accordance with 6.3 for buckling about the minor principal *y*-axis, with the effective length factor (k_{ey}) taken as 1.0 for both braced and sway members, unless a lower value is calculated for braced members from 4.8.3.1.

8.4.4.1.2 *Alternative design provision*

Alternatively, for members in which the cross-section geometry, design axial force and nature of applied loads comply with 8.1.5, M_{ox} may be calculated as follows:

$$M_{ox} = \alpha_{bc} M_{bxo} \sqrt{\left[\left(1 - \frac{N^*}{\phi N_{cy}}\right)\left(1 - \frac{N^*}{\phi N_{oz}}\right)\right]} \leq M_{rx}$$

where

$$\frac{1}{\alpha_{bc}} = \frac{1-\beta_m}{2} + \left(\frac{1+\beta_m}{2}\right)^3 \left(0.4 - 0.23\frac{N^*}{\phi N_{cy}}\right)$$

Amd 1 June '01 M_{bxo} = the nominal member moment capacity of the segment or member calculated using a_m = 1.0 in accordance with 5.6

N_{cy} = the nominal member capacity in axial compression, determined in accordance with 6.3 for buckling about the minor principal *y*-axis

β_m = as defined in 8.4.2.2.2

N_{oz} = the nominal elastic torsional buckling capacity of the member, calculated as follows:

$$N_{oz} = \frac{GJ + \left(\pi^2 EI_w / L_z^2\right)}{(I_x + I_y)/A}$$

E, *G* = the elastic moduli

A, I_w, I_x, I_y, and J = the section constants

Amd 1 June '01 L_z = the distance between restraints which effectively prevents twist of the section about its centroid (i.e. full or partial restraints).

NOTE – Values of *E* and *G*, and expressions for I_w and *J* are given in Appendix H.

8.4.4.2 Tension members

A member subject to a design axial tension force (N^*) and a design bending moment (M_x^*) about its major principal x-axis, and which does not have full lateral restraint in accordance with 8.1.2.3, shall satisfy the following:

$$M_x^* \leq \phi M_{ox}$$

where

ϕ = the strength reduction factor (see table 3.3)

M_{ox} = the nominal out-of-plane member x-axis moment capacity

$$= M_{bx}\left(1 + \frac{N^*}{\phi N_t}\right) \leq M_{rx}$$

M_{bx} = the nominal member moment capacity defined in 8.4.4.1.1

N_t = the nominal section capacity in axial tension determined in accordance with 7.2

M_{rx} = the nominal section moment capacity, reduced by axial tension, determined in accordance with 8.3.2.1 or 8.3.2.2, as appropriate.

8.4.5 Biaxial bending capacity

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8.4.5.1 Compression members or members subject to zero axial force

A member subject to a design axial compressive force (N^*) or to $N^* = 0$, and design bending moments $M_x{}^*$ and $M_y{}^*$ about the major x- and minor y- principal axes respectively shall satisfy the following :

$$\left(\frac{M_x^*}{\phi M_{cx}}\right)^{1.4} + \left(\frac{M_y^*}{\phi M_{iy}}\right)^{1.4} \leq 1.0$$

where

for a member which has full lateral restraint in accordance with 8.1.2.3

M_{cx} = M_{ix} (M_i about the major principal x-axis), from 8.4.2.2; or

for a member which does not have full lateral restraint

M_{cx} = the lesser of the nominal in-plane member moment capacity (M_{ix}) and the nominal out-of-plane member moment capacity (M_{ox}) for bending about the major principal x-axis, determined in accordance with 8.4.2.2 and 8.4.4.1 respectively

M_{iy} = the nominal in-plane member moment capacity, determined in accordance with 8.4.2.2, for bending about the minor principal y-axis.

8.4.5.2 Tension members

A member subject to a design axial tension force (N^*) and design bending moments (M_x^*) and (M_y^*) about the major *x*- and minor *y*- principal axes shall satisfy the following:

$$\left(\frac{M_x^*}{\phi M_{tx}}\right)^{1.4} + \left(\frac{M_y^*}{\phi M_{ry}}\right)^{1.4} \leq 1.0$$

where

for a member which has full lateral restraint in accordance with 8.1.2.3

M_{tx} = M_{rx}, from 8.3.2; or

for a member which does not have full lateral restraint

M_{tx} = the lesser of M_{rx}, from 8.3.2, and M_{ox}, from 8.4.4.2

M_{ry} = the nominal section moment capacity reduced by axial tension, determined in accordance with 8.3.3.1 or 8.3.3.2, as appropriate.

8.4.6 Double bolted or welded single angles eccentrically loaded in compression

8.4.6.1

Single angle members loaded only in compression, which are connected with at least 2 bolts or welded at their ends, are loaded through one leg (see figure 8.4.6) and which are not eligible from 8.1.6.1 for design in accordance with 6.6.2, shall be designed to satisfy 8.3 and the following:

$$\frac{N^*}{\phi N_{ch}} + \frac{M_h^*}{\phi M_{bx} \cos\alpha} \leq 1.0 \quad \text{(Eq. 8.4.6)}$$

where

N^* = the design axial compression force in the member

M_h^* = the design bending moment acting about the rectangular *h*-axis parallel to the loaded leg

ϕ = the strength reduction factor (see table 3.3)

N_{ch} = the nominal member capacity in axial compression, determined in accordance with 6.3, of a single angle compression member buckling with $L_e = L$ about the rectangular *h*-axis parallel to the loaded leg

M_{bx} = the nominal member moment capacity, determined in accordance with 5.6, for an angle bent about the major principal *x*-axis, using a factor α_m appropriate to the distribution of design bending moment along the member

α = the angle between the *x*-axis and the *h*-axis.

8.4.6.2

For equal leg angles, where $L/t \leq (210 + 175\beta_m)\left(\frac{250}{f_y}\right)$, M_{bx} may be taken as M_{sx},

where

L = the member length

t = the thickness of the angle

β_m = as defined in 5.3.2.4.

8.4.6.3

For other equal leg angles, M_{bx} may be determined by using 5.6.1.1.1 with:

$$M_o = \left(\frac{525t}{L}\right)\left(\frac{250}{f_y}\right) M_{sx}$$

8.4.6.4

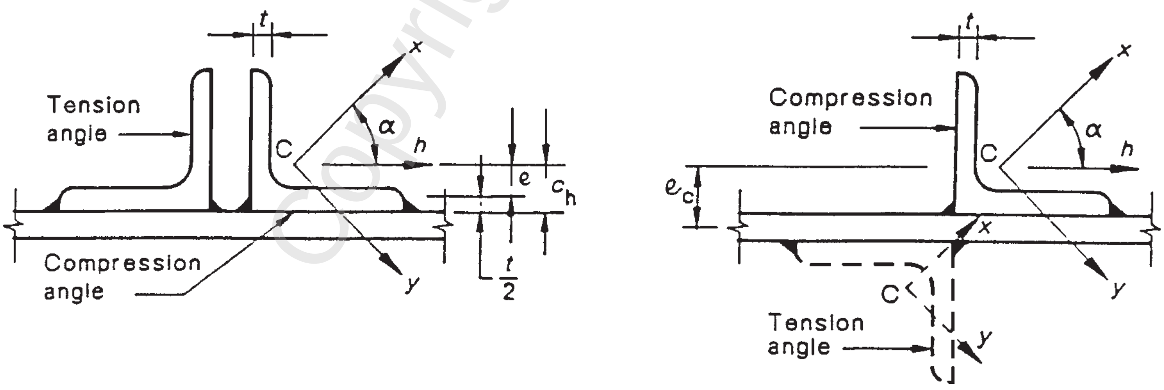
The design end bending moment (M_h^*) shall be calculated from a rational elastic analysis, or shall be taken as equal to N^*e, resulting from the out-of-plane eccentricity (e) of the design axial force (N^*) in the member,

where

e = $\left(c_h - \frac{t}{2}\right)$, for angles on the same side of the truss chord

= e_c, for angles on opposite sides of the truss chord, or for discontinuous angles connected to a gusset plate.

(See figure 8.4.6).



$e = c_h - \frac{t}{2}$

$e = e_c$

(a) Angles on same side

(b) Angles on opposite side or angle connected to gusset plate

Figure 8.4.6 – Single angles loaded through one leg and designed to 8.4.6

9 CONNECTIONS

9.1 GENERAL

9.1.1 Requirements for connections

Connection elements consist of connection components (cleats, gusset plates, brackets, connecting plates) and connectors (bolts, pins, and welds). The connections in a structure shall be proportioned so as to be consistent with the assumptions made in the analysis of the structure and to comply with this section and, where appropriate, with 12.3 and 13.7. Connections shall be capable of transmitting the calculated design actions and of achieving any ductility demand as required from section 12.

9.1.2 Classification of connections

9.1.2.1 Connections in rigid construction

The connections shall comply with 4.2.2.1.

The joint deformations shall be such that they have no significant influence on the distribution of design actions nor on the overall deformation of the frame.

9.1.2.2 Connections in semi-rigid construction

The connections shall comply with 4.2.2.2.

Connections between members in semi-rigid construction shall provide a predictable degree of interaction between members, based on the actual load-deformation characteristics of the connection as determined experimentally.

9.1.2.3 Connections in simple construction

The connections shall comply with 4.2.2.3.

Connections between members in simple construction shall be capable of deforming to provide the required rotation at the connection. The connections shall not develop a level of restraining bending moment which adversely affects any part of the structure. The rotation capacity of the connection shall be provided by the detailing of the connection and shall have been demonstrated by appropriate experimental testing and/or rational analysis. The connection shall be considered as subject to reaction shear forces acting at an eccentricity appropriate to the connection detailing.

9.1.2.4 Connections in structures analysed by the method of elastic analysis with moment redistribution

Connections in structures analysed by the method of elastic analysis with moment redistribution shall comply with 4.5.7.2, in addition to the requirements of this section.

9.1.2.5 Connections in structures analysed by the plastic method

Connections in structures analysed by the plastic method shall comply with 4.6.3.3, in addition to the requirements of this section.

9.1.3 Design of connections

9.1.3.1

Each element in a connection shall be designed so that the structure is capable of resisting all design actions in accordance with 9.1.2. The design capacities of each element shall be not less than the calculated design actions.

9.1.3.2
Connections and the adjacent areas of members shall be designed by distributing the design actions so that they comply with the following requirements:

(a) The distributed design actions are in equilibrium with the design actions acting on the connection;

(b) The deformations in the connection are within the deformation capacities of the connection elements;

(c) All of the connection elements and the adjacent areas of members are capable of resisting the design actions acting on them, including inelastic demand from severe seismic effects where appropriate;

(d) The connection elements shall remain stable under the design actions and deformations, including inelastic demand from severe seismic effects where appropriate.

9.1.3.3
Design shall be on the basis of a rational procedure supported by experimental evidence.

9.1.3.4
Residual actions due to the installation of bolts need not be considered.

9.1.3.5
Gusset (and other) unstiffened plates designed to transfer compression forces shall be subject to rational analysis to :

(a) Satisfy the strength and stability requirements of section 8;

(b) Incorporate allowances for eccentric load effects;

Amd 2 Oct. '07 in accordance with 9.1.5.1

9.1.4 Minimum design actions on connections

9.1.4.1 Minimum design actions on connections not subject to earthquake loads or effects
Connections carrying design actions, except for lacing connections and connections to sag rods, purlins and girts, shall be designed to transmit the greater of:

(a) The design action in the member; and

(b) The minimum design actions expressed as a factor times the member design capacity for the minimum size of member required by the ultimate limit state, specified in Items (i) to (vii) below:

(i) Connections in rigid construction – a bending moment of 0.5 times the member design moment capacity.

(ii) Connections to beams in simple construction – a shear force of 0.15 times the member design shear capacity.

(iii) Connections at the ends of tension or compression members – a force of 0.3 times the member design capacity, except that for threaded rod acting as a bracing member, the minimum tension force shall be equal to the member design capacity.

(iv) Splices in members subject to axial tension – a force of 0.3 times the member design capacity in tension.

(v) Splices in members subject to axial compression – for ends prepared for full contact in accordance with 14.4.4.2, it shall be permissible to carry compressive actions by bearing on contact surfaces. When members are prepared for full contact to bear at splices, there

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shall be sufficient fasteners to hold all parts in place. The fasteners shall be sufficient to transmit a force of 0.15 times the member design strength in axial compression. Fasteners shall also be sufficient to transmit a minimum of 0.15 times the member flexural design strength in tension friction mode.

In addition, splices located between points of restraint shall be designed for the design axial force (N^*) plus a design bending moment not less than the design bending moment (M^*)

where

$$M^* = \frac{\delta N^* L_s}{1000}$$

δ = appropriate amplification factor δ_b or δ_s determined in accordance with 4.4.3.2 or 4.4.3.3

L_s = distance between points of effective lateral support.

When members are not prepared for full contact, the splice material and its fasteners shall be arranged to hold all parts in line and shall be designed to transmit a force of 0.3 times the member design capacity in axial compression.

(vi) Splices in flexural members – a bending moment of 0.3 times the member design capacity in bending. This provision shall not apply to splices designed to transmit shear force only.

A splice subjected to a shear force only shall be designed to transmit the design shear force together with any bending moment resulting from the eccentricity of the force with respect to the centroid of the connector group.

(vii) Splices in members subject to combined actions – a splice in a member subject to a combination of design axial tension or design axial compression and design bending moment shall satisfy (iv), or (v) and (vi) simultaneously.

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(c) Members shall be connected in such a way to meet the robustness requirements of AS/NZS 1170.0 Clause 6.2.3, with the design action applied along the longitudinal axis of the supported member.

9.1.4.2 Minimum design actions on connections subject to earthquake loads or effects
These are specified in 12.9.2.

9.1.5 Intersections

9.1.5.1
Members or components meeting at a joint shall be arranged to transfer the design actions between the parts and, wherever practicable, with their centroidal axes meeting at a point. Where there is eccentricity at joints, the members and connections shall be designed for the design bending moments which result, except as permitted by 6.6.

9.1.5.2
The disposition of fillet welds to balance the design actions about the centroidal axis or axes for end connections of single angle, double angle and similar type members is not required for statically loaded or seismically loaded members but is required for members and connection components subject to fatigue loading.

9.1.5.3
Eccentricity between the centroidal axes of angle members and the gauge lines for their bolted end connections may be neglected in statically loaded members, but must be considered in members and connection components subject to fatigue loading.

9.1.6 Choice of fasteners

9.1.6.1
Where slip in the serviceability limit state must be avoided in a connection, high-strength bolts in a friction-type joint (bolting category 8.8/*TF*), fitted bolts or welds shall be used.

9.1.6.2 Use of fully tensioned bolts
Fully tensioned bolts shall be used in the following instances:

(a) Where no joint slip whatsoever is allowed under the design loads (tension friction mode required).

This requires design for the serviceability limit state in accordance with 9.3.3.1, using the appropriate serviceability limit state design loads from the Loadings Standard.

(b) Where the connection is used instead of a welded connection which would require Class SP welds or welds in accordance with AS/NZS 1554.5. (Tension bearing mode generally required if replacing Class SP welds, tension friction mode if replacing welds in accordance with AS/NZS 1554.5).

(c) In the connections of lateral force – resisting systems which exceed 2 storeys in height, unless consideration is given to second-order effects arising from the use of snug-tight bolts in the connections of these systems.

9.1.6.3
For connections subject to earthquake loads, refer to the requirements of 12.9.3, 12.9.4 and 12.9.6.

9.1.6.4
Where a joint is subject to vibration, or repeated impact loading, high-strength bolts in a friction-type joint (bolting category 8.8/*TF*), locking devices or welds shall be used.

9.1.7 Combined connections

9.1.7.1
When non-slip fasteners (such as high-strength bolts in a friction-type connection or welds) are used in a connection in conjunction with slip-type fasteners (such as snug-tight bolts, or tensioned high-strength bolts in bearing-type connections), all of the design actions shall be assumed to be carried by the non-slip fasteners.

9.1.7.2
Where a mixture of non-slip fasteners is used, sharing of the design actions may be assumed. However, when welding is used in a connection in conjunction with other non-slip fasteners:

(a) Any design actions initially applied directly to the welds shall not be assumed to be distributed to fasteners added after the application of the design actions; and

(b) Any design actions applied after welding shall be assumed to be carried by the welds.

9.1.7.3
In connections forming part of a seismic-resisting system, sharing of actions between bolts and welds on the same element is not permitted.

9.1.8 Prying forces
Where bolts are required to carry a design tensile force, when specified by the design procedure, the bolts shall be proportioned to resist any additional tensile prying force (see definition in 9.2).

9.1.9 Connection components

9.1.9.1
Connection components (cleats, gusset plates, brackets, etc.) other than connectors shall have their capacities assessed using the provisions of sections 5, 6, 7, 8 and the appropriate provisions of section 12 (especially 12.9.7) as applicable.

9.1.9.2
When using sections 6 and 7 for design of connection components, take $\alpha_b = 0.5$ and determine the net area (A_n) by means of a rational design procedure which accounts for the distribution of axial force into the component.

9.1.10 Deductions for fastener holes

9.1.10.1 Hole area
In calculating the deductions to be made for holes for fasteners (including countersunk holes), the gross areas of the holes in the plane of their axes shall be used.

9.1.10.2 Holes not staggered
For holes that are not staggered, the area to be deducted shall be the maximum sum of the areas of the holes in any cross sections at right angles to the direction of the design action in the member.

9.1.10.3 Staggered holes
When holes are staggered, the area to be deducted shall be the greater of:

(a) The deduction for non-staggered holes; or

(b) The sum of the areas of all holes in any zig-zag line extending progressively across the member or part of the member, less ($s_p{}^2 t/4s_g$) for each gauge space in the chain of holes.

where

s_p = staggered pitch, the distance measured parallel to the direction of the design action in the member, centre-to-centre of holes in consecutive lines, (see figure 9.1.10.3(1))

t = thickness of the holed material

s_g = gauge, the distance measured at right angles to the direction of the design action in the member, centre-to-centre of holes in consecutive lines, (see figure 9.1.10.3(1)).

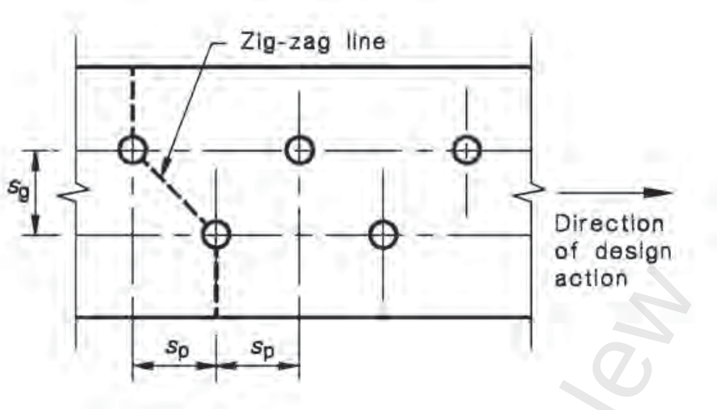

Figure 9.1.10.3(1) – Staggered holes

For sections such as angles with holes in both legs, the gauge shall be taken as the sum of the back marks to each hole, less the leg thickness (see figure 9.1.10.3(2)).

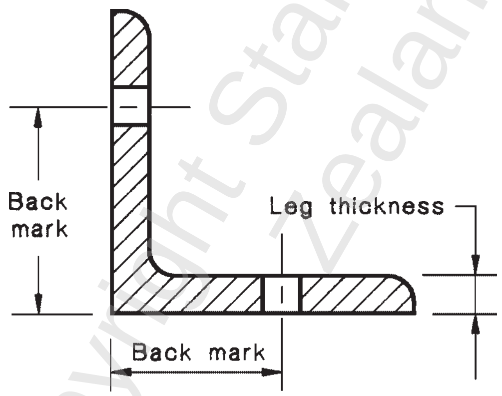

Figure 9.1.10.3(2) – Angles with holes in both legs

9.1.11 Hollow section connections

When design loads or effects from one member are applied to a hollow section at a connection, consideration shall be given to the local effects on the hollow section.

9.2 DEFINITIONS

For the purpose of this section, the definitions below apply.

BEARING-TYPE CONNECTION. Connection effected using either snug-tight bolts, or high-strength bolts tightened to induce a specified minimum bolt tension, in which the design load or effect is transferred by shear in the bolts and bearing on the connected parts at the ultimate limit state.

FRICTION-TYPE CONNECTION. Connection effected using high-strength bolts tightened to induce a specified minimum bolt tension such that the resultant clamping action transfers the design shear forces at the serviceability limit state acting in the plane of the common contact surfaces by the friction developed between the contact surfaces.

FULL TENSIONING. A method of installing and tensioning a bolt in accordance with 15.2.4 and 15.2.5.

IN-PLANE LOADING. Loading for which the design forces and bending moments are in the plane of the connection, such that the design actions induced in the connection components are shear forces only.

NON-SLIP FASTENERS. Fasteners which do not allow slip to occur between connected plates or members at the serviceability limit state so that the original alignment and relative positions are maintained.

OUT-OF-PLANE LOADING. Loading for which the design forces or bending moments result in design actions normal to the plane of the connection.

PIN. An unthreaded fastener manufactured out of round bar.

PRYING FORCE. Additional tensile force developed as a result of the flexing of a connection component in a connection subjected to tensile force. External tension force reduces the contact pressure between the component and the base, and bending in part of the component develops a prying force near the edge of the connection component.

SNUG TIGHT. The tightness of a bolt achieved by a few impacts of an impact wrench or by the full effort of a person using a standard podger spanner.

9.3 DESIGN OF BOLTS

9.3.1 Bolts and bolting category

9.3.1.1
The bolts and bolting categories listed in table 9.3.1 shall be designed in accordance with this clause, 9.4 and, where appropriate, with 12.9.4.

9.3.1.2
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Other property classes of bolts conforming to AS 1110, AS 1111 and AS 1559 may be designed in accordance with the provisions of 9.3, 9.4 and, where appropriate, with 12.9.4.

Table 9.3.1 – Bolts and bolting category

Bolting category	Bolt standard	Bolt property class	Method of tensioning	Minimum tensile strength (f_{uf}) MPa (Note (2))
4.6/*S*	AS/NZS 1111	4.6	Snug tight	400
8.8/*S*	AS/NZS 1252	8.8	Snug tight	830
8.8/*TB*	AS/NZS 1252	8.8	Full tensioning	830
8.8/*TF* (Note (1))	AS/NZS 1252	8.8	Full tensioning	830

NOTE –
(1) Special category used in connections where slip in the serviceability limit state is to be restricted (see 3.4.5 and 9.1.6).

(2) f_{uf} is the minimum tensile strength of the bolt as specified in a material supply Standard from 2.3.

9.3.2 Bolt ultimate limit states

9.3.2.1 Bolt in shear

A bolt subject to design shear force (V_f^*) shall satisfy:

$$V_f^* \leq \phi V_f$$

where

ϕ = strength reduction factor (see table 3.3)

V_f = nominal shear capacity of a bolt.

The nominal shear capacity of a bolt (V_f) shall be calculated as follows:

$$V_f = 0.62 f_{uf} k_r (n_n A_c + n_x A_o)$$

where

f_{uf} = minimum tensile strength of the bolt as specified in a material supply standard from 2.3 (see table 9.3.1)

k_r = reduction factor given in table 9.3.2.1 to account for the length of a bolted lap connection (L_j). For all other connections, k_r = 1.0

n_n = number of shear planes with threads intercepting the shear plane

A_c = minor diameter area of the bolt as defined in AS 1275

n_x = number of shear planes without threads intercepting the shear plane

A_o = nominal plain shank area of the bolt.

Table 9.3.2.1 – Reduction factor for a bolted lap connection (k_r)

Length mm	**L_j < 300**	**300 ≤ L_j ≤ 1300**	**L_j > 1300**
k_r	1.0	1.075 – L_j / 4000	0.75

9.3.2.2 Bolt in tension

A bolt subject to a design tension force (N_{tf}^*) shall satisfy:

$$N_{tf}^* \leq \phi N_{tf}$$

where

ϕ = strength reduction factor (see table 3.3)

N_{tf} = nominal tension capacity of a bolt.

The nominal tension capacity of a bolt (N_{tf}) shall be calculated as follows:

$$N_{tf} = A_s f_{uf}$$

where A_s is the tensile stress area of a bolt as specified in AS 1275.

9.3.2.3 Bolt subject to combined shear and tension

A bolt required to resist both design shear (V_f^*) and design tension forces (N_{tf}^*) at the same time shall satisfy:

$$\left(\frac{V_f^*}{\phi V_f}\right)^2 + \left(\frac{N_{tf}^*}{\phi N_{tf}}\right)^2 \leq 1.0$$

where

ϕ = strength reduction factor (see table 3.3)

V_f = nominal shear capacity calculated in accordance with 9.3.2.1

N_{tf} = nominal tension capacity calculated in accordance with 9.3.2.2.

9.3.2.4 Ply in bearing

9.3.2.4.1 *General*

A ply subject to a design bearing force (V_b^*) due to bolt in shear shall satisfy:

$$V_b^* \leq C_1 \phi V_b$$

where

ϕ = strength reduction factor (see table 3.3)

V_b = nominal bearing capacity of a ply

C_1 = 1.0 for bolts in connections not designed for loads including earthquake loads
= as given by table 12.9.4.3 for bolts in connections designed for loads including earthquake loads.

The nominal bearing capacity of ply (V_b) shall be calculated as follows:

$$V_b = 3.2 d_f t_p f_{up} \quad \text{(Eq. 9.3.2.4(1))}$$

provided that, for a ply subject to a component of force acting towards an edge, the nominal bearing capacity of a ply (V_b) shall be the lesser of that given by equation 9.3.2.4(1) and that given by equation 9.3.2.4(2).

$V_b = a_e t_p f_{up}$ .... (Eq. 9.3.2.4 (2))

where

d_f = diameter of the bolt

t_p = thickness of the ply

f_{up} = tensile strength of the ply

a_e = minimum distance from the edge of a hole to the edge of a ply, measured in the direction of the component of a force, plus half the bolt diameter. The edge of a ply shall be deemed to include the edge of an adjacent bolt hole.

9.3.2.4.2 *Use of quenched and tempered steel splice plates.*
In accordance with the conditions of 1.1.4 (b), quenched and tempered steel plated designated for general structural use (the Bisalloy 80 or equivalent grade) may be used as splice cover plates in fully bolted column splices, provided equation 9.3.2.4 (1) is replaced by equation 9.3.2.4 (3).

$V_b = 1.1\, d_f\, t_p\, f_{up}$ .... (Eq. 9.3.2.4 (3))

where all notation is as defined in 9.3.2.4.1.

9.3.2.5 Filler plates
For connections in which filler plates exceed 6 mm in thickness but are less than 20 mm in thickness, the nominal shear capacity of a bolt (V_f) specified in 9.3.2.1 shall be reduced by 15 %. For multi-shear plane connections with more than one filler plate through which a bolt passes, the reduction shall be determined using the maximum thickness of filler plate on any shear plane through which the bolt passes.

9.3.3 Bolt serviceability limit state

9.3.3.1 Design
For friction-type connections (bolting category 8.8/*TF*) in which slip in the serviceability limit state is required to be limited, a bolt subjected only to a design shear force (V^*_{sf}) in the plane of the interfaces shall satisfy:

$V^*_{sf} \leq \phi V_{sf}$

where

ϕ = strength reduction factor (see table 3.3(2) and 3.4.5)

V_{sf} = nominal shear capacity of a bolt, for a friction-type connection.

The nominal shear capacity of a bolt (V_{sf}) shall be calculated as follows:

$V_{sf} = \mu_s n_{ei} N_{ti} k_h$

where

μ_s = slip factor as defined in 9.3.3.2

n_{ei} = number of effective interfaces

N_{ti} = minimum bolt tension at installation as specified in 15.2.5.1.2

k_h = factor for different hole types, as specified in 14.3.5.2
= 1.0 for standard holes
= 0.85 for short slotted and oversize holes
= 0.70 for long slotted holes.

The ultimate limit state shall also be separately assessed in accordance with 9.3.2.

9.3.3.2 Contact surfaces

9.3.3.2.1
Where the surfaces in contact are clean 'as-rolled' surfaces, the slip factor (μ_s) shall be taken as 0.35. If any applied finish, or other surface condition, including a machined surface, is used, the slip factor shall be based upon test evidence. Tests performed in accordance with the procedure specified in Appendix K shall be deemed to provide satisfactory test evidence.

9.3.3.2.2
A connection involving 8.8/*TF* bolting category shall be identified as such, and the drawings shall clearly indicate the surface treatment required at such a connection and whether masking of the connection surfaces is required during painting operations (see 14.3.6.3).

9.3.3.3 Combined shear and tension
Bolts in a connection for which slip in the serviceability limit state must be limited, which are subject to a design tension force (N^*_{tf}), shall satisfy:

$$\left(\frac{V^*_{sf}}{\phi V_{sf}}\right) + \left(\frac{N^*_{tf}}{\phi N_{tf}}\right) \leq 1.0$$

where

V^*_{sf} = design force on the bolt in the plane of the interfaces

N^*_{tf} = design tension force on the bolt

ϕ = strength reduction factor (see table 3.3(2) and 3.4.5)

V_{sf} = nominal shear capacity of the bolt as specified in 9.3.3.1

N_{tf} = nominal tension capacity of the bolt.

The nominal tension capacity of the bolt (N_{tf}) shall be taken as:

N_{tf} = N_{ti}

where N_{ti} is the minimum bolt tension at installation as specified in 15.2.5.1.2.

The ultimate limit state shall also be separately assessed in accordance with 9.3.2.

9.4 ASSESSMENT OF THE STRENGTH OF A BOLT GROUP

9.4.1 Bolt group subject to in-plane loading

9.4.1.1
The design actions in a bolt group shall be determined by analysis based on the following assumptions:

(a) The connection plates shall be considered to be rigid and to rotate relative to each other about a point known as the instantaneous centre of the bolt group;

(b) In the case of a bolt group subject to a pure couple only, the instantaneous centre of rotation coincides with the bolt group centroid.

In the case of a bolt group subject to an in-plane shear force applied at the group centroid, the instantaneous centre of rotation is at infinity and the design shear force is uniformly distributed throughout the group.

In all other cases, either the results of independent analyses for a pure couple alone and for an in-plane shear force applied at the bolt group centroid shall be superimposed, or a rational method of analysis shall be used.

(c) The design shear force in each bolt shall be assumed to act at right angles to the radius from the bolt to the instantaneous centre, and shall be taken as proportional to that radius.

9.4.1.2
Each bolt shall satisfy the requirements of 9.3.2.1 using the appropriate factor (ϕ) for a bolt group (see table 3.3) and the ply in bearing shall satisfy 9.3.2.4.

9.4.2 Bolt group subject to out-of-plane loading

9.4.2.1
The design actions in any bolt in a bolt group subject to out-of-plane loading shall be determined in accordance with 9.1.3.

9.4.2.2
Each bolt shall comply with 9.3.2.1, 9.3.2.2 and 9.3.2.3 using the appropriate factor (ϕ) for a bolt group (see table 3.3) and the ply in bearing shall comply with 9.3.2.4.

9.4.3 Bolt group subject to combinations of in-plane and out-of-plane loading

9.4.3.1
The design actions in any bolt in a bolt group shall be determined in accordance with 9.4.1 and 9.4.2.

9.4.3.2
Each bolt shall comply with 9.3.2.1, 9.3.2.2 and 9.3.2.3 using the appropriate factor (ϕ) for a bolt group (see table 3.3), and the ply in bearing shall comply with 9.3.2.4.

9.5 DESIGN OF A PIN CONNECTION

9.5.1 Pin in shear

A pin subject to a design shear force (V_f^*) shall satisfy:

$$V_f^* \leq \phi V_f$$

where

ϕ = strength reduction factor (see table 3.3)

V_f = nominal shear capacity of the pin.

The nominal shear capacity of a pin (V_f) shall be calculated as follows:

$$V_f = 0.62 f_{yp} n_s A_p$$

where

f_{yp} = yield stress of the pin

n_s = number of shear planes

A_p = cross-sectional area of the pin.

9.5.2 Pin and ply in bearing

A pin and a ply subject to a design bearing force $\left(V_b^*\right)$ shall satisfy:

$$V_b^* \leq \phi V_b$$

where

ϕ = strength reduction factor (see table 3.3)

V_b = nominal bearing capacity of the pin or ply, whichever is the least.

The nominal bearing capacity (V_b) shall be calculated as follows:

$$V_b = 1.4 f_{yb} d_f t_p k_p$$

where

f_{yb} = yield stress of the pin or ply, whichever is the least

d_f = pin diameter

t_p = connecting plate thickness(es)

Amd 1 June '01 k_p = 1.0 for pins without rotation, or
= 0.5 for pins with rotation.

9.5.3 Pin in bending

A pin subject to a design bending moment (M^*) shall satisfy:

$$M^* \leq \phi M_p$$

where

ϕ = strength reduction factor (see table 3.3)

M_p = nominal moment capacity of the pin.

The nominal moment capacity of a pin (M_p) shall be calculated as follows:

$$M_p = f_{yp} S$$

where

f_{yp} = yield stress of the pin

S = plastic section modulus of the pin.

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(Text deleted)

9.6 DESIGN DETAILS FOR BOLTS AND PINS

9.6.1 Minimum pitch

The distance between centres of fastener holes shall be not less than 2.5 times the nominal diameter of the fastener.

NOTE – The minimum pitch may also be affected by 9.3.2.4.

9.6.2 Minimum edge distance

9.6.2.1

In a connection not subject to earthquake loads, edge distance shall be as specified in table 9.6.2, where d_f is the nominal diameter of the fastener. The edge distance shall be the distance from the nearer edge of the hole to the physical edge of the plate or rolled section plus half the fastener diameter (d_f).

Table 9.6.2 – Minimum edge distance

Sheared or hand flame cut edge	Rolled plate, flat bar or section: machine flame cut, sawn or planed edge	Rolled edge of a rolled flat bar or section
$1.75d_f$	$1.50d_f$	$1.25d_f$

NOTE – The edge distance may also be affected by 9.3.2.4.

9.6.2.2

For connections subject to loads or effects including earthquake loads, the minimum edge distance shall comply with 12.9.4.4.

9.6.3 Maximum pitch

The maximum distance between centres of fasteners shall be the lesser of $15t_p$ (where t_p = thickness of thinner ply connected) or 200 mm. However, in the following cases, the maximum distances shall be as follows:

(a) For fasteners which are not required to carry design actions in regions not liable to corrosion – the lesser of $32t_p$ or 300 mm.

(b) For an outside line of fasteners in the direction of the design action – the lesser of $(4t_p + 100)$ mm, or 200 mm.

9.6.4 Maximum edge distance

The maximum distance from the centre of any fastener to the nearest edge of parts in contact with one another shall be 12 times the thickness of the thinnest outer connected ply under consideration, but shall not exceed 150 mm.

9.6.5 Holes

Holes for bolts shall comply with 14.3.5 and, where appropriate, with 12.9.4.5.

Holes for pins shall comply with 14.3.7.

9.7 DESIGN OF WELDS

9.7.1 Scope

9.7.1.1 General

Welding shall comply with AS/NZS 1554.1 (except as modified by 9.7.1.4 and Appendix D of this Standard), AS 1554.2 or AS/NZS 1554.5, as appropriate.

9.7.1.2 Weld types

For the purpose of this Standard, welds shall be butt, fillet, slot or plug welds, or compound welds.

9.7.1.3 Weld quality

Weld quality shall be either SP or GP as specified in 9.7.1.4, except that where a higher quality weld is required by 10.1.5, weld quality conforming with AS/NZS 1554.5 shall be used. Weld quality shall be specified on the design drawings or in the design specification.

9.7.1.4 Selection of weld category in accordance with AS/NZS 1554.1

9.7.1.4.1

When using AS/NZS 1554.1 in accordance with the requirements of this Standard, the two weld categories shall be designated as follows:

(1) GP (general purpose)

(2) SP (structural purpose)

9.7.1.4.2

Category GP is appropriate where the weld is essentially statically loaded and where the design actions on the weld do not exceed its design capacity for this category, as calculated in accordance with 9.7 using the strength reduction factor from table 3.3, or where the welding application is other than structural.

9.7.1.4.3
Category SP should be specified for:

(a) Welds subject to essentially static loading where the design actions on the weld exceed the design capacity for a category GP weld, as calculated in accordance with 9.7 using the strength reduction factor from table 3.3; or

(b) Welds subject to earthquake loads or effects and which constitute part of the main load-carrying path in the seismic-resisting or associated structural system, irrespective of the level of design actions on the weld.

Welds subject to earthquake loads or effects and which do not constitute part of the main load-carrying path in the seismic-resisting or associated structural system should have their category selected according to the level of design loading as given by (a) above; or

(c) Welds subject to high-cycle dynamic loading within the limits specified in 1.1 of AS/NZS 1554.1.

9.7.2 Butt welds

9.7.2.1 Definitions
For the purpose of this clause, the definitions below apply.

COMPLETE PENETRATION BUTT WELD. A butt weld in which fusion exists between the weld and parent metal throughout the complete depth of the joint.

INCOMPLETE PENETRATION BUTT WELD. A butt weld in which the depth of penetration is less than the complete depth of the joint.

PREQUALIFIED WELD PREPARATION. A joint preparation prequalified in terms of AS/NZS 1554.1.

9.7.2.2 Size of weld

9.7.2.2.1
The size of a complete penetration butt weld, other than a complete penetration butt weld in a T-joint or a corner joint, and the size of an incomplete penetration butt weld shall be the minimum depth to which the weld preparation extends from its face into a joint, exclusive of reinforcement.

9.7.2.2.2
The size of a complete penetration butt weld for a T-joint or a corner joint shall be the thickness of the part whose end or edge butts against the face of the other part.

9.7.2.3 Design throat thickness
Design throat thickness shall be as follows:

(a) *Complete penetration butt weld*
The design throat thickness for a complete penetration butt weld shall be the size of the weld.

(b) *Incomplete penetration butt weld*
The design throat thickness for an incomplete penetration butt weld shall be as follows:

(i) Prequalified preparation for incomplete penetration butt weld, except as otherwise provided in (iii), as specified in AS/NZS 1554.1

(ii) Non-prequalified preparation for incomplete penetration butt weld, except as provided in (iii):

(A) where $\theta < 60°$ .... $(d - 3)$ mm, for single V weld;
$[(d_3 + d_4) - 6]$ mm, for double V weld.

(B) where $\theta \geq 60°$ .... d mm, for single V weld;
$(d_3 + d_4)$ mm, for double V weld.

where

d = depth of preparation (d_3 and d_4 are the values of d for each side of the weld)

θ = angle of preparation.

(iii) For an incomplete penetration butt weld made by an automatic arc welding process for which it can be demonstrated by means of a macro test on a production weld that the required penetration has been achieved, an increase in design throat thickness up to the depth of preparation may be allowed. If the macro test shows penetration beyond the depth of preparation, an increase in design throat thickness up to that shown in figure 9.7.3.4 may be allowed.

NOTE – It is only necessary to specify the design throat thickness required, leaving the fabricator to determine the welding procedure necessary to achieve the specified design throat thickness.

9.7.2.4 Effective length
The effective length of a butt weld shall be the length of the continuous full size weld.

9.7.2.5 Effective area
The effective area of a butt weld shall be the product of the effective length and the design throat thickness.

9.7.2.6 Transition of thickness or width

9.7.2.6.1
Butt welded joints between parts of different thickness or unequal width that are subject to tension shall have a smooth transition between surfaces or edges. The transition shall be made by chamfering the thicker part or by sloping the weld surfaces or by any combination of those, as shown in figure 9.7.2.6.

9.7.2.6.2
The transition slope between the parts shall not exceed 1:1. However, the provisions of section 10 require a lesser slope than this or a curved transition between the parts for some fatigue detail categories and the provisions of 12.14.3 and 12.14.4 apply for members subject to earthquake loads or effects.

9.7.2.7 Strength assessment of a butt weld
Strength assessment of a butt weld shall be as follows:

(a) *Complete penetration butt weld*

The design capacity of a complete penetration butt weld shall be taken as equal to the nominal capacity of the weaker part of the parts joined, multiplied by the appropriate factor (ϕ) for butt welds given in table 3.3, provided that the welding procedures are qualified in accordance with AS/NZS 1554.1 or AS/NZS 1554.5.

➤

The butt weld shall be made using a welding consumable which will produce butt tensile test specimens in accordance with AS 2205.2.1 for which the minimum strength is not less than the corresponding values for the parent material (see 2.2.1).

For welds within or connecting category 1 or 2 members (see 4.5, 12.3), $f_{uw} \geq f_u$ is required; where:

f_{uw} = nominal tensile strength of weld metal
f_u = tensile strength of parent metal

(b) *Incomplete-penetration butt weld*

The design capacity of an incomplete penetration butt weld shall be calculated as for a fillet weld (see 9.7.3.10) using the design throat thickness determined in accordance with 9.7.2.3(b).

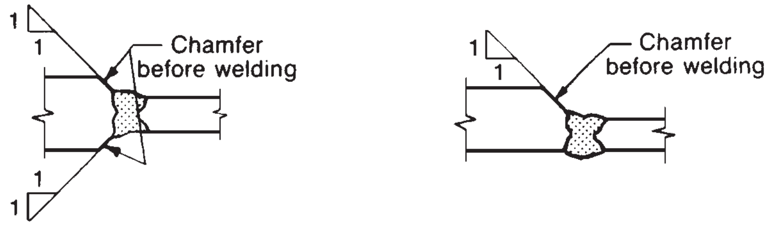
9.7.3 Fillet welds

9.7.3.1 Size of a fillet weld

The size of a fillet weld shall be specified by the leg lengths. The leg lengths shall be defined as the lengths (t_{w1}, t_{w2}) of the sides lying along legs of a triangle inscribed within the cross section of the weld (see figures 9.7.3.1(a) and (b)). When the legs are of equal length, the size shall be specified by a single dimension (t_w).

Where there is a root gap, the size (t_w) shall be given by the lengths of the legs of the inscribed triangle reduced by the root gap, as shown in figure 9.7.3.1(c).

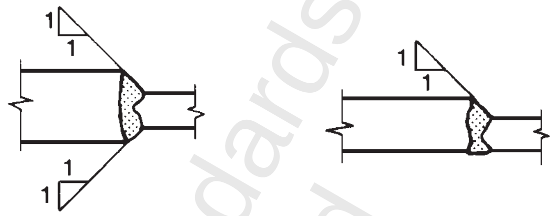
NOTE – The preferred sizes of a fillet weld less than 15 mm are – 3, 4, 5, 6, 8, 10 and 12 mm.



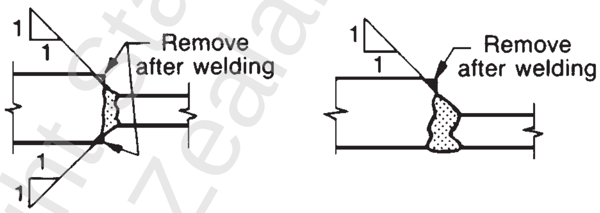
Centreline alignment
(Particularly applicable to web plates)

Offset alignment
(Particularly applicable to flange plates)

(i) Transition by chamfering thicker part

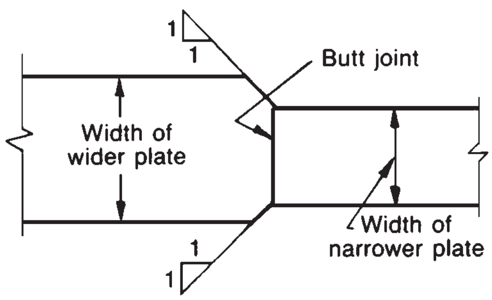


(ii) Transition by sloping weld surface



(iii) Transition by sloping weld surface and chamfering

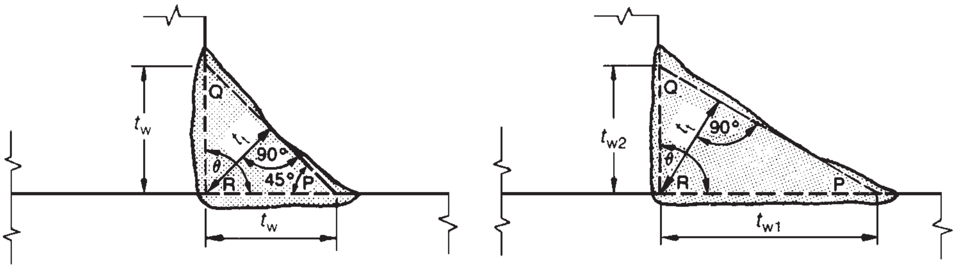
(a) Transition of butt joints in parts of unequal thickness



(b) Transition of butt joints in parts of unequal width—Transition by chamfering wider part

NOTE – Transition slopes shown in (a) and (b) are the maximum permitted.

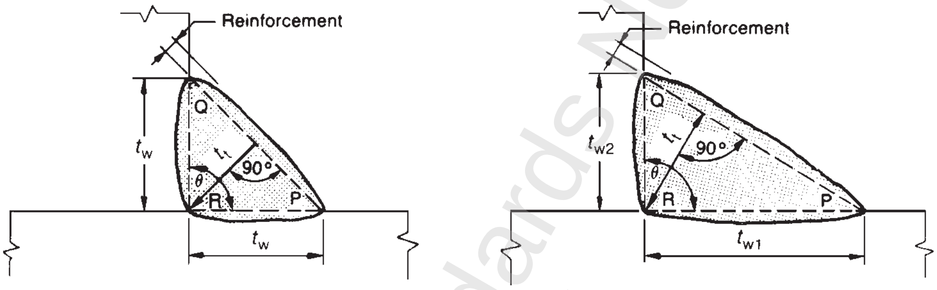
Figure 9.7.2.6 – Transition of thickness or width for butt welds subject to tension



Equal leg fillet weld

Unequal leg fillet weld

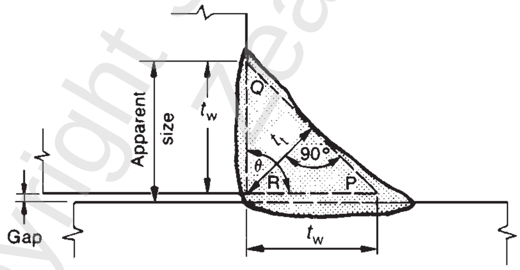
(a) Concave fillet welds



Equal leg fillet weld

Unequal leg fillet weld

(b) Convex fillet welds



(c) Fillet weld with root gap

LEGEND:

PQR	=	triangle inscribed within the cross section of the weld
t_w, t_{w1}, t_{w2}	=	sizes of fillet welds
t_t	=	design throat thickness

Figure 9.7.3.1 – Fillet weld size

9.7.3.2 Minimum size of a fillet weld

The minimum size of a fillet weld, other than a fillet weld used to reinforce a butt weld, shall conform with table 9.7.3.2, except that the size of the weld need not exceed the thickness of the thinner part joined.

Table 9.7.3.2 – Minimum size of a fillet weld

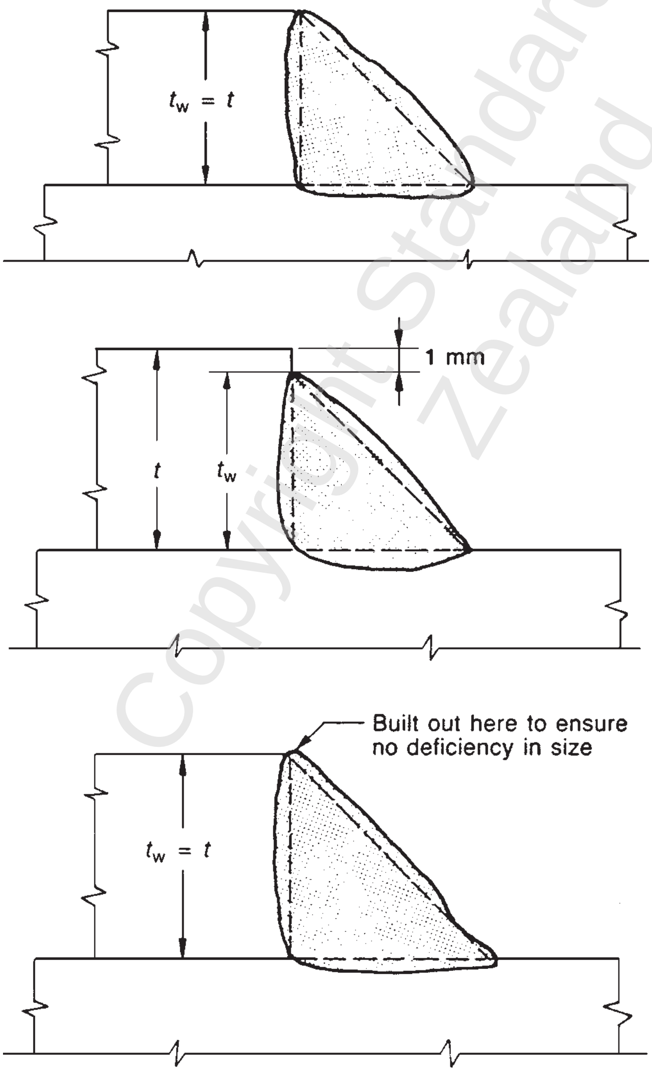
Thickness of thickest part (t) millimetres	Minimum size of a fillet weld (t_w) millimetres
$t \le 7$	3
$7 < t \le 10$	4
$10 < t \le 15$	5
$15 < t$	6

9.7.3.3 Maximum size of a fillet weld along an edge

The maximum size of a fillet weld along an edge of material shall be:

(a) For material less than 6 mm in thickness, the thickness of the material (see figure 9.7.3.3(a));

(b) For material 6 mm or more in thickness, 1 mm less than the thickness of the material, (see figure 9.7.3.3(b)), unless the weld is designated on the drawing to be built out to obtain the design throat thickness (see figure 9.7.3.3(c)).



(a) $t_w = t$ for $t < 6$ mm

(b) t_w not greater than $(t - 1)$ mm for $t \ge 6$ mm

(c) $t_w = t$ for all thicknesses where edge is built out

LEGEND: t_w = size of fillet weld
t = thickness of thinner part joined

Figure 9.7.3.3 – Maximum sizes of fillet welds along edges

9.7.3.4 Design throat thickness

9.7.3.4.1
The design throat thickness (t_t) of a fillet weld shall be as shown in figure 9.7.3.1.

9.7.3.4.2
For a weld made by an automatic arc welding process, an increase in design throat thickness may be allowed as shown in figure 9.7.3.4, provided that it can be demonstrated by means of a macro test on a production weld that the required penetration has been achieved. Where such penetration is achieved, the size of weld required may be correspondingly reduced to give the specified design throat thickness.

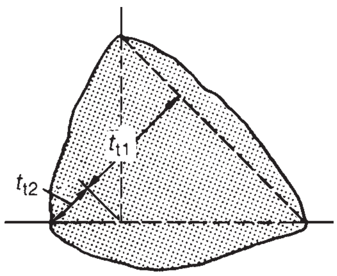


Figure 9.7.3.4 – Deep penetration weld

9.7.3.5 Effective length

9.7.3.5.1
The effective length of a fillet weld shall be the overall length of the full-size fillet, including end returns. No reduction in effective length need be made for either the start or crater of the weld if the weld is full size throughout its length.

9.7.3.5.2
The minimum effective length of a fillet weld shall be 4 times the size of the weld. However, if the ratio of the effective length of the weld to the size of the weld does not comply with this requirement, the size of the weld for design purposes shall be taken as 0.25 times the effective length. The minimum length requirement shall also apply to lap joints.

9.7.3.5.3
Any segment of intermittent fillet weld shall have an effective length of not less than 40 mm or 4 times the nominal size of the weld, whichever is the greater.

9.7.3.6 Effective area

The effective area of a fillet weld shall be the product of the effective length and the design throat thickness.

9.7.3.7 Transverse spacing of fillet welds

9.7.3.7.1
If 2 parallel fillet welds connect 2 components in the direction of the design action to form a built-up member, the transverse distance between the welds shall not exceed $32t_p$, except that in the case of intermittent fillet welds at the ends of a tension member, the transverse distance shall not exceed either $16t_p$ or 200 mm, where t_p is the thickness of the thinner of the 2 components connected.

9.7.3.7.2
It shall be permissible to use fillet welds in slots and holes in the direction of the design action in order to satisfy this clause.

9.7.3.8 Intermittent fillet welds
Except at the ends of a built-up member, the clear spacing between the lengths of consecutive collinear intermittent fillet welds shall not exceed the lesser of:

(a) For elements in compression $16t_p$ and 300 mm; and

(b) For elements in tension $24t_p$ and 300 mm.

9.7.3.9 Built-up members – intermittent fillet welds
If intermittent fillet welds connect components forming a built-up member, the welds shall comply with the following requirements:

(a) At the ends of a tension or compression component of a beam, or at the ends of a tension member, when side fillets are used alone, they shall have a length along each joint line at least equal to the width of the connected component. If the connected component is tapered, the length of weld shall be the greater of:

   (i) The width of the widest part; and

   (ii) The length of the taper.

(b) At the cap plate or baseplate of a compression member, welds shall have a length along each joint of at least the maximum width of the member at the contact face.

(c) Where a beam is connected to the face of a compression member, the welds connecting the compression member components shall extend between the levels of the top and bottom of the beam and in addition:

   (i) For an unrestrained (simple) connection, a distance (d) below the lower face of the beam; and

   (ii) For a restrained (rigid or semi-rigid) connection, a distance (d) above and below the upper and lower faces of the beam.

   where d is the maximum cross-sectional dimension of the compression member.

9.7.3.10 Ultimate limit state design capacity for fillet welds

9.7.3.10.1
A fillet weld subject to a design force per unit length of weld (v_w^*) shall satisfy:

$$v_w^* \leq \phi v_w$$

where

ϕ = strength reduction factor (see table 3.3)

v_w = nominal capacity of a fillet weld per unit length.

9.7.3.10.2
The design force per unit length (v_w^*) shall be the vectorial sum of the design forces per unit length on the effective area of the weld.

9.7.3.10.3
The nominal capacity of a fillet weld per unit length (v_w) shall be calculated as follows:

$$v_w = 0.6 f_{uw} t_t k_r \qquad \text{(Eq. 9.7.3.10)}$$

where

f_{uw} = nominal tensile strength of weld metal (see table 9.7.3.10(1)), except that for welds within or connecting category 1 or 2 members (see 4.5, 12.3), $f_{uw} \geq f_u$ is required.

t_t = design throat thickness

k_r = reduction factor given in table 9.7.3.10(2) to account for the length of a welded lap connection (L_w). For all other connection types, k_r = 1.0.

Table 9.7.3.10(1) – Nominal tensile strength of weld metal (f_{uw})

Manual metal arc electrode (AS/NZS 1553.1)	**Submerged arc (AS 1858.1) Flux cored arc (AS 2203) Gas metal arc (AS/NZS 2717.1)**	**Nominal tensile strength of weld metal (f_{uw})** (see Note 2) **MPa**
E41XX	W40X (see Note 1)	410
E48XX	W50X	480

NOTE –

(1) Not included in AS/NZS 2717.1

(2) The increased design capacity available from weld metals with a nominal tensile strength exceeding that given herein may be used, provided that failure at the interface between the weld and the parent metal is precluded.

Table 9.7.3.10(2) – Reduction factor for a welded lap connection (k_r)

Length of weld (L_w) metres	$L_w \leq 1.7$	$1.7 < L_w \leq 8.0$	$L_w > 8.0$
k_r	1.00	$1.10 - 0.06L_w$	0.62

9.7.4 Plug and slot welds

9.7.4.1 Plug and slot welds in the form of fillet welds around the circumferences of the hole or slot
These plug and slot welds shall be regarded as a fillet weld, with an effective length as defined in 9.7.3.5, and a nominal capacity as defined in 9.7.3.10. The minimum size shall be as for a fillet weld (see 9.7.3.2).

9.7.4.2 Plug and slot welds in hole filled with weld metal

9.7.4.2.1
The effective shear area (A_w) of a plug or slot weld in a hole filled with weld metal shall be considered as the nominal cross-sectional area of the hole or slot in the plane of the faying surface.

9.7.4.2.2
Such a plug or slot weld subject to a design shear force (V_w^*) shall satisfy:

$$V_w^* \leq \phi V_w$$

where

ϕ = strength reduction factor (see table 3.3)

V_w = nominal shear capacity of the weld.

The nominal shear capacity (V_w) of the weld shall be calculated as follows:

$$V_w = 0.60 f_{uw} A_w$$

9.7.4.3 Limitations
Plug or slot welds may only be used to transmit shear in lap joints or to prevent buckling of lapped parts or to join component parts of built-up members or to prevent out-of-plane buckling of doubler plates in joint panel zones, the latter in accordance with 12.9.5.3.3.

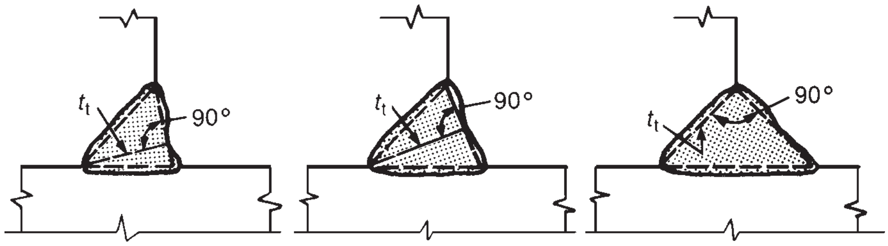
9.7.5 Compound weld

9.7.5.1 Description of a compound weld
A compound weld, defined as a fillet weld superimposed on a butt weld, shall be as specified in AS 1101.3.

9.7.5.2 Design throat thickness
The design throat thickness of a compound weld, for use in design calculations, shall be:

(a) For a complete penetration butt weld, the size of the butt weld without reinforcement; and

(b) For an incomplete penetration butt weld, the shortest distance from the root of the incomplete penetration butt weld to the face of the fillet weld as determined by the largest inscribed triangle in the total weld cross section, with a maximum value equal to the thickness of the part whose end or edge butts against the face of the other part (see figure 9.7.5.2).



NOTE – The design throat thickness (t_t) of a weld is the minimum distance from the root of a weld to its face, less any reinforcement. The three sketches above illustrate this concept.

Figure 9.7.5.2 – Design throat thickness of compound welds

9.7.5.3 Ultimate limit state
The weld shall satisfy the requirements of 9.7.2.7.

9.8 ASSESSMENT OF THE STRENGTH OF A FILLET WELD GROUP

9.8.1 Fillet weld group subject to in-plane loading

9.8.1.1 General method of analysis
The design force per unit length in a fillet weld group subject to in-plane loading shall be determined in accordance with the following:

(a) The connection plates shall be considered to be rigid and to rotate relative to each other about a point known as the instantaneous centre of rotation of the weld group.

(b) In the case of a weld group subject to a pure couple only, the instantaneous centre of rotation coincides with the weld group centroid.

In the case of a weld group subject to an in-plane shear force applied at the group centroid, the instantaneous centre of rotation is at infinity and the design force per unit length (v_w^*) is uniformly distributed throughout the group.

In all other cases, either the results of independent analyses for a pure couple alone and for an in-plane shear force applied at the weld group centroid shall be superimposed, or a rational method of analysis shall be used.

(c) The design force per unit length (v_w^*) at any point in the fillet weld group shall be assumed to act at right angles to the radius from that point to the instantaneous centre, and shall be taken as proportional to that radius.

A fillet weld shall satisfy the requirements of 9.7.3.10 at all points in the fillet weld group using the appropriate factor (ϕ) for a weld group (see table 3.3). In the case of a fillet weld group of constant throat thickness, it will be sufficient to check only that point in the group defined by the maximum value of the radius to the instantaneous centre.

9.8.1.2 Alternative method of analysis

9.8.1.2.1
The design force per unit length in the fillet weld group may alternatively be determined by considering the fillet weld group as an extension of the connected member and by proportioning the design force per unit length in the fillet weld group to satisfy equilibrium between the fillet weld group and the elements of the connected member.

9.8.1.2.2
A fillet weld shall satisfy the requirements of 9.7.3.10 at all points in the fillet weld group using the appropriate factor (ϕ) for a weld group (see table 3.3).

9.8.2 Fillet weld group subject to out-of-plane loading

9.8.2.1 General method of analysis
The design force per unit length in a fillet weld group subject to out-of-plane loading shall be determined in accordance with the following:

(a) The fillet weld group shall be considered in isolation from the connected element; and

(b) The design force per unit length in the fillet weld resulting from a design bending moment shall be considered to vary linearly with the distance from the relevant centroidal axes. The design force per unit length in the fillet weld group resulting from any shear force or axial force shall be considered to be uniformly distributed over the length of the fillet weld group.

A fillet weld shall satisfy the requirements of 9.7.3.10 at all points in the fillet weld group, using the appropriate factor (ϕ) for a weld group (see table 3.3).

9.8.2.2 Alternative method of analysis

9.8.2.2.1
The design force per unit length in a fillet weld group may alternatively be determined by considering the fillet weld group as an extension of the connected member and distributing the design forces among the welds of the fillet weld group so as to satisfy equilibrium between the fillet weld group and the elements of the connected member.

9.8.2.2.2
A fillet weld shall satisfy the requirements of 9.7.3.10 at all points in the fillet weld group, using the appropriate factor (ϕ) for a weld group (see table 3.3).

9.8.3 Fillet weld group subject to in-plane and out-of-plane loading

9.8.3.1 General method of analysis
The design force per unit length as determined from analyses in accordance with 9.8.1.1 and 9.8.2.1 shall satisfy 9.7.3.10 at all points in the fillet weld group, using the appropriate factor (ϕ) for a weld group (see table 3.3).

9.8.3.2 Alternative method of analysis
The design force per unit length as determined from analyses in accordance with 9.8.1.2 and 9.8.2.2 shall satisfy 9.7.3.10 at all points in the fillet weld group, using the appropriate factor (ϕ) for a weld group (see table 3.3).

9.8.4 Combination of weld types
If 2 or more types of weld are combined in a single connection, the design capacity of the connection shall be the sum of the design capacities of each type, determined in accordance with this section.

9.9 PACKING IN CONSTRUCTION

9.9.1
Where packing is welded between 2 members and is less than 6 mm thick, or is too thin to allow provision of adequate welds or to prevent buckling, the packing shall be trimmed flush with the edges of the element subject to the design action and the size of the welds along the edges shall be increased over the required size by an amount equal to the thickness of the packing.

9.9.2
Otherwise the packing shall extend beyond the edges and shall be welded to the piece to which it is fitted.

NOTES

10 FATIGUE

10.1 GENERAL

10.1.1 Requirements

10.1.1.1
This section applies to the design of structures and structural elements subject to fatigue.

10.1.1.2
The following effects are not covered by this section:

(a) Reduction of fatigue life due to corrosion or immersion;

(b) High stress–low cycle fatigue (e.g. due to seismic actions);

(c) Thermal fatigue;

(d) Stress corrosion cracking.

10.1.1.3
The design shall verify that at each point of the structure the requirements of this section are satisfied for the design life of the structure.

10.1.1.4
A structure or structural element which is designed in accordance with this section must also comply with the requirements of this Standard for the ultimate and serviceability limit states.

10.1.1.5
An alternative to the use of section 10 is to undertake a fracture assessment in accordance with 2.6.5.

10.1.2 Definitions
For the purposes of this section, the definitions below apply.

CONSTANT STRESS RANGE FATIGUE LIMIT. Highest constant stress range for each detail category at which fatigue cracks are not expected to propagate (see figure 10.6.1).

CUT-OFF LIMIT. For each detail category, the highest variable stress range which does not require consideration when carrying out cumulative damage calculations (see figures 10.6.1 and 10.6.2).

DESIGN LIFE. Period over which a structure or structural element is required to perform its function without repair.

DESIGN SPECTRUM. Sum of the stress spectra from all of the nominal loading events expected during the design life.

DETAIL CATEGORY. Designation given to a particular detail to indicate which of the S-N curves is to be used in the fatigue assessment.

NOTE –

(1) The detail category takes into consideration the local stress concentration at the detail, the size and shape of the maximum acceptable discontinuity, the loading condition, metallurgical effects, residual stresses, the welding process and any post weld improvement.

(2) The detail category number is defined by the fatigue strength at 2×10^6 cycles on the S-N curve (see figures 10.6.1 and 10.6.2).

DISCONTINUITY. An absence of material, causing a stress concentration.

NOTE – Typical discontinuities include cracks, scratches, corrosion pits, lack of penetration, slag inclusions, cold laps, porosity and undercut.

FATIGUE. Damage caused by repeated fluctuations of stress leading to gradual cracking of a structural element.

FATIGUE LOADING. Set of nominal loading events described by the distribution of the loads, their magnitudes and the numbers of applications of each nominal loading event.

FATIGUE STRENGTH. The stress range defined in 10.6 for each detail category varying with the number of stress cycles (see figures 10.6.1 and 10.6.2).

MINER'S SUMMATION. Cumulative damage calculation based on the Palmgren-Miner summation or equivalent.

NOMINAL LOADING EVENT. The loading sequence for the structure or structural element.

NOTE – One nominal loading event may produce one or more stress cycles depending on the type of load and the point in the structure under consideration.

STRESS CYCLE. One cycle of stress defined by stress cycle counting.

STRESS CYCLE COUNTING METHOD. Any rational method used to identify individual stress cycles from the stress history.

STRESS RANGE. Algebraic difference between two extremes of stress.

STRESS SPECTRUM. Histogram of the stress cycles produced by a nominal loading event.

10.1.3 Notation

For the purposes of this section –

d_x, d_y = distances of the extreme fibres from the neutral axes

f_c = fatigue strength corrected for thickness of material

f_f = uncorrected fatigue strength

f_{rn} = detail category reference fatigue strength at n_r – normal stress

f_{rs} = detail category reference fatigue strength at n_r – shear stress

f_y = yield stress

f_3 = detail category fatigue strength at constant amplitude fatigue limit (5 x 10^6 cycles)

f_5 = detail category fatigue strength at cut off limit (10^8 cycles)

f^* = design stress range

f_i^* = design stress range for loading event *i*

n_i = number of cycles of nominal loading event *i*, producing f_i^*

n_r = reference number of stress cycles (2 x 10^6 cycles)

n_{sc} = number of stress cycles

t_f = flange thickness

t_p = plate thickness

α_s = inverse of the slope of the S-N curve

ϕ = strength reduction factor

10.1.4 Limitation

In all stress cycles, the magnitude of the design stress shall not exceed f_y and the stress range shall not exceed $1.5f_y$.

10.1.5 Designation of weld category

10.1.5.1

The welds in the welded details given in tables 10.5.1 (2) and (4) for Detail Categories 112 and below shall conform with Category SP as defined in 9.7.1.4 of this Standard.

10.1.5.2

The welds in the welded details given in table 10.5.1 (2) for Detail Category 125 shall have a weld quality conforming to that defined in AS/NZS 1554.5.

10.1.6 Strength reduction factor

10.1.6.1

The strength reduction factor (ϕ) shall depend on whether or not the detail is located on a redundant load path (i.e. in a position where failure at that point alone will not lead to overall collapse of the structure) and on the following factors:

(a) The stress history is estimated by conventional methods;

(b) The load cycles are not highly irregular;

(c) The detail is accessible for, and subject to regular inspection.

10.1.6.2
When the detail is located on a redundant load path:

(a) If any two of 10.1.6.1 (a) – (c) apply, $\phi = 1.0$;

(b) If only one of 10.1.6.1 (a) – (c) applies, $\phi = 0.85$;

(c) If none of 10.1.6.1 (a) – (c) apply, $\phi < 0.85$ and shall be determined by rational design.

10.1.6.3
When the detail is located on a non-redundant load path:

(a) If all of 10.1.6.1 (a) – (c) apply, $\phi = 1.0$;

(b) If any two of 10.1.6.1 (a) – (c) apply, $\phi = 0.8$;

(c) If one or none of 10.1.6.1 (a) – (c) apply, $\phi < 0.8$ and shall be determined by rational design.

10.1.7 Thickness effect

10.1.7.1
The uncorrected fatigue strength (f_f) of a transverse fillet or butt welded connection involving a plate thickness (t_p) greater than 25 mm shall be reduced to a corrected fatigue strength (f_c) using:

$$f_c = f_f \left(\frac{25}{t_p} \right)^{0.25}$$

10.1.7.2
For all other details and for a transverse fillet or butt welded connection involving a plate thickness less than or equal to 25 mm, the corrected fatigue strength is given by:

$$f_c = f_f$$

10.1.7.3
Similar correction of f_{rn}, f_{rs}, f_3 and f_5 to f_{rnc}, f_{rsc}, f_{3c} and f_{5c}, respectively, shall be performed by substituting these variables into the equations above to replace f_f and f_c respectively.

10.2 FATIGUE LOADING

10.2.1
The loading used in the fatigue assessment shall be the expected service loading over the design life of the structure, including dynamic effects.

10.2.2
The fatigue loading shall be obtained from the appropriate Standard or document. For cranes or bridges, the following shall be used:

Amd 2
Oct. '07

(Text deleted)

AS 1418	Cranes (including hoists and winches)
Transit New Zealand	Bridge Manual: Design and Evaluation (for road bridges)
New Zealand Rail Ltd	Railnet Code, Part 4, Code Supplements Bridges and Structures, Section 2: Design (for rail bridges)

10.3 DESIGN SPECTRUM

10.3.1 Stress determination

10.3.1.1
The design stresses shall be determined from an elastic analysis of the structure or from the stress history obtained from strain measurements.

10.3.1.2
The design stresses shall be determined as normal or shear stresses taking into account all design actions on the member but excluding stress concentrations due to the geometry of the detail as described in tables 10.5.1(1) to 10.5.1(4). The effect of stress concentrations which are not characteristic of the detail shall be taken into account separately.

10.3.1.3
Unless noted otherwise, each arrow in tables 10.5.1(1) to 10.5.1(4) indicates the location and direction of the stresses acting in the base material on a plane normal to the arrow for which the stress range is to be calculated.

10.3.1.4
For the fatigue assessment of trusses made of open sections in which the connections are not pinned, the effects of secondary bending moments shall be taken into account unless:

$L / d_x > 40$
or
$L / d_y > 40$

as appropriate, where d is the depth in the plane of the truss.

10.3.1.5
For truss connections using hollow sections, the stress range in the members may be calculated without consideration of the effects of connection stiffness and eccentricities, subject to the following:

(a) For a truss connection involving circular hollow sections, the stress range shall be multiplied by the appropriate factor given in table 10.3.1(1).

Table 10.3.1(1) – Multiplying factors for calculated stress range – circular hollow sections

Type of connection		Chords	Verticals	Diagonals
Gap connections	K type	1.5	N/A	1.3
	N type	1.5	1.8	1.4
Overlap connections	K type	1.5	N/A	1.2
	N type	1.5	1.65	1.25

(b) For a truss connection involving rectangular hollow sections, the calculated stress range shall be multiplied by the appropriate factor given in table 10.3.1(2).

Table 10.3.1(2) – Multiplying factors for calculated stress range – rectangular hollow sections

Type of connection		Chords	Verticals	Diagonals
Gap connections	K type	1.5	N/A	1.5
	N type	1.5	2.2	1.6
Overlap connections	K type	1.5	N/A	1.3
	N type	1.5	2.0	1.4

(c) The design throat thickness of a fillet weld shall be greater than the wall thickness of the connected member. The use of fillet welds shall be limited to members with wall thickness ≤ 8 mm. Complete penetration butt welds shall be used when the wall thickness of the thinner member being connected exceeds 8 mm.

For truss connections using hollow sections in which (a) – (c) above are not applied, the calculated stress range shall include the effects of connection stiffness and eccentricities.

10.3.2 Design spectrum calculation

The stress of a nominal loading event producing irregular stress cycles shall be obtained by a rational stress cycle counting method, such as rainflow counting.

10.4 EXEMPTION FROM ASSESSMENT

Fatigue assessment is not required for a member, connection or detail, if the normal and shear design stress ranges (f^*) satisfy:

$$f^* < \phi\, 27 \text{ MPa} \quad \text{.......... (Eq. 10.4.1)}$$

or if the number of stress cycles (n_{sc}) satisfies

$$n_{sc} < 2 \times 10^6 \left(\frac{\phi \times 36}{f^*}\right)^3 \quad \text{.......... (Eq. 10.4.2)}$$

10.5 DETAIL CATEGORY

10.5.1 Detail categories for normal stress

10.5.1.1

A detail category for normal stress shall be assigned for each structural member, connection or detail in the structure. The detail categories are defined in tables 10.5.1(1) to (4).

The classifications in these tables are divided into four parts which correspond to four basic groups:

Group 1: Non-welded details – plain material and bolted plates. (See table 10.5.1(1))

Group 2: Welded details – not in hollow sections. (See table 10.5.1(2))

Group 3: Bolts. (See table 10.5.1(3))

Group 4: Welded details – in hollow sections. (See table 10.5.1(4))

10.5.1.2
Details not classified in tables 10.5.1(1) to (4) shall be treated as the lowest detail category of a similar detail, unless a superior fatigue strength is proved by experimental testing or by rational design.

10.5.2 Detail categories for shear stress

A detail category for shear stress shall be assigned for each relevant detail in the structure. The detail categories for shear stress are given in table 10.5.1(2) (Constructional details 39 and 40) and in table 10.5.1(3) (Constructional detail 41).

10.6 FATIGUE STRENGTH

10.6.1 Definition of fatigue strength for normal stress

The uncorrected fatigue strength (f_f) for each category (f_{rn}) subject to normal stress is defined by:

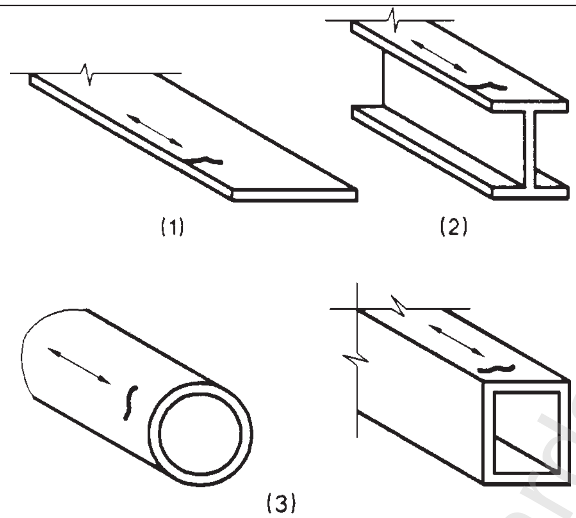
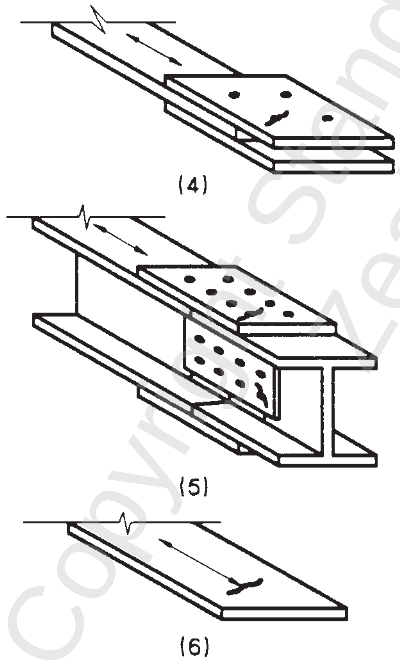
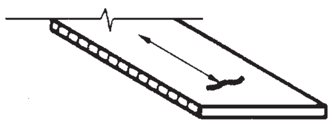
$$f_f^3 = \frac{f_{rn}^3 \times 2 \times 10^6}{n_{sc}} \qquad \text{when } n_{sc} \leq 5 \times 10^6$$

$$f_f^5 = \frac{f_5^5 \times 10^8}{n_{sc}} \qquad \text{when } 5 \times 10^6 < n_{sc} \leq 10^8$$

where n_{sc} is the number of stress cycles.

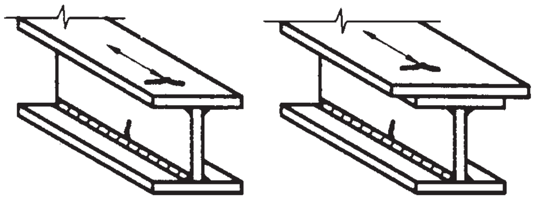
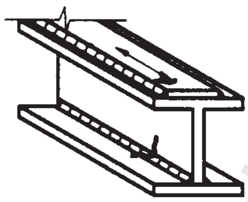
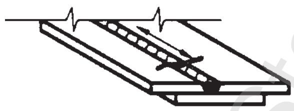
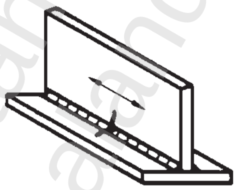
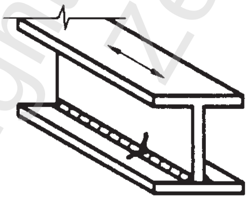
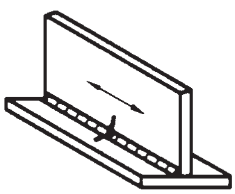
Values of f_f, f_3 and f_5 are shown in figure 10.6.1 for each detail category (f_{rn}).

Table 10.5.1(1) – Detail category classification Group 1 non-welded details

Detail category	Constructional details – Illustration (See Note)	Constructional details – Description
160	 (1) (2) (3)	**ROLLED AND EXTRUDED PRODUCTS** (1) Plates and flats (2) Rolled sections (3) Seamless tubes Sharp edges, surface and rolling flaws to be removed by grinding in the direction of applied stress.
140	 (4) (5) (6)	**BOLTED CONNECTIONS** (4) & (5) Stress range calculated on the gross section for 8.8/*TF* bolting category and on the net section otherwise. Unsupported one-sided coverplate connections shall be avoided or the effect of the eccentricity taken into account in calculating stresses. **MATERIAL WITH GAS-CUT OR SHEARED EDGES WITH NO DRAGLINES** (6) All hardened material and visible signs of edge discontinuities to be removed by machining or grinding in the direction of applied stress.
125	 (7)	**MATERIAL WITH MACHINE GAS-CUT EDGES WITH DRAGLINES OR MANUAL GAS-CUT MATERIAL** (7) Corners and visible signs of edge discontinuities to be removed by grinding in the direction of the applied stress.

NOTE – The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

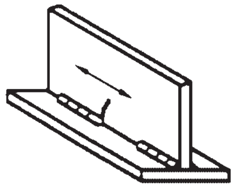
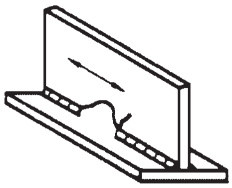
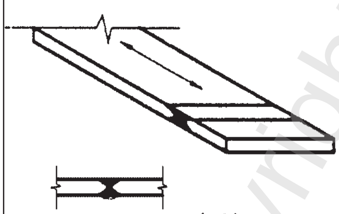
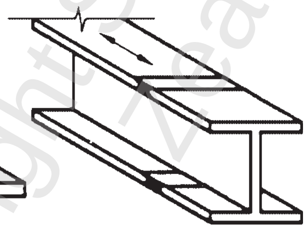
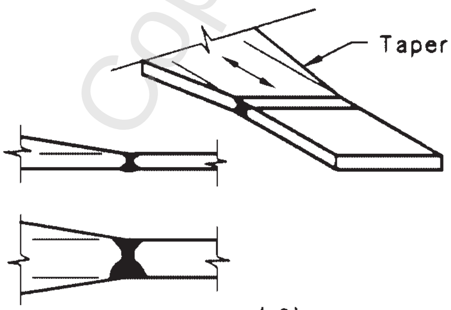
Table 10.5.1(2) – Detail category classification
Group 2 welded – not in hollow sections

Detail category	Constructional details: Illustration (See Note)	Description
125	 (8)  (9)	**WELDED PLATE I-SECTION AND BOX GIRDERS WITH CONTINUOUS LONGITUDINAL WELDS** (8) & (9) Zones of continuous automatic longitudinal fillet or butt welds carried out from both sides and all welds not having unrepaired stop-start positions. NOTE – See Clause 10.1.5 regarding weld category.
112	 (10)  (11)  (12)	**WELDED PLATE I-SECTION AND BOX GIRDERS WITH CONTINUOUS LONGITUDINAL WELDS** (10) and (11) Zones of continuous automatic butt welds made from one side only with a continuous backing bar and all welds not having unrepaired stop-start positions. (12) Zones of continuous longitudinal fillet or butt welds carried out from both sides but containing stop-start positions. For continuous manual longitudinal fillet or butt welds carried out from both sides, use Detail Category 100.
90	 (13)	**WELDED PLATE I-SECTION AND BOX GIRDERS WITH CONTINUOUS LONGITUDINAL WELDS** (13) Zones of continuous longitudinal welds carried out from one side only, with or without stop-start positions.

NOTE – The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

(continued)

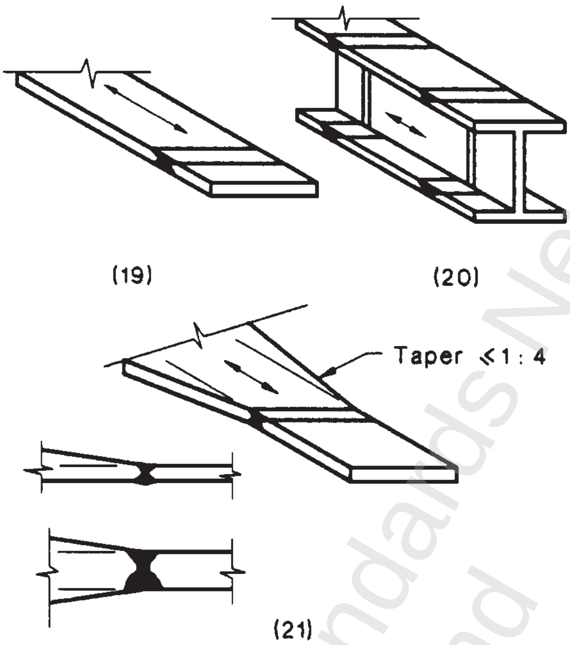
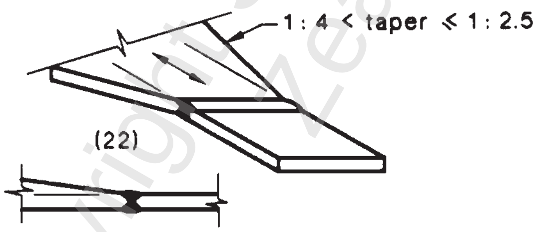
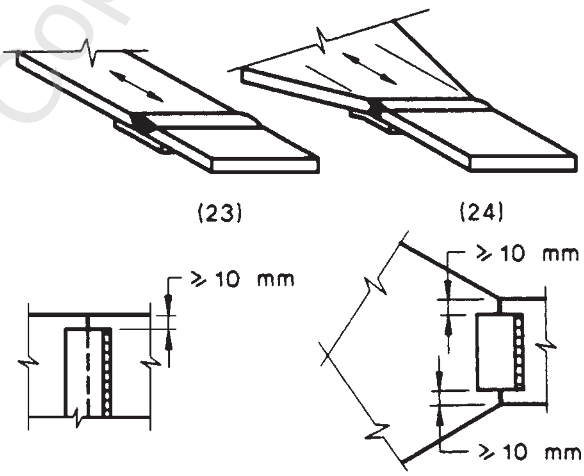
Table 10.5.1(2) (Continued)

Detail category	Constructional details: Illustration (See Note)	Description
80	 (14)	**INTERMITTENT LONGITUDINAL WELDS** (14) Zones of intermittent longitudinal welds.
71	 (15)	**INTERMITTENT LONGITUDINAL WELDS** (15) Zones containing cope holes in longitudinally welded T joints. Cope holes not to be filled with weld.
112	 (16)  (17)  (18)	**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from two sides. (16) Transverse splices in plates, flats and rolled sections having the weld reinforcement ground flush to plate surface. 100 % NDT inspection, and weld surface to be free of exposed porosity in the weld metal. (17) Plate girders welded as (16) before assembly. (18) Transverse splices as (16) with radiused or tapered transition with taper ≤ 1:4.

NOTE – The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

(continued)

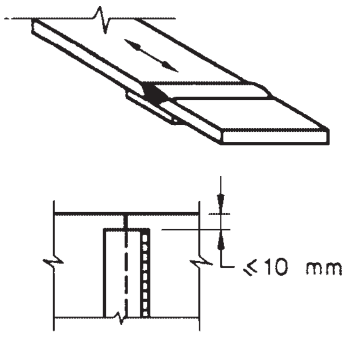
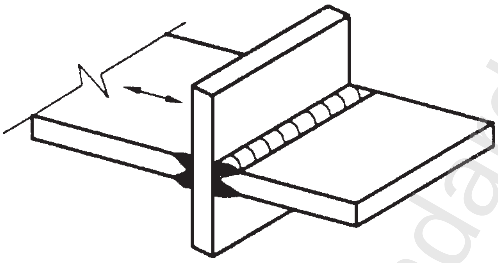
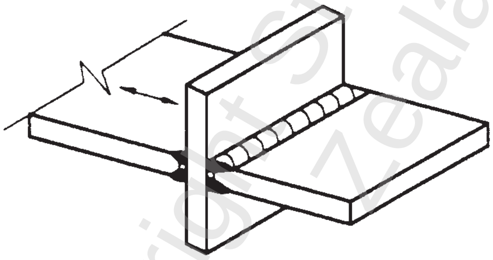
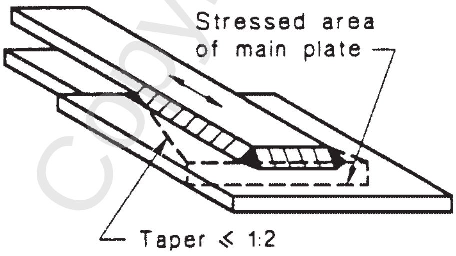
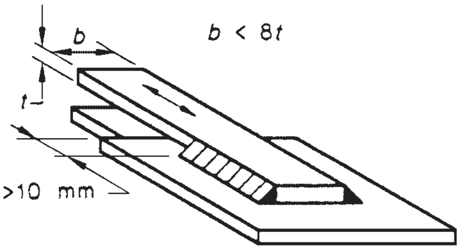
Table 10.5.1(2) (Continued)

Detail category	Constructional details: Illustration (See Note)	Description
90		**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from 2 sides. (19) Transverse splices of plates, rolled sections, or plate girders. (20) Transverse splices of rolled sections or welded plate girders, without cope hole. With cope hole use Detail Category 71, as for (15). (21) Transverse splices in plates or flats being tapered in width or in thickness where the taper is ≤ 1:4.
80		**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from 2 sides. (22) Transverse splices as for (21) with taper in width or thickness > 1:4 and ≤ 1:2.5.
71		**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** (23) Transverse butt welded splices made on a backing bar. The end of the fillet weld of the backing strip must be greater than 10 mm from the edges of the stressed plate. (24) Transverse butt welds as for (23) with taper on width or thickness < 1:2.5.

NOTE – The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

(continued)

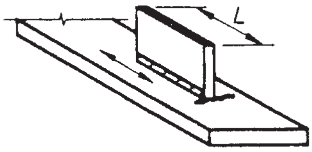
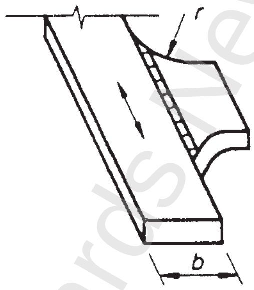
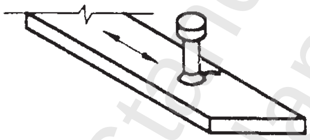
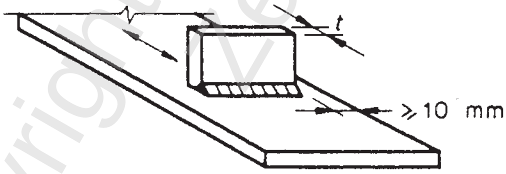
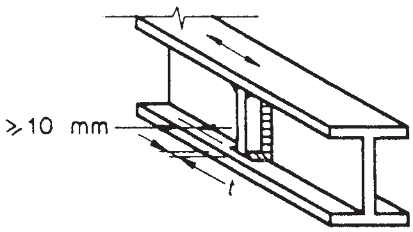
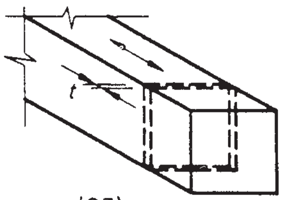
Table 10.5.1(2) (Continued)

Detail category	Constructional details: Illustration (See Note)		Description
50	 (25)		**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** (25) Transverse butt welds as (23) where fillet welds end closer than 10 mm to plate edge.
71	 (26)		**CRUCIFORM JOINTS WITH LOAD-CARRYING WELDS** (25) Full penetration welds with intermediate plate NDT inspected and free of defects. Maximum misalignment of plates either side of joint to be < 0.15 times the thickness of intermediate plate.
56	(27)		(27) Partial penetration or fillet welds with stress range calculated on plate area.
36	(28)		(28) Partial penetration or fillet welds with stress range calculated on throat area of weld.
63	 (29)		**OVERLAPPED WELDED JOINTS** (29) Fillet welded lap joint, with welds and overlapping elements having a design capacity greater than the main plate. Stress in the main plate to be calculated on the basis of area shown in the illustration.
56	(30)		(30) Fillet welded lap joint, with welds and main plate both having a design capacity greater than the overlapping elements.
45	(31)		(31) Fillet welded lap joint, with main plate and overlapping elements both having a design capacity greater than the weld.

NOTE – The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

(continued)

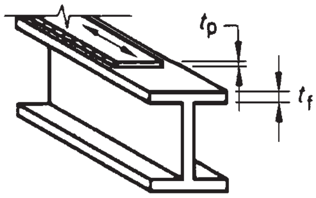
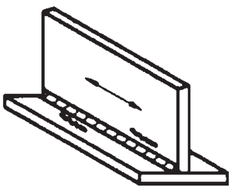
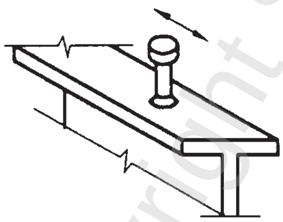
Table 10.5.1(2) (Continued)

Detail category	Constructional details: Illustration (See Note) (32)	(33)		Description
90	—	$\frac{1}{3} \leqslant \frac{r}{b}$	(32) (33)	**WELDED ATTACHMENTS (NON-LOAD CARRYING WELDS) LONGITUDINAL WELDS** (32) Longitudinal fillet welds. Class of detail varies according to the length of the attachment weld as noted. (33) Gusset welded to the edge of a plate or beam flange. Smooth transition radius (r) formed by machining or flame-cutting plus grinding. Class of detail varies according to r/b ratio as noted.
80	$L \leqslant 50$ mm	—		
71	$50 < L \leqslant 100$ mm	$\frac{1}{6} \leqslant \frac{r}{b} < \frac{1}{3}$		
50	100 mm $< L$	—		
45	—	$\frac{r}{b} < \frac{1}{6}$		
80	(34)			**WELDED ATTACHMENTS** (34) Shear connectors on base material (failure in base material).
80	$t \leqslant 12$ mm	(35) (36) (37)		**TRANSVERSE WELDS** (35) Transverse fillet welds with the end of the weld ≥ 10 mm from the edge of the plate. (36) Vertical stiffeners welded to a beam or plate girder flange or web by continuous or intermittent welds. In the case of webs carrying combined bending and shear design actions, the fatigue strength shall be determined using the stress range of the principal stresses. (37) Diaphragms of box girders welded to the flange or web by continuous or intermittent welds.
71	$t > 12$ mm			

NOTE – The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

(continued)

Table 10.5.1(2) (Continued)

Detail category	Constructional details: Illustration (See Note)		Constructional details: Description
50	t_r and $t_p \le 25$ mm	 (38)	**COVER PLATES IN BEAMS AND PLATE GIRDERS** (38) End zones of single or multiple welded cover plates, with or without a weld across the end. For a reinforcing plate wider than the flange, an end weld is essential. (See Description 31 for the fatigue check in the weld itself).
36	t_r and $t_p > 25$ mm		
80		 (39)  (40)	**WELDS LOADED IN SHEAR** (39) Fillet welds transmitting shear. Stress range to be calculated on weld throat area. (40) Stud welded shear connectors (failure in the weld) loaded in shear (the shear stress range to be calculated on the nominal section of the stud).

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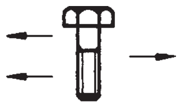
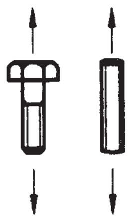
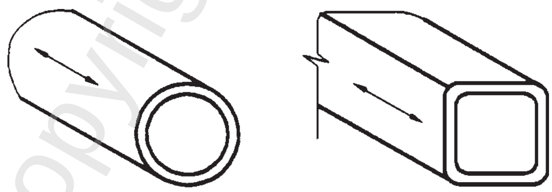
NOTE – The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

Table 10.5.1(3) – Detail category classification
Group 3 bolts

Detail category	Constructional details: Illustration (See Note)	Constructional details: Description
100	 (41)	(41) **BOLTS IN SHEAR (8.8/TB BOLTING CATEGORY ONLY)** Shear stress range calculated on the minor diameter area of the bolt (A_c). NOTE – If the shear on the joint is insufficient to cause slip of the joint (see 9.3.3), the shear in the bolt need not be considered in fatigue.
36	 (42)	(42) **BOLTS AND THREADED RODS IN TENSION** (tensile stress to be calculated on the tensile stress area A_s). Additional forces due to prying effects must be taken into account. For tensioned bolts (8.8/*TF* and 8.8/*TB* bolting categories), the stress range depends on the connection geometry. NOTE – In connections with tensioned bolts (see 15.2.4 and 15.2.5), the change in the force in the bolts is often less than the applied force, but this effect is dependent on the geometry of the connection. It is not normally required that any allowance for fatigue be made in calculating the required number of bolts in such connections. However, Standards New Zealand is not prepared to recommend methods for calculation of the stress range in tensioned bolts.

NOTE – The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

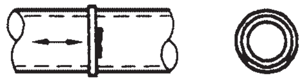
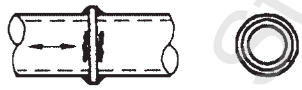
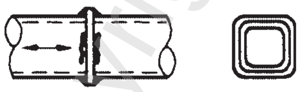
Table 10.5.1(4) – Detail category classification
Group 4 welded details – in hollow sections

Detail category	Constructional details: Illustration (See Note)	Constructional details: Description
140	 (43)	**CONTINUOUS AUTOMATIC LONGITUDINAL WELDS** (43) No stop-starts, or as manufactured.
90 71	 (44)	**TRANSVERSE BUTT WELDS** (44) Butt-welded end-to-end connection of circular hollow sections.
71 56	 (45)	(45) Butt-welded end-to-end connection of rectangular hollow sections.

NOTE – The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

(continued)

Table 10.5.1(4) (Continued)

Detail category	Constructional details: Illustration (See Note)	Constructional details: Description
56 50	 (46)	**BUTT WELDS TO INTERMEDIATE PLATE** (46) Circular hollow sections, end-to-end butt welded with an intermediate plate.
50 41	 (47)	(47) Rectangular hollow sections, end-to-end butt welded with an intermediate plate.
71	 (48)	**WELDED ATTACHMENTS** (Non-load-carrying) (48) Circular or rectangular hollow section, fillet welded to another section. Section width parallel to stress direction ≤ 100 mm.
45 40	 (49)	**FILLET WELDS TO INTERMEDIATE PLATE** (49) Circular hollow sections, end-to-end fillet welded with an intermediate plate.
40 36	 (50)	(50) Rectangular hollow sections, end-to-end fillet welded with an intermediate plate.

NOTE – The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.

10.6.2 Definition of fatigue strength for shear stress

The uncorrected fatigue strength (f_f) for each detail category (f_{rs}) subject to shear stress is defined by:

$$f_f^5 = \frac{f_{rs}^5 \times 2 \times 10^6}{n_{sc}} \qquad \text{when } n_{sc} \leq 10^8$$

where n_{sc} is the number of stress cycles.

Values of f_f and f_5 are shown in figure 10.6.2 for each detail category (f_{rs}).

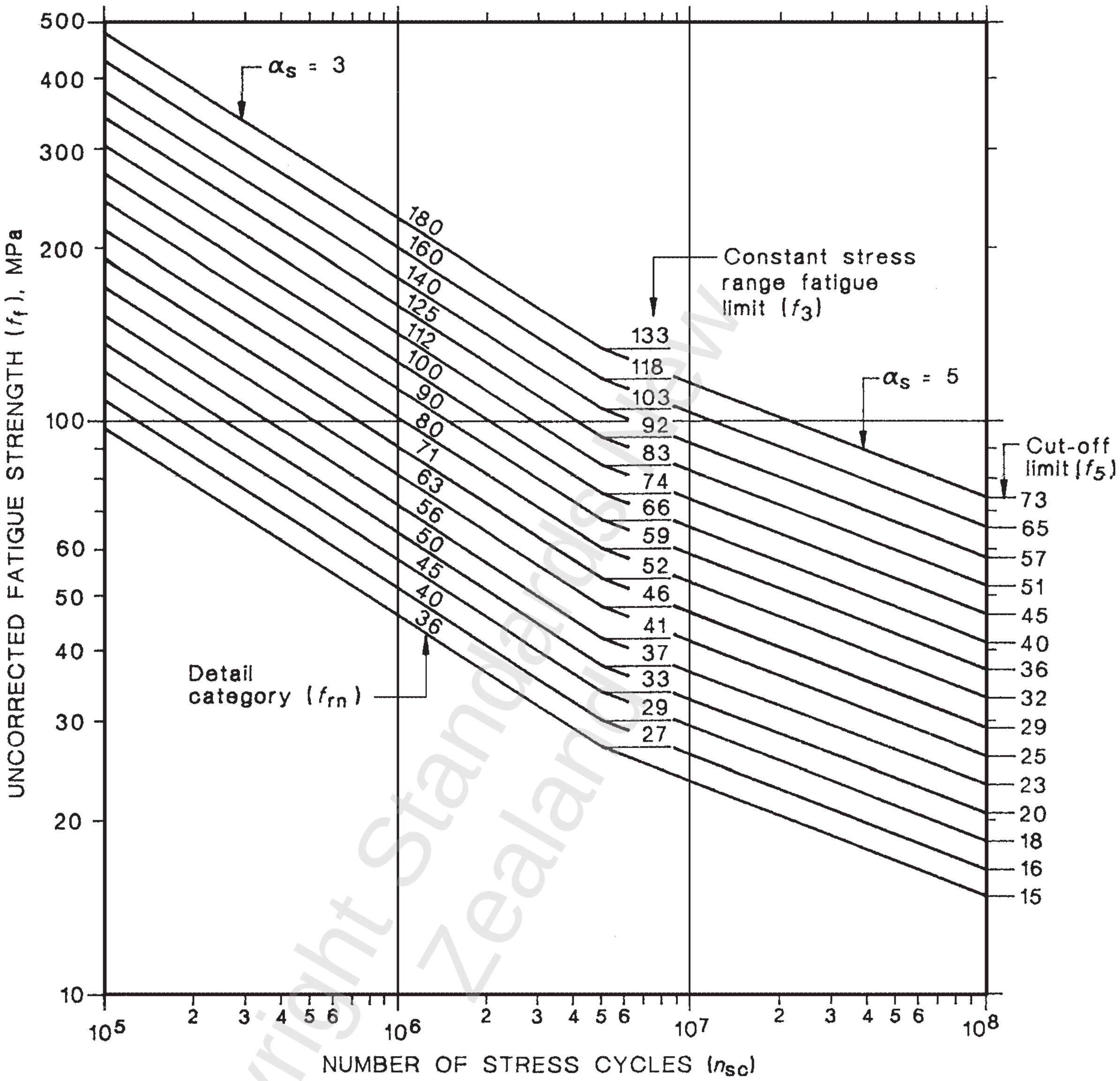


Figure 10.6.1 – S-N curve for normal stress

10.7 EXEMPTION FROM FURTHER ASSESSMENT

At any point in the structure at which all normal stress ranges are less than the constant normal stress range fatigue limit (ϕf_3) for the relevant detail category, no further assessment at that point is required.

10.8 FATIGUE ASSESSMENT

10.8.1 Constant stress range

The design stress range (f^*) at any point in the structure subject only to constant stress range cycles shall satisfy:

$$\frac{f^*}{\phi f_c} \leq 1.0$$

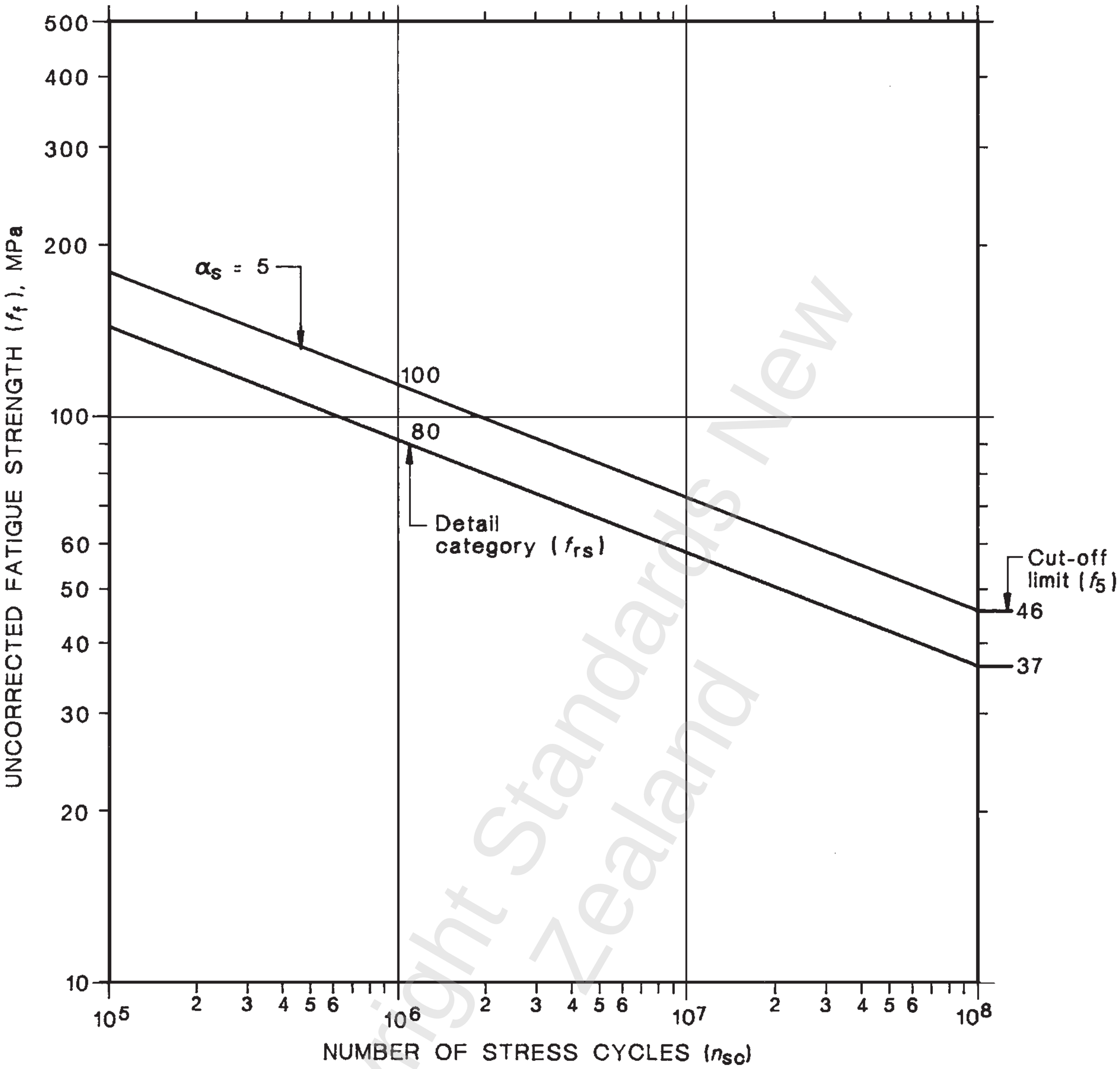


Figure 10.6.2 – S-N curve for shear stress

10.8.2 Variable stress range

The design stress range (f_i^*) at any point in the structure at which the stress range varies shall satisfy:

(a) For normal stresses:

$$\underbrace{\frac{\Sigma_i n_i \left(f_i^*\right)^3}{5 \times 10^6 \left(\phi f_{3c}\right)^3}}_{} \quad + \quad \underbrace{\frac{\Sigma_i n_i \left(f_i^*\right)^5}{5 \times 10^6 \left(\phi f_{3c}\right)^5}}_{} \quad \leq \quad 1.0$$

for $n_i(f_i^*)$ pairs in which $f_i^* \geq \phi f_{3c}$

for $(n_i f_i^*)$ pairs in which $\phi f_{3c} > f_i^* \geq \phi f_{5c}$

(b) For shear stresses:

$$\frac{\Sigma_i n_i \left(f_i^*\right)^5}{2 \times 10^6 \left(\phi f_{rsc}\right)^5} \leq 1.0$$

for n_i (f_i^*) pairs in which $f_i^* \geq \phi f_{5c}$

10.9 PUNCHING LIMITATION

For members and connections requiring assessment for fatigue in accordance with this section, a punched hole shall only be permitted in material whose thickness does not exceed 12.0 mm.

NOTES

11 FIRE

11.1 REQUIREMENTS

11.1.1
This section applies to steel building elements required to have a fire resistance rating (FRR).

For protected steel members and connections, the thickness of protection material shall be greater than or equal to that needed to give a period of structural adequacy (PSA) equal to the required FRR.

For unprotected steel members and connections, the section factor (SF) shall be less than or equal to that required to give a PSA equal to the required FRR.

The period of structural adequacy (PSA) shall be determined in accordance with 11.3, using the variations of the mechanical properties of steel with temperature as specified in 11.4.

Connections and beam web penetrations shall be treated in accordance with 11.10.

11.2 DEFINITIONS

For the purpose of this section, the definitions below apply.

CONFIGURATION. A term relating the type and general arrangement of protection material adjacent to the steel.

FIRE PROTECTION SYSTEM. The fire protection material and its method of attachment to the steel member.

FIRE EXPOSURE CONDITION. Either:

(a) *Three-sided fire exposure condition* – a steel member incorporated in or in contact with a concrete or masonry floor or wall

NOTE –

(1) Different configuration categories of three-sided exposure to fire shall be considered separately unless otherwise specified in 11.9.

(2) Members with more than one face in contact with a concrete or masonry floor or wall may be treated as three-sided fire exposure.

(b) *Four sided fire exposure condition* – a steel member exposed to fire on all sides.

FIRE-RESISTANCE RATING (FRR). The fire resistance grading period for structural adequacy only, in minutes, which is required to be obtained in the standard fire test or in a calculated time equivalent to that obtained in the standard fire test.

PERIOD OF STRUCTURAL ADEQUACY (PSA). The time (t), in minutes, for the member to reach the limit state of structural adequacy in the standard fire test.

PROTOTYPE. A test specimen representing a steel member and its fire protection system which is subjected to the standard fire test.

SECTION FACTOR (SF). Either:

(a) *Exposed surface area to mass ratio* (k_{sm}). The ratio of the surface area (m^2/m run) exposed to the fire to the mass of the steel (kg/m run); or

(b) *Exposed perimeter to area ratio* (H_p/A). The ratio of heated perimeter (m) of the surface exposed to fire to the area of cross section (m^2) of the steel.

NOTE – In the case of members with fire protection material applied, the exposed surface area or exposed perimeter is to be taken as the internal surface area or internal perimeter, respectively, of the fire protection material.

STANDARD FIRE TEST. The fire-resistance test specified in one of ISO 834, AS 1530.4 or BS 476 Parts 20-23.

STICKABILITY. The ability of the fire protection system to remain in place as the member deflects under load during a fire test, as specified in the relevant standard fire test method.

STRUCTURAL ADEQUACY. The ability of the member exposed to the standard fire test to carry the test loads within the deflection limits, as defined in the standard fire test method used in the assessment of the fire protection system.

11.3 DETERMINATION OF PERIOD OF STRUCTURAL ADEQUACY

The period of structural adequacy (PSA) shall be determined using one of the following methods:

(a) By calculation:

(i) By determining the limiting temperature of the steel (T_l) in accordance with 11.5; and then

(ii) By determining the PSA as the time from the start of the test (t) to the time at which the limiting steel temperature is attained in accordance with 11.6 for unprotected members or 11.7 for protected members; or

(b) By direct application of a single test in accordance with 11.8; or

(c) By structural analysis in accordance with section 4, using mechanical properties which vary with temperature in accordance with 11.4. Calculation of the temperature of the steel shall be by using a rational method of analysis which has been confirmed by test data.

11.4 VARIATION OF MECHANICAL PROPERTIES OF STEEL WITH TEMPERATURE

11.4.1 Variation of yield stress with temperature

The influence of temperature on the yield stress of steel shall be taken as follows:

$$\frac{f_y(T)}{f_y(20)} = 1.0 \qquad \text{when } 0\ °C < T \leq 215\ °C\text{; and}$$

$$= \frac{905 - T}{690} \qquad \text{when } 215\ °C < T \leq 905\ °C$$

where

$f_y(T)$ = yield stress of steel at T °C

$f_y(20)$ = yield stress of steel at 20 °C

T = temperature of the steel in °C.

This relationship is shown by Curve 1 in figure 11.4.

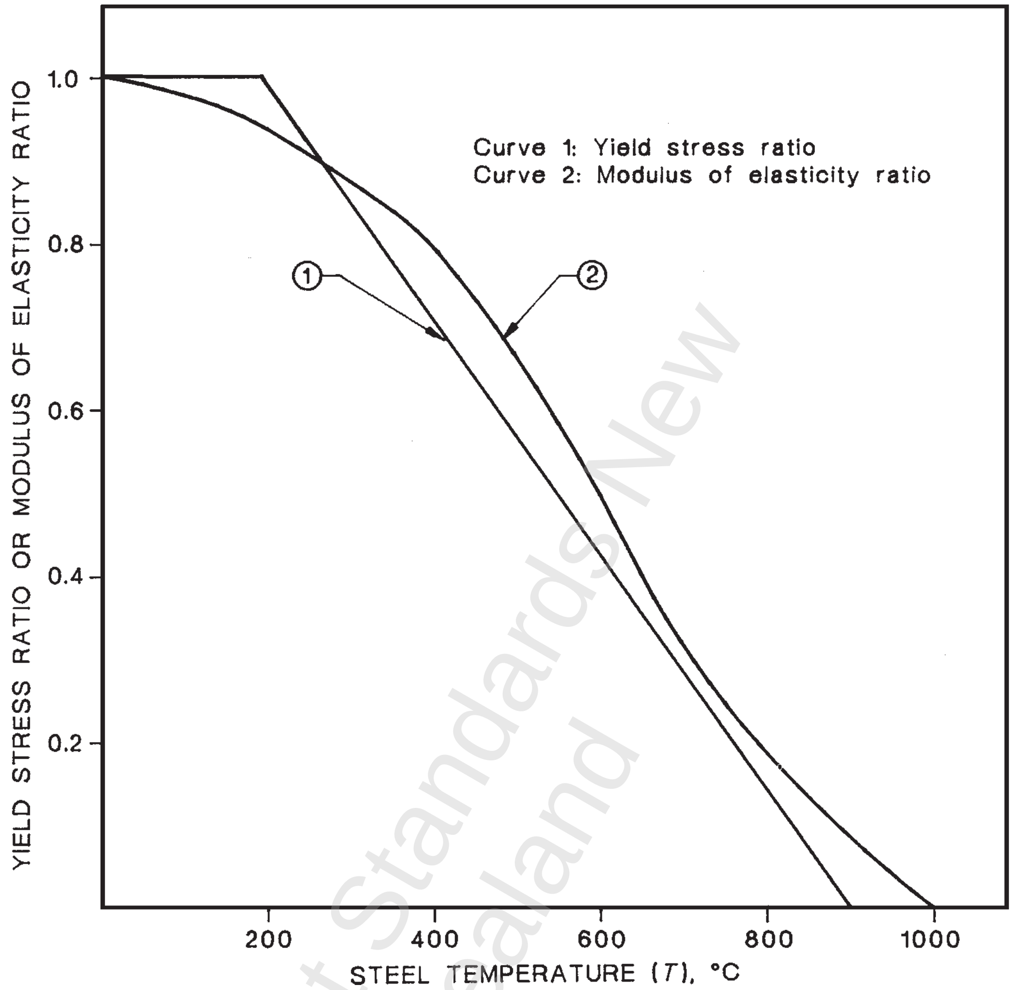


Figure 11.4 – Variation of mechanical properties of steel with temperature

11.4.2 Variation of modulus of elasticity with temperature

The influence of temperature on the modulus of elasticity of steel shall be taken as follows:

$$\frac{E(T)}{E(20)} = 1.0 + \left\{ \frac{T}{2000\left[\mathrm{Ln}\left(\frac{T}{1100}\right)\right]} \right\} \quad \text{when } 0\ ^\circ\mathrm{C} < T \le 600\ ^\circ\mathrm{C}; \text{ and}$$

$$= \frac{690\left(1 - \frac{T}{1000}\right)}{T - 53.5} \quad \text{when } 600\ ^\circ\mathrm{C} < T \le 1000\ ^\circ\mathrm{C}$$

where

$E(T)$ = modulus of elasticity of steel at T °C

$E(20)$ = modulus of elasticity of steel at 20 °C

Ln = Naperian logarithm ($\log_e$).

This relationship is shown by Curve 2 in figure 11.4.

11.5 DETERMINATION OF LIMITING STEEL TEMPERATURE

The limiting steel temperature (T_l) shall be calculated as follows:

$$T_l = 905 - 690 r_f$$

where r_f is the ratio of the design action on the member under the design load for fire specified in NZS 4203 to the design capacity of the member (f_{fire} R_u) at room temperature, for fire emergency conditions. The strength reduction factor for fire emergency conditions, $f_{fire} = (f/0.85) \leq 1.0$, where f is given by table 3.3(1) or table 13.1.2(1), as appropriate. Amd 1 June '01

11.6 DETERMINATION OF TIME AT WHICH THE LIMITING TEMPERATURE IS ATTAINED FOR UNPROTECTED MEMBERS

The time (t) at which the limiting temperature (T_l) is attained shall be calculated for:

(a) Three–sided exposure as follows:

$$t = -5.2 + 0.0221 T_l + \left(\frac{0.433 T_l}{SF}\right) \quad \text{(Eq. 11.6.1)}$$

(b) Four–sided exposure as follows:

$$t = -4.7 + 0.0263 T_l + \left(\frac{0.213 T_l}{SF}\right) \quad \text{(Eq. 11.6.2)}$$

where

t = time from the start of the test, in minutes

T_l = steel temperature, in degrees Celsius, 500 °C $\leq T \leq$ 850 °C

SF = section factor, within prescribed ranges

SF = k_{sm} (exposed surface area to mass ratio, within range of 2 m^2/tonne $\leq k_{sm} \leq$ 35 m^2/tonne)

= $\left(\frac{H_p / A}{7.85}\right)$ (exposed perimeter to area ratio, within range of 15 m^{-1} $\leq H_p/A \leq$ 275 m^{-1})

For temperatures below 500 °C, linear interpolation shall be used based on the time at 500 °C and an initial temperature of 20 °C at $t = 0$.

11.7 DETERMINATION OF TIME AT WHICH THE LIMITING TEMPERATURE IS ATTAINED FOR PROTECTED MEMBERS

11.7.1 Methods

11.7.1.1

The time (t) at which the limiting temperature (T_l) is attained shall be determined from the results of a single test in accordance with 11.7.2, or by calculation on the basis of a suitable series of fire tests in accordance with 11.7.3.

11.7.1.2

For beams and for all members with a four-sided fire exposure condition, the limiting temperature (T_l) shall be taken as the average of all of the temperatures measured at the thermocouple locations shown in the standard fire test method Standard.

11.7.1.3

For columns with a three-sided fire exposure condition, the limiting temperature (T_l) shall be taken as the average of the temperatures measured at the thermocouple locations on the face farthest from the wall. Alternatively, the temperatures from members with a four-sided fire exposure condition and the same section factor may be used.

11.7.2 Temperature based on single test

The variation of steel temperature with the time measured in a standard fire test may be used without modification provided:

(a) The fire protection system is the same as the prototype;

(b) The fire exposure condition is the same as the prototype;

(c) The fire protection material thickness is equal to or greater than that of the prototype;

(d) The section factor is equal to or less than that of the prototype; and

(e) Where the prototype has been submitted to a standard fire test in an unloaded condition, stickability has been separately demonstrated.

11.7.3 Temperature based on a test series

Calculation of the variation of steel temperature with time shall be by interpolation of the results of series of fire tests using equation 11.7.3.1, subject to the limitations and conditions of 11.7.3.2.

11.7.3.1 Regression analysis

The relationship between temperature (T) and time (t) for a series of tests on a group shall be calculated by least-squares regression as follows:

$$t = k_o + k_1 h_i + k_2\left(\frac{h_i}{SF}\right) + k_3 T + k_4 h_i T + k_5\left[\frac{h_i T}{SF}\right] + k_6\left(\frac{T}{SF}\right) \quad \text{.................... (Eq. 11.7.3.1)}$$

where

t = time from the start of the test, in minutes

k_o to k_6 = regression coefficients, determined for use in equation 11.7.3.1

h_i = thickness of fire protection material, in millimetres

T = steel temperature, in degrees Celsius, $T > 250$ °C

SF = section factor, determined for use in equation 11.7.3.1

SF = k_{sm} (exposed surface area to mass ratio, in square metres/tonne); or

= $\left(\frac{H_p / A}{7.85}\right)$ (exposed perimeter to area ratio, in m^{-1})

➤

NOTE –

(1) If the regression is carried out at a constant steel temperature, only the first 3 terms on the right-hand side of equation 11.7.3.1 are used and only the 3 regression coefficients $k_o - k_2$ are determined.

(2) Equation 11.7.3.1 is set up to utilize the section factor expressed in square metres/tonne. The same form of this equation may also be used with the section factor expressed in metres^{-1}. The magnitude of the regression coefficients will differ in each instance and they must be determined and used as appropriate to the units chosen for the section factor.

11.7.3.2 Limitations and conditions on use of regression analysis

11.7.3.2.1
Test data to be utilized in accordance with 11.7.3.1 shall satisfy the following:

(a) Steel members shall be protected with board, sprayed, blanket or similar insulation materials having a dry density less than 1000 kg/m^3;

Amd 2 Oct. '07 NOTE – Intumescent coatings do not fulfil this criterion and hence do not come within the scope of 11.7.3. They may be tested and assessed in accordance with section 5 of the document *Fire Protection for Structural Steel in Buildings,* (4th edition). In that publication, intumescent paints are referred to as reactive fire protection systems.

(b) All tests shall incorporate the same fire protection system;

(c) All members shall have the same fire exposure condition;

(d) The test series shall include at least 9 tested members;

(e) The test series may include prototypes which have not been loaded provided that stickability has been demonstrated;

(f) All members subject to a three-sided fire exposure condition shall be within a group in accordance with 11.9.

11.7.3.2.2
The regression equations shall only be used for interpolation. The window defining the limits for interpolation for equation 11.7.3.1 shall be determined as shown in figure 11.7.3.2. Section 3 of the document *Fire Protection for Structural Steel in Buildings* (4th edition), provides details of how to undertake this interpolation as part of an assessment of the performance of passive fire protection products. Amd 2 Oct. '07

11.7.3.2.3
The regression equation obtained for one fire protection system may be applied to another system using the same fire protection material and the same fire exposure condition provided that stickability has been demonstrated for the second system.

11.7.3.2.4
A regression equation obtained using prototypes with a four-sided fire exposure condition may be applied to a member with a three-sided fire exposure condition provided that stickability has been demonstrated for the three-sided case.

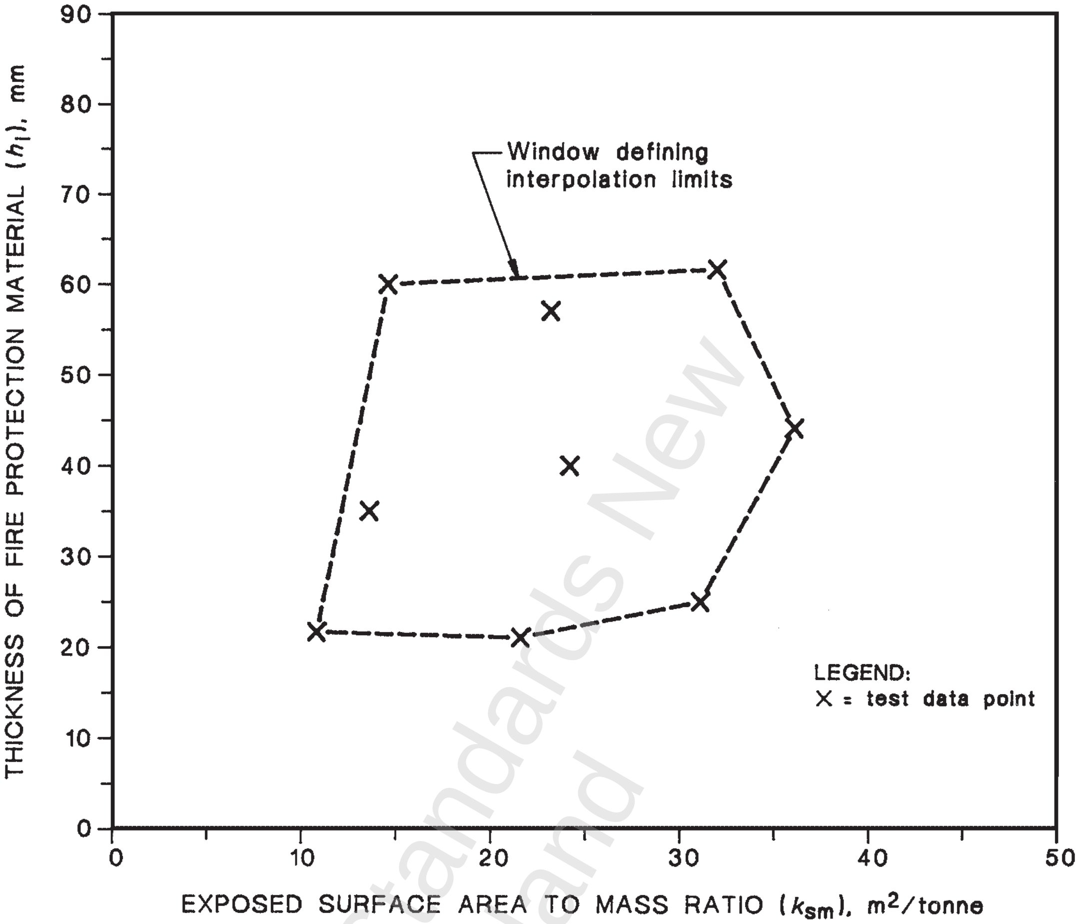


Figure 11.7.3.2 – Definition of window for interpolation limits for equation 11.7.3.1

11.8 DETERMINATION OF PSA FROM A SINGLE TEST

The period of structural adequacy (PSA) determined from data obtained from a single standard fire test may be applied without modification provided:

(a) The fire protection system is the same as the prototype;

(b) The fire exposure condition is the same as the prototype;

(c) The fire protection material thickness is equal to or greater than that of the prototype;

(d) The section factor is less than or equal to that of the prototype;

(e) The conditions of support are the same as the prototype and the restraints are not less favourable than those of the prototype; and

(f) The ratio of the design load for fire to the design capacity of the member is less than or equal to that of the prototype.

11.9 THREE-SIDED FIRE EXPOSURE CONDITION

Members subject to a three-sided fire exposure condition shall be considered in separate groups (configuration categories) unless the following conditions are satisfied:

(a) The characteristics of the members of a group shall not vary one from the other by more than:

(i) Concrete density: $\left(\dfrac{\text{highest in group}}{\text{lowest in group}}\right) \leq 1.25$, and

(ii) Effective thickness (h_e): $\left(\dfrac{\text{largest in group}}{\text{smallest in group}}\right) \leq 1.25$

where, for hollow slabs the effective thickness is equal to the cross-sectional area per unit width (excluding voids), or for composite slabs with profiled steel sheet the effective thickness is the average depth of concrete, as shown in figure 11.9 (a).

(b) Rib voids where profiled steel sheet is supported on beams shall, within any test series, be either:

(i) All open; or

(ii) All blocked, as shown in figure 11. 9 (b).

Concrete slabs may incorporate permanent steel deck formwork.

11.10 SPECIAL CONSIDERATIONS

11.10.1 Connections

11.10.1.1 Connections to protected members

Connections shall be protected with the maximum thickness of fire protection material required for any of the members framing into the connection to achieve their respective fire-resistance ratings. This thickness shall be maintained over all connection components, including bolt heads, welds and splice plates.

11.10.1.2 Connections to unprotected members

Connections transferring actions from a supported member, which is required to achieve a specified FRR and does so through compliance with 11.6, shall comply with (a) and (b) below:

(a) Connection components shall comply with 11.6, using the limiting temperature calculated from 11.5 and the Section Factor for the exposed cross section of the connection component.

(b) Connectors shall achieve the same or lower value of (r_f) for the connectors as that for the member being supported, where r_f is defined in 11.5.

11.10.2 Beam web penetrations

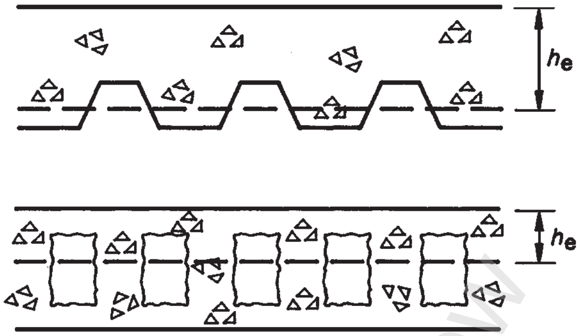
Unless determined in accordance with a rational fire engineering design, the thickness of fire protection material at and adjacent to web penetrations shall be the greatest of:

(a) That required for the area of beam above the penetration considered as a three-sided fire exposure condition (SF_1) (see figure 11.10.2);

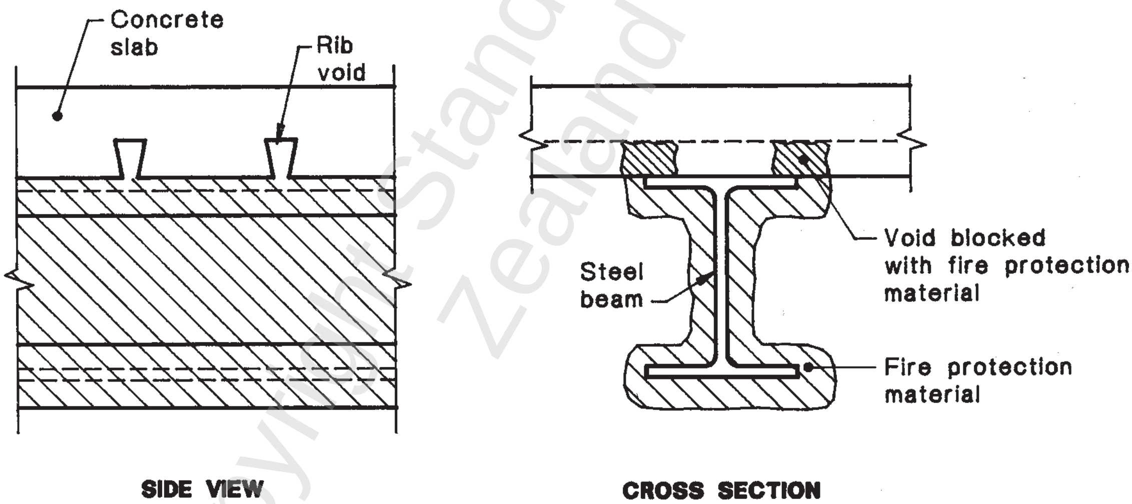
(b) That required for the area of beam below the penetration considered as a four-sided fire exposure condition (SF_2) (see figure 11.10.2); and

(c) That required for the section as a whole considered as a three-sided fire exposure condition (SF) (see figure 11.10.2).

This thickness shall be applied over the full beam depth and shall extend each side of the penetration for a distance at least equal to the beam depth, and not less than 300 mm.



(a) Effective thickness



(b) Blocking of rib voids

Figure 11.9 – Three-sided fire exposure condition requirements

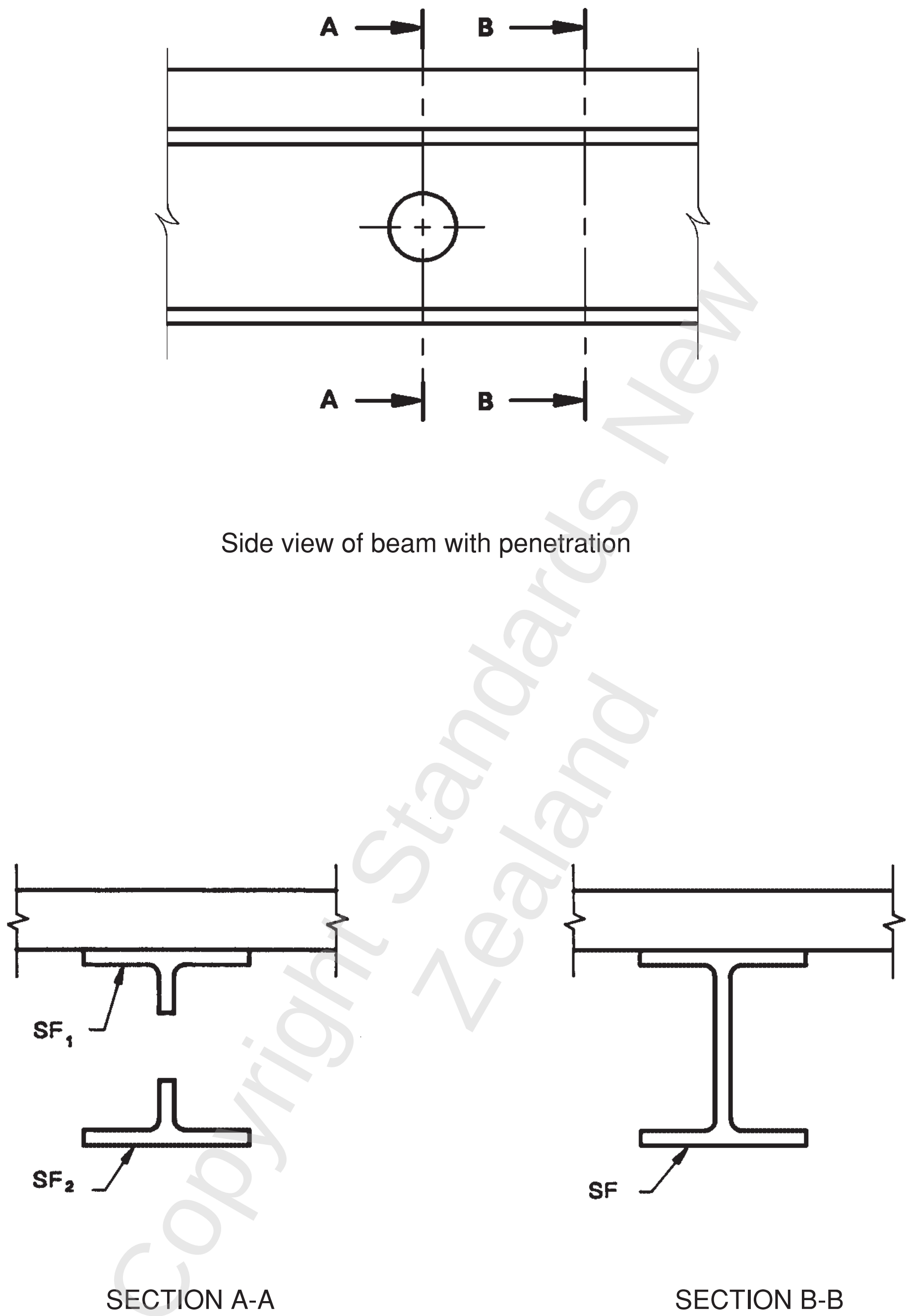


Figure 11.10.2 – Web penetrations

12 SEISMIC DESIGN

12.1 DEFINITIONS

For the purpose of this section, the definitions below apply.

COLLECTOR BEAM (as used in 12.12). A horizontal beam in a concentrically braced frame which transfers the horizontal component of the brace action developed in the concentrically braced frame seismic-resisting system.

MEMBER CATEGORY. A category required for application of the provisions of this section to individual members.

STRUCTURE CATEGORY. A category required for application of the provisions of this section to structural systems.

STOREY. The part of a structural system between 2 logical consecutive horizontal divisions.

12.2 GENERAL DESIGN AND ANALYSIS PHILOSOPHY

12.2.1 Scope

Section 12 presents detailed concepts and requirements that shall be followed when analysing and designing steel structures for earthquake resistance in accordance with this Standard. Unless specifically mentioned otherwise, all provisions of section 12 relate to design for the ultimate limit state, in accordance with sections 3 to 9 inclusive and section 13 of this Standard.

12.2.2 Structural performance factor and structural ductility demand

12.2.2.1 Structural performance factor values

The structural performance factor, S_p, shall be taken as equal to:

(a) For the serviceability limit state:............. $S_p = 0.7$

(b) For the ultimate limit state:

(i) For category 1 structures .... $S_p = 0.7$

(ii) For category 2 structures .... $S_p = 0.7$

(iii) For category 3 structures .... $S_p = 0.9$

(iv) For category 4 structures .... $S_p = 0.9$

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For the ultimate limit state for category 3 or 4 systems, if all the elements of the system meet the material, section geometry, member restraint and connection requirements of section 12 for category 2 members, then $S_p = 0.7$.

12.2.2.2 Structural ductility demand

12.2.2.2.1

In order to provide the level of earthquake resistance required by the Loadings Standard, the structure as a whole and all the elements that resist earthquake loads or effects shall be designed to possess an appropriate level of ductility as well as to satisfy the earthquake loading provisions of the Loadings Standard.

12.2.2.2.2
Adequate ductility may be considered to have been provided if all elements resisting earthquake loads or effects are designed and detailed in accordance with the relevant requirements of this Standard (principally section 12).

12.2.3 Classification of structural systems

Clause 12.2.3 applies to seismic-resisting systems, in accordance with 12.3.2 and 12.3.3. Parts of 12.2.3 also apply to associated structural systems in accordance with 12.3.4.

12.2.3.1 Categories of ductility demand
All steel seismic-resisting systems shall be classified into one of four categories for the purposes of seismic design. These four categories are:

(1) *Fully ductile systems* (Category 1 systems)
These are to be capable of sustaining structural displacement ductility demands sufficient to strain plastic hinges in the primary seismic-resisting members or elements into the strain-hardening region under severe earthquake loads or effects.

(2) *Systems of limited ductility capacity or subject to limited ductility demand* (Category 2 systems)
These are to be capable of sustaining structural displacement ductility demands sufficient to form plastic hinges in the primary seismic-resisting members or elements under severe earthquake loads or effects.

(3) *Nominally ductile systems* (Category 3 systems)
These are to be capable of sustaining structural displacement ductility demands sufficient to yield the flanges of primary seismic-resisting members or elements under the design level ultimate limit state earthquake loads or effects and to resist collapse under a maximum considered earthquake as directed by the Loadings Standard.

(4) *Elastic systems* (Category 4 systems)
These are expected to respond with minimal structural displacement ductility demand under the design level ultimate earthquake loads or effects and to resist collapse under a maximum considered earthquake as directed by the Loadings Standard. Elastic systems are not brittle systems: brittle systems are outside the scope of this Standard.

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12.2.3.2 Maximum structural displacement ductility demand
Appendix B gives maximum levels of structural ductility factor applicable for a range of seismic-resisting systems that shall be used in conjunction with the provisions of this section.

12.2.3.3 Application of structural classifications
The following requirements are applicable to each seismic-resisting system classification for design to the ultimate limit state.

(1) *Category 1 and 2 systems*
Capacity design procedures are required and the effects of concurrent action on columns which form part of a two-way seismic-resisting system shall be considered in accordance with 12.8.4(a) or (b).

(2) *Category 3 systems*
Capacity design procedures are required for category 3 structures which do not comply with table 12.2.6 Case Number 4 or as required by 12.11.1.1 or 12.12.5.3 and design for concurrent action is required by 12.8.4(b) or (c).

(3) *Category 4 systems*
Capacity design procedures are not required. Design for concurrent action is required by 12.8.4 (c).

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For systems including columns subject to concurrent actions (see 12.8.4) or for dual systems (see 12.13) the category of each system shall not differ numerically by more than one.

12.2.4 Structural displacement ductility demands

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Structural displacement ductility demands on the 4 categories of seismic-resisting systems for the ultimate limit state are specified in table 12.2.4.

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Table 12.2.4 – Relationship between category of structure and structural displacement ductility demand for the ultimate limit state

Category (12.2.3)	Description	Displacement ductility demand
1	Fully ductile	$\mu > 3.0$(See Note)
2	Limited ductile	3.0(See Note) $\geq \mu > 1.25$
3	Nominally ductile	$\mu = 1.25$
4	Elastic	$\mu = 1.0$

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NOTE – The maximum value of μ for category 2 systems may be period dependent for short period structural systems, in accordance with Note (5) to Appendix B.

For the serviceablility limit states defined in NZS 1170.5, the structural displacement ductility factor shall be $1.0 \leq \mu \leq 1.25$ for SLS1 and $1.0 \leq \mu \leq 2$ for SLS2.

12.2.5 Classification of members

All steel members which form part of a seismic-resisting system shall be classified into one of 4 categories for the purpose of seismic design. These categories are:

(1) *Members subject to high ductility demand* (Category 1 members).
These members shall be capable of sustaining high displacement ductility demands to the magnitude required as the primary seismic-resisting members in a category 1 seismic-resisting system.

(2) *Members subject to limited ductility demand* (Category 2 members).
These members shall be capable of sustaining low displacement ductility demands to the magnitude required as the primary seismic-resisting members in a category 2 seismic-resisting system or as the secondary seismic-resisting members in a category 1 seismic-resisting system.

(3) *Members subject to nominal ductility demand* (Category 3 members).
These members shall be capable of developing their nominal section capacity, where required to in bending, either as the secondary seismic-resisting members in a category 2 seismic-resisting system or as the primary seismic-resisting members in a category 3 seismic-resisting system.

(4) *Members subject to no ductility demand* (Category 4 members).
These members need not be designed to sustain any displacement ductility demand.

The same member classification system shall apply to associated structural systems, where appropriate, in accordance with 12.3.4.1.3.

12.2.6 Relationship between structure category and member category

This clause applies to seismic-resisting systems in accordance with 12.3.2 and 12.3.3 and where appropriate, to associated structural systems in accordance with 12.3.4.

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For systems outside the scope of table 12.2.6 the member category for each member of the seismic-resisting system shall be determined from matching the plastic hinge demand determined from analysis to 12.3.2.1 to the plastic hinge rotation limits of 4.7.2.

Other than an EBF active link (refer 12.11.3.3.1), plastic hinge rotation limits of 4.7.2 must also be checked for the members of any seismic-resisting system in which the clear length L of any primary member is $\leq 3\ M_s/V_w$.

Table 12.2.6 specifies the relationship between structural system and member category for the following applications:

(a) For seismic-resisting systems in which capacity design to prevent soft storey mechanisms from forming has been undertaken, except for category 1 to 4 concentrically braced frame systems which are covered by table 12.12.5.1.

(b) For category 3 seismic-resisting systems not exceeding the critical height (see 1.3) in which capacity design is not undertaken and for which the building is not irregular when assessed to the requirements of the Loadings Standard.

(c) For category 4 seismic-resisting systems not exceeding the critical height (see 1.3) and for which the building is not irregular when assessed to the requirements of the Loadings Standard.

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Table 12.2.6 – Relationship between structure category and member category
(Except as specified in limits on application (a) to (c) above and Notes 5,6)

Case number	Structural ductility category	Capacity design to prevent soft storey mechanism undertaken	Type of member of structural system (see Note 1)	Minimum member ductility category
1	1	Yes	Primary Secondary	1 2 (3) (see Note 2)
2	2	Yes	Primary Secondary	2 2 (3) (see Note 2)
3	3	Yes	Primary Secondary	3 3
4	3	No	Columns All other members	2 (see Notes 1, 3) 3
5	4	No	Columns All other members	3 (see Note 4) 4 (see Note 4)

NOTE –

(1) When a capacity design to prevent a soft storey mechanism is not undertaken, all members of the structural system are classified as primary (seismic-resisting) members. Because a column sidesway mechanism is not suppressed, more stringent minimum member ductility category requirements apply to the columns than to the other members of a structural system for which capacity design is not undertaken.

(2) The unbracketed value applies to secondary structural members which yield as the structure deforms laterally to obtain the desired mechanism (e.g. at the bottom of the columns of the storey immediately above the base). The bracketed value applies to other secondary members (e.g. in columns which may undergo lower levels of yielding above the base due to dynamic effects).

(3) Case number 4 is restricted by the limit on application (b) above. For single storey buildings, columns can be category 3.

(4) Case number 5 is restricted by the limit on application (c) above. For single storey buildings, columns can be category 4.

(5) For concentrically braced frame seismic-resisting systems, the relationship between structure category and member category is given in table 12.12.5.1.

(6) For eccentrically braced frames, see 12.11.3.2 for application of this table.

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12.2.7 Capacity design

12.2.7.1 General philosophy

In capacity design of a seismic-resisting system, the principal energy dissipating elements of mechanisms (the primary seismic-resisting elements) shall be chosen and suitably designed and proportioned to the requirements of this Standard, while all other elements of the seismic-resisting system (the secondary seismic-resisting elements) are provided with sufficient reserve strength to ensure that the chosen energy dissipating mechanisms within the seismic-resisting system are maintained throughout the deformations that may occur.

12.2.7.2

In the design of an associated structural system, the principle of capacity design shall be utilized, where necessary, to ensure that only elements of the associated structural system which can dependably resist anticipated inelastic demand (due to the behaviour of the seismic-resisting system under severe seismic loading) are required to do so. These elements are the primary seismic-resisting elements of the associated structural system.

12.2.7.3

The design capacity of the primary seismic-resisting elements shall be used to resist the design actions obtained from the design loads.

12.2.7.4

The design capacity of the secondary seismic-resisting elements shall be used to resist the overstrength design actions obtained through the capacity design process specified in this Standard.

12.2.7.5

When a capacity design is not undertaken, the design capacity of all elements (i.e. members and connections) of the structural system shall be used to resist the design actions.

12.2.8 Overstrength

Overstrength factors for use in section 12 are given in tables 12.2.8.

Table 12.2.8(1) – Overstrength factors for beams, braces and columns

	Category 1 members		**Category 2 members**				**Category 3 members**				
Steel grade	300[3]	300[2,3] from A/NZ	300[3]	300[2,3] from A/NZ	350	350[4] from Aust.	300[3]	300[2,3] from A/NZ	350	350[4] from Aust.	450
Strain hardening (ϕ_{os})	1.15	1.15	1.05	1.05	1.10	1.10	1.00	1.00	1.00	1.00	1.10
Material variation (ϕ_{om})	1.30	1.20	1.30	1.20	1.30	1.25	1.30	1.20	1.30	1.25	1.30
Overstrength[1] (ϕ_{oms})	1.35	1.25	1.25	1.15	1.30	1.25	1.15	1.10	1.15	1.15	1.30

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Table 12.2.8(2) – Overstrength factors for active links in EBFs

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	Category 1 members		Category 2 members				
Steel grade	300(3)	300(2,3) from A/NZ	300(3)	300(2,3) from A/NZ	350	350(4) ex Aust	
Strain hardening (ϕ_{os})	1.30	1.30	1.20	1.20	1.25	1.25	
Material variation (ϕ_{om})	1.30	1.20	1.30	1.20	1.30	1.25	
Overstrength(1) (ϕ_{oms})	1.50	1.40	1.40	1.30	1.45	1.40	Non-composite active links
Overstrength(1,5) (ϕ_{oms})	1.65	1.55	1.55	1.45	1.60	1.55	Composite(5) active links

NOTE –

(1) The values of ϕ_{oms} incorporate the reciprocal of the ideal capacity factor (see 1.3) and hence must be used in conjunction with the design capacity, ϕR_u, of the member or connection resisting the capacity design derived action, S^* (see 12.2.7.4).

(2) These values apply only to BHP 300PLUS sections, flat bar and plate produced by BHP Australia or to 300MOD welded sections produced by BHP New Zealand Steel.

(3) Use these values for Grade 250 steel.

(4) These values apply only to grade 350 sections, flat bar and plate produced by BHP Australia.

(5) A composite active link is one which is designed and detailed to act compositely with a concrete slab; the typical practice of not applying welded shear studs over the active link region is sufficient to classify the active link as "non-composite" for the determination of ϕ_{oms}, even though the adjacent beams may be detailed to act compositely with a concrete slab.

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(6) For moment-resisting frames and for long link D braced EBF seismic-resisting systems comprising beams supporting a concrete slab, refer to 12.10.2.4 and 12.11.7.2.

12.2.9 Damping values and changes to basic design seismic load

12.2.9.1 Damping values

Values of initial viscous damping (η) for use in the seismic design of steel structures are presented in table 12.2.9.

Table 12.2.9 – Initial damping values for steel structures
(given as a percentage of critical viscous damping)

Case number	Type of structure	Behaviour (3)	Welded connections	Bolted connections
1	Clad	Elastic	4 — 6	5 — 10
2	Clad	Inelastic	5 — 7	10 — 15
3	Unclad	Elastic	2 — 3	5 — 7
4	Unclad	Inelastic	5 — 7	10 — 15

NOTE –

(1) The terms clad and unclad are defined in 1.3.

(2) The lowest value of damping given for each case should be used in design unless a higher value can be justified in a particular instance.

(3) Inelastic behaviour applies wherever attainment of yield of the primary seismic-resisting members is expected to occur.

12.2.9.2 Influence of damping values on the ultimate limit state design seismic load for category 3 and 4 structural systems

All values of basic seismic acceleration coefficient (C_h) given by the Loadings Standard relate to an initial (viscous) damping level of 5 %. Where a value of damping greater than 5 % is applicable from 12.2.9.1, and the zone factor Z, from NZS 1170.5 is greater than 0.16 , the appropriate value of C_h for category 3 or 4 structural systems, as derived in accordance with the Loadings Standard, shall be modified as follows:

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$$\frac{C_{h\eta}}{C_{h5}} = \left[0.5 + \left(\frac{1.5}{0.4\eta + 1}\right)\right]$$

where

η = percentage of initial viscous damping from 12.2.9.1

C_{h5} = basic seismic acceleration coefficient from the Loadings Standard determined from the spectrum curve for $\mu = 1.0$ or $\mu = 1.25$ as appropriate, for the fundamental period, T_1

$C_{h\eta}$ = C_h applicable to $\eta \neq 5$ %.

This clause is not applicable to category 1 or 2 structural systems, for which the basic seismic acceleration coefficient as determined from the Loadings Standard shall be used without modification.

12.2.9.3 Influence of damping values on the serviceability limit state design seismic load for any category of structural system

Apply 12.2.9.2 for all categories of seismic-resisting system, using the damping values from (a) or (b) as appropriate;

(a) For category 1 systems, use the damping values from table 12.2.9 associated with inelastic behaviour.

(b) For category 2, 3 or 4 systems, use the damping values from table 12.2.9 associated with elastic behaviour.

12.3 METHODS OF ANALYSIS AND DESIGN

12.3.1 Scope

Clause 12.3.1 directs designers to the appropriate sections and/or specific clauses of this Standard for the following applications:

(a) Methods of analysis and design of seismic-resisting systems and their individual members (12.3.2, 12.3.3).

(b) Methods of analysis and design of associated structural systems and their individual members (12.3.4).

12.3.2 Methods of analysis of seismic-resisting systems

12.3.2.1 Ultimate limit state

12.3.2.1.1
Analysis shall be in accordance with section 4, subject to the following requirements:

(a) Elastic analysis in accordance with 4.4, or elastic analysis with redistribution in accordance with 4.5 may be undertaken. In applying 4.5 to seismic-resisting systems, the member category selected shall satisfy the requirements of 12.2.6 for the structural classification chosen and also the requirements of 4.5.4 and 4.5.5 for the level of redistribution applied.

(b) Plastic analysis in accordance with 4.6 may be used for moment-resisting framed systems which do not exceed the critical height (see 1.3) and which have a structural category of 2 in accordance with 12.2.3.

(c) Plastic analysis may be used for eccentrically braced frames with active links designed to respond to severe seismic forces in a flexural mode and which do not exceed the critical height (see 1.3).

12.3.2.1.2
The basic design seismic load shall be determined in accordance with the provisions of the Loadings Standard, utilizing a structural ductility factor (μ) applicable to the seismic-resisting system under consideration.

12.3.2.1.3
For moment-resisting and eccentrically braced frames, the design seismic load shall be taken as the basic design seismic load.

12.3.2.1.4
For concentrically braced frames, the design seismic load for the ultimate limit state shall be taken as the basic design seismic load multiplied by the factor, C_S, given in 12.12.3 or 12.12.6 as appropriate.

12.3.2.1.5
The lateral deflection of a seismic-resisting system shall be determined in accordance with the provisions of the Loadings Standard and shall not exceed the limits specified therein. For category 1 and 2 concentrically braced frames, the lateral deflection shall be increased as required by 12.12.5.2 (h).

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12.3.2.2 Serviceability limit state

12.3.2.2.1
Elastic analysis in accordance with 4.4 or elastic analysis with redistribution in accordance with 4.5.5 shall be used for any category of seismic-resisting system.

12.3.2.2.2
The design seismic load shall be the basic design seismic load determined in accordance with the provisions of the Loadings Standard.

12.3.2.3 Assessment of P-delta effects (P – Δ and P – δ)

12.3.2.3.1
Any increase required in design actions on the members of the seismic-resisting system due to $P-\Delta$ effects shall be determined in accordance with the provisions of the Loadings Standard.

12.3.2.3.2
The structural displacement ductility demand, μ, used in implementing the provisions of the Loadings Standard may be that associated with the actual seismic-resisting system as designed (μ_{act}) rather than that associated with initial choice of structure category (μ_{des}) from 12.2.3.1.

12.3.2.3.3
Any increase in design actions due to $P-\delta$ effects shall be determined in accordance with 4.4.2, 4.5.2 or 4.6.4 as appropriate for the method of structural analysis used and the braced or sway status of the member from 4.1.2.

12.3.3 Design of members and connections of seismic-resisting systems

12.3.3.1 Scope of this clause

Clauses 12.3.3.2 to 12.3.3.5 list the relevant sections and clauses of this Standard which apply to the design of beams, columns and connections of seismic-resisting systems.

12.3.3.2 Beams

12.3.3.2.1
The category of a beam shall be chosen from 12.2.5, consistent with the choice of system category from 12.2.3 and the requirements of 12.2.6. The material used shall comply with 12.4.

12.3.3.2.2
The design of a beam shall be in accordance with section 5 and also in compliance with the section geometry requirements of 12.5 and the member restraint requirements of 12.6.

12.3.3.2.3
Webs of beam within yielding regions shall comply with 12.7.2.

12.3.3.2.4
Maximum design actions are given in 12.3.3.4.

12.3.3.3 Columns

12.3.3.3.1
The category of a column shall be chosen from 12.2.5, consistent with the choice of system category from 12.2.3 and the requirements of 12.2.6. The material shall comply with 12.4.

12.3.3.3.2
The design of a column shall be in accordance with section 6 for axial compression and section 8 for combined actions and also in compliance with the section geometry requirements of 12.5 and the member restraint requirements of 12.6.

12.3.3.3.3
The effective length of a column, for use in sections 4, 6 and 8 shall be determined in accordance with 12.8.2.

12.3.3.3.4
The axial force on a column shall comply with the limits of 12.8.3.

12.3.3.3.5
Concurrent action in columns forming part of a two-way seismic-resisting system shall be considered in accordance with 12.8.4.

12.3.3.3.6
Webs of columns within yielding regions shall comply with 12.8.5.

12.3.3.3.7
Maximum design actions are given in 12.3.3.4.

12.3.3.4 Maximum design actions for members of seismic-resisting systems
The members of seismic-resisting systems need not be designed to transmit design actions greater than (a) or (b) below:

(a) Those generated by nominally ductile seismic response in conjunction with the appropriate design gravity loads for category 1 seismic-resisting systems or for category 2 seismic-resisting systems with $\mu_{act} \geq 1.8$. The value of S_p used shall be 0.7.

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(b) Those generated by elastic seismic response in conjunction with the appropriate design gravity loads for category 3 or 4 seismic-resisting systems or for category 2 seismic-resisting systems with $\mu_{act} < 1.8$. The value of S_p used shall be 0.7 for category 2 systems, 0.9 for category 3 seismic-resisting systems and 0.9 for category 4 seismic-resisting systems.

12.3.3.5 Connections

The following requirements apply:

(a) Connection design shall be in accordance with section 9 and the appropriate requirements of 12.9;

(b) Connection design shall comply with the design philosophy and general requirements of 12.9.1;

(c) Minimum design actions for connections in seismic-resisting or associated structural systems are specified in 12.9.2;

(d) Welds shall be designed in accordance with the requirements of 9.7 and 12.9.3;

(e) Bolts shall be designed in accordance with the requirements of 9.3 and 12.9.4;

(f) The design of elements of moment-resisting connections shall be in accordance with 12.9.5;

(g) Design of splices shall be in accordance with 12.9.3, 12.9.4, 12.9.6 and the appropriate requirements of section 9;

(h) Design of gusset plates shall be in accordance with 12.9.3, 12.9.4, 12.9.7 and the appropriate requirements of section 9;

(j) Design of connections between built up members shall comply with 12.9.3, 12.9.4, 12.9.8 and the appropriate requirements of section 9;

(k) Specific requirements for connections in moment-resisting, eccentrically and concentrically braced frame seismic-resisting systems are contained in 12.10, 12.11 and 12.12 respectively.

12.3.4 Design and detailing of members and connections of associated structural systems

12.3.4.1 General requirements

12.3.4.1.1
Clause 12.3.4 presents general requirements for the design and detailing of members and connections of structural systems which are not specifically designed for load combinations including earthquake loads, but which are subjected to actions resulting from the deformation of the seismic-resisting system under a severe seismic level of loading.

12.3.4.1.2
Assess the extent of the actions and inelastic demands on the elements of the associated structural system due to the total deformations of the seismic-resisting system under severe seismic loading.

12.3.4.1.3
The anticipated level of inelastic demand for the associated structural system as a whole may be taken as that determined for one category less severe than that for which the seismic-resisting system has been designed and detailed, with a minimum severity of category 3 applicable when the seismic-resisting system is designed for either limited ductility (category 2) or nominally ductile (category 3) response under severe seismic loading.

12.3.4.2 Inelastic demands on elements

Determine which elements of the associated structural system may be subject to inelastic demands, utilizing the principle of capacity design from 12.2.7, where necessary, to ensure that only elements which can dependably resist the anticipated inelastic demands are required to do so.

Note that, with an associated structural system, all 3 forms of construction specified in 4.2.2 may be used and inelastic demand may be concentrated into the connections, provided that they comply with 12.3.4.4.

12.3.4.3 Application to members

12.3.4.3.1
For an element, selected in accordance with 12.3.4.2, which is a member, choose the appropriate member category from 12.2.5 consistent with the anticipated level of inelastic displacement ductility demand and design and detail the member in accordance with the appropriate requirements of 12.4 to 12.8 inclusive.

12.3.4.3.2
The inelastic demand may be directed into specified members of the associated structural system through the application of capacity design (see 12.2.7).

12.3.4.4 Application to connections

12.3.4.4.1
For an element, selected in accordance with 12.3.4.2, which is a connection, design and detail the connection in accordance with the appropriate requirements of 12.9.

12.3.4.4.2
Connections in associated structural systems in which the lateral load is resisted by a category 1 or 2 seismic-resisting system shall be designed and detailed to accommodate the anticipated inelastic rotation without compromising their capacity to resist the design gravity loads, determined from the Loadings Standard, that act in conjunction with the earthquake loads.

12.4 MATERIAL REQUIREMENTS

12.4.1 Material requirements

12.4.1.1

Steels in seismic-resisting systems must meet the criteria in table 12.4:

Table 12.4 – Material requirements

Item		Category 1, 2 and 3 members	Category 4 members
1	Maximum specified grade reference yield stress (see Note 1)	360 MPa	450 MPa
2	Minimum % total actual elongation (see Notes 2, 3)	25	15
3	Maximum actual yield ratio (f_y/f_u) (see Note 2)	0.80	0.90
4	Maximum actual yield stress (see Note 2)	≤ 1.33f_y (see Note 4)	–
5	Minimum Charpy V-Notch impact energy (see Notes 3, 5, 6)	70J @ 0 °C – Average of three tests 50J @ 0 °C – Individual test	No special earthquake provisions required

NOTE –

(1) The limits in item 1 are based on a grade reference steel thickness of 12 < t ≤ 20 mm from the appropriate materials supply Standard from 2.2.1.

(2) For items 2, 3 and 4, the mechanical properties are those recorded on the certified mill test report or test certificate.

(3) f_y is the specified yield stress from 2.1.1.

(4) Tensile and Charpy V-Notch testing shall be completed and assessed for compliance in accordance with the provisions for selection, position and orientation, preparation for testing and testing procedures found in AS/NZS 3679.1 for hot rolled steel sections, AS/NZ 3678 for plate used in welded steel sections and AS 1163 for structural steel hollow sections.

(5) Charpy V-Notch testing is only required for sections greater than 12 mm thick.

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(6) These impact requirements are for steel in environments where the basic service temperature (see 2.6.3) is ≥ 5 °C.

12.4.1.2

Category 1 or 2 members shall be hot-rolled or fabricated by welding from hot-rolled plate, except that category 2 members may be cold-formed, provided that adequate ductility capacity of the member and its connections is established by experimental testing or rational design.

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(Text deleted)

12.5 SECTION GEOMETRY REQUIREMENTS

12.5.1 General requirements

12.5.1.1
Elements of category 1, 2, 3 and 4 members shall comply with the plate element slenderness limitations presented in table 12.5. The limitations given for category 1, 2 or 3 members need be applied only within the yielding regions of these members; for the non-yielding regions of category 1, 2 or 3 members the relevant plate element slenderness limits for category 4 members may be applied, provided yielding is prevented in these regions by application of the capacity design philosophy (see 12.2.7).

12.5.1.2
The plate element slenderness limits in table 12.5 apply to the physical properties of the section; calculation of section moment capacity shall be in accordance with 5.2, utilizing the relevant plate element slenderness limits of table 5.2 for this purpose.

12.5.1.3
Calculation of section properties for determination of member and structure deflection shall be in accordance with Appendix N.

12.5.2 Cross section geometry requirements for members other than braces in concentrically braced frames

12.5.2.1
The yielding regions of category 1 or 2 members shall be doubly symmetric sections.

12.5.2.2
The yielding regions of category 3 members shall be doubly or singly symmetric sections.

12.5.2.3
The braces in eccentrically braced frames shall be doubly or singly symmetric sections.

12.5.3 Cross section geometry requirements for members in concentrically braced frames

12.5.3.1 Braces effective in tension and compression

12.5.3.1.1
The plate element slenderness limits for uniform compression, as given in table 12.5, shall apply to all elements of the brace cross section.

12.5.3.1.2
The braces shall be doubly or singly symmetric sections.

12.5.3.2 Braces effective in tension only

The plate element slenderness limits given in table 12.5 do not apply to these sections used as braces (either as individual members or as built-up members).

Table 12.5 – Values of plate element slenderness limits (for members subject to design actions including earthquake loads or effects or as required by 4.5.7.1)

Case number	Plate element type	Longitudinal edges supported	Residual stresses (see Notes)	Category 1 Members (λ_{e1})	Category 2 Members (λ_{e2})	Category 3 Members (λ_{e3})	Category 4 Members (λ_{e4})
1	Flat (Uniform compression)	One	SR HR LW, CF HW	10 9 8 8	10 9 8 8	11 10 9 9	25[3] 25[3] 22[3] 22[3]
2	Flat (Maximum compression at unsupported edge, zero stress or tension at supported edge)	One	SR HR LW, CF HW	10 9 8 8	10 9 8 8	11 10 9 9	25 25 22 22
3	Flat (Uniform compression)	Both	SR HR LW, CF HW	25 25 25 25	30 30 30 30	40 40 40 35	60[3] 60[3] 60[3] 60[3]
4	Flat (Either non-uniform compression or compression at one edge, tension at the other)	Both	Any	82[5] 30[6]	82[5] 40[5,6]	101[4,5] 55[5,6]	161[4] 75[6]
5	Circular hollow sections		SR HR, CF LW HW	35 35 30 30	50 50 42 42	65 65 60 60	170[3] 170[3] 170[3] 170[3]

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NOTE –

(1) SR – stress relieved
HR – hot-rolled
CF – cold formed
LW – lightly welded longitudinally
HW – heavily welded longitudinally

(2) Welded members whose compressive residual stresses are less than 40 MPa may be considered to be lightly welded.

(3) As this limit exceeds the yield limit (λ_{ey}) from tables 5.2 or 6.2.4, effective section modulus or effective width concepts may apply in the calculation of section moment or compression capacity (see 5.2.5 or 6.2.4).

(4) Unstiffened webs which exceed the slenderness limit $(d_1/t)\sqrt{\left(\frac{f_y}{250}\right)} = 82$ will have their nominal shear strength reduced from the nominal shear yield capacity by 5.11.5.1.

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(5) Members that have unstiffened webs which exceed the ratio of $(d_1/t)\sqrt{\left(\frac{f_y}{250}\right)} = 30$ for category 1 or 2 members, or 40 for a category 3 member, shall comply with the axial compression force limitation of 12.8.3.1(c).

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(6) These limits are applicable to the webs of rectangular and square hollow section members.

12.6 MEMBER RESTRAINT

12.6.1 General

12.6.1.1 Scope of application

Section 12.6 covers the restraint of category 1 to 4 members, except that it does not apply to braces in concentrically braced framed seismic-resisting systems.

All of section 12.6, except for 12.6.2.4, applies to members subject to major principal *x*-axis bending. These provisions are applied on a segment by segment basis, consistent with 5.3.1.1. A segment is as defined in 5.3.1.2.

Clause 12.6.2.4 applies to members subject to minor principal *y*-axis bending.

12.6.1.2 Application to critical flange under major principal x-axis bending

The restraint requirements of section 12.6 shall apply to the critical flange (see 5.5) of a cross section.

NOTE – Under reversing loading, both flanges of a cross section may be critical.

12.6.1.3 Design bending moment for application for restraint to category 1, 2 and 3 members

When calculating the design moment magnitude and distribution along the member for application of clause 12.6.2;

(a) At the point of maximum moment in each yielding region, set the design bending moment for calculation of restraint equal to the design section moment capacity, reduced by axial force where appropriate, i.e.

$$M^*_{res,max} = \phi M_s$$

(b) At each point along the member,

$$M^*_{res} = M^*\left(\frac{\phi M_s}{M^*_{max}}\right) \quad \text{.......(Eq. 12.6.1)}$$

where

M^*_{res} = design bending moment for calculation of restraint to clause 12.6.2

M^* = design bending moment from analysis at the point under consideration

M^*_{max} = maximum design bending moment from analysis at the adjacent yielding region(s)

ϕM_S = ϕM_S or ϕM_r as appropriate (see 1.4).

Amd 1 June '01 (c) Use the value of M^*_{res} from Equation 12.6.1 in the application of clauses 12.6.2.1 to 12.6.2.5.

12.6.2 Restraint of category 1, 2 and 3 members

12.6.2.1 Length of yielding region

The length of a yielding region within the member shall be obtained from table 12.6.1.

Table 12.6.1 – Length of yielding region

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Category of member	Calculated length of yielding region	Minimum length(2) required
1	L_{yr}(1)	1.5 d (3)
2	L_{yr}(1)	1.5 d (3)
3	1.0 d (3)	1.0 d (3)

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NOTE –

(1) L_{yr} is given by equation 12.6.2.

(2) This length is applied from the position of maximum design moment.

(3) d = depth of section at position of maximum design moment.

The calculated length of yielding region, L_{yr}, is given by:

L_{yr} = length of member for which $M_r^* > C_1 \phi M_s$ .... (Eq. 12.6.2)

where

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M_{res}^* = the design bending moment at the point under consideration for calculation of the length of yielding region

C_1 = 0.85 for $N^*/\phi N_s \leq 0.15$

C_1 = 0.75 for $N^*/\phi N_s > 0.15$

ϕ = strength reduction factor (see table 3.3)

M_s = M_s or M_r as appropriate (see 1.4).

12.6.2.2 Restraint of segments which contain one or more yielding regions

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Each segment containing one or more yielding regions shall have $(\alpha_m \alpha_s) \geq 1.0$, where:

α_m and α_s are determined in accordance with 5.6, but $\alpha_m \leq 1.75$ for application of this clause.

(Table 12.6.2 deleted)

12.6.2.3 Restraint requirements within the length of a yielding region, when subject to major principal x-axis bending

12.6.2.3.1

The maximum length between adjacent restraints, within the length of yielding region, L_{yr}, shall comply with:

$$\frac{L}{r_y} \leq k_{yr} \sqrt{\frac{250}{f_y}}$$

where

L = length of member between adjacent restraints within the yielding region, L_{yr}

r_y = radius of gyration about the minor principal y-axis

k_{yr} = yielding region restraint spacing coefficient, obtained from table 12.6.3.

12.6.2.3.2

The type of restraint required within the yielding region shall be as specified in table 12.6.3.

For category 1 and 2 members, at least one restraint of this type shall be provided within or at one end of each length of yielding region.

For category 3 members, at least one F restraint and as many other F or P restraints, as required, shall be provided within, or at one end of each length of yielding region.

Amd 1 June '01 NOTE – Where the yielding region is at the end of a member, the end connection will provide a suitable restraint to the yielding region. In most applications, this will be the only restraint required from 12.6.2.3.

Table 12.6.3 – Yielding region restraint requirements

Type of cross section	Category of member	Yielding region restraint spacing coefficient, k_{yr}	Type of restraint required within yielding region(3,4)
Equal flanged I-section	1 2 3	30 40 60	F F F or P
Equal flanged channel	1 2 3	N/A(1) N/A(1) 44	– – F or P
I-section with unequal flanges	1 2 3	N/A(1) N/A(1) $60\left[\frac{\rho A d_f}{1.25 Z_{ex}}\right]^{0.5}$	– – F or P
Rectangular or square hollow section	1 2 3	300 b_f/b_w 600 b_f/b_w 1200 b_f/b_w	F F F or P

NOTE –

(1) These cross sections cannot be used within this category of yielding region; see 12.5.2 and 12.5.3.

(2) Refer to 5.3.2.4 for the definitions of notation used in this table.

(3) Refer to 5.4.2.1 or 5.4.2.2 for the requirements for full or partial section restraints, respectively; except that

(4) Bolt holes in yielding region restraints shall comply with 14.3.5.2.1 and the bolts shall be tensioned property class 8.8 (category 8.8/*TB*) to 15.2.4 and 15.2.5.

12.6.2.4 Restraint requirements within the length of a yielding region, when subject to minor principal y-axis bending

One restraint, which prevents twist rotation of the cross section about the member's longitudinal axis, shall be provided within or at one end of the length of a yielding region subject to *y*-axis bending.

12.6.2.5 Restraint of category 1, 2 and 3 member segments which do not contain a yielding region

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(a) When the member is subject to bending, these segments shall satisfy $\phi M_{bx} \geq M^*_{res}$ (see 5.6 for ϕM_{bx} and Equation 12.6.1 for M^*_{res}) in order that the required inelastic action is able to be developed in the yielding region or regions within the member.

(b) When the member is subject to combined bending and significant axial actions (see 8.1.4), these segments are required to have full lateral restraint from 8.1.2.4.

12.6.3 Restraint of category 4 members

Restraint of category 4 members shall be sufficient to ensure that the design member capacity, (ϕM_{bx}), determined from 5.3 or 5.6 as appropriate, equals or exceeds the design moment (M^*_x) at all points along the member.

12.7 BEAMS

12.7.1 General

Beams required to resist design load combinations including earthquake loads shall comply with the requirements of 12.2.5 with regard to classification for level of ductility demand.

12.7.2 Webs of beams within yielding regions

12.7.2.1

The web thickness within the yielding region of a beam shall be not less than $(d_1/82)\sqrt{(f_y / 250)}$ for a category 1 or 2 member or less than $(d_1/101)\sqrt{(f_y / 250)}$ for a category 3 member.

12.7.2.2

Load bearing stiffeners shall be provided when a bearing load or shear force acts within $d_1/2$ of a yielding region and the design bearing load or shear force exceeds 0.1 times the design shear capacity (ϕV_v) of the member as specified in 5.11.

12.7.2.3

These stiffeners shall be located within a distance $d_1/2$ on either side of the yielding region and shall be designed in accordance with 5.14 to carry the greater of the design bearing load or the design shear force considered as a bearing load.

12.7.2.4

If the stiffeners are flat plates, their slenderness parameter (λ_s), as defined in 5.2.2 using the stiffener yield stress (f_{ys}), shall be less than the slenderness limit appropriate to the category of member ($\lambda_{e1,2,3}$) specified in table 12.5.

12.8 COLUMNS

12.8.1 General

Columns required to resist design load combinations including earthquake loads shall comply with the requirements of 12.2.5 with regard to classification for level of ductility demand.

12.8.2 Effective lengths of columns and elastic stability of the seismic-resisting system

12.8.2.1 General

The effective length, $k_e L$, of a column shall be determined independently about each principal axis of the member, according to the appropriate requirements of 12.8.2.2 to 12.8.2.6.

12.8.2.2 Effective length of columns for calculation of the frame elastic buckling load factor (λ_c)

(a) *For a moment-resisting framed seismic-resisting system*
When calculating λ_c from 4.9.2 for such a frame of rigid construction, the elastic effective length associated with the elastic buckled shape shall be used. When calculating λ_c from 4.9.2.3, the member effective length factor (k_e) in the plane of the frame shall be obtained from figure 4.8.3.3(b) and the γ-values shall be determined in accordance with 4.8.3.4.

(b) *For a concentrically or eccentrically braced framed seismic-resisting system*
When calculating λ_c from 4.9.2, the effective length factor (k_e) in the plane of the frame shall be taken as 1.0.

12.8.2.3 Calculation of the frame elastic buckling load factor (λ_c)

(a) *For a moment-resisting framed seismic-resisting system*
The elastic buckling load factor shall be calculated from 4.9.2. Where appropriate, the approximate method of 4.9.2.3 may be used.

(b) *For a concentrically or eccentrically braced framed seismic-resisting system*
The elastic buckling load factor shall be calculated from 4.9.2. Where appropriate, the approximate method of 4.9.2.2 and 4.9.2.3 may be used, with λ_c taken as the lowest value calculated from:

(i) Clause 4.9.2.3; and

(ii) Clause 4.9.2.2 with $N^* = N_g^*$ (the design axial compressive force generated by gravity loading alone).

12.8.2.4 Effective length factor for design of individual column members forming part of a seismic-resisting system

For any form of seismic-resisting system in which second-order effects have been evaluated in accordance with section 4, the effective length factor (k_e) for column member design in the plane of the seismic-resisting system shall be taken as 1.0.

12.8.2.5 Effective length of columns in dual seismic-resisting systems

When the lateral load is resisted by a dual system, comprising a moment-resisting frame in conjunction with one or more other seismic-resisting systems in accordance with 12.13, the effective length of the columns in the moment-resisting frame and the determination of elastic second-order effects on the structural system shall be based on the assumption that the frame depends on its own bending stiffness to provide the lateral stability of the system.

12.8.2.6 Effective length of members within a triangulated structure which forms part of a seismic-resisting system

The effective length of such members shall be determined in accordance with 4.8.3.5.

12.8.3 Axial force and transverse load limitations on columns and braces

12.8.3.1 Limitations on axial force

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The ratio of design axial force, N^*, to design section capacity, ϕN_s,(refer to 6.2) shall not exceed the values given in (a) and (b) below.

The ratio of design axial force generated by gravity loading alone, N^*_g, to design section capacity, ϕN_s, shall not exceed the value given in (c) below.

(a) The general limit given in table 12.8.1.

Table 12.8.1 – General limit on (N^* / ϕN_s) as a function of member category
(except as given by Note 3)

Category of member (see 12.2.5 or 4.5)	(N^* / ϕN_s) not to exceed:	
	In a column member	In a brace in an eccentrically braced frame
1	0.5	– (1)
2	0.7	– (1)
3	0.8	0.8
4	1.0	1.0

NOTE –
(1) This category of member is not appropriate for this application.

(2) N^* is compressive or tensile.

(3) This table does not apply to braces in concentrically braced frames with braces effective in tension and compression. For such members, the axial compression force is limited by 6.1, in accordance with 12.12.2.2. Member capacity from 6.3.3 will govern the design.

(b) In addition to (a), for category 1, 2 and 3 column members, excluding brace members of concentrically and eccentrically braced frames, the following limitation on design axial compression shall apply, unless waived according to 12.8.3.2.

Where capacity design is not undertaken:

$$\frac{N^*}{\phi N_s} \leq \left\{ \frac{0.263(\beta_m + 1)^{0.88}}{e^{\left(0.19/(\beta_m+1)\right)}} \right\}^{\lambda_{EYC}} \quad \text{(Eq. 12.8.3.1(1))}$$

Where capacity design is undertaken and the column is the secondary element to 12.2.7.1:

$$\frac{N^*_{oc}}{\phi N_s} \leq \left\{ \frac{0.263(\beta_m + 1)^{0.88}}{e^{\left(0.19/(\beta_m+1)\right)}} \right\}^{\lambda_{EYC}} \quad \text{(Eq. 12.8.3.1(2))}$$

where

$$\lambda_{EYC} = \sqrt{\frac{N_s}{N_{oL}}}$$

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N^* = the design axial compression force

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N^*_{oc} = the capacity design derived design axial compression force on the column when the column is a secondary element to 12.2.7.1

β_m = 0 for columns forming part of a seismic-resisting system

β_m = + 0.5 for columns forming part of an associated structural system

N_s = the nominal section capacity, determined in accordance with 6.2.1.1

N_{oL} = $\dfrac{\pi^2 EI}{L^2}$

I = the second moment of area for the axis about which the design moment acts (i.e. the axis perpendicular to the plane of the structural system)

L = the actual length of the member (see 6.3.1).

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The scope of application is as defined in the Standard.

Equation 12.8.3.1 shall be applied about the column major *x*-axis in all cases and about the column minor *y*-axis only if the column is acting as part of a seismic-resisting system in that direction.

(c) When the slenderness ratio for the member web exceeds that given in table 12.8.2 for the appropriate member category, the design axial force generated by gravity loading alone, N^*_g, shall comply with the axial force limitation equation given therein.

Table 12.8.2 – Limit on N^*_g as a function of web slenderness

Category of member (see 12.2.5 or 4.5)	Web slenderness ratio $\dfrac{d_1}{t}\sqrt{\left(\dfrac{f_y}{250}\right)}$	Axial force limitation equation to apply
1	30	8.4.3.3(1) or 8.4.3.3(2) as appropriate, with $N^* = N^*_g$ used therein
2		
3	40	12.8.3.2(1)or 12.8.3.2(2) as appropriate

For category 3 members, the following limits on N^*_g shall apply.

$$\frac{N^*_g}{\phi N_s} \le 1.02 - \left[\frac{\frac{d_1}{t}\sqrt{(f_y/250)}}{99}\right] \quad \text{for webs where } 70 \le \frac{d_1}{t}\sqrt{\left(\frac{f_y}{250}\right)} \le 101 \quad \text{....... Eq. 12.8.3.2(1)}$$

or

$$\frac{N^*_g}{\phi N_s} \le 1.90 - \left[\frac{\frac{d_1}{t}\sqrt{(f_y/250)}}{44.2}\right] \quad \text{for webs where } 40 \le \frac{d_1}{t}\sqrt{\left(\frac{f_y}{250}\right)} \le 70 \quad \text{....... Eq. 12.8.3.2(2)}$$

where

N^*_g = the design axial force generated by gravity loading alone (e.g. dead, live loading).

NOTE – Any additional source of axial force which remains constant on the member throughout an earthquake must be included in N^*_g.

12.8.3.2 Compression axial force limitation waiver

For columns which are secondary elements of seismic-resisting systems (see 12.2.7.1), the limitation of equation 12.8.3.1 may be waived when the design capacity of the member to section 8 is adequate to resist the design actions generated by elastic response of the seismic-resisting system in conjunction with the appropriate design gravity loads.

12.8.3.3 Limit on transverse loading on category 1, 2 and 3 columns

Except as noted in the following paragraph, direct transverse loading between the supports/restraints of category 1, 2 and 3 columns shall be limited to that which generates a design moment less than or equal to 10 % of the design section moment capacity (ϕM_S) of the member about the appropriate principal axis or axes.

If, in a member subject to direct transverse loading, the position and pattern of the yielding regions is dependably unchanged by the application of the transverse load, then this limitation does not apply.

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12.8.4 Concurrent action on columns

For columns (and their foundations) which are part of a two-way seismic-resisting system, concurrent action shall be considered (allowing for the sign of the action when a 3 dimensional analysis is undertaken) in accordance with (a) to (c) as applicable:

(a) *Both seismic-resisting systems are category 1 or category 2*

(i) The seismic-resisting systems shall be designed, using capacity design, for seismic forces applied separately along each principal direction; then

(ii) The capacity design derived design actions from (a)(i) shall be considered acting concurrently on the column; and

(iii) Column design shall be to the requirements of sections 5-8 and 13; and

(iv) The design strength of the column shall be not less than the concurrently acting design actions (S^*).

(b) *One seismic-resisting system is category 2, the other is category 3*

(i) The seismic-resisting systems shall be designed for the seismic forces applied separately along each principal direction, using capacity design derived actions for the category 2 system and capacity design derived actions or design actions, as appropriate, for the category 3 system; and

(ii) The category 3 concurrent design actions resulting from 100 % of the earthquake forces acting on the category 3 system shall be determined; then

(iii) 100 % of the capacity design derived design actions from the category 2 system in conjunction with 30 % the category 3 concurrent design actions from (ii) above shall be considered acting concurrently on the column; and

(iv) Column design shall be to the requirements of sections 5-8 and 13; and

(v) The design strength of the column shall be not less than the concurrently acting design actions (S^*).

(c) *Both seismic-resisting systems are either category 3 or category 4*

(i) The seismic-resisting systems shall be designed for seismic forces applied separately along each principal direction, using capacity design where appropriate; then

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(ii) The column shall be designed for concurrent design actions generated by application of the specified seismic forces (and gravity loads) from the Loadings Standard, with the seismic forces acting on the structure in the direction that produces the most unfavourable effect in that column member; and

(iii) Column design shall be to the requirements of sections 5-8 and 13; and

(iv) The design strength of the column shall be not less than the concurrently acting design actions (S*).

Amd 2 Oct. '07 The category of system for application of the concurrent actions shall be assessed to μ_{act} when this is calculated; otherwise assessed to μ.

12.8.5 Webs of columns within yielding regions

12.8.5.1
The web thickness within the yielding region of a column member shall be not less than that required from 12.8.3.1(c).

12.8.5.2
Load bearing stiffeners shall be provided when a bearing load or shear force acts within $d_1/2$ of a yielding region and the design bearing load or shear force exceeds 0.1 times the design shear capacity (ϕ V_v) of the member specified in 5.11.

12.8.5.3
These stiffeners shall be located within a distance $d_1/2$ on either side of the yielding region and shall be designed in accordance with 5.14 to carry the greater of the design bearing load or the design shear force considered as a bearing load.

12.8.5.4
If the stiffeners are flat plates, their slenderness parameter (λ_s), as defined in 5.2.2 using the stiffener yield stress (f_{ys}), shall be less than the slenderness limit appropriate to the category of member ($\lambda_{e1, 2, 3}$) specified in table 12.5.

12.9 CONNECTIONS AND BUILT-UP MEMBERS

12.9.1 Connection design philosophy and design actions

12.9.1.1 Design philosophy

12.9.1.1.1
Connections shall exhibit dependable strength and ductility, in order to maintain the integrity of the structural system throughout the expected range of seismic-induced deformations determined in accordance with this Standard and the Loadings Standard.

12.9.1.1.2
The influence of the choice of connections on the overall structural response shall be considered (see 4.2 and 9.1.2, 9.1.3).

12.9.1.1.3
The load path and strength heirachy within the connection shall be such as to avoid inelastic demand being concentrated into the connectors or connection components, except for the panel zones of moment-resisting connections as covered by 12.9.5.2(b) and 12.9.5.3.2.

12.9.1.2 Design actions for connectors and connection components

12.9.1.2.1
The design actions for panel zones of moment-resisting connections are given by 12.9.5.2(b).

12.9.1.2.2
Amd 2 Oct. '07 All other connectors and connection components between elements of seismic-resisting members shall be designed to resist the actions specified in (1), (2), (3) or (4) as appropriate.

(1) *Connectors and connection components between elements of primary category 1, 2 or 3 members or between elements of primary and secondary category 1, 2 or 3 members*

(a) *For complete penetration butt welds*
The design action shall be the section design capacity of the element being connected.

(b) *For incomplete penetration butt welds, fillet welds, bolts and pins*

(i) Where capacity design is undertaken, the design actions shall be the capacity design derived design actions acting on the elements being connected, except as given by (4) below.

(ii) Where capacity design is not undertaken, the design actions shall be calculated as:

$\phi_{oms} R_u$ between elements that form the principal load-carrying path through the connection, except as given by (4) below; or

$0.9\, R_u$ between other elements of the connection, except as given by (4) below

where

R_u = nominal section capacity as appropriate to the type of design action acting on the elements being connected

ϕ_{oms} = overstrength factor for the member from tables 12.2.8.

(c) The design capacity of the connectors or connection components shall be used to resist the design actions specified in (a) and (b) above.

(2) *Connectors and connection components between elements of primary category 4 members*

(a) The design actions shall be as determined from structural analysis in accordance with section 4 and the appropriate provisions of section 12, using $S_p = 1.0$.

Amd 2 Oct. '07 (b) The design capacity of the connectors or connection components shall be used to resist the design actions.

(3) *Connectors and connection components between elements of secondary members of any member category*

(a) The design actions shall be the capacity design derived design actions acting on the elements being connected, except as given by (4) below.

(b) The design capacity of the connectors or connection components shall be used to resist the design actions.

(4) *Upper limit design actions on connections in seismic-resisting structural systems*

Connections need not be designed to transmit actions greater than (a) or (b) below, and (c) where applicable:

(a) Those generated by nominally ductile seismic response in conjunction with the appropriate design gravity loads for category 1 seismic-resisting systems or for category 2 seismic-resisting systems with $\mu_{act} \geq 1.8$. The value of S_p used shall be 0.7.

(b) Those generated by elastic seismic response in conjunction with the appropriate design gravity loads for category 3 seismic-resisting systems or for category 2 seismic-resisting systems with $\mu_{act} < 1.8$. The value of S_p used shall be 0.7 for category 2 systems and 0.9 for category 3 seismic-resisting systems.

NOTE – Category 4 seismic-resisting system design actions given by 12.9.1.2.2(2) are upper limit actions.

(c) For connections in category 1, 2 or 3 seismic-resisting systems incorporating incomplete penetration butt welds, fillet welds, bolts and pins, when the seismic-resisting system requires capacity design in accordance with 12.2.6 and the capacity design derived design actions on any connection is limited by (a) or (b) above, the connection shall be designed to resist 1.25 times the actions generated by the design capacity of the primary member or members to which it is attached.

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12.9.2 Minimum design actions on connections subject to earthquake loads or effects

Clause 12.9.2 applies to all connections in seismic-resisting systems or associated structural systems except for lacing connections and connections to sag-rods, purlins and girts.

The minimum design actions required for all connections shall be given by the lesser of 12.9.1.2.2(4) or the relevant subclause below.

In addition, members shall be connected in such a way to meet the robustness requirements of AS/NZS 1170.0 clause 6.2.3, with the design action applied along the longitudinal axis of the supported member.

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12.9.2.1 General case

12.9.2.1.1

The minimum design actions required for all connections, except for splices in columns as specified in 12.9.2.2 or 12.9.2.3, shall be as follows:

(a) 50 % of the design section capacity of the member in compression or tension as appropriate ($0.5\ \phi\ N_s$ or $0.5\ \phi\ N_t$);

(b) 30 % of the design section moment capacity of the member ($0.3\ \phi\ M_s$);

(c) 15 % of the design shear capacity of the member ($0.15\ \phi\ V_v$).

12.9.2.1.2

Only the actions relevant to the design of a particular connection need be considered.

12.9.2.2 Splices in columns : general

12.9.2.2.1 *Splices in columns of category 1 and 2 seismic-resisting systems*
For splices in columns of category 1 and 2 seismic-resisting systems, the minimum design actions shall be applied as 2 design cases, namely:

(1) The lesser of:

(i) 50 % of the design section capacity of the smaller member in tension or compression as appropriate ($0.5\ \phi N_s$ or $0.5\ \phi N_t$); or

(ii) The design action, in tension or compression as appropriate, from the combination of elastic or nominally ductile seismic response of the structure as appropriate (refer to 12.9.1.2.2(4)) in conjunction with the appropriate design gravity loads.

(2) The combination of:

(i) 50 % of the design section moment capacity, reduced by N_g^*, of the smaller member ($0.5 \phi M_r$); and

Amd 1 June '01 (ii) 25 % of the design shear capacity of the smaller member ($0.25\ \phi\ V_v$).

12.9.2.2.2 *Splices in columns of category 3 and 4 seismic-resisting systems*
The minimum design actions applicable to these splices shall consist of 2 design cases, namely:

(1) As for 12.9.2.2.1 (1)

(2) The combination of:

(i) 30 % of the design section moment capacity, reduced by N_g^*, of the smaller member ($0.3 \phi M_r$); and

Amd 1 June '01 (ii) 15 % of the design shear capacity of the smaller member ($0.15\ \phi\ V_v$).

where

N_g^* = design axial force generated by gravity loading alone (e.g. dead, live loading).

12.9.2.3 Splices in columns subject to axial compression

12.9.2.3.1 *Splices prepared for full contact*
For ends prepared for full contact in accordance with 14.4.4.2, compressive actions may be carried by bearing on contact surfaces. When members are prepared for contact to bear at splices, there shall be sufficient fasteners to hold all parts securely in place. These fasteners shall be sufficient to transmit:

(a) For columns forming part of a seismic-resisting system:

The design actions from 12.9.2.2 as appropriate, except that compression actions generated by axial force or moment may be assumed to be transmitted directly across the splice by bearing action.

(b) For columns forming part of an associated structural system, the greater of:

(i) The shear force from 12.9.2.1.1 (c);

(ii) A force of 0.15 times the smaller member design capacity in axial compression ($0.15\ \phi\ N_c$).

12.9.2.3.2 *Splices not prepared for full contact*
When column splice members are not prepared for full contact, the splice material and its fasteners shall be arranged to hold all parts in line and shall be designed to transmit:

(a) For columns forming part of a seismic-resisting system, the actions from 12.9.2.2 as appropriate;

(b) For columns forming part of an associated structural system, the greater of:

(i) The shear force from 12.9.2.1.1 (c) in conjunction with the design axial compression force (N^*);

(ii) A force of 0.3 times the smaller member design capacity in axial compression ($0.3\,\phi N_c$).

12.9.3 Welds

12.9.3.1 General
Welds shall be designed in accordance with 9.7.

The design actions on welds shall be determined from 12.9.1.

12.9.3.2 Weld failure
Brittle fracture or lamellar tearing of welded steel members either prior to, or as a result of seismic loading shall be suppressed through selection of steels and welding consumables in accordance with 2.6 and use of appropriate connection details.

12.9.3.3 Classification of welds
The appropriate choice of weld category shall be made from 9.7.1.4.

12.9.3.4 Fillet welds
Fillet welds may be used in connections subject to inelastic seismic demand, subject to the following:

(a) Minimize the amount of intermittent welding to minimize potential stress raisers;

(b) A fillet weld shall have a leg length not less than half the thicknesses of the thinner plate in a joint;

(c) Two-sided fillet welds shall be used whenever possible.

12.9.4 Bolts

12.9.4.1 General
Bolts shall be designed in accordance with 9.3, except as modified by 12.9.4.2, 12.9.4.3 and 12.9.4.4.

The design actions on bolts shall be determined from 12.9.1.

12.9.4.2 Reduction in cross-sectional area

12.9.4.2.1 *General*
Any reduction in gross cross-sectional area made to category 1 or 2 or 3 members at a connection or in any location such as may lead to yielding under the applied actions shall be limited to that permitted by equation 12.9.4.1, except as specified in 12.9.4.2.2.

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$$\left(\frac{A_n}{A_g}\right)_{min.} = \left(\frac{\phi_{oms} f_y}{0.85 f_u}\right) \leq 1.0 \qquad \text{(Eq. 12.9.4.1)}$$

where

A_g = the gross cross-sectional area within the zone of intended yielding
A_n = the area of cross section at the connection or critical location
ϕ_{oms} = the overstrength factor from 12.2.8
f_u = the tensile strength of the member material
f_y = the yield stress of the member material.

12.9.4.2.2 *Connections to category 1 and 2 members of grade 250 or 300 steel*
The ratio of net to gross cross-sectional area shall comply with the following:

(a) For category 1 members $A_n \geq 0.95 A_g$

(b) For category 2 members $A_n \geq 0.85 A_g$

12.9.4.3 Maximum design bearing capacity
A ply subject to a design bearing force (V_b^*) generated by a category 1, 2 or 3 member shall satisfy:

$$V_b^* \leq C_1 \phi V_b$$

where

ϕ = the strength reduction factor (see table 3.3)

V_b = nominal bearing capacity of a ply calculated from equation 9.3.2.4 (1) or 9.3.2.4 (2) as appropriate

C_1 = factor given in table 12.9.4.3.

Table 12.9.4.3 – Reduction factor C_1 for determination of design bearing strength of a ply

Member category	Snug tight mode	Tension bearing mode
1	0.6	0.8
2	0.8	0.8
3, 4	1.0	1.0

12.9.4.4 Minimum edge distance requirement
The minimum edge distance for bolts in connections subject to earthquake loads or effects shall comply with 9.6.2.1, except that, for connections to category 1, 2 or 3 members, the minimum edge distance parallel to the direction of applied force (a_e) shall be not less than 2 times the bolt diameter (d_f) and the minimum edge distance in any other direction shall be not less than 1.5 times the bolt diameter.

12.9.4.5 Holes for bolts and rivets

12.9.4.5.1 *General*
The requirements of 14.3.5 apply except as noted in 12.9.4.5.2 and 12.9.4.5.3.

12.9.4.5.2 *Bolt holes for snug tight bolts*
For category 1 members connected by snug tight mode bolts, bolt holes shall be 0.5 mm oversize only.

12.9.4.5.3 *Punching*
All punched bolt holes in designated yielding regions of category 1, 2 and 3 members shall be punched 3 mm undersize and reamed to final size.

12.9.5 Moment-resisting beam to column connections

12.9.5.1 Scope
Clause 12.9.5 covers the design requirements for elements of welded and bolted end plate moment-resisting connections for rigid structural systems used in the following applications:

(a) Between members of a seismic-resisting system; or

(b) Where appropriate, between members of an associated structural system; or

(c) Where directed by 4.5.7.2.2, between members of a structural system in which the actions from an elastic analysis are modified by redistribution.

12.9.5.2 Design actions from beams

(a) *Axial forces generated by beam flanges*

These shall be determined from 4.5.7.2.2 or 12.9.1.2, as appropriate.

(b) *Design shear force for joint panel zone*

(i) The design shear force (V_p^*) for the panel zone of a moment-resisting connection shall be based on the unbalanced moment generated across the connection given by equation 12.9.5.2 (1).

$$V_p^* = \frac{M_L}{(d_b - t_{fb})_L} + \frac{M_r}{(d_b - t_{fb})_R} - V_{COL} - V_G^{(4)} \quad \text{............ (Eq. 12.9.5.2 (1))}$$

where

(1) The subscripts $_L$ and $_R$ denote the left and right hand beams at the connection.

(2) V_{COL} is the lesser of the column shear above or below the joint generated by the design moments acting on the columns.

(3) d_b, t_{fb} are the depth and flange thicknesses, respectively, of the incoming beams.

(4) V_G is applicable to associated structural systems only (i.e. $V_G = 0$ for connections in seismic-resisting systems). V_G makes allowance for the reduction in unbalanced shear force on the panel zone due to gravity action in connection supporting beams from 2 opposing directions, and shall be determined by a rational design procedure. ➤

(ii) For category 1, 2 and 3 members in rigid structural systems:

$M_L = M_R = C_2 M_s$; except that

$(M_L + M_R) \leq$ the lesser of (1) and (2) below;

(1) 0.9 times the unbalanced moment generated across the connection by nominally ductile seismic response in conjunction with the appropriate design gravity loads; or

(2) $C_2 \quad \Sigma M_{r\ COL}$

where

C_2 = 1.15 for category 1 primary members framing into the connection

C_2 = 1.1 for category 2 primary members framing into the connection

C_2 = 1.0 for category 3 primary members framing into the connection

$M_{r\ COL}$ = nominal section moment capacity, reduced by axial force, in accordance with 8.3.2 or 8.3.3 as appropriate.

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Where required by 12.10.2.3, for category 1, 2 and 3 MRFs of rigid construction (see 4.2.2.1), incorporating welded beam to column connections, the design actions on the panel zone shall include the strength increase due to the slab through multiplying the term C_2 by the factor :

$(1.0 + 0.54\ t_{ef}/d_b)$

where

d_b = the steel beam depth (if the beams are a different size, use the larger value)

t_{ef} = the thickness of the concrete rib in direct contact with the column

= t_o when the concrete rib is continuous along the beam (e.g. when the decking is parallel to the beam), otherwise

= t (from 13.1.2.5.1).

(iii) For connections in moment-resisting systems in which equation 12.9.5.2(1) is called up by 4.5.7.2.2 and therefore (ii) above does not apply, M_L and M_R are determined from 4.5.7.2.1(b).

12.9.5.3 Welded moment-resisting beam to column connections

12.9.5.3.1 *Web stiffening*
Pairs of web stiffeners shall be provided on the column adjacent to the beam flanges where the concentrated force delivered by the beam flanges will otherwise produce web crippling opposite the compression flange or high tensile stresses in the connection to the tension flange. Web stiffeners are required:

(a) Opposite the compression flange of the beam when:

$$t_{wc} < \frac{\phi_{oms} A_{fb}}{t_{fb} + 5t_{fc} + 2t_{ep} + 2t_{wf}} \left(\frac{f_{yb}}{f_{yc}} \right) \quad \text{(Eq. 12.9.5.3 (1))}$$

(b) Opposite the tension flange of the beam when:

$$t_{fc} < 0.54 \sqrt{(A_{fb} f_{yb} / f_{yc})} \quad \text{(Eq. 12.9.5.3 (2))}$$

where

t_{wc}, t_{fc} = thickness of column web, flange
ϕ_{oms} = overstrength factor for category of beam (tables 12.2.8)
t_{ep} = thickness of end plate on beam, if any
t_{wf} = leg length of fillet weld, if present, between beam flange and column flange or end plate
t_{fb} = thickness of incoming beam flange
A_{fb} = flange area of beam = $t_{fb} \cdot b_b$
f_{yb}, f_{yc} = yield stress of beam, column.

(c) The area of web stiffeners, A_s, with yield stress, f_{ys}, shall be such that:

(i) When stiffeners are required opposite the compression flange from (a):

$$A_s \geq \phi_{oms} A_{fb} \left(\frac{f_{yb}}{f_{ys}} \right) - t_{wc}(t_{fb} + 5t_{fc} + 2t_{ep} + 2t_{wf}) \left(\frac{f_{yc}}{f_{ys}} \right) \quad \text{(Eq. 12.9.5.3 (3))}$$

(ii) When stiffeners are required opposite the tension flange from (b):

$$A_s \geq (A_{fb} - t_{wc} \, t_{fb}) \left(\frac{f_{yb}}{f_{ys}} \right) \quad \text{(Eq. 12.9.5.3(4))}$$

NOTES ASSOCIATED WITH EQUATIONS 12.9.5.3 (1) TO 12.9.5.3 (4):

(1) The ratio of outstand width/thickness for an individual stiffener in compression shall not exceed $8 / \sqrt{(f_{ys} / 250)}$ for connections in which the incoming beams are category 1 or 2 members or $15 / \sqrt{(f_{ys} / 250)}$ for connections in which the incoming beams are category 3 or 4 members.

(2) In equations 12.9.5.3 (1) to 12.9.5.3 (4), the overstrength moment action is assumed to be generated in the beam. If, from 12.9.1.2 or 4.5.7.2, this is not the case, then:

(i) When ascertaining the need for stiffeners, substitute the term N^*_{fb} / f_{yb} for $\phi_{oms} A_{fb}$ in equation 12.9.5.3 (1) or for A_{fb} in equation 12.9.5.3 (2)

(ii) When determining the minimum area of stiffeners required, use $\phi_{oms} = 1.0$ in equation 12.9.5.3 (3) or substitute N^*_{fb} / f_{yb} for A_{fb} in equation 12.9.5.3 (4)

In (i) and (ii), N^*_{fb} is the design axial force in the beam tension or compression flange, as appropriate, generated by the design moment (M^*).

(3) The design action on the welds between each pair of tension or compression stiffeners and the column web or doubler plate(s), as appropriate, shall be the least of the incoming design axial forces from the beam flanges or the design section capacity of the stiffener ($\phi A_s f_{ys}$). The design capacity of these welds shall be used to resist the design action.

12.9.5.3.2 *Shear capacity of connection panel zone*
The nominal shear capacity of a panel zone (V_c) shall be determined from equation 12.9.5.3 (5). The design shear force (V_p^*) shall not exceed the design shear capacity ($\phi\, V_c$).

$$V_c = 0.6\, f_{yp}^* d_c\, (t_{wc} + t_p)\, \eta \left[1 + \frac{3 b_c\, t_{fc}^2}{d_b\, d_c\, (t_{wc} + t_p)}\right] \quad \text{.................................. (Eq. 12.9.5.3 (5))}$$

where

ϕ = as specified in table 3.3

η = $\sqrt{(1.15 - (N^* / \phi N_s)^2)} \le 1.0$

t_p = total thickness of doubler plate(s)

$N^* / \phi\, N_s$ = ratio of column design compression force to design section capacity

f_{yp}^* = design yield stress for joint panel zone

$$= \frac{t_{wc}\, f_{yc} + t_p\, f_{yp}}{t_{wc} + t_p}$$

f_{yp} = yield stress of doubler plate(s)

d_c, b_c = depth, breadth of column.

Doubler plate(s) must be fitted immediately adjacent to the column web for equation 12.9.5.3(5) to be valid.

Doubler plate(s), if required, should be detailed in accordance with C12.9.5.3.2.

12.9.5.3.3 *Slenderness limits on panel zone elements*
The web and any doubler plate(s) designed to 12.9.5.3.2 must satisfy the slenderness limits of equation 12.9.5.3(6).

$$\left(\frac{d_c - 2t_{fc}}{t_{wc} + k_1\, t_p}\right)\left(\sqrt{\frac{f_{yp}^*}{250}}\right) \le C_3 \quad \text{.. (Eq. 12.9.5.3 (6))}$$

where

C_3 = 125 if both tension and compression stiffeners (see 12.9.5.3.1(c)) are present

C_3 = 82 otherwise

k_1 = 0.25 if doubler plate(s) are not plug-welded to web

k_1 = 1.00 if doubler plate(s) are plug welded, with a minimum of 4 plug welds in accordance with 9.7.4.2 placed symmetrically about the centroid of the panel zone, in the middle third of the panel zone area

t_{wc}, d_c, t_{fc}, t_p = as defined in 1.4

f_{yp}^* = as defined in 12.9.5.3.2.

12.9.5.3.4 *Welds between beam and column*
The welds shall be adequate to ensure the appropriate performance criteria specified in 12.2.5 are met and shall comply with the requirements of 12.9.3.

12.9.5.4 Bolted moment-resisting beam to column connections

12.9.5.4.1
Bolted moment-resisting connections shall be designed so as to ensure that the appropriate performance criterion specified in 12.2.5 is achieved and shall comply with the requirements of 12.9.4.

12.9.5.4.2
The design philosophy for bolted end-plate moment-resisting connections shall be to achieve a balanced design joint capacity, with no significant ductility demand being forced into any one component of the connection (except for the panel zone) and with all potential brittle failure modes suppressed.

12.9.5.4.3
The design procedure used shall incorporate, as required, an allowance for increase in the bolt tensile loads due to prying action on the bolts.

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12.9.6 Design of splices

12.9.6.1 Column splices

(a) Splices in columns should be located within the middle third of the storey height and shall be located clear of any yielding regions. Splices in brace members should be located as near to the quarter points of the brace as practicable and shall be located clear of any yielding regions.

(b) Column splices should be designed as full contact splices in accordance with 14.4.4.2. If the column splice is not designed as a full contact splice the maximum gap between any point of abutting column lengths in a splice shall be 3 mm for category 1, 2 or 3 columns.

(c) Only property class 8.8 or higher structural bolts in the tension bearing mode shall be used in site-bolted column splices in columns forming part of the seismic-resisting system.

(d) The minimum edge distances to all bolts in bolted splices shall comply with 12.9.4.4.

(e) Quenched and tempered steel plate designated for general structural use may be used as splice cover plates in fully bolted column splices in accordance with 9.3.2.4.2.

(f) Design actions for column splices are given in 12.9.1.2 and 12.9.2.2.

12.9.6.2 Beam splices

(a) Splices should be located as close as is practical to the point of contraflexure as determined from static or modal analysis and shall be located remote from any yielding regions.

(b) The bolts used in beam splices in any moment-resisting frame which forms part of a seismic-resisting system and which exceeds 2 storeys in height must be property class 8.8 or higher structural bolts used in the tension bearing mode, unless a rational design of the structure taking account of the effects of additional deflections arising from bolt slip through the use of property class 8.8 or higher snug tight bolts in beam splices is undertaken.

(c) The edge distance requirements for bolts in beam splices shall be in accordance with 12.9.4.4.

(d) Design actions for beam splices are given from 12.9.1.2.

12.9.7 Design of gusset plates

12.9.7.1 General
Design of gusset plates shall be in accordance with a rational design method and the relevant provisions of this section and section 9.

12.9.7.2 Local effects

Design of gusset plates for local effects shall take account of:

(a) Minor axis bending; check for combined actions in accordance with 8.3.3.1 and 8.4.2.2.1;

(b) Buckling in compression; check member capacity to 6.3 using an effective length (L_e) of 0.7 times the clear length;

(c) Bearing; determine the bearing capacity from 12.9.4.3;

(d) Plate tearing; comply with the edge distance requirements of 12.9.4.4.

12.9.7.3 Accommodation of member inelastic rotation

12.9.7.3.1

Gusset plates connecting category 1 or 2 members should be detailed such that there is a clear length of gusset plate, beyond the end of the member being connected, equal in length to between 2 and 3 times the thickness of the gusset plate.

12.9.7.3.2

Gusset plates not detailed to 12.9.7.3.1 shall be designed to accommodate, at the connection, inelastic out-of-plane rotation in the member being connected.

12.9.7.4 Allowance for beam-column joint opening

Where a gusset plate is welded or bolted into both a beam and a column member, the connections between the gusset plate and the supporting members shall be designed and detailed to accommodate opening of the joint angle under inelastic seismic action. Amd 2 Oct. '07

12.9.8 Design of built-up members

12.9.8.1 General

Built-up members shall comply with 6.4, 6.5 or 7.4 as appropriate and also with the appropriate requirements of 12.9.8.2.

12.9.8.2 Specific design considerations

(a) Individual members shall comply with the section property requirements of 12.5 for the category of built-up member.

(b) For category 1 and 2 members, each intermediate welded connection to a component in a built-up member shall be designed to resist the combined shear and bending actions generated by a force equal to half the design tensile section capacity (ϕN_t) generated by the other component acting about its centroid.

(c) The effective length factor k_e may be approximated as 0.5 for in-plane buckling of built-up members and 1.0 for out-of-plane buckling.

(d) Intermediate bolted connections shall use property class 8.8 or higher structural bolts in the tension bearing mode only and such connections shall be avoided at yielding regions (generally yielding regions are located in the middle third of the member length).

12.10 DESIGN OF MOMENT-RESISTING FRAMED SEISMIC-RESISTING SYSTEMS

12.10.1 General

These provisions apply to the design of category 1 or 2 moment-resisting framed (MRF) seismic-resisting systems and to the design of category 3 and 4 moment-resisting framed seismic-resisting systems where specified by the relevant clause.

12.10.2 Design procedure

A rational design procedure, incorporating a capacity design procedure where required, shall be used in the design of moment-resisting frames. The capacity design procedure for moment-resisting steel frames must meet the following requirements:

12.10.2.1

The beams are the primary members, the columns and connections are the secondary members and elements.

12.10.2.2

Unidirectional beam hinging shall be suppressed.

12.10.2.3

The columns and connections shall be designed to resist the overstrength actions from the beams. These are developed at the supported end of the beams.

For beams in MRFs of rigid construction (see 4.2.2.1) in which the slab is isolated from the column, the overstrength actions from each beam developed at the column face for design of the column member for combined bending and axial force shall be given as $\phi_{oms}M_s$, where ϕ_{oms} is given by Table 12.2.8.(1) and M_s is the nominal section moment capacity of the beam from 5.2.

For category 1, 2 and 3 MRFs of rigid construction in which the slab is not isolated from the column, the influence of the slab shall be included in the overstrength actions for design of the column member for combined bending and axial force in accordance with 12.10.2.4.

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Where required by 12.10.2.3, for category 1, 2 and 3 for MRFs of rigid construction (see 4.2.2.1), or by 12.11.7.2 for D-braced EBFs with long active links, the overstrength moment from the composite beams at the column face, M^o, shall include the strength increase due to the slab as follows.

$$M^o = \Sigma M^o_i + N_{slab}(d_b/2+t_o-t_{ef}/2)$$

where

ΣM^o_i is the sum of the overstrength moments i considering N_{slab} as:

$$\Sigma M^o_i = \min\{1.18 \times (1 - N_{slab}/\Sigma(A_g f_y)_i) \times \Sigma M^o_{b,i};\ \Sigma M^o_{b,i}\}$$

and

N_{slab} is the axial force generated by the slab, given by:

$$N_{slab} = \min\{1.3 t_{ef} b_{sef}(f'_c + f'_{cos});\ \Sigma(A_g F_y)_i\}$$

where:

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A_g = area of steel beam framing into the column

b_{sef} = the width of compression action against the supporting column
= b_{fc} when the column is an I section and the beam frames into the column flange
= d_c when the column is an I section and the beam frames into the column web
= $0.9d_c$ when the column is a concrete filled circular steel tube
= the width of column perpendicular to the incoming beam when the column is a concrete filled square or rectangular hollow section

d_b = the steel beam depth (if the beams are different size, use the larger value)

f'_{cos} = the long term increase in concrete stress in the slab above the nominal 28 day strength, taken as 10 MPa

f_y = the yield strength of the beam flange

$M^o_{b,i}$ = the overstrength moment for beam i not considering any effect of axial force
= $\phi_{oms}M_s$;

ϕ_{oms} = overstrength factor from Table 12.2.8(1) for MRFs and Table 12.2.8(2) for D-braced EBFs with long links, as directed by 12.11.7.2

M_s = nominal beam section moment capacity to 5.2

t_o = overall slab thickness

t_{ef} = the thickness of the concrete rib in direct contact with the column
= t_o when the rib is continuous along the beam (e.g. when the decking is parallel to the beam)
= t (from 13.1.2.5.1), otherwise

$\Sigma()$ = indicates the summation of all beams having a moment connection at the joint in the direction of loading considered (either 1 or 2);

For calculating the seismic induced beam shears and the capacity design derived beam bending moment at the column centreline for each end of each beam, the column face overstrength moments at a joint with two beams framing into the column in the direction of loading considered shall be distributed between the beams on each side of the joint in proportion to their section moment capacities, M_s. For one beam, the column face overstrength moment shall be taken as M^o.

As an alternative to calculating M^o, for I section beams framing into the column flange of I section columns, the overstrength actions from each beam including slab participation shall be given as $\phi_{omss}M_s$, where:

$$\phi_{omss} = \phi_{oms}\,(1.0 + 1.08\; t_{ef}/d_b)$$

for $t_{ef}/d_b \le 0.4$

12.10.2.5
In the design of the beams to resist shear, the seismic shear component shall be determined from the overstrength moments.

12.10.2.6
When determining the design actions on the columns, it shall be assumed that the structure is displaced laterally so that the yielding regions form at the ends of all beams to give a yielding mechanism. The columns shall be designed for the overstrength actions from the beams from 12.10.2.3 or 12.10.2.4, as appropriate, and incorporating the dynamic magnification factor from 12.10.2.7, subject to these actions not exceeding the maximum actions required from 12.3.3.4, in conjunction with the actions from the permanent and combination imposed loads.

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12.10.2.7
The dynamic magnification factor required is 1.2 for shear actions in columns, 1.0 for all other actions.

12.10.2.8
Design for concurrency in columns shall be in accordance with 12.8.4.

12.10.2.9
Design of connections between the beam and column shall be to 12.9.5.

12.10.2.10
Amd 2 Oct. '07 Splices in columns shall be located clear of any potential yielding regions, in accordance with 12.9.6.1.

12.10.3 Shear strength within beam yielding regions

12.10.3.1
When a capacity design procedure is not used, at yielding regions in category 1 or 2 beams forming part of a seismic-resisting system, when designing for load combinations including earthquake loads the nominal web shear capacity of the beams shall be taken as 80 % of that calculated from 5.11.4.1 and the interaction of shear and bending moment shall satisfy:

$$M^* \leq \phi M_{sv}$$

where

$$M_{sv} = M_s \quad \text{for } V^* \leq 0.6\,\phi\,V_w$$

$$= M_s\left[1.38 - \frac{V^*}{1.6V_w}\right] \quad \text{for } 0.6\phi V_w \leq V^* \leq 0.8\phi V_w$$

where

M_s = the nominal section moment capacity for the complete section (see 5.2)

V_w = the nominal shear yield capacity of a category 1 or 2 member web (see 5.11.4.1).

12.10.3.2
When a capacity design procedure is used to derive the beam shear forces, the member shall satisfy:

$$M^* \leq \phi M_{sv}\text{; and}$$

$$V^* \leq 0.8\,\phi\,V_w$$

where

$$M_{sv} = M_s \quad \text{for } V^* \leq 0.8\,\phi V_w.$$

12.10.4 Calculation of lateral deflection of the seismic-resisting system

(a) Moment-resisting frame lateral deflection calculations shall consider bending and shear contributions from the clear column and beam spans plus joint rotation from out-of-balance shear actions;

(b) The effect of panel zone distortion on the lateral deflection of the structure shall be considered;

(c) The contribution to total lateral deflection due to column axial stresses shall be included if it amounts to more than 10 % of the total frame lateral deflection.

12.10.5 Yielding region formation and column strength

Any design procedure used for category 1, 2 or 3 moment-resisting frames must consider the following:

(a) In the derivation of vertical loading induced in the columns the most adverse likely location of yielding regions in the beams needs to be considered. Where yielding regions occur at the beam ends the distance between them for calculation of this loading may be taken as the length between column faces.

(b) The interaction of seismic-induced member actions with actions resulting from gravity loading and the effect that this interaction may have on member yielding region formation, especially on the ductility demand and position of plastic hinges in the beams.

(c) Concurrent action in columns forming part of a two-way seismic-resisting system shall be considered in accordance with 12.8.4.

(d) The coincident formation of yielding regions in beams framing into columns at an appropriate number of levels.

12.10.6 Concrete encasement of steel frames

The effect of concrete encasement and floor slabs on frame stiffness and earthquake response shall be considered in accordance with Appendix N. Any effect of the encasement on the location or behaviour of yielding regions shall be considered.

12.11 DESIGN OF ECCENTRICALLY BRACED FRAMED SEISMIC-RESISTING SYSTEMS

12.11.1 General

12.11.1.1

An eccentrically braced frame (EBF) seismic-resisting system is a braced frame as defined in 1.3.

Category 1, 2 and 3 EBFs shall be designed and detailed in accordance with a rational capacity design procedure in compliance with 12.11.7 and the provisions of 12.11.

Amd 2 Oct. '07 Category 4 EBFs shall be designed and detailed in accordance with a rational design procedure and the relevant provisions of 12.11.

12.11.1.2

Eccentrically braced frame systems in which the active links are intended to dissipate inelastic energy principally in a shear mode shall be designed and detailed in accordance with a rational procedure which incorporates the requirements of 12.11.

For such systems, $e \leq 1.6\, M_s / V_w$

where

e = clear length of active link
M_s = nominal section moment capacity (see 5.2)
V_w = nominal shear capacity (see 5.11.4.1).

12.11.1.3
Eccentrically braced frame systems in which the active links are intended to dissipate inelastic energy principally in a flexural mode ($e \geq 3\ M_s / V_w$) or in a combined shear and flexural mode ($1.6\ M_s / V_w < e < 3\ M_s / V_w$) shall be designed and detailed in accordance with a rational procedure and the appropriate performance requirements of 12.11.

12.11.1.4
Shear or moment redistribution may be undertaken in accordance with 4.5.6.

12.11.2 Exceptions

An eccentrically braced framed system not in accordance with any or all of the provisions of 12.11 may be designed on the basis of a special study.

12.11.3 Design requirements for EBF frames and components

12.11.3.1
EBF lateral deflection calculations shall include bending and shear contributions from the active link regions plus distortion of the joint panel zone, where applicable. The contribution to total lateral deflection due to column axial stresses shall be included if it amounts to more than 10 % of the total EBF lateral deflection.

The effect of any concrete encasement on the frame stiffness and earthquake resistance of the EBF shall be considered in accordance with Appendix N.

12.11.3.2
The relationship between the structure category and the category of the members comprising an EBF shall be as given in table 12.2.6, and applied as follows:

(1) The active link is the primary member

(2) The collector beam is also a primary member for compliance with 12.4, 12.5 and 12.6

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(3) The column and the brace are secondary members.

12.11.3.3

12.11.3.3.1
The geometry of the inelastically deformed EBF bay shall be such that the rotation angle between a beam and an active link, γ_p, for active links not attached to columns, shall not exceed the following:

γ_p	$= \pm 0.08$ radians	for $e \leq 1.6\ M_{sp}/V_w$
γ_p	$= \pm 0.03$ radians	for $e \geq 3\ M_{sp}/V_w$
γ_p	is determined by interpolation	for $1.6\ M_{sp} / V_w < e < 3M_{sp} / V_w$

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12.11.3.3.2
For active links attached to the column flanges, $e \leq 1.6\ M_{sp} / V_w$ is required and the limit on γ_p shall be $\pm$ 0.08 radians.

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12.11.3.3.3
For active links attached to the column webs, $e \leq 1.6\ M_{sp}/V_w$ is required and the limit on γ_p shall be $\pm$ 0.03 radians.

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12.11.3.4
The web of the active link shall be single thickness without doubler plate reinforcement.

12.11.3.5
The nominal capacity of the active link shall be determined as follows:

(a) The nominal shear capacity of the active link (V_{val}) shall be:

$$V_{val} = V_w$$

where V_w is given by 5.11.4.1.

(b) The nominal moment capacity of the active link (M_{sal}) shall be taken as either:

$$M_{sal} = 0.75\ M_s$$

where M_s is the nominal section moment capacity given by 5.2.1; or

M_{sal} shall be taken as the nominal moment capacity calculated for the flanges alone (refer C5.12, Part 2 of this Standard).

(c) The nominal axial compression capacity of the active link (N_{sal}) shall be taken as the section capacity, from equation 6.2.1, calculated for the member flanges only;

(d) The interaction of moment and shear need not be considered;

(e) The interaction of moment and axial actions shall be in accordance with 8.3.2.2, using the values of M_{sal} and N_{sal} from (b) and (c) above.

12.11.3.6
The size of active link shall be chosen on the basis that the design shear capacity of the active link ($\phi\ V_{val}$), as given by 12.11.3.5(a), equals or exceeds the design shear force (V^*).

12.11.3.7
The following requirements shall apply:

(a) No part of the brace to beam connection shall extend into the web area of an active link;

(b) The minimum clear length of active link, e_{min}, shall be not less than the beam depth, d_b.

12.11.3.8
Brace to beam connections shall develop the capacity design derived design actions of the brace and transfer these actions into the active link/beam web.

12.11.3.9
The design compression force on each brace shall be determined by a rational capacity design procedure. Axial forces from the appropriate design gravity loads shall be combined with the seismic-induced brace axial forces in design of the braces.

12.11.3.10
Columns shall be designed for the capacity design derived design actions generated by the active links plus the appropriate design gravity loads.

12.11.3.11
The first storey of an EBF bay may be concentrically braced if this storey has a design shear capacity greater than the overstrength shear capacity of the storey frames above the first storey.

12.11.3.12
Axial forces in beams of EBF frames due to braces and due to transfer of seismic force to the frames shall be included in the frame and connection calculations.

12.11.3.13
The beam connection to columns may be designed as pins in the plane of the beam web if the active link is not adjacent to the column, but such beams shall have full section restraint at the connection in accordance with 5.4.2.1 and 5.4.3.

12.11.3.14
Concurrent action in EBF columns forming part of a two-way seismic-resisting system shall be considered in accordance with 12.8.4.

12.11.3.15
The design of the columns shall take into account the actions generated by coincident formation of yielding regions in active links at an appropriate number of levels.

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12.11.4 Active link web stiffening requirements for EBFs

12.11.4.1
Active links shall have full depth web stiffeners on both sides of the beam web at the brace end of the link beam. The stiffeners shall have a combined width not less than ($b_b - 2\ t_{wb}$) and each shall have a thickness not less than 0.75 t_{wb} (b_b = beam flange width, t_{wb} = beam web thickness.)

12.11.4.2
Active links shall also have intermediate full depth web stiffeners, spaced at intervals not exceeding 38 $t_{wb} - d_b/5$ for a rotation angle of 8 % (0.09 radians) and 56 $t_{wb} - d_b/5$ for a rotation angle of 3 % (0.03 radians) or less. Interpolation may be used for values between 3 % and 8 %.

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12.11.4.3
The thickness, t_s, of these intermediate full depth web stiffeners shall be not less than t_{wb} for a stiffener on one side of the web or 0.75 t_{wb} for stiffeners on both sides of the web, and the stiffener width shall be not less than $(b_b/2) - t_{wb}$.

12.11.4.4
For active link members 610 mm in depth and greater, intermediate full depth web stiffeners are required on both sides of the web. Such web stiffeners are only required on one side of the web for active link members less than 610 mm in depth.

12.11.4.5
Fillet welds connecting the stiffener to the active link member web shall develop a stiffener force of $\phi A_s\ f_{ys}$. Fillet welds connecting the stiffener to the flanges shall develop a stiffener force of $\phi A_s\ f_{ys} / 4$, where $A_s = b.t$ of stiffener, and b = width of stiffener plate. The design capacity of the welds, from 9.7.3.10, shall be used to resist this force.

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12.11.5 Connections between an active link and a column for category 1, 2 and 3 EBFs

Where the active link is adjacent to the column and expected to yield in shear and/or flexure, the welds between the active link flanges and the column flange shall comply with 12.9.1.2.2(1) and the web connection shall be welded to develop the overstrength shear capacity of the member web. Where the active link member is connected to the column web, the flanges shall have complete penetration butt welds to the connection plates and the web connection shall be welded to develop the overstrength shear capacity of the member web.

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12.11.6 Lateral restraint requirements for the active links of EBFs

12.11.6.1

Top and bottom flanges of EBF active link members shall be laterally restrained at the ends of the active link. Restraints shall have the capacity to resist a design axial force equal to 2.5 % of the beam flange design capacity, computed as $\phi f_y A_{fb}$, with a total lateral displacement of less than 4 mm. The design capacity of the lateral restraining system shall be used to resist this force.

12.11.6.2

Amd 2 Oct. '07 Top and bottom flanges of the active link between the braces for category 1, 2 and 3 EBFs shall be restrained at intervals as required for the yielding regions of the member from 12.6.2.3.1. Restraints shall have a strength and stiffness to resist a design axial force equal to 2.5 % of the active link member flange force at the restraint point at active link member yield with a total lateral displacement, calculated using elastic member properties, of less than 4 mm at the restraint point. The design capacity of the lateral restraining system shall be used to resist this force.

12.11.7 Capacity design requirements for EBFs

The capacity design procedure for eccentrically braced frames shall meet the following requirements:

12.11.7.1

The active links are the primary members; the braces, collector beam and columns the secondary members.

12.11.7.2

The braces and columns and all their connections shall be designed to resist the overstrength actions from the active links. The overstrength actions incorporate the overstrength factors, ϕ_{oms}, from table 12.2.8(2).

Also, for D-braced EBFs with $e \geq 3M_{sp}/V_w$ in 12.11.3.3.1, the overstrength moment generated at the face of the column shall incorporate the slab participation as described in 12.10.2.4.

The collector beams shall be designed to resist 0.8x the overstrength actions from the active links.

12.11.7.3

When determining the design actions on the secondary members, it shall be assumed that the structure is displaced laterally so that yielding hinges form in all the active links to give a yielding mechanism. The secondary members shall be designed for actions from 12.11.7.2 and incorporating the dynamic magnification factor from 12.11.7.4, subject to these actions not exceeding the maximum actions required from 12.3.3.4, in conjunction with the actions from the permanent and combination imposed loads.

12.11.7.4

The dynamic magnification factors required for design of columns are:

(a) For V-braced EBFs, 1.0 in all instances;

(b) For D-braced EBFs with columns that are category 2 members, 1.3 for determination of capacity design derived moments and axial forces and 1.2 for determination of capacity design derived shear forces;

(c) For D-braced EBFs with columns that are category 3 members, 1.5 for determination of capacity design derived moments and axial forces and 1.3 for determination of capacity design derived shear forces.

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12.11.7.5
For the columns of category 1 and 2 EBF systems, the maximum capacity design derived design compression action N^*_{oc} on the column shall be:

$N^*_{oc} \leq 0.8\ \phi\ N_s$ where $N^*_g / \phi\ N_s \leq 0.3$.

or $N^*_{oc} \leq 0.7\ \phi\ N_s$ where $N^*_g / \phi\ N_s > 0.3$.

12.11.7.6
Design for concurrency in columns shall be in accordance with 12.8.4.

12.11.7.7
When braces are welded to the active link/collector beam junction, this connection shall be analysed as rigid and the braces designed for the capacity design actions.

12.11.7.8
Splices in columns shall be located clear of any potential yielding regions, in accordance with 12.9.6.1.

12.11.7.9
Amd 2 Oct. '07 Columns shall be designed as continuous past the incoming collector beam/active link at each storey of the EBF.

12.12 DESIGN OF CONCENTRICALLY BRACED FRAMED SEISMIC-RESISTING SYSTEMS

12.12.1 Scope and definitions

12.12.1.1 General
Clause 12.12 covers seismic design requirements for concentrically braced frames (CBFs). CBFs in which the compression brace carries a dependable design force are covered by 12.12.2 to 12.12.5. CBFs considered effectively tension braced only are covered by 12.12.6.

The requirements for notching of braces (to reduce the design section capacity in axial tension of the brace) are given in 12.12.7.

12.12.1.2 Exceptions
A concentrically braced frame system not in accordance with any or all of the provisions of 12.12 may be designed on the basis of a special study.

12.12.2 CBFs with bracing effective in tension and compression

12.12.2.1
The provisions of 12.12.2 to 12.12.5 shall apply only to CBFs containing bracing members with a slenderness ratio $[\ (k_e\ L/r)\sqrt{(f_y / 250)}\ ]$ not exceeding 120.

The design seismic load applicable to the category of CBF system chosen shall be determined from 12.12.3.

The maximum height limitations applicable to the category of CBF system chosen, when it is required to resist 100 % of the seismic load in a given direction of loading, shall be determined from 12.12.4.

The design of the CBF system shall be in accordance with the requirements of 12.12.5.

If notched tension members are to be utilized in the CBF system, the notched region shall comply with the requirements of 12.12.7.

12.12.2.2
All brace members shall be designed on the basis of their design compression capacity from 6.1, whether acting in tension or compression.

In each line of braced frames, the individual braces in a given storey shall be oriented and sized so that, for each direction of seismic loading, the magnitudes of the horizontal vector force components in the tension and compression braces do not differ by more than 20 % and the shear strength of the braced bay in the 2 directions of loading does not differ by more than 10 %.

12.12.2.3
Built-up members used as bracing shall comply with the requirements of 12.9.8.

12.12.2.4
The width-thickness ratio of compression elements used in braces shall comply with the requirements of 12.5.3 for the category of structure and brace member as defined in 12.12.5.1.

12.12.2.5 Additional requirements for chevron bracing (V bracing)
(a) Chevron braces have a brace configuration (as shown in figure C12.1(a)) where a pair of braces, located either both above or below a beam, intersects the beam at a single point within the centre half of the beam span;

(b) The beam intersected by the chevron braces shall be continuous between columns and shall have a member category as specified in table 12.12.5.1;

(c) When chevron braces intersect a beam from below, the beam shall be capable of supporting all tributary design gravity loads acting in conjunction with the earthquake loads presuming the bracing is not present, except that this provision need not apply to roofs and one-storey structures.

12.12.3 Design seismic loads for CBFs with bracing effective in tension and compression

12.12.3.1 Ultimate limit state design seismic loads
The ultimate limit state design seismic loads for CBFs with bracing effective in tension and compression shall be determined in accordance with steps (1) and (2) below.

Step 1: Determine the horizontal design action coefficient, appropriate for the chosen structural ductility factor μ .

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Step 2: Multiply C_d (T_1) by the appropriate value of C_S from tables 12.12.3 (1) – 12.12.3 (4).

12.12.3.2
The C_S factors in tables 12.12.3 are only for use with a rational limit state design procedure which complies with 12.12.5.

12.12.3.3 Serviceability limit state design seismic loads
The serviceability limit state design seismic loads shall be determined in accordance with the Loadings Standard, using C_S = 1.0.

Tables 12.12.3 – C_s factors for determination of ultimate limit state design seismic loads for CBFs with bracing effective in tension and compression

Table 12.12.3(1) – C_s factors for Category 1 CBFs

Category 1 systems	Compression brace slenderness ratio $\frac{k_e L}{r}\sqrt{\left(\frac{f_y}{250}\right)}$		
Number of storeys	≤ 30	≤ 80	≤ 120
1	1.0	1.3	1.6
2 – 3	1.1	1.45	1.75
4 – 5	1.2	1.55	1.9
6 – 8	1.3	1.7	2.1

Table 12.12.3(2) – C_s factors for Category 2 CBFs

Category 2 systems	Compression brace slenderness ratio $\frac{k_e L}{r}\sqrt{\left(\frac{f_y}{250}\right)}$		
Number of storeys	≤ 30	≤ 80	≤ 120
1	1.0	1.2	1.45
2 – 4	1.1	1.3	1.55
5 – 8	1.15	1.4	1.7
9 – 12	1.25	1.5	1.8

Table 12.12.3(3) – C_s factors for Category 3 CBFs

Category 3 systems	Compression brace slenderness ratio $\frac{k_e L}{r}\sqrt{\left(\frac{f_y}{250}\right)}$		
Number of storeys	≤ 30	≤ 80	≤ 120
1	1.0	1.05	1.1
2 – 8	1.0	1.1	1.15
9 – 16	1.05	1.1	1.15
17 – 24	1.1	1.15	1.2

Table 12.12.3(4) – C_S factors for Category 4 CBFs

Category 4 systems	**Compression brace slenderness ratio** $\frac{k_e L}{r}\sqrt{\left(\frac{f_y}{250}\right)}$		
Number of storeys	**≤ 30**	**≤ 80**	**≤ 120**
1 – 16	1.0	1.0	1.0
17 – 32	1.0	1.05	1.1

NOTES TO TABLES 12.12.3(1) – 12.12.3(4):

(1) Refer to tables 12.12.4(1) to 12.12.4(4) for limits on the maximum heights to which these systems may be used in stand-alone configuration.

(2) When part of a dual seismic-resisting systems, the C_S factor for the CBF part of the dual system shall be as given in the relevant table for the appropriate number of storeys.

(3) When the number of storeys in the dual system exceeds the limit in the relevant table, as is permitted by 12.13.2(b), use the maximum C_S factor given therein.

12.12.4 Maximum height limitations for stand-alone CBF systems with bracing effective in tension and compression

12.12.4.1 Maximum height limitations for category 1 and 2 systems
Category 1 or 2 CBF seismic-resisting systems resisting 100 % of the seismic load in a given direction of loading shall be used only up to the height limitations specified in table 12.12.4(1) for category 1 and table 12.12.4(2) for category 2 systems. (These tables are on page 257).

12.12.4.2 Maximum height limitations for category 3 systems
Category 3 CBF systems resisting 100 % of the seismic load in a given direction of loading shall be used only up to the height limitations specified in table 12.12.4(3) on page 258.

12.12.4.3 Maximum height limitations for category 4 systems
Category 4 CBF systems resisting 100 % of the seismic load in a given direction of loading shall be used only up to the height limitations specified in table 12.12.4(4) on page 258.

Table 12.12.4(1) – Maximum height limitations for category 1 CBF systems

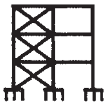
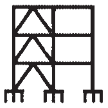
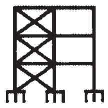
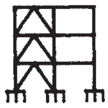
		Compression brace slenderness ratio $\frac{k_e L}{r}\sqrt{\left(\frac{f_y}{250}\right)}$		
Brace type		**≤ 30**	**≤ 80**	**≤ 120**
	x-brace	8 storeys	4 storeys	2 storeys
	v-brace (Chevron)	4 storeys	2 storeys	– (3)

Table 12.12.4(2) – Maximum height limitations for category 2 CBF systems

		Compression brace slenderness ratio $\frac{k_e L}{r}\sqrt{\left(\frac{f_y}{250}\right)}$		
Brace type		**≤ 30**	**≤ 80**	**≤ 120**
	x-brace	12 storeys	8 storeys	4 storeys
	v-brace (Chevron)	8 storeys	4 storeys	2 storeys

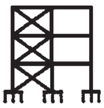
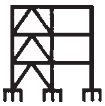
NOTES FOR TABLES 12.12.4(1) AND 12.12.4(2):

(1) If the combined roof and wall mass of the top storey weighs less than 150 kg/m^2 the height limits given in table 12.12.4(1) and 12.12.4(2) should be increased by 1 storey.

(2) These height limitations may be exceeded in a dual system designed in accordance with 12.13.

(3) Category 1 v-braced systems with brace slenderness ratios exceeding 80 are not permitted in systems of more than one storey.

Table 12.12.4(3) – Maximum height limitations for category 3 CBF systems

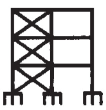
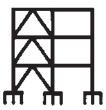
	Compression brace slenderness ratio $\frac{k_e L}{r}\sqrt{\left(\frac{f_y}{250}\right)}$		
Brace type	**≤ 30**	**≤ 80**	**≤ 120**
 x-brace	24 storeys	16 storeys	8 storeys
 v-brace (Chevron)	12 storeys	8 storeys	4 storeys

NOTES FOR TABLE 12.12.4(3):

(1) If the combined roof and wall mass of the top storey weighs less than 150 kg/m^2 the height limits given in table 12.12.4(3) should be increased by 1 storey.

(2) These height limitations may be exceeded in a dual system designed in accordance with 12.13.

Table 12.12.4(4) – Maximum height limitations for category 4 CBF systems

	Compression brace slenderness ratio $\frac{k_e L}{r}\sqrt{\left(\frac{f_y}{250}\right)}$		
Brace type	**≤ 30**	**≤ 80**	**≤ 120**
 x-brace	32 storeys	24 storeys	16 storeys
 v-brace (Chevron)	16 storeys	12 storeys	8 storeys

NOTES FOR TABLE 12.12.4(4):

(1) If the combined roof and wall mass of the top storey weighs less than 150 kg/m^2 the height limits given in table 12.12.4(4) should be increased by 1 storey.

(2) These height limitations may be exceeded in a dual system designed in accordance with 12.13.

12.12.5 Seismic design procedures

12.12.5.1 Relationship between structure category and member category for CBF systems

Table 12.12.5.1 specifies the relationship required between structure and member category for CBF systems.

Table 12.12.5.1 – Relationship between structure category and member category for CBF systems designed in accordance with clause 12.12[1]

Category of structure	Category of member for x-braced CBF			Category of member for v-braced CBF		
	Braces	Collector beams	Columns	Braces	Collector beams	Columns
1	1	3	2	1	1	2
2	2	3	3(2)[2]	2	2	2
3	3	4	3	3	3	3
4	3(4)[2]	4	4	3	3	4

NOTE –

(1) This table applies in conjunction with the seismic design procedures presented in 12.12.5.2 to 12.12.5.4 as appropriate.

(2) The unbracketed value applies to CBFs with bracing effective in tension and compression. The bracketed value applies to CBFs with bracing effective in tension only.

12.12.5.2 Seismic design procedures for category 1, 2 systems

(a) Capacity design is required on all category 1 and 2 CBF systems, with the braces chosen as the primary seismic-resisting elements;

(b) The design procedure shall take account of the force distribution in the elements of the CBF both in the elastic and the inelastic modes of response;

(c) Collector beams (see 1.3 for definition) shall be designed to develop the capacity design derived design actions from the braces in compression or tension as appropriate, except that the design axial force on the collector beams, generated by actions in the braces, need not be greater than that associated with nominally ductile seismic response of the structure from 12.12.3.1.

(d) (i) The design axial force on the column at a given level shall be determined on the basis that the braces at the level under consideration achieve their capacity design derived design actions ($\phi_{oms}N$) and all braces above that level achieve their design capacities (i.e. based on the design capacity of the bracing in compression or tension as appropriate) unless a special study is conducted to accurately determine the column axial forces.

(ii) However the design actions on the column at a given level need not be greater than those generated by nominally ductile seismic response of the structure from 12.12.3.1 in conjunction with the actions from the appropriate design gravity loads.

(iii) The design capacity of the column shall be used to resist the capacity design derived design actions.

(e) Connections shall be designed according to the provisions of 12.9, with design actions determined from 12.9.1.2;

(f) The effect of any concrete encasement on the frame stiffness and earthquake response of the CBF shall be considered in accordance with Appendix N;

(g) In the calculation of lateral deflection, the contribution to total deflection due to column axial stresses shall be included if it amounts to more than 10 % of the total frame lateral deflection;

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(h) Where the equivalent static method or the modal response spectrum method of analysis is used, the design lateral deflections, as determined from NZS 1170.5 clauses 7.2.1 and 7.3.1, shall be increased by the following factors:

(i) 1.5 for x-braced systems, with braces effective in tension and compression

(ii) 2.0 for x-braced systems, with braces effective in tension only (this is cross-referenced from 12.12.6.6)

(iii) 2.0 for v-braced systems, with braces effective in tension and compression.

(j) Concurrent action in columns forming part of a two-way seismic-resisting system shall be considered in accordance with 12.8.4.

12.12.5.3 Seismic design procedures for category 3 systems

(a) Capacity design is required on all category 3 CBF systems, with the braces chosen as the primary seismic-resisting elements;

(b) The design procedure shall take account of the force distribution in the elements of the CBF both in the elastic and the inelastic modes of response;

(c) Collector beams (see 1.3 for definition) shall be designed to develop the capacity design derived design actions from the braces in compression or tension as appropriate, except that the design axial load on the collector beams, generated by actions in the braces, need not be greater than that associated with elastic seismic response of the structure from 12.12.3.1.

(d) (i) The design axial force on the column at a given level shall be determined on the basis that the braces at the level under consideration achieve their capacity design derived design actions ($\phi_{oms}N$) and all braces above that level achieve their design capacities (i.e. based on the design capacity of the bracing in compression or tension as appropriate) unless a special study is conducted to accurately determine the column axial force.

(ii) However the design actions on the column at a given level need not be greater than those generated by elastic seismic response of the structure from 12.12.3.1 in conjunction with the actions from the appropriate ultimate limit state gravity loads.

(iii) The design capacity of the column shall be used to resist the capacity design derived design actions.

(e) Connections shall be designed according to the provisions of 12.9, with design actions determined from 12.9.1.2;

(f) The effect of any concrete encasement on the frame stiffness and earthquake response of the CBF shall be considered in accordance with Appendix N;

(g) In the calculation of lateral deflection, the contribution to total deflection due to column axial stresses shall be included if it amounts to more than 10 % of the total frame lateral deflection;

(h) Concurrent action in columns forming part of a two-way seismic-resisting system shall be considered in accordance with 12.8.4.

12.12.5.4 Seismic design procedures for category 4 systems

(a) Capacity design is not required on category 4 CBF systems;

(b) The design procedure shall take account of the force distribution in the elements of the CBF in the elastic mode of response;

(c) The collector beams and columns shall be designed for the actions generated by the elastic seismic response of the structure from 12.12.3.1, in conjunction with the actions from the appropriate design gravity loads, using their design capacities;

(d) Connections shall be designed according to the provisions of 12.9, with design actions determined from 12.9.1.2;

(e) The effect of any concrete encasement on the frame stiffness and earthquake response of the CBF shall be considered in accordance with Appendix N;

(f) In the calculation of lateral deflection, the contribution to total deflection due to column axial stresses shall be included if it amounts to more than 10 % of the total frame lateral deflection;

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(g) Concurrent action in columns forming part of a two-way seismic-resisting system shall be considered in accordance with 12.8.4.

12.12.6 Tension braced CBF systems

12.12.6.1

The provisions of 12.12.6 apply only to CBF systems with pairs of braces at each level framing between beam-column intersections, each brace having a slenderness ratio [$(k_e L/r)\sqrt{(f_y / 250)}$] exceeding 120. The seismic design load applicable to the category of tension braced CBF system chosen shall be determined from 12.12.6.3 or 12.12.6.4, with maximum height limitations given by 12.12.6.5.

12.12.6.2

The design of the tension-braced CBF system shall be in accordance with 12.12.6.6.

12.12.6.3 Ultimate limit state design seismic loads for tension braced CBF systems

12.12.6.3.1

The lateral force coefficient C or ordinate $C(T)$ shall be determined in accordance with the Loadings Standard, using the value of C_h as appropriate for the structural displacement ductility factor appropriate to the category of the system.

12.12.6.3.2

The ultimate limit state design seismic load for the tension braced system is then determined from the product of the lateral force coefficient or ordinate and the factor C_s, as specified below:

(a) For category 1 CBFs:
- (i) C_s = 1.7 for 1 storey systems
- (ii) C_s = 1.85 for 2 storey systems

➤

(b) For category 2 CBFs:
   (i) C_S = 1.5 for 1 storey systems
   (ii) C_S = 1.65 for 2 storey systems
   (iii) C_S = 1.8 for 3 or 4 storey systems

(c) For category 3 CBFs:
   (i) C_S = 1.1 for 1 – 2 storey systems
   (ii) C_S = 1.2 for 3 – 4 storey systems
   (iii) C_S = 1.25 for 5 – 8 storey systems

(d) For category 4 CBFs:
   (i) C_S = 1.0 for 1 – 3 storey systems
   (ii) C_S = 1.1 for 4 – 6 storey systems
   (iii) C_S = 1.2 for 7 – 12 storey systems

12.12.6.3.3
Where tension braced CBF systems forming part of a dual system are permitted to exceed the above number of storeys in accordance with 12.12.6.4, the value of C_S to be used in deriving the design seismic load shall be that taken from (a) (ii), (b) (iii), (c) (iii) or (d) (iii) above, as appropriate.

12.12.6.4 Serviceability limit state design seismic loads for tension braced CBF systems
The serviceability limit state design seismic loads shall be determined in accordance with the Loadings Standard, using C_S = 1.0.

12.12.6.5 Maximum height limitations for tension braced CBF systems

12.12.6.5.1
Tension braced CBF systems resisting 100 % of the seismic load in a given direction of loading are permitted only in x-braced systems up to 2 storeys for a category 1 system, 4 storeys for a category 2 system, 8 storeys for a category 3 system and 12 storeys for a category 4 system.

12.12.6.5.2
Tension-braced CBF systems exceeding these height limitations shall only be permitted as part of a dual system in accordance with 12.13.

12.12.6.6 Seismic design procedures and requirements for tension braced CBF systems
(a) Braces shall be arranged in pairs, so that one brace is always fully effective in tension under reversing lateral loading. V-brace (Chevron) configurations shall not be used in tension braced systems;

(b) All brace members in tension braced CBF systems shall be designed on the basis of their design tension capacity from 7.1;

(c) The seismic design procedures of 12.12.5.2, 12.12.5.3 or 12.12.5.4, as appropriate to the category of the system, shall apply, with the brace design compression capacity taken as zero for the implementation of these clauses. The category of members shall relate to the category of structure in accordance with 12.12.5.1.

(d) Built-up members used as bracing shall comply with 12.9.8.

12.12.7 Design of notched regions in braces

12.12.7.1 Scope

Notching of braces, whereby a length of brace is reduced in area, is a means of reducing the design tension capacity of the brace while maintaining dependable inelastic action.

12.12.7.2 Seismic design considerations

(a) The notched member shall be continuous through the notched region with no joins, either welded or bolted;

(b) The category of member within the notched region shall be taken as the category of the structure, in accordance with table 12.12.5.1;

(c) The grade of steel used in the notched member shall comply with 12.4.1 for the category of member chosen;

(d) The notched section shall comply with the section geometry requirements of 12.5.3.1 for the category of member chosen;

(e) The notched section shall be formed by cutting down the brace member with a slope through the transition zone not exceeding the ratio of 1:2.5 vertical:horizontal, where vertical is measured as the direction perpendicular to the longitudinal axis of the brace and horizontal is measured parallel to the brace longitudinal axis. All changes in slope shall be smoothly rounded to a radius of not less than 10 mm.

(f) The ratio of notched member area to gross member area shall be no greater than 0.80;

(g) The total permitted inelastic strain over the length of notched section, not including the length of transition zone, shall not exceed 42 x the yield strain for grade 250 or 300 members, 24 x the yield strain for grade 350 members or 18 x the yield strain for grade 450 members.

The total inelastic strain on the notched region of the brace shall be derived using the strain resulting from the structure achieving 2 times the total lateral deflection under the applied lateral loads as calculated from the Loadings Standard.

(h) The nominal inelastic buckling load (N_{on}) of the notched region shall be calculated as $N_{on}=\pi^2 E_n I_n / (k_e L_n)^2$, where E_n shall be taken as 5000 MPa, I_n is the second moment of area of the notched section, k_e = 1.2 and L_n is the clear length of the notched section. N_{on} shall be taken as the minimum of N_{onx} or N_{ony} determined separately about each principal member axis.

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N_{on} shall exceed 1.5 times the nominal member capacity of the brace as a whole, (N_c), determined in accordance with 6.3 and neglecting the effect of the notched region.

(j) Connections shall be designed to transmit the overstrength tensile section capacity ($\phi_{oms} N_t$) of the notched section for the appropriate category of member using the design capacity (see 12.9.1.2) of the connection components;

(k) Notched regions shall be located clear from any lengths of a member that would be yielding regions in the absence of a notch being provided.

12.13 DUAL SEISMIC-RESISTING SYSTEMS

12.13.1

Frames which resist lateral loads by a dual system, incorporating 2 or more seismic-resisting systems acting in parallel to resist seismic forces in a given direction of loading, shall be designed and detailed in accordance with the requirements of 12.10 to 12.12 as applicable. The seismic-resisting systems may include concentrically braced, eccentrically braced or moment-resisting frames.

12.13.2

The following requirements shall apply:

(a) The seismic design load for the dual system shall be calculated on the basis that the load is shared between lateral load-resisting systems in parallel on the basis of their relative stiffness, except that the moment-resisting frame in a dual system shall be designed to resist a minimum of 25 % of the base seismic design shear, including torsional effects. The base seismic design shear shall be determined for each system independently and the final seismic design loading for the dual system determined in accordance with a rational procedure.

(b) Dual systems comprising a concentrically braced frame system and a moment-resisting frame system in accordance with 12.12 and this clause may exceed the height limitations of 12.12.4 or 12.12.6.3, as appropriate, on the basis of a special study.

(c) In dual systems comprising a concentrically braced frame system, utilizing a v-brace (Chevron) configuration and a moment-resisting frame system, the v-brace system shall not be used to carry more than 50 % of the storey shear in structures exceeding the appropriate height limitations of 12.12.4.

12.14 FABRICATION IN YIELDING REGIONS

12.14.1 Shearing and gas cutting

12.14.1.1

In areas designated as yielding regions (1.3) sheared edges are not permitted. Material prepared by shearing shall be sheared 3 mm oversize with the excess material subsequently removed by machining.

12.14.1.2

A gas cut edge shall have a maximum surface roughness of 12 μm (Centre Line Average Method) (see 14.3.3.4).

12.14.2 Punching

In areas designated as yielding regions, fastener holes may only be punched in accordance with 12.9.4.5.3.

12.14.3 Transition of thickness

12.14.3.1

Transitions of thickness in members shall be undertaken in accordance with 9.7.2.6 for non-yielding regions in any category of member or for yielding regions in category 3 members.

12.14.3.2

For yielding regions in category 1 or 2 members, 9.7.2.6 shall also apply, except that the transition slope shall not exceed 1:2.5, where the former dimension is perpendicular to the line of applied force and the latter dimension is parallel to the line of applied force.

12.14.4 Transition of width

12.14.4.1
An abrupt transition of width is permitted where it occurs outside of a yielding region.

12.14.4.2
Where a transition of width occurs within a yielding region in category 1 or 2 members, the transition slope shall not exceed 1:2.5, where the former dimension is perpendicular to the line of applied force and the latter dimension is parallel to the line of applied force.

12.14.4.3
Where a transition of width occurs with a yielding region in a category 3 member, the transition slope shall not exceed 1:1.

NOTES

13 DESIGN OF COMPOSITE MEMBERS AND STRUCTURES

13.1 SCOPE AND GENERAL INTRODUCTION

13.1.1 Scope

This section covers the design of composite members of the following types and, in conjunction with other sections, the design of connections to these members:

(a) Structural steel sections of either open web or full web type encased in reinforced concrete;

(b) Structural steel sections supporting a reinforced concrete slab acting together to resist bending and shear;

(c) Steel encased reinforced concrete members or concrete-filled structural hollow sections with a reinforced or unreinforced concrete core.

13.1.2 Design assumptions and requirements

13.1.2.1

The following apply:

(a) Design capacity of composite members, including slabs cast onto formed steel deck, shall be calculated for the ultimate limit state using the appropriate design load combinations from the Loadings Standard;

(b) The methods of structural analysis specified in section 4 shall apply as appropriate, including all the assumptions for analysis specified in 4.3;

(c) The strength reduction factors specified in table 13.1.2 shall be used in calculating the design capacity of composite members.

13.1.2.2

Encased beams shall be proportioned to support, unassisted, all design dead loads and design construction live loads applied prior to hardening of the concrete (unless these loads are supported temporarily by propping) acting on the steel beam alone. The full design dead and live loads shall be assumed to be carried by the composite section, which shall include an effective width of slab where appropriate.

13.1.2.3

The following apply:

(a) When shear connectors are used in accordance with 13.3.2, for unpropped construction the steel beam alone shall be assumed to carry the design dead loads and design construction live loads prior to hardening of the concrete. The composite beam section shall be assumed to carry all design dead and live loads once the concrete has reached its specified concrete cylinder compression strength.

(b) For composite beams unpropped during construction, the stresses in the tension flange of the steel section due to the serviceability limit state design loads applied before the concrete reaches its specified cylinder compression strength plus the stresses at the same location, due to remaining specified serviceability limit state design loads considered to act on the composite section, shall not exceed 0.90 ϕf_y, where f_y is the yield stress of the steel beam and ϕ is specified in table 13.1.2 (2).

➤

(c) For propped beam construction, if 2 or more equally spaced props per beam span are provided, all design dead and live loads may be assumed to be carried by the composite section alone (without the propping). For propped beam construction with one prop only per beam span the design shall take into account the effects of prop removal.

(d) Reinforcement parallel to the beam within the effective width of the slab, when anchored to the requirements of NZS 3101, shall be included in computing the properties of composite sections, provided shear connectors are furnished in accordance with 13.3.2 or 13.3.3. The contribution of concrete in tension shall be neglected.

Table 13.1.2 – Strength reduction factors (ϕ) for composite members and connections

Table 13.1.2 (1) – Ultimate limit state strength reduction factors (ϕ) for composite members and connections

Design capacity at ultimate limit state for	Clauses	Strength reduction factor (ϕ)
Composite slabs subject to:	13.2.1	
- shear		0.75 (1)
- flexure		0.85 (1)
- bearing		0.65 (1)
Composite slabs subject to fire conditions	13.2.2.2	1.0
Longitudinal shear splitting in slab over beam	13.4.10	
- concrete (ϕ_c)		0.60
- reinforcement (ϕ)		0.90
Shear connectors (ϕ_{sc})	13.3.2.1	
- situated in positive moment regions	13.4.5.2	1.0
- situated in negative moment regions	13.4.7	0.75
Flexural strength of composite beams		
- subject to positive moment action	13.4.5.2	0.85 (1)
- subject to negative moment action	13.4.8.1	0.90 (1)
Composite beams subject to shear		
- shear resisted by steel section alone	13.6.1.1	0.90
- shear resisted by concrete in encased section	13.6.1.2	0.75 (2)
- shear resisted by encased open web members	13.6.2	0.75 (2)
Connections	13.7.1	
- steel components		from table 3.3 (3)
- concrete components		from NZS 3101 (2)
Composite columns		
- encased columns	13.8.2	from NZS 3101 (2)
- concrete-filled columns	13.8.3	from NZS 3101 (2) or alternative (1)

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Table 13.1.2 (2) – Serviceability limit state strength reduction factors (ϕ) for composite members

Design capacity at serviceability limit state for	Clauses	Strength reduction factor (ϕ)
Stress in tension flange of steel	13.1.2.3 (b)	1.0
Stress in steel beam or reinforcement	13.1.2.6	1.0

NOTES TO TABLES 13.1.2 (1) AND 13.1.2 (2):

(1) The strength reduction factor specified in table 13.1.2 is applied to the calculated nominal section capacity to derive the design section capacity. If a limit state design procedure is used which incorporates different partial strength reduction factors which are applied to the material strengths prior to calculation of the design section capacity, then these factors shall be used *in lieu* of the appropriate factor given in table 13.1.2 (1) to which this note refers.

(2) The strength reduction factor(s) is (are) as specified in NZS 3101 as the design capacity is calculated in accordance with NZS 3101.

(3) For steel components the strength reduction factor is specified in table 3.3.

13.1.2.4 Partial composite action

13.1.2.4.1

Partial composite action may be used within positive moment regions of composite beams or sections of composite beams, incorporating shear connectors, that are not required to form yielding regions under severe earthquake loads or effects. Partial composite action must also comply with the requirements of 13.4.6.

13.1.2.4.2

Partial composite action is not permitted in negative moment regions of composite beams.

13.1.2.5 Effective slab thickness

13.1.2.5.1

The effective slab thickness, t, of a concrete slab cast onto a profiled steel deck, for ribs oriented either parallel to or perpendicular to the supporting steel beam, shall be taken as the overall slab thickness, t_o, provided that:

(a) The slab is cast with a flat underside; or,

(b) The slab is cast on corrugated steel forms having a height of corrugation not greater than 0.25 times the overall slab thickness; or,

(c) The slab is cast on ribbed steel forms whose profile meets all of the following requirements: The minimum concrete rib width shall be 125 mm, the maximum rib height shall be 40 mm but not more than 0.4 times the overall slab thickness, the average width between ribs shall not exceed 0.25 times the overall slab thickness nor 0.2 times the minimum width of concrete ribs; or,

(d) The profile ribs between successive sheets interlock with no concrete between them.

13.1.2.5.2

In all other cases the effective slab thickness, t, shall be the overall slab thickness, t_o, minus the depth of rib (i.e. the cover slab thickness, which is equal to ($t_o - h_{rc}$)).

13.1.2.5.3

In no case shall the effective slab thickness be less than 65 mm, unless the adequacy of a lesser thickness is established by experimental testing. Except that, in the case of dove-tailed profiles that meet all the requirements of 13.1.2.5.1(c) above except the requirement for maximum rib height, the minimum effective slab thickness required shall be reduced to 50 mm.

13.1.2.6 Serviceability checks required

The following checks shall be carried out under serviceability limit state loads as specified in the Loadings Standard:

(a) The maximum stress in the tensile flange of a composite steel beam unpropped during construction shall not exceed 0.9 ϕ f_y as stipulated in 13.1.2.3(b);

(b) In continuous composite beams, the maximum stress in the tension reinforcement over support regions, due to the portion of the serviceability limit state loads which are applied to the section after the concrete reaches its specified cylinder compression strength, shall not exceed 0.6 ϕf_{yr}, where f_{yr} is the yield stress of the tension reinforcement and ϕ is specified in table 13.1.2 (2).

(c) Deflection of composite members under the short plus long-term serviceability limit state design loads shall be calculated, taking into account the effects of creep and shrinkage of concrete and increased flexibility resulting from interfacial slip and, where applicable, partial composite construction. The contribution of all long-term loads resisted by the composite section shall be considered in the calculation of long-term deflection.

(d) Short- and long-term deflection limits shall be chosen that are appropriate to the serviceability limit state for the type of finishings used. Deflections should comply with the recommendations of the Loadings Standard, where appropriate.

13.2 COMPOSITE SLAB DESIGN

13.2.1 Design methods

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The design capacity of composite slabs on profiled steel sheet decking shall be to BS EN 1994-1-1:2004 Section 9 and the design of the deck itself for the wet concrete loading and construction stages shall be to BS EN 1993-1-3:2006. Loads and load arrangements on the slab shall be determined from the AS/NZS 1170 set. Other loads shall be determined from the documents specified from BS EN 1994-1-1:2004. Where these requirements specify different ultimate limit state load factors from those specified by AS/NZS 1170.0 and/or different strength reduction factors from those specified in table 13.1.2(1), the factor of safety (reliability) against failure shall be consistent with that specified by NZS 3404:Part 2 clause C3.1(c) (see NOTE (5)).

NOTE –

(1) For design of composite slabs on profiled steel sheet decking, BS 5950-4:1994 may be used instead of BS EN 1994-1-1:2004 and BS 5950-6:1995 (with 1999 corrigendum) may be used instead of BS EN 1993-1-3:2006.

(2) Design of the profiled steel sheet decking itself for the wet concrete and construction stages may be to AS/NZS 4600: 2005.

(3) The yield stress and tensile strength used in design for steels conforming to AS 1397 Grades 450, 500 and 550 shall be limited as specified in the standard used for design of the steel decking, but shall not be greater than 9 0% of the minimum specified values from AS 1397.

(4) Design of composite slabs for concentrated loads closer than 500 mm to an end support requires rational design.

(5) This requirement will be met when the load factors are those specified by the standard used for the design of the composite slab or the design of the deck itself and the construction live loads are those specified by the standard used for the design of the composite slab.

13.2.2 Slab reinforcement

13.2.2.1

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Slabs shall be adequately reinforced to support all specified loads and to control cracking so as to meet the New Zealand Building Code performance requirements for durability, strength and fire safety. Reinforcement parallel to the span of the composite beam shall be anchored by embedment in concrete that is in compression. Reinforcement of slabs that are to be continuous over the end support of steel sections fitted with simple end connections (see 9.1.2.3) shall be given special attention.

13.2.2.2

Minimum amounts of reinforcement for each purpose shall be as specified in an appropriate design procedure or Standard, and as required for design for a specified period of structural adequacy (see 11.3) for fire conditions.

13.2.2.3

The possibility of longitudinal splitting due to composite action directly over the steel section shall be controlled by the provision of additional transverse reinforcement. For slabs supported by composite beams with shear connectors, the provisions of 13.4.10 and the limitations of 13.4.1.3 apply.

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(Clause 13.2.2.4 deleted)

13.2.2.5

In regions of a composite beam subjected to inelastic earthquake loads or effects, the contribution of the concrete to resistance of longitudinal splitting shall be neglected in calculating the amount of transverse reinforcement required.

13.2.3 Provisions for seismic design of floor slabs

13.2.3.1

The composite floor slab shall be designed and detailed, where required, to transfer horizontal seismic-induced diaphragm shear actions into the supporting beams in accordance with the requirements of NZS 3101.

13.2.3.2

These shear actions shall be resisted by suitable shear connections in accordance with 13.3.2 or 13.3.3 or by encasement in accordance with 13.3.1. Design actions induced in shear connectors by seismic-induced diaphragm effects shall be added to any design actions arising from composite action, using the appropriate design load combinations from the Loadings Standard.

13.2.4 Bases for design and construction

13.2.4.1

In the contract documents for construction, the following bases for design shall be stated:

(a) The extent of propping of the deck and any supporting beams;

(b) The precambering of any supporting beams;

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(c) The proposed method of screeding of the concrete surface, i.e. screeding to level or screeding to thickness.

13.2.4.2

Prior to construction commencing, if changes to these design bases are proposed, the adequacy of the floor slab and supports shall be determined to the approval of the Design Engineer or the Construction Reviewer.

13.2.4.3

When the deck and any supporting beams are unpropped, the maximum expected deflection, Δ_m of the floor system under the wet concrete shall be calculated and:

(a) The ponding effect shall be calculated when this deflection is greater than 1/10 of the specified effective thickness of concrete, h_e ;

Amd 2 Oct. '07 (b) The ponding effect shall be allowed for in calculating the unsupported deck span lengths and any propping requirements.

13.3 CONNECTIONS BETWEEN STEEL AND CONCRETE FOR COMPOSITE ACTION

13.3.1 Encasement of sections

Unpainted steel sections supporting slabs and totally encased in concrete do not require interconnection by means of shear connectors to achieve full composite action provided that:

(a) A minimum of 50 mm of concrete covers all portions of the steel section except as noted in item (c) of this clause; and

(b) The cover in item (a) is reinforced transversely as required by NZS 3101 including for the level of any ductility demand anticipated from the beam for load combinations including earthquake loads; and

(c) The top of the steel section is at least 40 mm below the top and 50 mm above the bottom of the slab.

13.3.2 Shear connectors

13.3.2.1 Nominal shear capacity of connectors

The nominal shear capacity, q_r, of a shear connector shall be established by experimental testing, or alternatively the following values may be used:

(a) End welded studs, headed or hooked with $h_{sc}/d_{sc} > 4$

(h_{sc} = height of stud after installation, d_{sc} = diameter)

$$q_r = \alpha_{dc} 0.13\sqrt{f'_c}\, A_{sc} f_u \le 0.8 f_u A_{sc} \quad \text{(Eq.13.3.2.1)}$$

Amd 2 Oct. '07 where f'_c is in MPa and there are at least 5 shear studs in the shear span, and

where

A_{sc} = shank area of stud

f'_c = specified concrete cylinder compression strength at 28 days

E_c = short-term elastic modulus for concrete

f_u = minimum tensile strength of stud

= 415 MPa for studs to AS 1443 grades 1010 to 1020 or ASTM A108 grades C1010 to C1020

α_{dc} = as determined from equations 13.3.2.3, 13.3.2.4 or 13.3.2.5 as applicable.

This value is limited to use in designs incorporating a solid concrete slab or designs incorporating a ribbed slab formed by casting a concrete slab onto a profiled steel deck.

(b) Channel connectors

$$q_r = 36.5\,(t_{fsc} + 0.5\,t_{wsc})\,L_{sc}\sqrt{f'_c}\ \text{(in Newtons)} \leq f_{usc}\,t_{wsc}\,L_{sc} \quad \text{(Eq. 13.3.2.2)}$$

where

t_{fsc} and t_{wsc}	=	average flange and web thickness of channel shear connector (mm)
L_{sc}	=	length of channel shear connector (mm)
L_{sc}	≥	100 mm required
f'_c	=	specified concrete cylinder compression strength (MPa).

Equation 13.3.2.2 is applicable only to use in designs incorporating a solid concrete slab of normal density concrete (2300 kg/m^3) with $f'_c \geq 20$ MPa.

(c) Other shear connectors

Shear connectors other than those described above must comply with the requirements of 13.3.3.

13.3.2.2 Stud shear connectors used with profiled steel deck

13.3.2.2.1 *General*

(a) Clause 13.3.2.2 is applicable to decks with a nominal rib height not exceeding 75 mm;

(b) The average width of a concrete rib or haunch, b_r, shall be not less than 50 mm, but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck;

(c) The concrete slab shall be connected to the steel beam with welded stud shear connectors 22 mm or less in diameter or equivalent studs complying with 13.3.3. Welded studs may be welded through the deck or directly to the steel member.

(d) Stud shear connectors shall extend not less than 40 mm above the top of the steel deck after installation;

(e) The slab thickness above the steel deck shall be not less than 50 mm.

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(f) When the deck rib is oriented transverse to the steel beam and is continuous across the steel beam, the stud shall be placed in the centre of the rib, unless there is an upstand in the deck at the centre of the rib that prevents this. Where there is such an upstand, the studs shall be placed as close as practicable to the upstand and alternatively on the left hand side and on the right hand side of the upstand throughout the length of the span.

13.3.2.2.2 *Reduction factors to be applied to stud capacity*

(a) Steel deck ribs oriented transverse to the steel beam

The nominal shear capacity, q_r, shall be multiplied by the reduction factor α_{dc}

$$\alpha_{dc} = \left(\frac{0.70}{\sqrt{n_{rc}}}\right)\left(\frac{b_r}{h_{rc}}\right)\left(\frac{h_{sc}}{h_{rc}} - 1.0\right) \quad \text{(Eq 13.3.2.3)}$$

but for $n_{rc} = 1$, $\alpha_{dc} \leq 1.0$; for $n_{rc} = 2$, $\alpha_{dc} \leq 0.8$ and for $n_{rc} = 3$, $\alpha_{dc} \leq 0.6$

where

h_{rc} = nominal height of steel deck rib, mm

h_{sc} = length of stud connector after welding, mm, not to exceed the value (h_{rc} + 75) in calculations, although the actual length may be greater

b_r = width at the base of the concrete rib, mm

n_{rc} = number of stud shear connectors on a beam in one rib, not to exceed 3 in calculations for α_{dc} or for the number of studs installed in one rib.

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(b) Steel deck ribs oriented parallel to steel beam

The nominal shear capacity, q_r, shall be multiplied by the reduction factor α_{dc}

$$\alpha_{dc} = 1.0 \text{ for } b_r/h_{rc} \geq 1.5 \qquad \text{(Eq. 13.3.2.4)}$$

Amd 1 June '01

$$\alpha_{dc} = 0.6\left(\frac{b_r}{h_{rc}}\right)\left(\frac{h_{sc}}{h_{rc}} \pm 1.0\right) \leq 1.0 \quad \text{for } b_r/h_{rc} < 1.5 \qquad \text{(Eq. 13.3.2.5)}$$

When the nominal depth of steel deck is 40 mm or greater, the average width, b_r, of the supported haunch or rib shall be not less than 50 mm for the first stud in the transverse row plus 4 stud diameters for each additional stud.

13.3.2.3 Detailing requirements for shear connectors

In addition to the detailing requirements contained in 13.3.2.1 and 13.3.2.2 the following requirements apply:

(a) The diameter of the head of a headed stud must be not less than 1.5 times the diameter of the shank;

(b) The distance between the edge of a shear connector shank and the edge of the plate to which it is welded shall be not less than 25 mm;

(c) The diameter of stud connector welded to a flange or web plate shall not exceed 2.5 times the plate thickness unless adequate performance is verified by experimental testing;

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(d) The horizontal distance between a free concrete surface (with or without steel decking) and any shear connector shall be not less than the greater of 50 mm or $2.2d_{sc}$ for an edge beam (L-beam) or not less than the greater of 30 mm or $2.2d_{sc}$ for an interior beam (T-beam). If, in the case of an edge beam, the actual horizontal distance is less than the overall slab thickness, the nominal shear capacity of the shear connector from 13.3.2.1 shall be reduced accordingly.

(e) The minimum concrete cover to the top of a shear connector shall be as specified by NZS 3101, unless the shear connector is suitably protected against corrosion, in which case the top of the connector may be flush with the surface of the concrete contributing to the flexural resistance of the member.

(f) The minimum centre-to-centre spacing of stud shear connectors shall be 6 diameters along the longitudinal axis of the supporting composite beam and 4 diameters transverse to the longitudinal axis of the supporting composite beam.

(g) The longitudinal spacing of channel shear connectors shall be such that a minimum clear distance of 100 mm or the height of the connector, whichever is the greater, is provided between adjacent connectors.

(h) The maximum longitudinal spacing of shear connectors shall not exceed 8 times the total slab thickness or 800 mm in regions of a composite member not subject to inelastic earthquake effects and 4 times the total slab thickness or 400 mm in regions subject to inelastic earthquake effects (yielding regions).

(j) In yielding regions of a composite member stud, shear connectors welded to the beam flange shall be staggered about the web centreline.

13.3.2.4 Ties

Mechanical ties shall be provided between the steel section and the slab or steel deck to prevent separation. Shear connectors, puddle welds, or powder-actuated shear connector fastener pins may serve as mechanical ties if suitably proportioned. The maximum spacing of ties shall not exceed 1000 mm, and the average spacing in a span should not exceed 600 mm nor be greater than that required to achieve a period of structural adequacy (see 11.3) as required for any specified fire resistance rating of the composite assembly.

13.3.2.5 Welding standards for shear connectors

Design and execution of welding for shear connectors shall be in accordance with 9.7 and AS/NZS 1554:Part 1, except that welding of proprietary steel studs shall be in accordance with AS 1554:Part 2.

13.3.3 Other methods of achieving composite action

Other methods of interconnection, which have been adequately demonstrated by experimental test and/or rational analysis, may be used to effect the transfer of forces between the steel section and the effective area of slab. Where the method of achieving composite action involves the use of a mechanical shear connection of some type, the design of the composite member shall conform to the design of a similar member employing shear connectors, in so far as practicable.

13.4 DESIGN OF COMPOSITE BEAMS WITH SHEAR CONNECTORS

13.4.1 Scope of application of 13.4

13.4.1.1 Stress distribution in steel and concrete

The design of composite beams for the ultimate limit state to 13.4 shall be based on a plastic distribution of stresses across the section.

An elastic distribution of stresses across the section in design for the ultimate limit state may only be used in conjunction with an appropriate limit state Standard or limit state design procedure.

13.4.1.2 Section geometry requirements

The steel beam section geometry requirements must satisfy 13.4.3.

13.4.1.3 Suppression of longitudinal shear failure

Suppression of failure due to slab splitting under longitudinal shear action is given by 13.4.10, except that the provisions of 13.4.10 may not dependably suppress potential longitudinal shear splitting failure in situations where all of the following apply:

(1) Edge beams (i.e. as given by 13.4.2.1.2) supporting slabs cast onto profiled steel deck in which the deck ribs are oriented at more than 15^{o} to the longitudinal axis of the beam (i.e. are deemed perpendicular to the beam); and

(2) Where the profiled steel deck extends across the top flange of the steel beam; and

(3) Where the width of profiled steel deck cantilevered slab outstand projecting beyond the outer edge of the top flange of the steel section is less than 550 mm wide.

Edge beams with a slab configuration conforming to (1) – (3) above shall either be designed as non-composite, or the design for longitudinal shear in a composite beam shall be to a rational limit state composite design procedure which includes provisions to dependably suppress longitudinal shear failure in these instances.

13.4.2 Design effective width of concrete slab

For composite beams with uniform loading or a series of 2 or more equal point loads, the effective width of a concrete slab is given by 13.4.2.1, 13.4.2.2 or 13.4.2.3 as appropriate.

13.4.2.1 Effective width of midspan positive moment regions

13.4.2.1.1

Slabs, or cover slabs extending on both sides of the steel section, for simply supported or continuous composite beam spans, shall be deemed to have a design effective width, b_{ec}, equal to the lesser of:

(a) 0.25 times the composite beam span; or

(b) The average distance from the centre of the steel section to the centres of adjacent parallel supports.

13.4.2.1.2

Slabs, or cover slabs extending on one side of the supporting section, for simply supported or continuous composite beam spans, shall be deemed to have a design effective width, b_{ec}, not greater than the width of the top flange of the steel section plus the lesser of:

(a) 0.1 times the composite beam span; or

(b) 0.5 times the clear distance between the steel section and the adjacent parallel support.

NOTE – If the slab, or cover slab, extends as a cantilever beyond the supporting section on the side opposite to the adjacent parallel support, then in lieu of a more detailed calculation the design effective width of that portion of slab shall be taken as the lesser of (a) above or the actual length of slab projecting beyond the outer edge of the top flange of the steel section.

13.4.2.2 Effective width of support negative moment regions

The effective width shall be taken as 0.6 times the value determined from 13.4.2.1. This width shall carry no internal tension actions except those generated by longitudinal reinforcement anchored to NZS 3101, which is contained within the effective width.

13.4.2.3 Effective width of support positive moment regions

13.4.2.3.1

The effective width shall be the width of the column flange or spreader plate.

13.4.2.3.2

This effective width shall be maintained along the beam span for a distance determined by the intersection of a pair of reference lines with the effective width in midspan positive moment regions, as determined from 13.4.2.1. The pair of reference lines shall be drawn, in plan, connecting from the centroid of the column section and oriented at 45° to the beam longitudinal axis, as shown in figure 13.4.2.3.

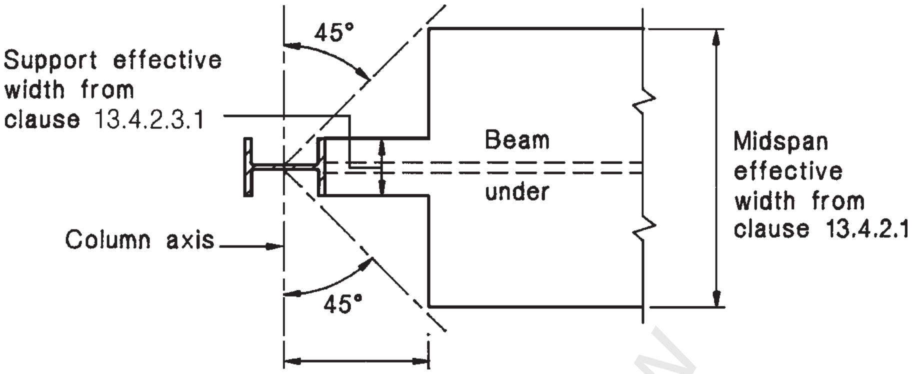

Figure 13.4.2.3 – Transition from effective width at support positive moment region to that at midspan positive moment region

13.4.3 Steel beam section geometry requirements

The steel beam section geometry requirements for the positive moment regions of simply supported or continuous beams are given by 13.4.3.1 and for the negative moment regions of continuous beams by 13.4.3.2 and 13.4.3.3.

13.4.3.1 Requirements for positive moment regions

(a) If the inelastic rotation demand (θ_p) required at the point of maximum moment does not exceed 10 milliradians (10×10^{-3} radians), the steel section shall comply with the following section geometry requirements:

$$b / t_f \quad \leq \quad 16 / \sqrt{\left(\frac{f_y}{250}\right)} \qquad \text{(see NOTE below)}$$

$$d_{1c} / t_w \quad \leq \quad 82 / \sqrt{\left(\frac{f_y}{250}\right)}$$

where

b = clear width of compression beam flange outstand from face of supporting web

t_f, t_w = thickness of beam flange, web

d_{1c} = twice the distance from the plastic neutral axis (PNA) of a composite section to the inside face of the compression flange (taken as zero if the PNA of the section lies above the inside face of the compression flange)

θ_p = inelastic rotation demand, being the ratio of the rotation at a plastic hinge location or yielding region to the relative elastic rotation of the far end of the segment containing the plastic hinge or yielding region.

(b) If the inelastic rotation demand (θ_p) required at the point of maximum moment exceeds 10 milliradians, the steel section shall comply with the following section geometry requirements:

$$b / t_f \quad \leq \quad 10 / \sqrt{\left(\frac{f_y}{250}\right)} \qquad \text{(see NOTE below)}$$

➤

$$d_{1c} / t_w \leq 65 / \sqrt{\left(\frac{f_y}{250}\right)}$$

NOTE – The limit for (b/t_f) is applicable to all except heavily welded members (i.e. welded members with compressive residual stresses exceeding 40 MPa). For heavily welded members, reduce 16 to 15 in the limit for (b/t_f) given in (a) above and 10 to 9 in the limit for (b/t_f) given in (b) above.

13.4.3.2 Requirements for negative moment regions

(a) If the inelastic rotation demand (θ_p) required at the point of maximum negative moment (assumed to be at a support) does not exceed 10 milliradians, the steel section shall comply with the following section geometry requirements:

(i) When shear connectors are terminated at least 1.5*d* from the column face at a support, the requirements for a category 3 member given in table 12.5 shall apply, for all load combinations.

(ii) When shear connectors are continued through to the column face at the support in accordance with 13.4.8;

$$b / t_f \leq 9 / \sqrt{\left(\frac{f_y}{250}\right)} \quad \text{(see NOTE below)}$$

$$\frac{d_{1c}}{t_w} \leq (82\, C_5) / \sqrt{\left(\frac{f_y}{250}\right)} \quad \text{and}$$

$$\frac{d_1}{t_w} \leq (101\, \alpha_b) / \sqrt{\left(\frac{f_y}{250}\right)}$$

where

d = depth of the section (taken at the support if variable)
C_5 = factor defined in equation 13.4.3.2
α_b = factor defined in equation 13.4.3.2.

(b) If the inelastic rotation demand (θ_p) required at the point of maximum negative moment exceeds 10 milliradians, the steel section shall comply with the following section geometry requirements:

(i) When shear connectors are terminated at least 1.5*d* from the column face at a support the plasticity limits given in table 5.2 shall apply to the elements of the section.

(ii) When shear connectors are continued through to the column face at the support in accordance with 13.4.8;

$$b / t_f \leq 9 / \sqrt{\left(\frac{f_y}{250}\right)} \quad \text{(see NOTE below)}$$

$$\frac{d_{1c}}{t_w} \leq (65\, C_5) / \sqrt{\left(\frac{f_y}{250}\right)} \quad \text{and}$$

$$\frac{d_1}{t_w} \leq (71\, \alpha_b) / \sqrt{\left(\frac{f_y}{250}\right)}$$

(c) In (a) (ii) and (b) (ii) above:

α_b = C_5 (1- 1.4 ψ) .... (Eq. 13.4.3.2)

C_5 = 1.0 if beam web unstiffened

C_5 = 2.0 if beam web stiffened to the requirements of 13.4.3.3

$$\psi = \frac{C_6 A_{rs} f_{yr}}{A f_y}$$

C_6 = 1.0 for members designed for load combinations not including earthquake loads

C_6 = ϕ_{os} from tables 12.2.8 for members designed for load combinations including earthquake loads

A_{rs} = area of slab reinforcement within effective width of slab

A = area of steel member cross section

d_1 = clear depth between flanges ignoring fillets or welds

d_{1c} = twice the distance from the plastic neutral axis of the composite section to the inside face of the compression flange.

NOTE – The limit for (b/t_f) in (a) or (b) is applicable to all except heavily welded members (i.e. welded members with compressive residual stresses exceeding 40 MPa). For heavily welded members, reduce 9 to 8 in the limit for (b/t_f) given in (a) and (b) above.

13.4.3.3 Web stiffeners for negative moment regions

Stiffeners shall comply with the following requirements:

(a) The stiffeners must be located at midheight of the steel beam and extend 1.5 times the total depth of the composite beam out from the supports;

(b) A pair of stiffeners is required;

(c) The section slenderness (λ_s) of each stiffener must not exceed the yield limit in table 5.2 and the stiffener(s) must have a second moment of area about the minor axis of the beam, taken about the centreline of the beam web, of not less than the corresponding second moment of area of one beam flange.

(d) The welds between stiffener and beam web must be capable of developing the design section capacity in axial compression of the stiffener.

13.4.4 Lateral restraint of steel beam

13.4.4.1

The design of the steel beam shall take account of the lateral restraint available at each stage of construction and its effect on the design capacity of the member.

13.4.4.2

A profiled steel deck shall not be considered to laterally support the top flange of the steel section until the shear connectors are in place or until other forms of direct connection with adequate strength are made between the beam flange and steel deck.

13.4.4.3

A profiled steel deck with ribs oriented perpendicular to the steel beam may be assumed to provide continuous lateral restraint to the top flange of the beam once rigidly connected to the beam. The degree of lateral restraint offered to the top flange of the steel beam by a profiled steel deck with ribs oriented parallel to the beam shall, if required, be determined by rational analysis or experimental tests but otherwise be neglected in design until the concrete is hardened.

➤

13.4.4.4
When the beam flange within a segment (see 5.3.1.2) is in contact with the concrete slab and is the critical flange (see 5.5), the segment may be considered to have full lateral restraint (see 5.3.1.5). When the beam flange within a segment is remote from contact with the concrete slab and is the critical flange, the member moment capacity for that segment shall be determined from 5.6.

13.4.5 Calculation of the positive moment capacity of the composite section

13.4.5.1
The properties of the composite section shall be calculated neglecting any concrete area in tension with the maximum effective area of concrete slab equal to the effective slab width multiplied by the effective slab thickness. If a steel truss is used, the area of its top chord shall be neglected in determining the properties of the composite section and only 13.4.5.2 (a) is applicable.

13.4.5.2
The design positive moment capacity (ϕM_{rc}) of the composite section shall be calculated from the nominal moment capacity (M_{rc}), as given by Cases 1, 2 or 3 below, multiplied by the strength reduction factor (ϕ) from table 13.1.2 (1).

(a) **Case 1 –** Full composite action (full shear connection) and plastic neutral axis in the slab, that is:

$\phi_{sc} R_{ss} \geq A f_y$ and $A f_y \leq 0.85 f'_c b_{ec} t$

$$M_{rc} = R_{tc} e' = A f_y e' \quad \text{(Eq. 13.4.5 (1))}$$

where

e' = the lever arm and is computed using

$$a_c = \frac{A f_y}{0.85 f'_c b_{ec}} \quad \text{(Eq. 13.4.5 (2))}$$

(b) **Case 2 –** Full composite action and plastic neutral axis in the steel section, that is:
$\phi_{sc} R_{ss} \geq 0.85 f'_c b_{ec} t$ and $0.85 f'_c b_{ec} t < A f_y$

$$M_{rc} = R_{sc} e + R_{cc} e' \quad \text{(Eq. 13.4.5 (3))}$$

$$R_{cc} = 0.85 f'_c b_{ec} t \quad \text{(Eq. 13.4.5 (4))}$$

$$R_{sc} = \frac{A f_y - R_{cc}}{2} \quad \text{(Eq. 13.4.5 (5))}$$

(c) **Case 3 –** Partial composite action (partial shear connection), that is:
$\phi_{sc} R_{ss} <$ least of ($0.85 f'_c b_{ec} t$ or $A f_y$)

$$M_{rc} = R_{sc} e + R_{cc} e' \quad \text{(Eq. 13.4.5.(6))}$$

$$R_{cc} = \phi_{sc} R_{ss} \quad \text{(Eq. 13.4.5 (7))}$$

$$R_{sc} = \frac{A f_y - R_{cc}}{2} \quad \text{(Eq. 13.4.5(8))}$$

where

e' = the lever arm and is computed using

$$a_c = \frac{R_{cc}}{0.85 f_c b_{ec}} \quad \text{(Eq. 13.4.5 (9))}$$

(d) In equations 13.4.5 (1) to 13.4.5 (9)

a_c = depth of equivalent concrete rectangular stress block
e = lever arm from centroid of steel area in compression block to centroid of steel area in tension
e' = lever arm from centroid of concrete compression block to centroid of steel area in tension
R_{ss} = nominal capacity of all shear connectors between points of maximum and adjacent zero moment
R_{ss} = $\sum q_r$
R_{tc} = internal tension force from area of steel section in tension
R_{cc} = internal compression force from area of concrete in compression
R_{sc} = internal compression force from area of steel section in compression
b_{ec} = concrete slab effective width from 13.4.2
t = slab effective thickness from 13.1.2.5
A = area of steel section
f_y = yield stress of steel section
ϕ_{sc} = strength reduction factor for shear connectors (see table 13.1.2(1)).

13.4.6 Limits of use of partial composite action

13.4.6.1

No composite action shall be assumed in computing moment capacity for the ultimate limit state when R_{ss} is less than 0.5 times the lesser of 0.85 f'_c b_{ec} t and Af_y. No composite action shall be assumed in computing deflections for the serviceability limit state when R_{ss} is less than 0.25 times the lesser of 0.85 f'_c b_{ec} t and Af_y.

13.4.6.2

Clause 13.1.2.4 limits the application of partial composite action to members not required to form yielding regions (see 1.3) under design loads or effects including earthquake loads.

13.4.7 Required shear connector strength

For full composite action (full shear connection) the design horizontal shear (R_h^*), at the junction of the steel section and the concrete slab or steel deck to be resisted by shear connectors distributed between the point of maximum bending moment and each adjacent point of zero moment shall be:

$$R_h^* = Af_y \quad \text{for Case 1 as defined in 13.4.5.2(a);} \quad \text{(Eq. 13.4.7(1))}$$

$$R_h^* = 0.85 f'_c b_{ec} t \quad \text{for Case 2, as defined in 13.4.5.2 (b)} \quad \text{(Eq. 13.4.7(2))}$$

➤

For Case 1 and Case 2;

$$\phi_{sc} R_{ss} \geq R_h^* \text{ is required} \qquad \text{(Eq. 13.4.7 (3))}$$

For partial composite action the total horizontal shear, as defined in 13.4.5.2 (c), shall be:

$$R_h^* = \phi_{sc} R_{ss} \qquad \text{(Eq. 13.4.7 (4))}$$

In equations 13.4.7 (3) and 13.4.7 (4), ϕ_{sc} is specified in table 13.1.2 (1).

13.4.8 Calculation of the negative moment capacity of the composite section

13.4.8.1 General

Composite beams may be designed as continuous members. The design moment capacity of the composite section, with the concrete slab in the tension area of the composite section, shall be the design moment capacity of the steel section alone, except that when sufficient shear connectors are placed in the negative moment region, concrete slab reinforcement which is parallel to the steel section, within the design effective width of the concrete slab and anchored in accordance with NZS 3101 may be included in computing the properties of the composite section. The total horizontal shear R_h^* to be resisted by shear connectors between the point of maximum negative bending moment and the adjacent point of zero moment shall be taken as $A_{rs} f_{yr}$,

where

A_{rs} = area of slab reinforcement within effective width of slab
f_{yr} = yield stress of tension reinforcement.

13.4.8.2 Calculation of design moment capacity

The design negative moment capacity shall be calculated from the plastic stress distribution in the composite or bare steel section as appropriate, except for category 3 members used in accordance with 13.4.3.2 (a) (i), where the design negative moment capacity shall be calculated from an elastic stress distribution (in the bare steel section).

13.4.9 Spacing of shear connectors

13.4.9.1

For full composite action, the number of shear connectors to be located on each side of the point of maximum design bending moment (positive or negative, as applicable) and distributed between that point and the adjacent point of zero moment shall be not less than:

$$n = \frac{R_h^*}{\phi_{sc}\, q_r} \qquad \text{(Eq. 13.4.9 (1))}$$

13.4.9.2

Shear connectors for full or partial composite action may be spaced uniformly, except that in a region of positive moment, the number of shear connectors required between any concentrated load applied in that region and the nearest point of zero moment shall be as determined by an appropriate limit state design procedure or alternatively shall be not less than:

$$n' = n \left[\frac{M^*_{m1} - \phi M_s}{M^*_m - \phi M_s} \right] \quad \text{(Eq. 13.4.9 (2))}$$

where

M^*_m = maximum calculated design positive bending moment within the region
M^*_{m1} = calculated positive bending moment at a concentrated load point
M_s = nominal section moment capacity of the steel section alone
ϕ = strength reduction factor for bending from table 3.3
n = as calculated from equation 13.4.9 (1)
n' = number of shear connectors required between a concentrated load and the nearest point of zero moment.

13.4.9.3
To determine when equation 13.4.9(2) needs to be applied, divide the length of positive moment region into thirds. Equation 13.4.9(2) shall be applied when:

(1) For the middle third, $F^*_{con} \geq 0.2\Sigma F^*_{uni}$

(2) For either of the two end thirds, $F^*_{con} \geq 0.1\Sigma F^*_{uni}$

where

F^*_{con} = magnitude of the largest single concentrated load applied within the part length under consideration

ΣF^*_{uni} = magnitude of all uniform load(s) applied within the total length of positive moment region.

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13.4.10 Longitudinal shear and post-cracking stud strength

13.4.10.1
The design longitudinal shear (V^*_l) in the slab of composite beams with solid slabs, or with cover slabs and steel deck parallel to the beam, shall be taken as:

$$V^*_l = \sum \phi_{sc} q_r - 0.85\, \phi_c f'_c A_{cs} - \phi A_{rl} f_{yr}$$

where

ϕ_{sc} = strength reduction factor for shear connectors from table 13.1.2(1)
ϕ_c = strength reduction factor for concrete under longitudinal shear from table 13.1.2(1)
ϕ = strength reduction factor for reinforcement under longitudinal shear from table 13.1.2(1)
A_{cs} = concrete cross-sectional area under compression between shear planes
A_{rl} = area of longitudinal reinforcement within the concrete area A_{cs}
q_r = nominal shear capacity of shear connectors from 13.3.2.1.

13.4.10.2
The design shear resistance for normal density concrete along any potential longitudinal shear surfaces in the concrete slab is given by:

$$V_r = (0.80 \; \phi A_{rt} f_{yr} + 2.76 \; \phi_c A_{cv}) \leq 0.50 \; \phi_c f'_c A_{cv}$$

where

A_{rt} = area of transverse reinforcement crossing shear planes
A_{cv} = area of concrete in shear planes.

13.4.10.3
To suppress longitudinal shear failure, $V_l^* \leq V_r$ is required.

Amd 2 Oct. '07 Transverse reinforcement required to suppress longitudinal shear failure shall be placed across the relevant failure plane, anchored so as to develop the yield strength of the reinforcement and shall be uniformly spaced along the composite beam span.

13.4.10.4
Where the side cover to the stud $c_{do} < 10d_{sc}$ then, to ensure shear stud strength is maintained in the event of cracking along the line of the shear studs, transverse reinforcement, A_{rt} shall be provided as follows:

$$A_{rt} \geq 430 \frac{d_{sc}^2}{s_{sc}} \frac{M^*}{\phi M_{rc}} \text{ (mm}^2\text{/metre length)}$$

where:

d_{sc} = diameter of shear stud (mm)

s_{sc} = average spacing along the line of the studs (mm)

c_{do} = the side cover to the stud, measured at the base of the stud

M^* = the maximum design moment along the line of shear studs (kNm)

ϕM_{rc} = the design moment capacity of the composite section (kNm)

This transverse reinforcement shall comply with the following:

(a) When placed as bars, the bar diameter shall not exceed 16 mm.

(b) In all beams except internal primary beams, this transverse reinforcement shall be placed within a vertical height from the base of the stud not greater than $2.5d_{sc}$.

(c) For internal primary beams, transverse reinforcement placed within the top 50 mm of the slab, including that for transverse flexural reinforcement, shall be considered contributing to A_{rt}.

(d) For edge beams, vertical hooked bars, where both the curve of the hook and the free end of the hook each extend more than $4d_{sc}$ mm past the centreline of the studs, shall be included. (Note that the bars may be inclined as required to fit within the depth of concrete available.)

(e) Decking that is transverse to the beam and continuous over the beam for at least 500 mm each side shall be considered contributing to A_{rt} .

(f) The development length for all bar or mesh reinforcement shall be as determined from NZS 3101 calculated for a reinforcement stress of $0.6f_{yr}$.

Appropriately placed reinforcement may contribute to both longitudinal shear and post-cracking resistance.

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13.4.11 Special seismic requirements for inelastic action

13.4.11.1

The total horizontal shear, R_h^*, to be resisted within a length of category 1 or category 2 composite beam containing a yielding region shall be based on the strain hardened capacity ($\phi_{os} R_u$) of any steel yielding in tension. The strain-hardening factor (ϕ_{os}) of steel beams for the appropriate category is given in 12.2.8.

13.4.11.2

The contribution of the concrete in the slab to the resistance of longitudinal splitting in a composite beam section, as covered by 13.2.2.3, shall be neglected within the yielding regions of category 1 and 2 beams.

13.4.11.3 Category 1 or 2 composite beam design requirements

13.4.11.3.1 *General*

Clause 13.4.11.3 contains specific performance provisions for ensuring that dependable inelastic flexural action from yielding regions of category 1 or 2 composite beams can be achieved at the following different locations along the beam span.

13.4.11.3.2 *Positive moment region within the span*

The properties of the composite section shall be such that the steel beam can achieve a maximum tensile strain of 10 times yield strain for category 2 plastic hinges and 24 times yield strain for category 1 plastic hinges prior to the ultimate compressive strength of the concrete slab being obtained and a maximum concrete strain of 0.003 being exceeded.

13.4.11.3.3 *Positive moment region at a support*

(a) Composite action may be curtailed at a distance of 1.5d away from the face of the support, where d = depth of steel section, and the design of the positive moment region adjacent to the support based on the design of the steel beam alone; or

(b) The area of concrete slab at the support equal to the effective slab width times the effective slab depth shall be reinforced with longitudinal compression reinforcement and confined to the requirements of NZS 3101, such that the steel beam can achieve a maximum tensile strain of 10 times yield strain for category 2 plastic hinges and 24 times yield strain for category 1 plastic hinges, prior to developing the nominal compression capacity of the concrete and compression reinforcement. The maximum compression concrete strain reached in the confined concrete region at a development of these tensile strains in the steel shall not exceed 0.004.

13.4.11.3.4 *Negative moment region at a support*

(a) Composite action may be curtailed in the negative moment region adjacent to a support and the design of the negative moment region based on the design of the steel beam alone; or

(b) The beam shall be designed as a fully composite beam in accordance with 13.4.8, provided slab reinforcement can be anchored to the requirements of NZS 3101, with the section geometry and lateral restraint to the steel beam complying with the requirements of 13.4.3 for section geometry and 12.6, 13.4.4 for lateral restraint, for the appropriate category of beam.

13.5 DESIGN OF COMPOSITE BEAMS WITHOUT SHEAR CONNECTORS

13.5.1

Unpainted steel sections supporting concrete slabs and encased in concrete in accordance with 13.3.1 may be proportioned on the basis that the composite section supports the total load.

13.5.2
The properties of the composite section for determination of member design capacity shall be computed by a rational design method applicable to the ultimate limit state, neglecting any area of concrete in tension.

13.5.3
The effective width of any slab cast monolithic with the beam shall be determined from 13.4.2.

13.5.4
Design of stirrups and any additional longitudinal reinforcement shall be to the requirements of NZS 3101:2006 clauses 9.3 and 9.4.

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13.6 SHEAR STRENGTH OF COMPOSITE BEAMS

13.6.1 Full web members

13.6.1.1
The web area of steel sections shall be proportioned to carry the total vertical shear in accordance with 5.11 and also with 12.7.2 where appropriate.

13.6.1.2
For encased members the contribution of the encasement may be included in calculating the design shear capacity provided the stirrups, ties and longitudinal steel are specified in accordance with NZS 3101.

13.6.2 Open web members
Alternatively to 13.6.1, for encased open web and battened members the member may be designed as a reinforced concrete member for shear in accordance with NZS 3101.

13.7 END CONNECTIONS TO COMPOSITE BEAMS

13.7.1 Design
End connections of steel sections shall be proportioned to transmit the total end reaction of the composite beam, and shall be designed to the requirements of 12.9 when subject to earthquake loads. Where concrete encasement is confined to the requirements of NZS 3101, allowance may be made for contribution from the concrete. For this purpose, the proportion of design load in the structural steel beam may be assumed to be transmitted to the steel column member and that in the concrete beam section transmitted separately to the concrete part of the column member.

13.8 DESIGN OF COMPOSITE COLUMNS

13.8.1 Scope
Clause 13.8 covers the design of columns for full composite action only.

13.8.2 Design of encased composite columns
Clause 13.8.2 relates to design of composite columns consisting of laterally tied concrete surrounding a structural steel core.

13.8.2.1 General

13.8.2.1.1
Calculation of section design capacity shall be in accordance with the requirements of NZS 3101. The section capacity shall equal or exceed the design actions determined from 13.8.2.2.1.

13.8.2.1.2
Any design axial force assigned directly to the concrete of the encased composite column from an incoming member at the point of connection shall be transferred to that concrete by direct bearing in accordance with NZS 3101.

13.8.2.2 Consideration of second-order effects/determination of effective length

13.8.2.2.1
For encased composite columns with geometrical slenderness ratios $(k_e L/r)_{cc} \leq 100$, second-order effects shall be determined in accordance with 4.4.2, 4.5.2 or 4.6.4 as appropriate to the method of structural analysis used, with the flexural stiffness of the encased composite column section $(EI)_{cc}$ from equation 13.8.2.2(1) replacing (EI) in 4.8.

$$(EI)_{cc} = \frac{(E_c I_g / 5) + EI_s}{1 + \beta_d} \geq (EI_s) \qquad \text{(Eq. 13.8.2.2(1))}$$

where

E_c = short-term elastic modulus of concrete
I_g = second moment of area of gross concrete section about principal axis of composite section
E = modulus of elasticity of steel
I_s = second moment of area of the steel member alone about its principal axis
β_d = ratio of maximum design moment from design dead load (G^*) alone to maximum design moment from full design load, taken as positive.

For encased composite columns with geometrical slenderness ratios $(k_e L/r)_{cc} > 100$, evaluation of second-order effects shall be by special study.

13.8.2.2.2
Effective lengths shall be determined in accordance with 4.8.3.1, substituting $(EI)_{cc}$ for I in equation 4.8.3.4 for composite members and substituting (EI) for I in equation 4.8.3.4 for bare steel members.

13.8.2.2.3
For calculation of the geometrical slenderness ratio, in the determination of second-order effects, the radius of gyration of the composite column (r_{cc}) shall be given by:

$$r_{cc} = \sqrt{\left[\frac{(E_c I_g / 5) + EI_s}{(E_c A_g / 5) + EA_s}\right]} \qquad \text{(Eq. 13.8.2.2(2))}$$

where

A_s = area of encased steel member section
A_g = area of gross concrete section.

13.8.2.2.4
The calculation of section properties to use in determining the lateral deflection of sway systems shall be in accordance with Appendix N.

13.8.2.3 General longitudinal and transverse reinforcement requirements

(a) In columns not designed for design loads or effects including earthquake loads the longitudinal and transverse reinforcement shall be designed in accordance with 13.8.2.4 herein when the design moment is equal to or less than 75 % of the design capacity of the section, as determined using a strength reduction factor $\phi = 0.85$. In regions where the design moment exceeds 75 % of the design capacity of the section for $\phi = 0.85$, the longitudinal and transverse reinforcement shall be designed in accordance with 10.3.10.5 or 10.3.10.6 of NZS 3101:2006 as appropriate, except that the centre-to-centre spacing across the cross section between cross-linked bars, or between a cross-linked bar and the zone of influence, as defined below, shall not exceed 200 mm. Amd 2 Oct. '07

(b) In columns designed for design loads or effects including earthquake loads, the longitudinal and transverse reinforcement shall be designed in accordance with in accordance with NZS 3101:2006 clauses 10.4.6 and 10.4.7, except that the minimum reinforcement requirements of 13.8.2.4 herein shall also apply at any point along the column length. Amd 2 Oct. '07

(c) When applying NZS 3101:2006 clauses 10.4.7.5.1 or 10.4.7.5.2 to composite columns, longitudinal reinforcement within the zone of influence shown by figure 13.8.2.3 need not be cross-linked. Amd 2 Oct. '07

Subclause (4)

Centre to centre spacing across the cross section between cross-linked bars shall not exceed 200 mm, except that bars situated within the zone of influence of the steel core need not be cross-linked.

The zone of influence is that region of concrete contained within the rectangular profile of the core steel section (*b.d*) plus the triangular areas of concrete contained within intersecting lines drawn at 45° from the four corners of the steel section, as shown in figure 13.8.2.3.

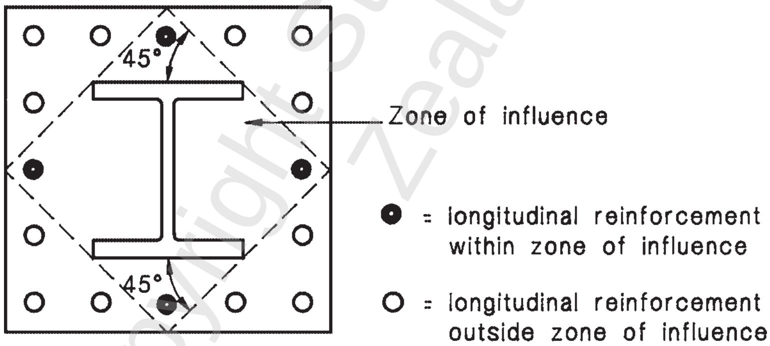


Figure 13.8.2.3 – Zone of influence of steel section in an encased composite column

Subclause (5)

Each longitudinal bar or bundle of bars shall be laterally supported by the corner of a hoop having an included angle of not more than 135° or by a supplementary cross-tie, except that the following 3 cases of bars are exempt from this requirement.

(i) Bars or bundles of bars which lie between 2 laterally supported bundles of bars supported by the same hoop where the distance between the laterally supported bars or bundles of bars does not exceed 200 mm between centres.

(ii) Bars situated within the zone of influence of the steel core as defined in subclause (4) above.

(iii) Inner layers of reinforcing bars within the concrete core centred more than 75 mm from the inner face of hoops.

13.8.2.4 Minimum longitudinal and transverse reinforcement requirements

13.8.2.4.1
An encased composite section incorporating concrete reinforced with spiral or circular hoop transverse reinforcement shall conform to the following:

Amd 2 Oct. '07 (a) Specified concrete cylinder compression strength, f'_c, shall be not less than 20 MPa;

(b) Design yield stress of the structural steel core shall be as specified by 2.1.1;

Amd 2 Oct. '07 (c) Spiral or circular hoop reinforcement shall conform with 10.3.10.5 of NZS 3101:2006;

(d) Longitudinal bars located within the spiral or circular hoop reinforcement shall be not less than 0.008 nor greater than 0.08 times the net area of the concrete section;

(e) Longitudinal bars located within the spiral or circular hoop reinforcement may be considered in computing A_s and I_s.

13.8.2.4.2
An encased composite column incorporating concrete transversely reinforced with lateral ties shall conform to the following:

Amd 2 Oct. '07 (a) Specified concrete cylinder compression strength, f'_c, shall be not less than 20 MPa;

(b) Design yield stress of the structural steel core shall be as specified by 2.1.1;

(c) Lateral ties shall extend completely around the structural steel core;

(d) Lateral ties shall be at least 16 mm diameter bars, or smaller bars with a diameter not less than 1/50 times the greater side dimension of the composite member, but not smaller than 10 mm diameter. Welded wire fabric of equivalent area may be used;

(e) Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 tie bar diameters, or one half times the least side dimension of the composite member;

(f) Each lateral tie shall be anchored by at least a 135^{o} stirrup hook;

(g) Longitudinal bars located within the ties shall not be less than 0.008 nor more than 0.08 times the net area of concrete section;

(h) Hoops or cross-ties shall be arranged so that lateral support from a corner of a hoop with an included angle of not more than 135^{o} or from a cross-tie is provided to:

(i) Every corner longitudinal bar; and

(ii) Alternate other longitudinal bars which are not within the zone of influence (see figure 13.8.2.3) and which have a clear transverse spacing between adjacent bars of not more than 150 mm; or

(iii) All other longitudinal bars which are not within the zone of influence (see figure 13.8.2.3) and which have a clear transverse spacing between adjacent bars greater than 150 mm.

(j) A longitudinal bar shall be located at every corner of a rectangular cross section, with other longitudinal bars spaced not further apart than one-half the least side dimension of the composite member;

(k) Longitudinal bars located within the ties may be considered in computing A_s and I_s.

13.8.2.5 Splices in composite columns

(a) Splices between the core steel members shall be designed to transmit the design actions applicable to the steel members . The minimum such design actions shall be determined from:

(i) Clause 9.1.4.1 (iv) or (v), as appropriate, for members designed for load combinations which do not include earthquake loads.

(ii) The tension or compression force requirements, as appropriate, of 12.9.2.2 and 12.9.2.3 for members designed for load combinations which include earthquake loads.

(b) Design of the reinforced concrete region of the splice shall be in accordance with the requirements of NZS 3101 for the splice forces in excess of (a) above.

13.8.3 Design of concrete-filled structural hollow sections

13.8.3.1 Scope

Clause 13.8.3 relates to design of composite columns consisting of structural hollow sections, either as-rolled or fabricated and filled with unreinforced or reinforced structural concrete with a specified 28 day compressive cylinder strength of not less than 17.5 MPa.

13.8.3.2 Section geometry requirements

For fully composite concrete-filled structural hollow sections subject to all load combinations, including any category of member for design loads or effects including earthquake loads, the section geometry requirements applicable to the section once the concrete has achieved its specified 28 day cylinder compression strength are as follows:

(a) *For rectangular members*
The section slenderness parameter of any plate element comprising the section, calculated in accordance with 5.2.2 for flat compression plate elements, shall not exceed the plate element slenderness limit for deformation given in table 5.2 for a flat plate under uniform compression with both edges supported (λ_e = 90).

(b) *For circular members*
The section slenderness parameter of the member, calculated in accordance with 5.2.2.4, shall not exceed the circular hollow section limit for yield (λ_e = 120) given in table 5.2.

The section geometry requirements shall also comply with any requirement from section 5 or 6 of the member to resist, as a bare steel member, design actions arising prior to the concrete achieving its specified 28 day cylinder compression strength.

13.8.3.3 Design procedure

13.8.3.3.1

The calculation of design capacity shall be in accordance with NZS 3101 or other appropriate limit state design procedure.

13.8.3.3.2

If the design is undertaken to a procedure other than that specified in NZS 3101, the factor of safety (reliability) against failure shall be consistent with that required for ultimate limit state design using the load factors and/or strength reduction factors applicable to the particular design procedure.

Amd 2 Oct. '07

13.8.3.4 Consideration of second-order effects/determination of effective length

13.8.3.4.1

Determination of effective length and consideration of second-order effects shall be in accordance with an appropriate limit state design procedure.

13.8.3.4.2

This may involve using the provisions of section 4 of this Standard, with the transformed stiffness $(EI)_{cc}$ of the composite column from equation 13.8.2.2(1) replacing the bare steel stiffness I_c.

13.8.3.5 Load transfer at connections to steel shell only

13.8.3.5.1

Shear connectors shall be provided if the design shear stress at a steel/concrete interface, due to ultimate limit state design loads, exceeds 0.7 MPa. The area over which load transfer takes place shall be determined from the connection detail.

13.8.3.5.2

The shear connectors, if required, shall be designed to transmit any design shear generated in excess of that transferred by direct contact at the maximum shear stress of 0.7 MPa.

13.8.3.5.3

Where concrete-filled columns are designed to achieve a period of structural adequacy in fire conditions without external fire protection being applied, connection details shall provide direct mechanical load transfer of all the design shear into the concrete through shear connectors or other appropriate means.

NOTES

14 FABRICATION

14.1 REJECTION OF A FABRICATED ITEM

14.1.1
A fabricated item shall be liable to rejection if:

(a) The material does not satisfy the requirements of 14.2; or

(b) The fabrication does not satisfy the requirements of 14.3; or

(c) It does not satisfy the tolerances specified in 14.4.

The fabricated item may be accepted nonetheless if:

(d) It can be demonstrated to the approval of the design engineer that the structural adequacy and intended use of the item are not impaired thereby; or

(e) It passes testing in accordance with the appropriate clauses of section 17.

14.1.2
Fabricated items which do not satisfy either 14.1.1 (d) or (e) and which do not satisfy either 14.2, 14.3 or 14.4 shall be rejected.

14.2 MATERIAL

14.2.1 General

14.2.1.1
All material shall satisfy the requirements of the appropriate material Standard specified in 2.2, 2.3, 2.4 and 2.5.

14.2.1.2
Where required by 12.4.1.1, the actual yield stress of steel, as recorded on the certified mill test report or test certificate, shall not exceed the maximum value specified by table 12.4.

14.2.1.3
Surface defects in the steel shall be removed using the methods specified in the appropriate Standards listed in 2.2.1.

14.2.2 Identification
The steel grade shall be identifiable at all stages of fabrication, or the steel shall be classed as unidentified steel and only used in accordance with 2.2.3. Any marking of steelwork shall be such as to not damage the material.

14.3 FABRICATION PROCEDURES

14.3.1 Methods

14.3.1.1
All material shall be straightened or formed to the specified configuration by methods that will not reduce the properties of the material below the values used in design. Steel may be bent or pressed to the required shape by either hot or cold processes.

14.3.1.2
Local application of heat or mechanical means may be used to introduce or correct camber, sweep and out-of-straight. The temperature of heated areas shall not exceed 650 °C.

14.3.2 Full contact splices

14.3.2.1
Full contact splices may be produced by cold saw cutting or machining.

14.3.2.2
The surface of such splices shall be such that, when the ends of the 2 members are abutted, the alignment of the members and the gap shall be within the tolerances specified in 14.4.4.2.

14.3.3 Cutting

14.3.3.1
Cutting may be by sawing, shearing, cropping, machining and thermal cutting processes, as appropriate.

14.3.3.2
Shearing of items over 16 mm thick shall not be carried out when the item is to be galvanized and subject to tensile force or bending moment unless the item is stress relieved subsequently.

14.3.3.3
Shearing of items of any thickness in designated yielding regions of category 1, 2 or 3 members is only permitted in accordance with 12.14.1.

14.3.3.4
Any cut surface not incorporated in a weld shall have a roughness not greater than the appropriate value given in table 14.3.3. A cut surface to be incorporated in a weld shall comply with AS/NZS 1554.1.

Table 14.3.3 – Maximum cut surface roughness

Application	Maximum roughness (CLA) μm
Normal applications, i.e. where the face and edges remain as-cut or with minor dressing (see Note 1)	25
Fatigue applications (Detail categories as defined in 10.5)	
detail category ≥ 80 MPa	12
detail category < 80 MPa	25
Yielding regions of category 1, 2 or 3 members (12.14.1)	12

NOTE –

(1) Roughness values may be estimated by comparison with surface replicas, such as the WTIA Flame Cut Surface replicas.

(2) Suitable techniques of flame cutting are given in WTIA Technical Note 5.

(3) CLA ≡ Centre Line Average Method (see AS 2382).

14.3.3.5
Cut surface roughness exceeding these values shall be repaired by grinding to give a value less than the specified roughness. Grinding marks shall be parallel to the direction of the cut.

14.3.3.6
Notches and gouges, not closer spaced than 20t (where t = component thickness) and not exceeding 1 % of the total surface area on an otherwise satisfactory surface, are acceptable provided that imperfections greater than t/5 but not exceeding 2 mm in depth are removed by machining or grinding. Imperfections outside the above limits shall be repaired by welding in accordance with AS/NZS 1554.1.

Amd 1 June '01

14.3.3.7
A re-entrant corner shall be shaped notch free to a radius of at least 10 mm.

14.3.4 Welding
Welding shall comply with AS/NZS 1554.1 or AS/NZS 1554.5 as appropriate (see 10.1.5) and welding of studs shall comply with AS 1554.2.

14.3.5 Holing

14.3.5.1 General

14.3.5.1.1
A round hole for a bolt shall either be machine flame cut, or drilled full size, or subpunched 3 mm undersize and reamed to size, or punched full size (subject in this latter case to the provisions of 12.9.4.5.3).

14.3.5.1.2
A slotted hole shall be either machine flame cut, or punched in one operation, or formed by drilling 2 adjacent holes and completed by machine flame cutting.

14.3.5.1.3
Hand flame cutting of a bolt hole shall not be permitted except as a site rectification measure for holes in column base plates.

14.3.5.1.4
A punched hole shall only be permitted in material whose yield stress (f_y) does not exceed 360 MPa and whose thickness does not exceed (5600/f_y) mm. For punched holes in applications subject to fatigue assessment refer to 10.9.

14.3.5.2 Hole size

14.3.5.2.1 *Nominal sized holes*
The nominal diameter of a completed hole other than a hole in a base plate shall be 2 mm larger than the nominal bolt diameter for a bolt not exceeding 24 mm in diameter, and not more than 3 mm larger for a bolt of greater diameter.

14.3.5.2.2 *Holes in base plates*
For a hole in a base plate, the hole diameter shall be not more than 6 mm greater than the anchor bolt diameter. A special plate washer with minimum thickness of 4 mm shall be used under the nut if the hole diameter is 3 mm or more larger than the bolt diameter.

14.3.5.2.3 *Oversize or slotted holes*

An oversize or slotted hole shall be permitted, provided that the following requirements are satisfied:

(a) An oversize hole shall not exceed $1.25d_f$ or $(d_f + 8)$ mm in diameter, whichever is the greater, where d_f is the nominal bolt diameter, in millimetres;

(b) A short slotted hole shall not exceed the appropriate hole size of this clause in width and $1.33d_f$ or $(d_f + 10)$ mm in length, whichever is the greater;

Amd 1 June '01 (c) A long slotted hole shall not exceed the appropriate hole size of this clause in width and $2.5d_f$ in length.

14.3.5.2.4 *Limitations on use of oversize or slotted holes*

The use of an oversize or slotted hole shall be limited so that the following requirements are satisfied:

(a) *Oversize hole*

An oversize hole may be used in any or all plies of bearing-type and friction-type connections, provided hardened or plate washers are installed over the oversize hole under both the bolt head and the nut;

(b) *Short slotted hole*

A short slotted hole may be used in any or all plies of a friction-type or a bearing-type connection, provided hardened or plate washers are installed over the holes under both the bolt head and the nut.

In a friction-type connection subject to a shear force, a short slotted hole may be used without regard to the direction of loading.

In a bearing-type connection subject to a shear force, a short slotted hole may be used only where the connection is not eccentrically loaded and the bolt can bear uniformly, and where the slot is normal to the direction of the design action.

(c) *Long slotted hole*

A long slotted hole may be used only in alternate plies of either a friction-type or bearing-type connection, provided a plate washer not less than 8 mm thick is used to completely cover any long slotted hole under both the bolt head and the nut.

In a friction-type connection subject to a shear force, a long slotted hole may be used without regard to direction of loading. In a bearing-type connection subject to a shear force, a long slotted hole may be used only where the connection is not eccentrically loaded and where the bolt can bear uniformly, and where the slot is normal to the direction of the load.

14.3.6 Bolting

14.3.6.1 General

14.3.6.1.1

All bolts and associated nuts and washers shall comply with the appropriate bolt material Standard specified in 2.3.1. All material within the grip of the bolt shall be steel and no compressible material shall be permitted in the grip.

14.3.6.1.2
The length of a bolt shall comply with (a) and either (b) or (c) as required:

(a) For all bolts, at least one clear thread shall show above the nut after tightening.

(b) For snug tightened bolts to 15.2.5.2(a), at least one clear thread run out shall be clear beneath the nut after tightening.

(c) For tensioned bolts to either 15.2.5.2 or 15.2.5.3, the minimum number of clear threads run out beneath the nut after tightening shall be:

(i) Five threads for a bolt length (see Table 15.2.5.2) up to and including 4 diameters;

(ii) Seven threads for a bolt length over 4 diameters but not exceeding 8 diameters;

Amd 2 Oct. '07 (iii) Ten threads for a bolt length over 8 diameters.

14.3.6.1.3
One washer or equivalent approved by the design engineer shall be provided under the rotated part.

14.3.6.1.4
Where the slope of the surfaces of parts in contact with the bolt head or nut exceeds 1: 20 with respect to a plane normal to the bolt axis, a suitably tapered washer shall be provided against the tapered surface and the non-rotating part shall be placed against the tapered washer.

14.3.6.1.5
The nuts used in a connection subject to vibration shall be secured to prevent loosening. (See 9.1.6.4).

14.3.6.2 Tensioned bolt
A tensioned high strength bolt when installed during fabrication shall be installed in accordance with 15.2.4 and 15.2.5. The contact surfaces of a joint using a tensioned bolt shall be prepared in accordance with 14.3.6.3.

14.3.6.3 Preparation of surfaces in contact
Preparation of surfaces in contact shall be as follows:

(a) *General*
All oil, dirt, loose scale, loose rust, burrs, fins and any other defects on the surfaces of contact which will prevent solid seating of the parts in the snug-tight condition shall be removed.

NOTE –

(1) If cleaning is necessary to meet these requirements, reference should be made to AS 1627.2 and AS 1627.7.

(2) A clean 'as-rolled' surface with tight mill scale is acceptable without further cleaning.

(3) Snug-tight is defined in 15.2.5.2.

(b) *Friction-type connection*
For a friction-type connection, the contact surfaces shall be clean 'as-rolled' surfaces and, in addition to satisfying the provisions of Item (a), shall be free from paint, lacquer, galvanizing or other applied finish unless the applied finish has been tested in accordance with Appendix K to establish the friction coefficient (see 9.3.3.2).

In a non-coated connection, paint including any overspray shall be excluded from areas closer than one bolt diameter to any hole but not less than 25 mm from the edge of any hole and from all areas within the bolt group.

(c) *Bearing-type connection*
For a bearing-type connection, an applied finish on the contact surfaces shall be permitted.

14.3.7 Pinned connection
Pins and holes shall be finished so that the forces are distributed evenly to the joint plies.

14.4 TOLERANCES

14.4.1 General
The tolerance limits of this clause shall be satisfied after fabrication is completed and any corrosion protection has been applied. Unless otherwise specified, the tolerance on all structural dimensions shall be ±2 mm.

14.4.2 Notation
For the purpose of this clause:

a_0, a_1 = out-of-square dimensions of flanges

a_2, a_3 = diagonal dimensions of a box section

b = lesser dimension of a web panel

b_f = width of a flange

d = depth of a section

d_o = overall depth of a member including out-of-square dimensions

d_1 = clear depth between flanges ignoring fillets or welds

e = web off-centre dimension

L = member length

Δ_f = out-of-flatness of a flange plate

Δ_v = deviation from verticality of a web at a support

Δ_w = out-of-flatness of a web

14.4.3 Cross section

14.4.3.1 Tolerances of rolled sections or plates
After fabrication, the tolerances on any cross section of rolled section or a plate shall be those specified in the relevant material supply standard from 2.2.1(a), (b) or (c), in respect of depth, flange width, flange thickness, web thickness, out-of-square, and web off-centre.

14.4.3.2 Tolerances of built-up sections
For any built-up section, the deviations from the specified dimensions of the cross section shall not exceed the following:

(a) Depth of a section (d) (see figure 14.4.3.1)

for $d \le 900$,	$\pm$ 3 mm
for $900 < d \le 1800$,	$\pm \left[3 + \dfrac{(d - 900)}{300}\right]$ mm
for $d > 1800$,	+8 mm, –6 mm

(b) Width of a flange (b_f) (see figure 14.4.3.1)

for all b_f,	$\pm$ 6 mm

(c) Out-of-square of an individual flange (a_0 or a_1) (see figure 14.4.3.1)

for $b_f \le 1000$ mm,	$\pm$ 5 mm
for $b_f > 1000$ mm,	$\pm \left(\dfrac{b_f}{200}\right)$ mm

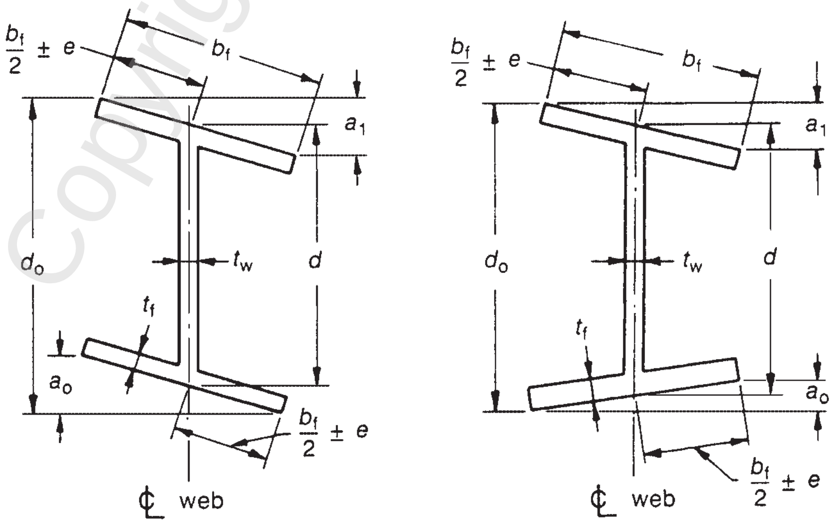
In the case of members supporting lightweight roofs

for $b_f < 210$, mm	$\pm$5 mm
for $b_f > 210$, mm	$\pm 1.5^{\circ}$ off horizontal

(d) Total out-of-square of 2 flanges ($a_0 + a_1$) (see figure 14.4.3.1)

80 % of the maximum possible combined values in (c), i.e.

for $b_f \le 1000$ mm,	$\pm$ 8 mm



NOTE –

(1) Dimensions d, d_o, a_o and a_1 are measured parallel to the centreline of the web. Dimensions b_f and ($0.5b_f \pm e$) are measured parallel to the plane of the flange.

(2) Dimension d is measured at the centreline of the web.

Figure 14.4.3.1 – Tolerances on a cross section

(e) Out-of-flatness a of web (Δ_w) (see figure 14.4.3.2)

d_1/150 mm for unstiffened web,

b/100 mm for stiffened web with intermediate stiffeners,

measured on a gauge length in the direction of d_1 or b, as appropriate.

(f) Deviation from verticality of a web at a support (Δ_v) (see figure 14.4.3.2)

for $d \leq 900$ mm, ± 3 mm

for $d > 900$ mm, $\pm \left(\frac{d}{300}\right)$ mm

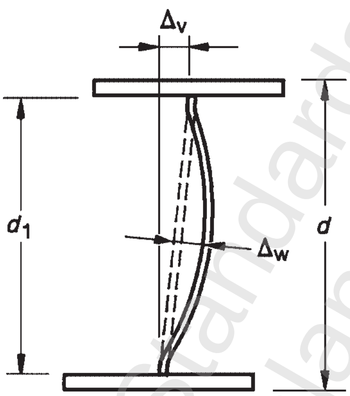


Figure 14.4.3.2 – Tolerances on a web

(g) Tolerance on shape of a built-up box section (see figure 14.4.3.3).

A built-up box section shall not deviate at the diaphragm from the prescribed shape by more than ±5 mm or $\pm[(a_2 + a_3)/400]$ mm, whichever is greater, unless connection requirements necessitate more stringent tolerances.

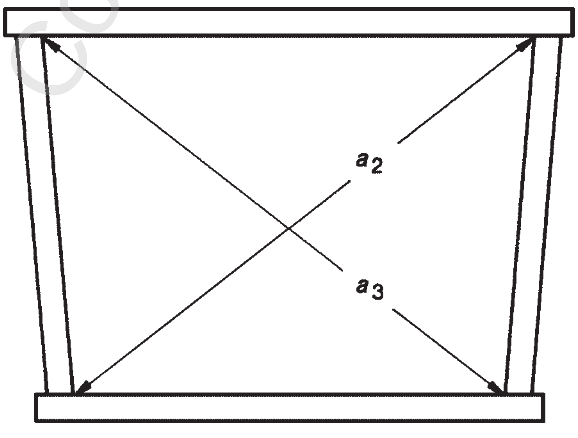


Figure 14.4.3.3 – Tolerance on shape of a box section

(h) Off-centre of a web (e) (see figure 14.4.3.4) ±6 mm

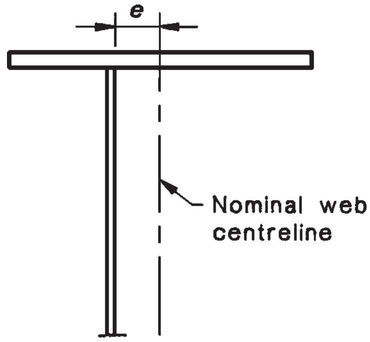


Figure 14.4.3.4 – Tolerance on off-centre of a web

(j) Out-of-flatness of a flange (Δ_f) (see figure 14.4.3.5)

In general application

for $b_f \leq 450$ mm	$\pm \left(\frac{b_f}{150}\right)$ mm
for $b_f > 450$ mm	± 3 mm

In the case of members supporting lightweight roofs

for $b_f < 210$ mm	± 5 mm
for $b_f > 210$ mm	± 1.5° off horizontal

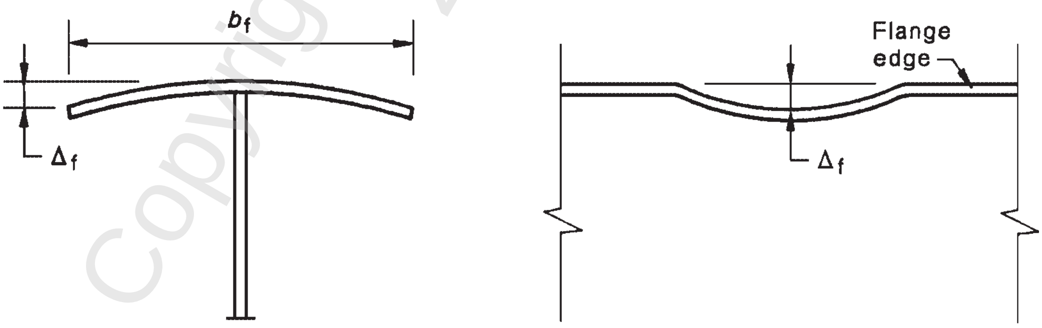


Figure 14.4.3.5 – Tolerance on out-of-flatness of a flange

14.4.4 Compression member

14.4.4.1 Straightness

A member shall not deviate about either principal axis from a straight line drawn between member end points by an amount exceeding $L/1000$ or 3 mm whichever is the greater.

14.4.4.2 Full contact splice

Amd 2 Oct. '07 If the ends of 2 butting lengths of a member, or the end of a member and the contact face of an adjoining cap plate or baseplate, are required to be in full contact, the maximum clearance between the contact surface and a straight edge shall not exceed 1 mm, and shall also not exceed 0.5 mm over at least 67 % of the contact area. The alignment of the butting surfaces shall be such that, when erected, the tolerances of 15.3.4 (d) are met.

14.4.4.3 Length

The length of a member shall not deviate from its specified length by more than ±2 mm.

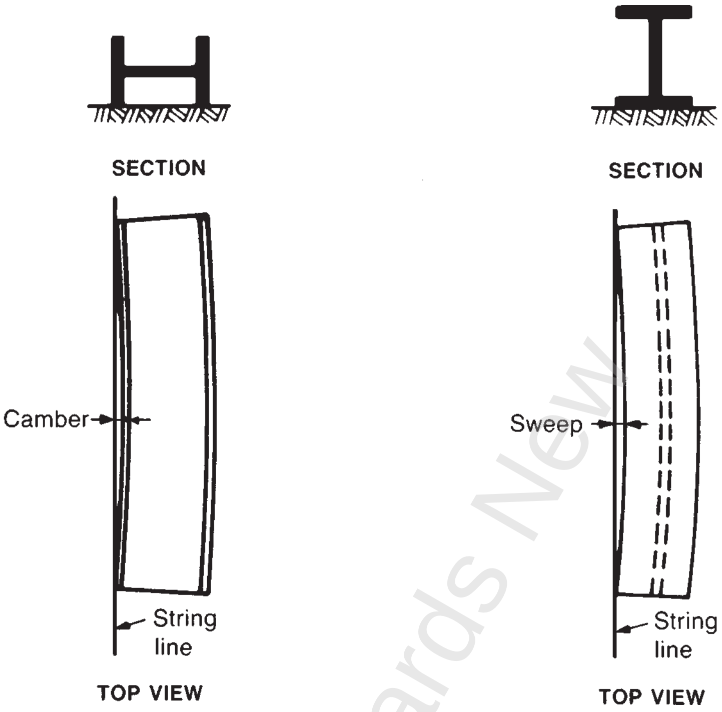
14.4.5 Beam

14.4.5.1 Straightness

A beam shall not deviate from a straight line drawn between the ends of the beam by more than the following:

(a) Camber–measured with the web horizontal on a test surface (see figure 14.4.5.1(a)). The tolerance on specified camber shall not exceed the limits given in table 14.4.5.

(b) Sweep–measured with the web vertical (see figure 14.4.5.1(b)). The sweep in plan shall not exceed the limits given in table 14.4.5.



(a) Camber (b) Sweep

Figure 14.4.5.1 – Measurement of camber and sweep

Table 14.4.5 – Tolerance on camber and sweep in beams

Nominal size	Sweep	Camber
I-sections with a flange width less than 150 mm	$\frac{\text{Length}}{500}$	$\frac{\text{Length}}{1000}$
I-sections with a flange width approximately equal to the depth		
Lengths of 14 m and under	$\frac{\text{Length}}{1000}$ but not more than 10 mm	
Length over 14 m	10 mm + $\frac{\text{Length (mm)} - 14\,000}{1000}$	
All other I-sections	$\frac{\text{Length}}{1000}$	$\frac{\text{Length}}{1000}$

NOTE –

(1) For I-sections with a specified precamber, camber shall be +0, –10 mm.

Amd 1 June '01 (2) Owing to the extreme variation in the elastic flexibility of I-sections about the Y-Y axis, difficulty may be experienced in obtaining reproducible sweep measurements. Appendix C of AS/NZS 3679:Part 1 provides a means for measuring sweep.

14.4.5.2 Length

The length of a beam shall not deviate from its specified length by more than ±2 mm for lengths less than 10 m, and ±4 mm for lengths greater than 10 m.

14.4.6 Tension member

14.4.6.1 Straightness

A member shall not deviate from a straight line drawn between end points by more than $L/500$, where L is the length between end points.

14.4.6.2 Length

The length of a tension member shall not deviate from its specified length by more than ±2 mm for lengths less than 10 m, and ±4 mm for lengths greater than 10 m.

15 ERECTION

15.1 GENERAL

15.1.1 Rejection of an erected item

15.1.1.1
An erected item shall be liable for rejection if:

(a) The erection does not satisfy the requirements of 15.2; or

(b) It does not satisfy the tolerances specified in 15.3.

The erected item may be accepted nonetheless if:

(c) It can be demonstrated that the structural adequacy and intended use of the item are not impaired thereby; or

(d) It passes testing in accordance with the appropriate clauses of section 17.

15.1.1.2
Erected items which do not satisfy either 15.1.1.1 (c) or (d) and which do not satisfy either 15.2 or 15.3 shall be rejected.

15.1.1.3
Bolts, nuts and washers shall be liable to rejection if, in the erected structure, they do not comply with 14.3.6, 15.2.3, 15.2.4 and 15.2.5, unless it can be demonstrated to the approval of the design engineer that the structural adequacy and intended use of the item are not impaired thereby.

15.1.1.4
Grouting at supports which does not satisfy the requirements of 15.5 shall be rejected.

15.1.2 Safety during erection
During the erection of a structure, steelwork shall be made safe against erection loading, including loading due to erection equipment or its operation, and wind.

Amd 2 Oct. '07 NOTE – Erection practices and procedures shall comply with AS 3828.

15.1.3 Equipment support
Equipment supported on partly erected steelwork shall not induce actions in the steel greater than the design capacities permitted in this Standard.

15.1.4 Reference temperature
Dimensions shall be set out on the basis of a reference temperature of 20 °C.

15.2 ERECTION PROCEDURES

15.2.1 General

15.2.1.1
The requirements specified in 14.3 shall also be observed during the erection of the steel frame and during any modifications to the steelwork in the course of erection.

This requirement shall apply to:

(a) Full contact splices (see 14.3.2).

(b) Cutting (see 14.3.3).

(c) Welding (see 14.3.4).

(d) Holing (see 14.3.5).

(e) Bolting (see 14.3.6).

15.2.1.2
Throughout the erection of the structure, the steelwork shall be securely bolted or fastened to ensure that it can adequately withstand all loadings liable to be encountered during erection, including, where necessary, those from erection plant and its operation. Any temporary bracing or temporary restraint shall be left in position until such time as erection is sufficiently advanced as to allow its safe removal.

15.2.1.3
All connections for temporary bracing and members to be provided for erection purposes shall be made in such a manner so as not to weaken the permanent structure or to impair its serviceability. All welding of such connections and their removal shall be in accordance with AS/NZS 1554.1, subject to the modifications of 9.7.1.4 and Appendix D of this Standard.

15.2.2 Delivery, storage and handling

15.2.2.1
Members, components and fasteners shall be handled and stacked in such a way that damage is not caused to them. Means shall be provided to minimize damage to the corrosion protection on the steelwork.

15.2.2.2
All work shall be protected from damage in transit. Particular care shall be taken to stiffen free ends, prevent permanent distortion, and adequately protect all surfaces prepared for full contact splices. All bolts, nuts, washers, screws, small plates and articles generally shall be suitably packed and identified.

15.2.3 Assembly and alignment

15.2.3.1
All matching holes shall align with each other so that a gauge or drift, equal in diameter to that of the bolts, shall pass freely through the assembled contact faces at right angles to them. Drifting to align holes shall be done in a manner that will not distort the metal nor enlarge the holes.

15.2.3.2
Each part of the structure shall be aligned as soon as practicable after it has been erected. Permanent connections shall not be made between members until sufficient of the structure has been aligned, levelled, plumbed and temporarily connected to ensure that members will not be displaced during subsequent erection or alignment of the remainder of the structure.

15.2.3.3
Each bolt and nut shall be assembled with at least one washer. A washer shall be placed under the rotating component. Where the slope of the surfaces of parts in contact with the bolt head or nut exceeds 1: 20 with respect to a plane normal to the bolt axis, a suitable tapered washer shall be used against the sloping surface. The non-rotating component shall be placed against the tapered washer.

15.2.3.4
Bolting categories 4.6/*S* and 8.8/*S* shall be installed to the snug-tight condition specified in 15.2.5.2(a).

15.2.3.5
Hardened or plate washers shall be used under both the bolt head and nut for any slotted and oversize holes, as specified in 14.3.5.2.4.

15.2.4 Assembly of a connection involving tensioned bolts

15.2.4.1 Placement of a nut
The nut shall be placed so that the mark specified in AS/NZS 1252 to identify a high strength nut is visible after tightening.

15.2.4.2 Packing
Packing shall be provided wherever necessary to ensure that the load-transmitting plies are in effective contact when the connection is tightened to the snug-tight condition defined in 15.2.5.2(a). All packing shall be steel with a surface condition similar to that of the adjacent plies.

15.2.4.3 Tightening pattern

15.2.4.3.1
Snug-tightening and final tensioning of the bolts in a connection shall proceed from the stiffest part of the connection towards the free edges.

15.2.4.3.2
High strength structural bolts that are to be tensioned may be used temporally during erection to facilitate assembly, but if so used they shall not be finally tensioned until all bolts in the connection have been snug-tightened in the correct sequence.

15.2.4.4 Retensioning
Retensioning of bolts that have been fully tensioned shall not be permitted.

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15.2.5 Methods of tensioning

15.2.5.1 General

15.2.5.1.1
The method of tensioning shall be in accordance with either 15.2.5.2 or 15.2.5.3.

COMMENT: Other methods associated with the tensioning of specialist property class 8.8 bolt and nut assemblies manufactured for use in structural steel construction may be used, in accordance with the manufacturer's instructions and to the satisfaction of the construction reviewer (see 1.6.3).

15.2.5.1.2
In the completed connection, all bolts shall have at least the minimum bolt tension specified in table 15.2.5.1 when all bolts in the bolt group have been tightened.

Table 15.2.5.1 – Minimum bolt tension for property class 8.8 bolts to AS/NZS 1252

Nominal diameter of bolt	Minimum bolt tension, kN
M16	95
M20	145
M22	180
M24	210
M30	335
M36	490

NOTE – The minimum bolt tensions given in this table are approximately equivalent to the minimum proof loads given in AS/NZS 1252.

15.2.5.2 Part-turn method of tensioning
Tensioning of bolts by the part-turn method shall be in accordance with the following procedure:

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(a) On assembly, all bolts in the connection shall be first tightened to a snug-tight condition to ensure that the load-transmitting plies are brought into effective contact. This will require retightening to snug tight any bolts that become loose during the snug tightening of adjacent bolts.

Snug-tight is the tightness attained by a few impacts of an impact wrench or by the effort of a person using a standard podger spanner.

(b) After completing snug-tightening, location marks shall be established to mark the relative position of the bolt and the nut and to control the final nut rotation.

Observation of the final nut rotation may be achieved by using marked wrench sockets, but location marks shall be permanent when required for inspection.

(c) Bolts shall be finally tensioned by rotating the nut by the amount given in table 15.2.5.2. During the final tensioning, the component not turned by the wrench shall not rotate.

Table 15.2.5.2 – Nut rotation from the snug-tight condition

Bolt length (underside of head to end of bolt)	Disposition of outer face of bolted parts (see Notes 1, 2, 3 and 4)		
	Both faces normal to bolt axis	One face normal to bolt axis and other sloped	Both faces sloped
Up to and including 4 diameters	1/3 turn	1/2 turn	2/3 turn
Over 4 diameters but not exceeding 8 diameters	1/2 turn	2/3 turn	5/6 turn
Over 8 diameters but not exceeding 12 diameters (see Note 5)	2/3 turn	5/6 turn	1 turn

NOTE –

(1) Tolerance on rotation: for 1/2 turn or less, one-twelfth of a turn (30°) over and nil under tolerance; for 2/3 turn or more, one-eighth of a turn (45°) over and nil under tolerance.

(2) The bolt tension achieved with the amount of nut rotation specified in table 15.2.5.2 will be at least equal to the minimum bolt tension specified in table 15.2.5.1.

(3) Nut rotation is the rotation relative to the bolt, regardless of the component turned.

(4) Nut rotations specified are only applicable to connections in which all material within the grip of the bolt is steel.

(5) No research has been performed to establish the turn-of-nut procedure for bolt lengths exceeding 12 diameters. Therefore, the required rotation should be determined by actual test in a suitable tension measuring device which simulates conditions of solidly fitted steel.

15.2.5.3 Tensioning by use of direct-tension indication device

Tensioning of bolts using a direct-tension indication device shall be in accordance with the following procedure:

(a) The suitability of the device shall be demonstrated by calibration testing of a representative sample of not less than 3 bolts for each diameter and class of bolt in a calibration device capable of indicating bolt tension. The calibration test shall demonstrate that the device indicates a tension not less than 1.05 times the minimum bolt tension specified in table 15.2.5.1.

(b) On assembly, all bolts and nuts in the connection shall be first tightened to a snug-tight condition defined in 15.2.5.2(a).

(c) After completing snug-tightening, the bolt shall be tensioned to provide the minimum bolt tension specified in 15.2.5.1.2. This shall be indicated by the tension indication device.

NOTE – Tensioning of bolts using a direct-tension indication device should also be in accordance with the manufacturer's specification.

15.3 TOLERANCES

15.3.1 Location of anchor bolts

15.3.1.1
Anchor bolts shall be restrained in position both in a vertical and a horizontal direction during all setting-in operations.

15.3.1.2
Anchor bolts shall be set out in accordance with the erection drawings. They shall not vary from the positions shown in the erection drawings by more than the following: (See figure 15.3.1)

(a) 3 mm centre to centre of any 2 bolts within an anchor bolt group, where an anchor bolt group is defined as the set of anchor bolts which receives a single fabricated steel member;

(b) 6 mm centre-to-centre of adjacent anchor bolt groups;

(c) Maximum accumulation of 6 mm per 30 000 mm along an established column line of multiple anchor bolt groups, but not to exceed a total of 25 mm. The established column line is the actual field line most representative of the centres of the as-built anchor bolt groups along a line of columns.

(d) 6 mm from the centre of any anchor bolt group to the established column line through that group.

15.3.1.3
Anchor bolts shall be set perpendicular to the theoretical bearing surface, threads shall be protected and free of concrete and nuts shall run freely on the threads.

15.3.1.4
The projection of the end of the anchor bolt from the theoretical bearing surface shall not be more than 25 mm longer nor 5 mm shorter than that specified.

15.3.2 Column base

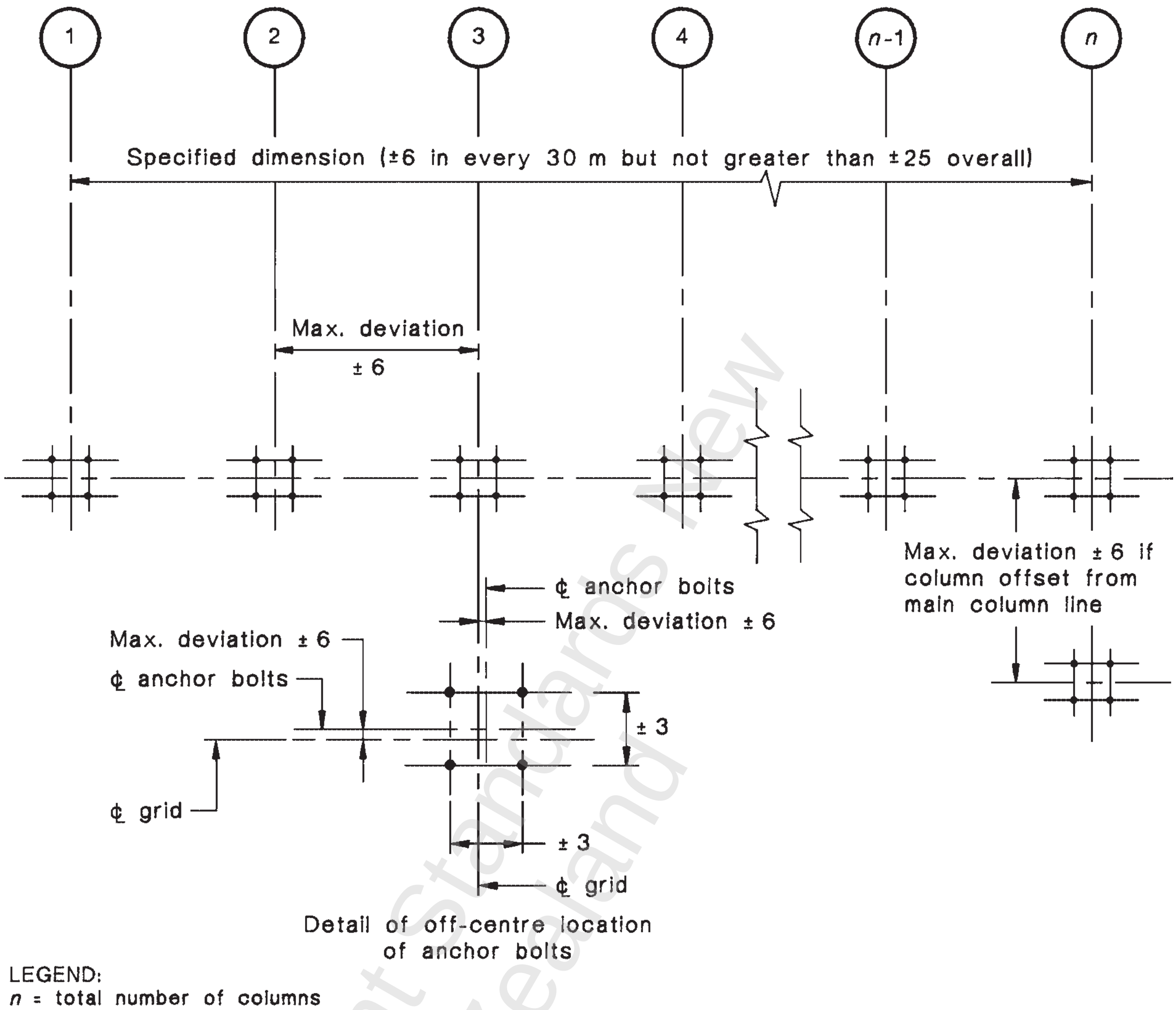
15.3.2.1 Position in plan
The position in plan of a steel column base shall not deviate from its correct value by more than 6 mm along either of the principal setting out axes.

15.3.2.2 Level
The level of the underside of a steel base plate shall not deviate from its correct value by more than ±10 mm.

15.3.2.3 Full contact

15.3.2.3.1
If full contact is specified, the requirements of 14.4.4.2 shall be satisfied, unless shims are used to reduce the measurable gaps to values specified in 14.4.4.2.

15.3.2.3.2
Packs, shims and other supporting devices shall be flat and of the same steel grade as the member. If such packings are to be subsequently grouted, they shall be placed so that the grout totally encloses them with a minimum cover of 50 mm.



DIMENSIONS ARE IN MILLIMETRES, UNLESS NOTED OTHERWISE.

Figure 15.3.1 – Tolerances in anchor bolt location

15.3.3 Plumbing of a compression member

The alignment and plumbing of a compression member shall be in accordance with both of the following requirements:

(a) The deviation of any point above the base of the compression member from its correct position shall not exceed height/500 or as follows, whichever is the lesser:

(i) For a point up to 60 m above the base of the member .... 25 mm.

(ii) For a point more than 60 m above the base of the member .... 25 mm plus 1 mm for every 3 m in excess of 60 m up to a maximum of 50 mm.

(b) The deviation of the top of the compression member from its correct position relative to the bottom of the member from one storey to the next shall not exceed storey height/500.

15.3.4 Column splice

A column splice shall conform to the following requirements:

(a) The level of the centre-line of a column splice shall not deviate from its correct level by more than ±10 mm;

(b) The position in plan of a column splice shall be in accordance with the plumbing tolerances specified in 15.3.3;

(c) The plan position of each spliced member relative to the other shall not deviate by more than 2 mm from their correct positions along either of the principal setting-out axes.

(d) For full contact column splices, the maximum clearance between the abutting surfaces shall not exceed:

(i) (d/1000 + 1) mm;

(ii) For gaps between (d/1000 + 1) and 6 mm, and the member is not part of a seismic frame, shims shall be required;

(iii) For gaps between (d/1000 + 1) and 3 mm, and the member is part of a seismic frame, shims shall be required;

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(iv) For gaps exceeding (ii) or (iii) an engineering assessment shall be required.

15.3.5 Level and alignment of a beam

In erecting a structure, a beam shall be deemed to be correctly positioned when:

(a) All connections including splices are completed;

(b) The maximum sweep in the beam is less than L_b/500, where L_b is the length between points of effective bracing or restraint;

(c) A beam is within ±10 mm of its correct level at connections to other members; and

(d) A web of a beam is within ±3 mm horizontally of its correct position at connections to other members.

15.3.6 Position of a tension member

A tension member shall not deviate from its correct position relative to the members to which it is connected by more than 3 mm along any setting-out axis.

15.3.7 Overall building dimensions

The overall building dimensions shall not deviate from the correct values by more than the following:

(a) Length (see figure 15.3.7.1)

for $\sum L_c \leq 30$ m, $\sum \Delta L_c \leq \pm 20$ mm

for $\sum L_c > 30$ m, $\sum \Delta L_c \leq \pm[20 \text{ mm} + 0.25 (\sum L_c - 30) \text{ mm}]$

(b) Height (see figure 15.3.7.2)

for $\sum h_b \leq 30$ m, $\sum \Delta h_b \leq \pm 20$ mm

for $\sum h_b > 30$ m, $\sum \Delta h_b \leq \pm[20 \text{ mm} + 0.25 (\sum h_b - 30) \text{ mm}]$

provided that:

(i) The distance between adjacent steel column centres (L_c) at every section does not deviate by more than ±15 mm from the correct length;

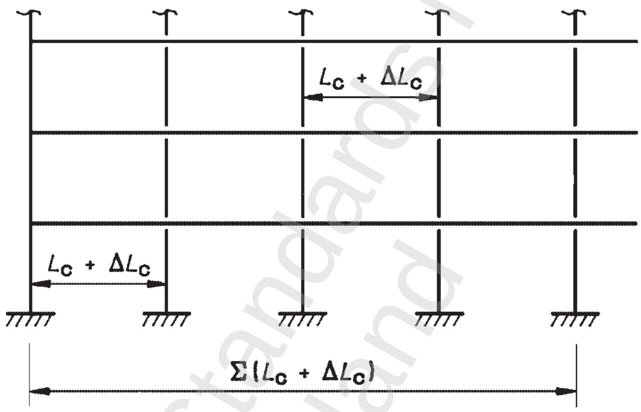
(ii) The vertical distance between tops of beams (h_b) at every section does not deviate by more than ±20 mm from the correct values; and

(iii) All other tolerances in this section are complied with.

For the purposes of this clause:

$\sum L_c$ = the correct overall length of steelwork, being the centre to centre distance of the extreme columns as shown in figure 15.3.7.1, at any location along the building (in metres), and

$\sum h_b$ = the correct overall height of steelwork, being the vertical distance from underside of column baseplate to the top of the finished floor level shown in figure 15.3.7.2, at any location along the building (in metres).



LEGEND:

L_c	=	distance between columns
ΔL_c	=	deviation from L_c
$\sum L_c$	=	correct overall length of steelwork
$\sum \Delta L_c$	=	deviation from $\sum L_c$

Figure 15.3.7.1 – Deviations in length (vertical section)

15.4 INSPECTION OF BOLTED CONNECTIONS

15.4.1 Tensioned bolts

The methods of tensioning specified in 15.2.5 shall comply with the following requirements:

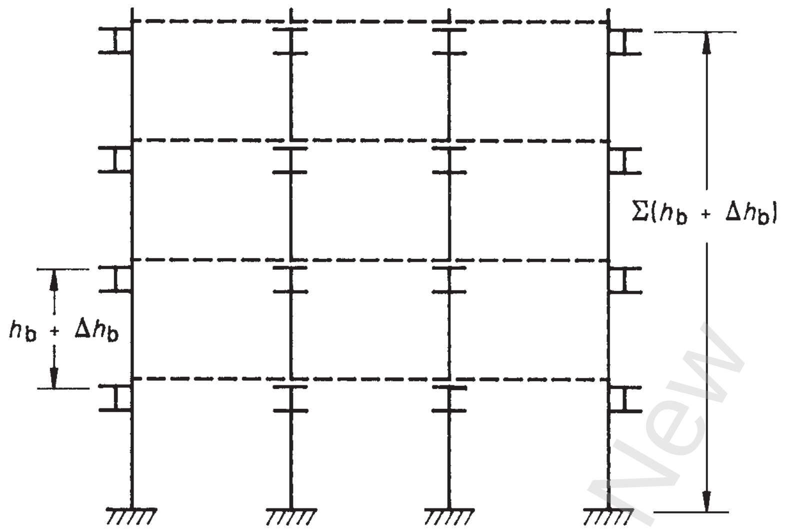
(a) Part-turn tensioning–the correct part-turn from the snug-tight position shall be measured or observed;

(b) Direct-tensioning indication device–the minimum tension developed in the bolt shall be indicated directly by the device.

NOTE –

(1) The manufacturer's recommendations for inspection procedures should be followed when using a direct-tensioning indication device.

(2) The use of a torque wrench for inspection is considered suitable only to detect gross undertensioning. A procedure for such use is detailed in Appendix L.



LEGEND:

h_b	=	distance between tops of beams
Δh_b	=	deviation from h_b
$\sum h_b$	=	correct overall height of steelwork
$\sum \Delta h_b$	=	deviation from $\sum h_b$

Figure 15.3.7.2 – Deviations in height (vertical section)

15.4.2 Damaged items

Bolts, nuts and washers which, on visual inspection, show any evidence of physical defects shall be removed and replaced by new items.

15.5 GROUTING AT SUPPORTS

15.5.1 Compression member base or beam

15.5.1.1

Bedding under a compression member base plate or the bearing of a beam on masonry and concrete shall be provided by grout or mortar.

15.5.1.2

Grouting or packing shall not be carried out until a sufficient portion of the structure (for multistorey buildings, a sufficient number of bottom column lengths) has been aligned, levelled and plumbed and adequately braced by other structural members which have been levelled and are securely held by their permanent fastenings. Steel packing or levelling nuts on the anchor bolts shall be under the base plate to support the steelwork. The space under the steel shall be thoroughly cleaned and be free from moisture immediately before grouting.

15.5.2 Grouting

Grout shall completely fill the space to be grouted and shall either be placed under pressure or placed by ramming against fixed supports.

Amd 2 Oct. '07 | Grout shall comply with NZS 3104.

16 MODIFICATION OF EXISTING STRUCTURES

16.1 GENERAL

All provisions of this Standard apply equally to the modification of existing structures or parts of a structure except as modified in this section.

16.2 MATERIALS

The types of base metal involved shall be determined before preparing the drawings and specifications covering the strengthening of, the repair of, or the welding procedures for an existing structure or parts of a structure.

16.3 CLEANING

Surfaces of existing material, which are to be strengthened, repaired, or welded shall be cleaned of dirt, rust and other foreign matter except adherent surface protection. The portions of such surfaces that are to be welded shall be cleaned thoroughly of all foreign matter, including paint film, for a distance of 50 mm from each side of the outside lines of the welds.

16.4 SPECIAL PROVISIONS

16.4.1 Welding and cutting

The capacity of a member to carry loads while welding or oxygen cutting is being performed on it shall be determined according to the provisions of this Standard, taking into consideration the extent of cross section heating of the member which results from the operation that is being performed.

16.4.2 Welding sequence

The welding sequence shall be chosen so as to minimize distortion of the member and ensure that its straightness remains within the appropriate straightness limits of 14.4.3, 14.4.4, 14.4.5, and 14.4.6.

NOTES

17 TESTING OF STRUCTURES OR ELEMENTS

17.1 GENERAL

17.1.1 Scope of section

The methods of test given in this section are applicable to proof tests and prototype tests of complete structures, sub-structures, individual members or connections. The methods are not applicable to the testing of structural models, nor to the establishment of general design criteria or data.

17.1.2 Circumstances requiring tests

Structures or parts of structures designed in accordance with this Standard are not required to be tested.

Where appropriate, tests will be accepted as an alternative to calculation.

Tests may be carried out as part of a special study.

17.2 DEFINITIONS

For the purposes of this section, the definitions below apply.

PROOF TESTING. The application of test loads to a structure, sub-structure, member or connection to ascertain the structural characteristics of only that one unit tested.

PROTOTYPE TESTING. The application of test loads to one or more structures, sub-structures, members or connections to ascertain the structural characteristics of that class of structures, sub-structures, members or connections which are nominally identical to the units tested.

17.3 TEST REQUIREMENTS

17.3.1

The test load shall be determined in accordance with 17.4.2 or 17.5.2, as appropriate.

17.3.2

Loading devices shall be calibrated, and care shall be exercised to ensure that no artificial restraints are applied by the loading systems. The test load shall be applied to the unit at a rate as uniform as practicable. The distribution and duration of loads applied in the test shall represent those loads to which the structure is deemed to be subjected under the requirements of section 3.

17.3.3

Deformations shall, as a minimum requirement, be recorded at the following times:

(a) Prior to the application of the test load;

(b) After the test load has been applied;

(c) After the removal of the test load.

17.4 PROOF TESTING

17.4.1 Application

This clause applies to the testing of a structure, sub-structure, member or connection to determine whether that particular structure, sub-structure, member or connection complies with the requirements for the ultimate or serviceability limit state, as appropriate.

17.4.2 Test load

The magnitude of test load shall be equal to the design load for the relevant limit state as determined from 3.2.3.

17.4.3 Criteria for acceptance

Criteria for acceptance shall be as follows:

(a) *Acceptance for strength; non-seismic applications*
The test structure, sub-structure, member or connection shall be deemed to comply with the requirements for strength if it is able to sustain the ultimate limit state test load for at least 15 minutes. It shall than be inspected to determine the nature and extent of any damage incurred during the test. The effects of the damage shall be considered and, if necessary, appropriate repairs to the damaged parts carried out.

(b) *Acceptance for strength and ductility; seismic applications*
Amd 2 Oct. '07 The test structure, sub-structure, member or connection shall comply with the requirements of ANSI/AISC 341-05 Appendices S or T or an equivalent test procedure.

(c) *Acceptance for serviceability*
The maximum deformation of the structure or member under the serviceability limit state test load shall be within the serviceability limits appropriate to the structure.

17.5 PROTOTYPE TESTING

17.5.1 Test specimen

The materials and fabrication of the prototype shall comply with sections 2 and 14, respectively. Any additional requirements of a manufacturing specification shall be complied with and the method of erection used shall simulate that which will be used in production.

17.5.2 Test load

The test load shall be equal to the design load for the relevant limit state determined in accordance with 3.2.3, multiplied by the appropriate factor given in table 17.5.2, unless a reliability analysis shows that a smaller value can be adopted.

Table 17.5.2 – Factors to allow for variability of structural units

No of similar units to be tested	Ultimate limit state	Serviceability limit state
1	1.5	1.2
2	1.4	1.2
3	1.3	1.2
4	1.3	1.1
5	1.3	1.1
10	1.2	1.1

17.5.3 Criteria for acceptance

Criteria for acceptance shall be as follows:

(a) *Acceptance for strength; non-seismic applications*
The test unit shall be deemed to comply with the requirements for strength if it is able to sustain the ultimate limit state test load for at least 5 minutes.

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(b) *Acceptance for strength and ductility; seismic applications*
The test unit shall comply with the requirements of ANSI/AISC 341-05 Appendices S or T or an equivalent test procedure.

(c) *Acceptance for serviceability*
The maximum deformation of the unit under the serviceability limit state test load shall be within the serviceability limits appropriate to the structure.

17.5.4 Acceptance of production units

Production-run units shall be similar in all respects to the unit or units tested.

17.6 REPORTING OF TESTS

The report of the test on each unit shall contain, in addition to the test results, a clear statement of the conditions of testing, including the method of loading and of measuring deflection, together with any other relevant data. The report shall also contain a statement as to whether or not the structure or part tested satisfied the acceptance criteria.

NOTES

APPENDICES TO NZS 3404:PART 1:1997

INTRODUCTION

Each Appendix to this Standard forms either a normative or an informative part. Within the context of this Standard, these terms have the following meaning:

Normative. A normative Appendix is one which establishes the required practice to use in accordance with this Standard.

Informative. An informative appendix contains material which is included for guidance, but is not required practice to use in accordance with this Standard.

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APPENDIX A
REFERENCED DOCUMENTS

(This Appendix forms a normative part of this Standard.)

The following documents are referred to in this Standard:

NEW ZEALAND STANDARDS

(Text deleted)

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NZS 1170.5:2007	Structural design actions, Part 5: Earthquake actions – New Zealand
NZS/AS 1657	Code for fixed platforms, walkways, stairways and ladders. Design, construction and installation
NZS 3101	Concrete structures Standard
NZS 3104	Specification for concrete production
NZS 3109	Concrete construction

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NZS 4332	Non-domestic passenger and goods lifts
NZS 4711	Qualification tests for metal arc welders

AUSTRALIAN AND JOINT AUSTRALIAN/NEW ZEALAND STANDARDS

AS 1101	Graphical symbols for general engineering
AS 1101.3	Part 3: Welding and non-destructive examination
AS 1110	ISO metric hexagon bolts and screws – Product grades A and B
AS 1111	ISO metric hexagon bolts and screws – Product grade C
AS 1112	ISO metric hexagon nuts
AS 1163	Structural steel hollow sections
AS/NZS 1170 set	Structural design actions
AS 1210	Unfired pressure vessels
AS/NZS 1252	High strength steel bolts with associated nuts and washers for structural engineering
AS 1275	Metric screw threads for fasteners

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Amd 2 Oct. '07	AS 1418 set	Cranes, hoists and winches
	AS 1443	Carbon steels and carbon-manganese steels – Cold-finished bars
	AS 1530 AS 1530.4	Methods for fire tests on building materials, components and structures Part 4: Fire-resistance test of elements of building construction
	AS 1544	Methods for impact tests on materials Part 2: Charpy V-notch
	AS/NZS 1553 AS/NZS 1553.1	Covered electrodes for welding Part 1: Low carbon steel electrodes for manual metal-arc welding of carbon steels and carbon-manganese steels
	AS/NZS 1554 AS/NZS 1554.1 AS 1554.2 AS/NZS 1554.4 AS/NZS 1554.5	SAA structural steel welding standard Part 1: Welding of steel structures Part 2: Stud welding (steel studs to steel) Part 4: Welding of high strength quenched and tempered steels Part 5: Welding of steel structures subject to high levels of fatigue loading
	AS 1559	Fasteners-bolts, nuts and washers for tower construction
	AS 1594	Hot-rolled steel flat products
Amd 2 Oct. '07	*(Text deleted)*	
	AS 1858 AS 1858.1	Electrodes and fluxes for submerged-arc welding Part 1: Carbon steels and carbon-manganese steels
	AS/NZS 1873	Powder-actuated (PA) hand-held fastening tools Part 1: Selection, operation and maintenance Part 2: Design and construction Part 3: Charges Part 4: Fasteners
	AS 2074	Steel castings
	AS 2203	Cored electrodes for arc welding
	AS 2205 AS 2205.2.1	Methods of destructive testing of welds in metal Tensile tests – Transverse butt tensile test
	AS/NZS 2312	Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings
	AS 2382	Surface roughness comparison specimens
	AS 2670	Evaluation of human exposure to whole-body vibration
	AS/NZS 2717 AS/NZS 2717.1	Welding – Electrodes – Gas metal arc Part 1: Ferritic steel electrodes
	AS/NZS 3678	Structural steel – Hot-rolled plates, floor-plates and slabs

AS/NZS 3679	Structural steel
AS/NZS 3679.1	Part 1: Hot-rolled bars and sections
AS/NZS 3679.2	Part 2: Welded I-sections
AS 3828	Guidelines for the erection of building steelwork (Amd 2 Oct. '07)
AS 4100	Steel structures
AS 4100	Supplement 1: Steel structures – Commentary
AS/NZS 4600	Cold-formed steel structures standard
AS/NZS 4671	Steel reinforcing materials (Amd 2 Oct. '07)

AMERICAN STANDARDS

ASTM A108	Specification for steel bars, carbon, cold-finished, standard quality
ASTM A514	Specification for high-yield-strength, quenched and tempered alloy steel plate, suitable for welding
ANSI/ASME	VIII: Boiler and pressure vessel code

BRITISH STANDARDS

BS 4	Structural steel sections Part 1: Specification for hot-rolled sections
BS 476	Fire tests on building materials and structures Part 20: Method for determination of the fire resistance of elements of construction (general principles) Part 21: Method for determination of the fire resistance of loadbearing elements of construction Part 22: Method for determination of the fire resistance of non-loadbearing elements of construction Part 23: Method for determination of the contribution of components to the fire resistance of a structure
BS EN 1993-1-3	Eurocode 3. Design of steel structures. General rules. Supplementary rules for cold-formed members and sheeting
BS EN 1994-1-1	Eurocode 4. Design of composite steel and concrete structures. General rules and rules for buildings (Amd 2 Oct. '07)
BS 4848	Hot-rolled structural steel sections Part 2: Specification for hot-finished hollow sections Part 4: Equal and unequal angles
BS 5500	Specification for unfired fusion welded pressure vessels
BS 7668	Weldable structural steels. Hot finished structural hollow sections in weather resistant steels. Specification (Amd 2 Oct. '07)
BS 7910	Guide to methods for assessing the acceptability of flaws in metallic structures (Amd 1 June '01)
BS EN 10002	Tensile testing of metallic materials Part 1: Method of test at ambient temperature

BS EN 10025 - - -	Hot rolled products of structural steels. Part 1 General delivery conditions Part 2 Technical delivery conditions for non-alloy structural steels. Part 3 Technical delivery conditions for long products Part 4 Technical delivery conditions for the thermomechanical rolled weldable fine grain steels Part 5 Technical delivery conditions for structural steels with improved atmospheric corrosion resistance Part 6 Technical delivery conditions for plates and wide flats of high yield strength structural steels in the quenched and tempered condition
BS EN 10029	Specification for tolerances on dimensions, shape and mass for hot rolled steel plates 3 mm thick or above.
BS EN 10210	Hot finished structural hollow sections of non-alloy and fine grain structural steels Part 1 Technical delivery requirements
BS EN 10219	Cold formed welded structural hollow sections of non-alloy and fine grain steels Part 2: Tolerances, dimensions and sectional properties

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INTERNATIONAL STANDARDS

ISO 834	Fire-resistance tests – Elements of building construction
ISO 2566	Steel – Conversion of elongation values

(Text deleted)

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JAPANESE STANDARDS

JIS G 3101	Rolled steels for general structure
JIS G 3106	Rolled steels for welded structure
JIS G 3114	Hot-rolled atmospheric corrosion resisting steels for welded structure
JIS G 3132	Hot-rolled carbon steel strip for pipes and tubes
JIS G 3136	Rolled steel for building structure
JIS G 3141	Cold-reduced carbon steel sheets and strip
JIS G 3192	Dimensions, mass and permissible variations of hot rolled steel sections
JIS G 3193	Dimensions, mass and permissible variations of hot rolled steel plates, sheets and strip

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OTHER PUBLICATIONS

Amd 1 June '01

(Text deleted)

ANSI/AISC 341-05 Seismic Provisions for Structural Steel Buildings

Fire protection for structural steel in buildings (4th edition). Published by the ASFPCM, Aldershot, England

Amd 2 Oct. '07

HERA Report R4-133 New Zealand Steelwork Corrosion Coatings Guide

WTIA Technical Note 5 "Flame Cutting of Steels" 1994

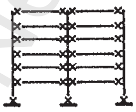
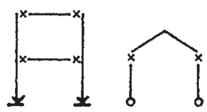
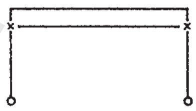
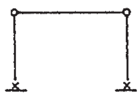
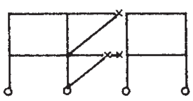
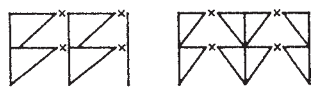
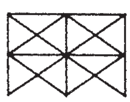
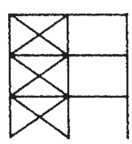
New Zealand Rail Limited : Railnet Code, Part 4, Code Supplements Bridges and Structures, Section 2 : Design

Amd 2 Oct. '07

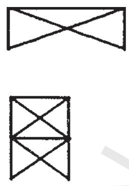
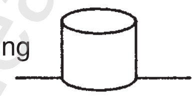
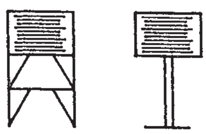
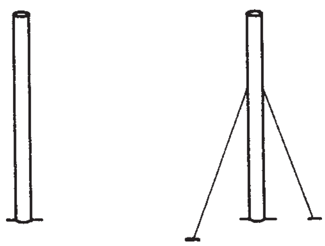
Bridge Manual (SP/M/022) second edition, Transit New Zealand, 2003, incorporating amendments dated June 2004, September 2004 and provisional amendment December 2004

APPENDIX B
MAXIMUM LEVELS OF DUCTILITY DEMAND ON STRUCTURAL STEEL SEISMIC-RESISTING SYSTEMS

(This Appendix forms a normative part of this Standard. It is to be read in conjunction with 12.2.4 and table 12.2.4.)

	Type of structure	Diagram	Maximum ductility demand, μ
1	**MOMENT-RESISTING FRAMES**		
1.1	Category 1 frame		6.0
1.2	Category 2 frame		3.0[5]
1.3	Frame designed by the plastic design method (not to exceed the critical height and must be category 2)		3.0[5]
1.4	Category 1 frame of 1 storey using truss as hinge forcing mechanism, hinge forms in columns		6.0
1.5	Category 3 frame		1.25
1.6	Category 4 frame		1.0
1.7	Category 2 vertical cantilever		3.0[5]
2	**ECCENTRICALLY BRACED FRAMES**		
2.1	Category 1 eccentrically braced frames designed and detailed in accordance with 12.11.	 	6.0
3	**CONCENTRICALLY BRACED FRAMES**		
3.1	Concentrically braced frames capable of yielding in tension and dependable inelastic action in compression, designed in accordance with 12.12.2 to 12.12.5.		
	(a) Category 1 frames (b) Category 2 frames (c) Category 3 frames (d) Category 4 frames		6.0[4] 3.0[4,5] 1.25[4] 1.0[4]

➤

Type of structure			Diagram	Maximum ductility demand, μ
3.2	Concentrically braced frames capable of yielding in tension only, designed in accordance with 12.12.6.			
	(a)	Category 1 frames		6.0[4]
	(b)	Category 2 frames		3.0[4,5]
	(c)	Category 3 frames		1.25[4]
	(d)	Category 4 frames		1.0
4	**STEEL SHEAR WALLS**			
	Shear walls of stocky or slender plates designed for yielding in shear			3.0[5]
5	**STEEL TANKS**			
5.1	Steel tanks on ground.			
	(i)	Ductile skirt pedestal		3.0[5]
	(ii)	Stepping foundations		2.0
	(iii)	Tensile yielding of holding down bolts		3.0
	(iv)	Nominally ductile		1.25
	(v)	Elastic		1.0
	(vi)	Tensile and compressive yielding of holding down bolts		6.0
5.2	Elevated steel tanks. Support frames as for relevant structure; tank to remain elastic.			–
6	**CHIMNEYS, STACKS etc.**			
6.1	Ductile skirt pedestal			3.0[5]
6.2	Stepping foundations			2.0
6.3	Tensile yielding of holding-down bolts			3.0
6.4	Guyed	(i) Nominally ductile		1.25
		(ii) Elastic		1.0
6.5	Cantilever	(i) Nominally ductile		1.25
		(ii) Elastic		1.0

NOTE –

(1) The above values of μ are the maximum values that can be used in conjunction with the appropriate design and detailing provisions of this Standard.

(2) A lower value of μ than that listed above or in note (5) may be used for a given type of structure in conjunction with the appropriate design and detailing provisions for the value of μ listed above.

(3) Use of a higher value of μ than listed above or in note (5) for a given type of structure will require additional evidence that the structure can satisfactorily resist the anticipated level of ductility demand.

(4) The inelastic behaviour of concentrically braced frames is dependant on the number of storeys (i.e. levels of bracing) the brace configuration and the slenderness ratio of the brace, and the design seismic force should be chosen accordingly. To account for these factors, the basic design seismic force for the ultimate limit state, as determined from the Loadings Standard for a given value of μ, must be multiplied by a factor $C_S \geq 1.0$. Details are given in 12.12.3.1 or 12.12.6.3. Maximum height limitations also apply for stand-alone concentrically braced framed systems (see 12.12.4 or 12.12.6.5).

(5) Use of structural displacement ductility factors greater than 3 for any category 2 seismic-resisting system identified by this note is permissible when the fundamental period (T_1) of the system is less than 0.7 seconds and the inelastic design spectrum is derived from the elastic design spectrum through the use of the equal energy concept. This applies for design to NZS 4203.

In such instances, the value of μ is dependent on T_1 and is given by:

$3 < \mu = 10\,(1 - T_1) \leq 5$ for a category 2 system.

The design and detailing requirements for the appropriate category of system from section 12 apply to the values of μ given above.

(6) Where indicated in the appropriate diagram:

x ≡ plastic hinge

o ≡ pin connection

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(7) For structural categories 1 and 2 the serviceability limit state requirements of NZS 1170.5 need to be considered.

APPENDIX C
CORROSION PROTECTION

(This Appendix forms a normative part of this Standard.)

C1 SCOPE

C1.1
This Appendix applies to the corrosion protection of steel members and connection components.

C1.2
The provisions of this Appendix provide a means of compliance with the requirements of the New Zealand Building Code Clause B2: Durability.

C2 SYSTEMS

C2.1
Amd 2 Oct. '07

Requirements for painting of steelwork in most environments are given in AS/NZS 2312 (including Amendment No 1) and HERA Report R4-133.

C2.2
The type of coating and surface preparation shall be specified, after proper account has been taken of the use of the structure, climatic or other local conditions, maintenance provisions, and of the effects of the fabrication processes on previously applied coatings.

C2.3
Note that the "specified intended life of the building" from Clause B2 of the NZBC need not be the life to first maintenance considered when selecting an appropriate method of corrosion protection. A shorter life to first maintenance may be selected in conjunction with a maintenance programme which together will meet the durability provisions of NZBC Clause B2.

C3 STANDARDS
All steelwork which is to be painted after fabrication and before erection shall be prepared and painted in accordance with relevant Standards. A list of such Standards may be found in C7.

Amd 2 Oct. '07

C4 INACCESSIBLE SURFACES AND TREATMENT OF CUT EDGES
Surfaces which will be in contact or near contact after fabrication or erection shall receive their specified surface preparation and treatment prior to assembly. Such surfaces should be dry before assembly.

This clause does not apply to the interior of hollow sections conforming to the relevant Standards listed in 2.2.1, or box sections, or connection surfaces for joints with friction type bolting category where bare steel interfaces are specified.

Amd 2 Oct. '07

Cut edges shall be dressed before application of coatings to AS/NZS 2312 clause 3.3.5.2.

C5 PROTECTION DURING TRANSPORT AND HANDLING AFTER CORROSION PROTECTION
Structural members shall be adequately protected during handling and transport to minimize damage to the corrosion protection.

Units which are transported in nested bundles should be separable without damage to the units or their coatings. Consideration shall be given to the use of lifting beams with appropriately spaced lifting points and slings, or to lifting with properly spaced fork-lift tines.

C6 REPAIRS TO CORROSION PROTECTION

Corrosion protection which has been damaged by welding or other causes shall be restored before the structure is put into service, unless this is considered not to be necessary by the design engineer or the construction reviewer.

The damaged area shall be dry and clean, free from dirt, grease, loose or heavy scale or rust before the corrosion protection is applied. The corrosion protection shall be applied as soon as practicable and before noticeable oxidation of cleaned surfaces occurs. Damaged zinc coating shall be restored by a suitable zinc paint conforming to AS/NZS 3750.9 or to AS/NZS 3750.15, or with thermal zinc spray.

Amd 1 June '01

C7 RELEVANT STANDARDS AND OTHER DOCUMENTS

NEW ZEALAND STANDARD

NZS 7703	The painting of buildings

AUSTRALIAN AND JOINT AUSTRALIAN/NEW ZEALAND STANDARDS

AS 1192	Electroplated coatings – Nickel and chromium
AS 1214	Hot-dip galvanized coatings on threaded fasteners (ISO metric coarse thread series)
AS/NZS 1580	Paints and related materials – Methods of test
AS/NZS 1580.108.1	Method 108.1 Determination of dry film thickness on metallic substrates – Non-destructive methods
AS 1627	Metal finishing – Preparation and pretreatment of surfaces
AS 1627.0	Part 0: Method selection guide for preparation and pretreatment of steel surfaces
AS 1627.1	Part 1: Cleaning using liquid solvents and alkaline solutions
AS 1627.2	Part 2: Power tool cleaning
AS 1627.3	Part 3: Flame descaling
AS 1627.4	Part 4: Abrasive blast cleaning
AS 1627.5	Part 5: Pickling, descaling and oxide removal
AS 1627.6	Part 6: Chemical conversion treatment of metals
AS 1627.7	Part 7: Hand tool cleaning of metal surfaces
AS 1627.8	Part 8: Wash primer pretreatment of metal surfaces
AS 1627.9	Part 9: Pictorial surface preparation standards for painting steel surfaces
AS 1627.10	Part 10: Cleaning and preparation of metal surfaces using acid solutions (non-immersion)
AS/NZS 1650	Hot-dipped galvanized coatings on ferrous articles (in preparation)
AS 1789	Electroplated coatings – Zinc on iron or steel
AS 1790	Electroplated coatings – Cadmium on iron or steel
AS 1791	Chromate conversion coatings – Zinc and cadmium

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AS 1856	Electroplated coatings – Silver
AS 1897	Electroplated coatings on threaded components (metric coarse series)
AS 1901	Electroplated coatings – Gold and gold alloys
AS 2105	Inorganic zinc silicate paint
AS 2239	Galvanic (sacrificial) anodes for cathodic protection
AS 2311	The painting of buildings
AS/NZS 2312	Guide to the protection of iron and steel against exterior atmospheric corrosion
AS 2672	Paints for steel structures – Chlorinated rubber, high build
AS 2673	Paints for steel structures – Alkyd/micaceous iron oxide
AS 2674	Paints for steel structures – Epoxy primer (two-pack)
AS 2832	Guide to the cathodic protection of metals
AS 3730	Guide to the properties of paints for buildings
AS 3730.6	Part 6: Solvent-borne – Exterior – Full-gloss enamel
AS/NZS 3750	Paints for steel structures
AS/NZS 3750.6	Part 6: Full gloss polyurethane (two-pack)
AS/NZS 3750.10	Part 10: Full gloss epoxy (two-pack)
AS/NZS 3750.11	Part 11: Chlorinated rubber – High-build and gloss
AS/NZS 3750.12	Part 12: Alkyd/micaceous iron oxide
AS/NZS 3750.13	Part 13: Epoxy primer (two-pack) (in preparation)
AS 3884	Etch primers (single pack and two-pack) for pretreating metal surfaces
AS 3887	Paints for steel structures – Coal tar epoxy (two-pack)
AS 3894.3	Determination of dry film thickness
AS 4025	Paints for equipment including ships
AS 4025.1	Solvent-borne – Interior and exterior – Full gloss enamel
AS 4089	Priming paint for steel – Single component – General purpose
AS 4169	Electroplated coatings – Tin and tin alloys

Amd 1 June '01

OTHER PUBLICATIONS

	CK13	Code of recommended practice for preparation of metal surfaces for electroplating
	CP1021	Cathodic protection
Amd 1 June '01	ISO 2063:1991 or BS EN 22063:1994	Metallic and other inorganic coatings – Thermal spraying – Zinc, aluminium and their alloys
	BS 7079	Preparation of steel substrates before application of paints and related products Part A1: Specification for rust grades and preparation grades of uncoated steel substrates and of steel substrates after overall removal of previous coatings
Amd 2 Oct. '07	HERA	HERA Report R4-133 New Zealand Steelwork Corrosion Coatings Guide

APPENDIX D
INSPECTION OF WELDING TO AS/NZS 1554.1

(This Appendix has been written specifically for the application of AS/NZS 1554.1 in accordance with this Standard. Clauses D1 to D2 form a normative part of this Standard, while table D1 forms an informative part of this Standard.)

D1 SCOPE

D1.1
When using AS/NZS 1554.1 in accordance with the requirements of this Standard, the inspection of welds shall be in accordance with section 7 of AS/NZS 1554.1, except that table D1 of this Appendix shall replace Appendix F of AS/NZS 1554.1.

D1.2
The authority as defined in 1.3 is also the inspecting authority as defined in 1.4 of AS/NZS 1554.1.

D1.3
The principal is defined in 1.4 of AS/NZS 1554.1.

D2 EXTENT OF NON-DESTRUCTIVE EXAMINATION
The extent of non-destructive examination required shall be determined by the principal to the approval of the design engineer.

NOTE –

(1) Further guidance on the extent of non-destructive examination is given in table D1.

(2) Refer to item (q) of AS/NZS 1554.1 Appendix E for references to the relevant clauses to consider in establishing the extent of NDE examination required on each project.

Table D1 – Suggested extent of non-destructive examination for welds to AS/NZS 1554.1
(Informative)

Weld category	Extent of NDE, % (See Notes 1 and 2)			
	Visual means (see Note 3)		Other means	
	Visual scanning (see clause 7.3 of AS/NZS 1554.1)	Visual examination to table 6.2 of AS/NZS 1554.1 (See Note 4)	Magnetic particle or liquid penetrant (See Notes 7 and 8)	Radiography or ultrasonics to table 6.3 of AS/NZS 1554.1 (See Notes 5 and 6)
GP SP	100 100	25 25 to 100	0 to 10 0 to 10	Nil 0 to 15

NOTE –

(1) Table D1 is intended to apply to routine testing of welds to determine the level of weld quality. Where routine testing reveals imperfections requiring further consideration in accordance with section 6 of AS/NZS 1554.1, the extent of further NDE required shall be determined in accordance with 6.7 of AS/NZS 1554.1.

(2) Where a proportion of NDE less than 100 % is required, a programme for testing should be drawn up by the principal, to the approval of the design engineer. This programme should involve full testing of the first major component or 5 % of welds, as appropriate, in order to pick up and be able to correct the cause of any major defects on commencement of welding. It should then involve a progressive reduction in frequency of testing on the basis of achieving compliance with each test. If non-compliance results from a test, return should be made to consecutive testing of the next 5 % of welds or the next major component, as appropriate.

(3) Visual means of NDE implies 2 levels of examination as follows:

(a) *Visual scanning* – To determine that no welds called for in the drawings are omitted and to detect gross defects.

(b) *Visual examination* – To examine a percentage of the welds to determine whether the required weld quality (see Table 6.2 of AS/NZS 1554.1) has been achieved.

(4) Welds which are category SP which constitute part of a seismic-resisting or associated structural system (see 9.7.1.4.3(b)) or because they are subject to fatigue loading (see 9.7.1.4.3(c)) should be subject to 100 % visual examination.

(5) Category SP welds in structures not exceeding 2 storeys in height and not supporting cranes may not require other than full visual examination on a routine basis as a means of NDE.

(6) The 15 % is applicable to category SP butt welds other than those covered by note (5). Category SP fillet welds should only be tested on a routine basis by ultrasonics when the inspection access is appropriate.

(7) The use of magnetic particle or liquid penetrant is unusual except for supplementary inspection after visual inspection for the types of structures and applications for which this Standard and AS/NZS 1554.1 is intended, and their use is usually restricted to repairs (see clause 6.2.1 of AS/NZS 1554.1).

(8) Liquid penetrant examination may be used as an alternative to magnetic particle examination, but magnetic particle examination is preferred, where convenient, for the inspection of ferromagnetic materials.

APPENDIX E
SECOND-ORDER ELASTIC ANALYSIS

(This Appendix forms a normative part of this Standard.)

E1 ANALYSIS

In a second-order elastic analysis, the members shall be assumed to remain elastic, and changes in frame geometry under the design loads and changes in the effective stiffnesses of the members due to axial forces shall be accounted for, except that:

(a) For a frame where the elastic buckling load factor (λ_c) of the frame, as determined in accordance with 4.9, is greater than 10, the second-order effects may be neglected;

(b) For a frame where the elastic buckling load factor (λ_c) of the frame, as determined in accordance with 4.9, is greater than 5, the changes in the effective stiffnesses of the members due to axial forces may be neglected.

When applying Appendix E to seismic-resisting systems being designed for load combinations which include earthquake loads, $P-\Delta$ effects shall be considered in accordance with the Loadings Standard. If $P-\Delta$ effects may be neglected in accordance with the Loadings Standard, then a second-order analysis to this Appendix for lateral deflection (sway) of the seismic-resisting system is not required. In this instance $P-\delta$ effects (on individual members) shall still be considered by one of the methods detailed in E2.

E2 DESIGN BENDING MOMENT

The design bending moment (M^*) shall be taken as the maximum bending moment in the length of the member. It shall be determined by one of (a) to (c) below:

(a) Directly from the second-order analysis, if it outputs $P-\delta$ and $P-\Delta$ effects; or

(b) Approximately, if the member is divided into a sufficient number of elements, as the greatest element end bending moment; or

(c) (i) By amplifying the maximum calculated design bending moment (M_m^*) taken as the maximum bending moment along the length of a member and obtained by superposition of the simple beam moments resulting from any transverse loading on the member with the second-order end bending moments (M_e^*) determined by the analysis.

(ii) For a member with zero axial force or a member subject to axial tension, the design bending moment (M^*) shall be calculated as follows:

$$M^* = M_m^*$$

(iii) For a member with a design axial compressive force (N^*) as determined from the analysis, the design moment (M^*) shall be calculated as follows:

$$M^* = \delta_b M_m^*$$

where d_b is the moment amplification factor for a braced member determined in accordance with 4.4.3.2.

APPENDIX F
MOMENT AMPLIFICATION FOR A SWAY MEMBER

(This Appendix forms a normative part of this Standard.)

F1

For a sway member which forms part of a rectangular frame, the design end bending moments (M_f^*), obtained from a first-order elastic analysis in which relative lateral displacements of the ends of members are not prevented, shall be separated into 2 components M_{fb}^* and M_{fs}^*,

where

M_{fb}^* = the design end bending moment obtained from a first-order elastic analysis of the frame with sway prevented (i.e. a braced frame), and

M_{fs}^* = $M_f^* - M_{fb}^*$

F2

For a frame where gravity design loads do not cause sway, it shall be permissible to calculate M_{fb}^* from the gravity design loads acting alone on the frame and M_{fs}^* from the transverse design loads acting alone.

F3

The amplified end bending moments (M_e^*) on a sway member shall be calculated as follows:

$$M_e^* = M_{fb}^* + \delta_s M_{fs}^*$$

where δ_s is the moment amplification factor for a sway member (see 4.4.3.3).

F4

The maximum calculated design bending moment (M_m^*) shall be taken as the maximum bending moment along the length of the sway member obtained by superposition of the simple beam bending moments resulting from any transverse loading on the member with the amplified end bending moments (M_e^*).

F5

For a sway member with zero axial force or a member subject to axial tension, the design bending moment (M^*) shall be calculated as follows:

$$M^* = M_m^*$$

F6

For a sway member with a design axial compressive force (N^*) as determined from the analysis, the design bending moment (M^*) shall be calculated as follows:

$$M^* = \delta_b M_m^*$$

where δ_b is the moment amplification factor for a braced member (see 4.4.3.2).

APPENDIX G
BRACED MEMBER BUCKLING IN FRAMES

(This Appendix forms a normative part of this Standard.)

G1

The member elastic flexural buckling load (N_{om}) of a braced compression member in a frame shall be determined as follows:

$$N_{om} = \frac{\pi^2 EI}{(k_e L)^2}$$

where k_e is the member effective length factor obtained from figure 4.8.3.3(a) and the values of γ_1 and γ_2 for each restrained end of the compression member under consideration shall be calculated as the ratio of the stiffness of that member to the total stiffness of the braced members restraining that end as follows:

$$\gamma = \frac{\left(\frac{I}{L}\right)_m}{\sum\left(\beta_e \alpha_{sr} \frac{I}{L}\right)_r}$$

$(I/L)m$ = stiffness in the plane of bending of the compression member under consideration

$\sum\beta_e\alpha_{sr}(I/L)r$ = summation of the stiffness in the plane of bending of all the braced restraining members rigidly connected at that end to the member under consideration (except the member itself)

β_e = modifying factor given in table 4.8.3.4 to account for the end conditions at the far end of the braced restraining member

α_{sr} = theoretical stability function multiplier, or the approximation shown in figure G1 to account for the effect of the design axial force (N_r^*) in the braced restraining member on its flexural stiffness.

G2

In figure G1, the value of ρ is calculated as follows:

$$\rho = \frac{N_r^*}{N_{oLr}}$$

where

$$N_{oLr} = \frac{\pi^2 EI_r}{L_r^{\,2}}$$

G3

For a braced restraining member in tension, α_{sr} may conservatively be taken as 1.0.

G4

Where a braced restraining member is connected by a detail with negligible moment transmitting capacity, the contribution of that member to the total stiffness shall be taken as zero.

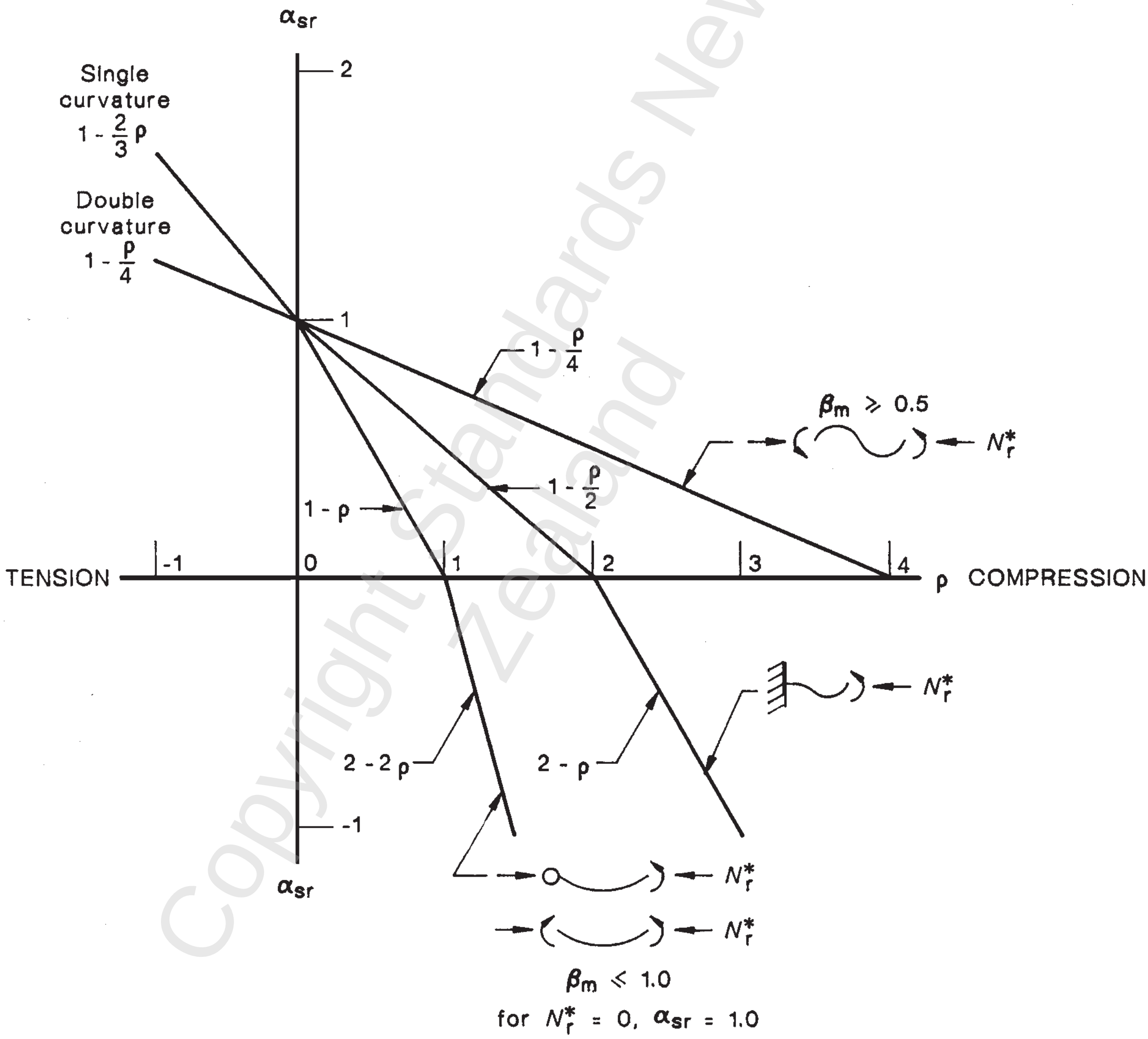


Figure G1 – Stability function multipliers

APPENDIX H
ELASTIC RESISTANCE TO LATERAL BUCKLING

Amd 2 Oct. '07 (This Appendix forms a normative part of this Standard.)

H1 GENERAL

H1.1

The elastic resistance of a beam to lateral buckling is influenced by many factors, including the beam geometry, the distribution of the loading on it, and the effects of end and intermediate restraints. Because of this, simple design rules can be formulated only for a limited number of situations. Such a set of simple rules is included in 5.6.1, 5.6.2 and 5.6.3.

H1.2

There are situations where these rules are overly conservative. When it is desirable to avoid undue conservatism, then 5.6.4 may be used, which requires the use of the results of an elastic flexural-torsional buckling analysis. This may be carried out by using computer programs such as those described in References (1 and 2).

H1.3

Alternatively, the published results of elastic flexural-torsional buckling analyses may be used. There are very many such publications, either in textbooks and surveys such as those listed in References (3 to 7), or in research publications such as References (8 to 10).

H1.4

However, it is often the case that suitable computer programs are not available, and that the designer is daunted by the complexity and scope of the research publications. In this case, it is desirable that there should be a second level of approximation, more general and more accurate than the provisions of 5.6.1, 5.6.2 and 5.6.3. Such a set of approximations is given in H2, H3 and H5. They may be used in conjunction with the method of design by buckling analysis of 5.6.4.

H2 SEGMENTS RESTRAINED AT BOTH ENDS

H2.1

The effects of geometry and loading distribution on the elastic flexural-torsional buckling of a uniform equal flanged segment restrained at both ends may be estimated by calculating approximately the maximum bending moment (M_{ob}) in the segment at elastic buckling as follows:

$$M_{ob} = \alpha_m \ \alpha_L \ M_o$$

where

α_m is given in 5.6.1.1.1 (b), or may be approximated in accordance with 5.6.4(b).

$$\alpha_L = \sqrt{\left\{1 + \left[\frac{0.4\alpha_m y_L}{M_o}\left(\frac{\pi^2 E I_y}{L_e^2}\right)\right]^2\right\}} + \left[\frac{0.4\alpha_m y_L}{M_o}\left(\frac{\pi^2 E I_y}{L_e^2}\right)\right] \quad \text{.......................(Eq. H2.1)}$$

L_e = $k_t L$, where k_t is determined from table 5.6.3(1)

M_o is given by equation H4.2.

y_L = the distance from the centroid of the gravity loading to the centroid of the beam (and is positive when the load acts below the centroid).

H2.2

Alternatively, for uniform equal flanged segments loaded so that $-d_o/2 \leq y_L \leq d_o/2$, where d_o is the overall section depth of the segment, the amended elastic buckling moment (M_{oa}) used in equation 5.6.1.1 (3) may be taken as:

$$M_{oa} = M_o + \left[0.4\alpha_m y_L \left(\frac{\pi^2 E I_y}{L_e^2}\right)\right] \quad \text{(Eq. H2.2)}$$

L_e = $k_t L$, where k_t is determined from table 5.6.3(1).

H2.3

Equations H2.1 and H2.2 are applicable to I-sections with unequal flanges, provided appropriate values of M_o (see equation 5.6.1.2) and I_y are used.

H3 SEGMENTS UNRESTRAINED AT ONE END

H3.1

The effects of geometry and loading distribution on the elastic flexural-torsional buckling of a uniform equal flanged segment unrestrained at one end and both fully or partially restrained and laterally continuous or restrained against rotation of the critical flange in plan at the other end may be estimated by calculating the maximum bending moment (M_{ob}) in the segment at elastic buckling as follows:

$$M_{ob} = \alpha_{mc}\,\alpha_{Lc}\,M_o \quad \text{(Eq. H3.1)}$$

where

$$\alpha_{mc} = \frac{(C_3 + C_4 K)}{\pi\sqrt{(1+K^2)}} \quad \text{(Eq. H3.2)}$$

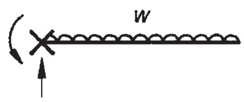
$$\alpha_{Lc} = 1 + \frac{\dfrac{2y_L}{d_f}\,\dfrac{K}{2}}{\sqrt{\left[1 + \left(\dfrac{2y_L}{d_f}\,\dfrac{K}{2}\right)^2\right]}} \quad \text{(Eq. H3.3)}$$

M_o	is given by equation H4.2
K	is given by equation H4.3
C_3, C_4	are given in table H3
d_f	is the distance between the flange centroids of the beam
y_L	is defined in H2.1.

H3.2

The elastic flexural-torsional buckling of a uniform equal flanged segment, unrestrained at one end, and both fully or partially restrained but unrestrained against rotation of the critical flange in plan at the other end, may be estimated by calculating the maximum bending moment (M_{ob}) in the segment at elastic buckling by using $C_4 = 0$ in equation H3.2.

Table H3 – Factors (C_3) and (C_4) for segments unrestrained at one end

Case number	Member segment	Moment distribution	Factor (C_3)	Factor (C_4)
1			1.6	0.8
2			4.0	3.7
3			7.0	8.0

NOTE – X $\equiv$ full or partial restraint

H4 REFERENCE ELASTIC BUCKLING MOMENT

The elastic buckling moment (M_o) of a simply supported segment in uniform bending may be used as a reference moment.

This moment is given by:

$$M_o = \sqrt{\left(\frac{\pi^2 EI_y}{L_e^2}\right)}\left\{\sqrt{\left[(GJ) + \left(\frac{\pi^2 EI_w}{L_e^2}\right) + \left(\frac{\beta_x^2}{4}\frac{\pi^2 EI_y}{L_e^2}\right)\right]} + \frac{\beta_x}{2}\sqrt{\left(\frac{\pi^2 EI_y}{L_e^2}\right)}\right\} \quad \text{(Eq. H4.1)}$$

For sections bent about an axis of symmetry, $\beta_x = 0$, and equation (H4.1) simplifies to:

$$M_o = \frac{\pi\sqrt{(EI_y GJ)}}{L_e}\sqrt{(1+K^2)} \quad \text{(Eq. H4.2)}$$

where the flexural-torsional buckling constant, K, is given by:

$$K = \sqrt{\left(\frac{\pi^2 EI_w}{GJL_e^2}\right)} \quad \text{(Eq. H4.3)}$$

In these equations:

L_e = $k_t L$, where k_t is determined from table 5.6.3(1).
E = 205 000 MPa
G $\approx$ 80 000 MPa

$$I_w = \frac{I_y (d_f)^2}{4} \qquad \text{for doubly-symmetric I-section,}$$

$$= I_{cy} d_f^2 \left(1 - \frac{I_{cy}}{I_y}\right) \qquad \text{for a monosymmetric I-section,}$$

$$I_w = \frac{b_f^3 t_f b_w^2}{48} \left(8 - \frac{3 b_f t_f b_w^2}{I_x}\right)$$ for a thin-walled channel section,

$$= 0$$ for an angle section, a tee-section, or a narrow rectangular section, and may be taken as 0 for a hollow section,

$$J \approx \sum \left(\frac{bt^3}{3}\right)$$ for an open section,

$$\approx \frac{4A_e^2}{\sum \left(\frac{b}{t}\right)}$$ for a hollow section, where A_e is the area enclosed by the hollow section,

$$\beta_x = \frac{1}{I_x} \int (x^2 y + y^3)\ dA - 2y_o$$

$$\approx 0.8 d_f \left(\frac{2I_{cy}}{I_y} - 1\right)$$ for a monosymmetric I-section

Expressions for the properties of other thin-walled sections are given in Reference (11), while more accurate approximations for J are given in Reference (12).

H5 EFFECTS OF END RESTRAINTS

H5.1 End restraints against twist rotation of the cross section

H5.1.1

The approximations given in H2 and H3 for the elastic buckling moments are for segments which are rigidly restrained at the points of restraint against twist rotations. When the torsional end restraints are elastic, the buckling twists increase, and the resistance to buckling decreases. The decreased resistance (M_{obr}) may be approximated as follows:

$$M_{obr} = M_{ob} \sqrt{\left[\frac{2\beta_t}{(1 + \beta_t)}\right]} \leq M_{ob}$$

in which β_t depends on the elastic stiffness (α_{rz}) of the end restraint against twist (i.e. the ratio of the restraining torque supplied to the twist rotation).

H5.1.2

For segments restrained at both ends:

$$\beta_t \approx \frac{\alpha_{rz} L_e / GJ}{5(1 + K^2)}$$

H5.1.3

For segments unrestrained at one end and both fully or partially restrained and either laterally continuous or restrained against rotation of the critical flange in plan at the other end:

$$\beta_t \approx \frac{\alpha_{rz} L_e / GJ}{25(1 + 2K^2)/(1 + K^2)}$$

and for segments unrestrained at one end and both fully or partially restrained but unrestrained against rotation of the critical flange in plan at the other end –

$$\beta_t \approx \frac{\alpha_{rz} L_e / GJ}{5(1 + 2K^2)/(1 + K^2)}$$

H5.2 End restraints against rotation in plan of the critical flange about the minor principal y-axis

H5.2.1 Segments restrained against rotation in plan at both ends

Continuity of a segment with adjacent segments may introduce restraining moments which reduce the lateral rotations and increase the elastic buckling moment. The restraint effects depend on the relative minor axis flexural stiffnesses of the adjacent segments, and these depend in turn on the moment distributions in these segments. The restraining effects may be calculated approximately by using the method referred to in References (6 and 9) to calculate the effective length (L_e), and by using this value of L_e in equations H4.2 and H4.3.

H5.2.2 Segments unrestrained against rotation in plan at one end

The appropriate elastic buckling moments obtained by using the values of C_3 and C_4 from table H3 in equations H4.1 to H4.3 are for segments which are prevented from rotating in plan at their ends which are restrained against twist rotation. For segments which are unrestrained against rotation in plan at their ends which are restrained against twist rotation, the value of C_4 used in equation H3.2 should be reduced to zero. For a segment with an elastic restraint against rotation in plan at its twist restrained end, a reduced value C_{4r} should be used, which may be approximated as follows:

$$\frac{C_{4r}}{C_4} = \frac{1.5\alpha_{ry} L_e / EI_y}{5 + (\alpha_{ry} L_e / EI_y)} \leq 1.0$$

in which α_{ry} is the elastic stiffness of the flexural end restraint (i.e. the ratio of the restraining minor axis moment supplied to the end rotation in plan of the segment).

H6 REFERENCES

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APPENDIX J
STRENGTH OF STIFFENED WEB PANELS UNDER COMBINED ACTIONS

Amd 2 Oct. '07 (This Appendix forms a normative part of this Standard.)

J1 YIELDING CHECK

The design bending, shear, axial and bearing actions (or reactions) (M_w^*), (V_w^*), (N_w^*) and (R_w^*) on a web panel (see figure J1) should satisfy the yielding criterion:

$$\left(\frac{R_w^*}{\phi b_{bf} t_w}\right)^2 - \frac{f_w^*}{\phi}\left(\frac{R_w^*}{\phi b_{bf} t_w}\right) + \left(\frac{f_w^*}{\phi}\right)^2 + \left(\frac{V_w^*}{0.6\phi A_w}\right)^2 \leq (f_y)^2$$

where

$$f_w^* = \frac{N_w^*}{0.6A_w} + 0.77\frac{M_w^*}{Z_{we}}$$

b_{bf} = width of the bearing load on the edge of the web dispersed at 2.5:1 through the flange as shown in figures 5.13.1.1 or 5.13.1.3

M_w^* = design bending moment in the web, calculated by elastic theory for sections with non-compact or slender flanges (see 5.2.4, 5.2.5), or by plastic theory for sections with compact flanges (see 5.2.3)

Z_{we} = elastic section modulus of the web panel

$$= \frac{t_w (d_p)^2}{6}$$

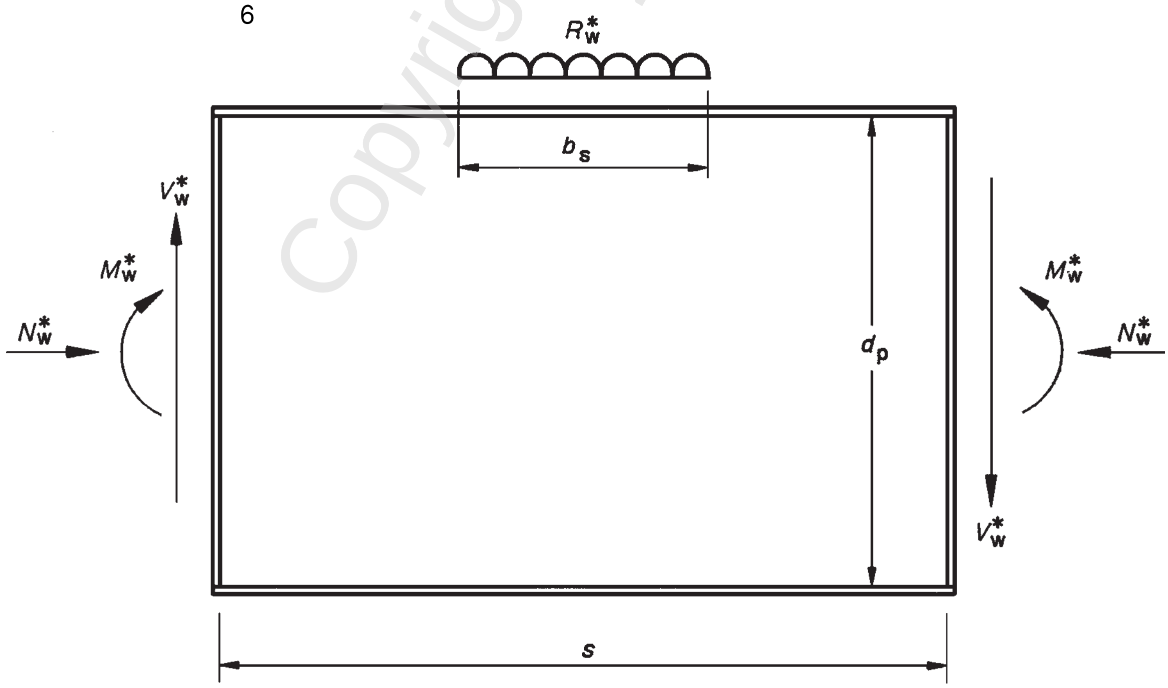


Figure J1 – Actions on web panel

J2 BUCKLING CHECK

The design bending, shear, axial and bearing actions (or reactions) (M_w^*), (V_w^*), (N_w^*) and (R_w^*) on a web panel should satisfy the buckling criterion:

Amd 2 Oct. '07

$$\left(\frac{R_w^*}{\phi R_{sb}}\right) + \left(\frac{N_w^*}{\phi N_{wo}}\right) + \left(\frac{V_w^*}{\phi V_v}\right)^2 + \left(\frac{M_w^*}{\phi M_w}\right)^2 \leq 1$$

where

N_{wo} = nominal axial capacity of the web panel if the web panel resisted axial force alone

$$= \frac{45 A_w f_y}{\left(\frac{d_p}{t_w}\right)\sqrt{\left(\frac{f_y}{250}\right)}} \leq A_w f_y$$

V_v = nominal shear capacity of the web panel if the web panel resisted shear alone, as specified in 5.11

M_w = nominal section moment capacity of the web if the web resisted bending alone, as specified in 5.2 (substitute d_p for b in 5.2 when performing this calculation)

R_{sb} = nominal buckling capacity of a transversely stiffened web in bearing alone

$$= \beta_w b_{bf} t_w f_y$$

$$\beta_w = 0.10 + \frac{20}{\left(\frac{d_e}{t_w}\right)\sqrt{\left(\frac{f_y}{250}\right)}}$$

$$d_e = \frac{1.9\sqrt{(b_{bf} d_p)}}{\alpha_w}\text{, and}$$

$$\alpha_w = \left[3.4 + \left(\frac{2.2 d_p}{s}\right)\right]\left[0.4 + \left(\frac{0.5 b_{bf}}{s}\right)\right]$$

s = spacing of transverse stiffeners.

APPENDIX K
STANDARD TEST FOR EVALUATION OF SLIP FACTOR

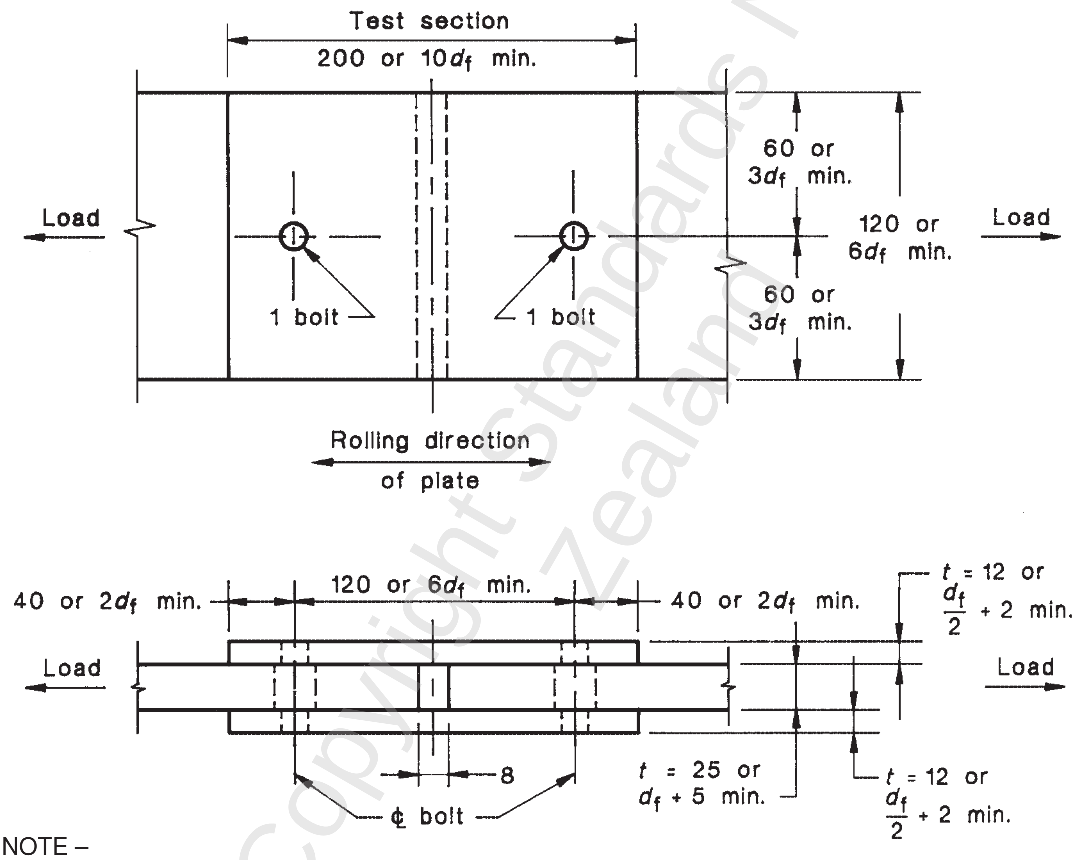
(This Appendix forms a normative part of this Standard.)

K1 TEST SPECIMENS

K1.1 Form

The standard test specimen shall be symmetrical double coverplated butt connection as shown in figure K1. The inner plates shall be equal to thickness.

NOTE – It is suggested that the use of M20 bolts will prove to be most convenient, with 25 mm inner plates and 12 mm outer plates.



NOTE –

(1) d_f is the diameter of the bolt.

(2) Holes in plates:
Cover plates .... 22 mm or (d_f + 2) mm
Inner plates .... 23 mm or (d_f + 3) mm

(3) Length and width of inner plates outside of test section may be increased to suit the laboratory's testing facilities.

(4) Dimensions are shown for the use of M20 bolts. Dimensions in parentheses are for use of bolts with nominal diameter d_f mm, which should not be less than 16 mm.

(5) All dimensions are in millimetres.

Figure K1 – Standard test specimen

K1.2 Assembly and measurement

K1.2.1

Care shall be taken in assembling the specimen to ensure that neither bolt is in bearing in the direction of loading, and that the surface condition of the friction faces is maintained in the same condition to be achieved in the field. If it is necessary to machine the ends of the inner plates to fit into the loading machine grip, machining oil shall be not allowed to contaminate the surfaces. Bolts shall be tensioned in the same manner as that to be used in the field and shall develop at least the minimum bolt tension given in table 15.2.5.1.

K1.2.2

Between snug-tightening and final tensioning, the bolt extension shall be measured using a dial gauge micrometer or a displacement transducer with a resolution of 0.003 mm or finer. The final measurement shall be made immediately prior to testing. The cone-sphere anvil measuring technique described in AS/NZS 1252 for proof load measurements or other equivalent technique is suitable.

K1.2.3

Bolt tension shall be ascertained from a calibration curve determined from load cell tests of at least 3 bolts or the test batch. In establishing the calibration curve, the bolt grip through the load cell shall be as close as practicable to that used in the specimens, the same method of extension measurement and tensioning shall be employed, and the calibration shall be based on the mean result. For the purposes of this test only, the initial snug-tight condition shall be finger tight.

K1.2.4

Alternatively, when a bolt tension load cell is not available, the bolts shall be tensioned to at least 80 % and not more than 100 % of their specified proof loads, and the tension induced in the bolts calculated from the following equation:

$$N_{ti} = \frac{E\Delta \times 10^{-3}}{\dfrac{a_o}{A_o} + \left(\dfrac{a_t + \dfrac{t_n}{2}}{A_s}\right)} \qquad \text{(Eq. K1)}$$

where

N_{ti} = tension induced in the bolt, in kilonewtons

E = Young's modulus of elasticity = 205 000 MPa

Δ = measured total extension of the bolt when tightened from a finger-tight condition to final tensioned condition, in millimetres.

a_o = length of the unthreaded portion of the bolt shank contained within the grip tensioning, in millimetres. In this context, the grip includes the washer thickness

A_o = plain shank area of the unthreaded portion of the bolt, in square millimetres

a_t = length of the threaded portion of the bolt contained within the grip before tensioning, in millimetres. In this context, the grip includes the washer thickness

t_n = thickness of the nut, in millimetres

A_s = tensile stress area of the bolt as defined in AS 1275, in square millimetres.

It is not necessary for both bolts in the one specimen to have identical tension induced in them.

K1.3 Number of specimens

Tests on at least 3 specimens shall be undertaken, but 5 is preferred as a reliable minimum number.

K2 INSTRUMENTATION

K2.1

Two pairs of dial gauge micrometers or displacement transducers having an effective resolution achieving 0.003 mm or finer shall be symmetrically disposed over gauge lengths of $3d_f$ on each edge of the specimen so as to measure the deformation between the inner plates from the bolt positions to the centre of the cover plates. The deformation of each half of the joint shall be taken as the mean of the deformation at each edge. The deformation so measured is therefore the sum of the elastic extension of the cover plates and any slip at the bolt positions.

K2.2

Figure K2 shows a typically instrumented test specimen. It is essential that the micrometers or transducers be securely mounted since they may be shock loaded as slip occurs.

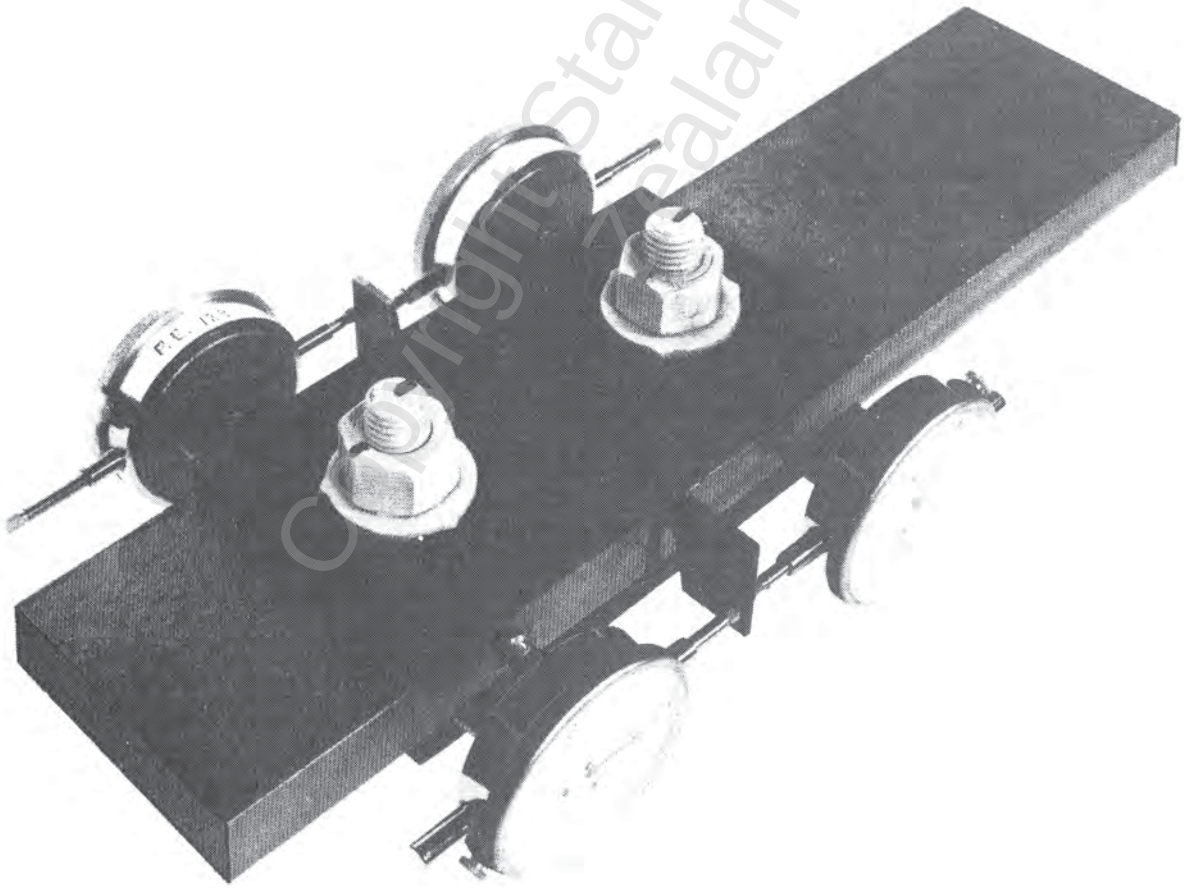


Figure K2 – Typically instrumented test assembly

K3 METHOD OF TESTING

The method of testing shall satisfy the following requirements:

(a) *Type of loading*
Specimens shall be tested only by tensile loading.

(b) *Loading rate*
Up to the slip load, force shall be applied in increments exceeding neither 25 kN nor 0.25 times of the slip load of the connection assuming a slip factor of 0.35 and the calculated bolt tension. The loading rate shall be approximately uniform at not more than 50 kN/min within each load increment. Slower loading rates are preferred. Each load increment shall be applied after creep at constant load due to the preceding load increment has effectively ceased.

Since slip will in all probability occur at one bolt position before the other, it is clear that the first bolt may slip into bearing before the slip load at the other bolt position is attained.

After attainment of the slip load at one bolt position, the loading rate and increment size may be adjusted at the discretion of the operator.

K4 SLIP LOAD

Slip is usually well defined and easily detected when a sudden increase in deformation occurs. One or more sharp clearly audible reports may also be heard. However, with some types of surface, and occasionally with normal surfaces, the incidence of slip is not so well defined. In these cases, the load corresponding to a slip of 0.13 mm shall be used to define the slip load.

K5 SLIP FACTOR

The slip factor (μ_s) to be used in design shall be calculated from:

$$\mu_s = k(\mu_{sm} - 1.64\delta)$$

where

k = 0.85 when 3 specimens are tested, or
= 0.90 when 5 or more specimens are tested

μ_{sm} = mean value of slip factor for all tests

δ = standard deviation of slip factor for all tests

$$\mu_{sm} = \frac{1}{2n}\left(\frac{V_{si}}{N_{ti}}\right)$$

$$\mu_{si} = \frac{1}{2}\left(\frac{V_{si}}{N_{ti}}\right)$$

$$\delta = \sqrt{\left[\frac{1}{2n-1}\sum_{i=1}^{2n}(\mu_{si} - \mu_{sm})^2\right]}$$

n = the number of specimens tested, each providing 2 estimates of μ

V_{si} = the measured slip load at the position of the i-th bolt

N_{ti} = the tension induced in the i-th bolt by the tensioning as calculated from equation K1.

However, if the calculated value of μ_s is less than the lowest of all values of μ_{si}, then μ_s may be taken as equal to the lowest value of μ_{si}.

APPENDIX L
INSPECTION OF BOLT TENSION USING A TORQUE WRENCH

(This Appendix forms an informative part of this Standard.)

L1 GENERAL

The correlation between the torque required to fully tension a calibration specimen and that which will be required on a bolt-nut assembly installed in a structural connection, will be materially affected by such factors as:

(a) The actual condition of the thread and the bearing face surface and their lubrication;

(b) The occurrence of galling during tensioning; and

(c) The time lapse between tensioning and inspection.

With due regard for these limitations, the procedure given in this Appendix is considered the most practical method for an independent assessment of whether gross undertensioning exists.

L2 CALIBRATION

L2.1

The inspection wrench may be either a hand-operated or an adjustable-torque power-operated wrench. It should be calibrated at least once per shift, or more frequently if the need to closely simulate the condition of the bolts in the structure so demands.

L2.2

The torque value determined during calibration may not be transferred to another wrench.

L2.3

At least 3 bolts, desirably of the same size (the minimum length may have to be selected to suit the calibrating device) and condition as those under inspection, should be placed individually in a calibrating device capable of indicating bolt tension. A hardened washer should be placed under the part turned.

L2.4

Each calibration specimen should be tensioned in the calibrating device by any convenient means to 1.05 times the minimum bolt tension specified for that diameter in table 15.2.5.1. The inspection wrench then should be applied to the tensioned bolt, and the torque necessary to turn the nut or bolt head 5° (approximately 25 mm movement at a 300 mm radius) in the tensioning direction should be determined. The average torque measured in the tests of at least 3 bolts should be taken as the job inspection torque.

L3 INSPECTION

Bolts represented by the sample which have been tensioned in the structure should be inspected by applying, in the tensioning direction, the inspection wrench with its job inspection torque to such proportion of the bolts in the structure as prescribed.

NOTE – For guidance, it is suggested that a suitable sample size would be 10 % of the bolts, but not less than 2 bolts in each connection.

L4 ACTION

L4.1

Where no further rotation occurs under the torque applied by the inspection wrench, the connection should be accepted as properly tensioned.

L4.2

Where any nut or bolt head is turned by the application of the job-inspection torque, this torque should then be applied to all other bolts in the connection and any bolt whose nut or head is turned by the job inspection torque should be tensioned and re-inspected. Alternatively, the fabricator or erector may retension all of the bolts in the connection and then resubmit the connection for inspection.

APPENDIX M

Amd 2
Oct. '07

(Deleted by Amendment No. 2)

APPENDIX N
SECTION PROPERTIES TO USE IN ULTIMATE AND SERVICEABILITY LIMIT STATE CALCULATIONS FOR DEFLECTION

(This Appendix forms a normative part of this Standard.)

For provisions requiring application of N1, refer to 12.3.2.1.5, 12.10.4, 12.10.6, 12.11.3.1, 12.12.5.2(f, g), 12.12.5.3(f, g), 12.12.5.4(e, f). For provisions requiring application of N2, refer to 3.4.2(b), 13.1.2.6.

N1 SECTION PROPERTIES FOR ULTIMATE LIMIT STATE DEFLECTION CALCULATIONS

N1.1 For lateral deflection from design load combinations including earthquake loads

N1.1.1 Bare steel members
For category 1, 2 or 3 bare steel beam and column members, use the gross properties of the cross section calculated in accordance with section 5 or section 6 as appropriate.

N1.1.2 Composite members
For composite beams and columns covered by this Standard, the appropriate section properties shall be as follows:

(a) For beams composite over the midspan region with shear connectors terminated at a distance of $1.5d$ away from the face of the support (refer to 13.4.11.3.3(a) or 13.4.11.3.4(a)), determine the section properties in accordance with one of (i), (ii) or (iii) below:

(i) Use 1.2 times the bare steel section properties; or

(ii) Determine the transformed gross properties of the composite section and non-composite section and combine 0.9 times the former with the latter in accordance with an appropriate design procedure (e.g. Reference (1)) to give section properties for the member as a whole; or

(iii) Model the variation in cross section properties calculated from (ii) directly in analysis.

(b) For beams composite over their full length, determine the section properties in accordance with one of (i), (ii) or (iii) below:

(i) Use 0.6 times the transformed gross properties of the composite section calculated for the effective width given in 13.4.2.1; or

(ii) Determine the transformed gross properties of the composite section in the negative and positive moment regions and combine 0.9 times these values in accordance with an appropriate design procedure (e.g. Reference (1)) to give the section properties for the member as a whole; or

(iii) Model the variation in cross section properties calculated from (ii) directly in analysis.

(c) For encased composite columns designed in accordance with 13.8.2, use 0.9 times the transformed gross properties of the composite section.

(d) For concrete-filled structural hollow columns designed in accordance with 13.8.3, use 1.0 times the transformed gross properties of the composite section.

In all instances, use the short term modulus of elasticity for concrete (E_c) when determining transformed composite section properties and include the influence of interfacial slip and partial shear connection, where appropriate.

N2 SECTION PROPERTIES FOR SERVICEABILITY LIMIT STATE DEFLECTION CALCULATIONS

N2.1 For lateral deflection from design load combinations including earthquake loads or other applied short term lateral loads

N2.1.1 Bare steel members

For all categories of member, use the gross properties of the cross section calculated in accordance with section 5 or section 6 as appropriate.

N2.1.2 Composite members

For composite beams and columns covered by this Standard, the appropriate section properties shall be as follows:

(a) For beams composite over the midspan region with shear connectors terminated at a distance of 1.5d away from the face of the support, determine the section properties in accordance with one of (i), (ii) or (iii) below:

(i) Use 1.3 times the bare steel section properties; or

(ii) Determine the transformed gross properties of the composite section and non-composite section and combine 1.0 times the former with the latter in accordance with an appropriate design procedure (e.g. Reference (1)) to give the section properties for the member as a whole; or

(iii) Model the variation in cross section properties calculated from (ii) directly in analysis.

(b) For beams composite over their full length, determine the section properties in accordance with one of (i), (ii) or (iii) below:

(i) Use 0.8 times the transformed gross properties of the composite section calculated for the effective width given in 13.4.2.1; or

(ii) Determine the transformed gross properties of the composite section in the negative and positive moment regions and combine 1.0 times these values in accordance with an appropriate design procedure (e.g. Reference (1)) to give the section properties for the member as a whole; or

(iii) Model the variation in cross section properties calculated from (ii) directly in analysis.

(c) For composite columns designed in accordance with 13.8, use 1.0 times the transformed gross properties of the composite section.

In all instances, use the short term modulus of elasticity for concrete (E_c) when determining transformed composite section properties and include the influence of interfacial slip and partial shear connection, where appropriate.

N2.2 For vertical deflection from design load combinations not including earthquake loads or applied short term lateral loads

N2.2.1 Bare steel members

Use the gross properties of the cross section, calculated in accordance with section 5 or section 6 as appropriate.

N2.2.2 Composite members

Use the appropriate short- or long-term transformed gross composite section properties applicable to the serviceability criterion being considered in accordance with 13.1.2.6.

N3 REFERENCE

1. Morrison, J. 1974. Design of Continuous Composite Beams for Buildings. Arup Journal, Vol. 9, No.2.

NOTES

APPENDIX P
ALTERNATIVE DESIGN METHOD

(This Appendix forms a normative part of this Standard.)

P1 SCOPE AND GENERAL

P1.1 Scope

P.1.1.1
Bare (non-composite) steelwork in applications within the scope of 1.1 may be designed using the alternative design method in accordance with this Appendix. Steelwork in structural systems which do not fall within the scope of this Appendix must be designed in accordance with the limit state design method.

P1.1.2
All sections and clauses of this Standard shall apply to the alternative design method unless directed otherwise by this Appendix.

P1.2 Definitions
The definitions in 1.3 shall apply, subject to the following modifications:

(1) Add the following new definition:

ALTERNATIVE DESIGN METHOD (alternatively known as the WORKING LOAD DESIGN METHOD) – a design method in which the required strength of a structural system is determined by matching the design working effects or loads against the permissible strength of the members of the system in accordance with the provisions of this Appendix.

(2) The definition of “Design capacity” is deleted and replaced with:

PERMISSIBLE STRENGTH (alternatively known as the ALLOWABLE STRENGTH) – the nominal capacity multiplied by the appropriate factor of safety from table P3.3.

(3) The definition of “Design effects or design loads” refers to the effects or loads derived for the alternative design method from the appropriate Loadings Standard.

(4) The definition of “Design resistance” is deleted and replaced with:

DESIGN RESISTANCE – the resistance computed from the loads and permissible strengths contributing towards the stability of the structure.

(5) The definition of “Loadings Standard” is deleted and replaced with:

LOADINGS STANDARD- a code of practice or other document approved by the authority which specifies nominal loads and load factors applicable to the alternative design method.

(6) The definition of “Nominal effect or load” is deleted and replaced with:

NOMINAL EFFECT OR LOAD – an effect or load as specified in P3.2.

(7) The definition of "Nominal capacity" is deleted and replaced with:

NOMINAL CAPACITY – the capacity of a member or connection computed excluding the factor of safety.

(8) The definition of "Severe earthquake loads" is deleted and replaced with:

SEVERE EARTHQUAKE LOADS OR SEVERE SEISMIC LOADS – the earthquake loads (resulting from application of earthquake – induced ground motion) applicable to the design for adequate strength to the alternative design method in accordance with the Loadings Standard.

(9) The definition of "Strength reduction factor" is deleted and replaced with:

FACTOR OF SAFETY – a factor used to multiply the nominal capacity to obtain the permissible strength.

(10) The definition of "Serviceability limit state" is deleted and replaced with:

DESIGN FOR SERVICEABILITY – the requirement under the alternative design method to ensure a satisfactory in-service condition is achieved.

(11) The definition of "Ultimate limit state" is deleted and replaced with:

DESIGN FOR STRENGTH – the requirement under the alternative design method to ensure that collapse, loss of structural integrity or loss of static equilibrium is prevented.

P1.3 Notation

The symbols in 1.4 shall apply, subject to the following modifications:

(1) A superscripted '*' placed after a symbol denotes a design action or a design force appropriate to the alternative design method.

(2) Delete the defined notation for ϕ and replace with:

Ω = factor of safety for obtaining the permissible strength from the nominal capacity.

P2 MATERIALS AND BRITTLE FRACTURE

Apply section 2 without modification.

P3 GENERAL DESIGN REQUIREMENTS

P3.2 Loads and other effects

P3.2.1 Loads

The design of a structure to the alternative design method for strength and stability shall account for the actions directly arising from the following loads:

(a) Dead, live, wind, earthquake, snow, rain, ice, soil and hydrostatic loads specified in the Loadings Standard;

(b) For the design of cranes, any additional or alternative relevant loads specified in NZS/BS 2573 or AS 1418 as appropriate;

➤

(c) For the design of fixed platforms, walkways, stairways and ladders, any additional or alternative relevant loads specified in AS 1657;

(d) For the design of lifts, any relevant loads specified in NZS 4332P;

(e) Other specific loads, as required.

NOTE –

(1) For the design of bridges, loads specified in the Transit New Zealand Bridge Manual: Design and Evaluation (for road bridges) or in the New Zealand Rail Ltd : Railnet Code, Part 4, Code Supplements Bridges and Structures, Section 2 : Design (for rail bridges), as applicable, should be used.

(2) For multi-storey building structures, see also P3.2.4.

P3.2.2 Other loads or effects

Any load or effect which may significantly affect the strength or serviceability of the structure, including the following, shall be taken into account:

(a) Foundation movements;

(b) Temperature changes and gradients;

(c) Axial shortening, both elastic and inelastic (under severe seismic forces);

(d) Dynamic effects, other than as already covered in P3.2.1;

(e) Construction loading.

P3.2.3 Design load combinations

The design load combinations for strength and serviceability shall be those applicable to the alternative design method as specified in the Loadings Standard.

P3.2.4 Notional horizontal loads (for design load combinations involving only vertical loads)

P3.2.4.1 *On multi-storey building structures*

P3.2.4.1.1

For building structures comprising 2 or more floor levels and being designed for load combinations involving only loads acting in the vertical direction, notional horizontal loads shall be considered to act in conjunction with the appropriate vertical loads at each floor level, in design for strength.

P3.2.4.1.2

The notional horizontal load at each level shall be taken as 0.002 times the total design vertical load, at that level, for the load combination under consideration.

NOTE – The substructure shown in figure C4.3.1, Part 2 of this Standard, offers an alternative approach to application of these notional horizontal forces for regular, rectangular internal frames being designed for load combinations involving only vertical loads.

P3.2.4.1.3

For a perimeter frame (see 1.3) the design vertical load for use in P3.2.4.1.2 shall be calculated for the tributary floor area supported laterally by the frame.

P3.2.4.1.4
The notional horizontal loads shall be applied at both the construction and occupancy stages.

P3.2.4.2 *On vertical cantilevers*

P3.2.4.2.1
For vertical cantilevers being designed for load combinations involving only vertical loads, notional horizontal loads shall be considered to act in conjunction with the appropriate vertical loads, at a given level up the height of the cantilever, in design for strength.

P3.2.4.2.2
The notional horizontal load at a given level shall be taken as 0.005 times the total design vertical load, at that level, from the load combination under consideration.

P3.3 Design for strength

The structure and its component members shall be designed for strength under the alternative design method as follows:

(a) The nominal loads and effects shall be determined in accordance with P3.2.1 and P3.2.2 and the design loads and effects shall be determined in accordance with P3.2.3 and P3.2.4;

(b) The design actions (S^*) resulting from the design loads for strength shall be determined by an analysis in accordance with P4 and, where appropriate, P12.3;

(c) The permissible strength (ΩR_u) shall be determined from the nominal capacity determined from P5 to P9 and P12 as appropriate, where the factor of safety (Ω) shall not exceed the appropriate value given in table P3.3;

(d) All members and connections shall be proportioned so that the permissible strength (ΩR_u) is not less than the design action (S^*), i.e.

$$S^* \leq \Omega R_u$$

(e) For structures that are required to respond in a nominally ductile manner to severe earthquake loads, the level of ductility demand on the structure and parts thereof shall be determined from P12.2 and provided for in accordance with P12 of this Standard;

(f) The structure as a whole (and any part of it) shall be designed to prevent instability due to overturning, uplift or sliding, as appropriate to the alternative design method, in accordance with the Loadings Standard.

Table P3.3 – Alternative design method factors of safety (Ω) for design of bare steel members and connections

Permissible strength for	Clause	Factor of safety (Ω)	
Member subject to bending	P5	0.60	
Member subject to shear	P5	0.62	
Member subject to axial forces	P6, P7	0.60	
Member subject to combined actions	P8	0.60	
Connection component other than a bolt, pin or weld	P9.1	0.60	
Bolted connection			
– bolt in shear, tension or combined shear and tension	P9.3	0.53	
– ply in bearing	P9.3	0.60	
– bolt group	P9.4	0.53	
Pin connection			
– pin in shear, bending, bearing	P9.5	0.53	
– ply in bearing	P9.5	0.60	
		SP Category	GP Category
Welded connection			
– complete penetration butt weld	P9.7	0.60	0.40
– all other forms of weld	P9.7	0.53	0.40
– weld group	P9.8	0.53	0.40

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P3.4 Design for serviceability

P3.4.1 General

The structure and its components shall be designed for serviceability by controlling or limiting deflection, vibration, bolt slip and corrosion, as appropriate, in accordance with the relevant requirements of P3.4.2.

P3.4.2 Method

The structure and its components shall be designed for serviceability under the alternative design method as follows:

(a) The loads and other effects shall be determined in accordance with P3.2.1 and P3.2.2, and the serviceability design loads shall be determined from P3.2.3 and P3.2.4.

(b) Deflections due to the serviceability design loads shall be determined by the elastic analysis method of 4.4, with all amplification factors taken as unity, except where limited moment redistribution is allowed by 4.5.5. Deflections shall comply with 3.4.3.

(c) Vibration behaviour shall be assessed in accordance with 3.4.4.

(d) Bolt slip shall be limited, where required, in accordance with 3.4.5.

(e) Corrosion protection shall be provided in accordance with 3.4.6.

P4 STRUCTURAL ANALYSIS

P4.1

Under the alternative design method, the design actions in a structure and its members and connections shall be determined by structural analysis using the provisions of 4.2 and 4.3 and one of the methods (a) or (b) following:

(a) Elastic analysis, in accordance with 4.4; or

(b) Elastic analysis with limited moment redistribution only, in accordance with 4.5.4.2.1 for a category 3 member.

P4.2

In applying the provisions of section 4, the terms “ultimate limit state” and “serviceability limit state” are replaced with the terms “design for strength” and “design for serviceability” in accordance with P1.2.

P5 MEMBERS SUBJECT TO BENDING AND SHEAR

P5.1 Design for bending moment

P5.1.1

A member bent about the section major principal *x*-axis shall satisfy:

$M_x^* \leq \Omega M_{sx}$, and

$M_x^* \leq \Omega M_{bx}$

where

M_x^* = the design bending moment appropriate to the alternative design method about the *x*-axis determined in accordance with P4

Ω = the factor of safety (see table P3.3)

M_{sx} = the nominal section capacity in bending, as specified in 5.2, for bending about the *x*-axis

M_{bx} = the nominal member capacity in bending, as specified in 5.3 or 5.6, for bending about the *x*-axis.

P5.1.2

A member bent about the section minor principal *y*-axis shall satisfy:

$M_y^* \leq \Omega M_{sy}$

where

M_y^* = the design bending moment appropriate to the alternative design method about the *y*-axis determined in accordance with P4

M_{sy} = the nominal section capacity in bending, as specified in 5.2, for bending about the *y*-axis.

P5.1.3
A member whose deflections are constrained to a non-principal plane shall be analysed as specified in 5.7.1, and shall satisfy 8.3.4 as modified by P8.

P5.1.4
A member which is bent about a non-principal axis and whose deflections are unconstrained shall be analysed as specified in 5.7.2, and shall satisfy 8.3.4 and 8.4.5 as modified by P8.

P5.1.5
A member subjected to combined bending and shear shall satisfy the requirements of this clause and 5.12 as modified by P5.12.

P5.1.6
A member subject to combined bending and axial compression or tension shall satisfy section 8, as modified by P8.

P5.11 Design of webs for shear

A web subject to a design shear force (V^*) shall satisfy:

$$V^* \leq \Omega V_v$$

where

Ω = the factor of safety (see table P3.3)

V_v = the nominal shear capacity of the web determined from either 5.11.2 or 5.11.3.

P5.12 Interaction of shear and bending

P5.12.1 General

The nominal web shear capacity in the presence of bending moment shall be calculated using the provisions of P5.12.2.

P5.12.2 Design requirement

P5.12.2.1
When the bending moment is assumed to be resisted only by the flanges and the design bending moment (M^*) satisfies:

$$M^* \leq \Omega M_f$$

where M_f is the nominal moment capacity calculated for the flanges alone, from P5.12.2.2, the member shall satisfy:

$$V^* \leq \Omega V_v$$

where

V_v is the nominal web shear capacity determined either from 5.11.2 or from 5.11.3.

P5.12.2.2 *Nominal moment capacity for the flanges alone*
The nominal moment capacity for the flanges alone is calculated as follows:

$M_f = A_{fm}\, d_f\, f_y$

where

A_{fm} = the lesser of the flange effective areas, determined using 6.2.2 for the compression flange and the lesser of A_{fg} and 0.85 $A_{fn} f_u / f_y$ for the tension flange

A_{fg} = the gross area of the flange

A_{fn} = the net area of the flange

d_f = the distance between flange centroids.

P5.13 Compressive bearing action on the edge of a web
Use the provisions of 5.13, substituting Ω from table P3.3 for ϕ from that clause and modifying section 6 in accordance with P6.

P5.14 Design of load bearing stiffeners
Use the provisions of 5.14, substituting Ω from table P3.3 for ϕ from that clause and modifying section 6 in accordance with P6.

P5.15 Design of intermediate transverse web stiffeners
Use the provisions of 5.15, substituting Ω from table P3.3 for ϕ from that clause and modifying section 6 in accordance with P6.

P5.15.8 Connection of intermediate stiffeners to web
Multiply the weld design shear force per unit length given in 5.15.8 by $\Omega = 0.60$.

P6 MEMBERS SUBJECT TO AXIAL COMPRESSION

P6.1 Design for axial compression
A concentrically loaded member subject to a design axial compression force (N^*) shall satisfy both:

$N^* \leq \Omega N_s$, and

$N^* \leq \Omega N_c$

where

Ω = the factor of safety (see table P3.3)

N_s = the nominal section capacity determined in accordance with 6.2

N_c = the nominal member capacity determined in accordance with 6.3.

P6.4 Laced and battened compression members

P6.4.1 Design forces
Use the provisions of 6.4.1, except that N^* is applicable to the alternative design method.

P6.4.2 Laced compression members
Use the provisions of 6.4.2.

P6.4.3 Battened compression members
Use the provisions of 6.4.3, except that V^* is calculated in accordance with P6.4.1.

P6.5 Compression members back to back
Use the provisions of 6.5, except that V^* is calculated in accordance with P6.4.1.

P6.6 Discontinuous angle, channel and tee section compression members not requiring design for moment action
Use the provisions of 6.6, subject to the following modifications:

(1) The design compression load (N^*) is that applicable to the alternative design method.

(2) Substitute the permissible member strength (ΩN_c) for (ϕN_c) in 6.6.2(b).

P7 MEMBERS SUBJECT TO AXIAL TENSION

P7.1 Design for axial tension
A member subject to an axial tension force (N^*) shall satisfy:

$$N^* \leq \Omega N_t$$

where

Ω = the factor of safety (see table P3.3)

N_t = the nominal section capacity in tension determined in accordance with 7.2.

P7.4 Tension member with 2 or more main components
Use the provisions of 7.4, modifying the referenced provisions of section 6, where appropriate, by those of P6.

P7.5 Members with pin connections
Use the provisions of 7.5, modifying referenced 9.5, where appropriate, by the provisions of P9.

P8 MEMBERS SUBJECT TO COMBINED ACTIONS

P8.1 General

P8.1.1
A member subject to combined axial and bending actions shall be proportioned so that its design actions specified in P8.2, in combination with the nominal section and member capacities (see P5-P7), satisfy P8.3 and P8.4.

P8.1.2
Eccentrically loaded single angles subject to design compression only and with slenderness ratios (L/r_y) < 150 shall be designed to P8.3 and P8.4.6.

P8.1.3
Eccentrically loaded single angles subject to design compression only and with slenderness ratios (L/r_y) ≥ 150 shall be designed to P6.6.

P8.2 Design actions

These shall be determined in accordance with 8.2 from loads applicable to the alternative design method.

P8.3 Section capacity

P8.3.1 General

The member shall satisfy P8.3.2, P8.3.3 and P8.3.4, as appropriate:

(a) For bending about the major principal *x*-axis only, sections at all points along the member shall have sufficient strength to satisfy P8.3.2;

(b) For bending about the minor principal *y*-axis only, sections at all points along the member shall have sufficient strength to satisfy P8.3.3;

(c) For bending about a non-principal axis, or bending about both principal axes, sections at all points along the member shall have sufficient strength to satisfy P8.3.4.

In this section:

M_{sx}, M_{sy} = the nominal section moment capacities about the *x*- and *y*- axes respectively, determined in accordance with 5.2.

N_s = the nominal section axial capacity determined in accordance with 6.2 for axial compression, or 7.2 for axial tension (for which $N_s = N_t$).

P8.3.2 Uniaxial bending about the major principal x-axis

Use the provisions of 8.3.2.1, subject to the following modifications:

(1) Substitute Ω from table P3.3 for ϕ from that clause.

(2) Do not apply the alternative provision of 8.3.2.2.

P8.3.3 Uniaxial bending about the minor principal y-axis

Use the provisions of 8.3.3.1, subject to the following modifications:

(1) Substitute Ω from table P3.3 for ϕ from that clause.

(2) Do not apply the alternative provision of 8.3.3.2.

P8.3.4 Biaxial bending

Use the provisions of 8.3.4.1, subject to the following modifications:

(1) Substitute Ω from table P3.3 for ϕ from that clause.

(2) Do not apply the alternative provision of 8.3.4.2.

P8.4 Member capacity

P8.4.1 General
The member shall satisfy P8.4.2 and P8.4.4, as appropriate:

(a) For a member bent about the major principal *x*-axis only and which has full lateral restraint in accordance with 8.1.2.3, or for a member bent about the minor principal *y*-axis only, the member shall satisfy the in-plane requirements of P8.4.2;

(b) For a member bent about the major principal *x*-axis only and which does not have full lateral restraint, the member shall satisfy both the in-plane requirements of P8.4.2 and the out-of-plane requirements of P8.4.4;

(c) For a member bent about a non-principal axis, or bent about both principal axes, the member shall satisfy the biaxial bending requirements of P8.4.5.

P8.4.2 In-plane capacity–elastic analysis
Use the provisions of 8.4.2.2.1 or 8.4.2.3, subject to the following modifications:

(1) Substitute Ω from table P3.3 for ϕ from that clause.

(2) Do not apply the alternative provision of 8.4.2.2.2.

P8.4.4 Out-of-plane capacity
Use the provisions of 8.4.4.1.1 or 8.4.4.2, subject to the following modifications:

(1) Substitute Ω from table P3.3 for ϕ from that clause.

(2) Do not apply the alternative provision of 8.4.4.1.2.

P8.4.5 Biaxial bending capacity
Use the provisions of 8.4.5, substituting Ω from table P3.3 for ϕ from that clause.

P8.4.6 Double bolted or welded single angles eccentrically loaded in compression
Use the provisions of 8.4.6, substituting Ω from table P3.3 for ϕ from that clause.

P9 CONNECTIONS

P9.1 General

P9.1.1 Requirements for connections
Connection elements consist of connection components (cleats, gusset plates, brackets, connecting plates) and connectors (bolts, pins and welds). The connections in a structure shall be proportioned so as to be consistent with the assumptions made in the analysis of this structure and to comply with section 9, as modified by P9, and 12.3, as modified by P12.3.

P9.1.2 Classification of connections
Use the provisions of 9.1.2, except that 9.1.2.5 is not applicable.

P9.1.3 Design of connections
The provisions of 9.1.3 apply.

P9.1.4 Minimum design actions on connections

Use the provisions of 9.1.4, subject to the following modification:

(1) The permissible member strength is substituted for the member design capacity in 9.1.4.1.

P9.1.5 Intersections; through to

P9.1.8 Prying forces

The provisions of 9.1.5 to 9.1.8, respectively, apply.

P9.1.9 Connection components

Connection components (cleats, gusset plates, brackets, etc.) other than connectors shall have their permissible strengths assessed using the provisions of P5, P6, P7, P8 and the appropriate provisions of P12 as applicable.

P9.1.10 Deductions for fastener holes

Use the provisions of 9.1.10.

P9.1.11 Hollow section connections

The provisions of 9.1.11 apply.

P9.3 Design of bolts

Use the provisions of 9.3, subject to the following modifications:

(1) For 9.3.1, 9.3.2 substitute Ω from table P3.3 for ϕ from that clause.

(2) For 9.3.3, use ϕ as specified therein.

(3) Any clauses referenced from 9.3.1 – 9.3.3 must be modified as required by this Appendix.

P9.4 Assessment of the strength of a bolt group

Use the provisions of 9.4, substituting Ω from table P3.3 for ϕ from that clause.

P9.5 Design of a pin connection

Use the provisions of 9.5, subject to the following modifications:

(1) Substitute Ω from table P3.3 for ϕ from that clause.

(2) Any clauses referenced from 9.5 must be modified as required by this Appendix.

P9.6 Design details for bolts and pins

The provisions of 9.6 apply.

P9.7 Design of welds

Use the provisions of 9.7, subject to the following modifications:

(1) Substitute Ω from table P3.3 for ϕ from that clause.

(2) Any clauses referenced from 9.7 must be modified as required by this Appendix.

P9.8 Assessment of the strength of a weld group

Use the provisions of 9.8, substituting Ω from table P3.3 for ϕ from that clause.

P10 FATIGUE

Fatigue assessments for the alternative design method must be undertaken in accordance with section 10.

P11 FIRE

The provisions of section 11 must only be applied as part of a limit state design procedure.

P12 SEISMIC DESIGN

P12.2 General design and analysis philosophy

Limited application of the alternative design method may be made, by applying the provisions of 12.2 in the following manner;

(1) Application is restricted to category 3 or 4 steel seismic-resisting systems in accordance with 12.2.3.1(3) or (4) and 12.2.3.3(3) or (4).

(2) Only category 3 or 4 members to 12.2.5 may be used.

(3) Clause 12.2.6 applies within the limited scope of (1) and (2) above.

(4) Capacity design (12.2.7) shall not be applied.

(5) Overstrength factors (12.2.8) are not necessary.

(6) The provisions of 12.2.9 shall be applied.

P12.3 Methods of analysis and design

Limited application of the alternative design method may be made, by applying the provisions of 12.3 in the following manner:

(1) Clauses 12.3.2, 12.3.3 are restricted in application to category 3 or 4 seismic-resisting systems not exceeding 2 storeys in height and which contain only category 3 or 4 members.

(2) Clause 12.3.4 is restricted in application to category 4 associated structural systems only.

P12.4 Material requirements

The provisions of 12.4 are applicable to category 3 or 4 members designed to the alternative design method.

P12.5 Section geometry requirements

The provisions of 12.5 are applicable to category 3 or 4 members designed to the alternative design method.

P12.6 Member restraint requirements

The provisions of 12.6.2 are applicable to category 3 members designed to the alternative design method.

The provisions of 12.6.3 apply.

P12.7 Beams

Clause 12.7 applies within the scope of P12.2 and P12.3, substituting Ω from table P3.3 for ϕ from that clause.

P12.8 Columns
Clause 12.8 applies within the scope of P12.2 and P12.3, substituting Ω from table P3.3 for ϕ from that clause.

P12.9 Connections and built-up members
Clause 12.9 applies within the scope of P12.2 and P12.3 and subject to the following modifications:

(1) Substitute Ω from table P3.3 for ϕ from that clause.

(2) The permissible member strength is substituted for the member design capacity in 12.9.2.

(3) Any clauses referenced from 12.9 must be modified as required by this Appendix.

P12.10 Design of moment-resisting framed seismic-resisting systems
The provisions of 12.10 are only applicable to category 3 or 4 systems.

P12.12 Design of concentrically braced framed seismic-resisting systems
The provisions of 12.12 are only applicable to category 3 or 4 systems.

P12.13 Dual seismic-resisting systems
The alternative design method shall not be used in the design of dual seismic-resisting systems.

P12.14 Fabrication in yielding regions
Clause 12.14 applies to category 3 members designed to the alternative design method.

P13 DESIGN OF COMPOSITE MEMBERS AND STRUCTURES
There are no provisions given in this Standard for design of composite members and structures to the alternative design method.

P14 FABRICATION
The provisions of section 14 apply.

P15 ERECTION
The provisions of section 15 apply.

P17 TESTING OF STRUCTURES OR ELEMENTS
Application of section 17 shall be in accordance with limit state design procedures only.

PA APPLICATION OF APPENDICES
When using the alternative design method, the following provisions apply to Appendices A-N of this Standard:

(1) Appendices A, C, D, K, L are generally applicable.

(2) Appendix B is limited in application by P12.

(3) Appendices E, F, G are limited in application by P4.

(4) Appendices H, J are limited in application by P5.

(5) Appendix M is applicable, with care, substituting Ω from table P3.3 for ϕ from that Appendix.

(6) Clause N1.1.1 of Appendix N applies when designing for strength and N2.1.1 or N2.2.1, as appropriate, apply when designing for serviceability.

(7) Appendix Q is not applicable.

APPENDIX Q
CORRESPONDING DETAIL FROM AS 4100

(This Appendix forms an informative part of this Standard)

Q1 SCOPE AND PURPOSE

This Appendix lists the corresponding clause (or equation, or section, or table, or figure) from AS 4100:1990, including Amendment Nos. 1, 2 and 3. This listing covers details from clause 2.6, sections 3 to 11 and from section 14 to Appendix L of this Standard.

This identification is made for the sole purpose of allowing design guidance written for use with AS 4100 to be used in conjunction with this Standard, by substituting the appropriate NZS 3404 clause, equation, section, table or figure for the corresponding AS 4100 detail referenced by the particular design guidance publication.

This identification does not mean that the corresponding AS 4100 detail is equivalent to that from NZS 3404 in all design applications, especially applications requiring design for earthquake loads or effects or applications involving composite action between steel sections and concrete. (Design guidance written for use with AS 4100 does not address the specific requirements of NZS 3404 sections 12 and 13).

The corresponding AS 4100 detail is given herein only for details from AS 4100 which are referenced in design guidance written for use with AS 4100 and which is routinely used in New Zealand. For this reason, only some of the clauses, equations, sections, tables or figures presented in this Standard appear in this Appendix.

Q2 CORRESPONDING AS 4100 CLAUSE

This listing is given in table Q2.

Table Q2 – Corresponding clauses

AS 4100 clause	Corresponding NZS 3404 clause
3.1 – 3.2	3.1 – 3.2
3.3 – 3.4	3.3
3.5	3.4
4.1	4.1
4.1.1 – 4.1.2	4.1.1 – 4.1.2
4.2	4.2
4.2.1	4.2.1
4.2.2 – 4.2.4	4.2.2
4.2.5	4.2.3
4.3	4.3
4.3.1 – 4.3.4	4.3.1 – 4.3.4
4.4	4.4
4.4.1.1	4.4.1
4.4.1.2	4.4.2
4.4.2	4.4.3
4.5	4.6
4.5.1	4.6.1
4.5.2	4.6.2
4.5.3	4.6.3
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5.10	5.10	9.6	9.6
5.10.1 – 5.10.7	5.10.1 – 5.10.7	9.6.1 – 9.6.5	9.6.1 – 9.6.5
5.11	5.11	9.7	9.7
5.11.1 – 5.11.5	5.11.1 – 5.11.5	9.7.1 – 9.7.5	9.7.1 – 9.7.5
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5.14	5.14	11.3	10.3
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Q3 CORRESPONDING AS 4100 EQUATIONS

This listing is given in table Q3.

Table Q3 – Corresponding equations

AS 4100 equation	Corresponding NZS 3404 equation
5.6.1.1(1)	5.6.1.1(1)
5.6.1.1(2)	5.6.1.1(3)
5.6.1.1(3)	5.6.1.1(4)

Q4 CORRESPONDING AS 4100 SECTIONS OR APPENDICES

This listing is given in table Q4.

Table Q4 – Corresponding sections or appendices

AS 4100 section or appendix	Corresponding NZS 3404 section or appendix	AS 4100 section or appendix	Corresponding NZS 3404 section or appendix
10	2.6	App. F	App. F
16	16	App. G	App. G
17	17	App. J	App. K
App. C	App. C	App. K	App. L
App. E	App. E		

Q5 CORRESPONDING AS 4100 TABLES

This listing is given in table Q5.

Table Q5 – Corresponding tables

AS 4100 table	Corresponding NZS 3404 table	AS 4100 table	Corresponding NZS 3404 table
4.6.3.4	4.8.3.4	6.2.4	6.2.4
5.2	5.2	6.3.3(1) & 6.3.3(2)	6.3.3(1)
5.6.1	5.6.1	6.3.3(3)	6.3.3(2)
5.6.2	5.6.2	7.3.2	7.3.2
5.6.3(1)	5.6.3(1)	10.4.1	2.6.4.1
5.6.3(2)	5.6.3(2)	10.4.4	2.6.4.4
5.6.3(3)	5.6.3(3)		

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Q6 CORRESPONDING AS 4100 FIGURES

This listing is given in table Q6.

Table Q6 – Corresponding figures

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4.6.3.2	4.8.3.2	5.13.1.2	5.13.1.2
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This Standard was prepared under the supervision of the Steel Structures Committee (P 3404) for the Standards Council established under the Standards Act 1988. The Committee consisted of:

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FOREWORD

This Commentary is intended to be read in conjunction with NZS 3404:Part 1:1997 *Steel Structures Standard* (hereinafter referred to as NZS 3404:Part 1).

The purposes of this Commentary are to:

(a) Provide background reference material to the clauses in NZS 3404:Part 1;

(b) Indicate the origins of particular requirements;

(c) Explain the application of certain clauses;

(d) Provide some assistance in the use of NZS 3404:Part 1;

(e) Where Part 1 specifies the use of appropriate or rational design procedures or member design provisions, Part 2 provides details which satisfy the intent of Part 1, either directly or by reference to relevant published documents;

(f) Provide a method of compliance for a number of clauses which specify performance levels only; and

(g) Reference appropriate sources of additional design guidance.

The Standard and Commentary draw on the provisions of AS 4100:1990 and for non-seismic applications the two Parts are generally compatible. A new Appendix (Appendix Q) has been added, giving more specific details of the corresponding AS 4100 detail to that given in this Standard.

The clause numbers and titles used in this Commentary are the same as those used in NZS 3404:Part 1 except that the clause numbers are prefixed by the letter C, e.g. C4.4.1. Not all clauses in Part 1 of this Standard have corresponding commentary guidance. Commentary Clauses C8.5, C14.5, C14.6, C15.6 and C15.7 have no corresponding clauses in Part 1.

The users of this Standard should ensure that their copies of the above-mentioned New Zealand Standards and referenced overseas Standards are the latest revisions or include the latest amendments. Such amendments are listed in the annual Standards New Zealand Catalogue which is supplemented by lists contained in the monthly magazine *Standards Update* issued free of charge to committee and subscribing members of Standards New Zealand.

NEW ZEALAND STANDARD
STEEL STRUCTURES – COMMENTARY

(Commentary to NZS 3404:Part 1:1997)

C1 SCOPE AND GENERAL

C1.1 SCOPE

The Standard sets out the minimum general requirements for the limit state design, fabrication, erection, and modification of safe, serviceable and durable steel structures. There may be additional requirements not specifically covered that also have to be considered by design engineers.

Design requirements for road bridges are covered by the Transit New Zealand Bridge Manual: Design and Evaluation, while railway bridges are covered by the New Zealand Rail Ltd : Railnet Code, Part 4; Code Supplements Bridges and Structures, Section 2 : Design. Both documents may not be directly compatible in all instances with the provisions of this Standard and some design engineer interpretation may be required in this regard.

This Standard gives member and structure design requirements for general application of steelwork. For some widely used forms of specialised steel structures, specific design requirements have been produced and gained wide acceptance as internationally accepted industry Standards. An example is transmission towers, where two publications (1.1, 1.2) give specific member and structure provisions that are more appropriate to use than the corresponding provisions of this Standard. As transmission tower designs require verification by test, or by a computer analysis method which has been confirmed by test data (1.3), these procedures (1.1, 1.2) can be used to meet the required level of reliability against failure for ultimate limit state design.

Steel elements less than 3 mm in thickness, unless designed to AS 1163, are excluded from this Standard. This is because such elements are usually cold-formed and have high slenderness ratios, thus necessitating a different design approach. Design of these elements is covered by AS/NZS 4600 *Cold-formed steel structures standard*.

The limit of 450 MPa for the yield stress used in design, except as noted in 1.1.4 (b), stems from a lack of research data on steel grades above this value, meaning that not all the member design provisions presented herein can be currently confirmed as applicable to steels with higher yield stress. The great majority can, however; designers wishing to use steels with $f_y > 450$ should read (1.5) and then use this Standard to the extent recommended therein. Alternatively, use a limit state design provision which covers high strength steels, e.g. the American Institute of Steel Construction LRFD Specification (1.4) which covers steels up to f_y = 690 MPa.

The clause does not preclude the use of steels having a specified yield stress greater than 450 MPa provided that the yield stress used in design (f_y) is limited to 450 MPa. Note, however, that the use of a steel having a specified yield stress greater than 350 MPa is specifically excluded from plastic design by 4.6.2 and the use of grade 450 MPa steel is considerably restricted for use in applications involving earthquake loads by 12.4.

Quenched and tempered steels used as splice cover plates in bolted connections have shown satisfactory behaviour and are permitted to be used in that application, by 1.1.4 (b), with the yield stress in design taken as appropriate to that grade of steel (f_y = 690 MPa). Suitably conservative criteria relating to bearing stresses have been applied, due to a lack of experimental data, to substantiate the use of the higher bearing stresses from 9.3.2.4 2 associated with lower strength grades of steel.

Hollow section members designed to AS 1163 are most commonly cold-formed, but have traditionally been designed using the 1989 edition of this Standard since they were, for many years, hot-formed. Tests carried out on members manufactured to AS 1163 confirm the applicability of the provisions of this Standard to such members. Similarly, cold-formed hollow members to BS 6363 or JIS G3141, with wall thickness over 3 mm, may be designed in accordance with this Standard.

This Standard is not intended to be used for thin-walled shell or plate structures, since such structures are subject to failure modes not addressed in this Standard. It is, however, considered reasonable to design floor plates using this Standard. (See Introduction to Commentary on section 5.)

C1.2 REFERENCED DOCUMENTS

The Standards listed in Appendix A are subject to revision from time to time and the current issue should always be used. The latest revision of any Standard may be checked with Standards New Zealand. Standards New Zealand can also advise on the status of other documents referenced in Appendix A.

C1.3 DEFINITIONS

Amd 2 Oct. '07

NOTE – Changes are required for alignment with the AS/NZS 1170 set and, in the case of design action, to correct an ambiguity in the Standard.

Technical definitions are provided in 1.3. Some technical definitions which are applicable to only one section are also given in the section in which they are relevant. A number of the terms defined are common to other Standards, such as the Loadings Standard, AS/NZS 1170 set. Amd 2 Oct. '07

Further guidance on certain of the terms defined in 1.3 now follows:

BRACED MEMBER. When undertaking structural analysis in accordance with section 4, members of structural systems must be classed as either braced or sway (see 4.1.2). For structural systems other than seismic-resisting systems, a braced member applies to triangulated frames and trusses or to frames where in-plane stiffness is provided by diagonal bracing, or by shear walls or by floor slabs or roof ducks secured horizontally to bracing systems parallel to the plane of buckling of the member.

For seismic-resisting structural systems under seismic loading, considerable lateral displacement may be generated, especially in a system responding inelastically to severe seismic effects, causing $P - \Delta$ effects to become significant. This applies to both conventional "sway" systems (e.g. moment-resisting frames) and conventional "braced" systems (e.g. concentrically or eccentrically braced frames). The provisions of this Standard only enable $P - \Delta$ effects to be evaluated for members in sway systems, hence most members of seismic-resisting systems are therefore classified as sway members in applying section 4 (structural analysis).

DESIGN ENGINEER. Throughout this Standard, the term designer may be replaced by design engineer, although the designer may also designate a person working under the authority of the design engineer.

IDEAL CAPACITY FACTOR. The ideal capacity factor applies to the ultimate limit state expression $S^* \leq \phi R_u$ (see 3.3(d)) when S^* is determined from application of the capacity design philosophy given in 12.2.7. It may be applied either directly to the design capacity, ϕR_u, of the secondary member or element resisting the capacity design derived design action S^*, or as the reciprocal value, (0.9/1.0) = 0.9, to the overstrength factor used to derive S^*. The former approach was used in NZS 3404:1992, while the latter approach is now adopted in this revision.

LOAD. The definition of load includes both direct externally applied load, such as live load, or an externally applied force, such as wind or soil load. It may also include equivalent static seismic forces, as calculated in accordance with the Loadings Standard, which are in fact generated by the building's inertial response to seismic-induced displacements although these are generally referred to as seismic forces.

Amd 2 Oct. '07

LOADINGS STANDARD. It will be normal practice for the Loadings Standard to be AS/NZS 1170 set. This Standard is written so as to be applicable to other loadings Codes or Standards, however, provided they are written in limit state format, or applied so as to give design reliability consistent with that from formal limit state design (see Commentary Clause C3.1 for these details).

P – DELTA EFFECT. There are two types of P – delta effect defined, namely that applicable to the lateral displacement of the structural system as a whole ($P - \Delta$) and that applicable to an individual member ($P - \delta$). Refer to Commentary Clause C4.4.2 for a more detailed explanation of the two effects.

PERIMETER FRAME. The definition as used in this Standard relates only to the role of the frame in resisting applied lateral loads. It may apply to any form of structural system used for that purpose, e.g. a moment-resisting, eccentric or concentrically braced frame.

PRYING FORCE. Prying force arises due to the flexing of a connection component in a connection subjected to external tension. The external tension force reduces the contact pressure between the component and its base, and bending in part of the component develops a prying force near the edge of the connection component. The magnitude of prying force is very dependant on the connection detail and the magnitude of the tension force; typically its influence becomes minimal as the ultimate limit state capacity of the connection is reached. Its action must be considered in design of bolted end-plate connections.

Amd 2 Oct. '07

RECTANGULAR FRAME. The amount by which a beam must deviate from the horizontal before axial forces become significant depends on the structural form and loading, with scope for design engineer judgement in each particular application.

Generally, any frame with beams oriented no greater than 5^{o} from the horizontal will be rectangular, whereas a frame with beams oriented at 10^{o} or more from the horizontal will be non-rectangular. For beams oriented between these limits, engineering judgement will apply.

For portal frames, the second-order provisions of section 4 (4.9.2.4) may be applied to beams of any orientation, so this distinction need not be made.

The definition of a rectangular frame may also be applied to braced systems in accordance with 12.8.2.

When tapered members comprising one straight flange and one inclined flange form part of a frame, provided the changing properties along the member are properly considered in the design, the orientation of the member can be taken as that of the straight flange in applying this definition. Examples include the top flange of tapered, triangular floor support beams or the vertical outer flange of tapered portal frame columns, where only the inner flange is tapered.

Amd 2
Oct. '07

SPECIAL STUDY. This definition of special study is compatible with that given in AS/NZS 1170 set and covers the situations where either the requirement is initiated by this Standard or where a special study is voluntarily undertaken by the design engineer.

While the authority may require additional information from the design engineer when checking a particular design, it cannot force a special study to be undertaken unless directed by the appropriate provisions of this Standard.

STRUCTURAL PERFORMANCE FACTOR. The value of structural performance factor for use in design is given in 12.2.2.1 and a brief background to its derivation and use is given in C12.2.2.1.

SWAY MEMBER. For non seismic-resisting systems, sway members are those in structural systems which depend on flexural action to limit the sway (e.g. moment-resisting frames). For seismic-resisting systems, in order to correctly account for second-order effects, all types of systems are considered as sway systems by 4.1.2.

C1.4 NOTATION

The change from the notation used in 1989 edition of this Standard to the ISO notation has been brought about by SNZ's policy of adopting the ISO recommendations on notation wherever practicable. This policy allows a unification of the notation used throughout all New Zealand Standards on structural design.

The basis of the notation is generally in accordance with ISO 3898, *Bases for design of structures – Notations – General symbols*.

The value specified for E is the 95 % confidence limit value from (1.6), rounded to the nearest 5×10^3 MPa.

Note that θ_p has three definitions. The definitions relating to use in 4.7.2 and 13.4.3 measure the same quantity, hence the use of the word and between the second and third definitions given for θ_p.

C1.5 USE OF ALTERNATIVE MATERIALS OR METHODS

Persons wishing to use materials other than those specified herein or methods of design or construction not covered herein should obtain expert advice. Contact the Building Industry Authority or HERA for advice in this regard.

C1.6 DESIGN AND CONSTRUCTION REVIEW

In the preparation of this Standard, the Committee made an assumption as to the level of knowledge and competence expected of users of this Standard. This assumption is that the user is either a professional engineer, experienced in the design of steel structures, or, if not, is under the supervision of such a person (termed the design engineer).

The design data and details required to be shown on the drawings or in the specification are the minimum information expected to be provided to ensure adequate documentation.

C1.6.3 Construction review

The aim of the Committee has been to make the monitoring requirements of this Standard consistent with those of the new Building Control System (BCS).

Three circumstances are identified and described in (1) – (3) below. The one to be chosen in a particular application will depend on:

(a) The criticality of the structural element concerned ;

(b) The status of the contractors quality assurance (QA) system;

(c) The particular requirements of the authority.

Adequate review, in the context of this clause, means such construction monitoring, which, in the opinion of the authority, is necessary to provide acceptable reliability that the construction has been carried out in accordance with the Building Consent. It also includes review, as required, of specialised work (e.g. welding, painting, etc).

The design engineer, after consultation and agreement with the owner, shall nominate the level of construction monitoring considered to be appropriate to the work in the plans and specifications. Final arrangements for construction review will rest with the authority or the building certifier.

Note that the requirements of the main contractor (builder) are equally applicable to the steel fabricator and erection sub-contractor. Because of this, the term "contractor" is used in the following elaboration on selection of an appropriate level of construction monitoring.

(1) *Contractor has a formal written independently certified quality assurance (QA) system*
For routine work, a minimum of Construction Monitoring Level 2 (i.e. CM2) in accordance with the joint IPENZ:ACENZ document "The Briefing and Engagement of Consultants" (1.8) should be provided by a suitably qualified and experienced construction reviewer. For larger, important projects, or projects involving other than routine procedures, an appropriately higher level of construction monitoring should be provided.

Relevant details of the contractor's quality assurance system (e.g. NZS 9000 series certification) will be submitted to the authority at the time of uplifting the Building Consent.

At the completion of work, the contractor shall supply to the owner a producer statement (PS) that their work has been carried out in accordance with the requirements of the Building Consent.

A similar producer statement, appropriate to the extent of the construction reviewer's engagement, may be required by the authority from the construction reviewer.

(2) *Contractor has a formal written quality assurance (QA) system, without independent audit*
For routine work, a minimum of Construction Monitoring Level 3 (i.e. CM3) in accordance with the joint IPENZ:ACENZ document "The Briefing and Engagement of Consultants" (1.8) should be provided by a suitably qualified and experienced construction reviewer. For larger, important projects, or projects involving other than routine procedures, an appropriately higher level of construction monitoring should be provided.

On completion of the work, the contractor shall supply to the owner a producer statement (PS) that the work has been carried out in accordance with the requirements of the Building Consent.

A similar producer statement, appropriate to the extent of the construction reviewer's engagement, may be required from the construction reviewer by the authority.

(3) *Contractor has no written quality assurance (QA) system*
For routine work, a minimum of Construction Monitoring Level 4 (i.e. CM4) in accordance with the joint IPENZ:ACENZ document "The Briefing and Engagement of Consultants" (1.8) should be provided by a suitably qualified and experienced construction reviewer. The frequency of review should be appropriate to the size and importance of the project. For larger,

important projects, or projects involving other than routine procedures, an appropriately higher level of construction monitoring should be provided.

Where the contractor has a demonstrable quality track record on similar works, a lesser level of construction monitoring may be appropriate, but this should not normally be less than Level 3, (i.e. CM3) to (1.8).

On completion of the work, the contractor shall supply to the owner a producer statement (PS) that the work has been carried out in accordance with the requirements of the Building Consent.

A similar producer statement, appropriate to the extent of the construction reviewer's engagement, may be required from the construction reviewer by the authority.

NOTE (2) – It is important not to confuse the role of the construction reviewer with that of the welding supervisor. The latter person's role and responsibilities are explicitly defined in 4.11 of AS/NZS 1554.1 or AS/NZS 1554.5 and the two jobs will not generally be undertaken by the same person.

C1.6.3.2.1 Use of welding supervisor as welding inspector

There are times when the welding supervisor may also be the welding inspector, as defined in Appendix D of the Standard. This should only be considered when:

(a) Visual inspection only of all welds is required, and

(b) The fabricator has a demonstrable proven quality track record from past jobs similar to the scope of work being undertaken, and

(c) The person nominated has performed this dual role satisfactorily on past jobs; and

(d) The approval of the design engineer is given.

The reasons for these restrictions lies in the potential conflict of interest in one person having this dual role. Such a situation will also only be possible where the fabricator has no formal written QA system, as these two roles must be kept separate under any formal written Quality Assurance System.

REFERENCES TO SECTION C1

1.1 ASCE. 1988. Guide for Design of Steel Transmission Towers, Manuals and Reports on Engineering Practice Manual No. 52. Published by the American Society of Civil Engineering.

1.2 ECCS. 1985. Recommendations for Angles in Lattice Transmission Towers, Publication No. 39. Published by the European Convention for Constructional Steelwork.

1.3 Kitipornchai, S. and Al-Bermani F.G.A. 1990. Predicting the Ultimate Structural Behaviour of Transmission Towers. AISC Journal, Volume 24, Number 4.

1.4 AISC. 1994. Load and Resistance Factor Design, Second Edition. Published by the American Institute of Steel Construction.

1.5 HERA. 1992. High Strength Steel Design and Fabrication, Second Edition. HERA Report R8-07. HERA, Manukau City.

1.6 Verrill, G. 1996. Statistical Variation of Young's Modulus for Steel. Values Supplied to HERA on request from BHP New Zealand Steel.

1.7 The Building Industry Authority. 1992. The New Zealand Building Code. Wellington, New Zealand.

1.8 IPENZ/ACENZ. 1994. The Briefing and Engagement of Consultants. Published by IPENZ/ACENZ.

NOTES

C2 MATERIALS AND BRITTLE FRACTURE

C2.1 YIELD STRESS AND TENSILE STRENGTH USED IN DESIGN

The yield stress and tensile strength given in material Standards are the minimum values for acceptance of a steel as satisfying the requirements of the appropriate material supply Standard from 2.2.1.

Both yield stress and tensile strength are defined since some clauses use one while other clauses use the other or both. Most structural steel used in New Zealand is of Australian or New Zealand origin and manufactured to the Australian material supply Standards specified in 2.2.1 (a). The values of yield stress and tensile strength for the full range of steels complying with these Standards are presented in table C2.2.1; these are the appropriate values for use in design in accordance with the provisions of this Standard.

Variations in the yield stress and tensile strength are accounted for in the derivation of the strength reduction factor (ϕ) (see C3.3). Because of this fact, the actual values of yield stress or tensile strength recorded on mill test reports or certificates cannot be used for design. The minimum specified values used in table C2.2.1 or from the other material supply Standards specified in 2.2.1 should not be exceeded in design or else the derived strength reduction factors of table 3.3 are rendered invalid.

If, however, a situation arises where the designer must use values of yield stress and/or tensile strength determined for specific steel to be used on a given project, a procedure for determining an appropriate value of f_y or f_u for use in design is given in C17.5.2.1.

Where the design yield stress differs between elements of a section, account may be taken of this in the calculation of nominal member capacity. In this instance the design yield stress used for a given type of design action should be that associated with the section element or elements which provide the principal resistance to this design action. In an I-section subject to major principal x-axis bending and shear, for example, the design yield stress of the flange is used to calculate nominal moment capacity, while the design yield stress of the web is used to calculate nominal shear capacity.

C2.2 STRUCTURAL STEEL

This Standard is written around structural steels of New Zealand, Australian, British or Japanese origin complying with the material supply Standards in 2.2.1. The Standards quoted are product type Standards.

All material specifications relevant to the product - chemistry, mechanical properties, methods of manufacture, supply requirements, tolerances and dimensions are contained in the appropriate Standard for that product. This applies irrespective of steel type, including ordinary weldable grades, weather-resistant grades, formable grades and impact-tested grades.

It is important that the test certificate (see clause 2.2.2) relates to the actual steel supplied. The supplier should, in the contract documents, be required to submit proof of this to the construction reviewer.

It is preferable that any unidentified steel should be tested in accordance with BS EN 10002-1, but if this is not possible, this Standard requires the severe assumption that a design yield stress not exceeding 170 MPa and a design tensile strength not exceeding 300 MPa be used, as appropriate. Clearly, many steels will have a yield stress and tensile strength in excess of these and testing may give an economic result.

Where inelastic demand is expected from a member, use of unidentified steels is restricted by 2.2.3.

Table C2.2.1 – Strengths of steels complying with AS 1163, AS 1594, AS/NZS 3678 and AS/NZS 3679.1

Steel Standard	Form	Steel grade	Thickness of material (t) (mm)	Yield stress (MPa)	Tensile strength (MPa)
AS 1163	Hollow sections	C450 C450L0	All	450	500
		C350 C350L0	All	350	430
		C250 C250L0	All	250	320
AS 1594	Plate, strip and floorplate	HA400	All	400	460
		HW350	All	340	450
		HA350	All	350	430
		HA300/1	All	300	430
		HA300 HU300	All	300	400
		HA250 HU250	All	250	350
		HA200	All	200	300
	Plate and strip	HA4N	All	170	280
		HA3	All	200	300
		HA1	All	(See Note 1)	(See Note 1)
		XF500	$t \leq 8$	480	570
		XF400	$t \leq 8$	380	460
		XF300	$t \leq 3$ $3 < t$	300 300	440 440
AS 3678	Plate	450 450L15	$t \leq 20$	450	520
		450 450L15	$20 < t \leq 32$	420	500
		450 450L15	$32 < t \leq 50$	400	500
		400 400L15	$t \leq 12$	400	480
		400 400L15	$12 < t \leq 20$	380	480
		400 400L15	$20 < t \leq 80$	360	480

Table C2.2.1 – Strengths of steels complying with AS 1163, AS 1594, AS/NZS 3678 and AS/NZS 3679.1 (continued)

Steel Standard	**Form**	**Steel grade**	**Thickness of material (t)** (mm)	**Yield stress** (MPa)	**Tensile strength** (MPa)
		350 350L15	$t \leq 12$	360	450
		350 350L15	$12 < t \leq 20$	350	450
		350 350L15	$20 < t \leq 80$	340	450
		350 350L15	$80 < t \leq 150$	330	450
		WR350 WR350LO	$t \leq 50$	340	450
		300 300L15	$t \leq 8$	320	430
		300 300L15	$8 < t \leq 12$	310	430
		300 300L15	$12 < t \leq 20$	300	430
		300 300L15	$20 < t \leq 150$	280	430
	Plate and floorplate	250 250L15	$t < 8$	280	410
		250 250L15	$8 \leq t \leq 12$	260	410
	Plate	250 250L15	$12 < t \leq 50$	250	410
		250L15	$50 < t \leq 150$	240	410
		250	$50 < t \leq 80$	240	410
		250	$80 < t \leq 150$	230	410
		200	$t \leq 12$	200	300
AS/NZS 3679.1	Flats and sections	350 350L0 350L15	$t \leq 12$	360	480
		350 350L0 350L15	$12 < t < 40$	340	480
		350 350L0 350L15	$40 \leq t$	330	480

➤

Table C2.2.1 – Strengths of steels complying with AS 1163, AS 1594, AS/NZS 3678 and AS/NZS 3679.1 (continued)

Steel Standard	**Form**	**Steel grade**	**Thickness of material (t)** (mm)	**Yield stress** (MPa)	**Tensile strength** (MPa)
AS/NZS 3679.1	Flats and sections	300 300L0 300L15	$t < 11$	320	440
		300 300L0 300L15	$11 \leq t < 17$	300	440
		300 300L0 300L15	$17 < t$	280	440
		250 250L0 250L15	$t \leq 12$	260	410
		250 250L0 250L15	$12 < t < 40$	250	410
		250 250L0 250L15	$40 \leq t$	230	410
	Hexagons, rounds and squares	350 350L0 350L15	$t \leq 50$	340	480
		350 350L0 350L15	$50 < t < 100$	330	480
		350	$100 \leq t$	320	480
		250 250L0 250L15	$t \leq 50$	250	410
		250	$50 < t$	230	410

NOTE –

(1) For design purposes, yield and tensile strengths approximate those of structural Grade HA200. For specific information contact the supplier.

(2) Welded I-sections complying with AS/NZS 3679.2 are manufactured from hot-rolled structural steel plates complying with AS/NZS 3678.

(3) Clause 1.1.4(b) does not permit the yield stress used in design (f_y) to exceed 450 MPa.

C2.3 FASTENERS

All fasteners are required to conform to Australian Standards endorsed by Standards New Zealand as suitable for use in New Zealand, as the design provisions of section 9 are based on this assumption. The relevant properties used for determining the nominal capacity of a fastener are found in section 9.

The equivalent high strength fastener provision in 2.3.2 reflects the fact that some new bolts are available on the New Zealand market which are essentially alternatives to bolts conforming with AS/NZS 1252 but with some added features. As yet these bolts are not covered by New Zealand Standards, but the clause permits their use.

Users of this Standard should note an important change from the 1989 edition with regard to welds, in that all welding consumables and deposited weld metal shall now comply with AS/NZS 1554.1, rather than with NZS 4701. The differences between the two welding Standards are relatively minor and are presented in detail in (2.2). For the designer, the only significant difference is in weld class selection, with classes B or A from NZS 4701 being broadly replaced by categories GP or SP from AS/NZS 1554.1 and Appendix D herein, and class S being replaced by weld selection to AS/NZS 1554.5. Guidance on selection of weld category to AS/NZS 1554.1 is covered further in C9.7.1.3.

C2.4 STEEL CASTINGS

AS 2074 is the only current Australian Standard for steel castings and is endorsed by Standards New Zealand as suitable for use in New Zealand. Steel castings to other specifications would fall under the provisions of 1.5.

C2.6 MATERIAL SELECTION TO SUPPRESS BRITTLE FRACTURE

Under certain conditions, structural steels may become liable to brittle fracture, and although the risk of this fracture is very small in normal steel structures, the design and construction of steel structures must avoid this mode of failure.

Even though a member or connection has been designed to satisfy the strength and serviceability limit states provided for in other sections of the Standard, and is fabricated from steel with high ductility at ambient temperatures, brittle fracture may occur when a critical combination of the following conditions exists:

(a) A notch or severe structural discontinuity giving rise to a severe stress concentration, such as a weld crack, lack of fusion or root penetration, or a punched square cornered hole;

(b) Significant tensile force across the plane of the notch (including any due to residual welding stresses);

(c) Low fracture toughness of the structural steel at the service temperature, or a service temperature below the transition temperature of the steel. The toughness of a steel grade is related to the thickness of the steel.

The transition temperature is the temperature above which a steel is predominantly notch ductile and below which it is predominantly brittle.

(d) Dynamic loading.

Improvement to a significant degree in any one of the above conditions will greatly reduce the risk of brittle fracture.

Guidance on material selection to suppress brittle fracture in the 1989 edition of this Standard was presented in terms of Charpy Impact energy criteria which the steel must meet. In general terms, these criteria were:

(i) For general applications in interior use or for exterior use in the warmer regions of New Zealand, the minimum impact resistance, to AS 1544.2, shall be 27 Joules at 0 °C.

(ii) For applications with design service temperatures down to –15 °C, the minimum impact resistance shall be 27 J at –30 °C (for thicker plates and elements of sections).

(iii) For applications with design service temperatures significantly below –15 °C, the designer was advised to consult appropriate experts for suitable choice of steel. In these instances the minimum impact resistance of 27 J was required to be met at temperatures significantly less than the expected design service temperature (e.g. 27 J at –45 °C for a design service temperature of –30 °C) or a higher impact resistance was required to be met at the design service temperature.

These criteria were applicable to plates and sections produced by the old ingot-cast, hot rolled method and to all thicknesses of material. They are therefore conservative for control-rolled plate and sections, especially for thinner elements. Furthermore, they were not presented in a form readily usable by designers, since the material supply codes do not list the actual Charpy impact resistances of the various grades of steel. In AS 4100 these criteria have been translated into a format readily usable by designers, making use of the statistical data available on the notch toughness characteristics of Australian-made steel of any grade and a range of thicknesses (2.3 to 2.7). These references refer to investigations made on a previous generation of steels manufactured in Australia. The current Australian Standards are believed to give at least the same notch toughness as those reviewed in (2.3 to 2.7). Evidence from (2.16), in fact, points to much improved performance. Limited testing on rectangular hollow sections of Australian and New Zealand origin (2.16) have indicated similar notch toughness between the two countries of origin and it is believed that this will apply to other comparable grades of steel produced by each country.

The material in section 10 of AS 4100 has been reproduced in 2.6 of this Standard. This covers steel of New Zealand or Australian origin. The necessary guidance for steels of British or Japanese origin, to the material supply Standards specified in 2.2.1, has been added to the same format.

Clause 2.6 is self-explanatory. General references dealing with the problem of brittle fracture (2.8 to 2.13) should be consulted as the need arises.

The principal method intended to be used in 2.6 is the selection of a steel which will operate in the notch-ductile temperature range where the steel is insensitive to all but the most gross notches, weld defects or structural discontinuities. If this is not practical, then a detailed fracture assessment in accordance with 2.6.5 must be carried out.

If stress relieving or normalising is to be undertaken, advice should be sought from the steel manufacturer as to the effects that either process may have on the material properties and those of any weldments. Stress relieving should be avoided wherever possible on the grounds of cost, and because there are no standard procedures yet available for carrying it out. It can also be difficult to normalise or stress relieve a steel weldment and stay within the fabrication tolerances of section 14.

The provisions of 2.6.4.3 illustrate the fact that the highest strains imposed on members of structural systems designed to respond inelastically under severe seismic loading may well occur during fabrication, especially where cold-bending is involved. This is not always recognized by designers.

The modifications listed in 2.6.4.3 are based on bending strain rather than uniaxial strain, because bending strain is more likely in actual fabrication than uniaxial strain. The values of strain specified as limits in 2.6.4 should be halved for cases of uniaxial strain.

To determine the bending strain is relatively straight forward except for the correction for springback. A good review of the calculation of bending strain may be found in (2.14).

For simple bending, as shown in figure C2.6, the maximum tangential strain may be derived as follows:

(a) Inside circumference of half circle = $\pi (r - t/2)$

(b) Outside circumference of half circle = $\pi (r + t/2)$

(c) Middle circumference of half circle = $\pi r = L_o$ = original length

(d) Elongation $\Delta = \pi (r + t/2) - \pi r = \pi t/2$

(e) Strain $\varepsilon = \dfrac{\Delta}{L_o} = \dfrac{\pi t/2}{\pi r} = \dfrac{t}{2r}$

This was the strain presented by Kervick and Springborn in (2.15). To account for any shifting of the neutral surface towards the compression side in severe plate bending, the following alternative method is mentioned in (2.14).

$$\text{Strain } \varepsilon = \frac{1}{1.8r/t + k}$$

where k = 0.82 or 1.0 according to different sources.

Springback occurs after cold bending so that the initial bend has to be increased to compensate for the subsequent springback. Ref. (2.14) canvasses a number of alternative methods of predicting springback but concludes that further study is required, such studies to include the effects of strain hardening and non-uniform curvature of the bent plate.

The material designations for steel grades to BS EN 10025 presented in the 1997 edition of table 2.6.4.4 were interim designations published in BS EN 10025:1990. The final designations were published in the 1993 edition of the BS EN 10025 and are incorporated into this Amendment No. 1. Designations for steels from Britain have changed significantly since 1986; details of the new material designations and a comparison between these and the pre-1986 designations can be obtained from HERA.

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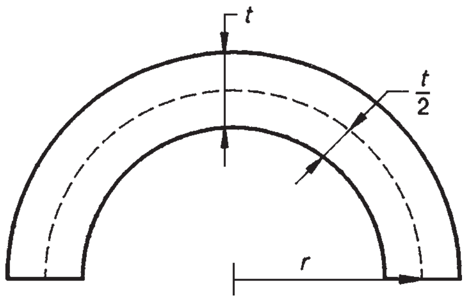

Figure C2.6 – Bending strain notation

C2.6.4.5 Selection of welding consumables

Clause 2.6.4.5.1 specifies the very important requirement that the Charpy impact resistance of welding consumables must be the same or greater than that of the parent metal. This is achieved through application of the selection requirements of AS/NZS 1554.1 or AS/NZS 1554.5. In addition, a limit is placed on the minimum Charpy impact resistance for welding consumables in joints subject to earthquake effects, of 47 J at 0 ^{o}C, by specifying an impact grading of 2 or 3 for these applications in 2.6.4.5.2.

Clause 2.6.4.6 extends this requirement to bolts. The Charpy impact resistance of bolts is available from the manufacturer.

C2.6.4.5.2

In table 4.6.1(A) of AS/NZS 1554.1 and table 4.6.1(A) of AS/NZS 1554.5, the grading numbers for SAW, GMAW and FCAW consumables are followed by up to three alphabetic qualifiers. The previous provisions were only applicable to electrodes from one welding process; linking to the steel type makes the provisions applicable to all welding processes given in that table.

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REFERENCES TO SECTION C2

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2.2 HERA. 1990. Notes Prepared for a Seminar on NZS 4701 Metal Arc Welding of Steel Structures. HERA Report R8-04. Published by HERA, Manukau City.

2.3 George, T.J. and Turner, C.J. 1968. The Impact Properties of AS 149 Structural Steel. AWRA Bulletin, Vol. 1, No. 2, pp. 13-23.

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2.5 Wade, J. 1972. The Weldability of Modern Structural Steels. Symposium on Modern Applications of Welding Technology in Steel Structures, Univ. of N.S.W., pp. 55-88.

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2.10 ECCS Committee 2. 1975. European Recommendations for the Selection of Steel Qualities for Welded Structures. Published by the European Convention for Constructional Steelwork, Brussels.

2.11 Gilmour, R.S. 1970. Brittle Fracture Performance of Bridge Steels. Australian Welding Research, Vol. 1, No. 8, pp. 13-20.

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2.13 Wells, A.A. 1976. Brittle Fracture and Ductile Tearing. Metals Technology Conference, AWRA/AWI/AISC, pp. 3-1-2 to 3-1-13.

2.14 Weng, C.C. and White, R.N. 1990. Cold-Bending of Thick High-Strength Steel Plates. Journal of Structural Engineering, ASCE, Vol. 116, No. 1, pp. 40-54.

2.15 Kervick, R.J. and Springborn, P.K. 1966. Cold-Bending and Forming Tube and Other Sections. Transactions, ASME, pp. 9-18.

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NOTES

C3 GENERAL DESIGN REQUIREMENTS

C3.1 DESIGN

Section 3 specifies structural design requirements in terms of the ultimate and serviceability limit states which must be considered by the designer. It indicates specific design procedures for these two limit states, with stability forming part of the ultimate limit state. For completeness, 3.6 to 3.9 refer to other possible limit state conditions of brittle fracture, fatigue, fire and earthquake which are treated in detail in other sections of this Standard. These involve criteria which must be satisfied for one or both of the ultimate or serviceability limit states.

The overall design objective, as stated in 3.1.1.1, is the safe and proper functioning of the Structure. With regard to the ultimate limit state, this involves meeting certain minimum factors of safety (reliability) against member or structure failure.

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The use of this Standard in conjunction with the Structural Design Actions Standard, AS/NZS 1170 set, establishes parameters for the determination of nominal loads, load factors, safety indices, nominal member capacity and strength reduction factors. These parameters are as follows:

(a) *Nominal loads*
The nominal loads are established for the return periods given in table 3.3 of AS/NZS 1170.0.

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(b) *Load factors*
The load factors are assessed in accordance with rational criteria derived for limit state design, taking into account the criteria on which the nominal loads are based. Suitable criteria are given in (3.17).

(c) *Safety indices* (b)
The target safety indices for limit state standard development at the ultimate limit state should be calculated in accordance with a rational procedure, e.g. as stipulated in (3.3, 3.17), and should exhibit the following values:

(1) For members subject to gravity loading – average value 3, range from 2.5 – 3.5

(2) For connections between members subject to gravity loading – average value 3.5, range from 3.0 – 4.0

(3) For sub-assemblages subject to gravity loading and wind loading – average value 2.5, range from 2.25 – 2.75

(4) For sub-assemblages subject to gravity loading and a specified level of earthquake lateral loading – average value 1.75, range from 1.5 – 2.0 (interim recommendations only pending further research)

(d) *Strength reduction factors*
The strength reduction factors should be established in accordance with a rational procedure, e.g. as stipulated in (3.2, 3.17), utilizing the nominal loads, load factors and safety indices as stipulated in (a)-(c) above.

Note that the limit states apply to complete structures, component members and connections.

C3.2 LOADS AND OTHER EFFECTS

C3.2.1 Loads

The structural design actions Standard most commonly used will be AS/NZS 1170 set, which is a structural design actions Standard only, whereas standards such as NZS/BS 2573, AS 1418 or AS 1657 provide loads but refer to a steel structures standard for the design of steel members and connections. Australian codes will generally refer to AS 4100 for which this Standard can be directly substituted. Care must be taken in substituting the steel structures standard referred to by other codes for this Standard, to ensure that the factor of safety (reliability) against failure is consistent with that required for limit state design from AS/NZS 1170. The material presented in Commentary C3.1 will be of assistance in meeting this requirement.

For cranes, the Power Crane Association of New Zealand references NZS/BS 2573, although AS 1418 also covers crane design.

Section 2.3.3 of (3.19) provides detailed guidance on combining crane loads with other loads for the design of structures supporting cranes.

Other specific loads mentioned in 3.2.1 (e) might include collision, explosion, or subsidence of subgrades.

C3.2.2 Other loads or effects

Attention is drawn to the critical period in the life of the structure during construction, when unusual load paths may be called into play in the partially completed structure.

No guidance is given in this Standard or in AS/NZS 1170 set regarding load factors for the effects listed in (a) - (c) because of uncertainties about the appropriate values and about their structural implications. Some of these loads or effects may not affect the strength of the member or the structure if sufficient ductility is available. However these loads or effects may need to be considered for the serviceability limit state.

Explanatory material relating to construction loads, especially for composite construction, may be found in HERA Design Guides Volume 2 (3.15). Guidance on the inelastic axial shortening that may occur under inelastic cyclic loading of steel beam-columns (carrying axial compression load and moment) is given in section 4.4 of (3.16) and how to make allowance for this in design is contained in each design section of (3.16).

C3.2.3 Design load combinations

Limited explanatory material relating to the load combinations of AS/NZS 1170.0 may be found in the Commentary to that Part.

C3.2.4

See the commentary to AS/NZS 1170.0 section C6.

(Text deleted)

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C3.3 ULTIMATE LIMIT STATE

For both loads and member capacities, it is necessary to distinguish between the nominal and design values. The distinction is best illustrated in the following diagrams, which relate to the situation where capacity design (12.2.7) is not applied:

Nominal loads (or effects)	Load factors ——>	**Design loads (or effects)**	Structural analysis ——>	**Design actions (S^*)**
Minimum specified values and specified parameters	Clauses of this Standard ——>	**Nominal capacity of member** (R_u)	Strength reduction factor (ϕ) tables 3.3(1) or 13.1.2(1) ——>	**Design capacity of member** (ϕR_u)

This diagram also illustrates the change in terminology used in this Standard compared with earlier editions. These changes should be noted carefully.

The terminology used in this Standard regarding actions and effects is also fundamentally different from that used in AS 4100. Compare with 1.3 of AS 4100 and Commentary Clause C3.4 of AS 4100 to see these differences.

The design actions due to the design loads or effects are determined by an analysis carried out in accordance with section 4.

Where the principle of superposition applies, as in the case of elastic analysis neglecting moment or shear redistribution, the designer has the choice of:

(a) Multiplying the loads by the load factors before calculating the design actions; or

(b) Calculating the actions from the unfactored loads and multiplying these by the load factors to arrive at the design actions.

For example, in the calculation of the elastic analysis derived bending moment from dead and live loads, the designer may either select the load combination for the ultimate limit state ($1.2G + 1.5Q$) and then calculate the design bending moment (M^*), or alternatively may calculate the separate dead and live load moments resulting from G and Q and then determine the design bending moment as $M^* = 1.2M^*_G + 1.5M^*_Q$.

If first-order elastic analysis incorporates moment redistribution to 4.5.4.2, then the structural analysis is firstly undertaken to produce the elastic analysis derived design actions. Redistribution is then carried out to produce the redistributed design actions, to which the members are initially sized. Second-order effects are then assessed and included in the design actions where required in accordance with 4.5.2. In cases where superposition does not apply, such as in plastic analysis, second-order elastic analysis with re-distribution, or an elasto-plastic analysis to 4.5.4.1 and 4.7.2, the design loads must be applied directly to the structural system to determine the design actions.

For non-seismic applications, the design capacity of the member or connection is determined in accordance with 3.3(c). Note that the strength reduction factor (ϕ) is always less than one for the ultimate limit state, except as given by clause 11.5 for fire emergency conditions.

For seismic applications where a capacity design is utilized, the capacity design derived design actions for the secondary members and connections are based on the overstrength primary member capacities. The ideal capacity factor (see 1.3 and C1.3) has been set at (1.0/0.9) times the design capacity in order to maintain the slightly increased level of reliability for connectors (bolts, pins, welds) relative to members that is embodied in the derivation of the strength reduction factor, while making the ideal capacity equal to the nominal capacity for members subject to any design action.

The strength reduction factor (ϕ) takes the following into account:

(a) The probability of understrength members or connections due to variations in material strength, material properties, sizes of members and connection elements, and homogeneity;

(b) The differences between the strengths in tests of isolated members, connections, or test pieces and the strength of the member in the structure;

(c) The inaccuracies in design equations related to member or connection design and inadequacies in our understanding of behaviour;

(d) The degree of ductility and reliability required of the member or connection element under the design actions being considered;

(e) The accidental eccentricities in columns, beams, and connections that are not otherwise allowed for in design.

The strength reduction factors were derived by a process called "code calibration" through the use of a "safety index". The safety index is a convenient measure of the notional safety taking into account the variabilities of the loads and the structural capacities. Safety indices have been computed for designs to NZS 3404:1989 (AS 1250). From a consideration of these safety indices, values of target safety indices were chosen for this Standard. These values are in accordance with those adopted by other limit states codes (3.17). The strength reduction factors were then selected so that for designs to this Standard, the associated safety indices are close to the chosen target values.

The procedure for analysis and the concept of a safety index are explained in (3.2, 3.3), while (3.4 to 3.8) detail the methods used to derive the strength reduction factors quoted in table 3.3(1) of this Standard. Refer to Commentary C3.1 for details of the values of safety index incorporated into the provisions of this Standard, when used in conjunction with AS/NZS 1170 set.

With regard to stability under the ultimate limit state, the requirements of 3.3(f) are implemented as follows:

(a) The loads or effects determined in accordance with 3.2 are subdivided into the components tending to cause instability and the components tending to resist instability;

(b) The design action (S^*) is calculated from the component of loads or effects tending to cause instability, combined in accordance with the load combinations for the ultimate limit state specified in the Loadings Standard. The most critical location of loads or effects must be considered.

(c) The design resistance is calculated from the loads and forces which can be dependably considered to resist the instability plus the design capacity (ϕR_u) of any elements contributing towards resisting the instability. The term (ϕR_u) is relevant only where it is mobilized towards resisting the instability. Unless specified otherwise in the appropriate loading standard, the magnitude of components of loads or effects tending to resist instability should be scaled to be consistent with a load factor of 0.9. (This is consistent with the provisions of AS/NZS 1170 set.)

(d) The whole or part of the structure, as appropriate, is proportioned so that the design resistance is equal to or greater than the design action.

The requirements of (a) – (d) can be exemplified for the general load condition $1.2G + 1.5Q$, as:

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$$1.2G^{I} + 1.5Q^{I} \leq (0.9)G^{S} + (\phi R)$$

where G^I and Q^I are the dead and live load components that tend to cause instability, G^S is the dead load component tending to resist instability, and (ϕR) is the design capacity mobilized to resist instability, (if any).

Determination of what constitutes stabilizing and destabilizing components of the loads is based on the postulated mechanism of instability established by the **total** loads, in this example $1.2G + 1.5Q$.

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C3.4 SERVICEABILITY LIMIT STATE

Amd 2 Oct. '07 NOTE – Not only is NZS 4203 replaced by the AS/NZS 1170 set, but many of the referenced publications given in this section have been superseded. It is beyond the scope of this amendment to give an updated reference list; contact HERA for advice on any updates.

C3.4.1 General

Serviceability requirements vary considerably from structure to structure and there is no simple codifiable way of dealing with these problems. It is the responsibility of the design engineer to address each individual condition by considering the specific performance requirements appropriate for the structure.

It is not possible to specify or even to list all of the possible requirements, and only specific problems are mentioned in this clause, namely structural deformation, vibration, bolt slip, and corrosion protection.

For the very specific case where moment redistribution at the serviceability limit state is permitted to 4.5.5, the guidance given in C4.5.3(c) on incorporating this redistribution into an elastic analysis should be followed.

C3.4.2 Method

Limited explanatory material relating to the load combinations of AS/NZS 1170.0 is found in the Commentary to that Part.

C3.4.3 Deflection limits

Appropriate performance limits are difficult to define because they depend on the nature of the building and perceptions of the occupants. Suggested serviceability limits for deflection are presented in table C1, Appendix C, AS/NZS 1170.0 (it should be noted that these are recommendations only). For light-weight steel framed industrial buildings, recommendations are given in section 14 of (3.15). For short-and long-term deflection of composite beams, refer to 13.1.2.6 and Commentary Clause C13.1.2.6. Further guidance is given in section 2.4 of (3.19).

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C3.4.4 Vibration of beams

Long-span floor structures used for rhythmic activities such as dancing, jumping exercises, or rock concerts are susceptible to vibration problems. These rhythmic activities create periodic forces with frequencies in the range 1 Hz to 4 Hz. When the fundamental vibration frequency of the structural system is less than 8 Hz, some dynamic analysis is advisable. Guidance on the problem can be found in (3.9, 3.10, 3.15).

Guidance on other specialist areas where vibration can be of concern is given in (3.11, 3.12). A design procedure for control of in-service floor vibration in composite or non-composite floor systems, incorporating a concrete slab on steel beams, as used in residential, office, commercial or gymnasium environments, is available in Appendix B13 of HERA Design Guides Volume 2 (3.15). This procedure is based on the method developed by T.M. Murray (3.13).

In 1997, the American Institute of Steel Construction published a revised joint American/ Canadian design procedure for control of in-service floor vibration that is wider in scope than the procedure contained in Appendix B13 of (3.15). This publication is available through HERA; details of it are contained in an article on pp. 25-27 of the HERA *Steel Design and Construction Bulletin*, Issue No. 56, 2000.

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C3.4.5 Bolt serviceability limit state

Under conditions described in C9.3.3.1, a design engineer may elect to use a no-slip bolted connection. The condition of preventing slip for the serviceability limit state load is strictly a serviceability condition, rather than a strength condition. This type of connection is the well-known friction-type (/*TF*) connection. Clause 9.3.3 details the method of design and requires a separate check for the ultimate limit state. This serviceability condition is discussed in (3.14).

C3.4.6 Corrosion protection

Guidance is given in Appendix C on corrosion protection, mainly by reference to other standards and publications which cover this subject.

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Note that the use of unprotected steel is an option under 3.4.6, provided that this decision is based on application of appropriate design criteria (e.g. in accordance with HERA Report R4-133 (3.21)). Unprotected steel may also be used in external environments on the basis of a properly evaluated sacrificial thickness of steel to allow for the expected material loss due to corrosion over the specified design life. The rate of corrosion applicable to a given location may be determined from site-specific data, when available, or estimated from published data, (e.g. (3.20, 3.21)).

C3.5 – C3.9

(No Commentary.)

REFERENCES TO SECTION C3

Amd 2 Oct. '07

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3.14 Galambos, T.V. et. al. 1982. Serviceability Limit States: Connection Slip. Journal of the Structural Division, ASCE, Vol. 108, No. ST12, pp. 1668-2680.

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3.20 Duncan, J.R. and Cordner, R.J. 1992. Atmosphere Corrosion Rates Over Two Years Exposure at 156 Sites in New Zealand. IPENZ Transactions, Vol. 18, No. 1/GEN.

3.21 HERA. 2005. New Zealand Steelwork Corrosion Coatings Guide, HERA Report R4-133. Published by HERA, Manukau City, New Zealand.

C4 METHODS OF STRUCTURAL ANALYSIS

INTRODUCTION

The methods used for the design of a steel structure are essentially element design methods, in that the design actions on the members and connections are predicted by a method of structural analysis which is appropriate to the assumptions made about the structure. The design capacities of these elements are then determined (using sections 5-9, 12 and 13) and compared with these design actions.

Section 4 deals with the methods of structural analysis, and the assumptions made for analysis. The methods of structural analysis are presented in many textbooks, such as (4.1, 4.2), while computer methods of analysis are discussed in more specialised texts, such as (4.3, 4.4). Trends in these methods of analysis are discussed in (4.5), and worked examples in accordance with the provisions of this Standard are presented in (4.18). Most of the member design provisions of this Standard are taken from AS 4100, hence design examples to that Standard (4.6) will also be generally relevant for non-seismic applications to this Standard.

New Zealand has a distinctly different design environment to Australia, which shows up in the requirements for analysis. Many New Zealand designs require explicit consideration of inelastic action within the structural system and this has led to widespread use of elastic analysis with redistribution, coupled with application of the strength method of design as the preferred pre-limit state design approach. Limit state design for the ultimate limit state simply carries on the strength method of design in a more formally calibrated and uniformly reliable manner. The most common methods of structural analysis for ultimate limit state design will be elastic analysis and elastic analysis with redistribution. Plastic analysis, in conjunction with the plastic method of design, will be used less often.

This is in contrast to Australian design practice, which involves no use of moment redistribution and limited use of plastic analysis.

These differences mean that the provisions of section 4 on structural analysis, as presented in this Standard, are presented in a different format, with considerably different emphasis, to the provisions of section 4 of AS 4100.

The provisions of section 4 apply to all structural systems; for seismic-resisting or associated structural systems the designer must also refer to 12.3 for additional requirements.

C4.1 METHODS OF STRUCTURAL ANALYSIS

C4.1.1 General

This clause directs the designer to subsequent clauses which govern the selection of structural form, assumptions for analysis and which lead on to each of the three methods of analysis covered by this Standard, namely:

(a) Elastic analysis;

(b) Elastic analysis with redistribution; or

(c) Plastic analysis.

As mentioned in the introduction, the material is presented on the basis of the first two methods being those most commonly used in both seismic and non-seismic applications. Note that the elastic analysis used may be either static or dynamic (modal) where appropriate for seismic effects or wind loads.

The method of elastic analysis (4.4) can be used quite generally, and there are a number of commercial computer programs available which allow the analysis of large structures. These include programs for three-dimensional structures, which usually ignore all warping effects in modelling torsional actions.

The actions from an elastic analysis may then be modified by moment or shear redistribution (4.5). Shear redistribution is only applicable to eccentrically braced framed systems and is covered by 4.5.6. Moment redistribution enables the moment pattern from elastic analysis to be modified and provides, typically, an intermediate position between elastic and plastic analysis. Moment redistribution generates limited inelastic demand on some members of a structural system; this is catered for by requiring these members to be designed and detailed by the necessary specific seismic provisions of section 12 in accordance with the extent of applied redistribution. Even though this approach imposes quite severe restrictions on member geometry and member restraint for the inelastic demand anticipated on a member from moment redistribution alone, elastic analysis with moment redistribution can be a cost-effective option for structural systems not designed for load combinations including earthquake loads, as well as for those that are.

Designers should note that the general option of elastic analysis with moment redistribution, introduced by 4.1.1(b) and detailed in 4.5.4.1, is a very general and powerful method of analysis, provided the analytical capability exists to output plastic hinge rotations. For example an elasto-plastic push-over method of analysis under static or equivalent static design loads may be used to satisfy 4.5.4.1 and hence 4.1.1(b).

The method of plastic analysis (4.6) allows greater levels of moment redistribution after initial yielding and can lead to more economical designs than those based on elastic analysis alone. The margin of economy is closed, however, if elastic analysis with subsequent moment redistribution is used. Also serviceability limit state considerations may limit the full strength reduction possible from plastic analysis.

Application of plastic analysis is restricted to frames which can safely redistribute their bending actions after yielding and allow plastic collapse mechanisms to develop fully. Plastic analyses are easily made for continuous beams and simple frames, while more complicated two-dimensional frames usually require computer programs (4.3, 4.4).

For any method of analysis, the effect of haunching or any variation of the cross section along the axis of a member shall be considered and, where significant, shall be taken into account in the analysis. Guidance on allowance for member haunching is given in (4.18).

C4.1.2 Braced or sway members

This clause requires members to be defined as braced or sway for application of section 4. In practice all members may sway, but the sway deflections are small in an adequately braced structure not subject to inelastic action, in which case the definition of a braced member allows the second-order effects of sway (the $P-\Delta$ effects) to be neglected. In seismic-resisting systems, especially those subject to inelastic action, the sway deflections may generate significant $P-\Delta$ effects (these effects are shown in figure C4.4.2), thus requiring almost all members of all seismic-resisting systems, when these are being designed for load combinations which include earthquake loads, to be considered as sway members. The exception to this is members within a triangulated structure which forms part of a seismic-resisting system. The triangulated structure as a whole will be considered as a sway member, while the individual members within that triangulated structure will be considered as braced members for design. This ensures that correct determination of design action is made both for the seismic-resisting system as a whole and for the individual members within the triangulated structure. An example is a portal frame with a truss rafter. The provisions of 4.4.3.3 and, where appropriate, of 4.4.3.2, then determine the appropriate allowance that must be made for $P-\Delta$ and $P-\delta$ effects in all members of the seismic-resisting system when it is being designed for load combinations which include earthquake loads.

For structural systems which are being designed for load combinations which do not include earthquake loads:

(a) In a braced system, all members should be taken as braced;

(b) In an unbraced rectangular frame which sways horizontally, the vertical members (columns) should be taken as sway members, and the horizontal members (beams) as braced members. This means that, for rectangular sway frames under non-seismic consideration, any increase in design moment required due to $P-\Delta$ effects is confined to the columns for design purposes, which is an acceptable simplification for such sway frames.

C4.2 STRUCTURAL FORM

C4.2.1 General

This clause defines the three alternative assumptions of rigid, semi-rigid, and simple construction, and permits combinations of these, with restrictions for use in seismic-resisting systems. These forms are classified in 4.2.2 according to the assumed rotational rigidities of the connections between members.

C4.2.2 Forms of construction

C4.2.2.2 Semi-rigid construction

Semi-rigid connections may be of two types. The first type includes connections which may allow some rotational slip because of the use of snug-tight bolts in shear and bearing connections. It is very difficult to predict accurately the bending moments transmitted by such connections for a given rotation, and the use of the semi-rigid assumption for these connections is effectively prohibited by the requirements of 9.1.2.2.

The second type of semi-rigid connection includes those which allow elastic rotations because of the flexibility of some of the plate elements used to transfer the bending actions through the connection. It is relatively easy to allow for such connection flexibility in elastic computer analysis methods, and there is an increasing amount of test information becoming available for both seismic and non-seismic applications of these systems (4.8, 4.22).

C4.2.2.3 Simple construction

Simple construction is very often assumed in order to simplify the manual analysis of triangulated structures which have joint loads only. In such cases, the differential member end rotations are very small, even when the connections are quite rigid, and the bending actions are therefore small and may be ignored.

Simple construction may also be assumed in order to simplify the manual analysis of flexural frames which do not rely on rigid frame action for lateral stability. In this case, the connections must be sufficiently flexible to ensure that adverse moments are not developed, but must still have the capacity to transmit any moments which are developed.

Simple construction also includes simply supported beams.

C4.2.4 Applicability of forms of construction for use in seismic-resisting systems

C4.2.4.1

Braced frame seismic-resisting systems (eccentrically or concentrically braced frames) may utilize simple construction, however the inelastic performance of eccentrically braced frames is improved through the use of semi-rigid or rigid joints as is detailed in (4.19). The columns of such systems must be rigid through floors and splices, in accordance with 12.9.6 of this Standard.

C4.2.4.2

Moment-resisting framed seismic-resisting systems may use either rigid or semi-rigid construction, however in the latter case, verification of the inelastic performance by special study is required. Semi-rigid construction, in the context of seismic-resisting systems, may be taken to include moment-resisting frames where the panel zone of the beam-column connection is designed and detailed as the primary seismic-resisting element. Information is available in the literature (e.g. (4.22)).

C4.2.4.3

The special study may be by static elasto-plastic analysis to the total design lateral deflection (elastic plus inelastic) for the frame, as determined in accordance with the Loadings Standard, for such frames which do not exceed the critical height (see 1.3).

For frames of semi-rigid construction which exceed the critical height, the special study should include numerical integration time history analysis in accordance with the Loadings Standard, using at least two earthquake records of acceleration versus time with an energy intensity and distribution as a function of period corresponding to the earthquake spectrum of the Loadings Standard or specific to the site (when available). An analysis under a 450 year return period event utilizing S_p = 1.0 to verify non-collapse should also be undertaken for semi-rigid frames which exceed twice the critical height. These requirements may be relaxed when the frame is part of a dual system (see 12.13).

C4.3 ASSUMPTIONS AND APPROXIMATIONS FOR ANALYSIS

C4.3.1 General

This clause allows a number of assumptions to be made which simplify the analysis. Clauses 4.3.1.1 and 4.3.1.2 allow a regular three-dimensional building to be decomposed into a series of two-dimensional frames which can be analysed independently of each other. This is not permitted when there are significant load redistributions between the two-dimensional frames, which may occur when these have different loads or stiffnesses, or under the action of wind or seismic loads, where torsional actions of the overall structural system need consideration.

Clause 4.3.1.3 allows the floors of a braced multi-storey building to be analysed independently of each other for the purposes of beam design, while 4.3.1.4 allows the separate analysis of the floor beams of a multi-storey building. These assumptions simplify the manual analysis of the beams and columns. Note that these assumptions cannot be applied in general to seismic-resisting systems, although a simplified sway model of one floor of a moment-resisting framed seismic-resisting system may be used in preliminary design, as detailed in section 5.2 of (4.19).

Note also that the substructure model of 4.3.1.3 and 4.3.1.4 used to determine design actions in the beams on a floor-by-floor basis may be unconservative in column design, as it will generate a reverse moment distribution in the columns in conjunction with reduced compression load. Studies (4.18) have shown that the live load distribution shown in figure C4.3.1 should be used to determine the appropriate design actions in the column. For further guidance, see section 6.6.5 of (4.18).

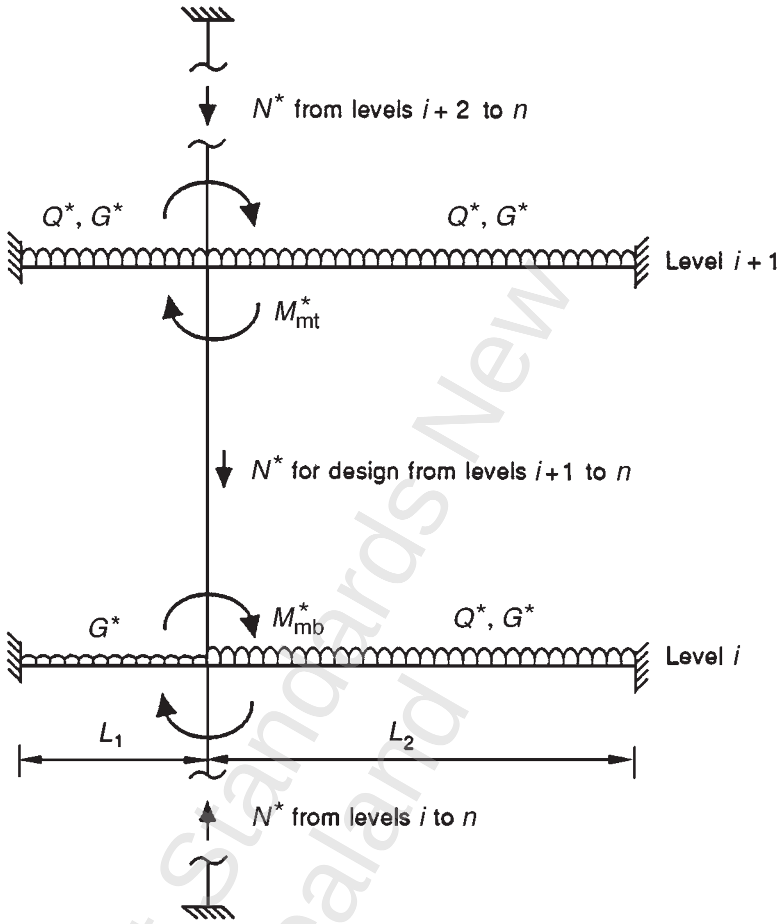

Figure C4.3.1 – Distribution of live (variable) loading for determination of first-order column design moments

NOTE –

(1) At the supports, only the bending moments in the beam-column are shown for clarity.

(2) M^*_{mt}, M^*_{mb} are the first-order design moments from analysis at the top and bottom respectively of the beam-column.

(3) The range of L_1 is $0 \leq L_1 \leq L_2$.

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(4) Q^*, G^* are the ultimate limit state design loads from AS/NZS 1170.0. For Q^*, G^* together, the design load combination is $1.2G + 1.5Q$. For G^* alone, the design load is $1.2G$ in this instance, not $1.35G$.

(5) The subassemblage shown is that required by 4.3.1.3.3.

(6) Refer to figure C4.4.3 for second-order effect determination.

C4.3.2 Span length

For simplicity, and perhaps conservatively, the span length for calculation of bending moments is taken as the centre-to-centre distance, rather than the clear distance between supports. Note that the design moments at the ends of continuous members or members in rigid or semi-rigid construction may then be taken as the value at the face of the supporting element.

C4.3.3 Arrangements of live loads for buildings

When the nominal live load exceeds three-quarters of the nominal dead load, at least three different arrangements must be considered for load combinations not including earthquake or fire. Live load on alternate spans will give maximum mid-span bending moments, while live load on two adjacent spans will give maximum support bending moments and reactions.

For ultimate limit state calculations involving earthquake or fire, it is the point- in- time or long term live load that forms the nominal live load for 4.3.3.2.

C4.3.4 Simple construction

The clauses set out a number of assumptions that may be made for the analysis of simple construction. Simple construction allows the assumption of joints which transmit negligible bending moments, which simplifies the manual analysis of many triangulated structures and beams.

In beam and column frames for which simple construction can be assumed, there will nevertheless be bending moments acting on the columns which are caused by the eccentricities of the beam reactions. The minimum distance of 100 mm which applies generally is a traditional limit that appears to have been satisfactory in the past, even though it would seem more logical to relate such a limit to the sizes of the members connected. This distance is measured from the face of the column, which is the surface of either the flange or the web of an I-section column, depending on whether the connection is made to the flange or the web.

An exception is given to the 100 mm limit, which corresponds to the case where a beam bears on top of a column cap, in which case rotation of the beam will cause the reaction force to act at or near the face of the column.

These approximate bending moments of uncertain accuracy should not be amplified for second-order effects, hence 4.3.4.4 states that they shall be used directly in design for combined actions.

C4.4 ELASTIC ANALYSIS

C4.4.1 General requirement

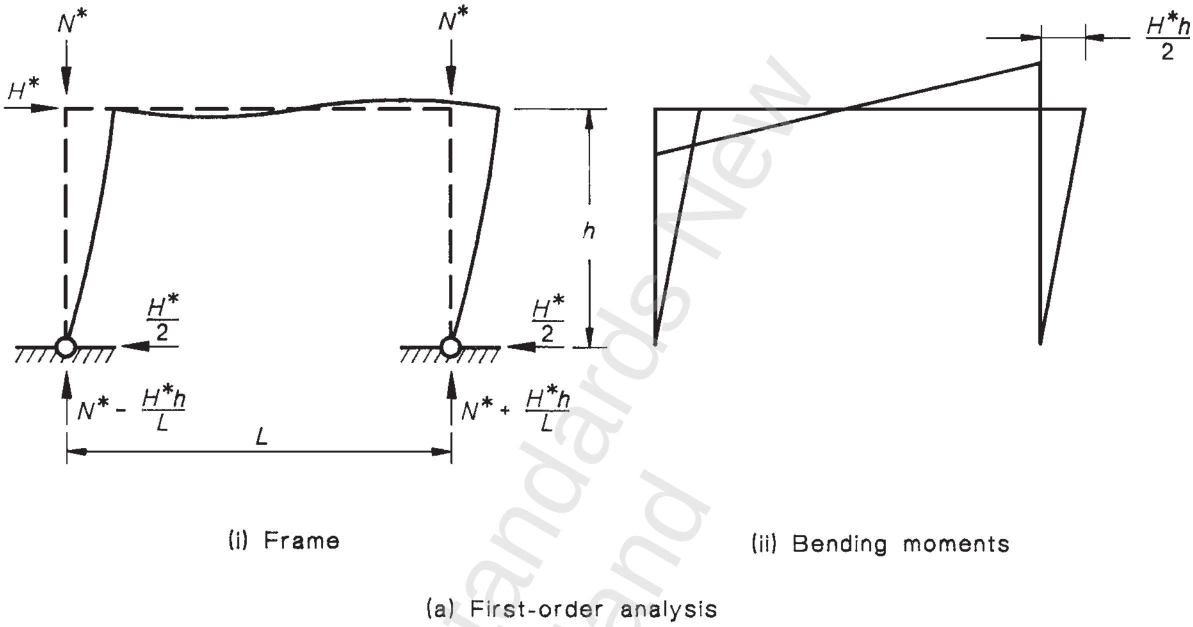
Elastic analysis in accordance with 4.4 requires that all members of the structural system remain essentially elastic under the design actions determined from an elastic analysis. The requirement for members to remain essentially elastic, rather than elastic, acknowledges that the nominal section capacity of members in bending may be based on a fully plastic stress distribution, if the member cross section is compact (5.2.3) or on a partially plastic stress distribution, if the member cross section is non-compact but not slender (5.2.4).

The method of elastic analysis (static or dynamic as appropriate) is intended to be applied to structures which transmit significant proportions of the applied loads by bending actions. It is not required for triangulated structures, which may be assumed to have axial force actions only and which become statically determinate as a result of the assumption of simple construction, since such structures can be analysed by statics alone.

C4.4.2 Second-order effects

Second-order effects are caused by axial forces acting on the structural system and its individual members. When the axial forces are compressive, the second-order effects may increase the first-order bending moments and so allowance for significant second-order effects must be made in design. Second-order effects are neglected in a first-order (linear) elastic analysis and so, where their effects are significant and a first-order analysis is undertaken, magnification of the first-order moments must be made. A more detailed background to second-order effects is given in section 7 of (4.18) and a useful summary in (4.26).

There are two forms of second-order effect, namely the $P - \Delta$ effect, which arises from lateral joint displacements of the structural system as a whole and the $P - \delta$ effect, which arises from member deflections away from the straight lines joining the member's ends. Refer to the definitions in 1.3 and to figure C4.4.2.



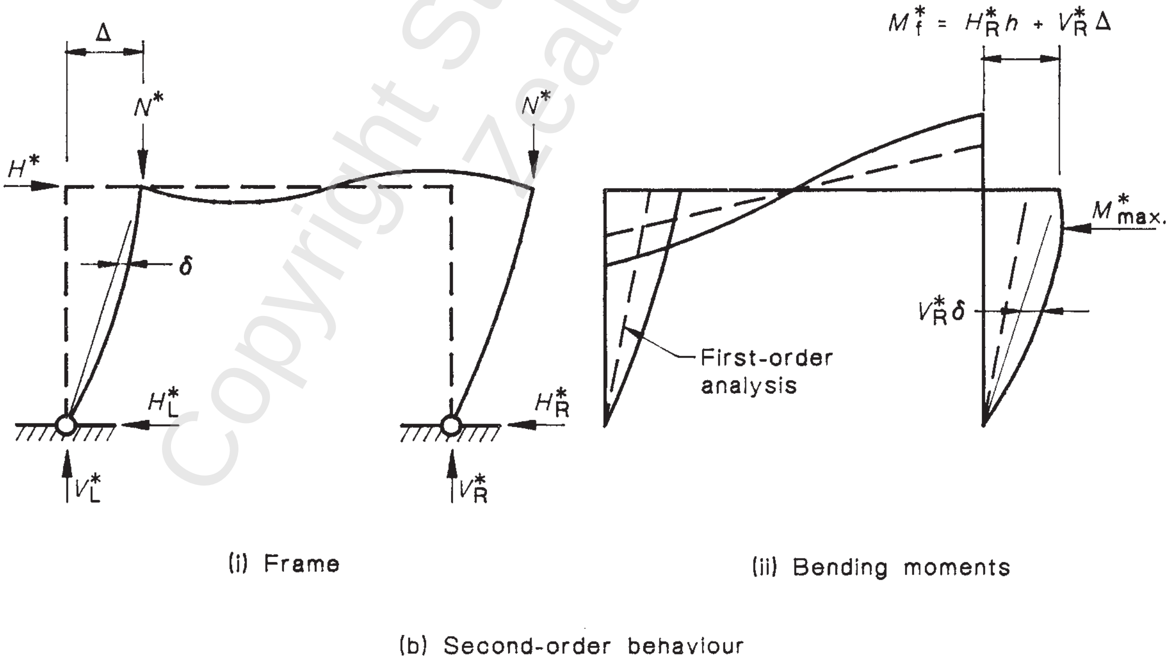


Figure C4.4.2 – First-order analysis and second-order behaviour

For braced structural systems not being designed for earthquake loads, the joint displacements Δ are small and only $P - \delta$ effects need be considered. For sway structural systems not being designed for earthquake loads, the $P - \Delta$ effects are important, often being larger than the $P - \delta$ effects.

For all types of seismic-resisting systems being designed for earthquake loads, the lateral deflection Δ must consider both elastic and inelastic components, in accordance with the Loadings Standard. This lateral deflection may be significant, irrespective of whether the system is a moment-resisting, eccentrically or concentrically braced frame and hence $P-\Delta$ effects must always be considered for the seismic design of a seismic-resisting system. This is the reason why seismic-resisting systems under seismic design are always classified as sway systems in accordance with 4.1.2 – refer to Commentary Clause C4.1.2 – and why they are not allowed to be excluded from specific second-order effect consideration through 4.4.2.2.2.

Second-order effects may be neglected in certain circumstances, as identified by 4.4.2.2.

Moderate second-order effects may be allowed for approximately by amplifying the bending moments obtained from a first-order elastic analysis (4.4.3). This approach will be appropriate for the majority of structural systems. The amplification factors δ used for this are based on estimates of the relative magnitudes of the design loads on the structural system compared with the elastic buckling loads.

The moment amplification method is often conservative and is unreliable for structural systems with higher second-order effects (i.e. where the elastic buckling load factor (λ_c) is less than 5 for sway systems or 3.5 for braced members or systems). In this latter case, a second-order elastic analysis method must be used (Appendix E). In practice it is unlikely that the moment amplification factor for a sway system will attain the upper limit of 1.2 for this method, although it is more possible, in a slender heavily loaded braced column, that δ_b may reach 1.4 (see Commentary Clause C4.4.3.2 for more details). Note that the upper limit only applies to members in indeterminate structures, as the amplification factors are of sufficient accuracy for members of determinate structures, whatever the value calculated.

The application of moment redistribution to a sway system is likely to slightly increase the sway second-order ($P-\Delta$) effects on that system. This is picked up in 4.5.3.

For sway structural systems, there is an important distinction between the application of increased design actions arising from $P-\Delta$ effects to seismic and non-seismic considerations. For seismic design of seismic-resisting systems, any increase in design actions required due to $P-\Delta$ effects by the Loadings Standard must be applied to all members of the system, whereas for non-seismic applications, moment amplification due to sway ($P-\Delta$) effects need be applied only to the column members. This is picked up in the wording of 4.4.3.3.2 (a) – (c) as appropriate and also in 4.1.2 (see Commentary Clause C4.1.2).

C4.4.2.2 Exclusions from second-order effect consideration

The 1992 edition of this Standard contained two types of structural system for which second-order effects may be excluded. These are a sub-set of the general provision that second-order effects may be excluded for a frame with an elastic buckling load factor, $\lambda_c \geq 10$ (4.18, 4.26). The 1997 revision has introduced this general exclusion and retained the existing provisions under the heading of specific exclusions. These specific exclusions do not require the calculation of λ_c.

It is important, in applying 4.4.2.2.1, that λ_c is calculated for the braced or sway status of the frame, as appropriate, and for the appropriate load set.

Second-order effects in non-seismic applications can be neglected in triangulated structures which may be assumed to carry axial forces only and which become statically determinate as a result of the assumption of simple construction. They may also be neglected in non-seismic design of systems carrying low levels of axial compression. In sway systems, the vertical loading must not generate significant axial forces in the beams for this exclusion clause to apply. Hence the restriction to **rectangular** moment-resisting framed systems in 4.4.2.2.2.

It is suggested (4.18) that second-order effects of less than 10 % on systems being designed for non-seismic load combinations should be neglected and a similar philosophy, but with the limit on elastic second-order effects reduced slightly to 9 %, applies to seismic design of seismic-resisting systems (4.20). Clause 4.4.2.2 translates these recommendations into easily applicable provisions for non-seismic design applications, while NZS 1170.5 *Earthquake actions* provides corresponding exclusion clauses for seismic design of seismic-resisting systems.

C4.4.3 First-order elastic analysis

C4.4.3.1 General

In a first-order elastic analysis, the second-order bending moments due to $P - \Delta$ and $P - \delta$ effects are not determined. Moderate but significant second-order moments may be allowed for approximately by amplifying the bending moments obtained from a first-order elastic analysis.

Flowchart guidance on applying 4.4.3 is given in figure C4.4.3.

The amplification is applied to the maximum bending moment M_m^* in the member calculated from the first-order analysis, which may occur within the member length, or at one of its ends. In sway frames, it will usually be an end moment which controls.

C4.4.3.2 Moment amplification for a braced member

This clause and 4.4.3.3 following provide for the determination of the design bending moment M^* which is to be compared with the appropriate design moment capacity ϕM. For a braced member, this design bending moment M^* is obtained by multiplying the maximum first-order bending moment M_m^* by the moment amplification factor δ_b. This is undertaken on a member by member basis, with each braced member considered individually.

For a tension member or for a member with zero axial force, the amplification factor δ_b is taken as unity. This is an approximation, since the second-order effects in a critical compression member will slightly increase the moments in an adjacent member, even if it has a zero or tensile axial force.

For a compression member, the amplification factor δ_b, which is never less than 1.0, depends on the ratio of its design compression action N^* and its member buckling load N_{omb}, determined using 4.8.2, and also on the factor c_m which varies with the shape of the bending moment diagram. When c_m takes its greatest value of 1.0, the amplification factor δ_b reaches the maximum value of 1.4 permitted for the method of moment amplification when $N^*/N_{omb} = 0.286$. This corresponds to a member with a slenderness ratio ($k_e L/r$) of around 180 and subject to significant design compression (N^*), which is possible but unlikely in practice.

For members with unequal end moments M and $\beta_m M$, the linear equation for the factor c_m and the $(1 - N^*/N_{omb})$ term provide a good approximation for the amplified bending moment (4.9).

For members with transverse loads, three alternative methods, which are of increasing accuracy and complexity, are given for approximating the effects of the bending moment distribution on the factor c_m. The third method is discussed in (4.10), where specific equations are given for members with uniformly distributed loads. The second method allows the determination of an equivalent end moment ratio β_t by matching the design bending moment distribution to one of those of figure 4.4.3.2, while the first method leads to the generally conservative value of $c_m = 1.0$.

SECOND-ORDER EFFECTS BY FIRST-ORDER MOMENT MAGNIFICATION

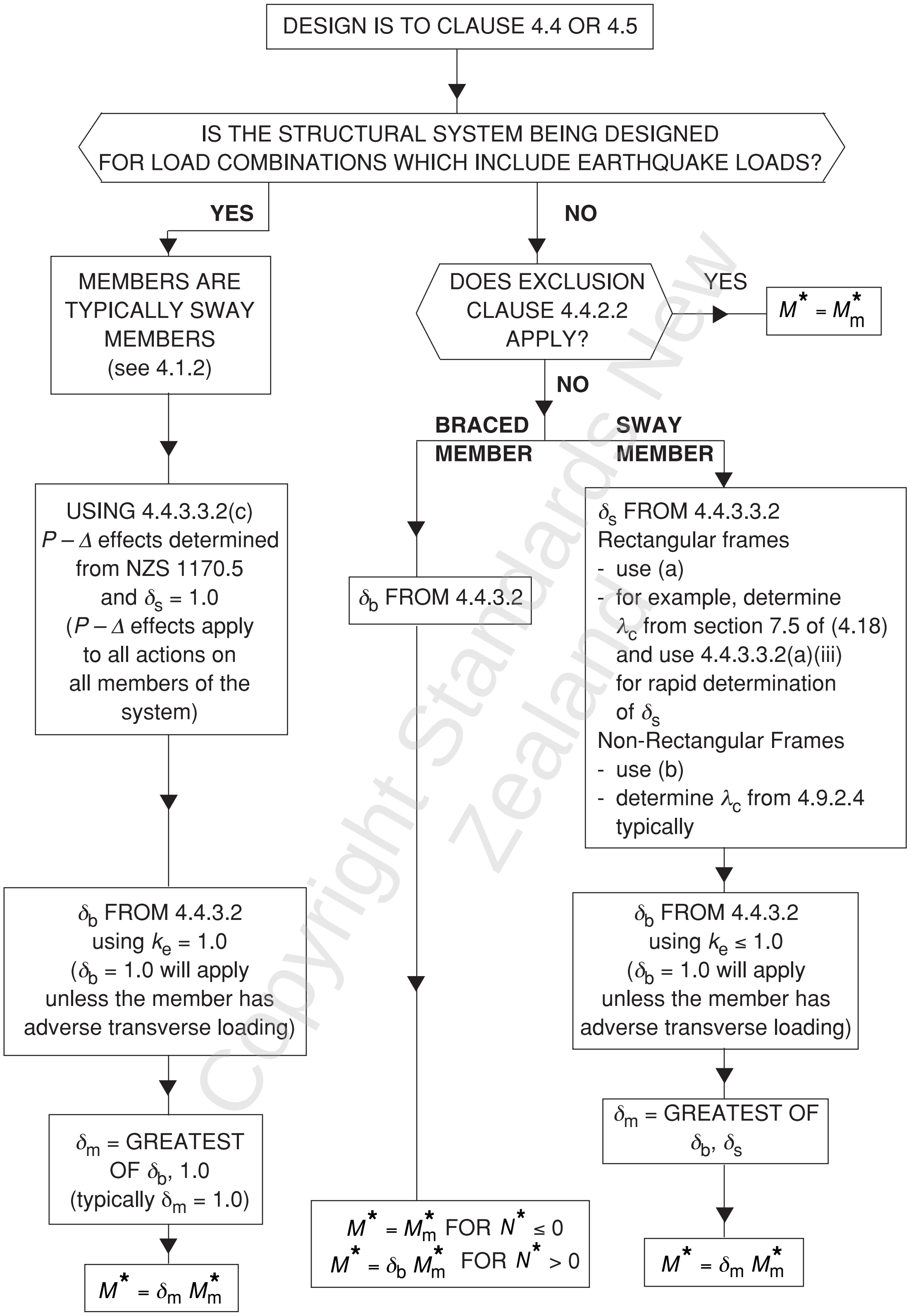


Figure C4.4.3 – Flowchart for second-order effect determination by first-order moment magnification (from (4.18))

NOTE – M_m^* = maximum bending moment from first-order elastic analysis (4.4.3.1)

$N^* \leq$ 0 signifies zero axial load or tension

$N^* >$ 0 signifies compression

The influence of transverse loading on a member may be ignored, if the design moment generated by this transverse loading is less than 10 % of the design moment capacity of the member about the appropriate principal axis or axes. (See also figures C8.1.1 and C8.1.2 and Commentary clause C12.8.3.3).

C4.4.3.3 Moment amplification for a sway member

This clause provides a generally conservative method of determining the moment amplification factor δ_m for a sway member (a more accurate method is given in Appendix F). In this conservative method, the amplification factor δ_m is taken as the greater of the braced member factor δ_b (4.4.3.2) and the sway member factor δ_s.

The determination of δ_s is undertaken on a storey-by storey or an overall frame basis, with all sway members in the storey or frame considered collectively in calculating δ_s. This is in contrast to calculating δ_b, which is applied on an individual member basis. It is recommended to determine δ_b using k_e = 1.0 initially, as the moment distribution along the member will usually mean that δ_b = 1.0 even when this conservative estimate of effective length for the (braced) member is used.

For lateral force-resisting systems, the selection of option (a) – (c) must be made appropriate to the load case under consideration. Where this includes earthquake loads, option (c) is appropriate. Where it does not include earthquake loads, options (a) or (b), as appropriate, are chosen. For example, in the case of a non-rectangular portal frame subject to both wind and seismic forces, option (b) is appropriate for the load combinations including wind and option (c) for the load combination including earthquake. Note that, for this example, option (b) will also apply for load combinations involving gravity loads alone and will typically give a larger value of δ_s in this instance.

For structural systems (frames) being designed for load combinations which do not include earthquake loads, there is one option for rectangular frames (option (a)) and one for non-rectangular frames (option (b)). The rectangular frame option gives three choices, no. (i) of which avoids the need to calculate an elastic buckling load factor. This choice cannot be used when V^* is arbitrarily linked to N^*, as is the case with the notional horizontal loads given by 3.2.4. The non-rectangular frame option requires the calculation of λ_c, however the only such application commonly encountered in design is to portal frames, where λ_c is given directly from 4.9.2.4.

A perimeter frame, by definition, supports a significantly smaller tributary floor area for determination of gravity loading than it supports laterally. When determining the second-order P-Δ effect from options (a) or (b) to apply to a perimeter frame, only the gravity load supported directly by the frame should be considered. The first-order $P-\Delta$ actions, however, will need to be determined from the gravity load on the total floor area supported laterally by the perimeter frame.

For frames being designed for load combinations which include earthquake loads (i.e. option (c)), $P-\Delta$ effects must be considered explicitly by the Loadings Standard and any increase in design actions required must be applied to all actions on all members of the system. This will mean magnifying both the moment and seismic-induced axial forces in, for example, the columns in eccentrically braced frames or the exterior columns in moment-resisting frames. This means that, when applying 4.4.3.3.2(c) of this Standard to seismic-resisting systems, the sway amplification factor is taken as 1.0.

When applying the provisions of the Loadings Standard to determination of $P-\Delta$ effects in seismic-resisting systems, in most instances no increase in design actions will be required, because of the need to meet prescribed strength and stiffness criteria. This is consistent with the general provisions of 4.4.3.3.2, in that if $P-\Delta$ effects on the same seismic-resisting systems were evaluated in accordance with one of the options (a) or (b) herein, the result would also give δ_s = 1.0.

When utilizing option (b), the frame buckling load factor (λ_c) may be found from an elastic buckling analysis (4.11), or for portal frames it should be determined directly from 4.9.2.4. This topic is covered in detail in section 7 of (4.18).

The expression for δ_s given by the buckling load factor method is consistent with that for plastic analysis given by 4.6.4. The numerator is increased to 0.95 to account for the use of more slender members and to avoid need for a two step requirement as is given by 4.6.4.1.

C4.5 ELASTIC ANALYSIS WITH MOMENT OR SHEAR REDISTRIBUTION

C4.5.1 General requirements

Elastic analysis followed by moment redistribution will be a common procedure utilized in ultimate limit state design for non-seismic and seismic applications.

Redistribution may only be undertaken for static loads and the static equilibrium between member actions and design loads must be maintained. This principally involves maintaining equilibrium between member moments and design loads; redistribution of moments will change the pattern of shear forces but account of this change is not necessarily made (4.19).

In the 1989 edition of this Standard, limits on moment redistribution were applied. Even though they applied to ultimate limit state design, they were set more by serviceability considerations, related to the incomplete handling of the serviceability limit state by the 1984 edition of the Loadings Standard. Now that serviceability limit state criteria are comprehensively covered in AS/NZS 1170 set, the limits on ultimate limit state redistribution applied in this Standard are set only by considerations of member and structure strength and ductility.

The application of moment redistribution imposes inelastic demand on a member and requires the application of additional design and detailing requirements consistent with the level of ductility demand imposed. The most straight-forward, if conservative manner of catering for the inelastic demand is to attach maximum levels of moment redistribution to the given categories of member for seismic application and to then require application of the appropriate design and detailing provisions relating to the seismic category of member. This is the approach taken in 4.5.4.

C4.5.2 Second-order effects

Redistribution of actions from an elastic analysis is undertaken in order to reduce the peak elastic analysis derived actions, thus reducing the design actions and resulting member sizes. This decrease in member size may increase the second-order effects.

If only moderate levels of moment redistribution are undertaken, then there will be little difference in the influence of second-order effects on the structural system before or after redistribution and the second-order provisions relating to elastic analysis (refer to 4.4.2) may be applied directly. This may mean simply applying the exclusion provisions of 4.4.2.2. If higher levels of redistribution are utilized, then second-order effects should be specifically considered and 4.4.2.2.2 cannot be applied.

Where second-order effects must be considered, it is important that they are determined for the structural system as sized for the redistributed design actions. Where second-order effects must be considered and moment redistribution is to be applied, then a two stage design approach is required. First, the elastic analysis is undertaken, redistribution is applied and the members sized. The second-order effects are then determined and applied to the redistributed design actions, with the members checked for their adequacy to resist the magnified redistributed design actions.

C4.5.2.3 Suppression of snap-through instability

Snap-through instability occurs in portal frames with inclined rafters. It is an elastic second-order effect, resulting in a change of deflected shape as shown in figure C4.5.2.

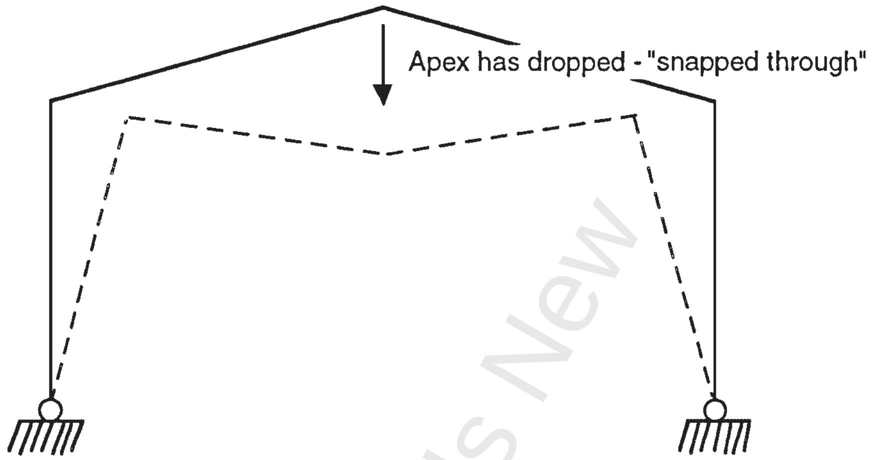


Figure C4.5.2 – Effect of snap-through

The second-order effect provisions of 4.4.3.3 (invoked by 4.5.2.3) will suppress snap-through in one or two bay portal frames. Therefore, for portal frame type structural systems, only when three or more bays exist are additional design procedures required to suppress the occurrence of snap-through. A suitable design procedure is given in BS 5950.1 (4.24); details are not given herein because of the very limited use in New Zealand of three or more bay portal frames with a configuration susceptible to snap-through instability.

C4.5.3 Calculation of lateral deflection for determination of second-order effects in sway members

The increased deflection resulting from moment redistribution may be determined in the following manner:

(a) When using 4.5.4.1 (a), use Δ_s as determined directly from the elasto-plastic analysis;

(b) When using 4.5.4.2, use Δ_s as obtained from an elastic analysis for the final member sizes selected multiplied by:

(i) 1.20 for up to 20 % redistribution in any member of the structural system: or

(ii) 1.50 for up to 50 % redistribution in any member of the structural system

(c) When using 4.5.5, use Δ_s from an elastic analysis multiplied by 1.20.

The provisions of (b) and (c) are conservative. The conservatism may be reduced by using a multiplier based on the average redistribution applied over all the bays in a given storey.

C4.5.4 Moment redistribution in moment-resisting systems of rigid construction at the ultimate limit state

These provisions are straight-forward to apply. Note that the extent of plastic hinge rotation or percentage of redistribution and the resulting category of member for section geometry and member restraint requirements are directly related. The designer chooses one and the provisions of this clause dictate the other.

As stated in Commentary Clause C4.1.1, the general application method of 4.5.4.1 allows a wide range of analysis methods to be used. Examples include the elasto-plastic push-over method as mentioned in C4.1.1, or the advanced method of analysis given in Appendix D of AS 4100:1990.

C4.5.6 Shear or moment redistribution in eccentrically braced framed systems at the ultimate limit state

These requirements are basically unchanged from the 1989 edition of this Standard and a detailed background to them is presented in section 11 of (4.19). MacRae's research (12.6) has not given any clear indication that the previous limits on redistribution can be relaxed, hence they remain the same.

The scope of this clause has been extended to cater for moment redistribution in EBFs with combined shear/flexural or flexural mode active links ($e > 1.6\ M_{sp}/V_w$) designed in accordance with references (12.3, 12.41).

C4.5.7 General design requirements

As stated in Commentary Clause C4.5.1, the inelastic demand imposed on a member through a reduction in elastic analysis derived actions by moment redistribution is catered for by attaching levels of moment redistribution to given categories of member for seismic application and then applying the design and detailing provisions relating to that category.

This clause presents the necessary requirements, which are straight-forward to apply. They call up specific provisions of section 12, irrespective of the application of the structural system to which the redistribution has been applied.

C4.6 PLASTIC ANALYSIS

The method of plastic analysis is intended to be applied to structures which transmit significant proportions of the applied loads by bending actions. The economic advantages of plastic design accrue for the bending moment redistributions which occur in statically indeterminate structures after yielding. There can be no bending moment redistribution in a determinate structure, and so there is no advantage in using plastic design in this case.

C4.6.1 Application

This clause requires the limitations of 4.6.2 to be satisfied, and also that the plastic analysis satisfy the equilibrium and boundary conditions. These conditions are illustrated in figure C4.6.1, along with the requirements from elastic analysis. This figure illustrates the fundamental differences between the two approaches.

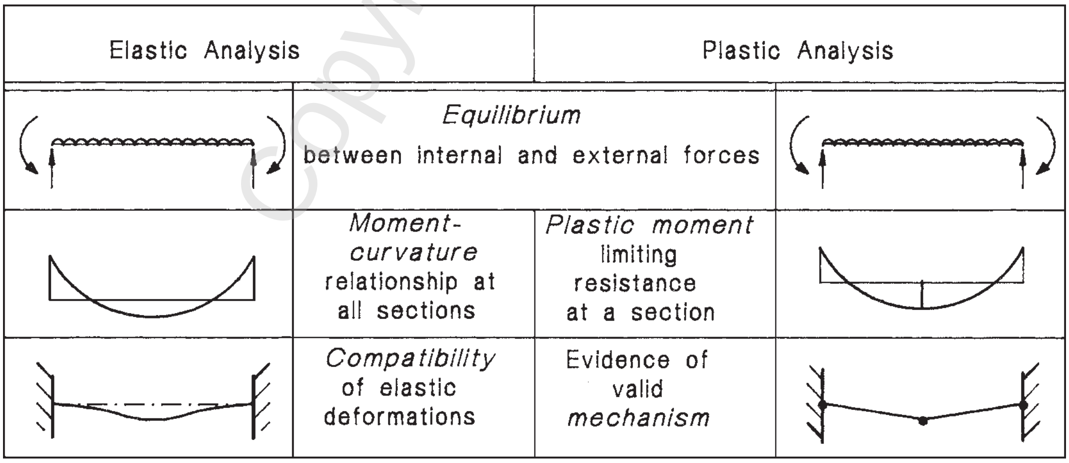


Figure C4.6.1 – Comparison of elastic and plastic design philosophies

Further details are given in section 2 of (4.18).

C4.6.2 Limitations

The method of plastic design requires the structure to remain limited ductile (to use the classification in section 12) under the design loading conditions. The method has been validated by extensive testing of a restricted range of member and material types and loading conditions.

The experimental validations were largely carried out under quasi-static and inelastic cyclic loading conditions on hot-formed, doubly symmetric, compact, I-section members with actual yield stresses not exceeding 450 MPa, and with stress-strain curves which displayed distinct elastic, plastic and strain-hardening regions. Seismic testing has also covered non-compact sections under varying levels of inelastic demand (12.7, 12.8, 12.49).

For plastic design to be applied to structures that do not satisfy all of the limitations specified in 4.6.2 (a) – (f), then this clause requires that adequate ductility and rotation capacity be demonstrated.

Its application to seismic-resisting systems is also restricted by 12.3.2.1.1, through 4.6.2 (g).

C4.6.3 Assumptions for analysis

These clauses require the member actions to be determined by a rigid plastic analysis. If an upper bound method is used in which the true collapse mechanism is not determined, then it must be followed by the lower bound method in which the loading is reduced so that the full plastic moment capacities of the members are not exceeded.

The application of plastic analysis to structures with partial strength connections is permitted, provided that the moment capacities of these are used in the analysis, and provided that they have adequate rotation capacities.

C4.6.4 Second-order effects

The usual plastic methods of analysis are first-order methods, in that any additional second-order moments are ignored. This clause allows the second-order moments to be neglected provided that the frame elastic buckling load factor λ_c is not less than 10, in which case the second-order 'elastic' moment components would be less than 10 %.

For frames with buckling load factor λ_c between 5 and 10, an amplification factor δ_p is used which varies between 1.0 and 1.13. For frames with buckling load factors λ_c less than 5, a second-order plastic analysis must be carried out (4.3), or else the structural system must be analysed elastically (4.4 or 4.5). Note that if the structural system is analysed elastically by a first-order analysis with second-order amplification to 4.4.3, then the moment amplification factors will be approaching their upper limits permitted.

Snap-through must also be considered, with design provisions additional to those contained in this Standard being necessary (invoked by 4.6.4.2) for portal frames with three or more bays. Referenced details are given in Commentary Clause C4.5.2.3.

C4.7 LIMITS ON PLASTIC HINGE ROTATION IN YIELDING REGIONS OF MEMBERS

The limits of θ_p given in tables 4.7 are derived from experimental testing of steel beam-columns undertaken for both unidirectional and cyclic inelastic loading. The values in the 1992 edition were derived from such testing up to 1990. Further testing has been undertaken over 1992, 1993 by Brownlee, who has published the results of all New Zealand testing since 1986 in (12.49) and made recommendations on changing many of the values given in the 1992 edition of tables 4.7.

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These changes have been incorporated into the 1997 revision.

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The values given for non-seismic application may also be used for seismic application involving no more than one cycle of inelastic loading to the specified value, e.g. as obtained from a numerical integration time-history analysis. Interpolation may be made for between one and four cycles of inelastic loading between the non-seismic and the seismic values.

For seismic-resisting systems, a minimum level of lateral stiffness will be required to meet the lateral deflection limits of the Loadings Standard. This will, in turn, dictate a minimum level of lateral shear strength for the system which will indirectly limit the plastic hinge rotation (θ_p) to which the members will be subjected. Designers must be aware that this lateral stiffness-imposed indirect limit may be lower than the material capability limits given in tables 4.7 and hence must check that a seismic-resisting system whose members achieve compliance with tables 4.7 also meets the lateral stiffness requirements imposed by the Loadings Standard.

C4.7.2 Plastic hinge rotaion limits

Minor changes have been made in Amendment No. 2 following a review of the experimental data and to take into account the reserve of rotation capacity required to meet the 2500 year criterion set by NZS 1170.5. The design rotation capacity is always less than the minimum experimental result for each band. The ratio of mean/design rotation capacity is higher for category 3 members than for category 1 and 2 members, and generally increases as the axial load increases. There is very limited data on which these tables have been based.

Two of the experimental tests incorporated axial loading with a constant and a variable component, with the variable component being a function of the enforced lateral displacement. The two axial load regimes involved were $n = 0 \pm 0.3$ and $n = 0.3 \pm 0.3$, where n = fraction of nominal section capacity and $n > 0$ signifies compression. The same section size was also tested to the same displacement controlled lateral loading regime but with the axial load held at the constant value. The behaviour and inelastic rotation capacity of the specimen with the constant + variable axial loading in each instance was very similar to that of the specimen with constant axial loading of $n = 0$ and $n = 0.3$, respectively. This is consistent with the first principles model of axial shortening developed by MacRae (12.6). This shows that it is the average level of compression load over a full cycle of inelastic demand that is important in determining the available inelastic rotation capacity. Note (4) expresses this, however it has been amended to give the correct requirement (the average load over a cycle of response) when the earthquake component is non-symmetrical over the cycle. The previous note stated the seismic load component could be ignored, which is only applicable when the earthquake loading component is symmetrical.

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C4.8 MEMBERS BUCKLING ANALYSIS

Introduction

The clause deals with the calculation of the elastic in-plane buckling loads N_{om} of compression members.

The buckling load N_{om} of a member depends on the restraining effects of the structural elements connected to its ends. The buckling load is determined in 4.8.2 from an effective length factor k_e, which is obtained from 4.8.3.

C4.8.3 Member effective length factor

C4.8.3.1 General

Clause 4.8.3.1 directs designers to the appropriate provisions for determining the effective length required. It also differentiates between the effective length required for elastic second-order effect determination (k_e > 1.0) and the effective length required for individual compression member design (k_e = 1.0) in sway structural systems/members. The clause also differentiates between non-seismic and seismic applications; compression member effective length requirements for the latter are given in 12.8.2.

The value of the member effective length factor (k_e) depends on the rotational restraints and the translational restraints at the ends of the member. In figure 4.8.3.3(a) for a braced member, the translational restraint has been assumed to be infinite. In figure 4.8.3.3(b) for a sway member, the translational restraint has been assumed to be zero. Effective length factors for members with intermediate (elastic) values of translational restraint can be obtained from the approximate method of (4.13).

C4.8.3.2 Members with idealised end restraints

This clause gives approximate values for the effective length factors of braced and sway members with idealized end restraints. The value of 0.7 given for a member fixed against rotation at both ends corresponds to the use of stiffness ratios γ of 0.6 approximately, and assumes that full fixity will not occur in practice. The approximate value of 0.85 given for a member fixed at one end also corresponds to a value of the stiffness ratio γ which is close to 0.6.

C4.8.3.3 Members in frames

The basis for figure 4.8.3.3 is given in (4.9) or in (4.18). The clause directs the designer either to 4.8.3.4 or Appendix G for determination of the values of γ for non-seismic applications and to 12.8.2 for seismic applications.

For the special case of in-plane buckling of a portal frame (i.e. each column buckling about its major principal *x*-axis) any enhancement of design actions due to frame instability is accounted for by 4.4.2 and 4.9.2.4. For the individual column members which form part of the rigid portal frame, in-plane design for combined axial and bending actions to 8.4.2 is not required. Section capacity to 8.3.2 and out-of-plane capacity to 8.4.4 (the latter only if the column does not have full lateral restraint to 8.1.2.3) will require checking for levels of axial force that are significant (as defined by 8.1.4). This means that the calculation of member effective length about the major principal *x*-axis for the outer columns of a portal frame is not required.

C4.8.3.4 Stiffness ratios in rectangular frames

These clauses give a method for calculating the stiffness ratios γ for a compression member in a rectangular frame with regular loading and negligible axial forces in the beams. A more accurate approximate method for members in non-rectangular braced frames or in rectangular braced frames which may have irregular loading or axial forces in the beams is given in Appendix G.

A method is not given for non-rectangular sway frames, nor is one given for sway frames which may have irregular loading or axial forces in the beams. In these cases the influence of second-order (P-Δ) effects on the (members of) the frame as a whole are either determined directly through second-order analysis (Appendix E) or through the calculation of the elastic buckling load factor (4.9.2) for use in 4.4.3.3.2(b). Neither procedure requires the designer to explicitly calculate the member effective length. For design of individual sway members that are subject to compression or to combined compression and moment actions in such a frame, the in-plane member effective length factor is always taken as 1.0.

The method given in 4.8.3.4 assumes that the frame and loading are regular so that all compression members are equally critical, and that the beams have zero axial forces so that their stiffnesses are not reduced by buckling effects. For a more detailed explanation, refer to section 7 of (4.18). Regular loading means uniform or near uniform gravity loading on the frame, i.e. no significant concentration of high levels of dead or live loading over only part of the frame.

The formulation given for calculating the stiffness ratios includes the factor β_e which allows for different conditions at the far end of the beam. In braced frames, the worst condition is when the end conditions are the same so that the beam restrains columns at both ends, and the best condition is when the far end is fixed. However, in sway frames, the worst condition is when the far end is pinned so that the beam is bent in single curvature, and the best is when the end conditions are the same, so that the beam is bent in double curvature.

The theoretical limiting values for the stiffness ratio γ are infinity for an unrestrained compression member and 0 for a built-in member. The restraints offered by footings at the bases of compression members are uncertain and not easily calculated, and so limiting values are given of 10 for members which do not have moment connections to the footing, and 0.6 for members which do.

C4.8.3.5 Members in triangulated structures

The designer must ensure that the effective lengths for in-plane and out-of-plane buckling are selected appropriately; for chord members of trusses they are unlikely to be the same. Guidance on applying 4.8.3.5.1 is given in section 7.6.1.2 of (4.18).

C4.9 FRAME BUCKLING ANALYSIS

Introduction

This clause deals with the calculation of the elastic buckling load factor λ_c of a frame.

For the analysis of elastic in-plane buckling, all member bending actions are ignored, and only the member axial force actions are considered (4.18). The buckling behaviour depends on the flexural interactions between members with moment connections, and so most members participate in the buckling action, and contribute to the buckling behaviour, especially in a rigid sway frame.

C4.9.1 General

The frame elastic buckling load factor λ_c for a particular load set is defined as the ratio of the axial force in a member of the frame at elastic frame buckling to its design axial force. Because all members of a sway frame with moment connections interact during buckling, this ratio is the same for all members. In a braced frame with moment connections, this interaction is very limited.

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The elastic buckling load factor is calculated for second-order effect determination, typically using the moment amplification method of clause 4.4. As described in C4.4.2, this method becomes unreliable for $\lambda_c < 3.5$, however this limit is not expressly stated in the code clause 4.9 for calculating λ_c and may be overlooked, hence its inclusion.

C4.9.2 In-plane frame buckling

C4.9.2.1

This clause directs the designer to one of 4.9.2.2, 4.9.2.3 or 4.9.2.4, or requires a rational buckling analysis of the whole frame. This will usually require the use of a suitable computer program (4.11). Unless such a program is used, it may be difficult to carry out an accurate manual analysis, although a relatively straight-forward hand method for a rectangular sway frame is given in section 7 of (4.18).

However, it is possible to determine a member's buckling load by considering the restraining effects of the structural elements connected to each end of the member. This member buckling load may be used to approximate the frame buckling load for a particular load set. The restraining effects depend on the axial forces in the structural elements at elastic frame buckling, and so the method is generally an iterative one (4.13, 4.14). For regular rectangular frames with negligible axial forces in the restraining beams, however, the frame buckling load may be approximated directly from the member buckling loads using 4.9.2.2 and 4.9.2.3.

For frames with plated columns and frames with stepped columns, the approximate methods of (4.15, 4.16, 4.17) may be used. These methods determine the frame effective length factors for the members, from which the axial forces in the members at elastic buckling, and hence the elastic buckling load factor (see C4.9.1), can be calculated.

C4.9.2.2 Rectangular frames with all members braced

For regular rectangular braced frames with regular loading and negligible axial forces in the beams, the frame elastic buckling load factor λ_c may be closely and conservatively approximated from the lowest of the member buckling load factors λ_m as calculated from the member buckling loads N_{om}, determined using 4.8.2 and 4.8.3. This method may be too conservative for other braced frames, in which case the iterative method of (4.14) may be used. Refer to (4.18) for a more detailed explanation.

C4.9.2.3 Rectangular frames with sway members

For these structures, a simple method is given of approximating the frame elastic buckling load factor λ_c by the lowest storey buckling load factor λ_{ms}. For other sway structures, except as covered by 4.9.2.4, a frame buckling analysis should be made, unless a solution is available in the literature (4.18 or 4.13 – 4.17).

C4.9.2.4 Portal frames of rigid construction

The one common form of non-rectangular sway frame is a pitched roof portal frame. The sloping rafters attract axial compression force from applied vertical loading and this decreases the structure's elastic stability. Thus in portal frames with rafters inclined at a significant angle to the horizontal (over 5^o – 10^o), the determination of second-order effects must take proper account of the effect of rafter compression force.

The method given is that proposed by Davies (4.23). It uses simple equations for the cases of pinned and fixed bases to determine an accurate value of elastic buckling load factor (λ_c), which is then used in 4.4.3.3.2(b) to determine δ_s. The method requires an initial elastic analysis to determine the axial forces in the (outer) rafter and column generated by the particular load combination under consideration, however this is straight-forward to undertake, either by computer analysis or through the use of published equations, e.g. (4.25).

Figure C4.9.2.4 gives a pictorial definition of the parameters used in 4.9.2.4.

Under non-seismic load combinations involving lateral loads, such as wind load, the axial compression forces in the column and rafter on one side of the portal frame will exceed those on the other side. The greater values are to be used in design, as they generate a lower value of λ_c.

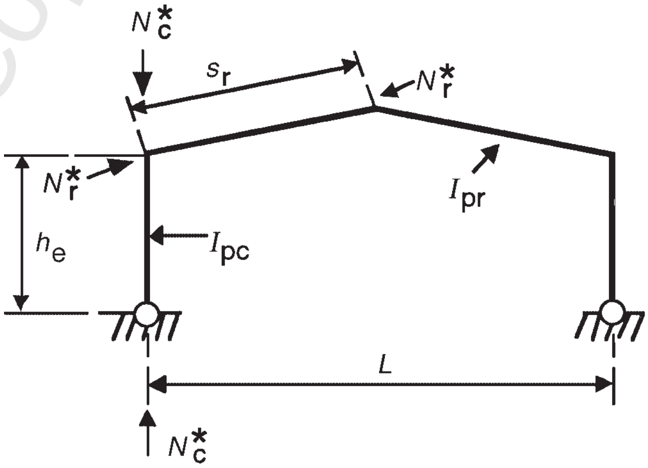

Figure C4.9.2.4 – Parameters used in 4.9.2.4
(pinned base example shown)

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NOTES

C5 MEMBERS SUBJECT TO BENDING AND SHEAR

INTRODUCTION

The strength design of members for bending moment is governed by 5.1 to 5.8. The strength design of member webs for shear, bending, and bearing is governed by 5.9 to 5.16. Further information is given in Appendices H and J, and in (5.1, 5.2). The behaviour on which these clauses is based is described in design guides (5.36) or in textbooks such as (5.3). This commentary should be read in conjunction with such a design guide or textbook.

Worked examples for the strength design of members subject to bending are presented in (5.4, 5.36), while computer programs are described in (5.5, 5.6).

Clause 5.1 directs the designer to the appropriate later clause for designing against each failure mode. Clause 5.2 defines the section moment capacity, which is governed either by yielding or local buckling. The section moment capacity controls the design of members with full lateral restraint, which are defined in 5.3. The lateral restraint systems are classified in 5.4. Clause 5.5 defines the critical flange. Clause 5.6 defines the member moment capacity which is governed by lateral buckling, and the material in the clause is amplified in Appendix H. Clause 5.7 deals with the biaxial bending of members, and 5.8 with separators and diaphragms. A flow chart for design for bending moment is given in figure C5.1.

Clause 5.9 directs the designer to the appropriate later clause for designing member webs against each failure mode including direction to the relevant provisions of section 12 for seismic applications. Clause 5.10 provides details of the web arrangement. Clause 5.11 governs the design of webs for shear, 5.12 for shear and bearing, and 5.13 for bearing. Appendix J provides information for the design of stiffened webs against combined shear, bending, compression, and bearing. Clause 5.14 deals with the design of load-bearing stiffeners, 5.15 with intermediate transverse stiffeners, and 5.16 with longitudinal stiffeners.

No specific clauses are given for floor plates, but these may be designed for the ultimate limit state as compact sections (with $Z_e = 1.5Z$) with full lateral restraint. The design of floor plates in two-way bending will usually be governed by the serviceability limit state.

The provisions of 5.1 – 5.6 look very different to the corresponding requirements of the 1989 edition of this Standard, but are just as simple to apply. The following brief guidance is taken from (5.41), which provides a simple introduction to all aspects of design to this Standard.

The guidance relates to the use of stock parallel flange sections that have a constant cross section along their length and are being subjected to major principal *x*-axis bending moment. More general guidance on member moment capacity determination is given in C5.3.1, see especially C5.3.1(9).

Having calculated the design actions, determining that the member has adequate moment capacity, in accordance with 5.1 – 5.6, will typically proceed as follows:

(1) Calculate the design section moment capacity, ϕM_{sx}.

Amd 1 June '01 $\phi M_{sx} = \phi f_y Z_{ex}$ from 5.2.1

(i) Z_e (Z_{ex}, Z_{ey}) is listed in steel suppliers handbooks.

(ii) f_y is taken for the flanges and is listed in steel suppliers handbooks.

(iii) Some handbooks list ϕM_{sx}, ϕM_{sy} directly.

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(2) Calculate the design member moment capacity for x-axis bending, ϕM_{bx}.

(2.1) The member moment capacity is determined for a particular segment, as described in 5.3.1.1 and 5.3.1.2.

Note that a segment with both ends restrained is defined as the length between **adjacent** cross sections which are restrained (see 5.3.1.2(a)). It is important to keep this in mind when determining ϕM_{bx}.

(2.2) In calculating ϕM_{bx}, the restraint provided against lateral movement of the critical flange must be considered. Restraint conditions are classified as full (F), partial (P), lateral (L) or unbraced (U) by 5.4. Figures 5.4.1 to 5.4.2.3 provide generic guidance as to the conditions required for each type of restraint. Restraint conditions for 42 "as-built" connection details are given in (5.40), which takes the guesswork out of determining the effectiveness of real restraint conditions.

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(2.3) Clause 5.3.2.1 gives a number of restraint conditions which constitute full lateral restraint, such that $\phi M_{bx} = \phi M_{sx}$. In conjunction with (5.40), it is straightforward to determine if one of these options applies to a member under consideration.

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Alternatively, for structures such as portal frames, the decision can be made to provide full lateral restraint to the columns and rafters, then restraint details from (5.40) are added to deliver this. The design moment capacity of the frame is then simply ϕM_{sx}.

Amd 1 June '01

(2.4) For general situations, $\phi M_{bx} = \alpha_m \, \alpha_s \, \phi M_{sx}$ from 5.6.

(2.5) α_m is typically obtained from table 5.6.1. Conservatively it can be set at 1.0. For a simply supported beam segment loaded with a uniformly distributed load, $\alpha_m = 1.13$; loaded with a point load, $\alpha_m = 1.35$. For a segment with a triangular moment distribution, from M^*_{max} to zero, $\alpha_m = 1.75$. These values should be memorized as they are commonly used in design.

(2.6) α_s is a function of section property and effective length, L_e, for lateral buckling. For hot-rolled and welded I-sections, α_s tables are available from HERA or the steel suppliers.

(2.7) L_e is determined from 5.6.3 for the segment under consideration. For typical building applications involving compact and non-compact uniform I-sections loaded in a plane of symmetry and with a ratio of segment length (L) to beam depth (d) exceeding 11, the following value of L_e can be conservatively used:

L_e = 1.0L for segments with F or L restraints at both ends
L_e = 1.1 L for segments with P restraints at both ends
L_e = 1.05 L for segments with F or L restraint conditions at one end and P restraint conditions at the other

(2.8) ϕM_{bx} is listed directly for a given L_e and for α_m = 1.0 in some published handbooks.

Details of the steel suppliers handbooks are referenced in (5.41) or available from HERA. Design examples (5.42) are also available from HERA.

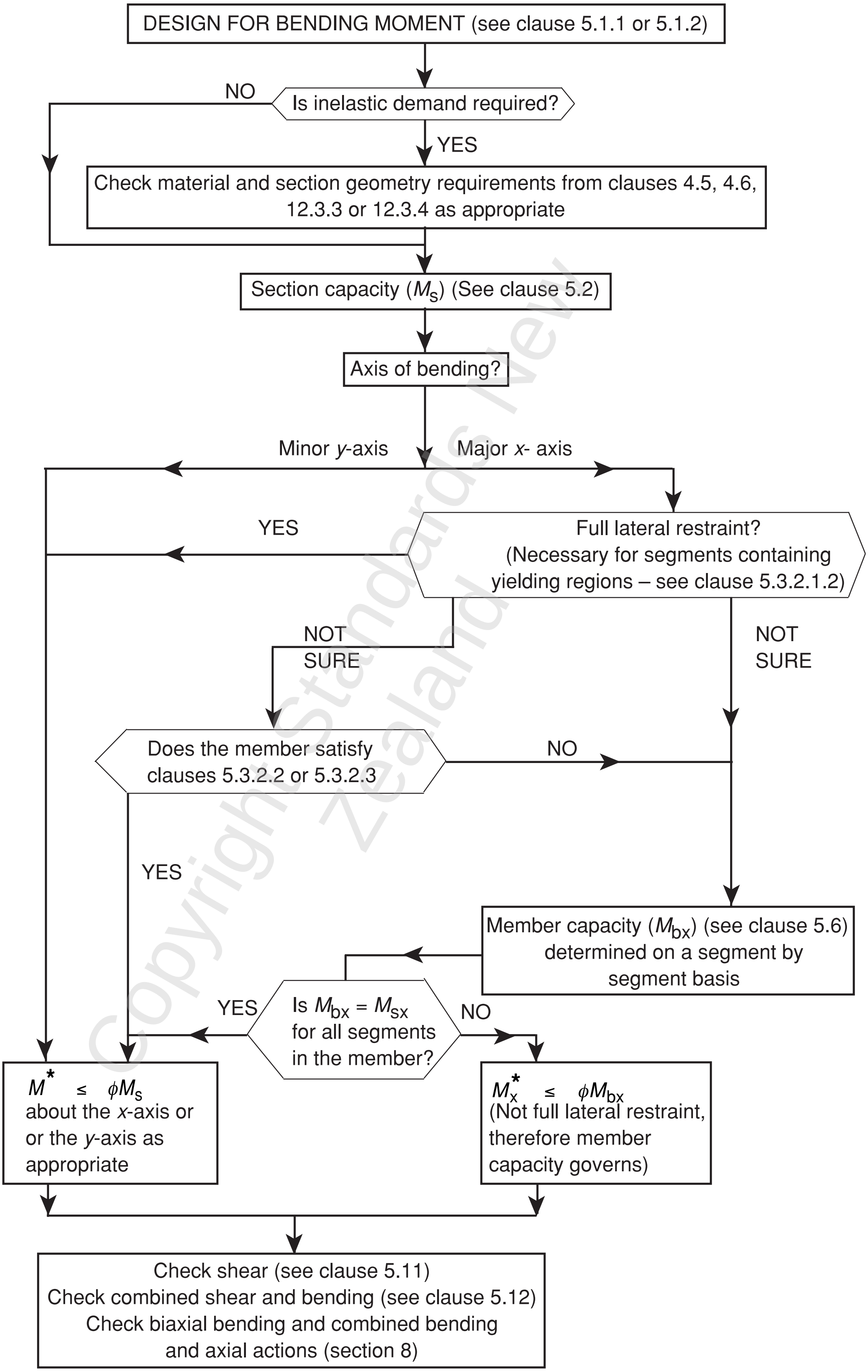

Figure C5.1 – Flowchart for member design for bending moment

C5.1 DESIGN FOR BENDING MOMENT

This clause provides the relationships between the design bending moment M^* and the nominal capacities of the member to resist cross section yielding or local buckling (section moment capacity, M_s) and overall flexural-torsional buckling under major principal x-axis bending (member moment capacity M_{bx}). These all involve a strength reduction factor of $\phi = 0.9$.

Note that members must satisfy the requirements of 5.12 for combined bending and shear. This will only affect members with high coincident moment and shear and whose webs are required to provide some resistance to bending moment.

C5.1.2

The NOTE to 5.1.2 requires designers to consider the destabilizing effect from a laterally unrestrained load generating minor principal y-axis bending and which is applied above the shear centre.

When this load is applied about an axis which is an axis of symmetry (this will be the case in all practical applications), then the destabilizing effect need only be considered when the laterally unrestrained load is applied at a distance $d_{ls} \geq d$ above the shear centre. The distance d_{ls} is measured from the centre of mass of the applied load to the shear centre of the section and d is the depth of the section.

In such instances, the member should be checked for combined y-axis bending and torsion, with the design torque, $M_z^* = 0.01F^*d_{ls}$, where F^* is the laterally unrestrained design load generating the bending and d_{ls} is the distance given above. This check is carried out to C8.5 herein; see also section 8 of (8.14) for design guidance on combined bending and torsion.

C5.2 NOMINAL SECTION MOMENT CAPACITY FOR BENDING ABOUT A PRINCIPAL AXIS

C5.2.1 General

The nominal section moment capacity (to resist yielding or local buckling) is defined by the yield stress and by the effective section modulus. The effective section modulus is defined in 5.2.3 – 5.2.5 in terms of the section slenderness parameter defined in 5.2.2, which provides a measure of the relative importance of yielding and local buckling. Note the terminology used; table 5.2 refers to the plate element slenderness (λ_e), which, when the plate elements being referred to are part of a section, becomes the section slendernesses parameter (λ_s) via application of 5.2.2.

Note that the calculation of section moment capacity is the same for seismic and non-seismic applications, i.e. to 5.2.1. For seismic applications, physical limitations (see clause 12.5) are placed on the section, appropriate to the category of member as defined in 12.2.5. A similar requirement applies to moment redistribution, through 4.5.4.

C5.2.2 Section slenderness parameter

The section slenderness parameter is used to determine the effects of local buckling on the effective section modulus. For sections consisting of flat plate elements, it is defined in terms of the width-thickness ratio of the most slender compression plate and its yield stress, and is used to classify a section as compact, non-compact, or slender. This and subsequent expressions involving the yield stress are arranged so that the yield stress term can be omitted altogether when $f_y = 250$ MPa. Flat compression plates include those supported on one or both longitudinal edges, as well as those in uniform and non-uniform compression.

Examples of typical plate elements for the application of tables 5.2, 8.1 and 12.5 are:

(a) The flange of an I or channel section is a flat element in uniform compression with one longitudinal edge supported when subject to compression from principal x-axis bending or axial compression force;

(b) The flange of a hollow box section is a flat element in uniform compression with both longitudinal edges supported when subject to compression from bending or axial compression force;

(c) The web of an I or channel section is a flat element with compression at one edge and tension at the other (both edges supported) when subject to principal x-axis bending.

Note that the provisions of 5.2.2 are applied to plate elements in either uniform or non-uniform compression. In the case of an I-section in bending about the major principal x-axis, they therefore apply both to the compression flange and to the web, one half of which is put into bending – induced compression. Thus both these elements will require checking against the appropriate plate element type limits from table 5.2. In the case of a section with compression flange slenderness not exceeding λ_{ey}, but with web slenderness exceeding λ_{ey}, the section will be classified as slender in accordance with 5.2.5 on account of the web slenderness. This will reduce the section moment capacity (M_s).

The section slenderness parameter of a circular hollow section is defined in terms of its outside diameter to thickness ratio and its yield stress.

Flat plates may be supported by webs and longitudinal stiffeners. When longitudinal stiffeners are used, they must be stiff enough to effectively prevent deflection of the stiffened plate. For a single stiffener between 2 supporting webs, it is suggested that the stiffener should have a second moment of area I_s which satisfies:

$$I_s \geq 4.5bt^3 [1 + (2.3A_s/bt)(1 + 0.5A_s/bt)]$$

in which b is half the plate width between supporting webs, t is the plate thickness, and A_s is the stiffener area (see also (5.7) or section 9 of (5.36)).

For an edge stiffener alone, it is suggested that the stiffener should have a second moment of area which satisfies:

$$I_s \geq 2.3bt^3 [1 + (4.6A_s/bt)(1 + 0.5A_s/bt)]$$

in which b is the plate width between the supporting web and the edge stiffener. For plates with more than a single stiffener, higher stiffness may be needed to prevent global buckling of the stiffened plate. Specific advice on this is given in (5.7, 5.8) and general guidance is given in (5.36).

The yield limit given in table 5.2 for an element with both longitudinal edges supported, compression at one edge and tension at the other, is that associated with the theoretical onset of elastic buckling at the yield stress (f_y) for a simply supported plate element in bending (5.3, 5.36). This limit does not make allowance for the significant post-buckling reserve of strength available in such an element nor for an increase in the resistance to elastic buckling due to flange restraint. In the case of the web of a doubly symmetric I-section in bending about its major principal x-axis, these 2 factors allow the yield limit for bending to be increased from 130 to 180 (5.3, 5.36). This increase is given in Note (5) to table 5.2. Clause 5.10.1.1 also limits the normalized slenderness ratio of the unstiffened web of a (doubly-symmetric) I-section to 180, which means that such an unstiffened web will not be slender in section bending about the major principal x-axis.

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Narrow rectangular sections without either longitudinal edge supported laterally and bent about their major principal *x*-axis will fail by lateral buckling (5.6) rather than by local buckling, hence the determination of their section capacity based on 5.2.3 is appropriate.

Table 5.2 differentiates between slenderness limits for welded members on the basis of sections that are lightly or heavily welded. This relates only to members formed by the longitudinal welding together of plate elements. The effects of welded attachments (e.g. cleats, stiffeners, studs) need not be considered.

A material supplier of welded sections will typically include the appropriate classification (HW, LW) in their product literature.

Rectangular and square hollow sections are much more susceptible to loss of bending strength from combined local flange and web buckling than is the case for I-sections. Because of this, their web slenderness, for a given flange slenderness, needs more stringent limits than are in table 5.2. Suitable limits for web slenderness on these types of cross section have been introduced in Amendment No. 1. A background to these limits is to be given in the HERA *Steel Design and Construction Bulletin*, Issue No. 55, April 2000.

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C5.2.3 Effective section modulus for compact sections

Compact sections are those for which the full plastic moment can be reached and maintained until a plastic collapse mechanism develops, without any decrease in section capacity due to local buckling effects. For these sections, the section moment capacity is determined by the lesser of the plastic section modulus or 1.5 times the elastic section modulus, the latter corresponding to the case of a solid rectangular section.

The plasticity slenderness limits for compact sections are given in table 5.2 for different plate support and stress conditions. Plates supported on one edge include the flange outstands of UB, UC, welded *I*, channel, angle and T-sections, while plates supported on both edges include the flanges of box sections and the webs of sections with 2 flanges. All UBs in Grade 250 steel of Australian origin are compact, as are most (but not all) UCs.

The plasticity limits for flat plates supported on one edge in uniform compression are also used when the elastic stress distribution is non-uniform, as in the case of the stems of tee-sections and the flanges of *I*-sections bent about the minor axis, since these stress distributions become more uniform after yielding. These limits may also be used for other stress distributions.

The plasticity limit for plates supported on both edges and in uniform compression may also be used for such plates in non-uniform compression.

C5.2.4 Effective section modulus for non-compact sections

Non-compact sections have section slendernesses lying between the plasticity and yield limits of table 5.2. The yield limits are the same as those used for compression members in table 6.2.4 (except for circular hollow sections), and are generally based on lower bound fits to the experimentally determined local buckling resistances of plate elements in uniform compression (5.8).

The resistance to local buckling is generally affected by the level of the residual compressive stresses induced in a member during manufacture or fabrication. Thus the plate slenderness yield limits given in table 5.2 are highest for stress relieved members, and lowest for heavily welded members. The distinction in table 5.2 between lightly and heavily welded members is set at 40 MPa. The magnitudes of the welding residual stresses increase with the rate of heat input to the weld, and so members with small welds or multi-pass welds can be expected to be lightly welded. Information for predicting the residual compression stresses induced by welding can be obtained from (5.9).

The yield limits given in table 5.2 for plates supported on one edge and in non-uniform compression are based on elastic buckling at the yield stress of plates with maximum compression at the free edge, and zero stress at the supported edge. The values for uniform compression may conservatively be used for other stress distributions, or reference made to more detailed guidance (5.36).

The yield limits for plates supported on both edges and in uniform compression may conservatively be used for plates in non-uniform compression. A higher limit is given for plates with compression at one edge and tension at the other (5.36).

The nominal section moment capacity of a section having a slenderness equal to the yield limit is based on the elastic section modulus.

The capacity of a section whose slenderness is in the range between the plasticity and yield limits is obtained by linear interpolation.

C5.2.5 Effective section modulus for slender sections

Slender sections have section slendernesses higher than the yield limits of table 5.2. None of the hot-rolled sections of New Zealand or Australian origin Grade 250 or 300 steel are slender when used in bending, except for some of the thinner angles, and some tees cut from UBs.

For a section with a flat plate element in uniform compression, the section capacity is based on the post-buckling capacity, and the effective section modulus is taken as being inversely proportional to the section slenderness. An alternative method, which is based on the effective width concept, is permitted for these sections, in which any widths in excess of those corresponding to the yield limits are ignored in the calculation of the effective section modulus.

A section with a flat plate element in non-uniform compression may conservatively be treated as if in uniform compression. For a section with a flat plate element with compression at a free edge and tension at a supported edge, the capacity is taken as equivalent to the elastic buckling capacity, and the effective section modulus is conservatively taken as being inversely proportional to the square of the section slenderness (5.10).

For circular hollow sections, 2 approximations are given for the effective section modulus, the first for moderate slendernesses, and the second for high slendernesses. The second is applicable for slenderness ratios up to around 450.

It should be noted that the section modulus is the only property that is reduced below the value for the full cross section in design. Other properties, such as those used in determining the lateral buckling capacity or the section deformation, are not reduced.

C5.2.6 Deformation of flat plate elements at the serviceability limit state

Some deformation slenderness limits are given in table 5.2 for flat plate elements in uniform compression. When these are exceeded, elastic local buckling may occur at the serviceability limit state, leading to noticeable distortions or deformations which do not affect the section moment capacity.

C5.2.7 Elastic and plastic section moduli

When calculating the elastic section modulus, Z, for a section in which the principal axis under consideration is not an axis of symmetry, 2 different values of Z are obtained, i.e. Z_t and Z_b (top and bottom). **For calculating Z_e, Z = minimum of (Z_t, Z_b) must be used.**

This clause allows the effects of small holes on the elastic and plastic section moduli to be neglected, while requiring that these effects be calculated for larger holes, either by a simple approximation, or from the net section. For Grade 350 steel to AS 3678, for example, loss of flange area of up to 8.5 % may be ignored in calculation of section modulus.

The clause is based on the results of experiments (Chapter 16 of (5.11)) which show that compact beams with bolt holes in their flanges can still reach the full plastic moment capacity of the gross section, because of local strain-hardening around the holes. The limit given for the maximum size of holes that can be neglected is consistent with that used in 7.2 for holes in tension members.

Note that if such members form part of a seismic-resisting system, the extent of holes permitted in yielding regions (see 12.1) is also limited by 12.9.4.2 to avoid premature failure at the bolt holes.

Fastener holes in webs have virtually no effect on the moment capacity, but reduced section moduli should be calculated from the net sections of beams with large openings in their webs.

C5.3 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS AND MEMBERS SUBJECT TO MAJOR PRINCIPAL X-AXIS BENDING AND WITH FULL LATERAL RESTRAINT

C5.3.1 General

Clauses 5.3.1.1 to 5.3.1.4 lay down the basis for determining the nominal member moment capacity of segments and members subject to major principal *x*-axis bending. Clause 5.3.1.5 then defines the condition of full lateral restraint, as a lead in to clause 5.3.2 which provides the requirements to be met in order to achieve full lateral restraint of segments and members. Clause 5.3.2 in turn leads into clause 5.6, which provides the general procedure for calculating the nominal member moment capacity.

The key points towards determining the nominal member moment capacity are as follows:

(1) The member moment capacity is required to be determined for *x*-axis bending. For *y*-axis bending, the section moment capacity only is required, except for the case of laterally unrestrained loads applied well above the shear centre, as covered by C5.1.2.

(2) Members span between supports or between a support and a free end (as defined in 5.3.1.3). The role of a support is defined in 1.3.

(3) Members are usually restrained at discrete points between their supports. The role of a restraint against bending is defined in 1.3.

(4) The effect of the restraints is to divide the member into segments. The definition of a segment is given in 5.3.1.2. Member moment capacity is then determined on a segment by segment basis, as invoked by 5.3.1.1.

(5) Supports must provide full or partial restraint to a cross section, as shown in the left hand diagram of figure 5.4.2.1(a) or of figure 5.4.2.2, and as given in (5.40).

(6) Restraints will provide full, partial or lateral restraint to a cross section, depending on the nature of the restraint and the location of the critical flange (as defined in clause 5.5) relative to the restraint. See figures 5.4.2.1 to 5.4.2.3 and details in (5.40).

(7) The member moment capacity of members which contain one or more yielding regions is determined in the same manner as detailed in (1) to (6) above, except that:

(7.1) If the member is subject to bending moment only (i.e. the axial force is insignificant as given by 8.1.4), then any segment containing a yielding region must have full lateral restraint, as invoked by 12.6.2.2 and 12.6.2.5(a).

(7.2) If the member is subject to combined bending and significant axial actions, then all segments within the member must have full lateral restraint, as invoked by 12.6.2.5(b).

(8) Common examples of members are:

- the columns of portal frames
- the length of portal frame rafter between the knee and the apex
- beams supporting floors

(9) The first point of consideration in determining the member moment capacity is to determine the position and type of supports and restraints to the member. Once these details are known, the procedures of 5.3 and 5.6 are applied to determine the member *x*-axis moment capacity.

C5.3.1.2
Designers should note the definition of a segment as being between adjacent points of restraint (for segments restrained at both ends). Typically it is conservative, sometimes overly so, to apply the segment based procedure between non-adjacent points of restraint. If, however, M^* is considerably less than ϕM_b for a segment based on adjacent points of lateral restraint (see 5.4.3.1), then the segment length can be taken as the length to the centreline of multiple points of lateral restraint and the restraining force then shared between the various individual lateral restraints in accordance with 5.4.3.1.2. See also C5.4.3.1 for further guidance.

For segments with one end unrestrained, the segment is defined as the length between that end and the adjacent cross section which is fully or partially restrained. Care must be taken in the application of this definition, especially to segments under gravity loading. Refer to (5.43) for guidance in relation to a length of portal frame rafter from the knee to the point of contraflexure when the portal frame is subject to gravity loading.

C5.3.2 Segments or members with full lateral restraint

C5.3.2.1 General requirements for full lateral restraint
Clause 5.3.2.1.1 provides the obvious definition of full lateral restraint in terms of preventing failure by lateral buckling, by ensuring that $M_{bx} = M_{sx}$. It also makes clear the relationship between full lateral restraint of an individual segment and full lateral restraint of an overall member.

Clause 5.3.2.1.2 specifies that segments containing a yielding region must have full lateral restraint and cross references to the appropriate clause in section 12.

Clauses 5.3.2.2 to 5.3.2.4 give 2 common situations where members supported at both ends can be considered to have full lateral restraint, without the need for designers to formally check for this on a segment by segment basis.

The first situation is covered in 5.3.2.2 and applies to, for example, a simply supported floor beam supporting a concrete floor slab off the top flange. In this case, the member in effect comprises a very large number of segments, each of near zero length.

The second situation is covered in 5.3.2.3 and 5.3.2.4. It applies to, for example, a simply supported floor beam supporting timber joists which in turn support a floor surface.

The second situation can also cover portal frame rafters, provided that the designer notes which flange is critical in relation to the change of sign of bending moment along the rafter.

C5.3.2.4 Spacing between adjacent restraints
This clause gives limiting segment slenderness ratios that are allowed between the adjacent restraints (including between the support and the first lateral restraint in from the support at

each end of the member). The ratios are given in terms of end moments M and $\beta_m M$ and are higher for lengths where the moment gradient is high.

The limits for channel section members are seemingly lower than those for I-section members because they have higher values of r_y. The limits for rectangular hollow section members, which may also be used for circular hollow section members by setting $b_f = b_w$, are high because these members rarely buckle laterally.

If the only transverse load over a given length of member under consideration is the self-weight of the member, then this length need not be considered subject to transverse load for the purpose of determining β_m and hence option (c) may be used. The moment induced by the self-weight of the member must be included in the calculation of design moment.

C5.3.3 Critical section

The critical cross section is the section which will control the design of a member with full lateral restraint. It is therefore defined in terms of the relative magnitudes of the design bending moment and the nominal section moment capacity.

C5.4 RESTRAINTS

C5.4.1 General

These clauses direct the designer to the appropriate following clauses.

An definition of an unrestrained cross section is given here for the sake of completeness.

C5.4.2 Restraints at a cross section

These definitions are of the various restraint conditions considered when designing against lateral buckling. Examples of these restraint categories are shown in figures in each sub-clause.

Amd 1 June '01 Designers will find the guidance given in (5.40) particularly useful in applying 5.4. See also section 2.5 of (5.40) for guidance on the strength requirements of practical restraining systems.

C5.4.2.1 Full section restraint (F)

While theoretically restraints must often be infinitely stiff to provide full restraint, a more realistic approach is taken by requiring the restraints to effectively prevent lateral deflection and twist of the cross section. The restraint requirements for full section restraint are given in 5.4.3.1 and 5.4.3.2.

The examples of full section restraint shown in figure 5.4.2.1 (a) are ones for which the lateral deflections of both flanges are effectively prevented, while those shown in figure 5.4.2.1 (b) are ones for which the lateral deflection of the critical flange (refer to 5.5) is effectively prevented, in which case the moment capacity required for effective twist restraint is typically available from any connection which prevents rotational slip between the restrained cross section and the restraining member(s).

The examples shown in figure 5.4.2.1 (c) are ones for which the lateral deflection of some other point of the cross section than the critical flange is effectively prevented, in which case effective restraint against twist rotation is required, and partial twist restraint alone is not sufficient.

The traditional connection between a purlin and a rafter, utilizing a purlin cleat and bolts in standard 2 mm oversize bolt holes, may be considered a moment connection, thus providing full restraint as shown in figure 5.4.2.1 (b), centre diagram. **If slotted holes or oversize holes**

greater than 2 mm are used in either the purlin or the purlin cleat, then the bolts in the purlin to purlin cleat connection (and flybrace) will need to be fully tensioned property class 8.8 bolts to prevent excessive twist due to bolt slip (5.36).

A purlin to rafter connection formed by bolting the bottom flange of the purlin directly to the rafter top flange does not provide significant twist rotation rigidity for full or partial restraint and must be considered as a pin connection only, providing lateral restraint to 5.4.2.3.

C5.4.2.2 Partial section restraint (P)

The distinction between full and partial section restraint is qualitative only, but some quantitative guidance on the effects of partial section restraints is given in H5 of Appendix H. The examples shown in figure 5.4.2.2 are ones for which lateral deflection of some other point of the cross section than the critical flange is effectively prevented, and for which there is only partial restraint against twist rotation of the cross section.

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In general, a steel purlin may be classified as "stiff" if its depth equals or exceeds half the depth of the member it is restraining and the ratio of purlin span length to depth does not exceed 30 for a simply supported purlin or 40 for a continuous purlin, where the purlins extend on both sides of the restrained member. If its depth is less than half the restrained member depth, or its span/depth ratio exceeds the limit given above, or the steel purlin is on one side only of the restrained member, classify it as providing a flexible restraint for application of 5.4.2.

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A web side plate connection can be considered to provide partial restraint when the length of web side plate cleat is not less than:

(a) 0.5 d for members with uniform depth, d, along the span; or

(b) 0.75 d_S for triangular tapered members, where d_S is the depth of section at the support.

If the beam being supported off the web side plate connection is carrying a rigid (e.g. concrete) floor slab, then full restraint (F) can be assumed for the top flange critical (see (5.40)).

The comment relating to use of fully tensioned bolts into oversize holes in purlins highlighted in C5.4.2.1 also applies to partially restrained connections.

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A background to the second option for partial restraint (P), as introduced through Amendment No. 1, is given in the HERA *Steel Design and Construction Bulletin*, Issue No. 48, 1999, pp. 1, 2.

C5.4.2.3 Lateral restraint to the critical flange (L)

When only lateral deflection of the critical flange of a cross section is effectively prevented, as indicated in figure 5.4.2.3, then the cross section may be considered to be laterally restrained. This cannot be assumed for segments which are unrestrained at one end, for which the effectiveness of lateral restraints is uncertain.

C5.4.2.4 Rotational restraint in plan

Rotational restraints at a cross section reduce lateral rotation of the segment about the minor principal axis of the cross section and increase the resistance to lateral buckling. The first example in figure 5.4.2.4 shows 2 parallel segments which are rotationally restrained by 2 transverse members with moment connections.

The second example in figure 5.4.2.4 shows the central segment of a laterally continuous member being rotationally restrained at both of its end cross sections by the adjacent segments. For this rotational restraint to be effective, these adjacent segments must have full lateral restraint.

C5.4.3 Restraining elements

The following sub-clauses provide the requirements which must be satisfied before a restraint at a cross section can be considered to be effective.

The restraining elements must transmit the restraining force to an anchorage or reaction point within the structural system in such a way as to be effective. Guidance for particular structural forms is given in a number of documents, (e.g. section 5.4.7 of (5.36) and design examples from (5.42)).

C5.4.3.1 Restraint against lateral deflection

At a cross section which is to be considered as fully, partially, or laterally restrained (5.4.2.1, 5.4.2.2 and 5.4.2.3), the restraint against lateral deflection out of the plane of loading is required to be able to transfer 2.5 % of the maximum critical flange force. A stiffness requirement is not given, even though there is a theoretical solution (5.13). This follows the finding (5.14) that the stiffness requirements for centrally braced columns are satisfied by practical braces which satisfy the 2.5 % rule.

The strength and stiffness requirements of a number of actual designs to the 1989 edition of this Standard (which contained both requirements) have been evaluated for these provisions and all show that the stiffness requirement is not necessary. Where the lateral restraint is provided by a very long slender member acting in tension, however, the stiffness requirement might govern if resistance to very high restraint forces is required. Refer to (5.36) for further details.

When the restraints are more closely spaced than is required to ensure that $M^* \leq \phi M_{bx}$, then an appropriate group of restraints is required as a whole to be able to transfer the 2.5 % of the flange force, rather than each individual restraint. This may also be used for continuous restraints (see 5.3.2.2).

When the restraints are only just sufficiently spaced to ensure that $M^* \leq \phi M_{bx}$, then each restraint must be designed to transfer the 2.5 % of the flange force.

C5.4.3.2 Restraint against twist rotation

At a cross section which requires effective restraint against twist rotation about the longitudinal axis of the segment, the restraint must be capable of transferring the moment action of a 2.5 % transverse force from the critical flange to the point of lateral restraint.

In the case of a section which only requires partial restraint against twist rotation, only qualitative indications are given of the stiffness required of the partial restraint, but some quantitative guidance is given in H5.1 of Appendix H.

C5.4.3.3 Parallel restrained members

This clause provides for a reduction in the rate of accumulation of the restraint forces for parallel members beyond the connected member from 2.5 % to 1.25 %. This reduction reflects the possibility that the crookedness or load eccentricity of any other member may act in the opposite sense, and reduce the total restraint force. Analysis of parallel restrained members supports this (5.38).

C5.4.3.4 Restraint against minor principal y-axis rotation

Only a qualitative indication is given of the stiffness required for a restraint against critical flange rotation in plan of the segment, but some quantitative guidance is given in H5.2 of Appendix H. If the length of the segment being restrained is less than around 4 metres, then the connection between the member being restrained and the restraining member should use either fully tensioned bolts or welds, unless the effects of bolt slip associated with snug tight bolts is allowed for. Note that a level of restraining moment need not be determined in applying 5.4.3.4.1.

Unless this quantitative guidance is followed, only a segment which itself may be considered to be fully restrained laterally should be considered as being capable of providing restraint against critical flange rotation in plan and then only if it is continuous with the segment it is to restrain.

The method of design by buckling analysis (5.6.4) allows the actual effective stiffness of the rotational restraint to be taken into account.

C5.5 CRITICAL FLANGE

The general definition in 5.5.1.1 identifies the flange to which restraint must be provided. This general definition is made more specific in 5.5.2 and 5.5.3.

C5.5.3.1

For segments unrestrained at one end and subjected to unrestrained gravity loads, the tension (top) flange will buckle the further, as illustrated in section 5.4.2 and in figure 5.1 of (5.36). In most instances, this flange will actually be restrained either by the system supplying the gravity load or by specific restraints. The restraint of the compression (bottom) flange then needs to be considered when determining the overall restraint of the segment. This means that, in practice, both flanges must be considered critical for this application.

C5.6 NOMINAL MEMBER MOMENT CAPACITY OF SEGMENTS SUBJECT TO X-AXIS BENDING AND WITH OR WITHOUT FULL LATERAL RESTRAINT

This clause applies only to members without full lateral restraint from 5.3 which are bent about the major principal axis, and which may buckle laterally by deflecting and twisting out of the plane of loading. Members which are bent about the minor principal axis do not buckle laterally, unless their loads act far above the shear centre, as covered by C5.1.2.

Each segment of the member must be designed against lateral buckling using the following sub-clauses. A segment is a length of the member which is restrained against lateral deflection and twist rotation out of the plane of loading, either at both ends (between adjacent restraints), or at one end when the other end is free, as it often is in cantilevers. It is required (see 5.3.1.4) that each support in the plane of loading will also provide full or partial restraint at the support out of the plane of loading. Suitable support restraint conditions are shown in (5.40).

A flowchart for the design of segments with or without full lateral restraint is given in figure C5.6.

C5.6.1 Segments restrained at both ends

C5.6.1.1 Open sections with equal flanges

C5.6.1.1.1 *Segments of constant cross section*

This is the most common case and applies to I-section members and channels. The member moment capacity may be reduced below the section moment capacity by the slenderness reduction factor α_s, which depends on the relative magnitudes of the section moment capacity M_s of the gross cross section and the elastic buckling moment M_{oa} (5.15). Values of α_s are given in table C5.6.1.1 for selected values of M_s/M_{oa}.

An equation is given for the elastic buckling moment M_{oa} in terms of the properties of the gross cross section and the effective length L_e. The effective length depends on the twist rotation and y-axis rotation restraint conditions and on the load height above the shear centre. Alternatively, the elastic buckling moment M_{oa} may be determined by an elastic buckling analysis (see 5.6.4). ➤

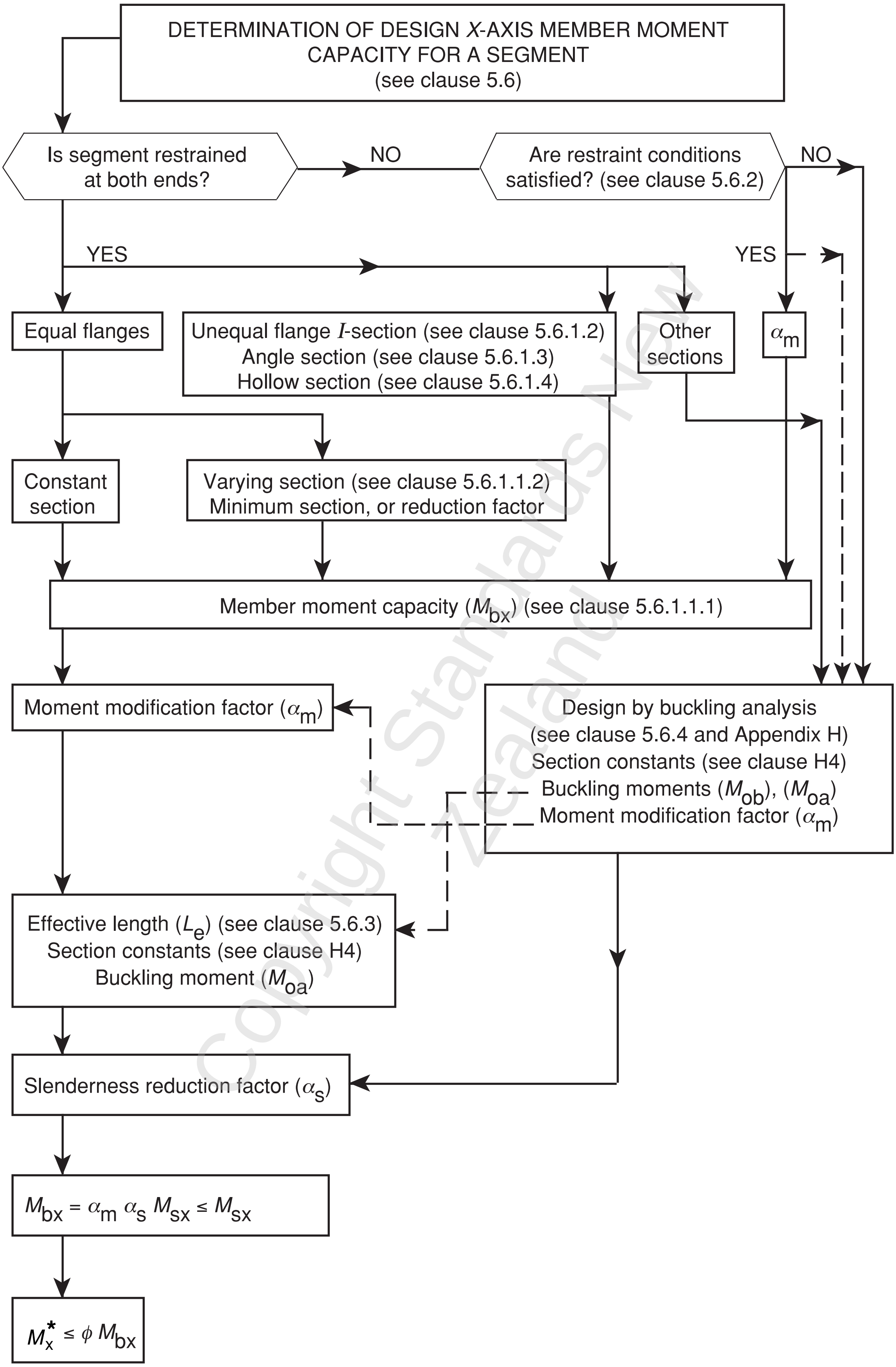

Figure C5.6 – Flowchart for determination of design member moment capacity of a segment subject to x-axis bending and with or without full lateral restraint

Moment modification factor, α_m

The member moment capacity may be increased by the moment modification factor α_m. The use of this factor will often lead to significant economies, especially when there are rapid variations in the bending moment along the segment.

Four different methods are given in 5.6.1.1.1(b) for approximating the moment modification factor. The simplest (and most conservative) is to use $\alpha_m = 1$, and the most complicated (and most accurate) is to use an elastic buckling analysis (see 5.6.4). The use of $\alpha_m = 1$ corresponds to the only option presented in the 1989 edition of this Standard.

The second method provides approximations of high accuracy through the formulae of table 5.6.1. It will be noted that cases 8 to 10 in this table correspond to segments which are cantilevered in the plane of loading, but are fully, partially or laterally restrained out-of-plane at both ends. Segments which are unrestrained out-of-plane at one end are dealt with in 5.6.2 and table 5.6.2. For members with small distributed loads, such as self-weight, these loads may be allowed for approximately by adding their moment to the maximum moment caused by the primary loads, without changing the calculated value of α_m.

If the bending moment distribution under consideration cannot be matched exactly with an option from table 5.6.1, either take the closest option that gives a conservative (lower) value of α_m or use method (iii) to determine α_m.

Note that the value of α_m for a simply supported beam with uniformly distributed loading is 1.13 and with central point loading is 1.35; these values are obtained from case numbers 6 and 4, respectively, of table 5.6.1 by making $\beta_m = 0$.

The third method allows a simple equivalent weighted average moment for the segment to be used. This covers a complete range of moment distributions, but is more time-consuming to use by hand than the second method.

The first three of these methods of determining the moment modification factor are for shear centre loading, and the effects of loading above the shear centre must be accounted for separately, by modifying the effective length, using 5.6.3. Only use of an elastic buckling analysis to 5.3.4 accounts directly for loading above the shear centre. A design example, illustrating the use of Appendix H in this regard, is included in (5.42).

Appropriate determination of α_s for I-sections with equal flanges

A simplified equation for M_{oa} for I-sections with equal flanges can be determined by making the following section geometry and elastic material property approximations:

$$I_y = A r_y^2$$

$$J = 0.25\ A t_f^2$$

$$I_w = I_y d_f^2 / 4$$

$$d_f = 0.9d$$

$$E = 2.5G = 205{,}000\ \text{MPa}$$

These approximations lead to M_{oa} given by:

$$M_{oa} = Ad\left(\frac{900\,000}{(L_e/r_y)^2}\right)\sqrt{\left[1+\frac{1}{20}\left(\frac{L_e t_f}{r_y d}\right)^2\right]} \text{ (Nmm)} \quad \text{(Eq. C5.6.1.1)}$$

where

L_e = effective length from 5.6.3 (mm)
r = radius of gyration about the minor principal y-axis (mm)
t_f = flange thickness (mm)
d = beam depth (mm)
A = area of the cross section

This expression can be used in conjunction with table C5.6.1.1 to rapidly determine α_s for I-sections with equal flanges. The value of M_{oa} determined is reasonably accurate for compact or non-compact I-sections, less so for slender I-sections.

In practice, the need for explicit calculation of M_{oa}, or even α_s, is usually avoided for stock sections through the use of manufacturer's published data.

Table C5.6.1.1 – Value of α_s

M_s/M_{oa}	α_s	M_s/M_{oa}	α_s
0.067	1.000	2.2	0.360
0.10	0.981	2.4	0.336
0.15	0.953	2.6	0.314
0.20	0.926	2.8	0.295
		3.0	0.278
0.25	0.900		
0.30	0.874	3.5	0.243
0.35	0.850	4.0	0.215
0.40	0.826	4.5	0.193
0.45	0.803	5.0	0.175
0.50	0.782	6	0.147
0.6	0.740	7	0.127
0.7	0.701	8	0.111
0.8	0.665	10	0.089
0.9	0.631	12	0.075
1.0	0.600	15	0.060
1.1	0.571	20	0.045
1.2	0.544	30	0.030
1.3	0.519	50	0.018
1.4	0.496	100	0.009
1.5	0.475		
1.6	0.455		
1.8	0.419		
2.0	0.387		

C5.6.1.1.2 *Segments of varying cross section*
Stepped or tapered members may be designed conservatively by using the gross properties of the minimum cross section, or more economically by using the gross properties of the critical cross section (as given by 5.3.3) and a reduction factor α_{st} which provides an approximation to the more accurate elastic buckling solutions (5.16–5.18). The reduction factor depends not only on the section properties at the minimum and critical cross sections, but also on the length L_r within the segment under consideration over which the cross section is reduced.

C5.6.1.2 *I*-sections with unequal flanges
For monosymmetric I-section members, the elastic buckling capacity depends on the monosymmetry of the cross section, and a member whose larger flange is in tension has a greatly reduced member capacity (5.36). An approximation for the monosymmetry section constant β_x is given, while some computer programs calculate both β_x and y_o. Simple approximations for the effects of monosymmetry have been developed (5.19).

C5.6.1.3 Angle sections
Although unequal leg angles bent about the major principal axis are really asymmetric and theoretically should be analysed for elastic buckling using the equations in 5.6.1.2, a sufficiently accurate answer will usually be obtained by using 5.6.1.1.1.

An angle which is bent in one of the planes of its legs is bent in a non-principal plane, and must be designed using 5.7.

C5.6.1.4 Hollow sections
Because rectangular hollow section members have very high torsional rigidities, J, they rarely buckle laterally, except for extreme sections with I_y much less than I_x, or when the load acts far above the shear centre. An approximate slenderness limit for lengths between restraints which may be considered to have full lateral restraint is given in 5.3.2.4. The elastic buckling of hollow section segments without full lateral restraint may be evaluated using 5.6.1.1.1 with I_w taken as zero. Similar comments also apply to circular hollow sections.

C5.6.2 Segments unrestrained at one end
Segments unrestrained at one end are usually cantilevered in the plane of loading, and have their free ends unrestrained against lateral deflection and twist rotation out of the plane of loading. It is also possible that one end of a segment may be supported in-plane on a skid so that it is unrestrained out of plane.

The simple rules given apply only to segments which are restrained at the support end both against rigid body twist rotations about the segment longitudinal axis (i.e. full or partial restraint) and against rigid body rotational restraint in plan out of the plane of loading.

Segments which are not so restrained, such as some double cantilevers, or segments coped at the support, do not easily satisfy the format of the elastic buckling capacity given in 5.6.1.1.1 (5.20). These segments may be designed by the method of buckling analysis (see 5.6.4), in which case the information given in H3.2 of Appendix H will be used.

C5.6.3 Effective length
This clause provides an approximate equation for the effective length L_e of the segment which incorporates factors for the end restraints against twist rotations, for the height of the load above the shear centre, and for any end restraints against rotations in plan out of the plane of loading. These latter restraints are often ineffective because any element providing the restraint usually has low geometrical stiffness which is reduced by any destabilizing loads carried by it. Because of this, 5.4.3.4 is referenced to ensure that such a restraint is effective before it can be allowed for.

The effective length factors use a basic value of 1.0. Partial section restraint (see 5.4.2.2) is accounted for by increasing the factor by appropriate multiples of (d_w/L) $(t_f/2t_w)^3$ (5.21). This factor will usually be close to 1.0, except for short deep beams with thick flanges and thin webs. Refer to commentary clause C5 (2.7) on page 59 for approximate values to use in routine design.

The approximate effective length factor of 1.4 for within-segment gravity loads at the top flange allows for the destabilizing effects of this type of loading by comparison with shear centre loading, as does the value of 2.0 used for top flange loads at the free ends (U) of cantilevers which are restrained at their supports against out-of-plane twisting and rotation. For less restrained cantilevers, the more accurate proposals of H5 of Appendix H are recommended. The beneficial effects of gravity loads acting below the shear centre may be approximated by using H2 or H3 of Appendix H.

Note that if the load is applied to the member through a structural system which is itself laterally restrained, then when the shear centre and centroid both lie on the same axis as the applied load, the load may be taken as acting at the shear centre for application of table 5.6.3(2). An example is the load transmitted to an *I*-section rafter from the purlin in a light-weight steel clad roofing system provided with adequate in-plane lateral bracing (e.g. to (5.37)).

For uplift loads such as those caused by wind loading, the critical loading position is at the bottom flange instead of the top. In wind-induced uplift loading, the loads are usually applied through the top flange from the roofing system, which, being on the opposite side of the shear centre to the critical flange (under negative moment) is itself beneficial. This is illustrated by a design example in (5.42).

In routine design, k_r = 1.0 can always be used without incurring significant conservatism.

The rotation restraint factors of 0.85 and 0.70 are for values of intermediate stiffness, and are based on the closed form solution reported in (5.12). The theoretical limiting values for rigid restraints are 0.7 approximately and 0.5, as for compression members.

C5.6.4 Design by buckling analysis

The method of design by buckling analysis provides an alternative to the method given in 5.6.1, 5.6.2, and 5.6.3. Although it is not as simple to use, it is of much wider application. It is likely to be used when it is desirable to have a more accurate prediction of the buckling capacity, as when designing critically important members, in repeated applications of a single design, when assessing an existing member, or when establishing reasons for failure.

The method generally requires the use of the results of an elastic analysis. Table 5.6.1, Appendix H and table H3 provide useful summaries of approximate equations for this purpose, as an alternative to the use of specialised computer programs (5.22, 5.23).

The results of the elastic buckling analysis may be used to determine the elastic buckling moment M_{ob} at the most critical section in the complete member, and from there to find the value of M_{oa} to be used in determining the slenderness reduction factor α_s of 5.6.1.1.1(a). For this purpose, the moment modification factor α_m may be taken as 1.0 for members in near uniform bending, which is conservative. More generally, advantage may be taken of the increased buckling capacity of a member with moment gradient by calculating an approximate value of the moment modification factor α_m from the results of elastic buckling analyses.

C5.7 BENDING IN A NON-PRINCIPAL PLANE

This clause applies to the common use of angle section members, channels and zeds as purlins and girts. Two different situations commonly arise, depending on whether the sheeting or other elements connected to the member are effective or not in preventing deflection in the plane of the sheeting. The clause also covers the general case of biaxial bending of members in which the resultant moment does not act in a principal plane.

An example relating to an unequal angle truss bottom chord is given in (6.31).

C5.7.1 Deflections constrained to a non-principal plane

When deflections are constrained to occur in a particular plane, then the resulting moment acting is the vector sum of the free moment exerted by the applied loads and the constraining moment exerted by the restraining elements. The constraining moment may be evaluated by a rational method (e.g. elastic analysis) by constraining the deflection to occur in the specified plane (5.24).

Because of the deflection constraints, there is no possibility of any lateral buckling, and the moment capacity is controlled by the section moment capacity. Thus the calculated principal axis moments are required to satisfy the biaxial bending section capacity requirement of 8.3.4.

C5.7.2 Deflections unconstrained

When the deflections are unconstrained, then the principal axis moments can be determined by elastic analyses of the bending in each principal plane. For short span members, the moment capacity will be controlled by the section moment capacity, and so the biaxial bending section capacity requirement of 8.3.4 is invoked. For long span members, lateral buckling effects will be important, and so the biaxial bending member capacity requirement of 8.4.5 is invoked.

One common example where an unconstrained member is bent in a non-principal plane is that of a crane runway girder, which is to be designed for both vertical forces and lateral forces acting above the top flange. In this case the lateral forces induce both minor axis bending and torsion in the girder. This Standard gives no guidance for designing against torsion, although some is given in section C8.5 of this Commentary and this is elaborated on in section 8 of (5.36).

In the case of a crane runway girder, it is suggested that each lateral force be replaced by statically equivalent lateral forces acting at the shear centres of the top and bottom flanges, and that the minor axis bending of these (in their own planes) be analysed independently. It is further suggested that the crane runway girder be designed by adapting 8.3.4 and 8.4.5 twice, one for each flange. In these adaptions, the girder major axis terms should remain unchanged, but the minor axis girder terms should be replaced by corresponding terms for the appropriate flange (assuming it to be braced by the girder web against buckling in the vertical plane).

C5.8 SEPARATORS AND DIAPHRAGMS

This clause gives the requirements for separators and diaphragms when either of these 2 methods is used to connect individual components together so as to form a single member.

C5.9 DESIGN OF WEBS

C5.9.1 General

This clause directs the designer to the relevant one of 5.10 to 5.16. In the design of a web to resist shear, several of these clauses may have to be considered in conjunction with one another.

It also directs the designer to the appropriate clauses in section 12 for the design of members in seismic-resisting or associated structural systems.

C5.9.2 Definition of web panel

Definitions are included of the longitudinal panel dimension s, and the clear transverse panel dimension d_p. Clarification of what constitutes an edge of a web panel is given in this clause.

C5.9.3 Maximum slenderness ratio of web panel

This clause directs the designer to clauses which give maximum slenderness ratios for webs which are unstiffened, webs stiffened transversely, webs stiffened both transversely and longitudinally, and webs of members designed plastically. A web thinner than that given by the clause is permitted, except as specified by 12.5, provided that it can be justified by a rational analysis which incorporates both stiffness (deflection) and strength (yielding and buckling) considerations. Guidance for this analysis is given in (5.25).

C5.10 ARRANGEMENT OF WEBS

A diagrammatic explanation of the provisions of this clause is given in figure C5.10.

C5.10.1 Unstiffened webs

This clause is based on the bending resistance of a web (5.3, 5.36). The effects of flanges on the local behaviour of the web in bending have been studied in detail in (5.8). The limits given in this clause allow for the restraining actions of the flanges on the web (5.36) and are consistent for webs of I sections with the limit given by Note (5) to table 5.2.

The limit for a web with a free edge is based on the assumption that the flange is in compression and the free edge of the web is in tension. The results of research in (5.26) show that in the unlikely event that the free edge is in compression, then a slenderness ratio of 90 will not allow the full bending strength of the web to be developed, and the provisions of this clause will be unconservative. However, the bending capacities of such webs with $(d_1/t_w)\sqrt{(f_y / 250)} > 22$ will be reduced by the section capacity requirements of 5.2.5.

C5.10.2 Load bearing stiffeners

Load bearing stiffeners are provided to transfer concentrated bearing loads or reactions which would otherwise lead to web yielding or buckling, or when an end post needs to be provided to anchor the tension field action in a transversely stiffened web. Tension field action is discussed in C5.11.5.2, and end posts in C5.15.9.

C5.10.3 Side reinforcing (doubler) plates

If side reinforcing (doubler) plates are used, then proper account must be taken of any lack of symmetry caused by placing the side reinforcing plate on one side only of the web. A shear flow analysis such as that described in (5.3, 5.36) may need to be undertaken to calculate the horizontal shear which is to be transmitted through the fasteners to the web and to the flanges.

Specific design requirements for such plates used in moment-resisting connections subject to earthquake loads are given in 12.9.5. The background to these requirements is given in Commentary Clause C12.9.5. They can also be used for non-seismic applications, as directed therein.

C5.10.4 Transversely stiffened webs

This clause allows smaller thicknesses than does 5.10.1 for unstiffened webs, and takes into account the stiffening effects of transverse stiffeners. The diagrammatic representation in figure C5.10 shows that $(d_1/t_w)\sqrt{(f_y / 250)}$ is limited to 200 or less when s/d_1 is between 1 and 3, but this limit becomes 270 for $s/d_1 < 0.74$. The limit of 270 is derived from experiments which show unfavourable results for webs whose slendernesses are greater than this value (5.27).

If transverse stiffeners are placed at spacings greater than $3d_1$, then the tension field action on which the increased shear resistance of the web is based is ineffective. Because of this, such a web is considered as being unstiffened transversely, and the design capacity is based on the elastic buckling capacity, as given by 5.11.5.1.

C5.10.5 Webs with longitudinal and transverse stiffeners

Longitudinal stiffeners increase the bending capacity of a web (5.8). Longitudinally stiffened webs must also be stiffened transversely.

Only one longitudinal stiffener is required when $(d_1/t_w)\sqrt{(f_y/250)} \leq 250$ in webs for which s/d_1 is between 1 and 2.4. It is placed at a distance of $0.2d_2$ from the compression flange, where the bending buckling deformations are high (5.7). The slenderness limits for a transversely stiffened web with a single longitudinal stiffener are illustrated in figure C5.10.

For a web with slenderness greater than the single longitudinal stiffener provisions, an additional longitudinal stiffener is required at the neutral axis, in which case the web slenderness limit increases to $(d_1/t_w)\sqrt{(f_y/250)} \leq 400$, as shown in figure C5.10.

C5.10.6 Webs of members designed plastically

In plastically designed members, sufficient rotation capacity must be available at each plastic hinge location to allow the collapse mechanism to form before local buckling. Webs with slenderness ratios greater than 82 will buckle locally before the nominal shear yield capacity V_w can be reached, and are therefore not allowed in members assumed to contain a plastic hinge or in yielding regions of category 1 or 2 members.

Load bearing stiffeners will generally need to be provided near any plastic hinge location where a significant concentrated load or shear force is present, even if such stiffeners are not required by 5.14. Note that the stiffener must be compact ($\lambda_s \leq \lambda_{sp}$), in order that the web-stiffener combination will possess the rotation capacity required for a plastic mechanism to form before local buckling.

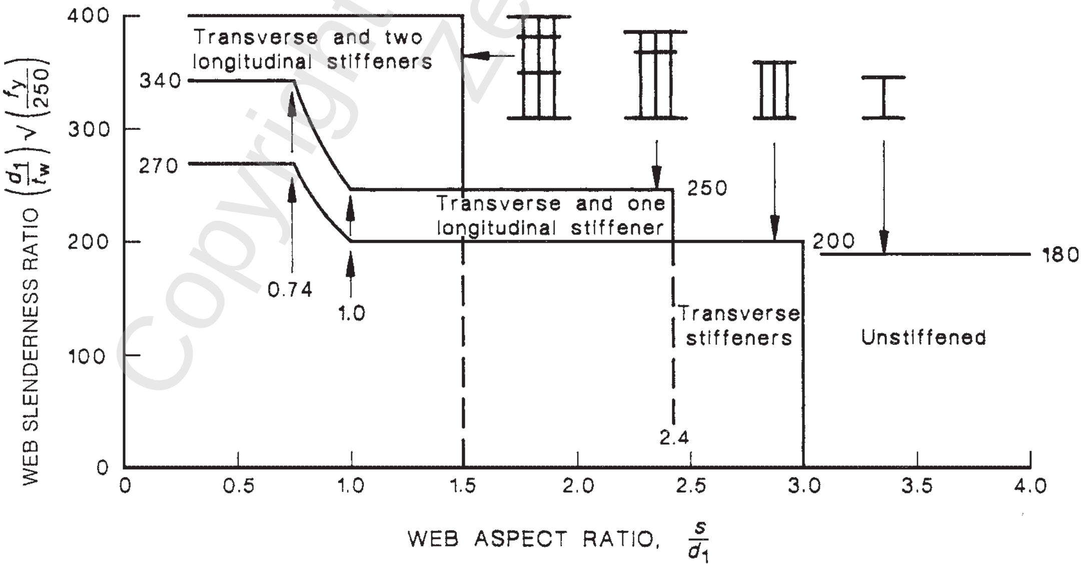

Figure C5.10 – Web arrangements

C5.10.7 Openings in webs

In many cases, webs with openings will satisfy the limits of 5.10.7.1 (a) or (b).

The design of webs with larger openings must be based on an appropriate limit state design procedure. While there have been many research findings on openings in webs in the last few ➤

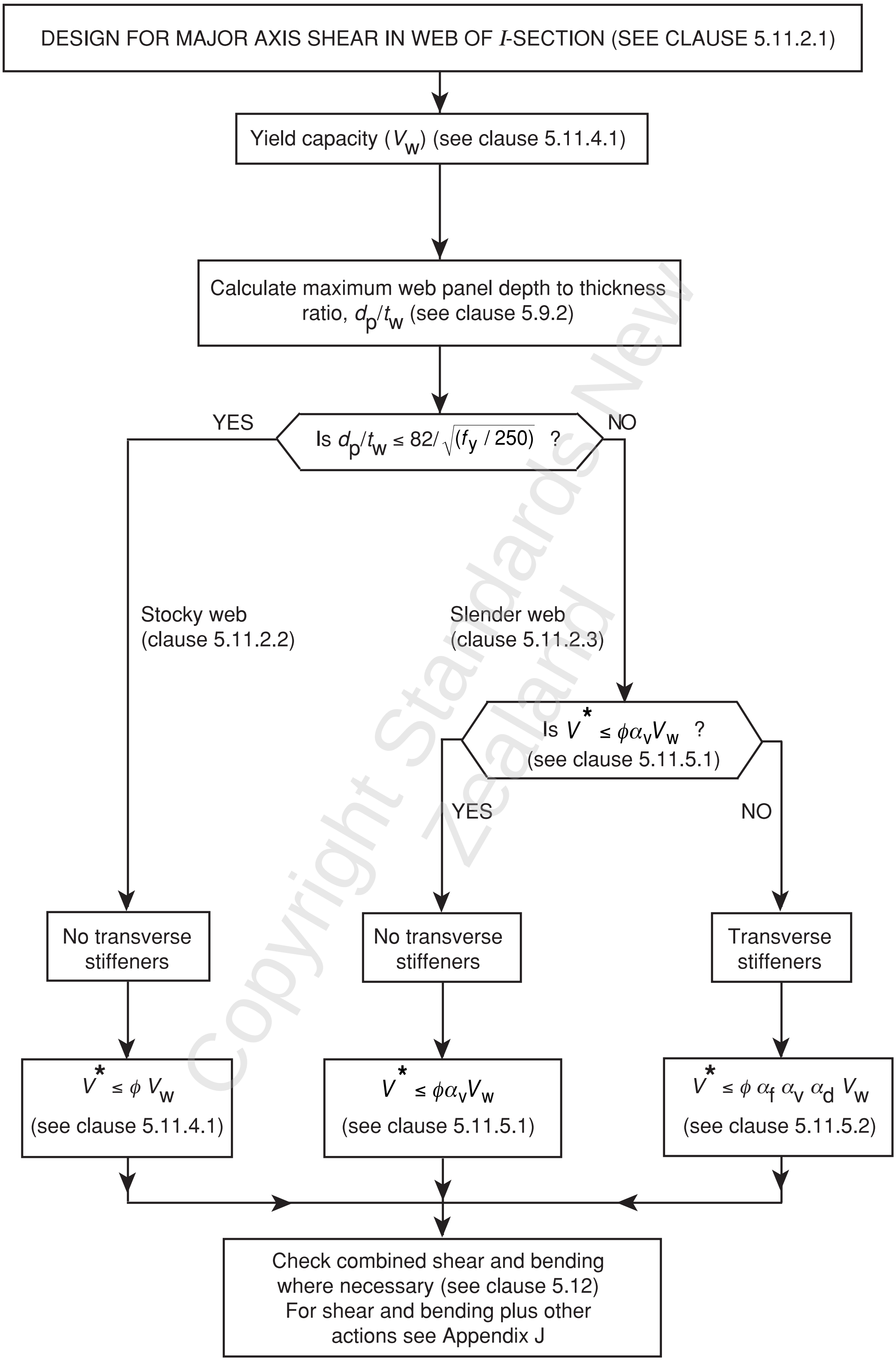

Figure C5.11.1 (1) – Flowchart for design for major principal x-axis shear in web of I-section

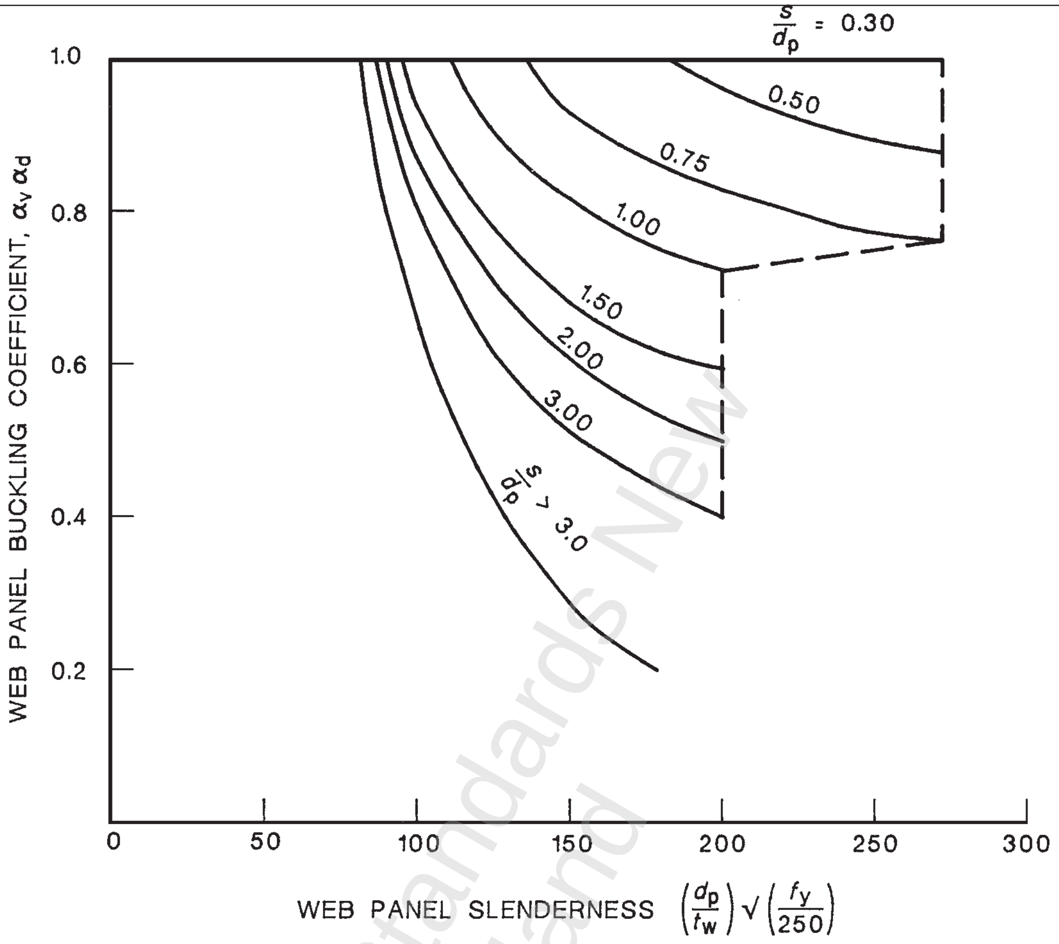

Figure C5.11.1 (2) – Web panel nominal shear capacity

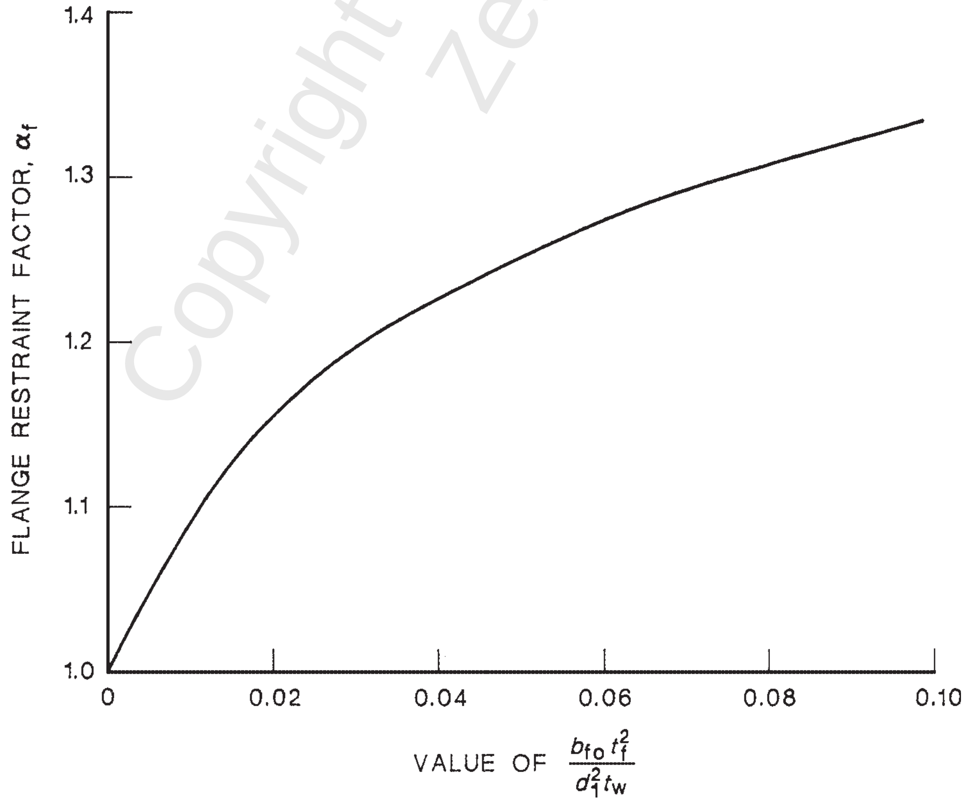

Figure C5.11.5.2 – Flange restraint factor α_f for shear buckling

decades, most of them relate to working stress design. However, the recommendations of (5.28) are in an ultimate strength format, and may be used for limit state design, provided a strength reduction factor ϕ of 0.9 is used. A list of references on research work carried out prior to 1973 is given in (5.29). Design guidance in limit states format for openings in webs of bare steel beams is given in section 5.5 of (5.36) and in webs of composite beams in (5.39). Design guidance for castellated beams is referenced from (5.36).

Amd 1 June '01 The general design philosophy and procedure given in (5.39) has now been adapted for New Zealand application. Details are given on pages 1-6 and Appendices A-C of the HERA *Steel Design and Construction Bulletin*, Issue No. 53, 1999.

C5.11 NOMINAL SHEAR CAPACITY OF WEBS

C5.11.1 Shear capacity

This clause provides the general inequality for shear design. The strength reduction factor of ϕ = 0.9 was obtained from the calibration studies of (5.30).

The nominal shear capacities of web panels are shown diagrammatically in figure C5.11.1(2). A flow chart for the design of web panels in an I-section for major principal x-axis shear is given in figure C5.11.1(1). Design examples covering application of 5.11 – 5.15 are given in (5.42).

C5.11.2 Shear capacity of a web with uniform shear stress distribution

In most I- section members loaded about their major principal axis, the ratio f_{vm}/f_{va} of the maximum and average shear stresses is less than 1.1, and the shear stress distribution in the web may be considered to be uniform. The uniform shear capacity V_{vu} is given by the shear yield capacity V_w of 5.11.4.1 for flat plate webs for which $(d_p/t_w)\sqrt{(f_y/250)} \leq 82$, which yield before local buckling. On the other hand, if the d_p/t_w ratio exceeds this limit, the web will buckle elastically before yielding. If such a web is unstiffened, then 5.11.5.1 applies, while 5.11.5.2 is used for stiffened webs.

C5.11.3 Shear capacity of a flat plate with non-uniform shear stress distribution

This clause covers those sections for which 5.11.2 does not apply. It should be used when the ratio f_{vm}/f_{va} of the maximum and average elastic shear stresses is greater than 1.1. The equation given lies approximately halfway between the first yield limit of an elastic shear stress distribution, and the fully plastic condition.

For a solid rectangular section, $f^*_{vm} = 1.5f^*_{va}$, giving $V_{vn} = 0.83\ V_{vu}$.

Calculations of the elastic shear stresses can be made by the methods of analysis in (5.3 or 5.36), and solutions for some particular cases are given in (5.31). A computer method of determining elastic shear stresses is given in (5.32).

C5.11.4 Shear yield capacity

The shear yield capacity will govern the design of flat plate web panels with $(d_p/t_w)\sqrt{(f_y/250)}$ $\leq$ 82. The yield capacity is also used in determining the capacity of more slender web panels.

The shear yield capacity is based on the yield stress of the web in shear ($f_y/\sqrt{3} \approx 0.6\ f_y$) and an approximate shear shape factor of 1.04.

For circular hollow sections, the appropriate shear yield stress of 0.6 f_y is multiplied by 0.6 times the effective area A_e so that the shear yield capacity corresponds approximately to the fully plastic capacity of the cross section.

C5.11.5 Shear buckling capacity

Webs with $(d_p/t_w)\sqrt{(f_y / 250)} > 82$ may reach a buckling rather than yield ultimate limit state, as shown in figure C5.11.1(2). Figure C5.11.1(1) presents a flowchart for the calculation of the major principal x-axis shear capacity of a web panel in an I-section.

C5.11.5.1 Unstiffened web

The shear buckling capacity of an unstiffened web assumes no post-local buckling capacity in shear, and is based on elastic buckling analyses such as those cited in (5.3, 5.36).

C5.11.5.2 Stiffened web

The favourable effect of tension field action may be utilized when the web is provided with intermediate transverse stiffeners. In tension field design, the factor α_v allows for the elastic buckling resistance, while the factor α_d allows for the tension field component. For slender webs, the provision of transverse stiffeners significantly enhances the shear capacity of the web, as illustrated in figure C5.11.1(2) and table 5.11.5.2, which give the ratio $V_b/V_w = \alpha_v\alpha_d$ of the nominal capacity V_b of the web to the yield capacity V_w when $\alpha_f = 1$.

The capacity of the web may be further enhanced by the rigidity of the flanges. This is allowed for by the introduction of the factor α_f, which depends on the ratio of plastic moment capacity of the flange $(f_y b_{fo} t_f^2/4)$ to that of the web $(f_y d_1^2 t_w/4)$. A curve showing values of α_f for values of $(b_{fo} t_f^2/d_1^2 t_w)$ is given in figure C5.11.5.2, and it can be seen that there may be a significant enhancement of the capacity of a transversely stiffened web by including the stiffening effect of the flange.

The flanges also have a beneficial effect on the post-elastic strength of a web panel zone, as detailed in 12.9.5.3.2 and discussed in Commentary Clause C12.9.5.3.2. Note the similarity of terms in the square brackets of 5.11.5.2 (b) and equation 12.9.5.3 (5).

The presence in a web of compression stresses caused by axial force decreases the shear capacity. Guidance on this is given in Appendix J.

The design of intermediate transverse stiffeners is governed by 5.15.

C5.12 INTERACTION OF SHEAR AND BENDING MOMENT

C5.12.1 General

When substantial bending moment is present, the shear capacity may be reduced, as for example at the interior supports of continuous beams.

A general method of accounting for this interaction is given in 5.12.2. Two specific applications that require different approaches are identified under (a) and (b) of 5.12.1.

C5.12.2 Shear and bending moment interaction

This clause provides a semi-empirical equation which reduces the shear capacity V_{vm} when the design bending moment M^* is greater than $0.75\phi M_s$. A graph of the provisions of this clause is given in figure C5.12.2. A more detailed background to these provisions is given in section 9.5.2 of (5.36).

The shear and bending moment interaction of clause 5.12.2 gives conservative results when applied to members with slender webs, as defined in 5.11.2.3. For such members, a less conservative approach is to limit the design bending moment to that which can be resisted by the flanges alone, leaving the web free to carry all of the design shear force. This is known as the proportioning method. Its application is as follows:

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Proportioning method (alternative method for considering the interaction of shear and bending moment for members with slender webs)

When the design bending moment (M^*) satisfies:

$M^* \leq \phi M_f$

where

M_f is the nominal moment capacity of the flanges alone

the member shall satisfy:

$V^* \leq \phi V_b$

where

V_b is the nominal shear buckling capacity for the slender web, from 5.11.2.3.

Nominal moment capacity of the flanges alone

The nominal moment capacity of the flanges alone, M_f, is calculated as follows:

$M_f \quad = \quad A_{fm} d_f f_y$

where

A_{fm} = the lesser of the flange effective areas, determined using 6.2.2 for the compression flange and the lesser of A_{fg} and 0.85 $A_{fn} f_u / f_y$ for the tension flange

A_{fg} = the gross area of the flange

A_{fn} = the net area of the flange

d_f = the distance between the flange centroids.

The nominal moment capacity must be based on that of the lesser flange. In order to allow for the effects of flange local buckling, the compression flange area is reduced to the effective area determined using 6.2.2, while the tension flange area may be reduced by holes.

Web panels subject to shear, bending and other actions

Web panels subject to shear and bending together with axial force and/or transverse loading may be designed in accordance with the provisions of Appendix J. Note, however, that the provisions of Appendix J are more conservative than those of 5.12.2 or C5.12.2 for applications subject only to shear and bending.

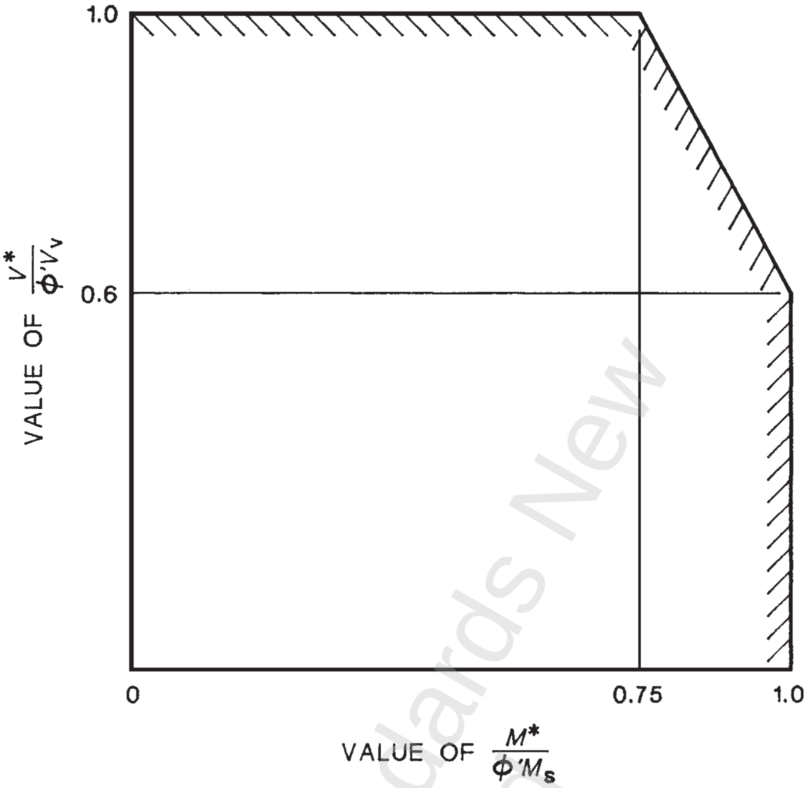

Figure C5.12.2 – Shear and bending moment interaction

C5.13 COMPRESSIVE BEARING ACTION ON THE EDGE OF A WEB

C5.13.1 Dispersion of force to web

The clause defines the length b_s of the stiff portion of flange under a bearing action R^* (figure 5.13.1.2), and the rate of dispersion of the bearing force through the flange to the flange-web junction (figure 5.13.1.1). New material has been added to the 1997 edition, making the provisions applicable to rectangular and square hollow sections with wall thicknesses below 3 mm. Design examples and guidance on the use of this new material is given in (5.44).

C5.13.2 Bearing capacity

This clause presents the general design for bearing capacity. The 2 bearing limit states of yielding and buckling need to be checked separately. Yielding is considered in 5.13.3 and buckling is considered in 5.13.4.

C5.13.3 Bearing yield capacity

The yield bearing width b_{bf} at the flange-web junction is obtained by the dispersion of 5.13.3.1 and is shown in figure 5.13.1.1 and figure 5.13.1.3. The factor of 1.25 has been introduced in equation 5.13.3(1) because of the favourable redistribution of bearing stress in the web which takes place after yielding, and because of the benign nature of bearing failure which leads only to local thickening in areas of high bearing stress.

New material has been added to the 1997 edition relating to rectangular and square hollow sections. This is covered in 5.13.3.1, 5.13.3.2 and figure 5.13.1.3.

C5.13.4 Bearing buckling capacity

For this clause, the web is considered as a compression member of height d_1, breadth b_b and thickness t_w. The value of b_b is obtained by a further dispersion at a slope of 1:1 of the bearing force through the web to the neutral axis.

In calculating the slenderness ratio, it is assumed that the effective length factor k_e is approximately equal to 0.7, which allows for the restraining effects of the flanges. This assumption leads to the slenderness ratios given in the clause.

If the transversely stiffened web panel has significant bending and axial actions, then recourse should be made to the provisions of Appendix J.

C5.13.5 Combined bending and bearing of rectangular square and hollow sections

The background to, and application of these provisions is covered in (5.44).

C5.14 DESIGN OF LOAD BEARING STIFFENERS

If the web alone does not meet the provisions of 5.13.2, then load bearing stiffeners must be provided. The stiffened web is then designed for the yield (5.14.1) and the buckling (5.14.2) ultimate limit states.

C5.14.1 Yield capacity

The total yield capacity is obtained by adding the stiffener yield capacity $A_s f_{ys}$ to the yield capacity of the web alone as given by 5.13.3.

C5.14.2 Buckling capacity

This clause requires the web-stiffener combination to be designed as a compression member. The effective section of this combination consists of the stiffener, plus a length of web on either side of the stiffener centreline equal to the lesser of $17.5t_w / \sqrt{(f_y / 250)}$ and $s/2$, if available. If the stiffener is close to the end of the beam, then a reduced length of web should be used. The radius of gyration of the web-stiffener combination is obtained from its area and second moment of area about the web mid-plane.

The effective length for calculating the slenderness ratio is taken as $0.7d_1$ when both flanges are restrained against rotation in the stiffener plane. If either is not so restrained, the effective length is taken conservatively as $1.0d_1$. In calculating the axial capacity, the column curve (clause 6.3.3) corresponding to $\alpha_b = 0.5$ should be used.

C5.14.3 Outstand of stiffeners

This clause provides a limit to the stiffener outstand in order to ensure that the stiffener does not buckle locally before it can transmit its full squash load. This limit is the same as the local buckling yield slenderness limit λ_{ey} given in table 6.2.4.

C5.14.4 Fitting of load bearing stiffeners

Because the web is designed to carry some of the bearing load, the stiffener connections need only to be able to transmit the stiffener's share of the design bearing force R^*. This share may be approximated by multiplying R^* by the ratio of the stiffener area to the area of the effective section of the web and stiffener.

C5.14.5 Design for torsional end restraint

When required to provide effective torsional restraint (see 5.4.3.2) at the support of a beam (so that the effective length factor k_t given in table 5.6.3 (1) can be taken as unity), then the load bearing stiffeners must possess a minimum stiffness. This clause requires that the load bearing stiffeners satisfy a stiffness limit as well as being designed for the yielding and buckling limit states of the web-stiffener combination. Stiffness requirements necessary for both full (F) and partial (P) restraint are given; the latter are obtained from (5.45).

C5.15 DESIGN OF INTERMEDIATE TRANSVERSE WEB STIFFENERS

Intermediate transverse web stiffeners may be provided to prevent local buckling of the web in shear, and must be provided when the factors α_v, α_d are used to calculate the shear capacity. In order to be effective, they require stiffness as well as strength. Stiffness is governed by 5.15.5, and strength by 5.15.3 and 5.15.4.

C5.15.1 General

Intermediate web stiffeners are often not connected to the tension flange, and need not be connected to the compression flange. It is permissible to provide stiffeners on one side of the web only, as long as the stiffness and strength criteria are met.

C5.15.2 Spacing

C5.15.2.1 Interior panels

This clause refers to the maximum requirements of 5.10.4 and 5.10.5. Clause 5.10.4 requires that effective web stiffeners must be spaced closer then $3d_1$. When spaced farther apart, then the tension field on which the increased shear resistance of the web is based is ineffective.

C5.15.2.2 End panels

An end post is required to anchor any unbalanced adjacent tension field in an end panel. The design of an end post (see 5.15.9) in an end panel may be avoided by reducing the length s of the end panel so that the tension field action is not required. In this case, $\alpha_d = 1.0$.

C5.15.3 Minimum area

This clause ensures that the stiffener has a yield capacity sufficient to transmit the actions induced by the tension field. This minimum area rule must be used in conjunction with the buckling requirement of 5.15.4 and the minimum stiffness requirement of 5.15.5 for adequate stiffener design.

The ratio $[V^*/\phi V_b]$ is included to avoid undue conservatism when the design shear V^* is only slightly greater than the unstiffened design web shear capacity.

C5.15.4 Buckling capacity

Because part of the applied shear is resisted by the elastic buckling capacity ϕV_b of the web, the buckling design of the stiffener need only be carried out for the net tension field component, which is equal to $(V^* - \phi V_b)$. The design stiffener capacity ϕR_{sb} is determined using the load bearing stiffener provisions of 5.14.2, but with an effective length of d_1.

C5.15.5 Minimum stiffness

The intermediate stiffeners must have sufficient stiffness to ensure that a web buckling node is maintained at the stiffener-web junction. This is deemed to be achieved by satisfying the appropriate limit on the second moment of area of the stiffener about the web centreline.

C5.15.6 Outstand of stiffeners

The outstand width should be limited so that the stiffener will not buckle locally when it is required to carry the tension field contribution, which may be close to its squash load. The limit of 5.14.3 for load bearing stiffeners is used, which is the same as the yield limit of table 6.2.4.

C5.15.7 External forces

Intermediate stiffeners are often used as connection plates for incoming beams, and this clause provides safeguards against the reduction of the intended stiffening effect by the external forces transmitted. Undesirable performance is guarded against by satisfying both the stiffness and the strength requirements of the following clauses.

C5.15.7.1 Increase in stiffness

The stiffness provision requires an increase in the second moment of area when the forces produce shears and moments in the plane of the stiffener.

C5.15.8 Connection of intermediate stiffeners to web

The shear flow calculated from this clause is used in the design of the connectors according to section 9.

C5.15.9 End posts

The requirement of an end post, consisting of an end plate and adjacent load bearing stiffener, is included in order to anchor any unbalanced tension field action in the panel. The dimensions of the end plate are obtained from the area provision of the clause, and the load bearing stiffener must be designed in accordance with the requirements in 5.14 for load bearing stiffeners. Further guidance is given in section 5.3 of (5.36).

C5.16 DESIGN OF LONGITUDINAL WEB STIFFENERS

The main function of longitudinal stiffeners is to allow increased web depths or decreased web thickness. The savings resulting from this, however, may be offset by increased fabrication costs.

A full treatment of longitudinal web stiffeners is not given, since these are used mainly for plate and box girder applications covered in specialised codes such as (5.33). Designers are referred to (5.25, 5.26 and 5.33) for a discussion of the effects of longitudinal stiffeners on web bending capacity. General guidance is also given in section 9 of (5.36).

C5.16.1 General

The clause requires that webs stiffened longitudinally have transverse stiffeners. If these are not necessary to increase the shear capacity, then they may be spaced at $3d_p$.

C5.16.2 Minimum stiffness

The requirements for the positioning of the longitudinal stiffeners and their stiffnesses are based on local buckling studies, as discussed in (5.3, 5.7, 5.34 and 5.36).

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NOTES

C6 MEMBERS SUBJECT TO AXIAL COMPRESSION

INTRODUCTION

The strength design of members subjected to compression actions only is governed by the clauses of this section. Clause 6.1 provides the design section and member inequalities which must be satisfied. Clause 6.2 is used to obtain the section capacity, and this makes allowances for the effects of local buckling. Clause 6.3 is used to obtain the member capacity, which reflects the effects of flexural or flexural-torsional buckling. Finally, 6.4 allows for laced and battened compression members, which are special types of members subjected to axial compression.

Further information on the basis of the clauses is given in (6.1 – 6.5, 6.28). The behaviour on which these clauses is based is described in design guides (6.28) or text books (6.6). This commentary should be read in conjunction with such a design guide or textbook.

Worked examples for the strength of members subject to axial compression are presented in (6.32), while a computer program is described in (6.8).

C6.1 DESIGN FOR AXIAL COMPRESSION

This clause provides the relationships between the design compression force (N^*) and the nominal capacities of the member to resist cross section yield or local buckling (section capacity N_s) and overall buckling (member capacity N_c). This is analogous to the provisions for members subjected to bending.

Note that, in most instances of members not subject to inelastic action, the nominal member capacity (N_c) will govern. Where a considerable local reduction in cross section area is made, however, N_s at that point may govern the design.

For members subject to inelastic action, the nominal compression capacity is reduced by a number of factors from 6.2.1.2 and may be less than N_c determined from 6.3.3.

C6.2 NOMINAL SECTION CAPACITY

C6.2.1 General

C6.2.1.1 Member not subject to inelastic action

The nominal section capacity (N_s) is the strength of a very short column which cannot undergo overall member buckling. If such a column does not buckle locally before squashing, its nominal capacity is ($A_n f_y$), where (A_n) is the net area of the section. Provided that unfilled holes or penetrations in the cross section are sufficiently small, the gross area may be used instead of the net area.

There is a significant range of columns composed of slender plate elements, and these will buckle locally before the squash load is reached. Because of this, the squash load capacity ($A_n f_y$) is modified by the local buckling form factor (k_f) calculated in accordance with 6.2.2.

C6.2.1.2 Member subject to inelastic action

If the compression member is also subject to inelastic action, through earthquake loads or effects, redistribution of elastic analysis derived design actions or design by the plastic method, then the nominal section compression capacity is limited by other factors, such as the web slenderness ratio. The design requirements for this are not contained in this section and may easily be overlooked. For this reason, they are specifically referenced by 6.2.1.2.

C6.2.2 Form factor

The concept of using a form factor (k_f) is discussed in (6.10). In order to calculate the form factor, it is necessary to obtain the effective area (A_e). This is done by summing the effective areas of each of the elements which comprise the cross section. Each of these elements has an effective width (b_e), which is calculated in accordance with 6.2.4, and a thickness equal to the actual thickness of the element.

Note that a member with a solid cross section has k_f = 1.

C6.2.3 Plate element slenderness

The calculation of the plate element slenderness (λ_e) is analogous to that for elements of members in bending (section 5), and incorporates the effect of the yield stress through the terms $\sqrt{(f_y / 250)}$ for flats and (f_y/250) for circular hollow sections.

C6.2.4 Effective width

The concept of using an effective width to calculate the section capacity, incorporating local buckling, is well-known, and is described in (6.6, 6.28). The effective width (b_e) of a plate element of width (b) is obtained by multiplying (b) by a reduction factor (λ_{ey}/λ_e). Values of the yield slenderness limit (λ_{ey}) are given in table 6.2.4, and are the same as the values given in table 5.2, except for circular hollow sections. The values of (λ_{ey}) for flat elements depend on the number of supported edges and on the residual stress category. A discussion of their derivation is given in publications such as (6.11 – 6.13). For a circular hollow section, the effective diameter d_e used to obtain the effective area is obtained by reducing the outside diameter (d_o) by the factor $\sqrt{(\lambda_{ey} / \lambda_e)}$, or by the factor $(3\lambda_{ey}/\lambda_e)^2$.

This clause also allows an enhanced effective width (b_e) to be used, which is dependent on the local buckling coefficient (k_b) for the plate element. Values of k_b may be obtained from computer programs such as that described in (6.14), or from tabulations or diagrams in texts such as (6.3, 6.6, 6.15, 6.30).

A plate element supported along both edges with simple supports has a theoretical elastic local buckling coefficient (k_{bo}) of 4.0 (6.15, 6.28). When a plate element is connected along its supported edges to other plate elements, its local buckling coefficient (k_b) may be greater than or less than 4.0, depending on the degree of restraint offered by the adjacent elements. When its local buckling coefficient (k_b) is greater than 4.0, then the effective width may be increased in accordance with this clause by multiplying by $\sqrt{(k_b / 4.0)}$. However, if k_b is less than 4.0, the value of (b_e) to be used is $b(\lambda_{ey}/\lambda_e)$.

A similar analysis is made for outstands, using k_{bo} = 0.425, as described in (6.6, 6.28).

In differentiating between lightly and heavily welded members, the effects of welded attachments (e.g. cleats) need not be considered.

A material supplier of welded sections will typically include the appropriate classification (HW, LW) in their product literature.

C6.2.4.4

For circular hollow members subject to significant design moment and very low design axial compressive force (N^*), the limit on effective diameter for compression given by 6.2.4.4 may be relaxed. It is recommended that this relaxation is applicable when $N^* \leq 0.1 A_n f_y$. In this instance, the calculation of section and member capacity in compression may be undertaken in accordance with section 6 by replacing the form factor given by 6.2.2 with $k_f = Z_e/Z$, as calculated from 5.2.5.3. Design for combined moment and compression is then undertaken in accordance with section 8.

C6.3 NOMINAL MEMBER CAPACITY

C6.3.1 Definitions

The definitions are included to clarify the subsequent clauses.

A definition of the actual length of the compression member is given in order to calculate the effective length (L_e) which is used in the member capacity calculations.

The geometrical slenderness ratio L_e/r is used to calculate the nominal member capacity (N_c) of the compression member based on buckling. **Note that (*r*) is computed using the gross section properties.** For most structural members loaded in axial compression, buckling may take place about either principal axis, and it is therefore necessary to calculate the geometrical slenderness ratios $(L_e/r)_x$ and $(L_e/r)_y$ about the principal *x* and *y* axes respectively. Note that for angle sections loaded in uniform compression, the radii of gyration r_x and r_y are taken about the principal axes, which are those inclined to the angle legs and not the axes parallel to the legs. For such members the radius of gyration about the axis parallel to the connected leg (r_h) may also be required in accordance with 6.6 or 8.4.6.1.

C6.3.2 Effective length

The effective length (L_e) is used to calculate the geometrical slenderness ratio (L_e/r). L_e is found from 4.8.3 or 12.8.2 as appropriate, and a discussion is given in Commentary Clauses C4.8.3 or C12.8.2.

C6.3.3 Nominal capacity of a member of constant cross section

The nominal capacity (N_c) accounts for the effects of flexural buckling (6.6, 6.28) and is calculated by multiplying the nominal section capacity (N_s) by a geometric slenderness reduction factor (α_c). The value of (α_c) depends not only on the geometrical slenderness ratio (L_e/r), but also on the yield stress and on the section type, the latter of which is represented by the member section constant (α_b). Five values of (α_b) are given, reflecting the various section types and distributions and magnitudes of the residual stresses. Justification for the selection of (α_b) for the ranges of Australian sections is given in (6.16 – 6.18); similar justification can be shown for sections of other origin.

The values of α_c given for modified member slenderness ratios from 340 to 600 under the section constant α_b = 0.5 are for use in determining the bearing buckling capacity of webs in accordance with 5.13.4. Values of modified slenderness ratio up to at least 540 are possible in this application.

When the nominal capacity (N_c) is plotted against the nominal slenderness ratio $\lambda_n = (L_e/r)\sqrt{k_f}\sqrt{(f_y / 250)}$, the five section constants (α_b) produce five strength curves (figure C6.3.3). The concept of using multiple column curves more correctly reflects the strength of compression members and their derivation and application is presented in (6.4).

The clause presents a set of equations which may be used to calculate the geometrical slenderness reduction factor α_c, based on the derivations in (6.4). These equations may be programmed simply on a programmable calculator or on a microcomputer. In lieu of using these equations, the tabulations in tables 6.3.3(1) and 6.3.3(2) may be used. The procedure used to calculate (N_c) is therefore as follows:

(a) Calculate the section capacity (N_s) from 6.2;

(b) Calculate the modified slenderness ratio $\lambda_n = (L_e/r)\sqrt{k_f}\sqrt{(f_y / 250)}$ about the relevant axis;

(c) Select the appropriate member section constant (α_b) from table 6.3.3(1);

➤

(d) Read the value of (α_c) from table 6.3.3(2). Linear interpolation may be used;

(e) Calculate the nominal capacity N_c about the relevant axis from $N_c = \alpha_c N_s$; and

(f) Select the lower value of N_c and N_s, to which ϕ is then applied.

The provisions of this clause are for members which buckle flexurally under applied compression. Some members, such as short cruciforms, tees and concentrically loaded angles, buckle in a torsional or flexural-torsional mode before flexural buckling. For these, the modified slenderness may be calculated as $\lambda_n = 90 \sqrt{(N_s / N_{om})}$, in which ($N_{om}$) is the elastic torsional or flexural-torsional buckling load. The nominal capacity (N_c) may then be determined in the usual way. This method of 'design by buckling analysis' is documented in (6.6, 6.28). The calculation of N_{om} is described in (6.3, 6.6, 6.19, 6.28).

C6.3.4 Nominal capacity of a member of varying cross section

This clause also uses the method of 'design by buckling analysis' documented in (6.6, 6.28). For this method, it is assumed that the interaction between yielding, elastic buckling, and geometric imperfection in compression members of varying cross section is the same as that for members of uniform cross section. Tabulations of elastic buckling loads (N_{om}) exist for a range of non-uniform members (6.20 – 6.22), a general computer program is described in (6.23) and a method for general derivation of N_{om} is given in section 4.7.3.2 of (6.28).

C6.4 LACED AND BATTENED COMPRESSION MEMBERS

C6.4.1 Design forces

To ensure that a compression member acts as a single member without premature failure of the shear-resisting elements (battens or lacing), the member must be designed for a transverse shear force (V^*) applied where it will have the most unfavourable effect. The expression for the design transverse shear force (V^*), which the main component-connecting elements and connections are designed to resist, is obtained from (6.5), with the addition of a minimum value of $0.01 N^*$ for low slenderness members to account for initial imperfections and eccentricities.

When laced and battened compression members are subjected to load combinations including earthquake loads, the design shear force (V^*) specified above is sufficient for category 3 and 4 members only.

C6.4.2 Laced compression members

This clause sets out the proportions and details for the main components and connecting elements to ensure that they act together as an integral member. These details are based on past experience (6.24) and modifications that have been made in line with the requirements of (6.25). For members subject to load combinations including earthquake loads, these requirements are applicable to category 3 and 4 members only. For category 1 and 2 members, refer to 12.9.8.2.

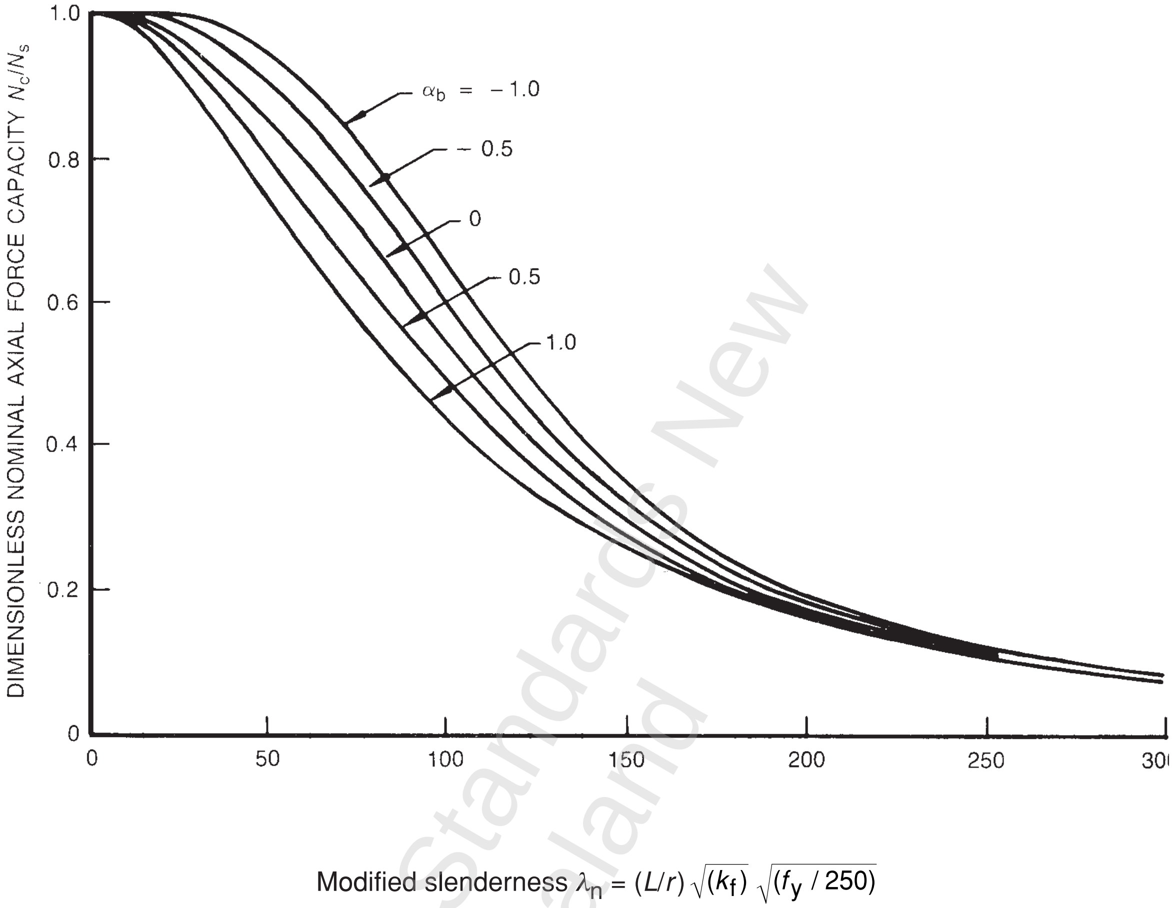

Figure C6.3.3 – Nominal axial force capacities in 6.3

C6.4.2.1 Slenderness ratio of a main component

The slenderness limit of 50 ensures that each main component is a relatively stocky member irrespective of the slenderness of the whole member. The slenderness limit of 0.6 times the slenderness ratio of the whole member not only ensures that each main component is less slender than the whole member, but also results in lacing being attached to at least 2 intermediate points within the length of the member.

C6.4.2.2 Slenderness ratio of a laced compression member

The limit of $1.4(L_e/r)_c$ is to prevent the possibility of a main component failing between consecutive lacing points.

C6.4.2.3 Lacing angle

Studies (6.5) have shown these angles to be the most effective for lacing members.

C6.4.2.4 Effective length of a lacing element

The different effective length values reflect the difference in the restraint provided by single and double lacing systems.

C6.4.2.5 Slenderness ratio limit of a lacing element

A nominal value is provided to ensure a reasonable stiffness and capacity for the lacing.

C6.4.2.6 Mutually opposed lacing

The asymmetry of 2 single lacing systems opposed in direction on opposite sides of a main member results in torsional effects under axial compression.

Significant second-order effects can arise if additional lacing elements (except tie plates specified in 6.4.2.7) are provided at 90° to the longitudinal axis of the member, and these effects cannot be neglected.

C6.4.2.7 Tie plates

These are used to anchor the lacing system at points where it is interrupted, such as at the member ends. They are designed as battens, in accordance with 6.4.3.

C6.4.3 Battened compression members

The details in this clause are based on past experience (6.24), with modifications in line with (6.25). For members subject to load combinations including earthquake loads, these requirements are applicable to category 3 and 4 members only. For category 1 and 2 members, refer to 12.9.8.2.

C6.4.3.1 Slenderness ratio of a main component

The slenderness limit of 50 ensures that each main component is a relatively stocky member irrespective of the slenderness of the whole member. The slenderness limit of 0.6 times the slenderness ratio of the whole member not only ensures that each main component is less slender than the whole member, but also results in 2 intermediate battens being provided within the length of the member in addition to the end battens.

C6.4.3.2 Slenderness ratio of a battened compression member

For a member with diagonal lacing which has geometries and proportions in accordance with 6.4.2, shear deformations are small and do not significantly reduce the strength below that for a similar member with a solid web. There are no webs or diagonals in battened columns, however, to resist transverse shear, and hence the main components and battens act together as a Vierendeel truss.

To account for the reduction in the strength of a battened member due to shear deformation, an increased slenderness ratio $(L_e/r)_{bn}$ is used, which depends on the relative values of the slenderness of the member as a whole, assuming the main components act together as an integral member, and the slenderness of a single main component between consecutive points where battens are attached.

C6.4.3.3 Effective length of a batten

The reduction in effective length for an intermediate batten reflects the degree of restraint provided by the main components.

C6.4.3.4 Maximum slenderness ratio of a batten

A nominal value is provided to ensure a reasonable capacity and stiffness for the batten.

C6.4.3.5 Width of a batten

This width of batten is required so that it will be stiff and be able to provide adequate connection to the main components, so as to ensure adequate Vierendeel action.

C6.4.3.6 Thickness of a batten

Where plate slenderness could reduce capacity, provision is made for a stiffener.

C6.4.3.7 Loads on battens

The transverse shear force (V^*) in the main components results in a design longitudinal shear force V_1^* and a design bending moment (M^*) which must be resisted by the battens. V_1^* and M^* are determined from statics assuming points of inflection in the main components midway between the battens and at the midspan of each batten (6.5). Diagrammatical guidance is given in (6.28).

C6.5 COMPRESSION MEMBERS BACK TO BACK

The details in this clause are based on past experience (6.24), with modifications in line with (6.25). For members subject to load combinations including earthquake loads, these requirements are applicable to category 3 and 4 members only. For category 1 and 2 members, refer to 12.9.8.2.

C6.5.1 Components separated

C6.5.1.1 Application

This clause is limited to double angles, channels or tees separated by no more than the distance required for end gusset connection in normal practice.

C6.5.1.3 Slenderness

The member is treated as an equivalent battened member (6.4.3.2).

C6.5.1.4 Connection

The member is treated as an equivalent battened member (6.4.3). A minimum of 2 fasteners or the equivalent is required at the ends of each main component.

C6.5.1.5 Design forces

The design longitudinal shear force (V_1^*) is a close approximation to that which can be derived from statics assuming points of inflection in the main components midway between the connections and at the connections.

C6.5.2 Components in contact

C6.5.2.4 Connection

The member is treated as an equivalent battened member. A minimum of 2 fasteners or the equivalent is required at the ends of each main component.

C6.6 DISCONTINUOUS ANGLE, CHANNEL AND TEE SECTION COMPRESSION MEMBERS NOT REQUIRING DESIGN FOR MOMENT ACTION

The requirements of 6.6 are taken principally from BS 5950:Part 1 (6.25). They are only applicable for use where there is no transverse loading on the member between supports and where the support conditions and, where relevant, the member slenderness, comply exactly with the relevant requirements of the clause.

A low level of applied transverse loading may be ignored in design if the simple beam bending moment that it would generate on the member in the plane of loading is less than either $0.1M_{bx}$ for that member, calculated from 5.6 using $\alpha_m = 1.13$, or $0.1M_{sy}$, calculated from 5.2, whichever is appropriate. Where the plane of loading is not a principal axis, the resulting moments about each principal axis should be determined and both the *x*- and *y*-axis checks made.

The provisions take account of the fact that the moment generated at the connection, due to eccentricity of load transfer, is also partially resisted by the rigidity of the connection detail, rather than having to be resisted solely by the compression member itself. ➤

The effective lengths required for 6.6.2 to 6.6.5 include allowance for the effect of joint eccentricity and the effect of joint fixity.

These provisions are formulated for application to typical relatively lightly loaded, long span truss and column sub-assemblages in which this type of member will most commonly be used.

This revision has seen the application of clause 6.6.2 restricted to use of slender single angle compression members (i.e. those with $L/r_y \geq 150$). This revision requires the moment generated by end eccentricity of the axial compression force to be explicitly considered, for stockier single angle compression members. This consideration is in accordance with the revised and simplified clause 8.4.6. More detailed background to this change is given in (6.31), along with 2 design examples.

The same concept has been applied to all configurations given in 6.6 which involve eccentric transfer of compression force perpendicular to the plane of the supporting member at the supports.

C6.7 RESTRAINING ELEMENTS

C6.7.1 General requirements

The design loads, including any notional horizontal loads from 3.2.4, can result in forces being induced in restraining members and connections, which must be designed for. Detailed guidance on the design of different restraining systems is given in section 4.6.2 of (6.28).

C6.7.2 Restraining members and connections

The restraint is required to be able to transfer 2.5 % of the axial compression force in the member being restrained, where this is greater than the force specified in 6.7.1. A stiffness requirement is not given even though there is a theoretical solution (6.26). This follows the finding (6.27, 6.29) that the requirements for centrally braced columns are satisfied by practical braces which satisfy the 2.5 % rule.

When the restraints are more closely spaced than is required to ensure that $N^* \leq \phi N_c$, then an appropriate group of restraints is required as a whole to be able to transfer 2.5 % of the force in the compression member, rather than each individual restraint.

When the restraints are only just sufficiently spaced to ensure that $N^* \leq \phi N_c$, then each restraint must be designed to transfer the 2.5 % of the force in the compression member.

C6.7.3 Parallel restrained compression members

This clause provides for a reduction in the rate of accumulation of the restraint forces for parallel members beyond the connected member from 2.5 % to 1.25 %. This reduction reflects the possibility, verified by analysis (6.29), that the crookedness or load eccentricity of any other parallel member may act in the opposite sense, and reduce the total restraint force.

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C7 MEMBERS SUBJECT TO AXIAL TENSION

C7.1 DESIGN FOR AXIAL TENSION

This section covers the design of members, subject to axial tension forces, which are statically loaded. Members subject to fatigue loading should also be assessed in accordance with section 10 and members subject to design loading combinations including earthquake loads should be assessed in accordance with section 12. A general coverage of the design of tension members may be found in (7.1, 7.10).

C7.2 NOMINAL SECTION CAPACITY

The failure criteria, on which the expressions for nominal section capacity are based, are yielding of the gross section ($A_g f_y$), given by equation 7.2.1, and fracture through the net section ($A_n f_u$), given by equation 7.2.2. Because of strain hardening, a ductile steel member loaded in axial tension can resist without fracture a force greater than the product of the gross area times the yield stress. Depending upon the ratio of the net area to the gross area, the member can fail by fracture through the net area at a load smaller than the load required to yield the gross area (7.2, 7.7, 7.8, 7.10).

Members for which ($A_n f_u > A_g f_y$) yield in the gross section before net section fracture occurs, and such a member has a considerable reserve of ductility, as evidenced by significant deformation. Alternatively, members for which ($A_n f_u < A_g f_y$) strain-harden at the net section and then fail by fracture before any significant yielding of the gross section occurs.

Hence, yielding of the gross section and fracture through the net area both constitute ultimate limit states. The 0.85 factor in equation 7.2.2 is intended to account for sudden failure by local brittle behaviour at the net section. The part of the member occupied by the net area at a bolted connection usually has a negligible length compared to the total member length. As a result, strain hardening at the net section is readily achieved and yielding of the net section does not constitute an ultimate limit state of practical significance, in either seismic or non-seismic applications.

A discussion of the effect of self weight on simple tension members (rods, angles, hollow sections) may be found in (7.5, 7.10).

Tensile stress areas for common merchant round bars which are provided with threads complying with AS 1275, are given in table C7.2.

C7.2.2 Area replacement plates for seismic applications

For tension members subjected to inelastic demand from severe seismic forces, it may be necessary to suppress failure at the net section in order to provide the ductility capacity required. This is achieved by limiting the reduction in cross section area within a yielding region in accordance with 12.9.4.2. The restriction on local reduction in cross section area from 12.9.4.2 is quite severe and in many instances will require the ratio of (A_n/A_g) to be 1.0. In such situations, any loss of cross section area due to penetrations or holes, including fastener holes, will have to be compensated for, at that location, by area replacement plates. These plates should extend a distance of ($d/2$) , but not less than 150 mm, past the closest penetration to the loaded face and the welds connecting each plate to the member should be sized to develop the design yield capacity of the plate ($\phi A_s f_{ys}$).

Table C7.2 – Tensile stress areas – round bars

Bar diameter (mm)	12	14	16	18	20	22	24	27	30	33	36	39	42	45	48
Tensile stress area (mm^2)	84.3	115	157	192	245	303	353	459	561	694	817	976	1121	1306	1473

C7.3 DISTRIBUTION OF FORCES

Some methods for attaching the ends of tension members to supports are shown schematically in figure C7.3. Also shown in this figure are the distributions of axial stress which would be expected in each case. The first 2 alternatives illustrate the eccentricity effect of connections which provide non-uniform force distribution, and the second 2 illustrate the shear lag effect in concentric end connections.

C7.3.1 End connections providing uniform force distribution

Where a connection is made by bolting or welding to all elements of the member cross section, the member may be assumed to have a uniform stress distribution across the cross section, and the correction factor (k_{te}) is taken as 1.0. The clause specifies the requirements of the end connection for this assumption to be valid. The behaviour of these types of members is discussed in (7.2).

C7.3.2 End connections providing non-uniform force distribution

Where the ends of members are connected such that not all elements of the member cross section are attached to the support, then additional stresses resulting from shear lag or eccentricity are induced and should be accounted for in the design. The 1989 and 1992 editions of the Standard have allowed for this effect by the use of a design equation to convert the actual cross-sectional area into an effective cross-sectional area. These design equations were based on work by Nelson (7.3), Munse and Chesson (7.4) and British and American practice of the time, and have been since confirmed (7.6, 7.9).

The behaviour of members with both these types of end connections is also discussed in (7.2). Various methods for empirically adjusting for the above effects have been investigated, and the use of the simple correction factor, k_{te}, given in table 7.3.2 was suggested as being sufficiently accurate (7.2). These correction factors are applied to the expression for the nominal capacity in 7.2 for net section fracture at the connection, because the non-uniform distribution of force is local to the connection. The correction factor is not applied to the equation for gross section yielding in 7.2, in recognition of the fact that the non-uniformities decrease with distance away from the connection, and are further reduced by redistribution after yielding.

In theory the same provisions apply to end connections to stocky compression members and this is recognized herein. However, the type of member presented in table 7.3.2 is covered for compression design of slender members by table 6.6, where reduction factors required for the given slenderness ratio will also cover the effect of end connection eccentricity in slender members subject to compression load only.

In seismic applications, the load transfer at the connection must not be too eccentric, irrespective of the magnitude of design action, when the member is designed for significant inelastic demand. The correction factor is therefore restricted to values of 0.85 or higher for category 1, 2 or 3 members, using the same philosophy as is applied in 12.9.4.2.

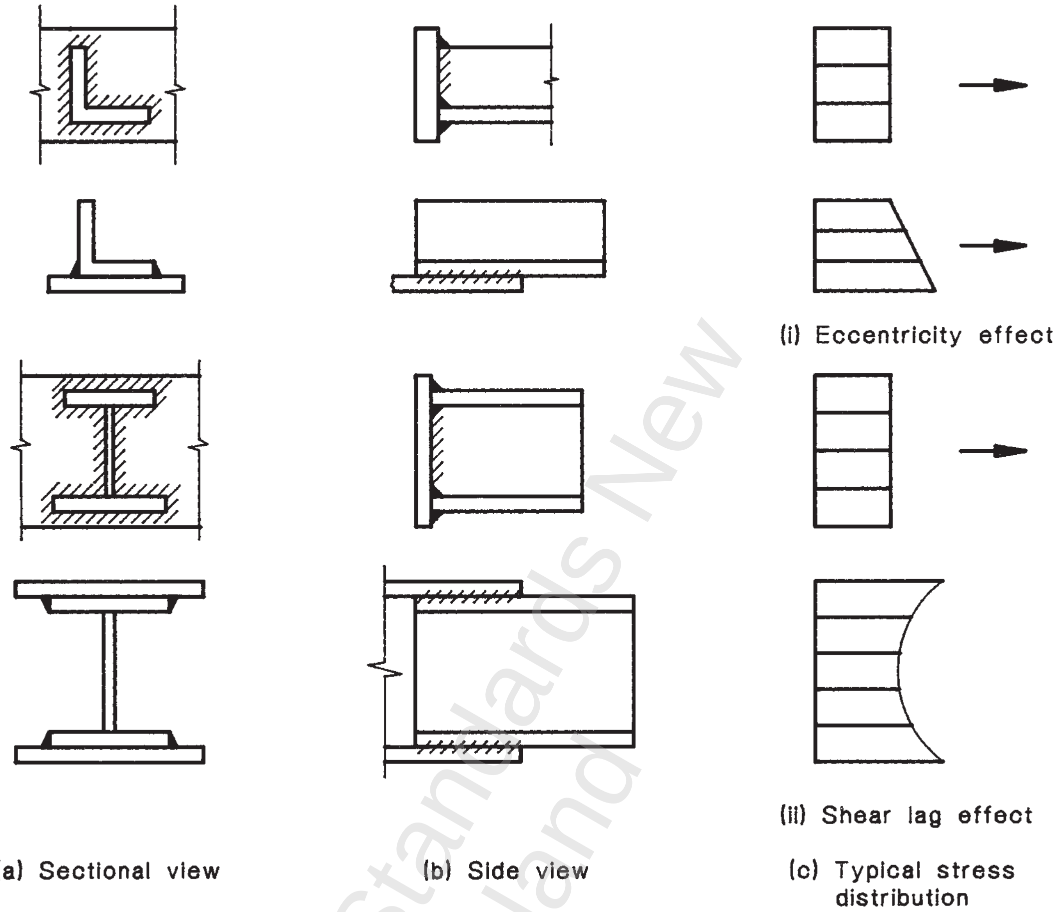


Figure C7.3 – Methods of end attachment

C7.4 TENSION MEMBERS WITH 2 OR MORE MAIN COMPONENTS

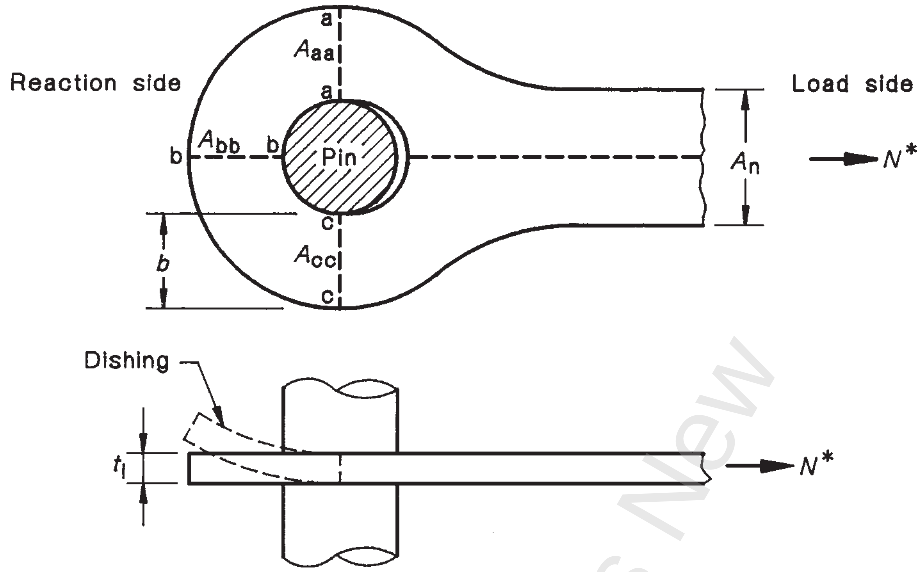
These design requirements are similar to provisions in the 1989 edition of this Standard and refer back to the comparable provisions for compression members in 6.4 and 6.5. Where the members are subject to inelastic seismic application as category 1 or 2 members, the requirements for design of the connecting elements given in this section are not sufficient and the more stringent requirements of 12.9.8 apply.

C7.5 MEMBERS WITH PIN CONNECTIONS

These design requirements are similar to provisions in the 1989 edition of the Standard. The provisions are intended to prevent tearing-through at the end of the eye-bar and dishing of the plate around the pin. The requirements are empirically based and derive from BS 5950:Part 1 and successful past practice.

The provisions are summarized in figure C7.5 for a single plate member. Provision (b) is intended to prevent tearing on the reaction side while provision (a) is intended to control dishing. Provision (c) is an empirical attempt to allow for stress concentration effects.

Pin connections to category 1, 2 or 3 members should be designed for the design actions from 12.9.1.2.2.



REQUIREMENTS:

(a) $t_i \geq 0.25b$

(b) $A_{bb} > A_n$

(c) $A_{aa} + A_{cc} \geq 1.33\ A_n$

where N^* is the design axial tensile force.

Figure C7.5 – Pin connection for single plate member

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C8 MEMBERS SUBJECT TO COMBINED ACTIONS

INTRODUCTION

The strength design of members subject to combined axial and bending actions is governed by clauses 8.1 to 8.4. The behaviour on which these clauses is based is described in textbooks such as (8.3) or in the HERA Design Guides Volume 1 (8.14). This commentary should be read in conjunction with such a reference. Worked examples are given in (8.4, 8.5, 8.14, 8.15), while computer programs are described in (8.6, 8.7).

Clause 8.2 defines the member actions which are to be designed for. Clause 8.3 defines the section capacity, which is governed either by yielding or local buckling, and which may control the design of fully restrained members. Clause 8.4 defines the member capacity, which often controls the design of members, especially those subject to *x*-axis bending and without full lateral restraint to 8.1.2.3.

The members of steel structures are often subjected to secondary torsional actions, in addition to the primary axial and bending actions. In the past, these actions were usually ignored, but designers who use three-dimensional analysis programs are becoming more aware of them. Because the Standard provides no guidance on designing against torsion (and neither do many other steel design standards around the world), there are no provisions for designing against combined torsion, axial and bending actions. This subject is discussed further in C8.5 and in more detail in section 8 of (8.14) or in (8.3).

It is of interest to designers to illustrate the parallels between the provisions of this Standard and the comparable provisions of the 1989 edition. The comparison is for ultimate limit state design, between this section (section 8) and the provisions of section 14 of NZS 3404: Part 2:1989. Broadly speaking:

(a) Clause 8.3 herein replaces 14.5.1 from NZS 3404:1989, except that the design provisions herein cover compact, non-compact and slender sections.

(b) Clauses 8.3.2, 8.3.3 and 8.3.4 herein replace 14.5.2.2 (a) and (b) and 14.5.3 from NZS 3404:1989. Use of the general design provisions will give design capacities similar to the provisions of NZS 3404:1989, while use of the more complex alternative design provisions, where applicable, will give enhanced design capacities.

(c) Clauses 8.4.4 and 8.4.5 herein replace 14.5.2.1 (a) and (b) and 14.5.3 from NZS 3404:1989. Use of the general design provisions will give similar design capacities to the provisions of NZS 3404:1989, while use of the more complex alternative design provisions, where applicable, will give enhanced design capacities.

(d) Clause 8.4.2 herein covers a specialised but not uncommon situation which replaces 14.5.2.1 (a) and (b) from NZS 3404:1989 for the case of axial force and minor principal axis bending, or for the case of axial force and major principal axis bending for members which have full lateral restraint to 8.1.2.3. This benefit of full lateral restraint was not recognized in the 1989 edition, nor was it clearly presented in the 1992 edition.

C8.1 GENERAL

C8.1.1

The first paragraph of 8.1.1 directs designers to check for the significance of the level of axial force, in 8.1.4. This is of importance in, for example, the rafters of portal frames, where the level of axial force is likely to be low and its significance will depend both on the magnitude of the design axial compression force and on the degree of restraint available against minor principal *y*-axis buckling in compression. Typically, only the length of rafter adjacent to the knee might have a significant level of axial compression in it and this only under load combinations generating the maximum design vertical load.

The second paragraph of 8.1.1 draws designers attention to the simplification of analysis under combined actions that can and should be applied for category 1, 2 or 3 members (i.e. subject to inelastic demand), and that are analysed in accordance with 4.4 or 4.5, when equation 12.8.3.1 of clause 12.8.3.1(b) is satisfied and the member has full lateral restraint to 8.1.2.4. The background to this simplification is given in Commentary Clause C12.8.3.1 and stems from the role of this equation, which is to determine if the critical location for design along the member is either at, or way from, the member ends. Compliance puts the critical location at the member ends, hence the section capacity from 8.3 governs for that direction of loading and the member capacity from 8.4 need not be checked.

In plastic design, the provisions of 8.4.3.2 will ensure that plastic hinges always form at the member ends (note that the requirement to ensure this for $N^{*}/\phi N_{s} > 0.15$ in 8.4.3.2.1 is the same as equation 12.8.3.1 except for the axial force limit) and the same simplification for combined action member design applies, with the requirements of 8.4.3.4 being the same as the section capacity requirements of 8.3.2 or 8.3.3 for compact sections which have their axial compression force suitably restricted according to the web slenderness.

Compliance with equation 12.8.3.1 is directional, and is often only required from 12.8.3.1(b) about the major principal *x*-axis of the member. If such a member is also subject to significant bending about the other axis (typically the *y*-axis), then the biaxial bending provisions for section capacity (8.3.4) will only apply if equation 12.8.3.1 is also satisfied about this other axis. If this is not the case, because either the check is not made or compliance is not achieved, then a suitable biaxial bending check combining the provisions of 8.3.4 and 8.4.5 must be made.

For example, in an *I*-section column member subject to biaxial bending and significant axial compression load, for which equation 12.8.3.1 is satisfied about the major principal *x*-axis only and which has full lateral restraint to 8.1.2.4, the following must be satisfied:

(a) If the section geometry and level of design axial force from 8.1.5 permits the alternative design provisions to be used:

$$\left(\frac{M_{x}^{*}}{\phi M_{rx}}\right)^{\gamma} + \left(\frac{M_{y}^{*}}{\phi M_{iy}}\right)^{1.4} \leq 1 \quad \text{(covers both section and member failure)}$$

where

γ and M_{rx} are calculated from the alternative design provisions of 8.3.4.2.

M_{iy} is calculated from the alternative design provisions of 8.4.2.2.2.

(b) If the section geometry or level of design axial force from 8.1.5 are such as to not permit the alternative design provisions to be used:

$$\left(\frac{M_x^*}{\phi M_{rx}}\right)^{1.0} + \left(\frac{M_y^*}{\phi M_{iy}}\right)^{1.4} \leq 1 \quad \text{(member failure)}$$

and

$$\left(\frac{M_x^*}{\phi M_{rx}}\right)^{1.0} + \left(\frac{M_y^*}{\phi M_{ry}}\right)^{1.0} \leq 1 \quad \text{(section failure)}$$

where

M_{rx}, M_{ry} are calculated from the general design provisions of 8.3.2.1 and 8.3.3.1 respectively.

M_{iy} is calculated from the general design provisions of 8.4.2.2.1.

C8.1.2 Definition of a member

In the 1997 edition, the definition of a member subject to combined actions has been re-written, from that of the 1992 edition, to provide consistency with the definition for a member subject to bending moment only (see 5.3.1.3) and a member subject to compression only (see 6.3.1).

Key points to note about the new provisions, for members subject to combined bending and axial compression (see 8.1.2.1), are as follows:

(1) Member moment capacity for *x*-axis bending is determined on a segment by segment basis, in accordance with 5.3.1.1.

(2) Restraint against compression is considered in accordance with 6.3.1.

(3) A point of support for bending may in addition be a point of restraint for compression, however the two locations may also be separate. Three options are given in 8.1.2.1. Examples of each are as follows:

(3.1) A portal frame column or a column in a typical multi-storey building are both examples of option (a) – i.e. the member spans between points of support/restraint at each end of the column.

(3.2) The length of portal frame rafter from the knee to a fly-brace just past the point of contraflexure would be an example of option (b). In this case, the fly-brace is a point of restraint for compression and also provides full or partial restraint for bending. The knee is a point of support/restraint.

(3.3) A vertically cantilevered *I*-section supporting a heavy load on the free end is an example of option (c).

(4) The provisions of 8.1.2.1 recognize that a restraint at the end of a segment of a member subject to combined actions will provide restraint for bending moment, but may not necessarily also be designed to provide restraint for compression, when the member is subdivided into more than one segment.

An example might be a portal frame column, subject to $N^*_{compression}$ and M_x^*, with supports for bending and restraints for *y*-axis compression at the knee and base and an intermediate height restraint for bending only. This column would be split into 2 segments for moment capacity design, however the design compression capacity would be based on member ➤

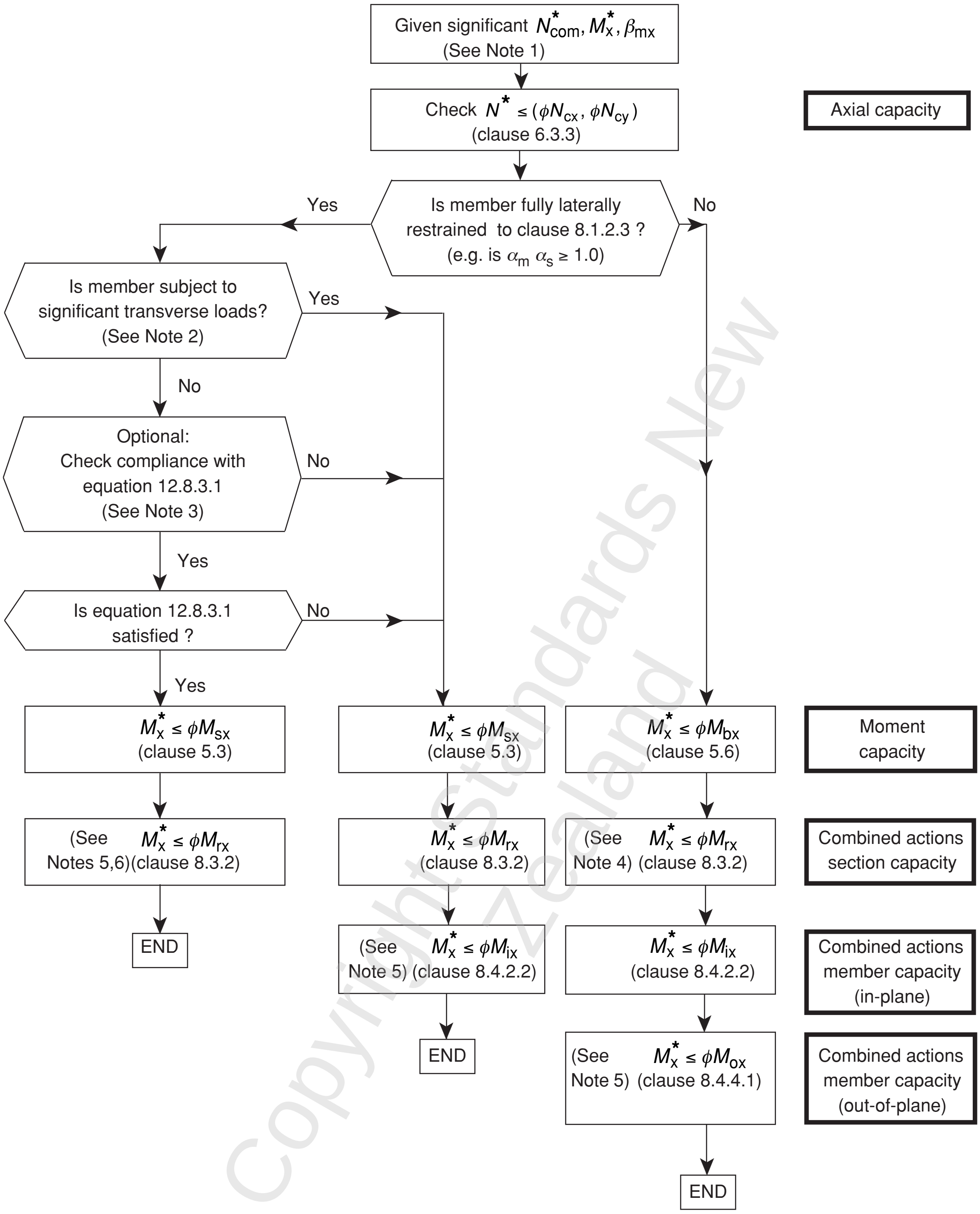


Figure C8.1.1 – Flowchart for members subject to major axis bending moment and compression combined actions and which are not subjected to inelastic demand

NOTE –

(1) When axial force is not deemed significant (see 8.1.4) combined action checks are not required.

(2) Transverse loading may be considered as not significant if it generates a simply supported design moment not exceeding 10 % of the design section moment capacity, ϕM_{sx}.

(3) Checking of this equation is optional. If checked, use the value of β_m from 4.4.3.2.3. If not checked, follow the "No" path.

(4) ϕM_{sx} is required for this check.

(5) When using the general linear interaction provisions of 8.3.2.1, 8.4.2.2.1 and 8.4.4.1.1, the last check for each route through the design flowchart governs. When the provisions of 8.1.5 permit the alternative provisions of 8.3.2.2, 8.4.2.2.2 and 8.4.4.1.2 to be used, then one of the earlier checks may govern.

(6) This option is appropriate to all portal frame column members which have full lateral restraint and insignificant transverse loading.

Amd 1
June '01

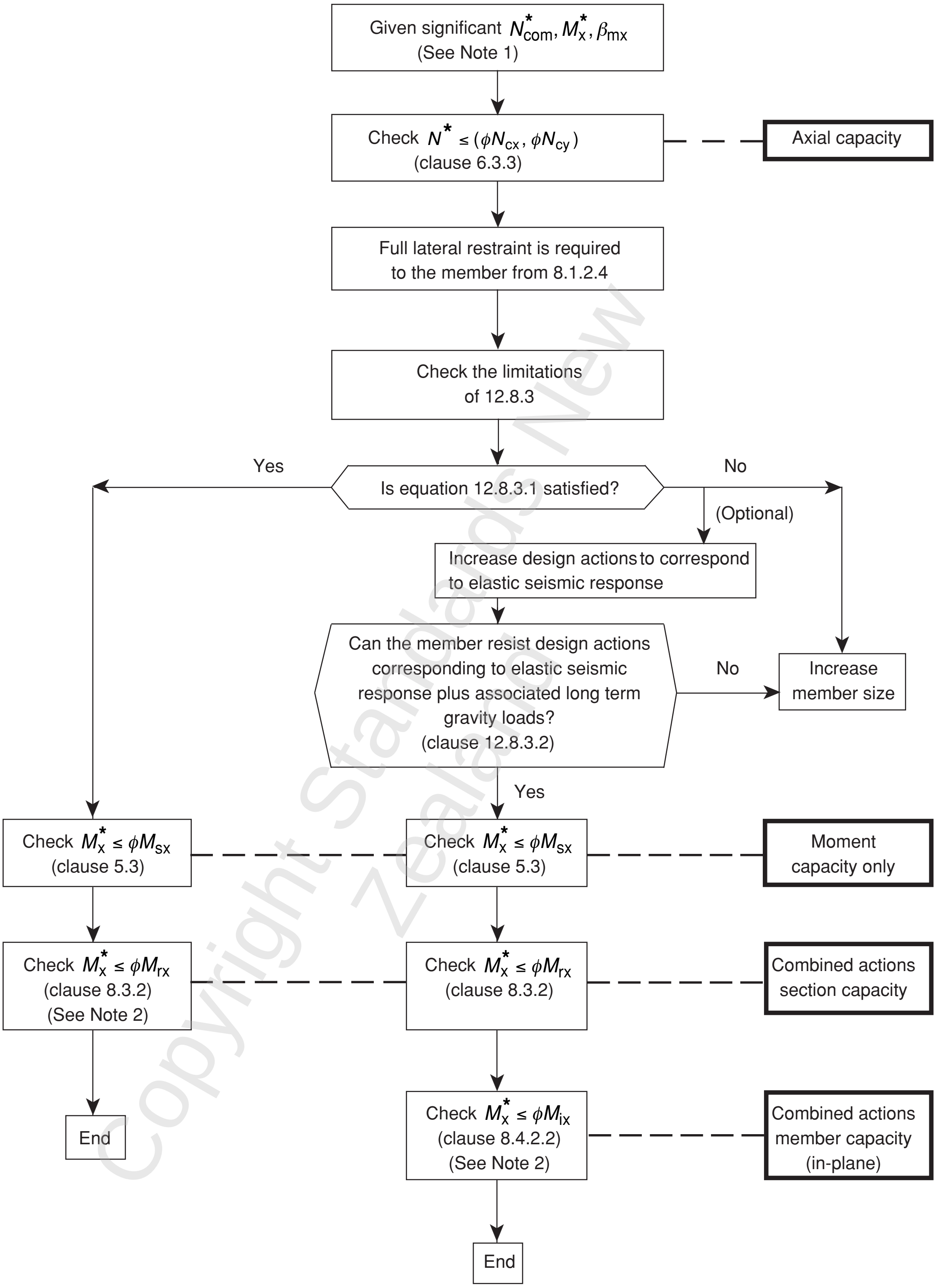


Figure C8.1.2 – Flowchart for members subject to major axis bending moment and compression combined actions and which are subjected to inelastic demand

NOTE –

(1) When axial force is not deemed significant (see 8.1.4) combined action checks are not required.

(2) When using the general linear interaction provisions of 8.3.2.1 and 8.4.2.2.1, the last check for each route through the design flowchart governs. When the provisions of 8.1.5 permit the alternative provisions of 8.3.2.2 and 8.4.2.2.2 to be used, then one of the earlier checks may govern.

buckling in compression between the knee and the base. If the design compression load is significant (see 8.1.4), then the combined actions check would be undertaken on each segment, using the segment length as the basis for design member moment capacity determination (ϕM_{bx}) and the member length for member compression capacity determination, ϕN_{cy}. This is covered further, for portal frame columns, in Issue No. 25 of the HERA Steel Design and Construction Bulletin (8.16).

If the intermediate restraint(s) in this example of the portal frame column are only designed to be effective in bending, then there are advantages in both segments having full lateral restraint, as this means that the interaction of *x*-axis bending and *y*-axis buckling does not need checking; the design check involves $N^* \leq \phi N_{cy}$ to 6.3.3 and combined ϕM_{sx} and ϕN action check, to either 8.3.2 or 8.4.2.2. See figure C8.1.1 for more details of which applies. Typically it will be 8.3.2, as identified through Note 6.

If either segment does not have full lateral restraint, then the interaction of ϕM_{bx} and ϕN_{cy} needs to be checked, to 8.4.4.1. ϕN_{cy} will be lowered because the intermediate restraint is not effective in compression and this will also reduce the combined actions member capacity.

Note that, if this portal frame column is a category 1, 2 or 3 member, then both segments will be required to have full lateral restraint to 8.1.2.4. Furthermore, such a column member of a portal frame will always achieve compliance with equation 12.8.3.1 (given that the transverse load on the member must be negligible; see 12.8.3.3) and hence the combined actions check to 8.3.2 would be the only section 8 check required for this category 1, 2 or 3 portal frame column subject to $N^*_{compression}$ and M^*_x.

For a member subject to combined bending and axial tension (see 8.1.2.2), it is the member length under bending that defines the member length for combined actions.

Clause 8.1.2.3 provides the definition for full lateral restraint of the member. The distinction between achieving full lateral restraint and not achieving full lateral restraint is important in applying 8.4.

Amd 1 June '01 Clause 8.1.2.4 then provides the important requirement that a member which contains one or more yielding regions and is subject to combined bending moment and significant axial actions must have full lateral restraint.

C8.1.3 Restraining elements

The purpose of this clause is simply to remind designers of the strength requirements that restraining elements must meet.

C8.1.4 Significant axial force

The threshold values at which the design axial force is deemed significant are consistent with similar criteria applied throughout this Standard.

For application of these provisions to the rafters and columns of portal frames, refer to (8.16). (Note that this article supersedes that written in Issue No. 23 (5.43)).

Amd 1 June '01 A background to the Amendment No. 1 changes to clause 8.1.4 (a) is given on page 5 of the HERA *Steel Design and Construction Bulletin*, Issue No. 43, 1998. The changes described therein have been rewritten when putting them into the amendment to make them more compact and to avoid the need for a new sub-clause.

C8.1.5 Use of alternative design provisions

The alternative design provisions of 8.3 and 8.4 are applicable only to members that can dependably develop plasticity within their critical cross section (8.14).

The various limits on section geometry and design axial force presented in 8.1.5 provide the maximum number of combinations that allow these much more efficient alternative provisions to be used.

The principal difference between these provisions and those of the 1992 edition is one of format, in that all the necessary requirements to access the alternative design provisions are now listed in 8.1.5, rather than cross-referenced to relevant clauses from other sections.

There has been one relaxation, that being to allow the plate element slenderness limits for the flanges of an *I*-section to be increased from those for a compact section to the limits for a category 3 section. This is because experimental testing of category 3 *I*-sections, as reported in (12.49), has shown that these flange slenderness limits are suitable to allow the alternative design provisions to be used. See also section 6.6.4 of (8.14) for further details. This is an important relaxation, as it allows a wider range of columns in both non-seismic and seismic applications to access the alternative design provisions, with a considerable resulting increase in member design efficiency.

C8.2 DESIGN ACTIONS

This clause first defines the member design actions N^*, M_x^*, M_y^*. For checking the section capacity, these are the values at the section under consideration. All sections in the member must satisfy the requirements of 8.3, but usually the most heavily loaded sections are obvious and only these need be considered.

For checking the member capacities, the design actions are the maximum values in the member. Each of the appropriate in-plane, out-of-plane and biaxial member capacity requirements of section 8 must be satisfied for the member as a whole. Note that only the appropriate requirements need be satisfied; not all requirements are appropriate in many instances as discussed in Commentary Clause C8.1 and illustrated in the flowcharts of figures C8.1.1 and C8.1.2.

C8.3 SECTION CAPACITY

C8.3.2 Uniaxial bending about the major principal x-axis

The nominal section capacity of a member subjected to bending alone is reduced by the presence of axial force. Axial compression and tension both reduce the yield capacity of the section, while axial compression also reduces the local buckling capacity.

A simple straight line approximation is given for the reduction in the section moment capacity caused by axial force. This can be applied to any type of member cross section. Substituting the approximation into the design inequality leads to the equivalent formulation:

$$\frac{M_x^*}{\phi M_{sx}} + \frac{N^*}{\phi N_s} \leq 1.0$$

The approximation is conservative for compact or nearly compact doubly-symmetric members either subject to design axial tension or with appropriate limits on design axial compression, and so a more accurate alternative is provided in 8.3.2.2.

C8.3.3 Uniaxial bending about the minor principal y-axis

This clause is similar to the previous one for bending about the *x*-axis, in that a simple straight line approximation for the reduction in the section moment capacity is given for general use in 8.3.3.1, and also a more accurate and economical alternative in 8.3.3.2 for compact or nearly compact doubly-symmetric members either subject to design axial tension or with appropriate limits on design axial compression.

C8.3.4 Biaxial bending

Clause 8.3.4.1 gives a simple linear approximation for the section capacity of members subjected to biaxial bending about both principal axes, which reduces to the earlier linear approximations for uniaxial bending.

This is often very conservative, and so a more accurate and economical power law approximation is given in 8.3.4.2 for compact or nearly compact doubly-symmetric members either subject to design axial tension or with appropriate limits on design axial compression. The requirements for member capacity under biaxial bending from 8.4.5.1 and 8.4.5.2 will usually be more critical (i.e. give lower nominal capacity) than the alternative provision for section capacity, unless there is a significant local reduction in section geometry at a heavily loaded cross section or member failure is otherwise suppressed.

C8.4 MEMBER CAPACITY

C8.4.1 General

This clause directs the designer to the appropriate later clauses for design against failure in a principal plane of bending, against lateral buckling failure out of a principal plane of bending, or against biaxial bending failure. The background to these provisions is presented in section 6 of (8.14).

C8.4.2 In-plane capacity-elastic analysis

This clause governs the in-plane capacity of members which are bent in one principal plane, and which fail in that plane. Members which are bent about the major principal *x*-axis and which do not have full lateral restraint in accordance with 8.1.2.3 to prevent buckling out of the plane of bending must also be checked for their out-of-plane member capacity using 8.4.4.

C8.4.2.2 Compression members

The destabilizing effects of axial compression reduce the member in-plane moment capacity. Clause 8.4.2.2.1 gives a simple linear approximation for the reduced member capacity.

This approximation is often conservative, and so a more accurate approximation is given in 8.4.2.2.2 for compact or nearly compact doubly-symmetric members with appropriate limits on design axial compressive force. The economy produced by the use of this more accurate approximation instead of the linear approximation is most marked for members with high moment gradient (high values of β_m).

The effective length factor (k_e) used in this clause to determine the in-plane compression member capacity (N_c) is taken as unity for a sway member, because the effects of end restraints which influence member buckling have already been taken into account where required in section 4, either in amplifying the first-order moment distribution, or in carrying out a second-order analysis.

C8.4.2.3 Tension members

Axial tension does not reduce the member in-plane moment capacity, and so the design of these members will be governed by their section capacities, determined using 8.3.

C8.4.3 In-plane capacity-plastic analysis

C8.4.3.1 Application

This clause only applies to compact doubly symmetric *I*-section members which form part of a frame which is analysed plastically. It governs the in-plane capacities of members which are bent in one principal plane, and which fail in that plane. Such members must have sufficient lateral restraint to prevent buckling out of the plane of bending.

The design bending moments in members assumed to contain a plastic hinge must not exceed ϕ times the reduced section moment capacity (M_{pr}) given in 8.4.3.4, in which ϕ is the strength reduction factor (table 3.3(1)). This condition will automatically be satisfied if (ϕM_{pr}) is used for the member plastic moment capacities in the plastic analysis carried out for the design loads and if the plastic collapse load factor is not less than 1.0. Note that the design moments determined by a first-order plastic analysis for a frame with an elastic buckling load factor which satisfies $5 \leq \lambda_c < 10$ will need to be amplified in accordance with 4.6.4.

C8.4.3.2 Member slenderness

Members which develop a plastic hinge according to the plastic analysis must satisfy the slenderness provisions of this clause. Members which do not satisfy these provisions are deemed to be unable to reach or maintain full plasticity while the collapse mechanism develops, and so must not develop a plastic hinge in the plastic analysis. Such members may be designed using the elastic analysis in-plane member capacity requirements of 8.4.2.2 and 8.4.2.3.

These requirements ensure that a plastic hinge will form at the ends of the member, where it can be adequately laterally braced. They have wider application than for plastic design, being also relevant to members subject to inelastic demand from earthquake effects or moment redistribution, and are presented in 12.8.3.1(b) in the required form for this application.

Amd 2 Oct. '07

A new and more accurate equation for end yielding criterion has been developed. Details are in the following report: *Plastic Hinge Location in Columns of Steel Frames*, by Peng B. et al, University of Canterbury Civil Engineering Research Report, 2006.

C8.4.3.3 Web slenderness

Axial compression increases the extent of plastic compressive yielding in the web, and consequently increases the possibility of premature local buckling which may prevent the collapse mechanism from developing. This clause restricts the axial forces which can be permitted in webs.

This situation is also of concern for members in seismic applications which are expected to undergo inelastic action. The provisions of this clause are therefore applicable to category 1 or 2 members and are specified for this purpose in 12.8.3.1(c). For category 3 members, the extent of anticipated inelastic demand is considerably lower, requiring less stringent restrictions on axial compression force as a function of web slenderness, as given by equations 12.8.3.2(1) and 12.8.3.2(2). Note that the force requiring restriction is the constant level of compression force; this is N_g^*.

C8.4.3.4 Plastic moment capacity

This clause gives approximations for the reduced design plastic moment capacities of members with axial compression or tension forces. These approximations are equivalent to the more accurate approximations used in 8.3.2.2 and 8.3.3.2 for the reduced section capacities of compact doubly-symmetric *I*-section members. The plastic moment capacities are only reduced when $N^*/\phi N_s > 0.15$ for sections bent about the major principal *x*-axis, or when $N^*/\phi N_s \geq 0.4$ for sections bent about the minor principal *y*-axis.

When the plastic moment capacities are reduced, then an iterative process will be required for the plastic analysis. It is suggested that the plastic moment capacities should not be reduced for the first cycle of this process. They should be reduced in the second cycle using the values of N^* determined from the first cycle. Often it will not be necessary to reduce the plastic moment capacities, in which case a second cycle is not required. When it is necessary, then a sufficiently accurate solution will usually be obtained from the second cycle.

C8.4.4 Out-of-plane capacity

This clause governs the design of members which are bent about the major principal *x*-axis and fail by buckling laterally out of the plane of bending, through not having full lateral restraint. The in-plane capacity of these members must also be checked using 8.4.2 but usually this will not be critical and does not need to be evaluated in detail. Members which are bent about the minor principal *y*-axis do not normally fail by buckling out of the plane of bending unless there are transverse loads acting far above the critical flange, hence 8.4.4 is written for major principal *x*-axis application only.

In calculating M_{bx} and M_{bxo} for application of 8.4.4.1 or 8.4.4.2, the length for which α_m and β_m are assessed is that derived in accordance with 8.1.2.1, for the segment under consideration (or for the member if it comprises only one segment).

C8.4.4.1 Compression members

The destabilizing effects of axial compression reduce the member lateral buckling moment capacity (M_{bx}). Clause 8.4.4.1.1 gives a simple linear approximation for the reduced member capacity which uses the out-of-plane compression member capacity (N_{cy}).

This approximation is often conservative, and so a more accurate approximation is given in 8.4.4.1.2 for compact or nearly compact doubly-symmetric members with appropriate limits on design axial compression. The economy produced by the use of this alternative instead of the linear approximation is most marked for members with high moment gradient (high values of β_m).

Note that L_z in 8.4.4.1.2 is the distance between partial or full restraints that prevent twist of the section about its centroid. Not all restraint conditions given in clause 5.4 do this; refer to section 4.6.2.2 of (8.14) for guidance on effective twist restraints.

Amd 1 June '01 The changes introduced in Amendment No. 1 to the definitions for M_{bxo} and L_z are for clarity; the meanings are not changed.

C8.4.4.2 Tension members

Axial tension increases the member lateral buckling moment capacity $M_{bx,}$ hence the positive sign in this interaction equation. This clause gives a simple linear approximation for the increased moment capacity which is conservative for small to moderate axial tensions. The design of members with high axial tensions will be governed by the section capacity requirements of 8.3.2.

C8.4.5 Biaxial bending capacity

This clause governs the biaxial capacities of members which are bent about both principal axes.

C8.4.5.1 Compression members or members subject to zero axial force

This clause gives a power law approximation for the biaxial bending capacities of members with axial compression or for biaxial bending alone ($N^* = 0$). For the term (M_{cx}) associated with bending about the major principal *x*-axis, the answer depends on whether or not the member has full lateral restraint to 8.1.2.3.

Amd 1 June '01

The power law inequality can be conservatively approximated by a linear relationship obtained by setting the index γ equal to 1.0, but often at the expense of significant economy.

C8.4.5.2 Tension members

This clause gives a power law approximation for the biaxial bending capacities of members with axial tension, which is similar to that of the previous clause for members with axial compression, except that the term (M_{tx}) associated with bending about the major principal *x*-axis is taken as the lower of the in-plane reduced section capacity (M_{rx}) and the out-of-plane member capacity (M_{ox}).

The power law inequality can again be conservatively approximated by a linear relationship obtained by setting the index γ equal to 1.0, but often at the expense of significant economy.

C8.4.6 Double bolted or welded single angles eccentrically loaded in compression

This clause provides a special design method for single angle members which are connected by at least 2 bolts or welded at each end, are loaded through one leg in compression only but require design for combined moment and compression. Single angle members require such a design when their slenderness ratio (L/r_y) is low to medium; a value of 150 is the limit set for application of this clause (6.31).

The method presented in (8.2) provides a conservative way of combining the effects of buckling in the minor principal plane with that of bending, perpendicular to the plane of the supporting structural system, caused by the eccentricity of compression load transfer at the supports.

Revisions to 8.4.6 made in the 1997 revision have reduced this conservatism. A background to these revisions and 2 design examples are given in (6.31).

For an equal leg angle whose length-thickness ratio L/t is less than the stated limit, lateral buckling effects may be ignored, and the member capacity (M_{bx}) may be taken as the section capacity. For other equal leg angles, a simple approximation is given for the elastic buckling moment (M_o) which can then be used in 5.6.1.1.1 to obtain the member buckling capacity (M_{bx}).

For eccentrically loaded single angles with L/r_y < 150 and a single bolt connection at each end, it is recommended (6.31) to use equation 8.4.6, with the right hand side reduced to 0.8.

C8.5 TORSION

C8.5.1 Introduction

There are no rules given in this Standard for designing against torsion, which reflects a similar situation in most overseas steel design standards.

The difficulties of predicting torsional effects and of providing recommendations for designing against torsion have also probably discouraged attempts to provide design guidance. At the same time, the incentive that would be provided by a significant number of failures which could be attributed to torsion has been missing.

However, the growing use of three-dimensional computer analysis programs has made many designers aware of the presence of torsional effects in their structures, and has led them to ask about their significance and how to design for them. In addition, there may be circumstances when designers would wish to transfer loads by primary torsion actions, but have been prevented from doing so by the lack of information on how to predict and design for torsion effects in steel structures.

This Commentary Clause briefly discusses the occurrence and significance of torsion in steel structures, and makes suggestions for the analysis of and design for torsion, when necessary. It should be read in conjunction with section 8 of the HERA Design Guides Volume 1 (8.14) or with a textbook which covers this subject, such as (8.3).

Clause C8.5.2 deals with the occurrence of pure torsion, and of combined bending and torsion. Torsion actions are classified as primary or secondary, and primary torsion actions as being restrained, free, or destabilizing. Clause C8.5.3 deals with the different types of torsion; uniform, warping, and non-uniform.

Clause C8.5.4 cites references on the analysis of pure torsion, including the determination of the section properties, the analysis of the member distributions of uniform and warping torsion, and the analysis of the stress distributions across the section. It also proposes methods of first and second-order analysis of combined bending and torsion; second-order analysis will rarely be required.

Finally, C8.5.5 provides recommendations for the strength design for uniform, warping, and non-uniform torsion, and for combined bending and torsion.

A detailed design example is given in section 8 of (8.14).

C8.5.2 Occurrence

Torsion may occur as pure torsion, or in combination with other actions, such as bending. Pure torsion, for which only torsion is present and there are no other actions, occurs very rarely in steel structures. When it does occur, it is likely to be of little or no significance, as in the case of secondary torsion of otherwise unloaded members. An example of this type of torsion occurs when differential rotations of the joints of primary frames cause corresponding (compatible) twisting of unloaded bracing members which are perpendicular to the primary frame (figure C8.5.2 (1)).

Most commonly, torsion occurs in combination with bending actions. The torsion actions may be classified as primary or secondary, depending on whether the torsion action is required to transfer the load (primary torsion), or whether it arises as a secondary action, as in the case of twist rotations compatible with the joint rotations in primary frames (figure C8.5.2 (1)). For example, the use of three-dimensional analysis programs commonly leads to the prediction of small torques in the minor members running between the main frames. These secondary torques are not unlike the secondary bending moments predicted in rigid-jointed trusses, but usually ignored in the design of the members (a procedure justified by many years of satisfactory experience, based on the long-standing practice of analysing trusses as if pin-jointed). Secondary torques are usually small when there are alternative load paths of high stiffness, such as those through the planes of shear walls and floors, in which case they may be ignored.

Primary torsion actions may be classified as being restrained, free, or destabilizing (figure C8.5.2 (2)). For restrained torsion, the member applying the torsion action also applies a restraining action to the member resisting the torsion. In this case, the structure is redundant, and compatibility between the members must be satisfied in the analysis if the magnitudes of the torques and other actions are to be determined correctly (figure C8.5.2 (2) (b)).

Free torsion occurs when the member applying the torsion action does not restrain the twisting of the torsion member, but does prevent both its lateral deflection and any destabilizing actions between the torsion action and the bending actions on the torsion member (figure C8.5.2(2)(c)).

Destabilizing torsion may occur when the member applying the torsion action does not restrain either the twisting or the lateral deflection of the torsion member. In this case, buckling actions caused by the in-plane loading of the torsion member amplify both the torsion and out-of-plane bending actions (figure C8.5.2 (2) (d)).

C8.5.3 Types of pure torsion

C8.5.3.1 Uniform torsion

In uniform torsion (figure C8.5.3.1), there is no warping torque present, either because the rate of change of the twist rotation along the member is constant (which happens infrequently), or else because the warping section constant is negligibly small (as in the case of angles, tees, cruciforms and hollow sections).

In open sections, the uniform torque is the resultant of components arising from shear stresses which vary almost linearly across the thickness of the section wall, and which have lever arms of the order of the wall thickness (figure C8.5.3.1 (b)). These small lever arms are responsible for the comparatively high shear stresses and the comparatively low torsional stiffness.

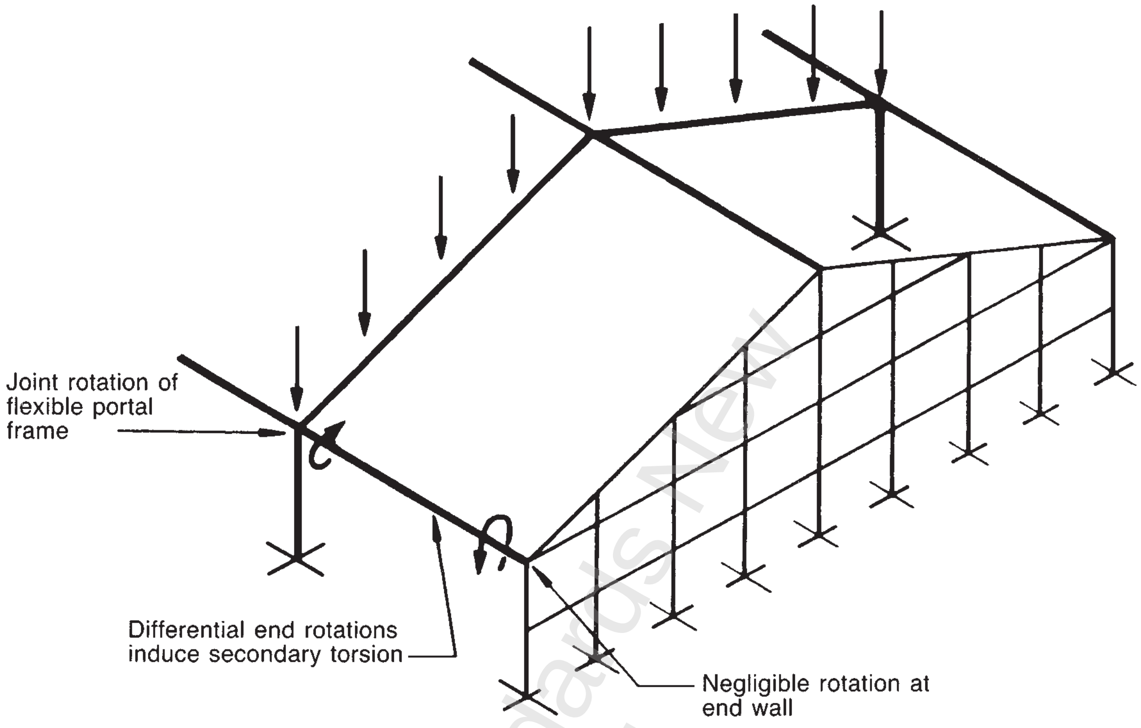


Figure C8.5.2(1) – Secondary torsion in an industrial frame

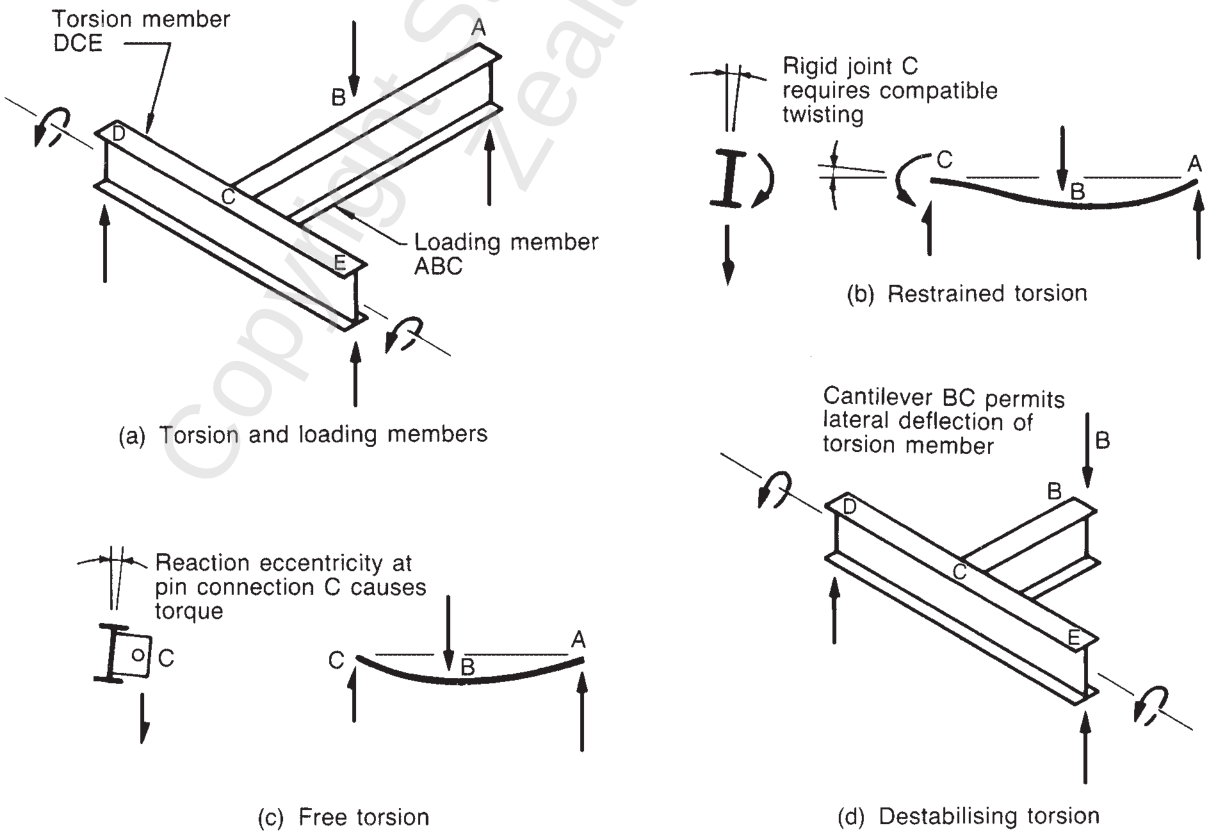
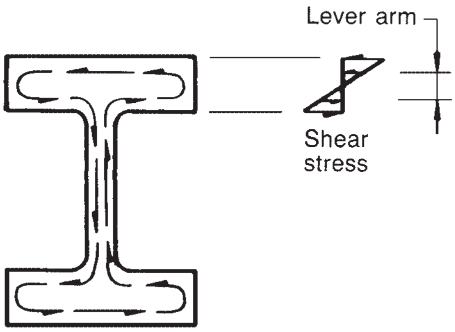
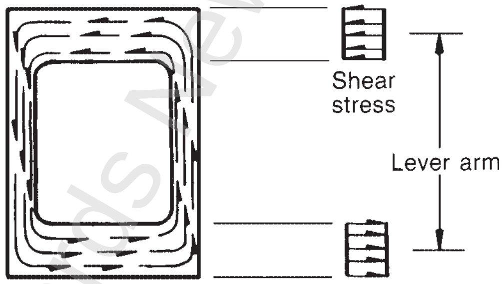


Figure C8.5.2(2) – Torsion actions

(a) Uniform torsion of an I-section member

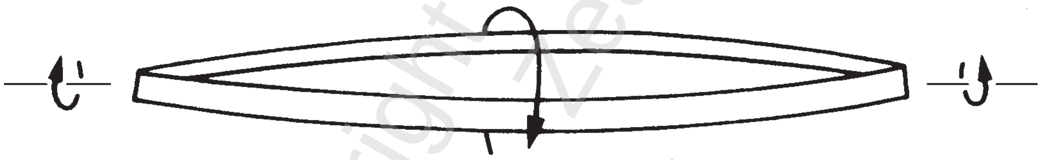


(b) Shear stresses in an I-section

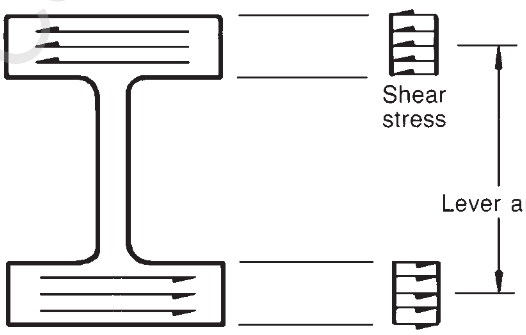


(c) Shear stresses in a box section

Figure C8.5.3.1 – Uniform torsion



(a) Non-uniform torsion of an I-section member



(b) Warping shear stresses

Figure C8.5.3.2 – Warping torsion

In closed sections, the uniform torque is the resultant of shear stresses which are almost constant across the thickness of the section wall, and which have lever arms of the order of the section width or depth (figure C8.5.3.1 (c)). Because of these large lever arms, the shear stresses are comparatively small and the torsional stiffness is usually very high.

C8.5.3.2 Warping torsion

In warping torsion, there is no significant uniform torque present because the torsion section constant is negligibly small (as it is for very thin-walled open sections).

In the warping torsion of open section members with parallel flanges, the torque is resisted by equal and opposite flange shears which arise from differential bending of the flanges, and which have a lever arm equal to the distance between the flanges (figure C8.5.3.2). Because of this large lever arm, the shear stresses are comparatively small. However, the complementary (warping) normal stresses resulting from flange bending are usually quite significant, and should not be ignored.

In some open section members, all the plate elements of the cross section meet at a common point (as in the case of angles, tees, cruciforms and narrow rectangular sections), so that there can be no torque resultant of any warping shear stresses in the elements. In such a case, the warping torque is negligibly small, and all the torque must be resisted by the uniform torque.

The warping torsion of closed section members is often ignored because the uniform torsional stiffness is very high.

C8.5.3.3 Non-uniform torsion

In non-uniform torsion, both uniform and warping torsion are present, and the applied torque is resisted by a combination of these 2 torque components. Non-uniform torsion is the general case for open section members, and particularly so for hot-rolled and welded members with parallel flanges, such as I-sections and channels. The relative contribution of uniform and warping torsion changes along the length of such members, as described in section 8 of (8.14).

C8.5.4 Analysis of torsion

C8.5.4.1 Pure torsion

(a) *Section properties*. The analytical determination of the uniform torsion section constant (J) and the warping torsion section constant (I_w) is discussed in (8.3, 8.14). Computer methods of section analysis are given in (8.8, 8.9). Summaries of equations for these properties are given in Appendix H herein, with a wider range of cross section properties covered in (8.12).

(b) *Member analysis*. The distributions of the uniform and warping torques along both determinate and statically indeterminate members may be determined by the analytical methods given in (8.3, 8.14), or by the computer method given in (8.13). Solutions for a number of loading cases are given in (8.8, 8.9).

(c) *Section stress distributions*. The stresses caused by the uniform and warping torques acting on a member cross section may be determined by the analytical methods given in (8.3, 8.14) or by the computer method given in (8.8, 8.9). Summaries of the stress distributions are given in (8.10, 8.11).

C8.5.4.2 Combined bending and torsion

(a) *First-order elastic analysis.* The first-order analysis of combined bending and torsion may be accomplished by the superposition of the results of first-order analyses of the separate effects of bending and of torsion. Thus the deformations and the stresses are obtained by appropriate additions of the values obtained from the separate analyses. For many applications, the deflections imposed by torsion will be significantly greater then those imposed by bending, especially in the case of primary torsion, and a first-order analysis is sufficient. For the rare instance where second-order effects due to combined torsion and bending must be considered, guidance is given in (b) and (c) below.

(b) *Approximate second-order analysis*. The first-order method of elastic analysis discussed above superimposes the results of independent analyses of bending and torsion. It therefore ignores any second-order components which arise from products of the first-order actions with the deflections or twists, such as the moment $M_x\Phi$ which is the minor axis component of the major axis moment M_x and which results from the twist rotation Φ of the member (figure C8.5.4.2 (b)).

One simple approximate method of second order analysis of combined bending and torsion is to add the term $M_x\Phi$ to any first-order minor axis moment M_y. Other second-order components which can exist but are not generally significant include the major axis component $M_y\Phi$, and the torque components $M_x u'$ and $M_y v'$ (figure C8.5.4.2 (b)). This approximate method is of reasonable accuracy while the major axis moment M_x is small compared with the flexural-torsional buckling moment at which the member becomes laterally unstable. When this is not the case, then the amplification method discussed in the next section will provide more accurate estimates of the second-order effects.

(c) *Amplification of the first-order effects*. When the major axis moment M_x is not small compared with the flexural-torsional buckling moment at which the member becomes laterally unstable, then the first-order minor axis deflections and the twists are amplified. These amplified deformations may be approximated by multiplying the first-order values by the amplification factor $[1/(1 - M_x/M_c)]$, in which M_c is the elastic lateral buckling moment. Similarly, the second-order minor axis bending moments and torques may be approximated by multiplying the first-order values by the same amplification factor.

C8.5.5 Design for torsion

C8.5.5.1 Pure torsion

(a) *Uniform torsion*. When the only action is that of uniform torsion, the design of an open section member is likely to be governed by deformation considerations, since the twist rotations Φ are likely to be large. For strength design, a conservative method is to limit the maximum shear stress τ_u^* predicted by elastic analysis under the design loads to $\phi(0.6f_y)$ with $\phi = 0.9$. This method is likely to be oversafe for compact I-sections, which have plastic shape factors equal to 1.5. For compact I-sections, it is therefore suggested that the 0.6 factor above be increased by 25 % to 0.75.

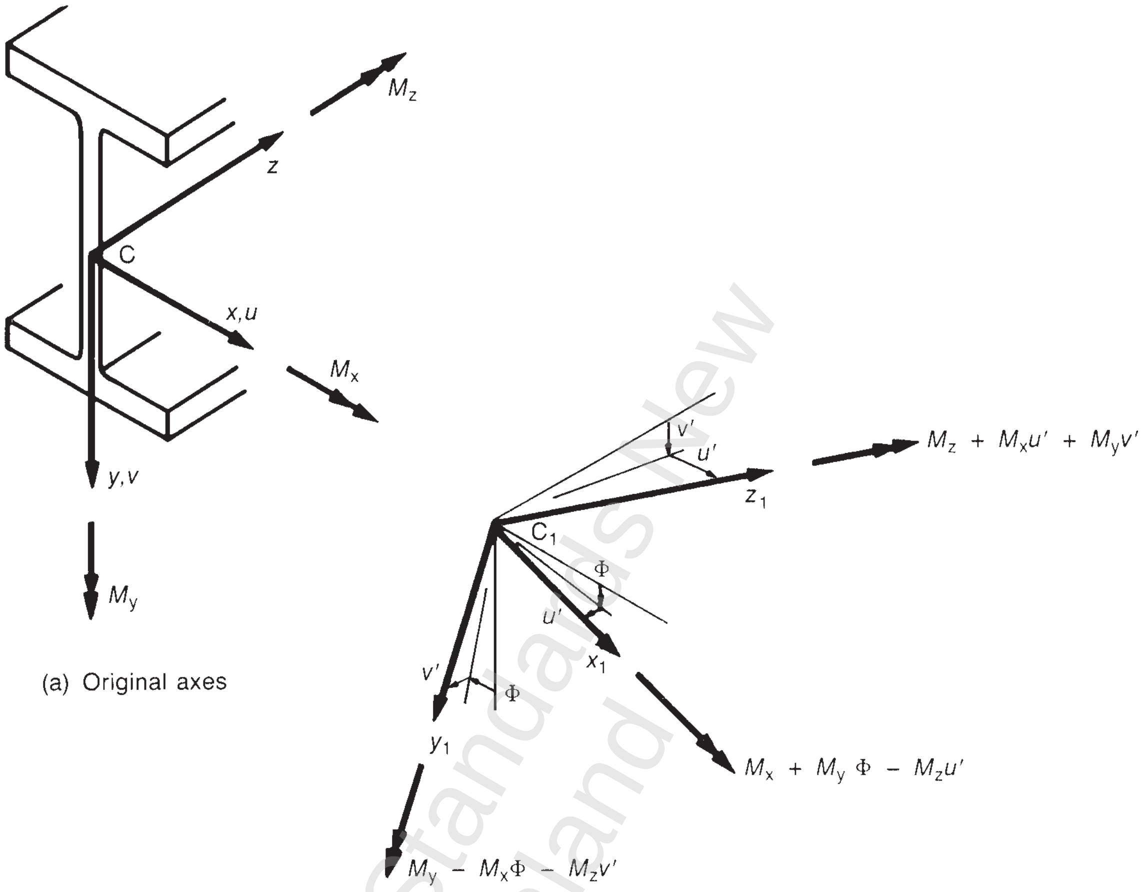


Figure C8.5.4.2 (b) – Second order moments

When the member is of closed section, the twist rotations are likely to be small. For strength design, it is suggested that the maximum shear stress τ_u^* predicted by elastic analysis under the design loads should be limited to $\phi(0.6f_y)$ for compact sections. For slender sections, significant distortions of the cross section may occur, and the effects of these should be allowed for in the analysis of the member. Designing for distortion is beyond the scope of this Commentary.

(b) *Warping torsion*. When the only action is that of warping torsion, the strength design of an open section member is usually governed by the warping normal stresses f_w^* developed by differential flange bending, since these are usually much larger than the warping shear stresses τ_w^*. A conservative method of strength design is to limit the warping normal stresses f_w^* predicted by elastic analysis under the design loads to ϕf_y with $\phi = 0.9$. This method is likely to be oversafe for the flanges of compact *I*-sections which have plastic shape factors equal to 1.5. For compact *I*-sections, it is therefore suggested that f_y be increased by 25 % to $1.25f_y$.

(c) *Non-uniform torsion*. In non-uniform torsion, there are usually significant shear stresses τ_u^* arising from the uniform torque and significant normal stresses f_w^* arising from the warping torque. However the maximum values of these usually occur at different cross sections along the member, and at different locations in the cross section. Because of this, it is often sufficient to ignore any interaction between the shear and normal stresses and to design separately for these using the methods suggested above. The distribution of warping and uniform torque along an open *I*-section member is described in detail in (8.14).

When it is found that there are significant stresses of both kinds occurring at the same point, then a circular interaction curve of the type:

$$(f_w^*/1.25\,\phi f_y)^2 \quad + \quad (\tau_u^*/0.75\,\phi f_y)^2 \quad \leq \quad 1.0$$

might be used for compact *I*-section members. For other members, this should be replaced by:

$$(f_w^*/\phi f_y)^2 \quad + \quad (\tau_u^*/0.6\phi f_y)^2 \quad \leq \quad 1.0$$

C8.5.5.2 Combined bending and torsion

For the strength design of members subjected to combined bending and torsion, the occurrence of coincident normal stresses due to bending and warping may be allowed for by replacing the design moment M^* at the section by an equivalent moment, M_{eq}^*.

$$M_{eq}^* \quad = \quad M^* \quad + \quad (f_w^*\, M_s/1.25\, f_y)$$

for compact *I*-section members, and by reducing the value of 1.25 to 1.0 for other members.

Similarly, the occurrence of coincident shear stresses due to uniform torsion and bending may be allowed for by replacing the design shear V^* on the cross section element by an equivalent shear, V_{eq}^*.

$$V_{eq}^* \quad = \quad V^* \quad + \quad (\tau_u^*\, V_w/0.75\, f_y)$$

for compact *I*-section members, and by reducing the value of 0.75 to 0.6 for other members.

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NOTES

C9 CONNECTIONS

C9.1 GENERAL

C9.1.1 Requirements for connections

The clause draws the distinction between connection components and connectors because the strength reduction factors for each are different (see table 3.3), and because most of section 9 deals with the design of the connectors. Only 9.1.9 mentions connection components.

It is essential that the connection design be consistent with the assumptions made in the method of structural analysis selected from section 4 and the structural form chosen from section 4.

The basic requirement is for the design method to demonstrate that the connections have the capacities to transmit the design actions calculated from the analysis performed and to satisfy any requirements for ductility.

C9.1.2 Classification of connections

Clause 9.1.2 details the design requirements for each connection classification. Two of the 3 forms of construction, rigid and simple, have appeared in the 1989 edition of this Standard. The third form, semi-rigid construction, is introduced in this edition, with restrictions on its use in a seismic-resisting system.

Semi-rigid connection design demands a knowledge of the true moment-rotation behaviour of the connection both to enable a frame analysis to be carried out and to allow the design of the connection itself.

Practical simple connections will transmit some bending moment to the supporting members. This may adversely affect the structure, in which case it must be allowed for in the design of those members so affected. A method of allowing for this is given in 4.3.4. Loss of rigidity in a rigid connection will cause a redistribution of bending moments in a frame, which may also adversely affect some members.

The rotation behaviour of practical simple connections is most commonly provided for by allowing one or more elements in the connection to deform appreciably. A discussion of the behaviour of typical flexible connections is contained in (9.1, 9.27). It is important that the detailing of the connection be such as to ensure that this assumed deformation actually takes place. Suitable detailing of flexible connections may be found in (9.28, 9.2).

C9.1.3 Design of connections

This clause nominates the basic requirements that any design model for any connection must satisfy if the model is to be acceptable. Not all published design models for connections satisfy all of these requirements. Suitable design models for Australian use, which are also generally suitable for New Zealand use, may be found in (9.1) and guidance on using these and other design models in seismic applications is given in section 10 of (9.27). These design models satisfy the requirements of the clause and are all supported by experimental evidence as required by the clause. Section 10 of (9.27) also provides guidance on adapting (9.1) to New Zealand use.

The reference to residual actions due to bolt installation not requiring consideration refers to tensioned bolts, which introduce local clamping actions in steel frames and may result in some local actions in individual members. The effects throughout the frame due to local actions or distortions are not significant, nor readily amenable to calculation, and may be ignored.

C9.1.3.5

Amd 2 Oct. '07

Unstiffened plates designed to transfer compression forces fail in a sway mode. If there is any eccentricity in the path of the compression force transfer, as will occur due to construction out of tolerance or when there is eccentricity between the plates transferring the compression, the connection will commence sway as soon as compression load is applied. The peak compression load that can be reached will be sensitive to construction tolerances and second-order effects.

For eccentrically loaded cleats in compression, involving one plate from the supporting member bolted into one plate from the supported member, a design procedure is presented in *Eccentric Cleats in Compression and Columns in Moment-resisting Connections*, HERA Report R4-142, Manukau City, New Zealand, 2007.

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C9.1.4 Minimum design actions on connections

The design actions nominated in the clause are the actions for the ultimate limit state for connections not subject to earthquake loads or effects. Minimum ultimate limit state design actions on connections subject to load combinations including earthquake loads are given in 12.9.2.

The provisions are intended to ensure that, even for a member subject to a low level of design actions, each connection has at least a minimum capacity.

The action to be designed for is the greater of the calculated design action or the minimum specified in (i) to (vii), as appropriate. The minimum is expressed as a factor times the design capacity (ϕR_u) for the minimum size of member required by the ultimate limit state. Hence, if a member is increased in size above the minimum size for whatever reason (rationalization of member sizes, slenderness considerations), it is only necessary to use the design capacity of the minimum size for the purposes of the clause. For columns which may be subject to large compressive forces and only minor tensile forces, any splice has to be designed both for the specified value for the minimum member size required to resist the compression, and for the specified value for the minimum member size required to resist the tension. Note carefully that the same philosophy does not extend to connections subject to earthquake loads or effects, where the minimum actions given by 12.9.2 are a function of the actual member size used.

The provisions for splices are covered by Items (iv) to (vii) of the clause. The basic minimum requirement of 0.3 times the member capacity for tension members and flexural members has been reduced from the requirement of 0.5 times in the 1989 edition of the Standard. These provisions, and those for compression members, are to prevent the situation of small splice elements being used to connect relatively thick plates. In the event of an excessive load, the weak splice elements will form a potential region of high deformation.

The full member design capacity required as the minimum design action at the end connection of a threaded rod used as a tension member comes from (9.3), which cites experiments where the pretensioning induced by a turnbuckle exceeded the nominal yield capacity at the threaded section. The same philosophy has been extended, in the 1997 edition, to all such threaded rods, as most of these are tightened on site to avoid sag.

For splices located between points of effective lateral support, the design bending moment (M^*) is taken as (see figure C9.1.4):

$$M^* = \begin{pmatrix} \text{amplification} \\ \text{factor } (\delta) \end{pmatrix} \times \begin{pmatrix} \text{design compression} \\ \text{force } (N^*) \end{pmatrix} \times \begin{pmatrix} \text{out-of-straightness} \\ \text{of member} \end{pmatrix}$$

Since 14.4.4.1 permits a maximum out-of-straightness of $L_s/1000$ for compression members, the design bending moment (see figure C9.1.4) becomes:

$$M^* = \frac{\delta N^* L_s}{1000}$$

$$M^* = \frac{\delta N^* L_s}{1000}$$

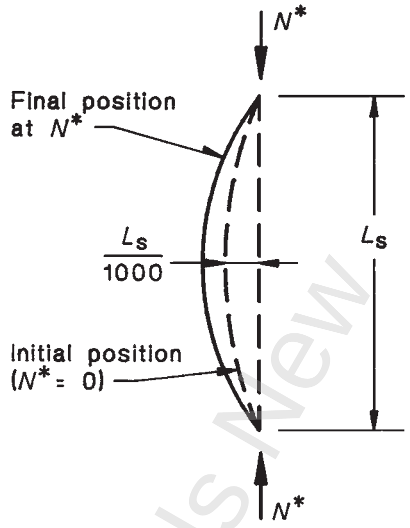

Figure C9.1.4 – Design moment at a column splice

C9.1.4.1 Minimum design actions on connections not subject to earthquake loads or effects

Amd 2 Oct. '07

For structures which incorporate a rigid floor diaphragm, that is composite with the supporting beams and in direct contact with the columns, the requirement of new sub-clause 9.1.4.1(c) can be deemed to be satisfied.

C9.1.5 Intersections

This clause states what is accepted good practice in detailing, and indicates what is to be done if the accepted good practice situation cannot be achieved.

Slight eccentricities between the centroidal axes of members and the centroidal axes of end connections have long been ignored as having negligible effect on the static or seismic strength of members, but it is known that these eccentricities can have deleterious effects on the fatigue strength (9.18). This is covered in section 10 of this Standard and, in more detail, in (10.8).

Three points should be made:

(a) Consideration should be given to the practicality of fabrication, inspection and erection in all connection detailing.

(b) Some eccentricity of centroidal axes may be desirable in achieving a practical connection detail (9.27).

(c) Some strange and doubtful connections are sometimes constructed due to too much emphasis being placed on centroidal concentricity, to the detriment of providing simple and direct transfer paths for the actions. This should be discouraged.

C9.1.7 Combined connections

The requirements of this clause are based on the fact that the action tends to be transferred by the stiffest fastener in the connection. Further discussion may be found in (9.4, 9.18). The following comments come from (9.18):

'Welds will not share the load equally with mechanical fasteners in bearing-type connections. Before ultimate loading occurs, the fastener will slip and the weld will carry an indeterminately larger share of the load.

Accordingly, the sharing of load between welds and bolts in a bearing-type connection is not recommended. For similar reasons, bolts and rivets should not be assumed to share loads in a single group of fasteners.

For high-strength bolts in slip-resistant connections to share the load with welds, it is advisable to properly tension the bolt before the weld is made. Were the weld to be placed first, angular distortion from the heat of the weld might prevent the faying action required for development of the slip-resistant force. When the bolts are properly tensioned before the weld is made, the slip-resistant bolts and the weld may be assumed to share the load on a common shear plane. The heat of welding near bolts will not alter the mechanical properties of the bolt.'

It should be noted that combination of fasteners as defined herein does not refer to connections such as shear plates for beam-to-column connections, which are welded to the column and bolted to the beam flange or web, and other comparable connections.

C9.1.7.3
The sharing of loads between bolts and welds in the same element of a seismic-resisting connection is not permitted, as such details have exhibited poor performance in recent major earthquakes (9.32).

C9.1.8 Prying forces
The concept of prying forces and methods of allowing for them in design are discussed in detail in (9.1, 9.4, 9.10 and 9.27).

C9.1.9 Connection components
The connection components other than connectors are treated in design as members subject to tension, compression, bending and shear as appropriate, using the strength reduction factor or the factor as appropriate for connection components given in table 3.3 and the nominal capacities given in sections 5 to 8.

Note that a gusset plate under compression loading is designed as a column member, with $\alpha_b = 0.5$ and $r_y = 3.46\, t_p$, where r_y = the minor principal y-axis radius of gyration and t_p = gusset plate thickness. The net area of gusset plate which transmits axial force (compression or tension) may be determined in accordance with the recommendations of section 10 of (9.27).

C9.1.10 Deductions for fastener holes
This clause applies to tension members, compression members, beams and connection components.

The requirements for staggered holes have been in this Standard and many other standards for a number of years and are due to work by Cochrane in 1922 (9.5). The method specified in this clause was originally based on research on holes in riveted members subject to tension.

C9.1.11 Hollow section connections
Suitable methods for designing hollow section connections in compliance with this clause may be found in (9.6 to 9.8) inclusive, or are referenced in (9.27, 9.28).

C9.2 DEFINITIONS
(No Commentary.)

C9.3 DESIGN OF BOLTS

C9.3.1 Bolts and bolting category

The bolting categories are those used in Australian and New Zealand publications (e.g. 9.1, 9.2, 9.9, 9.27, 9.28) and have been in use for some years in this country. They clearly identify both the type and property class of bolt to be used and the method of installation in one label.

The bolts nominated are those complying with the nominated standards of section 2. Installation methods are detailed in section 15 and in section 17 of (9.29).

Bolt areas which will prove useful in the remainder of 9.3 are given in table C9.3.1. The table applies to bolts with threads complying with AS 1275, and areas are given for common bolt diameters.

Table C9.3.1 – Bolt areas for bolts with ISO metric screw threads to AS 1275

Nominal diameter (mm)	Pitch (mm)	Areas (mm^2)		
d_f	p	A_c core	A_s tensile stress	A_o shank
12	1.75	76.2	84.3	113
16	2	144	157	201
20	2.5	225	245	314
22	2.5	278	303	380
24	3	324	353	452
30	3.5	519	561	706
36	4	759	817	1016

Clause 9.3.1.2 extends the design provisions of 9.3 to property class 10.9 and 12.9 bolts and nuts. Contact HERA for advice on the tensioning of property class 10.9 bolts and nuts; property class 12.9 are for snug tight use only.

C9.3.2 Bolt ultimate limit states

C9.3.2.1 Bolt in shear

Specifications for bolt manufacture do not usually require that that bolt be tested in shear as part of the quality control tests carried out by the manufacturer. Bolt shear strength is not normally stated in bolt standards such as AS 1111 and AS/NZS 1252. Amd 2 Oct. '07

The shear strengths of bolts have been obtained by a number of different investigators who have subjected bolts to double shear induced through plates subjected either to tension or compression. Kulak, Fisher and Struik (9.4) have examined the available data and concluded that, for ASTM A325 and A490 bolts (Property Class 8.8 bolts to AS/NZS 1252 are equivalent to ASTM A325 bolts), the average shear strength (f_{vf}) is 62 % of the tensile strength of the bolt (f_{uf}), so that:

$$f_{vf} = 0.62\, f_{uf}$$

Tests on bolted joints have also indicated that the level of any initial tension in the bolt has no significant effect on the ultimate shear strength. The factors responsible for this are detailed in (9.4). Consequently, the shear strength of a bolt which is snug tight (Category/S) is the same as that of the same class of bolt which is fully tensioned (Categories/T). ➤

The shear strength of a bolt is also directly proportional to the available shear area of the bolt, this being taken as the core area (A_c) when threads intercept the shear plane, and the plain shank area (A_o) when the plain shank intercepts the shear plane.

Hence, the nominal shear capacity of a single bolt (V_f) is given by:

V_{fn} = $0.62\ A_c\ f_{uf}$ for threads intercepting one shear plane; and

V_{fx} = $0.62\ A_o\ f_{uf}$ for a plain shank intercepting one shear plane.

The expression for the nominal shear capacity (V_f) given in this clause allows the capacity of a bolt intercepting a number of shear planes of each type to be determined. Typically, n_n and n_x will be either 0 or 1, depending on the application.

For bolted lap splice connections of the types shown in figure C9.3.2.1 (1) which are subject to applied force which gives rise to shear forces on the bolts, both theoretical and experimental studies have shown that the length of the joint is an important parameter influencing the total strength of the joint (9.4, 9.10). A reduction factor (k_r) has been introduced, accordingly, to account for this effect. Typical applications where a value of k_r less than 1.0 might be required include plate splices, flange splices, and gusset connections.

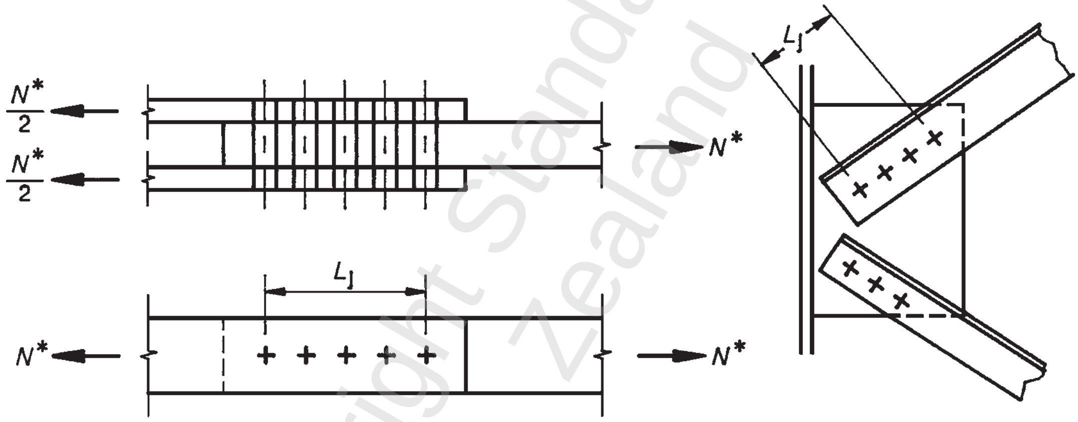

Figure C9.3.2.1 (1) – Lap joint and gusset connection

Depending on a number of factors related to connection geometry (including joint length) and material properties, either a simultaneous shearing of all bolts occurs or a sequential failure characterised by progressive unbuttoning takes place at failure of the lap joint, provided net section failure is avoided. In general, short lap joints exhibit the former failure mode, while for longer lap joints a decrease in the average bolt shear force at failure is detected. This occurs because the end bolts in the joint are more heavily loaded than the central bolts, and these end bolts may fail before a full redistribution of bolt forces occurs.

Kulak et. al. (9.4) propose using a reduction factor (k_r) for lap splice joints as follows:

k_r = 1.0 for $L_j \leq$ 1250 mm
= 0.8 for $L_j >$ 1250 mm

where L_j is the joint length (see figure C9.3.2.1 (1)).

This is a very simplistic approach with obvious limitations, and has a very sharp cut-off with no transition, as shown in figure C9.3.2.1 (2).

The ECCS (9.11) has proposed the following relationship between the reduction factor (k_r) and the joint length (L_j):

$$
\begin{aligned}
k_r &= 1.0 && \text{for } L_j \le 15d_f \ (d_f = \text{bolt diameter}) \\
&= 1.075 - \frac{L_j}{200d_f} && \text{for } 15d_f < L_j \le 65d_f \\
&= 0.75 && \text{for } L_j > 65d_f
\end{aligned}
$$

The ECCS and Kulak's proposals are plotted against the theoretical results from (9.4) in figure C9.3.2.1 (2).

A simplified version of the ECCS relationship has been chosen for use in this clause by taking d_f = 20 mm, as being a reasonable approximation to the theoretical results without undue complication.

Note that the shear capacity of a bolt (V_f) may have to be reduced by the presence of filler plates in accordance with 9.3.2.5.

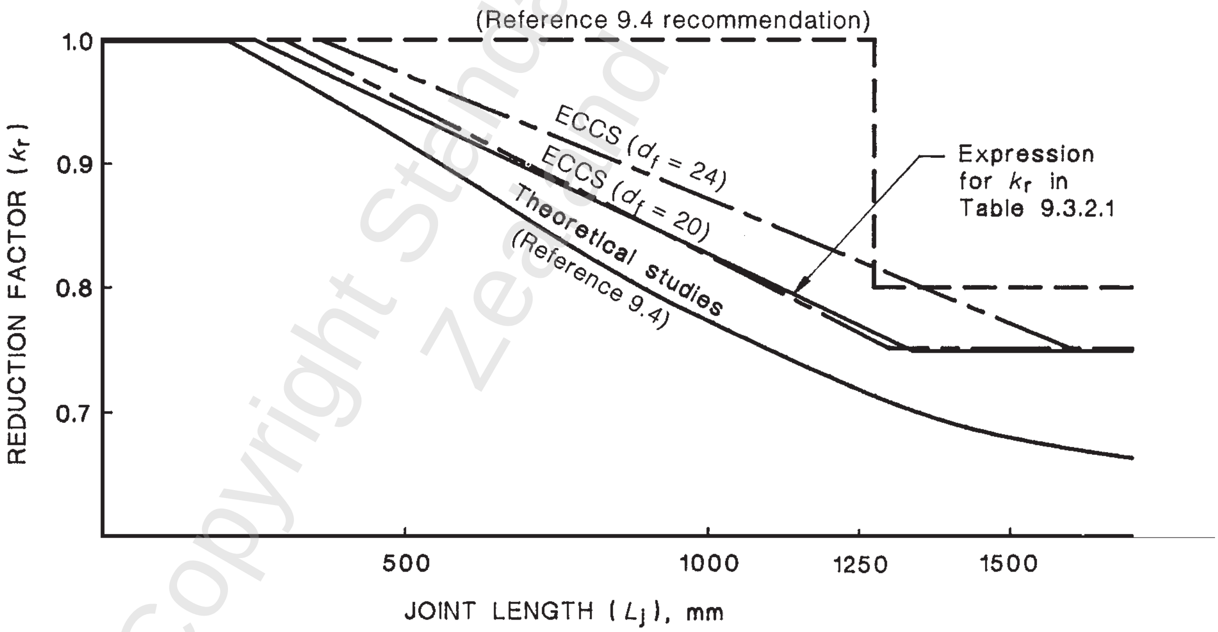

Figure C9.3.2.1 (2) – Relationship between the modification factor (k_r) and joint length (L_j)

No allowance need be made for long grips however, as a result of research summarized by Kulak, Fisher and Struik in (9.4). For the bearing-type joints covered by 9.3.2.1, they note that "for joints with up to 150 mm of grip, test results are in close agreement with the analytical solution. Joints with larger grips and longer bolts tend to give higher ultimate loads than predicted".

Longer grips could be expected to give lower ultimate loads due to bending of the bolts, but the bending deformation which occurs causes failure along an inclined shear plane of larger area, thus increasing the ultimate load and deformation capacity of the bolt.

C9.3.2.2 Bolt in tension

The behaviour and strength of a bolt subject to an axial tension is governed by the performance of the threaded part of the bolt. The relevant Standards specify as part of their requirements for the mechanical properties of the bolt:

(a) Minimum tensile strength;

(b) Minimum yield stress; and

(c) Proof stress.

In order to determine whether the specified mechanical properties are met, these Standards require direct tension tests on full size bolts as a quality control mechanism. The tensile capacity of a bolt (N_{tf}) is specified therein to be equal to:

$$N_{tf} = A_s f_{uf}$$

This expression is used in this clause for the nominal capacity of a bolt under tension.

Loading a bolt in tension, after preloading by tightening the nut (8.8/T categories), does not significantly decrease the ultimate tensile strength of the bolt (9.4). Apparently, the torsional stresses induced by turning the nut have a negligible effect on the tensile strength, and Kulak, Fisher and Struik (9.4) argue that tensioned bolts can sustain direct tension loads without any significant apparent reduction in their tensile strength.

C9.3.2.3 Bolt subject to combined shear and tension

Tests conducted on bolts subject to simultaneous shear and tension forces indicate that neither bolt diameter, bolt type nor ply material type have a significant effect on the ultimate load capacity of the bolt.

Test results are best approximated by an elliptical interaction relationship (after Kulak, Fisher and Struik, (9.4)), as adopted in this clause. The nominal tension capacity and the nominal shear capacity used in the denominators of the interaction equation are the respective nominal capacities of the bolt under the separate individual loads, with the nominal shear capacity being dependent upon the location(s) of the shear plane(s), as for a bolt subject to shear force alone.

C9.3.2.4 Ply in bearing

The bearing stress and edge distance requirements for plies in bolted joints are discussed in detail in (9.4 and 9.12).

Research generally indicates that shearing-tearing failure with considerable 'piling-up' of the ply material in front of a bolt (commonly referred to as a local bearing failure) occurs at a nominal bearing stress within the range 4.5 f_{yp} to 4.9 f_{yp}. Hence, using the lower limit and the conventional nominal bearing area ($d_f \times t_p$) leads to:

$$V_{bu} = 4.5\, f_{yp}\, d_f\, t_p$$

where V_{bu} is the ultimate bearing capacity of a ply. For most structural steels, $f_{yp} \approx 0.7\, f_{up}$ so that this limit is equivalent to a nominal bearing capacity of a ply (V_b) given by:

$$V_b = 3.2\, f_{up}\, d_f\, t_p, \quad \text{as used in the clause.}$$

Such a failure mode only occurs for relatively long end distances (a_e) in the direction of the applied force (generally $a_e > 3.5\, d_f$), as shown in figure C9.3.2.4 (1) (a).

For relatively short end distances (a_e) in the direction of the applied forces, failure occurs by longitudinal shearing of the connected ply along 2 practically parallel planes separated by a distance equal to the hole diameter (see figure C9.3.2.4 (1) (b)). This type of failure is commonly referred to as plate tearout failure. An important criterion in determining the nominal bearing stress for such a failure is the ratio a_e/d_f.

Kulak, Fisher and Struik (9.4) proposed the following lower bound equation:

$$\frac{f_{pu}}{f_{up}} = 1.40\,\frac{a_e}{d_f} - 0.70 \quad \text{or a simpler form:} \quad \frac{f_{pu}}{f_{up}} = \frac{a_e}{d_f}$$

The simpler form has been adopted as the design criterion, giving the expression for the nominal bearing capacity as:

$$V_b = f_{pu}\, d_f\, t_p\text{; or}$$

$$V_b = a_e\, t_p\, f_{up} \text{ on substituting } f_{pu} = \frac{a_e}{d_f} f_{up}\text{, as used in the clause.}$$

In order to allow for the use of slotted and oversize holes, a_e is defined as shown in figure C9.3.2.4 (2).

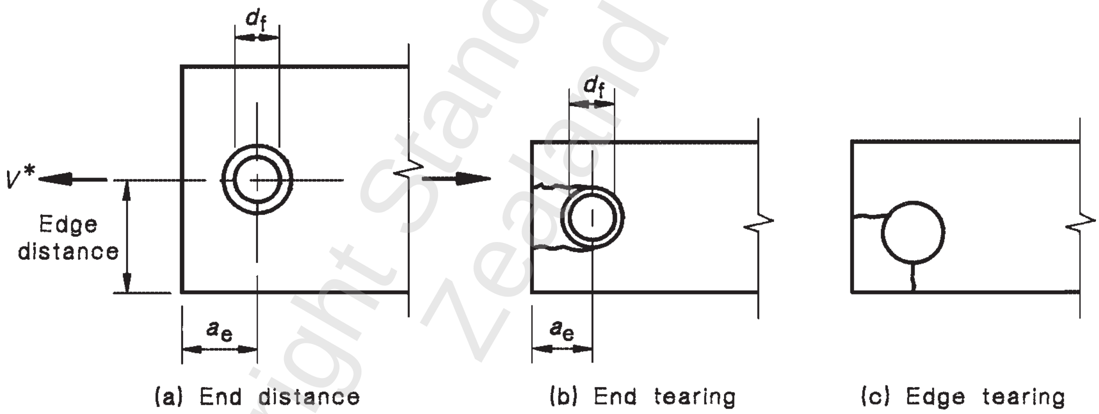

Figure C9.3.2.4 (1) – Failure by longitudinal shearing

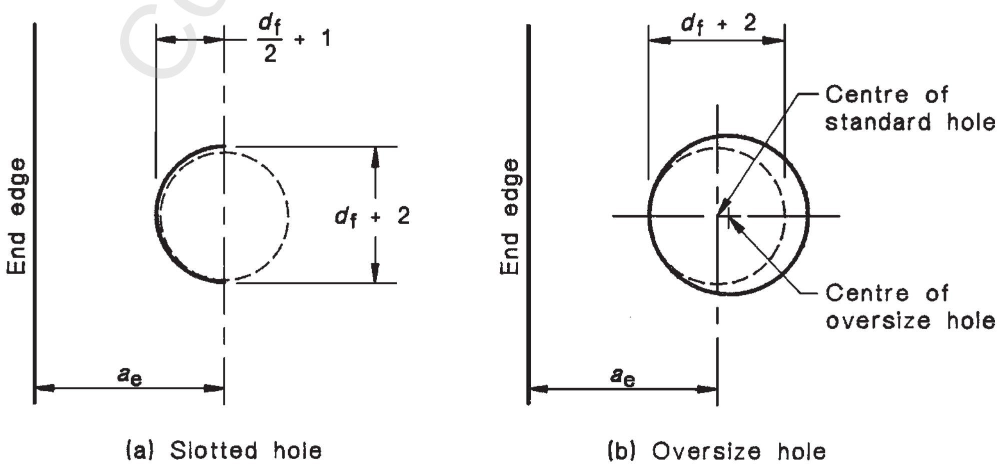

Figure C9.3.2.4 (2) – a_e for slotted and oversize holes

Detailed pull-out tests on bolts in shear have been undertaken for inelastic applications, to ensure that under such applications (e.g. earthquake loads) the form of bolt failure shown in figure C9.3.2.4 (1) is either suppressed or only occurs once the required ductility is attained in the connected member (12.25). The edge distances needed to achieve this are presented in 12.9.4.4 and are less than those required to achieve the value of V_b from equation 9.3.2.4 (1). The seismic requirements allow for significant ovalling of the bolt hole to occur under extreme inelastic demand, but are part of a package of connection design and detailing provisions formulated to ensure the critical element subjected to significant inelastic demand is never the bolt. This allows lower edge distances to be specified than those needed to develop the full nominal bearing capacity of the ply.

Bearing under bolts through quenched and tempered splice plates (equation 9.3.2.4(3)) is based on the general bearing yield capacity requirements of webs (5.13.3), because of a lack of research data on such grades of ply in bearing. The conversion from f_{yp} to f_{up} is based on *a* f_u/f_y ratio of 1.15, which is typical for this grade of steel.

C9.3.2.5 Filler plates

The provisions of the clause come directly from research reported by Kulak, Fisher and Struik (9.4). They report in Chapter 10.3 of (9.4) the following:

'For bearing-type joints, where the load is transmitted by shear and bearing of the bolts, loose fillers can be used as long as excessive bending of the bolts does not occur. It is suggested that single loose fillers up to 0.25 inch (6 mm) thick can be used without considering a reduction in bolt shear strength. If the loose filler thickness exceeds this, the bolt shear strength capacity should be reduced. A reduction of 15 % would be appropriate for a loose filler thickness of 0.75 inch (19 mm).'

C9.3.3 Bolt serviceability limit state

C9.3.3.1 Design

A limited range of bolted connections needs to be designed in such a manner that slip is limited under the serviceability loads. Such a design criterion has been reviewed elsewhere (9.13 to 9.15) and has long been a design condition for 8.8/TF category bolts.

The maximum amount of slip that can occur in connections that are not classified as slip-critical is limited theoretically to 2-3 mm. In most practical cases, however, the real magnitude of any slip would probably be less because the inaccuracies in the location of holes within a pattern of bolts would usually cause one or more bolts to be in bearing in the initial unloaded condition. Furthermore, in statically loaded structures, even with perfectly positioned holes, the usual method of erection would cause the weight of the connected elements to put the bolts into direct bearing at the time the member is supported on loose bolts and the lifting crane is unhooked. Subsequent additional gravity loading could not cause additional vertical connection slip.

Connections classified as needing slip to be limited include those cases where slip theoretically could exceed 2-3 mm, and thus possibly affect the serviceability of the structure by excessive distortion or reduction in strength or stability, even though the resistance to fracture of the connection may be adequate. Also included are those cases where slip of any magnitude should be prevented, as for example in connections subject to fatigue loading. Slip in connections subject to seismic loading is a special case and is covered by 9.1.6.2(c) and 12.9.6.

There is considerable variation in both μ_s and N_{ti}. The initial bolt tension (N_{ti}) depends on the bolt class and the method of installation. It is very variable and may be quite low for Category /S because no controlled method of tensioning is employed, and only Category /T with a Property Class 8.8 structural bolt, employing a controlled installation procedure as in 15.2.5, can be depended upon to give a reliable level of resistance to slip under the serviceability loads.

The value of μ_s is essentially a function of the surface condition of the interfaces, but also varies with the bolt class and the method of installation. The value of μ_s should be based on a series of tests conducted using the method specified in Appendix K. The value quoted in 9.3.3.2 for μ_s for bare steel surfaces is generally based on long experience, but is supported by figures quoted in (9.4), which reports a mean slip coefficient of 0.33 for clean mill scale surfaces. The effect of corrosion protection on the value of μ_s is also reviewed in (9.4).

No correction is required for long grips as a result of research reported by Kulak, Fisher and Struik in (9.4), who note that 'the grip length of bolts does not have a noticeable influence on the behaviour of friction-type joints. The only point of concern is the attainment of the desired clamping force.' The attainment of the specified initial bolt tension is ensured by increasing the amount of turn required in the part-turn of nut method, as a function of bolt length (see 15.2.5.2).

No correction is required for the presence of filler plates, again as a result of research reported by Kulak, Fisher and Struik in (9.4), who note that 'For slip-resistant joints, loose fillers with surface conditions comparable to other joint components are capable of developing the required slip resistance. Slip-resistant joints do not require additional fasteners when filler plates are used. The fillers become integral components of the joint, and filler thickness does not significantly affect the joint behaviour.'

The k_h factor for different hole types has been introduced to compensate for the loss of clamping area in the vicinity of the hole when other than a standard hole in employed. The clamping action, on which the frictional resistance is dependent, is highly localized around the bolt, and a loss of area in the zone of high clamping force affects the slip resistance at the interface. The values for k_h are based on recommendations contained in (9.4). The different hole types permitted in this Standard are defined in 14.3.5.2.

C9.3.3.2 Contact surfaces

The condition of the contact surfaces governs the value of the slip-factor, μ_s. Typical values for some common surface coatings obtained from tests (9.4) are shown in table C9.3.3.2.

Table C9.3.3.2 – Summary of slip factors (μ_s)

Surface treatment	Indicative slip factor (μ_s)
Uncoated	
Clean as-rolled	0.35
Flame cleaned	0.48
Abrasive blasted	0.53
Painted	
Red oxide zinc chromate	0.11
Inorganic zinc silicate	0.50
Hot-dip galvanized	
Clean as-galvanized	0.18
Lightly abrasive blasted	0.30 – 0.40

The values of the slip coefficient given in table C9.3.3.2 are indicative only, and actual values will vary within a generic type according to the formulation of the surface coating used. It is unrealistic to assign one value for a generic surface coating system, and major reliance has to be placed on results from the method of test specified in Appendix K.

Research (9.4) has indicated that the slip coefficient for galvanized surfaces is significantly improved by treatments such as hand (not power) wire brushing or light abrasive blasting, provided the treatment is controlled to achieve the necessary roughening of the surface.

Evidence conforming with the test method of Appendix K should be obtained from the manufacturers of a corrosion protection system in order to confirm the values to be used for corrosion protected surfaces.

C9.3.3.3 Combined shear and tension

When tensioned high-strength structural bolts of 8.8/TF category are subjected to applied tensile forces, the clamping forces are reduced, and a proportional reduction in the shear transferred by friction may occur. Connections of the type shown in figure C9.3.3.3 (a) suffer no overall loss of frictional resistance because the bolt tension produced by the moment, which leads to a loss of clamping force in one part of the connection, is coupled with a compensating compressive force on the other side of the connection. In a connection such as that in figure C9.3.3.3 (b), however, all the bolts receive applied tensions, which reduces the initial clamping force at the contact surface.

A linear interaction equation is used in this case, rather than the parabolic relationship used for the ultimate limit state (see 9.3.2.3), following the recommendations in (9.17).

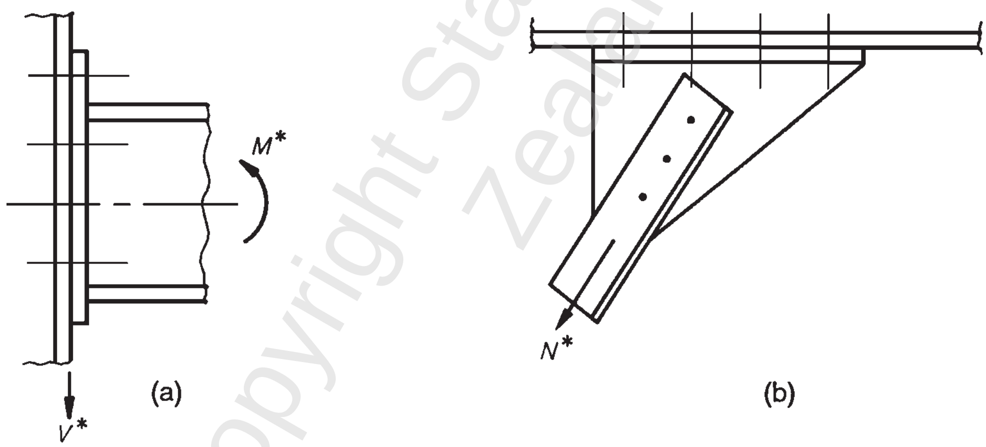

Figure C9.3.3.3 – Combined shear and tension

C9.4 ASSESSMENT OF THE STRENGTH OF A BOLT GROUP

C9.4.1 Bolt group subject to in-plane loading

The design assumptions listed in this clause are the conventional assumptions which are made for the analysis of bolt groups loaded by in-plane eccentric shear forces, in order to derive equations of equilibrium which can be solved for the bolt forces. For the bolt group shown in figure C9.4.1, force and moment equilibrium leads to 3 equations:

$$\sum V_n^* \cos\theta_n + V^* = 0 \quad \text{(Eq. C9.4.1(1))}$$

$$\sum V_n^* \sin\theta_n = 0 \quad \text{(Eq. C9.4.1(2))}$$

$$\sum V_n^* r_n + V^* (e - x_e) = 0 \quad \text{(Eq. C9.4.1(3))}$$

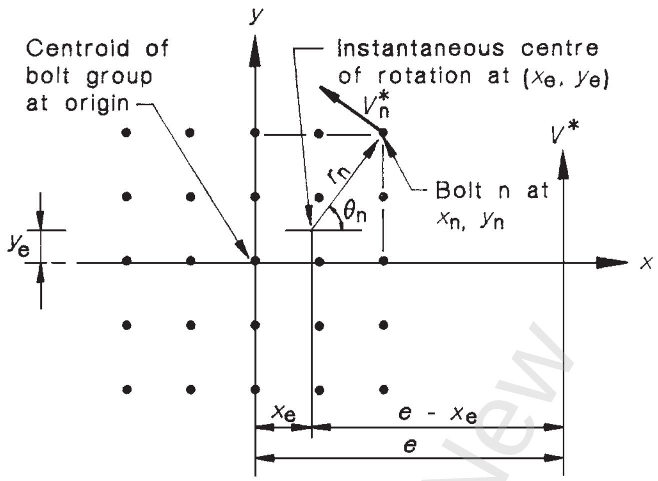

Figure C9.4.1 – Bolt group design actions

To solve these equations in order to evaluate the design shear capacity of the connection requires a further equation. The required equation depends on the analysis method used, which may be either:

(a) *Elastic analysis*. In this analysis it is assumed that the relationship between the force V_n^* on any bolt is linearly related to its distance (r_n) from the centre of rotation, and also to the force V_{of}^* on the bolt furthest from the centre of rotation, by the equation:

$$V_n^* = \frac{r_n}{r_{max}} V_{of}^* \quad \text{(Eq. C9.4.1(4))}$$

Solving equations C9.4.1(1) to C9.4.1(4) leads to the explicit expressions for the centre of rotation (x_e, y_e) and the maximum shear force (V_{of}^*) as follows:

$$x_e = \frac{-(\Sigma y^2{}_n + \Sigma x^2{}_n)}{n_b e} \quad \text{(Eq. C9.4.1(5))}$$

$$y_e = 0 \quad \text{(Eq. C9.4.1(6))}$$

$$V_{of}^* = \frac{-V^* r_{max}}{\Sigma(x_n - x_e)} \quad \text{(Eq. C9.4.1(7))}$$

where n_b is the number of bolts in the group (assumed of equal size).

This clause permits the design actions to be considered separately and then superimposed. In the case shown in figure C9.4.1, a design action (V^*) acting at the bolt group centroid and a couple (V^*e) may be considered separately and the calculated bolt forces superimposed.

Under a design action (V^*) acting at the bolt group centroid, the force on each bolt (V_{bv}^*) is given by:

$$V_{bv}^* = \frac{V_n^*}{n_b}$$

➤

Under a design action couple (V^*e), the force on each bolt (V^*_{bc}) is given by:

$$V^*_{bc} = \frac{(V^*e)r_n}{(\Sigma x^2_n + \Sigma y^2_n)}$$

(b) *Plastic analysis*. In the plastic analysis method, it is assumed that all bolts not at the centre of rotation are deformed sufficiently to become fully plastic, and that all transmit the same force at failure. In this case it is not possible to solve equations C9.4.1(1) to C9.4.1(3) for x_e, y_e and V^*e explicitly, and an iterative method must be used to evaluate these variables. This generally requires the use of a computer to obtain a solution.

(c) *Other methods*. Several authors have attempted to measure the relationship between the relative displacement of the connected components and the force developed by the bolt. They then use this relationship in solving equations C9.4.1(1) to C9.4.1(3). The method used to obtain a solution is again an iterative one, generally requiring the use of a computer to provide a satisfactory solution. Unfortunately, the relationship referred to above between the relative displacement and the bolt force is dependent on a number of factors including:

(i) The thickness of the connected components; and

(ii) The yield strengths of these components.

Because much of the deformation which occurs in realistic cases is due to bearing failure of the connected material, no simple relationship or single definition of this relationship is available.

Traditionally, design has been done using the elastic method of analysis, which is readily amenable to hand solution. More modern methods have become popular, as described in (9.4, 9.18). A review of available test results and methods of analysis used for bolt groups eccentrically loaded in-plane (9.16), has shown that elastic analysis, with its ease of calculation, provides a practical approach to the evaluation of the strength of bolt groups, and that there is little benefit arising from the use of the more complicated plastic analysis.

C9.4.2 Bolt group subject to out-of-plane loading

Methods of analysis for individual bolts in tension and bolts in simple hanging type tension connections may be found in (9.4 and 9.10). Methods of analysis for bolt groups subject to bending moments causing bolt tensions may be found in (9.1, 9.27) (for specific connection types) and (9.10) (for general bolt groups).

C9.4.3 Bolt group subject to combinations of in-plane and out-of-plane loading

The design actions are determined using the methods discussed in C9.4.1 and C9.4.2, and then combined in accordance with 9.3.2.3 or 9.3.3.3 as appropriate.

C9.5 DESIGN OF A PIN CONNECTION

C9.5.1 Pin in shear

The nominal shear capacity is based on a shear stress at failure of 62 % of the yield stress of the pin material.

C9.5.2 Pin in bearing

The approach here is different to that for a bolt in bearing. Failure of a bolt in bearing is not considered as a possible failure mode, as the bolt is usually equal to or greater in strength than the ply. Accordingly, attention is concentrated on bearing failure of the ply in 9.3.2.4. In contrast, in this clause the relatively low failure stress of 1.4 times the yield stress of the pin material reflects the critical nature of this load on a single pin.

The factor k_p of 0.5 for a pin that allows rotation reflects the fact that continual movement of the pin plates around the pin circumference creates a wearing effect. Amendment No. 1 now applies these provisions to the ply as well. See the new C9.5.4, introduced by this amendment, for the background to this change.

Amd 1
June '01

C9.5.3 Pin in bending

A pin is treated as a compact member under section 5, subject only to plastic yielding.

C9.5.4 Ply in bearing

Pin plates are designed for bearing using the provisions of 9.3.2.4, as for a ply in bearing due to a bolt.

The 1997 provisions treated the pin plates in bearing as for a ply in bearing due to a bolt. However, application of these to an experimental test rig component, in 1999, has shown them to be unconservative. The provisions introduced in Amendment No. 1 treat the pin plates in the same manner as the pin and have delivered good performance in the example quoted in the HERA *Steel Design and Construction Bulletin*, Issue No. 51, 1999, pp. 9-11, which presents the background to these changes. Because the pin plates are now treated the same as the pin, clause 9.5.2 has been modified to cover both pin and ply, with the original clause 9.5.4 being deleted.

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June '01

C9.6 DESIGN DETAILS FOR BOLTS AND PINS

The provisions in this clause have been taken from the 1989 edition of this Standard.

C9.6.1 Minimum pitch

The minimum pitch of 2.5 bolt diameters relates primarily to the tools required to install a bolt, and compares with a minimum of 2.67 bolt diameters in the AISC Specification (9.18). Most practical pitches are larger than this ((9.28) uses 3.75 bolt diameters for M20 bolts). The reference to 9.3.2.4 relates to the possibility of plate tearout between the bolt holes.

C9.6.2 Minimum edge distance

The minima specified are based on past successful practice and relate to the expected edge roughness. They are similar to those in comparable specifications, such as (9.18). The end distance may also be controlled by end plate tearout–hence the reference to 9.3.2.4.

These distances may need increasing for fasteners in connections subject to earthquake loads or effects, in accordance with 12.9.4.4. This is in order to suppress failure due to end or edge tearing (see figure C9.3.2.4(1) (b) and (c)) prior to attainment of the necessary inelastic ductility demand in the member. Refer also to Commentary Clause C9.3.2.4 for further details.

C9.6.3 Maximum pitch

The values specified are empirically based on successful past practice. Smaller pitches than the maximum may be preferred if corrosion between the connected piles could be a problem.

C9.6.4 Maximum edge distance

The values specified are empirically based on successful past practise, and are intended to provide for the exclusion of moisture between connected plies, thus preventing corrosion between the piles which might accumulate and force the plies apart. Lesser values should be considered in corrosive applications. The provisions are also intended to prevent any potential curling-up of plate edges.

C9.7 DESIGN OF WELDS

C9.7.1.1 General

When using AS/NZS 1554.1 in conjunction with this Standard, the following matters must be considered. These are included in the commentary because they do not relate directly to design, the subject of 9.7, however it is expected that the recommendations given herein will be followed in practice:

(a) *Welder qualification*
Welder qualification to AS/NZS 1554.1 remains with NZS 4711 (9.31). Welder testing/ certification to each weld process in accordance with NZS 4711 should be required on any job.

(b) *Weld procedure qualification*
This should only be undertaken by a suitably qualified laboratory – e.g. one that is TELARC registered.

(c) *Witnessing of procedure and welder qualification tests*
The (registered) testing laboratory which is issuing the welder or weld procedure qualification should ensure that all work undertaken is inspected and witnessed by a welding inspector qualified to clause 7.2 of AS/NZS 1554.1.

The brittle fracture provisions of AS/NZS 1554.1 (Appendix B) or AS/NZS 1554.5 (Appendix B) should be read in conjunction with the brittle fracture requirements of this Standard, as given in 2.6. There is considerable overlap between the 2 sets of provisions, however those from 2.6 herein give steel type numbers for steels not covered by AS/NZS 1554.

C9.7.1.4 Selection of weld category in accordance with AS/NZS 1554.1

Designers are directed to AS/NZS 1554.1 (12.4) for the appropriate choice of weld classification. Two weld categories, SP and GP, are provided for in AS/NZS 1554.1 and these are the 2 categories appropriate for seismic and non-seismic applications.

The choice of weld classification is specified in 1.5 of AS/NZS 1554. It is important that designers realise that the terminology "dynamic loading" used therein refers to high-cycle loading – i.e. fatigue loading – in accordance with section 10 of this Standard. For seismic applications, the weld class selection from AS/NZS 1554.1 is based on welds "essentially statically loaded" as defined therein.

The use of these 2 classifications for seismic design is appropriate, as discussed by McRobie (9.30). Note that the permissible defect limits for SP welds have been made more stringent from those in the previous edition of AS/NZS 1554.1 on which McRobie's paper (9.30) was based.

For designers used to selecting weld category (class) in accordance with NZS 4701:1981, (the welding Standard specified in the 1989 edition of this Standard) the following guidance will be of assistance:

(a) Category SP from AS/NZS 1554.1 can be substituted for Class A from NZS 4701:1981.

(b) Category GP from AS/NZS 1554.1 cannot always be substituted for Class B from NZS 4701:1981. In situations where a Class B weld would have been specified to NZS 4701:1981 and the design actions on that weld, as detailed, would exceed the design capacity for a Category GP weld, calculated in accordance with this clause, specify Category SP from AS/NZS 1554.1.

(c) In situations where a Class B weld would have been specified to NZS 4701:1981 and the design actions on that weld, as detailed, would not exceed the design capacity for a Category GP weld, calculated in accordance with this clause, specify Category GP from AS/NZS 1554.1.

C9.7.2 Butt welds

C9.7.2.2 Size of weld

The intention of this clause is indicated in figure C9.7.2.2.

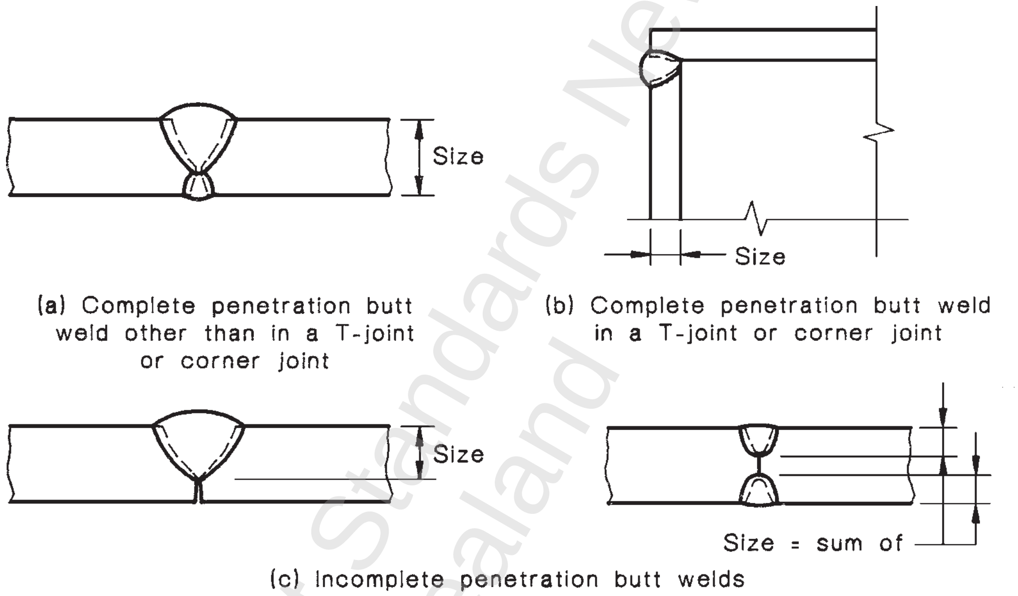

Figure C9.7.2.2 – Size of butt welds

AS/NZS 1554.1 requires the size of weld to be specified in the drawings. This presents no problem in respect of complete penetration butt welds, where the term ‘complete penetration butt weld’ or the appropriate symbol from AS 1101.3 describes the desired result.

However, for incomplete penetration butt welds, the designer determines the design throat thickness by calculation using 9.7.2.3 and 9.7.2.7, while the weld size is a function of:

(a) The design throat thickness;

(b) The welding process; and

(c) The details of the weld preparation.

Rather than specifying the size of an incomplete penetration butt weld, the drawings should show the required design throat thickness. This then allows the fabricator to produce the required design throat thickness by selecting a suitable weld preparation, welding process and welding position. This is particularly important in the case where a fully automatic welding process is to be used, as 9.7.2.3 (b) (iii) permits some advantage to be gained due to the deep penetration usually achievable.

C9.7.2.3 Design throat thickness

The design throat thickness is the minimum dimension of the weld throat used for purposes of strength assessment in 9.7.2.7.

For fully-automatic arc welding processes, 9.7.2.3 (b) (iii) permits advantage to be taken of the penetration achievable with such processes to reduce the size of the weld deposited, provided a macro test demonstrates the viability of the procedure (see figure C9.7.2.3).

C9.7.2.4 Effective length

The length of a continuous full size weld is not necessarily the actual weld length. In certain cases, it is necessary to use run-on and run-off tabs to ensure that a full size weld is present at the ends of a weld, otherwise the effective length may be reduced below the actual length.

Run-on and run-off tabs are not generally required in welds of connections in seismic-resisting systems, provided that the ends of welds are finished in accordance with the provisions of AS/NZS 1554.1 for the category of weld specified.

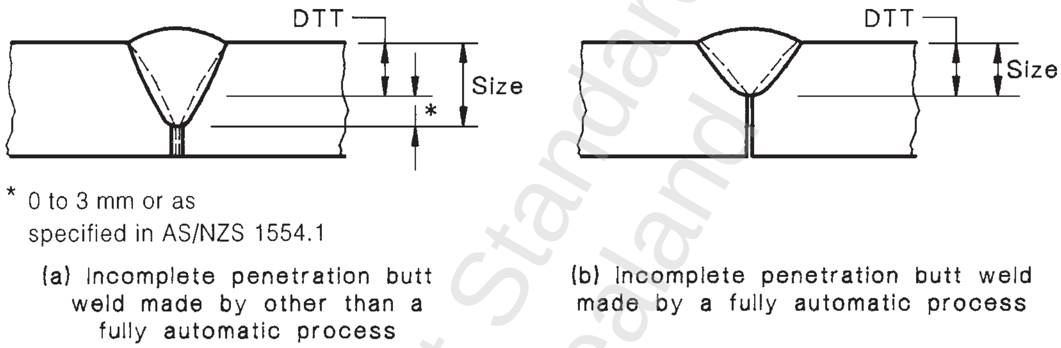

Figure C9.7.2.3 – Design throat thickness

C9.7.2.6 Transition of thickness or width

Where parts subject to tension are varied in thickness or width or both, the required smooth transition can be made by the methods given in figure 9.7.2.6. The maximum taper of 1:1 is a mandatory upper limit for either thickness or width transitions of parts in tension, although smaller tapers may be chosen, usually at some cost penalty. Some welded detail categories in section 10 (Fatigue) and the yielding regions in seismic category 1 or 2 members (section 12 – Seismic) require tapers no greater than 1:2.5, and 9.7.2.6 makes clear that this lesser taper should be observed in such cases. In parts subject to non-seismic induced compression, there is no need for a gradual transition, while for those subject to shear, a 1:1 maximum taper is recommended.

It is recommended that a taper less than 1:2.5 not be used, especially for thickness transitions, since in general the lesser the taper the greater the cost due to difficulties in preparation. Excessively low tapers on thickness transitions may need to be machined, which can be very costly.

The rationale for the 1:1 transition is related to the equivalent stress effect of weld defects and reinforcement permitted by AS/NZS 1554.1 for both GP and SP category welds. A more gradual transition is of little practical use if notches and stress concentration effects prevail adjacent to and in the weld.

Figure 9.7.2.6 (a) of this Standard illustrates the various methods of achieving the required thickness transition depending on whether the adjoining parts have centreline or offset alignment. When a large difference in thickness exists, there is little option but to prepare the parts to be joined with a special edge preparation. This will usually require a flame cut or machined edge with multiple faces, as shown in figure C9.7.2.6 (a).

Where the offset or thickness differential is less than the thickness of the thinner part connected, the transitions may be achieved by tapering the weld to the top surface of the thinner part (see figure C9.7.2.6 (b) and figure 9.7.2.6 (a) (ii)).

Alternatively, the weld may be tapered to the chamfered face of the thicker part (see figure C9.7.2.6 (c)) with subsequent tapering of the unfused top edge. The methods illustrated in figures C9.7.2.6 (b) and (c) are practical and economic, since they permit conventional edge preparations to be cut on the parts prior to welding operations.

The recommended method for width transitions of butt joints in parts of unequal width is by chamfering the wider part with the taper of the chamfer not being steeper than 1:1 (see figure 9.7.2.6 (b)).

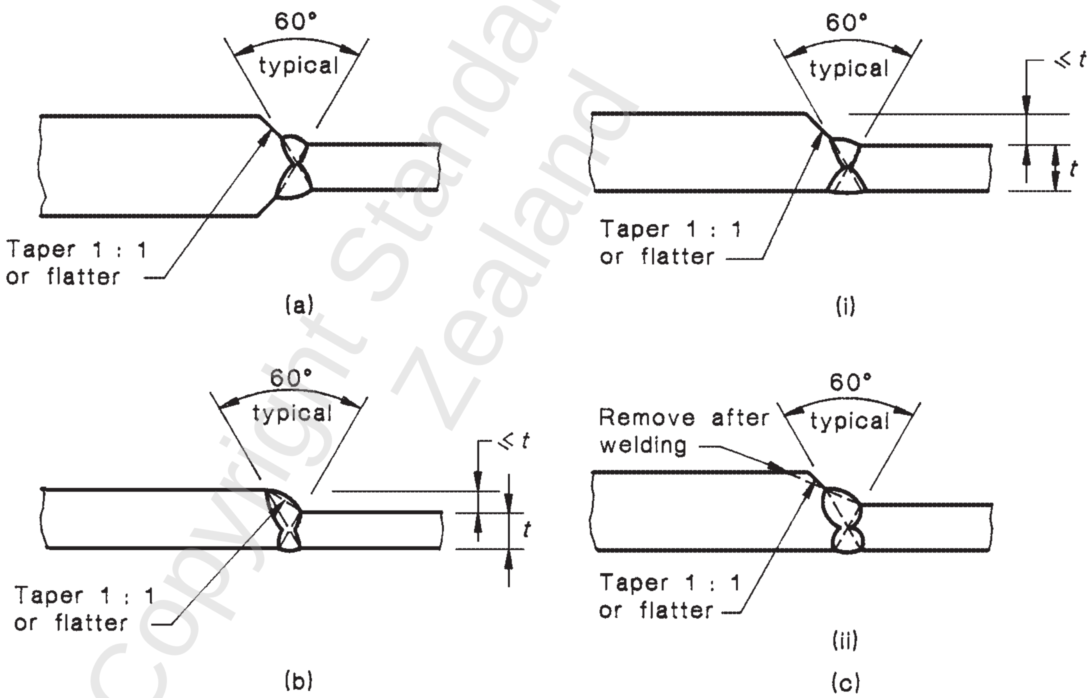

Figure C9.7.2.6 – Transition in thickness between unequal members

C9.7.2.7 Strength assessment of a butt weld

In a complete penetration butt weld, the throat thickness of the weld is equal to that of the thinner part joined, and since there is significant mixing of parent material and deposited weld metal, the design capacity is taken as that of the parts being joined, provided that the consumables are qualified in accordance with AS/NZS 1554.1.

Incomplete penetration butt welds are treated as fillet welds for design purposes, and accordingly the strength assessment is made using 9.7.3.10.

C9.7.3 Fillet welds

C9.7.3.1 Size of a fillet weld

The definition of fillet weld size is illustrated in figure 9.7.3.1, wherein t_w is the size (the leg length). Australasian and American practice is to denote the size of a fillet weld by the leg length, while European practice is to use the throat dimension (t_t).

Preferred fillet weld sizes have the advantage of setting a standard size range for designers to work to, and are sizes measurable with the available fixed weld gauges. There is no restriction implied on using non-preferred sizes.

C9.7.3.2 Minimum size of a fillet weld

The minimum sizes of fillet welds given in table 9.7.3.2 can all be made as single run welds. It is recommended that the provisions of table 9.7.3.2 also be used for the root run of multi-run welds, even though AS/NZS 1554.1 is not explicit in this regard.

The provisions of this clause are intended to ensure that sufficient heat input is provided in order to reduce the possibility of cracking occurring in either the heat-affected zone or in the fillet weld itself, especially in restrained joints. Thick material and small welds may result in a rapid cooling of the weld metal, due to the thick material acting as a heat sink, and this may result in cracking at or adjacent to the weld, with a consequent loss of ductility.

C9.7.3.3 Maximum size of a fillet weld along an edge

Note that in Case (b) of figure 9.7.3.3, the design throat thickness must be based on the size t_w which is less than t, while for Cases (a) and (c), the size t_w equals the thickness t. The reasons for the difference in Case (b) is that, if top edge melting occurs, it is difficult to determine the true size of the fillet weld.

C9.7.3.4 Design throat thickness

In a similar manner to butt welds, advantage may be taken of the increased penetration achievable with a fully automatic welding process, in order to reduce the size (but not the design throat thickness) of a fillet weld – 85 % of the penetration being considered as part of the design throat thickness. The viability of the welding procedure must be demonstrated by means of a macro test.

C9.7.3.5 Effective length

It is important to note that the effective length is the overall length of the full-size fillet weld. The 1989 edition of this Standard required a deduction of twice the weld size from the actual length, but experience has proved that this provision is unnecessary.

C9.7.3.7 Transverse spacing of fillet welds

The requirements of the clause are illustrated in figure C9.7.3.7. The provisions are empirical, based on successful past practice.

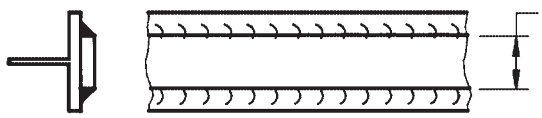

Figure C9.7.3.7 – Transverse spacing of fillet welds

C9.7.3.8 Intermittent fillet welds

The requirements of this clause are summarized in figure C9.7.3.8. The values specified are empirical, based on successful past practice.

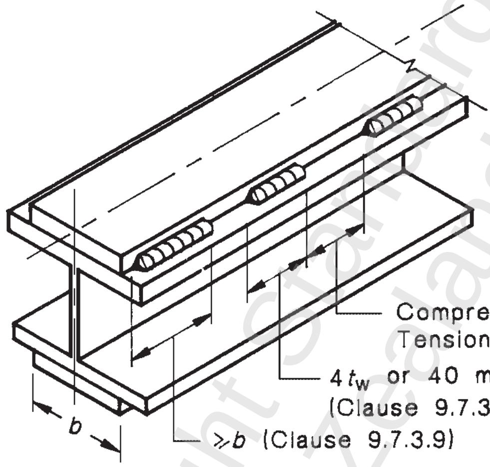

Figure C9.7.3.8 – Provisions for spacing of intermittent fillet welds

C9.7.3.9 Built-up members–intermittent fillet welds

The requirements of 9.7.3.9 (a) are included in figure C9.7.3.8. The remaining provisions are self-explanatory, generally being similar to Provision (a). The provisions are empirical, based on successful past practice.

C9.7.3.10 Ultimate limit state design capacity for fillet welds

The nominal capacity is based on a failure stress of 0.6 f_{uw} in shear on the weld throat (t_t) which is assumed to be the failure plane (see figure C9.7.3.10 (1)). Considering the design actions (v^*_n, v^*_{vt}, v^*_{vl}) on the fillet weld throat in figure C9.7.3.10 (1), a general form of a failure criterion may be written (9.19, 9.20) as:

$$\sqrt{\left[{v^*_n}^2 + k_v \left({v^*_{vt}}^2 + {v^*_{vl}}^2 \right) \right]} = \phi k_w \,(0.6\, f_{uw}\, t_t)$$

$$= \phi k_w\, v_w$$

where

v^*_n = design force per unit length of weld normal to the plane of the fillet weld throat ➤

v^*_{vl} = design shear force per unit length of weld longitudinal to the plane of the fillet weld throat

v^*_{vt} = design shear force per unit length of weld transverse to the plane of the fillet weld throat.

In AS 1250 –1972, the design criterion was based on a k_v value of 1.0 (which results in a vectorial addition method), while AS 1250 –1981 and NZS 3404:1989 used a k_v value of 3 (which results in a von-Mises combination criterion). For 9.7.3.10, values of k_v = 1.0 and k_w = 1.0 have been adopted, based on the studies reported in (9.19, 9.20).

An alternative approach is to use a load-deformation method, which recognizes that the weld has a finite deformation capacity and attempts to obtain the load-deformation curve for fillet welds by test. This data is then used to predict the failure load of any fillet weld (see for example (9.23)).

The influence of bending moments at the faces of the weld and of normal forces applied longitudinally to the weld cross section (see figure C9.7.3.10 (2)) have been shown to have little influence on the weld strength (9.21, 9.22).

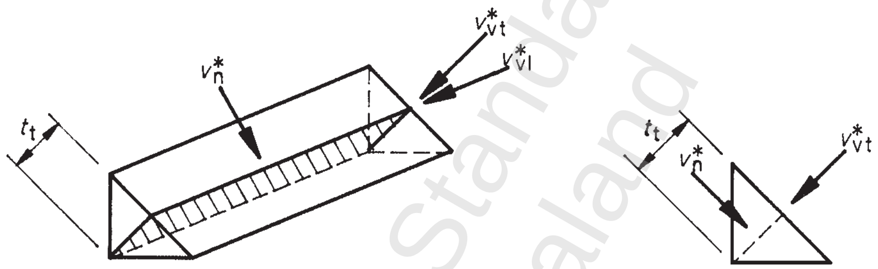

Figure C9.7.3.10 (1) – Design actions on a fillet weld

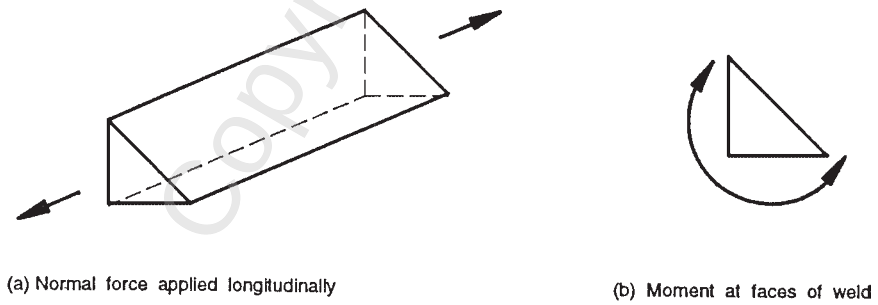

Figure C9.7.3.10 (2) – Design actions not usually considered in assessing the strength of a fillet weld

The reduction factor (k_r) essentially reduces the effective weld length (L_w) determined in accordance with 9.7.3.5 (see figure C9.7.3.10 (3)). The reduction in effective length applies to lap joints with long weld elements to account for non-uniformity in the stress distribution along the weld.

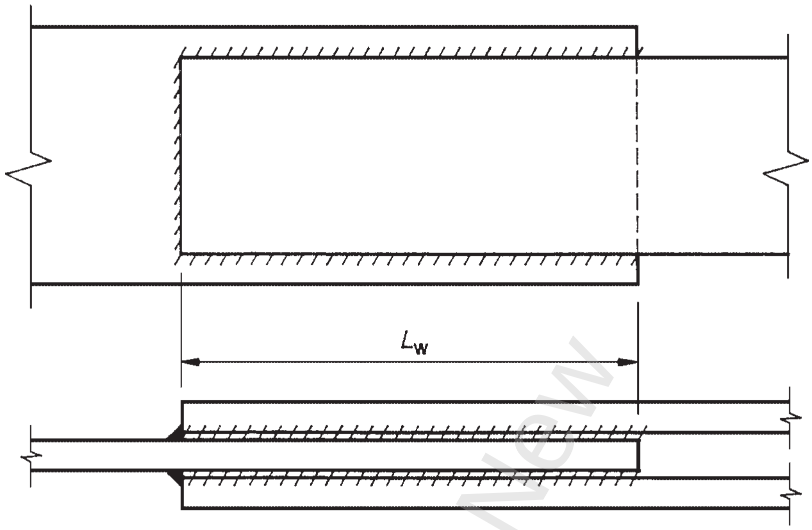

Figure C9.7.3.10 (3) – Lap joint with fillet welds

For fillet welded lap connections, there is no minimum length beyond that required by 9.7.3.5.2.

Where longitudinal fillet welds are used alone in a connection, the AISC specification (9.18) requires the length of each weld to be at least equal to the width of the connecting material, because of shear lag (see figure C9.7.3.10 (4)). By providing a minimum lap of 5 times the thickness of the thinner part of a lap joint, the resulting rotation of the joint when pulled will not be excessive, as shown in figure C9.7.3.10 (5).

When calculating the nominal capacity of a fillet weld from equation 9.7.3.10, note that the requirement in the 1992 edition to use the lesser of f_{uw} or f_u has been removed from the 1997 edition. The weld capacity is now based on f_{uw} alone. The additional requirement in the 1992 edition was intended to suppress failure in the parent metal if $f_{uw} \geq f_u$. The provisions of AS/NZS 1554.1 clause 4.5 regarding matching of welding consumables to parent metal suppress this form of failure, thus making the now-deleted requirement unnecessary and often overly conservative for calculation of weld capacity. For members subject to significant inelastic demand, however, $f_{uw} \geq f_u$ is still required.

Use of weld metal with increased tensile strength

Note (2) to table 9.7.3.10(1) allows the designer to use welding consumables with a nominal tensile strength, f_{uw}, greater than the 2 options given in that table, provided that failure at the interface between the weld and the parent metal is precluded.

This check can readily be made by determining the design capacity of the parent metal at the interface between the weld leg and the parent metal, based on a nominal capacity of ($0.6 f_u t_w k_r$), when t_w is the appropriate weld leg length, f_u is the nominal tensile strength of the parent metal connected to that leg and k_r is the reduction factor given by 9.7.3.10.3 for a welded lap connection. For an equal leg length fillet weld, $t_t = t_w / \sqrt{2}$, so this check would be needed when $f_{uw} > \sqrt{2} f_u$.

To use this option, designers must know the nominal tensile strength of the weld metal and must specify the appropriate weld metal on which the design has been based in the contract documents.

The benefit that is being sought through this approach is to reduce the fillet weld size required through the use of a higher strength weld metal.

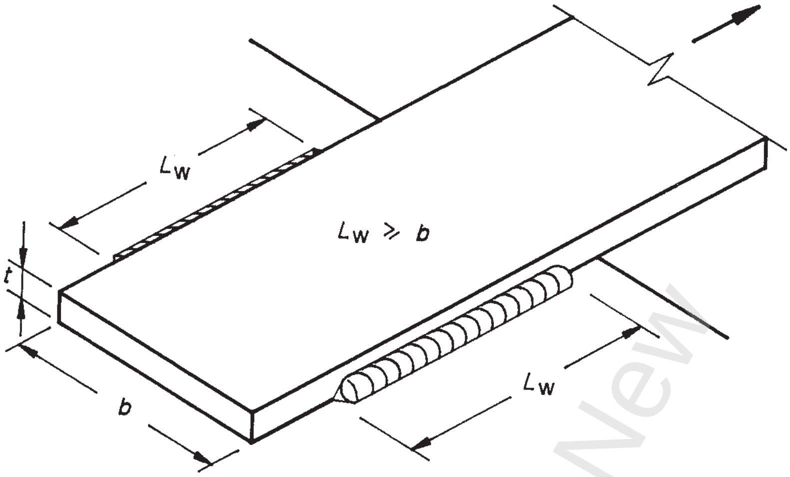

Figure C9.7.3.10 (4) – Longitudinal fillet welds

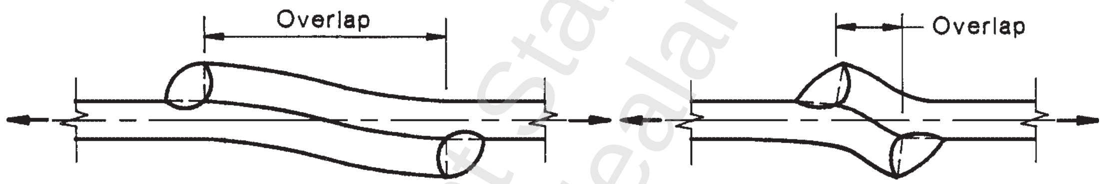

Figure C9.7.3.10 (5) – Minimum lap

C9.7.4 Plug and slot welds

Typical uses for plug and slot welds are to transmit shear in a lap joint or to prevent the buckling or separation of the plates in a lap joint. Their use is not extensive for structural applications.

The provisions of 9.7.4.2 are based on research reported in (9.25), which concluded that the traditional approach of using an average shear failure stress over the hole area is an acceptable design approach. The following detailing provisions are based on the provisions of the AWS Structural Welding Code (9.26). AISC (US) provisions are identical (9.18).

Plug welds. The diameter of the hole for a plug weld should be not less than the thickness of the part containing it plus 8 mm. The diameter should not exceed either the minimum diameter plus 3 mm, or 2.25 times the thickness of the part, whichever is the greater.

The minimum centre-to-centre spacing of plug welds should be 4 times the diameter of the hole.

The depth of the filling of plug welds in material 16 mm or less should be equal to the thickness of the material. For thicknesses over 16 mm, the depth should be at least one-half the thickness of the material, but not less than 16 mm.

Slot welds. The length of the slot for a slot weld should not exceed 10 times the thickness of the part containing it. The width of the slot should be not less than the thickness of the part containing it plus 8 mm. The width should not exceed either the minimum width plus 3 mm, or 2.25 times the thickness of the part, whichever is the greater.

The ends of the slot should be semicircular or should have the corners rounded to a radius not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

The minimum spacing of lines of slot welds in a direction transverse to their length should be 4 times the width of the slot. The minimum centre-to-centre spacing in a longitudinal direction on any line should be 2 times the length of the slot.

C9.8 ASSESSMENT OF THE STRENGTH OF A FILLET WELD GROUP

C9.8.1 Fillet weld group subjected to in-plane loading

C9.8.1.1 General method of analysis

In Commentary Clause C9.7.3.10, the strength and behaviour of an isolated element of weld was considered. A weld group may be considered as a collection of such elements, and it is necessary to consider how the nominal capacity of such a weld group may be assessed.

In the general method of analysis, the nominal capacity of a welded connection with a constant thickness weld group is assessed by treating that connection as a weld group of unit thickness in isolation from the attached elements or members.

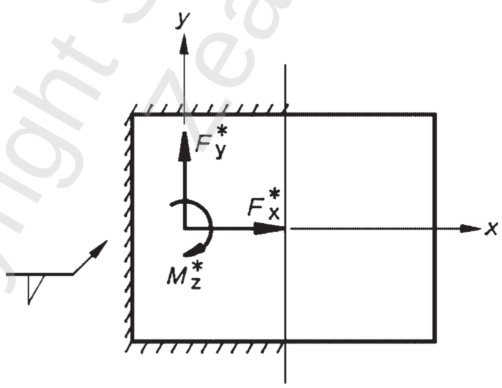

Figure C9.8.1.1 – Fillet weld group loaded in-plane

If a connection at the end of a member is viewed as a weld group in isolation from that member (see figure C9.8.1.1), then the nominal capacity of the weld group may be determined by either an elastic or an ultimate strength approach. Both methods are based upon assumptions (a) and (b) of the clause, rotation being assumed about an instantaneous centre.

The elastic or linear method is the traditional approach to the assessment of the load capacity of a weld group. The force per unit length of weld is considered to be proportional to the distance from the instantaneous centre.

Derivations of the fundamental equations are given in (9.23, 9.24).

Once the forces per unit length have been determined, the nominal capacity may be determined using the failure criteria of 9.7.3.10.

This method has been adopted in this Standard because reliability studies reported in (9.19, 9.20) have indicated that the method is sufficiently reliable, while having the virtue of being simpler to apply than the alternative methods and being amenable to hand calculation.

The ultimate strength analysis of a fillet weld group is described in (9.21, 9.23). For this type of analysis, the weld group is discretised into short elements of fillet weld. The load-deformation relationships determined by testing are considered to describe the behaviour of each element. Although the weld forces are still considered to act normal to the radius from the instantaneous centre, the magnitude of the force is not proportional to the radius. The instantaneous centre should therefore be determined by trial and error. The ultimate load capacity corresponding to the achievement of an ultimate displacement condition at some point in the weld group can then be determined.

C9.8.1.2 Alternative method of analysis

An alternative approach is offered in which a fillet weld group is designed as an extension of the connected member, by maintaining a consistent distribution of forces so that equilibrium is satisfied at the interface between the weld element and the parent plate. For example, in commonly adopted theory, only the web of the beam is assumed to resist vertical shear force, whereas in weld group theory, the shear force may be considered to be uniformly distributed over the length of the weld (see figure C9.8.1.2 (a)). A similar difference in the assumed force distribution exists for a beam subjected to torsion as illustrated in figure C9.8.1.2 (b). This alternative analysis allows the assumptions made in member design also to be used for the design of the fillet weld group.

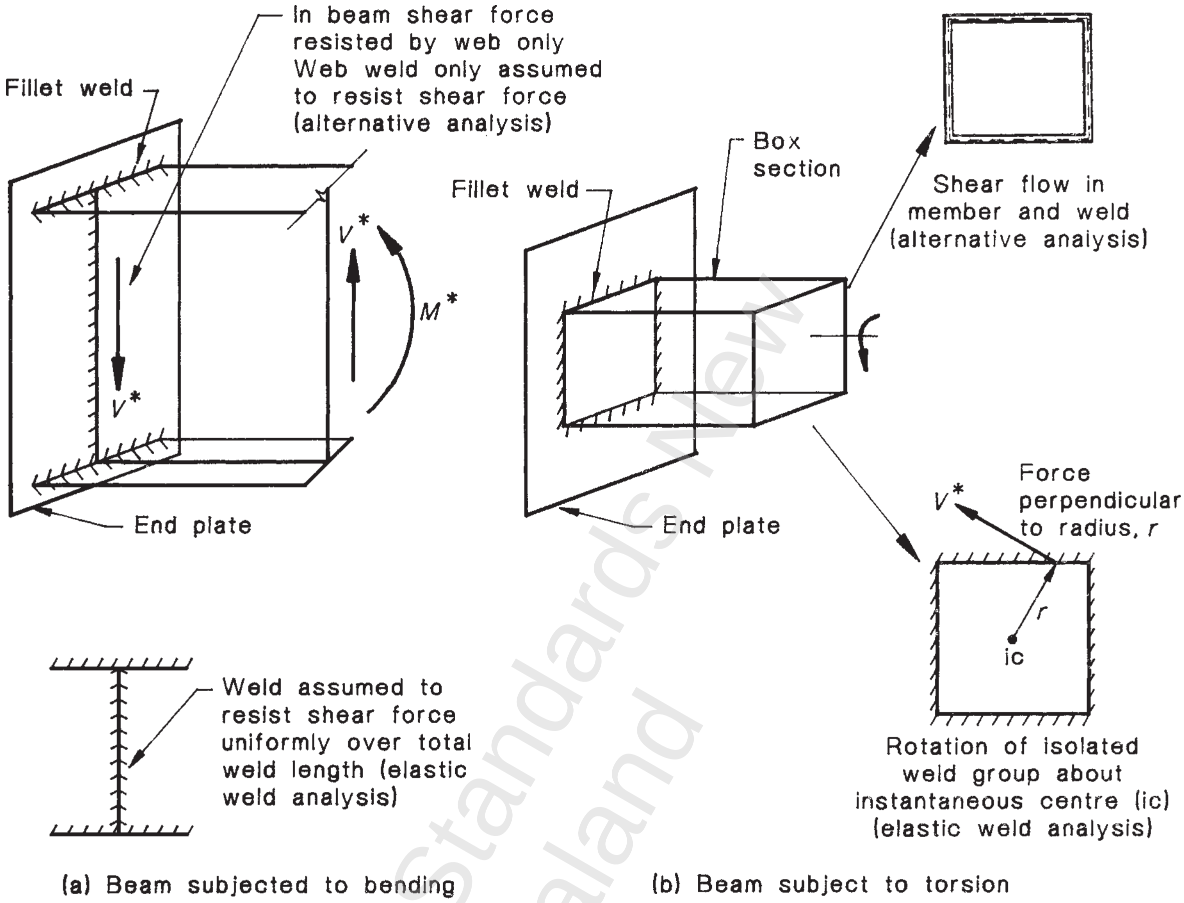

Figure C9.8.1.2 – Examples showing assumptions associated with analysis of weld group (9.19)

C9.8.2 Fillet weld group subject to out-of-plane loading

C9.8.2.1 General method of analysis

The same comments made in C9.8.1.1 apply. References containing analysis procedures within the provisions of this clause are outlined in (9.1 and 9.24).

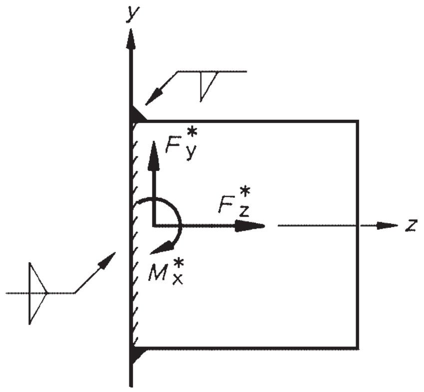

Figure C9.8.2.1 – Fillet weld group loaded out-of-plane

C9.8.2.2 Alternative method of analysis

The comments made in C9.8.1.1 apply.

C9.8.3 Fillet weld group subject to in-plane and out-of-plane loading

General expressions for such weld groups of constant thickness may be found in (9.1, 9.24, 9.27), as follows:

$$v_x^* = \frac{F_x^*}{s} - \frac{M_z^* y}{I_p}$$

$$v_y^* = \frac{F_y^*}{s} - \frac{M_z^* x}{I_p}$$

$$v_z^* = \frac{F_z^*}{s} + \frac{M_x^* y}{I_x} - \frac{M_y^* x}{I_y}$$

where v_x^*, v_y^*, v_z^* are the design forces per unit length in the *x*, *y*, *z* directions respectively on an elemental length of weld. The *x*- and *y*- axes are the principal axes of the weld group and the *z*-axis is perpendicular to the weld group and through the centroid.

F_x^*, F_y^*, F_z^* are the design forces applied to the weld group in the *x*, *y*, *z* directions respectively.

M_x^*, M_y^*, M_z^* are the design bending moments applied to the weld group about the respective axes, with M_z^* moments due to in-plane forces being determined relative to the weld centroid location.

I_x and I_y are the second moments of area of the weld group for a unit thickness of weld about the *x*- and *y*- axes respectively. $I_p = I_x + I_y$ is the polar moment of inertia about the *z*-axis, and *s* is the total length of weld.

C9.9 PACKING IN CONSTRUCTION

This clause has been brought forward from AS 1250 (NZS 3404:1989) and is based on successful past practice.

REFERENCES TO SECTION C9

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C10 FATIGUE

INTRODUCTION

The flowchart in figure C10 will assist users of the Standard to follow the layout and logic of this section. This section is set out in a manner intended to follow the normal procedure a designer would use in carrying out a fatigue assessment.

C10.1 GENERAL

C10.1.1 Requirements

The fatigue assessment is intended to verify that the required probability of survival for the structure is achieved for the spectrum of applied loads. An assessment should be made of every potential fatigue crack site. It should be established that failure will not occur during the design life of the structure by using 10.8.1 or 10.8.2 (or the equivalent provisions of (10.8)) as appropriate.

The fatigue assessment method of this section is based on the assumption that the structure is otherwise designed in accordance with the ultimate and serviceability limit state requirements. The fatigue clauses are additional to the other requirements of this Standard and are not intended to replace any other limit state condition.

The following effects are not covered by this section:

(a) *Reduction of fatigue life due to corrosion or immersion*. In corrosive environments the fatigue strength may be significantly reduced. Data appropriate to these environments is required to enable design to proceed. However, the *S-N* curves given in 10.6 are applicable to structures in mildly corrosive environments, such as normal atmospheric conditions, with suitable corrosion protection. The data on which this section is based is also not appropriate to structures or structural elements which are immersed, whether permanently or periodically.

(b) *High stress-low cycle fatigue*. If the stress is sufficiently high or if the stress range is such that the number of cycles necessary to produce cracking is less than approximately 10^5, the assessment procedures in this section are not applicable. This covers all severe seismic applications.

(c) *Thermal fatigue*. The *S-N* curves are not applicable to structures which are subject to temperatures above 150 ^{o}C.

(d) *Stress corrosion cracking*. Stress corrosion cracking is a phenomenon which occurs in conditions of high stress and a corrosive environment.

Examples of the structures which can come within the scope of section 10 are railway bridges, highway bridges, crane runway girders, machinery support structures, cranes and other similar structures.

The fatigue assessment of existing structures may also be carried out using the provisions of this section, but the fatigue loading must consist of the actual service loading for the entire design life of the structure (past and future).

In the fatigue assessment, it is important to check every point of the structure at which fatigue cracking may occur, because the structure may be damaged by cracking at any point which is not designed and detailed for the applied stress range.

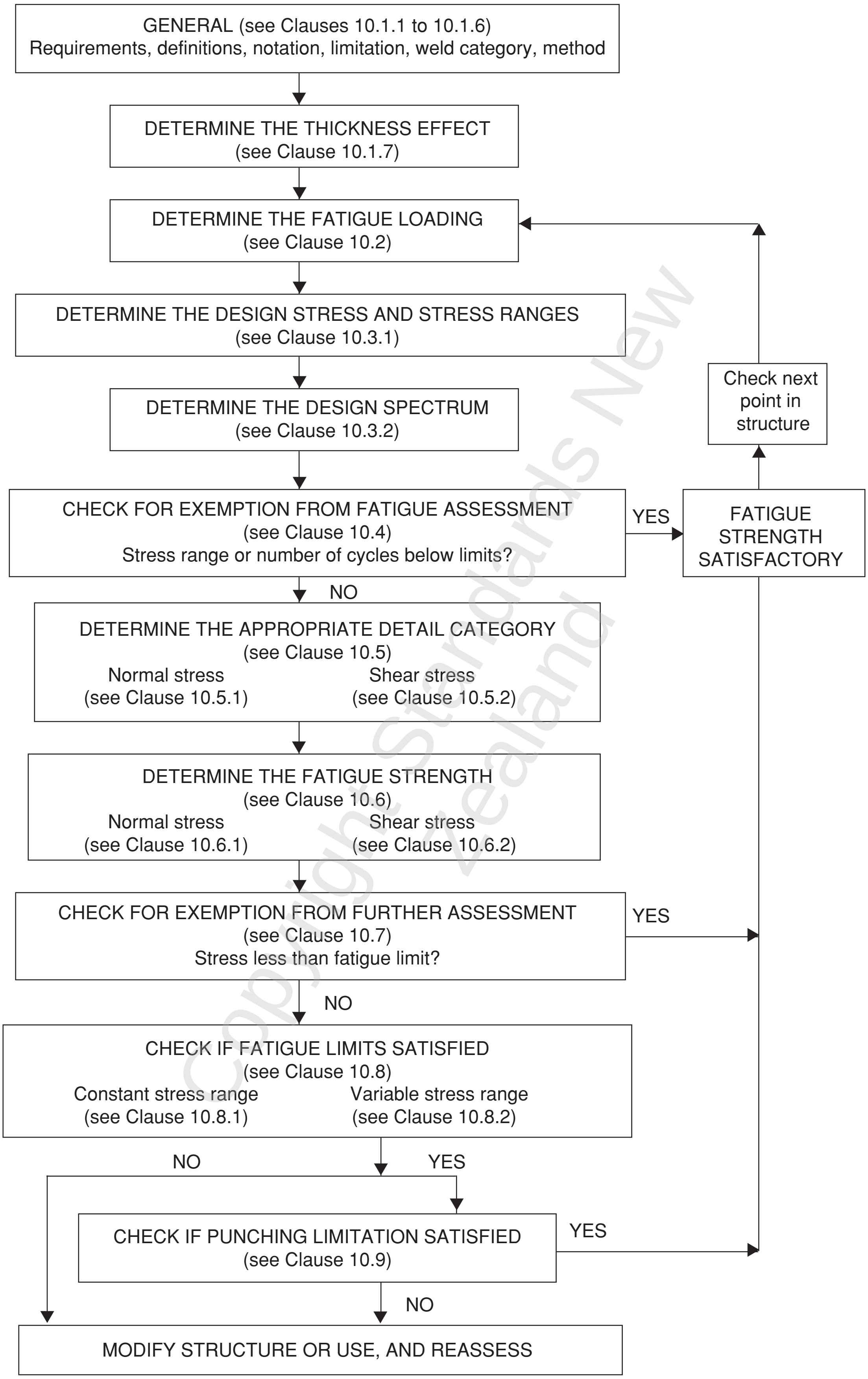

Figure C10 – Flowchart for fatigue assessment

An excellent reference, which expands considerably on the method used herein, is (10.8).

The application of a fracture assessment in accordance with e.g. BS 7910:1999 offers an alternative to the *S-N* design approach presented in this clause or in (10.8).

General guidance on all the aspects of design for fatigue resistance, including a comparison between the provisions of this Standard, BS 7608 (10.8) and other commonly used Standards is contained in HERA Report R8-19 *Seminar Notes on Fatigue in Welded Construction*, by Bayley, C. and Scholz, W., published by HERA in February 2000.

Amd 1 June '01

C10.1.2 Definitions

The definitions include some traditional terms which have been used for the sake of minimizing difficulties in the transition from the fatigue rules of the 1989 edition of the Standard to those in the current edition.

C10.1.4 Limitation

The *S-N* curves given in clause 10.6 are applicable to structural steel grades up to a maximum yield stress of 700 MPa, but are limited by 1.1 of the Standard to an upper limit of 450 MPa except as specified by 1.1.4 (b). The *S-N* curves are applicable for bolt property classes up to a maximum yield stress of 1000 MPa.

The fatigue assessment procedure in this section is not applicable if points in the structure are required to yield or if the stress range exceeds 1.5 f_y.

Information on the fatigue performance of Grade 690 quenched and tempered steels is available (10.6).

C10.1.5 Designation of weld category

Weld Category SP of AS/NZS 1554.1 is only considered applicable for Detail Category 112 and below (tables 10.5.1 (2) and (4)).

Detail Category 125 is thus the only welded detail affected by the restriction on the use of Weld Category SP (table 10.5.1 (2)). Detail Categories 160 and 140 do not involve welded details. Detail Category 125 may be designed by either:

(a) Using Detail Category 112 in lieu of 125 and designing the weld as Weld Category SP to AS/NZS 1554.1; or

(b) Using Detail Category 125 and specifying levels of acceptable imperfections which comply with those specified in AS/NZS 1554.5.

A three plate welded *I*-section can be considered to be Detail Category 112.

Repairs to Detail Category 125 must be undertaken strictly in accordance with AS/NZS 1554.5 in order not to compromise the fatigue performance of the weld.

C10.1.6 Strength reduction factor

The strength reduction factor ϕ should not be taken as greater than 1.0. By comparing the results of the fatigue design provisions herein with those from the more comprehensive BS 7068 (10.8) for specific constructional details, the relative importance of the factors given in 10.1.6.1 on the fatigue life has been able to be assessed. This has then been turned into more comprehensive recommendations on choosing ϕ than were given in the 1992 edition. These details are given in 10.1.6.2 and 10.1.6.3.

The value of ϕ required when conditions are very unfavourable, i.e. as given by 10.1.6.2(c) or 10.1.6.3(c), is a matter for experienced design engineer judgement. For example, in the fatigue design of off-shore oil platforms, where the detail is not practically accessible for inspection and is on a non-redundant load path, values as low as ϕ = 0.1 are used.

It is not practical for this Standard to prescribe values of the strength reduction factor for the very wide range of possible circumstances in fatigue design, and it is the responsibility of the design engineer to determine suitable values for use when either 10.1.6.2(c) or 10.1.6.3(c) apply.

C10.1.7 Thickness effect

The *S-N* curves were derived from experimental results of details involving plate thickness (and corresponding weld sizes) of approximately 15 mm. Testing of transverse butt welded connections involving plate thicknesses up to 100 mm has shown that these curves may be unsafe when plate thicknesses exceed 25 mm.

The equation for deriving f_c has been verified only for welded details oriented transverse to the direction of applied stress and for details which connect equal plate thicknesses. Reference (10.2) provides guidance which may assist the fatigue assessment for details which contain unequal plate thickness above 25 mm. Refer also to (10.8).

C10.2 FATIGUE LOADING

C10.2.1

Amd 2 Oct. '07

The loading used for the fatigue assessment should resemble, as closely as possible, the actual service loading envisaged over the design life of the structure. Factored loads to an alternative Loadings Standard are not appropriate.

Amd 2 Oct. '07

For loads, such as wind loads, in which the return period associated with the serviceability and the ultimate limit state levels of loading are given in AS/NZS 1170.0, this information can be used to determine the maximum load to be expected over the design life of the structure. For example, with wind load from AS/NZS 1170.2, the return period associated with the serviceability limit state wind is 20 years, while that associated with the ultimate limit state wind is 500 years. For a structure design life of 50 years, the maximum wind load for fatigue assessment is then obtained by scaling between the 2 limit state loads. For sites where the site-specific wind data required to determine the fatigue design spectrum is not known, this spectrum can be derived from the maximum service load determined above using (10.9).

C10.2.2

In determining the fatigue loading, dynamic effects should be taken into account as well as, in some types of structures, loads due to induced oscillations. For example, a study of the oscillations due to the structural response to moving loads of lightly damped structures, such as some cranes, is necessary for an accurate evaluation of the fatigue strength. Wind-induced oscillations in masts and chimneys should also be investigated.

Measured load histories may not reflect accurately the future fatigue loading. In some structures, such as bridges and cranes, consideration should be given to possible changes in usage, such as the growth of traffic or changes in the most severe loading.

Reference (10.1) provides more detailed information relating to modern concepts of fatigue loading. The fatigue loading may be composed of different load cases, each defined by the distribution and magnitude of the loads as well as their relative frequency of occurrence.

A loading event is a well defined loading sequence of the entire structure or structural element. This may be the approach, passage and departure of one train, or of a single bogie or axle in the case of a railway bridge, or of one vehicle in the case of a highway bridge. The effect of a loading event is best described by its stress history, which is the stress variation at a point in the structure during the loading event.

The effect of impact may be very important. Measured average impact factor values should be used whenever possible. In the absence of more accurate information, the generally applied impact factors used for the ultimate limit state should be employed. In many cases, these impact factors over-estimate the effect of impact on fatigue loading.

Simplified design calculations may be based on an equivalent fatigue loading which represents the fatigue effects of all loading events.

The equivalent fatigue loading should be obtained analytically from the summation of the cumulative damage of the design spectrum using design fatigue loads and an appropriate load cycle counting method. The equivalent fatigue loading may vary with the size and location of the structural element. For example, main bridge girders may not experience stress cycles due to individual axles, but these cycles may cause failure in smaller elements closer to the point of load contact.

The stress cycle counting method should be suitable for the analysis of the stress spectrum. Rainflow counting can be used together with the Miner's summation of 10.8.2 for many applications, including railway and highway bridges subject to the usual traffic loading.

The fatigue loadings from referred Standards are to be used as specified in those Standards. Comprehensive general guidance on deriving the fatigue loading is given in (10.8), including a description of the rainflow counting method.

C10.3 DESIGN SPECTRUM

C10.3.1 Stress determination

The configuration of K and N type joints is shown in figure C10.3.1.

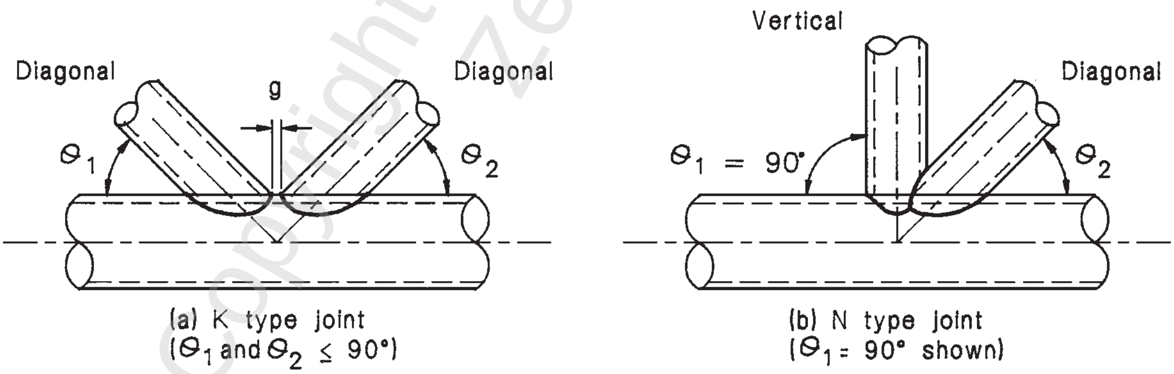

Figure C10.3.1 – Configuration of K and N type joints (shown for circular hollow sections)

The detail category allows for the effects of local stress concentrations due to weld shape, discontinuities and triaxiality. An evaluation of the stresses using the results of an elastic analysis generally provides the necessary information. Alternatively, measured strains can be used to derive stresses. Generally, the arrow on each detail in tables 10.5.1 (1) to (4) indicates where the stress is to be calculated, the plane on which the stress is calculated being normal to the arrow.

The effect of stress concentrations due to effects not already included, such as holes, cut-outs and re-entrant corners, which are not natural characteristics of the detail category itself, should be taken into account separately by the application of appropriate stress concentration factors. ➤

The effect of stresses arising from other effects, such as joint eccentricity, deformations, secondary bending moments, or partial joint stiffness, should be calculated and taken into account when determining the stress at the detail.

When the plane on which the stress range is calculated is subject to a combination of normal and shear stresses, the assessment should consider their combined effects. When normal and shear stresses cause the formation of fatigue cracks at 2 distinct locations, no combination of stresses is needed.

Otherwise, the assessment should consider the following:

(a) Principal stresses should be calculated when the fatigue loading originates from simple load cases. However, principal stresses should be calculated only when normal and shear stresses occur simultaneously (and at the same location) during the stress cycle or loading event.

(b) If normal (f_n^*) and shear (f_s^*) stress ranges do not occur simultaneously at the same location, the components of damage should be added using Miner's rule according to the following equation:

$$(f_n^* / \phi f_{rn})^3 + (f_s^* / \phi f_{rs})^5 \leq 1.0 \quad \text{(Eq. C10.3.1)}$$

where ϕ is given by 10.1.6.

C10.3.2 Design spectrum calculation

The various stress spectra and their relative frequencies of occurrence for each of the fatigue loading cases should be compiled. This compilation gives the design spectrum to be used for the fatigue assessment.

The constant amplitude fatigue limit (f_3) should not be used in the fatigue assessment unless it is certain that there will be no stress ranges which exceed it. The fatigue assessment procedure may not allow for a small number of high stress ranges which may occur during fabrication, transportation, erection or service of the structure, and so caution should be exercised in the use of the constant amplitude fatigue limit.

Compressive stress ranges should be considered to be as damaging as tensile stress ranges unless it can be shown to be otherwise.

C10.4 EXEMPTION FROM ASSESSMENT

A fatigue assessment is not usually necessary for buildings. However some parts of building structures, such as support structures for machinery or crane girders or cranes, may require a fatigue assessment. The exception permitted by this clause identifies structures or structural elements in which the stress ranges and numbers of stress cycles are low enough to ensure that even the worst details in tables 10.5.1 (1) to (4) would be satisfactory.

C10.5 DETAIL CATEGORY

C10.5.1 Detail categories for normal stress

The *S-N* curves for normal stress for the various detail categories are parallel and approximately equidistant from each other when examined on a log-log scale (figure 10.6.1 of the Standard). Each curve corresponds to a detail category defined by the fatigue strength at 2×10^6 cycles. The physical characteristics which correspond to each detail category are shown in tables 10.5.1 (1) to (4).

Some details do not behave exactly as categorised in tables 10.5.1 (1) to (4), but in order to ensure that unconservative conditions are avoided, some details are located in detail categories slightly lower than their fatigue strength at 2 x 10^6 cycles would require.

All physical characteristics of details must be defined by the designer and must not be altered in any way during fabrication or erection without the designer's approval. No attachments or cut-outs should be added to any part of the structure without notifying the designer.

C10.5.2 Detail categories for shear stress

More detailed procedures for estimating the fatigue endurances of stud shear connectors are available (10.7).

C10.6 FATIGUE STRENGTH

C10.6.1 Definition of fatigue strength for normal stress

The constant stress range limit (f_3) is taken as the fatigue strength at 5 x 10^6 cycles. However, if any stress range in the spectrum exceeds the constant amplitude fatigue limit, the stress ranges below the constant amplitude limit must also be considered in the assessment. The constant stress range limit is the point at which the slope (α_s) of the *S-N* curve changes from 3 to 5 for normal stress.

The cut-off limit (f_5) is taken as the fatigue strength at 10^8 cycles, and all stress cycles in the spectrum below the cut-off limit may be ignored in the fatigue assessment.

The *S-N* curves are based on a conservative interpretation of data taken mostly from tests of structural elements containing high tensile stresses at fatigue crack locations. Therefore, they apply to:

(a) Elements with high residual stresses;

(b) Elements with high values of the stress ratio f^*_{min} / f^*_{max}; or

(c) All elements, whether the level of mean stress, due to effects such as temperature, support settlements, erection, or misfit is known or not.

The curves are based on the mean experimental values minus 2 standard deviations. The vertical spacing between detail categories represents approximately a 10 % variation in fatigue strength.

C10.6.2 Definition of fatigue strength for shear stress

The *S-N* curve for cracking due to applied shear stresses is given in figure 10.6.2 of the Standard. The *S-N* curve for shear stress provides the relationship for a fatigue assessment of shear stress applied to weld throats or base material. Fatigue failure in these details occurs usually by crack propagation across the weld throat.

Calculations should be performed in a similar way to those for normal stress. The cut off limit (f_5) is as defined above. **No constant amplitude fatigue limit should be assumed, however.**

C10.7 EXEMPTION FROM FURTHER ASSESSMENT

(No Commentary.)

C10.8 FATIGUE ASSESSMENT

C10.8.1 Constant stress range

An alternative formulation in the similar terms to those in 10.8.2 is:

$$\frac{\Sigma_i n(f^*)\alpha_s}{n_r(\phi f_c)\alpha_s} \leq 1.0 \quad \text{(Eq. C10.8.1)}$$

where

$\sum_i n$ = number of stress cycles at constant stress range.

ϕf_c relates to the value at $n_{sc} = 2 \times 10^6$ cycles.

C10.8.2 Variable stress range

The inequality is similar to that in the previous commentary clause, however it incorporates the change of *S-N* curve slope at f_{3c} in the case of normal stresses. Thus 2 terms are necessary, as given in 10.8.2(a). The numerator represents a cumulative damage assessment based on Miner's rule.

C10.9 PUNCHING LIMITATION

(No Commentary.)

REFERENCES TO SECTION C10

10.1 International Association of Bridge and Structural Engineering. 1982. Fatigue of Steel and Concrete Structures. IABSE Reports, Vol. 37. Zurich.

10.2 European Convention for Constructional Steelwork, Committee IC6. 1985. Recommendations for The Fatigue Design of Steel Structures. Publication ICOM 141.

10.3 International Association of Bridge and Structural Engineering. 1990. Remaining Life of Steel Structures. Proceedings, Workshop. Lausanne.

10.4 Fisher, J.W. 1984. Fatigue and Fracture in Steel Bridges. Published by Wiley-Interscience.

10.5 Gurney, T.R. 1968. Fatigue of Welded Structures. Published by Cambridge University Press, Cambridge.

10.6 Corderoy, D.J.H. 1982. Report on Fatigue Tests on Bisalloy 80. Published by the School of Metallurgy, University of New South Wales, Sydney.

10.7 Oehlers, D.J. 1990. Methods of Estimating the Fatigue Endurances of Stud Shear Connections. IABASE Proceedings. P-145/90. Zurich.

10.8 BS 7608: Code of Practice for Fatigue Design and Assessment of Steel Structures, British Standards Institute, London.

10.9 European Convention for Constructional Steelwork. 1987. Recommendations For Calculating the Effects of Wind on Constructions. ECCS Publication No. 52. Brussels.

C11 FIRE

C11.1 REQUIREMENTS

Structural steel members may be required to have a Fire Resistance Rating (FRR) in order to meet the performance criteria of the New Zealand Building Code (11.23). The Approved Documents for Fire Safety (11.15) provide an Acceptable Solution to these performance criteria.

The Period of Structural Adequacy (PSA) is the time for which the structural member will support the applied loads when subjected to a standard fire test. The PSA for a member must equal or exceed the required FRR. To achieve the required FRR it might be necessary to protect the steelwork by the application of a suitable fire-protection material. The PSA for protected steel members is dependent on the thickness of the fire protection material applied, the section factor (SF) and the applied load. Protected members designed in accordance with (11.1, 11.2) or section 7 of (11.18) will satisfy the requirements of section 11. The procedure given in (11.2) allows for different levels of applied loading, whereas the procedure given in (11.18) generally is based on the applied loading being that required from the standard fire test.

The requirements of the section on fire in AS 4100:1990 are written around the Australian fire testing Standard AS 1530:Part 4 (11.3). In New Zealand, the corresponding ISO or British Standards (11.4, 11.6) are also means of compliance. The provisions of section 11 of this Standard, therefore, have been modified to allow equal use of (11.3, 11.4 or 11.6).

The Fire Resistance Rating may also be obtained from a calculation method which allows for the actual characteristics of the fire and fire compartment in determining the FRR required. Guidance is given in (11.17) and (11.21). The security (S) ratings for structural stability given in the Approved Documents (11.15) are based on this approach (11.21). This is an area under very active development and use of such methods calls for careful judgement on the part of the design engineer, coupled with a sound knowledge of the principles of Fire Engineering Design.

C11.2 DEFINITIONS

The differences in definitions between this section and the corresponding section in AS 4100:1990 reflect differences in terminology and practice between New Zealand and Australia. The basic procedures in designing for fire are effectively the same, however.

Figure C11.2 illustrates the different categories of configuration for three-sided and four-sided and four-sided fire exposure conditions.

The Fire Resistance Rating (FRR) in New Zealand is termed the Fire Resistance Level (FRL) in Australia.

The Australian practice is to express the Section Factor (SF) in units of exposed surface area to mass ratio (k_{sm}) while UK and international practice is to use the heated perimeter to area ratio (H_p/A). Both measure the same physical feature of the section and there is a constant ratio of 7.85 between them. Values for both H_p/A and k_{sm} for many common section sizes in 3-sided and 4-sided configuration are contained in section 7 of (11.18), which also presents methods for calculation of the SF for different configurations. Values of k_{sm} for common section sizes for 3-sided and 4-sided configurations are presented in (11.1, 11.2). ➤

Section factor

When calculating the section factor for members with varying cross section along the length of the member, the appropriate value to use is that associated with the cross section which:

(a) For members subjected principally to bending or axial tension, generates the highest value of (r_f) from 11.5.

(b) For members subjected principally to axial compression, would deflect the furthest under compression member buckling.

The methods for conducting standard fire tests on elements of building construction are set out in AS 1530:Part 4 (11.3), ISO 834 (11.4) or BS 476:Parts 20-23 (11.6). All methods are very similar and specify the heating curve for the test furnace, the specimen size, the loading and restraint to be applied and the criteria for failure to be used in the determination of the PSA.

The differences between standard fire test criteria do not, in practice, result in the results of a testing programme to one Standard being more reliable than those to another. Because of these differences, however, it is not recommended to derive product application data based on test results to a mix of fire test Standards, unless appropriate professional advice is sought.

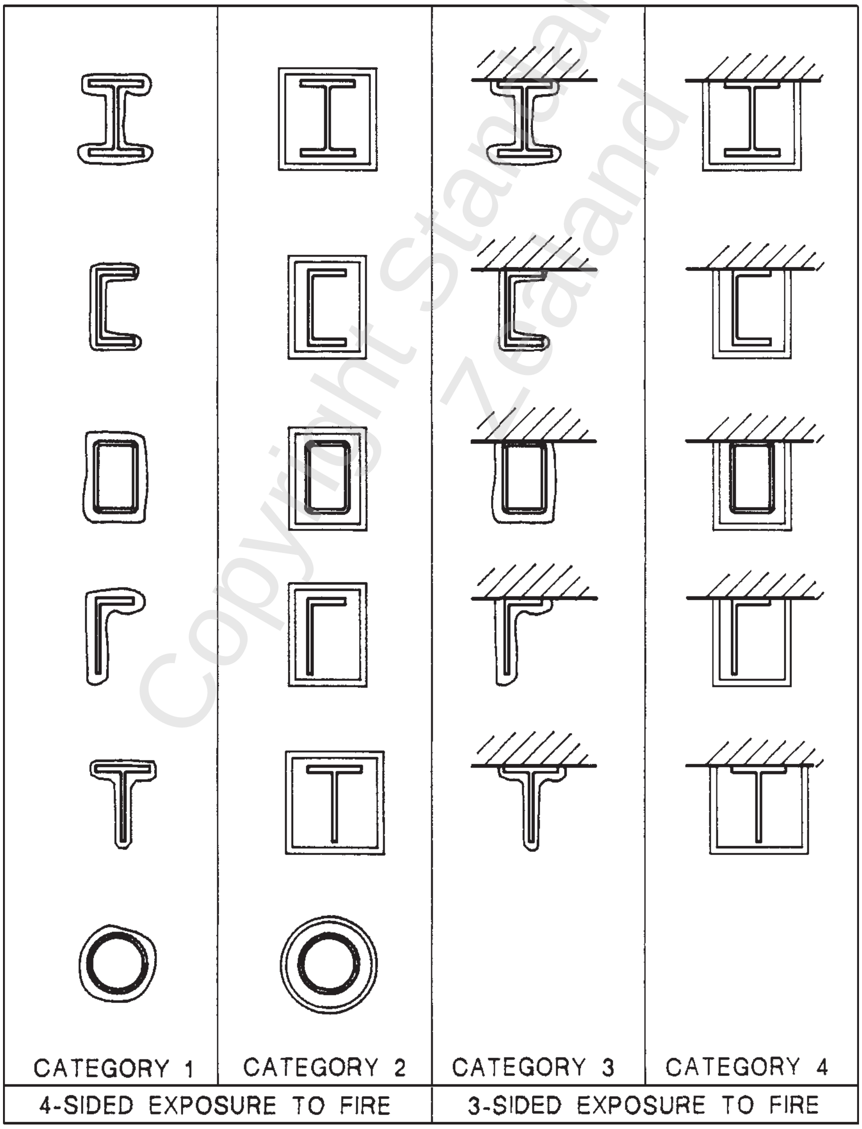

Figure C11.2 – Categories of configuration

Demonstration of stickability in accordance with one of these Standards (e.g. with 6.4.1.2 of AS 1530:Part 4) means satisfying the conditions under which loading of members may be waived for some fire tests based on the results of other tests on loaded members. The tests used to demonstrate stickability must have been continued to failure under the structural adequacy criterion and must have shown that the material remains substantially in place. For a test series, this must be demonstrated for the members with the smallest and greatest thicknesses of fire protection material applied. Some of the Standards (e.g. BS 476) are quite explicit on what constitutes material remaining "substantially in place".

C11.3 DETERMINATION OF PERIOD OF STRUCTURAL ADEQUACY

Three alternative methods of determining the period of structural adequacy (PSA) are permitted. These methods provide a tiered approach in which some methods require more calculation but give greater flexibility than others.

Method (a) provides for calculation of the structural behaviour based on measured steel temperatures. Temperatures may be obtained from a single fire test result or may be interpolated from temperature data obtained from a series of fire tests. The background and justification of this approach are given in (11.7-11.10).

Method (b) allows the direct use of a single test result where all test conditions are equal to or more severe than those for the member in service.

Method (c) allows alternative rational approaches, provided that the temperature data is confirmed by fire tests. Some guidance on suitable methods is provided in (11.11, 11.12, 11.14, 11.17, 11.21). Note that use of the UK codified provisions (11.14) is an acceptable option under this clause. A useful commentary to the UK provisions has been published by the Steel Construction Institute (11.22).

C11.4 VARIATION OF MECHANICAL PROPERTIES OF STEEL WITH TEMPERATURE

The relationship between yield stress ratio and temperature is based on regression analyses of data using elevated-temperature tensile tests conducted in Australia and the UK (11.8). The variation of the modulus of elasticity with temperature is based on CTICM recommendations (11.13).

C11.5 DETERMINATION OF LIMITING STEEL TEMPERATURE

The relationship between load ratio and the limiting steel temperature is applicable to members with either three-sided or four-sided exposure to fire, acting either as beams or columns. For members with four-sided exposure to fire, which have nominally uniform temperature throughout the steel, this relationship may be derived directly from the relationship between the yield stress ratio and the temperature. This formula may also be applied to members with three-sided exposure to fire, provided that the correct value of steel temperature is chosen, as specified in 11.6. The background and justification for this approach are given in (11.9).

The use of a more lenient strength reduction factor for emergency conditions (ϕ_{fire} = 1.0 for most applications) reflects an accepted lower safety index for individual member performance under fully developed fire conditions. This modification, introduced by this Amendment, makes the approach for structural steel and composite steel/concrete members in fire consistent with that applied to reinforced concrete (see clause 3.6 of NZS 3101 (13.13)), timber (see section 6.9 of (11.21)) and composite slabs on profiled steel decking (see section 3.4 of (11.20)). The specification of $\phi_{fire} = (\phi/0.85)$ makes the reduction in factor of safety constant across all actions and components, consistent with the approach for earthquake embodied in the Ideal Capacity Factor (see clause 1.3).

Amd 1
June '01

C11.6 DETERMINATION OF TIME AT WHICH THE LIMITING TEMPERATURE IS ATTAINED FOR UNPROTECTED MEMBERS

The formulae given are based on regression analyses of British temperature data for unprotected steel (11.14). The upper limit of application of these provisions has been increased to reflect the much greater knowledge of unprotected steel member behaviour in severe fires that is now (1997) available, e.g. as reported in (11.24).

C11.7 DETERMINATION OF TIME AT WHICH THE LIMITING TEMPERATURE IS ATTAINED FOR PROTECTED MEMBERS

Beams with three-sided exposure to fire develop a temperature gradient over the depth of the steel beam, and the temperature used must be the average measured by thermocouples located as shown in the appropriate Standard, which gives an average with greater weighting of the bottom flange temperature. For columns with a three-sided exposure to fire, a more conservative approach is necessary, and the temperature must be taken as that of the face furthest from the wall. This temperature can be taken as the temperature of a member with four-sided exposure to fire with the same Section Factor.

Temperature data for protected steel members may be obtained either from a single fire test or from regression data based on a series of fire tests. Such regression data are now available for many fire protection materials in Australia and are published in (11.2). The basis of the development of the regression equation and comparisons with the results of the tests are published in (11.7).

A group of members is defined as one where all members have the same configuration category, within the limits given by 11.9.

Equation 11.7.3.1 allows the use of regression data determined for a range of steel temperatures, as described in (11.7). If all the data is obtained for a limiting, constant steel temperature, then only the first 3 terms of equation 11.7.3.1 are applicable. The UK procedure, with regard to the derivation and use of regression data, takes this approach and uses a constant steel temperature of 550 ^{o}C. This approach, as detailed in (11.19), is equally applicable to New Zealand use. The fire protection data presented in section 7 of the HERA Fire Protection Manuals (11.18) is based largely on the UK data (11.19).

C11.8 DETERMINATION OF PSA FROM A SINGLE TEST

This method can be used only where the prototype, as tested, incorporates conditions which are equal to or more severe than those of the member in service, particularly in relation to the span, load restraint, support conditions, Section Factor, and the thickness of insulation. In practice there are likely to be differences in load and span, as it is usually necessary to test a shorter span than that of the in-service situation. Testing of a shorter span than that used in service, with the same support conditions, will comply with 11.8(e).

It should be noted also that no alteration can be made under this clause to the time-temperature response as a result of differences in thickness of insulation or Section Factor.

C11.9 THREE-SIDED FIRE EXPOSURE CONDITION

This clause makes an allowance for differences in the concrete or masonry construction which is in contact with the steel member and the resulting effects on steel temperatures. It is recognized that the steel temperature is not very sensitive to such differences, and within certain ranges there is no need to allow for their effect on temperatures. Where these ranges are exceeded, the members must be treated as separate cases of three-sided fire exposure.

C11.10 SPECIAL CONSIDERATIONS

Conservative approaches are given for ensuring the fire resistance of connections and web penetrations.

The fire resistance of composite slabs cast onto profiled steel deck should be calculated from (11.20).

REFERENCES TO SECTION C11

11.1 Bennetts, I.D., Proe, D.J. and Thomas, I.R. 1987. Guidelines for Assessment of Fire Resistance of Structural Steel Members. Published by the Australian Institute of Steel Construction, Sydney.

11.2 Proe, D.J., et.al. 1990. Handbook of Fire Protection Materials for Structural Steel. Published by the Australian Institute of Steel Construction, Sydney.

11.3 Standards Australia. 1990. Fire Tests on Building Materials and Structures, Fire Resistance Test of Structures. AS 1530.4. Sydney.

11.4 International Organization for Standardization. Fire-Resistance Test-Elements of Building Construction. ISO 834.

11.5 American Society for Testing and Materials. 1988. Method for Fire Tests of Building Construction and Materials. ASTM E119.

11.6 British Standards Institution. 1987. Fire Tests on Building Materials and Structures: Method for Determination of the Fire Resistance of Elements of Construction (General Principles), Methods for Determination of the Fire Resistance of Load Bearing Elements of Construction, Methods for Determination of the Fire Resistance of Non-Loadbearing Elements of Construction, and Determination of the Contribution of Components to the Fire Resistance of a Structure. BS 476:Parts 20, 21, 22, and 23. London.

11.7 Bennetts, I.D., Proe, D.J. and Thomas, I.R. 1986. Simulation of the Fire Testing of Structural Steel Elements by Calculation–Thermal Response. Steel Construction, Vol. 19, No. 3, pp. 2-28.

11.8 Proe, D.J., Bennetts, I.D. and Thomas, I.R. 1986. Simulation of the Fire Testing Structural Steel Elements by Calculation–Mechanical Response. Steel Construction, Vol. 19, No. 4, pp. 2-18.

11.9 Proe, D.J., Bennetts, I.D. and Thomas, I.R. 1986. Simulation of the Fire Testing of Structural Steel Elements by Calculation–Overall Behaviour. Steel Construction, Vol. 19, No. 4, pp. 19-32.

11.10 Bennetts, I.D., Proe, D.J. and Firkins, A. 1984. Steel Buildings Design for Fire Performance. Seminar Notes, Australian Institute of Steel Construction, Sydney.

11.11 Working Party on Fire Engineering. 1989. Fire Engineering for Building Structures and Safety. Published by The Institution of Engineers, Australia, National Committee on Structural Engineering.

11.12 CIB W14. 1986. Design Guide: Structural Fire Safety. Fire Safety Journal, Vol. 10, No. 2.

11.13 'Methode de Prevision par le Calcul du Comportement au Feu des Structures en Acier'. (DTU). 1982. Construction Metallique, Vol. 19, No. 3, pp. 39-79.

11.14 British Standards Institution. 1990. The Structural Use of Steelwork in Building:Code of Practice for the Fire Protection of Structural Steelwork. BS 5950:Part 8. London.

11.15 Building Industry Authority. 1992. Approved Documents for Fire Safety, Comprising C2: Means of Escape, C3: Spread of Fire and C4: Structural Stability in Fire , plus References, Definitions and Appendices. Wellington.

11.16 Standards New Zealand. 1989. MP 9. Fire Properties of Building Materials and Elements of Structure. Wellington.

11.17 ECCS. 1985. Design Manual on the European Recommendations for the Fire Safety of Steel Structures. ECCS Publication No. 35. Published by the European Convention for Constructional Steelwork, Brussels.

11.18 HERA. 1996. Fire Protection Manuals, Sections 7 and 8, 3rd Edition. Published by HERA, Manukau City.

11.19 Association of Structural Fire Protection Contractors and Manufacturers Ltd. 1992. Fire Protection For Structural Steel in Buildings, Rev. 2nd Edition. Published by ASFPCM et. al. Aldershot.

11.20 HERA. 1995. Calculation of the Fire Resistance of Composite Concrete Slabs with Profiled Steel Sheet. HERA Report R4-82. Published by HERA, Manukau City.

11.21 Buchanan, A.H. (editor). 1994. Fire Engineering Design Guide. Published by the CAE, University of Canterbury, Christchurch.

11.22 Lawson, R.M. and Newman, G.M. 1990. Fire Resistant Design of Steel Structures. Published by the Steel Construction Institute. Ascot, England.

11.23 Building Industry Authority. 1992. The New Zealand Building Code. Wellington.

11.24 Clifton, G.C. and Forrest, E. 1996. Notes Prepared for a Seminar on Design of Steel Buildings for Fire Emergency Conditions. HERA Report R4-91. Published by HERA, Manukau City.

C12 SEISMIC DESIGN

C12.1 DEFINITIONS

Member category is defined in performance terms in 12.2.5, following the definition of structure category in 12.2.3. These definitions apply principally to design of seismic-resisting structural systems and their members. They also apply, in certain specified instances, to design of associated structural systems and their members and to design of structural systems and their members which are not subject to inelastic earthquake effects but which are subjected to inelastic effects through being designed by elastic analysis incorporating moment redistribution.

Figure C12.1 illustrates the definition of "collector beam" and "storey" as given in 12.1. This clarifies what constitutes a storey for the purposes of this section. Note that the storey must be chosen as appropriate to the mechanism of inelastic response selected for the seismic-resisting system; i.e. if a horizontal beam supports loading from an additional floor to those shown in figure C12.1(a) and this additional floor is positioned between the levels of collector beam shown, but does not itself contribute to the seismic-resisting system's lateral capacity (in the case of a frame shown in figure C12.1(a) it does not act as a collector beam), then it does not add an extra storey to the structure for application of this section.

Structures such as masts, poles and the like with no logical structural divisions and with uniform or near-uniform mass distribution are considered single storey for the purposes of applying section 12. Some of the detailed provisions of 12.12 will not be applicable, in general, to such structures; although the philosophy and general provisions of section 12 as a whole are applicable.

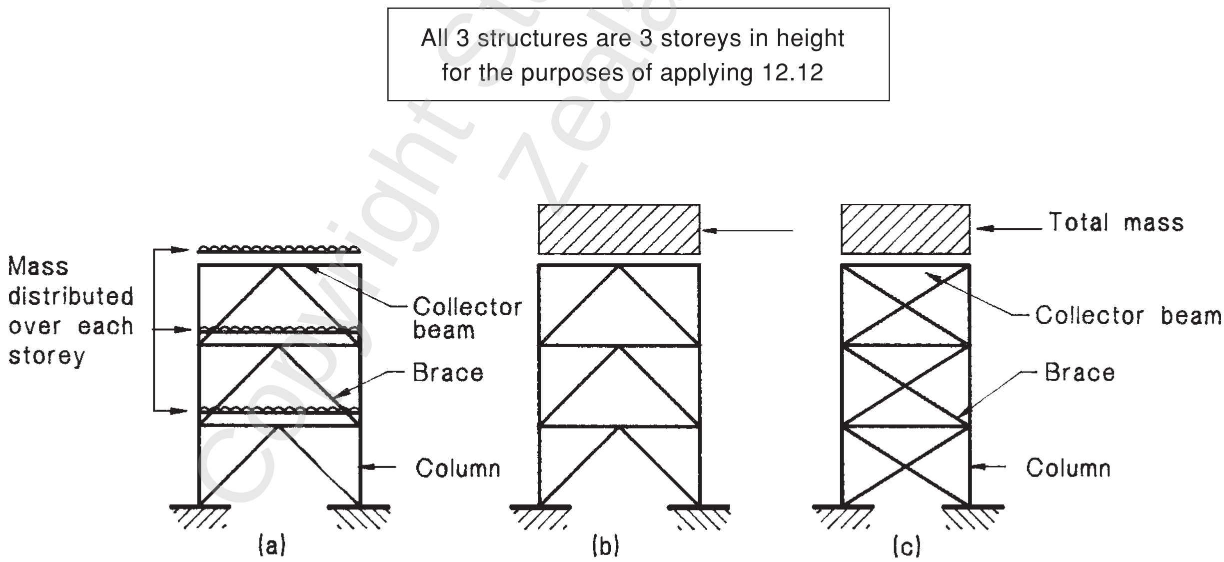

Figure C12.1 – Definition of storey height for different arrangements of applied loads

C12.2 GENERAL DESIGN AND ANALYSIS PHILOSOPHY

C12.2.1 Scope

In general terms, the material contained in section 12 of this edition is derived from the 1989 edition. These provisions have been expanded, in some instances clarified and considerably updated, reflecting very significant advances in the level of understanding of seismic-resisting systems and members that have been made (12.1, 12.3, 12.49) since the 1989 edition was prepared. These advances have led to improved design procedures, details of which are presented, where appropriate, in this section or referenced in the commentary. The principal source of these design procedures is HERA Report R4-76 (12.3).

Section 12 of the 1989 edition presented the first detailed seismic design requirements for structural steelwork in the format of a New Zealand Standard. Publication of this edition and of (12.3) has enabled shortcomings of the 1989 material to be rectified. The most significant shortcoming related to the format of the 1989 edition as a whole, with the general member design provisions in that edition contained in a separate document (AS 1250:1981) from the seismic and composite design provisions. This made it impossible to properly cross-reference between the seismic design requirements and the member design provisions as a whole.

In addition to this change, other significant developments in section 12 include:

(a) Detailed coverage is extended to associated structural systems and, through 4.5, to members of structural systems subject to inelastic demand through moment redistribution;

(b) The capacity design procedure is made more straight-forward to apply, allowing the design capacity of primary and secondary members to be used directly in design;

(c) Material requirements and overstrength factors have been revised in light of current research and the introduction of new grades of steel (12.2, 12.3, 12.49, 12.50, 12.55);

(d) A detailed clause on methods of analysis and design has been added, providing cross-referencing guidance into relevant provisions of the Standard for all structural systems subject to inelastic action;

(e) The requirements for member restraint have been clarified;

(f) Requirements for connection design have been updated, clarified and expanded in scope;

(g) Weld design is to AS/NZS 1554.1 (12.4) instead of to NZS 4701 (now superseded);

(h) Lateral deflection requirements have been made more concise and directly attainable from NZS 1170.5 for both the ultimate and the serviceability limit states;

(j) The provisions for derivation of design seismic load and the requirements for design and detailing of concentrically braced frames have been expanded in scope in both the 1992 and then the 1997 editions;

(k) Fabrication requirements for yielding (inelastic) regions have been made more concise and tied into the section on fabrication;

(m) Terminology changes to system and member categories have been made in the 1997 edition (from those used in the 1992 edition) to more logically describe the nature and extent of ductility demand on the system and its members.

These developments, in conjunction with the detailed design provisions in (12.3), considerably enhance the scope of application and ease of use of the seismic provisions for structural steelwork design in this Standard.

C12.2.2 Structural performance factor and structural ductility demand

C12.2.2.1 Structural performance factor

The changes to S_p are to align with NZS 1170.5 and be consistent with treatment of systems of similar ductility from NZS 3101:2006. See C4.4 of that Standard for the background to the new requirements for S_p.

(Text deleted)

Amd 2
Oct. '07

C12.2.2.2 Structural ductility demand

Clause 12.2.2 is a general clause directing designers to the need to provide an adequate level of ductility to any category of structural system, from fully ductile (category 1) to nominally ductile (category 3). Even elastic (category 4) structural systems (either seismic-resisting or associated systems) must not exhibit brittle modes of behaviour and the provisions of this Standard are aimed at achieving non-brittle behaviour.

Other methods of achieving the appropriate level of ductility may be used in lieu of those specified herein. If an alternative method is used, however, the onus is on the design engineer to ensure that the method can deliver the performance levels (design capacity, ductility capacity) that are required.

C12.2.3 Classification of structural systems

Note that 12.2.3, in whole or in part, applies to all structural systems which are subject to inelastic demand.

This clause requires a seismic-resisting system to be classified into one of 4 categories [1, 2, 3 or 4]. The final decision as to which category to adopt is dependent on the ductility capability of the structural system, the detailing constraints, the client's preference, a comparison of the associated costs and member sizes associated with each given category and the scope of the design method available coupled with the ease of design.

In the case of a moment-resisting framed seismic-resisting system, the structural form is inherently capable of resisting any level of global ductility demand. An eccentrically braced frame is most commonly used as a fully ductile (category 1) seismic-resisting system, but can equally well be a category 2 system. A concentrically braced frame exhibits less desirable inelastic behaviour, thus necessitating limits on the structural ductility factor that can be utilized; these limits are contained in 12.12.

All steel structures designed and detailed in accordance with this Standard will exhibit non-brittle behaviour and hence some structural ductility capacity. The level of inelastic action expected for category 3 structural systems under ultimate limit state loading or deformation is very minor. A category 3 seismic-resisting system responds with only nominal inelastic action, therefore, but can be designed for a significantly lower level of seismic base shear than that associated with elastic response. Such a system is likely to be rapidly and cost-effectively repairable after an earthquake of severe seismic intensity.

For seismically isolated structures (e.g. base isolated) the same classification system is used. In this case the design engineer, through a special study which takes into account the nature of the earthquake and the degree of isolation provided by the proposed seismic isolation system, must ascertain the level of inelastic demand that the seismically isolated structural system should dependably be designed to resist and select one of the 4 categories accordingly.

C12.2.3.1 Categories of ductility demand

Amd 2 Oct. '07

These changes are to make the performance requirements more quantifiable, for compliance with NZS 1170.5.

C12.2.3.3 Application of structural classifications

If a multi-storey seismic-resisting system above the critical height is designed for category 3 response, care must be taken to avoid soft-storey formation in the lower half height of the system, which is where inelastic demand will typically be concentrated. This can be achieved by applying a pseudo-capacity design approach to the lower half height of the system in accordance with the provisions of this Standard, in conjunction with a rational design procedure and this is recognized in 12.2.6. For moment-resisting frames, a suitable procedure is given in sections 5.4 and 7 of (12.3), while for concentrically braced frames, sections 16.4 and 17.4 of (12.3) provide the required guidance.

The changes are made to ensure consistent performance, to meet the requirements placed on the materials standards by NZS 1170.5 and to ensure that the rotation demand on members of the seismic-resisting system will not exceed the capacities of tables 4.7 in the cases where an explicit check for rotation using this table is not required.

The restriction on the system category that can be used for two seismic-resisting systems in a concurrent or dual configuration introduced in 12.2.3.3 brings into the Standard an existing provision in HERA Report R4-76 (12.3) and in the existing commentary clause for Dual Systems, being the third paragraph of Commentary Clause C12.13.

Amd 2 Oct. '07

C12.2.4 Structural displacement ductility demands

Table 12.2.4 presents the structural ductility demands expected on the structural system as a whole for a given category. These demands are based on the requirements of NZS 4203:1992. A fully ductile, or category 1 system, is required to achieve a structural ductility demand of 6 over 8 load reversals without the base shear capacity of the system being reduced by more than 20 % (12.5). The corresponding level of structural ductility demand expected from category 2 and 3 systems is 3 and 1.25 respectively.

The change-over value between fully ductile and limited ductile response is altered from the 1989 edition of this Standard (from $\mu = 2$ to $\mu = 3$). This returns it to half the value adopted for fully ductile systems (μ = 6), as specified in the 1976 edition of NZS 4203:1992.

The period dependance of μ for short period category 2 structures, as given by Note (5) to Appendix B, is in accordance with the philosophy and provisions of NZS 4203:1992.

Amd 2 Oct. '07

The changes are made to ensure consistent performance, to meet the requirements placed on the materials standards by NZS 1170.5 and to ensure that the rotation demand on members of

Amd 2 Oct. '07

the seismic-resisting system will not exceed the capacities of tables 4.7 in the cases where an explicit check for rotation using this table is not required.

C12.2.5 Classification of members

Members of structural systems are classified into 4 categories in the same manner as for complete systems. In the case of seismic-resisting systems, the ductility demand on the primary seismic-resisting members will be greater than that on the system as a whole. The ratio of maximum ductility demand on a primary member to that on the system is mainly dependent on the structural form and to a lesser extent on the characteristics of the earthquake.

Detailed information on this topic for moment-resisting frames and limited information for eccentrically braced frames is provided by MacRae (12.6). For internal moment-resisting frames, the ratio is typically 1.5 (12.1, 12.6). For perimeter moment-resisting frames, where the tributary floor area for deriving seismic weight is typically 2-5 times the tributary area for deriving design gravity loading, the ratio decreases to around 1.2. In these moment-resisting frames, as with all seismic-resisting systems, only a few of the primary seismic-resisting members are likely to be subjected to the maximum anticipated member ductility demand during a severe earthquake.

In a category 1 eccentrically braced frame, the primary seismic-resisting element is the active link (refer to definition, 1.3). A (displacement) ductility demand of 6 on the eccentrically braced framed system can generate a member (displacement) ductility demand of around 40-50 in an active link responding in a shear mode. Such inelastic demand is readily sustainable in an active link designed and detailed in accordance with the provisions of this Standard.

Primary and secondary seismic-resisting members designed and detailed to the requirements of this Standard will achieve the ductility demands required and the designer is not required to explicitly determine ductility demands on these members, unless the provisions of 4.7 are used. It is important for researchers in this area to work to established, performance oriented guidelines, however, and the loading regimes specified in, for example, ANSI/AISC 341-05 are there principally for researchers, in order that members and sub-assemblages may be experimentally tested under conditions that reflect that Standard's requirements for structure and member ductility. An improved inelastic cyclic loading regime, from which the code performance levels of ductility capacity for members and sub-assemblages can be readily derived, is available (12.1) and has been widely used in experimental research in recent times (12.6, 12.7, 12.8, 12.9, 12.49).

C12.2.6 Relationship between structure category and member category

Application of the capacity design procedure concentrates ductility demand into the primary (seismic-resisting) members of the structural system and provides considerable protection against excessive ductility demand occurring in the secondary members.

Studies undertaken, e.g. (12.6) indicate that, in general terms, application of the capacity design philosophy will provide a high degree of protection against inelastic demand in the secondary members of the seismic-resisting system. This is recognized in table 12.2.6, where the secondary members are assigned 2 categories of ductility demand less than that assigned to the primary members, except where inelastic demand could be expected in the secondary members because of the form of the structural system. The distinction is introduced through note 2 to the table. More detailed requirements for specific structural forms of seismic-resisting systems are given in notes 3 and 4 to table 12.2.6.

The relationship between structure category and member category for all framed seismic-resisting systems is given in figure 3.2 of (12.3).

Amd 1 June '01

Depending on the degree of foundation flexibility, the extent of ductility demand in secondary members whose position makes them susceptible to inelastic action (e.g. column bases with rigid connections into the foundations) may sometimes be greater than that allowed for in table 12.2.6. ➤

Amd 1 June '01 The two changes introduced in Amendment No. 1 will ensure that the ductility demand on the secondary member does not exceed the ductility capability of that member.

The changes are made to ensure consistent performance, to meet the requirements placed on the materials standards by NZS 1170.5 and to ensure that the rotation demand on members of the seismic-resisting system will not exceed the capacities of tables 4.7 in the cases where an explicit check for rotation using this table is not required.

These provisions have been developed on the assumption of rigid plastic deformation and rigid foundations, both of which generate maximum plastic rotation demand in the elements of the superstructure subject to inelastic demand.

Short steel members may yield in shear, rather than, or in addition to, flexure. Studies on steel shear links indicate that principally shear yielding occurs in members with a length less than $3M_p/V_w$. These members have a lower strength than those subject to flexural yielding only. Members detailed as active links according to clause 12.11 have the design strength given in clause 12.11.3.5 and they can undergo rotations greater than that for regular members in flexure according to clause 12.11.3.3.1.

For case number 4, the required column category has been determined through calculating the plastic rotation associated with inelastic response being concentrated into the lowest storey, for a building at the critical height, when the inelastic drift limit from NZS 1170.5 is reached at the top of the frame and selecting the required category from table 4.7(3) for a design axial compression force of $0.5\phi N_s$. In this case Category 2 is required. Use of table 4.7 gives sufficient reserve of strength to meet the 2500 year criterion set by NZS 1170.5.

For case number 5, there is no inelastic demand on the columns at the design ultimate limit state and the 2500 year criterion is accommodated principally through the use of $S_p = 0.9$ for these non brittle elastically responding systems. Additional protection for multi-storey building columns is afforded by the requirements for column design, which, for axial loads $> 0.15\phi N_s$, will dictate that the column cross section elements are typically below the yield slenderness limit for section efficiency and hence will provide some dependable inelastic rotation capacity. Amd 2 Oct. '07

C12.2.7 Capacity design

Capacity design procedures are required as part of the limit state design method for the ultimate limit state where the ductility demand on the structural system is sufficient to require careful control of the position of yielding regions within the system as a whole and the individual members.

Note that capacity design procedures may be applied to associated structural systems as well as to seismic-resisting systems.

The 1992 edition of this Standard introduced explicit guidance on the use of the ideal capacity of a secondary member to resist the capacity design derived design actions, in accordance with the philosophical requirements of NZS 4203. The 1997 revision does not change the philosophy of this approach, nor does it change the ratio of design action to design capacity for the secondary elements. It simply applies the concept of ideal capacity in a more practical and direct manner, allowing the design capacity (ϕR_u) to be used for the design of all members and elements of a seismic-resisting system. The change in design approach is described in commentary clause C1.3, under the definition of the *Ideal Capacity Factor* and the procedures given in (12.3) incorporate the revised approach.

The design capacity of secondary members comprising structural materials not covered by this Standard should be used to resist the capacity design derived design actions as determined by this revision (e.g. as is specified in (12.3). (Alternatively, these design actions can be factored by the Ideal Capacity Factor (1.0/0.9) and then resisted by the ideal capacity of these secondary members).

C12.2.8 Overstrength

The overstrength factors presented in tables 12.2.8 are based on the work of Erasmus and Smaill (12.2) and later work by Smaill (12.50) and data from BHP New Zealand Steel (12.55). Refer to (12.3) for a background to these values.

The material variation factor is derived as the ratio of the 97.5 percentile yield stress to the 2.5 percentile yield stress. The strain hardening factor is based on (12.2) for Grade 250 or 350 steels. Steels of up to 300 MPa nominal yield stress are suitable for use in the yielding regions of category 1 members (12.51) (this includes the BHP 300 PLUS and BHP New Zealand Steel 300 MOD steels).

For Grade 450 steels, as applicable to category 3 members, the inelastic demand will be very small and any strain hardening increase will be negligible. In the absence of any explicit data for Grade 450 steels, the same material variation factor as derived for Grade 350 steels has been adopted.

In table 12.2.8(2), a distinction is made between non-composite and composite active links. The latter will have increased overstrength capacity, due to the influence of the composite action. A factor of 1.1 is included to allow for this. This value is derived from (12.39 – 12.42).

C12.2.9 Damping values and changes to basic design seismic load

C12.2.9.1 Damping values

The values for viscous damping are obtained from (12.10). They illustrate the considerable differences in damping levels possible between different types of connection and structural form.

The values of damping for ductile behaviour are applicable to any levels of anticipated inelastic response – i.e. they apply to category 3, 2 or 1 seismic-resisting systems and may also be applied to category 4 systems at the discretion of the designer. Note that a distinction is made between values to be used for the ultimate and the serviceability limit states.

For unclad fully welded structures responding elastically, values of damping as low as 0.5 % have been measured. The value of 2 % given in table 12.2.9 for such structures will be suitable for all typical applications, however for structures where elastic response is imperative, a lower value of initial elastic damping may need to be considered in assessing the design level of seismic loading. Guidance is given in (12.10).

The average value of damping applies to mixed categories of construction, where some bolted connection movement under severe seismic conditions might reasonably be expected. For example, a category 3 eccentrically braced framed seismic-resisting system, with shop welded sub-assemblages which are site bolted together, will have a design value of initial viscous damping of 7.5 %, which can be used in 12.2.9.2.

Note that the damping values given in table 12.2.9 are **not** appropriate for use in design for building serviceability response under wind loading, which is associated with much lower lateral movement and therefore much lower levels of damping.

C12.2.9.2 Influence of damping values on the ultimate limit state design seismic load for category 3 and 4 structural systems

The equation presented in 12.2.9.2 is a simplified version of an equation relating the response spectral acceleration for a given period and given percentage of critical damping to the response spectral acceleration for the same period and 5 % of critical damping. It is obtained from (12.11).

The level of initial viscous damping exerts a major influence on the elastic response of a structure to seismic loading and a lesser, but still significant influence on the nominally ductile response. As a structure responds to seismic loading in an increasingly inelastic manner, the level of damping becomes less significant in determining the structural response (12.12).

Therefore making allowance for the damping value in determining the design seismic load is valid for category 3 and 4 systems, which respond in a nominally ductile or elastic manner. However for category 1 or 2 systems at the ultimate limit state, the response is primarily inelastic and changes in damping have little influence. To use the provisions of 12.2.9.2 in determining the design seismic load for category 1 or 2 systems would therefore alter the ultimate limit state design seismic load more than is justifiable.

C12.2.9.3 Influence of damping values on the serviceability limit state design seismic load for any category of structural system

Elastic or nominally ductile behaviour is required from all seismic-resisting systems at the serviceability limit state, hence the adjustment in design seismic load given by 12.2.9.2 is applicable to any category of system for the serviceability limit state.

The seismic provisions of NZS 1170.5 will typically produce ultimate limit state design seismic loads associated with $\mu = 6$ that are similar in magnitude to the serviceability limit state design seismic loads associated with $\mu = 1.0$ or $\mu = 1.25$. Thus some slight yielding of category 1 seismic-resisting systems may be expected at the serviceability limit state, allowing use of the higher damping values associated with the “inelastic” classification from table 12.2.9.

For category 2, 3 or 4 seismic-resisting systems, the design seismic loads for the ultimate limit state will considerably exceed those for the serviceability limit state, leading to elastic behaviour at the serviceability limit state.

C12.3 METHODS OF ANALYSIS AND DESIGN

C12.3.1 Scope

Clause 12.3 fulfils a cross-referencing role in directing designers to appropriate provisions of this Standard for the design of seismic-resisting or associated structural systems. As such it overcomes one of the most serious problems associated with the 1989 edition of the Standard, namely a lack of adequate cross-referencing between the provisions for seismic design and the other provisions of the Standard. Editorial changes from the 1992 edition to this edition have further improved the useability of this section.

The application of 12.3.1 is self-explanatory.

C12.3.2.3 Assessment of P-delta effects

C12.3.2.3.1

The most important of these effects are those resulting from the displacement of the structural system as a whole; i.e. the $P-\Delta$ effects as defined in 1.3 and 4.4.2 (refer to 4.4.2 and Commentary Clause C4.4.2). In design of category 1 and 2 or 3 seismic-resisting systems, in accordance with NZS 1170.5, the Δ that must be considered is the inelastic deflection, Δ_i, which is determined as $\mu\Delta_e$, where Δ_e is the elastic deflection derived from an elastic analysis. Assessment of the appropriate $P-\Delta_i$ effects is especially important in the design of category 1 or 2 moment-resisting framed seismic-resisting systems.

The assessment and design treatment of design actions generated by P-Δ_i effects must be in accordance with the Loadings Standard. In most instances, this will be to NZS 1170.5 and the provisions of 12.3.2.3 and C12.3.2.3 relating to selection of μ are written for use in accordance with the $P-\Delta_i$ provisions of NZS 1170.5. Other Loadings Standards may be used in accordance with this Standard, however they must be in limit state format and should include specific provisions for assessing and designing for $P-\Delta_i$ effects in seismic-resisting systems.

C12.3.2.3.2

Because of the importance of μ in determining Δ_i effects to NZS 1170.5, it is important that the value of μ used in the final determination of $P-\Delta_i$ effects is that associated with the actual ductility demand on the as-designed seismic-resisting system (μ_{act}). Various factors in the design process, especially with moment-resisting frames (12.3, 12.6) will ensure that the as-designed system is always stronger than that just necessary to resist the design seismic base shear associated with μ_{des}.

For moment-resisting framed seismic-resisting systems of rigid construction, μ_{act} may be taken as 0.75 μ_{des} for internal frames and 0.85 μ_{des} for perimeter frames in preliminary design. This will also be generally conservative in final design of these systems. For category 2 systems with $\mu_{des} > 2.0$, it is likely that $\mu_{act} > 1.5$ will apply.

In final design, an assessment of μ_{act} for any seismic-resisting system can be made simply and with sufficient accuracy by equating the design overturning moment with the base moment resistance of the seismic-resisting system as sized:

(a) The design overturning moment is that calculated from the equivalent static design base shear, V, acting about its centroid (taken as $h_{eq} = 0.64H$, where H is the height of the seismic-resisting system above its base – see 1.3). The magnitude of V includes the structure ductility factor, μ, for which the final value, μ_{act}, is sought. The value of h_{eq} is derived from (12.56).

(b) The base moment resistance is calculated from the design column axial resistance as one or more tension / compression couples, multiplied by the appropriate lever arm, plus the sum of the design moment capacity of each column at the base (reduced by the level of design axial force on the column).

(c) The external design overturning moment and internal structure overturning moment resistance are equated, from which μ_{act} is obtained. This is then used in assessment of $P-\Delta_i$ effects and (Δ_i / μ_{act}) gives the elastic stiffness.

The procedure is elaborated on below, with reference to the two-bay moment-resisting frame shown in figure C12.3.2.3. This frame assumes regularity in member sizes and loading.

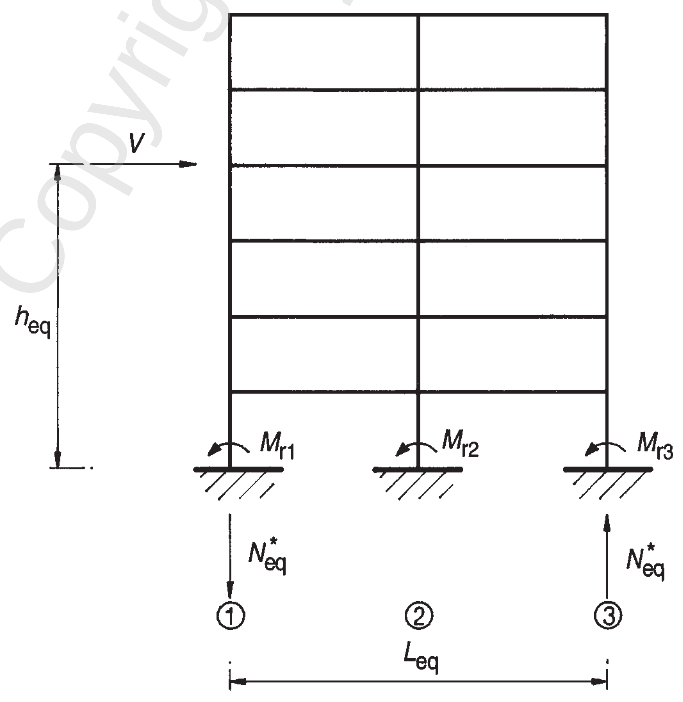

Figure C12.3.2.3 – Calculation of μ_{act} for a regular two-bay moment-resisting frame

N^*_{eq} is calculated for the column under both seismic-induced compression and compression due to gravity loading.

$$N^*_{eq} = \text{least of} \begin{cases} (N_{cr} - N^*_g) \quad ; \text{or} & \text{(Eq. C12.3.2(1))} \\ (\sum_1^n (V_{bo} / \phi_{oms}) - N^*_g) & \text{(Eq. C12.3.2(2))} \end{cases}$$

where

V = design seismic base shear from the Loadings Standard (function of μ)

ϕ = the strength reduction factor (see table 3.3(1))

N_{cr} = the limiting axial force on the column member from 12.8.3.1, as appropriate to its member category from 12.2.5

$\sum_1^n (V_{bo} / \phi_{oms})$ = the sum of the nominal shear forces (i.e. the overstrength shear forces divided by the overstrength factor) transferred into the column from all beam levels above the base of the seismic-resisting system

N^*_g = the design axial compressive force generated from the gravity loading associated with severe earthquake loads as specified in the Loadings Standard

M_{rn} = the nominal column section moment capacity reduced by axial force for column n. For column 1 in figure C12.3.2.3, the appropriate level of axial force to consider is $(N^*_{eq} - N^*_g)$, as the gravity load and seismic-induced force are in opposition. For column 3, the appropriate level of axial force is $(N^*_{eq} + N^*_g)$, thus M_{rn} will be effectively zero. For column 2, it is N^*_{g2}. M_{rn} is calculated from 8.3 about the member axis perpendicular to the plane of the frame (typically the x-axis).

Equating the external and internal moments gives:

$$Vh_{eq} = \phi\ (N^*_{eq}\ L_{eq} + M_{r1} + M_{r2} + M_{r3}) \quad \text{(Eq. C12.3.2(3))}$$

From this, μ_{act} can be determined directly.

For multi-bay frames where the beams spanning into an interior column differ in moment capacity, the seismic action will generate a compression component in the interior column, which must be added to N^*_g when calculating M_r for that column. The magnitude of this seismic-induced component must be determined in a rational manner by the designer. Guidance is given in (12.3).

A simple method of calculating μ_{act} is through Equation C12.3.2(4)

$$\mu_{act} = \mu \left(\frac{\sum S^*_E}{\sum \varphi R_{u,E}} \right) \geq 1.0 \quad \text{(Eq. C12.3.2(4))}$$

where

μ = the design ductility

S^*_E = design actions from analysis for the design ductility, μ, at the yielding regions of the primary members. These design actions which include redistribution were applied.

$\phi R_{u,E}$ = design capacity of the primary members at the yielding regions

$\sum$ = summation of all yielding regions in the yielding mechanism.

Amd 2
Oct. '07

Use of μ_{act} in NZS 1170.5

Once μ_{act} is determined, it is important that the increased seismic design loads resulting from the reduction in μ, from μ_{des} to μ_{act}, are applied in determining the elastic deflections to be increased in accordance with NZS 1170.5 clause 7.2.1 to give the inelastic deflections. The decreased ductility demand will also be beneficial in reducing any P-Δ effects from clause 6.5 of NZS 1170.5 and in meeting the concurrency requirements of 12.8.4.

Amd 2 Oct. '07

C12.3.2.3.3

As shown in figure C4.4.3.2, there will be no increase in design actions on the member due to $P-\delta$ effects unless the member is subject to adverse transverse loading (which for columns is only possible for category 4 column members, because of the restriction on transverse loading on category 1, 2 or 3 column members applied by 12.8.3.3).

C12.3.3.4 Maximum design actions for members of seismic-resisting systems

When the upper limit design actions on secondary members are less than the capacity design derived design actions based on primary member overstrength, the level of protection against inelastic demand afforded to the secondary members is reduced. When setting the value of μ_{act} at which μ_{max} changes from nominally ductile to elastic, then if the ratio $\frac{\mu_{act}}{\mu_{max}} \geq \phi_{oms}$, the same strength hierarchy against inelastic response in the secondary members for the overall system will be afforded by using the upper limit actions as is given by the capacity design process. If the ratio $\frac{\mu_{act}}{\mu_{max}} < \phi_{oms}$, then using the upper limit actions for the secondary members will afford less strength hierarchy protection. This approach has been used to assess the adequacy of $\mu_{act} = 1.5$ as the change-over for μ_{max} from nominally ductile to elastic, as given in the 1997 Standard, and has shown that the change-over value should be increased to $\mu_{act} \geq 1.8$. The same applies in 12.9.1.2.2 (4) (a) and (b).

Amd 2 Oct. '07

C12.3.4 Design and detailing of members and connections of associated structural systems

Sub-clauses 12.3.4.1 to 12.3.4.4 present the general approach required for design and detailing of the members and connections of associated structural systems. The clauses are presented in a general step-by-step manner, in the order in which a typical design and analysis evaluation of such a system would be made.

The level of ductility demand for design purposes on an associated structural system may be reduced from that applied to the seismic-resisting system for 3 principal reasons. The first is that most associated structural systems are more flexible and hence attract less inelastic damage for a given lateral deflection. The second reason is that associated structural systems, as a whole, do not need to resist additional $P-\Delta_i$ forces. The third reason is that loss of lateral strength of the system during a severe earthquake is not critical. The reduction in design ductility demand deemed reasonable to allow for is specified in 12.3.4.1.3.

Connections designed and detailed in accordance with this Standard and suitable design, detailing provisions (e.g. sections 8.1, 13.1 and 19.1 of (12.3), section 10 of (12.13), 12.22) will perform satisfactorily in accordance with the requirements of 12.3.4.4. Detailed experimental tests on moment-resisting connections (12.14, 12.15) and on web side plate connections (12.16) subjected to inelastic cyclic loading confirm their good performance under levels of inelastic action associated with a severe earthquake.

Amd 2 Oct. '07

Changes have been made to the maximum design actions to incorporate the 2500 year criterion from NZS 1170.5 and also to enhance the level of protection to lower ductility capacity systems. ➤

C12.4 MATERIAL REQUIREMENTS

The material requirements of table 12.4 will ensure that the grade of steel chosen for a given category of member can meet the expected inelastic demand in both seismic and non-seismic applications. These requirements must be used in conjunction with the material selection provisions of clause 2.6 in order to ensure that the member will not exhibit brittle fracture under the design loads or forces.

Amd 2 Oct. '07

Significant changes have been made to the material requirements for seismic design, for the following reasons:

(a) The requirements for elongation, yield ratio and maximum yield stress are based on the actual steel to be used. This requires a check to be made on these properties in the supply chain for the steel to go into the seismic-resisting system,

(b) Research has shown that grade 350 steel is now suitable for category 1 applications, provided it meets all of the criteria of this table.

(c) The requirement for a minimum Charpy impact energy requirement has been shown to be necessary (12.66). Steels meeting these criteria will also have good elongation performance.

C12.4.1.2 Category 2 application of cold-formed sections

Cold-formed sections cannot automatically be considered suitable for category 2 application, as their inelastic capacity is reduced by the cold-forming process and they are prone to failure under inelastic action through the heat affected zone adjacent to the weld line. Their suitability for category 2 application must therefore be ascertained by test. Cold-formed grade C350 hollow sections of Australian and New Zealand origin to AS 1163 have been so tested (12.48) and are suitable for this application, provided that the heat input into any welds made to the member is kept below 0.75 kJ/mm (12.48). This should be specified in the contract documents; compliance is then monitored through the weld procedure sheets required by AS/NZS 1554.1 (see Note (4) to Clause 1.6.3 of this Standard).

Amd 1 June '01

C12.5 SECTION GEOMETRY REQUIREMENTS

The section geometry requirements of table 12.5 and Notes (1) – (5) are taken principally from the 1989 edition of the Standard and presented in a format compatible with the general member design provisions of this Standard. The suitability of the requirements for category 1 and 2 members has now been established by comprehensive experimental testing (12.6, 12.7, 12.49). The provisions for category 3 members are comparable with those of the LRFD American specification (12.17) and individual members conforming to the category 3 provisions have demonstrated dependable inelastic performance considerably in excess of that required (12.1, 12.7 and 12.49).

The limit for category 4 circular hollow section members is based on the upper cross section slenderness limit given in the column design curves of BS 5649.7 for tubular steel lighting columns. That standard is one used in New Zealand for design of these structures. They respond in an elastic manner under severe earthquake loading and have a history of successful performance in severe earthquakes. Thus, this limit is considered a practical upper cross section slenderness limit for circular hollow sections designed for elastic response.

The cross section shape restrictions of 12.5.2 and 12.5.3 are intended to ensure that the required inelastic performance of the member is not compromised by shape-induced effects. There are different requirements for braces in concentrically braced frames (see 12.5.3) compared with other members (see 12.5.2). These differences reflect the fact that inelastic demand on the former (clause 12.5.3) is produced principally by compression action, while inelastic demand on the latter (clause 12.5.2) is produced principally by bending action, sometimes in conjunction with axial compression (or tension).

Amd 1 June '01

The Amendment No. 1 changes to web slenderness limits for rectangular and square hollow sections (case number 4 in table 12.5) are introduced because these sections are more susceptible to loss of bending strength from combined local flange and web buckling than is the case for I-sections. Because of this, their web slenderness, for a given flange slenderness, needs more stringent limits than were in table 12.5. A background to the proposed new limits is given in the HERA *Steel Design and Construction Bulletin*, Issue No. 55, April 2000.

The changes to slenderness limits of category 4 elements subject to uniform compression, both edges supported (case number 3 in table 12.5) are to bring these limits into line with those for category 4 elements subject to uniform compression, one edge supported (case number 1 in table 12.5). The ratio of this category 4 limit to the yield limit is now very similar for both cases.

Amd 1 June '01

The changes to slenderness limits for category 1 circular hollow sections arise from USA research which shows the previous limits (also based on earlier USA research), to be unconservative.

C12.6 MEMBER RESTRAINT

The member restraint requirements presented in 12.6 will ensure comparable behaviour of members of all seismic categories with those designed to the member restraint requirements presented in the 1989 edition of this Standard. Detailed experimental testing of members designed to the 1989 provisions has demonstrated the suitability of these provisions (12.8).

The format of 12.6 in the 1997 edition appears very different from that of the 1992 edition, however there has been no change to the philosophy or to the basic design provisions. What has been done is to simply the application of the clause and to bring all the requirements for restraint of members subject to earthquake loads into 12.6, rather than cross-referencing between clauses 12.6 and 5.3.

Amd 1 June '01

A background to the changes to 12.6 made in Amendment No. 1 is given in the HERA *Steel Design and Construction Bulletin*, Issue No. 51, 1999, pp. 11, 12. The moment for restraint determination is termed M^*_{res} to avoid confusion with M^*_r, which relates to the section moment capacity reduced by axial force.

C12.6.1 General

The member restraint provisions of 12.6 for members containing a yielding region are applicable to yielding regions produced principally by inelastic bending action, sometimes in conjunction with axial compression. The provisions are not applicable to braces in concentrically braced frames, where yielding is produced principally by compression action Hence braces in CBFs are excluded from the requirements of 12.6 by the first paragraph of 12.6.1.1.

C12.6.2 Restraint of category 1, 2 and 3 members

The basic philosophy behind 12.6.2 as follows:

(a) Each category 1, 2 or 3 member is divided into segments. Each segment either contains one or more yielding regions or does not contain a yielding region.

(b) The yielding region in category 1 and 2 members is defined as the length of member over which the design moment exceeds either 0.75 or 0.85 times the section design moment capacity, depending on the level of applied axial compression force, but a minimum length is specified. For category 3 members, the inelastic demand is slight and only a minimum specified length is given.

(c) Each segment containing one or more yielding regions must have full lateral restraint. This is achieved by calculation to clause 5.6, which accounts for moment gradient and section slenderness. Dynamic effects are accounted for by limiting α_m to 1.75. The provisions are made more stringent for segments containing yielding regions in category 1 members through raising the required limit on ($\alpha_m \alpha_s$) in table 12.6.2 to ϕ_{oms}.

(d) The yielding region itself must be considered as a segment (within a segment). This involves application of 12.6.2.3 to the yielding region, with the ends of the region taken as restrained for the purposes of applying 12.6.2.3 if either partial or full restraint to the cross section is not physically provided at these locations.

(e) At least one full restraint of the type specified in table 12.6.3 must apply to the critical flange of the yielding region (this can and often will be the same restraint required in (c) and (d) above) and other actual restraints must be added, as required, to keep the unsupported length of critical flange within the yielding region below that allowed from table 12.6.3. Remember that the ends of the yielding region are effectively or actually restrained for this purpose (as per (d) above).

(f) Any segments not containing a yielding region must not undergo loss of moment capacity due to lateral buckling prior to the inelastic action being developed in the yielding region. For segments in members subject to bending only, this is given by 12.6.2.5(a) and allows such segments not to have full lateral restraint, provided that this does not compromise attainment of inelastic action within the yielding region.

(g) In members subject to combined bending and significant axial actions, all segments are required to have full lateral restraint by 12.6.2.5(b). This is to avoid the destabilizing effect of *x*-axis bending and *y*-axis compression interaction in the segments which do not contain a yielding region, as well as in those that do.

C12.6.2.1 Length of yielding region

Clause 12.6.1.2.1 gives the length of yielding region. Some elasto-plastic analysis computer programs allow the criterion given to be input directly into the program, in which case the length of yielding region is output from the analysis. Not all elasto-plastic programs offer this capability and where it is not available the designer's input will be required to ensure that this clause is satisfied.

In the case of initial elastic analysis with subsequent limited redistribution in accordance with 4.5.4.2, the length of yielding region may conservatively be determined by taking M_r^* as the moment either prior to or subsequent to moment redistribution, whichever gives the greatest length of yielding region.

C12.6.3 Restraint of category 4 members

For these members, it is sufficient to require the design member moment capacity to equal or exceed the design moment at all points along the member.

C12.7 BEAMS

(No Commentary.)

C12.8 COLUMNS

C12.8.2 Effective lengths of columns and elastic stability of the structural system

C12.8.2.1 General

The provisions of 12.8.2.2 to 12.8.2.6 apply the philosophy of effective length use and determination given in 4.8.3 to seismic-resisting systems being designed for load combinations including earthquake loads. These provisions differentiate between the determination of effective length for calculation of elastic second-order effects on the frame (which requires the elastic effective length associated with the elastic buckled shape) and the determination of effective length for individual column member design (which involves different considerations). These matters are elaborated on in Commentary Clauses C12.8.2.2 – C12.8.2.6.

C12.8.2.2 Effective length of columns for calculation of the frame elastic buckling load factor

These provisions are included for completeness, but will be seldom used, as they are only necessary if this Standard is being used in conjunction with a Loadings Standard which does not address the assessment of $P - \Delta_i$ effects (NZS 1170.5 does address this matter). In such cases, assessment of $P - \Delta_i$ effects will have to be made through the use of 4.4.3.3.2(a) or (b), as appropriate, which may involve calculation of λ_c.

For a moment-resisting framed seismic-resisting system, the provisions of 4.8.3 are directly applicable. For either a concentrically or an eccentrically braced system, the elastic buckling mode will be effectively a braced one. The columns of such frames will have low slenderness ratios, hence the use of k_e = 1.0 for these members is appropriate (refer to Commentary Clause C12.8.2.4 for an explanation of this).

C12.8.2.3 Calculation of the frame elastic buckling load factor (λ_c)

The requirements account for the fact that elastic buckling may be initiated by a storey or a column, thus requiring both the sway and braced modes of buckling to be checked. The provisions provide a conservative simplification to a rational elastic buckling analysis of the whole frame.

C12.8.2.4 Effective length factor for design of individual column members forming part of a seismic-resisting system

All seismic-resisting systems are classified as sway systems, and because second-order effects are accounted for prior to individual member design, then use of k_e = 1.0 is appropriate.

There is a further reason for adopting k_e = 1.0 for such members and this lies in their stocky nature. Columns of seismic-resisting systems will have slenderness ratios (L_e/r) of typically 50 or less in the plane of the frame and hence will fail in a predominantly inelastic manner. Since the regions of a column that become inelastic have effectively zero stiffness, inelastic column buckling increases the effective elastic support provided by the incoming members at the column ends. For a braced column, where k_e < 1.0, the inelastic action increases k_e to almost 1.0 for columns with low slenderness ratios (12.19). For a sway column, where k_e > 1.0 for elastic buckling, the inelastic action reduces k_e to around 1.0 for columns with low slenderness ratios (12.19). Thus use of k_e = 1.0 is appropriate for columns of seismic-resisting systems in general. This is also consistent with the SEAOC provisions (12.20).

C12.8.2.5 Effective length of columns in dual seismic-resisting systems

This clause requires a moment-resisting framed seismic-resisting system to be treated as a sway system in design, even when acting as a dual system, for a given direction of seismic loading.

Contrast this with a moment-resisting framed associated structural system, which is considered as effectively braced by the seismic-resisting system.

C12.8.2.6 Effective length of members within a triangulated structure which forms part of a seismic-resisting system

Such members are considered braced by 4.1.2 and their effective length is not effected by their incorporation into a seismic-resisting system. Thus the provisions of 4.8.3.5 apply.

C12.8.3 Axial force and transverse load limitations on columns and braces

C12.8.3.1 Limitations on axial force

In the 1992 edition, the limitations on axial force for column members were contained in 12.8.3 and in notes to table 12.5. The 1997 edition has brought all the limitations on axial force together under clause 12.8.3.1, for ease of use. There are 3 sets of provisions, given in 12.8.3.1(a), (b) and (c). A brief background to each set of provisions is given below.

Background to 12.8.3.1(a)

The general limits given in table 12.8.1 are taken from the 1989 edition and ensure that the absolute level of axial force, especially compression, is not too high to compromise the member ability to deliver inelastic demand. These limits have been comprehensively tested (12.49).

The general limits do not apply to brace members of concentrically braced frames, in which the inelastic demand is induced by compression action, rather than by bending. This exclusion is given in note (3) to table 12.8.1, which also gives the appropriate limit on compression action for such members.

Background to 12.8.3.1(b)
The background to 12.8.3.1(b) is contained in (section 6.7.2.2 of (12.13)). The purpose of equation 12.8.3.1 is to determine whether or not a plastic hinge will form at the end of a member or somewhere between the ends. If it forms between the ends, then its position cannot be accurately determined and hence it cannot be effectively restrained. This, in turn, is unacceptable for potential yielding regions in these columns and equation 12.8.3.1 eliminates the use of category 1, 2 or 3 columns unless the yielding regions will form at the ends of the member. The most significant factor in equation 12.8.3.1 is the moment gradient (β_m). If β_m = +1, a yielding region will form at the ends, if β_m = –1 it will form at midspan. Detailed studies by MacRae (12.6) have shown that β_m = 0 should be used in columns of a seismic-resisting system to account for higher-mode effects on the moment distribution. Category 1, 2 or 3 columns which are acted on by design axial compression forces exceeding the limit given by equation 12.8.3.1 will not necessarily be able to form plastic hinge(s) with dependable rotation capacity (12.18). In this instance, the column size may be increased until equation 12.8.3.1 is satisfied, or alternatively the column may be designed for the increased seismic actions corresponding to elastic (category 4) response. This waiver is introduced in clause 12.8.3.2. This design information is readily available, involving simply the input of an extra load case in the final analysis of the seismic-resisting system. It may also have been required as a cut-off upper limit for member or connection design from 12.9.1.2.2(4).

If the clause 12.8.3.2 waiver is used, it applies only to by-passing equation 12.8.3.1 and the column must continue to comply with 12.4 – 12.6 for its appropriate category (i.e. 1, 2 or 3).

In the 1989 edition of this Standard, equation 12.8.3.1 was applied to category 1 and 2 column members only. Detailed experimental testing in 1991 and 1992 (12.49) of category 3 members under high axial compression levels and inelastic bending has shown the necessity of extending its use to category 3 members, in order to dependably develop the plastic hinge rotation capacities presented in tables 4.7, especially table 4.7(4).

Compliance with equation 12.8.3.1 in a given direction of loading means that the critical location for design of the (beam-column) member will be at its ends. Thus, under combined axial and bending actions, only the section capacity (Clause 8.3) needs to be checked, as it is the section capacity (at the member ends) that will be critical. This important fact is specified in 8.1.1.2. Note that equation 12.8.3.1 is direction-specific; compliance with it about the *x*-principal axis does not automatically ensure compliance about the *y*-axis. Guidance is given in Commentary Clause C8.1 on applying the provisions of section 8 to members subject to biaxial bending where compliance with equation 12.8.3.1 is obtained about one of the principal axes only. See also figures C8.1.1 and C8.1.2 for flow-chart based guidance.

The provisions of 12.8.3.1(b) do not apply to brace members of concentrically or eccentrically braced frames. The design of these members and their inelastic behaviour under severe seismic actions is governed principally by compression forces and by the behaviour of the seismic-resisting system as a whole. It is neither practicable nor appropriate to select a design moment gradient that would accurately reflect the real moment gradient on these members under severe seismic actions (12.3), hence it is not appropriate to apply equation 12.8.3.1.

In the case of braces in concentrically braced frames, they are the primary members and are expected to undergo compression buckling. The effects of this are catered for directly in the design procedure (see clause 12.12 and (12.3)). In the case of braces in eccentrically braced frames, studies (12.6, 12.39, 12.41, 12.42) have documented the pattern of brace behaviour under severe seismic actions. The design of the brace members to deliver the ductility performance required is adequately covered in the capacity design requirements of (12.3), coupled with the general axial force limits of table 12.8.1 and the general design and detailing requirements of clause 12.11 of this Standard.

Background to 12.8.3.1(c)

The requirements for limiting the web slenderness ratio of category 1 and 2 members as a function of the constant applied design compression load (i.e. the design axial force generated by gravity loading alone, N_g^*) that were presented in the 1989 edition are almost identical to those of equations 8.4.3.3(1) and 8.4.3.3(2) herein and have been superseded by these 2 equations, as given in table 12.8.2.

Corresponding equations for webs of category 3 members were not given in the 1989 edition, but were subsequently derived and presented in the predecessor publication to HERA Report R4-76 (12.3). These equations are presented herein, in limit state design format, as equations 12.8.3.2(1) and 12.8.3.2(2) and their suitability has been experimentally verified (12.49).

The same experimental testing (12.49) has included tests with both constant compression force and variable compression force. With regard to the influence of web slenderness on reducing the inelastic member capacity, these tests have shown that it is the level of constant compression force that is critical. For example, the performance of a member subject to inelastic bending under a constant applied axial compression force of $0.3N_s$ was the same as that of a member subject to the same extent of inelastic bending and a fluctuating compression force of $(0.3 \pm 0.3)N_s$.

Under inelastic bending and constant applied axial compression, the yielding region of a member undergoes an inelastic shortening. The mechanism involved is explained in (12.1, 12.6). This inelastic shortening is dependent on the level of compression force applied over a complete cycle of bending. Its effects are to increase the extent of local buckling, to promote the onset of combined web and flange local buckling and hence to accelerate the loss of member capacity that this combined web and flange buckling causes. It is therefore important to control the level of constant compression force applied as a function of web slenderness rather than the fluctuating component of compression force caused by earthquake action.

The level of constant compression force is best expressed by N_g^* and hence this is used in table 12.8.2 and in equations 12.8.3.2(1) and 12.8.3.2(2).

The note at the end of the clause makes the important point that any other source of constant axial force on a member under consideration must be included in N_g^*.

The application of this clause has been elaborated on to remove uncertainties as to which design actions are required to be used for each of (a) to (c).

Amd 2 Oct. '07 A new and more accurate equation for end yielding criterion (Equation 12.8.3.1) has been developed. Details are (12.63) and these replace the first paragraph of the existing commentary on the background to 12.8.3.1(b).

C12.8.3.3 Limit on transverse loading on category 1, 2 and 3 columns

Amd 1 June '01 The design bending moment from transverse loading may always be calculated assuming simple end support conditions (i.e. no joint fixity) for checking 12.8.3.3. The reason for the current clause and background to the Amendment No. 1 addition is given in the HERA *Steel Design and Construction Bulletin*, Issue No. 45, 1998, pp. 7, 8.

C12.8.4 Concurrent action on columns

Amd 2 Oct. '07 The new clause makes the concurrency provisions consistent with the concurrency requirements of NZS 1170.5 and covers all options that are allowed under the new restrictions of 12.2.3.3 on the category of each system in a concurrent configuration.

C12.9 CONNECTIONS AND BUILT-UP MEMBERS

C12.9.1 Connection design philosophy and design actions

C12.9.1.1 Design philosophy

Connections generally exhibit significantly less ductility capacity than the members they connect. This is due primarily to 3 factors. First, some of the individual elements of the connection are not very ductile (especially connectors such as incomplete penetration butt welds or one-sided fillet welds transmitting axial tension load, which exhibit virtually no ductility capacity (12.33)). Secondly, there tend to be localised regions of very high stress and abrupt force transfer within the connection, factors which inhibit ductility capacity. Thirdly, any inelastic demand within a connection tends to be concentrated into individual connection components or connectors, generating very large localised inelastic strains that cannot be resisted.

Thus connections are typically more vulnerable to sudden, complete failure under inelastic earthquake effects than the members they connect. Once such failure occurs, behaviour of the seismic-resisting system becomes undesirably unpredictable (12.21, 12.47).

The philosophy and design requirements presented in 12.9 aim to ensure that reliable connection behaviour, for any level of structural and member ductility demand, is achieved without being unnecessarily conservative. Where connection components can provide dependable ductility capacity, this is permitted, within prescribed limits, to ensure the integrity of the connection is maintained.

C12.9.1.2 Design actions for connectors and connection components

The design actions are specified in accordance with the philosophy expressed by 12.9.1.1, namely to protect potentially non-ductile connectors or connection components from inelastic demand to a considerably greater extent than the protection afforded those which fail in a ductile manner. This leads to the differences between (1) (a) and (1) (b). The different consideration of connections between primary members, or between primary and secondary members – i.e. in 1 (a, b, c) – compared with those between secondary members – i.e. in 3(a, b) – is consistent with the philosophy of 12.2.7.

Further background to sub-clause (1)

Sub-clause (1) covers design of connectors and connection components which connect elements of members subject to potential inelastic demand, through their being the primary members in a seismic-resisting or associated structural system. Sub-clause (1)(a) addresses the design of complete penetration butt welds, which exhibit a ductile form of failure (12.33) and which will exhibit equal or greater strength than the elements they connect, through the requirement from 9.7.2.7(a) that $f_{uw} \geq f_u$ for category 1 or 2 members.

Sub-clause (1)(b) addresses the design of other forms of connector or connection component which may not exhibit ductile failure mechanisms (12.25, 12.33). These forms of connector or connection component require an enhanced degree of protection against being directly subjected to inelastic demand and this is provided through application of either the capacity derived design actions or the overstrength section capacity design actions, as appropriate.

Sub-clause (1)(b)(ii) differentiates between elements forming the principal load-carrying path through the connection and other elements. An example is a moment-resisting connection between an end-plate, for example, and an I-section member in principal x-axis bending. The connection is via fillet welds to the flanges and web. The fillet welds to the flanges must be designed to resist $\phi_{oms} R_{uf}$, where R_{uf} is the nominal axial capacity of the flange. The fillet welds to the web must be designed to transfer $0.9R_{uw}$, where R_{uw} is the nominal shear capacity of the web. In contrast, for a connection to a member subjected to axial tension force, all elements of the connection form the principal load-carrying path and must resist their share of $\phi_{oms} R_u$, where R_u is the nominal section tension capacity of the member.

Further background to sub-clause (4)

Sub-clause (4) gives the upper limit needed on design actions for connections. Detailed studies by MacRae (12.6) have indicated that the nominally ductile seismic forces provide an upper bound on seismic-induced actions for category 1 moment-resisting and eccentrically braced framed seismic-resisting systems in general and for category 2 systems designed for $\mu \approx 3$. For nominally ductile seismic-resisting systems, application of a similar philosophy produces a cut-off limit associated with elastic response.

For category 2 seismic-resisting systems, a differential is established between systems designed for $\mu_{act} > 1.5$ and systems designed for $\mu_{act} \leq 1.5$, in order to maintain a realistic strength hierarchy between primary and secondary members in the latter case. This may occur where a criterion such as minimum lateral stiffness controls the member sizes and requires a category 2 structure which is significantly stronger than that required for the design earthquake loads associated with $\mu = 3$.

Sub-clause 4(b) may be used as an upper bound for any category 2 seismic-resisting system design in lieu of calculating μ_{act}.

Refer to C12.3.2.3 and figure C12.3.2.3 for calculation of μ_{act}.

Amd 2 Oct. '07

The changes to maximum actions are to incorporate the 2500 year criterion from NZS 1170.5 and also to enhance the level of protection to lower ductility capacity systems. Also 12.9.1.2.2(4) is supposed to represent the upper limit earthquake induced actions, however the second half of the sentence contradicts this by stating that 12.9.2 governs if these actions are greater. The amendment to this sub-clause and to 12.9.2 corrects this error.

C12.9.2 Minimum design actions on connections subject to earthquake loads or effects

The minimum design actions specified in 12.9.2 are derived from those specified in the 1989 edition of this Standard. They have been converted to limit state design format and made compatible in format and presentation with the non-seismic minimum design actions of section 9. Their scope has been extended to cover both seismic-resisting and associated structural systems.

Amd 1 June '01

Analyses undertaken, principally by MacRae (12.6), show that these requirements are generally satisfactory, except that the minimum design actions for splices in the lower half-height of columns of category 1 and 2 systems, as required by the 1992 edition, have been extended to apply over all levels of these systems, based on the recommendations of Clifton (12.52). N_g^* is constant during an earthquake and acts to potentially reduce ϕM_s, therefore it can and should be taken into account in design of column splices. This is introduced through Amendment No. 1.

C12.9.3 Welds

C12.9.3.2 Weld failure

Failure of incomplete penetration butt welds or one-sided fillet welds under tension action is inherently brittle (12.33) and such failure must be avoided. In contrast, failure of full penetration butt welds or 2 sided fillet welds with adequate capacity is relatively ductile (12.33). It is important to avoid the use of inherently brittle weld details that could promote brittle weld failure where a realistic upper limit on design actions cannot be given.

Advice on design and detailing to suppress brittle weld failure is given in (12.3) and section 10 of (12.13).

Run-on and run-off tabs are not generally required in the welds of connections in seismic-resisting systems. Refer to Commentary Clause C9.7.2.4. The selection of both parent metal and weld material with non brittle characteristics (see 2.6.4) is particularly important.

C12.9.3.4 Fillet welds

This clause contains 2 recommendations for good design and detailing practice from section 3.1 of (12.24).

Item (b) is especially critical for fillet welds joining elements intersecting at relatively shallow angles.

Note that fillet welds connecting plates intersecting at less than 60° are not prequalified to AS/NZS 1554.1 and their suitability will therefore have to be verified in each instance by prior documentation or by tests in accordance with the requirements of AS/NZS 1554.1 (12.4).

Research undertaken into the inelastic behaviour of fillet welds at different orientations to the direction of applied loading has indicated that their ductility capacity is strongly dependant on the orientation to the applied loading (12.25). For example, transverse fillet welds perform better than longitudinal fillet welds in splice plate connections between 2 members, with the best performance gained from circumferential fillet welds to the splice plates.

C12.9.4 Bolts

C12.9.4.2 Reduction in cross-sectional area

The general requirement expressed in equation 12.9.4.1 is taken from the 1977 edition of the Standard, with the member overstrength factor added. Its use in many instances will result in a requirement of $(A_n/A_g) = 1$, thus necessitating area replacement plates to be applied to compensate for the loss of cross-sectional area due to bolt holes. This will avoid fracture being initiated through the bolt holes of bolted connections subject to inelastic demand, a form of failure observed in recent severe earthquakes, for example as reported in (12.21). Where area replacement plates are required, they should extend a clear distance of (d/2), but not less than 150 mm, past the reduction in cross section area that they are replacing.

The specific requirements of 12.9.4.2.2 result from research into the behaviour of simple bolted (and welded) splices using Grade 250 steel plates. The results of this research (12.25) show that the requirement for category 1 members results in failure occurring through the gross section of the member, while the category 2 member requirement results in final failure through the outer line of bolts, but only after appropriate ductility has been generated in the connected member. The provisions of 12.9.4.2.2 are consistent with the use of equation 12.9.4.1 and have been extended to apply to grade 300 steels.

C12.9.4.3 Maximum design bearing capacity

The requirements of table 12.9.4.3 are generally based on (12.24) as expressed in limit state design format compatible with the provisions of 9.3.2.4.1.

C12.9.4.4 Minimum edge distance requirement

These restrictions on edge distance are based on the results of inelastic cyclic testing (12.34, 12.25, 12.26, 12.16). These results show that the minimum edge distance of $2d_f$ commonly considered necessary to avoid a splitting failure of the plate under inelastic cyclic loading only needs to be applied in the direction of the applied force and even then may be conservative (12.26).

It should be noted that even with these edge distances applied, equation 9.3.2.4(2) rather than equation 9.3.2.4(1) will govern the calculation of nominal bearing capacity. Refer to Commentary Clause C9.3.2.4 for further details.

C12.9.4.5.3 *Punching*
The requirements are obtained from (12.27).

C12.9.5 Moment-resisting beam to column connections

C12.9.5.1 Scope

Clause 12.9.5 has been expanded in scope from the 1989 edition of the Standard, to cover the design of rigid welded or bolted endplate connections not only between members of seismic-resisting systems, but also, where appropriate, between members of rigid associated structural systems and systems subject to moment redistribution (in the latter case by selected cross-reference from 4.5.7.2).

The philosophy underlying the design and detailing requirements of the clause is expressed in 12.9.1.1.

C12.9.5.2 Design actions from beams

(a) *Axial forces generated by beam flanges*
For seismic applications, these are determined directly from 12.9.1.2. The reduction allowed in the 1989 edition of this Standard, where it could be shown that opposing gravity loads and seismic forces reduced the potential ductility demand, has been removed, as the effects of shakedown under repeated inelastic action will eliminate this effect over successive cycles of inelastic action (12.6).

(b) *Design shear force for joint panel zone*
It is prudent design procedure to spread inelastic demand to as many components of the structure as is safely possible, to avoid excessive demand on any one component. The panel zone is the one connection component that can exhibit significant dependable ductility capacity and the design philosophy for the panel zone is based on it being subjected to inelastic ductility demand up to, but not exceeding, that on the adjacent beam or beams.

In practice the dependable ductility demand of a panel zone is still the subject of ongoing research and the design provisions are therefore based on suitably conservative interim recommendations. A differentiation has been made between the requirements for incoming category 1, 2 and 3 members. An upper cut-off has been placed on the unbalanced seismic-induced component, as given by sub-clause (b)(ii)(1), based on the initial work by MacRae (12.6) and confirmed as being appropriate through on-going research, e.g. (12.52).

Variable description (2) to equation 12.9.5.2(1), relating to V_{COL}

V_{COL} is determined directly from the column design actions. If the point of contraflexure in the columns above and below the connection occurs at mid-height, and the storey heights are regular, then:

$$V_{COL} = (M_L + M_R)/L_c \quad \text{(Eq. C12.9.5(1))}$$

where

L_c = clear height of column

This expression for V_{COL} will be sufficiently accurate for many design applications.

Variable description (4) to equation 12.9.5.2(1), relating to V_G

The 1989 edition of this Standard gave an interim procedure for reducing the design shear force in the joint panel zone under the interaction of design gravity loads and seismic forces. ➤

A better understanding of the effect of shakedown (12.1, 12.6) has shown that such a reduction is not appropriate to apply to panel zones of connections in seismic-resisting systems, however it is applicable to panel zones in moment-resisting framed associated structural systems. This is covered by variable description (4) to equation 12.9.5.2(1), which is associated with the term (V_G).

Amd 2 Oct. '07

An element of slab participation is to be considered for panel zone design, when required by 12.10.2.3. See (12.65) for background to the provisions.

Suitable method for determination of V_G

A suitable method for determination of V_G follows. It is written for this Standard being used in conjunction with AS/NZS 1170 set. The expression for V_G is given by equation C12.9.5(2).

$$V_G = \frac{(F^*_{pg}L)_{min}}{8\,(d_b - t_{fb})} \quad \text{(Eq. C12.9.5(2))}$$

where

F^*_{pg} = 1.0G + 1.0Q_u (calculated as the **total** load on the beam)
G = dead load calculated in accordance with AS/NZS 1170 set
Q_u = combination ultimate limit state live load calculated in accordance with AS/NZS 1170.0 table 4.1
L = beam centreline length as defined in 4.3.2.

Notes regarding the use of equation C12.9.5(2)

(1) This term represents a moment imposed by gravity loading on the positive moment end of the smaller beam framing into a connection. The moment applying is that at the column face, however *in lieu* of explicitly calculating the gravity moment at the column face, the value of moment calculated at the column centre line and multiplied by 0.85 may be used. This is the approach taken herein, with the factor of 0.85 incorporated into equation C12.9.5(2).

(2) Equation C12.9.5(2) is applicable for uniform or predominately uniform applied load on the beam. When the loading is non-uniform, replace equation C12.9.5(2) with [(0.85 x 1.75) $(M^*_{pg})_{min}$ / $(d_b - t_{fb})$], where $(M^*_{pg})_{min}$ is the fixed end moment generated at the column centre line by the applied gravity load case 1.0G + 1.0Q_u on the beam.

(3) Where beams frame into the panel zone from both sides of the column, the value of $(F^*_{pg}L)_{min}$ relating to the shorter beam will normally be the critical value for use in design.

(4) Where only one fixed-ended beam frames into the panel zone (for example at the outer column of a moment-resisting frame), V_G = 0.

Equation C12.9.5(2) is based on a recommendation made to SEAOC on panel zone design (12.28) with a couple of important modifications. The starting point in deriving this equation is the assumption that the gravity load combination 1.0G + 1.0Q_u (which is considered operative at the time of the earthquake) generates a fixed end gravity moment at the supports of each beam framing into the connection. This is quite an accurate assumption. The next step is to take the smallest of the 2 fixed end moments generated in this manner as applying at the ends of both beams. This is a conservative, but necessary, step, given the cyclic nature of seismic loading. The next step is to consider the interaction between this gravity loading and the seismic forces on each beam framing into the connection, one of which will have its positive moment end at the connection and the other its negative moment end.

At the positive moment end the assumption is made that the gravity moment subtracts directly from the maximum possible design moment of $C_2\ M_s$ given by 12.9.5.2(b). At the negative moment end, the gravity and seismic-induced moments are cumulative, with the possibility of the maximum design moment of $C_2\ M_s$ being generated mostly by seismic-induced loading (joint rotation). Therefore only three-quarters of the fixed end gravity moment is considered to contribute to the generation of $C_2\ M_s$ at the negative moment end.

As the gravity induced moments are in balance across the joint (i.e. they do not generate any panel zone shear) then their contribution, which amounts to 1.75 times the (smallest) of the fixed end gravity moments generated by the 2 beams framing into the connection, must be subtracted from the maximum design out-of-balance moment across the joint. This is the origin of equation C12.9.5(2), with 12/(0.85 x 1.75) rounded off to 8. The 0.85 is applied in order to translate the moment calculated at the column centreline to the appropriate value at the column face, which is required for use in equation C12.9.5(2).

Equation C12.9.5(2) is written for application to the connections of rigid framed associated structural systems where the inelastic response of the seismic-resisting system imposes inelastic rotations on the members and connections of the associated structural system. As rigid framed associated structural systems often carry significant gravity loading, a significant reduction in panel zone design shear force is possible through its use.

Relaxation of panel zone design shear force for tall buildings
For tall buildings the member sizes required for strength and stiffness may be sufficient to ensure that the joints never exhibit inelastic behaviour. In such cases the requirements of 12.9.5.2(b) may be relaxed, provided it can be shown that the design capacity of the joint is adequate to resist design actions generated by a maximum credible level of earthquake loading associated with nominally ductile response of the seismic-resisting system. Such analysis should be undertaken in accordance with either the modal response spectrum or numerical integration time history analysis methods specified in the Loadings Standard. A maximum considered level of earthquake record corresponds to that associated with a 2500 year return period calculated in accordance with NZS 1170.5.

C12.9.5.3 Welded moment-resisting beam to column connections

C12.9.5.3.1 *Web stiffening*
An individual stiffener must be proportioned so as not to buckle [under compression loading] (12.20) and the limits given on the ratio of outstand width/thickness in Note (1) are to prevent this form of failure occurring.

The width of an individual stiffener can be ascertained from the requirement that the total width of the pair of stiffeners plus column web should be similar to the width of the critical incoming beam flange (within 10 % is recommended).

Equations 12.9.5.3(1) to 12.9.5.3(4) do not make allowance for a reduction in ductility demand due, for example, to member sizes being dictated by load combinations not including earthquake loads. In such instances, it is always conservative, however, to substitute the quantity N^*_{fb}/f_{yb} into equations 12.9.5.3(1) or 12.9.5.3(2) in place of the quantity $\phi_{oms}\ A_{fb}$, or A_{fb}, respectively, when ascertaining the need for stiffeners and this approach has been taken in the clause.

If compression stiffeners are needed, they must have a minimum area to prevent stiffener buckling or local bearing failure of the area of column bearing onto the stiffener under incoming compression loading. Compression stiffener buckling is controlled primarily through the limits on outstand width/thickness ratio required in the clause, however, as a further control on buckling/bearing failure, a minimum stiffener area restriction is put on equation 12.9.5.3(3) by using the equation as written with ϕ_{oms} = 1.0 for situations where the quantity N^*_{fbc}/f_{yb} is less than A_{fb} (see Note 2(ii)). Thus the minimum stiffener area calculated from equation 12.9.5.3(3) will be determined on the basis that the incoming beam flange reaches its nominal axial yield capacity.

A similar situation applies for the tension stiffeners, with N^*_{fb}/f_{yb} being substituted for A_{fb} in equation 12.9.5.3(4). Local buckling is not of concern in the case of tension stiffeners, however, which means that the outstand limit of Note (1) need not be applied to them. (Watch for reversing moment, however). The overstrength factor is not applied in equation 12.9.5.3(4), because comparable or greater protection against undesirable ductility demand on the tension stiffener is provided by neglecting, in equation 12.9.5.3(4), the contribution to tension stiffener strength from 2 significant sources. The first of these is the strength of the unstiffened column flange and the second is the biaxial stress flow through the tension stiffener, in which incoming tension stress is continuously transferred into the column web, as shear stress, along the full length of the stiffener.

The welds connecting the stiffener to the column web need only be designed to transmit the incoming design action from the adjacent beam flange(s), with the upper limit on this action being the design section capacity of the stiffener ($\phi A_s f_{ys}$). The latter will usually govern.

C12.9.5.3.2 *Shear capacity of connection panel zone*
Equation 12.9.5.3(5) relates to the shear strength of the panel zone at a (displacement) ductility factor of 4. This is in line with the design philosophy, expressed in 12.9.5.2(b), of allowing some yielding in the panel zone, but to a similar or lesser extent than that occurring in the beam. The background to equation 12.9.5.3(5) is given in (12.29), except that the shear yield stress is increased to $0.6f_y$ consistent with 5.11.4.1.

The term η, which accounts for the interaction of axial and shear forces within the panel zone, does not decrease from 1.0 until $N^*/\phi N_s$ exceeds 0.4. Experimental testing of panel zones in columns with varying ratios of $N^*/\phi N_s$ indicate that low ratios of applied axial force to nominal section capacity have no effect on panel zone behaviour (12.29, 12.30) and that the standard interaction equation for shear and axial loading is conservative to apply because of the relatively rapid strain-hardening rate of the panel zone (compared to the members framing into it) and its confined nature. This is allowed for in the term η by replacing the 1.0 value, which applies in the standard interaction equation, with 1.15.

Design and detailing recommendations for doubler plate(s)

Doubler plate(s) required from 12.9.5.3.2 should be designed in accordance with the recommendations given in the HERA *Steel Design and Construction Bulletin*, Issue No. 57, 2000. (The article in Issue No. 57 supersedes previous guidance in the DCB on doubler plate design.) Some of the key points of those recommendations are:

1. Do not use doubler plates that are too thin. As an approximate guideline only, if $t_p \leq t_{wc}/2$ or 8 mm is required, use one plate only, with a limit on the thickness of any one doubler plate of $t_p \leq t_{wc}$.

2. If a web side plate cleat frames into the doubler plate, ensure the design shear from the design gravity load on the web side plate cleat can be resisted. This shear does not need to be considered concurrently with the earthquake induced shears from the moment-resisting connection. The doubler plate to column welds must also be checked for this shear.

3. A minimum doubler plate thickness of 5 mm should be used.

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4. The doubler plate is sized to fit within the panel zone region once the tension/compression

stiffeners are in position. (In any practical joint detail, tension/compression stiffeners will always be required when doubler plate(s) are needed.) The fitting of the doubler plate and sizing of the welds between it and the tension/compression stiffeners and between the tension/compression stiffeners and the column web are covered in HERA *Steel Design and Construction Bulletin*, Issue No. 57, 2000.

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C12.9.5.3.3 *Slenderness limits on panel zone elements*
The changes made from the 1992 edition to the 1997 edition in this clause are as follows:

(1) For panel zones in connections which fall within the scope of 12.9.5.1, this clause now places an upper limit on element slenderness for the component(s) of the panel zone, such that the panel zone can develop its shear yield capacity.

(2) The contribution of yield stress from both the doubler plate and the column web has been included in equation 12.9.5.3(6).

(3) Panel zones supported on all 4 sides have enhanced resistance to buckling. This is recognized through setting 2 limits to C_3. The higher limit applies to the 4 sides restrained condition and has been derived from the expression for α_v in 5.11.5.2, for an aspect ratio s/d_p (= d_b/d_c) = 1.2, which represents a practical maximum limit on d_b/d_c.

The lower limit applies when either no tension and compression stiffeners are present (2 sides restrained) or one set only are present (3 sides restrained). The lower limit is set at the end of stocky action for an unstiffened web, from 5.11.5.1; this is more appropriate and slightly less restrictive than the 1992 limit, which was taken from (12.30).

Panel zones which are more slender than the limit of equation 12.9.5.3(6) cannot be used in structural systems that fall within the scope of 12.9.5.1. They may be used in elastically responding systems, or in systems which are not governed by the action of seismic forces or seismic-induced displacements, and should be designed to 5.11.5.2.

C12.9.5.3.4 *Welds between beam and column*
The beam tension flange to column flange weld must be suitably designed to accommodate the non-uniform stress distribution that will occur if the column tension flange is unstiffened.

For welds between a beam flange and a stiffened column flange, the stress distribution along the weld can be considered uniform (i.e. the same as the stress distribution across the beam flange). For welds between a beam tension flange and an unstiffened column flange, the flexibility of the unstiffened column flange causes a very non-uniform distribution of stress along the weld, with maximum stress being reached in the weld at the centreline of the column section (i.e. adjacent to the web/flange junction) and minimum stress near the column flange tips. The peak stresses on the weld to an unstiffened flange must be allowed for in design and a suitable design procedure is given in section 10 of (12.13). In most instances stiffeners will be needed due to the high beam tension flange action forces generated.

Suitable connection details are presented in (12.22), except that connection item 26d therein must comply fully with the design model given in section 8.1.2 of (12.3) if it is to be used in category 1, 2 or 3 systems.

The beam web should be fillet or butt welded to the column flange so as to transfer the horizontal and vertical actions generated adjacent to the column by seismic-induced yielding and gravity-induced shear. Guidance is given in section 8.1.2 of (12.3).

C12.9.5.4 Bolted moment-resisting beam to column connections

(a) *General*

The design of bolted moment-resisting connections is relatively complex, due to the interaction of the various connectors and connection components (column flange, bolts, endplate, welds and beam flanges and web) and the need to accommodate different configurations of bolts.

It is important in the design of a bolted moment-resisting connection to design all the components in a balanced manner, in order that no one component is either too strong or too weak. Furthermore the interactive nature of the connection must be considered in design, with allowance made, where appropriate, to change the design of any one component of the connection where it is affected by the subsequent design of another component. This is especially important in seismic applications where inelastic demand on the members framing into the connection is anticipated.

(b) *Suggested sequence of connection design*

The design of a bolted moment-resisting connection should proceed in the following manner:

1. Having sized the members being connected, find the design forces on the joint.
2. Determine the number of rows of bolts and their layout.
3. Check the column flange strength and introduce tension stiffeners if required. Recheck the bolt strength.
4. Calculate the endplate thickness.
5. Check the shear zone strength (Zone (C) in figure C12.9.5) and design doubler plates if required.
6. Check the compression zone for the following:

   (i) Buckling of web

   (ii) Crippling of web

   (iii) Yielding of web
7. Check the tension zone for the following.

   (i) Yielding of web

   (ii) Yielding or tearing of the connecting parts, including design of all welds required
8. Design and detail all remaining welds and stiffeners.

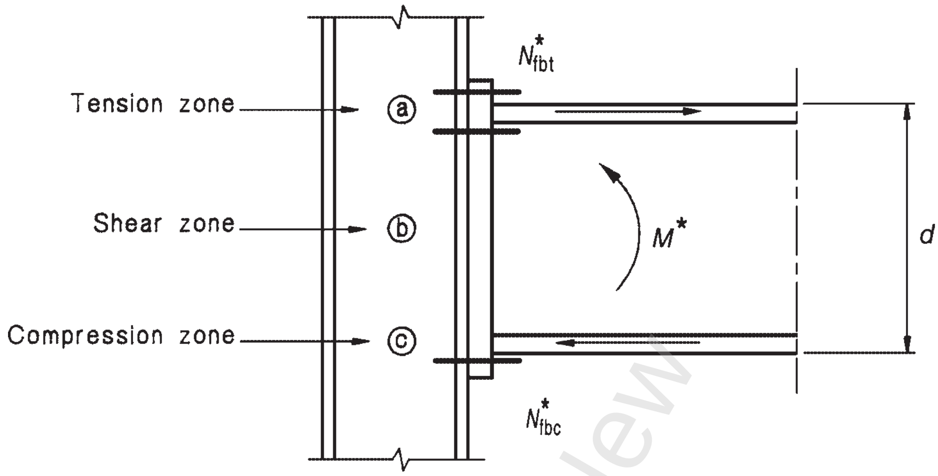

Figure C12.9.5 – Bolted moment-resisting connection detail

C12.9.6 Design of splices

A detailed design procedure for splices is presented in (12.32), written for non-seismic applications. It is equally applicable for seismic applications, when used in conjunction with the detailing requirements of this clause (12.9.6) and the appropriate design actions. Details of splices are presented in (12.22). The general principles of connection design given in section 10.5.3 of (12.13) are useful in splice design. Specific details for beam and column splices in seismic-resisting systems are given in (12.3).

C12.9.6.1 Column splices

(a) Column splice details may exhibit little dependable ductility, especially where incomplete-penetration butt welds are used to the flanges and these are subject to tensile actions, generated either by axial force or by bending action (12.33). Column splices must therefore be located clear of yielding regions. This is not an onerous requirement.

(b) There are 3 advantages in designing column splices for full bearing contact. First, the design compression action (which always exceeds the design tension action) need not be resisted by the bolts. Secondly, compression overloading of the column during an earthquake event, for example due to vertical ground acceleration, will not load the bolts beyond their design capacity and, thirdly, there is no need to hold the upper length of column on the crane during erection to line up the bolts.

Cold-sawing will usually meet the required accuracy of finish for the bearing surfaces (see 14.4.4.2 and Commentary Clause C14.4.4.2).

(c) This requirement is to suppress the column extension due to bolt slip under seismic-induced tension action that would occur with snug-tight bolts in standard oversized bolt holes (12.25).

C12.9.7 Design of gusset plates

A comprehensive procedure for gusset plate design for both seismic and non-seismic applications is presented in section 10.7.2 of (12.13). The procedure is confirmed by experimental testing (12.35).

C12.9.7.2 Local effects

(a) Minor axis bending – this is especially relevant to single shear gusset plates.

C12.9.7.3 Accommodation of member inelastic rotation

Where gusset plates are connecting pin-ended category 1, 2 or 3 members that may be expected to buckle out-of-plane inelastically under severe earthquake loads or effects, a clear distance of between 2 and 3 times the gusset plate thickness (figure C12.9.7) should be allowed for to accommodate joint rotation unless other specific steps are taken to accommodate any anticipated inelastic rotation.

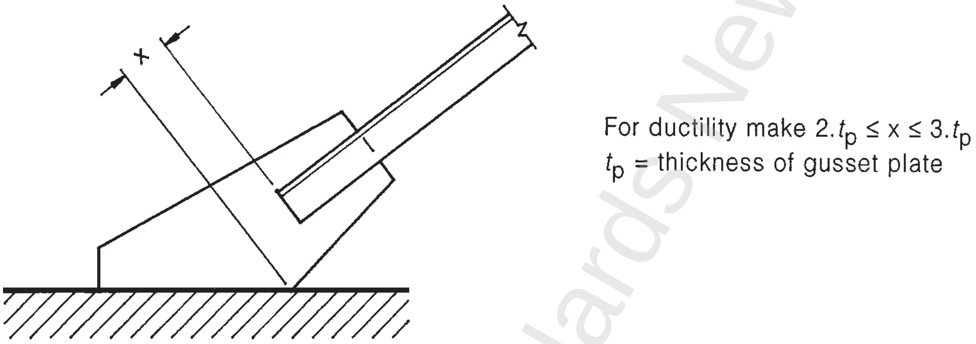

Figure C12.9.7 – Seismic design of gusset plates – A ductility requirement

In figure C12.9.7;
x = clear distance of gusset plate to allow for inelastic rotation of the connected member.

C12.9.7.4

A gusset plate which connects into both a beam and a column member will be stretched when the angle between these two members tries to increase under inelastic seismic action. This requirement is to ensure the connection between gusset plate and support does not fail in this situation.

For gusset plates that are welded into the supporting member (either the beam or the column), this can be achieved by sizing the weld between the gusset plate and the member to develop the design tension capacity of the gusset plate gross cross section at the connection.

For gusset plates that are bolted into the supporting member, this can be achieved by either designing the connection to accommodate opening of the joint angle under inelastic seismic action or by designing the bolted connection to develop the design tension capacity of the gusset plate gross cross section at the connection.

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C12.9.8 Design of built-up members

The provisions of 12.9.8.2 are obtained from research undertaken by Astaneh – Asl et al (12.36, 12.37).

Note that, as with the rest of this Standard, higher property classes of structural bolt – e.g. property class 10.9 – may be used *in lieu* of the property class 8.8, provided they are used in the tension bearing mode. The design engineer must check their suitability for full tensioning to 15.2.4. Advice on this may be obtained from HERA.

C12.10 DESIGN OF MOMENT-RESISTING FRAMED SEISMIC-RESISTING SYSTEMS

C12.10.1 General

Clause 12.10.1 defines the scope of 12.10. If applying this clause to associated structural systems, consider carefully which elements of the associated structural system may require special provisions to accommodate seismic effects, in accordance with 12.3.4.

C12.10.2 Design procedure

A comprehensive design procedure for category 1, 2, 3 and 4 moment-resisting framed seismic-resisting systems is presented in sections 5 – 7 of (12.3). It fulfils the requirements of 12.10.

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NZS 1170.5 requires the materials standards to specify the principles and key concepts required for capacity design. These concepts are already incorporated into the step by step requirements of HERA Report R4-76, with the exception of the slab participation factor now required by 12.10.2.3 and specified in 12.10.2.4.

C12.10.2.3

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The slab participation is required when determining the overstrength actions on the column member for combined actions design when the slab is not isolated from the column.

Suitable details for isolating the slab from the column are given in section 4 of (12.64).

C12.10.2.4

This method of accounting for slab participation in the overstrength joint moment is based on the assumption that the slab is infinitely rigid and strong axially, carrying force due to compression contact of the concrete against the column over a defined contact area. The slab contributes to both positive and negative moments in the adjacent beams, as in the negative moment case the tension force developed in the reinforcement has to be transferred into the column through the concrete slab. The distance between the centroid of compression contact area and that of the steel section generates the lever arm for determination of the moment induced by this slab action. The slab compression force has to be balanced by an axial tension in the steel beams and this reduces the beam moment capacity. The calculation of joint overstrength, $M°$, takes both factors into account.

The determination of bearing width against the column face is based on work by Civjan et al (12.59) for beams framing into the strong axis of an I section, with the same approach being applied to the other column types of cross section listed.

The value of b_{sef} for circular columns is from applying the principles of Blodgett (12.60) for distribution of transverse load from beam flanges into a circular hollow section column member. They are based on 80 % of the concrete compression force being resisted by the compression contact face of the slab onto the column.

Values of concrete stress at the concrete face ranging from less than 0.85 f'_c to 1.80 f'_c have been found in past tests Civjan et al. (12.59). Du Plessis and Daniels (12.61) and Lee and Lu (12.62) found that the concrete stress is usually 1.30 f'_c or less. Values less than a 1.30 result for partially composite slabs so a value of 1.30 is recommended for New Zealand application until tests on locally constructed slabs on beams are available.

The variable f'_{cos} takes account of the increase in concrete strength above the minimum specified that occurs in practice, due to both short and long term effects for a slab which has different compaction and curing than large concrete members.

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It should be noted that steel beams have an upper limit on the moment that they can resist as a result of beam axial tension and compression yielding which is different from reinforced/ prestressed concrete beams where large moments may result due to the slab.

For I section beams framing into the column flange of I section columns, the results from using the general equations have been determined for a typical range of beam and column sizes and ratios of slab effective thickness to beam depth. These have then been expressed as a function of the enhancement to the overstrength factor, ϕ_{oms}, against the ratio of slab effective thickness to beam depth. This function can be expressed as an enhancement to the overstrength factor ignoring slab participation in the format shown with only 3 % loss of accuracy within the practical range of t_{ef}/d_b. The enhanced overstrength factor, ϕ_{omss}, is used in place of ϕ_{oms} for determining the seismic induced beam shears and the column design moments, when applying the provisions of HERA Report R4-76 (12.3).

The slab participation is required when determining the overstrength actions on the column for flexural design. It may not be required for calculation of the column panel zone shear strength, as the slab effect on the panel zone is neutral in that it may increase both the panel zone shear forces and the panel zone shear strength.

When applying the capacity design provisions of section 6 of HERA Report R4-76 (12.3), if $M°$ is calculated directly, then it is used in place of $M°_{beam}$ in step 5.1 and subsequent steps of section 6. Where two beams frame into the column, $M°$ must be distributed into the beams as directed by the clause. If the enhanced overstrength factor option for I section beams framing into the flanges of I section columns is used, then this enhanced overstrength factor is used instead of ϕ_{oms}.

The effect of the slab is to increase overstrength demands on the columns. The aim of the slab participation factor is to incorporate this slab effect into the capacity design derived design actions on the columns, such that the inelastic demand on the columns is not underestimated due to unaccounted-for slab action. Advanced finite element modelling of a representative subassemblage with and without the slab has shown (12.65) that the proposed factor achieves this aim, both for the column above and below the beam (clause 12.10.2.4) and for the panel zone (clause 12.9.5.2 (b) (ii) (2)).

Example:

Beams framing into the column (one from each side):

460 UB74.6, f_y = 300 MPa, M_s = 498 kNm, $2M_s$ = 996 kNm,

$A_g f_y$ = 9520 mm^2 x 300 MPa = 2,856 kN, 2 $A_g f_y$ = 5,712 kN

MRF is category 2, Overstrength factor, bare steel ϕ_{oms} = 1.15

Column: 310 UC 118, f_y = 300 MPa

Frame configuration: Interstorey height is 3.6 m, and bay width is 7.0 m

Slab: A slab depth of 120 mm on a metal deck which runs in the same direction as the beam, thus forming a solid concrete rib down the beam. The concrete strength is f'_c = 30 MPa.

N_{slab} $= \min\{1.3 t_{ef} b_{sef}(f'_c + 10); \Sigma A_s F_y\}$

$= \min\{1.3*120 \text{ mm}*307 \text{ mm}*40 \text{ MPa} = 1{,}916 \text{ kN}; 5{,}712 \text{ kN}\}$

$= 1{,}916$ kN

$\Sigma M°_i$ $= \min\{1.18 \times (1 - 1916/5712) \times 996 \times 1.15; 996 \times 1.15\}$

$= \min\{898; 1145\} = 898$ kNm

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$M°$ $= 898 + 1{,}916\,(457/2+120-120/2) \times 10^{-3} = 1451$ kNm

Overstrength factor due to the slab effect alone is:

$$\phi_{oss} = M^{\circ} / \Sigma M^{\circ}_{b,i} = 1451/1145 = 1.27$$

Overstrength factor for the beam with slab participation

$$\phi_{omss} = M^{\circ}/\Sigma M_{s} = 1451/996 = 1.46$$

Alternatively,

$$\phi_{omss} = \phi_{oss} \Sigma M^{\circ}_{b,i} / \Sigma M_{s} = \phi_{oss} \Sigma(\phi_{oms} M_{s}) / \Sigma M_{s} = \phi_{oss} \phi_{oms} \Sigma M_{s} / \Sigma Ms = \phi_{oss} \phi_{oms}$$

$$= 1.27 \text{x}\ 1.15 = 1.46$$

C12.10.2.6

Inelastic time history studies carried out by MacRae (12.6), Clifton (12.64) and others have shown that a soft-storey mechanism does not develop in a steel moment resisting frame complying with the design requirements of this Standard and HERA Report R4-76 (12.3). For this reason the dynamic magnification factor for moment is 1.0. However, MacRae proposed (12.6) a shear magnification of 1.2 to limit the possibility of undesirable shear yielding in steel columns with unstiffened webs.

Amd 2 Oct. '07 These studies did not incorporate the strengthening effect of the slab when this is not isolated from the columns and this has been addressed through the slab participation factor in 12.10.2.3.

C12.10.3 Shear strength within beam yielding regions

When a capacity design procedure is used, the derivation of design shear force in the beam web is based on the overstrength capacity of the beam (see, for example, section 6.2 of (12.3)). The resulting design shear action has a sufficient factor of safety in-built to prevent a significant reduction in design moment capacity, even if the design shear equals the design shear capacity of the member as calculated from 5.11.4.

If a capacity design procedure is not used, then the design shear action must be kept to a level which will not reduce the nominal moment capacity of the beam.

This requires a different approach for the 2 cases, which is based on clauses 5.11 and 5.12.

The requirement applies only to category 1 or 2 beams, where the inelastic demand on the yielding region will involve the whole member cross section.

C12.10.4 Calculation of lateral deflection of the seismic-resisting system

(b) Panel zone distortion, if significant, will add an additional component of interstorey deflection to a moment-resisting frame, as shown in figure C12.10.

The relative strength of the panel zone to the incoming beam (or column) members can have a significant influence on the inelastic deflection of the structural system.

When rigid moment-resisting frame lateral deflection is based on clear member lengths, panel zone contribution to this deflection need not be considered if the design shear force (V^*_p) is as given by 12.9.5.2(b).

If the design shear force (V^*_p) is reduced below the value given by 12.9.5.2(b), then the effect of panel zone deformation must be included. A simple procedure for determining its influence on the frame elastic lateral deflection is presented in (12.38). The resulting frame elastic deflection, incorporating elastic panel zone deformation, is then factored by the structural ductility factor to provide the inelastic deflection, in accordance with the requirements of NZS 1170.5 as called up by 12.3.2.3.

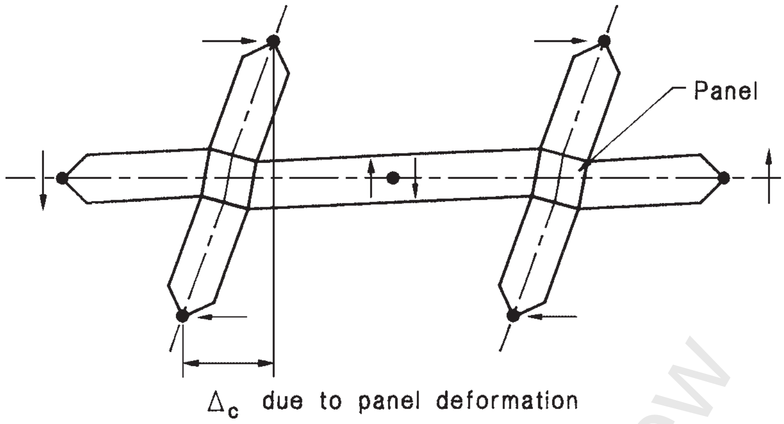

Figure C12.10 – Interstorey deflection component due to panel zone deformation

C12.10.5 Yielding region formation and column strength

The design procedure presented in sections 5 and 6 of (12.3) takes account of all these factors, except for (c).

Allowance for concurrent action must be made in accordance with 12.8.4.

C12.10.6 Concrete encasement of steel frames

Concrete encasement may alter the response of a steel frame, especially if it is extended to form a panel infill between columns. The effect of such infill is well documented, for example in (12.10).

The effect of encasement of members on the location and behaviour of yielding regions in members is covered in section 13, especially sections 13.3.5, 13.4 and 13.5, of (12.53).

The effect of encasement on deflection must be considered in accordance with Appendix N.

C12.11 DESIGN OF ECCENTRICALLY BRACED FRAMED SEISMIC-RESISTING SYSTEMS

Amd 2 Oct. '07

NZS 1170.5 requires the materials standards to specify the principles and key concepts required for capacity design. These concepts are already incorporated into the step by step requirements of HERA Report R4-76 and are presented in the elaboration of clauses 12.11.1.1, 12.11.3.2 and new sub-clauses in 12.11.7.

C12.11.1 General

The requirements for eccentrically braced framed (EBF) seismic-resisting systems presented in 12.11 are based on application to systems with active links responding inelastically in the shear mode. These requirements are obtained primarily from (12.20, 12.39, 12.40).

A series of V-braced and D-braced EBFs (refer to figure C12.11.1 for brace configuration) have been analysed by MacRae (12.6) and design requirements formulated. From these requirements, detailed design procedures for all categories of V-braced and D-braced EBFs, with active links responding in the shear mode, have been formulated and are presented in sections 10 – 12 of the HERA Report R4-76 (12.3).

The design procedures in (12.3) and all the design and detailing provisions of 12.11 apply to EBFs with active links responding in the shear mode. This requires short clear lengths (e) of active link, as specified in section 11.3 of (12.3) or in (12.39), such that $e \leq 1.6\, M_{sp}/V_w$.

This limitation on active link length to ensure shear mode behaviour can be restrictive on structural form. Popov et. al. have extended their initial research work on shear mode links to links responding inelastically in a combined shear and flexural mode ($1.6\, M_{sp}/V_w < e \leq 3\, M_{sp}/V_w$) and to links responding in a flexural mode ($e > 3\, M_{sp}/V_w$). The results of this work and the necessary design guidelines are contained in a detailed report (12.41). Careful use of (12.41) in conjunction with the general performance requirements of 12.11 will enable dependable design of EBFs with long active links to be undertaken. Guidance on use of long links is also referenced from section 11.3 of (12.3). A valuable summary of EBF design practice and developments in the USA is presented in (12.42).

The configuration of V-braced and D-braced EBFs is shown in figure C12.11.1, with the member terminology used herein shown applied to a V-braced EBF in figure C12.11.2.

The provisions of 12.11 may be applied to category 2 EBFs by making the active link category 2 and the beams, columns and braces category 3, then applying all other provisions directly as stated herein. Detailed procedures for category 2 EBF design are given in section 11 of (12.3).

Detailed procedures for category 3 and 4 EBF design are given in section 12 of (12.3). They incorporate the appropriate provisions of section 12.11 into a rational design procedure for each category of system.

C12.11.3 Design requirements for category 1 EBF frames and components

C12.11.3.1
It is very important to include the shear stiffness contribution to active link deformation when calculating the lateral drift of an EBF. In, for example, a moment-resisting frame the shear stiffness is so much greater than the flexural stiffness that the frame lateral drift due to beam deformation can be determined accurately from the beam flexural stiffness alone. However, in an active link, the elastic flexural and shear stiffnesses may have similar values, thus including the shear stiffness will decrease the active link stiffness and hence increase the relative elastic deformation across the active link by a factor of 2. The ratio of flexural to shear stiffness will vary with each EBF design, hence active link shear stiffness should be accurately modelled in any elastic EBF analysis. Guidance is given in section 13.2.1 of (12.3).

Similarly elastic change in length of columns due to axial force in an EBF may add a significant component to the overall lateral deflection at the top of the frame and should be allowed for in the analysis where required by this clause. Guidance is given in section 13.2.2 of (12.3).

The effect of concrete encasement on deflection must be considered in accordance with Appendix N of this Standard.

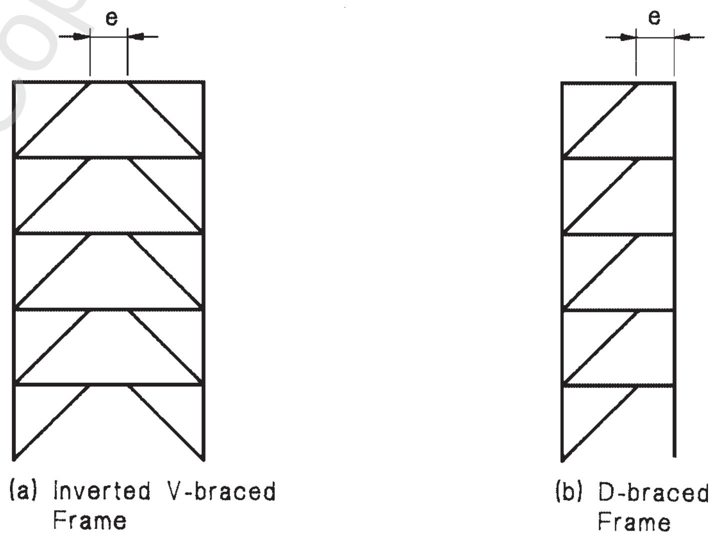

Figure C12.11.1 – Alternative bracing configurations for eccentrically braced frames

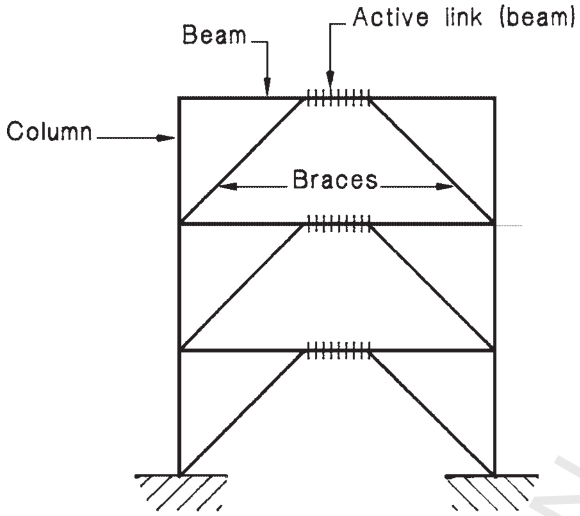

Figure C12.11.2 – Member terminology applied to a V-braced eccentrically braced frame

C12.11.3.2

The requirements for selection of member category in a category 1 EBF have been rationalized in this edition and an error contained in the 1989 edition of this clause corrected.

The active link is the primary seismic-resisting member and has the same member category as that for the system (i.e. category 1).

The beams and columns are subjected to combined actions, principally bending moment and axial force and will be subject to some inelastic demand under earthquake loading above the severe seismic level (12.6). For these reasons, category 2 is the appropriate member category for these members.

The braces are subject to principally axial force, with only slight moment action. Any inelastic demand on them under extreme earthquake loading will be minimal (12.6) and the section geometry requirements for category 3 elements from 12.5 (table 12.5) will provide dependable member resistance to these demands. The braces can therefore be made category 3 members.

C12.11.3.3

The requirements of 12.11.3.3.1 to 12.11.3.3.3 are illustrated in figure C12.11.3. In figure C12.11.3 (a) γ_p cannot exceed 0.09 radians for short links with reduced values for longer links (12.41). In figure C12.11.3 (b), γ_p cannot exceed 0.09 radians if the active link is connected to the column flanges (i.e. about the column strong axis) or 0.045 radians if the active link is connected to the column web (i.e. about the column weak axis) in accordance with 12.11.5.

The limit of 0.08 radians contained in the 1989 edition of this Standard to short links has been increased to 0.09 radians in line with the limit states design specification for EBF design in the United States (12.20). This reflects the increased confidence in the ductility capacity of these systems that has been gained through ongoing research and their use in design.

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The reduction in allowable plastic rotation limits for active links arises from the need to address non collapse in the 2500 year event from NZS 1170.5 and is consistent with the limits in the current American and European seismic design requirements.

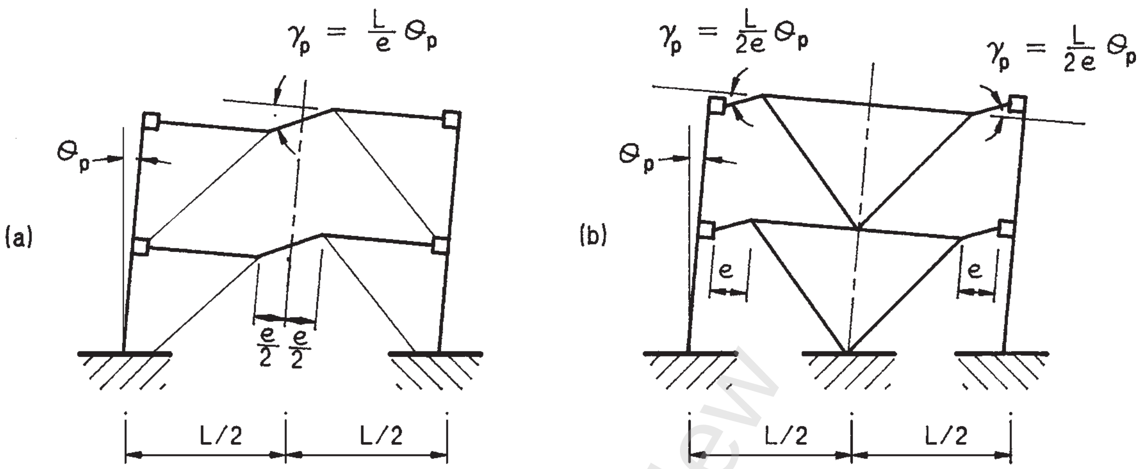

Figure C12.11.3 – Active link rotation angles

In figure C12.11.3

γ_p = inelastic storey rotation angle between the active link and the adjacent beam

θ_p = inelastic storey rotation angle of lateral deflection (assuming rigid column behaviour)

C12.11.3.4

It has been shown in tests (12.39) that welds connecting doubler plates to the beam flange/web junction do not perform reliably at the level of inelastic strain anticipated in the active link. Three plate sections with complete penetration butt welds between the web and flange plates may be used, however.

C12.11.3.5

The active link is subject to a specific set of design actions and inelastic demand that is quite different to that on other elements of a seismic-resisting system. The 1992 edition of the Standard contained some special requirements for active link design that were taken from USA practice (12.20, 12.39, 12.40).

In the 1997 edition, these requirements have been expressed in terms of the relevant member design provisions of sections 5-8. This will improve their useability, without compromising the active link performance.

C12.11.3.8

Braces should be designed for the capacity design derived design actions generated by the active link plus the axial forces from design gravity loads. Where an active link-beam segment of an EBF is carrying significant gravity floor loading, simply designing the braces to resist seismic-induced actions alone will not ensure protection against premature brace yielding. Where a concrete slab over an active link is made composite with the active link, the overstrength factor must account for this (see table 12.2.8.2 of this Standard).

C12.11.3.12

If a brace also frames into the beam-column connection in such a way that some or all of the brace axial force is transferred into the column through the beam, then the lateral restraint of the beam flange which carries the brace must be designed to incorporate 2.5 % of the component of brace design axial load transmitted through the beam flange into the column.

C12.11.4 Active link web stiffening requirements for category 1 EBFs

The terminology given in figure C12.11.4 will assist in the application of 12.11.4.1 to 12.11.4.5, which are derived from experimental results on EBF active links (12.39, 12.40).

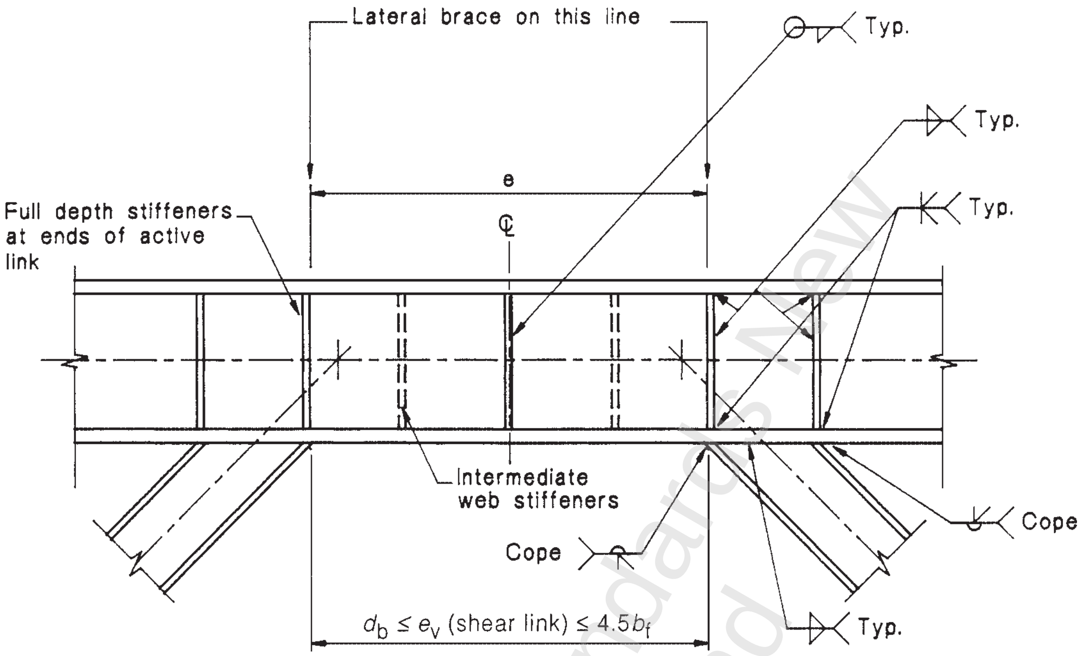

Figure C12.11.4 – Terminology for active link

Amd 2 Oct. '07 These requirements apply to shear mode active links ($e \leq 1.6\ M_{sp}/V_w$). Refer to (12.41) for the web stiffening requirements in EBFs with longer length active links. The changes are to make these clauses applicable to more than category 1 EBFs.

C12.11.5 Connections between an active link and a column for category 1 EBFs

These requirements are taken from (12.39) and apply to shear mode active links.

Amd 2 Oct. '07 There are restrictions on the use of active links with length exceeding 1.6 M_{sp}/V_w adjacent to a column (D-braced configuration). EBFs with active links exceeding a clear length of 1.6 M_{sp}/V_w may only be used in a V-braced configuration (12.41).The changes are to make these clauses applicable to more than category 1 EBFs.

C12.11.6 Lateral restraint requirements for the active links of category 1 EBFs

Amd 2 Oct. '07 Both the top and bottom flanges of the active link must be prevented from laterally deflecting. The active links are not usually long enough to require restraint between supports, however lateral restraint at the supports is always required. The strength requirement that must be met by the restraints, i.e. 2.5 % of the beam flange design capacity, has been increased from the 1.5 % value recommended by SEAOC to bring it into line with the lateral restraint strength and stiffness requirements of 5.4.3 and 6.6.2. The stiffness requirement, i.e. a minimum lateral deflection of 4 mm allowed under the design lateral force, is a metric translation of the 0.1 inch SEAOC requirement, increased to 4 mm in view of the increase from 1.5 % to 2.5 % made in the restraint lateral force design requirement.The changes are to make these clauses applicable to more than category 1 EBFs.

C12.11.7 Capacity design requirements for EBFs

Amd 2 Oct. '07 NZS 1170.5 requires the materials standards to specify the principles and key concepts required for capacity design. These concepts are already incorporated into the step by step requirements of HERA Report R4-76 (12.3), with the exception of the slab participation factor required for D-braced systems with long links by 12.11.7.2.

The collector beam may undergo inelastic action when the active link is fully inelastic. This is allowed for in (12.13) by designing the collector beam to resist 80 % of the capacity design derived design actions, rather than 100 % required for all other secondary members, as specified by 12.11.7.2. The collector beam is required by 12.11.3.2 (2) to have suitable properties to resist this inelastic demand.

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The dynamic magnification factors in 12.11.7.4 are obtained by MacRae(12.6). More recent studies (12.63) of EBFs designed to this Standard and HERA Report R4-76 (12.3) have confirmed the adequacy of these factors.

C12.12 DESIGN OF CONCENTRICALLY BRACED FRAMED SEISMIC-RESISTING SYSTEMS

C12.12.1 Scope and definitions

C12.12.1.1 General

The requirements for concentrically braced frame design are obtained primarily from (12.20, 12.31, 12.43, 12.44).

Since the 1992 edition of this Standard was published, detailed step by step design procedures for category 1, 2, 3 and 4 x-braced and v-braced concentrically braced framed (CBF) seismic-resisting systems have been developed and are presented in sections 14 to 19 of HERA Report R4-76 (12.3).

For category 1, 2 and 3 CBFs, these design procedures account for the force distribution in the elements of the seismic-resisting system in both the elastic and the inelastic modes of response.

C12.12.2 CBFs with bracing effective in tension and compression

The provisions of 12.12.2 to 12.12.5 inclusive apply to CBFs with braces which exhibit significant design compression capacity. This modifies and considerably improves the inelastic behaviour over that of a tension braced system alone, especially when the braces are stocky. Nordenson (12.43) describes in detail CBF behaviour with varying brace slenderness ratios.

Very slender brace members fail under compression loading by elastic buckling, with minimal capability for inelastic action and hence minimal energy absorption under severe earthquake loads or effects. Inelastic action starts to become significant when the slenderness ratio falls below the value given by $C_c = \sqrt{(2\pi^2 E / f_y)} = 126$ for $f_y = 250$ MPa. SEAOC (12.20) recommend a limiting value of 120 for Grade 250 steel, which has been adopted herein for all categories of CBF system.

The requirement that all brace members shall be designed on the basis of their compression capacity makes the sizing of the primary seismic-resisting members (the braces) straightforward. It does pose some problems with subsequent design of the secondary seismic-resisting members, as the overstrength capacities of the tension and compression braces will be different and this difference must be accommodated in the design of the secondary members. Brief guidance on this is given in Commentary Clause C12.12.5.2 and detailed guidance in sections 14 to 18 of (12.3).

C12.12.2.2

These requirements ensure that there will be tension braces in every braced frame unit and in both directions of loading and that the strength of the braces will be balanced for both directions of loading.

C12.12.2.4

The curvatures associated with inelastic compression buckling may be large enough to cause local buckling. This will be severe in non-compact members and lead to crack propagation and failure, hence all brace members in category 1 or 2 systems must comply with the section geometry requirements for category 1 or category 2 members as appropriate.

C12.12.2.5 Additional requirements for chevron bracing (V bracing)

The additional requirements of 12.12.2.5 (b) and (c) are necessary because of the inelastic behaviour of this type of bracing system. This behaviour is described in section 14 of (12.3) and the relevant aspects that necessitate these extra provisions and the more severe height limitations of 12.12.4 are briefly described herein.

The post-buckling strength of a *v*-braced system will deteriorate more rapidly than that of an *x*-braced system, because, unless the beam into which a pair of braces frame is very strong, once the compression brace has buckled, the additional strength of the tension brace will cause a plastic hinge to develop in the beam, rather than allow the full tensile strength of the tension brace to be developed. Under reversing loading, this means that the storey shear resistance of the bracing system will be governed by the post-buckling behaviour of 2 opposing braces. The consequence will be a deteriorating shear-storey drift relationship and less satisfactory inelastic behaviour than is exhibited by an *x*-braced CBF with braces having the same slenderness ratio. (Refer to section 14.5 of (12.3) for a detailed description of the inelastic behaviour of *v*-braced CBFs).

In the 1992 edition of this Standard, higher seismic design loads were specified for v-braced systems than for *x*-braced systems, through the C_S factor, as a crude method of allowing for the less satisfactory inelastic behaviour of the *v*-braced system. Since then, design procedures have been developed (12.3) which incorporate the influence of the inelastic behaviour of each system directly into the procedures. This is an improvement over simply raising the design seismic load for the *v*-braced system and has also allowed the same C_S factors to be specified for both systems in clause 12.12.3.

Requirements (b) and (c) relate to the formation of a beam plastic hinge due to unbalanced forces in the braces and are intended to ensure that the beam will continue to perform satisfactorily once the hinge forms.

C12.12.3 Design seismic loads for CBFs with bracing effective in tension and compression

Clause 12.12.3 presents the design seismic loads for each category of CBF system. The first step in deriving the design seismic load is to derive the lateral force co-efficient C or ordinate $C(T)$ in accordance with the Loadings Standard – clause 5.2 of NZS 1170.5 (12.5). For the ultimate limit state (see 12.12.3.1), this lateral force co-efficient or ordinate is then multiplied by a factor, C_S, which accounts for the less-than-ideal actual inelastic behaviour of CBF systems, as documented below and in section 14 of (12.3).

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Note that for calculation of serviceability limit state deflection, C_S = 1.0 (see 12.12.3.3).

With the publication, in 1994, of detailed seismic design procedures for all categories of *v*-braced and *x*-braced CBFs with braces effective in tension and compression and for typical brace configurations (see sections 16 and 17 of (12.3)), the philosophy in deriving the design seismic loads for CBF systems has been amended from that used in the 1992 edition. This amendment has involved removing the differential between design seismic loads for *v*-braced and for *x*-braced systems, a step that has been recommended in section 14.6 of (12.3).

This removal of the differential in design seismic loads for *v*-braced and *x*-braced systems has been made possible because of the greater rigorousness of design procedure now given in (12.3) in accounting for the system behaviour and distribution of forces in the elements of the seismic-resisting system, both in the elastic and the inelastic modes of response. The requirement for the latter to be included in any design procedure is also now given in 12.12.5.2(b) and 12.12.5.3(b), which are new clauses introduced into the 1997 revision.

In addition, Remennikov (12.58) has developed expressions for determining the post-buckling compression capacity of individual category 1, 2 and 3 braces. They are based on analysis of the data reported in (12.57), plus results from overseas research. These expressions are given as equations C12.12.3(1) and C12.2.3(2) below.

$\alpha_c' = 42.15 / \lambda_n^{1.1} \leq 1.0$ for braces in category 1 and 2 systems .... (Eq. C12.2.3(1))

Amd 1 June '01 $\alpha_c' = 7.7 / \lambda_n^{0.6} \leq 1.0$ for braces in category 3 systems .... (Eq. C12.2.3(2))

where

λ_n = modified member (brace) slenderness, given by 6.3.3

α_c' = post-buckling compression capacity of the brace

Equation C12.2.3(1) should be used, in applying section 16.2 of HERA Report R4-76 (12.3), instead of the expression for α_c' given in step 6.2 therein.

Equation C12.2.3(2) should be used, in applying section 16.4 of (12.3), instead of the expression for α_c' given in step 7 therein.

When $\lambda_n \leq 30, \alpha_c' = 1.0$ from equations C12.2.3(1) and C12.2.3(2). This recognizes the fact that stable hysteretic loops are obtained if the brace slenderness ratio is less than 30.

The variation of α_c' with λ_n is shown in figure 29.5 of (12.58).

With the development of the above expressions for brace post-buckling compression capacity and the greater rigorousness of the CBF design procedures required by this Standard and given in (12.3), the opportunity has been taken, in the 1997 edition, to derive values for the C_S factor which more accurately reflect the less-than-ideal actual inelastic behaviour of CBF systems than did the values in the 1992 edition.

The inelastic behaviour of a CBF is primarily dependent on 4 factors:

(a) The structural form of the design system (*x*-braced or *v*-braced);

(b) The inelastic demand on the system;

(c) The slenderness ratio of the braces;

(d) The number of storeys.

The effects of (a) are directly accounted for in the design procedures specified in sections 16 and 17 of (12.3), and in the design requirements of 12.12.5. Hence, this factor is not incorporated into the values of C_S given in tables 12.12.3.

The values of C_S in those tables are derived from 3 variables, which account for the effects of (b) – (d) above.

(i) Variable A accounts for the deterioration in inelastic performance of CBFs with increasing brace slenderness. The deterioration to be considered is that for the system as a whole. Based on the 1996/97 studies of CBF behaviour by Remennikov, a reasonable way of allowing for the effect on the CBF system of the individual brace slenderness is to use the following expression for variable A:

$$A = 1/\left[0.5\left(1+\alpha'_c\right)\right]$$

where α'_c is given by equations C12.2.3(1) or C12.2.3(2) as appropriate for the category of system (or brace) and the chosen brace slenderness ratio.

This expression reflects the fact that the brace slenderness ratio has considerably less effect on CBF system performance, for systems complying with this Standard and designed to (12.3), than it has on the post-buckled compression capacity of an individual brace member.

(ii) In applying variable A to deriving the C_s factors for tables 12.12.3, the following brace slenderness ratios, λ_n^*, have been chosen:

- for $\lambda_n \le 30$, $\alpha'_c = 1.0$, so a selected value of λ_n^* is not required
- for λ_n from 31-80, $\lambda_n^* = 55$
- for λ_n from 81-120, $\lambda_n^* = 100$
- for tension braced systems, to 12.12.6, $\lambda_n^* = 150$

The philosophy involved in selecting λ_n^* has been to select the value in the middle of the particular range of λ_n values. This slightly underestimates the influence of λ_n on CBF behaviour over half the given range and overestimates it over the other half.

(iii) Variable B accounts for the departure of the CBF system from the optimum *o*-mechanism system (weak beam-strong column mechanism) towards the less desirable *s*-mechanism system (strong beam-weak column mechanism). The extent of departure from the *o*-mechanism system in CBFs is successfully suppressed by the capacity design procedure requirements for column design (12.3, 12.54) and the requirements of this Standard to locate and design column splices away from the floors in order to achieve effective column continuity across the splice. Because of this, variable B, which is storey-dependent, is set at 1.1 for CBFs up to one-third of the maximum height limit specified in tables 12.12.4 (which is for *x*-braced systems with brace slenderness ratios up to 30), 1.2 for CBFs between one-third and two-thirds of this limit and 1.3 for CBFs higher than two-thirds of this limit.

(iv) Variable C accounts for the influence of inelastic demand on the system. This influence is accounted for by modifying the product of variables A and B to derive C_s as follows:

- for category 1 systems, C_s = A.B
- for category 2 systems, C_s = 0.75 (A.B – 1) + 1
- for category 3 systems, C_s = 0.25 (A.B – 1) + 1
- for category 4 systems, C_s = 1.0 for buildings up to 16 storeys in height and C_s = 0.1 (A.B – 1) + 1 for buildings above this height

(v) The value of C_s given in tables 12.12.3 is that derived from (iv), rounded to the nearest 0.05.

(vi) The value of C_s given in 12.12.6.3.2, for tension braced CBF systems, is derived on the same basis, using $\lambda_n^* = 150$ from (ii) above.

This detailed background to the derivation of C_s is given herein in order that designers may calculate an appropriate value of C_s for applications in which the brace slenderness falls just within the lower half of one of the ranges given; e.g. has a value of 40 and hence falls just within the 30-80 range of application.

Finally, in regard to selecting the value of C_S to apply, the situation may well arise in practice where the braces in the lower storeys have a lower slenderness ratio than those in the upper storeys. In determining which range of brace slenderness to apply when using tables 12.12.3, base this on the storey with the largest brace slenderness ratio for all the storeys up to $0.64H$, where H is the height of the CBF above its base. (The $0.64H$ is the height at which the centroid of the seismic force is considered to act; see C12.3.2.3.2). Alternatively, calculate a specific value of C_S for the actual value of the largest brace slenderness ratio used in the lower $0.64H$ of the building.

C12.12.4 Maximum height limitations for stand-alone CBF systems with bracing effective in tension and compression

The requirements of table 12.12.4(2) represent a transition from the SEAOC maximum height recommendations (12.20) which are considered applicable for category 2 CBF systems with stocky braces (slenderness ratio not exceeding 30) down to the SEAOC recommendations for systems with slender braces (slenderness ratio of 120).

The height limitations given in table 12.12.4(2) for CBFs with stocky braces are close translations of the SEAOC limitations of 160 ft and 120 ft respectively for stand-alone *x*- and *v*- braced systems, using a storey height of 3.75 m. From this benchmark, for category 2 CBF systems, a step-down in maximum height allowed for a given brace slenderness ratio has been made.

In the 1989 edition, the height limitations in metres were also expressed as number of storeys, based on a storey height of 3.75 m, in line with the storey height given in NZS 4203:1992. The philosophy applied to derive the height limitations of table 12.12.4(2) for category 2 systems was then used to derive the height limitations for category 1, 3 and 4 systems in the adjacent tables.

Expressing the height limitations in terms of both a storey height and an overall height causes complications, however, especially for low-rise CBFs which have widely varying storey heights. The actual value of storey height is not important; it is the number of storeys that is the important height parameter to consider.

In the 1997 edition, therefore, the height limitations in metres have been dropped from tables 12.12.4.

Note (1) to tables 12.12.4 incorporates the NZS 4203:1992 provision for an uppermost storey of light-weight construction.

Note (2) allows the height limitations to be exceeded for dual systems, which exhibit superior seismic response to stand-alone CBF systems (12.44).

C12.12.5 Seismic design procedures

C12.12.5.1 Relationship between structure category and member category for CBF systems

Table 12.12.5.1 presents the necessary requirements, which have been derived in accordance with the general philosophy of 12.2.6 applied to the particular characteristics of CBF system behaviour. The more severe requirements on the collector beams of v-braced systems to those of x-braced systems, for example, reflects the fact that a yielding region will usually form in a collector beam of a *v*-braced system at the brace intersection point under inelastic earthquake response.

These relationships are also given in figure 3.2 of (12.3).

C12.12.5.2 Seismic design procedures for category 1, 2 systems

(a) Capacity design is required on all category 1 and 2 CBF systems (and, to a limited extent on category 3 systems) to suppress the tendency for soft-storey action and hence avoid the need to consider individual inelastic storey sidesway mechanisms in design. CBF systems are particularly susceptible to a build-up of deflection in one direction when responding inelastically to severe seismic forces.

The requirement for capacity design, coupled with the requirement when sizing the braces that they be sized on the basis of their compression capacity (see 12.12.2.2) means that care must be taken in the second stage of the capacity design process (design of collector beams and columns) to rationally determine the design loads. Detailed guidance on this is given in section 16.2 of HERA Report R4-76 (12.3) for x-braced systems and in section 17.2 of (12.3) for v-braced systems.

(d) These are interim design requirements pending further detailed study of CBF systems. These interim provisions relate closely to the corresponding provisions for columns in moment-resisting and eccentrically braced framed systems, as recommended by MacRae (12.6) and specified in (12.3).

(e) This is a most critical requirement in ensuring satisfactory CBF performance in severe earthquakes; failure of the connections is the principal documented cause of CBF system failure or poor performance in severe earthquakes (12.21).

(h) An initial study of the inelastic response of CBF systems has been undertaken over 1996/97 at the University of Canterbury, The preliminary results from this study indicate that the inelastic lateral deflections of these systems, as calculated from NZS 1170.5 for the equivalent static or the modal response spectrum methods of analysis, underestimate the deflections generated from a numerical integration time history analysis for systems with $\mu > 2$. This underestimation can be allowed for by applying a factor, greater than 1.0, to the NZS 1170.5 calculated deflections and the factors given in sub-clause (h) have been determined as appropriate, pending more detailed studies.

The factor of 2.0 for tension braced systems, which is included in this sub-clause and cross-referenced from 12.12.6.4, was recommended in the commentary of the 1992 edition; it has now been transferred to the Standard and appropriate factors for systems with braces effective in tension and compression have been added.

C12.12.5.3 Seismic design procedures for category 3 systems
The same philosophy applies as for category 1 or 2 systems and the commentary to 12.12.5.2 is also applicable, except that the increase in inelastic deflection over that calculated from NZS 1170.5 is not necessary for the nominal level of ductility demand involved.

C12.12.5.4 Seismic design procedures for category 4 systems
These systems do not require capacity design and the design procedure is more straight-forward.

C12.12.6 Tension braced CBF systems

The 1989 edition of this Standard presented requirements for the design of category 2 tension braced CBF systems only. In line with the expansion of the scope of 12.12 in general, coverage of tension braced systems has been extended to all categories. The determination of the factor C_S and height limitations applicable to each category of system is based on careful assessment of the likely inelastic performance of tension braced systems in the light of research (12.46, 12.57, 12.58), observation of performance of actual systems in severe earthquakes (12.47) and consideration of the requirements of the detailed design requirements (12.3).

The requirement of 12.12.6.6(a) limits tension braced systems to an *x*-braced configuration. With the slender braces allowed for tension braced systems, the possibility of a build-up of inelastic deflection in one direction under severe seismic action should be considered in design. The effect of this build-up will be in enhanced $P-\Delta_i$ actions and this is allowed for simply but effectively through clause 12.12.5.2(h)(ii) by designing for 2 times the inelastic lateral deflections under the design loads as determined from NZS 1170.5. Allowance is then made for the $P-\Delta_i$ effects, in accordance with NZS 1170.5, using these enhanced values of Δ_i. Refer to section 18.2 of (12.3) for more detailed guidance.

Note that in a multi-bay CBF system, the pairs of braces do not have to be in the same bay, although there must be a pair of opposing braces on each storey.

C12.12.7 Design of notched regions in braces

Notching of braces to reduce the tensile area locally and hence reduce the design tension capacity of a brace is a recognized method of CBF design. However, the practice can result in the ductility capacity of the CBF being severely reduced unless the following aspects are considered:

(a) The notched region will be subject to very high inelastic strain and care must be taken to avoid brittle fracture adjacent to or within the notched region. Clauses 12.12.7.2 items (a) to (e) cover this aspect;

(b) Yielding must be concentrated into the notched region, even when strain-hardening occurs, and the notched region must be compact to avoid severe local buckling in compression. Clause 12.12.7.2 items (e), (f), (j) and (k) cover this aspect;

(c) The total accumulated inelastic strain within the notched region must not exceed the inelastic strain capability of the steel used. Clause 12.12.7.2 item (g) gives the necessary requirement.

The design inelastic strains are based on the conservative assumption that inelastic drift, in one direction only, may accumulate over 2 load cycles (see also 12.12.5.2(h)(ii)).

(d) The notched region, once it is strained inelastically in tension, must not buckle in compression when the brace is subjected to the compression cycles of the applied cyclic load. This requirement is covered in 12.12.7.2 item (h), where the inelastic buckling load of the notched region is required to exceed 1.5 times the compressive strength of the overall brace.

(e) The notched region is designed to yield in axial tension only and must not be placed in a region of the brace that could be required to undergo inelastic buckling under compression loading. This is covered by 12.12.7.2 items (h) and (k).

Amd 2 Oct. '07

C12.12.7.2 Seismic design considerations

The change to k_e reflects the sway failure mode of the notched region.

C12.13 DUAL SEISMIC-RESISTING SYSTEMS

The inelastic performance of dual seismic-resisting systems (typically a combination of concentrically braced or eccentrically braced frame and moment-resisting frame) is more dependable than that of a stand-alone braced system, principally due to the redundancy present in the dual system (12.44). Dual CBF/MRF systems are reasonably common in high seismicity regions of the USA, where the SEAOC provisions (12.20)) allow significant relaxation on the height limitations for stand-alone CBF systems. A similar approach has been adopted in this Standard.

The purpose of the special study required under 12.13.2(b) is to accurately assess the elastic and inelastic behaviour of the dual system and the ductility demand on the members of each system. This study must accurately account for any significant change in performance of each system over successive cycles of inelastic loading.

(Text deleted)

Where the height of the dual system exceeds twice the height limitations of 12.12.4 or 12.12.6.5 as appropriate, this special study should include numerical integration time history analysis in accordance with NZS 1170.5.

(Text deleted)

The requirements of minimum lateral load sharing given in 12.13.2 (a) and (c) are intended to ensure that the dual system's inelastic performance is improved, as intended, over that of a stand-alone system.

Dual systems by definition must act together to resist a given direction of seismic loading. They need not physically be connected, however, but may be bound together through a rigid floor diaphragm. A perimeter moment-resisting frame acting in conjunction with a concentrically or eccentrically braced frame enclosing an interior core of a building comprises a dual system, just as does a perimeter moment-resisting frame which includes a concentrically or eccentrically braced frame bay within its plane. More than 2 systems may be involved.

Where the seismic-resisting systems comprising the dual system have different categories (i.e. different structural ductility factors (μ)) then the assessment of a structural ductility factor for the dual system (μ_d) is required, in order to calculate the design seismic forces on the dual system as a whole. The assessment of (μ_d) may be undertaken by calculating the product of the structural ductility factor times the elastic stiffness for each system in parallel, summing these values and dividing by the total elastic stiffness to obtain the structural ductility factor (μ_d) for the dual system. The seismic design action for the dual system may then be determined from the Loadings Standard for (μ_d) and the seismic design actions for each system then determined, by factoring the design action for the dual system by the ratio:

$$\frac{\mu_d k_i}{\mu_i \Sigma k} \quad \text{(Eq. (C12.13))}$$

where

μ_d is the structural ductility factor for the dual system

μ_i is the structural ductility factor for system i

k_i is the elastic stiffness of system i

Σk is the total elastic stiffness of the dual system.

C12.14 FABRICATION IN YIELDING REGIONS

C12.14.1 Shearing and gas cutting

Gas cut edges are only permitted if they comply with the surface condition requirements for "Brittle Fracture Applications" given in the Welding Technology Institute of Australia (WTIA) Technical Note 5, "Flame Cutting of Steels" (14.2). Gas cut edges not complying with these requirements shall be repaired in accordance with the WTIA recommendations for "Brittle Fracture Applications" or ground to remove all cutting holes. This is covered further in section 14.

C12.14.3 Transition of thickness

The more gradual transition slopes required for the yielding regions of category 1 or 2 members are to prevent brittle fracture being initiated within the transition zone.

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C13 DESIGN OF COMPOSITE MEMBERS AND STRUCTURES

C13.1 SCOPE AND GENERAL INTRODUCTION

C13.1.1 Scope

This section applies only to members of steel and concrete in which the 2 materials are designed to act in combination.

Section 13 presents detailed requirements for the design of composite beams, principally in 13.4. As specified in 13.4.1, and in more general terms in 1.5, other rational design methods or standards may be used for composite beam design *in lieu* of the provisions contained herein.

The Steel Construction Institute have published a series of design guides on various specialized applications of composite beams, written for use in conjunction with the appropriate United Kingdom Standards (13.7, 13.9, 13.30). These design guides and associated Standards may be used in conjunction with the ultimate and serviceability limit state loads and load combinations from AS/NZS 1170.0, for non-seismic applications. For applications subject to earthquake loads or effects, the design engineer must ensure that relevant performance and prescriptive criteria from sections 12 and 13 of this Standard are also complied with. Clauses 12.3, 13.4.3 and 13.4.11 are particularly relevant in this regard.

The Steel Construction Institute publications referenced in this commentary provide coverage on:

(a) A commentary to the United Kingdom Standard (13.30) on composite beam design (13.32);

(b) Design guides for continuous composite bridges (13.36, 13.37);

(c) A design guide for fabricated composite beams in buildings (13.38);

(d) A design guide for haunched composite beams in buildings (13.39);

(e) A design guide for composite slabs and beams with steel decking (13.40).

A method of rapid preliminary design for composite beams supporting concrete floors is given in Appendix A2 of (13.16).

C13.1.2 Design assumptions and requirements

C13.1.2.1

Guidance as to the appropriate level of live and dead loads to apply during the various stages of composite construction can be obtained from several sources, the most comprehensive being (13.27 and 13.2).

Amd 2 Oct. '07 The strength reduction factors presented in table 13.1.2(1) are derived in accordance with limit state design philosophy utilizing the safety indices implicit in NZS 4203 and the load factors presented in NZS 4203. Factors for composite members presented in table 13.1.2(1) are determined in accordance with the provisions of (13.28).

In table 13.1.2(1), positive moment regions involve at least the top of the supported concrete slab in compression (see also definition in 1.3); negative moment regions involve all of the supported concrete slab in tension.

Amd 1 June '01 The product $\phi_{sc}q_r$ is not changed by Amendment No. 1, as described in the amendments to C13.3.2.1.

Amd 2 Oct. '07 These strength reduction factors do not need changing with the adoption of the AS/NZS 1170 set.

C13.1.2.3

In design for the ultimate limit state, the design capacity of a section is independent of its loading history. The steel beam alone has to support the ultimate design load applied prior to hardening of the concrete but the composite beam has to support the ultimate design load once the concrete has hardened. However, at serviceability loads, the total stress and strain at any point in the member are dependent on its loading history, with stresses and strains from each stage of loading being cumulative. Therefore, since the bottom flange of an unpropped steel beam experiences higher tensile strains during construction than that of a propped member, the total tensile strain in the bottom flange due to the serviceability design loads must be within the elastic range to prevent permanent deformation of the composite section under service load conditions due to steel yielding (13.2). To prevent this overstraining the total elastic stress in the bottom flange due to serviceability loads must not exceed 0.9 ϕf_y. This is equivalent to a maximum bottom flange stress of 0.60 ϕf_y multiplied by the shape factor for the composite beam, which can conservatively be taken as 1.5.

For propped beam construction, it can be shown (13.3) that, when 2 or more props per span are provided, considering all dead and live loads to be carried by the composite beam section involves only a small error (approximately 2.5 %) in determination of stresses and final deflections.

C13.1.2.4 Partial composite action

Composite beams with partial composite action have not been tested under inelastic cyclic loading, hence the use of partial composite action is restricted to applications that are not required to undergo inelastic deformations under severe seismic forces. A sub-assemblage comprising a moment-resisting frame with the beam composite over the mid-span regions only (shear studs stopped off 1.5d_b from the column face) has been tested under inelastic cyclic loading (13.41) and exhibited excellent response.

C13.1.2.5 Effective slab thickness

The profiled shape of most steel decks means that there are regular voids in the concrete below the level of the top of the steel deck. This may or may not lead to loss of efficiency in the composite section. The extent of loss of efficiency depends on the steel deck profile, properties of the composite section (particularly the position of the neutral axis) and the design method (elastic or plastic) used to determine stress and strain distribution within the section.

The recommendations of the Canadian limit state steel code (13.4) are the most suitable of the recommendations given in limit state Standards and have been adopted accordingly. They are simpler to apply than the slightly more accurate ECCS recommendations (13.5).

C13.1.2.6 Serviceability checks required

(a) The background to (a) is covered in C13.1.2.3.

(b) This requirement is to prevent excessive cracking in the slab over the support regions of continuous composite beams (13.6).

(c) The method given in CAN/CSA-S16.1 (13.4) to account for the effects of partial shear connections, interfacial slip, creep and shrinkage on the deflection of composite beams, *in lieu* of testing or a more detailed analysis, is straight-forward to use and sufficiently accurate. Details are as follows:

    (i) To account for increased flexibility resulting from interfacial slip and partial shear connection, calculate the deflections using an effective moment of inertia given by equation C13.1.2(1).

$$I_e = I_{st} + 0.85\, p^{0.25} (I_{tc} - I_{st}) \quad \text{(Eq. C13.1.2(1))}$$

where

I_{st} = second moment of area of steel beam alone
I_{tc} = second moment of area of composite beam transformed into an equivalent steel section
p = fraction of full shear connection
p = 1.00 for full shear connection.

(ii) To account for creep, either increase the elastic deflections due to dead and long term live loads, as computed in item (i), by 15 %, or alternatively determine the long term deflection from composite section properties calculated using a factored steel to concrete modular ratio equal to 2.5 times the elastic modular ratio (13.2). The former approach is applicable to floor systems loaded after the concrete has attained its 28 day strength; for systems loaded earlier (i.e. based on a 7 or 14 day strength), increased creep deflection needs to be considered.

(iii) To account for shrinkage, calculate deflection using a selected shrinkage strain and assuming the composite beam to be bent in single curvature by equal end moments. This approach leads to the following equation for deflection due to shrinkage (13.4).

$$\Delta_{sh} = \frac{\varepsilon_{sh} E_c A_c L^2 y_c}{8 E I_{tc}} \quad \text{(Eq. C13.1.2(2))}$$

where

ε_{sh} = shrinkage strain. A value of 300 microstrain is generally recommended for composite beams and 450 microstrain for stub-girder beams
E_c, E = moduli of elasticity of concrete, steel
A_c = effective area of concrete slab
L = span length of beam
y_c = distance from elastic neutral axis to assumed line of action of the shrinkage force.

Alternative means of accounting for shrinkage are available (13.27).

It is of fundamental importance in the design of composite floor systems for satisfactory in-service performance that long-term deflections are properly assessed and allowed for. The design procedures given in Parts 1 and 2 of this Standard and by reference from this commentary will ensure satisfactory performance, if followed correctly. A general discussion of long-term deflection of composite floor systems is given in (13.42). Factors which influence the magnitude of long-term deflection include:

(a) Whether or not supporting composite beams are propped during construction;

(b) Removal of props from propped beams before the minimum specified period;

(c) Magnitude of long-term applied load;

(d) Degree of shear connection used;

(e) Nature of concrete slab (decking profile);

(f) Amount of shrinkage strain that occurs;

(g) Span to depth ratio of supporting beams.

All these except (b) are accounted for by the design procedures specified and referenced herein.

C13.2 COMPOSITE SLAB DESIGN

A composite slab is defined as an *in situ* concrete slab poured onto a profiled steel sheet deck, with the decking containing sufficient means of mechanical shear connection to ensure that the 2 materials act together to resist applied loading (principally applied vertical loading).

A composite slab can, in turn, be made either composite or non-composite with the supporting beams.

C13.2.1 Design methods

The two Eurocode standards provide the most up to date design requirements and have the widest scope of application. However, the most recent edition of the British standard for the design of the composite deck and the associated standard for design of the bare steel deck are still applicable and so these options are given as NOTE (1).

AS/NZS 4600 is a suitable standard for design of the deck itself for the wet concrete and construction stages and this option is given as NOTE (2). AS/NZS 4600 is also cited in B1/VM1. The steel used for decks is typically a cold-rolled steel to AS 1397 with limited ductility, hence the limits on design yield stress and tensile strength in NOTE (3).

Neither BS 5950-4 or BS EN 1994-1-1 cover the design of the slab for concentrated loads close to the support, which is the reason for NOTE (4). One option in this situation is to ignore the effect of the decking and design the slab for moment and shear to NZS 3101. See design guidance including software in (13.51). This situation principally applies at decking end supports which are also at the free edge of the slab.

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C13.2.2 Slab reinforcement

The pattern and extent of cracking in a composite slab on profiled steel sheet deck is different to that in a reinforced concrete slab, hence the minimum reinforcement provisions for reinforced concrete slabs e.g. as given in NZS 3101) are not directly applicable.

Minimum reinforcement requirements are needed to control cracking due to shrinkage and temperature effects and these requirements are applicable to the depth of concrete above the ribs (i.e. the cover slab, t_o-h_{rc}). Reinforcement is also required to control cracking where the slab spans over supporting beams or into end supports.

The 1997 edition presented recommendations from BS 5950-4:1994 for interior applications only. However these have been superseded by latter standards and have been shown in practice to allow undesirable crack widths to develop. They also apply only to slabs that are fully enclosed within a building except during construction and this scope of application needs to be widened.

Working with the provisions of BS EN 1992-1-1:2004, AS 3600:2001, NZS 3101:2006 and BS 5950-4, the following minimum reinforcement requirements for composite slabs on steel deck are as follows (except where specific design for crack control is used as an alternative to the prescriptive provisions of (A.1, A.2) and (B)):

(A) Reinforcement parallel to the span of the decking

(A.1) *Over the supporting beams of nominally continuous slabs*

Where the slab is designed as simply supported, the area of reinforcement over the supporting

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beams shall be the greater of:

(a) 0.2 % of the cross-sectional area above the ribs (i.e. based on the depth (t_o-h_{rc})) where the slabs are unpropped and 0.4 % of the cross-sectional area above the ribs where the slabs are propped;

(b) That required from (B) below for the degree of crack control and exposure classification;

(c) That required for fire emergency conditions.

(A.2) *Over the supporting beams of continuous slabs where specific control of crack widths is not required*

The area of reinforcement shall be the greater of:

(a) That required for flexural strength for ambient temperature or fire emergency conditions.

(b) 0.2 % of the cross-sectional area above the ribs where the slabs are unpropped and 0.4 % of the cross-sectional area above the ribs where the slabs are propped.

(c) That required from (B) below for the degree of crack control and exposure classification.

(A.3) *Over the supporting beams of continuous slabs where specific control of crack widths is required.*

(a) Scope of application: this is required for critical tensile zones in exposure classification C from NZS 3101 or to critical tensile zones in exposure classifications A2, B1 or B2 where areas of reinforcement less than those specified for strong crack control in (B.3) are desired.

(b) A critical tensile zone is a region of slab where the design bending moment at the serviceability limit state, M_s^*, calculated using short term live loads, exceeds the uncracked moment capacity, M_{cr}. M_{cr} is calculated using gross cross section properties and a flexural tensile strength of concrete equal to $0.4\sqrt{f'_c}$.

(c) Using the cracked section properties, the stress, f_s, in the largest diameter of reinforcing bar subject to M_s^* shall be determined.

(d) The stress from (c) shall not exceed the lesser of (0.8 x the yield stress) for the reinforcement or that given in the table below:

Nominal bar dia. (mm)	**Maximum steel stress** (MPa)
≤ 6	375
6 – 8	345
10	320
12	300
16	265
20	240

(e) The minimum area of reinforcement in the critical tension zone (mm^2/m width of slab) shall be the greater of that required from (A.2) and:

$$A_{rmin} = 1.8\frac{A_{ct}}{f_s}$$

where

A_{ct} = the area of concrete in the tension zone of the uncracked section (mm^2/m width of slab)

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f_s = the stress from (d) above (MPa).

(f) The crack width shall be determined from NZS 3101 clause 2.4.4.6 and assessed against suitable recommended crack widths, such as given in NZS 3101 table C2.1.

(A.4) *Simply supported slabs and the positive moment regions of continuous slabs*

The area of reinforcement placed above the ribs of the deck in these regions shall be the greater of:

(a) That required for fire emergency conditions; and

(b) That required from (B) below for the degree of crack control and exposure classification.

(B) Reinforcement transverse to the span of the decking

(B.1) *For slabs fully enclosed within the building except during construction:*

$A_{r,\,min} = CR\,(t_o - h_{rc})$

where

$A_{r,\,min}$ = minimum area of reinforcement required (mm^2/m width of slab)

t_o = overall thickness of slab (mm)

h_{rc} = height of deck ribs (mm)

CR = 1.75 when a *minor* degree of control against cracking is required

CR = 3.5 when a *moderate* degree of control against cracking is required

CR = 6.0 when a *strong* degree of control against cracking is required.

(B.2) *For slabs in exposure classification A2 from NZS 3101:*

$A_{r,\,min} = CR\,(t_o - h_{rc})$

where

$A_{r,\,min}$ = minimum area of reinforcement required (mm^2/m width of slab)

t_o = overall thickness of slab (mm)

h_{rc} = height of deck ribs (mm)

CR = 3.5 when a *moderate* degree of control against cracking is required

CR = 6.0 when a *strong* degree of control against cracking is required.

(B.3) *For slabs in exposure classifications B1, B2 and C from NZS 3101:*

$A_{r,\,min} = CR\,(t_o - h_{rc})$

where

$A_{r,\,min}$ = minimum area of reinforcement required (mm^2/m width of slab)

t_o = overall thickness of slab (mm)

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h_{rc} = height of deck ribs (mm)

CR = 3.5 when a *moderate* degree of control against cracking is required (but see (D.2) for additional requirements)

CR = 6.0 when a *strong* degree of control against cracking is required.

(B.4) For slabs in exposure classification U from NZS 3101, specific design is required.

(C) Reinforcement spacing, type and bar size and concrete quality

(C.1) Reinforcement for shrinkage and temperature control only may be Ductility Class L, N or E to AS/NZS 4671. Reinforcement for flexural strength where inelastic response is not required may be Ductility Class L, N or E. Reinforcement for fire resistance shall be Ductility Class N or E or as specified by the fire design procedure. Reinforcement for earthquake resistance in potential inelastic regions shall be Ductility Class E.

(C.2) The centre to centre spacing of bars in each direction for crack control shall not exceed the lesser of $3(t_o - h_{rc})$ or 300 mm. The diameter of the largest bar used for crack control should not be greater than 2 times the diameter of the smallest bar used for crack control.

(C.3) The cover required to the top reinforcement specified in (A) or (B) above shall comply with NZS 3101 tables 3.6 or 3.7.

(C.4) The clearance between the transverse reinforcement required from (B) above and the top of the deck ribs shall be not less than the maximum size of the aggregate used in the slab concrete, except at splices in this reinforcement.

(C.5) Laps to reinforcement and curtailment of reinforcement shall comply with NZS 3101.

(C.6) Potential cracking due to plastic shrinkage of concrete shall be controlled by specification. Concrete placement and curing shall be to NZS 3109. See also sections 2.6 and 2.7 of HERA Report R4-107:2005 *Composite Floor Construction Handbook* for good practice requirements in these areas. Poor quality concrete production, placement or curing will jeopardise achieving the expected crack control for the area of reinforcement specified.

(D) Description of the crack control terms

What each of the terms minor, moderate and strong mean in terms of crack control as specified in (B) above is described below:

(D.1) *Strong* crack control will limit crack widths to generally < 0.3 mm. This is necessary for durability in certain circumstances as specified in (B) and is expected to be acceptable for visual appearance in exposed slabs.

(D.2) *Moderate* crack control will limit crack widths to generally < 0.5 mm. For cracks of this width, crack sealing admixtures are expected to dependably seal any cracks that develop. Without such admixtures, this crack width will not be suitable for durability in external environments B1, B2 or C to NZS 3101. This width of crack is not expected to be acceptable for high quality visual appearance.

(D.3) *Minor* crack control will give crack widths generally > 0.5 mm. This is only suitable for environment A1 (internal) and where the concrete slab will always have a floor covering in service.

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(E) Additional requirements for localised areas

(E.1) Additional reinforcement will be required where the slab is subject to localised high tensile stresses due to deformation compatibility. Such locations are:

(a) Where secondary beams frame into or over primary beams;

(b) Where beams frame into columns;

(c) Where slabs frame into shear walls and other rigid elements;

(d) Guidance on these is given in HERA Report R4-113, *Notes Prepared for a Seminar on Composite Design and Construction*, 2002.

(E.2) Additional reinforcement will be required around openings in floors and re-entrant corners, comprising additional bars placed diagonally to the corner and anchored in accordance with NZS 3101.

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(E.3) Slab depth to account for all reinforcement. Designers shall ensure that allowance is made within the depth of the slab and especially the depth of the cover slab (t_o-h_{rc}) for all covers and reinforcement including lapping (including corner lapping of mesh), trimmer and drag bars (to transfer diaphragm action or for dependable inelastic response in fire).

Reinforcement transverse to the line of shear connectors is also required when the slab forms part of a composite beam, to prevent longitudinal splitting of the slab due to transverse tensile forces generated in the slab by the longitudinal shear forces generated in the shear connectors. Catering for longitudinal shear splitting is not usually a problem over secondary beams, except for certain secondary edge beam configurations as detailed in 13.4.1.3, but can often require additional reinforcement in the slab over primary beams. Requirements for primary beams incorporating shear connectors are given herein in 13.4.10; note also the short-comings of these provisions for certain edge beam configurations, as detailed in 13.4.1.3.

Profiled steel decking can be considered as transverse reinforcement if suitably anchored and profiled to develop the tensile forces required. Profiled steel decking which is oriented with ribs perpendicular to the steel beam and which is either continuous across the beam or rigidly held with stud shear connectors welded through the steel deck can be considered fully effective as transverse reinforcement.

The concrete within the slab develops some shear resistance which will reduce the amount of transverse reinforcement required (13.2). This contribution from the concrete is neglected within yielding regions of a composite member.

C13.2.3 Provisions for seismic design of floor slabs

These recommendations are straight-forward to apply; guidance is given in section 13.3.3.7 of (13.27).

C13.2.4 Bases for design and construction

When the floor system is designed, decisions will be made on the extent of propping and precambering to be used and the method for placing the concrete to achieve the required surface finish. These decisions have important implications for deflection of the floor system under the wet concrete, loads on the floor system during construction and long-term deflections of the composite floor system.

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If changes are proposed to these design bases prior to construction, then the implications of these on the adequacy of the floor system during construction and in service must be determined and changes made to the design where required. See HERA Report R4-107 (13.50) for details. In general:

(a) Changing from propped to unpropped construction or from precambered to non-precambered beams will increase the depth of concrete placed and may reduce the unsupported deck spans.

(b) Changing from unpropped to propped construction may increase the long-term deflection of the composite floor system, may lead to increased crack widths and will influence the construction programme.

Amd 2 Oct. '07 A method for calculating the increased concrete loading from ponding is given in section 1.3 of (13.50). Alternatively the specified thickness of the concrete over the floor system may be increased by $0.7\Delta_m$, where Δ_m is required from 13.2.4.3.

C13.3 CONNECTIONS BETWEEN STEEL AND CONCRETE FOR COMPOSITE ACTION

C13.3.1 Encasement of sections

These requirements are obtained from the Canadian Standard CAN3 – S16.1 (13.4) and reflect traditional composite construction practice prior to the advent of the profiled steel deck and weld-through shear stud. The provisions will ensure fully composite action for beams (flexural members), however not necessarily for columns or beam-columns, due to the effect of axial force. Composite action of columns is covered in 13.8 herein and in (13.29).

C13.3.2 Shear connectors

C13.3.2.1 Nominal shear capacity of connectors

Equations 13.3.2.1 and 13.3.2.2 are obtained from CAN3 – S16.1 (13.4). Equation 13.3.2.1 is also found in the American Institute of Steel Construction LRFD specification (13.21). Equation 13.3.2.2 gives similar shear strength for channel connectors to the values recommended by Hogan (13.3) especially for the smaller sizes of channel.

The design capacity, $\phi_{sc}q_r$, of shear connectors given by Equations 13.3.2.1 and 13.3.2.2 has not been altered by Amendment No. 1. What has been done is to multiply ϕ_{sc} by (1/0.8) and to multiply q_r by 0.8. This brings the factor of 0.8 directly into the equation for determining shear stud nominal capacity, which provides better agreement with experimental results. For more details, see pages 25 and 26 of the HERA *Steel Design and Construction Bulletin*, Issue No. 55, 2000. The experimental work on which these changes are based has been undertaken on end welded studs, represented by Equation 13.3.2.1, with the concepts also applied to Equation 13.3.2.2, which covers channel connectors.

The lower value of ϕ given for shear connectors in table 13.1.2(1) which are situated in negative moment regions takes account of the reduced holding power of the stud in cracked concrete, as determined by Johnson et al (13.26) through experimental testing. It is incorporated into current UK design practice (13.9, 13.30).

Equation 13.3.2.1 is only applicable for studs with a ratio of length/diameter $(h_{sc}/d_{sc}) \geq 4.0$. Equations applicable to shorter studs, down to $h_{sc}/d_{sc} = 3.0$, are available, e.g. (13.25) and can be used. Alternatively equation 13.3.2.1 can be used if a 25 % reduction in stud strength determined from the first part of the equation (associated with concrete failure) is applied for studs with $h_{sc}/d_{sc} = 3.0$, with linear interpolation used for studs with values of h_{sc}/d_{sc} between 3.0 and 4.0. No reduction need be made in the second part of equation 13.3.2.1 (associated with failure of the stud itself) for studs with values of h_{sc}/d_{sc} between 3.0 and 4.0.

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The role of the shear connector is to provide sufficient strength and stiffness in resisting longitudinal shear at the steel/concrete interface, so that:

(i) The shear stud will yield prior to splitting of the concrete rib occurring;

(ii) There will not be an abrupt or significant loss of shear capacity when splitting occurs, but a gradual decline from the peak shear capacity with increasing longitudinal slip.

These performance requirements will need to be determined by experimental test and/or rational analysis for shear connectors not covered by the provisions of (a) or (b) or outside the scope of 13.3.2.2 and 13.3.2.3. The concepts involved are described in detail in the HERA *Steel Design and Construction Bulletin* Issue No. 55, 2000, pp. 18-28, with the background to these concepts given in (13.46).

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The replacement equation for shear stud capacity is a simplification of Equation 2.37 from (13.46) which provides better agreement with experimental tests, e.g. (13.47) and removes the artificial representation of shear stud failure as occurring either within the concrete or within the stud. That equation is given below. The simplification involves substituting for the elastic moduli of steel and concrete and solving for $n = 5$, which is a minimum realistic number of studs in the shear span.

$$q_r = \alpha_{dc}\left(4.3 - \frac{1.1}{\sqrt{n}}\right)A_{sc}f_u^{0.65}f_c^{0.35}\left(\frac{E_c}{E_s}\right)^{0.40} \leq 0.8\, f_u A_{sc} \quad \text{(Eq C13.3.2.1)}$$

where

n = the number of studs in the shear span from Equation 13.4.9 (1) or n' from Equation 13.4.9(2), when the latter equation is required.

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C13.3.2.2 Stud shear connectors used with profiled steel deck

The strength of a shear connector in a solid slab may be reduced in a ribbed slab formed by casting concrete onto a profiled steel sheet deck. The extent of any reduction depends on the orientation of the steel deck profile relative to the steel beam, the profile itself and the number of rows of shear connectors placed.

The provisions of 13.3.2.2 are based on the American Institute of Steel Construction specifications (13.10, 13.21) and the detailed background to these provisions is contained in the commentary to (13.21). The research work undertaken, from which these provisions were derived, was based on a range of steel deck profiles and welded shear studs of different diameter and the results are only applicable to the range of products tested. These limits are given in 13.3.2.2.1 and shown in figure C13.3.2. Provision (d) is intended to place the stud head in the compression zone of the concrete under positive moment.

The new sub-clause (f) in 13.3.2.2.1 addresses the approach to take when through deck studs in a secondary beam configuration cannot be welded in the centre of the rib. This requires studs to be placed on the "weak" side or the "strong" side of the rib. Push off tests (13.47) show a difference in stud capacity of some 20 % between these two positions. However, full scale tests (13.49) show that provided the studs are placed alternately on the weak side and the strong side along the beam, the net result is the same as if all studs had been placed in the centre.

Compared with test results (13.47), the existing Equation 13.3.2.3 is significantly unconservative, especially for two studs. The replacement equation is from Eurocode 4 (13.48).

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C13.3.2.3 Detailing requirements for shear connectors

These requirements complete the information necessary to designers to ensure shear connectors

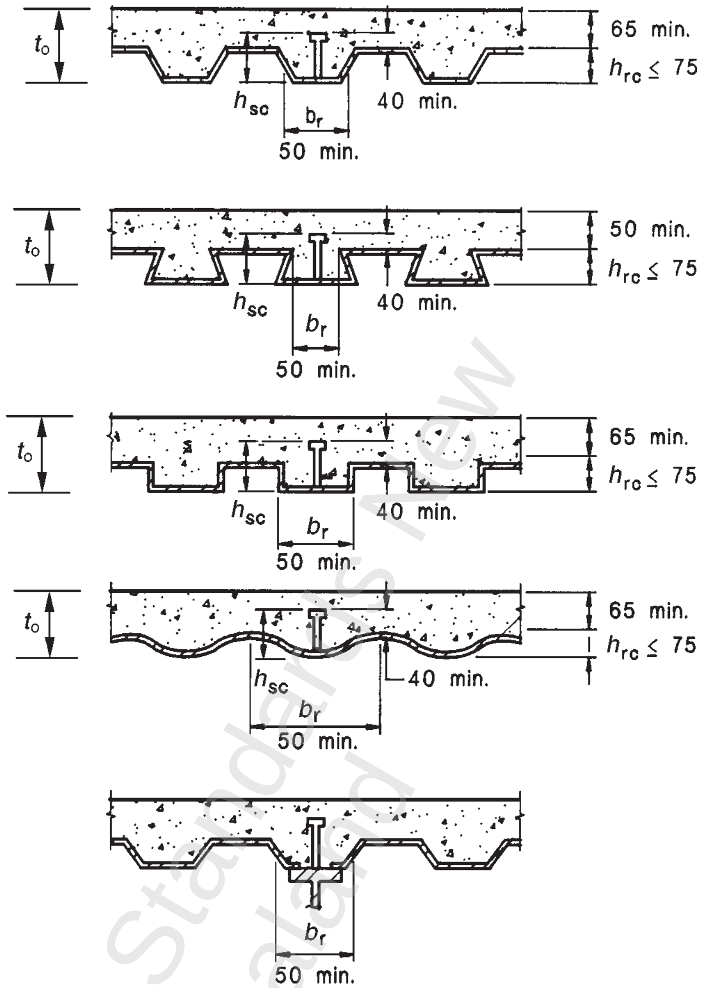

Figure C13.3.2 – Detailing/design limitations on stud shear connectors with profiled steel deck

NOTE – In figure C13.3.2, all dimensions shown are in millimetres.

perform satisfactorily in composite construction. The source of, and if necessary reason for, each of the recommendations (a) to (j) of 13.3.2.3 is given below:

(a) Is obtained from BS 5400: Part 5 (13.9). A welded shear stud obtains its shear strength through the generation of high tensile loads at the head of the stud, which transfer to shear loads at the base. Therefore a head on the stud is essential in developing the shear strength. This requirement is always satisfied for commercially available welded studs.

(b) And (c) are obtained from several references, especially Hogan (13.3). Recommendation (c) is to prevent a stud pulling out from a thin flange prior to obtaining its ultimate shear strength and to prevent burn-through of too thin a flange during stud welding. For components subject to severe fatigue actions, e.g. in bridge design, more stringent limits are recommended by (13.9).

(d) Is obtained from Chien and Ritchie (13.2). This reference also recommends further reduction factors in stud strength for L-beam construction when the stud is placed less than the overall slab thickness from the free surface of the concrete. Details are given in section 13.3.4.4 of (13.27).

(f) Is obtained from the AISC specification (13.21) and prevents overstressing of the slab concrete due to overlap of the zone of high shear stress around an individual shear connector. ➤

(h) Is obtained from 2 sources, (13.4 and 13.21).

(j) Is obtained from (13.1) and is intended to act as a further safeguard against transverse shear splitting of the slab, with subsequent loss of effective composite action, in yielding regions of composite beams.

C13.3.3 Other methods of achieving composite action

This clause allows for the use of shear connectors such as powder-actuated fasteners.

Externally reinforced concrete, where the reinforcement consists of unpainted steel plate encasing unreinforced or internally reinforced concrete, may also be designed in accordance with this Standard on the basis of 13.3.3. The performance of the composite section is dependent on shear transfer between the unpainted steel plates and the encased concrete and must be determined by experimental testing. Comprehensive testing of thin plate externally reinforced concrete (ERC) structural systems is ongoing at the University of Auckland (13.43) and more detailed design guidance will become available for ERC in the future.

C13.4 DESIGN OF COMPOSITE BEAMS WITH SHEAR CONNECTORS

C13.4.1.1 Stress distribution in steel and concrete

For design of simply supported or continuous composite beams, a plastic distribution of stresses across the section is required in design for the ultimate limit state to the provisions of 13.4. Details of such a stress distribution, utilizing a concrete stress block the same as that used in reinforced concrete design to NZS 3101 (13.13), are presented in section 13.3.6 of (13.27).

This is not the only permissible method of analysis of composite beams in accordance with 13.4.1 and AS/NZS 1170 set. An alternative plastic stress distribution and an elastic stress distribution are presented in BS 5950: Part 3.1 (13.30) and design of both simple and continuous beams may be undertaken to these provisions. In this case, the commentary to BS 5950: Part 3.1 (13.31) will be of assistance to designers. Design to the provisions of (13.30), utilizing the load combinations and load factors of AS/NZS 1170 set, will provide the necessary level of reliability against collapse consistent with that required for ultimate limit state design to AS/NZS 1170 set.

The use of a plastic distribution of stresses is dependent on the elements of the steel section having slenderness ratios sufficiently low to develop the plastic stress distribution without local buckling. The limiting slenderness ratios are presented in 13.4.3; if design is being undertaken to BS 5950: Part 3.1 (13.30), use the appropriate section geometry provisions therein for applications not subject to earthquake loads or effects or other inelastic actions. For applications which are subject to earthquake loads or other inelastic actions, use the appropriate section geometry provisions from this Standard, irrespective of the design method being followed.

C13.4.1.3 Suppression of longitudinal shear failure

The new sub-clause 13.4.1.3 (1997 edition) identifies areas where the longitudinal shear provisions of 13.4.10 may be deficient. In such applications, the beams shall either be designed as non-composite or designed to (13.44, 13.45) for longitudinal shear.

C13.4.2 Design effective width of concrete slab

The following parameters are important in determining the appropriate design effective width of concrete slab:

(a) The nature of the applied loading (concentrated or uniform);

(b) The type of beam span (simply supported or continuous);

(c) The ratio of flange thickness to beam depth;

(d) The presence of a haunch in the slab;

(e) The ratio of the beam span (or length between points of zero moment) to the distance between beam centres;

(f) The degree of interfacial slip.

The exact determination of effective width is complex and all design methods for determining effective width present a simplified procedure which sacrifices some accuracy for ease of use. Traditional design approaches for composite construction (13.4) and for reinforced concrete construction (13.13) give a method which uses a simple function of either the slab thickness, beam centre-to-centre spacing or beam length to determine the effective width. Alternatively, and much more accurately, BS 5400: Part 5 (13.9) presents several design charts of effective width factors accounting for variations in (a), (b), (c) and (e) above.

The method of determining effective width given in 13.4.2 conservatively underestimates the effective width for simply supported or continuous composite beams subjected to uniform loading or predominantly uniform loading. This covers primary and secondary beams in normal office or warehouse loading conditions. However, for composite beams subject to concentrated, heavy applied loading, the effective width procedure contained in section 5.2.3 of (13.9) should be followed.

C13.4.2.1 Effective width of midspan positive moment regions
The provisions of 13.4.2.1 are obtained from the Canadian limit state steel code (13.4). The effective width limit based on slab thickness has been removed. It was derived from elastic behaviour and is not applicable to ultimate limit state conditions (13.4, 13.21). The variation of effective width at ultimate load does not significantly affect the moment capacity of the composite beam (13.4).

C13.4.2.2 Effective width for support negative moment regions
There is no concrete in compression at support negative moment regions, so the only purpose for which the effective width at these locations is required is to determine the amount of slab reinforcement that acts in tension as part of the composite section. Nevertheless this area of reinforcement can have a significant impact on the section moment capacity at the support and on the section geometry and lateral bracing requirements for the steel member.

Morrison (13.6) indicates that the effective width over the supports in negative moment regions should be taken as 0.6 times the effective width at midspan, although he cites no supporting evidence. This approach has been adopted herein and is in good agreement with effective widths obtained from the much more comprehensive recommendations of BS 5400: Part 5 (13.9). However research work at Le High University (noted by reference 13.11) would indicate that the negative moment effective width at the supports is similar to the positive moment effective width.

C13.4.3 Steel beam section geometry requirements

The section geometry requirements for composite sections subject to both positive and negative moment actions are affected by more criteria than are required to set the requirements for bare steel members alone. The requirements of 13.4.3 are such as will ensure that a fully plastic stress distribution can be used in the section under consideration when calculating the moment capacity, except as noted in 13.4.8.2.

Designers may elect to use more slender members, in accordance with 13.4.1, if these can be shown to ensure satisfactory member action. The calculation of moment capacity will, in this instance, need to be based on an elastic stress distribution in the member and the inelastic rotation demand on the member will need to be prevented or severely limited as appropriate to the section geometry of the steel member chosen. Guidance compatible with this Standard and AS/NZS 1170 set is available from several publications, including (13.21, 13.30, 13.31). ➤

The section geometry requirements presented in 13.4.3 are based on the relevant requirements for seismic and non-seismic design given in the American LRFD specification (13.21), in BS 5950: Part 13.1 (13.30), and in 2 papers on continuous composite construction (13.5 and 13.23). The requirements have been amended slightly herein to align with the relevant section geometry requirements used elsewhere in this Standard and are presented in the same format.

A brief explanation of the limits presented in 13.4.3 is given below:

(a) *The flange limiting slenderness ratio* b/t_f

These are governed by the flange in compression, which will be the top flange under positive moment action and the bottom flange under negative moment action. The top flange derives considerable restraint against local buckling from the integral connection to the concrete slab in positive moment regions, thus enabling a slenderness ratio up to $16/\sqrt{(f_y / 250)}$ to be used in conjunction with a plastic stress distribution. Under negative moment this additional restraint is not available, resulting in flange slenderness ratios applicable to the bare steel beam alone being necessary to achieve a plastic stress distribution.

(b) *The web limiting slenderness ratios* d_1/t_w *and* d_{1c}/t_w

When a plastic stress distribution is used to calculate flexural strength, the web within the compression zone will be fully yielded in compression as well as carrying shear force. This can only be achieved if this region of web is stocky in both shear and compression. Conversely the web within the tension zone will be kept inherently stable in shear by virtue of the tension force generated through flexural action.

The appropriate slenderness ratio to control buckling of the web within the compression zone for a member subjected to only nominal (category 3) inelastic demand ($\theta_p \le 10$ milliradians) is given by table 12.5 as $40/\sqrt{(f_y / 250)}$. The limit given in 13.4.3.1 (a) is based on an effective web depth, d_{1c}, equal to twice the depth of the web in the compression zone, which increases the appropriate slenderness ratio limit to $80/\sqrt{(f_y / 250)}$. This has then been increased slightly to $82/\sqrt{(f_y / 250)}$, which is the limit at which full shear yield in the web can be sustained from 5.11.2.2. A similar procedure has been undertaken to give the limiting web slenderness ratio required for greater inelastic demand on the member ($\theta_p > 10$ milliradians).

Note that the inelastic rotation demand (θ_p) used in this clause is the same quantity as that used in tables 4.7 for non-seismic (non-cyclic) applications. The limiting value of θ_p = 10 milliradians is based on that of table 4.7(4), in recognition of the large internal compression force induced in the steel section by composite action of the member.

The above limits on web slenderness are the only limits needed for positive moment regions, however for negative moment regions – i.e. continuous construction – an additional limit on the actual depth of web between flanges is incorporated, in line with American and UK requirements (13.21, 13.30). In negative moment regions the first limit on web slenderness proposed was by Morrison (13.6), in 1974, which reduced the slenderness ratio (d_1/t_w) in proportion to the amount of compression force exerted on the steel section through composite action with the tension reinforcement in the concrete slab. Provision was made to double the slenderness ratios permitted with the addition of mid-height web stiffeners complying with certain requirements for stiffness and strength.

The most current guidance/requirements on continuous composite beam design (13.20, 13.31) recommends that both requirements be adhered to in view of the lack of comprehensive research results in this area.

Both limits have been given, therefore, for composite beams in negative moment regions, with Morrison's criterion regarding the effect of the additional compressive force in the web of the section on the web slenderness contained in equation 13.4.3.2. The beneficial effect on both limits of stiffening the web is incorporated into these provisions and the strength and stiffness criteria for the pair of stiffeners, if needed, are given in 13.4.3.3.

In negative moment regions, where the shear connectors are terminated a significant distance from the column face (not to be less than 1.5*d*) the bare steel beam alone carries the negative moment and the appropriate section geometry requirements, as given in table 12.5, apply for load combinations including earthquake loads and can be considered applicable in other situations as described below.

C13.4.3.1 Requirements for positive moment regions

All typical simply-supported beams will come under part (a) of this clause, as the inelastic rotation demand on these members will be negligible.

Part (b) of this clause is likely to apply only to regions of beams in rigid frame construction which are dominated by seismic actions adjacent to supports and hence will apply in conjunction with the requirements of 13.4.11.3.2.

C13.4.3.2 Requirements for negative moment regions

Part (a) is intended to apply to the seismic design of a category 3 member, or to non-seismic design based on an elastic analysis, either without moment redistribution or with moment redistribution limited to that given by table 4.5.4.2 for a category 3 member, as the inelastic demand on the member in both cases will be nominal only.

Part (b) is intended to apply to the seismic design of a category 1 or 2 member, or to non-seismic design of a member based on a plastic analysis of the continuous beam assemblage of which the member is a part, or based on an elastic analysis of that assemblage with moment redistribution to either that given by table 4.5.4.2 for a category 1 or 2 member, or to 4.5.4.1 with θ_p > 10 milliradians (10 x 10^{-3} radians).

C13.4.4 Lateral restraint of steel beam

Comprehensive design guidance on lateral restraint of the beam during the construction stage and in the final service condition is given in section 13.3.6 of (13.27) for steel beams in general and in (13.23) for continuous composite beams in particular.

C13.4.5 Calculation of the positive moment capacity of the composite section

Equations 13.4.5 (1) to 13.4.5 (9) are derived for the plastic stress distribution given in 13.4.1. A background to their derivation is presented in (13.2, 13.27).

C13.4.6 Limits on use of partial composite action

A composite beam with less than 50 % connection capacity may not behave as a composite member through the entire loading range up to the ultimate state as predicted by the ultimate limit state design procedure given in this Standard (13.2). Therefore a lower limit of 50 % shear connection capacity is stipulated for calculation of moment capacity for the ultimate limit state. Tests (13.33) support this limit; in fact indicate that it is conservative.

For deflection calculations, composite action is required for the serviceability limit state only and the minimum limit of shear connection capacity required decreases to 25 % (13.2).

C13.4.8 Calculation of the negative moment capacity of the composite section

In many instances, shear connectors will not be placed over negative moment regions and the design capacity will be that calculated for the steel section alone.

If shear connectors are placed over the negative moment region the calculation of negative moment capacity is straight-forward. Details are presented in (13.3, 13.23, 13.30 – 13.32).

C13.4.9 Spacing of shear connectors

The initial spacing of shear connectors was based on the shear stress distribution along a composite beam, which resulted in a variable connector spacing.

The work of Slutter and Driscoll (13.12) caused a change in philosophy when it was shown that, provided sufficient shear connector strength was provided between points of maximum and adjacent zero moment, the behaviour of composite beams utilizing uniformly spaced shear connectors under predominantly uniform loading was very similar to beams with variable spaced shear connectors. The design recommendations have been refined over the 2 decades since Slutter and Driscoll's initial research was undertaken to produce the recommendations given in this clause and derived in (13.4 and 13.21).

Under non-uniform loading, the shear connector spacing may need to be varied, with shear connectors concentrated in regions of high shear. Clause 13.4.9.2 presents the necessary details, taken from (13.4). The UK Standard (13.30) contains the same provisions. Less conservative, but more complicated provisions are given in (13.44).

A new subclause, 13.4.9.3, has been added to define what is meant by non-uniform moment conditions. The provisions of 13.4.9.3 are intended to activate the use of equation 13.4.9(2), thus invoking a non-uniform distribution of shear studs over the positive moment region, when the moment induced by concentrated load(s) at a given location exceeds the moment at that location, induced by uniform loading, by more than 10 %.

C13.4.10 Longitudinal shear

In order to develop the compressive force in the portion of the concrete slab outside the potential shear plane shown in figure C13.4.10, net longitudinal shear forces, of magnitude V_l^*, must be developed along these planes. The expressions given in this Standard are from the Canadian limit state standard (13.4). Values of (V_r) for semi-low density and low density concrete are given by Chien and Ritchie (13.2).

The contribution from the concrete towards the design shear resistance is neglected within the yielding regions of a composite member. This can be achieved in design by requiring the design longitudinal shear, V_l^*, to be resisted by a reduced length of shear plane which neglects the length of yielding region.

Profiled steel decking can be considered as transverse reinforcement if suitably anchored and profiled to develop the tensile forces required. Profiled steel decking which is oriented with ribs perpendicular to the steel beam and which is either continuous across the beam or rigidly held with stud shear connectors welded through the steel deck can be considered fully effective as transverse reinforcement, except as identified by 13.4.1.3.

Note the applications identified through 13.4.1.3 for which 13.4.10 may not provide adequate coverage for longitudinal shear and the referenced procedures required to overcome this (see C13.4.1.3).

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The design for longitudinal shear resistance involves a check for the adequacy to resist splitting of the concrete across potential longitudinal shear planes, such as those shown in figure C13.4.10. This form of splitting must not be confused with potential splitting of the concrete along the line of the shear connectors, due to high transverse tensile forces developed locally in the concrete near the bases of the connectors. The latter is an important factor to consider in the calculation of shear connector capacity; see 13.3.2. Detailed coverage of both forms of splitting is given in (13.46).

The addition of transverse reinforcement to resist longitudinal shear is generally only necessary in primary beams, which support the loads of incoming secondary beams. It is a likely requirement when 2 or more rows of shear studs are used; conditions as shown in figure C13.4.10.

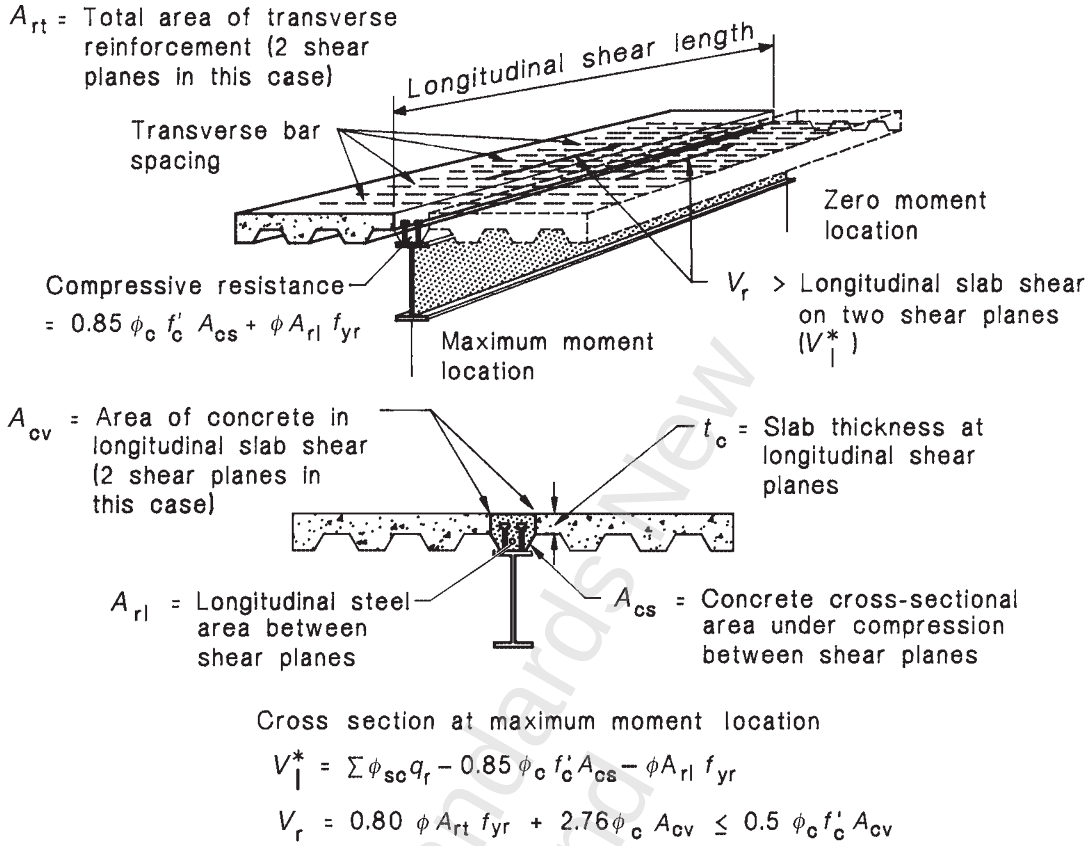

Figure C13.4.10 – Potential longitudinal shear planes in a primary tee-beam

The requirements to suppress a longitudinal shear failure in the cover slab have been recognised in past editions of the Standard and are not changed in this amendment.

However where the concrete rib is relatively narrow the shear stud strength is limited by cracking along the line of the studs. When this happens it is important that the post splitting capacity does not abruptly reduce. To prevent this, transverse reinforcement placed in specific locations is required and the relevant provisions are given in the new clause 13.4.10.4. They are based on (13.46), which provides guidance on the appropriate placement of this reinforcement.

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C13.4.11 Special seismic requirements for inelastic action

The provisions of 13.4.11.1 to 13.4.11.3 are obtained directly from the NZNSEE Study Group Paper I (13.1) and their purpose is to ensure that composite beams of a given category will provide the dependable ductile action required for that category from 12.2.5 and 12.3.

The provisions are presented as performance criteria. Section 13.3.5 of the HERA Design Guides Volume 2 (13.27) provides detailed design and detailing guidance on methods of achieving these performance criteria for a range of applications.

C13.5 DESIGN OF COMPOSITE BEAMS WITHOUT SHEAR CONNECTIONS

Externally reinforced concrete beams are an example of composite beams without shear connectors; refer to Commentary Clause C13.3.3 for further details.

Concrete encasement for passive fire protection remains a valid reason for encasing an *I*-section, however it does not provide composite column action. Detailing requirements for this purpose are presented in section 7 of (13.34). The effect of concrete encasement for fire protection alone on the seismic performance of the section is negligible, as covered in section 13.4.4 of (13.27).

Amd 2 Oct. '07

These requirements are to change specific clause numbers referenced from NZS 3101:1995 to the relevant clause from NZS 3101:2006.

C13.6 SHEAR STRENGTH OF COMPOSITE BEAMS

Generally shear strength of composite beams is not a limiting design criterion and the simple approach of designing the steel member to carry the shear is adequate. However the shear strength of a composite beam does exceed that of the steel beam alone (13.6) and the additional shear strength available can be used in limit state design if required.

C13.7 END CONNECTIONS TO COMPOSITE BEAMS

The general performance requirement of this clause directs designers to design connections as either bare steel or composite items. In the former case, recourse is made to the relevant parts of this Standard, especially section 9 and the relevant sub-clauses of 12.3 for seismic applications.

When the connection itself is to be designed as a composite item, detailed guidance is given in section 13.5 of (13.27).

C13.8 DESIGN OF COMPOSITE COLUMNS

C13.8.1 Scope

The scope of 13.8 is unchanged from that of the 1989 edition of this Standard, in that it applies to design of encased or concrete-filled columns for full composite action only. The 1989 edition of this Standard allowed for a low level of longitudinal and transverse reinforcement, for concrete-encased I-section columns, with an increased resistance to out-of-plane buckling imparted to the bare steel section but the concrete not considered as a load-sharing element. This option is no longer presented in this Standard and was rarely used in practice.

Amd 2 Oct. '07

These requirements are to change specific clause numbers referenced from NZS 3101:1995 to the relevant clause from NZS 3101:2006.

C13.8.2 Design of encased composite columns

C13.8.2.2 Consideration of second-order effects/determination of effective length

In the design of a bare steel column section the following aspects must be considered with regard to effective length determination (13.27).

(a) Effect of inplane restraint of the column through the calculation of the column effective length;

(b) Effect of local and lateral buckling of the column;

(c) Magnification of any applied forces on the column due to the ratio of design axial compression load to the elastic buckling load.

In the design of an (encased) composite column (a) and (c) are still important, while (b) is no longer of concern due to the suitably reinforced concrete encasement.

There are comprehensive provisions given in section 4 for the determination of second-order effects and these may be utilized to determine the effect of (a) and (c), provided flexural stiffness and radius of gyration values appropriate to the encased composite column section are used. The relevant requirements presented in 13.8.2.2.1 and 13.8.2.2.3 are taken from NZS 3101:1995

(13.13) and their background is given in (13.14). Once any appropriate magnification to the moment as determined from analysis is made, accounting for P-Δ and P-δ effects, then the section capacity is used to resist the combination of axial compression force and magnified moment.

NZS 3101 (13.13) requires a special study for reinforced concrete columns with geometrical slenderness ratios exceeding 100 and this requirement is included in 13.8.2.2.1 for encased composite columns. The factors that must be considered are detailed in NZS 3101. As an alternative, the reduction in section to member compression capacity caused by the slenderness ratio for slender encased composite columns can be determined by applying the provisions of 6.3 of this Standard, using k_f = 1.0 and α_b appropriate to the type of encased steel member.

C13.8.2.3, C13.8.2.4 Longitudinal and transverse reinforcement requirements

The aim of the longitudinal and transverse reinforcement requirements is to ensure that the column behaves as a fully composite member and that the required ductility capacity of any members subject to inelastic seismic action is achieved.

The requirements for transverse reinforcement are obtained from (13.1, 13.27) and are based on a comprehensive study of experimental tests on composite columns under inelastic cyclic and static loading. They incorporate some provisions from NZS 3101 (13.13) and cross-reference to others. The 1982 edition of NZS 3101 was revised into limit state format as NZS 3101:2006 (13.13.). Changes to clause numbering and content of NZS 3101 have necessitated changes to this Standard, which have been introduced in this revision.

Amd 2 Oct. '07

Detailed guidance on the application of 13.8.2.3 is given in section 13.4.2 of (13.27).

C13.8.3 Design of concrete-filled structural hollow sections

C13.8.3.1 Scope

The requirement for minimum concrete strength is obtained from (13.19), which also contains comprehensive design and detailing guidance for this type of composite member.

C13.8.3.2 Section geometry requirements

The wall thickness of the steel hollow section must be sufficient to effectively confine the concrete core and to achieve full composite action. The wall slenderness requirements presented for this purpose in the 1989 edition of the Standard were taken from NZS 3101 (13.13). They were considered sufficient to fulfil the 2 criteria for all non-seismic and seismic applications (13.14).

Detailed experimental testing undertaken in 1987 by Park et. al. (13.35) indicated that the limits in the 1989 Standard were very conservative and could be doubled without compromising achievement of the confinement and composite action criteria. The relevant limits for bare steel flat plate elements in compression presented in table 5.2 of this Standard represent slightly less than a doubling of the 1989 edition limits for concrete-filled composite columns and so have been adopted. When a concrete-filled column must function as a bare steel column prior to being concrete filled, the bare steel limits from table 5.2 or table 6.2.4 are also appropriate for this construction stage. The limits presented in 13.8.3.2 apply to the composite concrete-filled column for non-seismic applications and for all member categories of seismic application.

Bare steel tubes which support compression load during construction must comply with the effective width requirements of 6.2.4 in determining their nominal section capacity. The yield slenderness limit for this is 82 from table 6.2.4. Tubes with slenderness limits much in excess of this will not be able to carry a significant design compression load, because the effective outside diameter, d_e, calculated from 6.2.4, will be less than the internal diameter, d_i. Refer to C6.2.4.4 for guidance on what constitutes a significant level of design compression load.

C13.8.3.3 Design procedure

A composite concrete-filled column may be designed in accordance with the reinforced concrete design procedure specified in NZS 3101 (13.13). Computer software developed for reinforced concrete column design may be used by splitting up the effective area (A_e) of the steel section into discrete elements and inputting these as effective longitudinal reinforcement, although a nominal cover needs to be applied to make it work.

There is a design procedure (13.19) written expressly for concrete-filled composite column design, however, which may be used. This procedure utilizes partial strength reduction factors applied to each material, prior to calculation of the design section capacity directly. This is alluded to in table 13.1.2(1). Use of (13.19), in conjunction with the relevant load combinations and load factors from AS/NZS 1170.0, will fulfil the requirement expressed in 13.8.3.3 that the level of reliability against failure is consistent with that required for ultimate limit state design to AS/NZS 1170.0.

C13.8.3.5 Load transfer at connections to steel shell only

The exact mechanism of load transfer is complex and is still the subject of active research. The limit of 0.4 MPa contained in the 1989 edition of this Standard has been increased to 0.7 MPa herein after review of UK and New Zealand research (13.29, 13.35).

There is no specific guidance yet available to determine conclusively the most appropriate area of column wall within the concrete-filled structural hollow section for vertical load transfer to take place from the beam into the column. At this time, suitable guidance is to assume that this vertical load transfer occurs through an area of column wall equal to the width times depth of the incoming steel beam (i.e. (b.d)) for all types of connections used, unless the connectors or connection components within the connection to the column extend outside this area, in which case a larger area of column wall, which contains the connectors or connection components, may be used.

Tests (13.19) have indicated that local dependable shear transfer, without shear connectors being provided, is lost under heating of an unprotected steel section in fire and hence positive mechanical shear connection is required to carry the design shear load considered to be acting during a fire. This ultimate limit state design load is determined using the load combination for fire specified in AS/NZS 1170.0.

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C14 FABRICATION

C14.1 REJECTION OF A FABRICATED ITEM

Section 14 provides the minimum requirements for the standard of workmanship which should be achieved by the fabricator in order to ensure that the design assumptions fundamental to sections 5 to 13 are attained.

Clauses 14.1.1 and 14.1.2 specify under what conditions a member may be 'liable to rejection'. Rejection for failure to comply with 14.2, 14.3 or 14.4 is not automatic, as it is recognized that there may be sufficient reserve design capacity in the fabricated item for the item to be accepted nonetheless, or that more damage might be done by trying to rectify the non-compliance.

For a non-complying item, the options are given of either carrying out a revised assessment of the design capacity of the item in its 'as fabricated' state, or subjecting the item to a proof test carried out in accordance with 17.4.

The design provisions of sections 5 to 9, 12 and 13 are consistent with the provisions of section 14, particularly with the tolerance limits of 14.4. These tolerances have been considered when evaluating suitable expressions for the member or connection nominal capacities. Particularly important in this regard are the straightness provisions for members subjected to bending or compression.

The provisions of section 14 are also compatible with the HERA General Specification for the Fabrication and Erection of Structural Steel (14.15). This specification (14.15) is directly applicable, therefore, to designs undertaken in accordance with this Standard.

C14.2 MATERIAL

C14.2.1 General

A manufacturer's certificate of compliance with the specified material supply standard can be requested, but such a request is usually contained in the contract documents rather than being introduced during fabrication.

All material supply standards specified in section 2 provide methods of removing surface defects from as-rolled steel. The provisions of these standards should be followed in this regard. Requirements are also given in (14.15).

Two types of internal imperfections in steel material are of particular interest, although not mentioned in this Standard. They are:

(a) Laminar imperfections, and

(b) Lamellar tearing.

Laminar imperfections are discontinuities in planes parallel to the plane of rolling. Welding can result in shrinkage forces normal to a steel surface, and weld shrinkage may induce a level of force sufficient to cause the discontinuity to open up. Such an occurrence is termed lamellar tearing.

It is accepted that for lamellar tearing to occur, the welded joint detail must be such that the steel is subject to a high level of through-thickness strain due to high restraint and a high level of weld shrinkage force, and that the steel has reduced through-thickness ductility or a pre-existing discontinuity, such as a laminar imperfection (14.1).

The steel used in risk areas should be selected for properties which minimize the possibility of lamellar tearing. In most instances, correct design and detailing of a connection and correct fabrication will mitigate or remove the problem. Guidance on lamellar tearing may be found in (14.1).

New Zealand and Australian steels are now made by the continuous casting process, giving a fully killed steel in which laminations are no longer a major problem. It is possible to use ultrasonic testing of steel members in critical areas in order to identify discontinuities or laminations present in the steel. This allows critical connections involving significant size welds to be located in relatively clean areas of steel. The only 2 general areas of structural steel use where the possibility of lamellar tearing is greater than extremely remote are in welded moment-resisting beam to column connections involving thick column flanges (over 50 mm) or welded column to baseplate connections involving thick baseplates (over 50 mm) and the need to resist significant design moment or tension.

Positions on structural members or plates where the existence of a lamination may prove detrimental, and the extent of any ultrasonic testing required to identify laminations, should be stated in the contract documents.

C14.2.2 Identification

Steel is clearly identified by a marked designation after rolling. This takes the form of either rolled-in symbols or stencilled marking. For heavy plate, each item is stencilled, for coiled steel (16 mm thickness or less) or light plate (12 mm thick or less), each coil or pack of plate is identified by stencilling the outer surface of the coil or the top plate of the pack.

Colour coding is also used as a form of identification in distribution centres and steel fabrication yards. The colour code used varies amongst companies at this time.

The intention of this clause is to ensure that the identification is still evident during the fabrication process (i.e. after cutting etc.).

The steel fabrication industry recognizes the importance of identifying various items during processing and, as components are cut from mill material, an identifying number should always be applied (14.15). Thus, parts cut from a particular grade of material can be identified throughout fabrication and on to the erection phase.

Identification of the steel grade will normally be ensured as part of the fabricator's quality assurance procedures (14.15).

C14.3 FABRICATION PROCEDURES

C14.3.1 Methods

The Standard recognizes that, for steel complying with the standards and material selection provisions specified in section 2, steel members can be bent or formed by cold bending for member thicknesses up to the energy capacity of available equipment.

Heating can be used to remove out-of-straightness or bends in a member, or to deliberately camber or curve a member. This changes the residual stress pattern due to the plastic deformations, but these changes do not normally affect the design capacity as allowances for residual stress are incorporated in the expressions for design capacity.

Relying on heating for curving or cambering can result in some deviation from the desired result due to workmanship error or handling.

If hot processes are required to shape plates, camber beams, straighten members or correct out-of-tolerance, temperatures are required to be controlled to less than 650 °C. This is designed to ensure that the yield stress and tensile strength of the steel are not permanently reduced due to the heating process.

If heating above 650 °C is required, specialist advice from the steel manufacturer should be sought as to the permanent effect on the yield stress, tensile strength and ductility.

Temperatures should be checked by using temperature-indicating crayons during the application of heat.

For cold bending of plates and flats, the internal radius of a bend should not be less than that specified in AS/NZS 3678. For hot bending, the temperature (which normally should not exceed 650 °C), timing and cooling rate should be determined by agreement between the design engineer and the fabricator. Minimum bend radii for hot or cold structural sections can be obtained from specialist firms, these minima being intended to avoid local buckling failures during bending. Guidance is also available from the literature (14.16).

C14.3.2 Full contact splices

With the almost universal use of cold-sawing as a means of cutting members to length, this method is permitted in addition to traditional machining (end-milling) methods. The modern cold saw is a milling cutter and produces a surface finish equivalent to end milling.

C14.3.3 Cutting

Information on the flame cutting of steel may be found in (14.2), which discusses flame cutting procedures in detail. Adherence to these cutting procedures is recommended. The Welding Technology Institute of Australia (WTIA) Cut Surface Replicas provide a ready method of assessing cut surface roughness. Roughness classes of these replicas are defined in terms of centre-line-average values (CLA) as follows:

Class	Roughness, mm x 10^{-3}		
1	0	to	6.3
2	6.3	to	12.5
3	12.5	to	24

The actual roughnesses of the WTIA Replicas are 3, 6.3 and 19 mm x 10^{-3} for Classes 1, 2, 3 respectively. All classes should be readily obtainable with good equipment and correct techniques. Gouges may also be assessed using reference guides on the WTIA Replicas. Recourse might also be had to AS 2382 – Standard on Surface Roughness Comparison Specimens.

The restriction on the shearing of plates thicker than 16 mm when a part is to be galvanized, relates to strain-age embrittlement when severe cold-working is followed by galvanizing, resulting in a loss of ductility. Stress-relieving before galvanizing means the 16 mm limit can be waived. This clause requirement is based on data from galvanizers (14.3).

The restriction on shearing of plates of any thickness in yielding regions of category 1, 2 or 3 members (see section 12) also relates to loss of ductility capacity. Shearing 3 mm oversize and machining excess material is acceptable but is not generally likely to be cost-effective.

Defective cuts may be remedied by grinding, to obtain the appropriate limits of table 14.3.3.

Re-entrant corners are of special concern since such corners are natural points of stress concentration and, accordingly, require a minimum radius. The corners are to be notch-free and any notches are to be repaired using the methods specified in this clause.

C14.3.4 Welding

A commentary on AS/NZS 1554.1 and AS 1554.2 is available in WTIA Technical Note 11 (14.4). Note that some design provisions from AS/NZS 1554.1 and AS/NZS 1554.5 are now incorporated into section 9 of this Standard.

The weldability of steels is discussed in WTIA Technical Note 1 (14.5).

C14.3.5 Holing

C14.3.5.1 General

Holes may be produced by a variety of methods available to a fabricator, except by hand flame cutting. This prohibited method has a tendency to produce rough edges of unsatisfactory appearance. Hand flame cutting of holes is expressly forbidden by this clause for this reason, except as a site rectification measure for holes in column base plates, even though studies have shown that hand flame cutting does not adversely affect the connection performance (14.6), at least for non-seismic applications.

The studies in (14.6) involved a comparison of 18 bolted, double-lap connections with holes fabricated by one of the following methods:

(a) Punching;

(b) Punching with burrs removed;

(c) Sub-punching and reaming;

(d) Drilling;

(e) Hand flame-cutting;

(f) Hand flame-cutting and reaming.

Plates investigated in the study were 2 9.5 mm plates, lapped with a single 12.7 mm plate, with bolts in double shear. The question of hand flame-cutting of holes in thicker plate material still requires investigation.

Even though this study indicated that no undesirable consequences result from hand flame-cutting plates of these thicknesses in Grade 250 material, the use of this hole-making method is intended only for site situations involving corrective work. It is not a recommended method as a primary technique for structural connections and accordingly is prohibited. Its use for corrective applications on site should be at the discretion of the construction reviewer (see 1.6.3.1) in conjunction with agreement from the design engineer.

There are 2 reasons for placing a restriction on the use of full-size punching.

(a) First, to avoid an excessively dished area in the immediate vicinity of the hole, which may occur even under competent fabrication practices and which may impair the strength of the connection;

(b) Secondly, to avoid metallurgical defects such as severe work-hardening, which may impair the strength of the connection, particularly in the case of high-cycle dynamically loaded structures. Note that 10.9 restricts material to a minimum thickness of 12 mm which is not to be exceeded for a punched hole in an element subject to fatigue.

By exercising very close control over the punching process, the amount of local deformation and work-hardening can be reduced to a level which may not be critical in many applications, and contract documents may relax the restriction on punching in such cases. At the present time, generally inadequate data is available on which to base less restrictive requirements than those of 14.3.5.1.4. The restriction in this clause is in line with comparable overseas standards.

A background to the theory of punching is given in (14.7). Recent studies (14.8) indicate no decrease in the ultimate strengths of connections with punched holes for static loads. This study was conducted on plates 6 mm and 10 mm thick. The conclusions drawn from the study included:

(a) Punched holes should not be permitted in plastically designed structures where deformation capacity may be required at net sections in tension, nor in structures operating at low temperatures nor in structures subject to fatigue loading;

(b) Cracks around punched holes are arrested in the surrounding unhardened material and any loss of ductility will probably not impair the performance of an elastically designed structure under static loading.

In this Standard, restrictions have been placed on punched holes in elements subject to fatigue (see 10.9) and subject to earthquake (see 12.9.4.5.3).

The limit specified in 14.3.5.1.4 regarding the maximum thickness which can be punched recognizes that stronger plates can only be punched in thinner thicknesses. The expression $5600/f_y$ as the thickness limit is an empirical expression whose usage has proved satisfactory over a number of years. The limit is consistent with that in many overseas standards, which commonly relate the maximum thickness which can be punched in Grade 250 steel to the hole diameter. The limit is only valid for statically loaded members and connection components or for non-yielding regions of members and connection components subject to seismic forces or moment redistribution.

Sub-punching is permitted on an unrestricted basis since subsequent reaming removes any damaged or distorted material, which usually lies within 0.5 mm to 1.0 mm from the inside surface of the hole.

C14.3.5.2 Hole size

The nominal hole size specified by 14.3.5.2.1 is 2 mm or 3 mm larger than the bolt diameter, with the greater value for larger bolt diameters recognizing the greater difficulty and larger tolerance required to get such a bolt into the hole.

Larger oversize holes in column base plates are permitted by 14.3.5.2.2 in order to assist in the erection of columns. This hole size is linked to the tolerance on anchor bolt locations given in 15.3.1.

The use of oversize and slotted holes in 14.3.5.2.3 and 14.3.5.2.4 follows American and previous New Zealand and Australian practice, based on research reported in (14.9).

C14.3.6 Bolting

C14.3.6.1 General

The projection of a bolt from the nut face is required by 14.3.6.1.2 to be one clear thread, which is approximately one pitch of the thread. This provision is intended to ensure that full thread engagement over the total nut depth has been achieved. This is accepted practice for snug-tight bolting categories, but is of critical importance for tensioned bolting categories (8.8/*TF* and 8.8/*TB*), where the achievement of the initial bolt tension specified in 15.2.5.1 is only possible if full thread engagement is achieved.

Clause 14.3.6.1.2 also requires that for a bolt installed in a connection, one full thread plus thread run-out is clear on the inside face of the nut. This provision is intended to ensure that a nut is never run up to the thread run-out (end of the thread).

The requirement that nuts used in connections subject to vibration be secured against loosening is intended to cover both intermittent and continuous vibration applications. Intermittent applications (such as on monorail beams where vibration is occasional and not severe, being neither of high amplitude nor of long duration), usually only demand the use of proprietary self-locking nuts.

In applications with continuous vibration (machinery floors, screens, or crushers), it is recommended that fully tensioned bolting categories (8.8/*TF* or 8.8/*TB*) be employed. Since the minimum bolt tension specified in 15.2.5.1 is equivalent to the proof load of the bolt, the likelihood of bolt loosening is low due to the plastic deformation of the bolt during installation.

C14.3.6.2 Tensioned bolt

A connection using a tensioned high-strength bolt should be identified in the contract documents as either of:

(a) Bearing-type; or

(b) Friction-type.

In the absence of any indication, fabricators usually assume a friction-type connection is called for and this is expressly stated in (14.15).

The use of the notation for bolting categories that is specified in 9.3.1 on the contract drawing is sufficient identification.

C14.3.6.3 Preparation of surfaces in contact

Prior to assembly, surfaces should be checked, particularly the areas around the holes, to ensure that they are capable of achieving effective contact between the load-transmitting plies.

The removal of all burrs or fins is a relatively expensive procedure, and the US Specification was the first to change the absolute requirement to remove all burrs and fins. The 1985 edition of the US Specification (14.10) requires that only burrs or fins preventing solid seating of the faying surfaces at the snug-tight stage be removed. In view of research and testing conducted by the AISC(US), there appear to be no valid reasons for removing burrs and fins for fully tensioned bolted connections. This Standard requires their removal only if they prevent solid seating at snug-tightening.

For friction-type connections, it is necessary to check that the contact faces have the specified as-rolled surface finish in order to ensure that the required slip factor can be achieved in the assembled joint. When a slip factor of 0.35 is assumed without test evidence, painted members should be masked at the contact faces, and any cleaning to remove paint should be done by flame-cleaning or grit-blasting.

It has been shown that marking inks covering substantial portions of the contact faces can cause a reduction of the slip factor. It is recommended that marking inks be used no more than the absolute minimum necessary for marking out hole positions, and that any notes or other marks incidental to the fabrication or erection be made on an area adjacent to, and not on, the contact surfaces.

For any surface condition other than the clean as-rolled surface with tight mill scale, the slip factor adopted should be justified by test. Varying the slip factor permits the use of galvanizing, paint or other finishes, where warranted by the appropriate service conditions. Appendix K provides a satisfactory method of test for determining slip factors.

Apart from the general provisions of this clause, it should be noted that there is no restriction on the use of applied finishes on the contact surfaces of bearing-type connections.

C14.3.7 Pinned connection

In normal usage, this clause would be satisfied if the diameter of the pin hole is not more than 1 mm greater than the diameter of the pin. This clearance provides a good distribution of forces in connection plies and allows a reasonable tolerance to facilitate erection. Normal usage is intended to cover essentially statically loaded structures such as suspended, wide span roof structures where pins are now commonly used.

In normal fabrication practice, holes for pins will be drilled or bored. The finish so achieved will satisfy the intentions of this clause.

Material for a pin will usually be bright rolled bar stock of a standard size, or the pin will be machined to size from such stock. These surfaces will also satisfy the intentions of the clause. Under no circumstances should hot-rolled as-manufactured bar be used in pin connections.

Discussions on pinned connections may be found in (14.11, 14.12).

C14.4 TOLERANCES

C14.4.1 General

The tolerances specified are considered to be reasonable from the point of view of their effect on member capacity, and are considered to be efficiently and economically attainable by the fabrication industry. Tighter tolerances are generally only achievable at increased cost. The tolerance provisions are reviewed in (14.13) and compared to those in comparable overseas standards. They are also the same as those specified in (14.15).

The tolerances specified are applicable to all members, whether of hot-rolled steel sections or fabricated from plates.

Users should note that some of the cross section tolerances specified in this Standard are more lenient than the tolerance envelope which proprietary steel sections are manufactured to. Sections which comply with the tolerances herein may be outside the tolerance envelope required by some automated fabrication equipment. Also, some plant and equipment applications may require more stringent tolerances than those given herein. Advice can be obtained from HERA.

It should be noted that tolerances as close as those specified in this clause are not necessarily required for all structures. The tolerances are, however, consistent with the member design clauses, and should only be varied with the approval of the design engineer. The design engineer may decide to allow wider or require tighter tolerances in particular cases, and may specify accordingly in the contract documents. There may also be circumstances where deviation from the specified tolerance has occurred during fabrication, and where the design engineer may elect to accept the member provided that the structure is not adversely affected. When any deviation is permitted, an assessment should be made of the effect of the deviation on the member design capacity.

C14.4.2 Notation

See Commentary Clause C1.4.

C14.4.3 Cross section

The tolerances on specified cross section dimensions for built-up sections given in 14.4.3.2 are based on the following:

(a) AS 3679:Parts 1 and 2 requirements for rolled and welded sections;

(b) American Institute of Steel Construction 'Quality Criteria and Inspection Standards';

(c) American Welding Society, 'Structural Welding Standard – Steel' AWS D1.1.

The tolerance on plate thickness should be as specified in AS 1365 – Tolerances for flat-rolled steel products.

The tolerances in this clause are self-explanatory, and are readily measured on site. They are also the same as those specified in (14.15).

The tolerances for members supporting light-weight roofs, added in the 1997 revision, are adapted from the MBMA Low Rise Buildings Systems Manual (14.17), which specifies typical USA practice for such roofs.

C14.4.4 Compression member

C14.4.4.1 Straightness

The straightness provision is consistent with that required in AS/NZS 3679:Parts 1 or 2 for the manufacture of I–sections. The provision applies to both principal axes of a member (i.e. to camber and sweep – see discussion in Commentary Clause C14.4.5).

It is intended that a straightness check during fabrication be carried out to ensure that any out-of-straightness does not affect the ability of the erector to erect the structure.

C14.4.4.2 Full contact splice

This clause calls for the column to be plumbed to the requirements specified in 15.3.3 and then the gap measured. The restrictions on the gap are summarized in figure C14.4.4.2. These restrictions are intended to convey the reality that at splices, a perfect full-and-even contact fit cannot be achieved.

The 1 mm maximum gap restriction has been specified in the 1989 edition of this Standard and is consistent with a number of overseas standards. The 0.5 mm maximum gap over 67 % of the bearing area has been introduced in order to ensure that contact is relatively widely spread, and that no excessive settling of the column occurs during erection, which may alter the column alignment outside the tolerance limits of 15.3.3.

Experience has indicated that non-detrimental local yielding in a full contact splice compensates for imperfect abutting surfaces.

Where gap restrictions are exceeded, shimming may be used to correct the problem at the construction reviewer's discretion in conjunction with agreement from the design engineer.

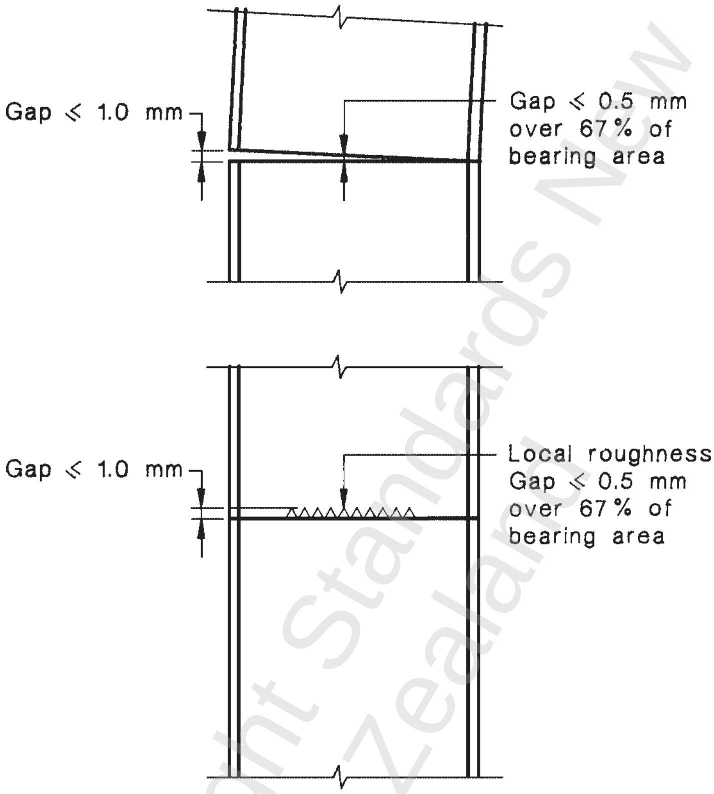

Figure C14.4.4.2 – Full contact splices

Full contact splices may be readily achieved using cold-saw cutting techniques, and end milling would only be resorted to in special cases (see Commentary Clause C14.3.2).

Tests (14.4) on spliced full-size columns with joints that had been intentionally milled out-of-square, relative to either the strong or weak axis, have demonstrated that their loading-carrying capacities were the same as those for similar unspliced columns. In the tests, gaps of 1.5 mm were not shimmed and gaps of 6 mm were shimmed with non-tapered mild steel shims. Minimum size incomplete penetration butt welds were used in all tests. No tests were performed on specimens with gaps greater than 6 mm. Accordingly, it seems reasonable to permit shimming on gaps up to 6 mm, with gaps larger than this being corrected by re-fabrication. This upper limit is restricted to 3 mm for seismic applications by 12.9.6.1 (b).

The criteria for fit of compression member connections are equally applicable to connections at column splices and connections between columns and base plates.

C14.4.4.3 Length
The length tolerance is consistent with the normal clearance in holes (2 mm or 3 mm, see 14.3.5.2.1), and with the normal tolerance on a welded butt joint in a compression member.

C14.4.5 Beam

The tolerances given in table 14.4.5 are compatible with those of AS/NZS 3679.1.

The measurement of camber is made with the web of the section to be tested horizontal. Camber is measured as illustrated in figure 14.4.5.1(a).

The measurement of sweep is made with the web of the section to be tested vertical. Sweep is measured as illustrated in figure 14.4.5.1(b). Although this method works satisfactorily for the majority of members, where sweep tends to be minimal, it may give unreliable answers on members with excessive sweep.

It should be noted that 'out-of straightness' about the major axis, within the limits of the material supply code, can be used to advantage in providing natural camber in rolled beams at no cost.

Amd 1 June '01

Experience has shown that the tolerances for camber given in table 14.4.5 are not always sufficiently stringent for I-section floor beams for which a specified precamber is given. An additional item has been added to rectify this.

C14.5 INSPECTION

The inspection of fabricated items should be appropriate to the importance of the member and the structure. Inspection of the following may be appropriate:

(a) Grades of material;

(b) Cut surfaces;

(c) Holes;

(d) Contact splices;

(e) Conditions of contact surfaces;

(f) Member sizes;

(g) Welding;

(h) Bolt classes and sizes;

(i) Tolerances;

(j) Corrosion protection.

The construction reviewer, or an appropriately qualified inspector representing him/her, should have access at all reasonable times to all places where the work is being carried out, and should be provided by the fabricator with all the necessary facilities, excluding specialized inspection equipment, for inspection while the work is in progress, unless specified otherwise in the contract documents. Inspection of welding should be in conformity with AS/NZS 1554.1 and Appendix D of this Standard, or as nominated in the contract documents.

Unless otherwise agreed prior to the inspection, the inspection should be carried out at the place of fabrication. The inspector should schedule his/her inspections by agreement with the fabricator.

Inspections are often best carried out by specialist inspection services under the direction of the construction reviewer.

C14.6 FABRICATION MATTERS AND CONTRACT DOCUMENTS

The following matters should be considered when drawing up contract documents relating to the fabrication of steelwork (refer also to (14.15)):

(a) Any additional requirements over and above those of section 14;

(b) Methods of correcting faults (see 14.3.1);

(c) Different tolerances to those specified in 14.4;

(d) Whether a certificate of compliance with the nominated material Standard is required and whether the actual yield stress is required (see 14.2.1.2 for the latter);

(e) Positions and extent of any ultrasonic examinations for any laminar imperfections;

(f) The roughness permitted on cut surfaces (see 14.3.3);

(g) Whether punching of holes is permitted in various thicknesses of steel used in a project (see 14.3.5.1);

(h) Permitted bolt hole sizes (see 14.3.5.2);

(i) When washers are to be provided (see 14.3.6.1);

(j) Identification of bolting categories, especially bearing-type or friction-type tensioned bolts (see 9.3.3.2 and 14.3.6.3);

(k) Whether an applied finish is permitted on contact surfaces of friction-type connections (see 14.3.6.3);

(l) Details of any limitations on bending;

(m) Inspection requirements – bolting and welding;

(n) Details of corrosion protection;

(o) Details of any stress relief treatment required.

Amd 1 June '01

The HERA Specification (14.15) contains a more comprehensive check list for fabrication and erection, and AS/NZS 1554.1 contains a check list of contractual matters related exclusively to welding. The HERA Specification (14.15) also references articles covering the frequency of inspection for welded connections, bolted connections and welded shear studs.

REFERENCES TO SECTION C14

14.1 Welding Technology Institute of Australia. 1985. Control of Lamellar Tearing, Rev. Edition. WTIA Technical Note 6. Published by the WTIA.

14.2 Welding Technology Institute of Australia. 1994. Flame Cutting of Steels, Rev. Edition. WTIA Technical Note 5. Published by the WTIA.

14.3 Galvanising Association of New Zealand. 1995. Hot Dip Galvanising, 14th Edition. Published by the Galvanising Association of New Zealand.

14.4 WTIA-AISC-AWI. 1992. Commentary on the Structural Steel Welding Code, Rev. Edition. WTIA Technical Note 11. Published by the WTIA.

14.5 Welding Technology Institute of Australia. 1994. The Weldability of Steels, Rev. Edition. WTIA Technical Note 1. Published by the WTIA.

14.6 Iwankin, N. and Schlafly, T. 1982. Effect of Hole-Marking on the Strength of Double Lap Joints. Engineering Journal, American Institute of Steel Construction, Third Quarter, pp. 170-178.

14.7 Brolund, T.F. 1974. Theory of Punching. Steel Fabrication Journal, Australian Institute of Steel Construction, No. 10, pp. 2-6.

14.8 Owens, G.W., Driver, P.G. and Kriye, G.J. 1981. Punched Holes in Structural Steelwork. Journal of Constructional Steel Research, Vol. 1, No. 3, pp. 34.47.

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14.10 American Institute of Steel Construction. 1985. Allowable Stress Design. Specification for Structural Joints Using ASTM A325 or A490 Bolts. Published by the American Institute of Steel Construction. Chicago.

14.11 Riviezzi, G. 1985. Pin Connections for Static Structures. Steel Construction, Australian Institute of Steel Construction, Vol. 19, No. 2, pp. 2-7.

14.12 Duerr, D. and Pincus, G. 1986. Pin Clearance Effect on Pinned Connection Strength. Journal of Structural Engineering, ASCE, Vol. 112, No. 7, pp. 1731-1736.

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14.14 Popov, E.V. and Stephen R.M. 1977. Capacity of Columns with Splice Imperfections. Engineering Journal, American Institute of Steel Construction, Vol. 14, 1st Quarter, pp. 16-23.

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14.15 HERA. 1998. HERA Specification for the Fabrication, Erection and Surface Treatment of Structural Steelwork. HERA Report R4-99. Published by HERA, Manukau City.

14.16 Riviezzi, G. 1984. Curving Structural Steel Members. AISC Journal of Steel Construction, Vol. 18, No. 3.

14.17 MBMA. Low Rise Buildings Systems Manual; 1986 Edition. Published by the Metal Building Manufacturers Association, Inc. Ohio, USA.

C15 ERECTION

C15.1 GENERAL

C15.1.1 Rejection of an erected item

Section 15 provides the minimum requirements for the standard of workmanship which should be achieved by the erector in order to ensure that the design assumptions fundamental to sections 5 to 13 are attained.

This clause specifies under what conditions a member may be 'liable to rejection'. Rejection for failure to comply with 15.1, 15.2 or 15.3 is not automatic, as it is recognized that there may be sufficient reserve design capacity in the erected item for the item to be accepted nonetheless, or that more damage might be done by trying to rectify the non-compliance.

For a non-complying item, the options are given of either carrying out a revised assessment of the design capacity of the item in its 'as-erected' state, or subjecting the item to a proof test carried out in accordance with 17.4.

The design provisions of sections 5 to 9 are consistent with the provisions of section 15, particularly with the tolerance limits of 15.3. These tolerances have been considered when evaluating suitable expressions for the member or connection nominal capacities. Particularly important in this regard are the plumbing provisions for a compression member (see 15.3.3) and the alignment provision for a beam (see 15.3.5).

The provisions of section 15 are also compatible with the HERA General Specification for the Fabrication and Erection of Structural Steel (15.7). This Specification (15.7) is directly applicable, therefore, to designs undertaken in accordance with this Standard.

C15.1.2 Safety during erection

The requirement for safety at all stages of erection should be understood by all persons dealing with the erection work. Any procedure specified should be strictly followed. Where temporary supports are specified, they should be of adequate design and construction and should only be used in the way intended. Improvised supports should not be employed.

Temporary bracing intended to provide stability during erection should be clearly shown in the contract drawings.

It is important that there should be a coordinated plan for all matters affecting erection and in, particular, account should be taken of any limitations that may exist at the site with respect to access, storage and the size and weight of components. Careful consideration should be given to planning the sequence of erection.

Guidance on safety during erection for steel buildings may be found in SAA/SNZ HB62 (15.1). This is a joint New Zealand/Australian Code of Practice, presented in 2 parts. Part 1 covers low-rise buildings and structures, Part 2 covers multi-storey buildings and structures.

Design engineers need to be aware of the responsibilities placed on them by Clause 2.4 of each part of SAA/SNZ HB62 and need to make appropriate provisions in each instance to either accept and discharge these responsibilities or to transfer them to more appropriate parties to the contract.

C15.1.3 Equipment support
No increase in the nominal capacity and no increase in the strength reduction factor is permitted under erection loading.

C15.1.4 Reference temperature
Due account should be taken of the effects of temperature on the structure and measuring instruments when measurements are made for setting-out and erection, and for dimensional checks carried out subsequently. Dimensions are required to be set out using a reference temperature of 20 ^{o}C, but other temperatures may be specified in the contract documents if the situation demands it.

Where this arises (for example in colder regions of New Zealand in other than summer the temperature may not reach 20 ^{o}C), clear instructions for temperature compensation are needed in the contract documents, including:

(a) The set-out and site checking of support locations;

(b) The shop fabrication of members at different temperatures to erection temperatures; and

(c) The site checking of the erected structure.

C15.2 ERECTION PROCEDURES

C15.2.1 General
See Commentary Clauses C14.3.2 to C14.3.6.

C15.2.3 Assembly and alignment
This clause requires that in a bolted connection, at least one steel washer is used for each bolt, and that it is placed under the rotated component. Depending on the types of holes used and the way in which the connection is assembled, the washer either may be of the type required when using oversize or slotted holes, or may be a normal washer. A washer must be placed under the rotating component to prevent the galling which would occur if either the bolt head or nut was turned on the softer structural steel. Hardened washers are supplied with bolts to AS/NZS 1252.

This clause also requires the use of tapered washers when surfaces are out of parallel by a slope of 1:20 or more. This is to ensure that the rotated component is tightened against a surface normal to the axis of the bolt. It is recommended that tapered washers be placed, if possible, under the non-rotating component, to avoid the possibility of the washer twisting during tensioning. This means that with a tapered section, it is desirable to use 2 washers, a tapered washer and a flat washer, with the flat washer placed under the rotating component.

C15.2.4 Assembly of a connection involving tensioned bolts

C15.2.4.1 Placement of a nut
In many structural applications, the nut is placed on a bolt by ‘feel’, and often is not visible to the operator. This means that it is not practicable to guarantee that identifying marks (on nuts marked on one face only) will always be visible after bolt installation, and so the clause reflects the ideal situation. In any case, nuts manufactured to AS/NZS 1252 can also be readily identified by their physical size and markings.

C15.2.4.2 Packing
Since a reduction in slip factor can result from the assembly of 2 faces having different surface conditions, this clause requires that packing be of steel with the same surface condition as the contact faces.

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Amd 1
June '01

C15.2.4.3 Tightening pattern

For both methods of tightening permitted by this clause, the observance of the correct tensioning sequence is important. Bolts and nuts should always be tightened in a staggered pattern, and, where there are more than 4 bolts in any one connection, they should be tightened from the centre or from the stiffest part of the connection onwards. In the permitted methods, this applies both to the initial snug-tightening and to the final tightening.

Where direct-tension indication devices are used, it is first necessary to ensure that the components are in effective contact at the snug-tight stage and secondly that the recommended tightening pattern is used. Proper tightening may be indicated by the breaking of a tail of the fastener or the closing of a gap on a washer. If the procedure specified in this clause is not followed, the tightening of subsequent fasteners may result in a loss of tension in those initially tightened, and this will not be revealed by the direct-tension indication device.

C15.2.4.4 Retensioning

While this clause recognizes the need to slacken and retension bolts in special circumstances, a warning is given because bolts may be tensioned beyond their proof load, and some plastic deformation may occur.

For either of the tightening methods permitted by 15.2.5, a bolt may be retensioned once only in the same hole without using the whole of its plastic deformation capacity, and without the associated danger of bolt breakage on re-tensioning.

Where special direct-tension indication fasteners are used, the fastener cannot be reused. Where load indicating washers are used, these must be replaced before re-tensioning a previously tensioned bolt.

Galvanized bolts cannot be satisfactorily retensioned due to the relatively soft zinc layer on the threads of the bolt and nut.

C15.2.5 Methods of tensioning

C15.2.5.1 General

It is accepted that the bolts will often be tensioned beyond their proof loads, as the minimum bolt tension specified in table 15.2.5.1 is approximately equal to the minimum proof load of the bolts.

If M30 or M36 bolts are specified, tensioning may be difficult depending on the capacity of the available equipment on site.

Hot-dip galvanized and zinc electroplated bolt-nut assemblies show a more variable torque-tension relationship than plain bolts, as the friction between the nut thread and the coated bolt is increased. AS/NZS 1252 therefore specifies that the nuts of hot-dip galvanized and zinc electroplated bolts be provided with supplementary lubrication.

The torque-control method of tensioning is not generally permitted. The reason for this is that experience since the introduction of high-strength bolting has shown that this method of achieving bolts tension is extremely unreliable in general structural applications. Torque-control tensioning has its origin in the mechanical engineering industry where bolts of higher quality surface finish are used. In addition, in these situations, bolts are normally stored under protected conditions and not exposed to weather. In these applications, therefore, the relationship between torque and tension is fairly constant and easily measured.

In the structural industry generally these conditions are rarely present. The bolts used very often are exposed to weather and general site contamination before being installed in structural connections. This leads to an extremely variable relationship between torque and induced shank tension due to the variable friction between the nut and the bolt threads, and the nut and the washer faces. Also, experience shows that torque wrenches of suitable capacity are not readily available on many sites, and load cells for wrench calibration are often not available when required nor for the necessary time for calibration to be carried out once per shift.

Torque control as a method is restricted in this Standard as an inspection method for the detection of gross undertensioning (see Appendix L) or for the tensioning of specialist bolt and nut assemblies as advised below.

Amd 1 June '01

Specialist class 8.8 structural bolt and nut assemblies exist which employ a break-off tail which is broken at a pre-determined torque. They are therefore being tightened by the torque-control method. The use of such bolts that are manufactured expressly for use in structural steel construction is permitted for 8.8/*TB* and 8.8/*S* applications, if approved by the design engineer and adequately supervised by the construction reviewer. Their use is not recommended for 8.8/TF applications. Such bolt and nut assemblies are supplied with detailed instructions for delivery, site storage and installation (15.9) and these instructions **must be strictly followed**.

A general review of bolt tensioning procedures may be found in (15.4).

C15.2.5.2 Part-turn method of tensioning

The objective is to draw the load-transmitting plies into effective contact and, to achieve this, all bolts in the joint should be brought to the snug-tight condition first. When snug-tightening by hand, the full effort of a person on a standard podger spanner is expected. Podger spanners have handles ranging from 400 mm to 800 mm in length, depending on the size of the bolt head. Where a pneumatic impact wrench is used for snug-tightening, the achievement of close contact between the plies is normally detectable as a distinct change in note as the wrench ceases to rotate freely and starts impacting.

With large connections, 2 runs over the bolts to check the snug-tight condition is suggested, as the load-transmitting plies will be drawn in gradually, tending to loosen those bolts which were snug-tightened first.

In the final tensioning, the non-rotating part should be held by a hand spanner to prevent it from turning.

The use of marked wrench sockets is a desirable visual aid for the operator to control unit rotation, whether or not the inspection procedure calls for permanent location marks. Where permanent location marks are required, they should remain visible until inspection is completed.

Part-turn tensioning may occasionally induce too high a bolt tension in very short bolts used in thin grips. The occurrence of this condition will be indicated by an abnormal number of bolt breakages during tensioning. If such a condition arises, it may be necessary to establish a reduced nut rotation from snug-tight by carrying out nut rotation-bolt tension tests.

The nut rotation values given in table 15.2.5.2 are based on AISC(US) values in (15.6), and reflect reduced rotation requirements in thin grips.

C15.2.5.3 Tensioning by use of direct-tension indication device

In making provision for this method of control of tensioning, note was taken of the marketing of devices for providing direct indications of bolt tension. It was further noted that the capability of such devices for indicating the achievement of minimum bolt tension could be checked by carrying out tensioning of sample bolts and nuts against a load cell or similar apparatus.

Design engineers and construction reviewers should satisfy themselves that the direct-tension indicators do actually indicate the correct bolt tensions, preferably by carrying out, or having available the results of, tests of the device in a load cell. American practice is to require that such devices indicate a tension not less than 105 % of the minimum bolt tension required by table 15.2.5.1. (Commentary to (15.6).)

It is important to note that the use of direct-tension indication devices still requires the observance of the two-stage procedure, namely initial snug-tightening to bring the plies into effective contact, followed by full tensioning. Observance of this procedure is imperative to ensure that the tensioning of subsequent bolts does not result in a loss of tension in those bolts tensioned previously. It should also be noted that incorporation of a tension indication device in the bolt-nut-washer assembly may require some slight addition to the bolt length allowances.

C15.3 TOLERANCES

The tolerances specified are considered to be reasonable from the point of view of their effects on member capacity, and are considered to be efficiently and economically attainable by steelwork erectors. Tighter tolerances are only achievable at increased cost. The tolerance provisions are reviewed in (15.2) and compared to those in comparable overseas standards. They are fully comparable with those specified in (15.7).

It should be noted that tolerances as close as those specified in this clause are not necessarily required for all structures. The tolerances are, however, consistent with the member design clauses, and should only be varied in special cases. The design engineer may decide to allow wider or require tighter tolerances in particular cases, and may specify accordingly in the contract documents. There may also be circumstances where deviation from the specified tolerance has occurred during erection, and where the design engineer may elect to accept the member provided that the structure is not adversely affected. When any deviation is permitted, an assessment should be made of the effect of the deviation on the member design capacity.

C15.3.1 Location of anchor bolts

One of the greatest problems faced by a steel erector on site is inaccuracy in the locations of the anchor bolts.

The tolerances quoted come from American and Canadian practice, and can be readily attained using reasonable care. It is intended that these be called-up in foundation drawings. These tolerances should ensure satisfactory site erection without site rectification measures being required, when using the 6 mm oversize hole permitted in column base plates in 14.3.5.2.2 with which these tolerances are compatible.

Several additional steps can be used to improve the site erector's position ((15.3), sections 14, 15, 18 of (15.8)):

(a) Caging of anchor bolt groups;

(b) Use of cored holes to allow adjustment in anchor bolt positions; and

(c) Checking of anchor bolt positions by a surveyor before placing concrete.

C15.3.2 Column base

C15.3.2.1 Position in plan

The 6 mm tolerance is intended to provide for accurate positioning of the base and, in conjunction with plumbing tolerances, to allow reasonable limits which still allow the steel frame geometry to be held. These tolerances should assist in the subsequent fitting of other building elements such as pre-cast or curtain wall facade panels to steel building frames.

C15.3.2.2 Level

The tolerance of ±10 mm is consistent with the tolerance on beam position given in 15.3.5.

C15.3.2.3 Full contact

Load bearing steel packs under column base plates should be placed so that they are always within the base plate plan dimensions and are enclosed by a minimum of 50 mm of grout. This provision guards against the possibility of a pack being close to the edge of the concrete pedestal, resulting in high local compressive stresses which may break off the concrete edge.

C15.3.3 Plumbing of a compression member

The tolerances in this clause are this same as those of the 1989 edition of this Standard, except that an overriding tolerance of storey height/500 has been included as a storey check. This is necessary to facilitate assembly during frame erection.

This clause is now in line with American practice. The angular misalignment of 1/500 should also strictly apply to lengths between splices.

Figure C15.3.3 explains diagrammatically the provisions of this clause.

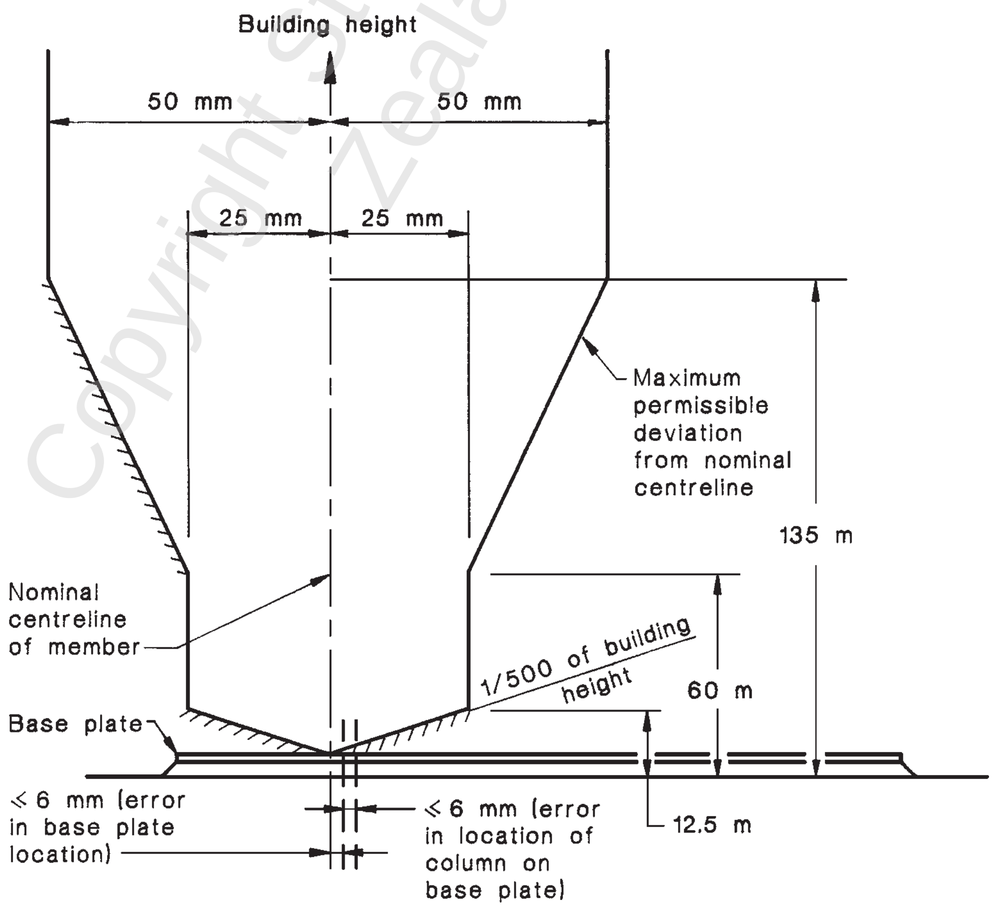

Figure C15.3.3 – Plumbing of a compression member

C15.3.4 Column splice

The ±10 mm level tolerance is consistent with the beam level tolerance of 15.3.5. The position of the splice in plan must lie within the tolerance envelope for plumbing (see figure C15.3.3). The tolerance on relative position of spliced members is the general ±2 mm tolerance, and guards against out of alignment of a splice (see figure C15.3.4).

Amd 2 Oct. '07

The new sub-clause (d) is based on the 2005 AISC specification and allows relaxation of overly conservative requirements which impact on erection practice.

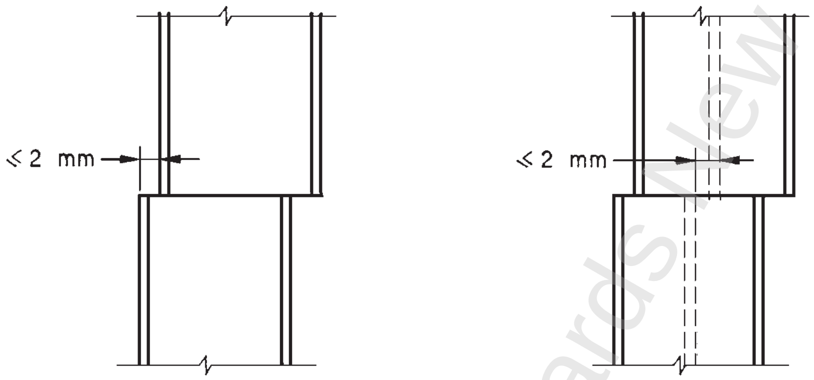

Figure C15.3.4 – Column splice

C15.3.5 Level and alignment of a beam

The tolerance of ±10 mm on beam level is the recommended ISO tolerance. The origin of this dimension is not clear, but the 10 mm is more liberal than in 1989 edition of this Standard, where the level tolerances were divided into ±7 mm for beams with depths less than or equal to 2000 mm, and ±10 mm for greater than 2000 mm. In any case, a beam will normally be held to level more accurately at its connection by virtue of the normal fabricating tolerances on the position of holes and cleats.

The tolerance on web position is the same as in the 1989 edition of this Standard, and is in line with ISO recommendations.

The tolerance on sweep of $L_b/500$ is greater than the sweep allowed in 14.4.5 during fabrication ($L/1000$). This extra tolerance allows for the pulling in of a beam at intermediate connections during erection, and is in line with 1989 edition of this Standard. The misalignment of 1/500 should also, strictly speaking, apply to lengths between any splices in the beam.

C15.3.7 Overall building dimensions

Although the 1989 edition of this Standard did not have a specific provision for tolerance on the overall building dimensions, it was considered that its introduction was desirable. This clause nominates limits on the combined effects of the tolerances on the fabrication, erection and construction of all steel elements incorporated in a building.

This clause is based on a similar clause in EuroCode 3.

C15.4 INSPECTION OF BOLTED CONNECTIONS

To ensure that all bolts are fully tensioned, the bolts that were tensioned first should be checked by the erector, as subsequent tensioning of other bolts may loosen them and this check will save considerable time during final inspection.

The number of bolts to be checked in each connection depends on the construction reviewer or his representative. A suitable sample would consist of 10 % of the bolts, but not less than 2 bolts at each connection selected at random.

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Guidance on the scope and frequency of inspection of bolts in bolted connections is presented in the HERA *Steel Design and Construction Bulletin*, Issue No. 46, 1998, pp. 8-10.

For part-turn tensioning, the construction reviewer or his representative should be satisfied that the specified procedure is being followed, and that either match marking, if specified, or marked wrench sockets are being used.

As a general inspection guide, the sides of bolt heads and nuts tensioned with an impact wrench will appear slightly peened, thus indicating that the wrench has been applied to the fastener.

C15.5 GROUTING AT SUPPORTS

The provisions represent good practice, and reference is made to NZS 3108 for details of the grout. Grout strength would typically vary from 10 to 25 MPa, and should be stated on the design drawings.

C15.6 INSPECTION

C15.6.1 General

Provision should be made for the construction reviewer or his representative to have access at all reasonable times to all places where work is being carried out. Facilities for inspection and testing of the work should be provided in accordance with the contract documents.

C15.6.2 Inspection of welded connections

Inspection of welding should be carried out in accordance with AS/NZS 1554.1 and Appendix D of this Standard. A commentary on the provisions of AS 1554.1 (the predecessor of AS/NZS 1554.1) may be found in (15.5); refer to Appendix CD herein to the commentary on Appendix D.

C15.7 ERECTION MATTERS AND CONTRACT DOCUMENTS

The following matters should be considered when drawing up contract documents relating to the erection of steelwork:

(a) Any additional requirements over and above those of section 15;

(b) Methods of correcting faults;

(c) Different tolerances to those specified in 15.3;

(d) Methods of inspecting bolting procedures and site welding;

(e) Methods of carrying out corrosion protection on site;

(f) Details of grouting.

REFERENCES TO SECTION C15

15.1 Standards Australia/Standards New Zealand. 1995. Code of Practice for Safe Erection of Building Steelwork. SAA/SNZ Handbook HB 62. Published by Standards Australia.

15.2 Hogan, T.J. and Firkins, A. 1985. Fabrication and Erection Provisions of AS 1250. Proceedings, Third Conference on Steel Developments, pp. 155-161.

15.3 AISC. 1991. Economical Structural Steelwork, 3rd Edition. Published by the Australian Institute of Steel Construction. Sydney.

15.4 Firkins, A. and Hogan, T.J. 1990. Bolting of Steel Structures, 3rd Edition. Published by the Australian Institute of Steel Construction. Sydney.

15.5 WTIA-AISC. 1992. Commentary on the Structural Steel Welding Code AS 1554. WTIA Technical Note 11. Published by the WTIA.

15.6 AISC. 1988. Load and Resistance Factor Design. Specification for Structural Joints Using ASTM A325 or A490 Bolts. Published by the American Institute of Steel Construction.

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15.7 HERA. 1998. HERA Specification for the Fabrication, Erection and Surface Treatment of Structural Steelwork. HERA Report R4-99. Published by HERA, Manukau City.

15.8 HERA. 1990. New Zealand Structural Steelwork Design Guides, Volume 2, Incorporating Amendment No. 3. HERA Report R4-49. Published by HERA. Manukau City.

15.9 HERA. 1990. Instructions for Site Storage, Handling and Installation of Torque Control Bolts (included as section 10.3.1.2 of HERA Report R4-80 or available separately from HERA).

C16 MODIFICATION OF EXISTING STRUCTURES

C16.1 GENERAL

Most of the material in section 16 and this Commentary is taken from (16.1) and its Commentary. The section contains only additional provisions to those of the remainder of this Standard which require consideration when carrying out modifications, such as repair or strengthening. Guidance on establishing the specification requirements for repair or strengthening is given in (16.2).

Repair and strengthening of existing structures differs from new construction, since both operations have to be executed with the structure or the structural element under load. There is presently little guidance with respect to the welding of structural members under load. Hence, each given situation should be evaluated on its own merits, and sound engineering judgement should be exercised as to the optimum manner in which repair or strengthening should be carried out.

Before completing the design, the following should be determined:

(a) The character and extent of damage to the parts and connections that require repair or strengthening; and

(b) Whether the repairs should consist only of restoring corroded or otherwise damaged parts, or of replacing members in their entirety.

A complete study of the design axial forces and bending moments in the structure should be made if the strengthening goes beyond the restoration of corroded or otherwise damaged members.

Allowance should be made for fatigue loading that members may have sustained in past service. Generally, in the case of high cycle-low stress dynamically loaded structures, sufficient data regarding past service is not available for estimating the remaining fatigue life. If such is the case, an inspection program designed to locate possible fatigue cracks in stable growth prior to their becoming critical is a reasonable alternative. The only practical method of extending the expected fatigue life of a member is to reduce the stress range, or to provide connection geometry less susceptible to fatigue failure.

Structural elements under load should not be removed or reduced in section except as specified by the design engineer. In (16.1), it is recommended that where rivets or bolts have insufficient design capacity to support the total load only the dead load should be assigned to them provided they have sufficient design capacity to support it. In such cases, sufficient welding should be provided to support all live and impact loads. If rivets or bolts have insufficient design capacity to support dead load alone, then sufficient welding should be added to support the total load.

C16.2 MATERIALS

The essential requirement in strengthening and repairing existing structures is the identification of the material.

When welding is anticipated for either operation, the weldability of the existing steel is of primary importance. Together with the mechanical properties of the material, it will provide information essential for the establishment of safe and sound welding procedures.

Mechanical properties are normally determined by tensile tests to ISO 2566.1 on a representative sample taken from the existing structure, or may be estimated using hardness testing.

If the chemical composition has to be established by test, then it will be advisable to take samples from the greater thickness, as these are more indicative of the extremes in chemistry.

It is important to recognize that in older structures, some or all members may be made from either:

(a) Cast iron; or

(b) Wrought iron; or

(c) Wrought steel; or

(d) Steels of special chemical composition

Such members may not be readily weldable, and may have been intended to be connected by riveting. The only way to positively identify them is by taking samples for microstructure examination by a metallurgist.

Many structural sections have mill markings on them which assist in identification, and it is often necessary to search through old steel section handbooks to identify both steel type and section properties. Old handbooks usually give 'allowable stress' values, from which it is possible to infer a yield stress.

C16.3 CLEANING

The provisions are a summary of good practice.

C16.4 SPECIAL PROVISIONS

C16.4.1 Welding and cutting

If material is added to a member carrying a stress from dead load in excess of 20 MPa, it is desirable (16.1) to relieve the member of dead load or to preload the material to be added. The design engineer should determine if propping to remove the dead load is necessary.

The extent of cross-sectional heating must be considered by the design engineer when determining whether live loads may be carried by the member during welding or oxygen cutting. The guidance of a Certified Welding Engineer may be sought in this regard.

The significance of this provision lies in the fact that the properties of steel are influenced by heat. It is the consensus of other reputable specifications that temperatures up to 345 °C have little or no reducing effect on the yield strength of the steel (see also figure 11.4).

Under such circumstances, the welding procedures should be adjusted in such a fashion that the total heat input per unit length of the weld for a given thickness and geometry of the material will keep the 345 °C isotherms relatively narrow and minor in relation to the cross section of the load-carrying member. The New Zealand Welding Centre can advise on suitable welding procedures for this application.

C16.4.2 Welding sequence

This is of particular importance if live load is permitted on the structure while the member under consideration is being strengthened or repaired.

Particular care should be given to the sequence of welding in the application of reinforcing plates on girder webs, and to the treatment of welds in the end connections of such plates where they abut stiffener assemblies or girder splice plates.

In strengthening members by the addition of material, it is desirable to arrange the sequence of welding so as to maintain a symmetrical section at all times.

REFERENCES TO SECTION C16

16.1 American Welding Society. 1996. Structural Welding Code – Steel. AWS D1.1-96.

Amd 1 June '01 16.2 HERA. 1998. HERA Specification for the Fabrication, Erection and Surface Treatment of Structural Steelwork. HERA Report R4-99. Published by HERA, Manukau City.

C17 TESTING OF STRUCTURES OR ELEMENTS

C17.1 GENERAL

C17.1.2 Circumstances requiring tests

Load tests are undertaken for several purposes, and it is important that in any particular load test the exact purpose of the test is clear.

The circumstances requiring tests referred to in 17.1.2 include:

(a) Where a structure or part of a structure is not designed by calculation;

(b) Where materials or design methods used are other then those of this Standard; or

(c) Where there is doubt or disagreement as to whether the structure or some part of it complies with provisions of this Standard.

The purpose of most load tests can be considered to lie in one of the following three broad classifications:

(i) To obtain the acceptance of a structure for a specific purpose;

(ii) To obtain information to assist the assessment of a structure; or

(iii) To provide a method of quality control in the construction of structures.

In a load test specification, it is important to define the structural states which are being assessed. In general, these will lie in one of the 2 following broad classifications:

(a) *Ultimate limit states.* These are limit states in which a structure is rendered unfit for further use. Typically, ultimate limit states follow the attainment of maximum load capacity. Usually it is desirable that there is only a small risk that a structure will reach the ultimate limit state during its design lifetime (refer to Commentary Clause C3.1 for details).

(b) *Serviceability limit states.* These are limit states in which a structure fails to perform satisfactorily but is still fit for further use. Examples of this are excessive deflections, vibrations and cracking. Usually it is acceptable for a structure to reach its serviceability limit state a few times during its design lifetime.

C17.2 DEFINITIONS

Two types of load test are considered. One is a proof load test which must be applied to every structure of a population of structures for them to be accepted. The other is a prototype load test which need be applied only to a portion of a population of structures for all structures of that population to be accepted.

It should be noted that different structures will be necessary to carry a given load, depending on whether the design is based on proof testing, prototype testing or computation. Partly, this arises because different components of variability are involved in each case.

Design by proof testing implies that every structure of a population is tested; only the components of variability associated with the loading condition need to be considered. Design by prototype testing only estimates the uncertainties due to structural analysis, while design by calculation (in accordance with this Standard) is based on the collective experience of all available test data. Hence in general, design by proof testing will require the smallest test load factor, and design by prototype testing the largest, in order to achieve consistent levels of reliability.

Where a structure is made up of several types of components, it may be more economical to subdivide the structure into various groups of components, and to test each such group individually. This may be effected by temporarily strengthening those parts of the structure that are not under test. However, if such temporary strengthening is carried out, care should be taken to ensure the components under test receive their correct loading, and that no artificial restraint or other form of strengthening is present that would not exist in the real structure.

C17.3 TEST REQUIREMENTS

The method by which the loading should be applied to the unit to be tested, and the positions at which deflections should be measured, can only be decided with special reference to the particular structure or element and to the particular loading conditions to be investigated.

The test loading should be applied and resisted in a manner which reasonably approximates the actual service conditions. Although, in general, both proof and prototype testing are likely to involve loading in a vertical plane, additional out-of-plane loading of a structure or element to simulate other loads or effects may be required. Lateral support to the unit as a whole or to individual members of the unit should also represent, as closely as possible, the actual service conditions.

Any eccentricities not inherent in the design of the structure or element, or not resulting from typical loading in service, should be avoided at points of loading and reaction, and care should also be taken to ensure that no inadvertent restraints are present. Where it is clear that the method of test involves a significant or appreciable divergence from service conditions, either in loading or restraint, due allowance should be made.

All likely combinations of permanent loads and imposed loads of shorter duration, including those due to wind and, where applicable, those due to impact or earthquake, should be taken into account when determining the worst loading conditions. The latter should be converted in accordance with 17.4 or 17.5, as appropriate, into an equivalent test load.

A load-deflection curve should be plotted during each test on each unit. Such a curve will serve not only as a check against observational errors, but also to indicate any irregularities in the behaviour under load of the structure or element. It is desirable that a minimum of 6 points, not including the zero load point, be obtained to define the shape of the load-deflection curve if the latter is predominantly linear, and a minimum of 10 points if the curve is significantly non-linear.

C17.4 PROOF TESTING

C17.4.1 Application

Two common types of proof load tests are:

(a) Proof testing an existing structure; and

(b) Proof testing of every new structure in a class.

There are many reasons for requiring proof testing. These include a doubt that the structure has the specified design characteristics because of errors in design, in construction or because of deterioration since construction, such as can occur owing to fire, chemical attack, or material degradation. It also often happens that a structure is to be put to a new use for which it was not originally designed, but for which nevertheless it may have an adequate structural capacity. In this case, a proof test may be used to demonstrate that the structure has the necessary capacity.

C17.4.3 Criteria for acceptance

A proof test is a method to prove that a structure or an element is adequate for certain load applications. It is not a method to establish the design load capacity of a structure or an element.

For a proof test on a structure to be successful, it is necessary that the structure does not reach its ultimate limit state during the test and also that it does not incur serious permanent structural damage. Suitable methods for detecting the onset of damage vary from one material to another, and include such techniques as the measurement of crack widths and acoustic emissions. One commonly used method is the measurement of recovery of the deformation on unloading the structure. A recovery value of 85 % is recommended by (17.1).

For the serviceability limit state, it is suggested that a 95 % recovery of deformation after removal of the test load will ensure that the structural unit was substantially elastic at the test load.

C17.5 PROTOTYPE TESTING

C17.5.1 Test specimen

In the application of prototype tests, the acceptance of a complete class of structure is based on the structural performance of a sample of such structures. The sample size is often quite small and a sample comprising a single structural unit is not uncommon. In these tests, structural units are usually, but not necessarily, loaded to failure.

C17.5.2 Test load

It is to be noted that the required load factor is a function of the number of prototypes to be tested and the estimated coefficient of variation of the structural characteristics of the individual units. The values given are based on the assumption that the coefficient of variation of the capacities of steel structures and elements is about 10 %, while the coefficient of variation of the deformation characteristics is assumed to be about 5 %.

Table C17.5.2 gives some guidance for the necessary adjustment if an estimated variation is significantly different from that assumed.

C17.5.2.1 Determination of f_y, f_u by test

If a situation arises where the designer has been given specific steel to be used on a project, for which it is not feasible or possible to use the specified minimum yield stress or tensile strength from the appropriate material supply standard (see 2.1.1, 2.1.2 and 2.2.1), then an appropriate value of f_y or f_u, for use in design, can be determined using the provisions of table C17.5.2. This process involves:

Step 1: Obtain test results for f_y, f_u for n number of samples of the specific steel to be used in the project.

Step 2: Determine the coefficient of variation for the test samples. For steel supply, this may be taken as 5 % unless a value is calculated from the test data.

Step 3: Determine the design value of f_y or f_u from equations C17.5.2(1) or C17.5.2(2).

$$f_y = \frac{f_{y,\,\text{tested, minimum}}}{C_{17.5.2}} \qquad \text{Eq. C17.5.2(1)}$$

$$f_u = \frac{f_{u,\,\text{tested, minimum}}}{C_{17.5.2}} \qquad \text{Eq. C17.5.2(2)}$$

where

f_y, f_u are the values for use in design

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$f_{y,\,\text{tested, minimum}}$ or $f_{u,\,\text{tested, minimum}}$ is the minimum value from the samples tested.

$C_{17.5.2}$ is the value of multiplier from table C17.5.2 for the given coefficient of variation (5 %, unless specifically calculated) and the number of samples tested.

Step 4: Use the values of f_y, f_u from step 3 in designing to this Standard instead of the values specified in 2.1.

Table C17.5.2 – Values of multiplier for test load for estimated coefficient of variation

No of units to be tested	Coefficient of variation of structural characteristics					
	5 %	10 %	15 %	20 %	25 %	30 %
1	1.20	1.46	1.79	2.21	2.75	3.45
2	1.17	1.38	1.64	1.96	2.36	2.86
3	1.15	1.33	1.56	1.83	2.16	2.56
4	1.14	1.30	1.50	1.74	2.03	2.37
5	1.13	1.28	1.46	1.67	1.93	2.23
10	1.10	1.21	1.34	1.49	1.66	1.85
100	1.00	1.00	1.00	1.00	1.00	1.00

C17.5.3 Criteria for acceptance

No requirement for deformation recovery is set because it is felt that it might be difficult for structures with some connection types to achieve the requirement. A recovery of 95 % of the deformation after the removal of the test load is a good indication that the test unit was still substantially elastic at the test load and that no extensive yielding has occurred.

C17.6 REPORTING OF TESTS

(No Commentary.)

REFERENCE TO SECTION C17

17.1 Bares, R. and Fitzsimons, N. 1975. Load Tests of Building Structures. Journal of the Structural Division, ASCE, Vol. 101, No. ST5, pp. 1111-1123.

NOTES

COMMENTARY TO THE APPENDICES

APPENDIX CA
REFERENCED DOCUMENTS

As part of the process of harmonising New Zealand and Australian Standards where practicable, the referenced Standards listed in AS 4100:1990 have been carefully assessed as to their suitability for use with this Standard. Where practicable, they have been adopted, although many have also been updated in the 1997 revision of the Standard.

Some of the referenced Standards herein do differ from those presented in Appendix A to AS 4100:1990. This is principally due to a different Standard being endorsed for use in New Zealand to that used in Australia or to revisions in the referenced Standards since 1992. A significant number of the referenced standards have been revised over that time.

A more common source of differences relates to the material supply Standards. This Standard includes not only Australian Standards for steel supply, but also British, Eurocode and Japanese standards, reflecting the more diverse sources of steel used in this country.

There are obvious differences relating to design guides/Standards for road and rail bridge design and lift design.

APPENDIX CB
MAXIMUM LEVELS OF DUCTILITY DEMAND ON STRUCTURAL STEEL SEISMIC-RESISTING SYSTEMS

The use of structural displacement ductility factors greater than $\mu = 3$ for category 2 structural systems respectively is specified through Note (5) in accordance with the philosophy of NZS 1170.5.

It is only applicable for use herein when this Standard is used in conjunction with a Loadings Standard in which the inelastic design spectra are derived from the elastic design spectrum, in the short period range, through the use of the equal energy concept. NZS 1170.5 uses this approach.

Use of the provision has been restricted to category 2 systems only by the 1997 revision of this Standard. It is considered not appropriate to category 3 systems.

REFERENCE TO APPENDIX CB

CB.1 Standards New Zealand. 2004. Structural design actions. AS/NZS 1170 set. Wellington.

APPENDIX CC
CORROSION PROTECTION

The 1997 revision has made this Appendix normative, in order that it can be cited as an acceptable means of compliance with Clause B2: Durability of the New Zealand Building Code (see 1.1.5.1).

Amd 2 Oct. '07 Incorporates reference to important new document on coatings specification for corrosion and also addresses a common area of poor performance.

APPENDIX CD
INSPECTION OF WELDING TO AS/NZS 1554.1

The decision having been made to replace NZS 4701:1981 with AS 1554.1 and then with AS/NZS 1554.1 for use with this Standard, the NZS 3404 Committee, in conjunction with HERA and the New Zealand Welding Centre, undertook a critical review of all differences between the 2 welding Standards in order to determine what differences there were between NZS 4701:1981 and AS 1554.1 and what, if any, changes were required in design related aspects to AS 1554.1 for use in conjunction with this Standard.

The New Zealand Welding Centre has produced a detailed report (CD.1) presenting all differences with regard to welder and weld detail qualification, choice of materials, prequalification of joint details etc. This report (CD.1) is available from HERA.

With regard to this Standard, the areas of weld category selection and weld inspection were the 2 aspects of AS 1554.1 considered necessary to revise for its use in conjunction with the 1992 edition of this Standard. Many of the items covered in the 1992 edition as specific departures from AS 1554.1 have been incorporated into AS/NZS 1554.1 and therefore deleted from this 1997 edition. The one important difference still retained herein relates to the suggested extents of non-destructive examination, for which differing values are given in table D1 herein from those given in Appendix F of AS/NZS 1554.1. The values given in table D1 herein represent a relaxation on the extent of visual examination for category SP or GP welds over that given in the 1992 edition, except for category SP welds forming part of a seismic-resisting system, an associated structural system or subject to fatigue loading, where the requirements are unchanged from the 1992 edition.

The principal as defined in AS/NZS 1554.1 is effectively equivalent to the owner as defined in this Standard.

Note that, as mentioned in Commentary Clause C1.6.3, in some limited circumstances the welding supervisor may also undertake the inspection of welds in accordance with Appendix D. This will most commonly apply to jobs where only visual inspection is required and requires the approval of the design engineer (see C1.6.3.2.1(d)).

The opportunity has also been taken to provide more explicit guidance on the suggested extent of non-destructive examination then is provided by AS/NZS 1554.1 (or was provided by NZS 4701:1981). Further guidance on extent of weld inspection, covering a recommended procedure for handling non-compliance, is given in (CD.2).

Amd 1 June '01 Recommendations on applying table D1 in practice are given in the HERA *Steel Design and Construction Bulletin*, Issue No. 44, 1998, pp. 2, 3.

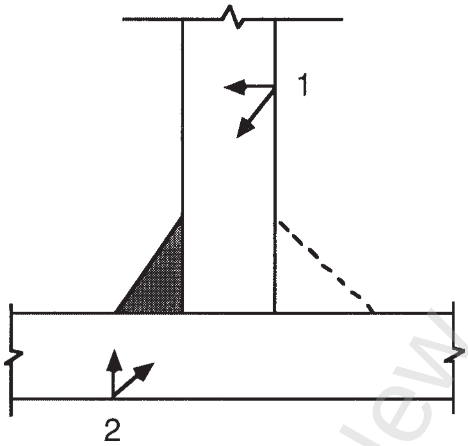

Figure CD1 – Inspection access for ultrasonic inspection of fillet welds

Note 6 to table D1 states that (category SP) fillet welds should only be tested on a routine basis by ultrasonic inspection when the inspection access is appropriate. Where ultrasonic inspection is required, the evaluation of the weldments should be made using the probe scanning positions shown in figure CD1. Scanning from position 1 may be limited when fillet welds to both sides of the joint are made.

REFERENCES TO APPENDIX CD

CD.1 Scholz, W. 1991. A Transfer From NZS 4701 to AS 1554.1; What Would This Mean to the New Zealand User? New Zealand Welding Centre. Report R8-06. Published by HERA, Manukau City.

CD.2 HERA. 1994. New Zealand Structural Steelwork Limit State Design Guides, Volume 1. HERA, Manukau City, New Zealand.

APPENDIX CE
SECOND-ORDER ELASTIC ANALYSIS

CE1 ANALYSIS

A second-order elastic analysis may be used to make a more precise analysis of the second-order effects in a sway frame than can be obtained by amplifying the results of a first-order analysis. Clause 4.4.3.1 requires a second-order analysis to be used when the amplification factor δ is greater then 1.4 for δ_b or 1.2 for δ_s, the latter of which corresponds to a frame buckling load factor (from clause 4.9) of less than 3.5.

Clause E1 permits a simple second-order analysis to be used when the frame buckling load factor λ_c (from 4.9) is greater than 5. In this simple method, the decreases in the member flexural stiffnesses caused by axial compressions are neglected, so that the member stiffnesses remain constant as the loads increase. An example of such a method is given in (CE.1, CE.2).

The simplification omits the $P-\delta$ second-order effects and accounts only for the $P-\Delta$ effects that occur in sway frames. Its application to a braced frame is therefore unnecessary, since it produces the same estimates of the member end bending moments as a first-order analysis.

When the frame buckling load factor λ_c is less than 5, then the reductions in the member flexural stiffnesses caused by the axial compressions must also be accounted for. Thus the stiffnesses of compression members decrease as the compressive loads increase, as explained in section 7 of (CE.3). When the frame buckling load factor λ_c exceeds 10, elastic second-order effects will be negligible and can be ignored, thus rendering use of this Appendix unnecessary (CE.3). Most seismic-resisting systems being designed for load combinations including earthquake loads come into this category.

CE2 DESIGN BENDING MOMENT

This paragraph defines the design bending moment M^* and gives 3 methods of determining it. In the first method, the maximum bending moment is determined directly from the second-order analysis. This may require a special routine to be included in the computer program for this purpose.

The second and third methods provide alternatives to this.

The second method requires a sufficient number of elements to be used so the greatest element end bending moment provides a close approximation to the maximum bending moment.

The third method uses the braced moment amplification factor δ_b to amplify the maximum bending moment M_m^* calculated by superposition of the simple beam bending moments with the second-order end moments M_e^*. In the third method, it is recommended to determine δ_b using $k_e = 1.0$ initially and to only refine the value of k_e below 1.0 if $\delta_b > 1.0$ from the initial assessment. This will only be the case when the member is subject to adverse transverse loading (see figure C4.4.3).

Warning note to designers:

It is important that users of elastic analysis computer programs which incorporate second-order effects determine which of E2 (a) – (c) are applicable and apply (c) if required to account for $P-\delta$ effects.

REFERENCES TO APPENDIX CE

CE.1 Wood, B.R., Beaulieu, D. and Adams, P.F. 1976. Column Design by P-Delta Method. Journal of the Structural Division, ASCE, Vol. 102, No. ST2, pp. 411-427.

CE.2 Wood, B.R., Beaulieu, D. and Adams, P.F. 1976. Further Aspects of Design by P-Delta Method. Journal of the Structural Division, ASCE, Vol. 102, No. ST3, pp. 487-500.

CE.3 HERA. 1994. New Zealand Structural Steelwork Limit State Design Guides, Volume 1. HERA Report R4-80. HERA, Manukau City.

APPENDIX CF
MOMENT AMPLIFICATION FOR A SWAY MEMBER

The simple analysis of sway compression members overestimates the amplification of the first-order moments caused by the second-order stability effects, except when the first-order deflected shape is the same as the buckled shape. This will not occur in multi-storey sway frames, as the number of changes to member size would be impractical. The alternative analysis permitted by this appendix allows more accurate estimates to be made (CF.1).

To take advantage of this, the first-order end moments M_f^* are separated into braced and sway components M_{fb}^* and M_{fs}^*. For frames where gravity loads produce negligible sway, the braced components M_{fb}^* are determined from the gravity loads, and the sway components M_{fs}^* from the transverse loads.

The sway component is then amplified using the sway moment amplification factor δ_s (4.4.3.3). The amplified sway component $\delta_s\ M_{fs}^*$ is then added to the unamplified braced component M_{fb}^* to obtain the sway amplified end moment M_e^*. This is used to compute the maximum bending moment M_m^* in the member, and this is amplified using the braced amplification factor δ_b (4.4.3.2) to obtain the final design bending moment M^*.

REFERENCE TO APPENDIX CF

CF.1 Lai, S-M. A. and MacGregor, J.G. 1983. Geometric Non-linearities in Unbraced Multi-Storey Frames. Journal of Structural Engineering, ASCE, Vol. 109, No. 11, pp. 2528-2545.

APPENDIX CG
BRACED MEMBER BUCKLING IN FRAMES

This Appendix gives a more accurate approximate method for calculating the stiffness ratio γ for a braced compression member in a frame than is given by the method of 4.8.3.4, for which the assumptions were made that the frame is rectangular with regular loading and negligible axial forces in the beams.

In this more accurate method, the stiffness ratio γ is expressed as the ratio of the stiffness of the compression member to the sum of the stiffnesses of the restraining members. The restraining member stiffness expressions include the stability function multiplier α_{sr} which accounts for the effect of the axial design actions N_r^* on the restraining member flexural stiffnesses. Linear approximations are given in figure G for this multiplier, which depend on the ratio ρ of N_r^* to the restraining member buckling load, $N_{oLr} = \pi^2 EI_r/L_r^2$.

As the member effective length factor k_e is used in the design of the compression member under consideration, the design values of the axial actions N_r^* are used in the restraining members.

However, this Appendix may also be used to determine the axial force $\lambda_c N^*$ in a compression member at frame buckling, and hence the frame buckling load set, provided the axial forces $\lambda_c N_r^*$ are used in the restraining members (i.e. use $\lambda_c \rho$ instead of ρ in figure G1). As it is the value of λ_c that is actually being determined, the method is iterative. The method with worked examples is given in (CG.1).

General information on the derivation of stability functions and their application to design of structural frames is given in section 7 of (CG.2).

REFERENCE TO APPENDIX CG

CG.1 Bridge, R.Q. and Fraser, D.J. 1987. Improved G-Factor Method for Evaluating Effective Lengths of Columns. Journal of Structural Engineering. ASCE, Vol. 113, No. ST6, pp. 1357-1372.

CG.2 HERA. 1994. New Zealand Structural Steelwork Limit State Design Guides, Volume 1. HERA Report R4-80. HERA, Manukau City.

APPENDIX CH
ELASTIC RESISTANCE TO LATERAL BUCKLING

Amd 2 Oct. '07

Change of status reflects the importance of the provisions to good performance.

CH1 GENERAL

This clause introduces Appendix H, which does not form an integral part of the Standard, but which gives useful information and references on lateral buckling.

CH2 SEGMENTS RESTRAINED AT BOTH ENDS

This clause provides 2 approximations for the effect of load height on the resistance to elastic buckling, the first being the more accurate, and the second the simpler. Either one of these may be used to estimate the beneficial effects of gravity loads which act below the centroid, and which are not accounted for in 5.6.3. They may also be used for uplift loading by reversing the sense of the load height y_L.

CH3 SEGMENTS UNRESTRAINED AT ONE END

For simplicity, clause 5.6.2 for segments unrestrained at one end uses the elastic buckling solution of equation 5.6.1.1 (3) for segments supported at both ends, even though this is not a very close approximation. Clause H3 gives more accurate approximations for segments unrestrained at one end, expressions which also allow approximately for the effects of load height (CH.1).

The set of approximations given in H3.1 is for beams built-in at the support so that rotation in plan (about the minor principal *y*-axis) is restrained. The set of approximations given in H3.2 is for beams whose flanges are unrestrained against this rotation in plan at the support, so that the beam ends are free to wrap, in which case the factor C_4 is taken as zero.

CH4 REFERENCE ELASTIC BUCKLING MOMENT

This clause provides the theoretical elastic lateral buckling solutions for simply supported beams in uniform bending (CH.2). The first solution (equation H4.1) is for mono-symmetric beams bent in the plane of cross section symmetry. The second solution (equations H4.2 and H4.3) is for doubly symmetric and mono-symmetric beams bent about an axis of cross section symmetry.

This clause also provides expressions for the section properties I_w, J, β_x for a range of commonly available sections.

CH5 EFFECTS OF END RESTRAINTS

CH5.1 End restraints against twist rotation of the cross section

The elastic buckling formulations of 5.6.1 and 5.6.2 and H2 and H3 are for segments whose twist rotations are prevented at one or both ends. In practice, the actual end conditions are often those of partial restraint against twist rotation. Approximations for the effects of partial restraint on the lateral buckling resistance are determined by using the twist restraint factor, k_t, from table 5.6.3(1), to determine the effective length, L_e.

This clause provides improved approximations (CH.1) for the effects of elastic end restraints against twist rotation which depend on the factor β_t, which is closely related to the ratio of the stiffness of the restraining element α_{rz} to that of the restrained segment GJ/L. Three formulations are given for the factor β_t, the first in H5.1.2 for segments restrained at both ends, the second in H5.1.3 for segments unrestrained at one end (as may be the case for cantilevers) and laterally continuous at the supported end (in which case their end warping is restrained), and the third also in H5.1.3 for segments unrestrained at one end and unrestrained against lateral rotation at the supported end (in which case the supported end is free to warp).

The approximations only take effect for values of β_t less than 1.0, and a 10 % reduction in elastic buckling resistance is predicted for β_t = 0.7. This latter value corresponds to approximate values of the stiffness ratio $\alpha_{rz}L/GJ$ of 4 for segments restrained at both ends, and 20 and 4 respectively for segments restrained at one end.

Practical applications of these provisions are given in (CH.3) and in design examples available from HERA.

CH5.2 End restraints against rotation in plan of the critical flange about the minor principal y-axis

CH5.2.1 Segments restrained against rotation in plan at both ends

The elastic buckling formulations of 5.6.1 and H2 are for segments which are unrestrained against rotation in plan at both ends. Approximations for the effects of this rotation restraint are given in 5.6.3, in which the segment effective length is affected by the value adopted from table 5.6.3(3) for the lateral rotation restraint factor k_r.

The difficulty of assessing whether the restraints against rotation in plan are significant is indicated by the restriction given in 5.4.3.4.3, which requires that a segment which does not have full lateral restraint be assumed to be unable to provide rotational restraints to an adjacent segment, unless the method of design by buckling analysis (5.6.4) is used.

This clause provides references to a relatively simple method of calculating the effective length of a segment whose ends are restrained against this rotation by adjacent segments which do not have full lateral restraint.

CH5.2.2 Segments unrestrained against rotation in plan at one end

This clause provides a method of interpolating between 0 and the value of C_4 given in table H3 for a segment with an end restraint against rotation in plan of known stiffness α_{ry} (CH.1). For values of the ratio of the restraint stiffness α_{ry} to that of the segment EI_y/L which are greater than 10, there is no reduction from the value of C_4 given in table H3, while a value of 2.5 leads to a reduction of 50 %.

CH6 REFERENCES

This paragraph provides references to computer programs for lateral buckling (H6.1 and H6.2), to textbooks and surveys of lateral buckling (H6.3 to H6.7), to research publications (Refs H6.8 to H6.10), and to methods of determining section properties (H6.11 and H6.12).

REFERENCES TO APPENDIX CH

CH.1 Trahair, N.S. 1983. Lateral Buckling of Overhanging Beams. Instability and Plastic Collapse of Steel Structures, ed. Morris, L.J. Published by Granada, London.

CH.2 Trahair, N.S. and Bradford, M.A. 1991. The Behaviour and Design of Steel Structures, Second Revision. Published by Chapman and Hall, London.

CH.3 HERA. 1994. New Zealand Structural Steelwork Limit States Design Guides, Volume 1. HERA Report R4-80. HERA, Manukau City.

APPENDIX CJ
STRENGTH OF STIFFENED WEB PANELS UNDER COMBINED ACTIONS

Amd 2 Oct. '07 The change of status from informative to normative reflects the importance of the provisions to good performance and corrects an error in one of the equations.

While this Appendix specifically refers to stiffened webs, the forms of the interaction equations given in Paragraphs J1 and J2 may also be used for unstiffened webs, provided the capacity terms in the denominations are evaluated for unstiffened webs instead of for stiffened webs.

A web is considered unstiffened when $s \geq 3\ d_p$.

The combined check offered by Appendix J should only be employed when bending and shear is combined with axial force and/or bearing. When bending and shear alone are present, the less conservative provisions of 5.12 should be used.

The effect of axial compression or bearing respectively may be neglected when $N_w^* \leq 0.1\phi N_{sw}$ or $R_w^* \leq 0.1\phi R_b$, where N_{sw} = nominal section compression capacity of the web from 6.2 and R_b = nominal bearing capacity of the web from 5.13.2.

CJ1 YIELDING CHECK

This paragraph ensures that a yield limit state is not reached under the action of the design moment M_w^*, shear V_w^*, axial force N_w^* and bearing force R_w^* acting on the web panel, as shown in figure J1. The strength reduction factor ϕ is equal to 0.9.

CJ2 BUCKLING CHECK

This clause ensures that the web panel design actions do not cause a buckling limit state to be reached. Clause J2 should be used in conjunction with clause J1. The strength reduction factor ϕ is again equal to 0.9.

APPENDIX CK
STANDARD TEST FOR EVALUATION OF SLIP FACTOR

The method adopted is that developed in (CK.1).

REFERENCE TO APPENDIX CK

CK.1 Lay, M.G. and Schmidt, L.C. 1972. Co-operative Slip Factor Tests for HSFG Joints. BHP Melb. Res. Lab. Rep. MRL 17/12.2.

APPENDIX CL
INSPECTION OF BOLT TENSION USING A TORQUE WRENCH

To ensure that all bolts are fully tensioned, the primary thrust of inspection should be to ensure that the correct bolts and nuts have been installed and that the chosen procedure has been correctly followed.

With the omission of the torque-control method of bolt tensioning from this Standard (except for specialist bolt and nut assemblies as described in 15.2.5.1 and Commentary Clause C15.2.5.1) because of its reliability under the conditions which typically prevail in the erection of steel structures, the logic of retaining this procedure as an inspection method was considered. It was decided that the procedure should be retained, but relegated to an appendix as a secondary inspection procedure, since neither the part-turn nor the direct-tension indication method is cheat proof if not properly supervised. It is emphasized that the torque wrench is considered reliable only for the detection of gross undertensioning.

Final inspection of bolt tightness by the use of an inspection wrench has been deliberately left optional, to allow the construction reviewer to exercise discretion on the amount of inspection practicable on small jobs. It follows that where the use of an inspection wrench is desired, it has to be specified by the construction reviewer. In spite of the variable torque-tension relationship of the bolts, the construction reviewer may still wish to use an inspection wrench to detect grossly undertensioned bolts.

It is envisaged that the inspection wrench may well be a manual torque wrench, and it is suggested that, where possible, this wrench be reserved for inspection purposes only.

When the method is used to check for gross under tensioning, the bolts that were tensioned first should be checked first as subsequent tensioning of other bolts may loosen them, and this check will save considerable time during an audit inspection.

The principle of setting the job inspection torque at a level which will cause a small movement of a fully tensioned bolt rather than the torque required to tighten it to the minimum tension, is to allow for the difference between static and dynamic friction.

APPENDIX CM
DESIGN PROCEDURE FOR BOLTED MOMENT-RESISTING ENDPLATE CONNECTIONS

Appendix M is deleted because it is too narrow in scope and contains errors. A design procedure for these connections is given in the SCI Publication P207:1995 *Joints in Steel Construction: Moment Connections*, published by the Steel Construction Institute, Ascot, England. A wide range of moment endplate connections are given in HERA Report R4-100:2004, *Structural Steelwork Connections Guide* and design of the columns for welded moment and moment endplate connections is covered by HERA Report R4-142, *Eccentric Cleats in Compression and Columns in Moment-resisting Connections*, Manukau City, New Zealand, 2007.

Amd 2 Oct. '07

APPENDIX CN
SECTION PROPERTIES TO USE IN ULTIMATE AND SERVICEABILITY LIMIT STATE CALCULATIONS FOR DEFLECTION

This Appendix has been included as normative in order to provide a standard on how section properties for deflection are determined for structural systems designed in accordance with this Standard. This is in line with the limit state design philosophy of providing more uniform and quantifiable factors of safety (reliability) in design.

Appendix N will typically be invoked from other provisions within this Standard and is referenced from the appropriate provisions. Cross-reference back to these provisions is provided prior to N1 for completeness.

APPENDIX CP
ALTERNATIVE DESIGN METHOD

CP1 SCOPE AND GENERAL

The alternative or working stress design method provisions contained in the 1989 edition of this Standard were based on applying a factor of safety to experimental and theoretical provisions that developed the nominal capacity of the member for the design action concerned. The resulting permissible strength was converted into a permissible stress through division by the cross-sectional area appropriate to the cross section and the design action under consideration. This was nowhere more obvious than in the provisions of section 6 of AS 1250:1981 (CP.1), where equation 6.1.1, an experimentally verified Perry-Robinson column buckling curve, was used to determine the limiting stress (F_L), which was then converted to the maximum permissible stress (F_{ac}) through multiplication by the factor of safety of 0.60.

The same philosophy is applied in Appendix P, adapted to the limit state provisions of this Standard as a whole. Thus the starting point is the nominal capacity of the member or connection. This is then converted into the permissible strength through application of the appropriate factor of safety. The permissible strength (which is an action, not a stress) is matched against the working load. Use of this philosophy allows the nominal capacities determined from the appropriate sections of this Standard (for use in ultimate limit state design) to be quickly and easily applied to the alternative design method.

The alternative design method presented in Appendix P is restricted in application to bare steel members in structural systems subject to very low or no levels of inelastic demand. This is consistent with the scope of the alternative design method presented in the 1989 edition of this Standard.

Appendix P contains only the modifications required to the other sections and clauses of this Standard in order to apply the alternative design method. This means that the clause numbering in Appendix P is not consecutive. In each instance the clause number used in this Appendix is that of the section or clause from the main body of the Standard to which this Appendix refers, prefixed with the letter “P”. Thus P1.2 refers to 1.2, while P5 refers to section 5.

CP3 GENERAL DESIGN REQUIREMENTS

CP3.2 Loads and other effects

The nominal effects or loads are generally the same for the ultimate limit state method of design and the alternative method of design. The difference in design loads for the 2 methods lies in the load factors applied to derive the design loads from the nominal loads. In design for the ultimate limit state, the load factors equal or exceed 1.0, except for dead load (G) opposing wind uplift load (W), where $0.9G$ is used. In the alternative design method, the load factors are ≤1.0 in all instances.

NZS 4203:1992 does not provide load combinations for the alternative design method, so recourse to an alternative Loadings Standard is required. The 1984 edition of NZS 4203 is a suitable example. Designers must follow the Loadings Standard requirements for general structural design as well as for determining the design loads.

In general, the design load factors for the alternative design method, to be applied to nominal loads calculated in accordance with the return period criteria of Commentary Clause C3.1(a) for the ultimate limit state, will be:

(1) 1.0 for combinations of loads acting vertically downwards to produce the design actions on a member.

(2) 0.8 for lateral loads in conjunction with 1.0 for loads acting vertically downwards, where these loads act in the same direction to produce the design actions.

(3) 0.7 for dead load acting in opposition to lateral load to produce the design actions, with the lateral load factor being 0.8.

CP3.3 Design for strength

The path to obtaining the permissible strength and the design actions for the alternative design method is best illustrated as follows:

Nominal loads (or effects)	Load ——> factors (≤ 1.0)	**Design loads (or effects)**	Structural ——> analysis (elastic)	**Design actions (S^*)**
Minimum specified values and specified parameters	Clauses of ——> Appendix P	**Nominal capacity of member (R_u)**	Factor of ——> safety (Ω) Table P3.3	**Permissible strength of member (ΩR_u)**

➤

In determining the design actions, an elastic analysis may be undertaken for each load case separately and the principle of superposition applied.

The factors of safety (Ω) for the alternative design method are given in table P3.3. They have been derived on the basis of achieving a similar ratio of permissible strength to design action under combinations of dead and live load as applied under NZS 4203:1984 and NZS 3404:1989 for the alternative design method. This involved a load factor of 1.0 and a material factor of safety of 0.6 (1/1.67) for axial force and bending or 0.67 (1/1.5) for shear. For connectors a material factor of safety of 0.6 applied. The approach taken in deriving the specific values given in table P3.3 has involved:

(1) For design of members subject to axial force and bending, the factor of 0.6 is applied directly.

(2) For design of members subject to shear, the factor of 0.67 is multiplied by (0.55/0.60) to account for the increase in nominal shear capacity given by this Standard over the corresponding provisions of NZS 3404:1989.

(3) For design of connectors, the factor of 0.60 is multiplied by (0.8/0.9) to maintain the extra increment of reliability for connection design over member design that is required from 3.3 for the ultimate limit state. The exception is Category GP welds, where the factor of safety is set at 0.4 to retain a consistent strength hierarchy between Category SP and Category GP welds between the alternative and the limit state design methods.

With regard to stability under the alternative design method, the requirements of P3.3(f) are implemented as follows:

(i) The loads or effects determined in accordance with P3.2 are subdivided into the components tending to cause instability and the components tending to resist instability.

(ii) The design action (S^*) is calculated from the component of loads or effects tending to cause instability, combined in accordance with the load combinations for the alternative design method specified in the Loadings Standard. The most critical location of loads or effects must be considered.

(iii) The design resistance is calculated from the loads and forces which can be dependably considered to resist the instability, plus the permissible strength (ΩR_u) of any elements contributing towards resisting the instability. Unless specified otherwise in the appropriate Loadings Standard, the magnitude of components of loads or effects tending to resist instability should be scaled to be consistent with a load factor of 0.6.

(iv) The whole or part of the structure, as appropriate, is proportioned so that the design resistance is equal to or greater than the design action.

The requirements of (i) – (iv) can be exemplified for the general load combination $1.0G + 1.0Q$ as:

$$1.0G^{I} + 1.0Q^{I} \leq (0.6)\, G^{S} + (\Omega R_u)$$

where G^I, Q^I, are the dead and live load components that tend to cause instability, G^S is the dead load component tending to resist instability, and (WR) is the permissible strength mobilized to resist instability (if any).

Determination of what constitutes stablizing and destabilizing components of the loads is based on the postulated mechanism of instability established by the **total** loads, in this example $1.0G + 1.0Q$.

CP3.4 Design for serviceability

The requirements to meet for serviceability under the alternative design method are effectively the same as those to be met under the limit state design method for the serviceability limit state. The methodology is given in P3.4.

CP4 STRUCTURAL ANALYSIS

As specified in P4, analysis under the alternative design method is undertaken in accordance with the elastic or nominally ductile analysis provisions of section 4. The Commentary to the relevant clauses of section 4 also applies.

Note that under the alternative design method provisions of this Appendix, the 33 % increase in permissible stresses to resist load combinations including wind or earthquake that was allowed by the 1989 edition of this Standard is no longer explicitly allowed. The same effect for these load combinations can also be obtained in indeterminate structures through moment redistribution to P4.1(b); this is applicable to any combination of loads for the alternative design method.

Note that P4.1(b) limits the maximum reduction in moment (from an elastic analysis) that is possible through moment distribution to 20 %, irrespective of the category of member actually supplied.

CP5 MEMBERS SUBJECT TO BENDING AND SHEAR; THROUGH TO CP9 CONNECTIONS

These sections apply the philosophy and requirements of CP3, as expressed in Commentary Clause CP3.3. No specific commentary to their application is given.

CP12 SEISMIC DESIGN

The alternative design method may be applied to seismic-resisting systems that respond in an elastic or a nominally ductile manner. Clause P12 references the appropriate provisions of section 12 for this purpose and specifies any modifications that are required to these provisions for application in accordance with the alternative design method.

REFERENCE TO APPENDIX CP

CP.1 Standards Australia. 1981. SAA Steel Structures Code (Working Stress Edition), AS 1250, Sydney.

APPENDIX CQ
CORRESPONDING DETAIL FROM AS 4100

CQ.1 SCOPE AND PURPOSE

The principal source of design guidance written for use with AS 4100, for which this identification is given, is the Design Capacity Tables [CQ.1]. It will also apply to other design guidance written for use with AS 4100. Details of relevant publications are available from HERA.

REFERENCE TO APPENDIX CQ

CQ.1 AISC 1994. Design Capacity Tables for Structural Steel, Second Edition, Volume 1: Open Sections. Published by the Australian Institute of Steel Construction, Sydney.

NOTES

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