Steel and Composite Structures, Vol. 5, No. 2-3 (2005) 87-101
DOI: http://dx.doi.org/10.12989/scs.2005.5.2_3.087

Pertinent issues on the strength design of steel structures to AS4100-1998

Mark A. Bradford†

School of Civil and Environmental Engineering, The University of New South Wales, UNSW, Sydney, NSW 2052, Australia

(Received July 4, 2004, Accepted November 15, 2004)

Abstract. This paper describes of an overview to the strength rules in the Australian AS4100 Steel Structures Code that was first issued in Limit States Format in 1990. It focuses on pertinent and characteristic issues, such as the means of analysis for second order effects in frames, and highlights how the tiered approach may lead to efficient design using advanced analysis techniques. It also considers design against buckling in some detail, and shows how advanced solutions may be readily incorporated into the design rules. Implicit in the formulations are the necessity for ductility of the steel, and the scope of the code is limited to steels that display this necessary ductility characteristic.

Key words: advanced analysis; Australian steel design; calibration; ductility; frames; limit states design; steel.

1. Introduction

The Australian steel structures standard AS4100 (SA 1998) was introduced in 1990 to replace and supersede the earlier AS1250 (SAA 1981), which was presented in working stress format. Not only did the AS4100 have a much wider scope than the AS1250, but it also adopted limit states (or load and resistance factor) design philosophy. Limit states design is probabilistically based, and is more rational than working stress design, and leads to more consistent margins of safety. The adoption of limit states design at that time also kept pace with other Australian structural engineering standards, such as the concrete code AS3600 (SAA 1989a), as well as with other national steel codes.

The familiar statement of the strength limit state is that the factored (design) load effects should be less than the factored (design) strength or capacity, that is

$$\Sigma(\gamma \times \text{Nominal load effects}) \leq \phi \times (\text{Nominal capacity}) \tag{1}$$

in which γ is a load factor for a particular type of loading and load combination (e.g. $\gamma = 1.2$ for dead loading when it acts in the same direction as wind loading), and where ϕ is the capacity (or resistance) reduction factor. The strength statement of Eq. (1) (Trahair and Bradford 1998) is different from that of other standards, such as the British Standard BS5950 (BSI 2000, Trahair, Bradford and Nethercot 2001), in which material properties (such as the yield stress) that are used to determine the nominal capacity by the use of prescriptive equations that contain partial factors to account for material variability. The AS4100

†Professor, E-mail: m.bradford@unsw.edu.au

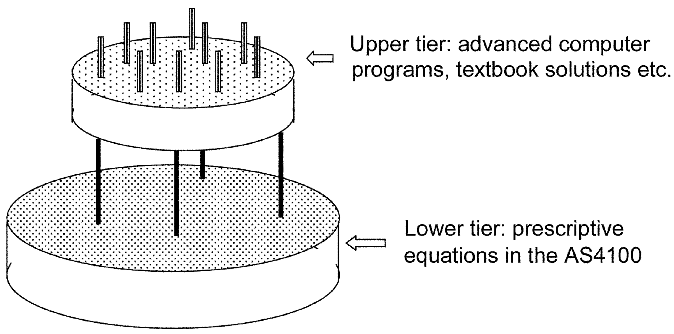


Fig. 1 Tiered (wedding cake) structure of the AS4100

approach uses mean values for material strengths in these prescriptive equations and the possibility of material strength variability in the AS4100 is accounted for in the probabilistic statement of Eq. (1) by the use of an appropriate capacity reduction factor ϕ.

The AS4100 is also presented in a tiered format, in which simple deterministic equations are presented within the code itself. However, in recognition in particular of the growing use of computer software when it was first released in 1990, the code allows for the option of 'advanced analysis', in which an upper tier option may be used for more economical design (Fig. 1) by making recourse to advanced computer analysis, or to graphical or tabulated solutions in reference textbooks.

At the time of its launch, AS4100 was accompanied by a number of design aids, primarily for educational use (Bradford 1997) and to assist design. These have taken the form of textbooks (Trahair and Bradford 1988, Bradford, Bridge and Trahair 1990, Woolcock, Kitipornchai and Bradford 1990, Gorenc, Tinyou and Syam 1996, AISC 1994) and software (Papangelis and Trahair 1989). The use of these aids, design software and in particular second order structural analysis software (Engineering Systems 1996, Integrated Technical Software 1995) is commonplace in contemporary Australian steel design.

This paper aims to provide a background to the strength provisions in the AS4100, by considering its scope and the necessity of ductile behaviour that is implicit in many of the design rules. It looks in some detail at the methods of analysis needed to obtain the second order load effects, which are separated from the strength of members. The limit state of buckling is treated in detail, and it is shown that the tiered approach may be utilised to obtain more efficient designs. The provisions within the AS4100 are comprehensive, and in many ways unique in comparison with other contemporary national limit states codes of practice.

2. Scope and limitations

An underlying assumption in the AS4100 is that of ductility. Because of this, the code does not allow for the use of steel whose yield stress exceeds 450 N/mm^2, as there is often insufficient data for these higher strength steels in terms of the extent of their plastic regions, and their ability to permit redistribution of bending moments within steel structures. This is also the case for cold-formed members (and for which a different standard is used), and for sections with steel elements less than 3 mm thick.

As an example, consider the bolted plate connection shown in Fig. 2. Using non-linear elastic analysis, the distribution of shear stress in the five bolts is as shown, and by using the stress-strain curve

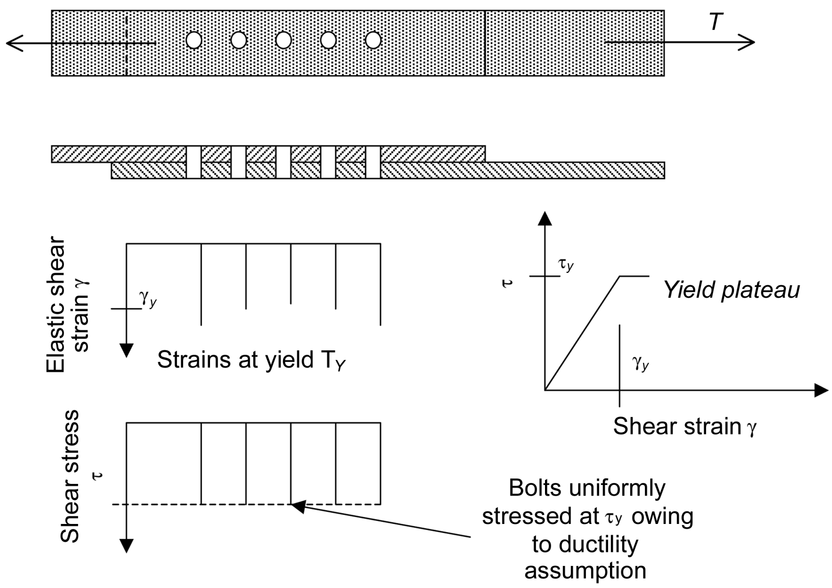


Fig. 2 Implicit assumption of ductility for the bolts in a bolted connection under in-plane forces

that is also shown in this figure, the stresses may also be determined. Hence, when the force T reaches its limit of yielding of the bolted connectors T_y, all of the bolts are equally loaded with a shear force of $T_y/5$. Prior to yield, this is not the case, and so the concept of ductility, viz. that of the plastic plateau in the stress-strain curve, is implicit in the design.

3. Calibration

Although the stochastic strength limit state is presented in deterministic form in Eq. (1), the load and capacity factors for use with the AS4100 are determined using probabilistic models based on appropriate statistical distributions of loadings and capacities. The probability of failure p_F, that is the probability that the inequality in Eq. (1) is violated, is related to the so-called safety index β according to the transformation.

$$\Phi(-\beta) = p_F \tag{2}$$

in which Φ is the cumulative probability distribution of a standard normal (or Gaussian) variate, with a value of $\beta = 2.5$ indicating a failure probability $p_F \approx 10^{-2}$ and $\beta = 5.5$ corresponding to $p_F \approx 10^{-8}$. The choice of the load factors, that are in the Australian loading code AS1170 (SAA 1989b) and that are common to design in steel and in concrete, and of the capacity reduction factor ϕ for steel design indicated in Eq. (1), is based on a calibration procedure. In this procedure, typical structures that had been designed according to the previous working stress code AS1250 were selected, and their safety indices were computed using idealised statistical models of their loads and structural capacities. The load and capacity factors for the limit states steel design method were then varied until the target safety indices were met with reasonable precision (Leicester *et al.* 1985). For example (Pham *et al.* 1986), the safety indices β for the strength limit state design according to the AS4100 were calibrated with those of the working stress AS1250 for steel beams and for columns. These comparisons were made for a dead load factor of 1.25, a live load factor of 1.5 and a capacity reduction factor of 0.9, and it was found that for all but the highest (and most unrealistic) dead load situations, the limit state formulation offers slightly safer designs with a reasonably consistent safety index in the range of 3.0 to 3.5.

The capacity reduction factors ϕ depend not only on the methods used to formulate the nominal capacities, but also on the methods of specifying the nominal loads, and on the values chosen for the load factors. The capacity reduction factors for the AS4100 are generally equal to 0.9 (for members and butt welds), except that 0.7 is used for bolts, pins and connection components.

4. Structural analysis

The AS4100 provides comprehensive guidance for determining the load effects that appear on the left hand side of Eq. (1). These methods of analysis may comprise of an elastic analysis, or a plastic analysis, and which form the lower tier of Fig. 1. In place of these methods, the standard allows for the higher tier of an advanced analysis.

The method of elastic analysis can be used quite generally, and it is intended to be applied to steel structures that transmit significant proportions of the applied loading by bending. The second-order bending moments that take place in members subjected to bending and compression have to be included in the design, and these may be the so-called P-Δ bending moments that arise from the joint displacements Δ, and the P-δ bending moments that arise from the member deflections δ from the straight line that joins the member's ends (Hibbeler 2002). Unlike to BS5950, braced frames in the AS4100 are considered to be fully braced, so that only P-δ effects are relevant, or unbraced, for which the P-Δ effects are usually larger.

Using the lowest tier in the AS4100, a first-order elastic analysis may be carried out that does not include the effects of the P-δ or the P-Δ moments. Most conveniently in hand calculation, this is carried out by elementary methods of structural analysis that are taught at undergraduate level, such as the method of moment distribution, and slope-deflection techniques (Hibbeler 2002). Whist in the past there was a suite of stiffness-based computer programs developed for first-order elastic analysis, those that have been available in Australia for the last decade (Engineering Systems 1996, Integrated Technical Software 1995) include a second order option and so will not be used by Australian engineers to undertake a first order elastic analysis in the lowest tier of the AS4100.

Notwithstanding this fact, considerable attention is directed in the AS4100 on this lowest tier. In this method, the first order moments are required to be amplified by an amplification factor δ_b for a braced frame, or for an amplification factor δ_s for a sway frame. The first order analysis is used to determine the maximum (design) moment M_m* for a given load combination with the appropriate load factors, and in a braced frame the design moment is taken as

$$M^* = \delta_b M_m^* \tag{3}$$

where the moment amplification factor δ_b for a braced member is calculated from:

$$\delta_b = \frac{c_m}{1 - \dfrac{N^*}{N_{omb}}} \geq 1 \tag{4}$$

In Eq. (4), N_{omb} is the elastic buckling load for the braced member that buckles about the same axis as that which the member in the frame is in bending, and c_m is a factor that accounts for the distribution of bending moments, given by

$$c_m = 0.6 - 0.4\beta_m \leq 1 \tag{5}$$

where β_m is the ratio of the smaller to the larger end moments that act on the member, and is taken as negative when the member is bent into single curvature. If the member in question has a transverse load applied to it, then Eq. (5) is used with β_m taken as -1.0 (the most conservative option), or determined from a number of bending moment distributions that are drawn in the AS4100, or from the equation:

$$\beta_m = 1 - 2\frac{\Delta_{ct}}{\Delta_{cw}} \tag{6}$$

provided that $-1 \leq \beta \leq 1$, in which Δ_{ct} is the mid-span deflection of the member that results from the transverse loading and any end moments which may also act, and Δ_{cw} is the mid-span deflection that results from the transverse loading. Only those end moments that produce a mid-span deflection in the same direction as the applied load are considered. In extreme cases as shown in Fig. 3, Δ_{ct} is equal to Δ_{cw} in Fig. 3(a) so that $\beta_m = -1$ in Eq. (6), whilst $\Delta_{ct} < \Delta_{cw}$ in Fig. 3(b) so that $\beta_m > -1$ in Eq. (6).

If the member in question is in a sway frame and the member is free to sway to that P-Δ effects would be expected to occur, then the AS4100 uses the amplification of the maximum first order moment M_m^*, stated as:

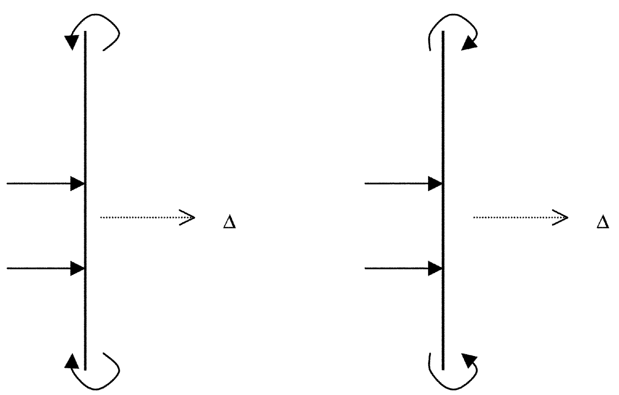
$$M^* = \delta_m M_m^* \tag{7}$$

where

$$\delta_m = \max[\delta_b; \delta_s] \tag{8}$$

in which δ_s is the amplification factor for the sway member. If the member is in a rectangular frame, then

$$\delta_s = \left[1 - \left(\frac{\Delta_s}{h_s}\frac{\Sigma N^*}{\Sigma V^*}\right)\right]^{-1} \tag{9}$$



(a) end moments in direction of Δ (b) end moments oppose Δ

Fig. 3 Definitions for Eq. (6)

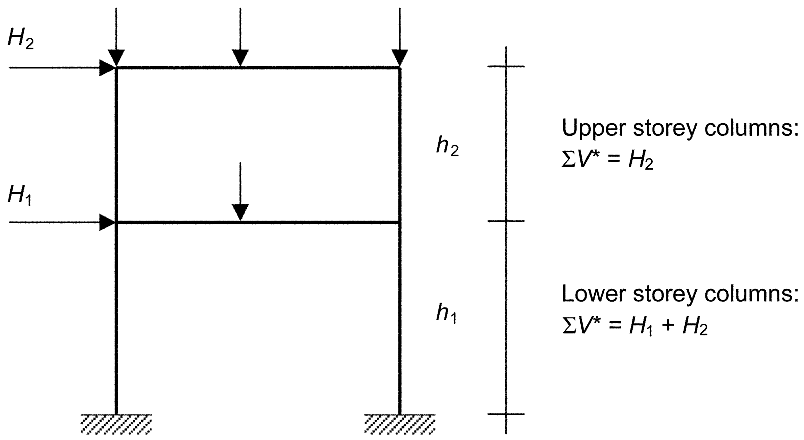


Fig. 4 Definitions for Eq. (9)

where Δ_s is the translational displacement of the top of the member relative to the bottom that is caused by design horizontal shears V^* at the column ends, and N^* is the design axial force in a column in the storey, with the summation being taken over all columns in that storey (Fig. 4).

The sway member amplification factor may also be determined in the AS4100 from:

$$\delta_s = \left[1 - \frac{1}{\lambda_{ms}}\right]^{-1} \tag{10}$$

or from:

$$\delta_s = \left[1 - \frac{1}{\lambda_c}\right]^{-1} \tag{11}$$

in which λ_{ms} is the buckling load factor for the storey in question, and λ_c is the buckling load factor for the whole frame. In Eq. (10):

$$\lambda_{ms} = \frac{\sum_{\substack{\text{columns}\\ \text{in storey}}} \left(\frac{N_{oms}}{h_s}\right)}{\sum_{\substack{\text{columns}\\ \text{in storey}}} \left(\frac{N^*}{h_s}\right)} \tag{12}$$

where N_{oms} is the elastic buckling load for the column in the sway frame. The design force N^* in a column is taken as negative if it is in tension. The frame buckling load factor in Eq. (11) can be taken as the lowest of all of the λ_{ms} values for all of the storeys in the frame, or can be determined using the higher tier elastic buckling analysis. If use is made of contemporary computer analysis software to determine the frame buckling load factor λ_c (as this value is an option in this software), Australian practice would not use Eqs. (7), (8) and (11) since this second order software automatically determines the bending moment diagrams that account for the P-Δ and P-δ bending moments.

The use of Eqs. (4) and (12) requires the calculation of the elastic buckling load of a braced or sway remember respectively. This is given by

$$N_{om} = \frac{\pi^2 EI}{(k_e l)^2} \tag{13}$$

where k_e is the effective length factor for the member. The effective length factor can be determined from charts, that allow for it to be obtained as a function of the ratio of the stiffnesses of the restraining members at each end of the member (Trahair and Bradford 1998).

Although not mentioned in the AS4100, the effective length factor for a member in a frame depends on the load in it by the following argument. At elastic buckling of a frame, all the members become critical at the load factor λ_c, and so if N^* is the load in the member, then the member becomes critical at the load $\lambda_c N^*$. If this load is equated to N_{om}, then rearranging this equation produces:

$$k_e = \frac{\pi}{L}\sqrt{\frac{EI}{N^* \lambda_c}} \tag{14}$$

This result is interesting and illustrates the lack of logic on which calculations in frames based on different member effective lengths are made, since a frame with no force in it (N^*=0) has an effective length factor of infinity, and so buckles at zero load. However, this is not paradoxical, since even when $N^* \rightarrow 0$ and so $k_e \rightarrow \infty$ and so $N_{om} \rightarrow 0$, the ratio N^*/N_{om} remains constant and finite at $N^*/N_{om} = \lambda_c$.

5. Design of members against instability

5.1. Local buckling

Local buckling of the plate elements of cross-sections of width b is accounted for in the AS4100 by use of von Karman's effective width concept, for which the yield strength of the effective section of width b_e is the same as the local buckling strength of the effective section at failure, viz.

$$(f_{ol})bt = (f_y)b_e t \tag{15}$$

in which the elastic buckling stress f_{0l} for a flat plate is (Trahair and Bradford 1998)

$$f_{ol} = k\frac{\pi^2 E}{12(1-\nu^2)}\frac{1}{(b/t)^2} \tag{16}$$

where k is the local buckling coefficient and ν is Poisson's ratio. Hence in Eq. (15) for the effective section

$$k\frac{\pi^2 E}{12(1-\nu^2)}\frac{1}{(b_e/t)^2} = f_y \tag{17}$$

and using Eq. (16) for a flat plate leads to the von Karman formula for the effective width as

$$\frac{b_e}{b} = \sqrt{\frac{f_{0l}}{f_y}} \tag{18}$$

and so

$$\frac{b_e}{b} = \frac{\text{const}}{\lambda_e} \tag{19}$$

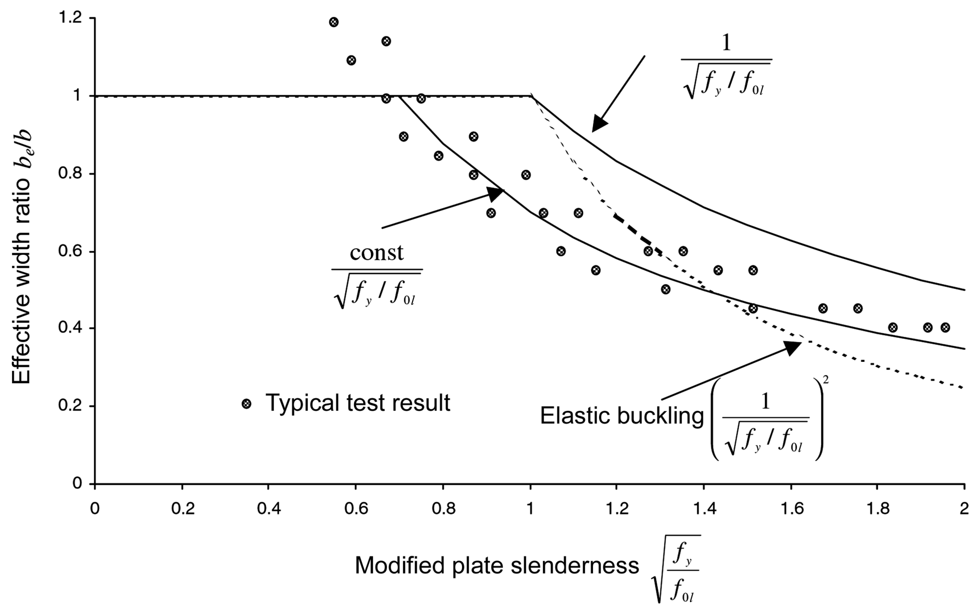


Fig. 5 Plate local buckling strengths

where the plate slenderness in the AS4100 is

$$\lambda_e = \frac{b}{t}\sqrt{\frac{f_y}{250}} \tag{20}$$

and where the yield stress is taken in units of MPa (N/mm^2). The constant in Eq. (19) is not only expressible deterministically from k, E and ν, but it accounts for the effects of plate imperfections, residual stresses etc. This can be illustrated conveniently by plotting Eq. (18), which is the counterpart of Eq. (17) as

$$\frac{b_e}{b} = \frac{\text{const}}{\sqrt{f_y / f_{0l}}}\left(\propto \frac{\text{const}}{\lambda_e}\right) \tag{21}$$

and this is done in Fig. 5, where the constant in Eq. (21) is adjusted to best fit the test results. The value of this constant then forms a delineation from yield to post local buckling at a value of the slenderness λ_{ey} (Bradford 1985), so that the plate strength in compression f_{ult} is

$$\frac{f_{ult}}{f_y} = \begin{cases} 1 & \lambda_e \le \lambda_{ey} \\ \dfrac{\lambda_{ey}}{\lambda_e} & \lambda_e > \lambda_{ey} \end{cases} \tag{22}$$

Without considering imperfection sensitivity in shell theory, the local buckling stress for a circular hollow section of outside diameter d_o and thickness t is (Timoshenko and Gere 1961, Bradford *et al.* 2001).

$$f_{0l} = \frac{2E}{\sqrt{3}\sqrt{1-\nu^2}}\frac{1}{d_0/t} \tag{23}$$

and the same process in Eqs. (15) to (21) again leads to the strength being established by Eq. (22), but in which the slenderness of the circular hollow section is defined as

$$\lambda_e = \frac{d_0}{t}\frac{f_y}{250} \tag{24}$$

Plates in compression members are effective when $\lambda_e \leq \lambda_{ey}$ and their effective widths equal their full widths b, whilst when $\lambda_e > \lambda_{ey}$ they are not effective and their effective widths must be computed from

$$b_e = \left(\frac{\lambda_{ey}}{\lambda_e}\right) b \leq b \tag{25}$$

The effective area for the cross-section, A_e, is then determined by summing the effective areas of all of the plate elements, and the form factor calculated from:

$$k_f = \frac{A_e}{A_g} \tag{26}$$

and the nominal strength of the cross-section for the limit state of local buckling is then:

$$N_s = k_f A_n f_y \tag{27}$$

It is well known from elementary plastic analysis that the moment to cause full yield in a ductile cross-section, the plastic moment M_p, is greater than the moment to cause first yield M_y, and the ratio M_p/M_y is known as the shape factor. For plates in beams with $\lambda_e > \lambda_{ey}$, local buckling will occur before first yield is attained, and the beam strength is less than M_y. In order for the full plastic moment to be reached, strains that are significantly higher than the yield strain, and it is possible that whilst $\lambda_e \leq \lambda_{ey}$ so that the yield moment M_y can be reached, the member may fail in local buckling (in the inelastic range) before the attainment of the fully plastic moment M_p. Because of this, the AS4100 has a lower plasticity limit λ_{ep}, so that sections composed of plates that all satisfy $\lambda_e \leq \lambda_{ep}$ are known as "compact" and their fully plastic moment can be reached before local buckling. Sections for which $\lambda_{ep} \leq \lambda_e \leq \lambda_{ey}$ are called "non-compact", while those for which $\lambda_e > \lambda_{ey}$ are called "slender". This effect is illustrated in Fig. 6 for columns and for beams.

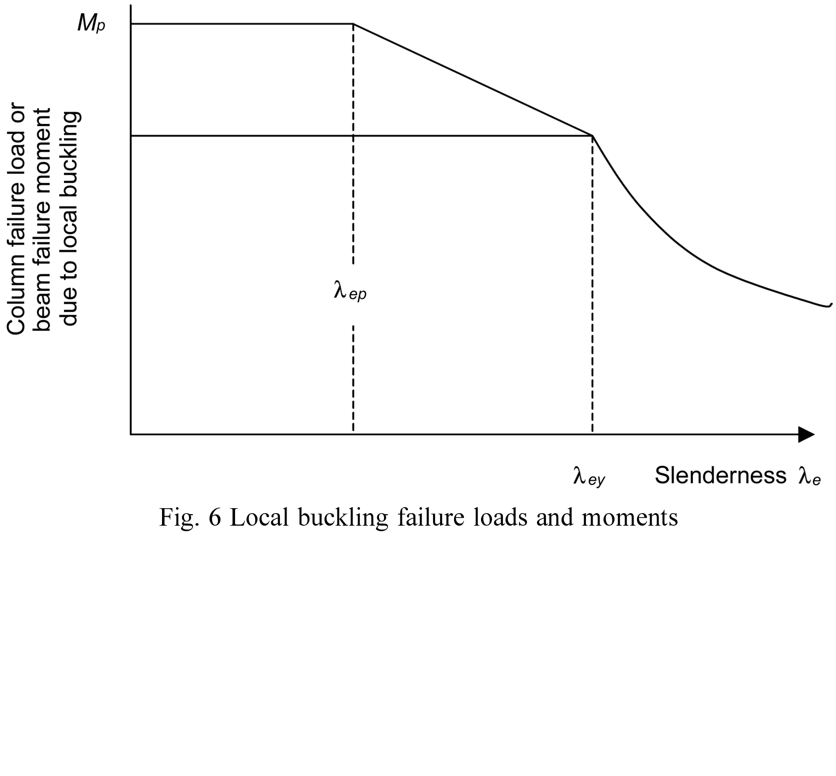


Fig. 6 Local buckling failure loads and moments

For a member in bending, the moment at the limit state of local buckling is:

$$M_s = f_y Z_e \tag{28}$$

where the effective section modulus Z_e is taken as the plastic section modulus S, while for non-compact sections, the interpolation that:

$$Z_e = Z + \left[\left(\frac{\lambda_{ey} - \lambda_e}{\lambda_{ey} - \lambda_{ep}}\right)(S - Z)\right] \tag{29}$$

is used, where Z is the elastic section modulus of the section. For slender sections,

$$Z_e = Z\left(\frac{\lambda_{ey}}{\lambda}\right) \tag{30}$$

which is equivalent to Eqs. (22) and (25).

For parts of a member subjected to shear, the yield stress in pure shear in accordance with von Mises theory is

$$\tau_y = \frac{f_y}{\sqrt{3}} \tag{31}$$

and noting that $1/\sqrt{3} \approx 0.6$, the yield strength of the web is

$$V_w = 0.6 f_w A_w \tag{32}$$

where A_w is the area of the web, usually taken as dt_w, where t_w is the web thickness. In Eq. (16), the local buckling coefficient k for a web in shear is approximately (Trahair and Bradford 1998)

$$k = \begin{cases} 5.35 + 4(d/L)^2 & L \geq d \\ 5.35(d/L)^2 + 4 & L < d \end{cases} \tag{33}$$

where L is the length of the web plate. Equating the shear force to cause local buckling to the yield value in Eq. (32), means that webs for which

$$\lambda_s = \frac{d}{t_w}\sqrt{\frac{f_y}{250}} \leq 82 \tag{34}$$

do not buckle locally in shear and so their strength is V_w in Eq. (32). If the web does not contain any stiffeners, the contribution of post-local buckling is ignored, and using $k = 5.25$ (Eq. (33)) results in the nominal shear buckling capacity being.

$$V_b = \left(\frac{82}{\lambda_s}\right)^2 V_w \leq V_w \tag{35}$$

while if the web has vertical stiffeners, use may be made of the tension-field action (Trahair and Bradford 1998), and the enhanced local buckling coefficient in Eq. (36) is given by:

$$V_b = \alpha_v\left[1 + \frac{1 - \alpha_v}{1.15\sqrt{1 + (s/d)^2}}\right]V_w \tag{36}$$

where

$$\alpha_v = \begin{cases} \left(\frac{82}{\lambda_s}\right)^2\left(\frac{0.75}{(s/d)^2}+1\right) & 1 \le s/d \le 3 \\ \left(\frac{82}{\lambda_s}\right)^2\left(\frac{1}{(s/d)^2}+0.75\right) & s/d \le 1 \end{cases} \tag{37}$$

in which s is the transverse spacing of the vertical web stiffeners.

5.2. Flexural buckling of columns

The design approach for columns in the AS4100 is based on the first yield of a column with a pre-determined bend is given by

$$N_L = \left[\frac{1+(1+\eta)N_{om}}{2}\right] - \left\{\left[\frac{1+(1+\eta)N_{om}}{2}\right]^2 - \frac{N_{om}}{Af_y}\right\}Af_y \tag{38}$$

where the imperfection parameter for this initially sinusoidally-bent member of width B, radius of gyration r and central imperfection δ_o is

$$\eta = \frac{B\delta_0}{2r^2} \tag{39}$$

Real compression members do not have idealised sinusoidal out of straightnesses, nor are the magnitude of their initial imperfection δ_o or their residual stresses often known. Because of this, Eq. (38) has to be calibrated against test results. Rotter (1982) devised a deft technique for presenting the flexural buckling load of the column in the generic fashion

$$\phi R = \phi(\alpha_R R_s) \tag{40}$$

where, generically, R is the nominal resistance against overall buckling, R_s is the nominal strength of the cross-section of zero length that does not buckle in a flexural mode, α_R is a reduction factor for buckling and ϕ is the capacity reduction factor in Eq. (1). In the AS4100, this equation takes the form

$$N_c = \alpha_c N_s \tag{41}$$

where N_s is the cross-section strength given in Eq. (27) and α_c is the member slenderness reduction factor given by

$$\alpha_c = \xi\left\{1 - \left[1 - \left(\frac{90}{\xi\lambda}\right)^2\right]^{1/2}\right\} \tag{42}$$

in which

$$\xi = \frac{(\lambda/90)^2 + \eta}{2(\lambda/90)^2} \tag{43}$$

$$\lambda = \lambda_n + \alpha_a\alpha_b \tag{44}$$

$$\eta = 0.00326(\lambda - 13.5) \ge 0 \tag{45}$$

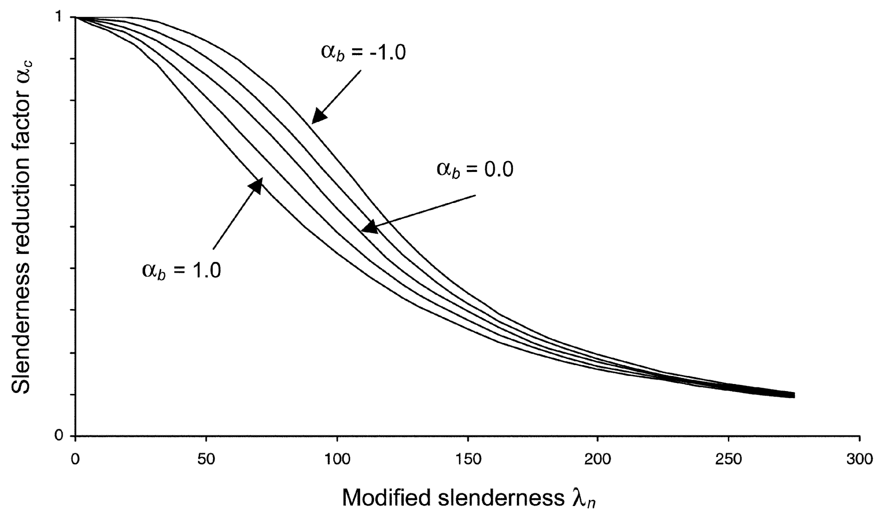


Fig. 7 Multiple column curves of the AS4100

$$\alpha_a = \frac{2100(\lambda_n - 13.5)}{\lambda_n^2 - 15.3\lambda_n + 2050} \tag{46}$$

where the member modified slenderness ratio is given by

$$\lambda_n = \left(\frac{l_e}{r}\right)\sqrt{k_f}\sqrt{\frac{f_y}{250}} \tag{47}$$

in which r is the radius of gyration, l_e is the effective length and k_f is the form factor given in Eq. (26). The AS4100 has five specific values for the member section constant α_b in Eq. (44), ranging from 1.0 for sections with the highest residual stresses to −1.0 for those with no residual stresses. The formulation of Eq. (42) to (47) in the AS4100 renders α_c as being a function of λ_n and the five values of α_b only, and is tabulated in the standard. The multiple column curves are depicted in Fig. 7.

For a column that is tapered, the AS4100 formulation permits an advanced elastic buckling analysis to be made to determine the elastic buckling load N_{om} for the member (e.g. Bradford and Cuk 1988). The slenderness in Eq. (47) is then replaced by

$$\lambda_n = 90\sqrt{\frac{N_s}{N_{0m}}} \tag{48}$$

5.3. Flexural-torsional buckling of beams

The AS4100 treatment for the lateral buckling of beams permits a generic presentation in the form of Eq. (40). The nominal buckling resistance in the code is

$$M_b = \alpha_m \alpha_s M_s \le M_s \tag{49}$$

where the factor α_m is intended to account for the bending moment distribution, and is given in a number of forms. The most popular expression with Australian designers, since bending moment diagrams are usually generated by analysis software, is

$$\alpha_m = \frac{1.7M_m^*}{\sqrt{M_2^{*2} + M_3^{*2} + M_4^{*2}}} \tag{50}$$

in which M_m^* is the maximum moment in the beam segment and M_2^*, M_3^* and M_4^* are the moments at the three quarter points within the beam segment.

The factor α_m is similar to the slenderness reduction factor for columns given in Eq. (42). It is given in AS4100 by

$$\alpha_s = 0.6\left[\sqrt{\left(\frac{M_s}{M_{0a}}\right)^2 + 3} - \left(\frac{M_s}{M_{0a}}\right)\right] \tag{51}$$

where M_s is the cross-section capacity given by Eq. (28), and the reference elastic buckling moment is

$$M_{0a} = \sqrt{\left(\frac{\pi^2 EI_y}{l_e^2}\right)\left[GJ + \frac{\pi^2 EI_w}{l_e^2}\right]} \tag{52}$$

with l_e is the effective length of the member. While Eq. (50) is considered to be a reasonably accurate way to quantify the effect of the distribution of bending moment, the effective length depends on a number of factors (twist and lateral restraints, and height of application of load). Therefore, the prescriptive equations to determine it that are given in the AS4100 are approximate. Because of this, the code allows for the upper tier of design by buckling analysis, where the elastic buckling moment M_{0b} that takes proper account of the member support, restraint and loading condition may be used. Since the effect of the moment gradient is already incorporated into the determination of M_{0b}, Eq. (50) , the value of M_{0a} used in Eq. (51) is M_{0b}/α_m. Generally, this method of design by buckling analysis requires recourse to finite element computer software (Hancock and Trahair 1978, Bradford and Cuk 1988) and so has not yet found widespread favour with Australian designers.

6. Beam columns and frames

The methods for the analysis of frames were discussed earlier in this paper, and once the maximum design moments M^* and axial forces N^* have been determined, with due account for second order effects, the members are normally sequentially checked for a number of strength limit states. One such limit state is the section capacity, which for a member bent around its major axis requires that

$$M^* \le \phi M_s\left(1 - \frac{N^*}{\phi N_s}\right) \tag{53}$$

AS4100 recognises that this equation is conservative for doubly symmetric I-sections and rectangular sections that are compact, and use may be made of the higher tier rule that, for $k_f = 1$,

$$M^* \le \phi 1.18 M_s\left(1 - \frac{N^*}{\phi N_s}\right) \tag{54}$$

or when $k_f < 1$,

$$M^* \le \phi M_s\left(1 - \frac{N^*}{\phi N_s}\right)\left[1 + 0.18\left(\frac{82 - \lambda_w}{82 - \lambda_{wy}}\right)\right] \tag{55}$$

where λ_w and λ_{wy} are the values of λ_e and λ_{ey} for the web.

The check for in-plane member capacity is that

$$M^* \le \phi M_s\left(1 - \frac{N^*}{\phi N_c}\right) \tag{56}$$

but the conservatism of Eq. (56) may be reduced by the use of a higher-tier equation for compact doubly symmetric I-sections and rectangular hollow sections whose form factor $k_f = 1$ given by

$$M^* \le \phi M_s\left\{\left[1-\left(\frac{1+\beta_m}{2}\right)^3\right]\left(1 - \frac{N^*}{\phi N_c}\right)+1.18\left(\frac{1+\beta_m}{2}\right)^3\sqrt{\left(1 - \frac{N^*}{\phi N_c}\right)}\right\} \tag{57}$$

The check for out-of-plane member capacity (lateral buckling) is:

$$M^* \le \phi M_b\left(1 - \frac{N^*}{\phi N_{cy}}\right) \tag{58}$$

where N_{cy} is the column capacity about the minor (y) axis and M_b is the lateral buckling capacity determined from Eq. (49). Again, the conservatism of this equation may be reduced for doubly symmetric compact I-section members that are fully or partially restrained at their ends, but are not loaded transversely, and have a form factor of unity by the use of

$$M^* \le \phi M_{b0}\alpha_{bc}\sqrt{\left(1 - \frac{N^*}{\phi N_{cy}}\right)\left(1 - \frac{N^*}{\phi N_{oz}}\right)} \tag{59}$$

in which

$$\frac{1}{\alpha_{bc}} = \frac{1-\beta_m}{2} + \left(\frac{1+\beta_m}{2}\right)^3\left(0.4 - 0.23\frac{N^*}{\phi N_{cy}}\right) \tag{60}$$

where N_{oz} is the elastic torsional buckling load and M_{bo} is the value of M_{oa} in Eq. (52) with the use of $\alpha_m = 1$ in Eq. (49).

7. Conclusions

This paper has presented an overview of the analysis and design rules for the strength limit state governed primarily by buckling in the Australian standard AS4100 that was introduced in limit states format in 1990. The code allows for the use of a tiered approach, with the higher tier often requiring recourse to a computer program for in-plane frame analysis. The prescriptive higher-tier equations given in AS4100 also find favour with Australian designers, as they can be implemented readily into spreadsheet calculations, and they lead to greater economy.

AS4100 also presents much guidance for plastic design. This finds far less favour with designers, usually because high wind loadings tend to result in lateral buckling considerations that preclude the use of plastic design.

References

Australian Institute of Steel Construction (1994), *Design Capacity Tables for Structural Steel*, 2nd edn., AISC, Sydney.

Bradford, M.A. (1985), "Local and post-local buckling of fabricated box members", *Civil Engineering Transactions*, Institution of Engineers, Australia, **CE27**(4), 391-396.

Bradford, M.A. (1997), "Five years of Australian limit states steel design", *Journal of Professional Issues in Engineering Education and Practice*, ASCE, **123**(1), 39-42.

Bradford, M.A. and Cuk, P.E. (1988), "Lateral buckling of tapered monosymmetric I-beams", *J. Struct. Eng.*, ASCE, **114**(5), 977-996.

Bradford, M.A., Bridge, R.Q. and Trahair, N.S. (1990), *Worked Examples for Steel Structures*, AISC, Sydney.

Bradford, M.A., Loh, H.Y and Uy, B. (2001), "Slenderness limits for CHS sections", *Ninth Int. Symp. on Tubular Structures*, Düsseldorf, Germany, 377-381.

British Standards Institute (2000), BS5950: *The Structural Use of Steelwork in Building: Part 1: Code of Practice for Design: Rolled and Welded Sections*, BSI, London.

Engineering Systems Pty Ltd (1996), *Microstran Users Manual*, Engineering Systems, Sydney.

Gorenc, B., Tinyou, R. and Syam, A. (1996), *Steel Designer's Handbook*, UNSW Press, Sydney.

Hancock, G.J. and Trahair, N.S. (1978), "Finite element lateral buckling of continuously restrained beam-columns", *Civil Engineering Transactions*, Institution of Engineers, Australia, **CE20**(2), 120-127.

Hibbeler, R.C. (2002), *Structural Analysis*, 5th edn. (International), Prentice Hall, New Jersey.

Integrated Technical Software Pty Ltd (1995), *Spacegass Reference Manual*, ITS, Werribee, Victoria, Australia.

Leicester, R.H., Pham, L. and Kleeman, P.W. (1985), "Use of reliability concepts in the conversion of codes to limit states design", *Civil Engineering Transactions*, Institution of Engineers, Australia, **CE27**(1), 1-6.

Papangelis, J.P. and Trahair, N.S. (1989), "A user-friendly program for the Limit States Design of steel structures", *Proceedings, 2nd Pacific Structural Steel Conf.,* Surfers Paradise, Australia, AISC, 155-169.

Pham, L., Bridge, R.Q. and Bradford, M.A. (1986), "Calibration of the proposed limit states design rules for steel beams and columns", *Civil Engineering Transactions*, Institution of Engineers, Australia, **CE28**(3), 268-274.

Rotter, J.M. (1982), "Multiple column curves by modifying factors", *J. Struct. Div.*, ASCE, **108**(ST7), 1665-1669.

Standards Australia (1998), *AS 4100 Steel Structures*, SA, Sydney.

Standards Association of Australia (1981), *AS 1250-1981*, *SAA Steel Structures Code*, SAA, Sydney.

Standards Association of Australia (1989a), *AS 3600 Concrete Structures*, SAA, Sydney.

Standards Association of Australia (1989b), *AS1170, Minimum Design Loads on Structures, Part 1 1989, Dead and Live Loads and Load Combinations*, SAA, Sydney.

Timoshenko, S.P. and Gere, J.M. (1961), *Theory of Elastic Stability*, McGraw Hill, New York.

Trahair, N.S. and Bradford, M.A. (1988), *The Behaviour and Design of Steel Structures*, 2nd edn., Chapman and Hall, London.

Trahair, N.S. and Bradford, M.A. (1998), *The Behaviour and Design of Steel Structures to AS4100*, 3rd edn. (Australian), E&FN Spon, London.

Trahair, N.S., Bradford, M.A. and Nethercot, D.A. (2001), *The Behaviour and Design of Steel Structures to BS5950,* 3rd edn. (British), Spon Press, London.

Woolcock, S.T., Kitipornchai, S. and Bradford, M.A. (1990), *Limit State Design of Portal Frame Buildings*, AISC, Sydney.

KC