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Establishing new brittle fracture provisions for the Australasian steel structures standards

Adolf F. Hobbacher [a], Michail Karpenko [b], Stephen J. Hicks [b,*], Patrick Schneider [a], Brian Uy [c]

[a] Jade University of Applied Sciences, Wilhelmshaven, Germany
[b] Heavy Engineering Research Association, HERA House, PO Box 76-134, Manukau 2241, Auckland, New Zealand
[c] School of Civil Engineering, The University of Sydney, Camperdown, NSW 2006, Australia

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ABSTRACT

This paper develops a new method to select steel grades manufactured to Australian and New Zealand standards. The current materials selection procedure is currently given in the design standards AS 4100, NZS 3404.1 and AS/NZS 5100.6, which is based on test data on the notch toughness characteristics from a previous generation of steel products originally manufactured in Australia or New Zealand. The existing procedure is limited to temperatures down to −40 °C. Moreover, it does not consider the effect of welding, detailing, stress utilisation, seismic loading rates, defects and other important factors. This paper includes a critical review of other international material selection procedures, before preparing a new design method based on fracture mechanics. The method extends the temperature range down to −120 °C, which is much lower than considered in many other international standards. It also includes New Zealand specific requirements for seismic loading rates. In comparison with the new method, it is demonstrated that the current materials selection procedure is much more conservative for plate thickness up to 75 mm for non-seismic design. The paper presents selection tables that can be considered for the development of new brittle fracture provisions for future versions of the Australian and New Zealand steel structures design standards.

1. Introduction

1.1. Nature of brittle fracture

A brittle fracture is defined as fracture with little, or no plastic deformation of the failed component. It can be initiated by an overload of a cross-section in combination with material properties and/or geometrical allocation of the stresses (i.e. triaxiality of stresses due to the structural detail).

The plastic deformation capability is essential for the avoidance of brittle fracture. It can be affected by: hardening in the weld heat affected zone; triaxiality of stress caused by the design of the structural detail; higher strain rates (e.g. during seismic events); or by neutron embrittlement. Another consideration is the possible inhomogeneity of the material, caused by sulphide inclusions leading to reduced mechanical properties in the through-thickness direction of the material. Other considerations include the loading imposed during fabrication and in service, design load, weld shrinkage and alignment at fabrication, erection stresses caused by poor fit-up, stresses by possible displacements of abutments and loadings caused by seismic events.

In the design office, usually only stresses due to design loads are verified by calculation. The other stresses (e.g. residual stresses from shrinkage), are covered by the assurance of a plastic deformation capacity. That assurance is of equal importance as the numerical verification by calculation.

The current provisions for brittle fracture in Australia and New Zealand are given in the steel structures design standard AS 4100 [1] and NZS 3404.1 [2], respectively. The current rules are based on notch toughness test data from a previous generation of steel products originally manufactured in Australia or New Zealand. The requirements are limited to the temperature ranges down to −40 °C. They do not consider the effect of welding, detailing, stress utilisation, seismic loading rates, defects and other important factors. Furthermore, the rules have sometimes caused a technical barrier to the introduction of new grades of steel, in particular, seismic S0 grade structural steel.

Following a review of metallurgical effects and mechanisms of brittle fracture, together with the materials selection methodology given in other international standards, a new design procedure for Australia and New Zealand is developed in this paper. The new procedure results in more competitive designs than the current AS 4100 and NZS 3404.1

* Corresponding author.
E-mail address: stephen.hicks@hera.org.nz (S.J. Hicks).

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for steel thicknesses lower than 40 mm. Conversely, the rules within this paper remove the existing unconservatism that exists for steel thicknesses >40 mm. The procedure has also been specifically developed to satisfy the onerous New Zealand seismic loading requirements.

2. Metallurgical effects and mechanisms of brittle fracture

2.1. Shrinkage

The heating by welding process and the subsequent cooling generates a thermal cycle, which is accompanied by a local thermal expansion and contraction. The thermal expansion coefficient of steel is about $\alpha = 1.2 \cdot 10^{-5} K^{-1}$ (for $T \le 100$ °C). The stress generated by that thermal expansion or contraction is about $\sigma_{th} = E \cdot \alpha \cdot \Delta T$. In other words, a temperature difference of only about $\Delta T = 150\ K$ can produce thermal stresses as high as the yield stress in a steel grade S355 (where $f_y = 355 MPa$). The local temperature differences are significantly higher at the weld locations and so a plastic deformation capacity in the weld metal and heat affected zone is needed to avoid cracks or ruptures.

The plastic deformation capacity is also considered in the design of bolted or riveted structures, but at a lower scale. A bolt hole has a stress concentration factor of $k_t = 3.0$, which is not considered for the static design and verification. At a usage of the material up to $0.75 \cdot f_y$, the fictitious elastic stresses would be higher than yield. However, it is limited due to plastification. At a cyclic load and at fatigue verification, the cyclic plastification would lead to a short fatigue life. It must be considered, and this is what is done in fatigue design codes.

The plastic deformation capacity is firstly used up by shrinkage during welding in the workshop, leaving local residual stresses as high as yield strength in tension. The first load in service, may be the proof load, introduces additional yield at locations of high residual stresses, which consumes a certain amount of plastic deformation capability. The remaining plastic deformation capability may be regarded as a safety against cracks or rupture.

2.2. Residual stress

Besides the residual stresses in the vicinity of the weld (the so called short range residual stresses), there are also additional stresses from assembly of the component, be it by additional welding apart from the weld under consideration or be it by a forced assembly due to poor fit-up. These are the so called long range residual stresses. Other sources of residual stresses are also possible (e.g. by a displacement of the abutments to a bridge).

2.3. Effect of triaxiality

The allocation of stress components is essential for welded joints, which can be visualised in Fig. 1. For a uniaxial load, the Poisson contraction is free in two dimensions, with the plastic deformation hardly obstructed. At a two-dimensional loading, the Poisson contraction can only be effective in one direction, which is in the thickness direction of the plates. Finally, for a three-dimensional load of equal magnitude, plastic deformation is not possible. That is a fundamental conflict with the requirement for a welded joint. At that condition, a brittle fracture cannot be avoided, even in very ductile materials.

The plastic deformation is governed by the maximum shear stress, which is present at the point of deformation. That shear stress can be estimated from the principle stresses by the Tresca criterion $\tau_{max} = (\sigma_1 + \sigma_2)/2$, where σ_1 and σ_3 are the maximum and minimum principle stress, respectively. In most cases of triaxiality, there is a difference in principle stresses, so that a shear stress can be developed which is necessary for a plastic deformation, but that will occur at a higher level of direct stress. That higher level of stress impedes the resistance against weld imperfection, which may be easily seen at fracture mechanics considerations.

The degree of triaxiality is described by the triaxiality ratio TR given in Eq. (1). Here the stress tensor is split into a hydrostatic and a deviatoric part. The hydrostatic part is algebraic mean of the three stress components, and the deviatoric part is the von Mises stress of all components. The quotient of both is the triaxiality ratio. Only the deviatoric part can produce shear according to the Tresca criterion. It can be seen in Fig. 2 that an increase in triaxiality ratio is associated with a pronounced decline of plastic deformation capacity.

$$TR = \frac{\frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)}{\frac{1}{\sqrt{2}}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}} \tag{1}$$

where σ_1, σ_2 and σ_3 are the principal stresses.

2.4. Materials

There is a general dependence on the deformation capacity of a material or steel grade, expressed by elongation in uniaxial tensile test and yield stress f_y (or ultimate tensile strength f_u). The dependence is visualised in the stress-strain diagram shown in Fig. 3. As can be seen from Fig. 3, the verification against brittle fracture becomes more important for higher strength steels.

The yield stress is usually determined by a uniaxial tensile test as given in the standards for materials testing. In real components there may be different conditions which affect the yield (e.g. wall thickness, cold forming, notches, triaxiality, or hardening), which is only partially considered in normal design work, since the verification calculations are mainly based on tested yield stress and a rise of yield might be considered as a benefit. For preventing brittle fracture, the situation is opposite, a rise in yield is unfavourable, because all necessary plastic deformations occur at higher direct stresses, if ever.

During a plastic deformation there is a strain hardening effect. It is not typically considered in the verification procedures for static stress and fatigue. In those conditions, especially in fracture mechanics verification of the ligament, some codes recommend the use of a yield stress f_{use}, which is the mean value between uniaxial yield from test and ultimate tensile stress, $f_{use} = (f_y + f_u)/2$. For more detailed analyses, the Ramberg-Osgood equation is often used which relates strain to stress,

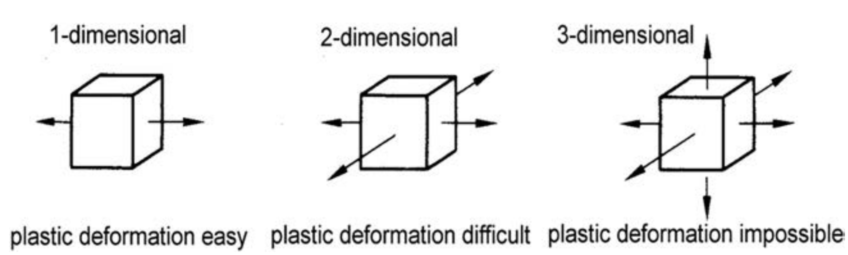


Fig. 1. Allocation of stress.

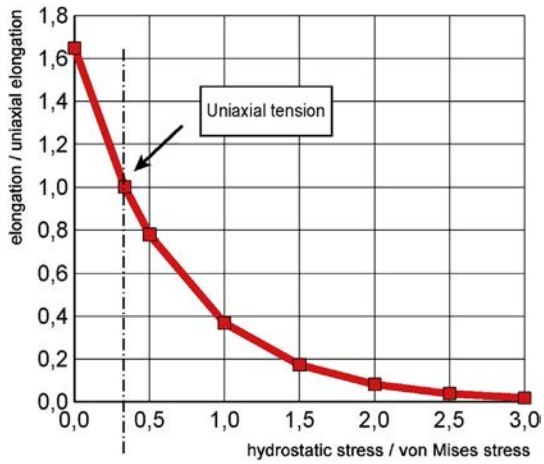


Fig. 2. Effect of triaxiality ratio TR on plastic deformation capacity [50].

as follows:

$$\epsilon = \frac{\sigma}{E} + \left(\frac{\sigma}{K}\right)^{\frac{1}{n}} \quad (2)$$

where K is the strain hardening coefficient and n is the strain hardening exponent.

In seismic engineering, it is conventional to introduce the overstrength factor, which is for most structural details $\gamma_{ov} = 1.25$ [3,4], which includes the increase of load carrying capacity by the transition from elastic to plastic conditions that are present in seismic actions.

2.5. Weld imperfections

Weld imperfections affect the resistance against brittle fracture in multiple ways. There is the stress raising effect of the notch action associated with most imperfections and, in addition, there is the fracture mechanics effect of cracks. Some types of imperfections as cracks, lack of fusion and lack of penetration offer an initial crack or a negligible service time of crack initiation, which may then propagate until final failure. Other stress raising imperfections, as misalignment, undercut, shape imperfections or porosity reduce the service time until a crack initiation. Up until now, no code has established a relationship between brittle fracture avoidance procedures and the type and amount of imperfections. The existing ISO 5817 [5] is more or less an abstract quality assurance system, with only a relation of imperfections to fatigue starting from the 2012 edition. There are fracture mechanics codes and recommendations for the assessment of fatigue at weld imperfections in BS 7910 [6] or API/ASME 751 [7]. Here, the imperfections are considered as cracks, which later are assessed by the methods of fracture mechanics.

The design tables of steel selection for avoidance of brittle fracture of in EN 1993-1-10 [8] are based on an abstract assumption of an initial crack, rather than real imperfections found by NDT. That initial crack is intended to cover all variations of detectability and types of possible imperfections.

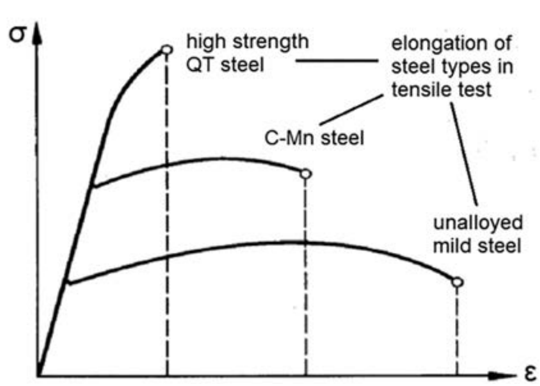


Fig. 3. Steel grades and elongation.

2.6. Fracture toughness

The fracture toughness of a material can be described by the stress intensity factor K (SIF), by the J-integral or by the crack opening displacement (COD). These terms can be converted to each other by established formulae, which depend of triaxiality or on the type of stress allocation as plane strain or plane stress. It is conventional to cover these effects under the term K_{mat}., where K_{mat} is derived from the elastic-plastic J-integral by $K_{mat} = \sqrt{J \cdot E}$. That term is dependent of temperature in a very uniform way at all steels, so a single point at a plot of K_{mat} versus temperature can describe the relationship. For that purpose, the temperature is taken at which K_{mat} takes the value of $K_{mat} =$ 100 MPa $m^{1/2}$. The temperature at this point is T_0. This Wallin [9] correlation is standardized in ASTM E1921–05 [10] (see Fig. 4), and is given by the following equation:

$$K = 20 + 70^{0.019(T-T_0)} \quad (3)$$

where K has the units of $MPa\sqrt{m}$

2.7. Sanz-correlation with Charpy energy

The Charpy energy is dependent on the test temperature [10]. As can be seen from Fig. 5, the dependence of temperature near the lower shelf of the energy (i.e. at 27 J) is almost congruent to the dependence of K_{mat}. This dependence can be expressed by the following simple equation by Sanz [11]:

$$T_{K100} = T_{27J} - 18\,^{\circ}C \quad (4)$$

Charpy-V-notch (CVN) transition temperature is usually taken from an impact energy of 20 ft. lbs. or 27 J. Other testing conditions may be converted by a relation, which was established by Burdekin [12,13], see Table 1. With those relations, all Charpy energies can be converted into K_{mat} with a reasonable accuracy.

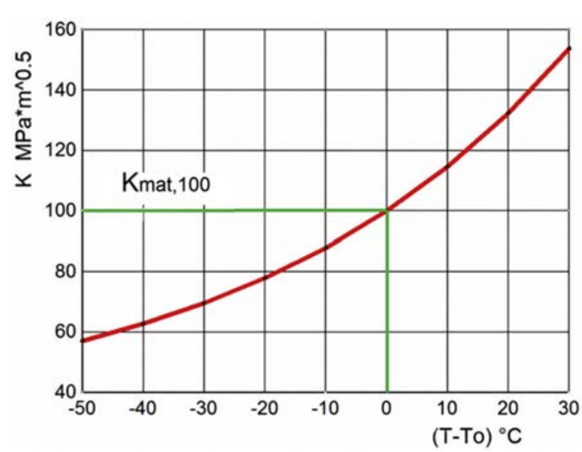


Fig. 4. Standardized dependence of fracture toughness on temperature [9].

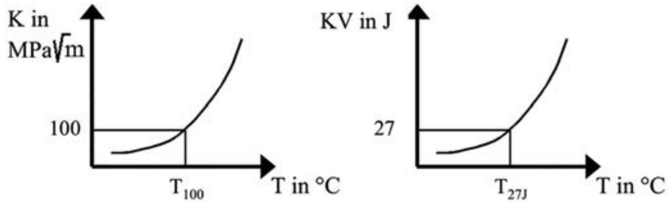


Fig. 5. Correlation between the Wallin master curve and Charpy-V-notch energy.

2.8. The R6 method

The load carrying capacity of a component under static load can be limited by an elastic-plastic overload, or by the release of a brittle fracture due to the onset of rapid crack propagation starting from a weld imperfection. The verification against the first failure mode is undertaken by a comparison of the stress in the component and the tensile resistance of the material, be it yield or ultimate tensile stress. The second verification is performed by the methods of fracture mechanics. For the simultaneous interaction of both failure possibilities, a special method was developed and commonly accepted, namely the R6 method [14]. The implementation of the method is shown graphically in Fig. 6 [8].

The fracture analysis diagram of the R6 method (Fig. 6) relates the normalized tensile loading $L_r = \sigma_{net}/f_y(t)$ and the normalized loading at the crack tip $K_r = K_{appl}/K_{mat}$ by the equation $K_r = (1 + L_r^2/2)^{-0.5}$, where σ_{net} is the tensile stress related to the remaining sectional area, $f_y(t)$ is the yield stress at the wall thickness t and K_{appl} is the stress intensity factor of the loading of the crack tip. Since the formulae for the determination of stress intensity factors have been derived from ideally elastic bodies, a correction for a partial plastification ρ is needed. This correction, together with the Sanz correlation given by Eq. (4), is used to compare the effective loading of the crack tip and the toughness requirement as:

$$K^*_{appl} = \frac{K_{appl}}{K_r - \rho} \leq 20 + \left[70\left(\exp\frac{T_{E,d} - T_{27J} + 18^{\circ}C + \Delta T_R}{52}\right) + 10\right]\left(\frac{25}{b_{eff}}\right)^{\frac{1}{4}} \quad (5)$$

where T_{Ed} is the service temperature, $T_{27\text{J}}$ is the transition temperature of the Charpy test at an energy consumption of 27 J, ΔT_{R} is a safety element, b_{eff} is a dimensional parameter of the crack front length (here the wall thickness is in mm since the reference thickness is 25 mm). For convenience, Eq. (5) may be rearranged for the service temperature T_{Ed} to give the following inequality:

$$T_{Ed} \geq \left(T_{27J} - 18^{\circ}C\right) + 52\ \ln\left(\frac{\left(K^*_{appl,d} - 20\right)\left(\frac{b_{eff}}{25}\right)^{0.25} - 10}{70}\right) + \Delta T_R \quad (6)$$

2.9. Inhomogeneity of material

Metallurgical impurities, such as sulphur oxides, manganese and aluminium oxides, are flattened and spread out by the rolling process of the steel ingots causing the effect of anisotropy of mechanical properties. The reduction of the mechanical properties in the through-thickness direction of the rolled plate creates a proneness to lamellar tearing. An indication to that proneness is the reduction in area in the thickness (Z) direction. The danger of a lamellar tear is dependent on the material property, structural detail, dimensions and heat treatment. The stressing in the *Z*-direction may be due to design or shrinkage caused by welding. A guide for avoidance is given in EN 1993-1-10 [8].

Table 1
Conversion of transition temperatures and CVN energy at 27 J after Burdekin [51].

Charpy impact energy in J	5	10	18	27	41	61	81	100
Temperature difference in CVN in °C	−30	−20	−10	0	+10	+20	+30	+40

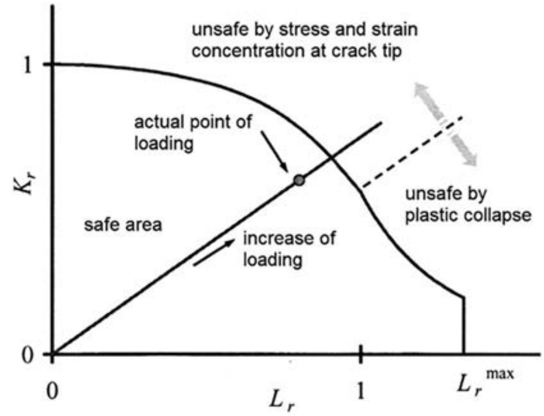


Fig. 6. Fracture analysis diagram.

2.10. Strain rate

Toughness properties of steel are dependent on the applied strain rate. The effect can be described as a shift in transition temperature (see Fig. 7). The shift of the permissible service temperature, resulting from high strain rates can be described by Eq. (7), which is valid for steels with a yield strength from 178 MPa to 890 MPa [15].

$$\Delta T_{\dot{\varepsilon}} = \frac{1140 - f_y(t)}{550} \cdot \left[\ln\left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_0}\right)\right]^{\frac{1}{5}} \quad (7)$$

where $\dot{\varepsilon}$ is the actual applied strain rate and $\dot{\varepsilon}_0$ is the reference strain rate.

3. Current international regulations on brittle fracture

Before the proposed design rules for Australia and New Zealand are developed in the next section, a review of brittle fracture provisions given in different international regulations is given for comparison purposes.

3.1. North American regulations

The LRFD bridge design specifications of the American Association of State Highway and Transportation Officials (AASHTO) were published in 2007 [16]. The main difference (and in most cases, the only difference), between AASHTO and ASME [7] requirements is the inclusion

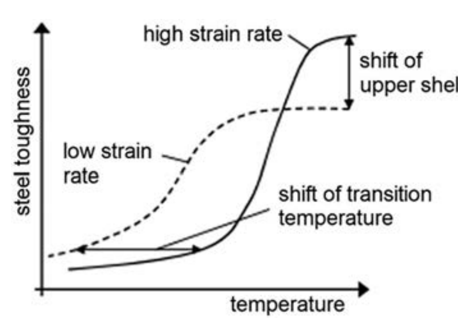


Fig. 7. Effect of strain rate.

of mandatory notch toughness and weldability requirements in the AASHTO Material Standards. These go further than the regulations in API/ASME, where only material parameters and methods are given. Structural steel satisfying the AASHTO Material requirements are considered prequalified for use in welded bridges.

AASHTO highlights the importance of notch toughness and the mechanical and physical properties of rolled steel, such as anisotropy, ductility, formability and corrosion resistance that may also be important to ensure the satisfactory performance of the structure.

Lamellar tearing as a consequence of anisotropy is only generally addressed and described in a special section, but there are no direct regulations to be followed. The yield stress is dependent on loading rate and temperature. The Steel Bridge Design Handbook [17] gives the following equation for estimating those effects.

$$\sigma_{\gamma D} = \left[\sigma_{\gamma S} + \frac{17400}{\log\left(2 \cdot 10^{10} \cdot t\right) \cdot (T + 459)}\right] - 27.4 \quad (8)$$

where σ_{yD} is the yield strength at a given rate and temperature (ksi), σ_{yS} is the room temperature 0.2% offset yield strength at static load rate (ksi), t is the load rise time from start of loading to maximum load in seconds and T is the temperature in °F.

According to ASTM E 399 [18], Eq. (8) is only useful for steels with $\sigma_{yS} \leq 70$ ksi (482.6 MPa) for evaluation of fracture resistance.

Three temperature zones, two importance levels of the component and welded versus un-welded are considered in requirements of toughness (see Table 2 and Table 3). According to the temperature zone, material and wall thickness, the required toughness should satisfy Table 2.

3.2. Seismic actions

For structural steel hot rolled shapes with flanges of 38 mm thick or greater, AISC 341–5 [19] requires tests to ensure that a minimum Charpy V-Notch toughness of 27 J at 21 °C is achieved. Conversely, for plates of 50 mm thick or greater, an identical minimum Charpy V-Notch toughness value is required to be demonstrated by tests when the plate is used in the following applications:

- Members built-up from plate.
- Connection plates where inelastic strain under seismic loading is expected.
- As the steel core of buckling-restrained braces.

No consideration is given to risk classes or wall thickness.

3.3. United Kingdom regulations

In the earlier steel bridge design standard BS 5400–3 [20], very detailed regulations for the selection of steel quality in terms of Charpy-V-notch energy were given in the following equation, using a classification of the structural and loading conditions:

$$k = k_d \cdot k_g \cdot k_o \cdot k_s \quad (9)$$

where k_d depends on structural detail according to notch effect in respect to fatigue (with values from 0.5 to 2.0), k_g takes account of stress concentrations not yet covered in the catalogue of structural details (e.g. in fatigue assessment), k_σ takes account of the stress level, depending of usage 25%, 50% of the yield stress or higher and k_S takes account of loading rate (usually $k_S = 1.0$, but reducing in value for elements at risk to potential traffic impact forces when $k_S = 0.5$).

Table 2
AASHTO temperature zones (converted to °C).

Minimum service temperature °C	−18 and above	−18 to −34	-35 to −51
Temperature zone °C	1	2	3

From the parameters of yield stress f_y and classification k, the largest permitted wall thickness t is given by the following inequality:

$$t \leq 50 \cdot k \cdot \left(\frac{355}{f_y}\right)^{1.4} \cdot 1.2^{\left(\frac{U - T_{27J}}{10}\right)}, \quad but\ U \geq T_{27J} - 20 \quad (10)$$

where U is the lowest service temperature (°C) and T_{27J} is the transition temperature where the Charpy test consumes 27 J of energy.

Table 4 presents the results for grade S355 steel with $k = 1.0$. A safety margin of $\Delta T = 5^\circ C$ must be considered. No special provisions against lamellar tearing are given. However, rules for corrosion in the vicinity of the shoreline are given, which is not the case in most of other codes [20,21].

3.4. Regulations given in the structural Eurocodes

3.4.1. General aspects

The structural Eurocodes [22] provide common rules for everyday use for the design of structures and components. Unusual forms of design or service conditions are not specifically covered, and additional expert considerations may be required. EN 1993-1-10 [8] is entirely based on fracture mechanics and has been calibrated by experiments and service experience [23–26]. An initial crack is assumed at a reference detail, then crack propagation is calculated from assumed loadings and finally the crack is assessed using the R6 method (see Section 2.8). The result is a minimal usable service temperature depending on the yield stress of the steel, the Charpy-V-notch properties and the wall thickness.

3.4.2. Reference detail and initial crack

The basis of this Eurocode is a reference structural detail, which is considered as the worst structural detail in usual bridge design. The reference structural detail that was considered was the end of a extended longitudinal stiffener as shown in Table 5 (the following dimensional parameters of the reference detail related to the wall thickness t were used: $L = 8.2\ t$; $B = 7.5\ t$ and $T = t$).

The choice of the initial crack is dependent on the detection probability of non-destructive testing. In EN 1993-1-10, the initial crack depth a_0 has been assumed to be dependent on the wall thickness t as follows (Table 9).

$$\alpha_0 = \begin{cases} 0.5 \cdot \ \ln(t) & t \geq 15\text{mm} \\ 0.5 \cdot \ \ln(1+t) & t < 15mm \end{cases} \quad (11)$$

The crack parameter c of the semi-elliptical surface crack was determined as $c = 2.5 \cdot a$.

3.4.3. Crack propagation calculation

The fracture mechanics stress intensity factors (SFI) have been calculated from the following:

$$K = \sigma \cdot \sqrt{\pi \cdot a} \cdot Y(a) \cdot M_k(a) \quad (12)$$

where a is the crack parameter, σ is the applied reference stress, Y(a) is the correction for semi-elliptical surface cracks by Newman and Raju and M_k(a) is the correction of the weld detail in consideration (for the end of a longitudinal stiffener, the M_k(a) correction by Hobbacher was used [27]).

The crack propagation itself was determined with the Paris-Erdogan power law:

$$\frac{da}{dN} = C_0 \cdot \Delta K^m \quad (13)$$

Table 3
AASHTO fracture toughness requirements (condensed and in SI units).

Welded or mech. Fast	Grade (Y.P./Y.S.)	Thickness (mm)	Fracture critical			
			Min. test value energy (J)	Zone 1 (J @ °C)	Zone 2 (J @ °C)	Zone 3 (J @ °C)
Welded	36	t ≤ 100	27	34 @ 21	34 @ 4	34 @ 4
	50/50S/50W	t ≤ 50	27	34 @ 21	34 @ 4	34 @ 4
		50 < t ≤ 100	33	41 @ 21	41 @ 4	41 @ -12
	HPS 50 W	t ≤ 100	33	41 @ 21	41 @ -12	41 @ -12
	HPS 70 W	t ≤ 100	38	48 @ -23	48 @ -23	48 @ -23
	100/100 W	t ≤ 75	38	48 @ -1	48 @ -18	48 @ -34
		75 < t ≤ 100	49	48 @ -1	61 @ -18	Not permitted

where N is the number of cycles, C_0 is a material constant and m is an exponent

For C_0 a value of $C_0 = 1.8 \times 10^{-13}$ [28] was chosen, which is the mean value of observed crack propagation in experiments. The mean was taken in order to have only one safety element at the end of an assessment. The exponent m was taken as a uniform exponent for crack propagation and fatigue assessment of welded joints, which was standardized to $m = 3.0$.

The lowest fatigue resistance according to EN 1993-1-9 is detail category 56 (see Table 6). A loading with a nominal net stress range of 56 MPa will lead to 2 million cycles. Four inspections in the lifetime of the component have been assumed to be provided, which gives 500,000 cycles within one inspection interval. At higher stresses, the number of cycles will be lower by fatigue assessment and so, the number of cycles between inspections also will be lower and vice versa. That holds true if four inspections are specified. At other numbers of inspection, different numbers of cycles for crack propagation calculations may be used. In EN 1993-1-9 [29] the crack propagation was calculated at constant amplitude loads as a worst case. At variable amplitude, an effective mean stress may be used, which is determined by the Palmgren-Miner rule. It should be noted that for the subsequent brittle fracture evaluation, the highest stress in spectrum must be applied.

For the reference detail, the assumed initial crack and an inspection interval of $N = 500{,}000$ cycles, the crack propagation can be determined for all relevant wall thicknesses. The results have been fitted in eq. (14).

$$a_d = 2 \cdot 10^{-6} \cdot t^3 + 6 \cdot 10^{-4} \cdot t^2 + 0.1341 \cdot t + 0.6349 \tag{14}$$

There are three different usages of the strength of the material, $0.25 \cdot f_y$, $0.5 \cdot f_y$ and $0.75 \cdot f_y$. An additional stress of 100 MPa has been added to cover possible residual stresses. The nominal yield stress was adjusted to wall thickness, as follows:

$$f_y(t) = f_{y,nominal} - 0.25 \cdot \frac{t}{t_0} \tag{15}$$

where t is the wall thickness and $t_0 = 1\ mm$.

A reference strain rate of $\dot{\varepsilon_0} = 4 \cdot 10^{-4} / \ sec$ was assumed, which covers most design situations. For other strain rates (e.g. at impact loads), a temperature shift may be applied using:

$$\Delta T_i = -\frac{1440 - f_y(t)}{550} \quad \ln\left(\frac{\dot{\varepsilon}}{\dot{\varepsilon_0}}\right)^{1.5} \tag{16}$$

For a material without cold forming $\varepsilon_{cf} = 0\%$ was assumed. For cold forming of non-ageing steels, a temperature adjustment of $\Delta T_{\varepsilon,\, cf} = -3 \cdot \varepsilon_{cf}\, [^{\circ}C]$ should be applied.

From that data and the application of the R6 method (see Section 2.8), the relationship between yield strength, CVN energy, wall thickness and lowest service temperature can be established, which was used to develop the design tables in EN 1993-1-10 [8]. However, if there are expected imperfections or flaws, or larger imperfections that those assumed above, a fracture mechanics analysis may be performed.

3.4.4. Lamellar tearing in Eurocode

EN 1993-1-10 [8] provides detailed rules for the avoidance of lamellar tearing. The required reduction in area of the material, the so-called*Z*-quality is dependent of joint detail, dimensions of weld and wall thickness, restraint conditions and post weld heat treatment. The present authors consider this to be the most elaborate guidance currently available.

3.4.5. Seismic actions

The chosen material and design detail should be capable to provide energy dissipative zones. For that effect, several conditions have to be met according to EN 1998–1 [4]. In static verification for seismic design, an over-strength factor γ_{OV} must be applied and the following condition should be satisfied $f_{y,\,max} \le 1.1 \cdot \gamma_{OV} \cdot f_y$. Additional higher yield strength is not required, because of the scatter of material resistance data towards higher and thus unfavourable values. The yield strength of the material varies considerably and so, the lower bound of the scatter is taken for static verification (which is a 95% probability or the so called characteristic value). At a seismic verification based on plastic deformation and energy dissipation, the upper bond value is governing.

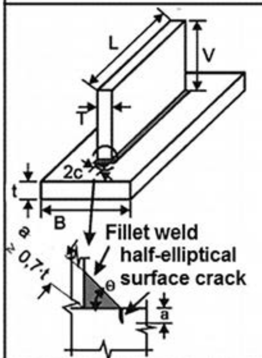
EN 1998-1 [4] provides no specific rules for the required toughness of structural steels in seismic conditions. As a consequence of this, the regulations for toughness of EN 1993-1-10 apply for both statically and fatigue loaded structures.

ECCS (European Convention of Constructional Steelwork) is currently supporting a joint working effort on the development of structural tubular structures with CIDECT, (Comité International pour le Développement et l'Étude de la Construction Tubulaire) which is in close contact with the corresponding groups for the structural Eurocodes (CEN/TC 250) and IIW commission XV-SC-E. The proposed method consists of establishing a basic table as in EN 1993-1-10, but reducing the temperature as low as −120 °C [26]. Starting from that, the effects of strain and cold-forming can be included by consideration of an adequate temperature shift, see Table 7. This temperature shift may

Table 4
The calculative results for $k = 1.0$ (partial and for S355).

Steel grad	Impact quality	Charpy test temp for 27 J	Maximum permitted thickness in mm at minimum service temperature in °C								
			0	−5	−10	−15	−20	−25	−30	−40	−50
S355	J2	−20	72	66	60	55	50	46	42	35	0
	N, K2, M	−30	86	79	72	66	60	55	50	42	35
	N, ML	−50	124	114	104	95	86	79	72	60	50
	N, M	−30	68	62	57	52	47	43	40	33	27

Table 5
Reference detail and fracture mechanics M_k formula.

Detail	Function	Reference
	$M_k = C \cdot \left(\frac{a}{t}\right)^k$ and $M_k \geq 1$	Hobbacher Eng.Fracture Mechanics 1993
Validity range $0.5 \leq \frac{L}{t} \leq 40$ $\quad 0.15 \leq \frac{T}{t} \leq 2$ $2.5 \leq \frac{B}{t} \leq 40$ $\quad 30° \leq \Theta \leq 60°$	$C = 0.9089 - 0.2357 \cdot \frac{T}{t} + 0.0249 \cdot \left(\frac{L}{t}\right) - 0.00038 \cdot \left(\frac{L}{t}\right)^2 + 0.0186 \cdot \frac{B}{t} - 0.1414 \cdot \frac{\Theta}{45°}$ $k = -0.02285 + 0.0167 \cdot \frac{T}{t} - 0.3863 \cdot \frac{\Theta}{45°} + 0.1230 \cdot \left(\frac{\Theta}{45°}\right)^2$	

be subtracted from the required service temperature on the actions side or added to the allowable service temperature on the resistance side.

3.5. Requirements in Australian and New Zealand Standards

3.5.1. Materials selection

The material selection requirements of NZS 3404.1 [30] have been drawn from AS 4100 [1], with some modifications. They have been updated to include 300S0 and 350S0 "seismic" steel grades in NZS 3404.1 [2]. These requirements are also available in a modified form in the bridge design standard AS/NZS 5100.6 [31] and the welding standard AS/NZS 1554.1 [32].

The different steel grades are grouped into steel types based on mechanical properties (see Table 8). The selection requirements for steel types are based on test data available on the notch toughness characteristics (both Charpy and CTOD) of the previous generation of steel products originally manufactured domestically [33–38]. Applicability of these requirements to imported steels is considered to be limited. Therefore, AS/NZS 1554.1 recommends considering verification testing in these cases, which can be restrictive [39].

The material selection requirements do not take into consideration the type of loading, effects from external actions and residual stresses on the member, strain rates, structural detailing and detectable size of defects that may be present in the structure. The lowest one-day mean ambient temperature (LODMAT) isotherms are plotted on maps of Australia and New Zealand in AS 4100, NZS 3404.1 and AS/NZS 5100.6. In New Zealand, the lowest temperature is given as −10 °C in the Southern Alps. However, given that Australia and New Zealand also maintain a presence in the Antarctic, there is a need to develop design rules for temperatures in the region of −60 °C (which is considered in the proposed design rules developed in the next section).

3.5.2. Seismic conditions

New Zealand is a seismic active area, which requires that the structural element categories 1, 2 be designed for sufficient energy dissipation, which is achieved by a plastic deformation capability. For

Table 6
Detail from Eurocode catalogue of structural details.

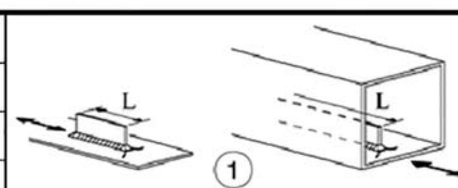
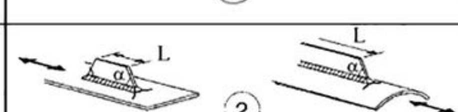
Detail category	Constructional detail		Description
80	L≤50mm		Longitudinal attachments: 1) The detail category varies according to the length of the attachment L.
71	50<L≤80mm		
63	80<L≤100mm		
56	L>100mm		
71	L>100mm α<45°		2) Longitudinal attachments to plate or tube.

Table 7
Rise of lowest permissible service temperature due to cold forming.

No.	Application	Proposal
1	General	$\Delta T[°C] = 3 \cdot \varepsilon[\%]$ but not higher than $\Delta T = 45$ °C
2	Circular hollow sections	$\Delta T[°C] = 3 \cdot \varepsilon[\%]$, ε calculated from d/t
3	Rectangular hollow sections	$\Delta T = 35$ °C for $t \le 16$ mm else $\Delta T = 45$ °C

Note: d is outer diameter of tube and t is wall thickness.

category 1 or 2 members, the permissible service temperature for each steel type is increased by 10 °C as compared with semi-static applications.

3.5.3. Strain hardening

Where a member or component is subject to >1.0% but <10.0% outer bend fibre strain during fabrication, the permissible service temperatures for each steel type is increased by 20 °C as compared with the standard value in NZS 3404.1; in excess of the outer bend strain of 10%, the permissible service temperature is further increased by 1 °C for every 1.0% increase in outer bend fibre strain. However, no modification is required after hot bending in the range from 500 °C to 620 °C.

3.6. Fabrication

The fabrication requirements are included AS/NZS 5131 [40] (similar to EN 1090 [41]), which introduces the fundamental concept of 'construction category' (CC) that is linked to the 'importance level' of the structure and provides the minimum levels of workmanship including welding required to ensure the design assumptions remain valid. High risk seismic structures are expected to be specified as executed Construction Category 3. This standard also includes inspection requirements.

Acceptance criteria for welds in terms of maximum allowable weld imperfections are defined in the AS/NZS 1554. Three weld categories can be specified GP,SP and FP. SP welds are commonly used for structural and seismic applications. SP weld category is comparable, but not identical with the weld quality level B of ISO 5817 [5]. For example, it permits a surface breaking lack of fusion, or incomplete penetration defect, as determined by visual examination or magnetic particle testing. This crack-like defect is not permitted in ISO 5817.

Table 8
Permissible service temperatures according to steel type and thickness.

Steel type	Wall thickness in mm						
	≤6	>6 ≤ 12	>12 ≤ 20	>20 ≤ 25	>25 ≤ 32	>32 ≤ 70	>70
1	−20	−10	0	0	0	0	5
2	−30	−20	−10	−1	−1	0	0
2S(*)	−35	−25	−15	−15	−5	−5	−5
3	−40	−30	−20	−15	−15	−15	−10
4	−10	0	0	0	0	0	5
5	−30	−20	−10	0	0	0	0
5S(*)	−35	−25	−15	−15	−5	−5	−5
6	−40	−30	−20	−15	−15	−15	−10
7A	−10	0	0	0	0	0	---
7B	−30	−20	−10	0	0	0	---
7C	−40	−30	−20	−15	−15	−15	---
8C	−40	−30	–	–	–	–	–

(*): AS/NZS 5100.6 and AS/NZS 1554.1 limit the permissible service temperature for steel type 2S and 5S to 0 °C.

4. Proposal for new material selection design rules in AS 4100, NZS 3404.1 and AS/NZS 5100.6

4.1. Basic assumptions and requirements

As discussed is Section 3.4.2, the reference detail of an end of a long longitudinal stiffener represents practically the worst case in bridge design (see Table 5). One-sided fillet welds without NDT, partial- or no-penetration welds and intermitted welds leading to a fatigue detail category lower than $\Delta\sigma_C = 56\,MPa$ are not permitted. Stress concentration factors such as k_g do not need to be considered (see Section 3.3), because they must be covered in the static analysis of the structure. A crucial point is the issue of residual stresses. The British and European standards consider only the residual stresses caused by the welding of the joint, the so called short range residual stresses. In reality there are also stresses originating from lack-of-fit at erection, or from displacements of abutments in service (e.g. by seismic actions). It seems reasonable to consider that issue and to assume a forced yielding and so a loading up to yield where it should be applied. Strain hardening due to cold forming, effect of strain rate and the possible over-strength effect at seismic loads also needs to be included.

4.2. Capabilities of NDT (POD)

The proneness to brittle fracture is strongly connected with the existence and the size of weld imperfections (in the worst case, with the existence of cracks). From the underlying assumptions given in EN 1993-1-9 (see Table 6), the initial crack depth a_0 and the distance of crack tips $2c_0$ may be calculated as a function of plate thickness, as shown in Table 9. Whilst not directly used in the brittle fracture avoidance procedure, BS 7910 [6] provides some examples of NDT capabilities that should be considered to detect a particular crack size.

In the EN 1993-1-9 procedure, there are different safety aspects inherent. One is the end of a longitudinal stiffener with no smoothening of the transition as a worst case (which is seldom met in existing design). As a consequence of this, a reasonable proposal may be to take the initial cracks as specified in EN 1993-1-9, since a vast experience is available. If other conditions should be assessed, the new parameters could easily be introduced into the procedure and the new material requirements or lowest service temperatures determined.

4.3. Material properties

The main material parameters are yield stress and Charpy-V-notch (CVN) transition temperature at an energy absorption of 27 J. It is assumed that CVN is the appropriate parameter to describe the toughness of the material. Recent experiments with thick walled components show that there may exist a brittle behaviour even though all CVN requirements are met. That could be controlled by COD measurements, or by the outdated weld bead bend test. In micrographs no major difference was found. The difference was the type of normalizing procedure integrated in steel manufacturing process. In several standards, normalizing is accepted at controlled temperatures at the end of rolling, which was shown to be insufficient in thick-walled plates [42]. Plates that have been separately heat treated for normalizing did not show that proneness to brittle fracture. Modern steel production is predominantly done by a continuous casting process. The rolled down product must be <25% of the wall thickness of the ingot.

Table 9
Assumed initial cracks depth a_0 and distance of crack tips $2 \cdot c_0$.

t [mm]	5	10	15	20	30	40	50	70	100
a_0 [mm]	0.9	1.2	1.4	1.5	1.7	1.8	2.0	2.1	2.3
$2c_0$ [mm]	4.5	6.0	7.0	7.5	8.5	9.0	10.0	10.5	11.5

Table 10
Maximum permissible wall thickness in mm at lowest service temperature °C, for $\sigma_{S,\,d} = 0.75 \cdot f_y(t)$

Steel grade and subgrade		Charpy energy		Lowest service temperature [°C]													
				Usage $\sigma_{S,d} = 0.75 \cdot f_y(t)$													
		°C	J	10	0	−10	−20	−30	−40	−50	−60	−70	−80	−90	−100	−110	−120
AS/NZS 3678 and AS/NZS 3679.1 structural steels																	
250	L0	0	27	78	66	55	46	39	33	28	24	21	18	16	14	13	12
	L15	−15	27	101	85	71	60	50	42	36	31	26	23	20	17	15	14
	L20	−20	27	110	92	77	65	55	46	39	34	28	25	21	18	16	15
	L40	−40	27	150	126	108	92	77	65	55	46	39	33	28	24	21	18
300	L0	0	27	70	58	48	40	34	29	24	20	18	16	14	12	11	10
	L15	−15	27	90	76	62	52	44	37	31	26	23	19	17	15	13	11
	L20	−20	27	98	82	68	57	48	41	34	29	24	21	18	16	14	12
	L40	−40	27	136	115	96	82	68	57	48	41	34	29	24	21	18	16
	S0	0	70	106	88	74	62	52	41	36	30	26	22	19	16	14	13
350	L0	0	27	62	50	42	35	30	25	21	18	15	13	11	10	9	8
	L15	−15	27	81	68	56	46	38	32	27	23	19	16	14	12	11	9
	L20	−20	27	90	75	60	50	40	35	30	25	21	18	15	13	12	10
	L40	−40	27	124	100	87	73	61	51	42	35	29	25	21	18	15	13
	S0	0	70	95	80	66	55	46	38	32	27	23	16	16	14	12	11
400	Y20	−20	40	96	80	66	55	45	37	31	26	22	18	15	13	11	10
450	Y40	−40	40	124	102	86	72	59	49	40	33	28	23	19	16	13	11
WR350	L0	0	27	62	50	42	35	30	25	21	18	15	13	11	10	9	8
AS 3597-2008 QT Steels																	
500 QT		−20	80	113	94	77	64	53	44	36	30	24	20	17	14	11	10
600 QT		−20	75	92	76	63	52	42	34	28	23	19	15	13	10	8	7
700 QT		−20	40	60	48	39	32	26	21	17	14	11	9	7	6	5	4
900 QT		−20	40	46	37	29	23	19	15	12	10	8	6	4	3	–	–
1000 QT		−20	40	41	33	26	21	16	13	10	8	6	5	3	–	–	–
AS/NZS 1163 steels for cold formed rectangular and circular hollow sections (un−formed)																	
C250	L0	0	27	78	66	55	46	39	33	28	24	21	18	16	14	13	12
C350	L0	0	27	62	50	42	35	30	25	21	18	15	13	11	10	9	8
C450	L0	0	27	50	41	33	28	23	19	16	13	11	10	8	7	6	5

Table 11
Maximum permissible wall thickness in mm at lowest service temperature, for $\sigma_{S,\,d} = 0.5 \cdot f_y(t)$

Steel grade and subgrade		Charpy energy		Lowest service temperature [°C]													
				Usage $\sigma_{S,d} = 0.50 \cdot f_y(t)$													
		°C	J	10	0	−10	−20	−30	−40	−50	−60	−70	−80	−90	−100	−110	−120
AS/NZS 3678 and AS/NZS 3679.1 structural steels																	
250	L0	0	27	114	98	84	73	63	54	48	42	37	33	30	27	25	23
	L15	−15	27	142	122	104	90	72	68	58	51	45	39	35	31	28	26
	L20	−20	27	152	130	112	97	84	73	63	54	48	42	37	33	30	27
	L40	−40	27	200	175	130	112	112	97	84	73	63	55	48	42	37	33
300	L0	0	27	107	92	78	67	58	50	43	38	33	30	27	24	22	20
	L15	−15	27	134	115	98	84	72	62	54	46	40	35	31	28	25	23
	L20	−20	27	144	124	106	91	78	67	58	50	43	38	33	30	27	24
	L40	−40	27	186	162	142	122	106	91	78	67	58	50	43	38	33	30
	S0	0	70	152	131	112	96	83	71	61	53	46	40	35	31	28	25
350	L0	0	27	100	84	72	61	52	45	39	34	30	26	24	21	19	18
	L15	−15	27	124	106	90	77	66	56	48	42	36	32	25	25	22	20
	L20	−20	27	134	115	98	84	72	61	53	46	39	34	30	26	24	21
	L40	−40	27	178	152	132	114	98	84	72	61	52	45	39	34	30	26
	S0	0	70	142	122	104	89	76	65	56	48	41	36	31	28	25	22
400	Y20	−20	40	146	124	106	90	77	65	56	47	40	35	30	27	23	21
450	Y40	−40	40	182	156	134	114	98	83	70	59	50	43	36	31	27	24
WR350	L0	0	27	100	84	72	61	52	45	39	34	30	26	24	21	19	18
AS 3597-2008 QT Steels																	
500 QT		−20	80	170	145	124	105	89	75	63	53	45	38	32	28	24	20
600 QT		−20	75	143	122	102	86	72	61	51	42	35	30	25	22	19	16
700 QT		−20	40	98	82	68	56	47	39	32	27	23	19	16	14	12	10
900 QT		−20	40	78	64	53	43	36	29	24	20	17	14	11	9	8	7
1000 QT		−20	40	74	58	47	39	32	26	21	17	14	12	10	8	7	6
AS/NZS 1163 steels for cold formed rectangular and circular hollow sections (un−formed)																	
C250	L0	0	27	114	98	84	73	63	54	48	42	37	33	30	27	25	23
C350	L0	0	27	100	84	72	61	52	45	39	34	30	26	24	21	19	18
C450	L0	0	27	85	71	59	50	43	36	31	27	24	21	18	16	15	14

Table 12
Maximum permissible wall thickness in mm at lowest service temperature for cold formed hollow sections.

Steel grade and subgrade		Charpy energy		Lowest service temperature [°C]															
				Usage $\sigma_{S,d} = 0.75 \cdot f_y(t)$								Usage $\sigma_{S,d} = 0.50 \cdot f_y(t)$							
		°C	J	10	0	−10	−20	−30	−40	−50	−60	10	0	−10	−20	−30	−40	−50	−60
AS/NZS 1163 cold formed rectangular hollow sections, welds around the corners, no heat treatment																			
C250	L0	0	27	28	24	21	18	16	14	13	12	48	42	37	33	30	27	25	23
C350	L0	0	27	21	18	15	13	11	10	9	8	39	34	30	26	24	21	19	18
C450	L0	0	27	16	13.5	11.5	9.7	8.4	7.7	6.3	5.5	31	27	24	21	18	16	15	14
AS/NZS 1163 cold formed circular hollow sections, no heat treatment, $d/t \geq 5$																			
C250	L0	0	27	46	39	33	28	24	21	18	16	73	63	54	48	42	37	33	30
C350	L0	0	27	35	30	25	21	18	15	13	11	61	52	45	39	34	30	26	24
C450	L0	0	27	28	23	19	16	13	11	10	8	50	43	36	31	27	24	21	18

Note

1. Steels for rectangular hollow sections with welds in or in vicinity of corners must be resistant against strain ageing. At cold formed hollow sections, a bending strain of 20% was assumed corresponding to $r/t < 2.0$ resulting in a temperature shift of $\Delta T = 60$ °C.
2. At circular hollow sections the bending strain varies according to d/t, so a conservative strain of 10% was assumed for all circular sections resulting in temperature shift of $\Delta T = 30$ °C.

4.3.1. Yield stress

The yield stress is one of the most important parameters in analysis and design. Most of the recent codes require a static verification by the partial safety concept (i.e. there are two safety factors, where one covers the uncertainty of load assumption and the other for variation of resistance values). In many cases, the strength of the material is only partially used and so, two levels of usage in relation to yield strength are considered, $0.75 \cdot f_y$ and $0.5 \cdot f_y$. The steel grades given in the following Australasian product standards were considered in the development of the proposed design tables: AS/NZS 1163 [43], AS/NZS 3678 [44], AS/NZS 3679.1 [45], AS/NZS 3679.2 [46] and AS 3597:2008 [47]. The latter is a standard for quenched and tempered steel up to yield strength of 1000 MPa. That is more than the corresponding European standard BS EN 1993-1-12 [48], which gives regulations up to 700 MPa. The tables have been expanded to 1000 MPa yield by the method described above.

4.3.2. Fatigue

Fatigue is the second important parameter in design. According to AS/NZS 5100.6, the fatigue resistance of an end of a longitudinal stiffener was chosen. The fatigue resistance at 2 million cycles of that detail is $\Delta\sigma_C = 56$ MPa.

4.3.3. Residual stresses

Residual stresses are usually not explicitly considered in practical design. It is relied on by the plastic deformation capability of the material, but in brittle fracture considerations that cannot be neglected. From Section 3.4.3, it was found that EN 1993-1-10 globally adds 100 MPa for that consideration. Given the vast experience of using this value, this assumption is implemented in the absence of specific Australian and New Zealand evidence.

4.3.4. Seismic actions

Seismic actions require a special consideration, since the plastic deformation must be secured. Three levels of verification modes are considered:

- The first level ensures that the nominal (or characteristic) yield stress can be attained without brittle fracture.
- The second level is for minor plastic deformations under seismic loads. In that case, a maximum yield strength of $1.33 \cdot f_y$ was taken (which is

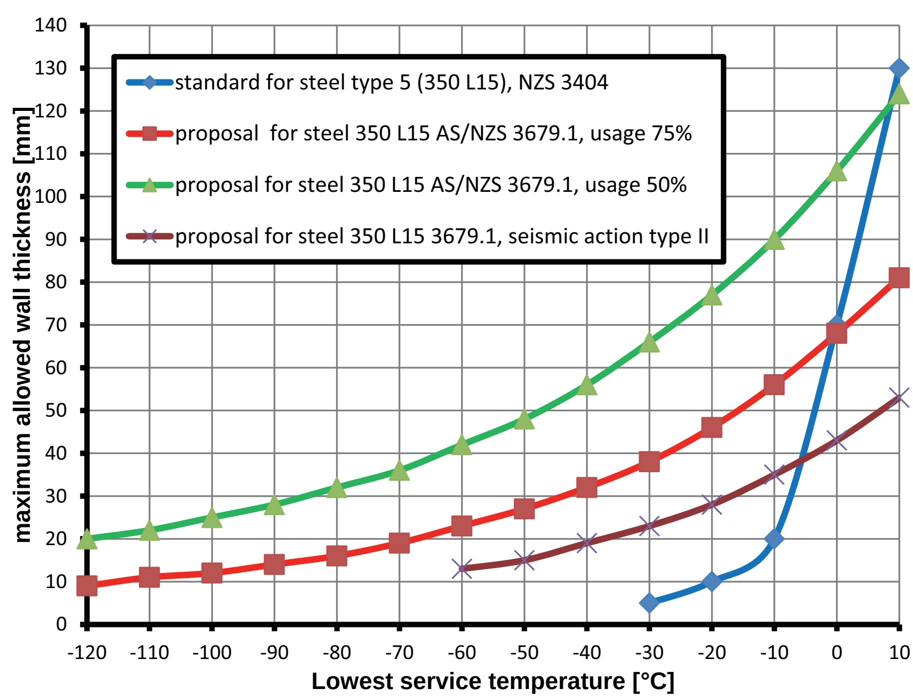


Fig. 8. Lowest permissible service temperature as a function of plate thickness for steel grade AS/NZS 3679.1350 L15 for the proposed selection criteria compared with current NZS 3404.1.

Table 13
Rise of lowest permissible temperature due to a bending radius

Bending radius r over wall thickness t in mm	Maximum plastic strain ε in %	Rise in lowest permissible service temperature	
		Stress relieved after bending	No heat treatment
≥25	≥2	0 °C	0 °C
10 ≤ r/t < 25	≥5	8 °C	15 °C
5.0 ≤ r/t < 10	≥9	14 °C	27 °C
3.0 ≤ r/t < 5.0	≥14	21 °C	42 °C
2.0 ≤ r/t < 3.0	≥20	30 °C	60 °C
1.5 ≤ r/t < 2.0	≥25	38 °C	75 °C
1.0 ≤ r/t < 1.5	≥33	50 °C	100 °C

currently included in NZS 3404.1 in order to cover the scatter of yield strength at the upper bond).

- The third is intended to cover a major deformation under seismic action of $\varepsilon = 10\%$.

4.3.5. Fracture mechanics

Fracture mechanics parameters for the Paris power law are taken from Section 3.4.3 with $C_0 = 1.8 \cdot 10^{-13}$ and $m = 3.00$. Owing to the fact that these are mean values, the safety margin is later achieved by an additional safety element to the lowest permitted service temperature of $T_S = 7^{\circ}C$.

4.3.6. Strain rate

The strain rate has an effect on the yield stress and so it influences the lowest allowable service temperature. From Section 3.4.3, the basic strain rate was taken to be $\dot{\varepsilon}_0 = 4 \cdot 10^{-4}\ s^{-1}$. Considering different strain rates in Eq. (7), it was found that there was no practical difference between the different wall thicknesses and the seismic steel grades 300S0 and 350S0. The existing steel structures design standard NZS 3404.1 specifies a rise in lowest permissible service temperature of 10 °C due to seismic actions. That corresponds to a strain rate of about $\dot{\varepsilon}_0 = 1 \cdot 10^{-2}\ s^{-1}$ (which implies a rise of stress from zero to yield in about 0.2 s). For other measured or specified strain rates Eq. (7) may be used, or the temperature shift may be looked up from Table 13.

4.3.7. Cold forming

Cold forming has two effects on proneness to brittle fracture. It may lead to a strain ageing embrittlement due to a possible precipitation of nitrogen, which could be avoided (e.g. by a free aluminium content of 0.02%). The strain ageing process can start at temperatures of 200–250C, which is usually present in the vicinity of welding. The other effect is caused by strain hardening in the cold formed areas. The latter effect can be reduced by a subsequent stress relieving thermal treatment. A good engineering approach for compensation of these effects is to raise the lowest allowable service temperature by a temperature shift (see Section 3.4.3), which was adopted in the proposed design procedure below.

In the product standard for cold formed rectangular hollow sections AS/NZS 1163 [43], the largest wall thickness is 12.5 mm with the ratio between the outer corner radius and the wall thickness is r/t ≥ 2.00. This ratio corresponds to an elongation at the outer surface of $\epsilon \approx 20\%$. It is important to check this ratio, since it is sensitive to brittle fracture behaviour. At circular hollow cold formed sections there is a variety of combinations of wall thickness and diameter. The highest possible strain is practically confined by $\epsilon \approx 10\%$. To compensate for this effect, the lowest allowable service temperature is raised by a temperature shift in the proposed design procedure below. Table 12 is based on that conservative approach. However, there are discussions and proposals to release that strict requirement according to Table 7.

4.3.8. Proposed tables for AS 4100, NZS 3404.1, AS/NZS 5100.6 and AS 3597

From the above considerations, the tables presented in this section have been specifically developed for Australian and New Zealand steel grades. The worst case of an end of a longitudinal stiffener was consistently used and no consideration of possibly more benign structural details is done. That is because the service conditions and the wall thicknesses are known when ordering the material but not the numerous design details. The acting design temperature should be evaluated using the following equation:

$$T_{Ed} = T_{md} + \Delta T_{\varepsilon,cf} + \Delta T_{\dot{\varepsilon}} + \Delta T_{\varepsilon,cf,EXC} \tag{17}$$

where T_{md} is the basic design temperature from AS 4100, NZS 3404.1, AS/NZS 5100.6 or AS 3597, $\Delta T_{\varepsilon,\,cf}$ is the temperature shift due to cold forming (see Table 13), $\Delta T_{\dot{\varepsilon}}$ is the temperature shift due to seismic strain rates $\dot{\varepsilon} > 4 \cdot 10^{-4}\ s^{-1}$ (see Table 15) and $\Delta T_{\varepsilon,\,cf,\,EXC}$ is the temperature shift due to cold forming during the fabrication/construction process (from Eq. (18), see also Table 15). Additional safety elements may be added according to special requirements. A safety element $\Delta Ts = 7$ °C is proposed according to the Eurocode and is included in the tables of the lowest service temperature. Note that the safety elements lower the acting design temperature, or rise if applied on the resisting (permissible) service temperatures which are given in the tables below.

4.3.8.1. Static and fatigue loads. For both static and fatigue loaded structures, the maximum permissible wall thicknesses at different design

Table 14
Maximum wall thickness at lowest temperature under seismic load at $\dot{\epsilon} = 4 \cdot 10^{-4}\ s^{-1}$

Steel grade	Charpy energy		VM	Lowest service temperature [°C]							
	°C	J		10	0	−10	−20	−30	−40	−50	−60
300 S0	0	70	I	125	105	87	73	62	52	43	37
			II	78	64	52	42	35	29	24	20
			III	42	35	29	24	20	16	13	11
350 S0	0	70	I	103	85	71	59	50	41	34	29
			II	64	52	42	34	28	22	18	15
			III	34	28	22	18	15	12	10	8
300 L15	−15	27	I	107	90	74	63	53	44	37	32
			II	66	54	43	36	29	24	20	16
			III	36	29	24	20	17	14	12	10
350 L15	−15	27	I	89	73	60	50	42	35	29	25
			II	53	43	35	28	23	19	15	13
			III	28	23	19	15	13	10	8	7

VM: Verification modes against seismic loads.

I *fy* static load: for stresses up to specified yield and negligible plastic deformation.
II *fy* seismic load: for minor plastic deformation at seismic actions (a factor of 1.33fy was applied).
III *fy* seismic load + $\varepsilon = 10\%$: for major plastic deformation at seismic actions ($\varepsilon = 10\%$ was assumed)

Table 15
Rise of lowest permissible temperature in °C due to seismic strain rates $\dot{\varepsilon}>4 \cdot 10^{-4} s^{-1}$

Strain rate $\dot{\varepsilon}$ [s^{-1}]	4e-4	7e-4	1e-3	3e-3	7e-3	1e-2	3e-2	7e-2	1e-1
Steel type: 300 and 350	0	1	2	6	10	12	19	24	27

temperatures T_{Ed} are presented in Table 10, and Table 11 for plates, hot rolled bars and sections (the lowest service temperature given is −120 °C, in order to allow for a temperature shift due to cold forming).

For cold formed hollow sections according to AS/NZS 1163, the maximum permitted wall thickness is given in Table 12 (the effect of cold bending and longitudinal welds required during manufacturing process of rectangular hollow sections have been considered in the development of this table). It was assumed that the product complies with AS/NZS 1163 requirements for aluminium killed steels, otherwise the distance between the longitudinal seam and the tangent to the inner radius should be at least twice the wall thickness [49]. The effect of fabrication welds has also been included (e.g. around the corners).

Any cold forming introduced during the fabrication/construction process should be considered by a rise of the lowest permissible service temperature according to the applied outer fibre plastic strain ε. The temperature shift ΔT should be taken to be:

$$\Delta T_{\varepsilon,cf,EXC} = \begin{cases} 3 \cdot \varepsilon_{cf} & \text{for cold formed} \\ 1.5 \cdot \varepsilon_{cf} & \text{for hot}-\text{formed or heat treated after cold forming} \end{cases} \quad (18)$$

where ε_{cf} is the degree of cold forming expressed as a percentage (see Table 7)

4.3.9. Seismic loads

As discussed in Section 4.3.4, seismic actions require a special consideration. The maximum permissible wall thicknesses at different design temperatures T_{Ed} for three levels of verification are presented in Table 14. Since fracture toughness is dependent on the strain rate $\dot{\varepsilon}$, temperature shifts for strain rates greater than $\dot{\varepsilon} = 4 \cdot 10^{-4}\ s^{-1}$ are presented in Table 15.

For the avoidance of lamellar tearing, the selection of material and weld details should be performed in accordance with AS/NZS 1554.1. The recommendations, however, do not cover welding consumables and fasteners, which should be selected to operate in the notch-ductile temperature range.

5. Comparison of the proposed selection criteria with Australian and New Zealand requirements

A comparison of the lowest permissible service temperature as a function of plate thickness is presented in Fig. 8. The diagram comprises the toughness requirements according to NZS 3404 [2] for a steel type 5, here 350 L15, the proposals for steel 350 L15 according to AS/NZS 3679.1 [46] for a usage of yield of 75% and 50% and finally the same steel at seismic design, where the yield was multiplied by a factor of 1.33 in order to cover the actual yield strength which is usually higher than specified.

From the values given in Table 10 and Table 11, it can be seen that grades 300S0 and 350S0 may be used in sub-zero applications, whilst the current requirements in NZS 3404.1 are very conservative. In addition, the current NZS 3404.1 requirements are more conservative than the present proposal for rectangular hollow sections (currently considered to be steel type 2, 4 and 5 in Table 8). Whilst the temperature range of NZS 3404.1 and AS/NZS 5100.6 is limited to −40 °C, this has been extended in the present proposal to −120 °C to allow for the shift in the temperature, if required. Moreover, the requirements of NZS 3404.1 and AS/NZS 5100.6 that the permissible service temperature should be modified for members subjected to an outer bend fibre strain during fabrication are conservative for a strain up to 6.6% when compared with the present proposal. Finally, permissible service temperature for stress relieved members after bending requires modification in the proposal (see Table 13), which is not considered in either NZS 3404.1 or AS/NZS 5100.6.

6. Conclusion

Following a review of metallurgical effects and materials selection requirements given in other international standards, a new brittle fracture design procedure for Australia and New Zealand has been developed. The proposed design procedure takes into account local requirements for the steel products and specific service conditions, such as seismic strain rates. The temperature range was extended down to −120 °C, which is much lower than considered in many other international standards, in order to allow for temperature adjustment due to service and fabrication conditions. It is demonstrated that the proposal gives a much more efficient utilisation of material than is possible in AS 4100, NZS 3404.1 and AS/NZS 5100.6, which will remove a great deal of conservatism that presently exists in current Australasian design practice. For other countries, the present paper presents the considerations that should be made when applying the fracture mechanics methodology for structural steel grades manufactured to different national standards.

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