yielding and buckling check of a stiffened web panel subject to bending moment, shear force, bearing and axial load. The commentary to AS 4100 [5.1] further notes that this guidance may also be applicable to unstiffened web panels.

An interaction check on open section webs may only be required for very slender webs in plate web girders with high bearing loads and high bending moments. Consequently, no additional advice is given on the matter though the above references may be used if further guidance is required on the topic.

5.3 Design Moment Capacities for Members without Full Lateral Restraint

5.3.1 General

Values of design member moment capacity (ϕM_b) are given in Tables 5.3-1 to 5.3-10 for various values of effective length (l_e in AS4100) based on the uniform moment case (α_m = 1.0) for members bending about the x-axis *without full lateral restraint*. The design section moment capacity (ϕM_{sx} – see Section 5.2.2.1) is also listed in these tables to allow easy calculation of ϕM_b for other moment distributions, as well as the design shear capacity (ϕV_v – see Section 5.2.2.4) for checking the interaction of shear force and bending moment. Additionally, the segment length for full lateral restraint (FLR) is also listed in these tables (see discussion in Section 5.2.2.2).

Structural Tees cut from Universal Columns (CT) are not included in the 5.3 series tables as they are not susceptible to flexible-torsional buckling when bending about the principal axis parallel to the flange. Hence, $\phi M_{by} = \phi M_{sy}$ for these sections, where ϕM_{sy} is given in Table 5.2-8.

Except for the BT sections, each of the 5.3 series Tables is immediately followed by a graph of ϕM_b versus Effective Length (l_e) based on the uniform moment case (α_m = 1.0).

5.3.2 Design Member Moment Capacity

Designers must ensure that the design bending moment (M^*) ≤ ϕM_b for all beam segments. The tabulated values of design member moment capacity (ϕM_b) are determined in accordance with Clause 5.6.1.1 of AS4100 as follows:

$$\phi M_b = \phi \alpha_m \alpha_s M_s \leq \phi M_s$$

where ϕ = 0.9 (Table 3.4 of AS 4100)

α_m = moment modification factor (Clause 5.6.1.1 of AS4100)

= 1.0 **(Assumed for all entries in Tables 5.3-1 to 5.3-10 & immediately following graphs – based on uniform moment case)**

α_s = slenderness reduction factor (Clause 5.6.1.1 of AS 4100)

$$= 0.6\left\{\sqrt{\left[\left(\frac{M_s}{M_{oa}}\right)^2+3\right]}-\left(\frac{M_s}{M_{oa}}\right)\right\} \qquad \text{(Equation 5.6.1.1(2) of AS 4100)}$$

M_{oa} = M_o – the reference buckling moment(Clause 5.6.1.1(a)(iv)(A) of AS4100)

$$= \sqrt{\left(\frac{\pi^2 EI_y}{l_e^2}\right)\left(GJ+\frac{\pi^2 EI_w}{l_e^2}\right)} \qquad \text{(Equation 5.6.1.1(3) of AS 4100)}$$

l_e = effective length of beam segment.