CHAPTER 6 WEBS

6.1 General

The design of webs for strength is usually governed by:

(a) the web subjected to shear force and undergoing shear buckling, or yielding in shear or a combination of the two, or

(b) the web subjected to in-plane bending and undergoing local buckling in the compression zone, or

(c) the web subjected to a concentrated transverse force (edge load) at a load or reaction point, and undergoing web crippling.

In all three cases, the buckling modes may interact with each other to produce a lower strength if the shear and bending or the bending moment and concentrated load occur simultaneously.

In AS/NZS 4600, the design of webs in shear is governed by Clause 3.3.4. The interaction between shear and bending is covered in Clause 3.3.5. The design of webs for bearing under point loads and reactions is governed by Clause 3.3.6 and combined bending and bearing is prescribed in Clause 3.3.7. The design of transverse stiffeners at bearing points and as intermediate stiffeners, is set out in Clause 3.3.8.

6.2 Webs in Shear

6.2.1 Shear Buckling

The mode of buckling in shear is shown in Fig. 6.1(a). This mode of buckling applies for a long plate which is simply supported at the top and bottom edges. Unlike local buckling in compression, the nodal lines are not perpendicular to the direction of loading and parallel with the loaded edges of the plate. Consequently, the basic mode of buckling is altered if the plate is of a finite length. The buckling stress for shear buckling is given by Eq. (4.1), with the value of the local buckling coefficient (*k*) selected to allow for the shear loading and boundary conditions. The theoretical values of *k* as a function of the aspect ratio (*a*/*b*) of the plate are shown in Fig. 6.1(b). As the plate is shortened, the number of local buckles is reduced and the value of *k* is increased from 5.34 for a very long plate to 9.34 for a square plate. An approximate formula for *k* as a function of the aspect ratio (*a*/*b*) is shown by the solid line in Fig. 6.1(b) and given by Eq. (6.1).

$$k = 5.34 + \frac{4}{\left(\frac{a}{b}\right)^2} \qquad (6.1)$$

Eq. (6.1) is a parabolic approximation to the garland curve in Fig. 6.1(b).

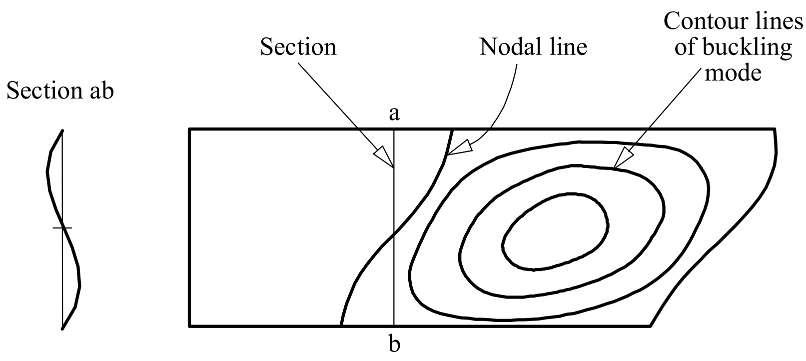
For plates with intermediate transverse stiffeners along the web, the effect of the stiffeners is to constrain the shear buckling mode between stiffeners to be the same as a plate of length equal to the distance between the stiffeners. Consequently the value of '*a*' in Eq. (6.1) is taken as the stiffener spacing and an increased value of the local buckling coefficient over that for a long unstiffened plate can be used for webs with transverse stiffeners.

As explained in Ref. 6.1, plates in shear are unlikely to have a substantial postbuckling reserve as for plates in compression unless the top and bottom edges are held straight. Since cold-formed members usually only have thin flanges, then it is unlikely that the postbuckling strength in shear can be mobilised in the web of a cold-formed member. Hence the elastic critical stress is taken as the limiting stress in Clause 3.3.4 of AS/NZS 4600. Clause 3.3.4 uses Eq. (4.1) to give a nominal shear capacity (V_v) of:

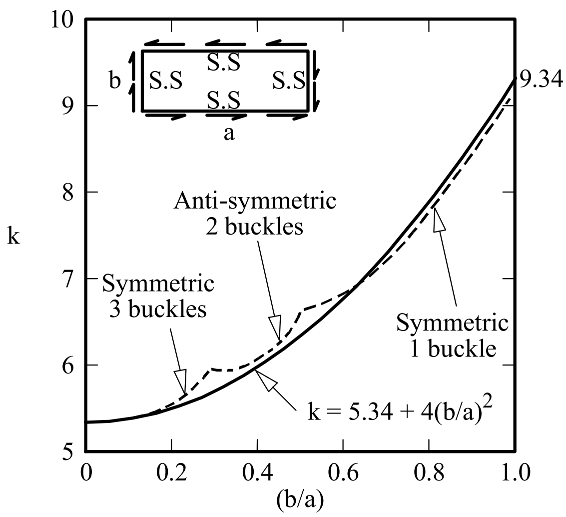
$$V_v = \frac{0.905 E k_v}{\left(\frac{d_1}{t_w}\right)^2}(d_1 t_w) = \frac{0.905 E k_v t_w^3}{d_1} \quad (6.2)$$

where d_1 is web depth which is taken as the depth of the flat portion of the web measured parallel to the web, and k_v is taken as the parabolic approximation to k given by Eq. (6.1). When the stiffener spacing is greater than the web depth such that a/d_1 is greater than 1.0, then b in Eq. (6.1) is replaced by d_1. When the stiffener spacing is less than the web depth, such that a/d_1 is less than or equal to 1.0, then the plate is taken as a plate of length (d_1) with a flat width ratio of a/t_w. Hence b/t in Eq. (4.1) is replaced by a/t_w and a/b in Eq. (6.1) is replaced by d_1/a. The resulting equation for k_v to be used with Eq. (6.2) is:

$$k_v = 4.0 + \frac{5.34}{\left(\frac{a}{d_1}\right)^2} \quad (6.3)$$



(a) Buckled mode in shear for long narrow plate



(b) Buckling coefficient (k) versus aspect ratio (a/b)

Fig. 6.1 Simply supported plates in shear

For plates without stiffeners, k_v is simply taken as 5.34. These formulae for k_v are specified in Clause 3.3.4.

6.2.2 Shear Yielding

A stocky web (small d_1/t) subject to shear will yield in shear at an average stress of approximately $f_y / \sqrt{3}$ as given by the von Mises' yield criterion (Ref. 6.2). The nominal shear capacity for yield (V_v) is therefore given by:

$$V_v = 0.64 \; f_y \, d_1 \, t_w \tag{6.4}$$

The value of 0.64, which is higher than $1/\sqrt{3}$, is consistent with a reduced factor of safety normally assumed for shear yielding in permissible stress design standards. Fig. 6.2 shows the failure stresses for a web in shear, including yield as given by Eq. (6.4) and buckling as given by Eq. (6.2). In the region where shear and buckling interact, the failure stress is given by the geometric mean of the buckling stress and 0.8 times the yield stress in shear (Ref. 6.3). The resulting equation for the nominal shear capacity (V_v) is given by Eq. (6.5) and is shown in Fig. 6.2 for a range of yield stresses.

$$V_v = 0.64 \; t_w^2 \sqrt{E k_v f_y} \tag{6.5a}$$

This equation applies in the range of web slenderness,

$$\sqrt{\frac{E k_v}{f_y}} < \frac{d_1}{t_w} \leq 1.415 \sqrt{\frac{E k_v}{f_y}} \tag{6.5b}$$

The capacity reduction factor (ϕ_v) for webs in shear is 0.90.

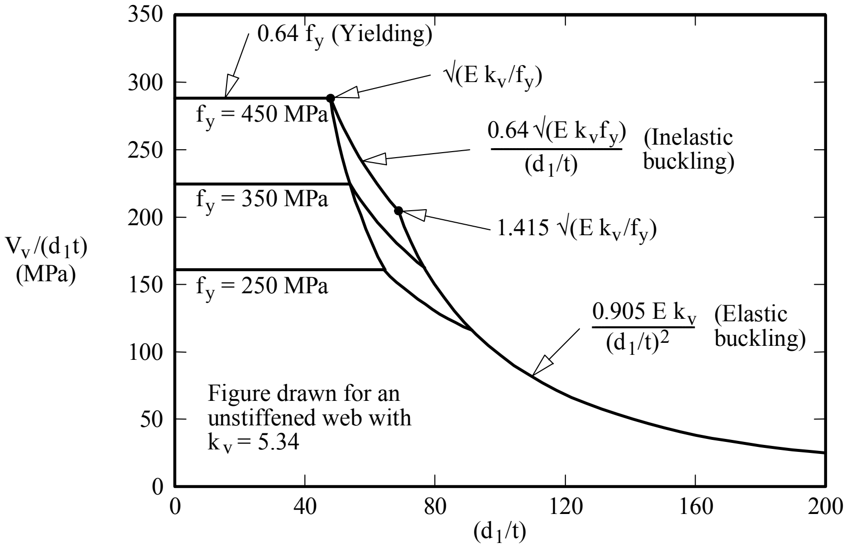


Fig. 6.2 Nominal shear capacity (V_v) of a web expressed as average shear stress

6.3 Webs in Bending

The elastic critical stress of a web in bending is given by Eq. (4.1) with k = 23.9 as shown as Case 5 in Fig. 4.1. As described in Section 4.4, the web in bending has a substantial postbuckling reserve. Consequently the design procedure for webs in bending should take account of the postbuckling reserve of strength. Two basic design methods have been described in Ref. 6.4. Both methods result in approximately the same design strength but require different methods of computation.

Design of Cold-Formed Steel Structures

(To Australian/New Zealand Standard

AS/NZS 4600:2005)

by

Gregory J. Hancock BSc BE PhD DEng

Bluescope Steel Professor of Steel Structures
Dean
Faculty of Engineering & Information Technologies
University of Sydney

fourth edition - 2007

CONTENTS