Steel Work design (1) to BS 5950- 1:2000

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References

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➢ Way, A. G. J. , Salter, P. R. (2003). “**Introduction to steelwork design to BS 5950-1:2000**”, *the steel construction institute, SCI*, UK.

➢ Case, J., Chilver, L., Ross, C.T.F. (1999). “**Strength of materials and structures**”, *John Willy & Sons Inc.*, fourth edition, London.

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➢ محمد أحمد السمارة (2003). “**تصميم المنشآت الفولاذية**”، الطبعة الثانية، دمشق.

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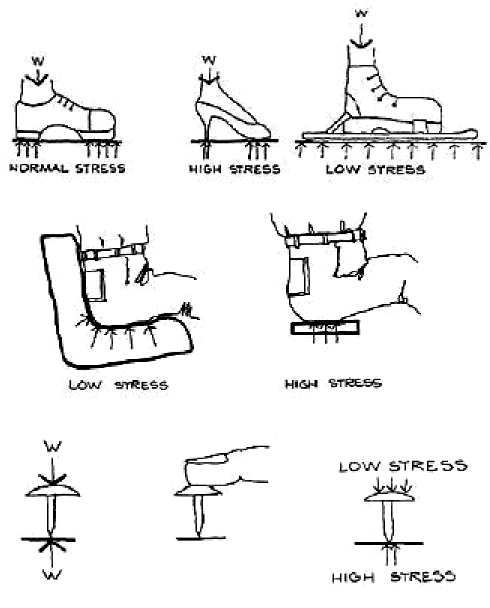
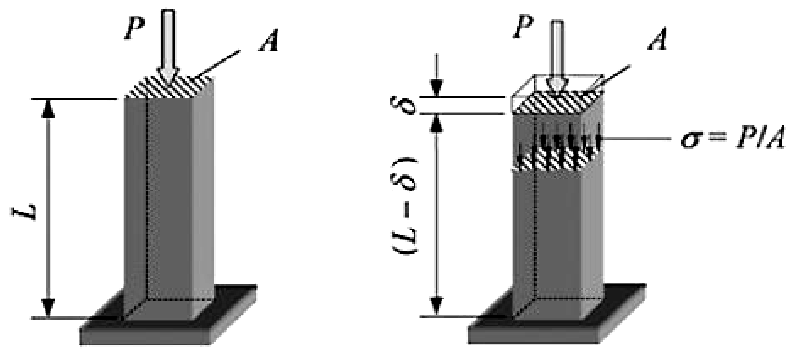
Design of Compression Members
تصميم عناصر الضغط

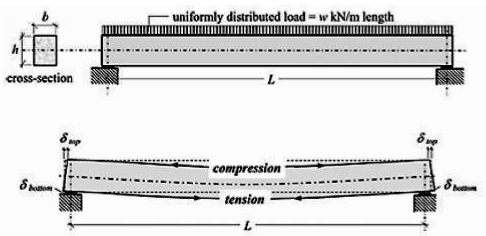
(The concept of stress) مفهوم الإجهاد



(Axial stresses) الإجهادات المحورية

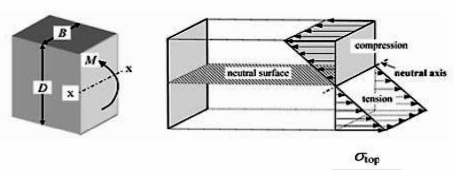


(Bending stresses) إجهادات الانعطاف



$$\sigma_{top} = \frac{M}{I} y_{top} = \frac{M}{Z_t} : Z_t = \frac{I}{y_{top}}$$

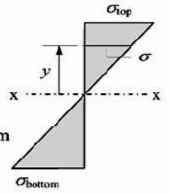
$$\sigma_{bottom} = \frac{M}{I} y_{bottom} = \frac{M}{Z_b} : Z_b = \frac{I}{y_{bottom}}$$



Z : Elastic Section Modulus

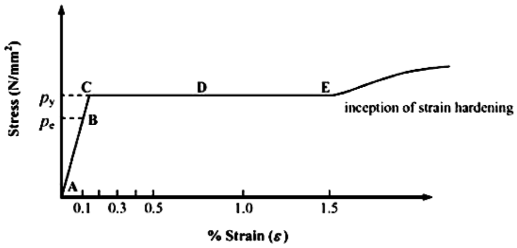
$$Z = \frac{I}{y_{\max}}$$

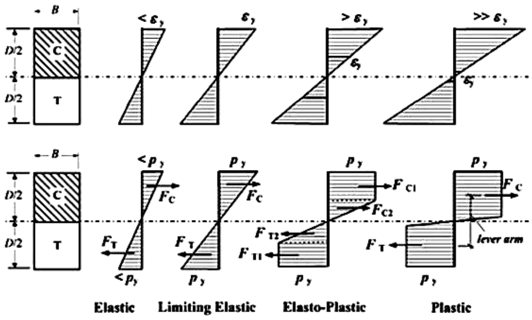
$$\sigma = \frac{M}{I} y$$



Bending Stress Diagram

Plastic Section Modulus, S





$F_c = F_T$

$A_c \cdot p_y = A_T p_y$ » $A_c = A_T$

$M_p = p_y S$

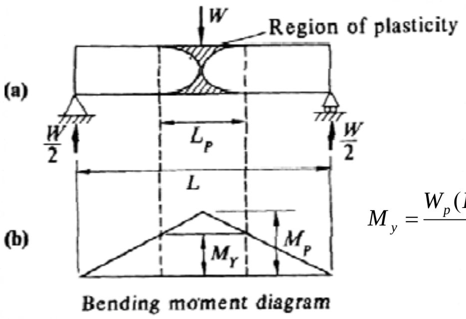
$$S = \frac{1}{2} A(y_c + y_T)$$

S: Plastic Modulus

$$Shape\ Factor\ f = \frac{S}{Z}$$

High Shape Factor → Early Yielding → Permanent Deformation

(The concept of plastic hinge) مفهوم المفصل اللدن



$$M_p = \frac{W_p L}{4} \Rightarrow W_p = \frac{4M_p}{L}$$

$$M_y = \frac{W_p(L - L_p)}{4} \Rightarrow W_p = \frac{4M_y}{(L - L_p)} = \frac{4M_p}{L} \Rightarrow L_p = L(1 - \frac{M_y}{M_p})$$

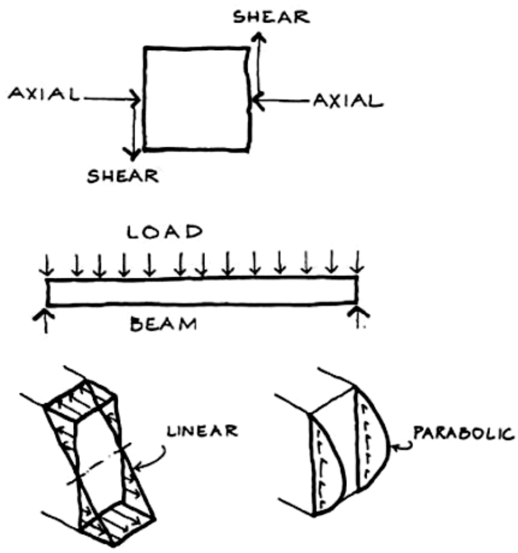
$$L_p = L(1 - \frac{1}{f}) : f = \frac{S}{Z}$$

Examples

For ***rectangular*** section $f = 1.5 \Rightarrow L_p = \frac{1}{3} L$

For ***I*** section of ***f=1.13*** $f = 1.13 \Rightarrow L_p = 0.12L$

إجهادات القص (Shear stresses)



$$\tau = \frac{V \times s}{I \times b}$$

$$\tau = \frac{M_T}{J} r$$

S=First moment of area

J=Torsion constant.

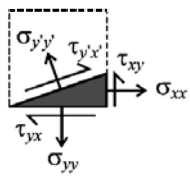
Final stresses

Normal stress ⇨ $\sigma = \pm \frac{N}{A} \pm \frac{M_x}{I_x} y \pm \frac{M_y}{I_y} x$

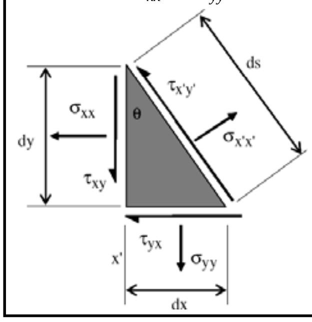
Shear stress ⇨ τ

⇩

Principal stresses



$$\tan 2\theta = \frac{2\tau_{xy}}{\sigma_{xx} - \sigma_{yy}}$$



$$\sigma_{x'x'} = \frac{(\sigma_{xx} + \sigma_{yy})}{2} + \frac{(\sigma_{xx} - \sigma_{yy})}{2}\cos 2\theta + \tau_{xy}\sin 2\theta$$

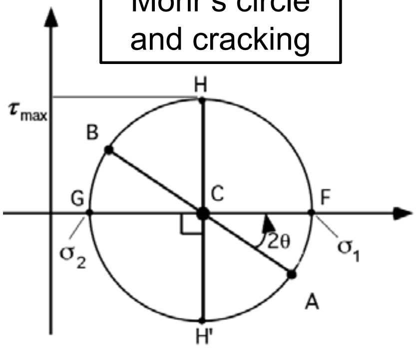
$$\sigma_{y'y'} = \frac{(\sigma_{xx} + \sigma_{yy})}{2} - \frac{(\sigma_{xx} - \sigma_{yy})}{2}\cos 2\theta - \tau_{xy}\sin 2\theta$$

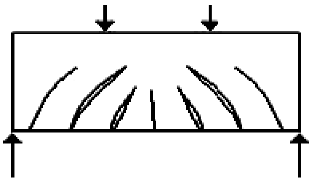
$$\tau_{x'y'} = -\frac{(\sigma_{xx} - \sigma_{yy})}{2}\sin 2\theta + \tau_{xy}\cos 2\theta$$

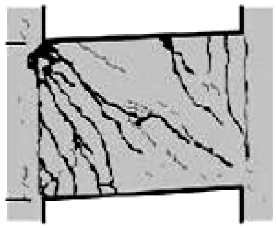
where θ is an anticlockwise rotation.

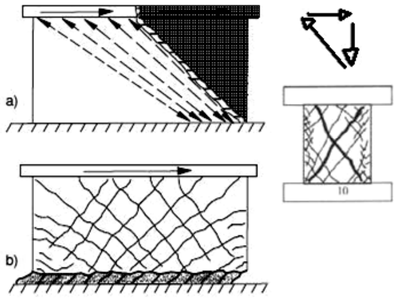
Principal stresses and the failure

Mohr's circle
and cracking





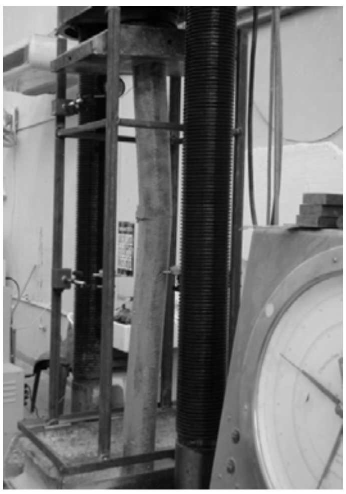


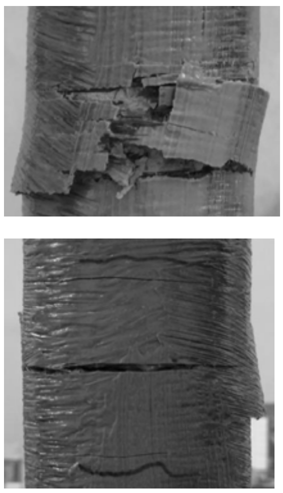


Crack is expected when the principal stresses have reached a critical strength

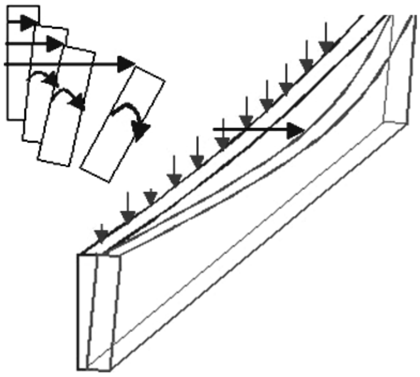
(The concept of Buckling) مفهوم التحنيب

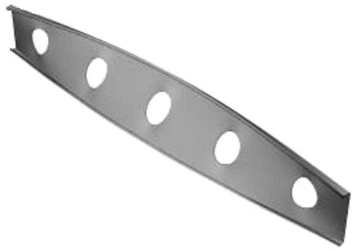
Buckling of columns



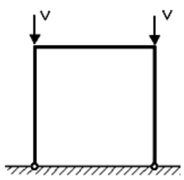


Lateral Torsional buckling of beams

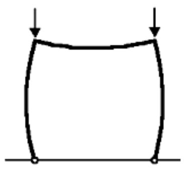




Buckling of Frames



(a) Partial and idealised loading for buckling analysis

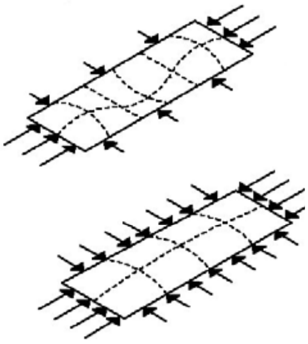


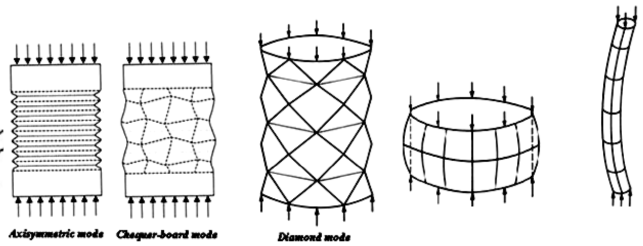
(b) Symmetrical (non-sway) mode of buckling

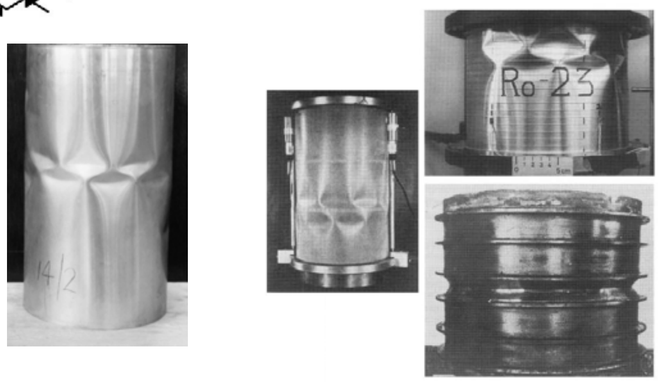
(c) Antisymmetrical (sway) mode of buckling

Local Buckling of yielding

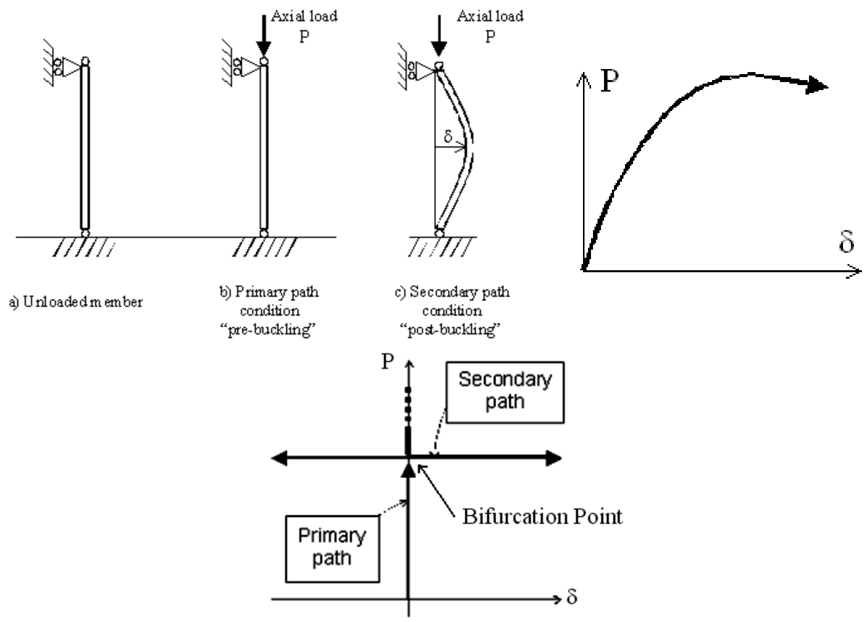
Buckling of plates and shells



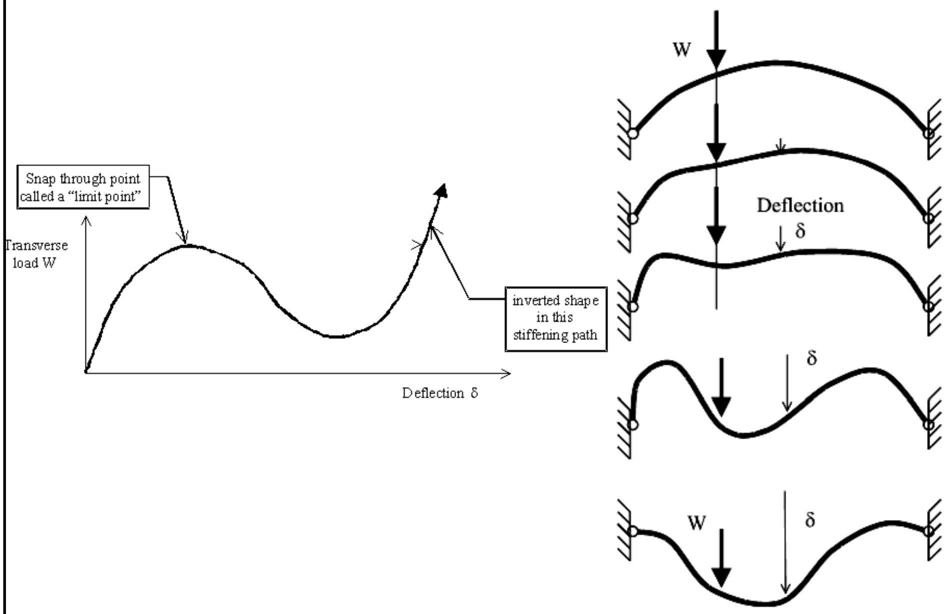




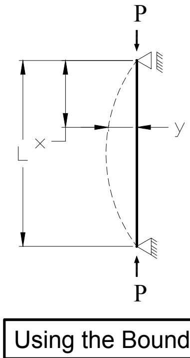
Concept of Bifurcation Buckling



Concept of Snap-Through Buckling



(Buckling of an Euler strut) تحنيب أويلر



$$M = P.y$$

$$y'' = -\frac{M}{EI} \Rightarrow y'' + \frac{P}{EI} y = 0 \Rightarrow y'' + K^2 y = 0 : K^2 = \frac{P}{EI}$$

General solution for the deflected shape ⇨ $y = A\cos kx + B\sin kx$

Using the Boundary Conditions

$$x = 0 \Rightarrow y = 0 \Rightarrow A = 0 \Rightarrow y = B\sin kx$$

$$x = L \Rightarrow y = 0 \Rightarrow B\sin kL = 0$$

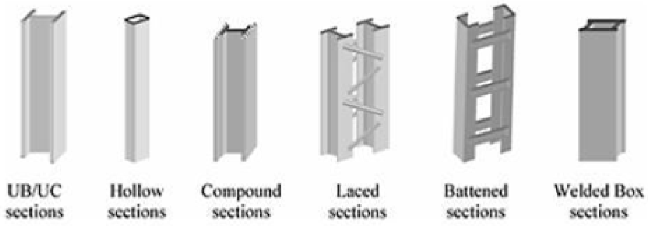
If KL≠0 » B=0 » always y=0 »No Buckling » **wrong assumption** » KL=0 or KL=nπ

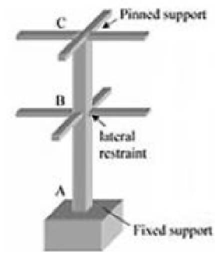
$$kL = 0 \Rightarrow k = 0 \Rightarrow Always\ \ y = 0 \Rightarrow kL = n\pi \Rightarrow k^2L^2 = n^2\pi^2 \Rightarrow \frac{n^2\pi^2}{L^2} = \frac{P}{EI} \Rightarrow P_E = \frac{n^2\pi^2 EI}{L^2}$$

$$For\ smallest\ \ load\,(Critical\ \ load) \Rightarrow n = 1 \Rightarrow P_E = \frac{\pi^2 EI}{L^2}$$

The concept of restraints مفهوم القيود الجانبية

Column types أنواع الأعمدة





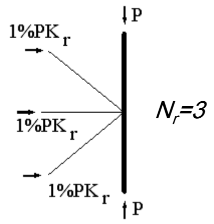
Horizontal Ties القيود الجانبية (مقاومة قوة شد مصعدة 2.4.5.2&2.4.5.3) Figure 1 & Figure 2

$$N_{ut} = \max[\,0.5q_u(\,factored\ \ vertical\ \ load\ \ on\ \ Tie\,), 75\,kN\,,\ 1\%\,N_{uc}\,(compressive\ \ force\ \ of\ \ edge\ \ column\,)\,]$$

4.7.1.2 → Compressed $N_{restraint}$=1% $N_{compression\ member}$

4.7.3-a → $M_u > 90\% M_r : M_r = p_y S_r$

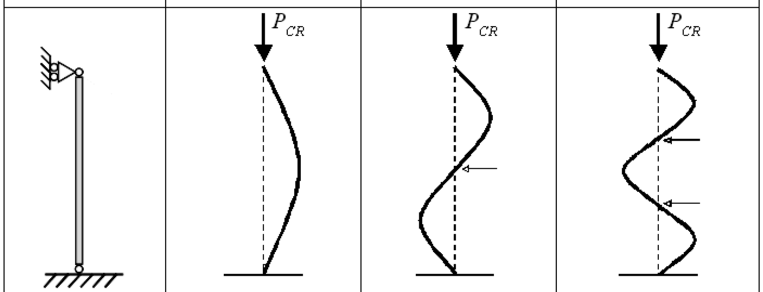
No directional restraint لاقدرة لمقاومة الدوران



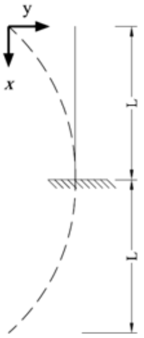
$$k_r = (0.2 + 1/N_r)^{0.5}$$

Critical buckling load of different deflection modes

$$P_{cr} = \frac{n^2\pi^2 EI}{L^2}$$

$n=0$	$n=1$	$n=2$	$n=3$
			
$(\lambda=\infty)$	$\lambda = L$	$\lambda=\frac{L}{2}$	$\lambda=\frac{L}{3}$
$v=0$	$v = A\sin\pi\frac{z}{L}$	$v = A\sin 2\pi\frac{z}{L}$	$v = A\sin 3\pi\frac{z}{L}$
$(P_{CR}=0)$	$P_{CR}=\frac{\pi^2 EI}{L^2}$	$P_{CR}=4\frac{\pi^2 EI}{L^2}$	$P_{CR}=9\frac{\pi^2 EI}{L^2}$

Columns under other boundary conditions and the concept of the effective length



$$M = P.y$$

$$y'' = -\frac{M}{EI} \Rightarrow y'' + \frac{P}{EI} y = 0 \Rightarrow y'' + K^2 y = 0 : K^2 = \frac{P}{EI}$$

General solution for the deflected shape ⇨ $y = A\cos kx + B\sin kx$

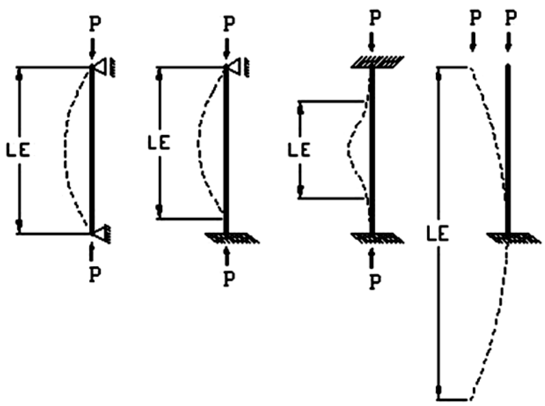
$$x = 0 \Rightarrow y = 0 \Rightarrow A = 0 \Rightarrow y = B\sin kx$$

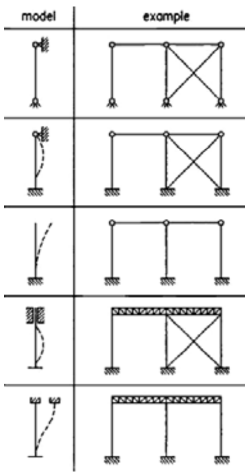
$$x = l \Rightarrow \frac{dy}{dx} = 0 \Rightarrow BK\cos kL = 0$$

$$B \neq 0, k \neq 0 \Rightarrow \cos kL = 0 \Rightarrow kL = n\frac{\pi}{2} \Rightarrow k^2L^2 = n^2\frac{\pi^2}{4} \Rightarrow \frac{n^2\pi^2}{4L^2} = \frac{P}{EI} \Rightarrow P_{cr} = \frac{n^2\pi^2 EI}{(2L)^2}$$

Note: The critical buckling load of a cantilever length ***L*** is as the critical load of simply-supported ends of ***2L***

The Effective Length, L_E





L_E *Theoretically*	**1.L**	**0.7L**	**0.5L**	**2L**
L_E *Practically*	**1.L**	**0.85L**	**0.7L**	**2L**

$L_E = K_e . L$

$$P_{cr} = \frac{n^2\pi^2 EI}{L_E^{\ 2}}$$

Major and Minor axis of buckling

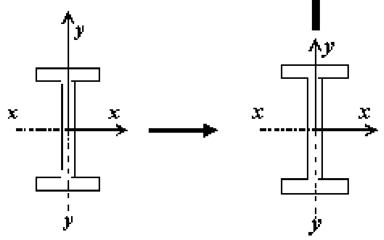
$$P_{cr} = \frac{n^2\pi^2 EI}{L_E^{\;2}} \Rightarrow n=1 \Rightarrow P_{cr} = \frac{\pi^2 EI}{L_E^{\;2}} \Rightarrow \sigma_{cr} = \frac{\pi^2 EI}{L_E^2 A} = \frac{\pi^2 E r^2}{L_E^2} = \frac{\pi^2 E}{\lambda^2}$$

$$\sigma_{cr} = \frac{\pi^2 E}{\lambda^2} : \; Slenderness\,\lambda = \frac{L_E}{r}$$

$$r_y < r_x \Rightarrow I_y < I_x$$

⇩

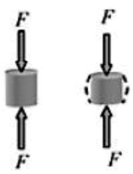
y is the ***minor axis***
x is the ***major axis***



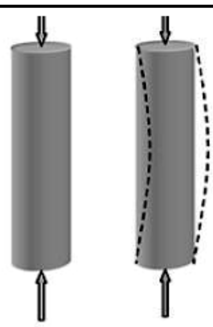
Buckling about ***y-axis*** is **more critical than** buckling *about* ***x-axis*** for the same length because the smallest radii of gyration is about y

(Buckling of a perfect column) تحنيب عمود لايحتوي تشوهات بدائية

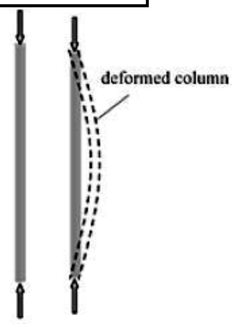
Short Element

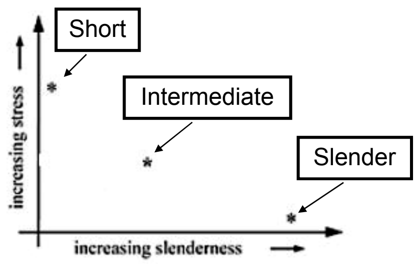


Intermediate Element



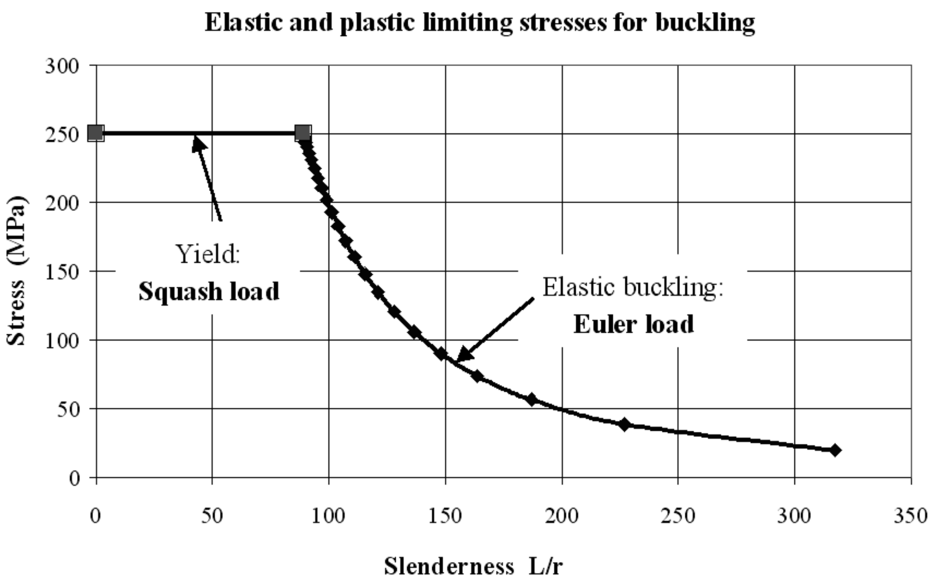
Slender Element





$$Slenderness = \lambda = \frac{L_E}{r}$$

$$Euler \quad stress, \sigma_E = \frac{\pi^2 E}{\lambda^2}$$



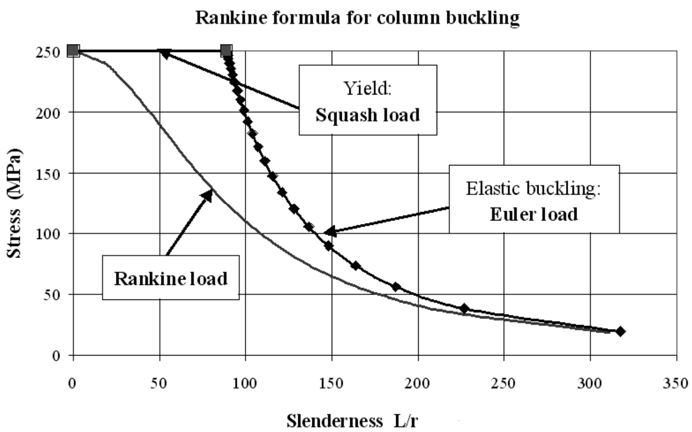
Empirical buckling of a perfect column

Rankine formula is the simplest safe empirical formula from test data

$$\frac{1}{P_F} = \frac{1}{P_E} + \frac{1}{P_Y}$$

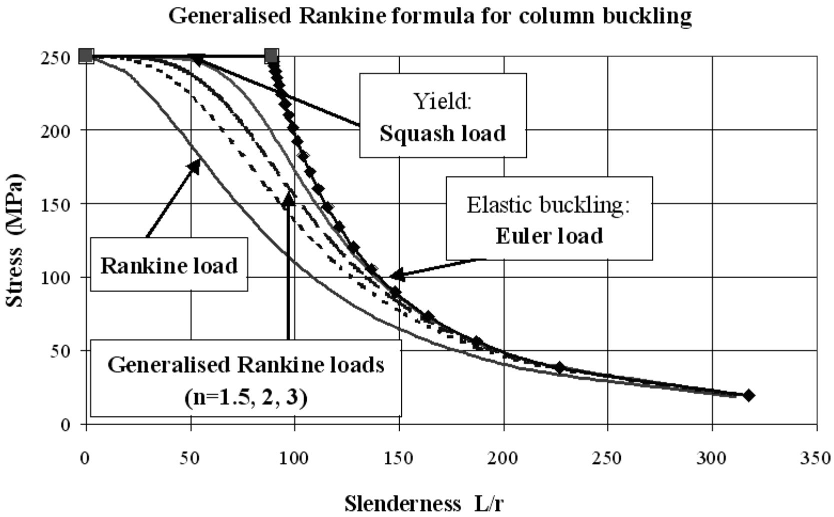
P_F = the **real** buckling failure strength
P_E = the ideal **Euler** buckling load
P_Y = the squash load

$$\sigma_F = \frac{\sigma_E \, \sigma_Y}{\sigma_E + \sigma_Y}$$

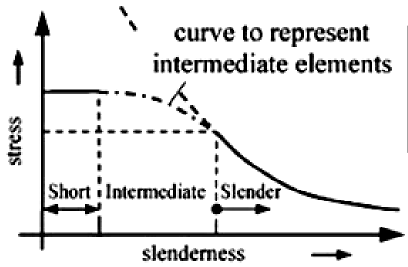


For less conservative treatments

$$\frac{1}{\sigma_F^{\ n}} = \frac{1}{\sigma_E^{\ n}} + \frac{1}{\sigma_Y^{\ n}} \quad or\ \sigma_F = \frac{\sigma_E\,\sigma_Y}{\sqrt[n]{\sigma_E^{\ n} + \sigma_Y^{\ n}}} : 1 \le n \le 3$$

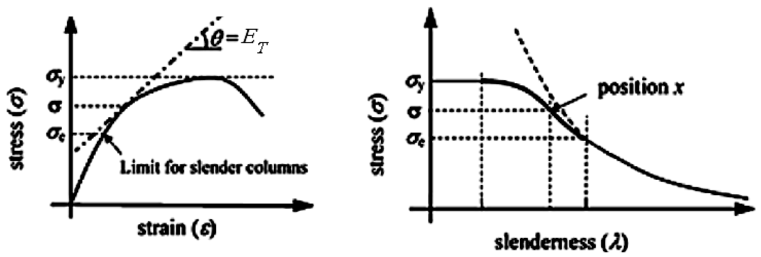


The effect of material non-linearity on buckling load

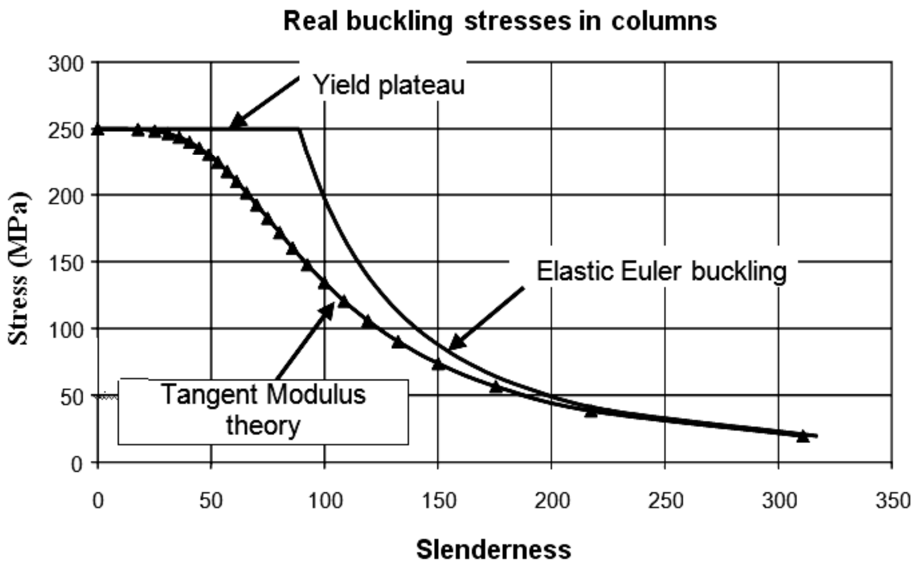


The **non-linearity of material** causes the drop in results between Euler theory and experiment data for intermediate columns

Tangent modulus theory is the simple safe estimate of buckling strength in **Elastic-Plastic region**

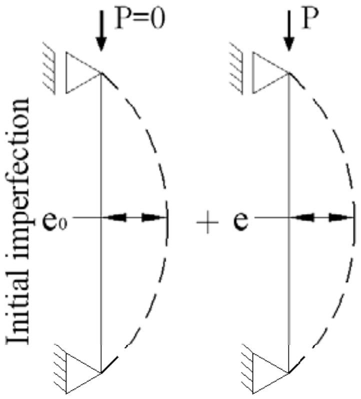


$$\sigma = \frac{\pi^2 E_T}{\lambda^2} \Rightarrow Modified\ slenderness\ at\ position\ x = \lambda = \frac{L_{cr}}{r} = \pi\sqrt{\frac{E_T}{\sigma}}$$



(Buckling of a imperfect column) تحنيب عمود يحوي على تشوهات بدائية

Perry Formula (1886)

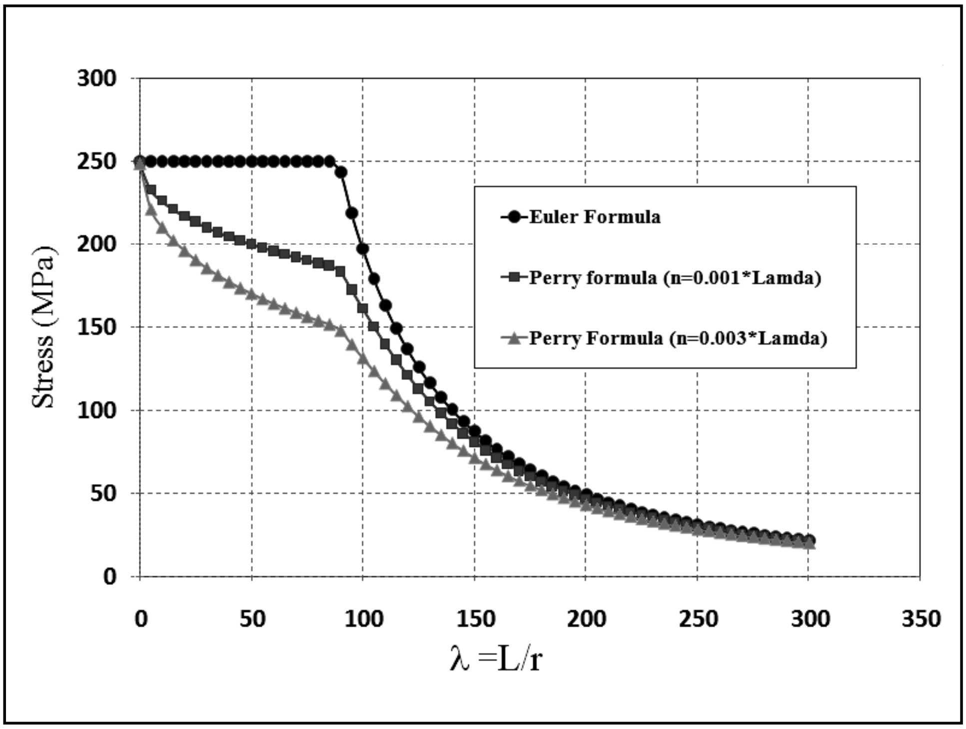


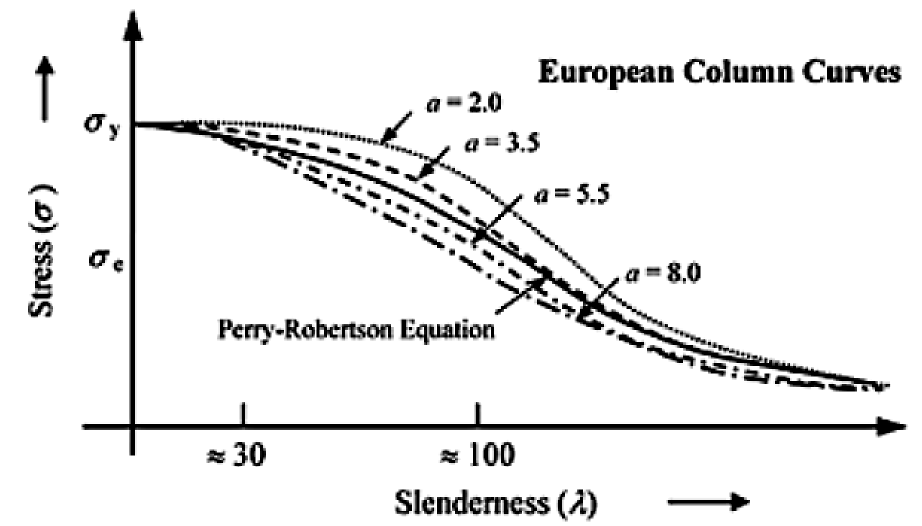
Perry Formula (BS 5950-1:2000) Annex C

$$(\sigma_E-\sigma_c)(\sigma_y-\sigma_c)=\eta\sigma_E\sigma_c : \eta=\frac{e_0 z}{r^2}$$

$$\sigma_c=\frac{\sigma_E\sigma_y}{\phi+\sqrt{\phi^2-\sigma_E\sigma_y}} : \phi=\frac{\sigma_y+(\eta+1)\sigma_E}{2}, \sigma_E=\frac{\pi^2 E}{\lambda^2}$$

z: the distance of the extreme fiber from the neutral axis of buckling.
r : Radii of gyration

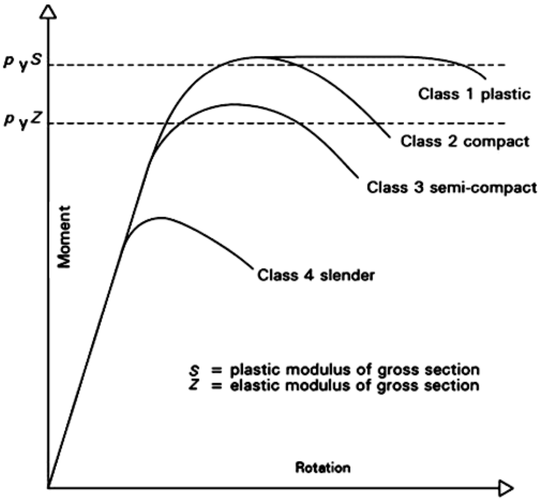


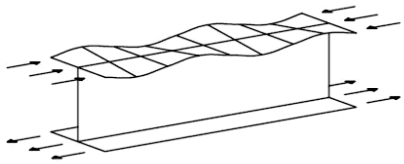


$$Perry\ Factor\ ,\eta = a(\lambda - \lambda_0)/1000 \geq 0 : a\ is\ Robertson\ \text{constant}$$

λ_0 is the limiting slenderness (short column)= $0.2(\pi^2 E/p_y)^{0.5}$

(Classification of sections) تصنيف المقاطع
(3.5.2)





Local Buckling تحنيب موضعي

- Element geometry أبعاد العنصر(السماكة/ الطول)
- Stress distribution توزع الإجهادات
- Support conditions شروط الاستناد
- Yield strength اجهاد السيلان

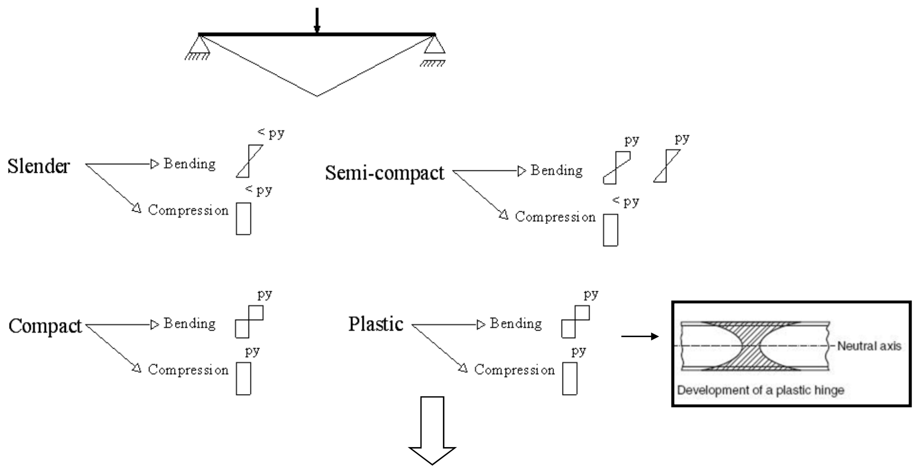


Table 11&12

- Element Geometry أبعاد المقطع : b/T, d/t (Figure 5 & 6)
- Stress distribution توزع الإجهادات :r_1, r_2 (Section 3.5.5)
- Yield strength إجهاد السيلان: ε
- Element Type نوع العنصر : Outstand element (External خارجي), Internal element (عنصر داخلي) (3.5.1)

Example 1:

S275, UB 457×152×52, Bending moment only about the major axis عزم انعطاف فقط حول المحور الرئيسي (القوي)

UB 457×152×52» t=7.6, T=10.9 <16mm » Table9 » p_y=275Mpa » $\varepsilon = \sqrt{\frac{275}{275}} = 1$

b/T=6.99 < 9ε » Class1 (Plastic)

d/t=53.6 < 80ε » Class1 (Plastic)

} Plastic section مقطع لدن

Example 2:

S275, UB 457×152×52, Bending moment + Axial compression force 800kN

b/T=6.99 < 9ε » Class1 (Plastic)

$$r_1 = \frac{F_c}{dtp_{yw}} = \frac{800\times10^3}{407.6\times7.6\times275} = 0.94 \text{ » } \frac{80\varepsilon}{1+r_1} = \frac{80}{1+0.94} = 41 < \frac{d}{t}$$

$$\frac{100\varepsilon}{1+1.5r_1} = \frac{100}{1+1.5\times0.94} = 41.5 < \frac{d}{t}$$

$$r_2 = \frac{F_c}{A_g p_{yw}} = \frac{800\times10^3}{6660\times275} = 0.44 \text{ » } \frac{120\varepsilon}{1+2r_2} = \frac{120}{1+2\times0.44} = 63.8 > \frac{d}{t}$$ » Semi-Compact

Example 3:

S355,HF RHS 250×150×5, Bending moment only about the major axis عزم انعطاف فقط حول المحور الرئيسي (القوي)

HF RHS 250×150×5» t=5<16mm » Table9 » p_y=355Mpa » $\varepsilon = \sqrt{\frac{275}{355}} = 0.88$

b/T=27 >28ε=25

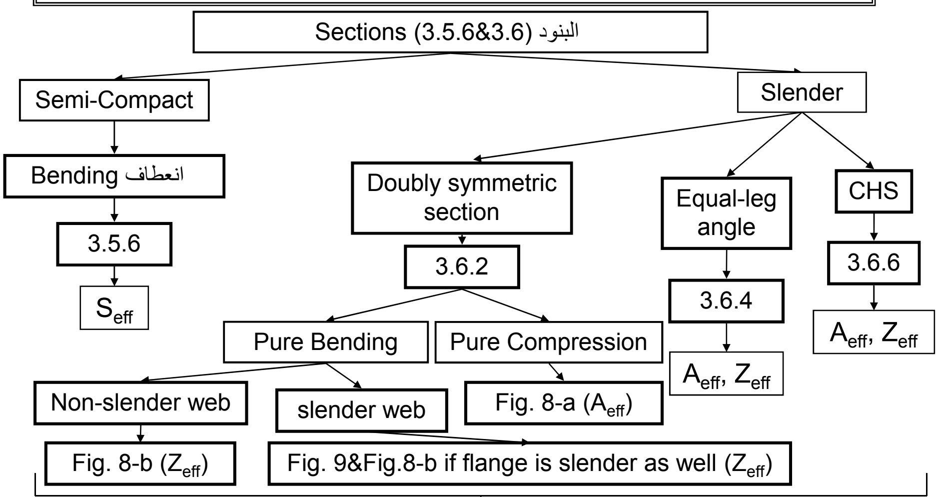
b/T<32ε=28 & b/T<62ε-0.5d/t=54.5-0.5×47=31 » Class2 (Compact)

d/t=47<64ε=56 » Class1 (plastic)

⇓

Compact section مقطع مكتنز

Effective section properties المواصفات الهندسية الفعالة للمقطع

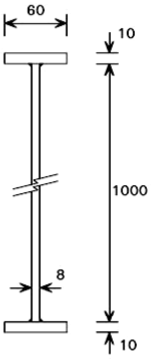


Alternative method (طريقة مغايرة) for slender section (3.6.5): $p_{yr} = (\frac{\beta_3}{\beta})^2 p_y$

Notice: Compression + Bending » Compression only (A_{eff})+ Bending only (Z_{eff})

Example:

S275, Welded section مقطع ملحوم, pure Bending عزم انعطاف فقط, Plastic flange, slender web



Properties

d/t = 125

p_y = 275 N/mm²

ε = 1.0

I_x = 97270 cm⁴

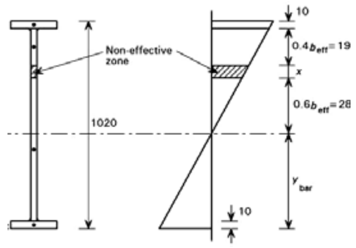
Z_x = 1907 cm³

Solution:

$f_{cw}=f_{tw}$ » $b_{eff}=60\varepsilon t=60\times1\times8=480mm$

$0.4\times b_{eff}=192$, $0.6b_{eff}=288$

Try x=40mm» First moment=$\sum \pm a_i y_i \neq 0$

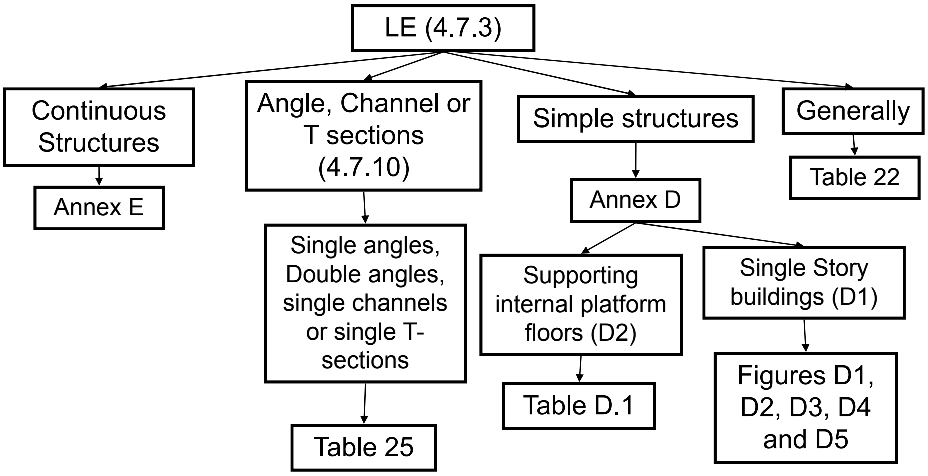


Try x=28mm» First moment=$\sum \pm a_i y_i \approx 0$

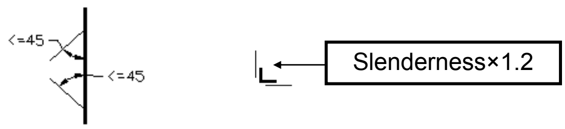
$I_x=95285cm^4$, $y_{max}=52cm$

$Z_{eff}=95285/52=1832cm^3$

Effective length to BS 5950-1:2000



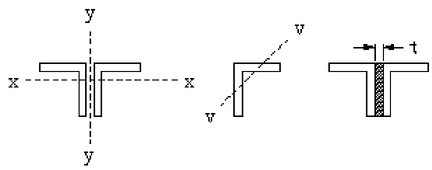
Notes: (4.7.10.1)



Conditions اشتراطات (4.7.9&4.7.13)

4.7.9

$$\begin{cases} \lambda_c = \dfrac{L_{vv}}{r_{vv}} \le 50 \\ \lambda_b = \sqrt{\lambda_m^2 + \lambda_c^2} \ge 1.4\lambda_c : \lambda_m = \dfrac{L_{Eyy}}{r_{yy}} \\ \lambda_{max} = \dfrac{L_E}{r} \le 180 \end{cases}$$



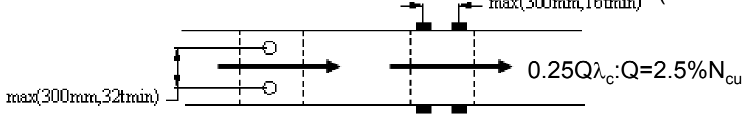
4.7.13

❖يتم تقسيم العنصر إلى ثلاثة أقسام متساوية تقريباً بواسطة وصلات وسطية (4.7.13.1.e). وعند النهايات يجب أن يوجد برغيين على الأقل باتجاه طول العنصر. ولايجب أن يوجد برغي واحد على أحد صفوف البراغي.

❖لاتقل براغي الوصل الوسطي عن 16mm في أي حال من الأحوال(4.7.13.1.f).

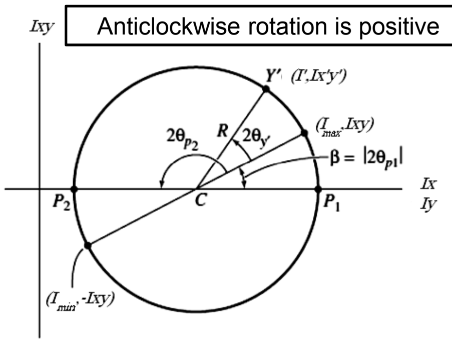
❖في الوصلة الوسطية بواسطة براغ يتم وضع برغيين بشكل عمودي على طول العنصر(4.7.13.1.g). والمسافة بين البرغيين لاتقل عن الأصغر بين 300mm أو 32t حيث t أصغر سماكة من الأجزاء المتصلة (4.7.13.2.b.1).

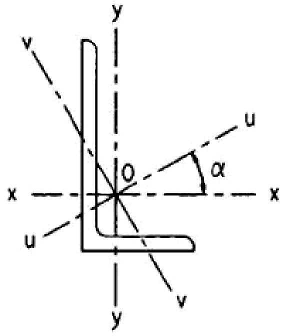
❖في الوصلة الوسطية بواسطة لحام، يجب أن يتم اللحام من الأعلى والأسفل للعناصر المتصلة مرتين على امتداد العنصر بمسافة لاتزيد بين مركزي اللحام عن 300mm أو 16t حيث t أصغر سماكة (4.7.13.2.b.2).



Mohr's Circle

Anticlockwise rotation is positive





$$\tan 2\alpha = \frac{-2I_{xy}}{I_{xx} - I_{yy}}$$

In Eqs, I_{xy} is always to be used as positive. The clockwise rotation is taken

$$I_{uu} = \frac{I_{xx} + I_{yy}}{2} + \frac{I_{xx} - I_{yy}}{2}\cos 2\alpha - I_{xy}\sin 2\alpha$$

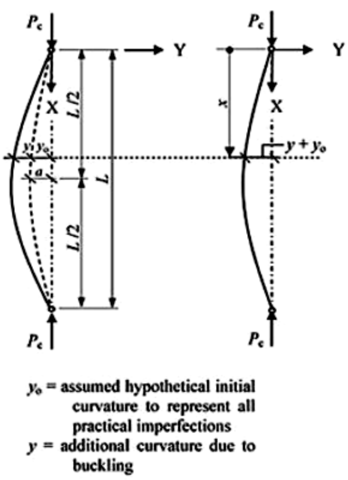
$$I_{vv} = \frac{I_{xx} + I_{yy}}{2} - \frac{I_{xx} - I_{yy}}{2}\cos 2\alpha + I_{xy}\sin 2\alpha$$

Compressive strength, p_c (4.7.5)
Perry and Robertson formula (Annex C)

The formula was developed by an assumption that practical imperfections may exist

هذه المعادلة تم تطويرها بافتراض وجود تشوهات مبدئية ناتجة عن الصنع قبل تطبيق الحمولة

Note (4.7.5): For **Welded** section in compression only p_y= p_y(table9)-20



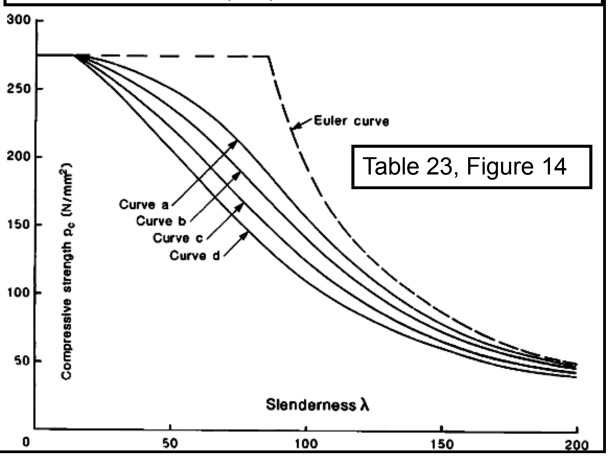
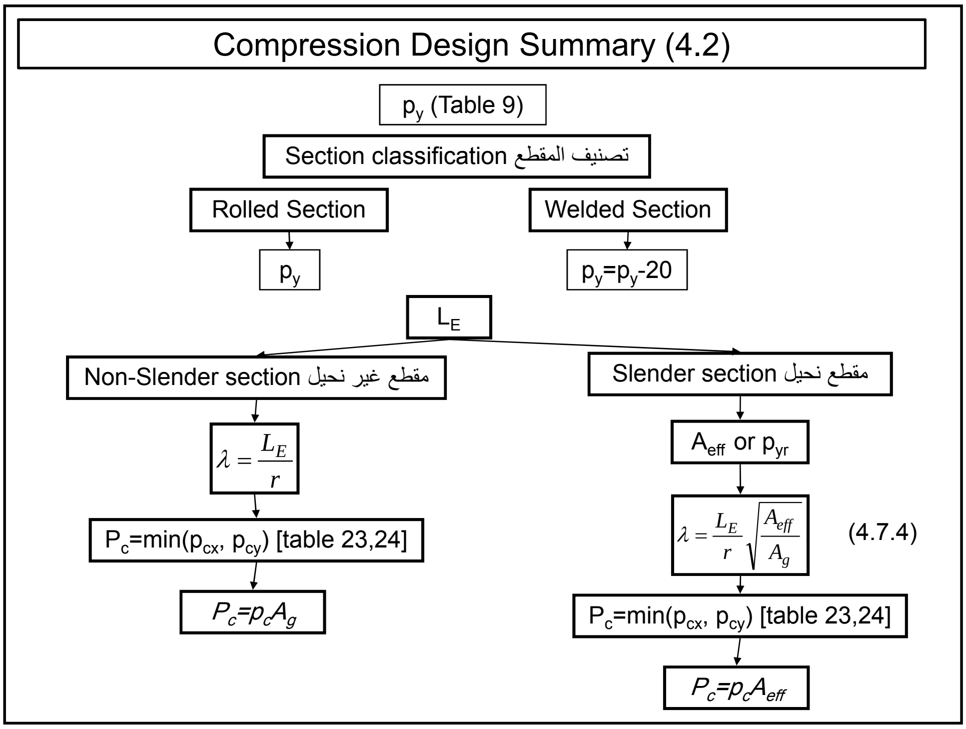


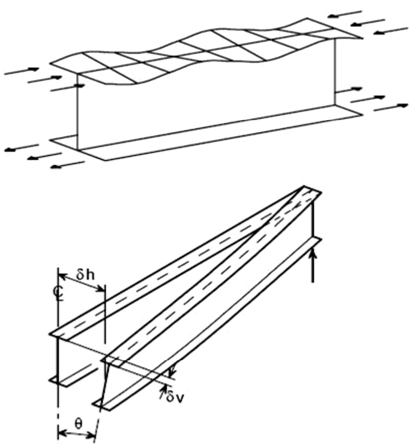
Table 23, Figure 14



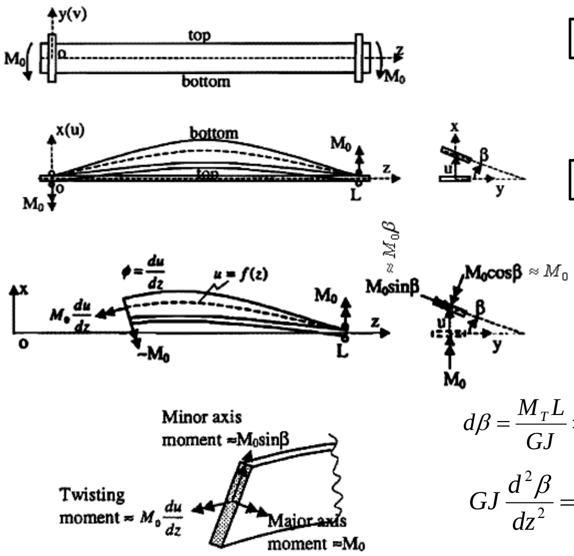
Design of Fully Restrained Beams
تصميم الجوائز المقيدة بشكل كلي

Lateral Torsional Buckling of beam

Lateral Torsional Buckling of Beams



Lateral torsional buckling (تحنيب الفتل الجانبي) =
Lateral deflection (إزاحة جانبية) + Twisting (فتل)



For bending about x axis

$$M_0 = -EI_x \frac{d^2v}{dz^2}$$

For bending about y axis

$$M_0\beta = -EI_y \frac{d^2u}{dz^2}$$

From Torsion

$$d\beta = \frac{M_T L}{GJ} \Rightarrow d\beta = \frac{M_0 \frac{du}{dz} dz}{GJ} \Rightarrow GJ \frac{d\beta}{dz} = M_0 \frac{du}{dz}$$

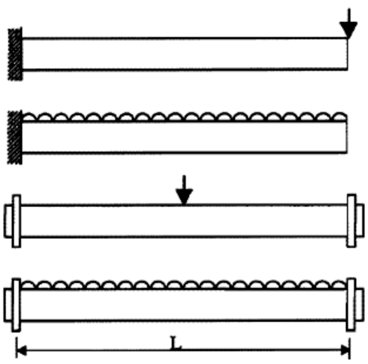
$$GJ \frac{d^2\beta}{dz^2} = M_0 \frac{d^2u}{dz^2} \Rightarrow \frac{d^2\beta}{dz^2} + \frac{M_0^2}{GJEI_y}\beta = 0$$

$$\beta = A\cos kz + B\sin kz : k^2 = \frac{M_0^2}{GJEI_y}$$

$$z = 0 \Rightarrow \beta = 0 \Rightarrow A = 0, z = L \Rightarrow \beta = 0 \Rightarrow kL = \pi \Rightarrow \boxed{M_{o,cr} = \frac{\pi}{L}\sqrt{GJEI_y}}$$

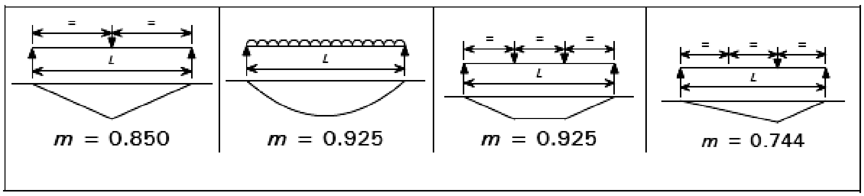
Other load cases

1. $M_{max,cr} = \frac{4.01}{\pi} M_{0,cr}$
2. $M_{max,cr} = \frac{6.42}{\pi} M_{0,cr}$
3. $M_{max,cr} = \frac{4.23}{\pi} M_{0,cr}$
4. $M_{max,cr} = \frac{3.5}{\pi} M_{0,cr}$

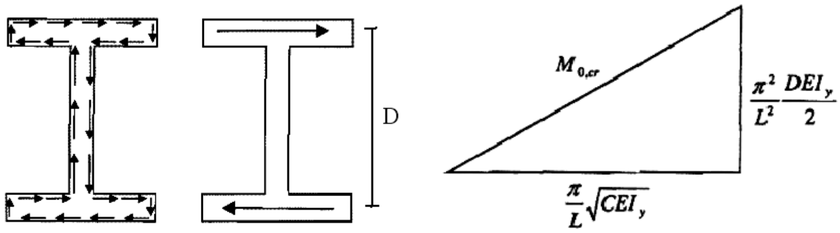


$$\boxed{M_{max,cr} = \frac{1}{m} . M_{0,cr}}$$

In BS 5950-1:2000 for steelwork design



Lateral Torsional Buckling of an I beam

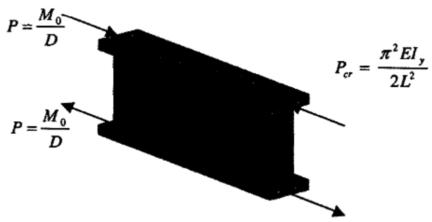


$$M_{0,cr} = \sqrt{\left(\frac{\pi}{L}\sqrt{CEI_y}\right)^2 + \left(\frac{\pi^2}{L^2}\frac{DEI_y}{2}\right)^2}$$

$$C = GJ = G\sum\frac{b_i t_i^3}{3}$$

Buckling of flange

$$P_{cr} = \frac{\pi^2 E(\frac{I_y}{2})}{L^2} = \frac{\pi^2 EI_{y,\,flange}}{L^2} = P_{Euler\,,\,flange}$$



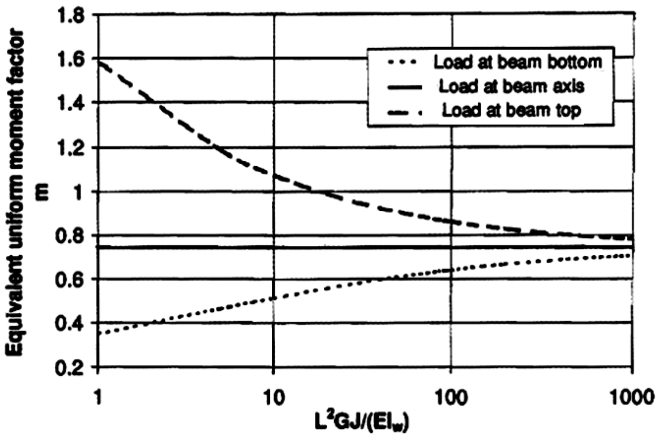
Second term of critical moment

The effect of load level

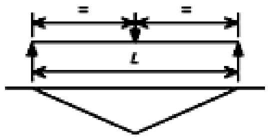
$$M_{\max,cr} = \frac{1}{m}.M_{0,cr}$$

m is dependent on the ratio L^2GJ/EI_w

$$I_w = I_y\frac{D^2}{4}$$



Example for concentrated load at mid span



Fully restrained beams الجوائز المقيدة كلياً ضد الفتل الجانبي

4.2.2 & 4.3.2.2

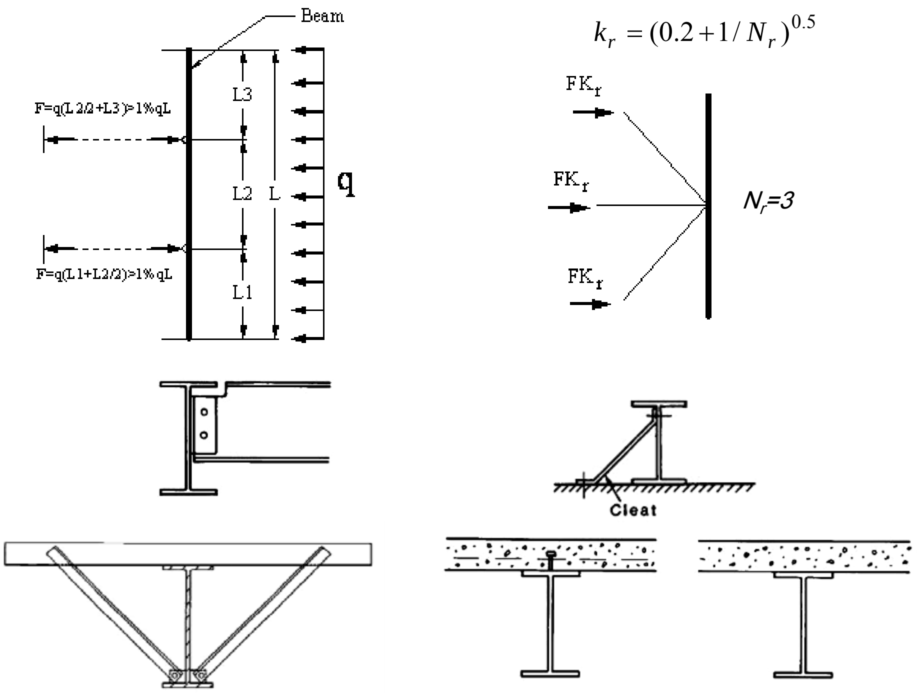
↓

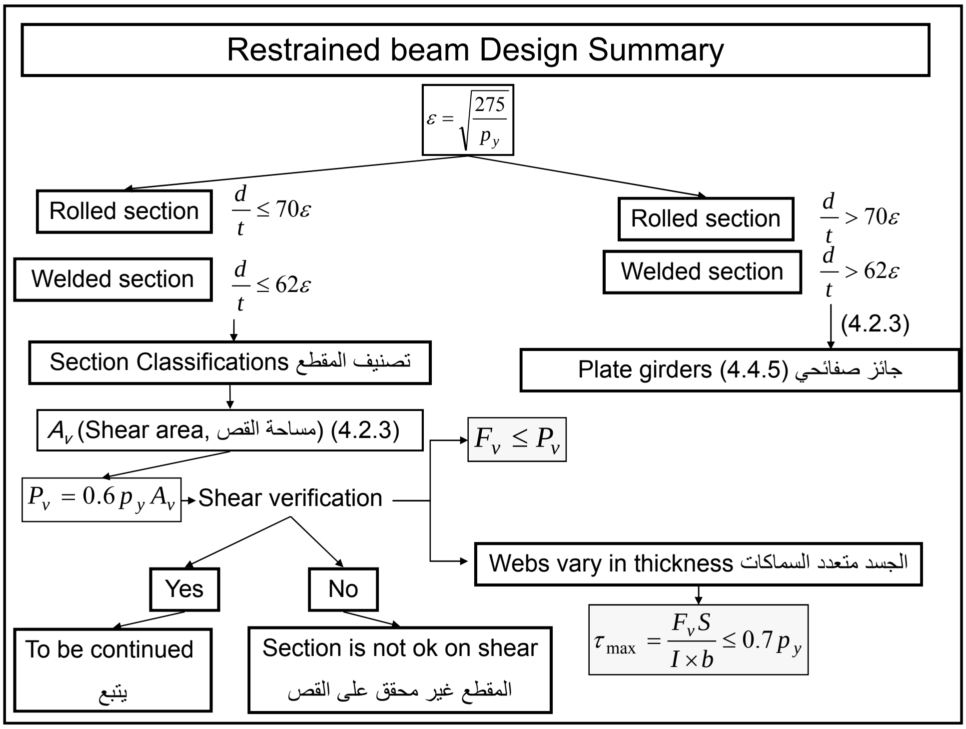
The restraint should resist a lateral force more than: ***2.5% F_{fc}***

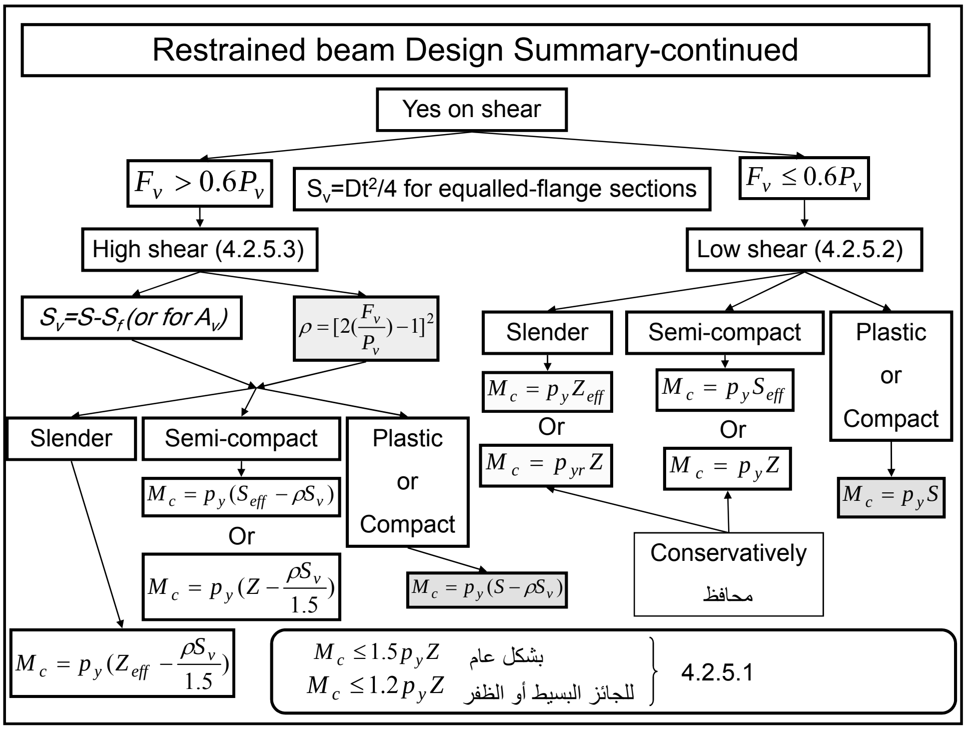
القيد الجانبي يجب أن يقاوم قوة ضغط جانبية أكبر من 2.5% من قوة الضغط في الجناح

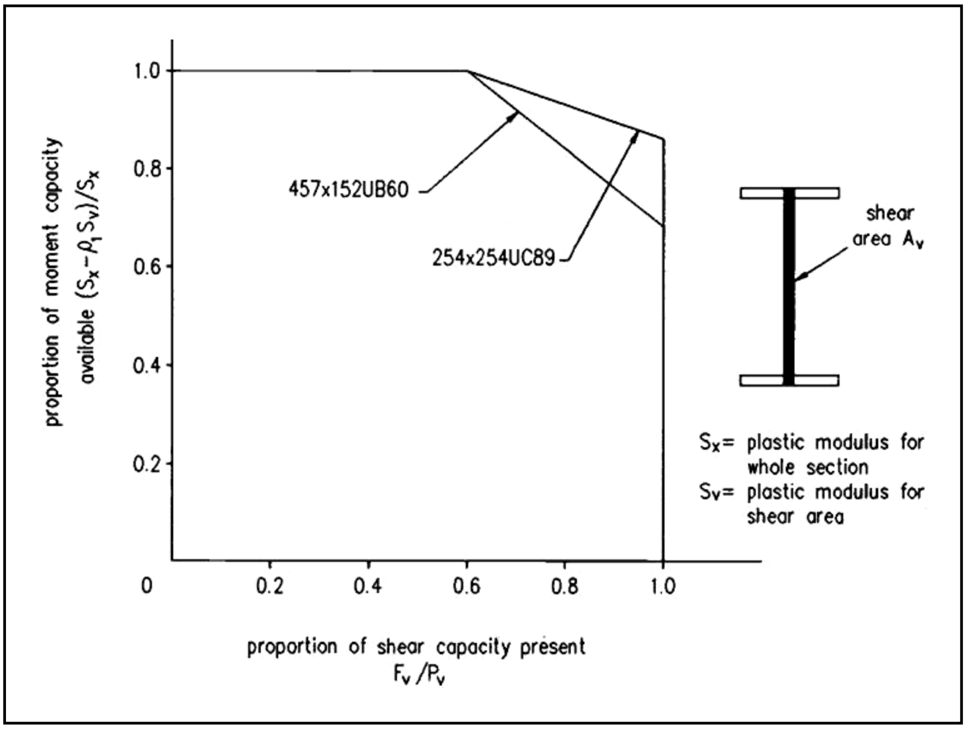
Force in compression flange F_{fc}	Frictional force
القوة في جناح الضغط	قوى احتكاك
$F_{fc}=M_{umax}/D$	q_1=2.5%(Load × coefficient of friction)/L
q_2=2.5% × F_{fc}/L →	$q=q_1+q_2$

$$k_r = (0.2 + 1/N_r)^{0.5}$$

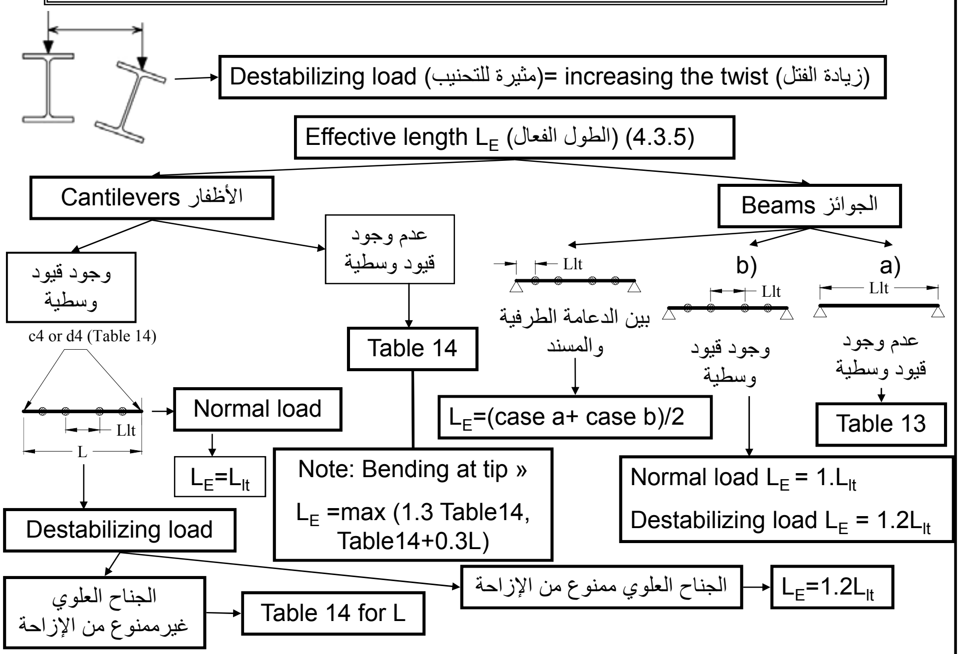


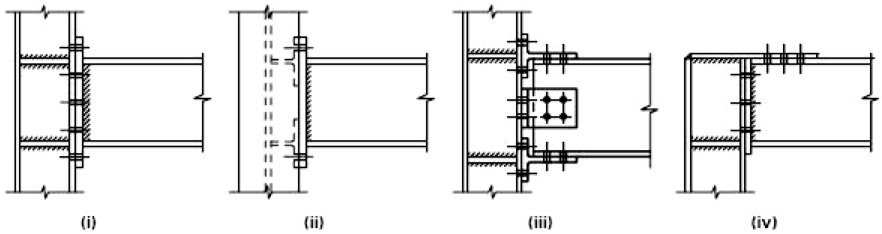

Restrained beam Design Summary
$\varepsilon = \sqrt{\frac{275}{p_y}}$
Rolled section $\frac{d}{t} \le 70\varepsilon$
Welded section $\frac{d}{t} \le 62\varepsilon$
Rolled section $\frac{d}{t} > 70\varepsilon$
Welded section $\frac{d}{t} > 62\varepsilon$
(4.2.3)
Section Classifications تصنيف المقطع
Plate girders (4.4.5) جائز صفائحي
A_v (Shear area, مساحة القص) (4.2.3)
$F_v \le P_v$
$P_v = 0.6 p_y A_v$
Shear verification
Webs vary in thickness الجسد متعدد السماكات
$\tau_{max} = \frac{F_v S}{I \times b} \le 0.7 p_y$
Yes
No
To be continued
يتبع
Section is not ok on shear
المقطع غير محقق على القص


Restrained beam Design Summary-continued
Yes on shear
$F_v > 0.6P_v$
S_v=Dt²/4 for equalled-flange sections
$F_v \le 0.6P_v$
High shear (4.2.5.3)
Low shear (4.2.5.2)
S_v=S-S_f (or for A_v)
$\rho = [2(\frac{F_v}{P_v}) - 1]^2$
Slender
Semi-compact
Plastic or Compact
$M_c = p_y Z_{eff}$
Or
$M_c = p_{yr} Z$
$M_c = p_y S_{eff}$
Or
$M_c = p_y Z$
$M_c = p_y S$
Conservatively
محافظ
Slender
Semi-compact
Plastic or Compact
$M_c = p_y (S_{eff} - \rho S_v)$
Or
$M_c = p_y (Z - \frac{\rho S_v}{1.5})$
$M_c = p_y (S - \rho S_v)$
$M_c = p_y (Z_{eff} - \frac{\rho S_v}{1.5})$
$M_c \le 1.5 p_y Z$ بشكل عام
$M_c \le 1.2 p_y Z$ للجائز البسيط أو الظفر
4.2.5.1

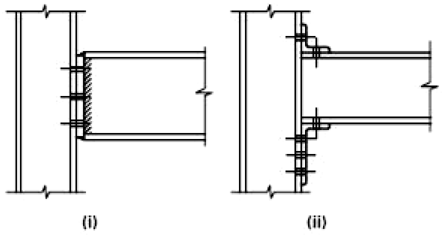


unrestrained beams الجوائز غير المقيدة ضد الفتل الجانبي (4.3)

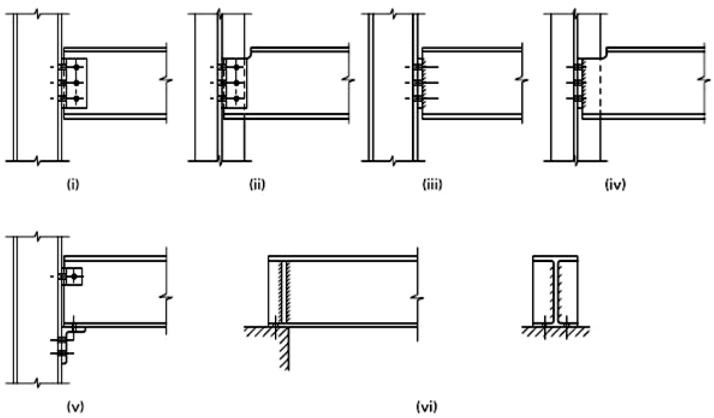




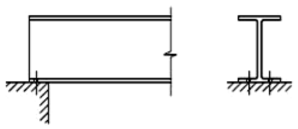
a) Both flanges fully restrained against rotation on plan



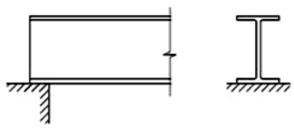
b) Both flanges partially restrained against rotation on plan



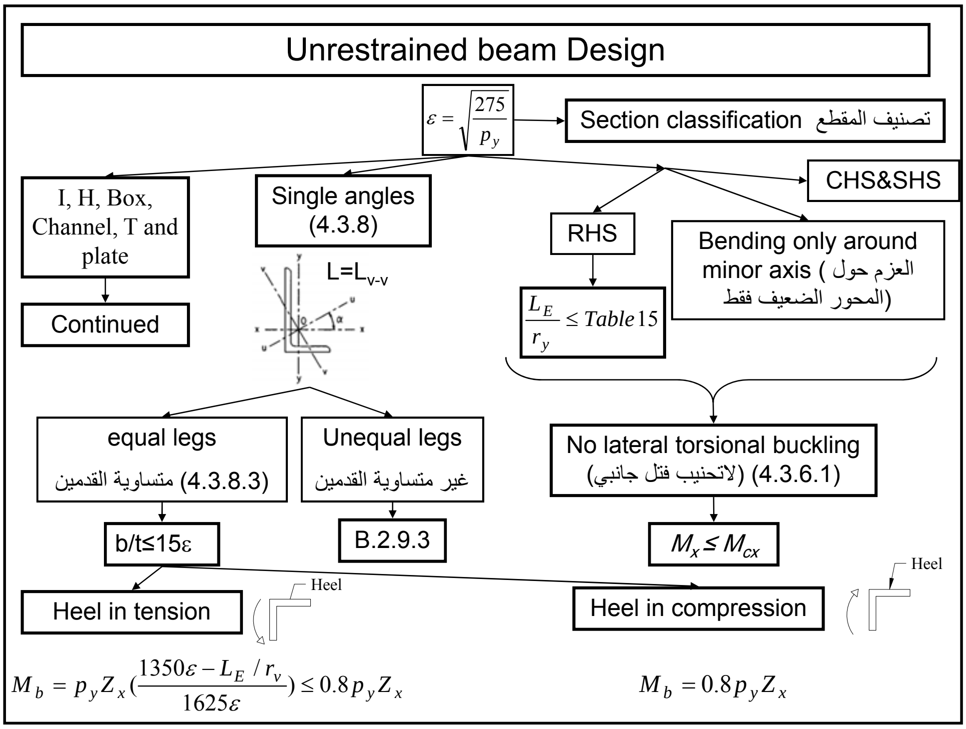
c) Both flanges free to rotate on plan

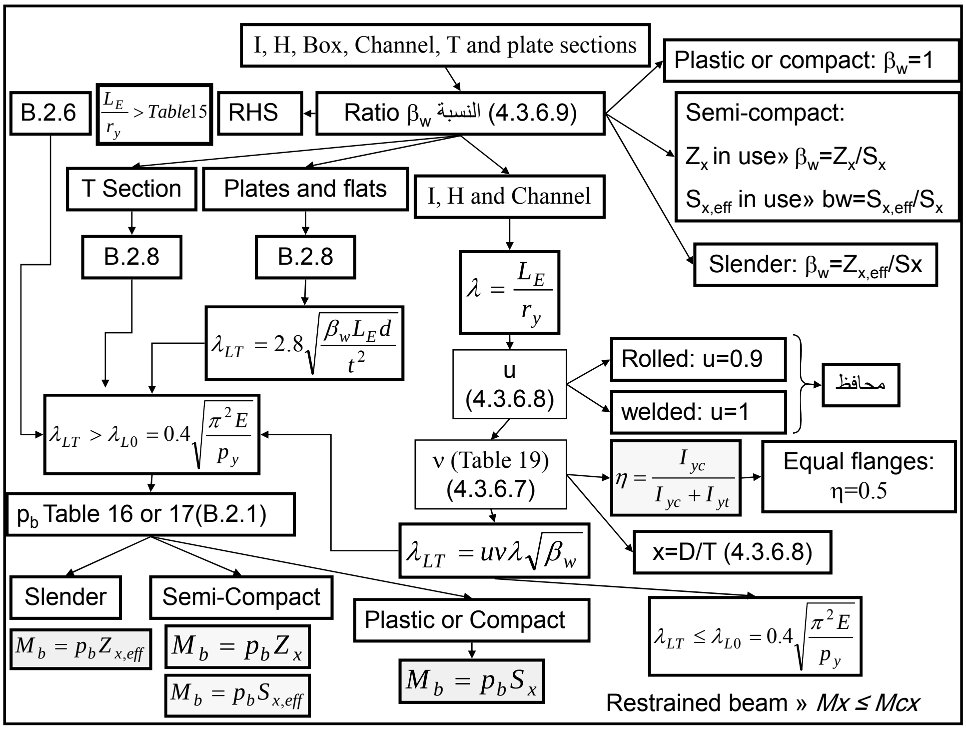


d) Partial torsional restraint against rotation about longitudinal axis provided by connection of bottom flange to supports

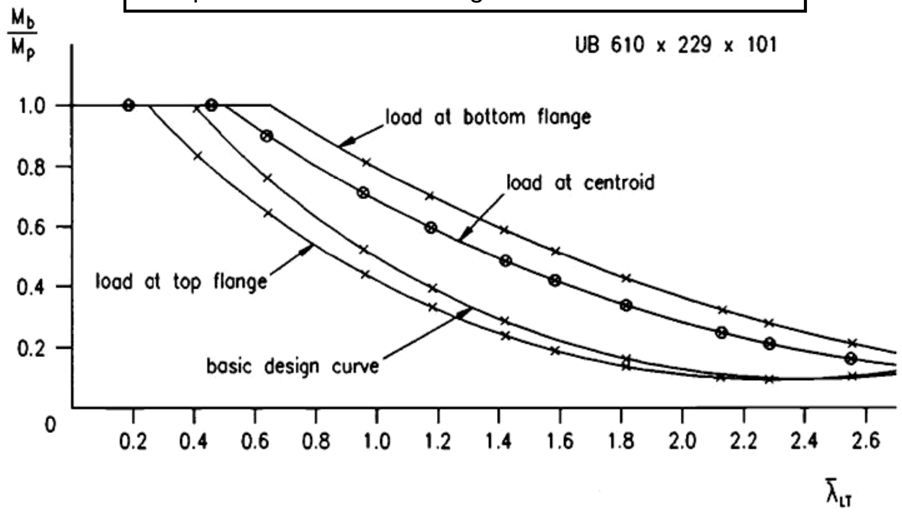


e) Partial torsional restraint against rotation about longitudinal axis provided by pressure of bottom flange onto supports

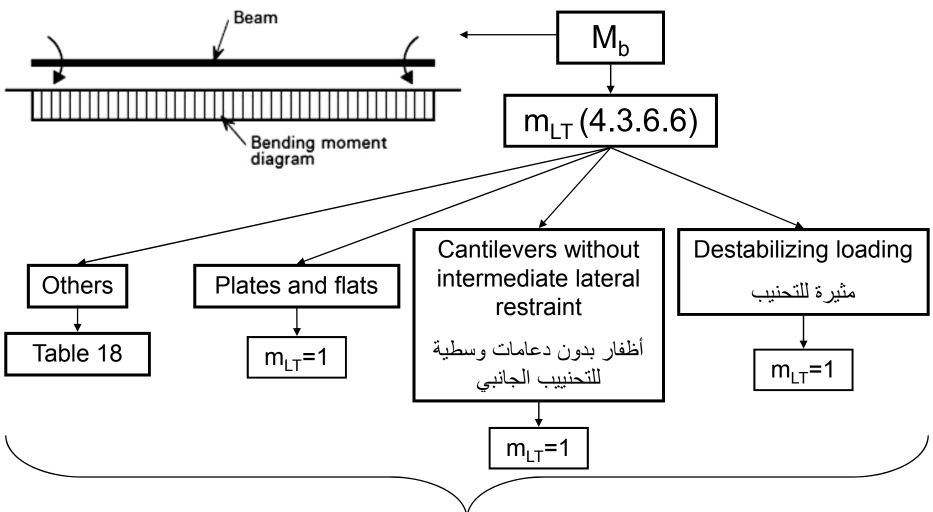

Unrestrained beam Design
$\varepsilon = \sqrt{\frac{275}{p_y}}$
Section classification تصنيف المقطع
I, H, Box, Channel, T and plate
Continued
Single angles (4.3.8)
L=L$_{v-v}$
CHS&SHS
RHS
$\frac{L_E}{r_y} \le Table 15$
Bending only around minor axis (العزم حول المحور الضعيف فقط)
equal legs
متساوية القدمين (4.3.8.3)
Unequal legs
غير متساوية القدمين
No lateral torsional buckling
(لاتحنيب فتل جانبي) (4.3.6.1)
b/t≤15ε
B.2.9.3
$M_x \le M_{cx}$
Heel in tension
Heel
Heel in compression
Heel
$M_b = p_y Z_x(\frac{1350\varepsilon - L_E / r_v}{1625\varepsilon}) \le 0.8 p_y Z_x$
$M_b = 0.8 p_y Z_x$


I, H, Box, Channel, T and plate sections
Plastic or compact: β_w=1
B.2.6
$\frac{L_E}{r_y} > Table 15$
RHS
Ratio β_w النسبة (4.3.6.9)
Semi-compact:
Z_x in use» $\beta_w=Z_x/S_x$
$S_{x,eff}$ in use» bw=$S_{x,eff}/S_x$
T Section
Plates and flats
I, H and Channel
B.2.8
B.2.8
$\lambda = \frac{L_E}{r_y}$
Slender: $\beta_w=Z_{x,eff}/Sx$
$\lambda_{LT} = 2.8\sqrt{\frac{\beta_w L_E d}{t^2}}$
u (4.3.6.8)
Rolled: u=0.9
welded: u=1
محافظ
$\lambda_{LT} > \lambda_{L0} = 0.4\sqrt{\frac{\pi^2 E}{p_y}}$
ν (Table 19) (4.3.6.7)
$\eta = \frac{I_{yc}}{I_{yc}+I_{yt}}$
Equal flanges: η=0.5
p_b Table 16 or 17(B.2.1)
$\lambda_{LT} = uv\lambda\sqrt{\beta_w}$
x=D/T (4.3.6.8)
Slender
Semi-Compact
Plastic or Compact
$\lambda_{LT} \le \lambda_{L0} = 0.4\sqrt{\frac{\pi^2 E}{p_y}}$
$M_b = p_b Z_{x,eff}$
$M_b = p_b Z_x$
$M_b = p_b S_{x,eff}$
$M_b = p_b S_x$
Restrained beam » $Mx \le Mcx$

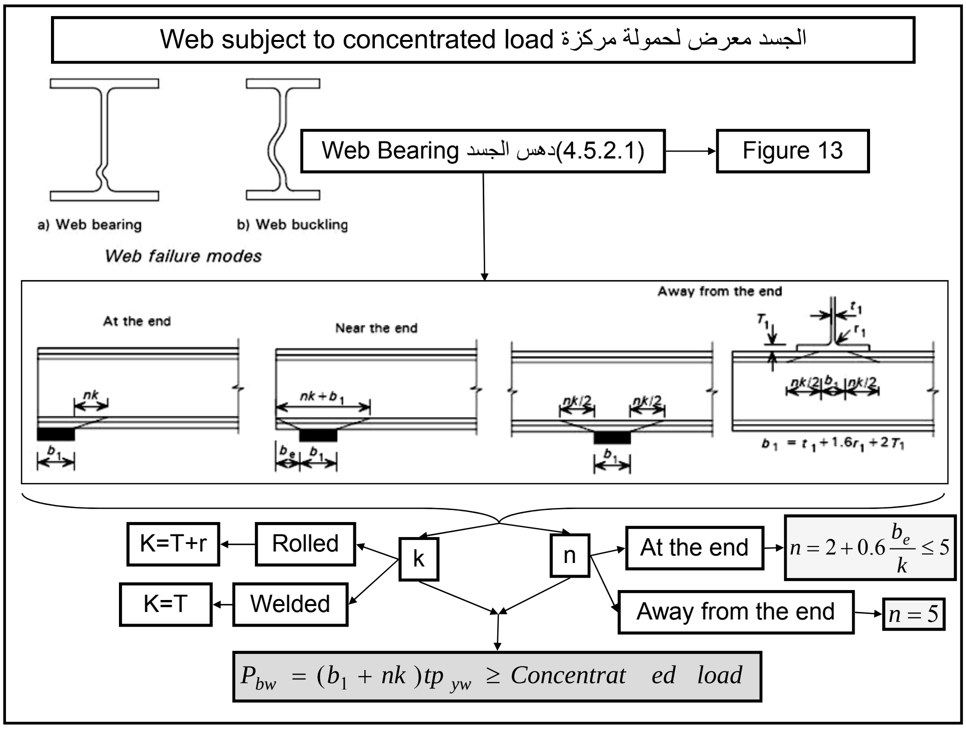
Compared between the design curve and research data

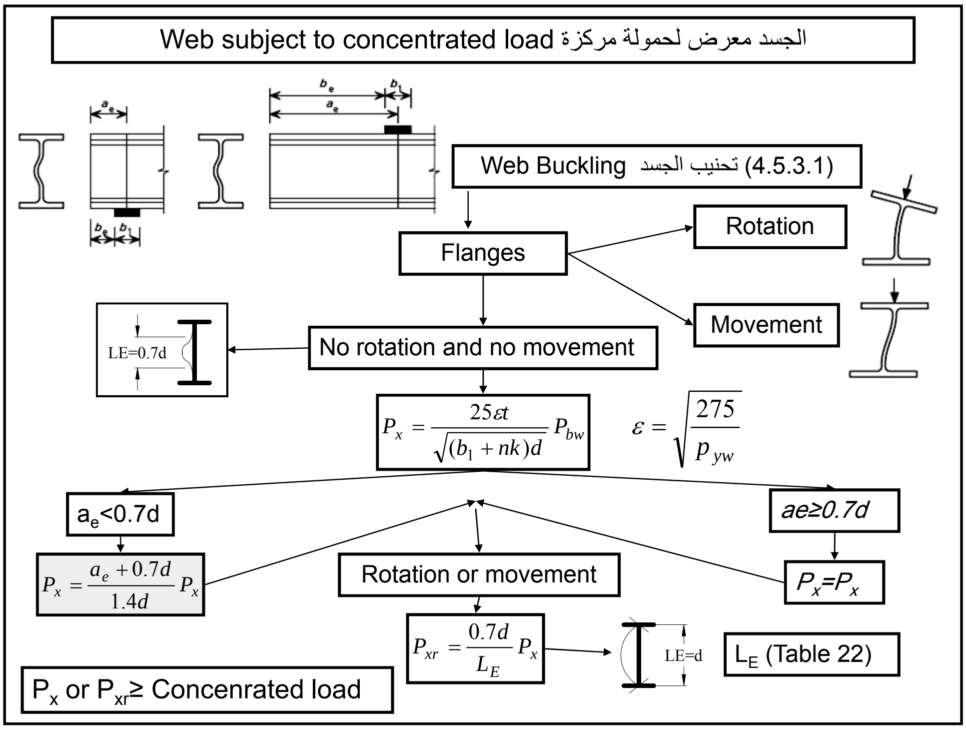


Lateral – torsional buckling of a tip-loaded cantilever



$$M_x \leq \frac{M_b}{m_{LT}}$$


Web subject to concentrated load الجسد معرض لحمولة مركزة
a) Web bearing
b) Web buckling
Web failure modes
Web Bearing دهس الجسد(4.5.2.1)
Figure 13
At the end
Near the end
Away from the end
$b_1 = t_1 + 1.6r_1 + 2T_1$
K=T+r
Rolled
K=T
Welded
k
n
At the end
$n = 2 + 0.6\frac{b_e}{k} \le 5$
Away from the end
$n = 5$
$P_{bw} = (b_1 + nk)tp_{yw} \ge Concentrated\ load$


Web subject to concentrated load الجسد معرض لحمولة مركزة
Web Buckling تحنيب الجسد (4.5.3.1)
Flanges
Rotation
Movement
No rotation and no movement
LE=0.7d
$P_x = \frac{25\varepsilon t}{\sqrt{(b_1 + nk)d}} P_{bw}$
$\varepsilon = \sqrt{\frac{275}{p_{yw}}}$
$a_e < 0.7d$
$ae \ge 0.7d$
$P_x = \frac{a_e + 0.7d}{1.4d} P_x$
Rotation or movement
$P_x = P_x$
$P_{xr} = \frac{0.7d}{L_E} P_x$
LE=d
L_E (Table 22)
P_x or $P_{xr} \ge$ Concenrated load

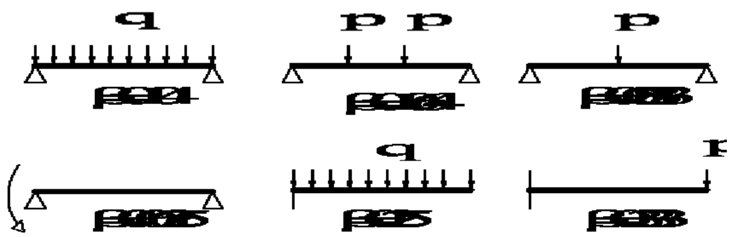
Deflection السهم (2.5)

Serviceability loads

a) LL
b) 0.8LL+0.8WL
c) WL
d) 0.8LL+0.8CL
e) CV
f) CH

Deflection limits حدود السهم (Table 8)

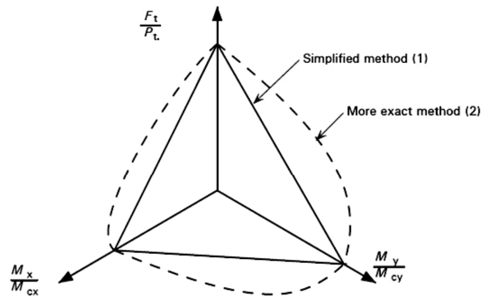
$$\delta = \beta \frac{ML^2}{EI} \leq Table\ \ 8$$

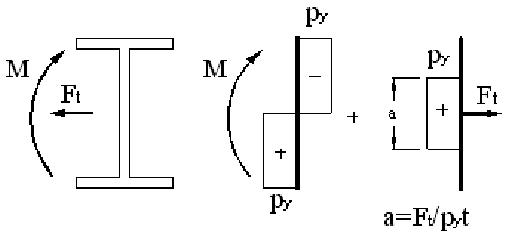


Members with Combined Moment and Axial force

العناصر المعرضة لعزم انعطاف وقوة محورية

Tension members with moment العناصر المعرضة لقوة شد وانعطاف (4.8.2)





a ≥ d » neglected moment
العزم مهمل

pure tension design تصميم على الشد الصافي

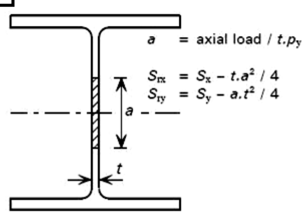
a ≤ 10%d » neglected tension load قوة الشد مهملة

Pure bending design تصميم على عزم الانعطاف الصافي

Tension members with moment design

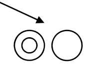
$\varepsilon = \sqrt{\frac{275}{p_y}}$ → Section classification تصنيف المقطع

- For plastic& compact sections
  - More exact method (4.8.2.3)
    - Tension with biaxial moments شد مع عزوم بالاتجاهين → Continued
    - Tension with minor axis moment only شد مع انعطاف حول المحور الضعيف → $M_{ry} = p_y S_{ry}$ → $M_y \le M_{ry}$
    - Tension with major axis moment only شد مع انعطاف حول المحور القوي → $M_{rx} = p_y S_{rx}$ → $M_x \le M_{rx}$
- General Case الحالة العامة
  - Simplified method (4.8.2.2)
    - $$\frac{F_t}{P_t} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \le 1$$



Tension members with moment design-continued

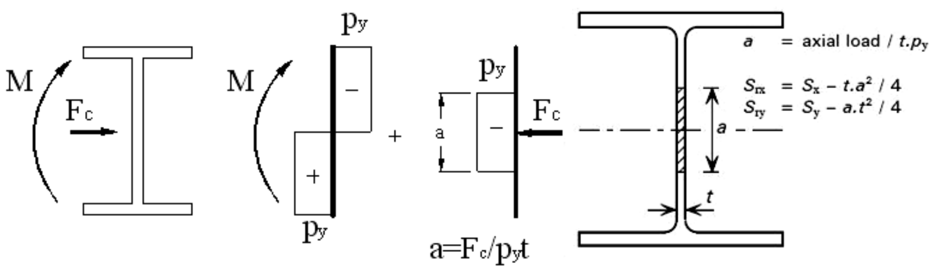
Tension with biaxial moments شد مع عزوم بالاتجاهين

- All others حالات أخرى → $Z_1=Z_2=1$
-  → $Z_1=Z_2=5/3$
-  → $Z_1=Z_2=2$
- I and H sections with equal flanges الأجنحة متساوية → $Z_1=2$, $Z_2=1$

$$\left(\frac{M_x}{M_{rx}}\right)^{Z_1}+\left(\frac{M_y}{M_{ry}}\right)^{Z_2}\le 1$$

Note: In all cases of tension members with moment, lateral-torsional buckling should be checked تحقيق تحنيب الفتل الجانبي (4.8.2.1) → $M \le \dfrac{M_b}{m_{LT}}$

Compression members with moment العناصر المعرضة لقوة ضغط وانعطاف (4.8.3)



a ≥ d » neglected moment العزم مهمل
pure compression design تصميم على الضغط الصافي

a ≤ 10%d » neglected compression load قوة الضغط مهملة
Pure bending design تصميم على عزم الانعطاف الصافي

Compression members with moment design

$\varepsilon = \sqrt{\frac{275}{p_y}}$ → Section classification تصنيف المقطع

For plastic& compact sections

More exact method (4.8.3.2)

- Compression with biaxial moments ضغط مع عزوم بالاتجاهين → Continued
- Compression with minor axis moment only ضغط مع انعطاف حول المحور الضعيف → $M_{ry} = p_y S_{ry}$ → $M_y \leq M_{ry}$
- Compression with major axis moment only ضغط مع انعطاف حول المحور القوي → $M_{rx} = p_y S_{rx}$ → $M_x \leq M_{rx}$

General Case الحالة العامة

Simplified method (4.8.3.2)

$$\frac{F_c}{A_g p_y} + \frac{M_x}{M_{cx}} + \frac{M_y}{M_{cy}} \leq 1$$

Slender section: $A_g = A_{eff}$

Compression members with moment design-continued

Compression with biaxial moments ضغط مع عزوم بالاتجاهين

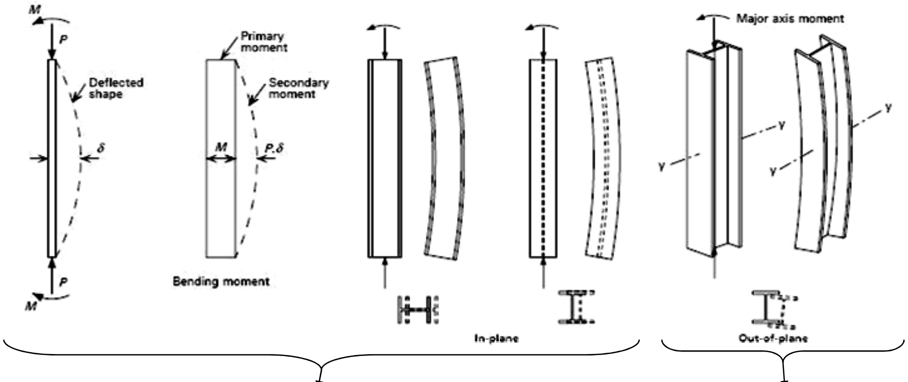
- All others حالات أخرى → $Z_1 = Z_2 = 1$
- Rectangular hollow / box sections → $Z_1 = Z_2 = 5/3$
- Circular hollow / solid sections → $Z_1 = Z_2 = 2$
- I and H sections with equal flanges الأجنحة متساوية → $Z_1 = 2$, $Z_2 = 1$

$$\left(\frac{M_x}{M_{rx}}\right)^{Z_1} + \left(\frac{M_y}{M_{ry}}\right)^{Z_2} \leq 1$$

Note: In all cases of compression members with moment, member buckling resistance should be checked تحقيق مقاومة العنصر على التحنيب (4.8.3.3)

Compression members with moment design-continued

Member buckling resistance مقاومة العنصر للتحنيب (4.8.3.3)



Flexural buckling: تحنيب ناتج عن الزيادة في الانعطاف نتيجة الانزياح الحاصل تحت تأثير قوة الضغط (في نفس مستوي العزم المطبق)

Lateral-torsional buckling: تحنيب الفتل الجانبي (ليس في مستوي العزم المطبق)

Compression members with moment design-continued

Member buckling resistance مقاومة العنصر للتحنيب (4.8.3.3)

m_x: معامل التكافؤ للعزم حول المحور القوي (**M_x**) على الجزء (**L_x**) والذي به تتحدد قوة الضغط الحدية حول x (**p_{cx}**). (Table 26) وعلى هذا الجزء نوجد **$M_{xmax}=M_x$**

m_y: معامل التكافؤ للعزم حول المحور الضعيف (**M_y**) على الجزء (**L_y**) والذي به تتحدد قوة الضغط الحدية حول y (**p_{cy}**). (Table 26) وعلى هذا الجزء نحدد **$M_{ymax}=M_y$**

m_{yx}: معامل التكافؤ للعزم حول المحور الضعيف (**M_y**) على الجزء (**L_x**) والذي به تتحدد قوة الضغط الحدية حول x (**p_{cx}**). (Table 26) (**m_{yx}**:To be used with out-of-plane buckling).

m_{LT}(unrestrained beam), **P_c=Min(p_{cx},p_{cy})**, **M_{LT}=M_{xmax}** in the segment where **M_b** occurs

- For plastic& compact sections
  - More exact method (4.8.3.3.2-4)
- Simplified method (4.8.3.3.1)
  - Minor axis buckling

    $$\frac{F_c}{P_{cy}}+\frac{m_{LT}M_{LT}}{M_b}+\frac{m_yM_y}{p_yZ_y}\le 1$$

  - General buckling

    $$\frac{F_c}{P_c}+\frac{m_xM_x}{p_yZ_x}+\frac{m_yM_y}{P_yZ_y}\le 1$$

Notes

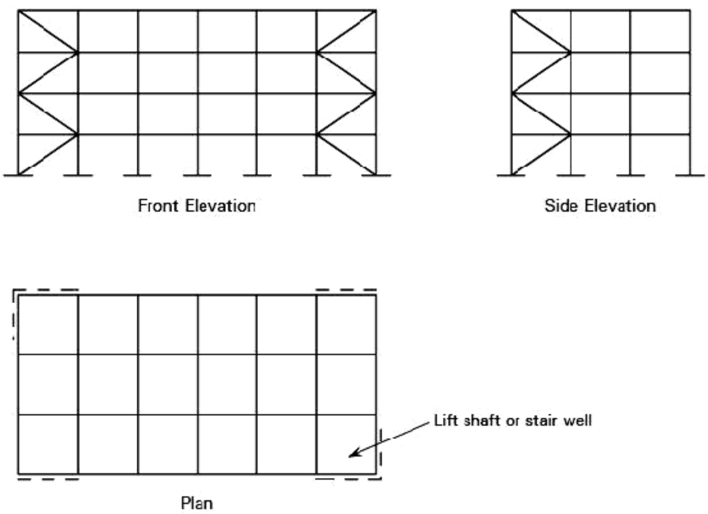
Slender Section: $Z=Z_{eff}$

$\mathbf{F_t=F_c=0}$, $\mathbf{M_x \neq 0}$ and $\mathbf{M_y \neq 0}$ » Biaxial moments (4.9)» The design is according to compression with moments case with $F_c=0$

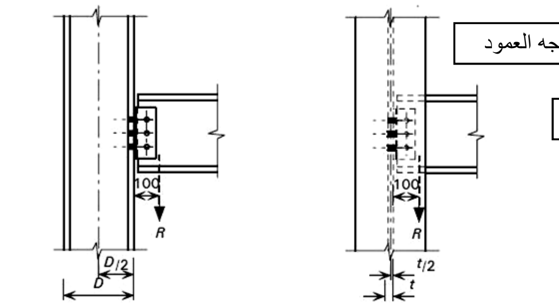
In sway mode حالة الإزاحة في المستوي للمنشأ ككل: $\mathbf{m_x}$,$\mathbf{m_y}$ and $\mathbf{m_{yx}} \geq 0.85$

Columns in simple structures الأعمدة في المنشآت غير الإطارية (4.7.7)

Simple structures= pinned columns + bracing or shear wall for horizontal resistance



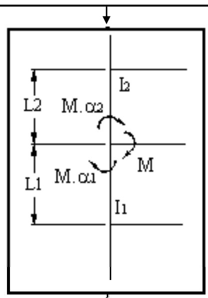
Columns in simple structures الأعمدة في المنشآت غير الإطارية



Nominal moment = R (D /2 + 100 mm)　　Nominal moment = R(t /2 + 100 mm)

نفرض أن رد فعل الجائز يبعد 100mm عن وجه العمود

Nominal moment=M



$$\alpha_i = \frac{I_i / L_i}{\sum I_i / L_i}$$

$$Note : \frac{\alpha_{max}}{\alpha_{min}} \le 1.5 \Rightarrow M_1 = M_2 = M / 2$$

$$\frac{F_c}{P_c} + \frac{M_x}{M_{bs}} + \frac{M_y}{P_y Z_y} \le 1$$

L_E (Table 22), Typically=(0.85L or L)

P_c=min(P_{cx}, P_{cy})

M_{bs} as unrestrained beam, λ_{LT}=0.5L/r_y for simplicity