NZS3404.1 Steel Structures - Materials, Fabrication and Construction

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Date: *21st March 2009*
Ref.: *GEN2002*

Key Words
Standards, bridges, documentation, corrosion, steel types, welding, seismic, heat straightening, WQMS, fracture Critical Member, tolerances, holding down bolts, fatigue, crane beams, AESS, inspection, k-area, bolting

Introduction

New Format of NZS 3404
The steel structures standard is being separated into seven inter-related parts as follows. The first part, NZS3404.1 is due for release mid-2009:

NZS 3404.1 Steel structures – Materials, fabrication, and construction
NZS 3404.2 Steel structures – Structural analysis
NZS 3404.3 Steel structures – General design of members and connections
NZS 3404.4 Steel structures – Design of composite members
NZS 3404.5 Steel structures – Design for fire
NZS 3404.6 Steel structures – Design for fatigue
NZS 3404.7 Steel structures – Design for earthquakes

The commentary section is being brought into the body of the Standard and is located against each relevant clause. This will facilitate interpretation and usability, removing the isolation of the commentary from every day use of the standard.

Highway and Railway Bridge Provisions
New provisions specific to highway and railway bridges are being added throughout the parts, conforming to the requirements of AS5100.6 (SAA, 2004) and BS 5400.5 (BSI, 2005) for highway bridges, and the AREMA guidelines and the AWS D1.5 Bridge Welding Code for railway bridges. This allows a harmonised set of design, fabrication and construction provisions for building, general and bridge structures to be available in a single New Zealand Standard suite.

Benefits
The benefits of the new format for NZS3404 is that specialist sub-committees with the right mix of relevant technical experience can be utilised to develop the drafts of each part. It will be easier to keep the standard up to date in the future as revisions can be made to the relevant parts rather than to the full set. The revision is also deliberately Trans-Tasman friendly, making it easy to draw on design tools and aids currently available and allowing an easy transition into a joint AS/NZS Standard series in the future.

NZS 3404.1 is expected to become the key reference document for specifying and building steel structures that will be used by architects, structural engineers, builders, fabricators, steel erectors, highway and railway agencies, and building control officials. While the remaining parts focus on structural engineering design matters and will be primarily of use to structural engineers, NZS3404.1 will be used by all.

1. Drawings & Specification
Clarification of documentation requirements:

- Corrosion protection and maintenance schedules
- Steel grades
- Seismic frame and member categories
- Fatigue sensitive members

- Weld failure consequence category
- Seismic weld demand category
- Fracture control plan for fabrication of bridges
- Location of fracture critical members in bridges
- Design constraints on construction
- Camber and propping of deck and beams
- Method of concrete screeding of toppings
- Uncoated surfaces
- Welding quality management system requirements

2. Materials

Acceptance of Steels

Evidence of compliance of steel supplied to the relevant material supply standards shall now be by test reports or test certificates prepared by a laboratory accredited by signatories to the International Laboratory Accreditation Cooperation (ILAC) Mutual Recognition Agreement (MRA) on behalf of the manufacturer. This is to allow confidence in the properties stated in the test reports or certificates, which may originate from many different countries and jurisdictions.

Steel Types

- Variation of Properties between Steel Types

  The Steel Type is an important classifier within the steel structures, material supply and welding standards (Table 1). It shows the general relationship of steels manufactured to varying international standards. The steel type numbers have three bands related to the three Grade nominal yield stresses of 1-3 being G250/300, 4-6 being G350 and 7 being G450 MPa. Within each numbering band the numbers increase with the level of Charpy V-Notch (CVN) absorbed energy at various testing temperatures. The first number in each band, (1, 4 and 7A) being steel grades without CVN rating; the second number in each band, (2, 4 and 7B) being grades with 27J@0^{o}C; the third number in the G250/300 and G350 bands, (2S and 5S) being seismic grade steel rated at 70J@0^{o}C; and the last number in each grade band, (3,6 and 7C), achieve 27J@-15 ^{o}C.

Steel type	Steel grade					
	AS 1163 ASTM A106 API 5L	AS 1594	AS/NZS 3678 AS/NZS 1679.2	AS/NZS 3679.1	BS EN 10025	JIS G 3106 JIS G 3136
1	C250 Grade B	200 250 300	200 250 300	250 300	S275 S275JR	SM 400A SN 400A
2	C250L0	–	–	250L0 300L0	S275J0	SM 400B SN 400B
2S	–	–	250S0 300S0	250S0 300S0	–	–
3	–	–	250L15 300L15	250L15 300L15	S275J2G3/ S275J2G4	SM 400C
4	C350	HA350 HA440	350 WR350L0	350	S355 S355JR	SM 490YA
5	C350L0	–	WR350L0	350L0	S355J0	SM 490YB SM 520B

Table 1 Portion of Steel Types Table

- Relationship to Welding Requirements

  The steel type is used to determine welding and thermal cutting procedures and consumable selection in the welding standard AS/NZS 1554. Welding heat input and matching of appropriately Charpy V-Notch rated consumables is related to the CVN absorbed energy of the parent steel. The welding consumable will typically be required to achieve 27J or more at a lower test temperature than the parent steel. The welding heat input is also required to be reduced when welding CVN rated steels to minimise reductions in the notch toughness in eth heat affected zones adjacent to the welds.

Seismic Steel

- S0 grade

  Earthquakes in Japan and USA in the mid-1990's resulted in unexpected fractures in steel structures. Studies have since shown there to be a correlation between the Charpy V-Notch characteristics of the steel and their ability to prevent fast fracture occurring under inelastic cyclic loading.

  In October 2007 Amendment 2 of NZS 3404 introduced the performance requirements for steel used in steel members required to be able to develop cyclic inelastic deformation during the design level earthquake. These requirements are also being incorporated into the material supply standard for hot rolled steel sections, AS/NZS 3679.1 and the welding standard AS/NZS 1554.1. The G300 seismic steel is designated as steel type 2S.

  OneSteel has introduced a 300Plus S0 grade to meet the requirements of Type 2S steel. Type 5S steel is currently not available as a grade.

- Performance requirements of Seismic Steel

  Where seismic steel is required S0 steel may be used or else steel needs to be tested or have mill test certificates showing compliance with the material performance requirements of NZS3404.1 (Table 3). These include. In accordance with normal engineering practice, an allowance of + or – 3% may be allowed as appropriate, in assessing compliance of steel properties based on manufacturer's mill test certificates.

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

Table 2 Steel Types for Seismic Members

- Seismic category 3 members
  Type 2 or 5 steel may be used for category 3 members (Table 2). These steel types are Charpy V-Notch rated to the lower level of 27J@0°C. The other limits on elongation, maximum yield ratio and maximum yield stress applied to steel for category 1 and 2 members are not applied.

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

Table 3 Material Performance Requirements for Category 1 and 2 Seismic Members

- Seismic category 1 and 2 members
  The requirement to use seismic steel is limited to Category 1 and 2 members in seismic resisting frames.

- Elongation
  Minimum total elongation after fracture is 25%. This is a direct measure of the ductility of the steel.

- Maximum yield ratio
  The ratio of yield stress to tensile strength affects the length of plastic hinge that can develop in a section. The longer the plastic hinge zone the greater the rotational ductility that can be developed in the plastically deforming frame. An upper bound is therefore set on the yield ratio

- Maximum yield stress
  The maximum yield stress as noted on the manufacturer's mill certificate shall not be greater by 33% of the allowed grade minimum yield stress. This requirement has remained the same as in the 1997 revision but has been reformatted. The limit affects the material variation component of the over-strength factor used in the seismic design provisions.

- Charpy V-Notch
  Seismic steel is required to be able to absorb a minimum of 70 Joules of energy during Charpy V-notch testing at 0^oC, ($70J@0^oC$). During cyclic strain hardening the CVN transition curve of steel shifts to increased temperatures reducing its ability to prevent propagation of cracks at notch defects.

Steel for Bridges

- Fracture Critical Members

  The definition for fracture critical members (FCM) is drawn from the AREMA guidelines and the AWS D1 Bridge Welding Code. It applies to tension members or components of bending members subject to tensile flexural stresses, the failure of which would be expected to result in collapse of the bridge.

  FCMs are required to be fabricated from Type 2 or 5 steels, which can absorb $27J@0^oC$ during CVN testing.

- Other members

  The other members in bridges may be fabricated from Type 1, 4 or 7 steels which are not rated for CVN absorbed energy

Line-pipe
Grade B ASTM A106 and API 5L line pipe is now recognised as acceptable to use a s a Type 1 steel, suitable for Cat 4 elastically responding members. This reflects the reality of the use of these products in structural applications

Fracture Assessment

- Permissible service temperature approach

  This approach has not changed except that Type 2S and 5S steels have been added in which allows a useful option for low temperature applications down to -15^oC (Table 4).

- Fracture mechanics approach

  Fracture mechanics based assessment methodologies are proposed to be covered in NZS 3404.6.

End of Life Re-use of Steel
There is significant environmental benefit in steel being reused at the end of its life in a structure. What typically limits reliable and effective re-use of steel is a lack of confidence in the mechanical properties of the steel that has been salvaged. To overcome this and to allow full advantage of end of life re-use in environmental assessment of steel structures a protocol has been developed for marking steelwork and keeping material properties records that may be used upon salvage of the steel at the end of the life of the structure:

- The member mark number shall be hard stamped on each section in a designated location not less than 500 mm from an end.

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

<table>
<tr><th rowspan="3">Steel type</th><th colspan="6">Permissible service temperatures (°C)</th></tr>
<tr><th colspan="6">Thickness (mm)</th></tr>
<tr><th colspan="6">0 5 10 20 30 40 50 ≥70</th></tr>
<tr><td>1</td><td>–20</td><td>–10</td><td colspan="3">0</td><td>+5</td></tr>
<tr><td>2</td><td>–30</td><td>–20</td><td colspan="2">–10</td><td colspan="2">0</td></tr>
<tr><td>2S</td><td>–35</td><td>–25</td><td>–15</td><td colspan="3">–5</td></tr>
<tr><td>3</td><td>–40</td><td>–30</td><td>–20</td><td colspan="3">–10</td></tr>
<tr><td>4</td><td>–10</td><td>0</td><td>0</td><td colspan="2">0</td><td>+5</td></tr>
<tr><td>5</td><td>–30</td><td>–20</td><td>–10</td><td colspan="3">0</td></tr>
<tr><td>5S</td><td>–35</td><td>–25</td><td>–15</td><td colspan="3">–5</td></tr>
<tr><td>6</td><td>–40</td><td>–30</td><td>–20</td><td colspan="3">–10</td></tr>
<tr><td>7A</td><td>–10</td><td colspan="5">0</td></tr>
<tr><td>7B</td><td>–30</td><td>–20</td><td>–10</td><td colspan="3">0</td></tr>
</table>

Table 4 Permissible Service Temperatures According to Steel Type and Thickness

3. Fabrication

Fracture Control Plan for Railway Bridges

A fracture control plan (FCP)shall be prepared prior to fabrication identifying how fabrication will be controlled in accordance with the requirements set out in the Appendices. This is based on the AREMA guidelines.

Use of Heat to Straighten, Curve and Cambering

The use of heat to directly distort steel to preferred profiles requires temperatures well above the recrystallisation temperature of steel of around 920 °C. For relatively thin elements of modern steels this can be acceptable. As the steel becomes thicker greater care is required to ensure that the cooling rate of the steel is not excessive. The welding standard AS/NZS 1554.1 may be used to develop a reliable procedure for pre-heating, prior to applying a heat spot, and the necessary post-heating. This is based on the carbon equivalent of the steel and its thickness.

For Fracture Critical Members in bridges this use of heat is not allowed, to minimise the risk of micro-crack development adjacent to the heat spots that could develop into fatigue cracks. The use of heat to assist forming of steel in FCMs is limited to 600°C. This is a sufficient temperature to improve workability of the steel and remove most of the residual stresses that can develop during cold working. However it is also not a high enough temperature to affect the micro-structure characteristics of the steel.

Caution is required when using heat with cold formed sections such as C350 and C450 tubes and hollow sections as the mechanical properties of these sections are assessed after cold forming during their manufacture. A reduction in their mechanical properties may therefore result due to the stress relieving effects of heat applied to 600 °C.

Cutting and Punching of Steel

- Thermal Cutting

  Flame cutting of steel should be undertaken utilising the guidance of the Welding Standard AS/NZS 1554.1 for pre and post-heating.

- Yielding regions

  Punched holes, guillotined or sheared edges are highly cold worked and have residual tensile stresses up to the UTS of the steel and plastic strains up to the material's elongation limit. Therefore cracks can readily initiate and propagate from these edges during cyclic deformation in earthquakes. Therefore shearing, cropping and punching is not allowed in yielding regions of seismic resisting members.

- Bridges

  Thermal cutting induces residual stresses and carbonisation of the edge that can allow fatigue cracks to initiate. Therefore limitations on the surface roughness of the cut are set. Stress relieving of cut edges, at temperatures not greater than 600 ^{o}C , or machining in fatigue sensitive zones can significantly reduce the tensile residual stress levels adjacent to the cut surface and potential tensile stress range of fluctuating loads.

  Shearing, cropping and punching is not allowed on bridge members for the same reasons as in yielding regions. The proviso being that it is permitted if the edge is subsequently dressed 3 mm by machining to remove the affected edge material.

Welding
Welding consumables are to be selected in accordance with AS/NZS 1554.1 to match the steel Type required. For welds to seismic resisting members the heat input is reduced to minimise the reduction of notch toughness of the heat affected zone.

Weld Access Holes
Weld access holes are now required to be formed in such a way as to minimise the development of cracks into the section flanges. Weld access holes in sections with elements thicker than 50 mm ground to bright metal and inspected for cracks by either magnetic particle or dye penetrant methods.

Welding of Continuity Stiffeners in Seismic Members
Corners of continuity stiffener plates placed in the webs of rolled sections with elements greater than 20 mm thick shall be clipped to avoid the k-areas (Figure 1). This is to avoid crack development into the k-area of the hot rolled sections.

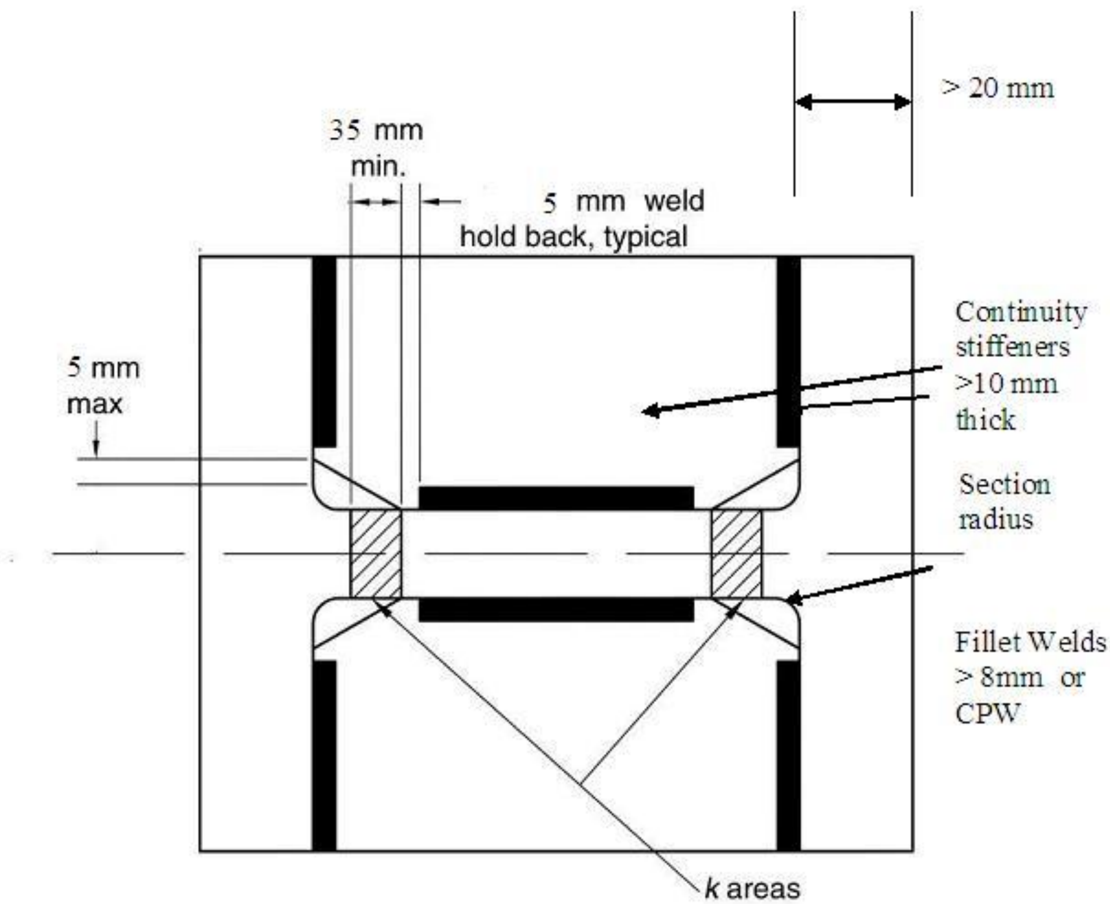


Figure 1 Continuity Stiffeners Detailing Requirements

Welding Quality Management Systems
With the development of the Licensed Building Practitioner (LBP) framework there has been increased focus on quality management of construction. The LBP system is focussed on ensuring companies undertaking construction work have appropriately qualified staff and supervisors executing the designated restricted work. With structural steelwork there is the added dimension of quality management that is applied by the company employing the qualified staff and supervisors.

AS/NZS 1170.0 Importance Level	**3 and 4**	**2**
Elements	**AS/NZS ISO 3834-3**	**AS/NZS ISO 3834-4**
Contract review	Less extensive review	Establish that capability and information is available
Design review	Design for welding to be confirmed	
Subcontractor	Treat like a main fabricator	Shall comply with all requirements
Welders, operators	Approved in accordance with AS/NZS 1554.1	
Welding coordination	Welding coordination personnel with appropriate technical knowledge,	Not required but personal responsibility of manufacturer
Inspection personnel	Sufficient and competent personnel to be available	Sufficient and competent, access for third parties, as needed
Production equipment	Required to prepare, cut, weld, transport, lift, together with safety equipment and protective clothes	No specific requirements
Equipment maintenance	No specific requirements, shall be adequate	No requirements
Production plan	Restricted plan necessary	No requirements
Welding procedure specification (WPS)	Instructions to be available to welder AS/NZS 1554.1	No requirements
Welding procedure approval	In accordance with the appropriate part of AS/NZS 1554.1, approved as application standard or contract demands	No specific requirements
Work instructions	Welding procedure specification (WPS) or dedicated work instructions to be available	No requirements
Documentation	Not specified	No requirements
Batch testing of consumables		
Storage and handling of welding consumables	According to supplier's recommended minimum	
Storage of parent materials	Protection required from influence by the environment; identification shall be maintained	No requirements
Post-weld heat treatment	Conformation to specification necessary	No requirements
Inspection before, during, after welding	As required for specified operations	Responsibilities as specified in contract
Non-conformances	Procedures shall be available	
Calibration	Not specified	
Identification	Required, when necessary	Not specified
Traceability		
Quality records	Shall be available to meet the rules for product liability	As required by contract

Table 5 AS/NZS ISO 3834 Welding Quality Management Systems

To this end recommended levels of welding quality management system have been identified that are relevant to the importance category of the structure. The Importance categories are linked to the definitions in the Loadings Standard AS/NZS 1170.0. They rank the expected impact on human life or critical community functions of the structures after an extreme event such as an earthquake or hurricane.

AS/NZS ISO 3834 is used as the basis for the welding quality management systems (Table 5). This allows international benchmarking of the welding quality management. For Importance Level 1 & Houses < 3 storeys a WMQS is not required. For Importance Level 2 a WQMS to AS/NZS ISO 3834.4 is recommended. For Importance Level 3 & 4 structures a WQMS to AS/NZS 3834.3 is recommended.

Weld Categories for Fatigue Applications
The level for weld quality performance is linked to the fatigue detail classification (Figure 2). This may be SP in accordance with the imperfection levels permitted in AS/NZS 1554.1 or more stringent levels set in AS/NZS 1554.5. In principle the higher the stress range the better the weld required. To make it easier for structural engineers, fabricators and inspectors to identify the level of inspection required for particular connections and fabrication components in structures designed for fatigue, such as bridges, the fabricated fatigue detail diagrams have been brought into NZS 3404.1. Other fatigue details only relevant to the structural engineering design will be provided in NZS 3404.6 Design for Fatigue. Some corrections have been made to the details to be consistent with the current IIW Fatigue Recommendations.

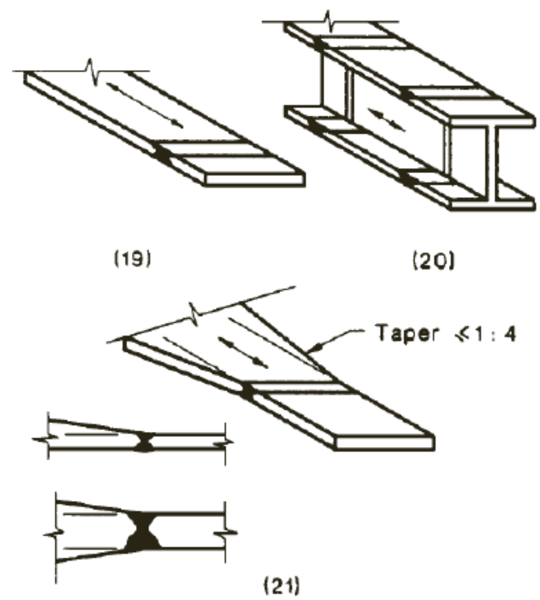
Detail category	Construction details	
	Illustration (see Note)	Description
90		**TRANSVERSE BUTT WELDS (COMPLETE PENETRATION)** Weld run-off tabs to be used, subsequently removed and ends of welds ground flush in the direction of stress. Welds to be made from 2 sides. (19) Transverse splices of plates, rolled sections, or plate girders. (20) Transverse splices of rolled sections or welded plate girders, without cope hole. With cope hole use Detail Category 71, as for (15). (21) Transverse splices in plates or flats being tapered in width or in thickness where the taper is ≤ 1:4.

Figure 2 Sample Fatigue Weld Detail

4. Erection

Erection Method Statement
Erection Method Statements may be prepared by Contractor to address three principal safety issues:

- Stability of works during construction
- Safe positioning and operation of lifting equipment
- Safety of erectors

To assist with its development the Design Engineer shall provide details of assumptions with respect to erection and associated temporary works. This doesn't require the Design Engineer to design or provide loadings for the temporary works, but is rather a statement of designer intent with respect to the construction of the steel structure. AS3828 (SAA,1998) gives further guidance on the development of erection method statements.

Throughout the erection of the structure, the steelwork shall be securely bolted or fastened to ensure that it can adequately withstand all loadings liable to be encountered during erection, including, where necessary, those from erection plant and its operation. Any temporary bracing or temporary restraint shall be left in position until such time as erection is sufficiently advanced as to allow its safe removal.

Bolting

There are no significant changes to the bolting section. The issue of ensuring the correct minimum grip lengths are provided where bolts are to be tensioned, was brought in Amendment 2.

Tolerances

- Differences to NZS 3109 Concrete Construction

  There are some aspects of the tolerances in NZS 3404.1 that differ from those in NZS3109 (SNZ, 1997). This arises at the interface between the construction trades. The steel structures standard has been written with the view that it must identify the tolerances that are required of concrete construction at these interfaces to enable steelwork to be reliably fabricated and installed with minimal disruption and alteration on site. At the same time concrete fixing details that allow adequate adjustment to suit the steelwork are also allowed.

- Holding Down Bolts

  - Fixed bolts
    Steel constructor preference is for holding down bolts that are unable to be adjusted after pouring of concrete are required to be installed with tighter tolerances than previously of +/- 3 mm (Figure 3). This level of setting out requires the use of precision surveying equipment and techniques.

  - Sleeved bolts or adjustable details
    Where polystyrene sleeved cones and other adjustable details are provided then setting out tolerances are relaxed to +/- 10 mm.

  - Precision surveying requirements
    There have been major improvements and reduction in cost of computerised laser surveying equipment in the last ten years. This has led to huge improvements in the accuracy of setting out of holding down bolts that is reasonably achievable. However it is important that the Contractor clearly identifies whether fixed or sleeved holding down bolts will be used to achieve the setting out requirements for holding down bolts. Where fixed bolts are to be cast in then the surveyor needs to be informed of the requirement to use precision surveying techniques.

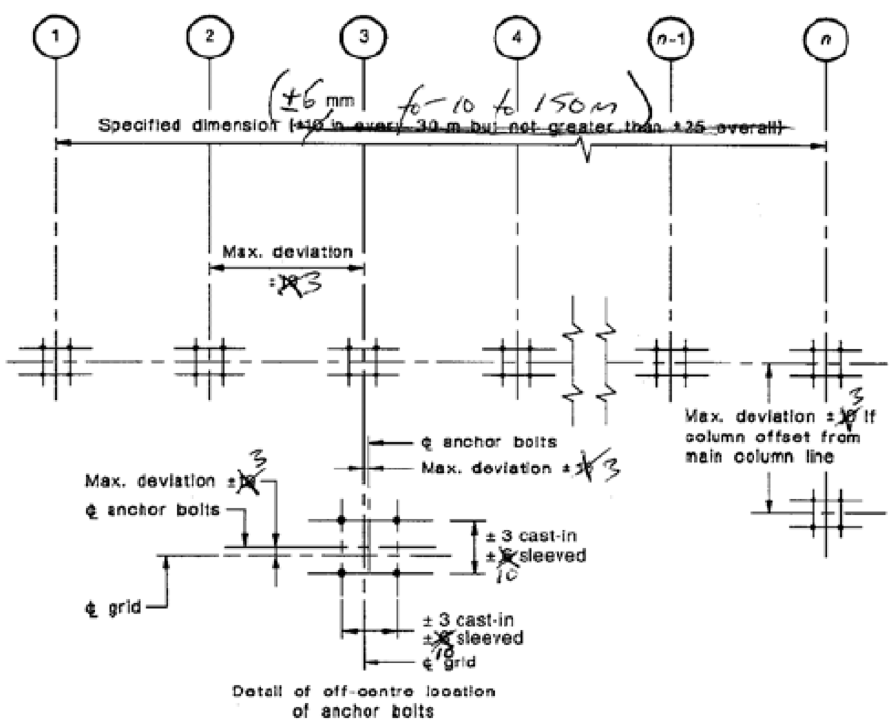


Figure 3 Holding Down Bolt Setting Out Tolerances Preferences

- Connections to Walls

  The same issues apply to cast in bolts and plates in walls (Figure 4, Figure 5 and Figure 6). If the tolerances are not able to be achieved by the concreter then details need to be fabricated into the steelwork connecting to the fixings to allow adequate adjustment. This is somewhat easier to achieve with wall to beam fixings than with column base plates to holding down bolts. The base plates tend to be much thicker and more constrained in terms of adjustability.

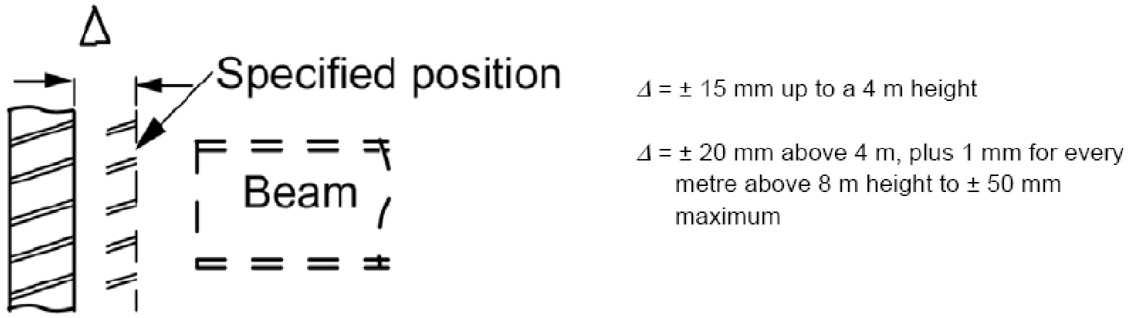


Figure 4 Deviation of Vertical Wall

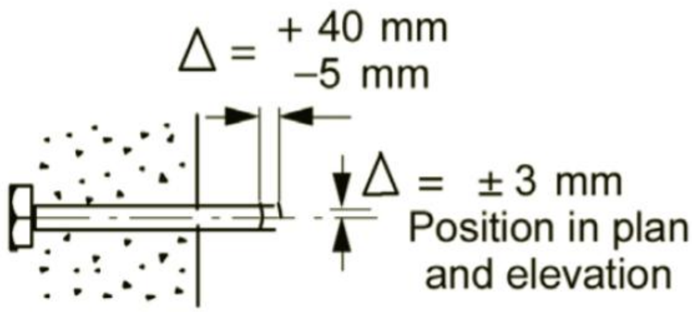


Figure 5 Fixed Position Bolt in Wall

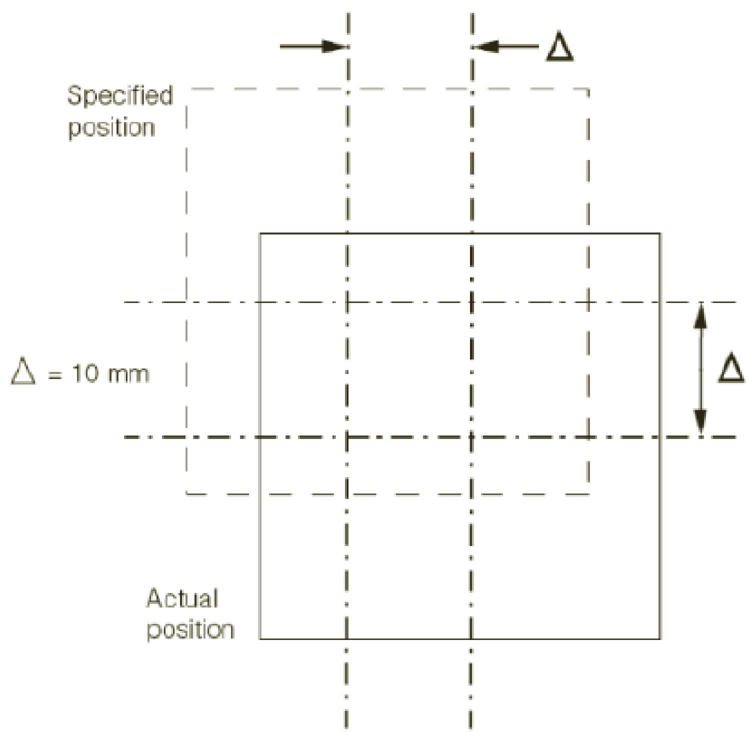


Figure 6 Tolerances on Cast-in Fixing Plates

- Crane runway beams and rails

  A new section has been added to identify permissible deviations for crane rails and runway girders (Table 6).

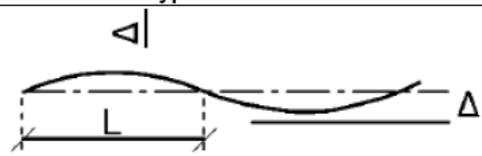
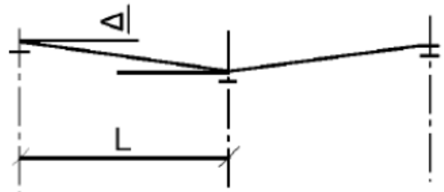
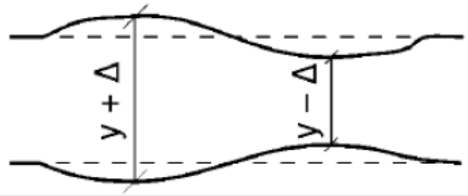
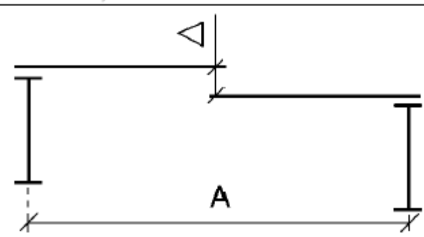
No.	Type of deviation	Description	Permitted deviation
(a)		Local alignment of rail over gauge length (*L*) of 2 m	Horizontally: $\Delta = \pm 1$ mm Vertically: $\Delta = \pm 2$ mm
(b)		Level of rail over length (*L*) between column supports	$\lvert \Delta \rvert$ = greater of: $\begin{bmatrix} A/1000 \\ 10\,\text{mm} \end{bmatrix}$
(c)		Alignments of parallel rails	For y ≤ 15 m: $\Delta = \pm 5$ mm For y > 15 m: $\Delta = \pm 10$ mm
(d)		Difference in level of rails	$\lvert \Delta \rvert$ = greater of: $\begin{bmatrix} A/1000 \\ 10\,\text{mm} \end{bmatrix}$

Table 6 Portion of Permitted Deviations for Crane Rails and Runway Girders

- Combining tolerances

  When it is desired to assess a fabricated assembly for compliance with a number of relevant tolerances the method set out in BS5606:1990 Accuracy in Building Method Section 9 (BSI,1990) is recommended. The method considers that the maximum limits on individual tolerances are highly unlikely to occur in combination. This therefore uses a "square root of the sum of the squares" combination of each relevant deviation (1). While there will on occasion be some assemblies that theoretically could have a higher combined deviation, a reasonable assessment can be made for building purposes as to the typically achievable total deviation that can be expected. An example is shown in Table 7 for a pre-cambered secondary beam. (Cowie, Hyland, 2008)

$$\Delta_{Total} = \sqrt{\Delta_1^2 + \Delta_2^2 + \Delta_i^2} \quad \textbf{(1)}$$

	Variation (mm)
Top of footing	Assumed to be at correct level
Underside of steel base plate	±10
Connection of primary beam to column	±10
Connection of secondary beam to primary beam	±10
Fabricated precamber of secondary beam	+0, -10 i.e. ±5
Total expected variation	$=\pm\sqrt{(10^2+10^2+10^2+5^2)}$ $=\pm 18.0$
Maximum permitted variation in overall height to top of steel	±20

Table 7 Example of Combining of Tolerances for Secondary Floor Beam

5. Corrosion Protection

Maintenance

Normal maintenance in terms of the NZ Building Code B2 Durability allows for recoating of paint systems. The terminology in AS/NZS 2312 unfortunately has caused some confusion as it talks about time to first major maintenance, as being the time to recoating or significant patch repair of a coating system. The wording in NZS3404.1 has therefore been adapted to match the terminology in the NZBC B2 as this is the terminology used by building control officials. Apart from that NZS 3404.1 is fully compatible with AS/NZS 2312 and in fact opens up the applicability of AS/NZS 2312.

5.1.2 Time to First Maintenance

The time to first maintenance of the coating system does not determine the durability of the structure. It is simply the expected time at which significant patch repair or recoating is required to the coating system. If normal maintenance is continued at regular intervals then the durability of the structure is theoretically unlimited. The durability of the structure can be assessed using the method for assessment of steel loss during the design life in combination with the life of the coating systems.

5.2 Protective Coating System Selection

Up until now the limitation has been the assessment of the correct atmospheric corrosivity zone to use when selecting a coating system. Micro-climate effects weren't covered either in any deterministic way. NZS3404.1 draws upon the method used in the HERA Coatings Guide (Clifton, El Sarraf, 2005) to determine surface specific corrosivity zone ratings based on geographic location on macroclimate corrosivity maps, and tabulated adjustment criteria for surface specific conditions.

- Determine Macroclimate Atmospheric Corrosivity Zone

  The first step is to identify the location of the site on the various maps of New Zealand or the localised city maps (Figure 7). The maps show macroclimate corrosivity based on 1st year mild steel corrosion data, correlated to relative humidity, distance from sea shore, temperature and rainfall.

- Macroclimate Corrosion Map Zones

  Seaspray: > 40 m/a
  Zone 1: 25-40 m/a
  Zone 2: 12.5-25 m/a
  Zone 3: < 12.5 m/a
  Zone 4: Taupo Volcanic Zone (TVZ)

- Surface Specific Corrosivity Determination

  The macroclimate zone is converted into a surface specific zone by identifying the particular location of the surface in that environment. For example External, Internal, Exposed, Sheltered, Wet, Dry, Damp, or High Humidity. The resulting categories are directly compatible with AS/NZS 2312 A, B, C, D, E-I, E-M (Table 8).

- Select Corrosion Protection System

  Once the surface specific rating is determined then coating systems may be selected from the range of common systems tabulated in NZS 3404.1. These are a small subset of the greater range in AS/NZS 2312 and use the same system names. The tables in NZS 3404.1 show systems with 15 Years to 1st maintenance and 25+ Years to 1st Maintenance.

  Pre-construction primers and coatings for internal steelwork are also discussed, including guidance on durability of primers while under construction. Surfaces prepared by wire-brushing are typically less durable during construction than those prepared by grit blasting by which the mill scale is fully removed prior to the coating being applied. The construction life is typically assessed as half the expected time to first maintenance of that coating in the exterior environment to which it is exposed during construction, prior to it being closed in permanently. This allows for the primer coating to be in adequate condition for a top coat to be applied without significant remedial work needing to be done to the coated surface.

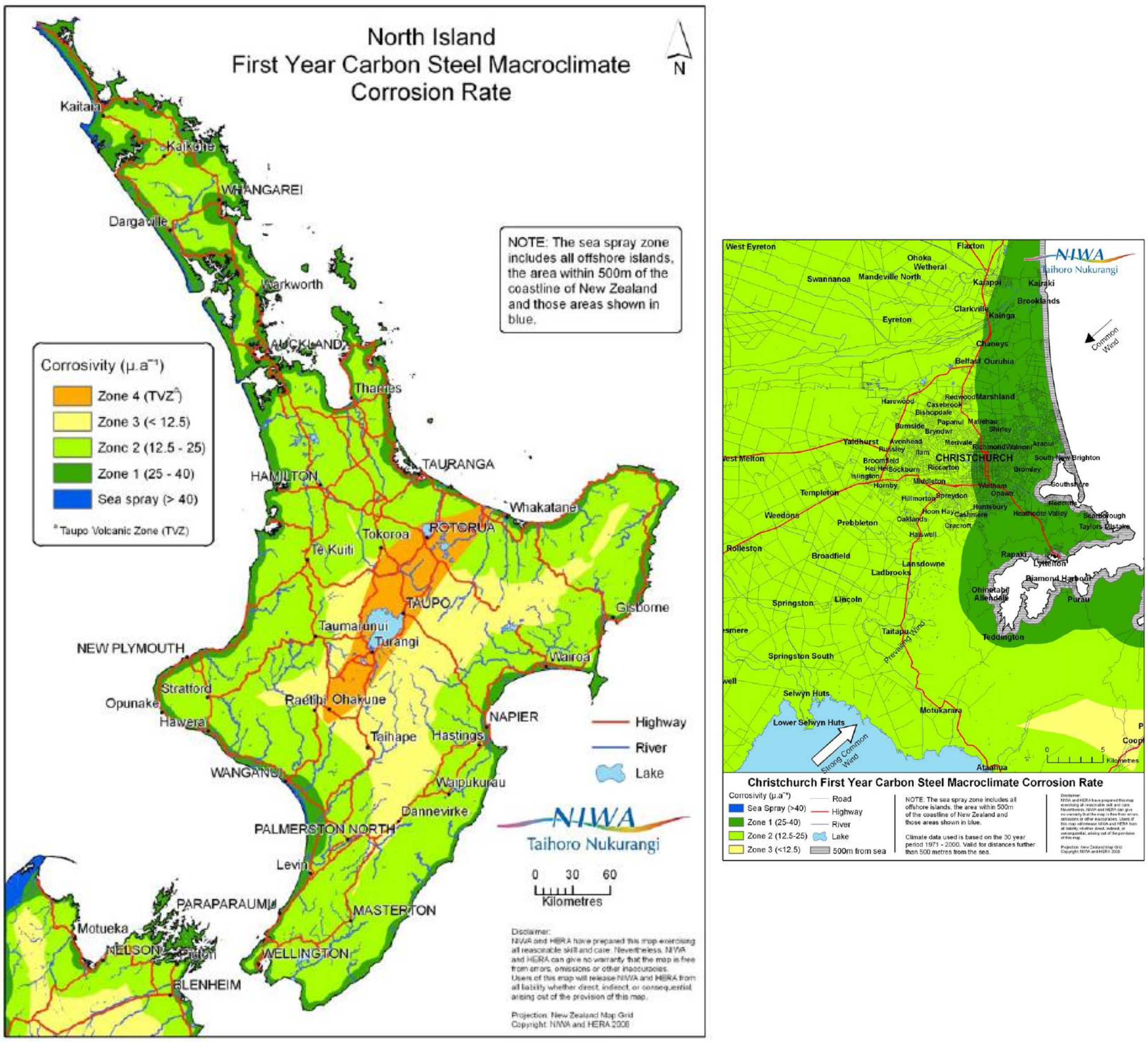


Figure 7 Macroclimate Corrosivity Zone Maps

Corrosion Map Zone (Figures 18 to 23)	Macroclimate corrosion category AS/NZS 2312	Typically	Location	Characterised by	Surface specific corrosion category: External – Exposed	External – Sheltered	External – Wet	Internal – Dry	Internal – Damp	Internal – High humidity
Seaspray	E-M	Within 200 metres from breaking surf on the West Coast of the South Island. Within 100 metres from breaking surf on West Coast of the North Island. Within 50 metres from breaking surf of all other coasts. This environment may be extended inland by prevailing winds and local conditions.	All coasts.	Heavy salt deposits. Almost constant smell of salt spray in the air	E	E	E	A	C	D
	E-M	200 metres up to 500 metres or more inland from breaking surf. In the immediate vicinity of calm salt water such as harbour foreshores. This environment may be extended inland by prevailing winds and local conditions.	West Coast of the South Island.	Heavy salt deposits. Almost constant smell of salt spray in the air	E	E	E	A	C	C
	D	50 metres up to 500 metres or more inland from breaking surf. In the immediate vicinity of calm salt water such as harbour foreshores. This environment may be extended inland by prevailing winds and local conditions.	All coasts except West coast of the South Island.	Medium salt deposits. Frequent smell of salt in the air.	D	E	E	A	B	C
	D	500 metres to 1 km from breaking surf. In the immediate vicinity of calm salt water such as estuaries.	West Coast of the South Island.	Little salt deposits. Occasional smell of salt in the air.	D	E	E	A	B	C
Zone 1	C	500 metres to 1 km from breaking surf. In the immediate vicinity of calm salt water such as estuaries.	All coasts except West coast of the South Island.	Little salt deposits. Occasional smell of salt in the air.	C	E	E	A	B	C
	C	More than 1 km to 20 km from salt water	West Coast of both Islands and South Coast of South Islands	Minor salt deposits. No smell of salt in the air.	C	E	E	A	B	C
		More than 1 km to 5 km from salt water	East Coast both Islands, South Coast of North Island and all harbours		C	D	E	A	B	C
Zone 2	B	More than 20 km to 50 km from salt water	West Coast of both Islands and South Coast of South Islands	No marine influences	B	C	D	A	B	C
		More than 5 km to 50 km from salt water	East Coast both Islands, South Coast of North Island and all harbours		B	C	D	A	B	C
Zone 3	B	Inland, more than 50 km from salt water.	Both Islands		B	B	C	A	B	C
Zone 4	E	Close to the geothermal source < 150 m	Taupo Volcanic Zone	Major geothermal influence	E–I	E	E	A	B	C
	B	Not closer than 150 m to geothermal source		Mild geothermal influence with no marine influences	B	D	D	A	B	C

Table 8 Surface Specific Corrosivity Selection

Coatings for 15 years durability - time to first maintenance, surface specific corrosivity category B

System designation	Type	Surface preparation	Number of coats	Hardness	Typical colour	Initial gloss	Colour and gloss retention on weathering
EPM3	Epoxy mastic	Sa2	2	Very Good	Wide range	Low to semi-gloss	Fine
ACC2	Acrylic	Sa2½				Semi-gloss to full gloss	Very good
PUR2	Polyurethane			Excellent	Mostly grey		Excellent
IZS2SB	Inorganic zinc silicate		1			Flat	Fine

Table 9 Sample Coatings System Selection Table

- Assessment of Steel Loss During Design Life

  The average steel loss, net of coating life may be calculated and the values used to assess structural adequacy at the end of the desired design life (2). The assessment method assumes uniform corrosion loss and excludes localised pitting. As a result the method is limited to systems with 10 mm minimum element thickness.

$$t_{sl} = (T_{DL} - T_{FM})*C_r \qquad \textbf{(2)}$$

where

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

T_{DL} Steelwork design life in years

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

C_r Carbon steel corrosion rate.

Metal	Design corrosion rate (10^{-3} mm/steel surface/year) in various atmospheric corrosivity categories				
	A	B	C	D	E
Carbon steel	0.02	2.2	7.2	22	92

Table 10 50 Year Average Steel Corrosion Rates

Tables list the average steel rates over 50 years of exposure at the most severe end of each surface specific environmental zone for steel in air (Table 10). Average corrosion rates for steel in water or soil are tabulated (Table 11). Typical coating systems for steel piles are also shown.

Water	Below the sea bed	Undisturbed, natural soils.	0.015
	Permanent immersion in sea water	–	0.035
	Permanent immersion in fresh water	For lengths of the pile further than around 300 mm below the surface of the water.	0.025
		For length of pile in the top 300 mm depth of water.	0.050[1]
	Low-water zone corrosion	Bottom of the tidal range where a lack of marine growth occurs but oxygen is quite readily available.	0.075
	Tidal zone	Tidal zones tend to accumulate marine growths, which reduce the supply of oxygen to the steel surface.	0.035
	Splash and marine atmospheric zones	Above the tidal range, subject to wave action and high chloride concentrations. The height of the zone depends on the degree of shelter from wave action.	0.075-0.14[2]
		Above peak wave height and where the pile is sheltered from direct wind flow (e.g. underside of wharfs, above the wharf skirt).	0.035

Table 11 Bare Steel Corrosion Rates in Water

6. Architecturally Exposed Steelwork
Architecturally exposed steelwork requires careful specification. Many different levels of finish may be required in differing circumstances . The level of finish required is strongly affected by the finishing of the fabrication prior to the application fo coating systems. To clarify the specification requirements NZS 3404.1 has defined a range of architecturally exposed structural steelwork categories and the fabrication requirements associated with each (Table 12).

The requirements are based on Canadian and USA developed approaches.

SSS Standard Structural
Workmanship to NZS3404.1Chapter 3

AESS 1 Basic Elements
Enhanced workmanship
20-60% cost premium

AESS 2 Feature Elements Viewed > 6 m
Enhanced treatment of weld, connection and fabrication detail, tolerances for gaps and copes.
40-100% cost premium

AESS 3 Feature Elements Viewed < 6 m
Viewer can see art of metalworking-welds are smooth but visible, some grinding marks; tolerances tighter; subject to touch by public.
60-150% cost premium

AESS 4 Showcase Elements
The form is the only feature showing; all welds ground and filled edges ground square and true; tolerances half of normal; glove smooth surfaces.
100-250% cost premium

AESS C Custom Elements
Special requirements
20-250%+
Additional fabrication, delivery and erection requirements

7. Modification of Existing Structures
No changes from previous versions

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

Table 12 Architecturally Exposed Structural Steelwork Matrix

8. Inspection of Welding and Bolting

Significant improvements have been made to clarify decision criteria for setting a NDE inspection plan and the extent of NDE required after non-compliance. K-areas at the junction of web to flanges in hot rolled sections also require inspection in seismic frames.

A Major	• Steel moment frames with low redundancy (4 or fewer beams per floor resisting lateral forces in each principal direction). • Steel EBF and CBF frames with low redundancy (2 frames in each principle direction). • Any shear joint supporting gravity loads from two or more floors. • Splices resisting applied tension or bending. • Fracture critical members of bridges. • Crane girders S4 and above classification in accordance with AS 1418.1.
B Moderate	• Joints in steel moment frames with redundancy. • Joints in EBF and CBF frames with redundancy. • Joints in steel moment frames with a designed secondary system for lateral loading. • Any shear joint supporting gravity loads from more than one member at a given floor level. • Splices resisting only shear or compression or both. • Non-fracture critical members portions of bridge girders.
C Minor	• Shear, compression and tension joints supporting single members only, and not part of the lateral force-resisting system. • Joints not required to carry gravity loads.

Table 13 Weld and Bolt Failure Consequence Category

Demand	
H High	Welds in which connected member element stresses are expected to be above the yield level, with significant strain hardening development. (Member Categories 1 and 2.)
M Medium	Welds in which connected member element stresses are anticipated to be at or slightly exceed yield level. (Member category 3.)
L Low	Welds in which stresses are anticipated to remain below yield stresses or will remain in compression. (Member category 4.)

Table 14 Seismic Weld Demand Category

Inspection of Welding

Extent of Non-Destructive Examination (NDE)

The process for setting the extent of NDE requires determination of the consequence of weld failure and assessment of inelastic demand on the weld (Table 16). Specific guidance is given for some typical connection configurations, which also act to illustrate how to use the criteria.

- Determine weld failure consequence category (Table 13)
  - A : Major
    This applies to frames with low redundancy, where failure of the weld could lead to collapse; Shear connections supporting 2 or more levels; Tension and bending splices; Fracture Critical Members in bridges; and crane girders S4 and above to AS1418.1
  - B : Moderate

Joints in seismic resisting moment resisting frames (MRF), eccentrically braced frames (EBF), and concentrically braced frames (CBF), where there is considered to be redundancy, such that should failure of the weld occur, then collapse is unlikely to result.

Shear connections supporting more than one member at a floor; Shear and compression splices; and non-fracture critical members in bridges

– C : Minor
  Shear, compression and tension members of single members not part of lateral bracing system. Joints not carrying gravity loads

- Weld Demand Levels (Table 14)
  - Seismic
    High: Significant strain hardening expected

    Medium: Some yielding

    Low: Elastic
  - Non-seismic
    Non-seismic joints are considered to have low weld demand levels.
  - Fatigue
    Scope is tabulated relative to weld failure consequence. Reference is also required to the specific fatigue category of the detail, which in some cases require UT. The permissible defects allowed are also determined by the fatigue detail category.
  - Specific Weld Demands for MRF, EBF and CBF
    Diagrams are provided to show specific application of the seismic weld demand criteria that are related to the location of the weld and the structural ductility category of the frame (Figure 8, Table 16).

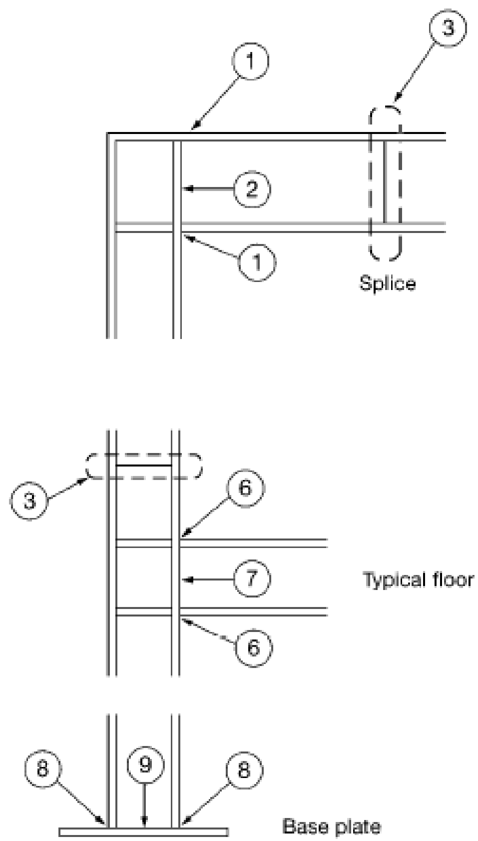


Figure 8 Specific Weld Demand Locations for Moment Resisting Frames MRF

Moment resisting frame weld demand designation

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

NOTE – The figure in the bracket applies if the principal yielding region is located in the beam adjacent to the column.

Table 15 MRF Weld Demand Designations

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

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

Table 16 Extent of NDE Required

– Inspection of k-area adjacent to seismic connections

The potentially work hardened k-areas are required to be checked for cracks within 75 mm of seismic connections

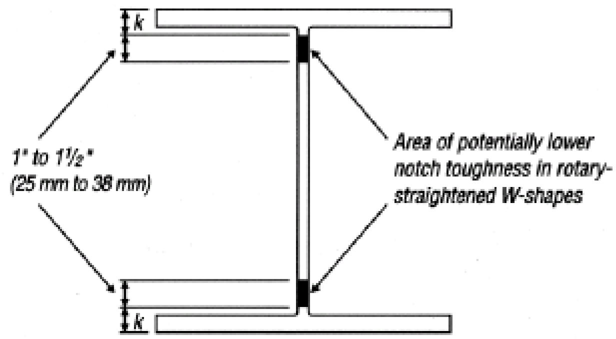


Figure 9 K-areas in Hot Rolled Sections

- Plate thickness limits for Ultrasonic testing at welds

  The requirement for ultrasonic or radiographic testing at welds is dependent on the thickness of the plates welded. Thinner plates in many cases may only require visual inspection. However where visual inspection indicates that a lapse in quality may have occurred then the use of NDE outside the limits shown is not precluded.

- NDE after non-compliance

  The provisions reward regular testing and inspection throughout the fabrication programme when non-compliance is found. If regular testing and welder traceability is documented then after conformity of three consecutive joints the inspection programme may return to the original testing regime. However if there hasn't been a programme of regular testing or traceability of welder to weld is not possible then it is much more difficult to develop confidence in the welding that has been completed prior to the non-compliance. More stringent re-testing requirements are usually required at discretion of Design Engineer.

Inspection of Bolting

Traceability of Bolts

A system of tracing the manufacturer mill test reports to the locations of bolts installed in the structure where fully tensioned /TB or /TF bolts are required in connections assessed to have Major or Moderate consequence upon failure.

Verification of Bolt Tensioning

Tensioning may be verified by direct observation during tensioning or by one of the following methods:

- Part-turn match-marking

  The nut and the bolt shaft are match marked using a chisel after snug tightening, allowing the magnitude of the part-turn applied to be clearly identified at a later time.

- Direct tensioning indication devices

  These are not commonly used. It is important that the butting plys are properly snugged up in contact before tensioning commences, or the connection will not be correctly pre-tensioned even if the bolt is at tension.

- Removal and measurement of sample bolts

  If structural bolts have been correctly tensioned then permanent plastic stretch will have occurred within the grip threads length of the bolt. (Figure 10) This can be measured by removing the bolt and using vernier callipers. A bolt that has been removed and been found to be permanently stretched should not be re-installed and re-tensioned as it may fracture. However if no permanent stretch has occurred then the bolt may be replaced and tensioned. By definition a snug tightened bolt should have suffered no plastic deformation.

- Ultrasonic devices

  Several manufacturers produce equipment specifically for this application. The use of appropriate techniques, which include calibration, can produce a very accurate measurement of the actual pretension. The method involves measurement of the change in bolt length during the release of the nut, combined with either a load calibration of the removed fastener assembly or a theoretical calculation of the force corresponding to the measured elastic release.

The number of bolts to be checked in each connection depends on the construction. A suitable sample would consist of 10 % of the bolts, but not less than 2 bolts at each connection selected at random.

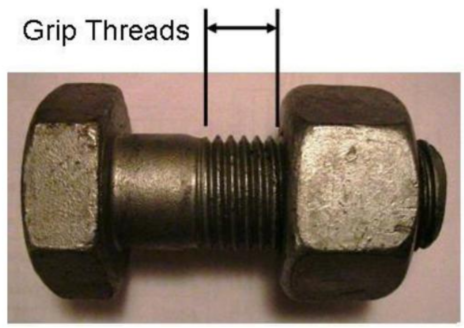


Figure 10 Grip Threads Length

9. Composite Construction

Installation of Stud Shear Connectors
The setting out requirements of shear studs are described, including heights of studs relative to decking profiles (Figure 11).

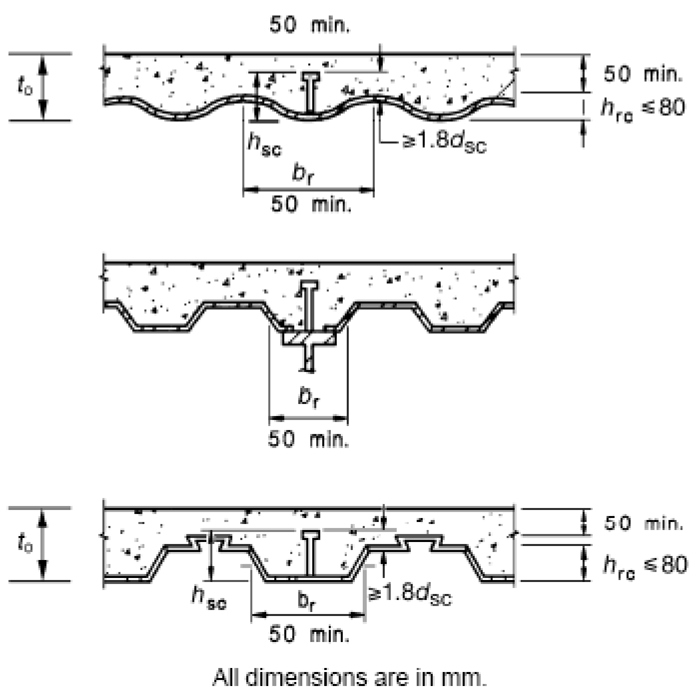


Figure 11 Limitations for Stud Shear Connectors on Profiled Steel decks.

Concrete Encased Steel Columns
The spacing of ties and reinforcing are described.

Conclusion
NZS3404.1 contains significant new material and is formatted in such a way so that all parties involved in steel construction will find it an authoritative source of technical requirements for effective steel construction in New Zealand.

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