PART C

C6

Structural Steel Buildings

Document Status

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This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

Document Access

This document may be downloaded from www.building.govt.nz in parts:

1 Part A – Assessment Objectives and Principles

2 Part B – Initial Seismic Assessment

3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

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Contents

C6. Structural Steel Buildings

C6.1 General

C6.1.1 Scope and outline of this section

This section provides guidance on the Detailed Seismic Assessment (DSA) of existing steel framed buildings. It does not address earthquake damaged steel framed buildings or the retrofitting of existing buildings.

The following topics are addressed in this section:

- Factors affecting the seismic performance of steel buildings and their observed behaviour in past earthquakes (Sections C6.2 and C6.3)
- Structural steel material properties and testing (Section C6.4)
- Assessment of member and connection probable strength and deformation capacities (Sections C6.5 and C6.6)
- Philosophy and assumptions for the evaluation of existing steel seismic-resisting systems, including the evaluation procedure for steel moment resisting frames (MRFs), steel MRFs with infill panels, and braced frame buildings (Sections C6.7 and C6.8).

C6.1.2 Useful publications

The following publications will be of particular assistance to designers making seismic assessment of steel framed buildings.

ASCE 41-13 (2014). *Seismic evaluation of existing buildings*, American Society of Civil Engineers, and Structural Engineering Institute, Reston, Virginia, USA.

Clifton, G.C. and Cowie, K. (2013). *Seismic design of eccentrically braced frames,* HERA Publication P4001:2013.

Clifton, G.C. and Ferguson, W.G. (2015). *Determination of the post-earthquake capacity of an eccentrically braced frame seismic resisting system*, The University of Auckland, report to the Natural Hazards Research Platform.

Feeney, M.J. and Clifton, G.C. (2001). *Seismic design procedures for steel structures*, HERA Report R4-76, Manukau City, NZ. HERA, 1995. To be read with Clifton, G.C.; *Tips on Seismic Design of Steel Structures*, Notes from Presentations to Structural Groups mid-2000; HERA, Manukau City, 2000.

FEMA 273 (1997). *NEHRP guidelines for the seismic rehabilitation of buildings*, Federal Emergency Management Agency, FEMA Report 273, Washington, DC.

FEMA 356 (2000). *Prestandard and commentary for the seismic rehabilitation of buildings*, Federal Emergency Management Agency, FEMA Report 356, Washington, DC.

NZS 1170.5:2004. *Structural design actions, Part 5: Earthquake actions - New Zealand*, NZS 1170.5:2004. Standards New Zealand, Wellington, NZ.

NZS 3404 Part 1:1997. *Steel structures standard, incorporating Amendments 1 and 2,* NZS 3404:1997. Standards New Zealand, Wellington, NZ.

C6.1.3 Definitions and acronyms

Category 1 buildings	Fully ductile buildings ($\mu > 3$)
Category 2 buildings	Limited ductile buildings ($1.25 < \mu \leq 3$)
Category 3 buildings	Nominally ductile buildings ($1 < \mu \leq 1.25$)
Category 4 buildings	Elastic buildings ($\mu = 1$)
Concentrically braced frame (CBF)	A braced frame where the members are subjected primarily to axial forces
Connection	The entire assemblage of connection components and connectors where two members intersect
Connector	An item within a connection that transfers forces from one member or connection component to another (e.g. bolts, rivets and welds)
Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part C of these guidelines
Eccentrically braced frame (EBF)	A braced frame in which at least one end of each brace frames only into a beam in such a way that at least one stable, deformable link beam is formed in each beam if the elastic limit of the frame is exceeded. In this event, energy is dissipated through shear and/or flexural yielding in the link beams (termed the active link regions) and the bracing members and columns have sufficient capacity to remain essentially elastic.
Full restraint against lateral buckling (FLR)	Restraint that effectively prevents lateral deflection and twist of a member
Lateral force-resisting system	The part of a structural system that provides resistance to earthquake induced forces
Lateral restraint	An element that prevents lateral movement of the critical flange of a member
Local buckling	A local instability involving a change of shape of the member cross section along a relatively short length of member under compression
Moment resisting frame (MRF)	A building frame system in which lateral loads are resisted by shear and flexure in members and joints of the frame
Overstrength	The maximum strength that a member or a connection can develop due to variations in material strengths, and strength gain due to strain hardening, if applicable
Plate slenderness	The ratio of the critical unsupported width of a steel plate to the average plate thickness
Primary seismic-resisting member	An energy dissipating member of a seismic-resisting system
Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects this is determined using probable material strengths. For geotechnical issues the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
Rolled steel joist (RSJ)	I-sections that have tapered flanges
Segment	The length between adjacent cross sections which are fully, partially or laterally restrained, or the length between an unrestrained end and the adjacent cross section which is fully or partially restrained
Tensile strength	The probable breaking strength in tension

Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings
XXX%ULS shaking (demand)	Percentage of the ULS shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members/elements for the same site. For general assessments 100%ULS shaking demand for the structure is defined in the version of NZS 1170.5 (version current at the time of the assessment) and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016. For engineering assessments undertaken in accordance with the EPB methodology, 100%ULS shaking demand for the structure is defined in NZS 1170.5:2004 and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016 (with appropriate adjustments to reflect the required use of NZS 1170.5:2004). Refer also to Section C3.
Yielding region	The region of a member which is anticipated to be subjected to nonlinear deformations under earthquake induced forces

C6.1.4 Notation, symbols and abbreviations

Symbol	Meaning
a	Distance between the centre of connectors and a flange cleat angle leg
a_1	Distance between the centre of connectors and the top edge of a flange cleat angle
A_g	Gross area of the cross section
A_n	Net area of the cross section
A_o	Plain shank area of a rivet
A_s	Tensile stress area
A_w	Area of a web
b	Distance between the centroid of tension and compression forces in a web cleat
b_1	Width of contact between beam flange and welds and column
b_{eff}	Effective beam flange width
B_f	Length of an angle
b_{fb}	Beam flange width
b_{fc}	Column flange width
C_s	Factor that accounts for the potential for deterioration in performance of CBFs with increasing inelastic demand
d	Depth of a steel section
D_b , d_b	Depth of a beam section
d_c	Depth of a column section
d_p	Depth of a web
E	Modulus of elasticity
e	Clear length of an active link

Symbol	Meaning
f	Residual capacity factor
f_u	Probable tensile strength
f_{uf}	Probable tensile strength of a rivet
f_{uw}	Tensile strength of weld metal
f_y	Probable yield strength
f_{yb}	Yield strength of a beam flange
f_{yc}	Yield strength of a column flange
f_{yw}	Yield strength of a web
G	Shear modulus of elasticity for steel, 80,000 MPa
G	Permanent action
h	Storey height
h_{eq}	Effective height of a frame
H_i	Height of floor i
H_v	Vickers Hardness
I_b	Second moment of area of a beam
I_c	Second moment of area of a column
k	Distance between bolt centreline and a web cleat angle leg
k	Column base flexural stiffener modifier
k_e	Member effective length factor
k_f	Form factor for members subject to axial compression
k_r	Rotation restraint reduction factor for lap connections
k_{te}	Correction factor for distribution of stresses in a tension member
k_θ	Rotational stiffness of column bases
L	Width of the braced bay
l	Member length
l_a	Length of a web cleat angle face
L_b	Length of critical brace
l_b	Clear span of beam
L_{bi}	Bay width
l_c	Clear length of column
L_{eq}	Total width of frame
L_j	Length of a bolted lap-splice connection
m	Distance from centre of bolt hole to radius root at web
m	Number of columns fixed at the base
m	Number of braces

Symbol	Meaning
M^*	Bending moment demand
M_{b}	Member flexural strength
$M_{\mathrm{bi,l}}$	Probable beam flexural strength to the left of a joint
$M_{\mathrm{bi,r}}$	Probable beam flexural strength to the right of a joint
m_{i}	Mass of floor i
M_{prob}	Probable flexural strength
$M_{\mathrm{prob,bare}}$	Probable flexural strength of a bare connection
$M_{\mathrm{prob,bl}}$	Probable beam flexural strength to the left of a joint
$M_{\mathrm{prob,br}}$	Probable beam flexural strength to the right of a joint
$M_{\mathrm{prob,c}}$	Probable member flexural strength
$M_{\mathrm{prob,ca}}$	Probable column flexural strength above a joint
$M_{\mathrm{prob,cb}}$	Probable column flexural strength below a joint
$M_{\mathrm{prob,encased}}$	Probable flexural strength of an encased connection
$M_{\mathrm{prob,s}}$	Probable section flexural strength
$M_{\mathrm{prob,w}}$	Probable tensile strength of a web cleat angle
$M_{\mathrm{prob,web}}$	Probable flexural capacity of a beam web to column connection
M_{ri}	Probable flexural strength at the base of column i
$M_{\mathrm{ri,b}}$	Probable flexural strength at the base or bottom of column i
$M_{\mathrm{ri,t}}$	Probable flexural strength at the top of column i
n	Number of connectors
n	Number of storeys
n_1	Length obtained by a 45° dispersion though half of the depth of a column
n_2	Length obtained by a 1:2.5 dispersion though column flange and root radius
N^*	Axial force, compressive or tensile
N^*_{eq}	Earthquake induced axial force
N_{fbc}	Probable compression capacity of beam flange
N^*_{fbc}	Compression demand on beam flange
N_{fbt}	Probable tension capacity of beam flange
N^*_{fbt}	Tension demand on beam flange
N_{fct}	Probable tension capacity of column flange
$N^*_{\mathrm{G+\psi_{\mathit{E}}Q}}$	Axial force demand due to gravity load
$N_{\mathrm{prob,c}}$	Probable member capacity in compression
$N_{\mathrm{prob,ci}}$	Probable compression capacity of brace i
$N_{\mathrm{prob,cr}}$	Probable limiting axial force

Symbol	Meaning
$N_{\mathrm{prob,s}}$	Probable section capacity of a compression member
$N_{\mathrm{prob,tf}}$	Probable tension capacity of a connector
N_{t}	Probable section capacity of a tension member
N_{tfw}	Probable tension capacity of beam flange weld
N_{w}^{*}	Axial force acting on a web panel
N_{wcc}	Probable compression capacity of column web
N_{wct}	Probable tension capacity of column web
n_{x}	The number of connector shear planes intercepting a shear plane
Q	Imposed action
r	Radius of gyration; or transition radius; or the root radius of a section
r_{c}	Column root radius
S	Plastic section modulus
s_{f}	Weld leg length to beam tension flange
S_{i}	Sway potential index
T	Tensile force in web cleat bolts/rivets
t	Thickness
t_{1}	Flange cleat angle leg thickness
t_{2}	Web cleat angle leg thickness
T_{c}	Probable tensile strength of column flange
t_{c}	Thickness of column flange
t_{fb}	Beam flange thickness
t_{fc}	Column flange thickness
t_{p}	Total thickness of doubler plates
t_{w}	Thickness of a web
t_{wc}	Column web thickness
V_{base}	Probable base shear capacity
V_{bi}	Storey i beam seismic shear demand determined from beam probable capacity
V_{c}	Probable panel zone shear capacity
V_{prob}	Probable shear capacity
$V_{\mathrm{prob,f}}$	Probable shear capacity
V_{v}	Shear capacity of a web
V_{w}	Shear capacity of a web
α'_{c}	Residual compressive strength factor
α'_{ci}	Residual strength factor for brace i

Symbol	Meaning
α_{b}	Compression member section constant
γ	Rotation angle of an active link
γ_{p}	Plastic rotation of an active link
γ_{u}	Ultimate rotation of an active link
γ_{y}	Yield rotation of an active link
δ/t	Dimensionless transverse deflection of plate
Δ	Displacement
Δ_{b}	Displacement capacity of a brace
Δ_{c}	Displacement at buckling of a brace
Δ_{cap}	Probable displacement capacity
Δ_{i}	Lateral displacement of floor i
Δ_{p}	Probable plastic displacement before deterioration
Δ_{t}	Displacement at tension yield
Δ_{y}	Probable yield displacement
θ	Chord rotation
θ_{cap}	Probable plastic hinge rotation capacity
θ_{i}	Angle between a brace and beam at the top end of the brace
θ_{p}	Plastic hinge rotation before deterioration
θ_{y}	Probable yield rotation
λ_{n}	Modified compression member slenderness
μ	Structural displacement ductility factor
μ_{act}	Actual structural displacement ductility demand
ϕ	Strength reduction factor
Ψ_{E}	Earthquake combination factor

C6.2 Factors Affecting the Seismic Performance of Steel Buildings

C6.2.1 General

Structural steel members are generally considered capable of dissipating significant amounts of energy when subjected to inelastic demands as the base material is inherently ductile. Because of this expected ductile response of the members, steel buildings are considered suitable for regions of high seismicity. However, the seismic performance of steel buildings can be affected by factors such as:

- imperfections and the fabrication process
- load paths through connections
- building condition (deterioration over time)
- member restraints
- P-delta effects
- slab participation, and
- building age (materials and design procedures).

Each of these factors is discussed below. Also refer to Appendix C6A for general guidance on the typical pre-1976 steel building systems used in New Zealand.

C6.2.2 Imperfections and fabrication process

Imperfections in structural steel generally cause stress concentrations that may result in a sudden loss in strength and hence a poor seismic performance. Imperfections may be created during fabrication processes, such as welding, or may be already present in the base material. It is rare for fabrication imperfections to be sufficiently severe in themselves to cause building failures during earthquakes.

Note:

The weld materials used and fabrication processes adopted were some of the minor factors that led to brittle fractures of welded connections in over 200 buildings during the 1994 earthquake in Northridge, California.

C6.2.3 Load paths through connections

Inadequate load paths through connections is the most common cause of local failures in steel buildings during earthquakes. Inadequate load paths through connections was the principal cause of welded connections failures during the 1994 Northridge earthquake (refer to Section C6.3.2.1 for more details).

Inadequate load paths through connections was also considered to be the principal cause of most local failures in multi-storey steel buildings in the 2010/2011 Canterbury earthquake sequence.

Note:

When undertaking a seismic assessment of a steel framed building, assessing load paths through connections is likely to be the most important aspect of the evaluation process.

C6.2.4 Building condition (deterioration over time)

Deterioration due to environmental effects such as corrosion may have a major effect on the seismic performance of steel framed buildings. When exposed to aggressive environments that facilitate corrosion, structural steel members/connections may sustain significant deterioration such as reduction in member strength due to loss of base material to oxidation. The ductile capacity of corroded members may be significantly reduced if the members sustain localised corrosion as the zone of yielding will be limited to the reduced cross section.

Column bases and hold down bolts are the elements most prone to severe localised loss of material due to long term corrosion. There were several reported failures of industrial structural systems in the 1987 Edgecumbe earthquake due to column failures at the base from corrosion. In addition, reduction in member strengths due to corrosion was reported as one of the main factors contributing to failure of braces during this earthquake (Butcher et al., 1998).

Note:

A condition assessment, particularly of pre-1976 steel framed buildings, is recommended as part of the DSA. Refer to Section C6.4 for more details.

C6.2.5 Member restraints

Structural steel members are made up of plates that are hot rolled, cold formed, welded, bolted, or riveted together. The slenderness and the boundary conditions of the constituting plates may significantly affect the seismic performance of a steel member by limiting the local and lateral torsional buckling capacity of the member.

Local buckling of steel members occurs due to plate slenderness, while lateral torsional buckling of steel members occurs when there is inadequate lateral bracing of compression flanges. The elastic resistance to lateral buckling of a steel member is influenced by several factors such as: unbraced length of the compression flange, geometric and material properties of the member, and moment gradient along the member.

Experimental evidences indicate that local plate buckling generally results in a gradual degradation of strength and stiffness in compact cross sections, while lateral torsional buckling causes a rapid loss of strength and stiffness (Gupta and Krawinkler, 1999). Local buckling of slender members causes a rapid loss of section and hence member capacity.

C6.2.6 P-delta effects

Steel MRF buildings are generally more flexible than other building types and hence are subjected to relatively large lateral displacement demands. Therefore, gravity induced loads acting on a laterally displaced building (P-delta effects) can be pronounced on flexible steel MRFs.

Note:

When large ductility demands that may result in significant deterioration in member strength and stiffness are likely, P-delta effects will be worsened.

C6.2.7 Slab participation

Typically, floor slabs have been constructed with no separation from columns. This causes the slab to contribute to the seismic capacity of framed buildings. Slab participation results in development of increased seismic demands in columns due to increased beam flexural overstrength capacity.

Slab participation may induce column flexural yielding, column shear failure or beam shear failure modes in steel MRFs, depending on the relative strength of the members and the connections. Slab participation may also cause damage to floor slabs and compromise the capacity of the floor system to transfer seismic demands to the lateral force resisting members; although the evidence from the 2010/2011 Canterbury earthquake sequence is that the influence on composite slabs (concrete on steel deck on steel or concrete supporting beams) is minimal.

Note:

When the connections of a steel framed building are semi-rigid, slab participation may considerably increase the stiffness and strength of the connections (Roeder et al., 1994). Slab participation may be beneficial in such buildings if it does not result in localised column failures.

C6.2.8 Building age (materials and design)

C6.2.8.1 Materials

The earliest steel framed buildings in New Zealand are believed to have been constructed in the 1880s, with steel being the preferred ferrous material for structural members from then onwards.

Cast iron columns are found in some of the oldest New Zealand buildings and, until the early 1900s, were often used as gravity carrying elements. Cast iron is a low strength and brittle material not suitable for use in a seismic-resisting system or in a gravity system that is required to sustain significant deformations. The tensile strength of cast iron is significantly less than its compressive strength due to the presence of voids and cracks within the iron matrix (Rondal and Rasmussen, 2003). The consequence of these non-ductile characteristics is that the performance of cast iron columns is likely to be poor if they are part of the lateral force resisting system and/or are subjected to significant lateral displacements.

Cast iron columns can be dependably retained in an existing building if they are used as a propped gravity column, with the supports for the beams assessed and reinforced if necessary (e.g. with steel bands) to avoid local fracture under seismic-induced rotations. However, the strength of a cast iron column cannot be determined using the provisions for steel columns in these guidelines as cast iron has a different stress-strain relationship to steel. Guidance on the assessment of cast iron columns can be found in Bussell (1997) and Rondal and Rasmussen (2003).

Wrought iron was also used to a limited extent for structural members in early New Zealand buildings. However, its use largely ended around the 1880s and 1890s as these items were costly to manufacture. The principal disadvantage of wrought iron as a building material was the small quantities made in each production item (bloom), being only 20-50 kg. This meant that the use of wrought iron in structural members required many elements to be joined by rivets.

Wrought iron has good compressive and tensile strength, good ductility, and good corrosion resistance. The performance of wrought iron members is considered comparable to that of steel members from the same era.

C6.2.8.2 Design

Despite their apparent advantage over other building types of the same era such as unreinforced masonry buildings, steel buildings designed before the introduction of NZS 4203:1976 suffer from the fundamental drawback of being not designed according to capacity design procedures.

Note:

Pre-1976 design methods generally assumed an elastic response, with no consideration given to likely failure modes and with no ductile detailing requirements to ensure that potential plastic hinge regions can dependably accommodate earthquake induced ductility demands. In addition, no attention was generally given to load paths through connections under inelastic response. Structural members of these buildings that should remain elastic to avoid undesirable failure mechanisms may not have the capacity to resist overstrength actions originating from potential plastic hinge regions and slab participation. Additionally, structural members and connections that are provided to resist gravity induced loads may not have the capacity to accommodate earthquake induced displacement demands; although most early gravity systems with bolted or riveted connections are considered to have high ductility capacity but very limited strength.

The pattern of damage observed during the 1995 earthquake in Kobe, Japan indicates that three factors play a significant role in ensuring a good overall seismic performance of a steel frame building not designed following the capacity design method:

- The beam-column connections of the frames of a building should be able to retain their shear and axial force carrying capacity when the connections are sustaining flexural actions from earthquake demands.
- The inelastic demand in the columns should be kept to a minimum. This demand is principally due to local buckling or crippling failure, and also to general plastic hinge formation.
- The inelastic response of the building should be essentially symmetric in nature and not lead to a progressive movement of the building in one direction only.

Note:

Details of the damage sustained during the Kobe earthquake are provided in reconnaissance reports such as that by Park et al. (1995).

In buildings constructed before the 1950s the structural members of steel frames are usually encased in lightly reinforced concrete as fire protection (refer to Figure C6.1). The reinforcement of the encasement is often inadequate and poorly detailed (Bruneau and Bisson, 2000), which results in a significant increase in stiffness and a relatively modest increase in strength of the encased members.

Spalling of the encasement concrete, particularly in the end regions of members, has the potential to increase the nonlinear demands in the steel members if they are required to be loaded beyond yield.

Figure C6.1: A typical riveted beam-column connection

Even older steel framed buildings constructed before the 1936 New Zealand standard model building by-law introduced seismic design requirements typically contain beams that are deeper than the columns. The frames of these buildings generally contain simple and semi-rigid riveted connections that have a modest flexural capacity. In addition, these connections generally exhibit poor energy dissipation capability with lack of adequate strength and stiffness and may serve as the weakest link during inelastic earthquake demands. However, the seismic performance of similar structures dating back to the 1906 San Francisco earthquake has generally been high.

C6.3 Observed Behaviour of Steel Buildings in Past Earthquakes

C6.3.1 Overall performance

Steel buildings have been observed to perform generally well during major international earthquakes. The only steel framed buildings to have been reported to have collapsed were during the 1985 Michoacan, Mexico earthquake. However, these collapses were attributed to factors such as resonance and local soil conditions. The collapsed buildings were between 10 and 15 storeys high, in the resonance range of the strongly harmonic earthquake that struck Mexico City. Another source of collapse was very light welds between built-up members that “unzipped” during the earthquake.

Consequently, steel framed buildings have been generally regarded as ductile and resilient against earthquake induced collapse. However, the significant damage observed during the Northridge (1994) and Kobe, Japan (1995) earthquakes emphasises the vulnerability of even recently constructed steel framed buildings and the need for attention to load paths.

C6.3.2 Moment resisting frame buildings

C6.3.2.1 Performance in the 1994 Northridge earthquake

The 1994 Northridge earthquake caused considerable damage to steel MRFs that had been designed on the basis that they would behave in a ductile manner. The rigidly welded connections of these frames were observed to have fractured at low levels of ductile demand.

Although hundreds of MRF buildings suffered this unexpected overload form of connection damage, most of the buildings displayed no visible signs of distress after the earthquake (such as permanent lateral deflections); nor was there significant damage to non-structural components and contents. However, the capacity of these buildings to resist further earthquake induced demands was significantly compromised and costly repairs were required.

The main reason for the unexpectedly poor performance was the inability of the load paths between the beams and the columns of the frames to transfer actions generated by plastically responding beams into the columns. The inadequacy of these load paths caused fractures of the beam flange to column flange connections. The majority of the fractures were observed to occur at the bottom beam-column flange connections due to slab participation. In some instances these bottom fractures were even observed to trigger web connection failures (Krawinkler, 1995). Refer to Figure C6.2.

Note:

Details of the damage sustained during the Northridge earthquake have been widely reported in reconnaissance reports such as that by Norton et al. (1994).

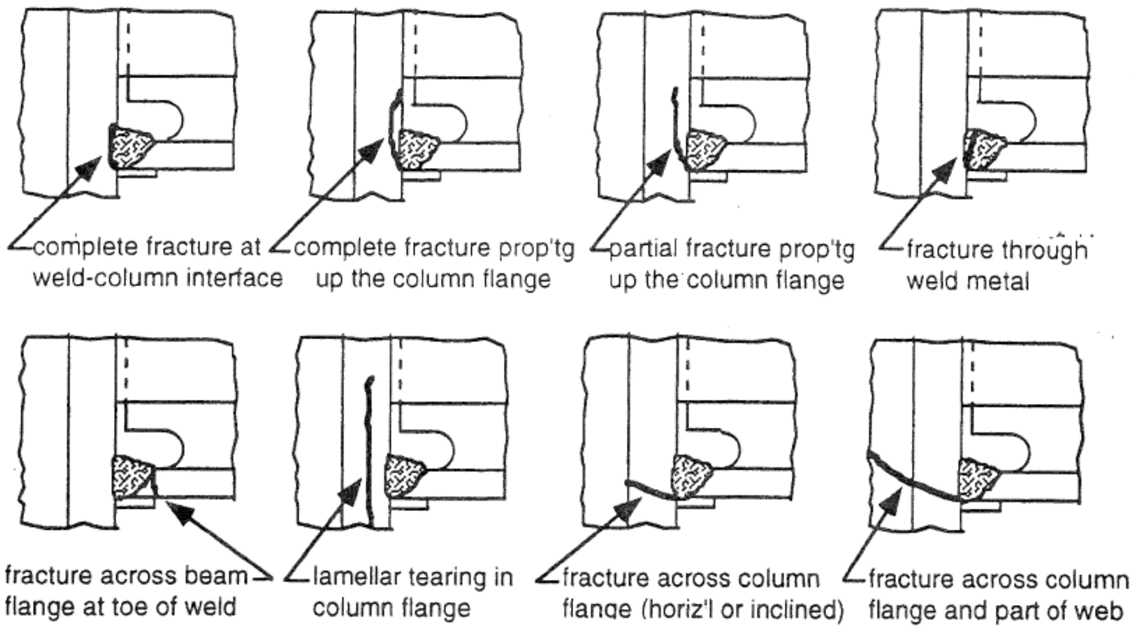


Figure C6.2: Welded connection fracture modes observed during the 1994 Northridge earthquake (Krawinkler, 1995)

The inadequacy of the load paths of "Pre-Northridge" connections meant that even the best fabricated beam to column connections were not able to develop plastic hinges in beams that exceeded a depth of approximately 360 mm. However, the following factors were considered to have minor contributions to the failures of "Pre-Northridge" connections (FEMA 355E, 2000):

- The welding practice was such that bottom flange weld passes were interrupted at beam webs, resulting in weld defects that served as crack initiators.
- The configuration of the connections made detection of hidden bottom weld defects difficult, particularly at the beam webs.
- The filler metal employed was typically developed for high deposition rate welding and had very low notch toughness as a result.
- There was use of large size beams in buildings that had few lateral force resisting frames. The deeper the beam, the greater the web contribution to flexural strength and therefore the greater the likelihood of ductile overload of the beam flange to column flange connection. The use of large size beams also meant higher deposition rate large welds which were more prone to fractures than small size welds (Krawinkler, 1995).
- The mean yield strength of members fabricated in the 1980s was observed to be generally significantly greater than the nominal values.
- The geometry of weld access holes was, in some cases, observed to hinder ease of filler metal deposition and weld inspections.

Immediately after the Northridge Earthquake, the New Zealand Heavy Engineering Research Association (HERA) and the University of Auckland looked at the possibility of similar types of failures in New Zealand buildings and found no examples of this type of construction. A series of large scale beam/column inelastic cyclic tests were performed on typical New Zealand type MRF connections which showed that they were not vulnerable to this type of failure (Butterworth, 1995).

C6.3.2.2 Performance in the 2010-11 Canterbury earthquake sequence

During the Canterbury earthquake sequence of 2010/11 no significant damage appeared to have been sustained by any post-1976 MRFs. Minor panel zone yielding of an MRF (refer to Figure C6.3) was observed in a 12 storey, predominantly eccentrically braced frame (EBF) building.

Provided the beams adjacent to the panel zone did not exhibit any signs of yielding, the yielding of the panel zone was not expected. The yielding of this panel zone was considered to result from the combination of elevated levels of compression force in the columns due to high vertical ground accelerations and the expected and significant bending demands imposed on the adjoining beams.

Figure C6.3: Panel zone of an MRF showing minor inelastic action (Clifton and El Sarraf, 2011)

C6.3.3 Braced steel frame buildings

C6.3.3.1 Eccentrically braced frame buildings

EBF multi-storey buildings generally performed very well during the Canterbury earthquake sequence. Generally, the observed damage was minor and limited principally to non-structural items. A 22 storey EBF building required replacement of seven active links due to nonlinear overload and, in one case, brittle fracture (refer to Figure C6.4(a)). Another 35 active links were replaced due to the steel having unacceptably low Charpy impact energy. More active links would have been expected to be replaced as the magnitude of the excitation during the February 22, 2011 earthquake was such that it was significantly above the 500 year return design spectrum of NZS 1170.5:2004 that is the basis for ULS design of typical new buildings. One 12 storey EBF building was returned to service with no structural repairs needed. It was the only multi-storey building in the Christchurch CBD for which this was the case, including base isolated structures.

The good performance of multi-storey EBF buildings in the Canterbury earthquake sequence can be attributed to:

- the significant effects of soil-foundation-structure-interaction (on reducing the seismic demand on the superstructure of these relatively heavy multi-storey buildings built on soft soil (Storie et al., 2014))
- factors contributing to overstrength in steel frames such as actual yield strengths significantly exceeding nominal values, modelling assumptions, etc.
- the contribution of the composite floor slab action to the shear resistance that was not allowed for in the design of the frames, and
- the contribution of solid partition walls and non-structural items.

A fractured active link of the 12 storey EBF building is presented in Figure C6.4(a). This active link appeared to have undergone at least one full cycle of web panel yielding prior to fracture. The fracture appeared to have propagated from one top corner across the active link region and resulted in significant residual deformations. Detailed evaluations of this and other links in the EBF braced bay concerned showed that the Charpy impact energy of this steel was well below that specified by NZS 3404:1997, with the material having a transition temperature of around 12^{o}C. This particular link also had a shear stud welded to the flange immediately above the left hand visible stiffener, which is believed to have acted as a crack initiation site.

(a) A fractured active link in a 12 storey building

(b) A fractured active link in a low-rise parking building

Figure C6.4: Fractured EBF active links during the February 22, 2011 Christchurch earthquake (Clifton and El Sarraf, 2011)

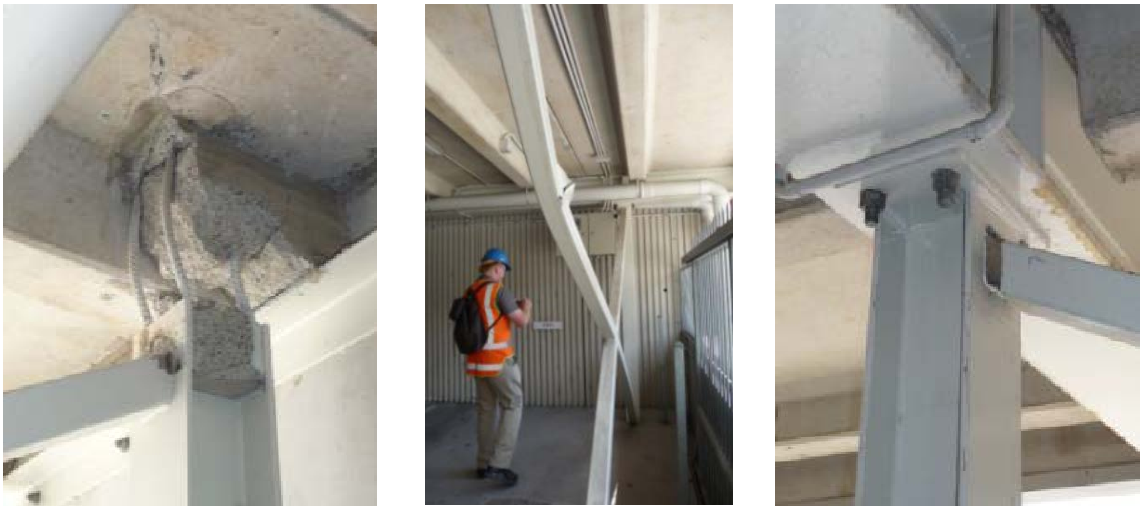
Fractures of two active links in a low-rise EBF building (refer to Figure C6.4(b)) were attributed to detailing/fabrication errors. The flanges of the two braces were observed to be offset from the stiffeners of the active links. The offset lead to fracture of unstiffened collector beam flanges located between the active link stiffeners and the flanges of braces.

C6.3.3.2 Concentrically braced frame buildings

Observations made during the 1995 Kobe earthquake have reinforced the expectation that concentrically braced frames (CBFs) that are not designed following the capacity design method are not likely to perform as intended in the event of an earthquake.

In New Zealand, non-capacity designed (pre-1976) CBFs are typically X-braced, while very few are believed to be V-braced. Pre-1976 CBFs in New Zealand were typically designed to resist lower levels of lateral forces than required by NZS 1170.5:2004. In Kobe, several such CBF buildings were reported to sustain buckled braces or failed connections during the 1995 earthquake. However, none of these buildings were reported to have collapsed (Clifton, 1996).

Most CBF buildings performed as expected during the 1994 Northridge earthquake, but with no collapses reported. Similar to the connection weld fractures of MRFs discussed in Section C6.3.2, fractures of brace-collector beam and column-base plate welded connections were prevalent. In addition, excessive local buckling of thin-walled tubular braces of CBFs was observed (Krawinkler, 1995).



(a) A poorly detailed connection **(b) A buckled brace** **(c) A fractured connection**

Figure C6.5: Damaged CBFs in a single-storey car park building during the February 22, 2011 Christchurch earthquake (Clifton and El Sarraf, 2011)

Significant damage to a single-storey CBF building was observed during the Canterbury earthquake sequence (refer to Figure C6.5). However, the connections of the CBFs to the columns appeared to have been poorly detailed. One of the CBFs appeared to have been connected to a column that had a non-ductile reinforced concrete extension (refer to Figure C6.5(a)), while the welded connection of the second CBF did not appear to have been designed by following capacity design principles (refer to Figure C6.5(b)).

C6.3.4 Portal frame buildings

Most portal frame buildings generally performed well during the Canterbury earthquake sequence. Observed damage was mainly attributed to ground instability or limited to failure of bracing systems, while frame moment connections exhibited no visible signs of damage.

Many of the portal frame buildings in Christchurch were industrial facilities designed to resist high wind induced forces, which were typically the controlling design case. These buildings typically have light roofs that are braced using light rod braces with proprietary end fittings. A few fractures and thread stripping of the proprietary brace connectors were reported following the February 22, 2011 earthquake (refer to Figure C6.6).

Figure C6.6: Proprietary brace connectors that failed during the February 22, 2011 Christchurch earthquake (Clifton and El Sarraf, 2011)

Figure C6.7: Roof bracing that failed during the February 22, 2011 Christchurch earthquake (Clifton and El Sarraf, 2011)

In one building, failure of a roof bracing was observed following the February 22, 2011 earthquake (refer to Figure C6.7). This failure was considered to be a result of excessive movements of tilt up panels that were likely to have been caused by ground liquefaction (Clifton and El Sarraf, 2011).

C6.4 Material Properties and Testing

C6.4.1 General

Note:

Assessments in accordance with these guidelines are intended to be carried out using probable material strengths. Typically, the probable material strengths may be taken as the (lower) characteristic/nominal (as referred to in design standards) material strengths but enhanced by the material strength modification factors given in Section C6.4.4.

Mechanical properties of the steelwork within existing structural steel framed buildings may be determined from:

- drawings, specifications or other construction records
- historical steel grades and nominal strengths, and/or
- steel material tests.

The mechanical properties of structural steelwork are best determined from original construction records supplemented by laboratory or in-situ tests of selected critical components to confirm the expected steel grade.

If the source of steelwork can be confirmed from the designations on original construction records, but the steel grade is not identified and testing is not practicable, default mechanical properties corresponding to the source and age of the steelwork can be adopted from those outlined in historical specifications. Refer to Appendix C6B for typical sources of historical New Zealand structural steelwork.

In the absence of construction records, the source of a structural steelwork can be identified from the mill markings generally present on historical structural steel sections and from section geometric properties contained in literature on historical structural steelwork (e.g. Bates, 1991; Bussell, 1997; and Ferris, 1954).

Note:

Older steelwork exhibits greater variability than modern steelwork. Accordingly, a minimum degree of non-destructive testing is recommended to gain assurance of the mechanical properties for the members in the primary structure. This is particularly the case when the steel is “of unknown origin”.

If the steelwork cannot be identified from construction records, mill markings or section geometric properties, the default yield strengths for steel “of unknown origin” provided in Appendix C6B may be adopted.

If tensile tests are undertaken, default strengths corresponding to the grade, potential source and age of the steelwork should be adopted from Appendix C6B.

Members with steel of unknown origin may exhibit non-ductile behaviour if all of the following conditions apply:

- from an assessment of the strength hierarchy of the building, the members with steel of unknown origin are the weakest links, not the connections, and
- the members with steel of unknown origin are located in an external steelwork or on the cold side of the building envelope so that the members could be below their transition temperature at the time of an earthquake, and
- a notch, a significant crack, or any stress raiser is present in a critical location.

If all these conditions contributing to potential member brittle responses are present and potential brittle failure has not already been ruled out through physical testing, fracture toughness tests should be undertaken on selected critical members as per Section C6.4.5 to rule out potential brittle responses.

Note:

Another key concern with members with "steel of unknown origin" is the undefined upper bound on yield strength, which may be significantly greater than the characteristic values. Primary members of unknown origin may develop strengths that are significantly higher than allowed for using overstrength factors. Large member overstrengths may lead to overloading other aspects of the structure and loss of assumed hierarchical behaviours and/or protection.

C6.4.2 Identifying the building materials: are they cast iron, wrought iron or steel?

As outlined in Section C6.2.8.1 the earliest steel framed buildings in New Zealand are believed to have been constructed in the 1880s. While steel was the preferred ferrous material for structural members from then onwards, cast iron and, to a lesser extent, wrought iron were also used in New Zealand buildings before the early 1900s. Identifying the building materials and their age is an important aspect of the seismic assessment process.

Cast iron

The use of cast iron from the 1880s until its discontinuance around 1910 was limited to columns. Cast iron columns would have been used typically for gravity load carrying purposes. These columns are typically "chunky" with thick sections, often having ornate or complex profiles (fluted, plain hollow circular, or cruciform shaped). The surface of these columns is typically pitted with small blowholes.

Wrought iron

If a building is constructed before 1900 and contains members built up from many short-length I-sections, channels and/or flats, then the possible use of wrought iron in these members should be considered. Guidance for the assessment of wrought iron members is provided in Bussell (1997).

Note:

Detailed visual assessment criteria for iron and steel members are presented in Bussell (1997).

C6.4.3 Cast iron and wrought iron: probable strengths

In the absence of specific material data, the probable yield strengths of cast iron and wrought iron should be taken as the values provided in Table C6.1 if members in buildings constructed before the early 1900s are identified to be made of cast iron or wrought iron.

Table C6.1: Probable strengths of historical cast iron and wrought iron

Material	Tensile strength (MPa)	Compressive strength (MPa)	Modulus of elasticity (GPa)
Cast iron	47	247	93
Wrought iron	162	124	185

Note:

Cast iron and wrought iron are generally only found in buildings constructed prior to 1900. Due to the lack of available specific data, the probable strength of cast iron and wrought iron is taken as one half of published breaking strengths such as those by Fidler (1879). Table C6.1 is based on the lower characteristic strength values published in 1879 (Bates, 1991).

C6.4.4 Structural steel: historical grades and probable strengths

Before the 1960s most structural steelwork was imported from Australia (historical evidence indicates this was from the late 1930s onwards) and the UK. A small quantity of steel is also believed to have been imported from the USA and continental Europe.

From the 1960s on most rolled sections have been manufactured in Australia, while plates and welded sections have been mainly produced in New Zealand.

The structural steel properties outlined in relevant historical standard specifications are summarised in Appendix C6B. Default characteristic/nominal strengths are also provided for steel of unknown origin.

Note:

The first New Zealand structural steel standard specifications are believed to be NZS 309 and NZS 310, published in 1941. These standards and their subsequent editions were based on their British equivalents until the first joint AS/NZS standard specifications were introduced in the mid-1990s. The joint specifications were revisions of previous Australian standard specifications.

Mechanical properties provided in construction documentation and default mechanical properties specified in standard specifications should be taken as (lower) characteristic or nominal strengths. Probable strengths can be determined from these by applying the appropriate strength modification factor from Table C6.2. The factors provided in this table are applicable to steelwork produced in New Zealand and to steelwork imported from Australia or the United Kingdom.

Table C6.2: Factors to convert lower characteristic material strengths to probable strengths (based on tests undertaken by Baker, 1969; Erasmus, 1984; and Erasmus and Smaill, 1990)

Period	Steel grade	Factor
Pre-1960	All	1.1
1960-Now	300 and below	1.15
	350 and above	1.1

C6.4.5 Test methods to determine the mechanical properties of structural steel

C6.4.5.1 General

Testing to determine the mechanical properties of structural steel components of an existing building is generally recommended. This is especially the case when the properties of the primary structure cannot be identified from original construction records and mill markings.

Tests should at least identify the likely steel grades. They should also identify unexpectedly high or low strength materials and materials that may exhibit brittle behaviour when subjected to earthquake loading.

Note:

If the intention is to strengthen an existing steel building and the strengthening involves welding to an existing steel, the weldability of the existing steel parent material also needs to be determined.

C6.4.5.2 Tensile strength tests

The probable tensile strength of a structural steel component can be determined from tensile tests undertaken on a representative material removed from the component. Alternatively, hardness tests may be undertaken on the component in situ.

There is an approximate relationship between material hardness and probable tensile strength. The best relationship for the range of steel material strengths of interest (400 to 700 MPa) is given by Vickers Hardness, H_{v}. The relationship between Vickers Hardness and tensile strength of a steel material is tabulated in ASM International (1976) and can be expressed in equation form as:

$$f_{\mathrm{u}} = 3.09\, H_{\mathrm{v}} + 21.2 \qquad \text{…C6.1}$$

where:

H_{v} = Vickers Hardness from test
f_{u} = probable tensile strength

This expression is valid for $100 \leq H_{\mathrm{v}} \leq 300$, corresponding to $330 \leq f_{\mathrm{u}} \leq 950$ MPa.

Note:

Testing for Vickers Hardness is carried out to AS 1817:1991 Metallic Materials – Vickers Hardness Test (1991). There are a number of materials testing organisations in New Zealand that can undertake Vickers Hardness tests.

The key steps for determining what components to test and how many tests to conduct are as follows:

Step 1

Determine the members/elements to be tested, i.e. beams, columns, critical connection components and connectors. The elements identified as critical from the connection evaluation in Section C6.6.1 and the strength hierarchy evaluation in Section C6.7 should be subjected to the most detailed testing.

Step 2

Determine the frequency of testing. The aim is to cover at least 5% of the total sample of each type of critical component.

Step 3

Use Equation C6.1 or refer to Nashid et al. (2015) for the relationship between Vickers Hardness and tensile strength.

Note:

Nashid et al. (2015) presents the findings of comprehensive recent research on the hardness-tensile strength relationship of structural steel members.

Step 4

Compare the tensile strengths with the expected steel grades. Any material with $H_{\mathrm{v}} < 100$ or $H_{\mathrm{v}} > 230$ should be investigated more thoroughly by tensile sampling and visual inspection. Any material with $H_{\mathrm{v}} > 230$ should also be treated as potentially prone to brittle fracture.

Note:

There is no direct relationship between tensile strength and brittle fracture. However, the susceptibility to brittle fracture increases with increasing tensile strength. The elongation capacity of steel also decreases with increasing strength. Accordingly, the guidance provided above is a threshold requiring more appropriate testing for potential brittle fracture performance.

C6.4.5.3 Fracture toughness tests

As discussed above and in Section C6.4.1, the potential for member brittle fracture in an existing building becomes an issue for further investigation if the structural components are the weakest links and if any of the following are applicable:

- the components are “steel of unknown origin” and are located in an external steelwork or on the cold side of the building envelope, or

- the Vickers Hardness test of the components identifies steel with $H_v > 230$, or
- the thickness of any component is > 32 mm.

If any of these apply, material from those components should be removed for Charpy impact tests, as specified in NZS 3404:1997, to determine whether the steelwork satisfies energy absorption requirements. Test material may be removed from the less critical regions of a member/element; e.g. from the web of beams away from high shear zones.

A minimum of three Charpy impact tests should be undertaken on material removed from each type of critical component. For the energy absorption requirements to be satisfied, the average Charpy impact energy absorption capacity of a steelwork from the three tests should exceed 27 J at 0°C, while the minimum of the three tests should exceed 20 J at 0°C.

If the steel does not satisfy the above energy absorption requirements a more detailed evaluation should be undertaken.

Note:

For brittle fracture of steel to occur during an earthquake, the steel has to have a low Charpy impact energy absorption capacity at service temperature (or the steelwork has to be below its transition temperature at the time of the earthquake) and a stress raiser has to be present in a critical location.

C6.4.6 Probable yield and tensile strengths of fasteners and weld metals

In the absence of any physical test data, probable strengths of fasteners and weld metals provided in Table C6.3 can be used.

Note:

In the absence of specific data, the probable strengths shown in Table C6.3 have been taken as the lower characteristic strengths based on Bussell (1997) and ASCE 41-13 (2014) except for pre-1961 rivets which have been taken as 1.1 times their characteristic/ nominal strength.

Table C6.3: Probable strengths of fasteners and weld metals

Time period	Material	Origin	Yield strength (MPa)	Tensile strength (MPa)
1901–60	Rivets	USA	228	380
1934-42	Rivets to BS 548 (High tensile steel)	UK	*	510
1948-61	Rivets to BS 15 (Mild steel)	UK	*	425
All	Bolts	All	240	400
All	Weld metals	All	-	410

Note:
*The probable yield strength of these rivets can be taken as half of their probable tensile strength.

C6.5 Component Capacities

C6.5.1 General

This section covers the assessment of the probable strength and deformation (rotation) capacities of members/elements of moment resisting and braced steel frames including:

- beams
- columns
- concrete encased steel beams and columns
- braces
- active links of eccentrically braced frames.

The probable strength of structural steel members/elements should be determined using the probable material strengths as outlined in Section C6.4. A strength reduction factor is not required to be applied (i.e. a strength reduction factor, ϕ, of 1.0 is used).

C6.5.2 Beams

C6.5.2.1 General

The probable strength of steel beams of seismic-resisting frames is generally governed by flexural strength.

The flexural strength of a steel beam is dependent on the length of the beam between adjacent cross sections that may be either restrained or unrestrained (segments) and the restraint condition provided at the ends of the segments (full, partial or lateral restraint).

Guidance provided in NZS 3404:1997 and guidelines such as those outlined in Clifton (2009) provide methods to determine bracing required against lateral torsional buckling and plate slenderness limits to ensure local and lateral buckling of steel members do not occur prematurely.

The effect of combined actions of shear and flexure should be assessed at cross sections where both shear and flexure are expected to be high.

C6.5.2.2 Shear strength

For a stocky web of a structural steel section satisfying a web panel slenderness ratio (d_p/t_w) of:

$$d_p/t_w \leq {}^{82}\Big/\sqrt{{}^{f_y}/_{250}} \qquad \text{…C6.2}$$

where:

d_p = depth of web
t_w = thickness of web
f_y = probable yield strength

the probable shear yield capacity of the web (V_v) should be taken as (NZS 3404:1997):

$$V_v = 0.6 f_y A_w \quad \ldots C6.3$$

where:

f_y = probable yield strength
A_w = area of web.

If the above web slenderness criterion is not satisfied and the web is slender, the web is likely to buckle instead of yielding in shear. The probable shear buckling strength of slender webs should be determined from Clause 5.11.5 of NZS 3404:1997.

C6.5.2.3 Flexural strength

The probable section flexural strength, $M_{prob,s}$, and probable member flexural strength, $M_{prob,c}$, of steel beams that are subjected to bending about their major principal axis should be determined from Clause 5.2 and Clause 5.3 of NZS 3404:1997 using probable material strengths.

The sections of a steel beam should be compact and not prone to local plate buckling in order to have flexurally yielding regions in the beam that are able to develop and maintain their full plastic section strength until the deformation capacity is reached.

In addition to having compact sections, steel beams or segments of steel beams need to have full restraint against lateral buckling (FLR) to develop and maintain their full section plastic strength.

Beams supporting a concrete slab are considered to have FLR and develop their section flexural strength, while beams supporting timber floors generally achieve member flexural strength only. Restraint offered to steel beams by timber floors or other lateral restraint conditions are provided in HERA Report R4–92 (Clifton, 1997).

The probable strength of beams having slender sections is limited to their probable yield strength due to local plate buckling. However, unlike elastic buckling of compression members, buckling of slender plates of flexural members does not lead to immediate loss of load-carrying capacity or excessive deflections, as shown in Figure C6.8, as redistribution of in-plane stresses occurs within the plates.

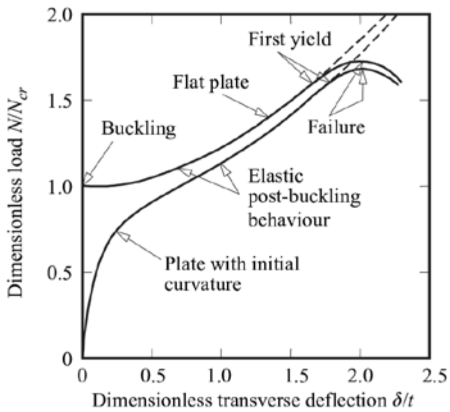


Figure C6.8: Post-buckling behaviour of thin plates (Trahair et al., 2008)

A generic relationship between the probable flexural strength, M_{prob}, and probable chord rotation, θ, capacity of steel beams with FLR is provided in Figure C6.9. The parameters for the generic relationship for this type of beam should be taken from Table C6.4 using the highest possible member category. The member category for steel beams should be determined based on steel material and section geometry requirements outlined in Clause 12.4 and Clause 12.5 respectively of NZS 3404:1997.

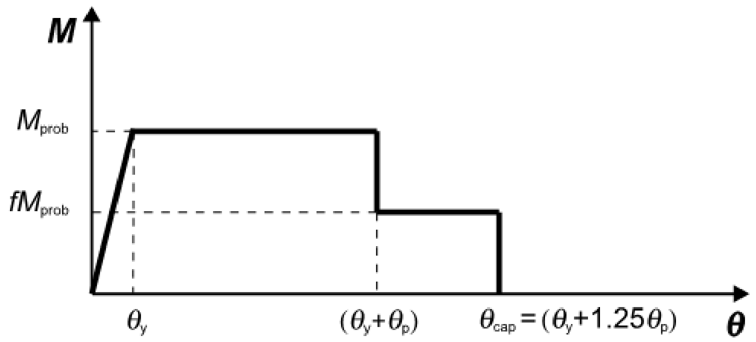


Figure C6.9: Moment-rotation relationship for steel beams and columns with FLR

Table C6.4: Parameters for the moment-rotation relationship for steel beams with FLR

Category of member	$(\boldsymbol{\theta_y} + \boldsymbol{\theta_p})$ (mrad)	Residual strength factor f
1 & 2	45	0.5
3	30	0.5

Note:
If the beam under consideration cannot support gravity loading $(G + \Psi_E Q)$ in a simply supported condition, halve the rotational capacity of the beam for both the full and the residual strength capacity.

The probable yield rotation, θ_y, of seismic governed steel beams that are rigidly connected to columns at both ends can be determined from:

$$\theta_y = \frac{M_{\text{prob,s}}\, l_b}{6EI_b} \qquad \ldots\text{C6.4}$$

where:

$M_{\text{prob,s}}$	=	probable section flexural strength
l_b	=	clear span of beam
E	=	modulus of elasticity
I_b	=	second moment of area of beam.

C6.5.3 Columns

Steel columns in seismic-resisting buildings are generally subjected to a combination of flexure and axial forces. Both axial tension and compression reduce the flexural capacity of steel columns, while axial compression reduces the local buckling capacity.

The probable strength of steel columns may be limited by the various member shear and flexural yield mechanisms outlined in Section C6.5.2. In addition, the flexural capacity of steel columns may be limited by column buckling. The probable section and member flexural capacities of steel columns should be determined from Clause 8.3 and Clause 8.4 of NZS 3404:1997 using probable material strengths.

When determining the rotation capacity of steel columns, the axial force used should be that from the gravity load associated with earthquake action ($N^*_{\mathrm{G+\psi_E Q}}$) and the seismic contribution should be ignored.

Note:

Experimental tests (MacRae, 1990 and Brownlee, 1994) have shown that the inelastic behaviour and rotation capacity of a steel beam-column subject to compression and major axis bending is dependent on the magnitude of the constant component of the compression force – i.e. that from $N^*_{\mathrm{G+\psi_E Q}}$ – rather than on the total compression force that includes the seismic component.

Steel columns that are subjected to inelastic demand should satisfy the axial load limitations of Clause 12.8.3 of NZS 3404:1997. This clause is intended to ensure that the level of compression in a column is not too high to compromise the capacity of the column to dependably accommodate inelastic earthquake demands.

A typical moment-rotation relationship for steel columns that have FLR is provided in Figure C6.9. The parameters for the generic relationship for this type of column should be taken from Table C6.5 using the highest possible member category. The member category for steel columns should be determined based on steel material and section geometry requirements outlined in Clause 12.4 and Clause 12.5 respectively of NZS 3404:1997.

Table C6.5: Moment-rotation parameters for steel columns with FLR

Category of member	$(\theta_{\mathrm{y}}+\theta_{\mathrm{p}})$ $(mrad)$				Residual strength factor, f
	$N^*/N_{\mathrm{prob,s}} \le 0.15$	$0.15 < N^*/N_{\mathrm{prob,s}} \le 0.3$	$0.3 < N^*/N_{\mathrm{prob,s}} \le 0.5$	$0.5 < N^*/N_{\mathrm{prob,s}} \le 0.8$	
1 & 2	50	45	20	15	0.5
3	35	30	15	10	0.5

The probable yield rotation of steel columns that have a point of contraflexure at mid height and are subjected to both flexure and compression can be determined from:

$$\theta_{\mathrm{y}} = \frac{M_{\mathrm{prob,s}}\, l_{\mathrm{c}}}{6EI_{\mathrm{c}}}\left(1 - \frac{N^*}{N_{\mathrm{prob,c}}}\right) \quad \text{…C6.5}$$

where:

$M_{\mathrm{prob,s}}$	=	probable section flexural strength
l_{c}	=	clear length of column
E	=	modulus of elasticity
I_{c}	=	second moment of area of column
N^{*}	=	axial force from analysis
$N_{\mathrm{prob,c}}$	=	probable member capacity in compression.

Comparisons of the provisions in NZS 3404:1997 with physical tests undertaken recently in Canada (Clifton (not published at time of preparation)) on a medium heavy I-section column type cross section and an I-section beam type cross section indicated that the following modifications needed to be made to the provisions in NZS 3404:1997 when determining the probable capacity of steel members:

- The rotation restraint factor (k_{r}) should be taken as 0.85, consistent with a plastic hinge forming at one end only at a particular point in time.
- The member effective length factor (k_{e}) should be taken as 0.85 instead of the NZS 3404:1997-specified 1.0, consistent with a plastic hinge forming at one end only at a particular point in time.

The physical tests undertaken in Canada also showed that the moment-rotations parameters presented in Table C6.5 for highly axially loaded columns are on the conservative side.

Members that are subjected to bending about their minor principal axis should be considered capable of developing their probable plastic section flexural strength about their minor principal axis.

C6.5.4 Concrete encased steel beams and columns

C6.5.4.1 General

If the concrete encasement of steel members complies with the requirements of NZS 3404:1997 for composite member action, the assessment of such members should be undertaken in accordance with NZS 3404:1997, consistent with the determination of probable strength as specified in these assessment guidelines.

The probable capacity of encased steel members not satisfying the requirements of NZS 3404:1997 should be determined as discussed below.

Note:

The structural members of old steel frames are generally encased in lightly reinforced concrete. In some cases the concrete encasement is unreinforced and has low compressive strength, and is therefore generally considered to play a fire protection role only (Bruneau and Bisson, 2000).

If the concrete encasement of old steel frames is reinforced, the reinforcement is often nominal and consists of plain round bars and thin wire meshes. Inadequately reinforced concrete encasement results in a significant increase in stiffness and a relatively small increase in strength of the encased members.

C6.5.4.2 Concrete encased steel beams, solid sections

The concrete encasement should be assumed to suppress local buckling. The probable strength of such beams should be based on the strength of the steel member only, with slight strength enhancement allowed for due to the concrete encasement:

$$M_{\mathrm{prob,s}} = 1.1 S f_{\mathrm{y}} \qquad \text{...C6.6}$$

where:

S = plastic section modulus
f_{y} = probable yield strength.

The moment-rotation relationship of concrete encased steel beams is similar to that provided in Figure C6.9. The parameters for the generic relationship for this type of member should be taken from Table C6.4 and the probable yield rotation from Equation C6.4.

C6.5.4.3 Concrete encased steel columns, solid sections, small changes in cross section area or moment of inertia of the encased steelwork within a storey height

The concrete encasement should be assumed to suppress local buckling of the encased steel elements and lateral buckling for moment. However member buckling in compression needs to be considered in accordance with Clause 6.3 of Wood (1987). Alternatively, use the column design curve from NZS 3404:1997 for $\alpha_{\mathrm{b}} = 0.0$ to determine the slenderness reduction factor, with the effective length factor $k_{\mathrm{e}} = 1$ in accordance with Clause 12.8.2.4 of NZS 3404:1997.

The probable flexural capacity of such columns should be based on the probable flexural capacity of the steel members only.

The moment-rotation relationship of concrete encased steel columns is similar to that provided in Figure C6.9. The probable plastic hinge rotation capacity (θ_{cap}) should be determined from Table C6.4 (Category 2) and the probable yield rotation capacity from Equation C6.5. However, for the columns to be considered to have ductile capacity they should satisfy the axial load limitations of Clause 12.8.3 of NZS 3404:1997.

C6.5.4.4 Concrete encased steel columns, laced and battened sections or solid sections with significant changes in the cross section area or moment of inertia of the encased steelwork within a storey height

The response of encased laced and battened columns is considered nominally ductile and, as such, θ_{p} should be limited to that for a Category 3 member in Table C6.4.

The probable flexural capacity of this type of column should be based on the probable flexural capacity of the steel elements only, while the probable yield rotation may be taken as 5 mrad in lieu of a detailed analysis.

The probable compression capacity of laced and battened columns should be determined from Clause 6.4 of NZS 3404:1997 using probable material strengths.

C6.5.5 Braces

C6.5.5.1 Compression capacity

The performance of braces that are subjected to earthquake induced compression forces principally depends on the slenderness ratio of the braces.

Braces with a slenderness ratio $\frac{k_e l}{r}\sqrt{\frac{f_y}{250}}$ that is:

> 120	generally do not have the capacity to carry compressive inelastic earthquake demand and their capacity is exceeded typically through elastic buckling
$\leq$ 120	should be expected to buckle inelastically through local yielding under the combined actions of compression and bending.

where:

k_e	=	member effective length factor
l	=	member length
r	=	radius of gyration
f_y	=	probable yield strength

The probable capacity of braces in compression, $N_{prob,c}$, should be determined from Chapter 6 of NZS 3404:1997 using probable material strengths. Note that the flexural demand due to the self-weight of the brace and any other gravity load acting on the brace should be allowed for when determining the capacity of a brace acting in a horizontal plane (e.g. roof bracing).

When a compression brace buckles inelastically the same peak compression capacity as achieved in a previous cycle is generally not likely to be achieved during subsequent cycles of loading. A typical force-displacement relationship for a brace in compression is presented in Figure C6.10. The values of the parameters in the figure are provided in Table C6.6.

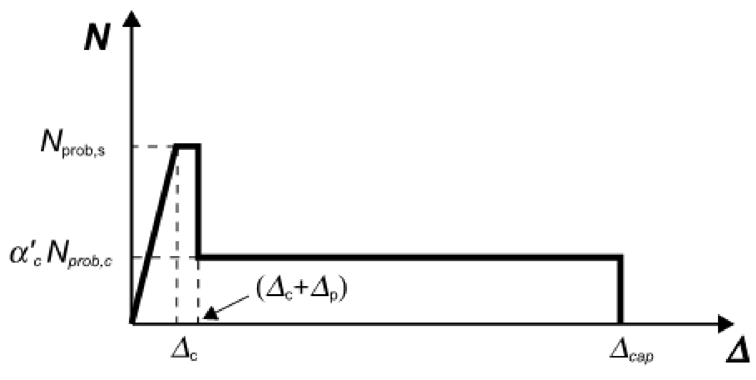


Figure C6.10: Force-displacement relationship for steel braces in compression

Table C6.6: Force-displacement parameters for steel braces in compression

Modified slenderness ratio λ_n	Component type	Deformation		Residual strength factor α_c'
		Δ_p	Δ_{cap}	
≤ 60	I-section, double angle (2L) in-plane	Δ_c	$8\Delta_c$	*
	Hollow section, double angle (2L) out-of-plane	Δ_c	$7\Delta_c$	*
≥ 120	I-section, double angle (2L) in-plane	$0.5\Delta_c$	$9\Delta_c$	*
	Hollow section, double angle (2L) out-of-plane	$0.5\Delta_c$	$8\Delta_c$	*
	Single angle	$0.5\Delta_c$	$10\Delta_c$	*
$60 < \lambda_n < 120$	All	Linearly interpolate		*

Note:

* given by Equation C6.7 or C6.8 as appropriate

λ_n should be determined from Clause 6.3.3 of NZS 3404:1997 as: $\lambda_n = \left(\frac{k_e l}{r}\right)\sqrt{k_f}\sqrt{\frac{f_y}{250}}$

Δ_c is the probable elastic axial deformation of a brace at buckling (N^*=$N_{prob,c}$)

Δ_p is the probable plastic axial deformation capability of a brace before degradation of strength

Δ_{cap} is the probable deformation capacity of a brace.

The residual strength factor (α_c') for compression braces that are likely to remain nominally ductile due to XXX%ULS shaking is given as:

$$\alpha_c' = 7.7/\lambda_n^{0.6} \leq 1.0 \quad \text{...C6.7}$$

and for compression braces that are likely to be subjected to ductile demand at XXX%ULS shaking as:

$$\alpha_c' = 42.15/\lambda_n^{1.1} \leq 1.0 \quad \text{...C6.8}$$

C6.5.5.2 Tension capacity

A typical force-displacement relationship of braces in tension is provided in Figure C6.11. The values of the parameters in the figure are given in Table C6.7. The probable capacity of braces in tension $N_{prob,s}$ should be determined from Chapter 7 of NZS 3404:1997.

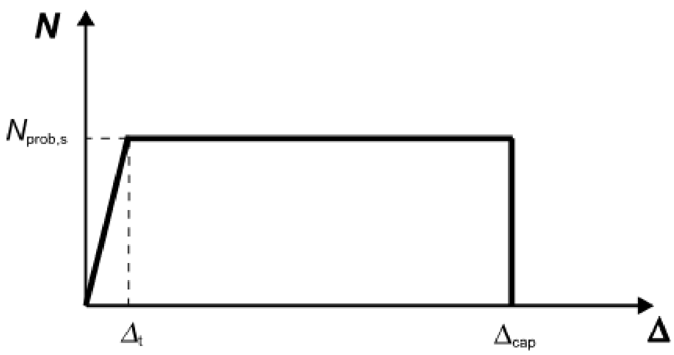


Figure C6.11: Force-displacement relationship for steel braces in tension

Braces in tension are not considered to have any residual capacity after their displacement capacity, Δ_{cap}, is exceeded.

Note:

The probable tensile capacity of a brace should not be taken as greater than the probable capacity (in tension and/or in shear) of the connections at either end.

Table C6.7: Force-displacement parameters for steel braces in tension

Component type	Deformation
	Δ_{cap}
I-section	$11\Delta_t$
Hollow section	$8\Delta_t$
Single angle	$9\Delta_t$
Double angle (2L)	$10\Delta_t$
Rod bracing	$8\Delta_t$

Note:
Δ_t is the axial deformation of a brace at yield (N^*=$N_{prob,s}$).

C6.5.6 Active links of eccentrically braced frames

When subjected to earthquake induced forces, an active link of an EBF responds in either a shear ($e \leq 1.6M_s/V_w$), flexural ($e \geq 3M_s/V_w$) or combined shear and flexural ($1.6M_s/V_w < e < 3M_s/V_w$) mode depending on the clear length of the active link (e).

The probable shear and flexural capacities of an active link should be determined from Section C6.5.2.

The force-rotation relationship of active links is similar to that provided in Figure C6.11. However, $N_{prob,s}$, Δ_y and Δ_{cap} in Figure C6.11 should be replaced with V_w, γ_y and γ_{cap} respectively to obtain the force-rotation relationship for active links. The values of the parameters in the force-rotation relationship are provided in Table C6.8.

Table C6.8: Force-rotation parameters for active links of EBFs

Active link length, e	Deformation* $mrad$
	γ_{cap}
$e \leq 1.6M_s/V_w$	$\gamma_y + 140$
$e \geq 3M_s/V_w$	Same as beams
$1.6M_s/V_w < e < 3M_s/V_w$	Linearly interpolate

Note:
γ_y is the rotational deformation of an active link at yielding.

C6.6 Connection Capacities

C6.6.1 General

Assessing the capacity of a steel frame connection involves determining the load path through the connection, identifying weak links, and then evaluating the probable strength and ductility capacity of those weak links.

The following advice should help when determining the load path through a connection and the weakest link in a load path:

- Determine the internal forces that could be generated in the attached members during an earthquake.
  - An I-section beam responding elastically under flexure will deliver axial forces through the flanges (tension and compression) and vertical shear through the web.
  - An I-section beam responding inelastically under flexure will deliver axial yield forces through the flanges and axial yield forces plus vertical shear through the web.
  - A brace will deliver axial forces (tension is critical) through all its elements.
- Trace the transfer of forces from elements of the supported member into elements of the supporting member that lie parallel to the incoming force. For example, the incoming axial forces from an I-section beam flange connected to an I-section column should be transferred through the column flange into the column web.
- Calculate the probable capacity of all elements along the identified load path in accordance with the provisions of Sections C6.6.3 and C6.6.4.
- If there are no tension and compression stiffeners in a column adjacent to incoming beam flanges in a moment-resisting beam to column connection, then tensile distortion of the flange of the column or compression buckling of the web of the column web are likely to occur before the beam can develop its full flexural capacity.
- The strength and ductility capacity of a load path is determined by the strength and ductility capacity of the weakest component in the load path.
- If various load paths exist through a connection, the stiffest of the load paths will attract the most force.
- Be particularly aware of situations where the connectors (rivets, bolts or welds) may be the weakest component, as their ductility capacity will be limited. One sided fillet welds in tension or bending are particularly vulnerable in this regard, showing no ductility.
- Be aware of component forces introduced when an applied force changes direction along the load path.

Note:

The article by Blodgett (1987) on welds explains the concept of load paths through welded connections and illustrates this with a number of examples.

C6.6.2 Strength modification coefficients

Probable strength of structural steel connections should be taken as the values determined using probable material strengths reduced by the strength modification coefficients provided in Table C6.9.

Table C6.9: Strength modification coefficients for steel connections

Component	Action	Strength modification coefficient	
Bolted connections	Ply in bearing	1.0	
	Bolt shear, tension, and combined actions	0.9	
Pin connections	Ply in bearing	1.0	
	Pin shear, tension, and combined actions	0.9	
		SP	GP
Welded connections	Complete penetration butt welds	1.0	0.7
	Incomplete penetration butt, fillet, plug and slot welds	0.9	0.7

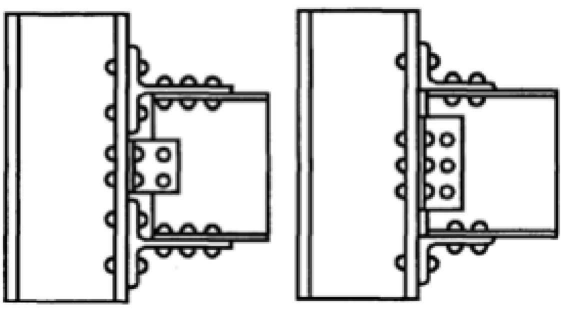
Note:

Strength modification coefficients are not to be confused with strength reduction factors, which for assessment are taken as 1.0. Strength modification coefficients are intended to better define the probable capacity of the defined components.

C6.6.3 Bolted and riveted connections

C6.6.3.1 General

Most old riveted or bolted beam to column connections in New Zealand are believed to be clip angle connections (refer to Figure C6.12(b)). While riveted connections were common in many pre-1950s steel frame buildings, rivets were gradually phased out after this and replaced with bolts as riveting was labour intensive.



(a) Tee-stub **(b) Clip angle**

Figure C6.12: Typical riveted connections (Roeder et al., 1996)

A simplified procedure for determining the moment-rotation relationship of clip angle connections is provided in Section C6.6.3.2. Assessment of other types of historical bolted and riveted connections may be determined using the procedure outlined by Roeder et al. (1996).

The assessment of bolted and riveted connections should be undertaken in accordance with the following:

- Probable shear capacity of rivets, $V_{\text{prob,f}}$, can be determined from Barker (2000). The key equation is derived from the bolt shear capacity provisions of NZS 3404:1997 and is given as:

$$V_{\text{prob,f}} = 0.75 f_{\text{uf}} k_{\text{r}} n_{\text{x}} A_{\text{o}} \qquad \ldots\text{C6.9}$$

  where:

  f_{uf} = probable tensile strength of the rivet
  k_{r} = reduction factor given in Table 9.3.2.1 of NZS 3404:1997 to account for the length of a lap connection (L_{j}). $k_{\text{r}} = 1.0$ for $L_{\text{j}} < 300$ mm and for all other type of connections
  n_{x} = number of connector shear planes intercepting the shear plane
  A_{o} = nominal plain shank area of the rivet.

- Probable tension capacity of rivets should be determined using Clause 9.3.2.2 of NZS 3404:1997, with the value of probable tensile strength of the rivet (f_{uf}) determined from Section C6.4.6 as:

$$N_{\text{prob,tf}} = A_{\text{s}} f_{\text{uf}} \qquad \ldots\text{C6.10}$$

where:

A_{s} = gross tensile stress area of the rivet.
f_{uf} = probable tensile strength of the rivet

- The diameter of a rivet shank should be determined from the diameter of the rivet head in accordance with Figure C6.13.
- Be aware that some less scrupulous erectors made up some dummy rivets from moulded putty covered in paint on larger groups of rivets. Hitting each rivet with a hammer will soon identify any dummy ones!
- Assume that concrete encasement, if present and with any amount of confining reinforcement, will prevent local buckling of the steel members. This assumption may not hold for members in regions subject to significant inelastic demand and will need to be assessed more closely for such regions.

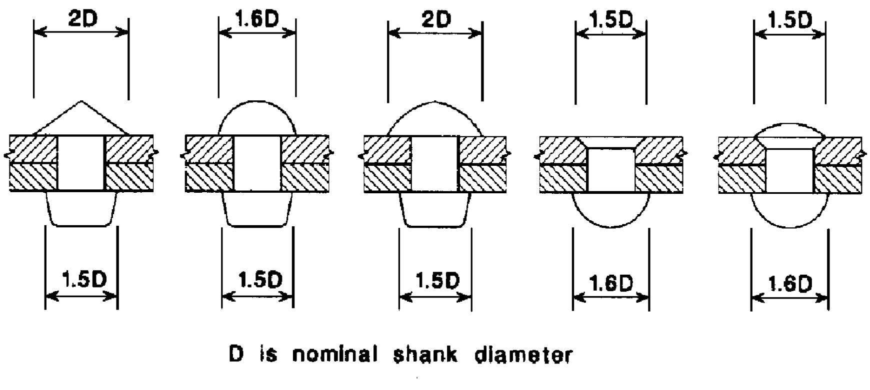


Figure C6.13: Typical rivet shank and head diameters (Bussell, 1997)

- When determining the capacity of a connection, assume that:
  - the connections to beam flanges develop and transfer flexure-induced axial forces from the beam to the column
  - the connections of the beam web to the column flange transfer gravity and earthquake-induced vertical forces and will also transfer horizontal forces, if a suitably stiff and strong horizontal load path from the beam web into the column flange is available, and
  - if there is a direct connection between the beam web and the column flange via welded or bolted plates or cleats, and if this connection is independent of the beam flange to column connection, then for seismic assessment the vertical shear capacity can be assumed to be adequate.

C6.6.3.2 Behaviour of clip angle connections

Clip angle connections are generally weaker and more flexible than other semi-rigid connections and behave as partially restrained connections. The hysteretic behaviour of clip angle connections is relatively poor, but the connections are often able to sustain large deformation demands (Roeder et al., 1996).

The experimental tests undertaken on historical riveted connections by Roeder et al. (1996) revealed that the mode of failure of clip angle connections under cyclic loading was similar to that under monotonic loading. Both monotonic and cyclic load tests deteriorate and fail at similar levels deformation demands, as shown in Figure C6.14. The monotonic tests typically provided an upper bound envelope for the cyclic tests.

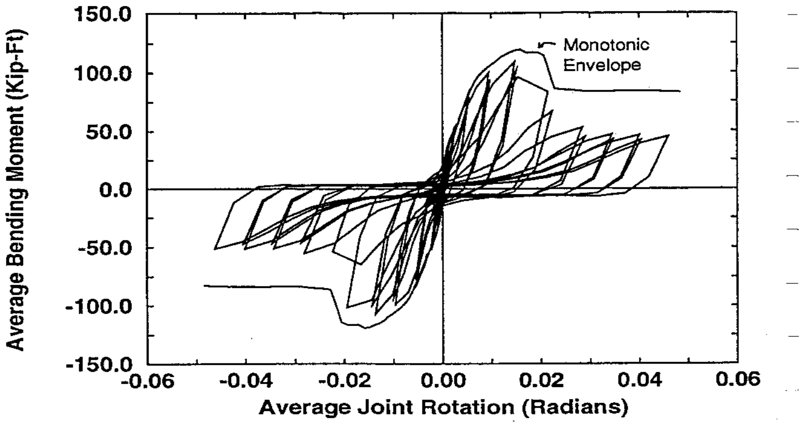


Figure C6.14: Comparison of monotonic and cyclic moment–rotation behaviour

Both concrete encased and bare connections were observed to experience strength degradation at rotations in the order of 20-25 milliradians. It was also observed that concrete encasement improved performance by suppressing any local deformation until the concrete was crushed at larger deformation demands due to lack of adequate confinement.

The capacity enhancement provided by the composite action of concrete encasement and floor slabs to connection capacity was observed to be substantial and in the range of 30-100%. Concrete encasement significantly increased the strength and stiffness of the weaker and more flexible connections such as clip angle connections (refer to Figure C6.15). The capacity of the bare connections was observed to deteriorate significantly when the clip angles to the beam flanges failed. However, flexural capacity was not completely lost because of the resistance provided by the web cleat angle connections.

It should be noted that bolted clip angle connections would be stiffer and would have more rotational capacity than comparable riveted connections. However, the limits on the overall system inelastic displacement would be such that bolted connections cannot attain their full capacity. For example, when the connections are the weakest element, rotational demand on the connections will be around 30 milliradians maximum for an inter-storey drift of 2.5%. Therefore, a 40 milliradians limit on rotation is considered a practical upper limit for the system as a whole, even if the individual connection is capable of greater rotations while maintaining a dependable level of flexural capacity.

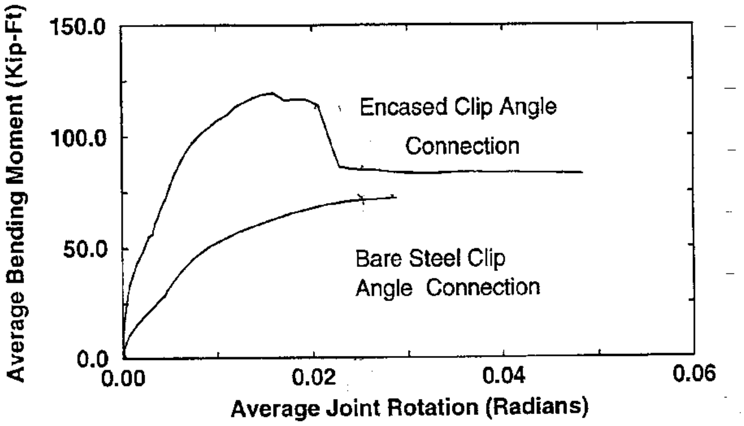


Figure C6.15: Comparison of bare steel and encased moment–rotation behaviour of clip angle connections (Roeder et al., 1994)

C6.6.3.3 Simplified assessment procedure for clip angle connections

General

The strength and rotation capacity of bolted and riveted clip angle connections (illustrated in Figure C6.16) can be determined from first principles and using the guidance presented in this section. The procedure includes a method for determining the probable flexural strength, along with expressions for estimating the probable rotational capacity. Both flexural strength and degradation threshold are considered to be a function of the expected mode of failure of the connections to the beam flanges.

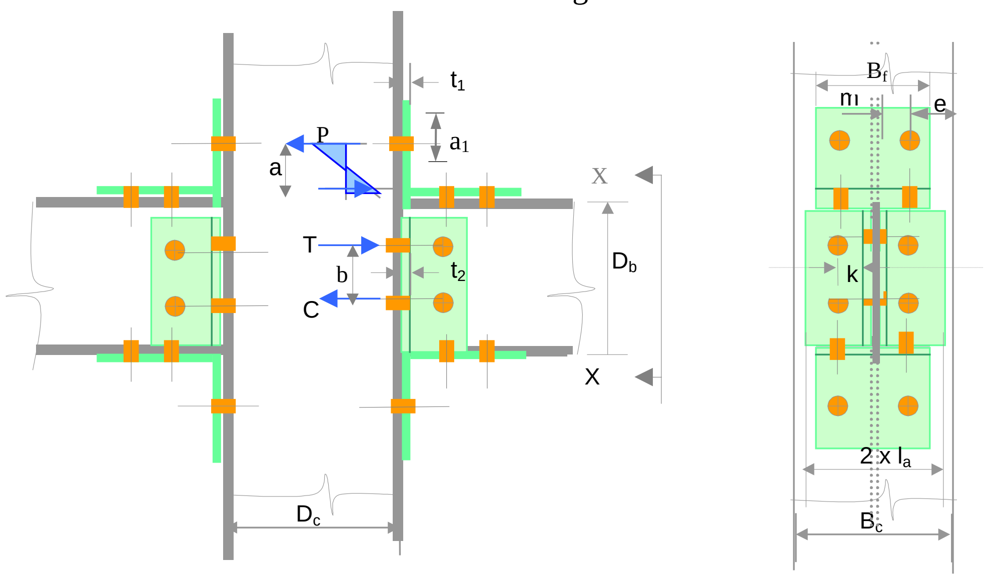


Figure C6.16: A clip angle riveted connection

The flexural strength of a clip angle connection is limited by the smallest demand required to form one of the following four yielding/shear failure modes (Roeder et al., 1996):

- shear yielding/failure of the connectors
- tensile capacity of flange cleat angles
- tensile capacity of connectors, or
- flexural yielding of connection elements (flange cleat angles and/or web cleat angles).

Shear yielding/failure of connectors

Shear yielding/failure of connectors that are provided between the horizontal leg of the flange cleat angles and the beam flange often dictates the flexural capacity of clip angle connections in old buildings.

The probable shear strength of connectors, $V_{\mathrm{prob,f}}$, can be determined from Equation C6.9. The probable flexural strength of a clip angle connection limited by the shear strength of the connectors, M_{prob}, can be determined from:

$$M_{\mathrm{prob}} = nV_{\mathrm{prob,f}}D_{\mathrm{b}} \qquad \text{…C6.11}$$

where:

n	=	the number of connectors
D_{b}	=	the depth of the beam.

Tensile capacity of flange cleat angles

The strength of the horizontal leg of the flange cleat angle in tension may limit the flexural capacity of clip angle connections. The probable tensile strength of a flange cleat angle $N_{\mathrm{prob,t}}$ can be determined from (NZS 3404:1997):

$$N_{\mathrm{prob,t}} = A_{\mathrm{g}}f_{\mathrm{y}} \leq 0.85k_{\mathrm{te}}A_{\mathrm{n}}f_{\mathrm{u}} \qquad \text{…C6.12}$$

where:

A_{g}	=	gross area of the cross section
f_{y}	=	probable yield strength of the section
k_{te}	=	tcorrection factor in accordance with Clause 7.3 of NZS 3404:1997
A_{n}	=	net area of the cross section
f_{u}	=	probable tensile strength of the section.

The probable flexural strength of a clip angle connection limited by the tension capacity of the flange angles, M_{prob}, can be determined from:

$$M_{\mathrm{prob}} = N_{\mathrm{prob,t}}\left(D_{\mathrm{b}} + {}^{t_1}/_{2}\right) \qquad \text{…C6.13}$$

where:

t_1	=	the thickness of the flange cleat angle leg.

Tensile capacity of connectors

The tensile capacity of the connectors provided between the vertical leg of the flange cleat angle and the column flange may also control the flexural strength of a clip angle connection. Experimental tests have shown that this failure mode is the least ductile with a rapidly deteriorating capacity.

The probable tensile strength of connectors, $N_{\mathrm{prob,tf}}$, can be determined from Equation C6.10 and the probable flexural strength of a clip angle connection limited by the probable tensile strength of the connectors, M_{prob} , can be determined from:

$$M_{\mathrm{prob}} = nN_{\mathrm{prob,tf}}\,(d_{\mathrm{b}} + a) \qquad \text{…C6.14}$$

where:

n = the number of connectors

a = the distance between the centre of the connectors and the flange cleat angle leg.

Flexural yielding of flange cleat angles

Flexural yielding of the vertical leg of the flange cleat angle connected to the column flange is the fourth mode that may limit the flexural strength of clip angle connections.

Flexural yielding of the flange cleat angle requires development of prying actions. However, the prying forces that develop in connections that use mild steel connectors are typically not likely to cause the capacity of the connectors to be exceeded.

The probable flexural strength of a clip angle connection reduced by prying actions, M_{prob}, is given as:

$$M_{\mathrm{prob}} = \left(\frac{\frac{B_{\mathrm{f}} t_1^2}{4a} f_{\mathrm{y}} + \frac{a_1}{a} n N_{\mathrm{prob,tf}}}{1 + \frac{a_1}{a}} \right) \left(D_{\mathrm{b}} + {t_1}/{2} \right) \qquad \text{…C6.15}$$

where:

B_{f} = the length of the angle

a_1 = the distance between the centre of the connectors and the top edge of the flange cleat angle.

If the connectors are strong enough to induce flexural yielding of the flange cleat angles, the probable flexural strength can be determined from:

$$M_{\mathrm{prob}} = \frac{B_{\mathrm{f}} t_1^2}{2a} f_{\mathrm{y}}\,(D_{\mathrm{b}} + t_1/2) \qquad \text{…C6.16}$$

Flexural yielding of web cleat angles

If flexural yielding of the flange cleat angle governs the probable flexural strength of a clip angle connection, the flexural strength of the web cleat angle can be considered to contribute to the overall connection strength.

The probable flexural capacity of the web cleat angle can be determined from:

$$M_{\mathrm{prob}} = \frac{l_{\mathrm{a}} t_2^2}{2} f_{\mathrm{y}} \quad \text{...C6.17}$$

where:

l_{a}	=	the length of the web cleat angle face
t_2	=	thickness of the web cleat angle leg
f_{y}	=	probable yield strength.

From Equation C6.17, the tensile force in the web cleat bolts/rivets is:

$$T = \frac{2M_{\mathrm{prob}}}{k} \quad \text{...C6.18}$$

where:

k	=	the distance between bolt centreline and the web cleat angle leg.

Probable tensile strength of the column flange is given as:

$$T_{\mathrm{c}} = (4m + 1.25e) t_{\mathrm{c}} f_{\mathrm{yc}} \quad \text{...C6.19}$$

where:

m	=	distance from centre of bolt hole to radius root at web
e	=	distance from rivet centre to flange edge
t_{c}	=	thickness of the column flange
f_{yc}	=	probable yield strength of the column flange.

The contribution of the web cleat angle to the probable flexural strength of the connection is:

$$M_{\mathrm{prob}} = Qb \quad \text{...C6.20}$$

where:

Q	=	either T from Equation C6.18 or T_{c} from Equation C6.19, whichever is less
b	=	the distance between the centroid of tension and compression forces in the web cleat.

C6.6.3.4 Moment-rotation behaviour of riveted clip angle connections

The moment-rotation behaviour of riveted clip angle connections is provided in Figure C6.17 based on the experimental studies undertaken by Roeder et al. (1996). The values of the parameters in the figure are provided in Table C6.10.

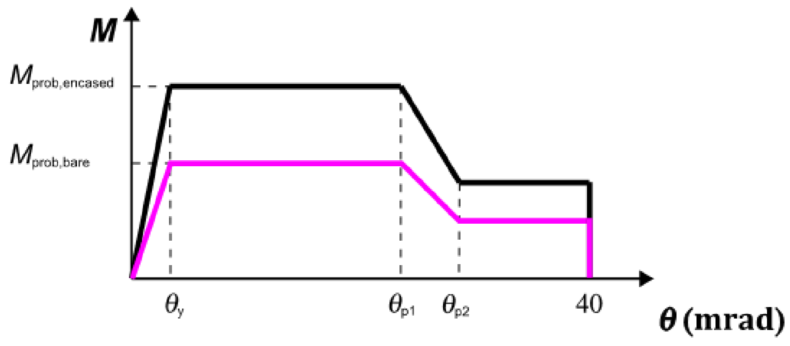


Figure C6.17: Moment-rotation behaviour of clip angle connections

Table C6.10: Moment-rotation parameters for clip angle connections

Mode of failure	Probable yield rotation $(mrad)$	Plastic rotation $(mrad)$		Residual strength
	θ_y	θ_{p1}	θ_{p2}	
Tensile yielding of connectors	5	$3.75/d_b$	$\theta_{p1} + 5$	$0.5M_{prob}$
Shear yielding of connectors	5	$7.5/d_b$	$\theta_{p1} + 5$	$0.5M_{prob}$
Flexural yielding of connecting elements	5	$12.5/d_b$	$\theta_{p1} + 5$	$0.5M_{prob}$

Note:
d_b is depth of beam (m)
$M_{prob,encased} = 2M_{prob,bare}$

C6.6.4 Welded connections

Welded connections are able to transfer the moment-induced beam actions into columns if the various components along the load path have the required capacity as indicated in Table C6.12. The required checks are outlined in Figure C6.18 and Table C6.11.

Note:

As discussed in Section C6.3.2, fractures of welded beam-column connections were widely reported after the 1994 Northridge earthquake, with the majority of these fractures observed at the bottom beam-column flange connections. Refer to that section for more discussion, including a list of the factors considered to have contributed to the brittle failures of "Pre-Northridge" connections (FEMA 355E, 2000).

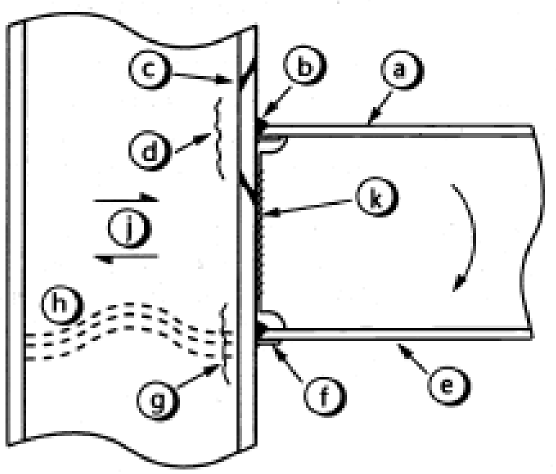


Figure C6.18: Components of welded connections requiring checks (SCI, 1995)

Table C6.11: Components of welded connections requiring checks (SCI, 1995)

Zone	Reference on Figure C6.18	Checklist item
Tension	a	Beam flange
	b	Flange weld
	c	Column flange in bending
	d	Column web in tension
Compression	e	Beam flange
	f	Flange weld
	g	Column web crushing
	h	Column web buckling
Horizontal shear	j	Column web panel shear
Vertical shear	k	Fin plate or direct weld to column

Table C6.12: Probable capacities of components of welded connections requiring checks (NZS 3404:1997; SCI, 1995)

Item	Equation	Equation number
Beam flange capacity	$N_{\text{fbt}} = N_{\text{fbc}} = 1.2[\min(b_{\text{fb}}, b_{\text{fc}})]t_{\text{fb}}f_{\text{yb}}$	…C6.21
Column flange tension capacity*	$N_{\text{fct}} = b_{\text{eff}}t_{\text{fb}}f_{\text{yb}}$ $b_{\text{eff}} = t_{\text{wc}} + 2r_{\text{c}} + 7t_{\text{fc}}$ $b_{\text{eff}} \leq t_{\text{wc}} + 2r_{\text{c}} + 7\left[\frac{t_{\text{fc}}^2 f_{\text{yc}}}{t_{\text{fb}} f_{\text{yb}}}\right]$ $b_{\text{eff}} \leq b_{\text{fb}} \leq b_{\text{fc}}$	…C6.22
Column web tension capacity	$N_{\text{wct}} = [t_{\text{fb}} + 2s_{\text{f}} + 5(t_{\text{fc}} + r_{\text{c}})]t_{\text{wc}}f_{\text{yc}}$	…C6.23
Column web crushing	$N_{\text{wcc}} = (b_1 + n_2)t_{\text{wc}}f_{\text{yc}}$ $b_1 = t_{\text{fb}} + 2s_{\text{f}}$ $n_2 = 5(r_{\text{c}} + t_{\text{fc}})$	…C6.24
Column web buckling	$N_{\text{wcc}} = (b_1 + n_1)t_{\text{wc}}f_{\text{yc}}$ $n_1 = d_{\text{c}}$	…C6.25
Column web panel shear (unstiffened web)+	$V_{\text{c}} = 0.6d_{\text{c}}t_{\text{wc}}\, f_{\text{yc}}\, \eta \left[1 + \frac{3b_{\text{fc}}\, t_{\text{fc}}^2}{d_{\text{b}}d_{\text{c}}t_{\text{wc}}}\right]$ $\eta = \sqrt{(1.15 - (N^*/N_{\text{s}})^2} \leq 1$	… C6.26
Beam flange weld	$N_{\text{tfw}} = b_{\text{eff}}f_{\text{uw}}$	…C6.27

Note:

*If $b_{\text{eff}} < 0.7b_{\text{fb}}$, tension stiffeners are necessary to avoid weld tearing at the point of peak stress.

b_1, n_1, & n_2 should be reduced if the column projection is insufficient for full dispersal.

+ If doubler plates are provided, $(t_{\text{wc}} + t_{\text{p}})$ should replace t_{wc} in Equation C6.26.

where:

b_1	=	width of contact between beam flange and welds and column
b_{eff}	=	effective beam flange width
b_{fb}	=	beam flange width
b_{fc}	=	column flange width
d_{b}	=	beam depth
d_{c}	=	column depth
f_{yb}	=	probable yield strength of beam
f_{yc}	=	probable yield strength of column
f_{uw}	=	probable strength of weld metal
n_1	=	length obtained by a 45° dispersion though half the depth of the column
n_2	=	length obtained by a 1:2.5 dispersion though column flange and root radius
N^*	=	axial load in column below joint
N_{fbc}	=	probable compression capacity of beam flange
N_{fbt}	=	probable tension capacity of beam flange
N_{fct}	=	probable tension capacity of column flange
N_{s}	=	probable column section compression capacity
N_{tfw}	=	probable tension capacity of beam flange weld
N_{wcc}	=	probable compression capacity of column web
N_{wct}	=	probable tension capacity of column web
r_{c}	=	column root radius
s_{f}	=	weld leg length to beam tension flange (when available)
t_{fb}	=	beam flange thickness
t_{fc}	=	column flange thickness
t_{p}	=	total thickness of doubler plates
t_{wc}	=	column web thickness
V_{c}	=	probable shear capacity of panel zone.

The demand on beam flanges of welded beam-column connections is determined from:

$$N^*_{\mathrm{fbt}} = \frac{M^*}{d_{\mathrm{b}} - t_{\mathrm{fb}}} - \frac{N^*}{2} \qquad \ldots\mathrm{C6.28}$$

$$N^*_{\mathrm{fbc}} = \frac{M^*}{d_{\mathrm{b}} - t_{\mathrm{fb}}} + \frac{N^*}{2} \qquad \ldots\mathrm{C6.29}$$

where:

N^*_{fbt} = tension demand on beam flange
N^*_{fbc} = compression demand on beam flange
N^* = axial load in column below joint
M^* = moment in beam.

If the various components of a welded connection do not have the required capacity to resist beam/column overstrength demand, as would be the case for an unstiffened column that is typical of old buildings, the moment-rotation behaviour of the connection should be taken from Table C6.13 and the general shape of the moment-rotation curve should take the form of Figure C6.17.

Table C6.13: Moment-rotation parameters for welded connections

Mode of failure	Probable yield rotation ($mrad$)	Plastic rotation ($mrad$)		Residual strength
	θy	θ_{p1}	θ_{p2}	
Flange weld failure	3	$3.75/d_{\mathrm{b}}$	$\theta_{\mathrm{p1}} + 5$	$M_{\mathrm{prob,w}}$
Note: d_{b} is depth of beam (m) $M_{\mathrm{prob,encased}} = 1.3M_{\mathrm{prob,bare}}$				

$M_{\mathrm{prob,w}}$ is the probable flexural capacity of the beam web column connection and needs to be determined from the particular connection detail adopted. This capacity is determined from:

- the probable capacity of the connection, if the beam web is connected to the column flange using clip angles, or
- the probable plastic flexural capacity of the beam web, if the beam web is connected using balanced, double sided fillet welds, or butt welds of sufficient strength to yield the web in tension.

If a beam-column connection is suspected of being welded but the connection is not visible (e.g. due to concrete encasement) and if no drawings are available, the encasement material should be removed from a representative connection so that a reasonable assessment can be undertaken. The difference in connection moment-rotation capacity between a connection that can transfer the beam flange axial forces induced by inelastic beam action dependably into the column and one that cannot is significantly large that the capacity should be determined and not guessed.

Similarly, the existing state of welds needs to be assessed using visual inspection techniques. Engineers undertaking weld inspections should be familiar with visual inspection techniques such as those outlined by Hayward and McClintock (1999).

C6.7 Global Capacity

C6.7.1 Assumptions

Guidance provided in this section for determining the global capacity of steel framed buildings assumes the following:

- The form of the connections is such that the strengths and the elastic and post-elastic stiffness of the connections can be determined by rational assessment.
- The steel members consist of either solid I-sections or sections built up by plates, which are connected by rivets, bolts or welds, and where the strength of the connectors can be determined by rational assessment.
- The member sizes and connection details can be ascertained with sufficient accuracy to undertake the assessment. This will typically require the availability of structural drawings containing critical details or selective removal of non-structural and concrete encasements surrounding the frames to expose critical members and connections.
- Concrete encasement to the steel frames is considered to play a fire protection role only and is not sufficiently reinforced to contribute significantly to the strength or stiffness of the frames.

If the concrete encasement is well reinforced and is likely to contribute to the strength and stiffness of the steel frame, the contribution of the composite section should be determined. Note that this is very unlikely in pre-1976 building encased beams and is more likely in pre-1976 building encased columns. Column encasement is advantageous as it increases column strength relative to beam strength.

C6.7.2 Global capacity of steel moment resisting frames

C6.7.2.1 General procedure

Determining the global capacity of a steel MRF principally involves identifying the governing inelastic mechanism and the associated deformation capacity, which entails assessing the strength hierarchy throughout the frame.

The influence of inelastic response on overall response is considered to be insignificant on steel MRFs exhibiting the following "good features":

- The strength hierarchy at all floor levels is beam sidesway except on the uppermost seismic mass level.
- If the connections are the weakest links, the evaluation of the connections in accordance with Section C6.6 shows the following:
  - The weakest components of the connections are not the connectors (welds, rivets and/or bolts). In addition, the capacity of the connections is not limited by the net tension failure of components.
  - When the peak flexural strength of the connections is exceeded, the connections are able to retain their integrity and maintain their shear and axial force carrying capacity.
- None of the beam to column connections has the potential to introduce local buckling or tearing failure in the columns (e.g. lack of stiffeners adjacent to an incoming beam flange in a welded beam to column connection).

- The assessed inelastic response of the system (this assessment is qualitative rather than quantitative) is essentially symmetrical in nature and does not contain features that will inevitably lead to a progressive deformation of the building in one direction only.

If the ductility demand on a steel frame due to XXX%ULS shaking is not significant ($\mu \leq 1.5$) and the frame exhibits the four "good features" listed above, the inelastic response of the frame does not need to be assessed.

A step-by-step hand procedure is provided below on a rapid determination of the global capacity of steel MRFs having either beam sidesway or column sidesway as the governing inelastic mechanism. This procedure is applicable to regular frames that have similar bay widths, floor heights, and floor seismic weights. Refer to Section C2 for the assessment of irregular frames.

Step 1

Determine the probable material strength of the members, the elements of the connections and the connectors. Use probable strengths provided in Section C6.4 in the absence of original construction documentation and physical test data.

Step 2

Determine and assemble the probable capacity of the individual members and connections located on potentially critical floor levels. Refer to Sections C6.5.2 and C6.5.3 for beams and columns respectively, and Section C6.6 for connections.

If the individual beams of the frame on each level under consideration cannot support gravity loading ($G + \psi_{\mathrm{E}} Q$) in a simply supported condition, then halve the plastic rotation capacity of the beams (refer to Section C6.5.2) and of the connections (refer to Section C6.6). The reduction in rotational capacity reflects the monotonic, cumulative nature of inelastic demand on the yielding regions of such members.

If the slab is placed in contact with the columns of a frame or insufficient separation is provided, the contribution of the slab to the flexural strength of the beams should be taken into account.

The assessment should include the first level above the seismic ground level, the uppermost seismic mass level, and floor levels where member sizes and/or connection types change.

Step 3

Determine the governing inelastic mechanism of the frame: i.e. beam sidesway or column sideway mechanism.

A sway potential index (S_{i}) can be employed to determine the potential sway mechanism of a frame. A sway potential index can be defined at a storey of the frame by comparing the sum of the probable flexural strengths of the beams (or connections, whichever are smaller) and the columns at the centroid of every joint:

$$S_{\mathrm{i}} = \frac{\Sigma(M_{\mathrm{bl}}+M_{\mathrm{br}})}{\Sigma(M_{\mathrm{ca}}+M_{\mathrm{cb}})} \qquad \ldots\text{C6.30}$$

where:

M_{bl}	=	probable beam (or connection, whichever is smaller) flexural strength to the left of the joint, extrapolated to the centroid of the connection
M_{br}	=	probable beam flexural strength (or connection, whichever is smaller) to the right of the joint, extrapolated to the centroid of the connection
M_{ca}	=	probable column flexural strength above the joint, extrapolated to the centroid of the connection
M_{cb}	=	probable column flexural strength below the joint, extrapolated to the centroid of the connection.

If:

- $S_{\mathrm{i}} < 0.85$, a beam sidesway mechanism is likely to form. It should be noted that a significant change in storey heights increases the likelihood of a column sidesway mechanism.
- $0.85 < S_{\mathrm{i}} < 1$, either a beam sidesway or column sidesway mechanism is likely to form. The effect of both mechanisms need to be assessed.
- $S_{\mathrm{i}} > 1$, a column sidesway mechanism is likely to form.

Note:

When a frame has semi-rigid connections and these connections are flexurally weaker than the beams or the columns, a beam sidesway mechanism forms.

Step 4

Determine the probable base shear capacity, V_{prob}, of the frame.

If the potential inelastic mechanism of a frame is beam sidesway, the probable base shear capacity, V_{prob}, of the frame can be determined from (refer to Figure C6.19):

$$V_{\mathrm{prob}} = \frac{\sum_{\mathrm{i=1}}^{\mathrm{m}} M_{\mathrm{ri}} + \sum_{\mathrm{i=1}}^{\mathrm{n}} V_{\mathrm{bi}} L_{\mathrm{eq}}}{h_{\mathrm{eq}}} \qquad \ldots\text{C6.31}$$

where:

$M_{\mathrm{prob,I}}$	=	probable flexural strength of column i at the base
V_{bi}	=	storey i beam seismic shear demands determined from probable beam flexural strengths as:

$$V_{\mathrm{bi}} = \frac{M_{\mathrm{prob,il}} + M_{\mathrm{prob,ir}}}{L_{\mathrm{b}}} \qquad \ldots\text{C6.32}$$

$M_{\mathrm{prob,il}}$	=	probable beam (or connection, whichever is smaller) flexural strength to the left of an internal joint on floor i, extrapolated to the centroid of the connection
$M_{\mathrm{prob,ir}}$	=	probable beam (or connection, whichever is smaller) flexural strength to the right of an internal joint on floor i, extrapolated to the centroid of the connection
L_{b}	=	bay width
n	=	number of storeys
m	=	number of columns that are fixed at the base
L_{eq}	=	total width of frame

h_{eq}	=	effective height of frame to be determined from the displaced shape of the frame as:

$$h_{\mathrm{eq}} = \frac{\sum_{\mathrm{i=1}}^{\mathrm{n}} m_{\mathrm{i}} H_{\mathrm{i}} \Delta_{\mathrm{i}}}{\sum_{\mathrm{i=1}}^{\mathrm{n}} I} \qquad \text{…C6.33}$$

mI_{i}	=	mass of floor i
I_{i}	=	lateral displacement of floor i
I	=	height of floor i.

Note:

Equation C6.31 provides an upper bound base shear capacity. For the frame to achieve this upper bound base shear capacity, all of the beams and the bases of the columns should start to yield before the rotational capacity of the critical hinge is exceeded.

If the rotational capacity of the critical hinge is likely to be exceeded before some of the beams and/or the bases of the columns start to yield, the flexural resistance developed in the potential plastic hinges that have not started to yield should replace probable flexural strengths in Equation C6.32.

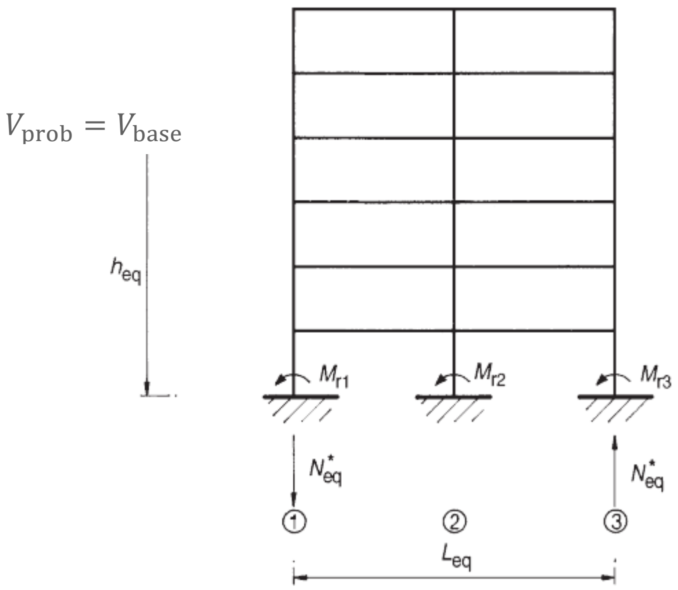


Figure C6.19: Base overturning demand on a beam sidesway governed frame

If the potential inelastic mechanism of a frame is column sidesway, the probable base shear capacity, V_{prob}, of the frame can be estimated from:

$$V_{\mathrm{prob}} = \frac{\sum_{\mathrm{i=1}}^{\mathrm{m}} M_{\mathrm{ri,b}} + \sum_{\mathrm{i=1}}^{\mathrm{m}} M_{\mathrm{ri,t}}}{h} \qquad \text{…C6.34}$$

where:

$M_{\mathrm{prob,ib}}$	=	probable column flexural strengths at the base or bottom of column i extrapolated to the centroid of the connection
$M_{\mathrm{prob,it}}$	=	probable column flexural strengths at the top of column i extrapolated to the centroid of the connection
h	=	storey height.

Step 5

Ensure the axial force demand on the external columns does not exceed the probable limiting axial force (N_{cr}) from Section 12.8.3.1 of NZS 3404:1997:

$$N^*_{\mathrm{eq}} + N^*_{\mathrm{G}+\psi_E\mathrm{Q}} \leq N_{\mathrm{cr}} \qquad \ldots\text{C6.35}$$

where:

N^*_{eq} = earthquake-induced axial force demand

$N^*_{\mathrm{G}+\psi_E\mathrm{Q}}$ = axial force demand due to gravity loading.

If the above equation is not satisfied, reduce the base shear capacity of the frame until it is.

Step 6

Determine the deformation capacity of the frame.

Refer to Section C2 for methods on determining the deformation capacity of frames.

C6.7.2.2 Steel moment resisting frame systems with infill panels

The interaction between steel MRFs and infill panels should be assessed using the guidance provided in Section C7.

The assessment of infilled steel MRFs should allow for the stiffening effect of infill panels on the overall system response. In addition, the presence of infill panels induces increased shear demands on the frame members by creating short column effects. The increased shear demands are unlikely to exceed the capacity of bare steel or concrete encased solid section columns. However, elements of encased laced and battened members may not have sufficient shear capacity. In addition, if the columns have a better shear capacity than the infills and the infills are likely to sustain significant damage, the potential for a soft-storey formation should be taken into consideration.

Steel moment-resisting infilled frames with weak connections should be assessed for the potential for diagonal compression struts formed in infill panels pulling apart beam to column connections as the frames deform laterally. External beam to column connections are likely to be more critical than internal connections.

Note:

The assessment of weak beam to column connections involves comparison of the tension capacity of the connections with the peak compression capacity of the infill panels (capacity prior to deterioration due to panel crushing/shear failure). If the infill panel compression strut capacity is greater than the beam to external column connection tension capacity, failure of this connection needs to be considered for the response of that end bay.

C6.7.3 Global capacity of concentrically braced steel buildings

Concentrically braced frames (CBFs) are braced frames where the centrelines of the braces intersect at a node. CBFs are commonly X-braced or V-braced (refer to Figure C6.20) and rely primarily on the axial strength and stiffness of the braces to resist lateral forces.

The lateral force capacity of CBFs is dependent on:

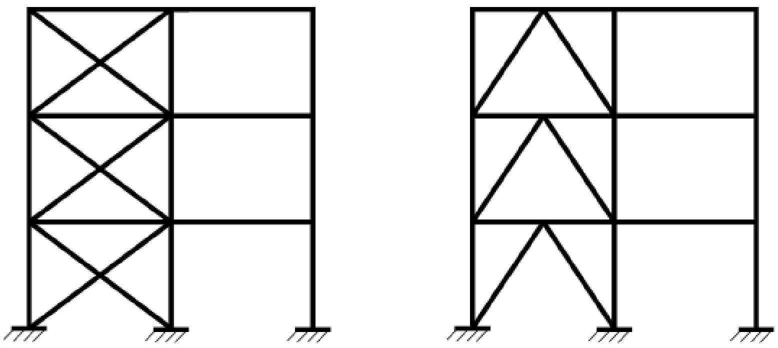
- bracing configuration – X-braced CBFs have an advantage over V-braced CBFs (refer to Figure C6.20, as the inelastic capacity of V-braced CBFs is likely to be governed by the capacity of the collector beam and the post-buckling capacity of the braces only
- the slenderness ratio of the braces – as discussed in Section C6.5.5.1, the slenderness ratio has a significant influence on the deformation capacity and residual strength of compression braces, and
- the capacity of brace connections to the beams and columns of the frame – the connections of the braces should have sufficient capacity to resist demand due to braces yielding in tension or buckling in compression.

Note:

When a compression brace of a V-braced CBF buckles, the capacity of the tension brace may not be fully utilised as the collector beam may not have the capacity to resist the unbalanced vertical force acting at the brace-collector beam joint. Note that the collector beam will have to resist demands due to gravity loads in addition to the unbalanced vertical force.

The buckling of compression braces of V-braced CBFs results in significant reductions in frame lateral stiffness and strength, as the system changes to a D-braced EBF with a long flexural link. In such situations a plastic hinge is likely to form in the collector beam before the tension brace yields in tension.

During the subsequent reversing cycle of earthquake demand, the previously tension brace generally buckles before the braces that buckled during the preceding half cycle fully straighten up (Tremblay and Robert, 2000). Therefore, the inelastic capacity of V-braced frames is limited by the post-buckling capacity of the braces.



(a) X-braced CBFs **(b) V-braced CBFs**

Figure C6.20: Concentrically braced frames (CBFs)

The following steps outline an assessment procedure for X-braced and V-braced CBFs.

Step 1

Determine and assemble the probable capacity of the individual members and connections located on potentially critical floor levels. The capacities to be determined are:

- axial force capacity of the braces
- post-buckling capacity of the braces
- flexural and compression capacity of collector beams
- axial force and flexural capacity of columns
- axial force capacity of connections and splices.

Step 2

Determine the weakest member and the expected mode of failure; i.e. brace, brace connection, collector beam, column, etc.

Step 3

Check whether the frame exhibits the following good features:

- The strength hierarchy involves weak braces at all levels except the uppermost seismic level (rather than weak columns or weak collector beams).
- The columns are continuous over two consecutive storeys.
- The collector beams, columns and the beam to column connections have sufficient capacity to resist the loads generated by the system at the point of brace yielding in tension and brace buckling in compression. In many old braced buildings the brace to beam/column connections are likely to be the weakest components.
- For all beam to column connections the connections should not be of a type that has the potential to introduce local buckling or tearing failure in the column under inelastic rotation due to lack of column tension/compression stiffeners.
- The assessed inelastic response of the system (this assessment is qualitative rather than quantitative) should be essentially symmetrical in nature and not contain features that will inevitably lead to a progressive displacement of the building in one direction.

Step 4

If the ductility demand on the frame due to XXX%ULS shaking is not significant ($\mu \leq 1.5$) and the frame exhibits the above “good features”, the inelastic response of the frame need not be assessed.

If the braces or brace connections are not the weakest component, the capacity of the frames should be limited to the capacity of the weakest member/element if the failure of that member/element constitutes loss of gravity load carrying capacity.

If the brace connections are the weakest component resulting in a rather low lateral force capacity, the frame can be assessed as a moment resisting frame. However, the failure of the brace connections is unlikely to lead to loss of gravity load carrying capacity on their own.

If the ductility demand on the frame due to 100%ULS shaking is significant ($\mu > 1.5$) and the braces are the weakest component, proceed to the next step.

Step 5

Determine the probable base shear capacity of the frame.

The capacity of CBFs can be determined from first principles and the brace member capacity relationships provided in Section C6.5.5.

There is an inherent potential for soft-storey formation in CBFs constructed without following the provisions of NZS 3404:1997. For a typical case of a soft storey forming in the bottom storey of a CBF, the probable base shear capacity, V_{prob} , of a CBF that is effective both in tension and compression can be determined from the post-buckling capacity of the braces in the bottom storey as:

$$V_{\mathrm{prob}} = \sum_{\mathrm{i=1}}^{\mathrm{m}} \alpha'_{\mathrm{ci}} N_{\mathrm{prob,ci}} \; cos\, \theta_{\mathrm{i}} \qquad \text{…C6.36}$$

where:

α'_{ci}	=	residual strength factor for brace i from Section C6.5.5.1
$N_{\mathrm{prob,ci}}$	=	probable compression capacity of brace i
θ_{i}	=	angle between brace i and beam at the top end of the brace
m	=	number of braces.

If a soft storey forms in one of the upper storeys of a CBF, the calculated base shear capacity should allow for the resistance mobilised in the braces that are located in the storeys below the soft-storey level before the capacity of the critical brace is exceeded.

Step 6

Determine the displacement capacity of the frame.

The yield displacement, Δ_{y} , of a CBF may be determined from an elastic analysis of the frame based on the displacements at which the first brace yields in tension (if a tension-only brace) or buckles in compression and a mechanism develops in a storey.

The probable displacement capacity of a single-storey CBF or the probable inter-storey displacement capacity of a multi-storey CBF that is likely to form a soft storey can be determined from the displacement capacity of the critical brace as:

$$\Delta_{\mathrm{cap}} = \sqrt{(L_{\mathrm{b}} + \Delta_{\mathrm{cap,b}})^2 - h^2} - L \qquad \text{…C6.37}$$

where:

L_{b}	=	length of the critical brace
$\Delta_{\mathrm{cap,b}}$	=	displacement capacity of the critical brace from Section C6.5.5.1
h	=	storey height
L	=	width of the braced bay.

C6.8 Assessment of Steel Framed Buildings

C6.8.1 General

Detailed seismic assessments of steel framed buildings, especially those that are considerably old, should not rely solely on drawings. A condition assessment is recommended as part of the DSA and may include inspections to determine:

- any deterioration due to environmental effects
- the physical conditions of members and connections
- configuration and presence of members and connections
- load paths through connections, splices and between members, and
- workmanship.

The global assessment of steel framed buildings may be undertaken using either a displacement or force based assessment procedure as appropriate. This section covers factors specific to the analysis and assessment of these buildings. Refer to Section C2 for overall procedures and appropriate global analysis methods.

C6.8.2 Stiffness of frames

The rotational stiffness of column base connections should be taken into account when undertaking an analysis of steel framed buildings. Fixed column base connections are never infinitely stiff, while pinned column base connections have some rotational stiffness.

Rotational stiffness of column base connections can be determined from NZS 3404:1997 as:

$$k_{\theta} = \frac{kEI_{c}}{L_{c}} \qquad \text{…C6.38}$$

where:

k	=	1.67 for fixed base connections
k	=	0.1 for pinned base connections
I_c	=	second moment of area of the column about the direction under consideration
L_c	=	length of column.
E	=	modulus of elasticity

Note:

Experimental tests undertaken on typical seismic-resisting system foundations have confirmed that the fixed base rotational stiffness recommendation of NZS 3404:1997 is a reasonable value to adopt (AISC, 2012; Borzouie et al., 2016).

When undertaking an elastic analysis of a steel framed building, rigid end blocks having dimensions equal to one half of the beam depth and one half of the column depth should be used at each beam to column connection of the lateral force resisting frame. The use of one half the depth as a rigid end block instead of the full member depth takes account of the flexibility of the panel zone of the connections.

C6.8.3 Seismic actions to ensure ductile mechanisms

As discussed in Section C6.7 a key step in the assessment of an existing steel framed buildings is to check whether non-yielding members and connections (members and connections located outside potential plastic hinge regions) of the primary structure of the building are protected from undergoing inelastic deformations if an overall ductile response of the building is to be assumed.

Comparison of the probable strength of the assumed non-yielding members and connections with the actions generated by the overstrength of potential plastic hinge regions determines whether the members and connections outside these hinge regions of the building are protected. If they are not, development of the full ductile mechanism may not be possible and the overall capacity of the mechanism may reduce.

To ensure that a ductile mechanism can develop as assumed, the assumed non-yielding members and connections should have a probable capacity that is greater than required to resist the actions resulting from yielding the plastic hinge regions at overstrength.

In order to meet the objectives of these guidelines the overstrength actions should be determined using the maximum overstrength factor defined for the particular steel grade in Table 12.2.8(1) of NZS 3404:1997 irrespective of the Category designation.

C6.8.4 Actions on concentrically braced systems

C6.8.4.1 Vertical concentrically braced frames

Non capacity designed CBFs with inelastically responding braces are vulnerable to soft-storey formation. The C_s factor, which needs to be included when determining seismic demand on CBFs in accordance with NZS 3404:1997, accounts for this potential for soft-storey formation and the deterioration in inelastic performance of compression braces with increasing slenderness. The application of the C_s factor limits the ductility demand on CBFs and therefore pushes the seismic response of capacity designed CBFs towards a reliable overall mechanism.

The inelastic demand on braces of Category 3 CBFs is expected to be minimal. Therefore, the C_s factor may be taken as 1.0 when assessing single-storey Category 3 CBFs. The C_s factor for multi-storey Category 3 CBFs and all Category 1 and Category 2 CBFs should be determined in accordance with the provisions of NZS 3404:1997.

C6.8.4.2 Roof X-bracing

The seismic performance of X-braced roof diaphragms is likely to be better than similar vertical CBFs as the roof sheeting system potentially contributes significantly to the stiffness and strength of such diaphragms, especially in light weight systems. However, quantifying this contribution is not straightforward, particularly when the sheeting system has significant openings such as skylights or translucent sheeting.

An X-braced roof diaphragm that remains close to elastic is likely to fulfil its role more reliably than one that yields (noting that there will be a level of earthquake that will cause actions beyond elastic levels unless the actions are limited by a reliable mechanism in the lateral force resisting system supporting the roof). It should be also recognised that a diaphragm that can reliably yield will likely perform better than one that cannot.

Maximum actions on an X-braced roof diaphragm are principally dependent on the capacity of the lateral force resisting system supporting the roof and the capacity of the connections of the roof bracing system.

They may be taken as one of the following:

- The nominally ductile ($\mu = 1.25$) diaphragm actions determined in accordance with Section C2, in which case $C_s = 1$ should be used.
- If the diaphragm has brace connections capable of yielding the braces in tension at overstrength, diaphragm actions corresponding to a limited ductile response ($\mu = 3$) and $C_s = 1.35$ should be used.
- If a ductile mechanism exists in the lateral force resisting system supporting the roof that would limit the actions in the roof system, the overstrength actions generated by the lateral force resisting system including demand due to out-of-plane response of walls, etc. (if there are any) may be used, provided that the diaphragm is then assumed to be only capable of nominally ductile behaviour.

C6.8.5 Concurrency effects

Columns and their foundations that are part of a two-way seismic-resisting frame should be assessed against concurrent actions as specified in Clause 12.8.4 of NZS 3404:1997.

References

AISC (2012). *Column base connections*, www.aisc.org/content.aspx?id=31382, American Institute of Steel Construction, Chicago, USA.

AS 1163-1973. *Structural steel hollow sections,* Standards Association of Australia, North Sydney, New South Wales, Australia.

AS 1163-1981. *Structural steel hollow sections,* Standards Association of Australia, North Sydney, New South Wales, Australia.

AS 1163-1981 (Amdt 2). *Structural steel hollow sections,* Standards Association of Australia, North Sydney, New South Wales, Australia.

AS 1163-1991. *Structural steel hollow sections*, Standards Australia, Homebush, New South Wales, Australia.

AS 1204-1972. *Structural steels – Ordinary weldable grades,* Standards Association of Australia, North Sydney, New South Wales, Australia.

AS 1204-1980. *Structural steels – Ordinary weldable grades*, Standards Association of Australia, North Sydney, New South Wales, Australia.

AS 1205-1980. *Structural steels - Weather-resistant weldable grades*, Standards Association of Australia, North Sydney, New South Wales, Australia.

AS 1405-1973. *Carbon steel plates of structural quality*, Standards Association of Australia, Sydney, Australia.

AS 1817:1991. *Metallic materials – Vickers hardness test,* Standards Australia, Sydney, Australia.

AS 3678-1990. *Structural steel – Hot-rolled plates, floorplates and slabs*, Standards Association of Australia, North Sydney, New South Wales, Australia.

AS 3679-1990. *Hot-rolled structural steel bars and sections*, Standards Association of Australia, North Sydney, New South Wales, Australia.

AS A1-1928. *Structural steel and Australian Standard rolled steel sections for structural purposes*, Australian Commonwealth Engineering Standards Association, Sydney, Australia.

AS A1-1956. *Structural steel (excluding plates) and Australian Standard rolled steel sections for structural purposes*, Standards Association of Australia, Sydney, Australia.

AS A135-1965. *Notch ductile steel for general structural purposes,* Standards Association of Australia, Sydney, Australia.

AS A149-1965. *Mild steel for general structural purposes,* Standards Association of Australia, Sydney, Australia.

AS A151-1966. *Structural steel of high yield stress (welding quality),* Standards Association of Australia, Sydney, Australia.

AS A157-1966. *Low and intermediate tensile strength carbon steel plates of structural quality,* Standards Association of Australia, Sydney, Australia.

AS A186-1971. *Structural steels ordinary weldable grades,* Standards Association of Australia, Sydney, Australia.

AS A187-1971. *Structural steels (weather-resistant weldable grades)*, Standards Association of Australia, Sydney, Australia.

AS A33-1937. *Carbon steel plates for general structural engineering purposes*, Standards Association of Australia, Sydney, Australia.

AS A33-1955. *Carbon steel plates for general structural engineering purposes*, Standards Association of Australia, Sydney, Australia.

AS/NZS 1163:2009. *Cold-formed structural steel hollow sections*, Standards Australia/Standards New Zealand.

AS/NZS 3678:1996. *Structural steel – Hot-rolled plates, floorplates and slabs,* Standards Australia/Standards New Zealand.

AS/NZS 3678-2011. *Structural steel—Hot-rolled plates, floorplates and slabs*, Standards Australia/Standards New Zealand.

AS/NZS 3679.1:1996. *Hot-rolled bars and sections*, Standards Australia/Standards New Zealand.

AS/NZS 3679.1:2010. *Hot-rolled bars and sections*, Standards Australia/Standards New Zealand.

ASCE 41-13 (2014). *Seismic evaluation of existing buildings,* American Society of Civil Engineers, and Structural Engineering Institute, Reston, Virginia, USA.

ASM International (1986). *Guide to materials engineering data and information,* ASM International, Ohio, USA.

Baker, M.J. (1969). *Variations in the mechanical properties of structural steel - British early 1960s*, IABSE reports of the working commissions. No. 4. 165-174.

Barker, G.F. (2000). *Assessment of existing structures: a strength limit state appraisal of hot driven rivets*, Steel Construction 34(1): 1–15, March 2000.

Bates, W. (1991). Historical Structural Steelwork Handbook. 4th ed. The British Constructional Steelwork Association Limited.

Blodgett, O.W. (1987). *Weld failures: they could be the result of violating simple design principles*, Australia Welding Journal 32(2).

Borzouie, J., MacRae, G.A., Chase, J.G., Rodgers, G.W. and Clifton, G.C. (2016). *Experimental studies on cyclic performance of column base strong axis aligned asymmetric friction connections*, Journal of Structural Engineering 142(1).

Brownlee, S.A. (1994). *Axial force and plate slenderness effects on the inelastic behaviour of structural steel beam-columns*, University of Canterbury, Christchurch, NZ.

Bruneau, M. and Bisson, M. (2000). *Hysteretic behavior of existing and retrofitted concrete-encased riveted stiffened seat angle connections*, Engineering Structures 22(9): 1086-1096.

BS 15:1948. *Structural steel*, British Standards Institution, London, UK.

BS 15:1961. *Mild steel for general structural purposes*, British Standards Institution, London, UK.

BS 4360:1968. *Weldable structural steels*, British Standards Institution, London, UK.

BS 548:1934. *High tensile structural steel for bridges and general building construction*, British Standards Institution, London, UK

BS 968:1943. War time amendment No. 1, *(War emergency standard) High tensile (fusion welding quality) structural steel for bridges and general building purposes,* British Standards Institution, London, UK.

Bussell, M. (1997). *Appraisal of existing iron and steel structures,* The Steel Construction Institute, Ascot, England, SCI Publication 138.

Butcher, G., Andrews, L. and Cleland, G. (1998). *The Edgecumbe earthquake, a review of the 2 March 1987 Eastern Bay of Plenty earthquake*, Centre for Advanced Engineering, University of Canterbury, Christchurch, NZ.

Butterworth, J.W. (1995). *Inelastic rotation capacity of Grade 300 and Grade 350 universal columns*, Department of Civil and Resource Engineering, University of Auckland. School of Engineering Report No. 554.

Clifton, G.C. (1996). *Steel building performance in two recent major earthquakes*, IIW Asian Pacific Welding Congress, Auckland, NZ.

Clifton, G.C. (1997). *Restraint classifications for beam member moment capacity determination to NZS 3404:1997*, HERA, Manukau City, HERA Report R4–92.

Clifton, G.C. (2009). *Lateral restraint of yielding regions in columns and beams in multi-storey buildings*, Steel Construction New Zealand. Steel Advisor MEM2001.

Clifton, G.C. and Cowie, K. (2013). *Seismic design of eccentrically braced frames*, HERA Publication P4001:2013.

Clifton, G.C. and El Sarraf, R. (2011). New Zealand Steelwork Corrosion Coatings Guide. HERA, Manukau City, HERA Report R4–133.

Clifton, G.C. and Ferguson, W.G. (2015). *Determination of the post-earthquake capacity of an eccentrically braced frame seismic resisting system*, The University of Auckland, Report to the Natural Hazards Research Platform.

Erasmus, L.A. (1984). *The mechanical properties of structural steel sections and the relevance of these properties to the capacity design of structures*, Transactions Vol. II, No. 3/CE.

Erasmus, L.A. and Smaill, J.S. (1990). *The mechanical properties of BHP structural sections*, Transactions Vol. 17, No. 1/CE, No. 1/EMCh.

Feeney, M.J. and Clifton, G.C. (2001). *Seismic design procedures for steel structures*, HERA Report R4-76, Manukau City, New Zealand. HERA, 1995. To be read with Clifton, G.C.; *Tips on seismic design of steel structures*, Notes from Presentations to Structural Groups mid-2000; HERA, Manukau City, 2000.

FEMA 273 (1997). *NEHRP guidelines for the seismic rehabilitation of buildings*, Federal Emergency Management Agency, FEMA Report 273, Washington, DC.

FEMA 355E (2000). *State of the art report on past performance of steel moment-frame buildings in earthquakes*, Federal Emergency Management Agency, FEMA Report 355E, Washington, DC.

FEMA 356 (2000). *Prestandard and commentary for the seismic rehabilitation of buildings*, Federal Emergency Management Agency, FEMA Report 356, Washington, DC.

Ferris, H.W. (1954). *Rolled Shapes – Beams and columns – period 1873 to 1952*, American Institute of Steel Construction, New York, USA.

Fidler, H. (1879). *Notes on building construction*, Vol. 3, Rivingtons.

Gupta, A. and Krawinkler, H. (1999). *Seismic demands for performance evaluation of steel moment resisting frame structures,* The John A. Blume Earthquake Engineering Center, Report No. 132.

Hayward, P. and McClintock, A. (1999). *What every engineer should know about welding and inspection*, CBIP/NZWC, HERA, Manukau City.

Hyland, C. (1999). *Structural steelwork connections guide*, HERA, Manukau City, HERA Report R4–100.

Kotwal, S. (2000). *Evolution of Australian Standards for structural steel*, 1st ed. Standards Australia International.

Krawinkler, H. (1995). *Earthquake design and performance of steel structures*, Pacific Conference on Earthquake Engineering.

MacRae, G.A. (1990). *The seismic response of steel frames*, University of Canterbury Research Report 90–6. University of Canterbury, Christchurch, NZ.

Nashid, H., Clifton, G.C., Ferguson, G and Choi. J. (2015). *Relationship between hardness and plastically deformed structural steel elements*, Earthquakes and Structures, Vol. 8, No. 3.

Norton, J.A., King, A.B., Bull, D.K., Chapman, H.E., McVerry, G.H., Larkin, T.J. and Spring, K.C. (1994). *Northridge earthquake reconnaissance report*, Bulletin of the NZSEE, Vol. 27, No. 4.

NZS 1170.5:2004. *Structural design actions, Part 5: Earthquake actions - New Zealand*, NZS 1170.5:2004. Standards New Zealand, Wellington, New Zealand.

NZS 3404 Part 1:1997. *Steel structures standard, incorporating Amendments 1 and 2,* NZS 3404:1997. Standards New Zealand, Wellington, NZ.

NZS 4203:1976. *Code of practice for the general structural design and design loadings for buildings,* NZS 4203:1976. Standards New Zealand, Wellington, NZ.

Park, R., Billings, I.J., Clifton, G.C., Cousins, J., Filiatrault, A., Jennings, D.N., Jones, L.C.P., Perrin, N.D., Rooney, S.L., Sinclair, J., Spurr, D.D., Tanaka, H. and Walker, G. (1995). *The Hyogo-ken Nanbu earthquake (the Great Hanshin earthquake) of 17 January 1995,* Report of the NZSEE Reconnaissance Team. Bulletin of the New Zealand National Society for Earthquake Engineering Vol. 28(1): 1–98.

Roeder, C.W., Knechtel, B., Thomas, E., Vaneaton, A., Leon, R.T. and Preece, F.R. (1996). *Seismic behaviour of older steel structures,* Journal of Structural Engineering, Vol. 122, No. 4, April 1996, 365-373.

Roeder, C.W., Leon, R.T. and Preece, F.R. (1994). *Strength, stiffness and ductility of older steel structures under seismic loading*, University of Washington Department of Civil Engineering, Washington, USA, Report No. SGEM 94–4.

Rondal, J. and Rasmussen, K. (2003). *On the strength of cast iron columns,* University of Sydney, Research Report No. R829.

SB4.6 (2007). *Improved assessment methods for static and fatigue resistance of old steel railway bridges,* Background document D4.6 to *Guideline for load and resistance assessment of railway bridges*. Prepared by Sustainable Bridges - a project within EU FP6.

SCI (1995). *Joints in steel construction – Moment connections,* Steel Construction Institute, Ascot, U.K., Publication No. 207/95.

Storie, L.B., Pender, M.J., Clifton, G.C. and Wotherspoon, L. (2014). *Soil-foundation-structure-interaction for buildings on shallow foundations in the Christchurch earthquake,* 10th US National Conference on Earthquake Engineering, Anchorage, Alaska, NCEE.

Trahair, N.S., Bradford, M.A., Nethercot, D.A. and Gardner, L. (2008). *The behaviour and design of steel structures to EC3*, 4th ed. Taylor & Francis. Abingdon, Oxon, UK.

Tremblay, R. and Robert, N. (2000). *Seismic design of low- and medium-rise chevron braced steel frames,* Canadian Journal of Civil Engineering. 27: 1192–1206.

Wood, P.J. (1987). *Cyclic testing of a concrete encased riveted beam column joint*, Ministry of Works and Development, Wellington, New Zealand, Report Number 5–82/2.

Appendix C6A: Typical Pre-1976 Steel Building Systems Used in New Zealand

C6A.1 General

This section gives general guidance on the typical pre-1976 steel building systems used in New Zealand.

Note:

This information is based on published material and details supplied by design engineers.

C6A.2 Moment Resisting Frames

C6A.2.1 Beams

Beams were typically rolled steel joist (RSJ) sections. These are I-sections where the inside face of the flanges is not parallel to the outside face, being at a slope of around 15%. This makes the flanges thicker at the root radius than at the tips.

The flange slenderness ratios of RSJ sections are always compact when assessed to NZS 3404:1997.

These beams were typically encased in concrete for fire resistance and appearance. This concrete contained nominal reinforcement made of plain round bars or, sometimes, chicken wire.

C6A.2.2 Columns

Columns formed from hot-rolled sections

These columns were either RSJs used as columns or box columns formed by connecting two channels, toes out, with a plate to each flange. The columns were encased in lightly reinforced concrete containing nominal reinforcement made of plain round bars.

Compound box columns

These columns were also formed from plates, joined by riveted or bolted angles into a box section and encased in concrete. Examples of this type of construction are shown in Figures C6A.1 and C6A.2.

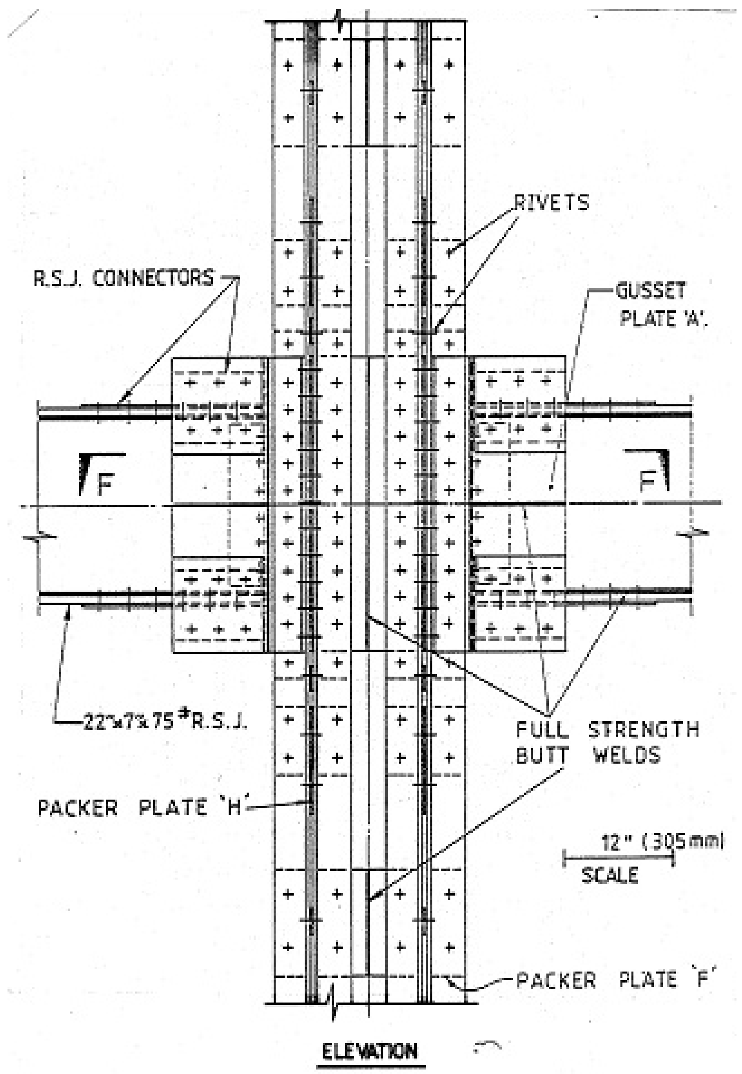


Figure C6A.1: Riveted steel fabrication details, Government Life Insurance Building, 1937 (Wood, 1987)

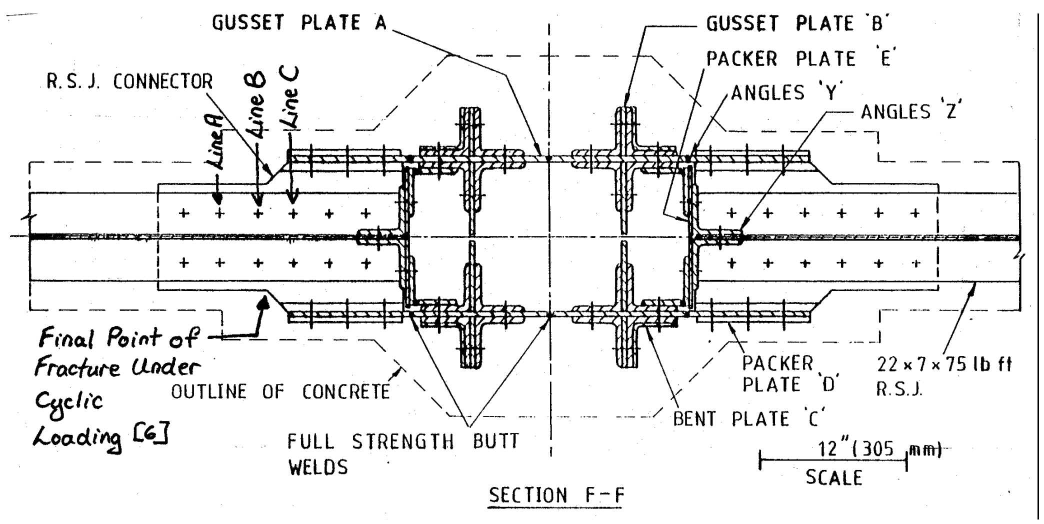


Figure C6A.2: Riveted steel fabrication details, Government Life Insurance Building, 1937 (Wood, 1987)

C6A.2.3 Beam to column connections

Rivets and bolts

Beam to column connections in the earlier moment frames typically comprised semi-rigid riveted or bolted connections. The RSJ beam flanges were bolted to Tee-stubs or angles bolted to the column flanges or to lengths of RSJ bolted to side extensions of the column plates. An example of the latter is shown in Figure C6A.2.

The RSJ beam web was connected by a double clip angle connection to the column flanges, also as shown in Figure C6A.2.

A simpler version of a semi-rigid connection used in some pre-1976 buildings is shown in Figure C6.12.

These joints generally involved the use of rivets up to 1950 and HSFG bolts after 1960, with a changeover from rivets to bolts from 1950 to 1960.

Arc welding

Beam to column connections from about 1940 onwards were also arc welded.

The strength and ductility available from welded connections will need careful evaluation and attention to load path. This topic is addressed in Section C6.6.1 and its importance is illustrated in Figure C6A.3. This figure is taken from a building that collapsed in the Kobe earthquake of 1995 (while this example is from Japan, the details are relevant to some early New Zealand buildings and the concept is certainly relevant). It shows a failed beam to column minor axis connection, forming part of a moment resisting frame in that direction. The beam was welded to an endplate which was fillet welded to the column flange tips.

Unlike the connection detail shown in Figure C6A.2, there was no way to transfer the concentrated axial force in the beam flanges induced by seismic moment reliably from the beam into the column. As a result, the weld between endplate and column flange unzipped under the earthquake action.

Figure C6A.3: Failed beam to column weak axis welded connection from the 1995 Kobe earthquake

Figure C6A.4: Braced frame with light tension bracing showing damage but no collapse from the 1995 Kobe earthquake

C6A.2.4 Splices in columns

These typically involved riveted (pre-1950) or bolted (post-1950) steel sections, with the rivets or bolts transferring tension across the splice and compression being transferred by direct bearing.

Figures C6A.1 and C6A.2 show plated box columns connected by riveted angles. Figure C6A.3 shows a bolted UC splice detail in the column, a forerunner of the bolted column splice details of HERA Report R4-100 (Hyland, 1999). Such bolted splices generally perform well.

C6A.3 Braced Frames

For the pre-1976 buildings covered by this document, braced frames incorporating steel bracing involved concentrically braced framing (CBF): either X-braced CBFs or V-braced CBFs.

Figure C6A.4 shows an X-braced CBF with relatively light bracing, while Figure C6A.5 shows a V-braced CBF. While both examples are from Kobe, Japan they have similar details to early New Zealand buildings.

Figure C6A.5: V-braced CBF showing damage but no collapse from the 1995 Kobe earthquake

Appendix C6B: Historical Steel Grades and Characteristic/Nominal Strengths

C6B.1 United Kingdom

The characteristic (lower)/nominal material properties of historical UK steelwork are given in Tables C6B.1 to C6B.4. Geometric properties of UK sections can be obtained from publications such as that by Bates (1991).

Table C6B.1: Characteristic/nominal properties of mild structural steels from the UK (Bates, 1991 and Bussell, 1997)

Period	Plate thickness (mm)	Yield strength (MPa)	Tensile strength (MPa)	Ultimate strain (mm/mm)
<1906	All	-*	432	-
1906-48*	All	-*	432	0.2
1948-68	≤19	247	432	0.16
	t >19	232	432	0.16

Note:
*A nominal yield strength of 210 MPa may be used for steel manufactured before 1948 in the UK.

Table C6B.2: Characteristic/nominal properties of mild structural steels from the UK manufactured to BS 4360:1968 (1968-86)

Grade	Plate thickness, *t* (mm)	Yield strength (MPa)	Tensile strength (MPa)	Ultimate strain (mm/mm)
40 A, B & C	$t \le 16$	232	402	0.22
	$16 < t \le 38$	224	402	0.22
40 D & E	$t \le 16$	263	402	0.22
	$16 < t \le 38$	247	402	0.22
43 A, B & C	$t \le 16$	247	432	0.20
	$16 < t \le 38$	239	432	0.20
43 D & E	$t \le 16$	278	432	0.20
	$16 < t \le 38$	270	432	0.20

Table C6B.3: Characteristic/nominal properties of high tensile structural steels from the UK (Bussell, 1997 and Bates, 1991)

Period	Plate thickness, *t* (mm)	Yield strength (MPa)	Tensile strength (MPa)	Ultimate strain (mm/mm)
1934-65 (BS 548)	$t \le 32$	355	571	0.14
1943-62	$t \le 19$	324	541	0.14
(BS 968)	$t > 19$	293	510	0.14
1962-68	$t \le 16$	355	494	0.15
(BS 968)	16<t≤32	340	494	0.15

Table C6B.4: Characteristic/nominal properties of high tensile structural steels from the UK manufactured to BS 4360:1968 (1968-86)

Grade	Plate thickness, *t* (mm)	Yield strength (MPa)	Tensile strength (MPa)	Ultimate strain (mm/mm)
50 A, B, C, & D	$t \leq 16$	355	494	0.18
	$16 < t \leq 38$	347	494	0.18
55 C & E	$t \leq 16$	448	556	0.17
	$16 < t \leq 25$	432	556	0.17
	$25 < t \leq 38$	417	556	0.17

C6B.2 Australia

The characteristic (lower)/nominal properties of steelwork provided in Australian standard specifications before the introduction of joint AS/NZ standards in 1996 are given in Tables C6B.5 and C6B.6.

Table C6B.5: Characteristic/nominal strengths of mild structural steels from Australia (Kotwal, 2000)

Period	Grade	Plate thickness, *t* (mm)	Yield strength (MPa)	Tensile strength (MPa)	Standard
1928-56	A1	All	-*	432	AS A1-1928 (Sections)
1928-37	A1	All	-*	432	AS A1-1928 (Plates)
1937-55	D	All	216	432	AS A33-1937 (Plates)
	E	All	193	386	
	F	All	162	324	
1955-65	D	≤19	236	432	AS A33-1955 (Plates)
	D	>19	228	432	
	E	All	193	386	
	F	All	162	324	
1956-65	A1	≤19	236	432	AS A1-1956 (Sections)
		>19	228	432	
1965-71		$t \leq 19$	247	417	AS A149-1965 (Plates & sections)
		$19 < t \leq 38$	232	417	
		$t > 38$	228	417	
1965-71	A	$t \leq 19$	232	394	AS A135-1965 (Notch ductile steel - plates) (Toughness test requirement introduced)
		$t > 19$	220	394	
	B	$t \leq 19$	247	425	
		$t > 19$	236	425	

Period	Grade	Plate thickness, *t* (mm)	Yield strength (MPa)	Tensile strength (MPa)	Standard
1966-71	A151	$t < 16$	355	478	AS A151-1966
		$16 < t \leq 32$	348	478	
		$t > 32$	339	478	
1966-73	20	> 6.4	178	309	AS A157-1966 (Plates)
	24	> 6.4	208	371	
1971-80	250, 250L0	$t \leq 12.5$	262	414	AS A186-1971 & AS A187-1971 & AS 1204-1972 (Sections & flat bars)
		$< 12.5 < t \leq 38$	248	414	
	350, 350L0	$t \leq 12.5$	359	483	
		$12.5 < t \leq 38$	345	483	
	WR350	All	345	483	
1971-80	250	$t \leq 9.5$	276	414	AS A186-1971 & AS A187-1971 & AS 1204-1972 (Plates)
		$9.5 < t \leq 12.5$	262	414	
		$12.5 < t \leq 19$	248	414	
		$19 < t \leq 38$	232	414	
	300	$t \leq 12.5$	310	448	
		$t > 12.5$	296	448	
	350	$t \leq 12.5$	365	483	
		$t > 12.5$	345	483	
	400	$t \leq 12.5$	414	517	
	500	$t \leq 9.5$	483	552	
	WR350	All	345	483	
	WR400	All	414	517	
	WR500	All	483	552	
1973-80	180	>6	180	310	AS 1405-1973 (Plates)
	210	>6	210	370	
1980-90	200	All	200	300	AS 1204-1980 (Sections, flat bars and plates)
	A revision of AS 1204-1972 and AS 1405-1973. Grades 180 and 210 plates were replaced by new grade 200 plates. Grades 300,400 and 500 plates were removed. The rest remained the same as in AS 1204-1972.				
1980-90	WR350	All	345	483	AS 1205-1980 (Sections, flat bars and plates)
1990-96	200	$t \leq 12$	200	300	AS 3678-1990 (Plates)
	250	$t \leq 8$	280	410	
		$8 < t \leq 12$	260	410	

Period	Grade	Plate thickness, *t* (mm)	Yield strength (MPa)	Tensile strength (MPa)	Standard
		$12 < t \leq 50$	250	410	
	300	$t \leq 8$	320	430	
		$8 < t \leq 12$	310	430	
		$12 < t \leq 20$	300	430	
		$12 < t \leq 150$	280	430	
	350	$t \leq 12$	360	450	
		$12 < t \leq 20$	350	450	
		$20 < t \leq 80$	340	450	
	400	$t \leq 12$	400	480	
		$12 < t \leq 20$	380	480	
		$20 < t \leq 50$	360	480	
	WR350	$t \leq 50$	340	450	
1990-96	250*	$t \leq 12$	260	410	AS 3679-1990 (Sections & flat bars)
		$< 12 < t \leq 40$	250	410	
	350*	$t \leq 12$	360	480	
		$t < 12 \leq 40$	340	480	
	WR350	All	340	480	
	BHP Australia replaced most of their Grade 250 sections with new Grade 300 sections in 1994, while BHP New Zealand replaced their Grade 350 sections with new Grade 300 sections				

Note:
*A nominal yield strength of 210 MPa may be used for steel manufactured before 1937 in Australia.

Table C6B.6: Characteristic/nominal strengths of hollow structural steels from Australia

Period	Grade	Yield strength (MPa)	Tensile strength (MPa)	Standard
1973-81	200	210	-	AS 1163-1973
	250	250	-	
	350	360	-	
1981-91	C200 and H200	200	320	AS 1163-1981
	C250 and H250	250	350	
	H350	350	450	
1981-88	C350	350	450	AS 1163-1981
1988-91	C350	350	430	AS 1163-1981 (Amd 2)
1991-09	C250	250	320	AS 1163-1991
	C350	350	430	
	C450	450	500	

C6B.3 Australia/New Zealand

The first joint Australian and New Zealand structural steel specifications were introduced in 1996. Characteristic (lower)/nominal strengths outlined in these joint specifications are given in Tables C6B.7 and C6B.8.

Table C6B.7: Characteristic/nominal strengths of mild structural steels to AS/NZS 3678 and AS/NZS 3679

Period	Grade	Plate thickness (mm)	Yield strength (MPa)	Tensile strength (MPa)	Standard
1996-now	Same as 3678-1990, but a new grade 450 is added.				AS/NZS 3678:1996 AS/NZS 3678:2011
	450	$t \le 20$	450	520	
		$20 < t \le 32$	420	500	
		$32 < t \le 50$	400	500	
1996-2010	250	$t \le 11$	260	410	AS/NZS 3679:1996 (Plates)
		$11 < t \le 40$	250	410	
	300	$t \le 11$	320	440	
		$11 < t \le 17$	300	440	
		$17 < t \le 40$	280	440	
	350	$t \le 11$	360	480	
		$11 < t \le 40$	340	480	
	400	$t \le 17$	400	520	
		$t > 17$	380	520	
2010-now	Grade 250 and 400 sections removed. A new S0 grade introduced. The rest remained the same as AS/NZS 3679:1996.				AS/NZS 3679:2010 (Plates)

Table C6B.8: Nominal strengths of hollow structural steels to AS/NZS 1163:2009

Period	Grade	Yield strength (MPa)	Tensile strength (MPa)	Standard
2009-now	C250	250	320	AS/NZS 1163:2009
	C350	350	430	
	C450	450	500	

C6B.4 USA and Continental Europe

Material and geometric properties of historical continental sections can be obtained from publications such as those by Bates (1991) and SB4.6 (2007).

Structural steelwork imported from the USA before the 1960s is likely to have a lower yield strength than that imported from the UK (refer to Table C6B.9). Geometric properties of US sections can be obtained from publications such as that by Ferris (1954).

Table C6B.9: Characteristic/nominal strengths for steels manufactured in the USA for buildings, based on Ferris (1954) and ASCE 41-13 (2014)

Period	Yield strength (MPa)	Tensile strength (MPa)
<1900	165	248
1901–08	207	414
1909–23	193	379
1924–31	207	379
1932–60	228	417

C6B.5 Steels of Unknown Origin

When the origins of structural steelwork cannot be confirmed, the default nominal strengths in Table C6B.10 should be used.

Table C6B.10: Nominal strengths for structural steels of unknown origin

Time period	Yield strength (MPa)	Tensile strength (MPa)
Pre-1948	210	-
1948–Now	230	-