5.7 Design strength

Based on the studies of Ayrton & Perry (1886), the compressive strength of the column can be obtained from the following equation.

$(f_y - f_c)\ (f_e - f_c) = \eta . f_e . f_c$ ***(5.10)***

Where, f_y = yield stress, f_c = compressive strength, f_e = Euler buckling stress, λ = Slenderness ratio (l/r) and η = a parameter allowing for the effects of lack of straightness and eccentricity of loading and can be expressed as aλ where α is a function of the shape of the cross section. Since Robertson evaluated the mean values of α for many sections, the design method is termed "Perry-Robertson method". Equation (5.8) will result in column strength values lower than f_y even in very low slenderness cases as indicated by the Robertson's curve in Fig. 5.19. By modifying the slenderness, λ to (λ - λ_o), a plateau to the design curve can be introduced for low slenderness values. This has the effect of shifting the curve to the right by a value equal to λ_o. The value of λ_o may be taken as 0.2($\pi\sqrt{E/f_y}$). Thus, the elastic critical stress can be calculated as $f_e = \pi_2 E/(\lambda - \lambda_o)_2$. Note that calculations for f_e is not needed when $\lambda \le \lambda_e$ as the column would fail by squashing at f_y.

Fig 5.19 Column strength curves

5.7.1 Design Strength as per the Code

Common hot rolled and built-up steel members, used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial bow and accidental eccentricities of load. To account for all these factors, strength of members subjected to axial compression are given by multiple design curves corresponding to buckling class a, b, c, or d, as given below. The design compressive strength of a member is given by (Cl.7.1)

$P_d = A_e f_{cd}$ ***(5.11)***

Where, A_e = effective sectional area and
f_{cd} = design stress in compression, obtained as per the following equation:

y m0
0.3 y m0 y m0 2 2
m
f /
fcd X f / f /

γ

= = γ ≤ γ

φ + [φ − λ] [ ]

(5.12)

Where

$\varphi = 0.5[1+\alpha (\lambda - 0.2) + \lambda_2]$

≠ f = 0.5[1=a (1-0.2) + 1_2]

λ_* = non-dimensional effective slenderness ratio

2
e
y 2
y u
f L f f r E

= = π ***(5.13)***

f_{cc} = Euler buckling stress = $\pi_2 E/(KL/r)_2$

L_e/r = effective slenderness ratio, ratio of effective length L_e, to appropriate radius of gyration, r

α = imperfection factor given in Table 5.2

χ = stress reduction factor

c = stress reduction factor

l_{m0} ≠ (γ_{m0})

λ_{mo} = partial safety factor for material strength = 1.1

Table 5.2 Imperfection factor, α

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

The classifications of different sections under different buckling class a, b, c or d, is given in Table 5.3 of the Code. Note that thicker sections and welded sections which are likely to have more residual stresses are assigned lower buckling classes. The curves corresponding to different buckling class are presented in non-dimensional form, in Fig 5.20. The selection of an appropriate curve is based on cross section and suggested curves are listed in Table 5.3. Although both hot rolled sections and welded sections have lock-in residual stresses, the distribution and magnitude differ significantly. Residual stresses due to welding are very high and can be of greater consequence in reducing the ultimate capacity of compression members.

Fig. 5.20 Column Buckling Curves

Table5.3 Buckling class of cross sections
(Section 7.1.2.2)

Cross Section	**Limits**	**Buckling about axis**	**Buckling Class**
	h/b > 1.2 : t_f < 40 mm		

40 mm< t_f < 100 mm
z-z
y-y
z-z
y-y
a
b
b
c
Rolled I-Sections
h/b < 1.2 : t_f <100 mm
t_f >100 mm
z-z
y-y
z-z
y-y
b
c
d
d
Welded I-Section t_f <40 mm
t_f >40 mm
z-z
y-y
z-z
y-y
b
c
c
d
Hollow Section Hot rolled Any a
Cold formed Any b
Generally
(Except as below)
Any b
Welded Box Section
Thick welds and b/t_f
< 30
h/t_w < 30
z-z
y-y
c
c
Channel, Angle, T and Solid Sections Any c

5.7..2 Design of angle compression members

When angles are loaded in compression through their centroid, they can be designed as per the procedure described above using curve c. However, angles are usually loaded eccentrically by connecting one of its legs either to a gusset or to an adjacent member. Such angles will buckle in flexural-torsional mode in which there will be significant twisting of the member. Such twisting may be facilitated by the flexibility of the gusset plate and the other members connected to it. To simplify the design, the code considers only two cases – gusset fixed and gusset hinged. The other parameter which will influence the strength of the angle strut is its width-thickness ratio of either leg. Thus, to account for the reduction in strength due to flexural-torsional mode, the code gives an equivalent slenderness ratio as a function of the overall slenderness ratio and the width-thickness ratio. In general, the equivalent slenderness ratio will be less than or equal to the slenderness ratio for flexural buckling λ_{vv}

The flexural torsional buckling strength of single angle loaded in compression through one of its legs may be evaluated using the equivalent slenderness ratio, λ_{eq}, as given below (Cl. 7.5.1.2)

2 32

$\lambda_{eq} = k_1 + k_2\lambda_{w} + k \lambda \varphi$ ***(5.14)***

Where

k_1, k_2, k_3 = constants depending upon the end condition, as given in Table 5.4,

Built-up Member Any c

w (1 2)
w 2 2
l
r b b
and
E Ex 2t
250 250
φ
+ λ = λ =
π π
ε ε

(5.15 a,b)

Where

l = centre to centre length of the supporting member

r_{vv} = radius of gyration about the minor axis

b_1, b_2 = width of the two legs of the angle

t = thickness of the leg

ε = yield stress ratio $(250/f_y)^{0.5}$

Table 5.4 Constants k_1, k_2 and k_3
(Section 7.5.1.2)

No. of bolts at the each end connection	**Gusset/Connecting member Fixity†**	k_1	k_2	k_3
> 2	Fixed	0.20	0.35	20
	Hinged	0.70	0.60	5
1	Fixed	0.75	0.35	20
	Hinged	1.25	0.50	60

Stiffeness of in-plane rotational restraint provided to the gusset/connecting member. For partial restraint, the λ_{eq} *can be interpolated between the* λ_{eq} *results for fixed and hinged cases.*

For double angle discontinuous struts, connected back to back, on opposite sides of the gusset or a section, by not less than two bolts or rivets in line along the angles at each end, or by the equivalent in welding, the load may be regarded as applied axially (Cl. 7.5.2). The effective length, L_e, in the plane of end gusset can be taken as between 0.7 and 0.85 times the distance between intersections, depending on the degree of the restraint provided. The effective length, L_e, in the plane perpendicular to that of the end gusset, shall be taken as equal to the distance between centers of intersections.