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Recent Developments in the Australian/New Zealand Standard AS/NZS 4600 for Cold-Formed Steel Structures

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Abstract

The Australian/New Zealand Standard AS/NZS 4600 is currently under revision based in part on the latest edition of the North American Specification AISI S100:2012 and partly based on the latest research in Australia and New Zealand. The Direct Strength Method (DSM) of design has undergone substantial research since the 2005 edition of AS/NZS 4600 and this research is now incorporated in the revised edition. The new areas in the DSM include shear, combined bending and shear, combined bending and compression, sections with holes and inelastic reserve capacity. Further, the prequalified sections now include most sections with longitudinal web and flange stiffeners based in part on Australian research on high strength sections with multiple stiffeners.

New areas in the Australian/New Zealand Standard include extension of Section 8 Testing to design based on testing, Section 9 Design for Fire, Appendix B Methods of Analysis including advanced analysis, and Appendix D Buckling moments and stresses for local, distortional and global buckling. Revisions of design rules for net section tension and block shear rupture at bolted connections based on Australian research, inclusion of oversize and slotted holes, and screwed connections in tension and shear now are also included. The paper includes the research basis of the latest revisions with the supporting references.

The Australian Buildings Code Board (ABCB), which regulates buildings in Australia by way of the National Construction Code (NCC 2015), has recently changed the loading data for wind, snow and earthquake from 50 year to annual probability of occurrence. This has the effect of increasing the target safety indices. The paper describes the recalibration process for test based design “using the revised loading data.

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INTRODUCTION

The Limit States Australian/New Zealand Standard for Cold-Formed Steel Structures was originally published in 1996 based party on the American Iron and Steel Institute Specification at that time, and partly on Australian research on high strength steels to AS 1397 (Standards Australia, 2011). The higher strength G450, G500 and G550 steels result in more severe stability problems including new modes such as distortional buckling which was incorporated in the 1996 edition. In 2005, a revision of AS/NZS 4600 (Standards Australia 2005) occurred which included design for low ductility G550 steel as commonly used in steel framed housing. The more recent editions of the North American Specification have included distortional buckling and higher strength low ductility steels. The AISI has published the 2012 Edition (AISI S100:2012) of its specification which substantial updates to the DSM which are also being incorporated in the revised edition of AS/NZS 4600.

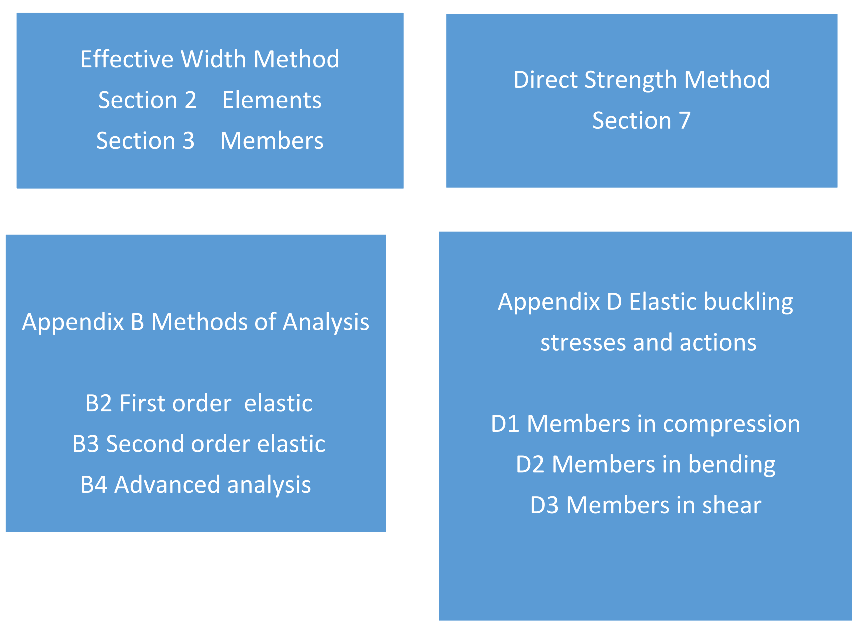


Figure 1. Design and Analysis Modules in AS/NZS 4600

Currently, two basic design methods for cold-formed steel members are available in the Australian/New Zealand Standard for Cold-Formed Steel

Structures (AS/NZS 4600:2005) (Standards Australia, 2005). They are the traditional Effective Width Method (EWM) specified in Section 2 Elements and Section 3 Members and the newly developed Direct Strength Method of design (DSM) as specified in Chapter 7 as shown in Fig. 1. The EWM has been "grand-fathered" in the revised edition on the basis that Committee BD/82 of Standards Australia required that all existing design methods are maintained in their current form without restriction. The DSM has undergone extensive revision and extension in line with AISI S100:2012 as described later in the paper. A new Appendix B has been added to the Standard to clearly specify B2 First Order Elastic Analysis, B3 Second Order Elastic Analysis and B4 Advanced Analysis as described later in the paper. Australian research on advanced analysis methods at the University of Sydney has been used to develop the new Appendix B4. Appendix D, which previously included only buckling solutions for distortional buckling, has been extended to include all elastic buckling solutions for local, distortional and global buckling of sections with and without holes.

The new testing methodology in Section 8 Testing is closely linked to the National Construction Code (NCC) of the Australian Buildings Code Board (ABCB, 2015). Two significant changes have been made to Section 8 in the new edition. They are the determination of design values based on prototype testing where the average of the test results can now be used, and calibration of a strength prediction model based on prototype testing. It also includes members in compression, bending and shear.

A new Section 9 Fire Design has been added to the standard using research at Queensland University of Technology, Australia. The methodology has been developed for Australian high strength steels to AS1397 assuming protected cold-formed steel building members. A new informative Appendix G for members subject to non-uniform temperature distribution is included. The new methodology is described in the paper.

Significant research has been performed recently in Australia at the University of Wollongong on net section fracture and block shear rupture. New equations have been developed and included in Section 5 Connections for net section fracture and block shear rupture where new shear lag factors have been incorporated. Further, the shear planes in block shear rupture are now based on average shear planes rather than gross or net sections at bolted connections.

Additionally, new areas in AS/NZS 4600 based mainly on AISI S100:2012 include

1. Uniformly compressed stiffened elements with non-circular holes
2. Uniformly compressed elements restrained by intermittent connections
3. Combined bending and torsional loading
4. Compression members composed of two sections in contact
5. New equations for C- and Z-beams with neither flange connected to sheeting
6. Modification factors for bearing of bolted connections with oversize and short slotted holes
7. Screwed connections in shear and tension
8. Power actuated fasteners (PAFs)
9. Screwed connections in roof battens

In addition, failure of the screws in shear or tension is now a permitted limit state where the capacities of the screws are based on testing.

SECTION 7 DIRECT STRENGTH METHOD

Pre-qualified sections

Thin-walled sections are becoming more complex with additional longitudinal web stiffeners and return lips as demonstrated in Fig. 2. For the EWM, the calculation of effective widths of the numerous sub-elements leads to severe complications with decreased accuracy. In some special cases, no design approach is even available for such a section using the EWM. The DSM appears to be more beneficial and simpler by using the elastic buckling stresses of the whole section. Consequently, the pre-qualified sections for use in the DSM have been extended to include up to 4 intermediate stiffeners in stiffened compression elements and webs and 2 in the flanges of edge stiffened elements which may include return lips.

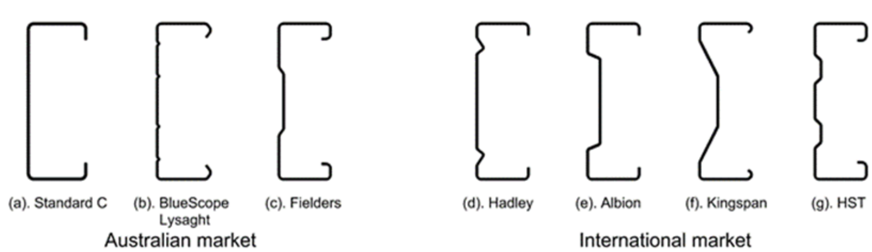


Figure 2 Channel sections with additional stiffeners

In order to validate the extension of the range of complex sections to larger intermediate stiffeners and multiple intermediate stiffeners as occurs in practice (see Fig. 2), experimental programs were performed at the University of Sydney for bending by Pham and Hancock (2014), and for shear by Pham, Bruneau and Hancock (ASCE, 2015).

Sections with Holes

The inclusion of holes in the DSM calculations requires the calculation of the elastic buckling loads and stresses for perforated sections. Equations for this purpose are included in Appendix D of AS/NZS 4600 based on research by Moen and Schafer (ASCE, 2011) as included in Appendix 1 and the Commentary of AISI S100:2012

DSM Design Rules for Shear

The recent development of the Direct Strength Method (DSM) of design of cold-formed sections in pure shear was included in the 2012 Edition of the North American Specification for Cold-Formed Steel Structural Members (AISI S100-2012) based mainly on Australian research (Pham and Hancock, 2012). It is now included in AS/NZS 4600 as follows:

DSM design rules in shear without Tension Field Action

The nominal shear strength (V_v) of beams without holes in the web and without transverse web stiffeners is as follows:

For $\lambda_v \leq 0.815 : V_v = V_y$ (1)

For $0.815 < \lambda_v \leq 1.227 : V_v = 0.815\sqrt{V_{cr}V_y}$ (2)

For $\lambda_v > 1.227 : V_v = V_{cr}$ (3)

$$V_y = 0.6A_w f_y \quad (4)$$

$$V_{cr} = \frac{k_v \pi^2 E A_w}{12(1-\nu^2)\left(\frac{d_1}{t_w}\right)^2} \quad (5)$$

where V_y is the yield load of web based on an average shear yield stress of $0.6f_y$;

V_{cr} is the elastic shear buckling force of the whole section derived by integration of the shear stress distribution at buckling over the whole section; $\lambda_v = \sqrt{V_y / V_{cr}}$;

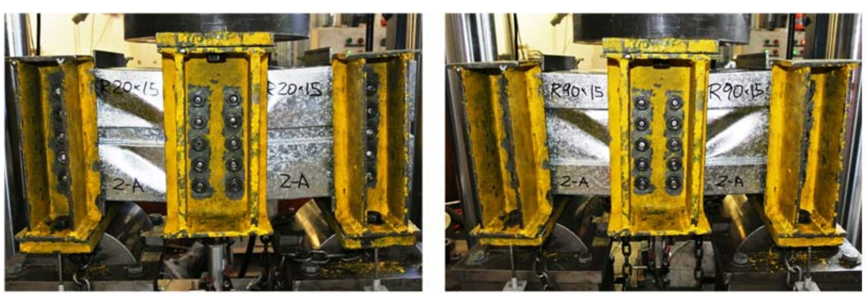
k_v is the shear buckling coefficient of the whole section based on the Semi-Analytical Finite Strip Method (SAFSM) (Hancock and Pham, 2013a). Alternatively, shear buckling coefficients have been tabulated in Appendix D for a range of sections including Lipped Channel Beams (LCB), LiteSteel Beams (LSB), Hollow Flange Beams (HFB) (Keerthan and Mahendran, 2015)

DSM design rules in shear with Tension Field Action

The nominal shear strength (V_v) of beams without holes in the web including tension field action is determined using Equation 6 which is based on Appendix 1, Section 1.2.2.2.1 of NAS-2012 (AISI, 2012) as follows:

$$V_v = \left[1 - 0.15\left(\frac{V_{cr}}{V_y}\right)^{0.4}\right]\left(\frac{V_{cr}}{V_y}\right)^{0.4} V_y \tag{6}$$

A photograph of a web-stiffened channel under test in shear (Bruneau et al. 2015) is shown in Fig. 3 and demonstrates the different tension field action with different depth intermediate stiffeners.



(a). SWC-R20x15 (b). SWC-R90x15

Figure 3 Web-stiffened sections under test in shear

DSM Design Rules for Combined Bending and Compression

The design axial compression (N^*), and the design bending moments $\left(M_x^* \text{ and } M_y^*\right)$ about the x- and y-axes of the gross section, respectively are required to satisfy the following linear interaction equation:

$$\frac{N^*}{\phi_c N_c} + \frac{M_x^*}{\phi_b M_{bx}} + \frac{M_y^*}{\phi_b M_{by}} \leq 1.0 \quad (7)$$

where

N_c = nominal member capacity of the member in compression

M_x^*, M_y^* = design bending moment about the x- and y-axes of the gross section, respectively including the second order moments in accordance with Appendix B3 Second Order Elastic Analysis

M_{bx}, M_{by} = nominal member moment capacity about the x- and y-axes, respectively

In the application of Equation 7 including second order moments, the effective lengths are taken as the actual length L or the length between brace points. The two key developments are the use of actual lengths with the second order elastic analysis and the use of gross sections rather than the effective sections when calculating the line of action of the axial forces.

SECTION 8 TESTING

Section 8.2 Testing for Assessment or Verification of AS/NZS 4600 has undergone significant revision by providing separate Clauses 8.4.1 Prototype Testing and 8.4.2 Strength Prediction Model. The former applies to a design value R_d for a specific product or assembly, and the latter applies to the calibration of a design equation according to the Australian National Construction Code (NCC) (ABCB, 2015).

The design value (R_d) for a specific product or assembly is required to satisfy either….

$$R_d = \left({R_{min}}/{k_{t-min}}\right) \quad (8)$$

$$R_d = \left(R_{ave} / k_{t-ave} \right) \tag{9}$$

where:

> R_{min} is the minimum value of the test results and $k_{t\text{-}min}$ is the sampling factor given in Table 8.2.3(a) of AS/NZS 4600.
> R_{ave} is the average value of the test results and $k_{t\text{-}ave}$ is the sampling factor as given in Table 8.2.3(b) of AS/NZS 4600.

The values of $k_{t\text{-}min}$ and $k_{t\text{-}ave}$ depend upon the coefficient of variation of structural characteristics V_{sc} given by:

$$V_{sc} = \sqrt{(V_f^2 + V_m^2)} \tag{10}$$

The coefficient of variation of structural characteristics (V_{sc}) refers to the variability of the total population of the production units. It includes the total population variation due to fabrication (V_f) and material (V_m). By way of example, if V_{sc} is 10% and 5 units are tested, then $k_{t\text{-}min}$ is 1.28 and $k_{t\text{-}ave}$ is 1.34.

The alternative approach using a strength prediction model for the resistance R is given by

$$R = k_m . k_f . k_t . R_n \tag{11}$$

where:

> R_n = nominal design strength
> k_m = factor to account for variation in material properties
> k_f = factor to account for variation in fabrication
> k_t = factor to account for the accuracy of the prediction

An assessment of the mean value and the coefficient of variation of (R/R_n) is required to derive the capacity factor φ to be used. In this case, V_{sc} is given by:

$$V_{sc} = \sqrt{(V_f^2 + V_m^2 + V_t^2)} \tag{12}$$

The coefficient of variation of structural characteristics (V_{sc}) refers to the variability of the total population of the production units. This includes the total population variation due to fabrication (V_f), material (V_m) and variation of the prediction (V_t). The value of V_t is established to reflect the difference between the test results and the strength prediction model.

The capacity factor φ is determined to satisfy the verification method BV1 of the National Construction Code (ABCB, 2015). In the 2015 revision of the NCC, the loading data has been modified to annual probability of exceedance with a consequent increase in the required safety indices. The safety index in the NCC is given by:

$$\beta = \ln\left[\left(\frac{R_m}{Q_m}\right)\sqrt{\left(\frac{C_Q}{C_R}\right)}\right] / \sqrt{(\ln(C_R C_Q))} \tag{13}$$

where

$$\left(\frac{R_m}{Q_m}\right) = \left(\frac{\gamma}{\emptyset}\right)\left(\frac{R_m}{R_n}\right) / \left(\frac{Q_m}{Q_n}\right) \tag{14}$$

$$C_R = 1 + V_R^2 \tag{15}$$

$$C_Q = 1 + V_Q^2 \tag{16}$$

Q_m = mean action
Q_n = nominal design action
R_m = mean resistance
R_n = nominal design resistance
V_Q = coefficient of variation with respect to action
V_R = coefficient of variation with respect to resistance

For example, for permanent (dead load) and imposed (live load) actions, the safety index β is 3.8 when annual probability data is used. This may be reduced by 0.3 for other than primary structural components which may be applicable to cold-formed steel design. The corresponding β values for wind, earthquake and snow actions are 3.2, 3.4, 3.6 and 3.8 at Importance Levels 1, 2, 3 and 4 respectively.

SECTION 9 FIRE DESIGN

The protected cold-formed steel structural members are designed to have a Period of Structural Adequacy (PSA) equal to or greater than the required Fire Resistance Level (FRL). The FRL is the fire resistance period in minutes required to be attained in a standard fire test. The PSA is normally determined using the elevated temperature mechanical properties of cold-formed steels and the temperature-time relationship of cold-formed steel structural members in the standard fire test. Since thin-walled cold-formed steel structural members have a high exposed surface area to mass ratio, temperature development is likely to be rapid and high. Hence they are normally located within or protected by fire-resistant barriers when they are required to have a FRL. Section 9 of AS/NZS 4600 applies to such protected cold-formed steel structural members. The PSA can be determined by a simple fire test, by calculations by determining the limiting temperature of the cold-formed steel structural member and then determining the time from the start of the standard fire test to the time at which the limiting temperature is reached using the temperature time relationships, or by advanced analysis.

The PSA can also be determined using the elevated temperature capacities of members at a given time in the standard fire test based on their temperature-time relationships. For members subject to uniform or near uniform temperature distributions in applications such as beams or columns, ambient temperature design capacity rules are used with appropriately reduced mechanical properties as described below. For members subject to non-uniform temperature distributions, the net eccentricity due to neutral axis shift and thermal bowing and their magnification effects are used in calculating the resulting bending moment on a wall stud. Appendix G gives guidance on the determination of the load bearing capacity of cold-formed steel structural members used in floors or load bearing walls under non-uniform temperature distribution. Section 9 and Appendix G are based on research at Queensland University of Technology (Gunalan and Mahendran, 2014)

The influence of temperature (T) on the yield stress is defined by a reduction factor $\frac{f_{y,T}}{f_{y,20}}$ as follows:

For high strength steels (G450, G500 and G550) to AS 1397,

$20 \leq T < 300^{o}C$ $$\frac{f_{y,T}}{f_{y,20}} = -0.000179T + 1.00358 \quad (17)$$

$300 \leq T < 600^{o}C$ $$\frac{f_{y,T}}{f_{y,20}} = -0.0028T + 1.79 \quad (18)$$

$600 \leq T < 800^{o}C$ $$\frac{f_{y,T}}{f_{y,20}} = -0.0004T + 0.35 \quad (19)$$

For low strength steels (G250, G300 and G350) to AS 1397,

20≤T<300°C $$\frac{f_{y,T}}{f_{y,20}} = -0.0005T + 1.01 \quad (20)$$

200≤T<800°C $$\frac{f_{y,T}}{f_{y,20}} = 25(1.16 - T^{0.022}) \quad (21)$$

where:

$f_{y,T}$ = yield stress of steel at T˚C
$f_{y,20}$ = yield stress of steel at 20˚C

The influence of temperature (T) on the modulus of elasticity is defined by a reduction factor $\left(\frac{E_T}{E_{20}}\right)$ as follows:

For all steels to AS 1397

20≤T<200°C $$\frac{E_T}{E_{20}} = -0.000835T + 1.0167 \quad (22)$$

20≤T<300°C $$\frac{E_T}{E_{20}} = -0.00135T + 1.1201 \quad (23)$$

where:

E_T = modulus of elasticity of steel at T˚C
E_{20} = modulus of elasticity of steel at 20˚C

The influence of temperature (*T*) on the stress-strain relationship for cold-formed steel is as follows:

$$\varepsilon_T = \frac{f_T}{E_T} + \beta\left(\frac{f_{y,T}}{E_T}\right)\left(\frac{f_T}{f_{y,t}}\right)^{\eta_T} \quad (24)$$

where: ε_T is the strain corresponding to a given stress f_T at temperature (*T*), E_T and $f_{y,T}$ are modulus of elasticity and yield stress at temperature (*T*) respectively, and η_T and β are two parameters.

For high strength steels (G450, G500 and G550) to AS 1397,
For 20 ≤T<800°*C*
β = 0.86
$$\eta_T = -3.05 * 10^{-7}T^3 + 0.0005T^2 - 0.2615T + 62.653 \quad (25)$$

For low strength steels (G250, G300, G350) AS 1397,
For 300≤T<800°C
β = 1.5
$$\eta_T = 0.000138T^2 - 0.085468T + 19.212 \tag{26}$$

NET SECTION TENSION AND BLOCK SHEAR RUPTURE

Net Section Tension

A new equation for net section tension has been included in Clause 5.3.3 of AS/NZS 4600 based on research at the University of Wollongong (Teh and Gilbert 2014). The new equation better accounts for shear lag in flat sheets and has also been balloted in the new edition of AISI S100. The design tensile capacity (ϕN_f) of the connected part is determined as follows:

$$N_f = \left[0.9 + \left(0.1\frac{d_f}{s_f}\right)\right] A_n f_u \tag{27}$$

where: ϕ = 0.8
d_f = diameter of fastener
s_f = spacing of bolts perpendicular to the line of the force, or width of sheet, in the case of one bolt
A_n = net area of the connected part

The improved equation is more reliable and therefore allows a higher capacity factor of 0.8 to be used for design than previously at 0.55 – 0.65.

Block Shear Rupture

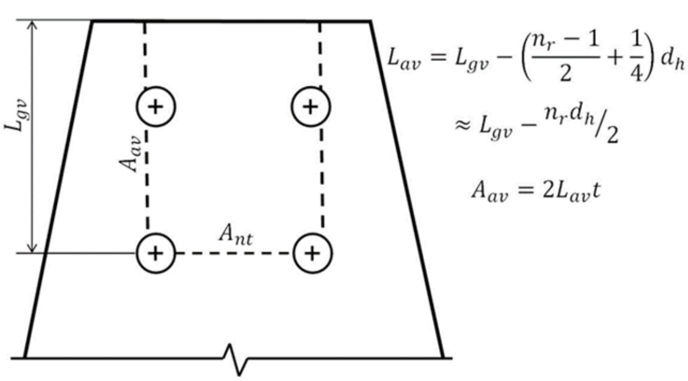


Figure 4 Shear and tension failure planes in block shear rupture

A new equation for block shear rupture has been included in Clause 5.7.3 of AS/NZS 4600 based on research at the University of Wollongong (Teh and Clements, 2012). The new equation better accounts for shear lag in flat sheets as for Equation 27 and also more accurately represents the shear failure planes shown in Fig. 4. The design tensile capacity (ϕR_n) of the connected part is determined as follows:

$$R_n = 0.6 f_y A_{av} + f_u A_{nt} (0.9 + 0.1 \left(\frac{d_f}{s_f}\right)) \quad (28)$$

where $\phi = 0.8$

A_{av} is the active shear area in block shear rupture defined in Fig. 4, L_{gv} is the distance from the free edge to centerline of bolt furthest from the edge*,* n_r is the number of rows of bolts, s_f is the spacing of bolts perpendicular to the line of the force, A_{nt} is the net area subject to block shear tension, and d_f is the diameter of the fastener. The capacity factor ϕ is increased from 0.65 to 0.8 due to the better reliability of the revised equation.

APPENDIX B METHODS OF ANALYSIS

The new Appendix B contains provisions for the structural analysis of cold-formed steel framing systems comprised of braced frames, unbraced frames, portal frames, braced compression members, or combinations thereof. The design action effects in a structure and its members and connections caused by the design loads are determined by structural analysis using one of the methods of—

(a) first order elastic analysis, in accordance with Clause B2;

(b) second order elastic analysis, in accordance with Clause B3; or

(c) advanced analysis, in accordance with Clause B4.

Appendices B2 First Order Elastic Analysis and B3 Second Order Elastic Analysis

The first order elastic analysis, also referred to as linear analysis (LA), and second order elastic analysis, also referred to a geometric non-linear analysis

(GNA), follow the same assumptions and methodology as in the Australian Steel Structures Standard AS4100:1998 (Standards Australia, 1998). The only significant difference is in the frame geometric imperfections which have been based on Eurocode 3 Part 1.1.

Appendix B4 Advanced Analysis

Advanced structural analysis, also referred to as geometric and material nonlinear analysis with imperfections (GMNIA), of a cold-formed steel framing system is required to consider all of the following effects:

a) Flexural, shear and axial member deformations, and connection deformations that contribute to displacements of the structure;

(b) Second-order effects arising from displacements of the structure and its members;

(c) Geometric imperfections, comprising:

— frame imperfections (out-of-plumbness),

— member imperfections (out-of straightness), and

— cross-sectional imperfections (distortions of cross-section);

(d) Stiffness reductions due to axial forces and inelasticity including the effect of residual stresses and partial yielding of the cross-section;

(e) Stiffness reductions due to cross-section deformations or local and distortional deformations;

(f) Uncertainty in system, member, and connection stiffness and strength.

For the strength and stability limit states, the frame is required to support the factored limit states actions multiplied by $1/\phi$, where values of ϕ are given in Table B4 for prequalified frames. For steel storage racks, $\phi = 0.90$ and for pitched roof portal frames, $\phi = 0.85$. Connections are required to have adequate strength and ductility to ensure the structure fails within the members. The design capacity (R_d) of connections is determined as per Section 5 of AS/NZS 4600 and needs to be shown to equal or exceed the design actions to which the connections are subjected as predicted by the advanced analysis. Appendix B4 is based on research by Cardoso et al. (2015). at the University of Sydney.

CONCLUSIONS

A substantial revision of the Australian/New Zealand Cold-Formed Steel Structures Standard AS/NZS 4600 is in progress. At the time of writing this paper, it was at public review stage. The revised standard is based partly on the 2012 Edition of the AISI North American Specification AISI S100:2012, and partly on recent Australian research. New areas in the standard include fire design in Section 9 and advanced analysis in Appendix B. Substantial revisions have been made to the Direct Strength Method (DSM) to include shear, combined bending and compression, sections with holes and inelastic reserve capacity, as well as a much wider range of prequalified sections. Significant changes in the connections Section 5 include oversize and short slotted holes, screwed connections subject to combined tension and shear, power actuated fasteners (PAFs), new rules for net tension rupture and block shear rupture in bolted connections, and welding of G550 and G500 sheet steels.

ACKNOWLEDGEMENT

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