Except for Structural Tees, the FLR values listed in Tables 5.2-1 to 5.2-10 are calculated using β_m = -1.0 which is the most conservative case. However, β_m = -0.8 may be used for segments with transverse loads (as in the case of Tables 5.1-1 to 5.1-12) or β_m may be taken as the ratio of the smaller to larger end moments in the segment length (l in AS4100) for segments without transverse loads (positive when the segments is bent in reserve curvature).

For Structural Tees cut from Universal Beams (BT sections), the value of FLR has been determined by alternate means as AS4100 does not provide specific guidance for these sections. The alternate method calculates FLR by evaluating the maximum effective length for which ϕM_b equals ϕM_S assuming α_m = 1.0. This is listed in Tables 5.2-7 as "Effective Length". However, for BT sections bending about the x-axis causing compression in the stem, there is no effective length listed as $\phi M_b < \phi M_S$ generally – i.e. these types of loaded sections require continuous restraint if the design section moment capacity is to be attained. Additionally, Structural Tees cut from Universal Columns (CT sections) and some BT sections bent about the principal axis parallel to the flange do not buckle laterally and no "Effective Length" values are given in Tables 5.2-7 and 5.2-8.

5.2.2.3 Design Torsional Moment Section Capacity

As noted in Ref[5.1], there is generally not much guidance available on the design of open section members subject to torsion loading. However, further information on analysis and design of such loaded members is given in Refs.[5.1, 5.2].

5.2.2.4 Design Shear Capacity of a Web

Designers must ensure that the design shear force (V^*) ≤ ϕV_v along the beam.

For I-sections and channels the shear stress distribution in the web is approximately uniform and the design shear capacity ϕV_v can be taken as (Clause 5.11 of AS4100):

$$\phi V_v = \phi\,(0.6\, f_{yw} A_w) \qquad \text{for } \frac{d_1}{t_w}\sqrt{\left(\frac{f_{yw}}{250}\right)} \le 82$$

$$\phi V_v = \phi\,(0.6\, f_{yw} A_w)\left[\frac{82}{\left(\frac{d_1}{t_w}\right)\sqrt{\left(\frac{f_{yw}}{250}\right)}}\right]^2 \qquad \text{for } \frac{d_1}{t_w}\sqrt{\left(\frac{f_{yw}}{250}\right)} > 82$$

Where
- ϕ = 0.9 (Table 3.4 of AS 4100)
- d_1 = clear depth of the web
- A_w = area of the web = $d\, t_w$ (for hot-rolled sections)
  = $d_1 t_w$ (for welded sections)
- f_{yw} = yield stress of the **web**
- d = depth of the section

For Structural Tees the shear stress distribution is non-uniform and the web shear capacity is given by

$$\phi V_v = \min\left[\phi V_u, \frac{2\phi V_u}{0.9 + \left(\frac{f^*_{vm}}{f^*_{va}}\right)}\right]$$

Where f^*_{vm}, f^*_{va} = the maximum and average design shear stresses respectively determined from an elastic analysis.

$$\frac{f^*_{vm}}{f^*_{va}} = \frac{b_f t_f\left(y_c - \frac{t_f}{2}\right) + \left(y_c - \frac{t_f}{2}\right)^2 \frac{t_w}{2}}{\left[b_f t_f\left(y_c - \frac{t_f}{2}\right) - \frac{t_w}{6}\left(d - \frac{t_f}{2}\right)^2 + \frac{t_w}{2}\left(y_c - \frac{t_f}{2}\right)\left(d - \frac{t_f}{2}\right)\right]}$$ (from Ref.[5.3])

b_f, t_f = flange width and thickness

t_w = web thickness

d = depth of Tee section

y_c = distance of section centroid to flange outer face

ϕV_u = the design shear capacity assuming a uniform shear stress distribution

= ϕV_v (as noted above for I-sections and channels)

Note for Welded Sections: Clause 6.3.1 of AS/NZS 3679.2 states that the flange-web welded joint for welded sections shall develop the minimum tensile strength of the web (for $t_w \le 16$ mm), or the minimum tensile strength of a 16 mm web (for $t_w > 16$ mm), therefore

ϕv_{wj} = $\phi f_u t_w$ (for $t_w \le 16$ mm)

= 16 ϕf_u (for $t_w > 16$ mm)

Where ϕ = 0.8 (Table 3.4 of AS4100)

ϕv_{wj} = the design capacity of the welded joint

t_w = web thickness

f_u = web tensile strength (see Table T5.2)

Table T5.2: Flange-web joint details for welded sections

Steel Grade (AS/NZS 3679.2)	Web Thickness t_w	Web Tensile Strength f_u	Design Capacity of Welded Joint ϕv_{wj}
	mm	MPa	kN/mm
300	10	430	3.44
300	12	430	4.13
300	≥ 16	430	5.50
400	10	480	3.84
400	12	480	4.61
400	≥ 16	480	6.14

Table T5.2 lists the design capacities of the welded joint for the range of web thicknesses of Welded Beams and Welded Columns. The design shear flow at the welded joint may be evaluated by elastic shear flow principles and must be less than or equal to ϕv_{wj} in Table T5.2. Evaluation of design shear flow should recognise that fillet welds are on both sides of the web. If this is not satisfied, the design shear capacity of the section must be reduced to the value of ϕv_{wj} at the joint. This has been included in the ϕV_v listings for the above sections.

For some Welded Columns (specifically the Grade 400 500WC440 and 400WC361 sections) the design shear capacity (ϕV_v) is governed by the capacity of the welded joint, being given by (d_1 x ϕv_{wj}).

5.2.2.5 Design Web Bearing Capacities

Designers must ensure that the design bearing force (R^*) ≤ ϕR_b at all locations along a beam where bearing forces are present.

The design bearing capacity (ϕR_b) is calculated in accordance with Clause 5.13 of AS4100 and taken as the lesser of ϕR_{by} and ϕR_{bb} as noted below.

The design bearing yield capacity of a web (ϕR_{by}) is calculated in accordance with Clause 5.13.3 of AS4100. The design bearing yield capacity per mm of bearing width (b_{bf}) is given by:

$$\frac{\phi R_{by}}{b_{bf}} = \phi(1.25\, t_w f_{yw})$$

Where b_{bf} is shown in Figure 5.2 and ϕ = 0.9 (Table 3.4 of AS4100).

The design bearing buckling capacity (ϕR_{bb}) is determined using Clause 5.13.4 of AS4100. This is equal to the design axial compression capacity of a member with area = $t_w b_b$ and slenderness, $l_e/r = 2.5\, d_1/t_w$ with $\alpha_b = 0.5$ and $k_f = 1.0$ (Section 6 of AS4100).

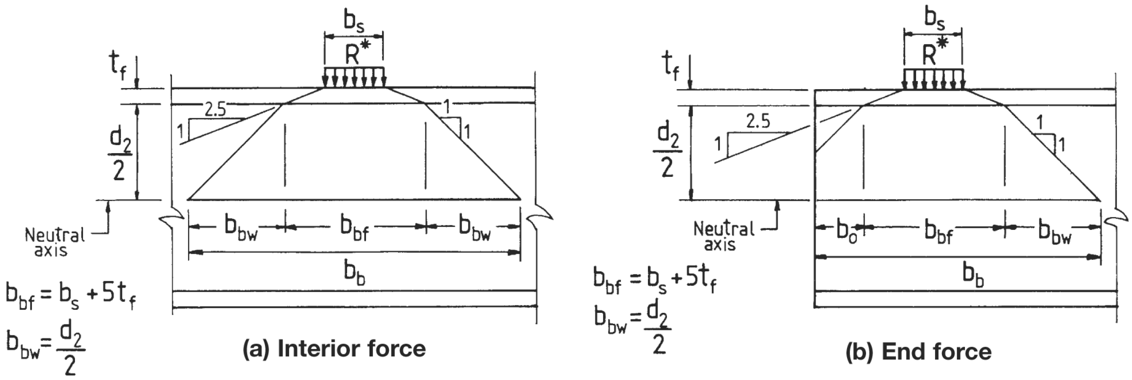

Figure 5.2: b_{bf} and b_b for dispersion of Force Through Flange and Web (after Figure 5.13.1.1 of AS4100)

5.2.3 Example - Web Bearing

A 530UB92.4 – Grade 300 steel beam as shown in Figure 5.3 is subjected to a design end reaction of $R^* = 300$ kN. Check the bearing capacity of this section to resist this design reaction.

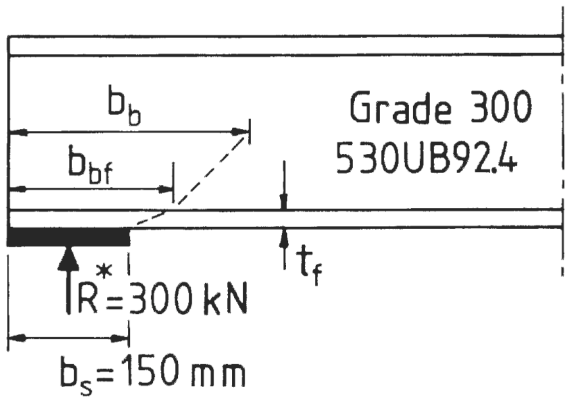

Figure 5.3: Web bearing design example.

<u>Design Data:</u>

Design bearing force	R^*	=	300 kN
Design shear force	V^*	=	300 kN
Stiff bearing length	b_s	=	150 mm
Half clear web depth	$d_1/2$	=	251 mm (Table 3.1-3(A) = $d_2/2$ in Fig. 5.2

<u>Solution:</u>

(1) Check shear capacity

V^* = 300 kN

ϕV_v = 939 kN (Table 5.2-5)

$> V^*$ COMPLIES