NZS 3404:Part 1:2009

New Zealand Standard

Steel structures Standard

Part 1: Materials, fabrication, and construction

Supersedes in part NZS 3404 Parts 1 and 2:1997

NZS 3404:Part 1:2009

COMMITTEE REPRESENTATION

This Standard was prepared under the supervision of the P 3404 Committee for the Standards Council established under the Standards Act 1988.

The committee consisted of representatives of the following:

Nominating Organisation	Committee member
Association of Consulting Engineers New Zealand	David Cooke
Cement and Concrete Association of New Zealand	Win Clark
Department of Building and Housing	Stephen Lee
Institution of Professional Engineers New Zealand	Barry Brown
New Zealand Heavy Engineering Research Association	Stephen Hicks
Steel Construction New Zealand	Clark Hyland
Steel Construction New Zealand	Malcolm Hammond
Structural Engineering Society New Zealand	John Dale
University of Auckland	Charles Clifton
University of Canterbury	Gregory MacRae

ACKNOWLEDGEMENT

Standards New Zealand gratefully acknowledges the contribution of time and expertise from all those involved in developing this Standard. We wish to particularly thank New Zealand Heavy Engineering Research Association and National Institute of Water and Atmospheric Research for the use of Figures 20 to 25.

The P 3404 Committee and Standards New Zealand also wishes to thank the P 3404 Part 1 Subcommittee:

Alistair Fussell	Bruce Mellsop	Don Silvester	John Frederickson	Raed El Sarraf
Bradley Wynn	Colin Beavis	Geoffrey Verrill	Kevin Cowie	Wayne Carson

for its drafting work in the preparation of this Standard.

Cover photo courtesy of Beca and Mighty River Power.

Published by Standards New Zealand, the trading arm of the Standards Council, Private Bag 2439, Wellington 6140. Telephone: (04) 498 5990, Fax: (04) 498 5994, Website: **http://www.standards.co.nz**.

AMENDMENTS

No.	*Date of issue*	*Description*	*Entered by, and date*

NZS 3404.1:2009

New Zealand Standard

Steel structures Standard – Part 1: Materials, fabrication, and construction

Supersedes in part NZS 3404 Parts 1 and 2:1997

ISBN 978-1-86975-126-5

NOTES

CONTENTS

Appendix

Table

Figure

REFERENCED DOCUMENTS

Reference is made in this document to the following:

NEW ZEALAND STANDARDS

NZS 1170: - - - -	Structural design actions
Part 5:2004	Earthquake actions – New Zealand
NZS 3101: - - - -	Concrete structures Standard
Part 1:2006	The design of concrete structures
Part 2:2006	Commentary on the design of concrete structures
NZS 3104:2003	Specification for concrete production
NZS 3109:1997	Concrete construction
NZS 3404:- - - -	Steel structures Standard
Part 1:1997	Steel structures Standard
Part 2:1997	Commentary to the steel structures Standard
NZS 3404: - - - -	Steel structures Standard
Part 2: - - - -	Structural analysis (in preparation)
Part 3: - - - -	General design of members and connections (in preparation)
Part 4: - - - -	Design of composite members (in preparation)
Part 5: - - - -	Design for fire (in preparation)
Part 6: - - - -	Design for fatigue (in preparation)
Part 7: - - - -	Design for earthquakes (in preparation)

JOINT AUSTRALIAN/NEW ZEALAND STANDARDS

AS/NZS 1170: - - - -	Structural design actions
Part 0:2002	General principles
Part 1:2002	Permanent, imposed and other actions
Part 2:2002	Wind actions
Part 3:2003	Snow and ice actions
AS/NZS 1252:1996	High-strength steel bolts with associated nuts and washers for structural engineering
AS/NZS 1365:1996	Tolerances for flat-rolled steel products
AS/NZS 1554: - - - -	Structural steel welding
Part 1:2004	Welding of steel structures
Part 2:2003	Stud welding (steel studs to steel)
Part 5:2004	Welding of steel structures subject to high levels of loading
AS/NZS 1559:1997	Hot-dip galvanized steel bolts with associated nuts and washers for tower construction
AS/NZS 1594:2002	Hot-rolled steel flat products
AS/NZS 1873: - - - -	Powder-actuated (PA) hand-held fastening tools
Part 1:2003	Selection, operation and maintenance
Part 2:2003	Design and construction
Part 3:2003	Charges
Part 4:2003	Fasteners
AS/NZS 2312:2002	Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings
AS/NZS 3678:1996	Structural steel – Hot-rolled plates, floorplates and slabs
AS/NZS 3679: - - - -	Structural steel
Part 1: 1996	Hot-rolled bars and sections

Part 2: 1996	Welded I-sections
AS/NZS 3750: - - - -	Paints for steel structures
Part 9:2009	Organic zinc-rich primer
Part 15:1998	Inorganic zinc silicate paint
AS/NZS 4680:2006	Hot-dip galvanized (zinc) coatings on fabricated ferrous articles
AS/NZS ISO 3834: - - - -	Quality requirements for fusion welding of metallic
Part 2:2008	Comprehensive quality requirements
Part 3:2008	Standard quality requirements
Part 4:2008	Elementary quality requirements
AS/NZS ISO 9000: - - - -	Quality management systems set
9000:2006	Fundamentals and vocabulary
9001:2008	Requirements
9004:2000	Guidelines for performance improvements

INTERNATIONAL STANDARDS

ISO 2566: - - - -	Steel – Conversion of elongation values
Part 1:1984	Carbon and low alloy steels
ISO 9224:1992	Corrosion of metals and alloys – Corrosivity of atmospheres – Guiding values for the corrosivity categories

AMERICAN STANDARDS

ANSI/AISC 341:2005	Seismic provisions for structural steel buildings
ASTM A106/A106M 2008	Standard specification for seamless carbon steel pipe for high-temperature service
ASTM A193/A193M 2009	Standard specification for alloy-steel and stainless steel bolting materials for high temperature or high pressure service and other special purpose applications
ASTM A514/A514M 2005	Standard specification for high-yield-strength, quenched and tempered alloy steel plate, suitable for welding

AUSTRALIAN STANDARDS

AS 1110: - - - -	ISO metric hexagon bolts and screws – Product grades A and B
Part 1:2000	Bolts
Part 2:2000	Screws
AS 1111: - - - -	ISO metric hexagon bolts and screws – Product grade C
Part 1:2000	Bolts
Part 2:2000	Screws
AS 1112: - - - -	ISO metric hexagon nuts
Part 1:2000	Style 1 – Product grades A and B
AS 1163:1991	Structural steel hollow sections
AS 1210:1997	Pressure vessels
AS 1275:1985	Metric screw threads for fasteners
AS 1391:2007	Metallic materials – Tensile testing at ambient temperature
AS 1418: - - - -	Cranes, hoists and winches
Part 1:2002	General requirements
AS 1576: - - - -	Scaffolding

Part 1:1995	General requirements
AS 1627: - - - -	Metal finishing – Preparation and pretreatment of surfaces
Part 1:2003	Removal of oil, grease and related contamination
Part 2:2002	Power tool cleaning
Part 4:2005	Abrasive blast cleaning
Part 7:1988	Hand tool cleaning of metal surfaces
AS 2074:2003	Cast steels
AS 2159:1995	Piling – Design and installation
AS 2382:1981	Surface roughness comparison specimens
AS 2673:1983	Paints for steel structures – Alkyd/micaceous iron oxide
AS 3828:1998	Guidelines for the erection of building steelwork
AS 3894: - - - -	Site testing of protective coatings
Part 3:2002	Determination of dry film thickness
AS 4100:1998	Steel structures
AS 5100: - - - -:	Bridge design
Part 6:2004	Steel and composite construction

BRITISH STANDARDS

BS 4:2005	Structural steel sections
Part 1	Specification for hot-rolled sections
BS 7668:2004	Weldable structural steels. Hot finished structural hollow sections in weather resistant steels. Specification
BS 8004:1986	Code of practice for foundations

EUROPEAN COMMITTEE FOR STANDARDIZATION (CEN)

EN 1090-2:2008	Execution of steel structures and aluminium structures – Part 2: Technical requirements for steel structures
EN 1991-1-6:2005	Eurocode 1 – Actions on structures – Part 1-6: General actions – Actions during execution
EN 1993-1-1:2005	Eurocode 3 – Design of steel structures – Part 1-1: General rules and rules for buildings
EN 1993-5:2007	Eurocode 3 – Design of steel structures – Part 5: Piling
EN 10025:2004	Hot rolled products of structural steels
Part 1	General technical delivery conditions
Part 2	Technical delivery conditions for non-alloy structural steels.
Part 3	Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels
Part 4	Technical delivery conditions for thermomechanical rolled weldable fine grain structural steels
Part 5	Technical delivery conditions for structural steels with improved atmospheric corrosion resistance
Part 6	Technical delivery conditions for flat products of high yield strength structural steels in the quenched and tempered condition
EN 10029:1991	Hot rolled steel plates 3 mm thick or above – Tolerances on dimensions, shape and mass
EN 10210:2006	Hot finished structural hollow sections of non-alloy and fine grain steels
Part 1	Technical delivery conditions

EN 10219:2006	Cold formed welded structural hollow sections of non-alloy and fine grain steels
Part 2	Tolerances, dimensions and sectional properties
EN 12812:2008	Falsework – Performance requirements and general design

JAPANESE STANDARDS

JIS G 3101:2004	Rolled steel for general structure
JIS G 3106:2008	Rolled steels for welded structure
JIS G 3114:2008	Hot-rolled atmospheric corrosion resisting steels for welded structure
JIS G 3132:2005	Hot-rolled carbon steel strip for pipes and tubes
JIS G 3136:2005	Rolled steel for building structure
JIS G 3192:2008	Dimensions, mass and permissible variations of hot rolled steel sections
JIS G 3193:2005	Dimensions, mass and permissible variations of hot rolled steel plates, sheets and strip

OTHER PUBLICATIONS

American Petroleum Institute. *API 5L Specification for line pipe.* 42nd ed. Washington: API, 2000.

American Institute of Steel Construction, Quality criteria and inspection Standards.

AREMA. *AREMA Manual for railway engineering*. Chapter 15. Lanham, MD: American Railway Engineering and Maintenance-of-Way Association, 2009.

AWS D1.5:2008 *Bridge welding code*

IIW, 2007. XIII-2151-07 *Recommendations for fatigue design of welded joints and components*. International Institute of Welding.

IPENZ/ACENZ. *The briefing and engagement of consultants*. IPENZ/ACENZ, 1994.

NZTA. *Bridge manual* SP/M/022. 2nd ed. New Zealand Transport Agency, 2003.

NEW ZEALAND LEGISLATION

Building Act 2004 and New Zealand Building Code

Chartered Professional Engineers Act 2002

WEBSITES

New Zealand Legislation	http://www.legislation.govt.nz
New Zealand Welding Centre	http://www.hera.org.nz/nzwc

LATEST REVISIONS

The users of this Standard should ensure that their copies of the above-mentioned New Zealand Standards are the latest revisions. Amendments to referenced New Zealand and Joint Australian/New Zealand Standards can be found on **http://www.standards.co.nz**.

REVIEW OF STANDARDS

Suggestions for improvement of this Standard will be welcomed. They should be sent to the Chief Executive, Standards New Zealand, Private Bag 2439, Wellington 6140.

Foreword

NZS 3404 has now been in use since the first edition in 1989, the second in 1992, and the third in 1997. With increasing use of steel structures and changes to the regulatory framework a full review of the Standard was therefore commissioned by the Department of Building and Housing.

The committee made the decision to divide NZS 3404 into seven parts that interrelate but also can be used largely in a stand-alone manner by the relevant user groups. This revision of NZS 3404 has been divided into separate parts so that users can better access the requirements of this Standard, and the knowledge that underpins them. To this end NZS 3404.1 was devised to be the first part. Its user base includes architects, structural engineers, steel distributors, steel constructors, builders, and building control authorities. It was recognised that when many of the provisions were bound within the design standard in previous editions, the construction end users often did not recognise the relevance of the document to them. The potential benefits of having commonly recognised requirements were therefore not fully achieved. In addition the commentary clauses have been brought into the body of the Standard to facilitate access to the background knowledge and principles embodied in the provisions. It is hoped that the new format will lead to greater realisation of those benefits by all users.

A specialist subcommittee was formed to develop provisions for the committee, including representatives from architectural, engineering, fabrication, precast concrete, and building companies, as well as from Steel Construction New Zealand Inc., and the New Zealand Heavy Engineering Research Association.

In addition to reformatting the Standard significant new provisions have been added. These include specific guidance for identifying corrosivity of steelwork and selection of protective coatings in the New Zealand environment. This complements the coatings Standard AS/NZS 2312. A means of categorising the finishing requirements of architecturally exposed steelwork drawing on work by the Canadian Institute of Steel Construction has been established.

The extent of weld testing is now more consistently and easily determined by use of a loadings demand, and consequence of a weld failure assessment approach. Seismic grade steel types 2S and 5S have been introduced following the performance specification requirements set in Amendment No.2 of the 1997 edition, and in line with subsequent changes to AS/NZS 3679.1 and AS/NZS 1554.

The tolerances for steel fabrication have been reviewed and compared with current international Standards and codes of practice and adjusted and expanded accordingly. Alignment has been sought with related materials Standards where practicable, however more stringent construction tolerances at the interface of reinforced concrete and structural steelwork are now required by this Standard.

Composite construction provisions have been included.

At the request of the New Zealand Transport Agency and Ontrack, provisions for highway and railway bridges have been significantly updated. These now incorporate the relevant requirements of AS 5100.6, the American AREMA design guidelines and

AWS D1.5 *Bridge welding code* contextualised for the New Zealand market. This includes the introduction of the fracture critical member (FCM) designation and the requirements for fracture control plans (FCP) necessary for quality management control in the fabrication of railway bridges.

Knowledge of the content of this Standard will be one of the indicators of competency for steel structures licensed building practitioners (LBP), for which restricted work requirements will commence in 2012.

To assist users of NZS 3404 Parts 1 and 2:1997, Appendix B outlines the clauses, figures, and tables from the 1997 version which are superseded by NZS 3404.1:2009.

Outcome Statement

NZS 3404.1 provides an authoritative single source of guidance to design and construction practitioners, and building control officials. Better communication and understanding of the requirements for achieving good quality, durable, and earthquake resistant steel construction for building and bridge structures will result in improved practice for the selection of materials, the fabrication, and construction of steel structures in New Zealand.

1 GENERAL

1.1 Scope

1.1.1 Inclusions

NZS 3404.1 sets out minimum requirements for the selection of materials, corrosion protection systems, and the fabrication, erection, and construction of steel structures. It will supersede the relevant provisions in NZS 3404 Part 1 and 2:1997 and will be referenced by NZS 3404.2 to NZS 3404.7 once those parts are complete.

This Standard applies to building structures; crane support girders; highway, railway, and pedestrian bridges; and composite steel and concrete beams and columns.

1.1.2 Exclusions

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

(a) Steel elements less than 3 mm nominal thickness, except for packers and square or rectangular hollow sections to 2.2.1; and

(b) Steel members for which the value of 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, and the appropriate grade for general structural use is selected (ASTM A514 or equivalent grade). Any welding of such steel shall only be to the approval of the Design Engineer and in accordance with AS/NZS 1554, with due regard for the changes in mechanical properties that will result from the fabrication process.

1.2 Use of this Standard as a means of compliance with the New Zealand Building Code (NZBC)

It is intended that once all parts of the revised steel structures Standard (that is, NZS 3404.1 to NZS 3404.7) are published, this Standard will be referenced in the Compliance Document for the NZBC Clause B1 Structure, Verification Method B1/VM1, and Clause D2 Durability, Acceptable Solution D2/AS1.

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

1.3 Interpretation

For the purposes of this Standard, the word 'shall' refers to requirements that are essential for compliance with the Standard, while the word 'should' refers to practices that are advised or recommended.

Clauses prefixed by '*C*' and printed in italic type are intended as comments on the corresponding clauses. They are not to be taken as the only or complete interpretation. The Standard can be complied with if the comment is ignored.

The terms 'Normative' and 'Informative' have been used in this Standard to define the application of the Appendix to which they apply. A 'Normative' Appendix is an integral part of the Standard and contains requirements. An 'Informative' Appendix gives additional information, and is only for guidance. It does not contain requirements.

1.4 Definitions

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

Associated structural system	A structural system which is not specifically designed for load combinations including earthquake loads but which is subject to earthquake effects resulting from the deformation of the seismic-resisting system under its design actions
Authority	A body having statutory powers to control the design and erection of a structure for example, a territorial authority, for applications within the scope of 1.1
Bearing-type connection	Connection effected using either snug-tight bolts (-/S), or high strength bolts tightened to induce a specified minimum bolt tension (-/TB) , in which the design action effect 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
Buckling restrained brace frame (BRB)	Ductile buckling restrained braced frames can develop significant inelastic deformation through axial yielding in tension and compression of the core of the buckling restrained bracing members
Capacity design	Design to direct inelastic demand into selected members of a seismic-resisting system under severe seismic action
Capacity reduction factor	A factor used to multiply the nominal capacity to obtain the design capacity
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 weld (CPW)	A weld in which fusion exists between the weld and parent metal throughout the complete depth of the joint
Concentrically braced frame (CBF) system	A braced frame in which the members are subject primarily to axial forces
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
Construction reviewer	The person responsible for review of construction
Design Engineer	An engineer with relevant experience and skills in structural engineering as applied to New Zealand who is responsible for interpretation of the requirements of this standard when used for building structure design. A structural engineer who is chartered under the Chartered Professional Engineers of New Zealand Act 2002 would satisfy this requirement

Design life	Period over which a structure or structural element is required to perform its function without repair
Doubler plate	Additional side reinforcing plate, fixed to the web of a member, provided to increase the shear capacity of the web
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
Eccentrically braced frame (EBF) system	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
Fastener	A prefabricated item which transfers load, that is a bolt, rivet, or turnbuckle
Fatigue	Damage caused by repeated fluctuations of stress leading to gradual cracking of a structural element
Fatigue purpose welds	Fatigue purpose welds should be used for structures subject to fatigue with high cycle dynamic loading and conform to the requirements of the welding standard AS/NZS 1554.1 or AS/NZS 1554.5 as appropriate to the detail category and the application described in 3.2.3
Fracture control plan (FCP)	The fabrication and inspection plan developed to control fracture in non-redundant members of a bridge (see Appendix A)
Fracture critical members (FCM)	Bridge fracture critical members or member components are tension members or components of bending members subject to tensile flexural stresses, the failure of which would be expected to result in collapse of the bridge
Fracture critical member (FCM) welds	All welds to FCMs in bridges shall be considered fracture critical and shall conform to the requirements of the FCP
Fracture critical member (FCM) attachments	Any attachment welded to a tensile zone of a fracture critical member in a bridge shall be considered a FCM when any dimension of the attachment exceeds 100 mm in the direction parallel to the calculated tensile stress in the FCM. Attachments designated FCM shall meet all the requirements of the FCP
Friction-type connection	Connection effected using high-strength bolts tightened to induce a specified minimum bolt tension so that the resultant clamping action effect 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 (-/TF)
Full tensioning	A method of installing and tensioning a bolt in accordance with 4.2.5 and 4.2.6

General purpose (GP) welds	GP welds should only be used for redundant or non-structural applications where design capacity and failure of the weld will have minimal structural significance
Incomplete penetration butt weld	A fusion weld in which the depth of penetration is less than the complete depth of the plate
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
Lateral buckling	An instability phenomenon that reduces the major axis flexural strength of certain types of steel sections from a combination of flexural action effects and twists
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
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 as 'permanent action'
Load, live	Is referred to in AS/NZS 1170 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 actions for buildings and approved by the authority. For the use of this Standard as a verification method in Approved Document B1, the 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
Liquid penetrant examination (LP)	LP examination of welds in accordance with AS/NZS 1554
Member	A steel section spanning between supports or cantilevered beyond a support
Moment-resisting frame (MRF) system	A structural system of rigid or semi-rigid construction capable of resisting design actions principally through the bending resistance of its members and connections
Magnetic particle inspection (MT)	Magnetic particle inspection of welds in accordance with AS/NZS 1554
Overstrength	The maximum strength that a member can generate, taking into account higher than specified steel yield stresses and an increase in strength from strain hardening, where relevant
Owner	The owner as defined in the Building Act 2004
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
Prequalified weld procedure	A welding procedure prequalified in AS/NZS 1554.1 or AS/NZS 1554.5
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 constructed or a nominated representative
Proof testing	The application of test loads to a structure, substructure, member or connection to ascertain the structural characteristics of only that one unit under test
Prototype testing	The application of test loads to one or more structures, substructures, members or connections to ascertain the structural characteristics of that class of structures, substructures, members or connections which are nominally identical to the units tested
Reduced beam section connection (RBS)	In a reduced beam section moment connection, portions of the beam flanges are selectively trimmed in the region adjacent to the beam to column connection. Yielding and plastic hinge formation are intended to occur primarily within the reduced section of the beam and thereby limit the inelastic deformation demands at the face of the column and capacity design actions on the column
Rectangular frame	A frame comprising beam and column members in which the orientation of any beam member is sufficiently close to the horizontal so 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 if designed for the appropriate restraint forces, for a member subject to combined bending and axial compression.
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

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
Shear wall	A wall designed to resist lateral forces parallel to the plane of the wall
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
Structural purpose (SP) welds	SP welds should be used for most structural applications in which failure of the weld could lead to structural failure and collapse, or damage of an element or structure leading to loss of service
Stability (limit state)	Part of the ultimate limit state corresponding to the loss of static equilibrium of a structure considered as a rigid body
Steel plate shear wall (SPSW)	A steel plate shear wall is a lateral load resisting system consisting of steel plate infills connected to surrounding horizontal (beams) and vertical (column) boundary elements to form a cantilever wall
Storey	The part of a structural system between two logical consecutive horizontal divisions
Structural system	An assemblage of members and connections acting together to resist the actions due to the specified design actions
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 (that is, it provides full or partial section restraint)
Tensile strength (f_u)	The minimum ultimate strength in tension (R_m) as defined in AS 1391. This is the stress at which localised necking begins.
Time to first maintenance	See 5.1.2
Transverse load	The 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
Ultrasonic or radiographic testing (UT)	Ultrasonic or radiographic testing of welds in accordance with AS/NZS 1554
Welding procedure specification (WPS)	Specifies the parent material, welding standard, welding process, edge preparation, welding direction, applicable range, preheat temperature and method, inter-run temperature, welding consumables, weld run details, welding parameters, and other matters necessary to achieve a specific weld

Yield stress (f_y) — The minimum lower yield stress in tension (R_{eL}) or 0.2% proof stress ($R_{o.2p}$) in steels not exhibiting a clear yield plateau, such as for hollow sections, as defined in AS 1391.

Yielding region — That length of a member which is intended to yield under earthquake loads or from the redistribution of design action effects or in which yield under earthquake loads or effects may occur. For the purposes of this part the yielding region shall be approximated as the region measured from the face of the supports to 1.5 x depth of member and shall also include all link zones of eccentrically braced frames, scalloped sections of reduced beam section connections, wall plates of steel shear walls, and the core and connections of buckling restrained braces

1.5 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_o — Plain shank area of a bolt

A_{sc} — Shank area of a stud

a_o — Length of unthreaded portion of the bolt shank contained within the grip

a_t — Length of threaded portion of the bolt contained within the grip

a_0, a_1 — Out-of-square dimensions of flanges

a_2, a_3 — Diagonal dimensions of a box section

b — Width;
Lesser dimension of a web panel;
Clear width of an element outstand from the face of a supporting plate element; or
Clear width of a support element between faces of a supporting plate elements

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_{es} — Stiffener outstand from the face of a web

b_f — Width of a flange

b_r — Average width of concrete rib in a composite slab cast onto a profiled steel deck

b_w — Web depth

b_1, b_2 — Greater and lesser leg lengths of an angle section

d — Depth of a section;
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_f	Diameter of a fastener (bolt or pin); or Distance between flange controids
d'_f	Diameter of bolt hole
d_o	Overall section depth including out-of-square dimensions; Over 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 the deepest web panel in a length
d_{sc}	Diameter of a stud shear connector
d_1	Clear depth between flanges ignoring fillets or welds
d_3, d_4	Depths of preparation for incomplete penetration butt welds
d_5	Flat width of web
e	Eccentricity; Web off-centre dimension; Distance between an end plate and a load-bearing stiffener; 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
f'_c	Specified concrete cylinder compression strength
f_u	Tensile strength of the element under consideration
f_{uf}	Minimum tensile strength of a bolt
f_{uw}	Nominal tensile strength of weld metal
f_y	Yield stress of the element under consideration
h_b	Vertical distance between tops of beams
h_{rc}	Nominal height of steel deck rib
h_s	Storey height
h_{sc}	Length of stud connector after welding (mm)
L	Span; Member length; or Segment or subsegment length
L_b	Length between points of effective bracing or restraint
L_c	Distance between adjacent column centres
L_s	Distance between points of effective lateral support
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)

r	Radius of gyration; Transition radius; or The root radius of a section
R_m	Tensile strength
$R_{0.2p}$	0.2% proof tensile stress
R_{eL}	Lower yield tensile stress
/S	Snugtight mode of bolt action
s_g	Gauge of bolts
s_p	Staggered pitch of bolts
/TB	Tension bearing mode of bolt action
/TF	Tension friction mode of bolt
t	Thickness; Thickness of thinner part jointed; Wall thickness of a circular hollow section; Thickness of an angle section; Effective thickness of a composite slab; or Time
t_f	Thickness of a flange; or Thickness of the critical flange
t_n	Thickness of a nut
t_o	Overall thickness of a composite slab
t_p	Thickness of a ply; Thickness of thinner ply connected; or Thickness of a plate (including a doubler plate)
t_r	Thickness of a backing plate
t_{sl}	Steel thickness loss
t_t, t_{t1}, t_{t2}	Design throat thickness of a weld
t_w	Thickness of a web
v_w	Nominal shear yield capacity of a web; Nominal shear capacity of a plug or slot weld
z	Specific fillet weld dimension for UT in accordance with Table 32
Δ	Deflection; Deviation from nominated dimension; or Measured total extension of a bolt when tightened
Δ_f	Out-of-flatness of a flange plate
Δh_b	Deviation from h_b
ΔL_c	Deviation from L_c
Δ_s	Translation 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
μ	Structural displacement ductility factor
μ_s	Slip factor
μ_{sm}	Mean value of the slip factor

1.6 Abbreviations

The following abbreviations are used in this Standard:

AESS	Architecturally exposed structural steel
BRB	Buckling restrained braced frame
CBF	Concentrically braced frame
CHS	Circular hollow section
CPW	Complete penetration weld
EBF	Eccentrically braced frame
FCM	Fracture critical member
FCP	Fracture control plan
GMAW	Gas metal arc welding
GP	General purpose weld
HSS	Hollow structural steel
LBP	Licensed building practitioner
LODMAT	Lowest one-day mean ambient temperature
LP	Liquid penetrant examination of welds
MRF	Moment resisting frame
MT	Magnetic particle
NDT	Non-destructive testing
NDE	Non-destructive examination
NZBC	New Zealand Building Code
PS	Producer statement
QA	Quality assurance
RBS	Reduced beam section moment connection
RHS	Rectangular hollow section
SP	Structural purpose weld
SPSW	Steel plate shear wall
UT	Ultrasonic or radiographic testing of welds
WPS	Welding procedure specification
WQMS	Welding quality management system

1.7 Design and documentation

1.7.1 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 their representative.

C1.7.1

An appropriate level of knowledge and competence is 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.

1.7.2 Drawings or specification requirements

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

(a) The corrosion protection requirements, inspection and maintenance regime;
(b) The steel grades;
(c) The size and designation of each member;
(d) Seismic frames and seismic member categories;
(e) Members requiring fabrication conforming to fatigue provisions;
(f) Weld failure consequence category;
(g) Seismic weld demand category;
(h) Fracture control plan for railway bridges in accordance with Appendix A;
(i) The location of any FCMs in bridges. Each portion of a bending member that is fracture critical shall be clearly described giving the limits of the FCM;
(j) The sizes, types, and categories of welds used in the connections;
(k) The sizes, property class, and tightening categories of the bolts used in the connections;
(l) The sizes of the connection components;
(m) The locations and details of planned joints, connections, and splices;
(n) Any constraint on construction assumed in the design;
(o) The camber of any members;
(p) The required extent of propping of the deck and any supporting beams;
(q) The required precamber of beams;
(r) The required method of screeding of the concrete surface, that is, screeding to level or screeding to thickness;
(s) The surfaces not to be coated, for example areas with shear studs to be welded on later, or faying surfaces of friction grip joints;
(t) The welding quality management system requirements, including the AS/NZS 1170.0 importance level of the structure;
(u) The extent of any steel elements subject to architecturally exposed structural steel (AESS) requirements including AESS designations;
(v) Permanent marking requirements for end of life traceability and reuse in accordance with 2.2.6;
(w) Specific fabrication or erection tolerances more stringent than otherwise required by 3.3 and 4.2; and
(x) Designated items requiring precision setting out of anchor bolts in accordance with Figure 11.

1.8 Construction review

C1.8

The aim of the Committee has been to make the monitoring requirements of this Standard consistent with those of the New Zealand building control system.

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 contractor's quality assurance (QA) system; and

(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 building consent 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 (for example welding, painting, and so on).

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 building consent authority.

Note that the requirements of the main contractor (builder) are equally applicable to the steel fabricator and erection subcontractor. Because of this, the term 'contractor' is used in the selection of an appropriate level of construction monitoring.

(1) Contractor has a formal, written, independently certified QA system appropriate to steel fabrication and construction (see C3.2.3). For routine work, a minimum of Construction Monitoring Level 2 (CM2) in accordance with the joint IPENZ:ACENZ document 'The briefing and engagement of consultants' 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 QA system (for example NZS 9000 series certification) will be submitted to the building consent 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 PS, 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 QA system appropriate to steel fabrication and construction (see C3.2.3), without independent audit. For routine work, a minimum of Construction Monitoring Level 3 (CM3) in accordance with the joint IPENZ:ACENZ document 'The briefing and engagement of consultants' 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 should supply to the owner a PS that the work has been carried out in accordance with the requirements of the building consent.

A similar PS, 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 QA system (see C3.2.3). For routine work, a minimum of Construction Monitoring Level 4 (CM4) in accordance with the joint IPENZ:ACENZ document 'The briefing and engagement of consultants' 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, (CM3) in the joint IPENZ:ACENZ document 'The briefing and engagement of consultants'.

On completion of the work, the contractor should supply to the owner a PS that the work has been carried out in accordance with the requirements of the building consent.

A similar PS, appropriate to the extent of the construction reviewer's engagement, may be required from the construction reviewer by the authority.

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.

There are times when the welding supervisor may also be the welding inspector. This should only be considered when:

(d) Visual inspection only of all welds is required;

(e) The fabricator has a demonstrable proven quality track record from past jobs similar to the scope of work being undertaken;

(f) The person nominated has performed this dual role satisfactorily on past jobs; and

(g) 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 QA system (see C3.2.3).

1.8.1

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.)

1.8.2

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.

C1.8.2

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

Either AS/NZS 1554.1 or AS/NZS 1554.5 require qualification of welding procedure specifications (WPSs) by the fabricator. Prior to commencement of welding, these should be approved for the particular job by the Design Engineer, or by the construction reviewer, or by their nominated representative.

1.8.3

For railway bridges, a fracture control plan shall be prepared in accordance with Appendix A as part of the construction review.

1.8.4 Falsework and temporary works

Structural steelwork for falsework or temporary works shall be designed and constructed to have sufficient strength and stiffness to safely carry the imposed loads for the duration of its use to support construction loads.

Consideration shall be given to:

(a) Construction sequence and duration;
(b) Construction static loads;
(c) Construction loads that change with time such as:
    (i) Temperature effects on the supported elements and the falsework
    (ii) Dynamic and vibration effects
    (iii) Vehicle movement
    (iv) Concrete placement
    (v) Post stressing of supported precast concrete elements;
(d) Accuracy of load assessment;
(e) Risk of individual member failure; and
(f) Consequence of individual member failure.

C1.8.4

The structural analysis should assess the imposed loads and the proposed structural system to a level of accuracy that is appropriate for the defined loads, complexity, and redundancy of the structural system, and the consequences of failure.

Guidance for loadings for falsework and temporary works structures is found in AS/NZS 1170.0, BS EN 12812, and AS 1576.1. Guidance for loadings to profiled steel decking is found in Eurocode 1 (EN 1991-1-6).

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 minimum lower yield stress in tension (R_{eL}) or 0.2% proof stress ($R_{0.2p}$) specified for the grade and thickness of steel in the appropriate Standard as listed in 2.2.

2.1.2 Tensile strength

The tensile strength (f_u) used in design shall not exceed minimum tensile strength specified for the grade and thickness of steel in the appropriate Standard as listed in 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.

(a) Australian or Joint Australian/New Zealand Standards:
AS 1163
AS/NZS 1594
AS/NZS 3678
AS/NZS 3679 (Part 1 and Part 2);

(b) British Standards:
BS 4 (Part 1)
BS 7668
BS EN 10025 (Parts 1 to 6)
BS EN 10029
BS EN 10210 (Part 1)
BS EN 10219 (Part 2);

(c) Japanese Standards:
JIS G 3101
JIS G 3106
JIS G 3114
JIS G 3132
JIS G 3136
JIS G 3192
JIS G 3193;

(d) American Standards:
API 5L
ASTM A106/A106M.

If structural steels or shapes other than those referred to in (a), (b), (c), and (d) above are used, they shall comply with an internationally recognised Standard that is approved by a qualified metallurgist or materials engineer, as being equivalent to those listed above. Such an approval is outside the scope of this Standard as part of a Verification Method for the NZBC.

2.2.2 Acceptance of steels

For acceptance of steels sufficient evidence of compliance with the relevant material supply standards specified in 2.2.1 shall be test reports or test certificates prepared by a laboratory accredited by signatories to the International Laboratory Accreditation Cooperation (ILAC) Mutual Recognition Agreement (MRA) on behalf of the manufacturer.

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 AS 1391 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.

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 elongation requirements of the conforming steel types in Table 1 relevant to the seismic member category.

Unidentified steel shall not be used in members of an associated structural system which are subject to inelastic demand or in members which are subject to moment redistribution, **unless** it is shown by tests that the steel complies with the respective elongation requirements of Type 1 hot rolled sections or plate in Table 2.

2.2.4 Steel for seismic applications

Steel types used in seismic members shall conform with the requirements of Table 1 or otherwise comply with the requirements of Table 3. See Table 2 for steel grade relationships to steel types and Table 4 for steel types for highway or railway bridges.

C2.2.4

The materials requirements for category 1 and 2 members are more stringent than those for category 3 members, which in turn are more stringent than those required for category 4 members. Therefore steel types suitable for category 1 and 2 members may also be used for category 3 and 4 members. Similarly, steel types suitable for category 3 members may also be used for category 4 members.

Table 1 – Steel types for seismic members

Seismic member category	Conforming steel types
1 and 2	2S, 3, 5S, 6
3	2, 5
4	1, 4, 7A, 7B, 7C

Table 2 – Steel type relationship to steel grade
(For steels from 2.2.1)

Steel type	Steel grade					
	AS 1163 ASTM A106 API 5L	**AS/NZS 1594**	**AS/NZS 3678 AS/NZS 3679.2**	**AS/NZS 3679.1**	**BS EN 10025**	**JIS G 3106 JIS G 3136**
1	C250 Grade B	200 250 300	200 250 300	250 300	S275 S275JR	SM 400A SN 400A
2	C250L0	–	–	250L0 300L0	S275J0	SM 400B SN 400B
2S	–	–	250S0 300S0	250S0 300S0	–	–
3	–	–	250L15 300L15	250L15 300L15	S275J2G3/ S275J2G4	SM 400C
4	C350	HA350 HA440	350 WR350	350	S355 S355JR	SM 490YA
5	C350L0	–	WR350L0	350L0	S355J0	SM 490YB SM 520B SM490B
5S	–	–	350S0	350S0	–	–
6	–	–	350L15 400L15	350L15	S355J2G3/ S355J2G4	SM 520C
7A	C450	–	–	–	–	–
7B	C450L0	–	–	–	–	–
7C	–	–	450L15	–	–	–

Table 3 – Category 1 and 2 seismic member material requirements

Item		Category 1 and 2 members
1	Maximum grade yield stress (see Note 1)	360 MPa
2	Minimum % elongation after fracture (see Notes 2, 3)	25
3	Maximum yield to tensile ratio (f_y/f_u) (see Note 2)	0.80
4	Maximum yield stress (see Note 2)	≤ 1.33 f_y (see Note 1)
5	Minimum Charpy V-Notch impact energy (see Notes 2, 4, 5, 6)	70J @ 0°C – Average of three tests 50J @ 0°C – Individual test

NOTE –

(1) The limits in item 1 and the 1.33 f_y value in item 4 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) Elongation after fracture shall be determined from proportional test pieces in accordance with AS 1391.

(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.

(6) Steel conforming with Table 3 may be considered to be equivalent to steel type 2S or 5S for permissible service temperature and welding requirements.

2.2.5 Steel for highway or railway bridges

2.2.5.1

Steel for non-FCMs shall be Types 1 or 4 (see Table 4).

2.2.5.2

Steel for FCMs shall be Types 2 or 5 for highway bridges (see Table 4). For railway bridges FCM steel shall be Types 2 or 5 for thicknesses less than or equal to 17 mm and Types 3 or 6 for thicker elements. All steel for FCMs shall be manufactured using killed fine-grain practice. No welded repairs to the steel shall be performed by the producing mill. If heat numbers and other identification numbers are to be applied by die stamping, low stress dies shall be used.

Table 4 – Conforming steel types for highway or railway bridges

Member category	Conforming steel types
Non-FCMs	1, 4
Highway bridges FCM	2, 5
Railway bridges FCMs < 17 mm thick	2, 5
Railway bridges FCMs ≥ 17 mm thick	3, 6

2.2.6 Reuse of steel

Where end of life reuse of steel sections is required, the following requirements shall apply:

(a) The member mark number shall be hard stamped on each section (subject to the requirements of 2.2.5.2), in a designated location not less than 500 mm from an end; and

(b) As-built shop drawing member schedules and assembly drawings, including section size, grade, heat number, and mill certificates shall be certified by the contractor and lodged with the owner of the structure.

2.3 Fasteners

2.3.1 Steel bolts, nuts, and washers

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

(a) AS 1110;
(b) AS 1111;
(c) AS 1112:Part 1;
(d) AS/NZS 1252; and
(e) AS/NZS 1559.

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 4.2.4.1.1;

(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; and

(c) The method of tensioning and the inspection procedure for equivalent fasteners may differ in detail from those specified in 4.2.6 and 8.2 respectively, provided that the minimum fastener tension is not less than the minimum bolt tension specified in Table 12 and that the tensioning procedure is able to be checked.

2.3.3 Welds

All welding consumables and deposited weld metal shall comply with 3.2.3.

2.3.4 Welded studs

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

2.3.5 Explosive fasteners

All explosive fasteners shall comply with, and shall be installed in accordance with AS/NZS 1873 (Parts 1 to 4).

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

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

C2.6

The provisions of 2.6 are drawn from AS 4100 with minor modifications. Brittle or fast fracture may occur when a critical combination of the following conditions exist:

(a) A stress raising notch or crack in the presence of active or residual tensile stress;

(b) Low fracture toughness or Charpy V-Notch energy absorption of parent metal or welds. This is related to a number of variables including steel thickness, chemical composition, and density of non-metallic particles. In welds this is strongly linked to the effectiveness of shielding gases during welding;

(c) Cyclic plastic deformation occurs degrading the CVN characteristics of the steel, increasing its transition temperature (see 2.7(a)).

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 rational fracture assessment method.

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, and 0.

2.6.2.3

The steel grade shall be selected to conform with the required steel type in accordance with 2.6.4.

2.6.2.4

The welding consumables shall be selected in accordance with 3.2.3.

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 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*

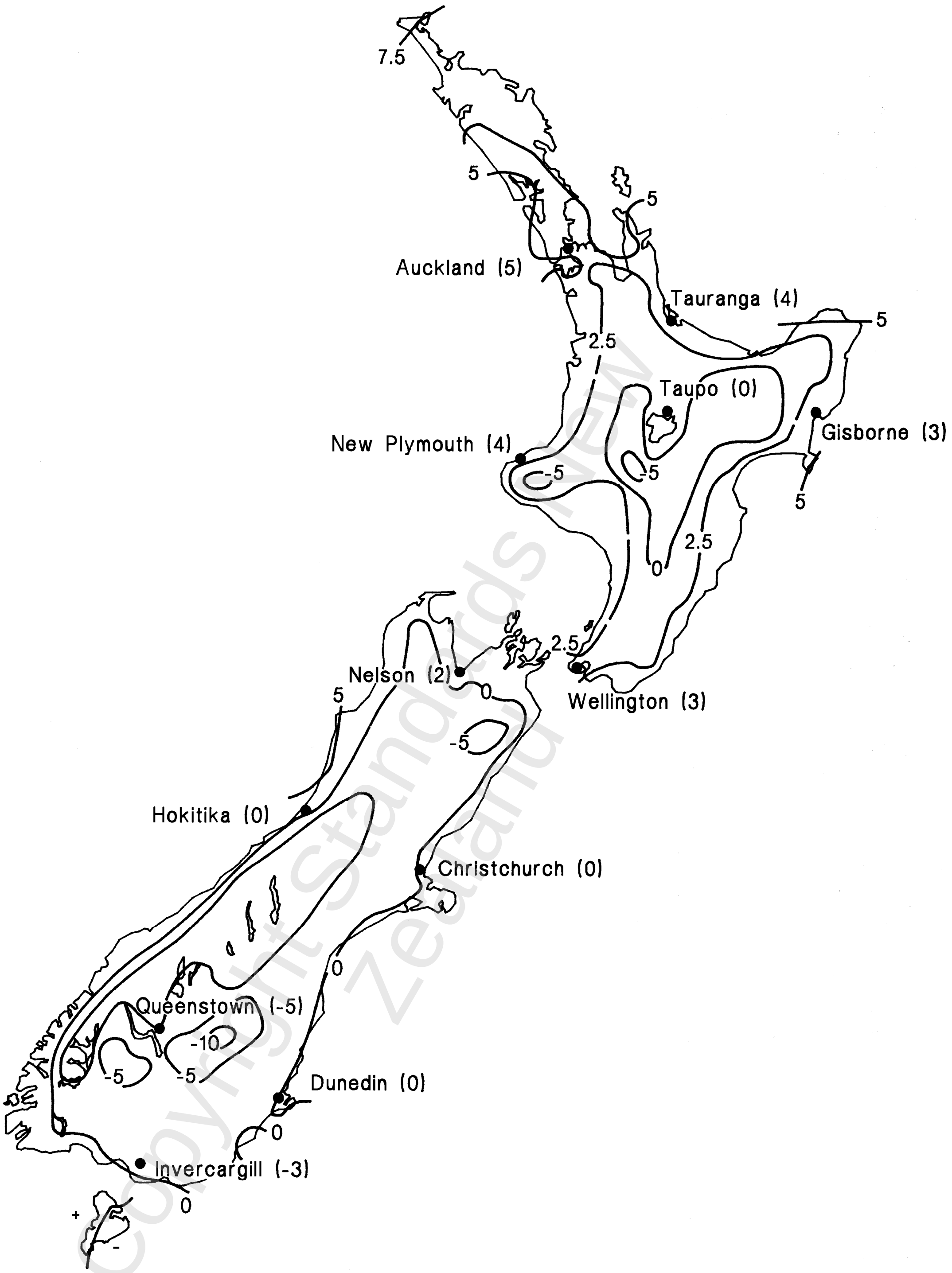
The selection of the design service temperature shall also make allowance for the following conditions:

(a) For structures which may be subject 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; and

(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.



(1) The isotherms show the LODMAT 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 1.

Figure 1 – LODMAT isotherms

2.6.4 Material selection

C2.6.4

Table 5 is based on statistical data available for the notch toughness characteristics of Australian-made steel of any grade and range of thicknesses (see 2.7(b) to (f)). These references refer to investigations on a previous generation of steel products, however current manufacturing processes in Australia are believed to result in the same or improved performance (see 2.7(g)). Limited testing of rectangular hollow sections of Australian and New Zealand origin (see 2.7(h)) has shown 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.

2.6.4.1 *Selection of steel type*

The steel type for the material thickness shall be selected from Table 5 so that the permissible service temperature listed in Table 5 is lower than the design service temperature determined in accordance with 2.6.3. The permissible service temperatures listed in Table 5 shall be subject to the limitations and modifications specified in 2.6.4 and 2.6.4.3 respectively.

2.6.4.2 *Limitations*

2.6.4.2.1

Table 5 shall only be used without modification for members and components which comply with the fabrication and erection provisions, and with the provisions of AS/NZS 1554.1 or AS/NZS 1554.5 as appropriate.

2.6.4.2.2

Table 5 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 from weld distortion shall be disregarded.

Table 5 – Permissible service temperatures according to steel type and thickness

<table>
<tr><th rowspan="3">Steel type
(Table 2)</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>2S</td><td>–35</td><td>–25</td><td>–15</td><td colspan="3">–5</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>5S</td><td>–35</td><td>–25</td><td>–15</td><td colspan="3">–5</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>
<tr><td colspan="7">NOTE –
Table 5 applies for:
(a) Elements of a member or connection component subject to tensile stress; and
(b) Plates or sections, the flange thickness being used for the latter.</td></tr>
</table>

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 subject 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 5.

NOTE – Disregard local strain due to weld distortion.

2.6.4.3.2 *Steel used in category 1, 2 or 3 members (seismic applications)*
For category 1 or 2 members, the permissible service temperature for each steel type shall be increased by 10°C above the value given in Table 5.

For category 3 members, Table 5 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 5 by 20°C plus 1°C for every 1.0% increase in outer bend fibre strain above 10.0%.

NOTE – Disregard local strain due to weld distortion.

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 5.

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

C2.6.4.3.4

The modifications are based on plastic bending strain rather than uniaxial strain, as bending strain is more likely to occur during fabrication. The values of strain limits specified should be halved for cases of uniaxial strain. To determine plastic bending strain see 2.7(h). If stress relieving or normalising is to be undertaken, advice should be sought from the steel manufacturer and Design Engineer on the effects that either process may have on the material properties and those of the weldments.

2.6.4.4 *Selection of steel grade*

The steel grade shall be selected to match the required steel type given in Table 2.

2.6.4.5 *Selection of welding consumables*

See 3.2.3.2.

2.6.4.6 *Selection of bolts*

See 4.2.4.1.1.

2.7 References to section 2

(a) Hyland, C W K. *Assessment of ductile endurance of earthquake resisting steel members*. Ph.D Thesis, University of Auckland, 2008.

(b) George, T J, and Turner, C J. ‘The impact properties of AS 149 structural steel.’ *AWRA Bulletin* 1, no. 2 (1968): 13 – 23.

(c) Wade, J B. ‘Comparison of Australian steel to AS A149 and imported steels to BS 15.’ *AWRA Bulletin* 1, no. 4 (1968): 5 – 12.

(d) Wade, J. ‘The weldability of modern structural steels.’ Symposium on modern applications of welding technology in steel structures, University of NSW (1972): 55 – 88.

(e) Banks, E. ‘Notch toughness testing and the specification of steel to avoid brittle fracture.’ Proceedings, conference on steel developments, AISC, Newcastle (1973): 233 – 238.

(f) Banks, E E. 'A fracture assessment of the HAZ properties of Australian structural steels.' *Australian Welding Journal* 18, no. 5 (1974): 59 – 67.

(g) HERA. *Investigation of the brittle fracture resistance of cold-formed rectangular hollow sections.* HERA Report R4-39. Manukau City: HERA, 1987.

(h) Weng, C C, and White, R N. 'Cold-bending of thick high-strength steel plates.' *Journal of Structural Engineering*, *ASCE* 116, no. 1 (1990): 40 – 54.

3 FABRICATION

3.1 Material

3.1.1 General

C3.1.1

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 introduced during fabrication.

All material supply Standards specified in section 2 provide methods of removing surface defects from as-rolled steel.

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 from 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 (3.5(a)).

The steel used in risk areas should be selected for properties which minimise 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 3.5(a).

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 possibility of lamellar tearing occurring should be considered in welded moment-resisting beam to column connections involving thick column flanges (over 50 mm), and in welded column to base plate connections involving thick base plates (over 50 mm). In both cases the effect of lamellar tearing will be to limit the reliability of the connection to resist 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.

3.1.1.1

All material shall satisfy the requirements of the appropriate material Standard specified in 2.2, 2.3, 2.4, and 2.5.

3.1.1.2

For category 1, 2, or 3 members of seismic frames the steel shall comply with the requirements of 2.2.4.

3.1.1.3

Surface defects in the steel shall be removed using the methods specified in the appropriate Standards listed in 2.2.1.

3.1.2 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.

C3.1.2

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 among companies at this time.

The intention of this clause is to ensure that the identification is still evident during the fabrication process (that is, after cutting and so on).

The steel fabrication industry recognises the importance of identifying various items during processing and, as components are cut from mill material, an identifying number should always be applied. 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.

3.2 Fabrication procedures

3.2.1 Straightening, curving, and cambering

3.2.1.1

For general structures and non-fracture critical members (FCMs) in bridges, the local application of heat spots, mechanical cold working or heat assisted mechanical working, to introduce pre-camber or correct camber, sweep and out-of-straight, shall be subject to the limitations of the relevant material supply Standard and the requirements of the Design Engineer.

C3.2.1.1

Heat spots with temperatures > 600°C may be used to camber and straighten hot rolled steel plate and shapes, subject to appropriate care being taken to ensure the cooling rate of the steel. Cooling should be controlled in a manner similar to that for welding in accordance with AS/NZS 1554.1, appropriate to the steel type and thickness. This will avoid undesirable changes to the material properties of the steel and crack development.

Guidance on curving structural steel members may be found in 3.5(b).

Where heat is used temperatures should be checked using temperature-indicating crayons or a pyrometer.

The application of heat to cold-formed steel tubes and pipes will locally reduce the nominal yield stress of the section, affecting structural capacity.

Cold working of steel without stress relieving heat treatment will result in a reduction of ductility and an increase its Charpy V-Notch nil ductility transition temperature. Particular consideration of these effects should be given for members of seismic resisting frames. AS/NZS 3678 sets limitations on the internal bend radii of hot rolled flats and plate.

The application of stress relieving heat at temperatures < 600°C may result in unwanted changes in shape and curvature of a steel assembly, with the release of localised residual stresses caused by welding and cold working.

3.2.1.2

For FCMs in bridges mechanical cold working shall not be used. The temperature of heated areas shall not exceed 600°C. Finished FCMs constructed of as-rolled and normalised steels may be heated to produce slight reductions in excessive camber or to produce a curvature equivalent to a circular radius not less than 300 m.

C3.2.1.2
For FCMs in bridges, the use of heat above 600°C to assist working is prohibited, to further minimise the risk of such effects occurring and initiating fatigue crack development.

3.2.1.3

Precamber specified for sections shall be fabricated to a tolerance of ± 5 mm.

C3.2.1.3
The achievement of precamber is an inexact process that can be time consuming. It is therefore not recommended to specify precambering of hot rolled sections of less than 20 mm. For a built-up section the lower achievable limit is 10 mm.

3.2.2 Cutting

3.2.2.1

Cutting may be by sawing, shearing, cropping, machining, and thermal cutting processes, as appropriate.

3.2.2.2

Shearing of items over 16 mm thick shall not be carried out when the item is to be galvanised and subject to tensile force or bending moment unless the item is stress relieved subsequently.

3.2.2.3

In areas designated as yielding regions, sheared edges are not permitted. Material prepared by shearing shall be sheared 3 mm oversize with the excess material subsequently removed by machining.

For highway and railway bridges:

(a) Edges and ends of plates shall be cut to size by thermal cutting. Universal mill and sheared plates shall have a minimum of 5 mm of materials removed from rolled or sheared edges and from ends by thermal cutting prior to assembly and welding. This provision shall not apply to edges of bars and shapes in accordance with AS/NZS 3679.1, or the ends of stiffeners and connection plates where there is no calculated gross tensile stress;

(b) No visually detected discontinuities shall be inspected further using magnetic particle (MT) inspection. Laminar discontinuities shall be allowed in the fusion face of groove welds in butt joints subject to gross tensile stress normal to the weld axis, or in the sides (edges) of contiguous parent metal within 300 mm of such welds; and

(c) Parent metal at the fusion face of groove welds in butt, T, and corner joints may have discontinuities as allowed by AS/NZS 3679.1 for hot-rolled sections, AS/NZS 3679.2 for welded sections and AS/NZS 3678 for plate.

3.2.2.4

Any cut surface not incorporated in a weld shall have a roughness not greater than the appropriate value given in Table 6. A cut surface to be incorporated in a weld shall comply with AS/NZS 1554.1.

Table 6 – Maximum cut surface roughness

Application	Maximum roughness (CLA) (μm)
Normal applications, that is, where the face and edges remain as-cut or with minor dressing (see Note 1)	25
Fatigue applications (Detail categories)	
detail category ≥ 80 MPa	12
detail category < 80 MPa	25
Yielding regions of category 1, 2, or 3 members	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 3.5(c).

(3) CLA = Centre-line average method (see AS 2382).

3.2.2.5

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.

Steelwork with surface specific atmospheric corrosivity D and E, determined in accordance with 5.2.1, shall have cut edges and corners chamfered or dressed to a minimum radius of 1.5 mm.

C3.2.2.5

Radiusing cut edges gives improved paint build and is recommended in all cases.

3.2.2.6

Notches and gouges, not closer spaced than 20*t* (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 or AS/NZS 1554.5 as appropriate.

For railway bridges, welding shall not be permitted to repair notches and gouges.

3.2.2.7

A re-entrant corner shall be shaped notch free to a radius of at least 10 mm.

C3.2.2.7

Information on the flame-cutting of steel may be found in 3.5(c), 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 have centre-line-average values (CLA) defined 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, and 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. AS 2382 also provides guidance.

The restriction on the shearing of plates thicker than 16 mm when a part is to be galvanised, relates to strain-age embrittlement when severe cold-working is followed by galvanising, resulting in a loss of ductility. Stress-relieving before galvanising means the 16 mm limit can be waived. This clause requirement is based on data from galvanisers (3.5(d)).

The restriction on shearing of plates of any thickness in yielding regions of category 1, 2, or 3 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 6.

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 3.2.2.6.

3.2.3 Welding

Welding and welding consumables shall comply with AS/NZS 1554.1 or AS/NZS 1554.5 as appropriate and welding of studs shall comply with AS/NZS 1554.2. For highway and railway bridges only manual metal arc welding (MMAW), shielded metal arc welding (SAW), flux cored arc welding (FCAW), and gas metal arc welding (GMAW) welding processes shall be used to construct or repair FCMs.

C3.2.3

A commentary on AS/NZS 1554.1 and AS 1554.2 is available in WTIA Technical Note 11 (3.5(e)). 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 (3.5(f)).

A welding quality management system (WQMS) is an important part of ensuring reliability in welded structures. It is therefore recommended that a WQMS appropriate to the importance level of the structure be implemented. Importance levels have been derived from AS/NZS 1170.0 with a modification for residential houses reflecting LBP site supervision limits. An appropriate WQMS should generally conform to the requirements set out in the table below.

The selection of consumables in accordance with AS/NZS 1554.1 and AS/NZS 1554.5 will ensure appropriate matching of mechanical properties to those of the parent steel type.

GMAW performed using dip-transfer is prone to lack of fusion, and care needs to be taken to verify adequacy of welding procedures and fusion using NDT.

Recommended welding quality management system

<table>
<tr><th>Importance level</th><th>Typical examples</th><th>WQMS</th></tr>
<tr><td>Low</td><td>Farm buildings, isolated structures, towers in rural situations, fences, masts, residential houses less than 3 storeys</td><td>Elementary (AS/NZS ISO 3834-4)</td></tr>
<tr><td>Medium</td><td>Buildings of medium importance not included in Importance Level 1, 3, or 4</td><td rowspan="2">Standard (AS/NZS ISO 3834-3)</td></tr>
<tr><td>High</td><td>Buildings and facilities as follows:
(a) Where more than 300 people can congregate in one area;
(b) Day care facilities with a capacity greater than 150;
(c) Primary school or secondary school facilities with a capacity greater than 250;
(d) Colleges or adult education facilities with a capacity greater than 500;
(e) Healthcare facilities with a capacity of 50 or more resident patients;
(f) Airport terminals, principal railway stations with a capacity greater than 250;
(g) Correctional institutions;
(h) Multi-occupancy residential, commercial (including shops), industrial, office and retail buildings designated to accommodate more than 5000 people and with a gross area greater than 10,000 m^2; and
(i) Public assembly buildings, theatres, and cinemas of greater than 1000 m^2.
Emergency medical and other emergency facilities.
Power generating facilities, water treatment and wastewater treatment facilities, and other public utilities.
Buildings and facilities containing hazardous materials.
Highway and rail bridges.</td></tr>
</table>

Summary comparison of welding quality management system requirements based on AS/NZS ISO 3834 Parts 3 and 4

Importance level	**Medium and high**	**Low**
WQMS	***Standard***	***Elementary***
Contract review	*Less extensive review*	*Establish that capability and information is available*
Design review	*Design for welding to be confirmed*	
Welders, operators	*Approved in accordance with AS/NZS 1554.1*	*Responsibility of contractor*
Welding coordination	*Welding coordination personnel with appropriate technical knowledge*	*Not required but personal responsibility of manufacturer*
Inspection personnel	*Sufficient and competent personnel to be available*	*Sufficient and competent, access for third parties, as needed*
Production equipment	*Required to prepare, cut, weld, transport, and lift together with safety equipment and protective clothes*	*No specific requirements*
Equipment maintenance	*No specific requirements, shall be adequate*	*No requirements*
Production plan	*Restricted plan necessary*	*No requirements*
Welding procedure specification	*In accordance with AS/NZS 1554.1*	
Welding procedure approval	*In accordance with AS/NZS 1554.1 and the requirements of the Design Engineer*	
Work instructions	*WPS or dedicated work instructions to be available*	*No requirements*
Batch testing of consumables	*No requirements*	
Storage and handling of welding consumables	*According to supplier's recommended minimum*	
Storage of parent materials	*Protection required from influence by the environment; identification shall be maintained*	*No requirements*
Post-weld heat treatment	*Conform to specification*	
Inspection before, during, after welding	*As required for specified operations*	*Responsibilities as specified in contract*
Non-conformances	*Procedures shall be available*	
Calibration	*If required*	*No requirements*
Identification during process		
Traceability		
Quality records	*Shall be available to meet the rules for product liability*	*As required by contract*
	Retained for five years minimum	

A comprehensive welding quality management system such as AS/NZS ISO 3834.2 is recommended when welding fatigue detail classifications (see Table 7 and Table 8) are required to conform with the quality requirements of AS/NZS 1554.5.

3.2.3.1 *Welding quality for fatigue applications*

3.2.3.1.1 *General structures*

The welds in the welded details given in Table 7 and Table 8 for detail classifications 112 and below shall have quality conforming with Category SP. The welds in the welded details given in Table 7 for detail classification 125 and above shall have a weld quality conforming to that defined in AS/NZS 1554.5.

3.2.3.1.2 *Railway bridges*

The welds in the welded details given in Table 7 and Table 8 for detail classifications below 71 shall have quality conforming with Category SP. The welds in the welded details given in Table 7 for detail classifications 71 and above shall have a weld quality conforming to that defined in AS/NZS 1554.5.

C3.2.3.1.2

The requirements for railway bridges are consistent with AS 5100.6. The fatigue life of detail classifications 90 or lower may be increased by applying post weld improvement techniques described in 3.5(g).

3.2.3.2 *Welding consumables for earthquake resisting structures*

For welds subject to earthquake loads or effects, 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 2S for Grade 300 steel, Steel Type 5S for Grade 350 steel and Steel Type 7C for Grade 450 steel;

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

3.2.3.3 *Weld access holes*

Weld access holes shall have a length from the toe of the weld preparation not less than 1.5 times the thickness of the material in which the hole is made. The height of the access hole shall be not less than 1.5 times the thickness of the material, and not less than 25 mm nor greater than 50 mm. Cut surfaces of the access hole shall comply with 3.2.2.4. No arc of the weld access hole shall have a radius less than 10 mm. Weld access holes in sections with elements thicker than 50 mm shall be ground to bright metal and inspected for cracks by either magnetic particle or dye penetrant methods.

3.2.3.4 *Welding of continuity stiffeners in earthquake resisting members*

Corners of continuity stiffener plates placed in the webs of rolled sections with elements greater than 32 mm thick shall be clipped to avoid the *k*-areas (see Figure 2) as follows. Along the web, the clip shall extend a distance of 35 mm beyond the tangent of the web-to-flange radius. Along the flange, the clip shall extend 12 mm beyond the tangent of the web-to-flange radius. The welds to the continuity stiffener shall be terminated 5 mm back from the clipped corners.

C3.2.3.4

The provisions are intended to avoid welding in the k-area and terminations in the section radii of hot rolled sections in highly restrained joints. This includes continuity plates in columns at moment connections and stiffeners in link beams. Where welding in the k-area cannot be avoided or has been done in error then it is recommended that the area is checked for post-weld cracking using MT or LP inspection. This is not intended to prevent welding of thin plates with small weld sizes near the k-area or in sections with thin flanges (refer to ANSI/AISC 341).

A limit is placed on welding consumables in joints subject to earthquake effects of 47J @ 0°C, by specifying a Ships' Classification Societies Grade 3 approval.

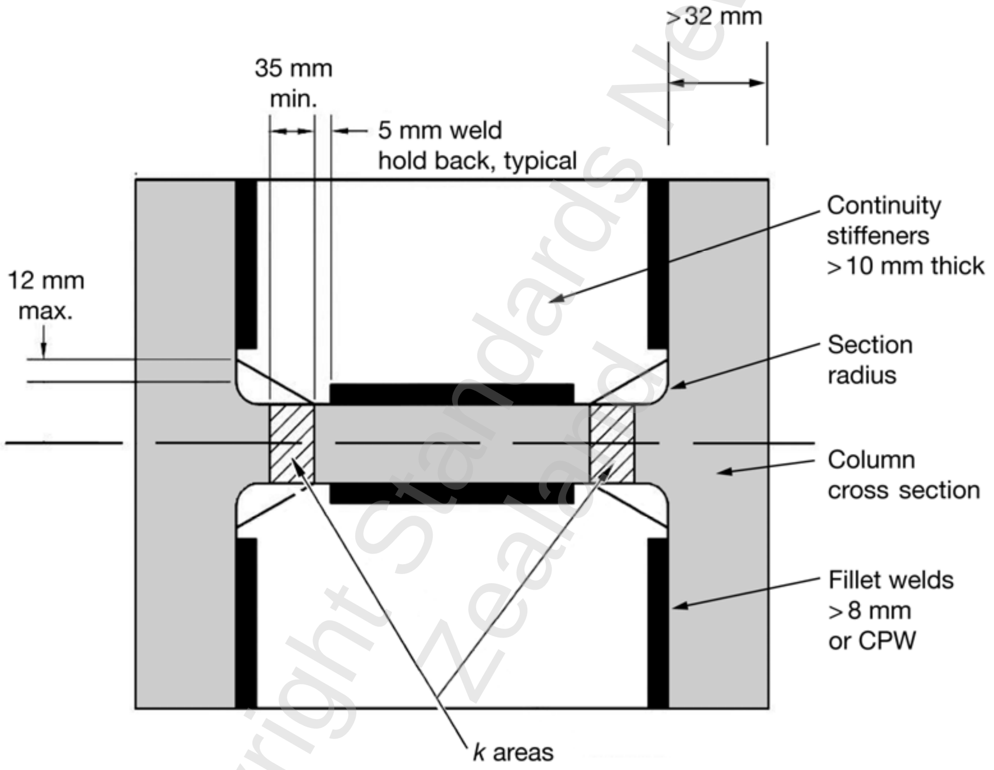


Figure 2 – Welding of continuity stiffeners in seismic members

3.2.4 Transition of thickness and width

3.2.4.1 *Non-seismic members*

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 Table 7.

The transition slope between the parts shall not exceed 1:1. However, the provisions of Table 7 require a lesser slope than this or a curved transition between the parts for some fatigue detail categories.

3.2.4.2 *Seismic members*

Members subject to earthquake loads or effects shall comply with the following:

(a) Transitions of thickness in members shall be undertaken in accordance with 3.2.4.1 for non-yielding regions in any category of member or for yielding regions in category 3 members;

(b) For yielding regions in category 1 or 2 members, 3.2.4.2 shall also apply, except that the thickness 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;

(c) An abrupt transition of width is permitted where it occurs outside a yielding region; and

(d) 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.

3.2.5 Holing

3.2.5.1 *General*

C3.2.5.1

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 not permitted by 3.2.5.1.3 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 (3.5(h)), at least for non-seismic applications.

The study in 3.5(h) 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; or

(f) Hand flame-cutting and reaming.

Plates investigated in the study were two 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 showed 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. Its use for corrective applications on site should be at the discretion of the construction reviewer in conjunction with agreement from the Design Engineer.

There are two reasons for placing a restriction on the use of full-size punching.

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;

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.

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 3.2.5.1.4. The restriction in this clause is in line with comparable overseas Standards.

A background to the theory of punching is given in 3.5(i). Studies (3.5(j)) 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:

(g) 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; and

(h) 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 and subject to earthquake.

The limit specified in 3.2.5.1.4 on the maximum thickness which can be punched recognises 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.

3.2.5.1.1

A round hole for a bolt shall only be either machine flame-cut, or drilled full size, or sub-punched 3 mm undersize and reamed to size, or punched full size.

3.2.5.1.2

A slotted hole shall be either machine flame cut, or punched in one operation, or formed by drilling two adjacent holes and completed by machine flame-cutting.

3.2.5.1.3

Hand flame-cutting of a bolt hole shall not be permitted except as a site rectification measure for holes in column base plates.

3.2.5.1.4

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.

3.2.5.1.5

For members and connections in fatigue sensitive members a punched hole shall only be permitted in material whose thickness does not exceed 12.0 mm.

3.2.5.1.6

All punched bolt holes in designated yielding regions of category 1, 2, and 3 members for seismic applications shall be punched 3 mm undersize and reamed to final size.

3.2.5.1.7

For railway bridges, holes shall not be machine flame-cut full size or punched full size on any members.

3.2.5.2 *Hole size*

C3.2.5.2

The nominal hole size specified by 3.2.5.2.1 is 2 mm or 3 mm larger than the bolt diameter, with the greater value for larger bolt diameters recognising 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 3.2.5.2.2 in order to assist in the erection of columns. This hole size is linked to the tolerance on anchor bolt locations given in 4.3.1.

The use of oversize and slotted holes in 3.2.5.2.3 and 3.2.5.2.4 follows American and previous New Zealand and Australian practice, based on research reported in 3.5(k).

3.2.5.2.1 *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.

3.2.5.2.2 *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.

3.2.5.2.3 *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 1.25 d_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 1.33 d_f or (d_f + 10) mm in length, whichever is the greater; and

(c) A long slotted hole shall not exceed the appropriate hole size of this clause in width and 2.5 d_f in length.

3.2.5.2.4 *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 effect; and

(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 irrespective of the 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, where the bolt can bear uniformly, and where the slot is normal to the direction of the load.

Table 7 – Fatigue detail classifications – Welded details to open sections

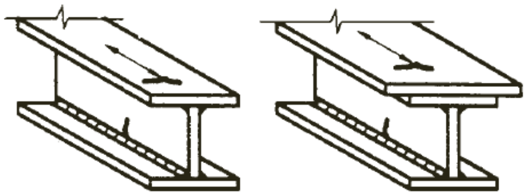
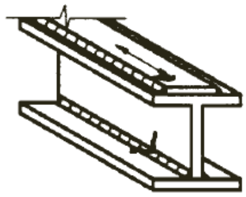
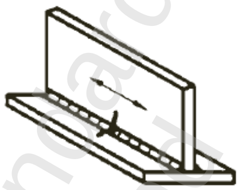
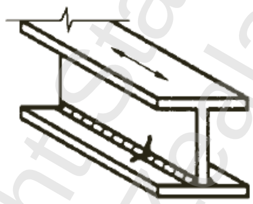
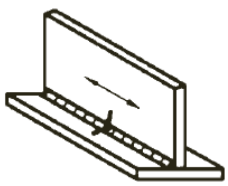
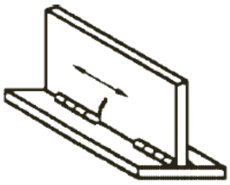
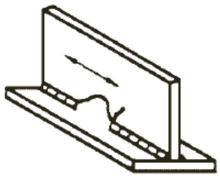
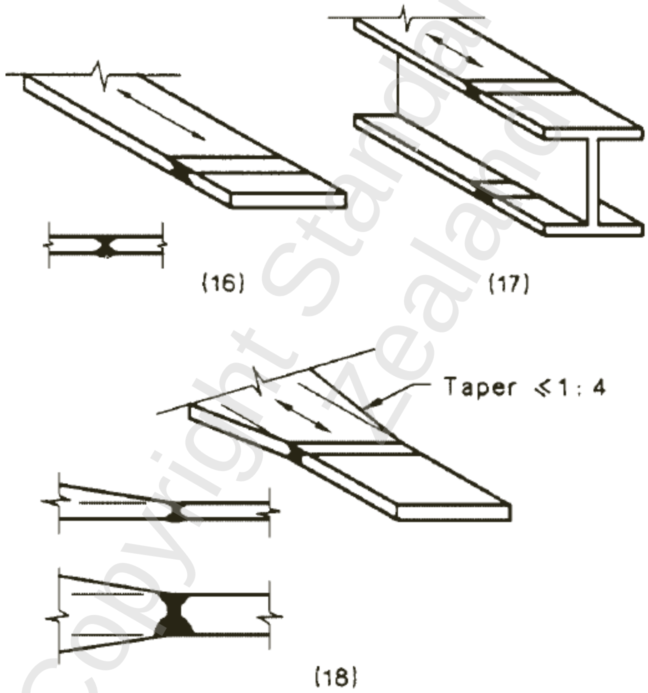
Detail classification	Construction details	
	Illustration	**Description**
125	 (8)  (9)	**WELDED PLATE I-SECTION AND BOX GIRDERS WITH CONTINUOUS LONGITUDINAL WELDS** (8) and (9) zones of continuous automatic longitudinal fillet or butt welds carried out from both sides and all welds not having unrepaired stop-start positions. NOTE – See 3.2.3.1 for 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.		

Table 7 – Fatigue detail classifications – Welded details to open sections (continued)

Detail classification	Construction details	
	Illustration	**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		**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% UT 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 overleaf)

Table 7 – Fatigue detail classifications – Welded details to open sections (continued)

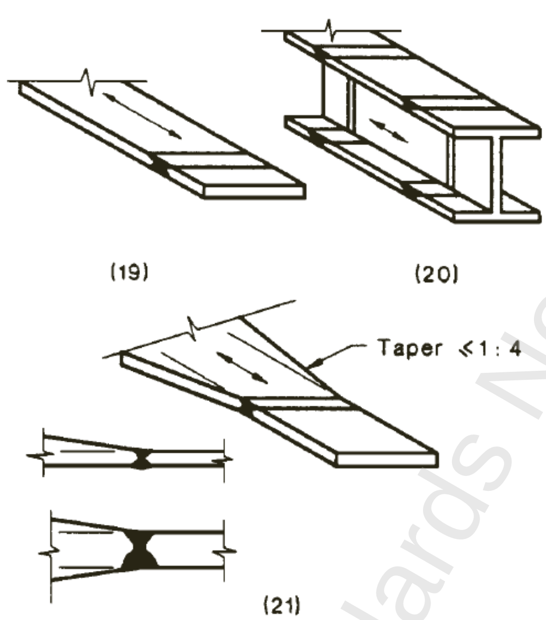
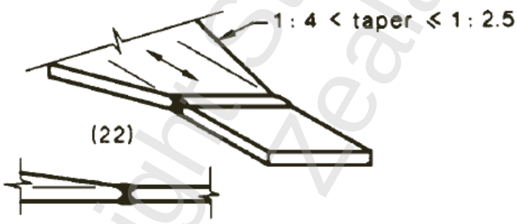
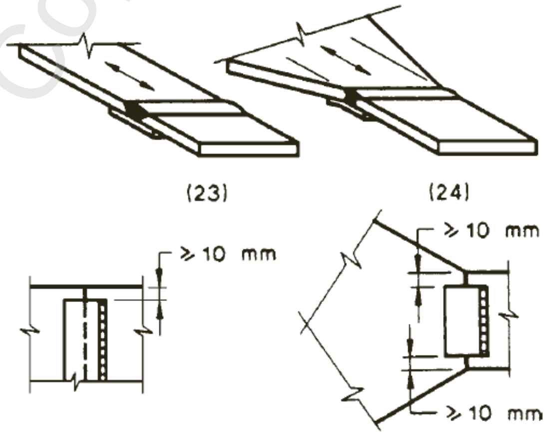
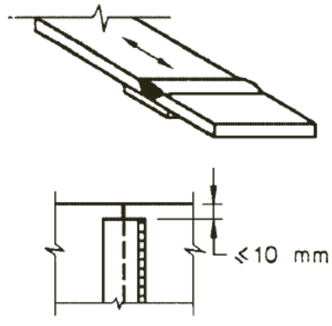
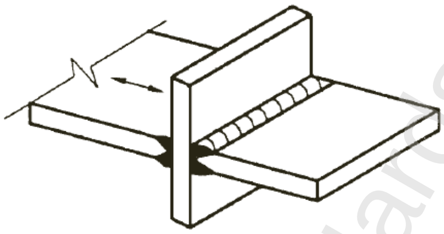
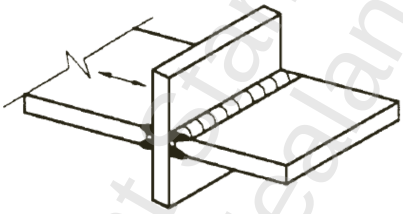
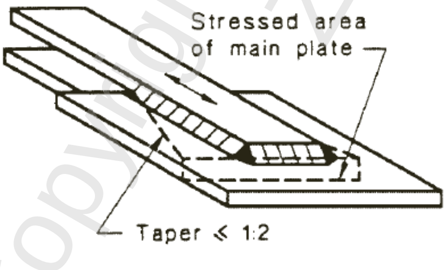
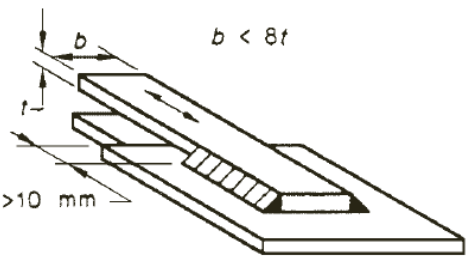
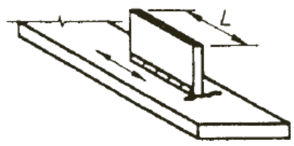
Detail classification	Construction details	
	Illustration	**Description**
90		**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from 2 sides. (19) transverse splices of plates, 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 shall 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.		

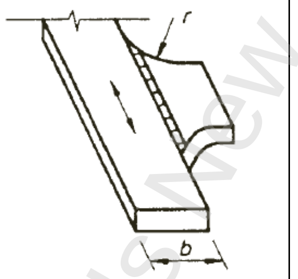
Table 7 – Fatigue detail classifications – Welded details to open sections (continued)

Detail classification	Construction details	
	Illustration	**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 UT inspected. 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 overleaf)

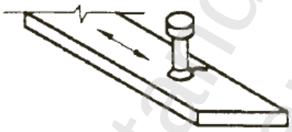
Table 7 – Fatigue detail classifications – Welded details to open sections (continued)

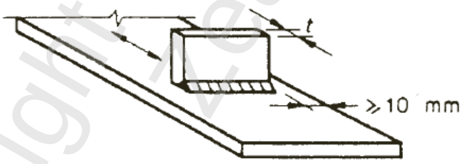
<table>
<tr><th rowspan="2">Detail classification</th><th colspan="4">Construction details</th></tr>
<tr><th colspan="3">Illustration</th><th>Description</th></tr>
<tr><td>90</td><td>(32)</td><td>(33)</td><td rowspan="6">

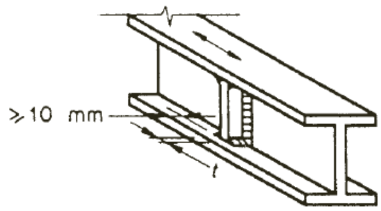
(32)


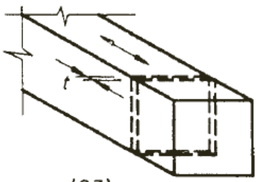
(33)</td><td rowspan="6">WELDED ATTACHMENTS (NON-LOAD CARRYING WELDS) LONGITUDINAL WELDS

(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.</td></tr>
<tr><td></td><td>–</td><td>$\frac{1}{3} \leq \frac{r}{b}$</td></tr>
<tr><td>80</td><td>L ≤ 50 mm</td><td>–</td></tr>
<tr><td>71</td><td>50 < L ≤ 100 mm</td><td>$\frac{1}{6} \leq \frac{r}{b} < \frac{l}{3}$</td></tr>
<tr><td>50</td><td>100 mm < L</td><td>–</td></tr>
<tr><td>45</td><td>–</td><td>$\frac{r}{b} \leq \frac{1}{6}$</td></tr>
<tr><td>80</td><td colspan="3">
(34)</td><td>WELDED ATTACHMENTS

(34) shear connectors on base material (failure in base material).</td></tr>
<tr><td>80</td><td>t ≤ 12 mm</td><td colspan="2" rowspan="2">

(35)


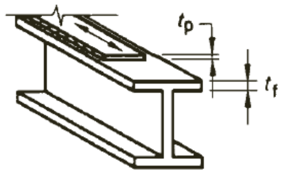
(36)


(37)</td><td>TRANSVERSE WELDS

(35) transverse fillet welds with the end of the weld ≥ 10 mm from the edge of the plate.</td></tr>
<tr><td>71</td><td>t < 12 mm</td><td>(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 action effects, the fatigue strength shall be determined using the stress range of the principal stresses.

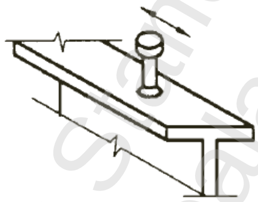
(37) diaphragms of box girders welded to the flange or web by continuous or intermittent welds.</td></tr>
<tr><td colspan="5">NOTE – The arrow indicates the location and direction of the stresses acting in the basic material for which the stress range is to be calculated on a plane normal to the arrow.</td></tr>
</table>

Table 7 – Fatigue detail classifications – Welded details to open sections

<table>
<tr><th rowspan="2">Detail classification</th><th colspan="3">Construction details</th></tr>
<tr><th colspan="2">Illustration</th><th>Description</th></tr>
<tr><td>50</td><td>t_r and $t_p \leq 25$ mm</td><td rowspan="2">

(38)</td><td rowspan="2">COVER PLATES IN BEAMS AND PLATE GIRDERS

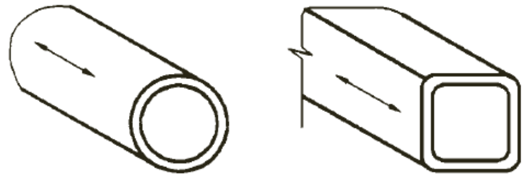
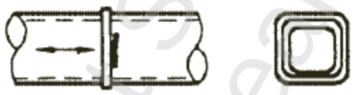
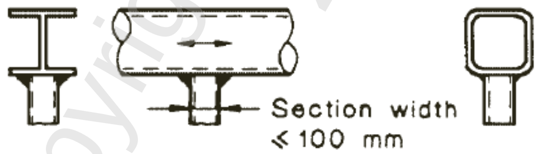
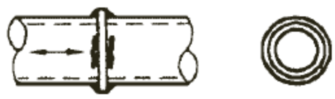
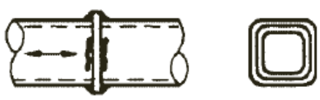
(38) end zones of single or multiple welded cover plates, with or without a weld across the end. For a reinforcing plate wider than the flange, an end weld is essential. (See Description 31 for the fatigue check in the weld itself).</td></tr>
<tr><td>36</td><td>t_r and $t_p > 25$ mm</td></tr>
<tr><td>80</td><td colspan="2">
(39)


(40)</td><td>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).</td></tr>
<tr><td colspan="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.</td></tr>
</table>

Table 8 – Fatigue detail classifications – Welded hollow sections

Detail classification	Construction details – Illustration	Construction details – Description
140	 (43)	**CONTINUOUS AUTOMATIC LONGITUDINAL WELDS** (43) no stop-starts, or as manufactured.
71 (with UT) 36 (no UT)	 (44)	**TRANSVERSE BUTT WELDS** (44) butt-welded end-to-end connection of circular hollow sections.
56 (with UT) 36 (no UT)	 (45)	(45) butt-welded end-to-end connection of rectangular hollow sections.
56 (t ≥ 8 mm) 50 (t < 8 mm)	 (46)	**BUTT WELDS TO INTERMEDIATE PLATE** (46) circular hollow sections, end-to-end butt welded with an intermediate plate.
50 (t ≥ 8 mm) 45 (t < 8 mm)	 (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 (t ≥ 8 mm) 40 (t < 8 mm)	 (49)	**FILLET WELDS TO INTERMEDIATE PLATE** (49) circular hollow sections, end-to-end fillet welded with an intermediate plate.
40 (t ≥ 8 mm) 36 (t < 8 mm)	 (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.

CTable 7 and Table 8

Detail classifications are consistent with IIW Recommendations (XIII-2151-07).

3.2.6 Pinned connection

Pins and holes shall be finished so that the forces are distributed evenly to the joint plies.

C3.2.6

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 3.5(l) and 3.5(m).

3.2.7 Full contact splices

3.2.7.1

If the ends of two butting lengths of a member, or the end of a member and the contact face of an adjoining cap plate or base plate, 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 (see Figure 3 (a)).

C3.2.7.1

The 1 mm maximum gap to a straight edge restriction 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 4.3.3 (see 3.5(n)).

Experience shows 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.

3.2.7.2

Full contact splices may be produced by cold-saw cutting or machining.

C3.2.7.2

Cold-sawing as a means of cutting members to length is permitted in addition to traditional machining (end-milling) methods. The modern cold-saw, as distinct from a band-saw, is a milling cutter and produces a surface finish equivalent to end milling.

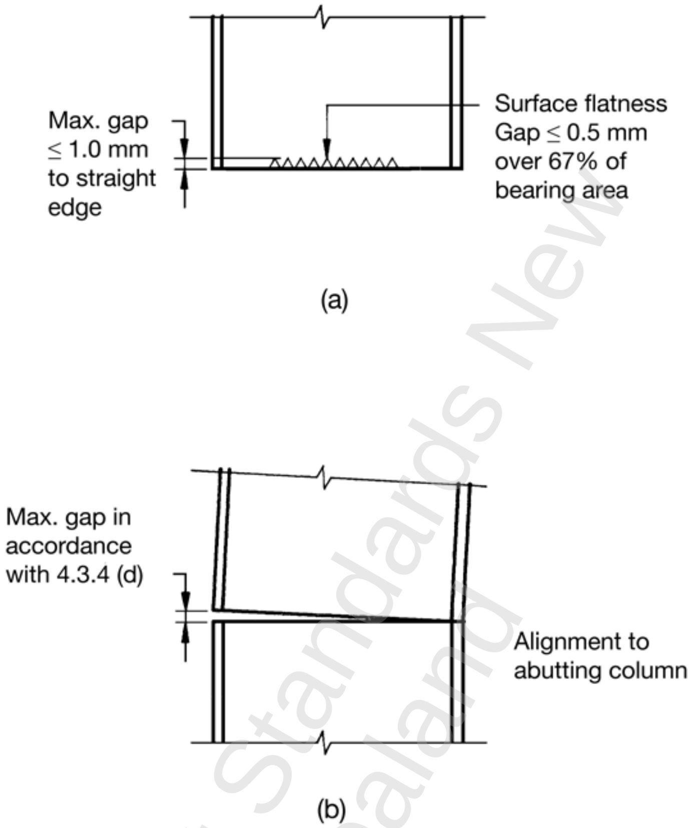


Figure 3 – Full contact splices

3.2.8 Highway and railway bridges welding, flame-cutting, and shearing procedure trials

When specified by the Design Engineer and before fabrication is commenced, welding, flame-cutting and shearing trials shall be carried out using representative samples of materials to be used in the work. The samples of materials shall be agreed with the Design Engineer.

C3.2.8

Where the Design Engineer wishes to exceed the requirements of this Standard the acceptance criteria should be clearly set out in the drawings or specification in accordance with 1.7.2.

AS/NZS 1554 provides for qualification of welding procedures using special test pieces to simulate particular conditions.

3.3 Tolerances

3.3.1 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.

C3.3.1

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 3.5(o) and compared to those in comparable overseas Standards.

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 to which proprietary steel sections are manufactured. Sections which comply with the tolerances specified 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 here.

It should be noted that tolerances as close as those specified in 3.3.1 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.

3.3.2 Notation

For the purpose of 3.3, the following notations apply:

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_{sx}	In-plane straightness of stiffener
Δ_{sy}	Out-of-plane straightness of stiffener
Δ_v	Deviation from verticality of a web at a support
Δ_w	Out-of-flatness of a web

3.3.3 Cross section

3.3.3.1 *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), (c), or (d) for depth, flange width, flange thickness, web thickness, out-of-square, and web off-centre.

3.3.3.2 *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 4)
for $d \leq 900$, ± 3 mm
for $900 < d \leq 1800$, $\pm\left[3+\frac{(d-900)}{300}\right]$mm
for $d > 1800$ + 8 mm, – 6 mm

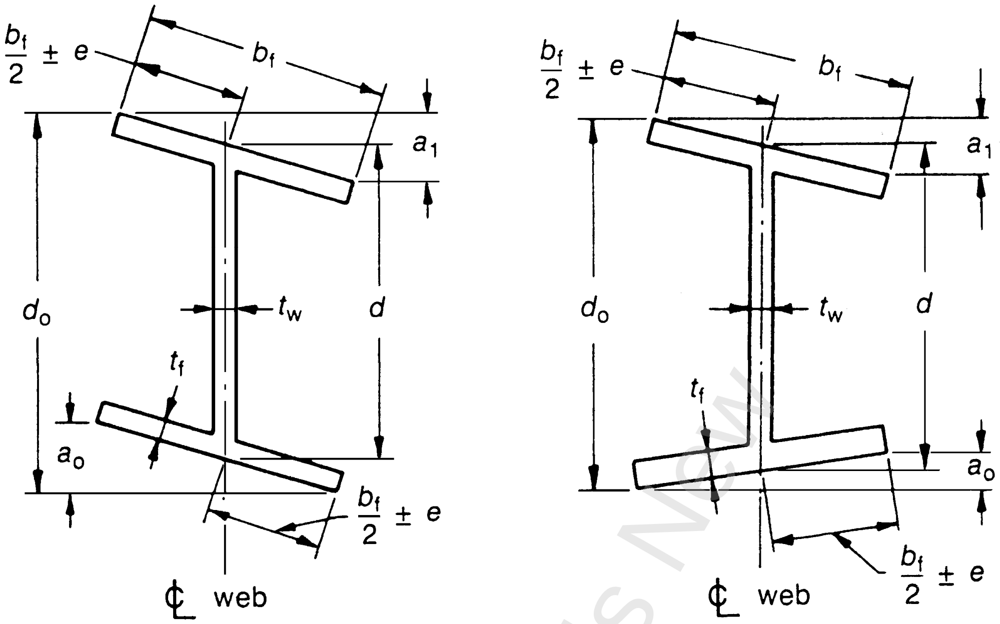
(b) Width of a flange (b_f) (see Figure 4)
for all b_f, ± 6 mm

(c) Out-of-square of an individual flange (a_0 or a_1) (see Figure 4)
for $b_f \leq 1000$ mm, ± 5 mm
for $b_f > 1000$ mm, $\pm\left(\frac{b_f}{200}\right)$mm

In the case of members supporting lightweight roofs
for $b_f < 210$ mm, ± 5 mm
for $b_f > 210$ mm, $\pm 1.5°$ off horizontal

(d) Total out-of-square of 2 flanges ($a_0 + a_1$) (see Figure 4)
80% of the maximum possible combined values in (c), that is,
for $b_f \leq 1000$ mm, ± 8 mm

(e) Pre-camber specified for built-up I-sections shall be fabricated to a tolerance of ± 5 mm.



NOTE –

(1) Dimensions d, d_o, a_o, and a_1 are measured parallel to the centre-line of the web. Dimensions b_f and ($0.5\ b_f \pm e$) are measured parallel to the plane of the flange.

(2) Dimension d is measured at the centre-line of the web.

Figure 4 – Tolerances on a cross section

(f) Out-of-flatness a of web (Δ_w) (see Figure 5)
$d_1/150$ 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;

(g) Deviation from verticality of a web at a support (Δ_v) (see Figure 5)

for $d \leq 900$ mm, ± 3 mm

for $d > 900$ mm, $\left(\frac{d}{300}\right)$ mm

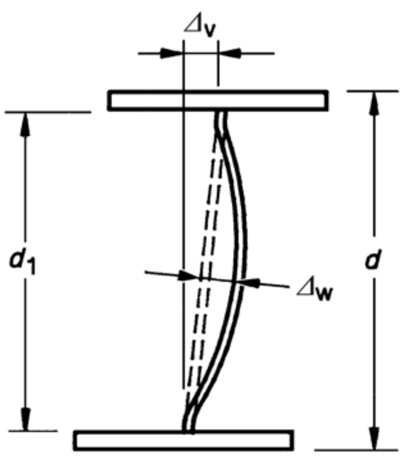


Figure 5 – Tolerances on a web

(h) Tolerance on shape of a built-up box section (see Figure 6).

A built-up box section shall not deviate at the diaphragm from the prescribed shape by more than ± 5 mm or ± [(a_2 + a_3)/400] mm, whichever is greater, unless connection requirements necessitate more stringent tolerances.

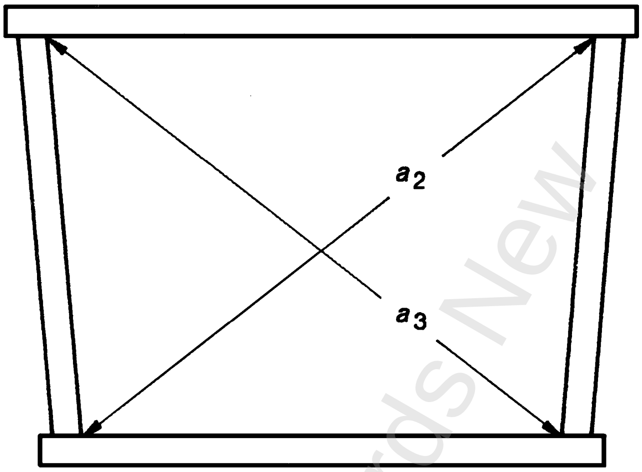


Figure 6 – Tolerance on shape of a box section

(i) Off-centre of a web (e) (see Figure 7) ± 6 mm. For transom top railway bridges ± 2 mm.

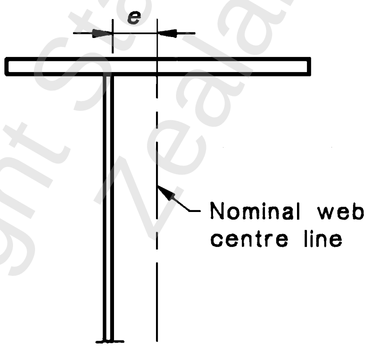


Figure 7 – Tolerance on off-centre of a web

(j) Out-of-flatness of a flange (Δ_f) (see Figure 8)
In general application

for $b_f \le 450$ mm $\quad \pm\left(\dfrac{b_f}{150}\right)$mm

for $b_f > 450$ mm $\quad$ ± 3 mm

In the case of members supporting lightweight roofs
for $b_f < 210$ mm $\quad$ ± 5 mm
for $b_f > 210$ mm $\quad$ ± 1.5° off horizontal

For railway bridges, flanges shall not pond water. Presetting of flanges may be required.

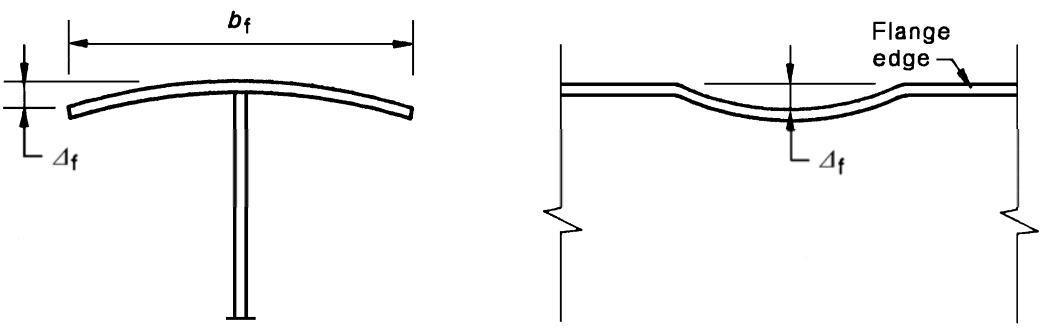


Figure 8 – Tolerance on out-of-flatness of a flange

C3.3.3

The tolerances on specified cross section dimensions for built-up sections given in 3.3.3.2 are based on the following:

(a) AS/NZS 3679:Parts 1 and 2 requirements for rolled and welded sections;

(b) American Institute of Steel Construction 'Quality criteria and inspection Standards'; and

(c) American Welding Society, 'Structural Welding Standard – Steel' AWS D1.1.

The tolerance on plate thickness should be as specified in AS/NZS 1365.

The tolerances in this clause are self-explanatory, and are readily measured on site. They are also the same as those specified in 3.5(p).

The tolerances for members supporting lightweight roofs are adapted from the Metal Building Manufacturers' Association (MBMA) Low Rise Buildings Systems Manual (3.5(q)), which specifies typical USA practice for such roofs.

3.3.4 Compression member

3.3.4.1 *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.

C3.3.4.1

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 (that is, to camber and sweep – see discussion in C3.3.5).

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.

3.3.4.2 *Length*

The length of a member shall not deviate from its specified length by more than ±2 mm.

C3.3.4.2

The length tolerance is consistent with the normal clearance in holes (2 mm or 3 mm, see 3.2.5.2.1), and with the normal tolerance on a welded butt joint in a compression member.

3.3.5 Beam

C3.3.5

The tolerances in Table 9 are compatible with those of AS/NZS 3679.1. Tolerances on pre-cambering are covered in 3.2.1.3 and 3.3.3.2(e).

The measurement of sweep is made with the web of the section to be tested vertical. Sweep is measured as illustrated in Figure 9. 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.

3.3.5.1 *Straightness*

A straight beam shall not deviate from a straight line drawn between the ends of the beam, after fabrication, by more than the following:

(a) Camber-measured with the web horizontal on a test surface (see Figure 9(a)). The tolerance on camber shall not exceed the limits in Table 9;

(b) For transom top railway bridges, the difference in camber between adjacent girders parallel to the girders shall not be greater than 6 mm;

(c) Sweep-measured with the web vertical (see Figure 9(b)). The sweep in plan shall not exceed the limits given in Table 9; and

(d) Twist measure with the web vertical (see Figure 9(c)). For girders directly supporting transoms in railway bridges, the twist deviation shall not be more than L/4,000,000, where L may also be the length between any two points on the beam.

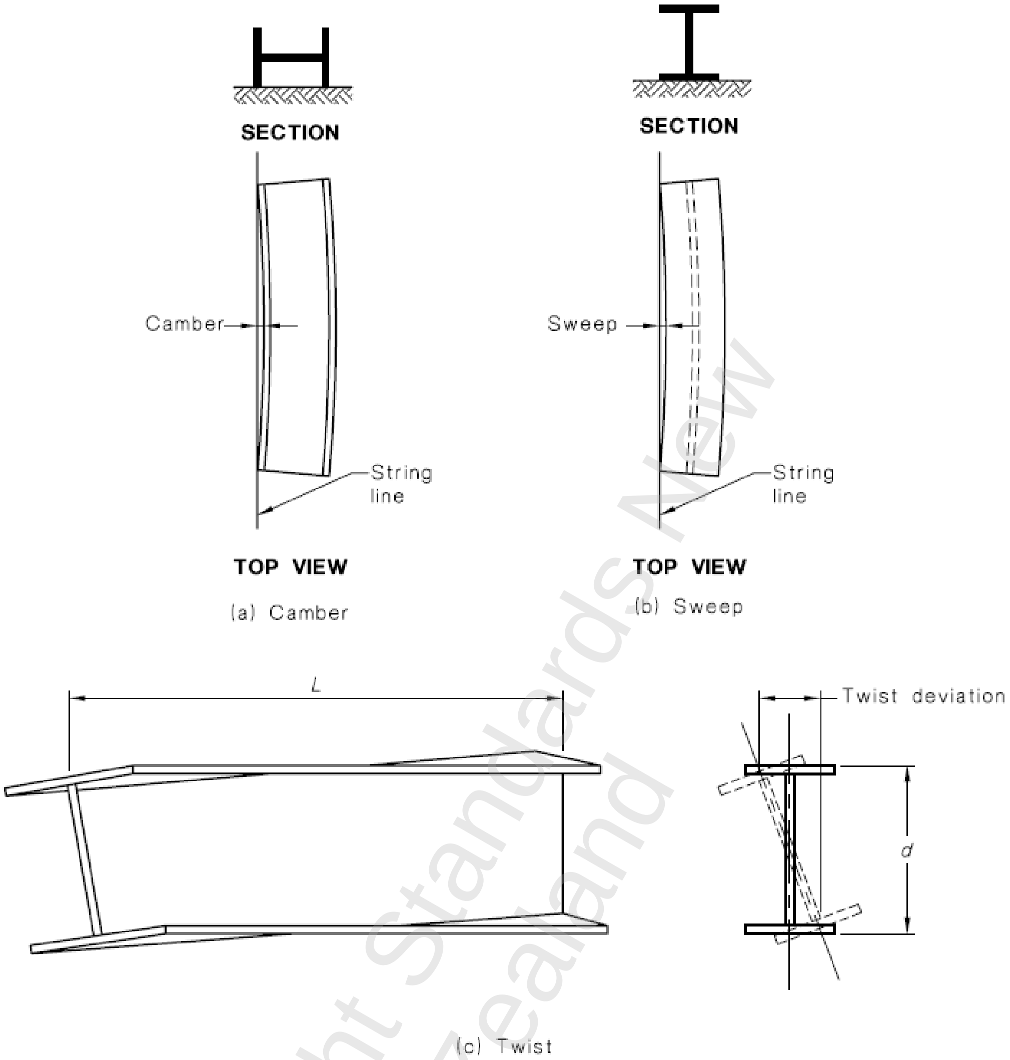


Figure 9 – Measurement of camber and sweep

Table 9 – Tolerance on straightness of beams after fabrication

Normal 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		
Length of 14 m and under	$\frac{\text{Length}}{1000}$ but no more than 10 mm	
Length over 14 m	$10\,\text{mm} + \frac{\text{Length (mm)} - 14\,000}{1000}$	
All other I-sections	$\frac{\text{Length}}{1000}$	$\frac{\text{Length}}{1000}$

NOTE – 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 3679.1 provides a means for measuring sweep.

3.3.5.2 *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.

3.3.6 Tension member

3.3.6.1 *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.

3.3.6.2 *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.

3.3.7 Tolerance for plate elements of highway and railway bridges

Plate panels in the web of plate and box girders in stiffened compression flanges and in box compression members shall not exceed the tolerances for flatness at right angles to the plate surface, measured parallel to the longer side in either direction in Table 10.

Table 10 – Tolerance for plate elements

Grade	*b*/*t*	Tolerance
250	>30	$\frac{G}{200}$ but not less than 3 mm
	≤30	No requirement
350	>25	$\frac{1.8\,G}{200}$ but not less than 3 mm
	≤25	No requirement

where

a = length of the longer side of a plate panel

b = length of the shorter side of a plate panel

G = gauge length

= *a*, where $a < 2b$

= $2b$, where $a > 2b$

> ***C3.3.7***
> *These tolerances are consistent with AS 5100.6.*

3.3.8 Tolerance for box girder compression flange and web stiffeners in highway or railway bridges

Longitudinal compression flange stiffeners in box girders and box compression members and all web stiffeners in plate and box girders shall meet the following tolerances:

(a) For in-plane straightness of stiffener, measured perpendicular to plate surface being stiffened

$$\Delta_{sx} = \frac{G}{750} \text{ or 2 mm, whichever is greater}$$

where the gauge length *G* is taken as *L*; and

(b) For out-of-plane straightness of stiffener, measured parallel to plate surface being stiffened

$$\Delta_{sy} = \frac{G}{450} \text{ or 2 mm, whichever is the greater}$$

where *G* is taken as 2*b* or *L*, whichever is the lesser

a, *b* = as specified in 3.3.7

Δ_{sx}, Δ_{sy} = maximum deviations from flatness within a specified gauge length, measure perpendicular to and parallel to the plate surface respectively

L = clear length of stiffener between adjacent transverse stiffeners, cross frames, cantilevers, or diaphragms.

> ***C3.3.8***
> *These tolerances are consistent with AS 5100.6.*

3.3.9 Checking of deviations in rolled and built-up sections for highway and railway bridges

Member/components of rolled and built sections shall be checked for compliance with the tolerance requirements of this Standard in accordance with Table 11. Fifty per cent of the checks shall be made in critical areas specified by the Design Engineer. The remainder of the checks shall be made in areas selected at random by the Design Engineer.

Table 11 – Tolerance examination and checking requirements

Member/component	Form of construction	Visual examination	Quantitatively checked
Plate panels in webs of plate and box girders in stiffened compression flanges and in box columns Longitudinal compression flange stiffeners in box girders, box columns and orthotropic decks. All web stiffeners in plate and box girders Cross girders, cross frames and cantilevers in orthotropic decks or in compression flanges of box girders or on all sides of stiffened box column	Plate and box girders Orthotropic decks All other forms	100% 100% 100%	5% 5% 10%
Columns and struts Webs of rolled sections in the regions of the internal supports of continuous beams and elsewhere (as shown on the drawings)	All forms	100%	100%

3.4 Rejection of a fabricated item

A fabricated item shall be liable to rejection if:

(a) The material does not satisfy the requirements of 3.1;
(b) The fabrication does not satisfy the requirements of 3.2; or
(c) It does not satisfy the tolerances specified in 3.3.

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 by any of the shortcomings in 3.4(a) to (c); or
(e) It passes testing in accordance with the appropriate clauses of section 4.

3.5 References to section 3

(a) Welding Technology Institute of Australia. *Control of lamellar tearing*, rev. ed. WTIA Technical Note 6. WTIA, 1985.

(b) Riviezzi, G. 'Curving structural steel members.' *AISC Journal of Steel Construction* 18, no. 3 (1984).

(c) Welding Technology Institute of Australia. *Flame-cutting of steels*, rev. ed. WTIA Technical Note 5. WTIA, 1994.

(d) Galvanising Association of New Zealand. *Hot dip galvanising*. 15th ed. Galvanising Association of New Zealand, 1999.

(e) WTIA-AISC-AWI. *Commentary on the structural steel welding code*, rev. ed. WTIA Technical Note 11. WTIA, 1992.

(f) Welding Technology Institute of Australia. *The weldability of steels*, rev. ed. WTIA Technical Note 1. WTIA, 1994.

(g) International Institute of Welding. *Post weld improvement of steel and aluminium structures*. Welding Recommendations PXIII – 18115-00.

(h) Iwankiu, N, and Schlafly, T. 'Effect of hole-marking on the strength of double lap joints.' *Engineering Journal*, American Institute of Steel Construction, 3rd Quarter (1982): 170 – 178.

(i) Brolund, T F. 'Theory of punching.' *Steel Fabrication Journal*, Australian Institute of Steel Construction 10 (1974): 2 – 6.

(j) Owens, G W, Driver, P G, and Kriye, G J. 'Punched holes in structural steelwork.' *Journal of Constructional Steel Research* 1, no. 3 (1981): 34 – 47.

(k) Allen, R N, and Fisher, J W. 'Bolted joints with oversize or slotted holes.' *Journal of the Structural Division*, ASCE 94, no. ST9 (1988): 2061 – 2080.

(l) Riviezzi, G. 'Pin connections for static structures.' *Steel Construction*, Australian Institute of Steel Construction, Vol. 19, No. 2 (1985): 2-7.

(m) Duerr, D, and Pincus, G. 'Pin clearance effect on pinned connection strength.' *Journal of Structural Engineering*, ASCE 112, no. 7 (1986): 1731 – 1736.

(n) AISC, *Code of standard practice for steel buildings and bridges*. American Institute of Steel Construction, 2005.

(o) Hogan, T J, and Firkins, A. 'Fabrication and erection provisions of AS 1250.' Proceedings, Third Conference on Steel Developments, Australian Institute of Steel Construction, 1985: 155 – 161.

(p) HERA. *HERA specification for the fabrication, erection and surface treatment of structural steelwork. HERA Report R4-99.* Manukau City: HERA,1998.

(q) MBMA. *Low rise buildings systems manual*. Ohio, USA: Metal Building Manufacturers Association, 1986.

4 ERECTION

4.1 General

4.1.1 Safety during erection

During the erection of a structure, steelwork shall be made safe against erection loading, including loading by erection equipment or its operation, and wind.

NOTE – Erection practices and procedures shall comply with AS 3828.

C4.1.1

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 and affect 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 AS 3828.

4.1.2 Erection method statement

Where requested an erection method statement shall be prepared by the contractor to address three principal safety objectives:

(a) To ensure individual pieces and the part-erected structure stands up throughout the construction stage;
(b) To operate cranes and other plant to lift and position safely; and
(c) To provide safe working positions for erectors and safe access to/egress from these positions.

To assist with the development of the erection method statement the Design Engineer shall provide:

(d) Details and locations of any temporary works provided for by the Design Engineer in the design;
(e) An outline of the method of erection envisaged by the Design Engineer, giving the sequence for erecting the structure taking into account any phasing of the works, including positions on the structure where temporary bracing, metal decking, or other restraints are needed to provide stability to individual members or the structure until walls, floors, or other non-steel structures are in position; and
(f) A description of any temporary works and any special requirements for temporary bracing required by the Design Engineer to comply with 4.1.2(e); the stage when it is no longer necessary, or whether it is to be left in position after completion of the steelwork.

C4.1.2
Further guidance on the preparation of an erection method statement can be found in AS 3828.

4.1.3 Equipment support

Equipment supported on partly erected steelwork shall not induce actions in the steel greater than the design capacities permitted in this Standard.

C4.1.3
No increase in the nominal capacity and no increase in the strength reduction factor are permitted under erection loading.

4.1.4 Reference temperature

Dimensions shall be set out on the basis of a reference temperature of 20°C.

C4.1.4
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°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°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.

4.1.5 Highway and railway bridges temporary erection at contractor's works

Where specified by the Design Engineer, steelwork shall be temporarily erected at the contractor's works to the Design Engineer's specification.

4.2 Erection procedures

4.2.1 General

4.2.1.1

The requirements specified in 3.2 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 3.2.7);
(b) Cutting (see 3.2.2);
(c) Welding (see 3.2.3);
(d) Holing (see 3.2.5); and
(e) Bolting (see 4.2.4).

4.2.1.2

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.

4.2.1.3

All connections for temporary bracing and members to be provided for erection purposes shall be made in such a manner 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 8.1.

4.2.2 Delivery, storage, and handling

4.2.2.1

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 minimise damage to the corrosion protection on the steelwork.

4.2.2.2

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.

4.2.3 Assembly and alignment

4.2.3.1

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 not distort the metal nor enlarge the holes.

4.2.3.2

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.

4.2.4 Bolting

4.2.4.1 *General*

C4.2.4.1

The projection of a bolt from the nut face is required by 4.2.4.1 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 4.2.6.1 is only possible if *full thread engagement is achieved.*

Clause 4.2.4.1.2 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 demand only 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 4.2.6.1 is equivalent to the proof load of the bolt, the likelihood of bolt loosening is low.

Holding-down bolts incorporating high strength property class 8.8 threaded rods, should be designed with consideration of the low level of elongation available and the potential for notch induced localisation of fracture. For example, rods conforming with B7 of ASTM A193M have a minimum elongation after fracture of 16%. Capacity design principles should be applied in assessing the design action effects on high strength holding-down bolts with application of upper bound elastic design actions associated with a structural performance factor, Sp = 0.9.

4.2.4.1.1
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.

C4.2.4.1.1
In low temperature situations special care should be taken in the selection of bolt properties with adequate fracture toughness. Yield point tensioning, that is the part turn method, may not be appropriate in some situations, with a reduction in fracture toughness that may occur.

4.2.4.1.2
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; and

(b) For snug-tightened bolts to 4.2.6.2, at least one clear thread run out shall be clear beneath the nut after tightening; or

(c) For tensioned bolts to either 4.2.6.2 or 4.2.6.3, the minimum number of clear threads run out beneath the nut after tightening shall be:
   (i) Five threads for a bolt length (see Table 5) up to and including 4 diameters
   (ii) Seven threads for a bolt length over 4 diameters but not exceeding 8 diameters
   (iii) Ten threads for a bolt length over 8 diameters.

4.2.4.1.3
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 for 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.

4.2.4.1.4

Hardened or plate washers shall be used under both the bolt head and nut for any slotted and oversize holes, as specified in 3.2.5.2.4.

C4.2.4.1.4

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 that would occur if either the bolt head or nut were 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 two washers, a tapered washer and a flat washer, with the flat washer placed under the rotating component.

4.2.4.1.5

Bolting categories 4.6/*S* and 8.8/*S* shall be installed to the snug-tight condition specified in 4.2.6.2(a).

4.2.4.1.6

The nuts used in a connection subject to vibration shall be secured to prevent loosening.

4.2.4.2 *Tensioned bolt*

A tensioned high strength bolt designated 8.8/TF or 8.8/TB, shall be installed in accordance with 4.2.5 and 4.2.6. The contact surfaces of a joint using a tensioned bolt shall be prepared in accordance with 4.2.4.3.

C4.2.4.2

A connection using a tensioned high-strength bolt should be identified in the contract documents as either of:

(a) Bearing-type, -/TB; or

(b) Friction-type, -/TF.

The tensioning methods for bolts and rods of property classes greater than 8.8 are not covered by this Standard.

4.2.4.3 *Preparation of surfaces in contact*

Preparation of surfaces in contact shall be as follows:

(a) *General and snug-tightened connection (-/S)*

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 4.2.6.2(a).

(b) *Friction-type connection (8.8/TF)*

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, galvanising, or other applied finish unless the applied finish has been tested in accordance with 8.3 to establish the friction coefficient or slip factor.

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 (8.8/TB)*

For a bearing-type connection, an applied finish on the contact surfaces shall be permitted without testing to establish the slip factor.

C4.2.4.3

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 burrs is only required if they prevent solid seating of piles 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 galvanising, paint or other finishes, where warranted by the appropriate service conditions. Clause 8.3 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.

4.2.5 Assembly of a connection involving tensioned bolts

4.2.5.1 *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.

C4.2.5.1

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.

4.2.5.2 *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 4.2.6.2 (a). All packing shall be steel with a surface condition similar to that of the adjacent plies.

C4.2.5.2

Since a reduction in slip factor can result from the assembly of two faces having different surface conditions, this clause requires that packing be of steel with the same surface condition as the contact faces.

4.2.5.3 *Tightening pattern*

4.2.5.3.1

Snug-tightening and final tensioning of the bolts in a connection shall proceed from the stiffest part of the connection towards the free edges.

4.2.5.3.2

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.

C4.2.5.3.2

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.

4.2.5.4 *Retensioning*

Retensioning of bolts that have been fully tensioned shall not be permitted.

C4.2.5.4

While this clause recognises the need to slacken 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 4.2.6, 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 retensioning.

Where special direct-tension indication fasteners are used, the fastener cannot be reused. Where load indicating washers are used, these must be replaced before retensioning a previously tensioned bolt.

Galvanised bolts cannot be satisfactorily retensioned due to the relatively soft zinc layer on the threads of the bolt and nut.

4.2.6 Methods of tensioning

4.2.6.1 *General*

C4.2.6.1

Bolts will often be tensioned beyond their proof loads, as the minimum bolt tension specified in Table 12 is approximately equal to the minimum proof load of the bolts.

If only hand tools are available, the correct size of podger spanner must be used to snug-tighten, while the final tightening must be undertaken by the use of a heavy duty ratchet wrench with a lever length of about 1 m for M20 bolts. Experience shows that when an air wrench is used for M20 bolts, an air pressure of at least 700 kPa at the wrench is recommended. An air wrench generally used to tighten M20 bolts may not be suitable for M24 bolts. A higher powered air wrench may be required. Experience shows for bolts larger than M24 that a hydraulic torque wrench is required. For galvanised bolts, AS/NZS 1252 requires that the galvanised nuts be supplied with a lubricant coating; this may be supplemented by lubricant on-site.

Hot-dip galvanised 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 galvanised 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 are very often 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 caused by 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.*

A general review of bolt tensioning procedures may be found in 4.6(a).

4.2.6.1.1

The method of tensioning shall be in accordance with either 4.2.6.2 or 4.2.6.3.

NOTE – 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.

4.2.6.1.2

In the completed connection, all bolts shall have at least the minimum bolt tension specified in Table 12 when all bolts in the bolt group have been tightened.

Table 12 – Minimum bolt tension for specialist 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 are approximately equivalent to the minimum proof loads given in AS/NZS 1252.	

4.2.6.2 *Part-turn method of tensioning*

Tensioning of bolts by the part-turn method shall be in accordance with the following procedure:

(a) Snug-tightening: 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.

The bolts shall be tightened by a few impacts of an impact wrench or by the effort of a person using a standard podger spanner. If the load-transmitting plies cannot be brought into effective contact with this amount of tightening effort then shim plates or remedial work to the connection will be required;

(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 13. During the final tensioning, the component not turned by the wrench shall not rotate.

Table 13 – 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 a half turn or less, one-twelfth of a turn (30°) over and nil under tolerance; for a two-thirds 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 13 will be at least equal to the minimum bolt tension specified in Table 12.

(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.

C4.2.6.2

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, two runs over the bolts are suggested to check the snug-tight condition, as the load-transmitting plies will be drawn in gradually, tending to loosen those bolts that 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 shown 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 13 are based on AISC values in 4.6(b), and reflect reduced rotation requirements in thin grips.

4.2.6.3 *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 three bolts for each diameter and class of bolt in a calibration device capable of showing bolt tension. The calibration test shall demonstrate that the device shows a tension not less than 1.05 times the minimum bolt tension specified in Table 12;

(b) On assembly, all bolts and nuts in the connection shall be first tightened to a snug-tight condition defined in 4.2.6.2(a); and

(c) After completing snug-tightening, the bolt shall be tensioned to provide the minimum bolt tension specified in 4.2.6.1.2. This shall be shown 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.

C4.2.6.3

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 12 (see 4.6(b)).

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. The incorporation of a tension indication device in the bolt-nut washer assembly may require some slight addition to the bolt length allowances.

4.3 Tolerances

C4.3
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 4.6(c) and compared to those in comparable overseas Standards. They are fully comparable with those specified in 4.6(d).

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.

4.3.1 Location of anchor bolts

C4.3.1
One of the greatest problems faced by a steel erector on site is inaccuracy in the locations of the anchor bolts.

The tolerances set in this revision are tighter than previously, reflecting improved surveying technology and requirements of 3D steelwork modelling and construction programming, and can be readily attained using precision surveying equipment and techniques. 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 3.2.5.2.2 with which these tolerances are compatible.

Several additional steps can be used to improve the site erector's position (4.6(e), sections 14, 15, 18 of 4.6(f)):

(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.

4.3.1.1

Anchor bolts shall be restrained in position both in a vertical and a horizontal direction during all setting-in operations.

4.3.1.2

Anchor bolts shall be set out in accordance with the erection drawings using precision surveying equipment and techniques where necessary. They shall not vary from the positions shown in the erection drawings by more than the following (see Figure 10 and Figure 11):

(a) 3 mm centre to centre of any 2 bolts within an anchor bolt group, for bolts rigidly cast-in, or 10 mm for bolts in sleeves, or connecting to base plates with details allowing 25 mm lateral adjustment. An anchor bolt group is defined as the set of anchor bolts which receives a single fabricated steel member;

(b) 2 mm centre to centre of adjacent grid lines;

(c) 6 mm from the centre of any anchor bolt group to the grid line through that group; and

(d) Column base plates > 20 mm thick or designated items shall have each holding-down bolt set out directly from its designated grid referenced location with ±3 mm tolerance (see Figure 11).

C4.3.1.2

For sleeve anchor bolts prepared for adjustment there should be 25 mm clearance at top of concrete. The length of sleeve needs to vary based on the bolt diameter if the anticipated degree of bolt adjustment is to be realised.

The setting out tolerances on fixed holding-down bolts have been tightened considerably in this revision. However the tolerances for sleeved holding-down bolts incorporating polystyrene void formers or other devices that allow some adjustment of bolt positioning after concreting have been relaxed. In some instances the setting out of the centre of holding-down bolt groups relative to grid lines may be relaxed by agreement, such as for simple buildings where timber members are measured and cut on site to connect all the steel elements.

The changes are in recognition of the significant improvements in surveying technology now available for building setting out work and the accuracy demanded by modern fabrication technology. The utilisation of three dimensional steelwork modelling packages and modern computer numerical control (CNC) fabrication has also significantly improved the accuracy of steelwork fabrication. In addition project timelines are often requiring steelwork to be erected shortly after holding-down bolts are installed reducing the opportunity to make changes to fabrication based on as-built surveys of holding-down bolts after concreting.

It is recommended that the Design Engineer and the contractor, in conjunction with the steelwork subcontractor, identify whether bolts will be cast-in as fixed or sleeved early in the contract. Where fixed bolts are to be used then precision surveying techniques and equipment should be used to set out the holding-down bolts. The setting out in such cases may be assigned to the steelwork subcontractor.

4.3.1.3

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.

4.3.1.4

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.

4.3.1.5

Drilled anchors shall only be substituted for cast-in holding-down bolts with the approval of the Design Engineer.

4.3.1.6

Compressive cavity former material around sleeved holding-down bolts shall be removed, unless specified otherwise by the Design Engineer, and the cavity appropriately grout-filled.

4.3.2 Column base

4.3.2.1 *Position in plan*

The position in plan of a steel column base shall not deviate from its correct value by more than 10 mm along either of the principal setting out axes (see Figure 12).

C4.3.2.1
The 10 mm tolerance is intended to provide for accurate positioning of the base and, in conjunction with plumbing tolerances, to allow reasonable limits which still enable the steel frame geometry to be held. These tolerances should assist in the subsequent fitting of other building elements such as precast or curtain wall facade panels to steel building frames.

4.3.2.2 *Level*

The level of the underside of a steel base plate shall not deviate from its specified value by more than ±10 mm.

4.3.2.3 *Full contact*

4.3.2.3.1
If full contact is specified, the requirements of 3.2.7 shall be satisfied, unless shims are used to reduce the measurable gaps to values specified in 3.2.7.

4.3.2.3.2
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.

C4.3.2.3
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.

4.3.3 Plumbing of a compression member

The alignment and plumbing of a compression member shall be in accordance with the following requirements:

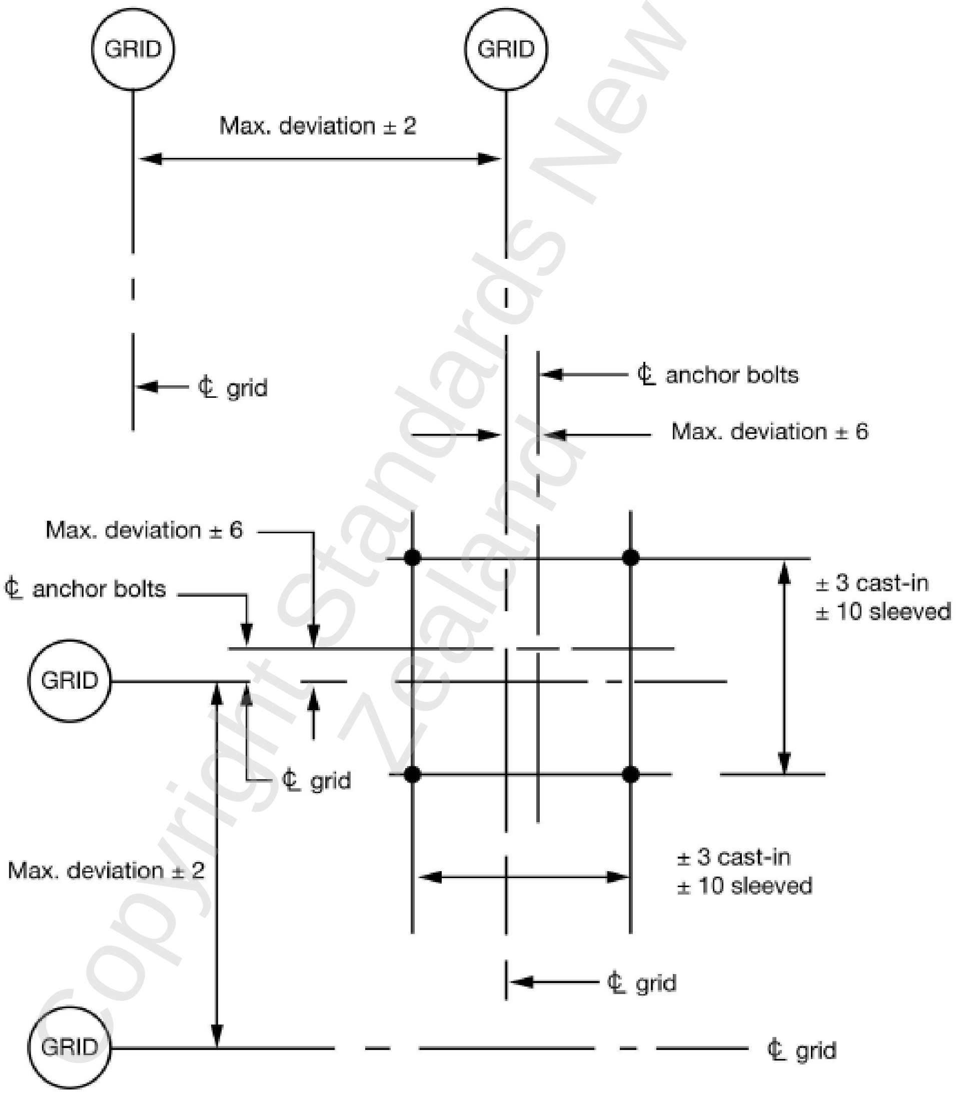
(a) The deviation of any point above the base of the compression member from its correct position shall not exceed height /500 or as follows (see Figure 12), whichever is the lesser:
   (i) For a point up to 60 m above the base of the member ……………….... 25 mm
   (ii) For a point more than 60 m above the 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; and

(c) In addition for columns adjacent to elevator shafts and crane gantry columns the deviation of any point above the base from its correct position shall not exceed height /1000.

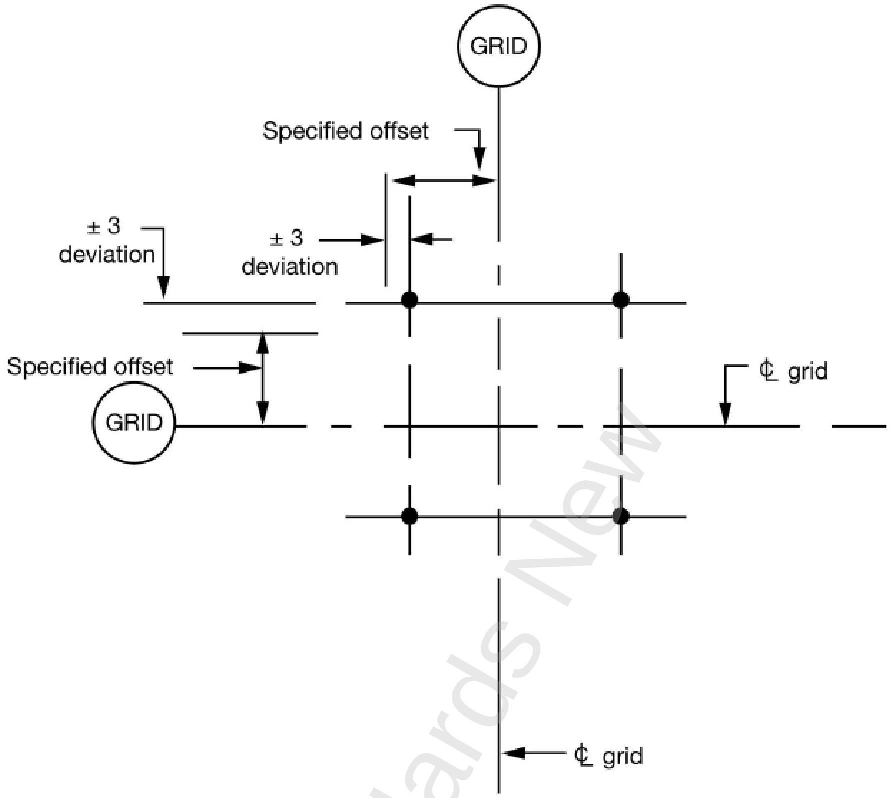
C4.3.3

Consideration should be given to adopting a tighter limit of height /1000 if tolerance sensitive cladding systems such as some curtain wall glazing systems are used. Consideration should also be given to specifying maximum deviation on plan of adjacent perimeter column members' working points for sensitive cladding systems.



All dimensions are in mm.

Figure 10 – Detail of off-centre location of anchor bolts for base plates ≤ 20 mm thick



All dimensions are in mm.

Figure 11 – Detail of deviation of anchor bolt locations for base plates > 20 mm or designated items

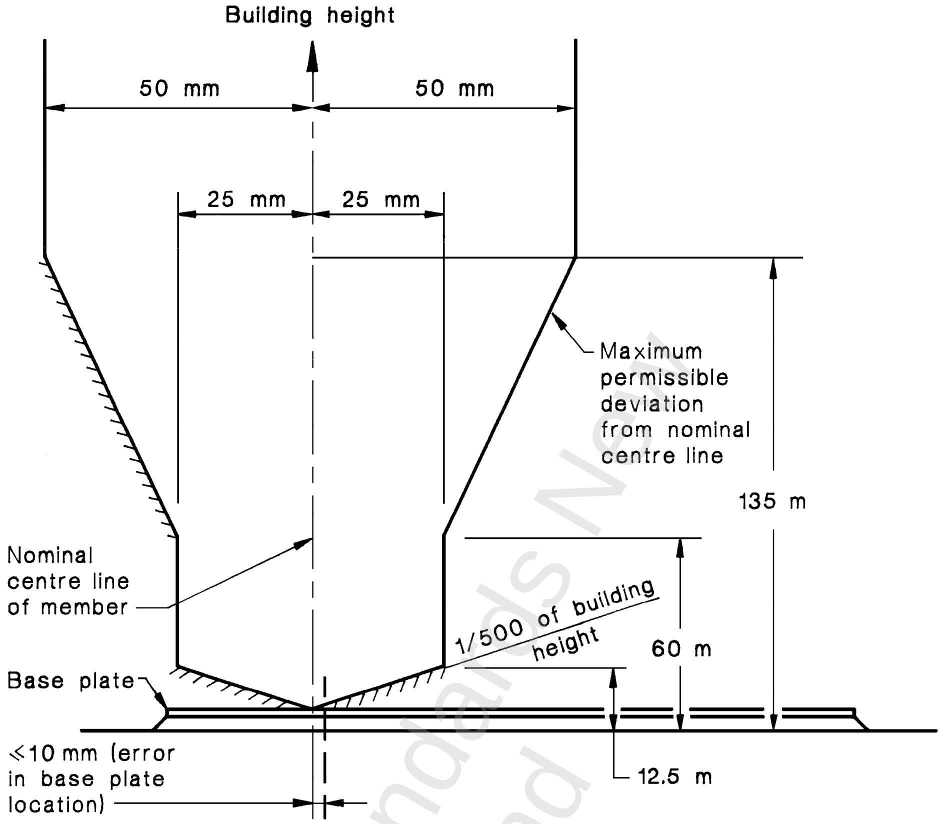


Figure 12 – Plumbing of a compression member

4.3.4 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 4.3.3 (see Figure 12);

(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 (see Figure 13); and

(d) For full contact column splices, the maximum clearance between the abutting surfaces on one edge (see Figure 3 (b)) 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
   (iv) For gaps exceeding (ii) or (iii) an engineering assessment shall be required.

C4.3.4

This clause calls for the column to be plumbed to the requirements specified in 4.3.3 and then the gap measured. The restrictions on the gap are summarised in Figure 3. These restrictions are intended to convey the reality that at splices, a perfect full-and-even contact fit cannot be achieved.

Tests (see 3.5(n)) 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.

The criteria for fit of compression member connections are equally applicable to connections at column splices and connections between columns and base plates.

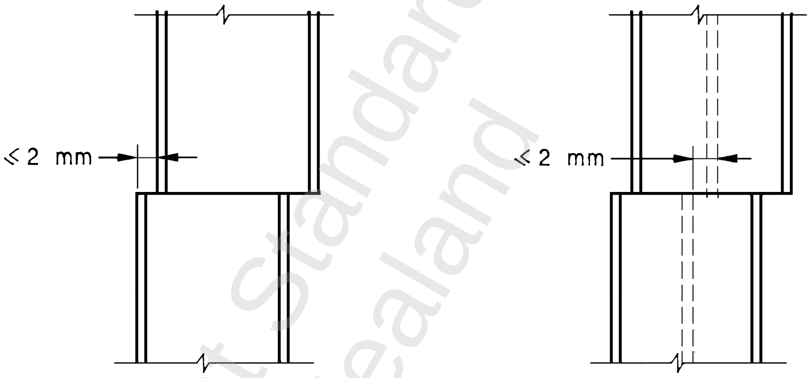


Figure 13 – Column splice

4.3.5 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 relative to supporting members.

C4.3.5

The tolerance on sweep of L_b/500 is greater than the sweep allowed during fabrication (L /1000). This extra tolerance allows for the pulling in of a beam at intermediate connections during erection. The misalignment of 1/500 should also apply to lengths between any splices in the beam.

4.3.6 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.

4.3.7 Connections to vertical concrete or masonry walls

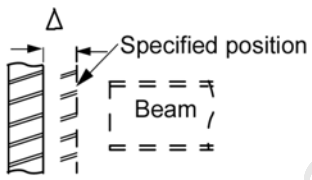
Steelwork shall be detailed and fabricated to accommodate the tolerances Δ for attachment points of steelwork and locations of embedded items in concrete walls as shown in Figure 14, Figure 15, and Figure 16.

C4.3.7

These tolerances may be different to, and supersede those specified in NZS 3109.

It is recommended that the builder confirms tolerances at the steel to concrete interfaces with the steel constructor and concretor prior to fabrication of steelwork commencing.

It is recommended that the contractor confirm concrete to steel interface tolerances with the concrete and steelwork subcontractors prior to work commencing.

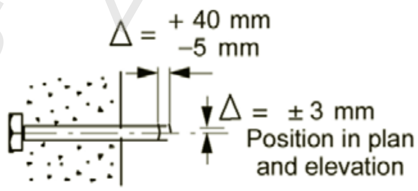


Δ = ± 15 mm up to a 4 m height

Δ = ± 20 mm above 4 m, plus 1 mm for every metre above 8 m height to ± 50 mm maximum

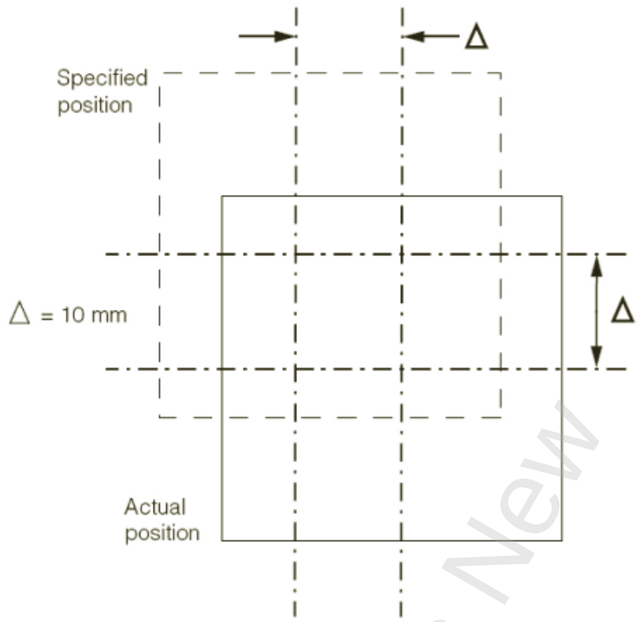
NOTE – Tolerances may be different to NZS 3109 requirements.

Figure 14 – Deviation of vertical wall



NOTE – Tolerances may be different to NZS 3109 requirements.

Figure 15 – Pre-set wall bolt not prepared for adjustment



NOTE – Tolerances may be different to NZS 3109 requirements.

Figure 16 – Embedded cast-in fixing plates

4.3.8 Crane rails and runway beams

The permitted deviations for the alignment and the gauge of crane rails shall be as given in Figure 17 (cases (a), (b), (c), and (e)).

The permitted deviation for the levels of adjacent crane rails or runway beams at a joint shall be as given in Figure 17 (cases (d), (f), and (g)).

C4.3.8

As with other special tolerances, the project specification should define any that are necessary for crane rails to match the requirements of the crane manufacturer.

Special details permitting adjustment should be provided in connections for crane rails and runway beams in order to accommodate the permitted deviation of the support steelwork.

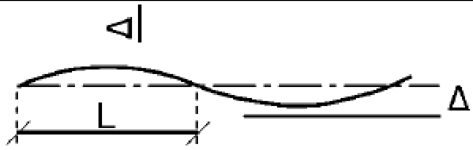
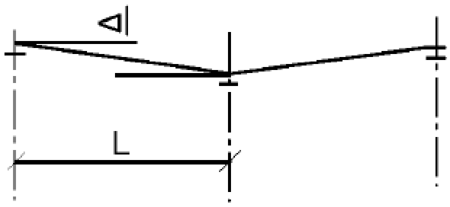
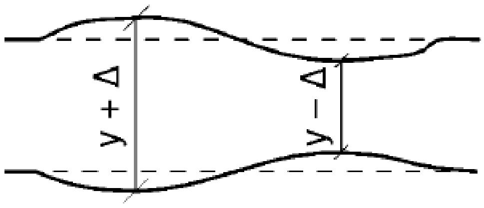
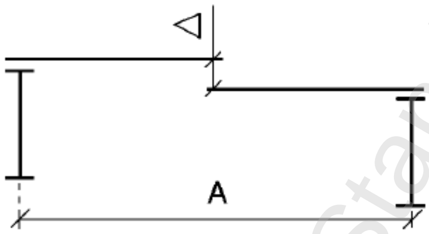
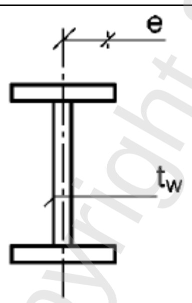
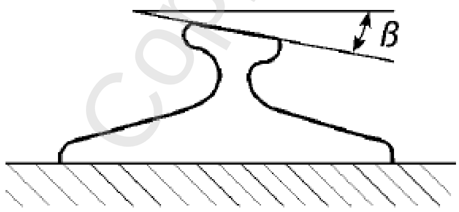
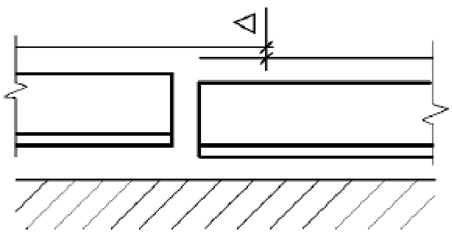
No.	Type of deviation	Description	Permitted deviation
(a)		Local alignment of rail over gauge length (*L*) of 2 m	Horizontally: $\Delta = \pm 1$ mm Vertically: $\Delta = \pm 2$ mm
(b)		Level of rail over length (*L*) between column supports	$\lvert \Delta \rvert$ = greater of: $\left[\begin{matrix} A/1000 \\ 10\,\text{mm} \end{matrix}\right]$
(c)		Alignments of parallel rails	For $y \leq 15$ m: $\Delta = \pm 5$ mm For $y > 15$ m: $\Delta = \pm 10$ mm
(d)		Difference in level of rails	$\lvert \Delta \rvert$ = greater of: $\left[\begin{matrix} A/1000 \\ 10\,\text{mm} \end{matrix}\right]$
(e)		Eccentricity of rail with respect to web	For $t_w \geq 12$ mm: $e = \pm 0.5\, t_w$ For $t_w < 12$ mm: $e = \pm 6$ mm
(f)		Flatness of rail cross-section with respect to horizontal	$\beta = \pm 1/100$ arc
(g)		Step at joint in rail	$\Delta = \pm 0.5$ mm

Figure 17 – Permitted deviations for crane rails and runway beams

4.3.9 Overall building dimensions

The overall building dimensions shall not deviate from the correct values by more than the following:

(a) Length (see Figure 18)
for $\Sigma L_c \leq 30$ m, $\Sigma \Delta L_c \leq \pm 20$ mm
for $\Sigma L_c > 30$ m, $\Sigma \Delta L_c \leq \pm [20 \text{ mm} + 0.25 (\Sigma L_c - 30) \text{ mm}]$

(b) Height (see Figure 19)
for $\Sigma h_b \leq 30$ m, $\Sigma \Delta h_b \leq \pm 20$ mm
for $\Sigma h_b > 30$ m, $\Sigma \Delta h_b \leq \pm [20 \text{ mm} + 0.25 (\Sigma h_b - 30) \text{ mm}]$

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:

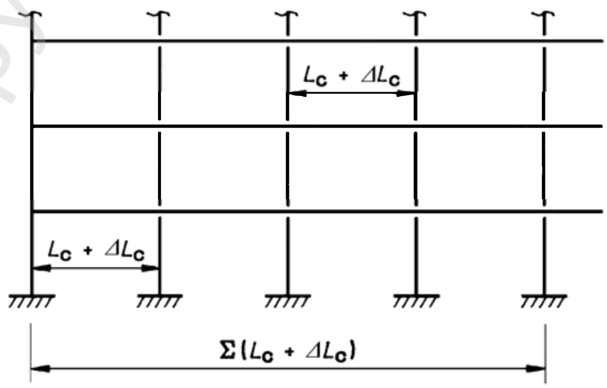
ΣL_c = the correct overall length of steelwork, being the centre to centre distance of the extreme columns as shown in Figure 18 at any location along the building (in metres), and

Σh_b = the correct overall height of steelwork, being the vertical distance from underside of column base plate to the top of the finished floor level shown in Figure 19, at any location along the building (in metres).

C4.3.9

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.

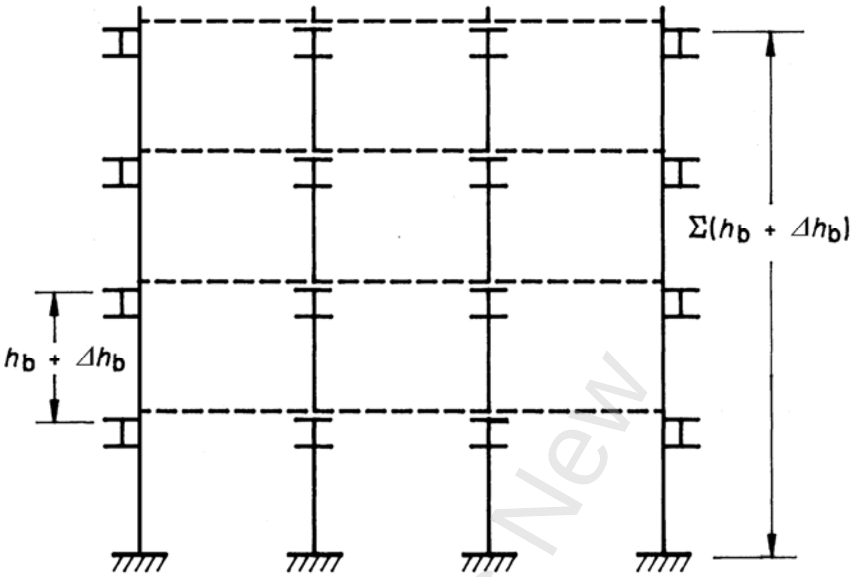
This clause is based on a similar clause in EN 1090-2.



where

L_c = distance between columns
ΔL_c = deviation from L_c
ΣL_c = correct overall length of steelwork
$\Sigma \Delta L_c$ = deviation from ΣL_c

Figure 18 – Deviations in length (vertical section)



where

h_b = distance between tops of beams

Δh_b = deviation from h_b

Σh_b = correct overall height of steelwork

$\Sigma\Delta h_b$ = deviation from Σh_b

Figure 19 – Deviations in height (vertical section)

4.4 Grouting at supports

> ***C4.4***
>
> *The provisions represent good practice, and reference is made to NZS 3109 for details of the grout. Grout strength would typically vary from 10 to 25 MPa, and should be stated on the design drawings.*

4.4.1 Compression member base or beam against wall

4.4.1.1

Bedding under a compression member base plate or the bearing of a beam on masonry and concrete shall be provided by grout or mortar.

4.4.1.2

Grouting or packing shall not be carried out until a sufficient portion of the structure (for multi-storey buildings, a sufficient number of bottom column lengths) has been aligned, levelled, 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; any compressible cavity former material around sleeved holding-down bolts shall be removed, and the space shall be free from water immediately before grouting.

4.4.2 Grouting

Grout shall completely fill the space to be grouted and shall either be placed under pressure or placed by ramming against fixed supports.

Grout shall comply with NZS 3109.

4.5 Rejection of an erected item

4.5.1

An erected item shall be liable for rejection if:

(a) The erection does not satisfy the requirements of 4.2; or
(b) It does not satisfy the tolerances specified in 1.1.

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 by any of the shortcomings in 4.5.1(a) or (b); or
(d) It passes testing in accordance with the appropriate clauses of section 4.

4.5.2

Bolts, nuts, and washers shall be liable to rejection if, in the erected structure, they do not comply with 4.2.2, 4.2.3, 4.2.5, and 4.2.6, 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.

C4.5.2

This clause specifies under what conditions a member may be 'liable to rejection'. Rejection for failure to comply with 4.1, 4.2, or 4.3 is not automatic, as it is recognised 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.

4.5.3

Grouting at supports which does not satisfy the requirements of 4.4 shall be rejected.

4.6 References to section 4

(a) Firkins, A, and Hogan, T J. *Bolting of steel structures.* 3rd ed. Sydney: Australian Institute of Steel Construction, 1990.

(b) AISC. *Load and resistance factor design – Specification for structural joints using ASTM A325 or A490 Bolts*. American Institute of Steel Construction, 1988.

(c) Hogan, T J, and Firkins, A. 'Fabrication and erection provisions of AS 1250.' Proceedings, Third Conference on Steel Developments (1985): 155 – 161.

(d) HERA. *HERA specification for the fabrication, erection and surface treatment of structural steelwork. HERA report R4-99*. Manukau City: HERA, 1998.

(e) AISC. *Economical structural steelwork*. 3rd ed. Sydney: Australian Institute of Steel Construction, 1991.

(f) HERA. *New Zealand structural steelwork design guides, volume 2, incorporating amendment No. 3. HERA Report R4-49*. Manukau City: HERA, 1990.

5 CORROSION PROTECTION

5.1 Scope

This section applies to the corrosion protection of steel members and connection components.

5.1.1 Specified intended life

The provisions of this section shall apply to the detailing and specifying for durability of steel structures and members with a specific intended life of not less than 50 years for building structures or 100 years for highway, road, and railway bridges. Compliance with this section is intended to ensure that the structural steelwork is sufficiently durable to satisfy the requirements of the NZBC Clause B2: Durability and/or NZTA Bridge Manual Section 2: Design – General Requirements, throughout the life of the structure, with only normal maintenance, including the recoating of the protective coating system, and without requiring reconstruction or major renovation.

C5.1.1

The 'specified intended life of the building' from Clause B2 of the NZBC need not be the time 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.

5.1.2 Maintenance of the steel structure

Time to first maintenance is the expected time from application of a corrosion protection system up to when patch repair or recoating is required as part of normal maintenance. Minor repairs to coatings within the construction maintenance period are not considered when determining the time to first maintenance. In addition, prior to the time of first maintenance, minor repairs to the coating system may also be required for aesthetic reasons to rejuvenate the finish and restore loss of gloss and colour.

The criteria for determining when the time to first maintenance is reached are:

(a) For scattered general breakdown of the coating system: when a specified percentage of rust is visible. This varies from under 0.5% of the total area for barrier coat systems, which exclude air and water from the steel surface, up to 2% of the total area for sacrificial systems which protect by cathodic action;

(b) For more severe localised breakdown of the coating system, for example caused by welding damage, missed or undercoated areas: when a specified percentage from 2% to 20% of the total area has occurred; and

(c) Where blistering, flaking or rusting under the paint surface is evident.

The expected time to first maintenance of a properly installed coatings system is determined from the surface specific corrosion category in Table 14 of this Standard, with specific examples given in Table 17 to Table 24 and also in AS/NZS 2312.

C5.1.2

The criteria for determining when the time to first maintenance of coating system has been reached are based on AS/NZS 2312 which includes pictorial examples.

All coatings systems may need minor repairs prior to the time to first maintenance, for example to areas welded after the coatings have been applied and at corners where the coating thickness is difficult to verify. Coatings systems also lose their appearance over time, for example with some external coatings from UV exposure, and this may necessitate recoating for aesthetic reasons prior to the loss of effective protection to the underlying steel surface implicit in the time to first maintenance provisions.

All the above constitute part of the normal maintenance specified in the NZBC.

For structures with a specified intended life greater than a multiple of the expected time to first maintenance of the coatings system, one or more recoats are expected to be needed.

5.1.3 Inaccessible surfaces

Members with surfaces that cannot be accessed for normal maintenance shall be assessed for structural adequacy for the specified intended life as a consequence of the effects of corrosion in accordance with 5.3.

5.1.4 Surface preparation and application of coating systems

Surface preparation and application of coating systems shall be in accordance with the requirements of AS/NZS 2312 for the system selected.

5.2 Protective coating system selection

5.2.1 Determining the surface specific atmospheric corrosivity category

To select a protective coating system, determine the surface specific atmospheric corrosivity category of the coated steelwork from Table 14 according to Figure 20 and Figure 21, or the relevant city maps (see Figure 22 to Figure 25).

For cases not covered in Table 14, use the guidance given in Table 15.

C5.2.1

For a more detailed calculated determination of the atmospheric corrosivity category and extent of exposure conditions use the guidance given in New Zealand Steelwork Corrosion Coatings Guide (see 5.8(a)). Weathering of the coating system during construction may be more severe than in its final state. This often is the case with single coat systems applied to wire-brushed steelwork. To ensure the intended in-service performance of the coating system is achieved, the allowable period of construction exposure is set at half the expected time to first maintenance.

5.2.2 Selecting a complying protective coating system

The surface specific atmospheric corrosivity category determined from 5.2.1 shall be used to select a complying protective coating system for the required time to the first maintenance in accordance to 5.1.2.

Table 16 to Table 22 provides a selection of complying coating systems for required time to first maintenance or refer to AS/NZS 2312 for more coating system options according to the required time to first maintenance.

The dry film thickness specified in AS/NZS 2312 shall comply with the requirements of clause 8.3 of AS 3894.3.

C5.2.2

Additional coating systems and guidance on the compatibility of various coating systems as an overcoat is given in AS/NZS 2312 and the New Zealand Corrosion Coatings Guide (5.8(a)).

Example selection of a protective coating system:

Steel sections supporting a roof canopy are to be built in Hamilton. The steel is sheltered from the rain but open to the wind. A time to first maintenance of 15 years is desired. High gloss appearance, and resistance to weathering and graffiti are required. To select a complying coating system the following steps are taken:

(1) The macroclimate corrosivity zone is determined from Figure 20. Hamilton is located in Zone 2.

(2) The surface specific corrosivity category is determined from Table 14. For steel in Zone 2 and in an external sheltered environment the surface specific corrosion category is C.

(3) Table 18 provides a number of coating systems that have 15 years durability for a surface specific corrosion category C. A complying coating system is PSL1. This coating system has excellent hardness, can be full gloss and has excellent colour and gloss retention on weathering.

Table 14 – Surface specific corrosion categories

Corrosion map zone (Figures 20 to 25)	Macroclimate corrosion category AS/NZS 2312	Typically	Location	Characterised by	Surface specific corrosion category – External – Exposed	External – Sheltered	External – Wet	Internal – Dry	Internal – Damp	Internal – High humidity
Seaspray	E-M	Within 200 metres from breaking surf on the West Coast of the South Island. Within 100 metres from breaking surf on West Coast of the North Island. Within 50 metres from breaking surf of all other coasts. This environment may be extended inland by prevailing winds and local conditions (Figures 20 – 25).	All coasts.	Heavy salt deposits. Almost constant smell of salt spray in the air.	E-M	E-M	E-M	A	C	D
Seaspray	E-M	200 metres up to 500 metres or more inland from breaking surf. In the immediate vicinity of calm salt water such as harbour foreshores. This environment may be extended inland by prevailing winds and local conditions (Figure 21).	West Coast of the South Island.	Heavy salt deposits. Almost constant smell of salt spray in the air.	E-M	E-M	E-M	A	C	C
Seaspray	D	50 metres up to 500 metres or more inland from breaking surf. In the immediate vicinity of calm salt water such as harbour foreshores. This environment may be extended inland by prevailing winds and local conditions (Figure 20 – 25).	All coasts except West Coast of the South Island.	Medium salt deposits. Frequent smell of salt in the air.	D	E-M	E-M	A	B	C
Seaspray	D	500 metres to 1 km from breaking surf. In the immediate vicinity of calm salt water such as estuaries.	West Coast of the South Island.	Little salt deposits. Occasional smell of salt in the air.	D	E-M	E-M	A	B	C
Zone 1	C	500 metres to 1 km from breaking surf. In the immediate vicinity of calm salt water such as estuaries.	East Coast of both Islands, South Coast of North Island and all harbours.	Little salt deposits. Occasional smell of salt in the air.	C	E-M	E-M	A	B	C
Zone 1	C	More than 1 km to 20 km from salt water.	West and South Coast of South Island.	Minor salt deposits. No smell of salt in the air.	C	E-M	E-M	A	B	C
Zone 1	C	More than 1 km to 5 km from salt water.	East Coast of both Islands, West Coast of North Island, South Coast of North Island, and all harbours.	Minor salt deposits. No smell of salt in the air.	C	D	E-M	A	B	C
Zone 2	B	More than 20 km to 50 km from salt water.	West and South Coast of South Island.	No marine influences.	B	C	D	A	B	C
Zone 2	B	More than 5 km to 50 km from salt water.	East Coast of both Islands, West and South Coast of North Island, and all harbours.	No marine influences.	B	C	D	A	B	C
Zone 3	B	Inland, more than 50 km from salt water.	North and South Islands.	No marine influences.	B	B	C	A	B	C
Zone 4	E-I	Close to the geothermal source < 150 m.	Taupo Volcanic Zone.	Major geothermal influence.	E–I	E-I	E-I	A	B	C
Zone 4	B	Not closer than 150 m to geothermal source.	Taupo Volcanic Zone.	Mild geothermal influence with no marine influences.	B	D	D	A	B	C

Table 15 – Exposure conditions not covered by Table 14

Exposure condition for steelwork	Use
Within the external wall and roof cavity with the steel on the cold side of the dew point. Steel in subfloor spaces.	High humidity.
Steelwork near openings in external walls.	Sheltered.
High humidity and corrosive atmosphere, such as chemical processing plant, swimming pool, dye works, paper manufacturing plants, boatyards over sea water, foundries, or smelters.	Specific engineering design required, refer to Table B1 of AS/NZS 2312.

NOTE to Table 14 and Table 15 – The following definitions apply:

External	Exposed to the weather
Internal	Protected from the weather by being located inside the structure
Exposed	Opened to airborne salts, is washed by the rain, and can dry quickly after wetting
Sheltered	Open to airborne salts but unwashed by the rain, such as an awning or the underside of a steel bridge
Wet	Often wet for extended periods of time, such as crevices, or in low points and pockets that are not drained
Dry	Dry internal environment, such as a fully enclosed office or apartment building
Damp	Damp internal environment where condensation may occur, such as non-air conditioned and poorly insulated vehicle depots and warehouses
High humidity	Internal, high humidity environment with some pollution, such as a food processing plant, breweries, and dairies

Table 16 – Internal steelwork – Coating required only for appearance, surface specific corrosivity category A

System designation	Type	Surface preparation	Number of coats	Hardness	Typical colour	Initial gloss	Colour and gloss retention on weathering	Allowable surface specific corrosivity during construction[1]
ALK4	Alkyd	Sa2½	2	Good	Wide range	Flat to full gloss	Good	D
PUR1	Polyurethane	St2		Excellent		Semi-gloss to full gloss	Excellent	D
ALK1	Alkyd		1	Good	Limited range	Flat to full gloss	Fine[2]	B[3]
ALK 3			2		Wide range		Good	C

NOTE –

(1) Based on a maximum of 1-year exposure during construction.

(2) The alkyd primer system ALK1 should not be used in grey colour because the breakdown of the system will be highly visible. Red oxide colour is preferred to reduce the visual impact of minor and structurally acceptable rusting that will occur on the ALK1 system in a few years.

(3) Based on a maximum of 4-weeks exposure during construction.

Table 17 – Coatings for 15 years – Time to first maintenance, surface specific corrosivity category B

System designation	Type	Surface preparation	Number of coats	Hardness	Typical colour	Initial gloss	Colour and gloss retention on weathering
EPM3	Epoxy mastic	Sa2	2	Very good	Wide range	Low to semi-gloss	Fine
ACC2	Acrylic	Sa2½				Semi-gloss to full gloss	Very good
PUR2	Polyurethane			Excellent			Excellent
IZS2 [1]	Inorganic zinc silicate		1		Mostly grey	Flat	Fine

NOTE –

(1) IZS2 is solvent borne.

Table 18 – Coatings for 15 years – Time to first maintenance, surface specific corrosivity category C

System designation	Type	Surface preparation	Number of coats	Hardness	Typical colour	Initial gloss	Colour and gloss retention on weathering
IZS2 [1]	Inorganic zinc silicate	Sa2½	1	Excellent	Mostly grey	Flat	Fine
EHB2	High build epoxy				Wide range	Flat to full gloss	Fair
PSL1	Polysiloxane		2			Semi-gloss to full gloss	Excellent
PUR4	Polyurethane		3				
ACC4	Acrylic			Very good			Very good
MCU2	Moisture cured urethane			Excellent	Limited range	Semi-gloss	

NOTE –

(1) IZS2 is solvent borne.

Table 19 – Coatings for 15 years – Time to first maintenance, surface specific corrosivity category D

System designation	Type	Surface preparation	Number of coats	Hardness	Typical colour	Initial gloss	Colour and gloss retention on weathering
IZS3 [1]	Inorganic zinc silicate	Sa2½	1	Excellent	Mostly grey	Flat	Fine
TSZ100	Thermal zinc spray				Grey (wide range if colour sealer is used)		
PUR5	Polyurethane		3		Wide range	Semi-gloss to full gloss	Excellent
ACC6	Acrylic			Very good			Very good

NOTE –

(1) IZS3 is solvent borne.

Table 20 – Coatings for 15 years – Time to first maintenance, surface specific corrosivity category E-M

System designation	Type	Surface preparation	Number of coats	Hardness	Typical colour	Initial gloss	Colour and gloss retention on weathering
EUH1	Ultra high build epoxy	Sa2½	1	Excellent	Limited	Flat to semi-gloss	Fair
IZS3 [1]	Inorganic zinc silicate				Mostly grey (limited colour range)	Flat	Fine
TSZ150S	Thermal zinc spray with sealer				Grey (wide range if colour sealer is used)		
HDG600P7	Hot dip galvanising with paint	Sweep abrasive blast	2		Wide range	Semi-gloss to full gloss	Excellent

NOTE –
(1) IZS3 (solvent borne) gives 13.5 years time to first maintenance; IZS3 (water borne) gives 15 years.

Table 21 – Coatings for 25+ years – Time to first maintenance, surface specific corrosivity category B

System designation	Type	Surface preparation	Number of coats	Hardness	Typical colour	Initial gloss	Colour and gloss retention on weathering
IZS2 [1]	Inorganic zinc silicate	Sa2½	1	Excellent	Mostly grey	Flat	Fine
EHB2	High build epoxy				Limited range	Flat to semi-gloss	Fair
MCU2	Moisture cured urethane		3			Semi-gloss	Very good
PUR3	Polyurethane				Wide range	Semi-gloss to full gloss	Excellent
ACC4	Acrylic			Very good			Very good

NOTE –
(1) IZS2 is solvent borne.

Table 22 – Coatings for 25+ years – Time to first maintenance, surface specific corrosivity category C

System designation	Type	Surface preparation	Number of coats	Hardness	Typical colour	Initial gloss	Colour and gloss retention on weathering
EUH1	Ultra high build epoxy	Sa2½	1	Excellent	Limited	Flat to semi-gloss	Fair
IZS3	Inorganic zinc silicate				Mostly grey	Flat	Fine
TSZ100	Thermal zinc spray				Grey (wide range if colour sealer is used)		
HDG600	Hot dip galvanising	See AS/NZS 4680	0		Grey		
HDG600P7	Hot dip galvanising with paint	Sweep abrasive blast	2		Wide range	Semi-gloss to full gloss	Excellent

Table 23 – Coatings for 25+ years – Time to first maintenance, surface specific corrosivity category D

System designation	Type	Surface preparation	Number of coats	Hardness	Typical colour	Initial gloss	Colour and gloss retention on weathering
EUH1	Ultra high build epoxy	Sa2½	1	Excellent	Limited	Flat to semi-gloss	Fine
TSZ150S	Thermal zinc spray with sealer				Grey (wide range if colour sealer is used)	Flat	
HDG900	Hot dip galvanising		0		Grey		
IZS3	Inorganic zinc silicate		1		Mostly grey		
HDG600P7	Hot dip galvanising with paint	Sweep abrasive blast	2		Wide range	Semi-gloss to full gloss	Excellent

Table 24 – Coatings for 25+ years – Time to first maintenance, surface specific corrosivity category E-M

System designation	Type	Surface preparation	Number of coats	Hardness	Typical colour	Initial gloss	Colour and gloss retention on weathering
TSA225S	Thermal aluminium spray with sealer	Sa2½	2	Excellent	Grey (wide range if colour sealer is used)	Flat	Fine
TSZ200S	Thermal zinc spray with sealer	Sa2½	2	Excellent	Grey (wide range if colour sealer is used)	Flat	Fine

NOTE – Thermal aluminium spray is mostly used for structures within 100 m from the sea due to the high corrosivity category and abrasiveness of the climate of that region, while thermal zinc spray is used for structures in the low to mid E-M category.

5.3 Assessment of steel loss during design life

Where an assessment of average steel loss on a surface is required over the design life of the structure, use equation 5.3.1 in conjunction with the surface specific corrosivity category determined from 5.2.

5.3.1 Corrosion of steel in air

The steel thickness loss (t_{sl}) shall be calculated as follows:

$t_{sl} = C_r\,(T_{DL} - T_{FM})$ .... Eq. 5.3.1

where

t_{sl} Steel thickness loss in mm/steel surface

T_{DL} Steelwork design life in years

T_{FM} Total time to first maintenance of initial and subsequent coating systems applications

C_r Carbon steel corrosion rate.

The carbon steel corrosion rate is given in Table 25 based on the actual surface specific corrosivity given in 5.2.1.

NOTE – Equation 5.3.1 does not account for localised pitting corrosion so is only applicable to a minimum steel thickness of 10 mm for steel in maritime environments or embedded in soil, and 8 mm for steelwork located elsewhere.

C5.3.1

Example equation:

Steel sections supporting a roof canopy will be recoated after 15 years with a 15-year time to first maintenance coating system. No further recoats will be applied. The design life of the structure is 50 years. The surface specific corrosivity category is C. The extent of steel thickness loss at the end of the design life is calculated using Eq. 5.3.1 as follows:

$t_{sl} = 7.2\,(50 - (15 + 15)) = 144 \times 10^{-3}$ *mm/steel surface.*

Table 25 – Atmospheric corrosion rates for carbon steel in various surface specific atmospheric corrosivity categories

Metal	**Corrosion rates for surface specific corrosivity categories** (10^{-3} mm/steel surface/year)				
	A	B	C	D	E
Carbon steel	0.02	2.2	7.2	22	92

NOTE –

(1) These values are derived from ISO 9224 and represent the average annual corrosion rate over 50 years of exposure at the most severe end of each environmental category.

(2) In ISO 9224, these categories are given as C1 to C5. These are related to the categories given in Table B1 of AS/NZS 2312.

5.3.2 Corrosion of steel in water and soil

5.3.2.1

Equation 5.3.1 shall be used to determine the average steel thickness loss (t_{sl}) for steel exposed to water or embedded in soil.

5.3.2.2 *Protective coatings of steel piles*

Protective coatings systems can be used to increase the steel pile design life. Table 26 describes recommended options.

C5.3.2.2

Appendix C of AS/NZS 2312, and AS 2159 have additional recommendations for protective coatings for piles. A number of methods can be used to increase the steel pile design life. These methods are:

(a) Use of a heavier section;

(b) Use of a high yield steel at mild steel stress levels;

(c) Apply a protective coating;

(d) Apply cathodic protection; and

(e) Use concrete encasement where practicable.

C5.3.2.1

Guidance on typical corrosion rates of steel in water and soil for use in design are given in the commentary table below:

Typical bare steel design corrosion rates in water and soil for use in design

Location	***Steel pile case***	***Type***	***Design corrosion rate*** *(mm/steel surface/year)*
Soil	*Buried in fill below the permanent water table*	–	*0.015*
	Buried in controlled fill above the permanent water table	–	*0.015*
	Buried in uncontrolled fill above the permanent water table	*Fill which includes more than small quantities of any or all of the following materials: cinders, fly ash, slag from steel making or the residue from concrete making, coal or organic waste.*	*0.050 (for ground with pH ≥ 4)* *0.075 (for ground with pH < 4)*
		Soil material or concrete, brick, and other inorganic building material rubble.	*0.025*
Water	*Below the sea bed*	*Undisturbed, natural soils.*	*0.015*
	Permanent immersion in sea water	–	*0.035*
	Permanent immersion in fresh water	*For lengths of the pile further than around 300 mm below the surface of the water.*	*0.025*
		For length of pile in the top 300 mm depth of water.	*0.050*[1]
	Low-water zone corrosion	*Bottom of the tidal range where a lack of marine growth occurs but oxygen is quite readily available.*	*0.075*
	Tidal zone	*Tidal zones tend to accumulate marine growths, which reduce the supply of oxygen to the steel surface.*	*0.035*
	Splash and marine atmospheric zones	*Above the tidal range, subject to wave action and high chloride concentrations. The height of the zone depends on the degree of shelter from wave action.*	*0.075*[2]
		Above peak wave height and where the pile is sheltered from direct wind flow (such as the underside of wharfs, above the wharf skirt).	*0.035*
Exposed	*Steel piles in the exterior atmosphere*	–	*0.035*[3]
	Steel soldier piles in retaining walls	*Zone at base of pile within drainage backfill.*	*0.050*

NOTE –

(1) The corrosion rate is sometimes observed to be higher (BS 8004) over the zone of pile in the top 300 mm depth of water and, for design purposes, a value of 0.050 mm/steel surface/year should be used in this band. This band of increased corrosion occurs when the water level is constant and becomes less significant with greater fluctuation of the water level (BS 8004).

(2) These are above the tidal range, subject to wave action and high chloride concentrations. The height of the zone depends on the degree of shelter from wave action. A corrosion rate of 0.075 mm/steel surface/year is recommended by 5.8(b), however higher rates can occur in localised conditions, especially where heavy wave action occurs. Localised conditions are especially affected by the shape of the steel pile, the temperature of the water and the exposure of the pile to the elements (wind and wave action). For example, localised corrosion is confined to some external corners of sheet piled walls in a zone at, or just below, the mean low water level. Other regions undergo very much lower corrosion rates. Peak corrosion rates of 0.3 – 0.8 mm/year have been observed in these circumstances.

The design corrosion rate for this application must be used in conjunction with specification of a supplementary corrosion protection system for the mean low water region and/or periodic inspection and a maintenance regime to eliminate structural degradation due to high localised corrosion in this zone.

(3) This design rate will be an overestimate for New Zealand conditions for all atmospheric classifications in AS/NZS 2312, except for the severe marine classification, however it will cover many severe marine sites.

The values in the above commentary table are from 5.8(a) – (d).

The corrosion rate has been shown to not be affected by the soil type or permeability between different soil types. Also, if the soil has a low pH there is a slight increase in the corrosion rate but not as high as previously believed. Finally, there appears to be no correlation between the corrosion rate and resistivity as previously believed (see 5.8(a)).

Whereas the corrosion rate for piles in soil in AS 2159 is dependent on the soil pH, chloride levels, and the resistivity, AS 2159 does not provide guidance on the make up of the fill and its effect on the corrosion rate.

Table 26 – Recommended protective coating systems for steel piles

AS/NZS 2312 system designation	Coating	Surface preparation	No. of coats	Nominal dry film thickness (microns)	Typical areas of use	Typical times to first maintenance (years)
EHB2	High build epoxy	Sa2½	1	450	Piers, jetties, bearing piles in corrosive soils	20+
–	Glass flake epoxy	Sa2½	1	450	Piers, jetties, bearing piles in corrosive soils. For soils and immersion conditions that also require abrasion resistance	20+
–	Glass flake polyester/ vinyl ester	Sa2½	1	450	Piers, jetties, bearing piles in very corrosive soils where abrasion resistance and chemical resistance area required.	20+
MCU1	Moisture cured urethane	St2	3	200	Piers, jetties, bearing piles in corrosive soils	20+
HDG600	Hot dip galvanising	See AS/NZS 4680	0	85	Retaining walls in non-marine environments	20+

5.4 Inspection of coatings

Inspection of protective coating systems shall be carried out in accordance with the recommendations in AS/NZS 2312.

C5.4

Coating inspection recommendations are covered in detail in AS/NZS 2312. These include confirmation of requirements and achievement of: specifications; standards of workmanship for each step in the system; surface preparation; method of application; suitability of equipment; drying and curing time intervals between coats and handling; coating thickness; method of handling; and reporting.

It is recommended that inspection of protective coatings systems in locations with surface specific corrosivity of C, D, E-I, or E-M, be performed by a Certification Board for Inspection

Personnel (CBIP) or National Association of Corrosion Engineers (NACE) certified coatings inspector.

In all surface specific corrosivity categories, the performance of coating systems requiring steel surface preparation to AS/NZS 2312 of Sa2½ or higher is very dependent on that quality of preparation being achieved. This includes attaining a specific surface roughness where required for a particular coating (for example, thermal metal sprays require a sharp angular profile to a specified roughness depending on the type of metal spray being deposited).

5.5 Inaccessible surfaces

Surfaces in contact or near contact after fabrication or erection shall receive their specified surface preparation and treatment prior to assembly. Such coatings should be cured before assembly.

This clause does not apply to the interior of sealed hollow or sealed box sections, or connection surfaces for joints with friction type bolting category where bare steel interfaces are specified.

5.6 Protection during transport and handling after corrosion protection

Structural members shall be adequately protected during handling and transport to minimise 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.

5.7 Repairs to corrosion protection

Corrosion protection which has been damaged by welding, erection or other causes shall be reinstated before the structure is put into service.

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 application of an equivalent thickness of a suitable zinc paint conforming to AS/NZS 3750.9 or to AS/NZS 3750.15, or with thermal zinc spray.

Damaged hot dip galvanising should be repaired to the requirements listed in AS/NZS 4680.

5.8 References to section 5

(a) HERA. *New Zealand steelwork corrosion coatings guide, HERA Report R4-133.* Manukau City: HERA, 2005.

(b) ArcelorMittal. *Piling handbook*. 8th ed. Esch-sur-Alzette, Luxembourg: ArcelorMittal, 2008.

(c) EN 1993-5 *Eurocode 3 – Design of steel structures – Part 5: Piling*. European Committee for Standardization, 2007.

(d) BS 8004 *Code of practice for foundations*. British Standards Institution, 1986.

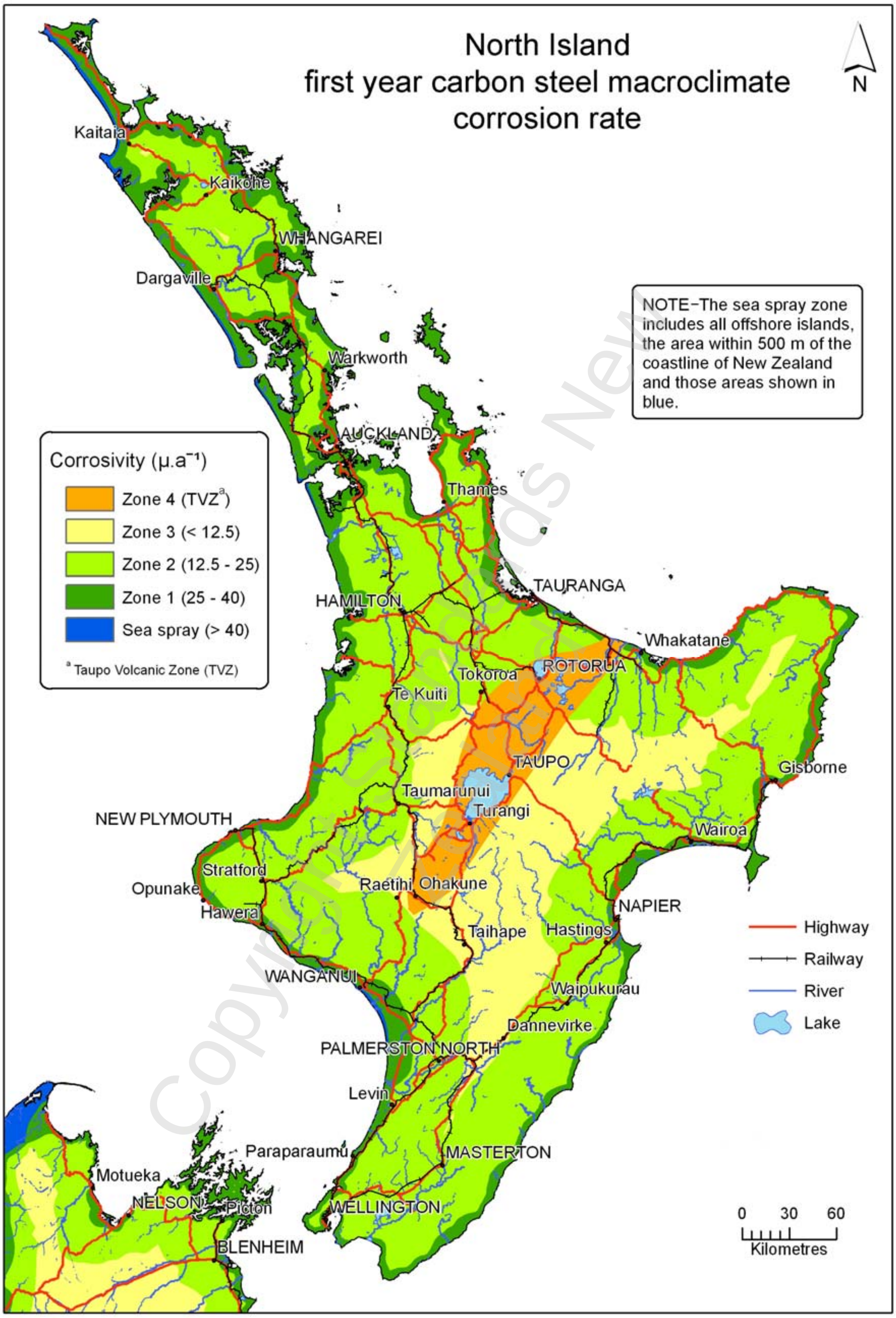


Figure 20 – North Island corrosivity zone map

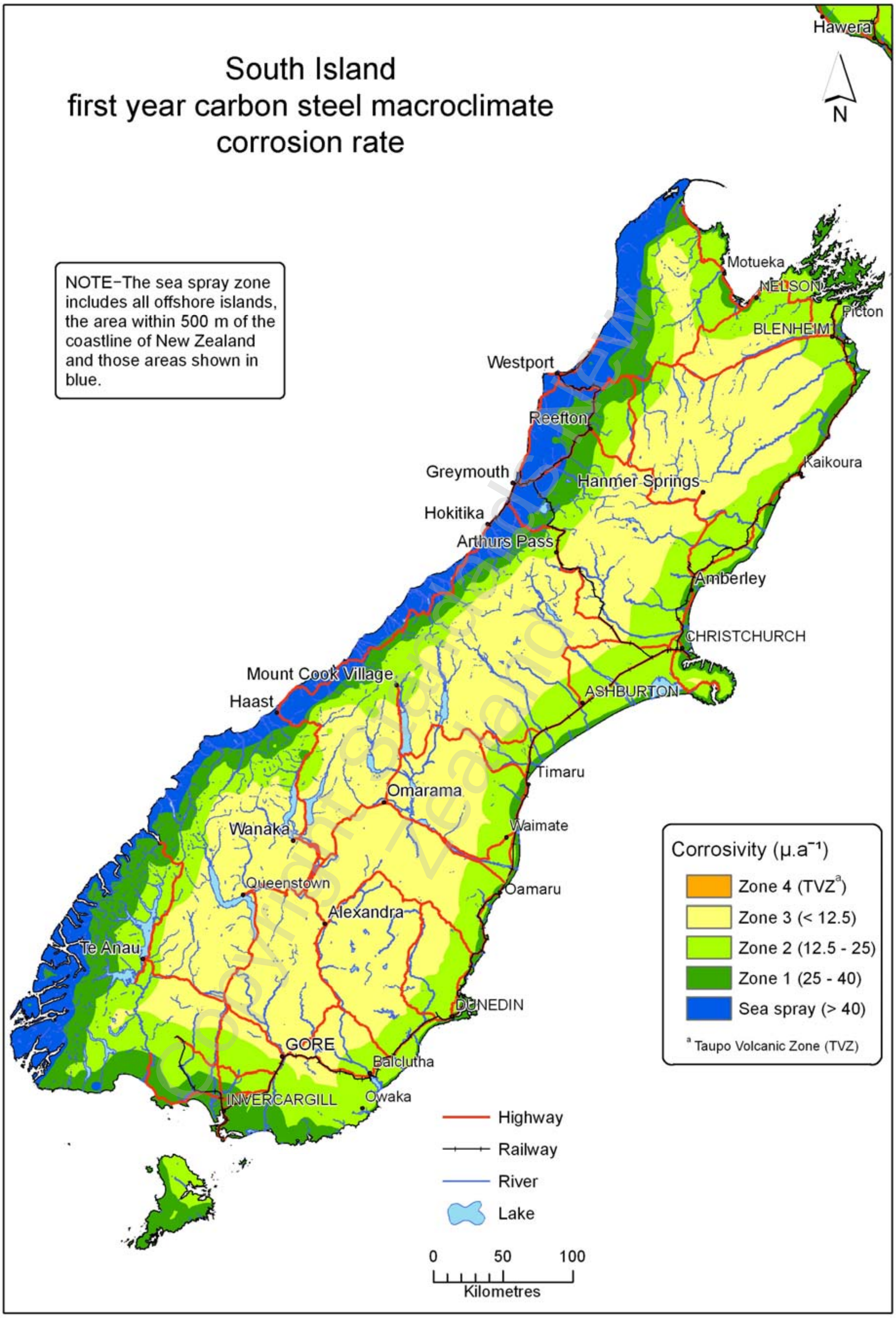


Figure 21 – South Island corrosivity zone map

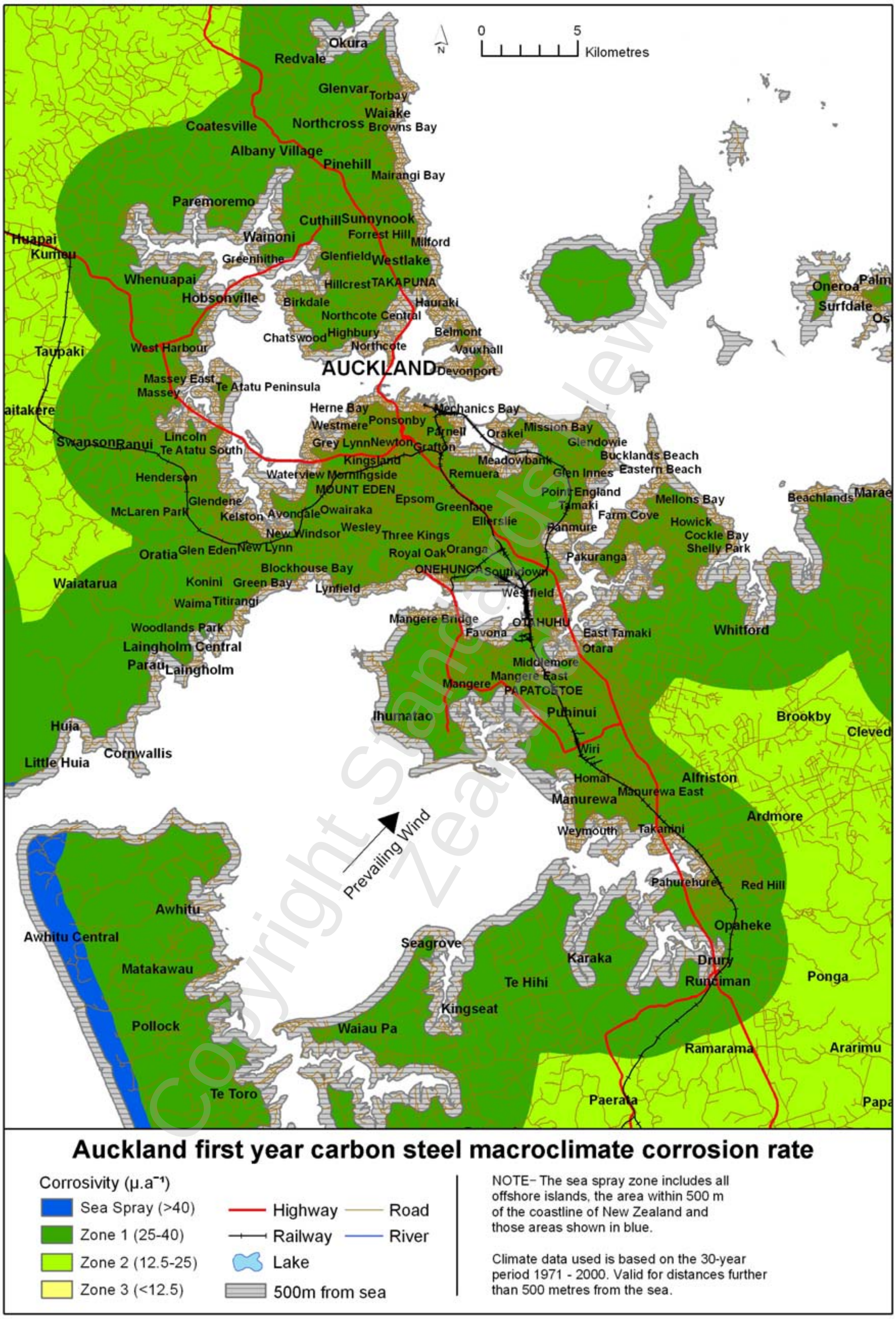


Figure 22 – Auckland corrosivity zone map

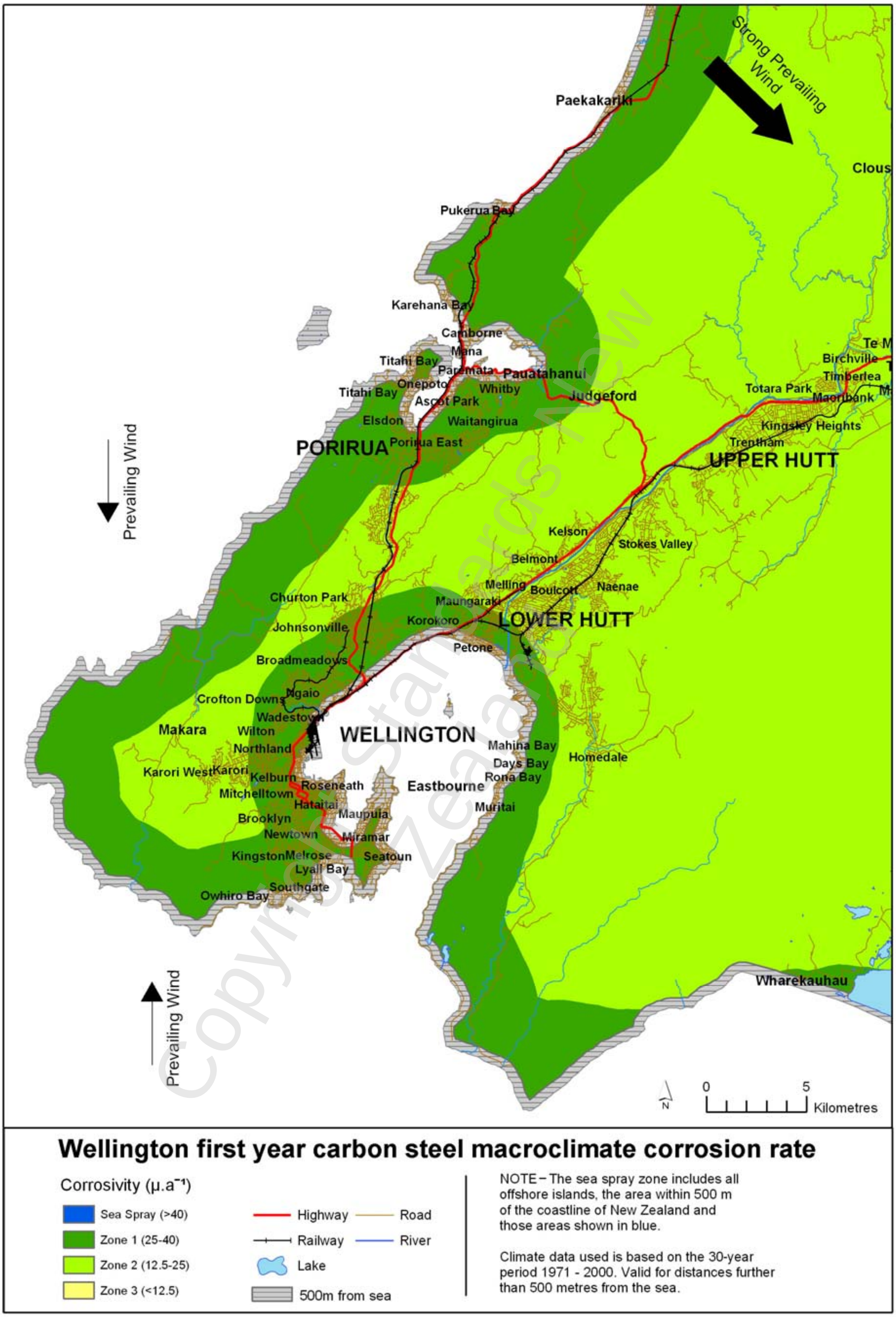


Figure 23 – Wellington corrosivity zone map

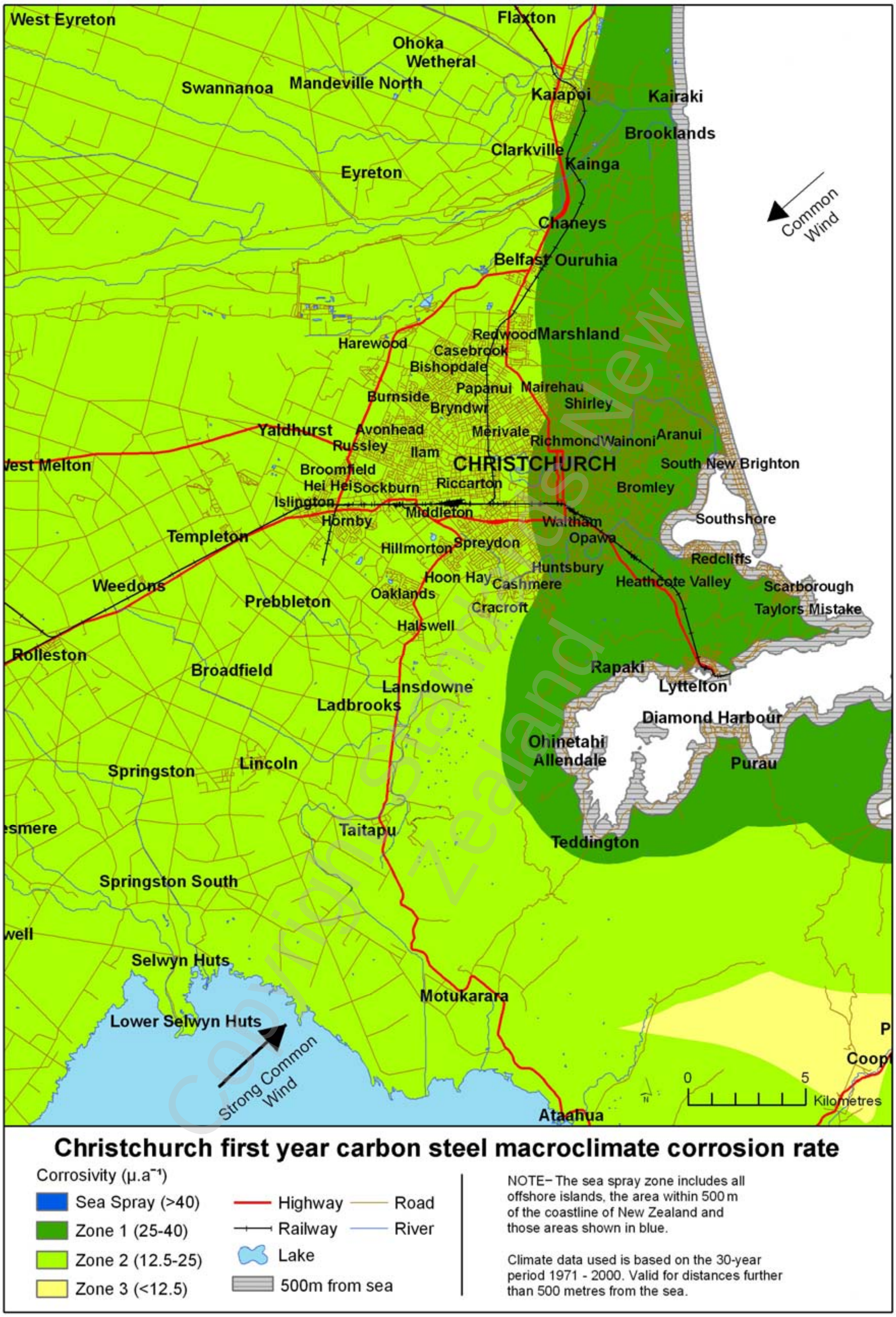


Figure 24 – Christchurch corrosivity zone map

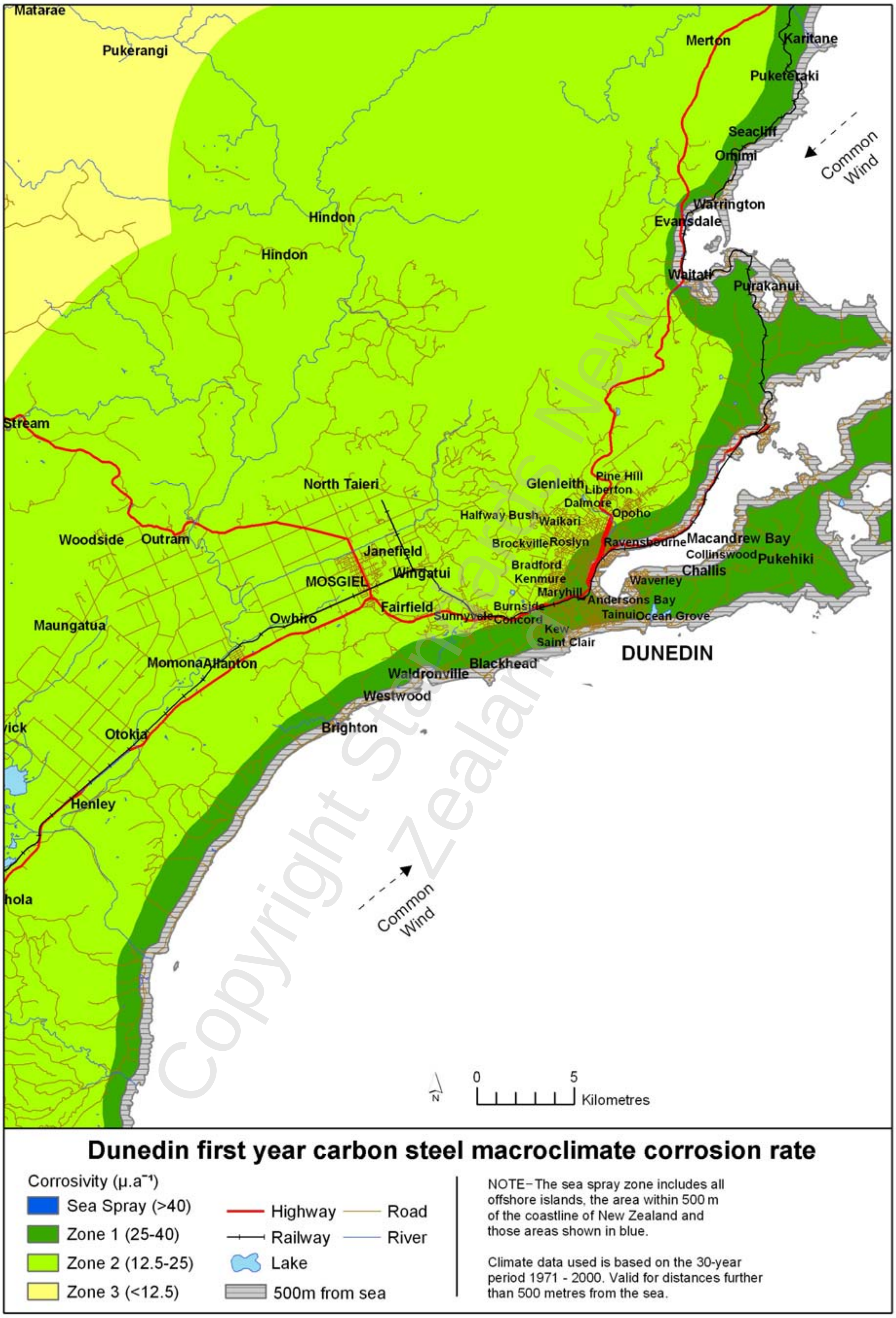


Figure 25 – Dunedin corrosivity zone map

6 ARCHITECTURALLY EXPOSED STRUCTURAL STEEL (AESS)

6.1 Scope and requirements

6.1.1 General requirements

When members are specifically designated as 'Architecturally Exposed Structural Steel' or 'AESS' in the contract documents, the requirements in sections 3 and 4 shall apply as modified by this section. AESS members or components shall be fabricated and erected with the care and dimensional tolerances that are stipulated in 6.2 to 6.5.

6.1.2 Definition of categories

Categories are listed in the AESS Matrix shown in Table 27 where each category is represented by a set of characteristics. The following categories shall be used when referring to AESS:

(a) AESS 1 – Basic elements
Suitable for 'basic' elements which require enhanced workmanship;

(b) AESS 2 – Feature elements viewed at a distance > 6 m
Suitable for 'feature' elements viewed at a distance greater than six metres. The process involves basically good fabrication practices with enhanced treatment of weld, connection, and fabrication detail, tolerances for gaps, copes;

(c) AESS 3 – Feature elements viewed at a distance ≤ 6 m
Suitable for 'feature' elements – where the designer is comfortable allowing the viewer to see the art of metalworking – welds are generally smooth but visible, some grind marks are acceptable. Tolerances are tighter than normal standards. The structure is normally viewed closer than six metres and is frequently subject to touch by the public;

(d) AESS 4 – Showcase elements
Suitable for 'showcase or dominant' elements – used where the designer intends that the form is the only feature showing in an element. All welds which have ground and filled edges are ground square and true. All surfaces are sanded/filled. Tolerances of fabricated forms are more stringent – generally half of standard tolerance. All surfaces to be 'glove' smooth; and

(e) AESS C – Custom elements
Suitable for elements which require a different set of characteristics as specified in Categories 1, 2, 3, or 4.

6.1.3 Additional information

The following additional information shall be provided in the contract documents when AESS is specified:

(a) Specific identification of members or components that are AESS using the AESS Categories listed in 6.1.2 (see Table 27);

(b) Fabrication and/or erection tolerances that are to be more restrictive than provided for in section 6;

(c) For Categories AESS 2, 3, 4 requirements, if any, of a visual sample or first-off component for inspection and acceptance standards prior to the start of fabrication; and

(d) For Category AESS C, the AESS Matrix included in Table 27 shall be used to specify the required treatment of the element.

6.2 Shop detail, arrangement, and erection drawings

6.2.1 Identification

All members designated as AESS members shall be clearly identified to a category, either AESS 1, 2, 3, 4, or C, on all shop detail, arrangement, and erection drawings.

6.2.2 Variations

Any variations from the AESS Categories listed shall be clearly noted. These variations could include machined surfaces, locally abraded surfaces, or forgings. In addition:

(a) If distinction is to be made between different surfaces or parts of members the transition line/plane shall be clearly identified/defined on the shop detail, arrangement, and erection drawings;

(b) Tack welds, temporary braces, or fixtures used in fabrication are to be shown on shop drawings; and

(c) All architecturally sensitive connection details shall be submitted for approval by the architect/Design Engineer prior to completion of shop detail drawings.

6.3 Fabrication

6.3.1 General fabrication

The fabricator shall take special care in handling the steel to avoid marking or distorting the steel members. In addition:

(a) All slings shall be nylon type or chains with softeners or wire rope with softeners;

(b) Care is also taken to minimise damage to any shop paint or coating;

(c) If temporary braces or fixtures are required during fabrication, during shipment, or to facilitate erection, care shall be taken to avoid and/or repair any blemishes or unsightly surfaces resulting from the use or removal of such temporary elements; and

(d) Tack welds are ground smooth.

6.3.2 Unfinished, reused, or weathering steel

Members fabricated of unfinished, reused, or weathering steel that are to be AESS may still have erection marks, painted marks, or other marks on surfaces in the completed structure. Special requirements shall be specified as Category AESS C.

6.3.3 Tolerances for rolled shapes

The permissible tolerances for depth, width, and out of square, camber, and sweep of rolled shapes shall be as specified in 3.3.3.1 and 3.3.4 or 3.3.5. The following exceptions apply:

(a) For Categories AESS 3 and 4 and otherwise specified in the contract documents: The matching of abutting cross sections shall be required; and

(b) For Categories AESS 2, 3, and 4: The as-fabricated straightness tolerance of a member is one-half of the standard camber and sweep tolerance in 3.3.4 or 3.3.5.

6.3.4 Tolerances for built-up members

The tolerance on overall profile dimensions of members made up from a series of plates, bars, and shapes by welding is limited to the accumulation of permissible tolerances of the component parts as provided by 3.3.3.2. For Categories AESS 2, 3, and 4, the as-fabricated straightness tolerance for the member as a whole is one-half of the standard camber and sweep tolerances in 3.3.4 or 3.3.5.

6.3.5 Joints

For Categories AESS 3 and 4, all copes, mitres, and butt cuts in surfaces exposed to view are made with uniform gaps, if shown to be open joint, or in uniform contact if shown without gap.

6.3.6 Surface appearance

For Categories AESS 1, 2, and 3, the quality surface as delivered by the mills should be acceptable. For Category AESS 4, the steel surface imperfections should be filled and sanded.

6.3.7 Welds

For corrosive environments, all joints should be seal welded. In addition:

(a) For Categories AESS 1, 2, and 3, a smooth uniform weld is acceptable. For Category AESS 4, the weld shall be contoured and blended;

(b) For Categories AESS 1, 2, 3, and 4, all weld spatter is to be avoided/removed where exposed to view; and

(c) For Categories AESS 1 and 2, weld projection up to 2 mm is acceptable for butt and plug welded joints. For Categories AESS 3 and 4, welds shall be ground smooth/filled.

C6.3.7

Refer to AS/NZS 1554.1 for general dressing of the surface of flush butt or plug welds. For AESS 4, more stringent requirements may need to be specified.

6.3.8 Weld show-through

It is recognised that the degree of weld show-through, which is any visual indication of the presence of a weld or welds on the opposite surface from the viewer, is a function of weld size and material thickness.

For Categories AESS 1, 2, and 3, the members or components are to be acceptable as produced.

For Category AESS 4, the fabricator shall minimise the weld show-through.

C6.3.8

Visual effects may arise from heating, distortion, and weld profile.

6.3.9 Surface preparation for painting

Unless otherwise specified in the contract documents, the fabricator shall clean AESS members to meet the requirement of Sa2 to AS 1627.4.

Prior to blast cleaning:

(a) Any deposits of grease or oil are to be removed by solvent cleaning in accordance with AS 1627.1;

(b) Weld spatter, slivers, and surface discontinuities are to be removed; and

(c) Sharp edges resulting from flame-cutting, grinding, and especially shearing are to be softened.

6.3.10 Hollow structural sections (RHS or CHS) seams

For Categories AESS 1 and 2, seams of hollow structural sections shall be acceptable as produced.

For Category AESS 3, seams shall be oriented away from view or as shown in the contract documents.

For Category AESS 4, seams shall be treated so they are not apparent.

6.4 Delivery of materials

6.4.1 General delivery

The fabricator shall use special care to avoid bending, twisting or otherwise distorting the structural steel. All tie downs on loads shall be either nylon strap or chains with softeners to avoid damage to edges and surfaces of members.

6.4.2 Standard of acceptance

The Standard for acceptance of delivered and erected members shall be equivalent to the Standard employed at fabrication.

6.5 Erection

6.5.1 General erection

The erector shall use special care in unloading, handling, and erecting the AESS to avoid marking or distorting the AESS. The erector shall plan and execute all operations in such a manner that allows the architectural appearance of the structure to be maintained. In addition:

(a) All slings shall be nylon strap or chains with softeners;

(b) Care shall be taken to minimise damage to any shop paint or coating;

(c) If temporary braces or fixtures are required to facilitate erection, care shall be taken to avoid and/or repair any blemishes or unsightly surfaces resulting from the use or removal of such temporary elements;

(d) Tack welds shall be ground smooth and holes shall be filled with weld metal or body filler and smoothed by grinding or filling to the standards applicable to the shop fabrication of the materials;

(e) All backing bars shall be removed and ground smooth; and

(f) All bolt heads in connections shall be on the same side, as specified, and consistent from one connection to another.

6.5.2 Erection tolerances

Unless otherwise specified in the contract documents, members, and components shall be plumbed, levelled, and aligned to a tolerance equal to the tolerance permitted for structural steel.

6.5.3 Adjustable connections

Specifically designated more stringent erection tolerances for AESS require that the owner's plans specify/allow adjustable connections between AESS adjoining structural elements, in order to provide the erector with means for adjustment and/or specify the method to be used to achieve the desired dimensions. Any proposed adjustment details desired by the erector shall be submitted to the Design Engineer for review.

Table 27 – Architecturally exposed structural steel matrix

Category	AESS C *Custom elements*	AESS 4 *Showcase elements*	AESS 3 *Feature elements*	AESS 2 *Feature elements*	AESS 1 *Basic elements*	SSS *Standard structural steel*
Characteristics			*Viewed at a distance ≤ 6 m*	*Viewed at a distance > 6 m*		NZS 3404.1
1.1 Surface preparation to Sa2 to AS 1627.4		✓	✓	✓	✓	
1.2 Sharp edges ground smooth		✓	✓	✓	✓	
1.3 Continuous weld appearance		✓	✓	✓	✓	
1.4 Standard structural bolts		✓	✓	✓	✓	
1.5 Weld spatters removed		✓	✓	✓	✓	
2.1 Visual samples		Optional	Optional	Optional		
2.2 One-half standard fabrication tolerances		✓	✓	✓		
2.3 Fabrication marks not apparent		✓	✓	✓		
2.4 Welds uniform and smooth		✓	✓	✓		
3.1 Mill marks removed		✓	✓			
3.2 Butt and plug welds ground smooth and filled		✓	✓			
3.3 RHS/CHS weld seam oriented for reduced visibility		✓	✓			
3.4 Cross-sectional abutting surface aligned		✓	✓			
3.5 Joint gap tolerances minimised		✓	✓			
3.6 All welded connections		Optional	Optional			
4.1 RHS/CHS seam not apparent		✓				
4.2 Welds contoured and blended		✓				
4.3 Surfaces filled and sanded		✓				
4.4 Weld show-through minimised		✓				
C.1						
C.2						
C.3						
C.4						
C.5						
Sample use:	Elements with special requirements	Showcase or dominant elements	Airports, shopping centres, hospitals, lobbies	Retail and architectural buildings viewed at a distance	Roof trusses for arenas, retail warehouses, canopies	
Estimated cost premium:	**Low to high** (20-250 %)	**High** (100-250 %)	**Moderate** (60-150 %)	**Low to moderate** (40-100 %)	**Low** (20-60 %)	**None** 0%

NOTE to Table 27 –

1.1 See 6.3.9.

1.2 Rough surfaces are to be deburred and ground smooth (see 6.3.9).

1.3 Intermittent welds are made continuous, either with additional welding, caulking, or body filler.

1.4 Bolts placed to one side (see 6.5.1 (f)).

1.5 See 6.3.9.

2.1 Visual samples are a 3-D rendering, a physical sample, a first-off inspection, a scaled mock-up, or a full-scale mock-up, as specified in the contract documents.

2.2 These tolerances are required to be one-half of those of standard structural steel as specified in sections 3 and 4.

2.3 Members marked with specific numbers during the fabrication and erection processes are to be made not visible.

2.4 See 6.3.7.

3.1 All mill marks are not to be visible in the finished product.

3.2 Caulking or body filler is acceptable (see 6.3.7 (c)).

3.3 See 6.3.10.

3.4 See 6.3.3 (a).

3.5 This characteristic is similar to 2.2 above. A clear distance between abutting members of 3 mm is required.

3.6 Hidden bolts may be considered.

4.1 HSS/CHS seams are to be treated so they are not visible (see 6.3.10).

4.2 In addition to a contoured and blended appearance, welded transitions between members are also required to be contoured and blended (see 6.3.7).

4.3 See 6.3.6.

4.4 Weld show-through on the back face of the welded element caused by the welding process can be minimised by hand grinding the back face (see 6.3.8).

7 MODIFICATION OF EXISTING STRUCTURES

7.1 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.

C7.1

Most of the material in section 7 and this commentary is taken from 7.5(a) 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 7.5(b).

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. At present there is little guidance on 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 on 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 on past service is not available for estimating the remaining fatigue life. If this is the case, an inspection programme 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 7.5(a), 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.

7.2 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.

C7.2

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 recognise that in older structures, some or all members may be made from either:

(a) Cast iron;

(b) Wrought iron;

(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.

7.3 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.

7.4 Special provisions

7.4.1 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-sectional heating of the member which results from the operation that is being performed.

C7.4.1

If material is added to a member carrying a stress from dead load in excess of 20 MPa, it is desirable (see 7.5(a)) 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. A Certified Welding Engineer may provide guidance.

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.

Under such circumstances, the welding procedures should be adjusted so 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 (http://www.hera.org.nz/nzwc) can advise on suitable welding procedures for this application.

7.4.2 Welding sequence

The welding sequence shall be chosen so as to minimise distortion of the member and ensure that its straightness remains within the appropriate straightness limits of 3.3.3, 3.3.4, 3.3.5, and 3.3.6.

C7.4.2
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 to maintain a symmetrical section at all times.

7.4.3 Replacement of rivets

The replacement of rivets with bolts shall comply with the following:

(a) Existing rivets that are removed to effect a repair or strengthening shall be replaced on a one for one basis with snug-tightened (-/S) high strength bolts of equal or greater diameter; and

(b) Where remaining safe fatigue life is a controlling limit state, existing rivet holes shall be reamed after removal of rivets, and the replacement high strength bolt shall be one size larger in nominal diameter than the replaced rivet. Alternatively if they are the same diameter, they shall satisfy the requirements for an oversize hole, unless the hole is examined and found to contain no significant flaws or stress raisers.

7.4.4 Building-up existing sections of highway and railway bridges

In strengthening existing steel bridge members the added material shall be applied in such a way as to minimise any change in eccentricity of the strengthened member. Where a change in member eccentricity cannot be avoided then the Design Engineer shall take into account the change of eccentricity in determining the resultant member design capacity and fatigue stress range.

C7.4.3 and 7.4.4
The requirements in 7.4.3 and 7.4.4 are based on the recommendations of AREMA Chapter 15.

7.5 References to section 7

(a) American Welding Society. *Structural welding code – Steel*. AWS D1.1-96. 1996.

(b) HERA. *HERA specification for the fabrication, erection and surface treatment of structural steelwork. HERA Report R4-99.* Manukau City: HERA, 1998.

8 INSPECTION OF WELDING AND BOLTING

8.1 Inspection of welding to AS/NZS 1554.1 and AS/NZS 1554.5

The minimum extent of inspection of welding shall be in accordance with the provisions of this section otherwise all inspections shall conform with the requirements of section 7 of AS/NZS 1554.1 and AS/NZS 1554.5.

If a welded structure or component is to be heat treated the final inspection shall be carried out after the heat treatment.

C8.1

In some limited circumstances the welding supervisor may also undertake the inspection of welds in accordance with 8.1. This will most commonly apply to jobs where only visual inspection is required and requires the approval of the Design Engineer.

8.1.1 Extent of non-destructive examination

The minimum extent of non-destructive examination (NDE) is a function of weld failure consequence category and weld demand level.

8.1.1.1 *Weld failure consequence category*

The selection of weld failure consequence category A, B, or C shall be determined by the Design Engineer in accordance with Table 28.

A – Major – Failure of weld would result in collapse of the structure
B – Moderate – Failure of weld would result in loss of service but not collapse
C – Minor – Failure may not cause immediate loss of service but would require remedial work.

Table 28 – Weld and tensioned bolt failure consequence category

A Major	Steel moment frames with low redundancy (four or fewer beams per floor resisting lateral forces in each principal direction). Steel EBF and CBF frames with low redundancy (two frames in each principal direction). Any shear joint supporting gravity loads from two or more floors. Splices resisting applied tension or bending. Fracture critical members of bridges. Crane girders S4 and above classification in accordance with AS 1418.1.
B Moderate	Joints in steel moment frames with redundancy. Joints in EBF and CBF frames with redundancy. Joints in steel moment frames with a designed secondary system for lateral loading. Any shear joint supporting gravity loads from more than one member at a given floor level. Splices resisting only shear or compression or both. Non-FCM portions of bridge girders. Joints in bracing frames of residential houses less than three storeys.
C Minor	Shear, compression and tension joints supporting single members only, and not part of the lateral force-resisting system. Joints not required to carry gravity loads.

8.1.1.2 *Seismic weld demand levels*

Seismic weld demand category shall comply with Table 29.

Table 29 – Seismic weld demand category

Demand	
H High	Welds in which connected member element stresses are expected to be above the yield level, with significant strain hardening development. (Member Categories 1 and 2.)
M Medium	Welds in which connected member element stresses are anticipated to be at or slightly exceed yield level. (Member Category 3.)
L Low	Welds in which stresses are anticipated to remain below yield stresses or will remain in compression. (Member Category 4.)
NOTE – For application of these weld demand categories to various types of seismic resisting systems see Table 30 and Table 31.	

8.1.1.3 *Seismic weld demand categories for MRF, EBF, and CBF*

Weld demand categories have been preselected in accordance with the principles of Table 29 for MRF, EBF, and CBF as tabulated in Table 30 and Table 31.

NOTE –

(1) Inspection of fatigue welds shall also comply with the specific requirements of the relevant fatigue detail classification in Table 7 and Table 8.

(2) The use of UT may be reduced on the basis of the plate thickness limits in Table 32.

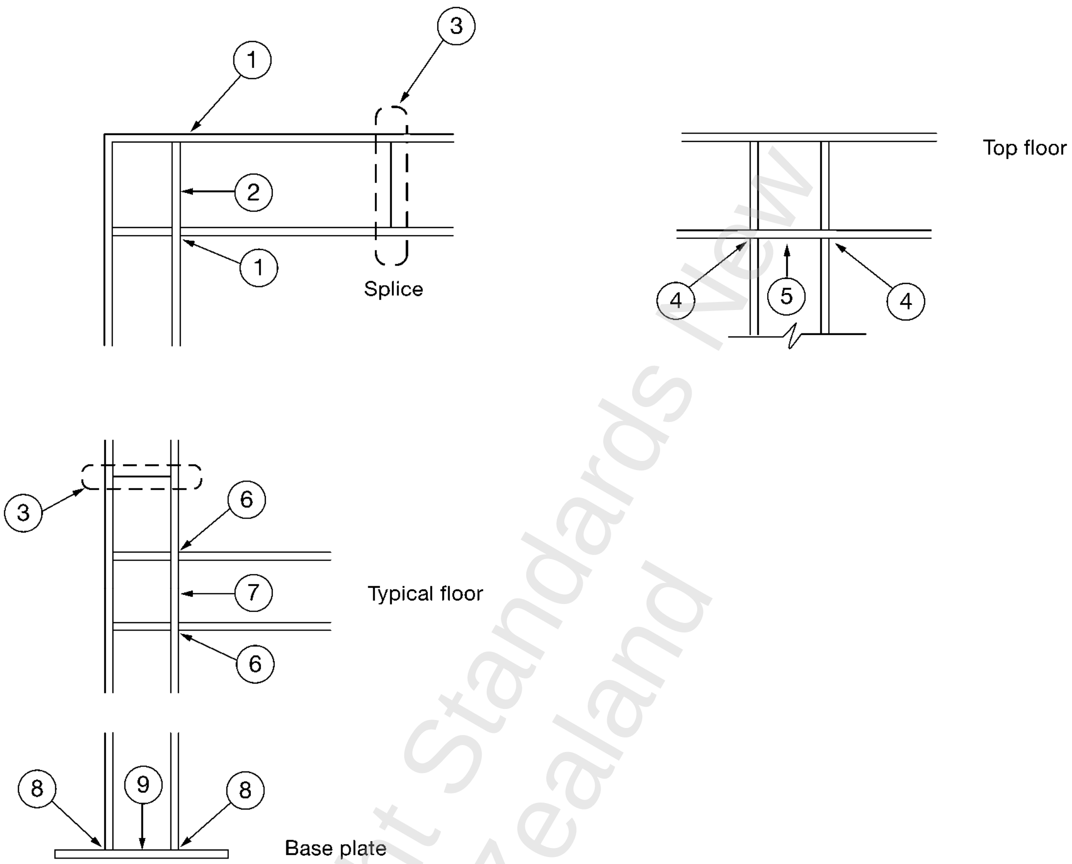


Figure 26 – Moment resisting frame weld location diagram

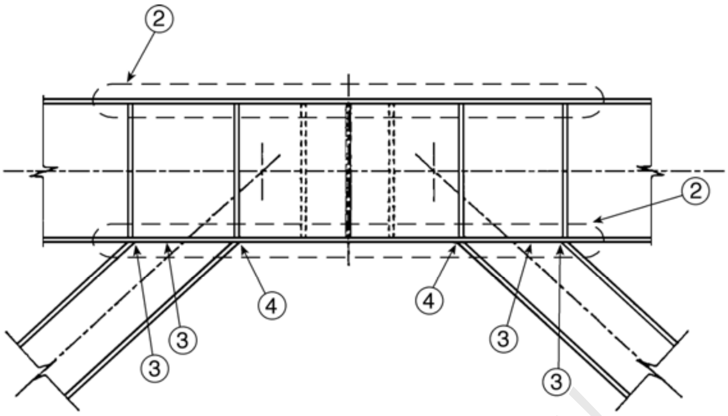
Table 30 – Moment resisting frame weld demand designation

Frame structural ductility category	**Weld location**								
	1	2	3	4	5	6	7	8	9
1, 2	M (H) (see Notes)	M	M	H (M) (see Notes)	M	H	M	H	M
3	M	L	M	M	L	M	L	M	L
4	L	L	L	L	L	L	L	L	L

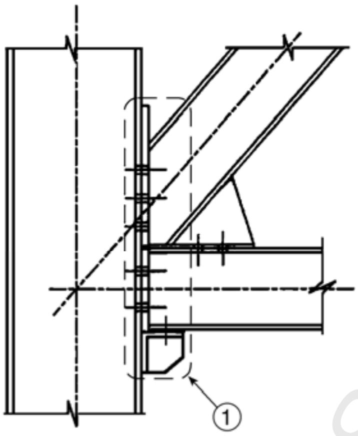
NOTE –

(1) The figure in the bracket applies if the principal yielding region is located in the beam adjacent to the column.

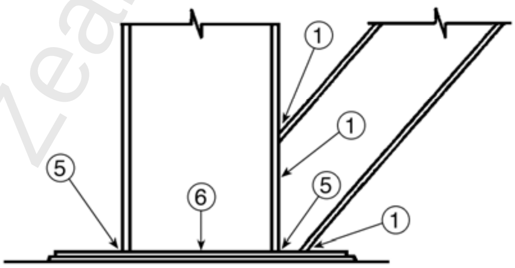
(2) Table 29 describes H, M, and L.



Link zone



Collector beam/Brace connection



Baseplate detail

Figure 27 – EBF weld location diagram

Table 31 – EBF weld demand designation

Frame structural ductility category	Weld location					
	1	2	3	4	5	6
1, 2	M	H (see Note)	M	H	H	M
3	M	M	M	M	M	M
4	L	L	L	L	L	L

NOTE –

(1) The requirement to test web to flange CP welds applies only to welded beams used as active links.

(2) Table 29 describes H, M, and L.

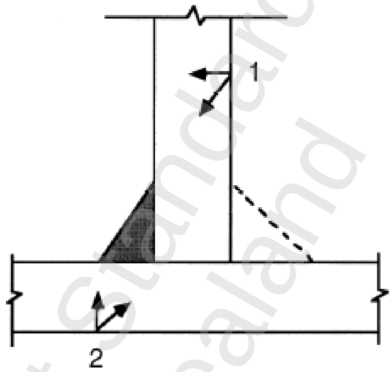


Figure 28 – Inspection access for ultrasonic inspection of fillet welds

8.1.1.4 *Extent of non-destructive examination after non-compliance*

Where a proportion of NDE less than 100% is required and non-compliance results from a test, retesting shall be carried out. Where testing has been done at regular intervals and welder traceability is documented then three consecutive joints shall be inspected. If conformity is achieved then frequency of testing shall reduce back to the inspection levels required in Table 34.

C8.1.1.4
If inspection is not done at regular intervals throughout the fabrication process or welder traceability cannot be mapped to specific joints then more stringent requirements may be required by the Design Engineer.

8.1.1.5 *Inspection of k-areas in members of seismic resisting frames*

When thermal cutting, or welding of doubler plates, continuity plates, or welding of stiffeners has been performed in the *k*-area, the web shall be tested for cracks using MT testing. The MT inspection area shall include the *k*-area base metal within (75 mm) of the weld or cut, see Figure 29.

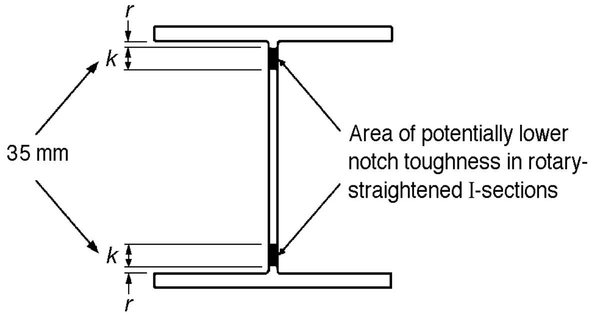


Figure 29 – Location of *k*-area zone

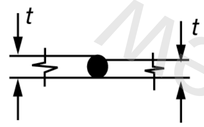
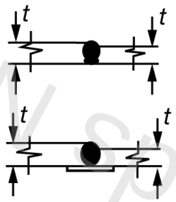
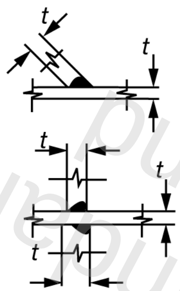
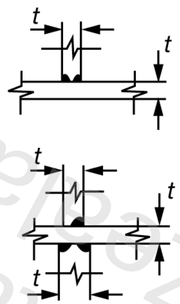
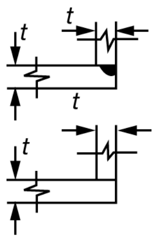
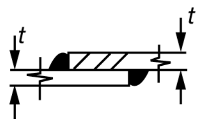
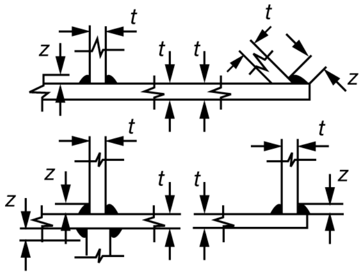
C8.1.1.5

The k-area of rotary straightened wide-flange sections may have reduced notch toughness. Where welds are to be placed in the k-area, inspection of these areas is needed to verify that cracking has not occurred. For doubler plates, where welding in the k-area is performed, MT in the k-area should be performed on the side of the member web opposite the weld location, and at the end of the weld. If both sides of the member web receive doubler plates in the k-area, MT of the member web should be performed after welding of one side, prior to welding of the opposite side. Cracking in the k-area is known to occur in a delayed manner, typically within 24 to 48 hours after welding. The cracks generally, but not always, penetrate the thickness of the base metal.

8.1.1.6 *Ultrasonic testing at welds*

For the minimum plate thickness requiring ultrasonic or radiographic testing (UT) at welds, see Table 32.

Table 32 – Thickness of plate requiring ultrasonic testing

Weld type	**Butts full and partial penetration (including butts with reinforcing fillets)**					**Fillet**		
Joint type	**In-line butt**		**Tee and cruciform**		**Corner**	**Lap**	**Tee, cruciform, and corner**	
Procedures	S/S	D/S and S/S+B	S/S	D/S and S/S+B	All	All	All	
Key *t* = greatest *t* at a joint *z* = greatest *z* at a joint S/S = single sided D/S = double sides +B = backing NM = not mandatory	**Examples** 							
NDT method	*t* (mm)	*t* (mm)	*t* (mm)	*t* (mm)	*t* (mm)	*t* (mm)	*t* (mm)	*z* (mm)
Ultrasonic testing (UT)	≥ 10	≥ 12	≥ 12	≥ 20	≥ 30	NM	NM	≥ 20

NOTE – The requirements of this table shall not preclude the use of non-destructive testing (NDT) outside the limits shown should the results of visual inspection of NDT show that a lapse in quality may have occurred in specific joints.

8.1.1.7 *Visual means*

The extent of NDE by visual means shall be in accordance with Table 33 and Table 34.

Table 33 – Extent of non-destructive examination for welds to AS/NZS 1554:Parts 1 and 5 (visual means)

Weld category	Weld failure consequence	Visual scanning	Visual examination
GP	C	100	25
SP	B or C	100	25
	A	100	100
Fatigue	A or B	100	100

NOTE –

Visual means of NDE implies two levels of examination:

(1) Visual scanning – To determine that no welds called for in the drawings are omitted and to detect gross defects.

(2) 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.

8.1.1.8 *Other NDT (ultrasonic, magnetic particle, liquid penetrant)*

The extent of NDE other than visual means shall be in accordance with Table 34.

C8.1.1.8

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 28. Scanning from position 1 may be limited when fillet welds to both sides of the joint are made.

8.1.2 Removal of temporary attachments

For the removal of temporary attachments refer to the provisions of 5.9 of AS/NZS 1554.1 or AS/NZS 1554.5, whichever is applicable.

Table 34 – Extent of non-destructive examination for welds to AS/NZS 1554:Part 1 (ultrasonic or radiographic (UT), magnetic particle (MT), liquid penetrant (LP) [1])

Weld failure consequence	Non-seismic	Seismic demand			Fatigue (2,3)
		H High	**M Medium**	**L Low**	
A Major	CPW UT 10% of joints	CPW UT 100% *k*-area MT 100% Fillet Weld MT 25%	CPW UT 25% including 10 of the first 10 joints *k*-area MT 100%	CPW UT 10% including 2 of the first 10 *k*-area MT 10%	CPW UT 100% *k*-area MT 100% Fillet Weld MT 25%
B Moderate	Nil	CPW UT 25% including 10 of the first 10 joints *k*-area MT 100%	CPW UT 25% including 10 of the first 10 joints *k*-area MT 100%	Nil	CPW UT 25% Including 10 of the first 10 joints *k*-area MT 100% Fillet Weld MT 10%
C Minor	Nil	CPW UT 25% including 10 of the first 10 joints *k*-area MT 10%	CPW UT 10% joints including 2 of the first 10 joints *k*-area MT 10%	Nil	

NOTE –

(1) The use of MT or LP is used for supplementary inspection in conjunction with visual inspection.

(2) Inspection of fatigue welds shall also comply with the specific requirements of the relevant fatigue detail classification in Table 7 and Table 8.

(3) The use of UT may be reduced on the basis of the plate thickness limits in Table 32.

8.2 Inspection of bolted connections

8.2.1 Traceability of bolts

Mill certificates shall be provided on request by the Design Engineer for tensioned bolts in the connections with moderate or high consequences of failure in Table 28.

8.2.2 Tensioned bolts

The methods of tensioning specified in 4.2.6 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 shown directly by the device; and

(c) Where the part-turn method has been used and the nut and bolt threads have been match-marked after snug-tightening, then visual scanning of each bolted connection is sufficient. Visual scanning shall also be sufficient if direct-tension indicating washers have been used. However if there is no visual evidence of tensioning then the Design Engineer may require verification of bolt tensioning in accordance with 8.2.3.

C8.2.2

The manufacturer's recommendations for inspection procedures should be followed when using a direct-tensioning indication device.

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.

8.2.3 Verification of bolt tension

Bolt tensioning may be verified by removal of a sample of bolts and measurement of the thread spacing within the installed grip zone beneath the base of the nut and the unthreaded portion of the bolt shank. If permanent stretch in the spacing of those threads is detected, compared to the spacing of the other threads in the bolt, then the bolt may be considered to have been adequately tensioned.

Alternatively suitably calibrated ultrasonic bolt extension testing equipment may be used.

Bolts that have been loosened off for verification of tensioning and have been found to have been permanently stretched shall be replaced with new bolts. Bolts shall only be removed for verification from locations approved by the Design Engineer.

C8.2.3

Several manufacturers produce equipment specifically for this application. The use of appropriate techniques, which include calibration, can produce a very accurate measurement of the actual pretension. The method involves measurement of the change in bolt length during the release of the nut, combined with either a load calibration of the removed fastener assembly or a theoretical calculation of the force corresponding to the measure elastic release or 'stretch'. Mechanical measurement can be done with the use of vernier callipers to measure the thread stretch. Reinstallation of the released bolt or installation of a replacement bolt is required.

The number of bolts to be checked in each connection depends on the construction reviewer or their representative. A suitable sample would consist of 10% of the bolts, but not fewer than two bolts at each connection selected at random.

8.2.4 Damaged items

Bolts, nuts, and washers which, on visual inspection, show any evidence of physical defects shall be removed and replaced by new items.

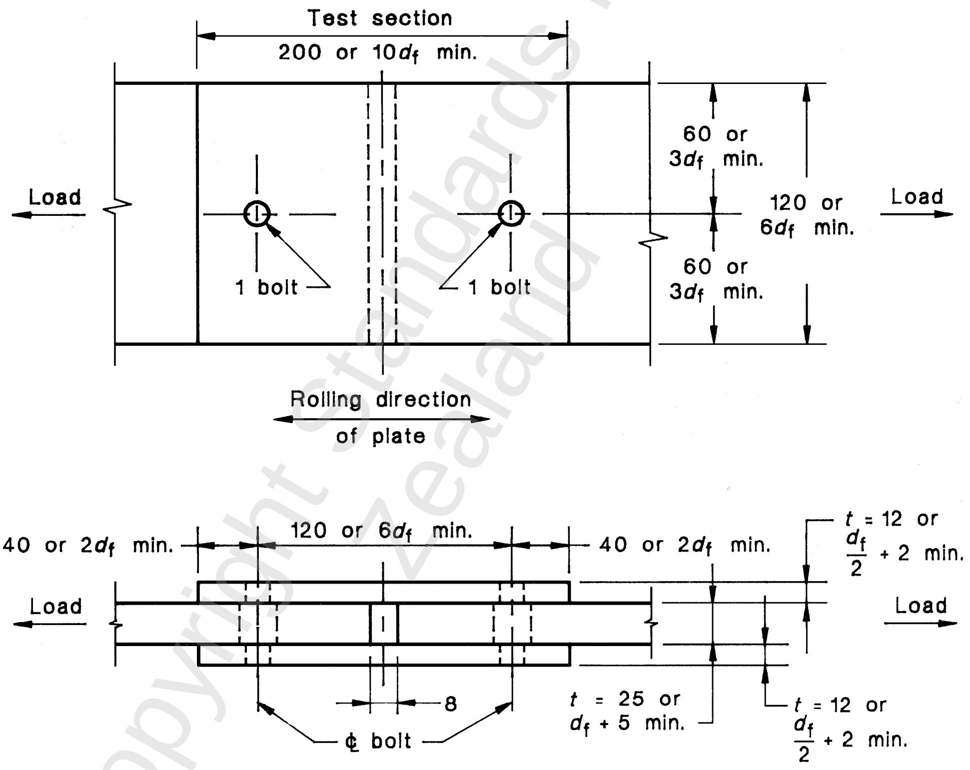
8.3 Standard test for evaluation of slip factor

8.3.1 Test specimens

8.3.1.1 *Form*

The standard test specimen shall be symmetrical double coverplated butt connection as shown in Figure 30. 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 a 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 30 – Standard test specimen

8.3.1.2 *Assembly and measurement*

8.3.1.2.1

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 not be 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 12.

8.3.1.2.2

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.

8.3.1.2.3

Bolt tension shall be ascertained from a calibration curve determined from load cell tests of at least three 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.

8.3.1.2.4

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:

$$N_{ti} = \frac{E\Delta \times 10^{-3}}{\dfrac{a_o}{A_o} + \left(\dfrac{a_t + \dfrac{t_n}{2}}{A_s}\right)} \quad \text{...Eq. 8.3.1.2.4}$$

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.

C8.3.1.2.4

The method adopted is that developed in Lay and Schmidt (see 8.4(a)).

8.3.1.3 *Number of specimens*

Tests on at least three specimens shall be undertaken, but five are preferred as a reliable minimum number.

8.3.2 Instrumentation

8.3.2.1

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.

8.3.2.2

Figure 31 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.

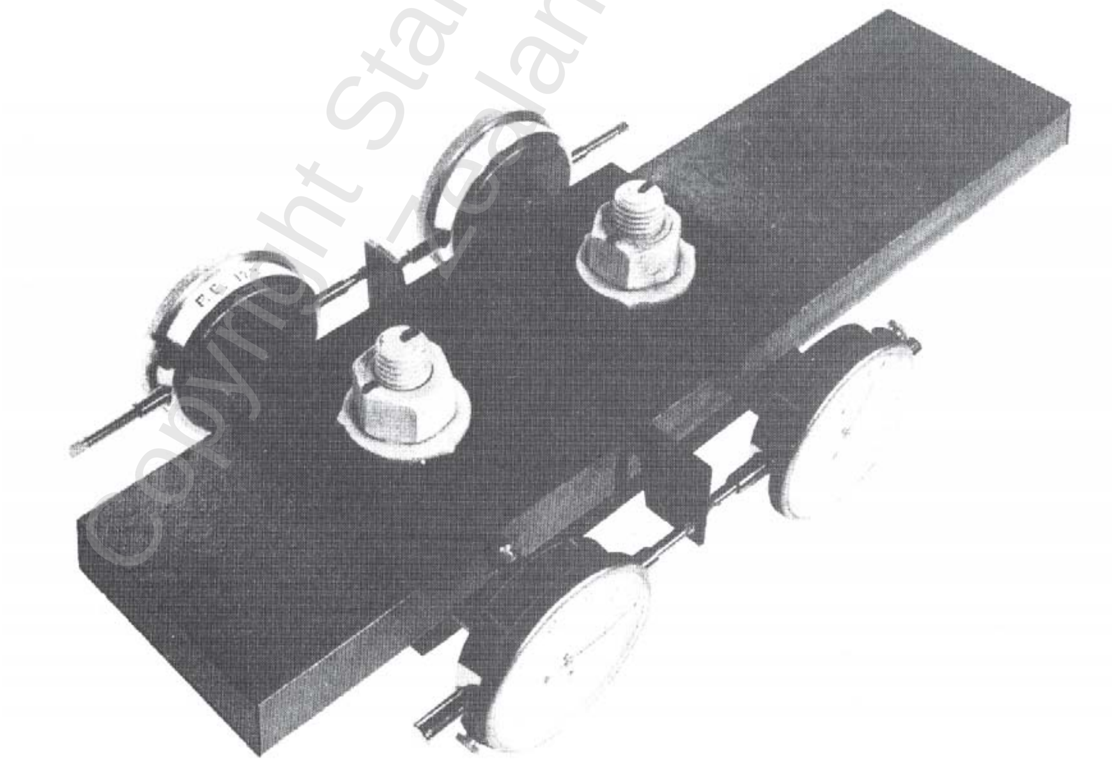


Figure 31 – Typically instrumented test assembly

8.3.3 Method of testing

The method of testing shall satisfy the following requirements:

(a) *Type of loading*
Specimens shall be tested only by tensile loading; and

(b) *Loading rate*
Up to the slip load, force shall be applied neither in increments exceeding 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 as 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.

8.3.4 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.

8.3.5 Slip factor

The slip factor (μ_s) to be used in design shall be calculated from:

$$\mu_s = k\,(\mu_{sm} - 1.64\delta) \quad \text{.......... Eq. 8.3.5}$$

where

k = 0.85 when 3 specimens are tested; or
= 0.90 when 5 or more specimens are tested

μ_{sm} = mean value for slip factor for all tests

δ = standard deviation of slip factor for all tests

$$\mu_{sm} = \frac{1}{2\,n}\left(\frac{V_{si}}{N_{ti}}\right)$$

$$\mu_{si} = \frac{1}{2}\left(\frac{V_{si}}{N_{ti}}\right)$$

$$\delta = \sqrt{\left[\frac{1}{2n-1}\sum_{I=1}^{2n}(\mu_{si}-\mu_{sm})^2\right]}$$

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 Eq. 8.3.1.2.4

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}.

8.4 References to section 8

(a) Lay, M G, and Schmidt, L C. *Co-operative slip factor tests for HSFG joints. BHP Melbourne. Res. Lab. Rep. MRL 17/12.2*. 1972.

9 COMPOSITE CONSTRUCTION

9.1 Installation of stud shear connectors

9.1.1 General

Installation of stud shear connectors shall take the following into account:

(a) The diameter of the head of a headed stud shall be not less than 1.5 times the diameter of the shank d_{sc} and a depth not less than $0.4d_{sc}$;

(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 20 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;

(d) The horizontal distance between a free concrete surface (with or without steel decking) and the centre-line of a shear connector shall be not less than $6d_{sc}$. For cases when the horizontal distance between a free concrete surface (with or without steel decking) and the centre-line of a shear connector is less than 300 mm, transverse U-bars with a diameter of not less than $0.5d_{sc}$ shall be placed as low as practicably possible below the head of the stud connector developed in accordance with the requirements of the Design Engineer;

(e) The minimum concrete cover to the top of a shear connector shall be as specified for the reinforcing steel, 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 six diameters along the longitudinal axis of the supporting composite beam and four diameters transverse to the longitudinal axis of the supporting composite beam;

(g) The longitudinal spacing of channel shear connectors shall have a minimum clear distance of 100 mm or the height of the connector, whichever is the greater, between adjacent connectors;

(h) The maximum longitudinal spacing of shear connectors shall not exceed four times the total slab thickness or 600 mm in regions of a composite member not subject to inelastic earthquake effects and four times the total slab thickness or 400 mm in regions subject to inelastic earthquake effects (yielding regions); and

(i) In yielding regions of a composite member, stud shear connectors welded to the beam flange shall be staggered about the web centre-line.

C9.1.1

The following relate to 9.1.1(a), (c), (f), and (i):

(a) A short welded shear stud develops rotations at the head of the stud. Therefore a head on the stud is essential in developing its strength;

(c) This prevents 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, for example, in bridge design, more stringent limits are recommended by 9.3(a), (b), and (c);

(f) This prevents localised overstressing of the slab concrete from overlap of the zone of high shear stress around an individual shear connector (see 9.3(b)); and

(i) This 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 (see 9.3(d)).

9.1.2 Stud shear connectors used with profiled steel deck

This section and Figure 32 apply to decks with nominal rib height not exceeding 80 mm. Installation of stud shear connectors used with profiled steel decks shall take the following into account:

(a) The concrete slab shall be connected to the steel beam with welded stud shear connectors 22 mm or less in diameter or equivalent studs. Welded studs may be welded through the deck or directly to the steel member;

(b) Stud shear connectors shall extend not less than $1.8d_{sc}$ above the top of the steel deck (as defined in Figure 32) after installation;

(c) The slab thickness above the steel deck shall be not less than 50 mm; and

(d) 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.

For trapezoidal profiles, the depth of the sheet may be calculated excluding any small re-entrant stiffener to the crest of the sheet, provided that the width of the crest of the profile b_r is not less than 110 mm and the stiffener does not exceed 15 mm in height and 55 mm in width.

9.1.3 Welding Standards for shear connectors

Welding of shear connectors shall be in accordance with AS/NZS 1554.1, except that welding of proprietary steel studs shall be in accordance with AS/NZS 1554.2.

9.1.4 Lateral restraint of steel beam by decking

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.

9.2 Concrete encased steel columns

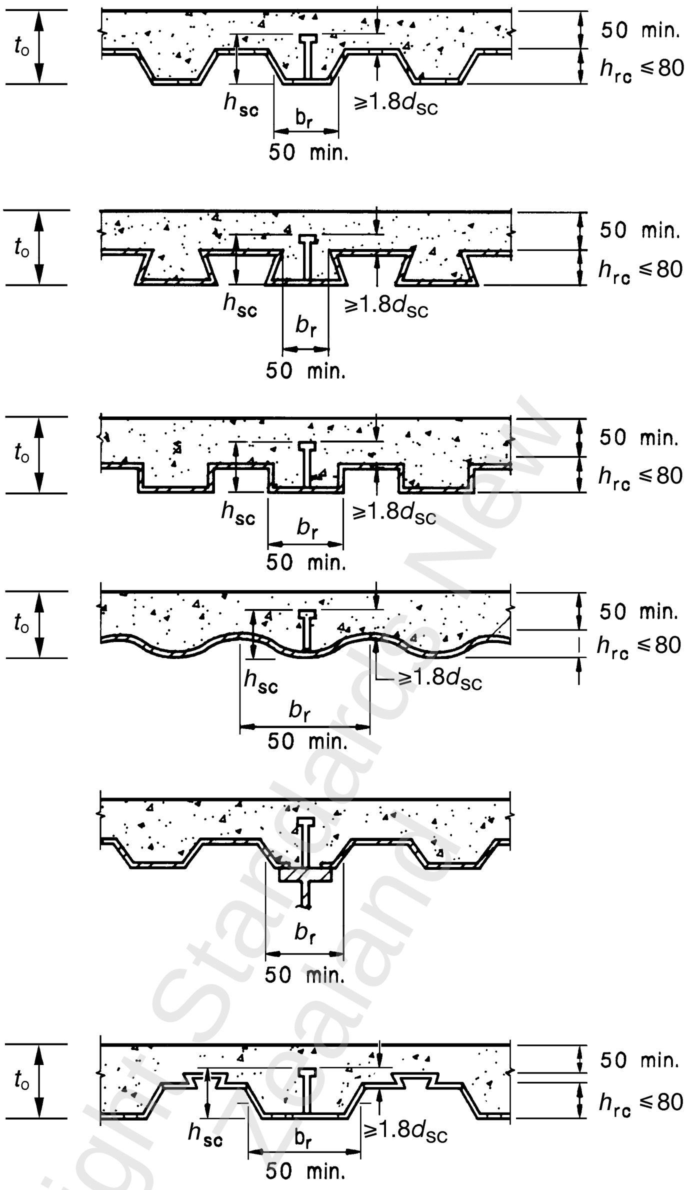
An encased composite column incorporating concrete transversely reinforced with lateral ties, spiral or circular hoop reinforcement, detailed in accordance with NZS 3109, shall conform to the following:

(a) Specified concrete cylinder compression strength, f'_c, shall be not less than 20 MPa nor greater than 60 MPa;
(b) Lateral ties shall extend completely around the structural steel core;
(c) Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 tie bar diameters, or half the least side dimension of the composite member;
(d) Each lateral tie shall be anchored by at least a 135° stirrup hook;
(e) 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° or from a cross-tie is provided to every corner longitudinal bar; and other bars as specified; and
(f) A longitudinal bar shall be located at every corner of a rectangular cross section, with other longitudinal bars spaced not further apart than half the least side dimension of the composite member.

C9.2

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 obtained from 9.3(d) and (e) 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 and cross-reference to others. The 1982 edition of NZS 3101 was revised into limit state format as NZS 3101:2006. Changes to clause numbering and content of NZS 3101 have necessitated changes to this Standard, which have been introduced in this revision.



All dimensions are in mm.

Figure 32 – Detailing/design limitations on stud shear connectors with profiled steel deck

9.3 References to section 9

(a) AISC. *Manual of steel construction load and resistance factor design.* 3rd ed. Chicago: AISC, 2001.

(b) Chien, E Y L, and Ritchie, J K. *Design and construction of composite floor systems.* Toronto, Canada: Canadian Institute of Steel Construction, 1984.

(c) Globe, G G. 'Shear strength of thin-flange composite specimens.' *Engineering Journal*, AISC 5. (April 1968).

(d) Clifton, G C. 'Composite design. Section I of the deliberations of the study group for the seismic design of steel structures.' *NZNSEE Bulletin* 18, no. 4 (1985).

(e) HERA. *New Zealand structural steelwork design guides volume 2, incorporating amendment no. 3. HERA Report R4-49.* Manukau City: HERA, 1990.

Appendix A –
Fracture control plan (FCP) for railway bridges

(Normative)

A fracture control plan shall be prepared prior to commencing fabrication identifying how fabrication will be controlled in accordance with the following:

(a) *Designation of FCMs*
The drawings shall designate the location of any FCMs in accordance with 1.7.2;

(b) *Design Engineer's review*
The Design Engineer shall review and record disposition to all shop drawings and welding procedure specifications (WPS) for construction and repair to ensure that the contractor has identified all FCMs and complied with the FCP;

(c) *Materials*
Steel shall conform with 2.2.5;

(d) *Welding processes*
Welding shall be in accordance with 3.3.4;

(e) *Consumables requirements*
Welding consumables shall comply with 2.6.4.5;

(f) *Welding procedure specification*
WPSs shall conform to the requirements of AS/NZS 1554.1 and AS/NZS 1554.5 as necessary;

(g) *Quality control systems and qualification*
Steelwork contractors shall have quality control systems in accordance with AS/NZS ISO 3834.3 or equivalent. Welders shall be qualified in accordance with AS/NZS 1554.1 to weld the procedures they execute;

(h) *As-received inspection of materials*
All parent steel surfaces and edges shall be visually inspected for imperfections in accordance with AS/NZS 3679.1 for hot-rolled sections, AS/NZS 3679.2 for welded I-sections and AS/NZS 3678 for plate;

(i) *Thermal cutting*
Thermal cutting of edges shall comply with the requirements of 3.2.2.3;

(j) *Repair of parent metal laminar discontinuities*
Parent metal laminar discontinuities adjacent to fracture-critical butt joints shall be repaired or replaced to conform with the following options:

   (i) *Rotation of parent metal*
   The parent metal may be rotated end for end when it is possible to remove discontinuities from areas subject to gross tensile design stresses

   (ii) *Thermal cutting*
   When the unacceptable laminar discontinuities are localised, these shall be removed by thermal cutting to sound metal, reducing the length of the affected plate, bar, or shape. This requires adjacent pieces of steel to be extended beyond their detailed length and mandates relocation of

the affected butt joint. Relocation of butt welds from their detailed position shall require approval from the Design Engineer. All changes in weld location shall be recorded on the shop drawings

(iii) *Repairs*
Repairs may be made by welding in accordance with 3.2.3

(iv) *Replacement*
With the Design Engineer's approval, a defective portion of the parent metal may be removed and replaced with new material of the same grade. Unless approved otherwise by the Design Engineer, the minimum replacement length shall be 1.5 m. All parent metal replacements shall be recorded in the inspection records and shop drawings;

(k) *Straightening, curving, and cambering*
Straightening, curving, and cambering procedures shall comply with 3.2.1;

(l) *Tack welds and temporary welds*
Track welds and temporary welds shall comply with the following requirements:

(i) *Tack welds*
All tack welds shall be located within the joint unless otherwise approved by the Design Engineer

(ii) *Temporary welds*
All welds not shown as permanent welds on the drawings or approved by the Design Engineer shall be removed

(iii) *Weld removal*
When required weld removal shall include all of the weld plus 3 mm of the adjacent parent metal to remove the heat affected zone. Weld and parent metal removal sites shall be faired to adjacent surfaces on a slope not steeper than 1 into the metal to 10 along the surface. The surface roughness shall comply with requirements of Table 6;

(m) *Preheat and interpass control*
Preheat and interpass control shall comply with the requirements of the approved WPS used in accordance with AS/NZS 1554.1 or AS/NZS 1554.5;

(n) *Postweld thermal treatments*
Postweld thermal treatment shall comply with the requirements of the approved WPS used in accordance with AS/NZS 1554.1 or AS/NZS 1554.5;

(o) *Weld inspection*
Inspection of welding shall comply with 8.1;

(p) *Repair welding*
Repair welding shall comply with the requirements of AS/NZS 1554.1 or AS/NZS 1554.5.

Appendix B – List of superseded clauses from NZS 3404.1 and NZS 3404.2:1997

(Informative)

This Appendix lists the corresponding superseded clause (or table, or figure) and related commentary in NZS 3404.1 and NZS 3404.2:1997 (including Amendment No. 1 and No. 2). This identification is made for the purpose of allowing the remaining clauses of NZS 3404:1997 to be used in conjunction with the provisions of this Standard.

Existing clause and commentary in NZS 3404:1997	Updated clause in NZS 3404.1:2009
1 GENERAL AND SCOPE	1 GENERAL
1.1 Scope	1.1 Scope
1.1.1	1.1.1
1.1.2	1.1.1
1.1.4 (a), (b)	1.1.2
1.2 Referenced documents	Referenced documents
1.3 Definitions	1.4 Definitions
1.4 Notations	1.5 Notations
1.6 Design and construction review	1.7 Design and documentation
	1.8 Construction review
2 MATERIALS AND BRITTLE FRACTURE	2 MATERIALS AND BRITTLE FRACTURE
2.1 Yield stress and tensile strength used in design	2.1 Yield stress and tensile strength used in design
2.2 Structural steel	2.2 Structural steel
2.2.1	2.2.1
2.2.2	2.2.2
2.2.3	2.2.3
2.3 Fasteners	2.3 Fasteners
2.4 Steel castings	2.4 Steel castings
2.5 Concrete	2.5 Concrete
2.6 Material selection to suppress brittle fracture	2.6 Material selection to suppress brittle fracture
2.6.1	2.6.1
2.6.2	2.6.2
2.6.3	2.6.3
2.6.3.1	2.6.3.1
2.6.3.2	2.6.3.2
Figure 2.6.3.1	Figure 1
2.6.4	2.6.4
Table 2.6.4.1	Table 5
Table 2.6.4.4	Table 2

Existing clause and commentary in NZS 3404:1997	Updated clause in NZS 3404.1:2009
10 FATIGUE	
Table 10.5.1(2)	Table 7
Table 10.5.1(4)	Table 8
10.9 Punching limitation	3.2.5.1.5
12 SEISMIC DESIGN	
Table 12.4	Table 1
	Table 3
12.4 Material requirements	
12.4.1	2.2.4
12.4.1.2	Deleted
12.4.2	Deleted
12.9.4.5.2	Deleted
12.9.4.5.3	3.2.5.1.6
12.14.1.1	2.2.4
12.14.1.2	3.2.2.4
12.14.2	3.2.5.1.6
12.14.3	3.2.4.2
12.14.4	3.2.4.2
13 DESIGN OF COMPOSITE MEMBERS AND STRUTURES	9 COMPOSITE CONSTRUCTION
13.3.2.3	9.1.1
13.3.1.1.1(c)	9.1.2(a)
13.3.2.2.1(d)	9.1.2(b)
13.3.2.2.1(e)	9.1.2(c)
13.3.2.2.1(f)	9.1.2(d)
14 FABRICATION	3 FABRICATION
14.1 Rejection of a fabricated item	3.4 Rejection of a fabricated item
14.1.1	3.4 Rejection of a fabricated item
14.1.2	Deleted
14.2 Material	3.1 Material
14.3.1.1	3.2.1.1
14.3.1.2	3.2.1.1
14.3.2.1	3.2.7.2
14.3.2.2	3.2.7.1
14.3.3	3.2.2
14.3.4	3.2.3
14.3.5	3.2.5
14.3.6.1.1	4.2.4.1.1
14.3.6.1.2	4.2.4.1.2
14.3.6.1.3	4.2.4.1.3

Existing clause and commentary in NZS 3404:1997	Updated clause in NZS 3404.1:2009
14.3.6.1.4	4.2.4.1.3
14.3.6.1.5	4.2.4.1.6
14.3.6.2	4.2.4.2
14.3.6.3	4.2.4.3
14.3.7	3.2.6
14.4.1	3.3.1
14.4.2	3.3.2
14.4.3	3.3.3
Figure 14.4.3.1	Figure 4
Figure 14.4.3.2	Figure 5
Figure 14.4.3.3	Figure 6
Figure 14.4.3.4	Figure 7
Figure 14.4.3.5	Figure 8
14.4.4.1	3.3.4.1
14.4.4.2	3.2.7.1
	4.3.4(d)
14.4.4.3	3.3.4.2
14.4.5.1	3.3.5.1
Figure 14.4.5.1	Figure 9
Table 14.4.5	Table 9
14.4.5.2	3.3.5.2
14.4.6.1	3.3.6.1
14.4.6.2	3.3.6.2
15 ERECTION	4 ERECTION
15.1.1.1	4.5.1
15.1.1.2	Deleted
15.1.1.3	4.5.2
15.1.1.4	4.5.3
15.1.2	4.1.1
15.1.3	4.1.3
15.1.4	4.1.4
15.2.1	4.2.1
15.2.2	4.2.2
15.2.3.1	4.2.3.1
15.2.3.2	4.2.3.2
15.2.3.3	4.2.4.1.3
15.2.3.4	4.2.4.1.5
15.2.3.5	4.2.4.1.4
15.2.4.1	4.2.5.1
15.2.4.2	4.2.5.2
15.2.4.3.1	4.2.5.3.1

Existing clause and commentary in NZS 3404:1997	Updated clause in NZS 3404.1:2009
15.2.4.3.2	4.2.5.3.2
15.2.4.4	4.2.5.4
15.2.5.1.1	4.2.6.1.1
15.2.5.1.2	4.2.6.1.2
15.2.5.2	4.2.6.2
15.2.5.3	4.2.6.3
15.3.1	4.3.1
15.3.2	4.3.2
15.3.3	4.3.3
15.3.4	4.3.4
15.3.5	4.3.5
15.3.6	4.3.6
15.3.7	4.3.9
15.4.1	8.2.2
15.4.2	8.2.4
15.5 Grouting at supports	4.4 Grouting at supports
16 MODIFICATION OF EXISTING STRUCTURES	7 MODIFICATION OF EXISTING STRUCTURES
16.1 General	7.1 General
16.2 Materials	7.2 Materials
16.3 Cleaning	7.3 Cleaning
16.4 Special provisions	7.4 Special provisions
APPENDIX A – REFERENCED DOCUMENTS	Referenced documents
APPENDIX C – CORROSION PROTECTION	5 CORROSION PROTECTION
C1.1	5.1 Scope
C1.2	5.1.1
C2.1	5.2.2
C2.2	5.2.1
C2.3	5.1.2
C3 Standards	5.1.4
C4 Inaccessible surfaces and treatment of cut edges	3.2.2.5
	5.1.3
	5.5 Inaccessible surfaces
C5 Protection during transport and handling after corrosion protection	4.2.2.1
	5.6 Protection during transportation and handling after corrosion protection
	6.4 Delivery of materials
	6.5.1
C6 Repairs to corrosion protection	5.7 Repairs to corrosion protection

Existing clause and commentary in NZS 3404:1997	**Updated clause in NZS 3404.1:2009**
C7 Relevant Standards and other documents	Referenced Documents
APPENDIX D – INSPECTION OF WELDING TO AS/NZS 1554.1	8 INSPECTION OF WELDING AND BOLTING
D1.1	8.1 Inspection of welding to AS/NZS 1554.1 and AS/NZS 1554.5
D1.2	Deleted
D1.3	Deleted
D2 Extent of non-destructive examination	8.1.1
APPENDIX K – STANDARD TEST FOR EVALUATION OF SLIP FACTOR	8.3 Standard test for evaluation of slip factor
K1 Test specimens	8.3.1
K2 Instrumentation	8.3.2
K3 Method of testing	8.3.3
K4 Slip load	8.3.4
K5 Slip factor	8.3.5
APPENDIX L – INSPECTION OF BOLT TENSIONING USING A TORQUE WRENCH	8.2.3

NOTES

NOTES

Approved by the Standards Council on 11 September 2009 to be a New Zealand Standard pursuant to the provisions of section 10 of the Standards Act 1988.

First published: 13 October 2009

The following Standards New Zealand references relate to this Standard:

Project No: P 3404
Draft for comment No. DZ 3404.1
Typeset by: Standards New Zealand
Printed by: The Colour Guy