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Design Of Steel Compression Members (According To Is: 800)

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ABSTRACT

Compression members are very important components for any building. All the kinds of loads such as dead load or live load are ultimately transferred to the columns (compression members) which in turn transfer it to the foundation. Thus, a column can be considered to be the main supporting unit for any kind of structure.

This article gives a brief description about the characteristics and the behaviour of steel compression members. The various design steps are taken in accordance to IS:800.

INTRODUCTION

A structural member which is subjected to compressive forces which tend to decrease its length is called a compression member.

If the net end moments are zero, then the load is said to be acting concentrically to the member and the structure is said to be axially loaded.

Compression members are usually given names: the vertical compression members in building frames are called columns, the inclined ones are called struts. The principal compression member in a crane is called a boom.

The strength of a column depends on the following parameters:-

- Material of the column or member.
- Cross-sectional configuration.
- Support conditions.
- Length of the column.
- Residual stresses.

POSSIBLE FAILURE MODES OF A COMPRESSION MEMBER (axially loaded):

- **LOCAL BUCKLING**

Failure occurs by buckling or deflection of one or more parts of the member, for example: flange or web of an I-section. No overall deflection is observed in this kind of buckling.

- **SQUASHING**

Squashing occurs in relatively small length columns. It occurs by yielding of a cross section of the column.

- **OVERALL FLEXURAL BUCKLING**

In this mode, failure of the member occurs by excessive deflection in the plane of weaker principal axis.(fig1)

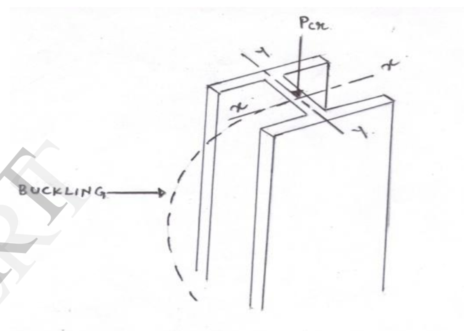


Fig. 1

In the above fig, x-x and y-y axis are shown. $I_{xx} > I_{yy}$, so the resistance about y-y axis is less as compared to x-x axis. Hence buckling will occur about y-y axis.

- **TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING**

Torsional buckling failure occurs by twisting of the column about shear center in the longitudinal axis. A combination of flexural and torsional buckling is called flexural-torsional buckling.

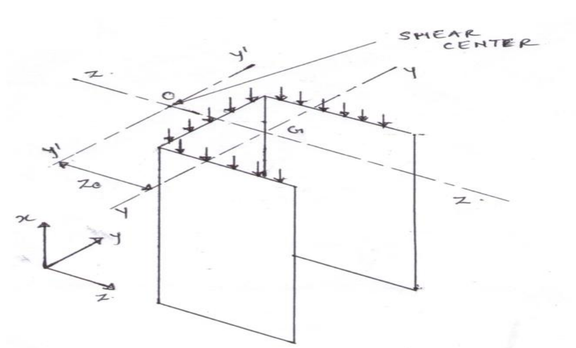


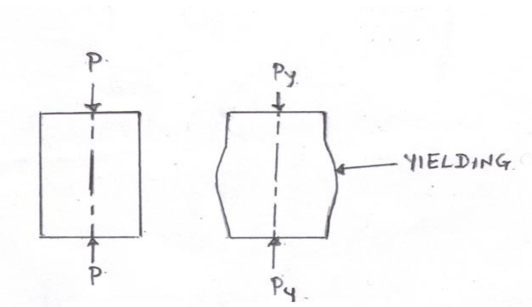
Fig. 2

CLASSIFICATION OF COLUMNS BASED ON THEIR LENGHTS AND THEIR BEHAVIOUR:

- **SHORT COLUMNS**

Short columns are very short a compression members. Slenderness ratio of such columns is very low.

The failure of such columns occurs by yielding and hence stresses at failure are yield stresses. No buckling is observed in such columns.



(a) (b)

Fig. 3

- **SLENDER OR LONG COMPRESSION MEMBERS**

The strength of any compression member decreases with the increase in its length.

Compression members with high slenderness ratio and which satisfy all the conditions of Euler's Formula for buckling are called slender or long compression members.

These columns will fail by elastic buckling. The stresses induced during failure (buckling) are well below the yield limit and lie in the elastic zone. So, the failure occurs elastically.

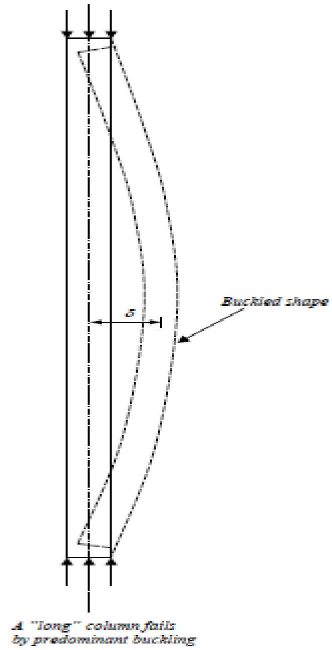


Fig. 4

- **INTERMEDIATE COLUMNS**

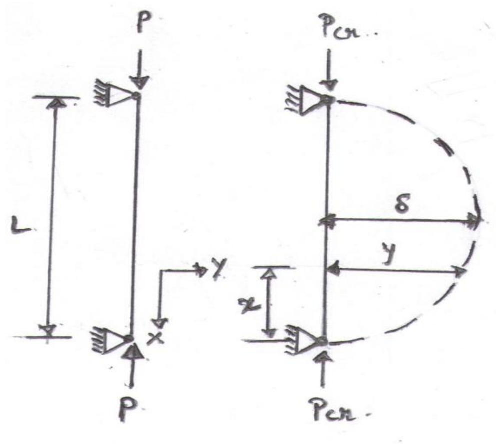
A column under an axial load has some fibers yielded some fibers in the elastic limit are known as intermediate columns.

These compression members would fail both by yielding and buckling. The failure would fall under the 'inelastic' category. Hence, Euler's formula is not applicable for such columns.

SLENDER COMPRESSION MEMBERS (ELASTIC BUCKLING):

The buckling of slender compression member or a column was first described by Euler. He was the first one to give remarks about the strength of a column. Euler considered an ideal column with the following properties-

- Material of the member is perfectly isotropic and homogeneous.
- Column has no imperfections.
- Column is pinned at both the ends.
- Column is initially straight and the load are acting concentrically.



(a) (b)

Fig. 5

Given in the above figure is a Euler column. The column will remain straight at load P. No deflection or buckling is observed then. But when the load reaches P_{cr} buckling of the column (about weaker axis) is observed. Such a load (when the column buckles) is termed as Critical Load or $P_{cr.}$

From figure (b), at any location at distance x, bending moment M is

$$M = P_{cr} * y$$

But, $$d^2y/dx^2 = - (M/EI)$$

So, $$-EI\ d^2y/dx^2 = P_{cr} * y$$

$$d^2y/dx^2 + (P_{cr}/EI)*y = 0$$

Now let $k^2 = P_{cr}/EI$

The solution of this differential equation is

$$y = A_1\sin kx + A_2\cos kx$$

where A_1 and A_2 are unknown co-efficient.

Applying boundary conditions,

1. At x=0 ; y=0

   $0 = A_1(0) + A_2(1)$

So, $A_2 = 0$

2. At x=L ; y=0

   $0 = A_1\sin kL$

The above equation is valid for $kL=n\pi$

The fundamental buckling mode considers or occurs at N = 1 (as both ends are pinned).

Hence, $$P_{cr} = \pi^2EI/L^2$$

The critical stress obtained at critical load will be

$$f_{cr} = P_{cr}/A_g = \pi^2E/\lambda \qquad (I=A_gr^2)$$

where L = effective length of the column(for pins at both ends)

r = radius of gyration of the axis about which buckling occurs.

A_g = gross cross section of the column.

(Note: For different end conditions, the value of effective length changes.)

For short columns, $f_{cr} = f_y$. They can be loaded in the strain hardening range and hence λ is replaced by λ_p.

Therefore, for short columns:

$$f_y=\pi^2E/\lambda_p^2$$

$$=> \lambda_p=\pi(E/f_y)^{1/2}$$

The strength curve for a column is shown below.

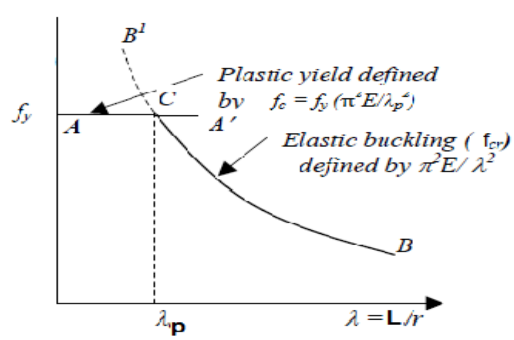


Fig. 6

This curve is obtained by the intersection of two curves i.e. Euler column buckling curve and the short column yield curve. The curve can also be defined in a non-dimensional form, where of λ is taken on X-axis and f_c/f_y on Y-axis.

$\overline{\lambda}$ = non dimensional slenderness ratio given by ($f_y/f_{cr})^{1/2}$

At point c, transformation of a column from short to slender is observed and the value of $\overline{\lambda} = 1$. This is also known as critical slenderness ratio.

STRENGTH OF COMPRESSION MEMBERS IN PRACTICE:

The highly idealized column cannot be achieved in actual practice. It was ignored because the test results did not agree with it. The column in actual practice tends to have initial crookedness, experience accidental eccentric loading, local or lateral buckling and may have residual stresses.

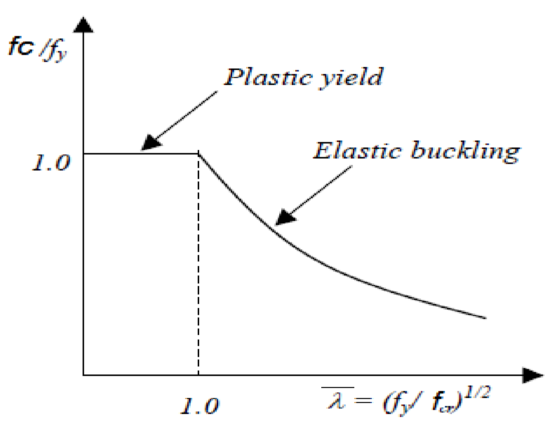
Due to these imperfections, the deflection curve of a real column will differ from the curve of an idealized column. Three main factors which result in reduction of the strength of the column are:

 # Initial Crookedness:-

Consider a pin ended member which consists of two rigid bars connected to each other by a linearly elastic torsion spring. The member is loaded with

compressive forces initially

When the force reaches P_{cr}, the member deflects as



shown on figure (Fig. 7

For equilibrium,

$P_{cr}\,\delta = 2K\theta$ (K is the spring constant)

⇨ $P_{cr}\, L\sin\theta/2 = 2K\theta$ (from fig. $\delta = L\sin\theta/2$)

Therefore,

$P_{cr} = 4K\theta/ L\sin\theta$ .... (1)

But when an initial crookedness is observed in the member such that $\theta = \theta_0$, P=0 and $\delta_0 = L\sin\theta_0/2$, on application of load P, a deflection of $\delta = L\sin\theta/2$

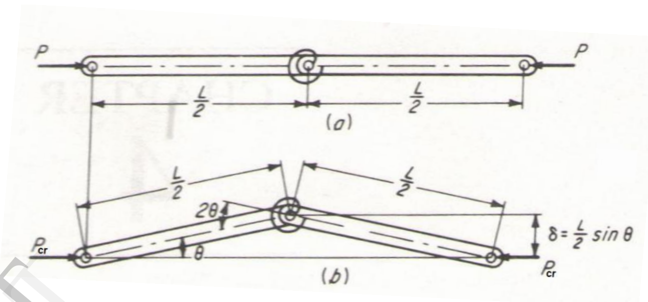


Fig. 8

would be achieved such that $\theta < \theta_0$.

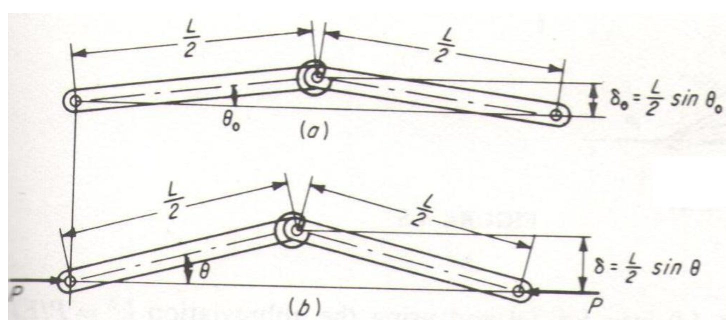


Fig. 9

Hence, $\delta < \delta_0$

For equilibrium,

$P\,\delta = 2K(\theta - \theta_0)$

$\Rightarrow P\, L\sin\theta/2 = 2K(\theta - \theta_0)$

$\Rightarrow P = 4K(\theta - \theta_0)/2\sin\theta$ ...... (2)

From equations (1) and (2);

$\theta = \theta_0 * (1/(1-P/P_{cr}))$.......... (3)

Now consider a column as shown in figure below,

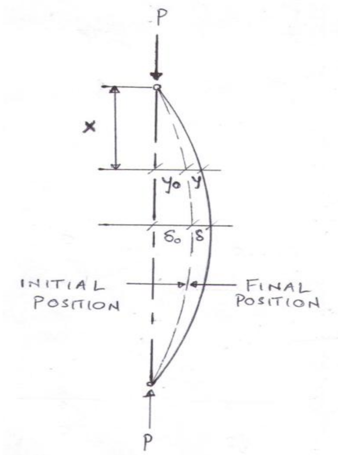


Fig. 10

If the deflection of the centroidal axis of the column after application of load is y, then

$$-EI(d^2y/dx^2) = P\ (y + y_0)$$

Applying boundary conditions y = 0 at x = 0 and y = 0 at x= L, the solution of equation:

$(d^2y/dx^2) + P\ (y + y_0)=0$ is

$y= \delta \sin(\pi x/L)$

where $\delta = \delta_0 * (1/(1-P/P_{cr}))$…….. (4)

(It is observed that eq^n. (3) and (4) are similar.)

Thus, the effect of axial thrust P would increase the amplitude of initial deflection by a factor $(1/(1-P/P_{cr}))$. This value is known as amplification factor.

EFFECT OF ECCENTRICITY OF APPLIED LOADING:

As discussed earlier, it is impossible to ensure that the load is applied at the exact centroidal axis of the column. The above figure shows a load P applied at an eccentricity e to the centroidal axis of the column which induces a bending moment (P * e) and causes lateral deflection of the column.

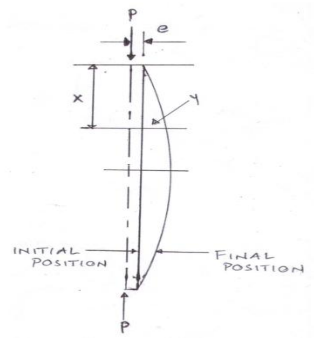


Fig. 11

EFFECT OF RESIDUAL STRESSES:

As a consequence of differential cooling of different parts of the member while forming process, residual stresses are formed in it.

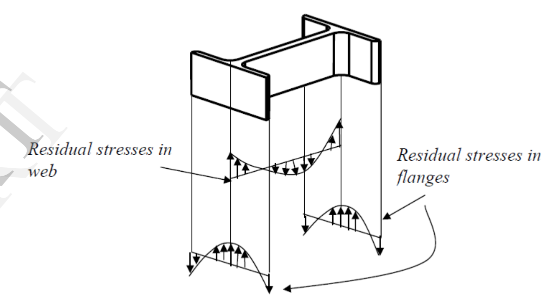


Fig. 12

So, when compressive forces are applied to the compression member with residual stresses, they add upto the residual stresses and may affect the strength of the member by phenomena like premature yielding (for short or stocky columns) or spontaneous buckling(for slender columns).

EFFECTS OF IMPERFECTIONS TAKEN TOGETHER (MULTIPLE COLUMN CURVES) ACCORDING TO IS:800

Fig shows a non-dimensional form of a strength curve of a Euler column. Considering all the imperfection factors encountered in a real compression member, the Indian Code(13800) has adopted the multiple column curves, as shown in the fig below.

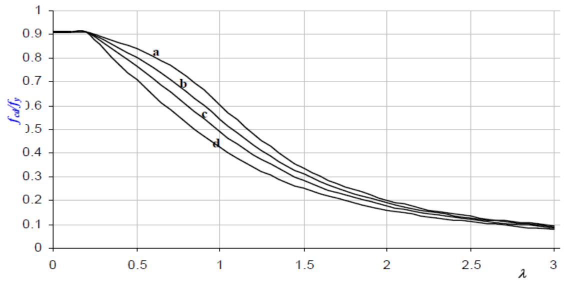


Fig. 13

The design compressive strength is given by

$$P_d = A_e f_c d$$

Where A_e = effective cross sectional area of the member.

f_{cd} = design compressive stress

The design stress f_{cd} is computed as:-

$$f_{cd} = (1/[\emptyset + (\emptyset^2 - \overline{\lambda^2})]^{1/2})\ (f_y/\gamma_{mo})$$

$$= \chi f_y/\gamma_{mo} <= f_y/\gamma_{mo}$$

Where, $\emptyset = 0.5(1 + \alpha\ (\lambda - \overline{0}.2) + \lambda^2\ \overline{)}$

$\overline{\lambda}$ = non-dimensional slenderness ratio = $(f_y/f_{cr})^{½}$

f_{cr} = Euler buckling stress = $\pi^2 E/(kL/r)^2$

KL/r is the effective slenderness ratio

α is the imperfection factor

X is the stress reduction factor.

γ_{mo} is partial safety factor = 1.10

On the basis of the value of α a particular buckling curve is selected.

Buckling Class	a	b	c	d
α	0.21	0.34	0.49	0.76

From the selected curve and the effective slenderness ratio, the value of design stress f_{cd} can be obtained.

STEPS FOR DESIGN OF AXIALLY LOADED COLUMNS:

(i) Assume a suitable trial section and classify the section in accordance with the classification as detailed in the Table (Limiting Width to Thickness Ratios) of IS: 800.

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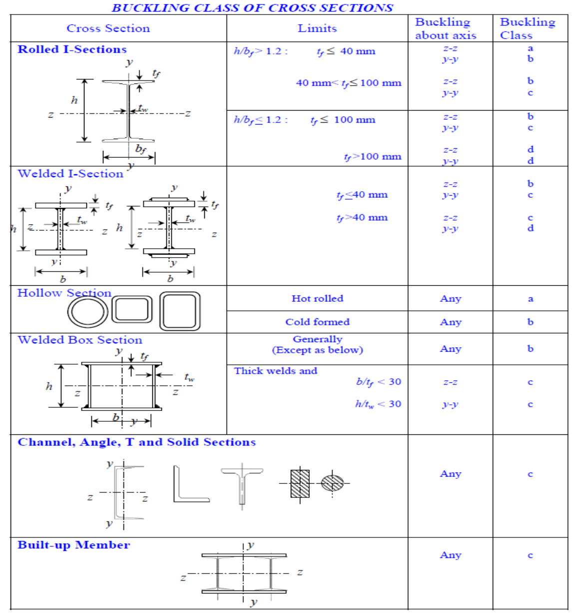


BUCKLING CLASS OF CROSS SECTIONS

Cross Section	Limits	Buckling about axis	Buckling Class
Rolled I-Sections	$h/b_f > 1.2$: $t_f \le 40$ mm	z-z y-y	a b
	40 mm< $t_f \le 100$ mm	z-z y-y	b c
	$h/b_f \le 1.2$: $t_f \le 100$ mm	z-z y-y	b c
	$t_f > 100$ mm	z-z y-y	d d
Welded I-Section	$t_f \le 40$ mm	z-z y-y	b c
	$t_f > 40$ mm	z-z y-y	c d
Hollow Section	Hot rolled	Any	a
	Cold formed	Any	b
Welded Box Section	Generally (Except as below)	Any	b
	Thick welds and $b/t_f < 30$	z-z	c
	$h/t_w < 30$	y-y	c
Channel, Angle, T and Solid Sections		Any	c
Built-up Member		Any	c

Fig. 14

(note: The section can be classified into 4 categories:

1. Plastic
2. Compact
3. Semi compact
4. Slender

In actual practice, totally plastic or slender sections are not available. But, if the section is more on the plastic side, the failure would occur by yielding of the section while it is on the slender side, failure would occur by buckling in the elastic zone.

Hence, sections more on the side of plastic limit have a better strength than the ones on the slender or elastic side.)

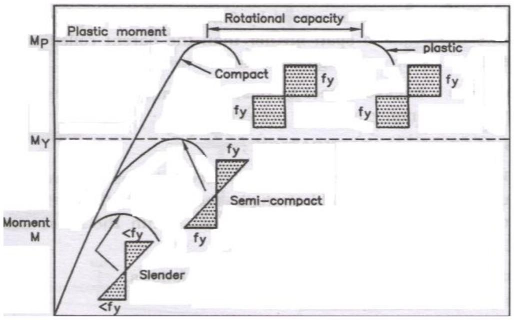


Fig. 15

(If section is slender then apply appropriate corrections.)

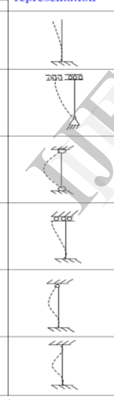
(ii) Calculate effective sectional area, A_e as defined in Clause 7.3.2 of IS: 800

(iii) Calculate effective slenderness ratio, *KL/r*, ratio of effective length *KL*, to appropriate radius of gyration, *r*

(Note: values of r for a particular section are given in the steel table.)

EFFECTIVE LENGTH OF PRISMATIC COMPRESSION ME

Boundary Conditions				Schematic representation
At one end		At the other end		
Translation	Rotation	Translation	Rotation	
Restrained	Restrained	Free	Free	
Free	Restrained	Restrained	Free	
Restrained	Free	Restrained	Free	
Restrained	Restrained	Free	Restrained	
Restrained	Restrained	Restrained	Free	
Restrained	Restrained	Restrained	Restrained	



Note – L is the unsupported length of the compression member

Fig. 16

(iv) Calculate λ from the equation, λ = non-dimensional effective slenderness ratio= $\sqrt{f_y / f_{cc}} = \sqrt{f_y \left(KL/r\right)^2 / \pi^2 E}$

(v) Calculate φ from the equation, $\varphi = 0.5[1+\alpha(\lambda - 0.2)+ \lambda^2]$ Where, α = Imperfection factors for various Column Buckling Curves a, b, c and d are given in the following Table: (of IS: 800)

IMPERFECTION FACTOR, α

Buckling Class	**a**	**b**	**c**	**d**
α	0.21	0.34	0.49	0.76

(vi) Calculate χ from equation $\sqrt{f_y / f_{cc}} = \sqrt{f_y \left(KL/r\right)^2 / \pi^2 E}$

(vii) Choose appropriate value of Partial safety factor for material strength, γ_{m0} from Table 5.2 of IS: 800

(viii) Calculate design stress in compression, f_{cd} , as per the following equation (Clause **7.1.2.1** of IS: 800):

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + \left[\phi^2 - \lambda^2\right]^{0.5}} = \chi f_y / \gamma_{m0} \leq f_y / \gamma_{m0}$$

(ix) Compute the load P_d, that the compression member can resist $P_d = A_e f_{cd}$

(x) Calculate the factored applied load and check whether the column is safe against the given loading. The most economical section can be arrived at by trial and error, i.e. repeating the above process.

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2. Design of Steel Structures by Edwin h. Gaylord Jr., Charles N. Gaylord, James E. Stallmeyer
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