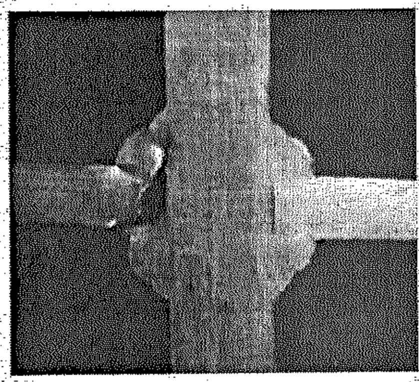
Problems in Weld placement



Kaynaklı Birleşimlerde Emniyet Gerilmeleri (kg/cm²)

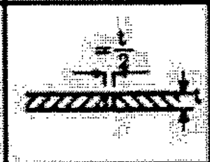
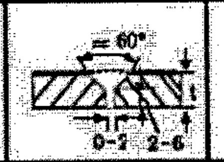
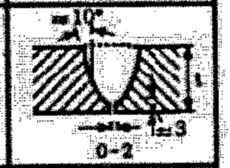
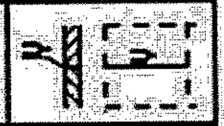
Dikiş Türü		Gerilme		St 37 H	St 37 HZ	St 52 H	St 52 HZ
KÜT	1	Basınç, Eğilmede basınç	σ_{kem}	1400	1600	2100	2700
	2	Çekme, Eğilmede çekme [illegible]	σ_{kem}	1100	1250	1700	1900
KÖŞE	3	Basınç, çekme, eğilmede basınç ve çekme	σ_{kem}	1100	1250	1700	1900
	4	Civar	σ_v	750	750	1200	1200
	5	Kıyaslama	σ_{kem}	1100	1250	1700	1900
Genel	6	Kayma	τ_{kem}	1100	1250	1700	1900

Madde profillerinin [illegible] eğilme çekmesine çalışan küt kaynaklı eklerinden [illegible] nispetinde kaçınılmalıdır. Aksi takdirde [illegible] değerlerin [illegible] kullanılmalıdır.

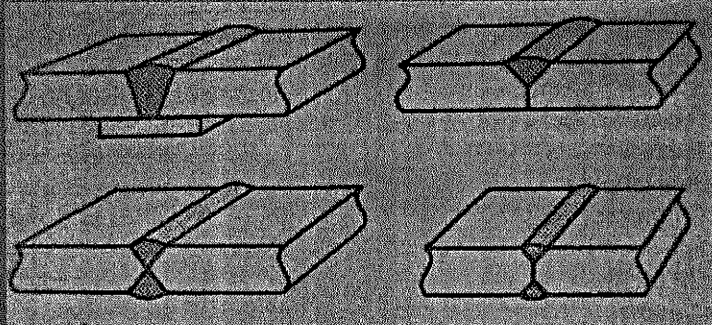
Prof.Cavidan Yorgun ders notları
Kıvanç Taşkın ders notları

GROOVE WELD (KÜT KAYNAK)

Types and symbols of Groove weld

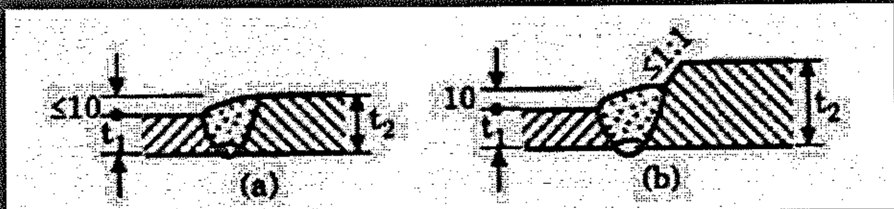
Kaynak dikişi ismi	I – Dikişi	V – Dikişi	Y – Dikişi	X – Dikişi	U – Dikişi
Levha kenarları şekli (DIN 8551) — Types of plate edges					
Kaynak işareti (DIN 1912) — Symbol of weld					
levha t kalınlığı — Thickness of plates	≤ 5	3–20	8–20	18–40	>16

For CJP welds, the effective throat equals the thickness of the thinner part joined.



Complete joint penetration groove welds

Partial joint penetration groove welds

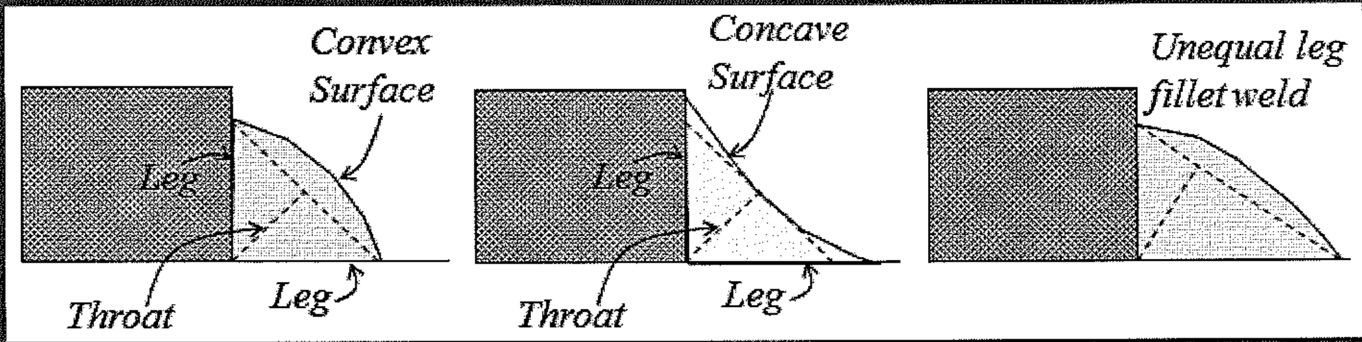


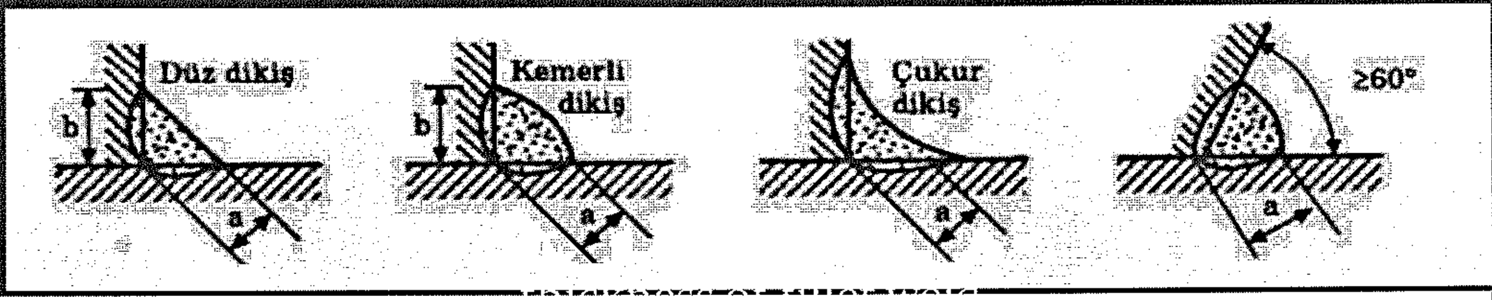
$t_2 - t_1 \leq 10$ $t_2 - t_1 > 10$

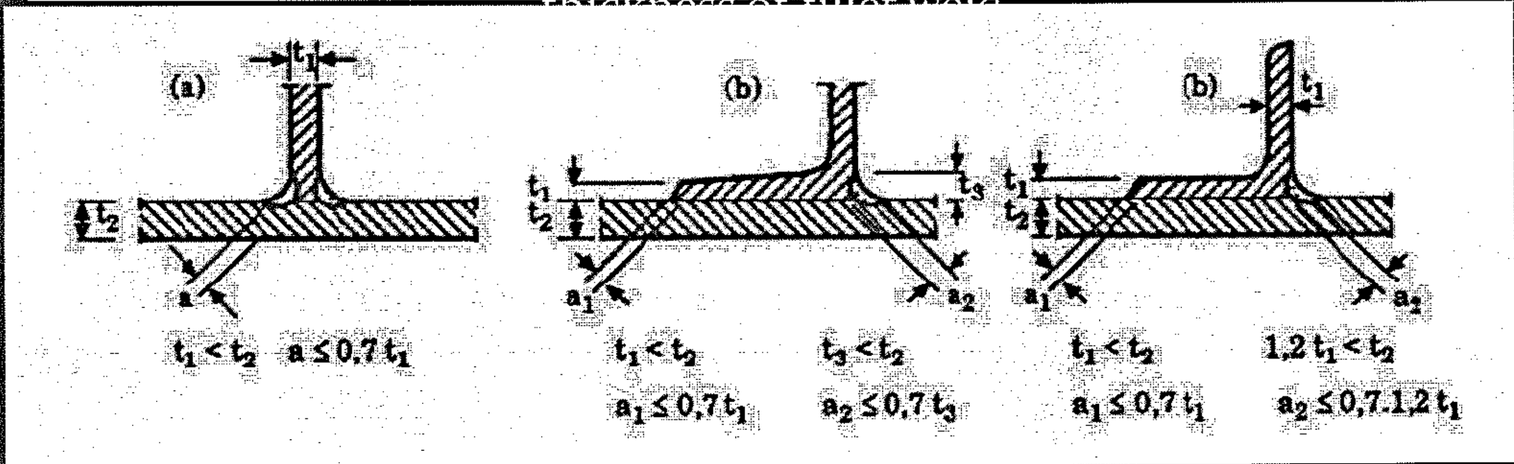
http://justinketterer.com/2010/04/03/welding/

Devrim Özhendekçi ders notları

FILLET WELD (KÖŞE KAYNAK)







Maximum thickness of fillet weld

- (ℓ) kaynak dikişi boyu

ℓ'; kaynak dikişinin ölçülen (gerçek) boyu

ℓ; kaynak dikişinin hesap boyu

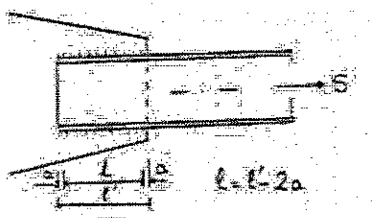
$$\ell = \ell' - \underset{\text{krater boyu}}{\underline{2a}}$$

Küt kaynak dikişlerinde $\ell = \ell'$ için

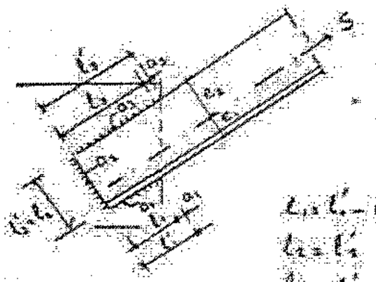
Köşe kaynak dikişlerinde, dikişin konumuna göre

$$\ell = \ell' - \begin{pmatrix} 2 \\ 1 \\ 0 \end{pmatrix} a$$

Kuvvet doğrultusunda sınırlamalar:



$15a \leq \ell' \leq 100a$



$\ell_1 = \ell_1' - a_1$
$\ell_2 = \ell_2'$
$\ell_3 = \ell_3' - a_3$

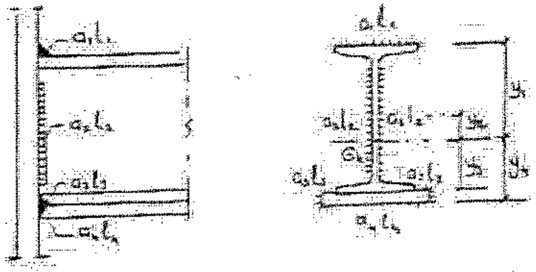
$10a \leq \ell_i' \leq 100a$

Kaynak Dikişlerinin Enkesit Değerlerinin Hesabı

- (F_k) kaynak dikişi alanı

$$F_k = \Sigma a_i \ell_i$$

- (I_k) kaynak dikişi eylemsizlik momenti



(yi'ler profil kenarından)

- Kaynaklı birleşimin G_s ağırlık merkezi bulunur.

$$I_{kx} = F_{k1} y_1^2 + 2\left(\frac{a_2 \ell_2^3}{12} + F_{k2} y_2^2\right) + 2F_{k3} y_3^2 + F_{k4} y_4^2$$

- (W_k) kaynak dikişi mukavemet momenti

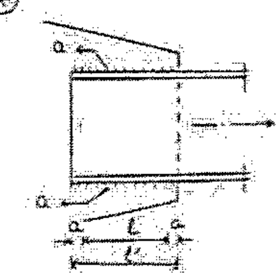
$$W_{kx} = \frac{I_{kx}}{y_{max}}$$

* Tek dikişte

$$F_k = a\,\ell \quad , \quad I_k = \frac{a\ell^3}{12} \quad , \quad W_k = \frac{a\ell^2}{6}$$

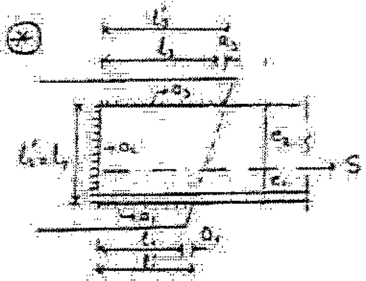
Kaynak Dikişlerinde Gerilmelerin Hesaplanması

(*)



$$\tau_k = \frac{S}{F_k} \leq \tau_{kem} \quad ; \; F_k = 2al$$

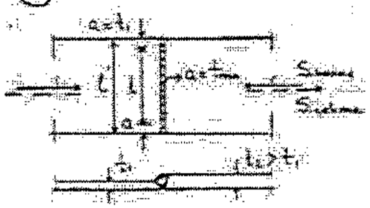
(*)



$$\tau_k = \frac{S}{F_k} \leq \tau_{kem} \quad , F_k = l_1 a_1 + l_2 a_2 + l_3 a_3$$

$$\left[\left(l_1 a_1 e_1 + a_2 \frac{e_2^2}{2} = l_3 a_3 e_2 + a_2 \frac{e_2^2}{2} \right) \text{ ise } \right]$$

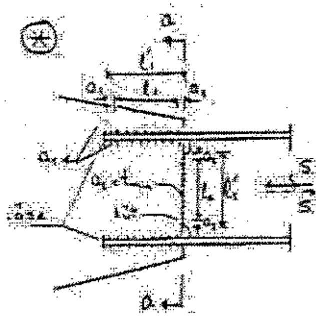
(*)



$$\sigma_k = \frac{S}{F_k} \leq \begin{cases} \frac{1}{2}\sigma_{kem} & ; (S_{\text{çekme}}) \\ \sigma_{kem} & ; (S_{\text{basınç}}) \end{cases}$$

$F_k = a.l \quad (a = t_{min})$

(*)



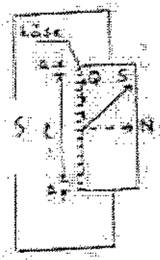
a-a kesiti

- Küt ve köşe kaynağın birlikte kullanılması

$$\tau_k = \frac{S}{F_k} \leq \tau_{kem}$$

$$F_k = \begin{cases} 8a_1 l_1 + a_2 l_2 & (S_{\text{basınç}}) \\ 8a_1 l_1 + \frac{1}{2} a_2 l_2 & (S_{\text{çekme}}) \end{cases}$$

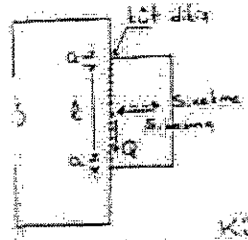
(*)



$$\left. \begin{aligned} \sigma_k &= \frac{N}{F_k} \\ \tau_k &= \frac{Q}{F_k} \end{aligned} \right\} \geq \sigma_z \text{ ise } \sigma_v = \sqrt{\sigma_k^2 + \tau_k^2} \leq \sigma_{vem}$$

$F_k = a.l$

(*)

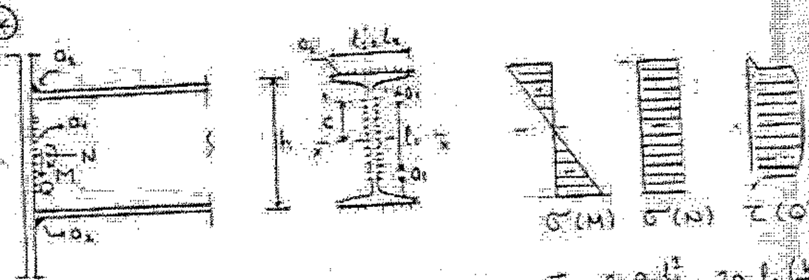


$$\sigma_k = \frac{S}{F_k} \leq \begin{cases} \frac{1}{2}\sigma_{kem} & (S_{\text{çekme}}) \\ \sigma_{kem} & (S_{\text{basınç}}) \end{cases}$$

$$\tau_k = \frac{Q}{F_k} \leq \tau_{kem} \qquad F_k = a.l$$

Küt kaynak dikişlerinde $\sigma_v \leq \sigma_{vem}$ irdelemesine gerek yoktur.

(*)



$$I_k = 2\frac{a_1 l_1^3}{12} + 2a_2 l_2 \left(\frac{h}{2}\right)^2$$

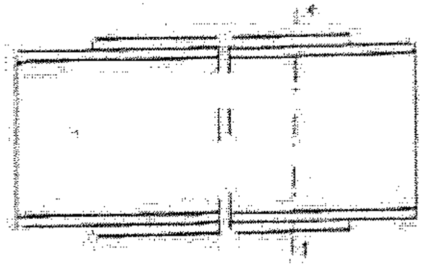
$$W_k = \frac{I_k}{h/2}$$

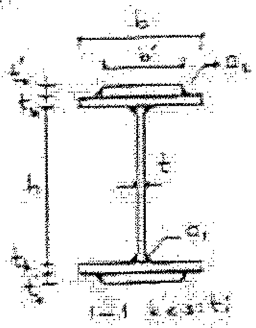
$$\sigma_k = \mp \frac{M}{W_k} + \frac{N}{F_k} \leq \sigma_{kem}$$

$$\left. \begin{aligned} \sigma_k &= \mp \frac{M}{I_k} c + \frac{N}{F_k} \\ \tau_k &= \frac{Q}{F_{k_g}} \end{aligned} \right\} \geq \sigma_z \text{ ise } \sigma_v = \sqrt{\sigma_k^2 + \tau_k^2} \leq \sigma_{vem}$$

$$F_{k_g} = 2a_1 l_1 \quad , \; F_k = 2(a_1 l_1 + a_2 l_2)$$

⊛ (M + Q) etkisindeki yapma kirişlerin boyuna köşe kaynakları





I–I kesiti

Dikiş Türü		Gerilme		St 37		St 52	
				H	HZ	H	HZ
KÜT	1	Basınç, Eğilmede basınç	σ_{kem}	1400	1600	2400	2700
	2	Çekme, Eğilmede çekme ($\frac{1}{2}$ V ≤ K.Ü.)	σ_{kem}	1100	1250	1700	1900
KÖŞE	3	Basınç, çekme, eğilmede basınç ve çekme	σ_{kem}	1100	1250	1700	1900
	4	Sınır	σ	750	750	1200	1200
	5	Kayışlama	σ_{kem}	1100	1250	1700	1900
Genel	6	Kayma	τ_{kem}	1100	1250	1700	1900

(a_1) kalınlıklı dikişlerde:

$$\tau_k = \frac{Q \cdot S_x}{I_x \cdot 2a_1} \leq \tau_{kem}$$

(a_2) kalınlıklı dikişlerde:

$$\tau_k = \frac{Q \cdot S_x'}{I_x \cdot 2a_2} \leq \tau_{kem}$$

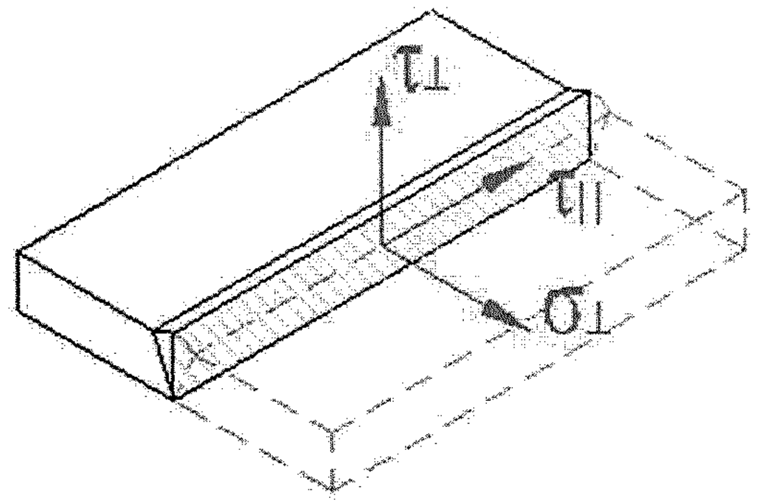
(a_1) ve (a_2) kalınlıklı köşe kaynak dikişlerinde (σ_k) gerilmesi kontrolüne gerek yoktur.

$$I_x = \frac{t h^3}{12} + 2 b t_b \left(\frac{h}{2} + \frac{t_b}{2}\right)^2 + 2 b' t_b' \left(\frac{h}{2} + t_b + \frac{t_b'}{2}\right)^2$$

$$S_x = b t_b \left(\frac{h}{2} + \frac{t_b}{2}\right) + b' t_b' \left(\frac{h}{2} + t_b + \frac{t_b'}{2}\right)$$

$$S_x' = b' t_b' \left(\frac{h}{2} + t_b + \frac{t_b'}{2}\right)$$

Gerilme	St 37 çeliği için	St 52 çeliği için
σ ve $\sqrt{\tau_{//}^2+\tau^2}$ her biri kendi başına	7,5	12
$\sigma + \sqrt{\tau_{//}^2+\tau^2}$ için	11	17



Kaynak Dikişlerinde Emniyet Gerilmeleri

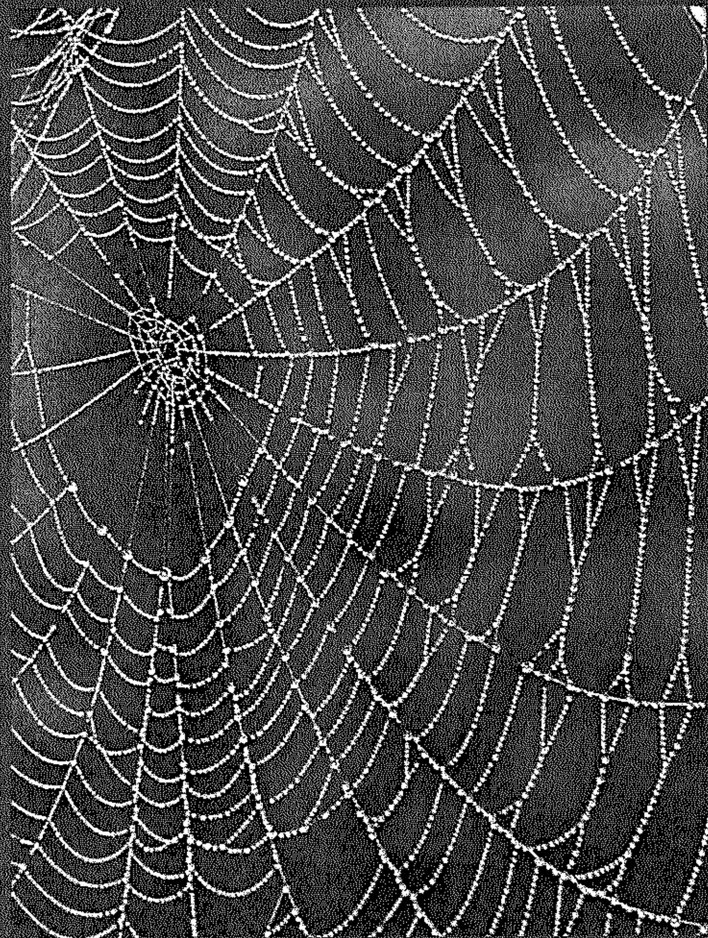
1	2	3	4	5	f	7	8
2	Dikiş çeşidi	Dikiş niteliği	Gerilme	Çelik cinsi			
				Fe 37		Fe 52	
				EY	EIY	EY	EIY
3	• Küt kaynak	Muayene edilmiş veya edilmemiş	Basınç ve eğilme basma	14	16	24	27
4	• Çift taraflı yarım Y tipi köse kaynağı*	Çatlak, bağlantı ve kök hatası olmadığı muayene ile saptanmış	Çekme ve kaynak yönüne dik eğilme çekmesi	14	16	24	27
5	Kökü kaynatılmış yarım Y tipi köşe kaynağı	Muayene edilmemiş		11	12,5	17	19
6	• Köşe kaynağı • Yarım Y tipi köşe kaynağı • Çukur ve yarık kaynağı	Muayene edilmiş veya edilmemiş	Basınç ve eğilme basıncı, çekme ve eğilme çekmesi, kıyaslama değeri	11	12,5	17	19
7	Bütün dikiş çeşitleri		Kayma	11	12,5	17	19
*							

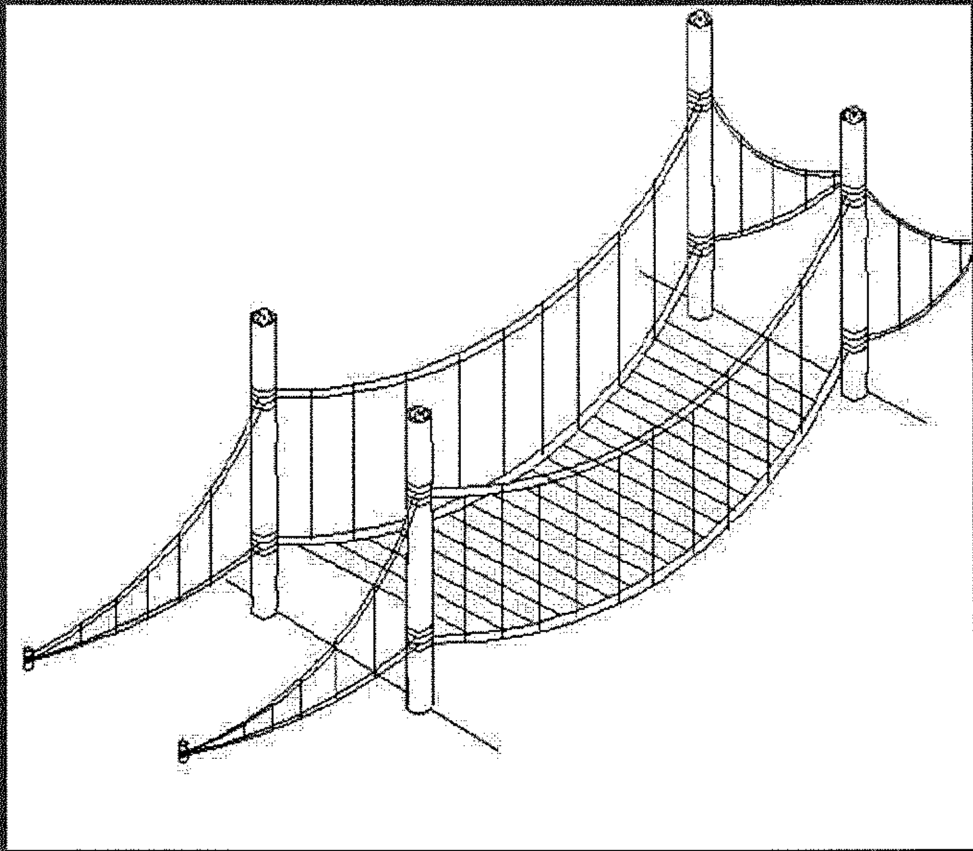
TENSION MEMBERS

Tension members are structural elements that are subjected to axial tensile forces.
Examples of tension members are bracing for buildings and bridges, truss members, andcables in suspended roof systems.

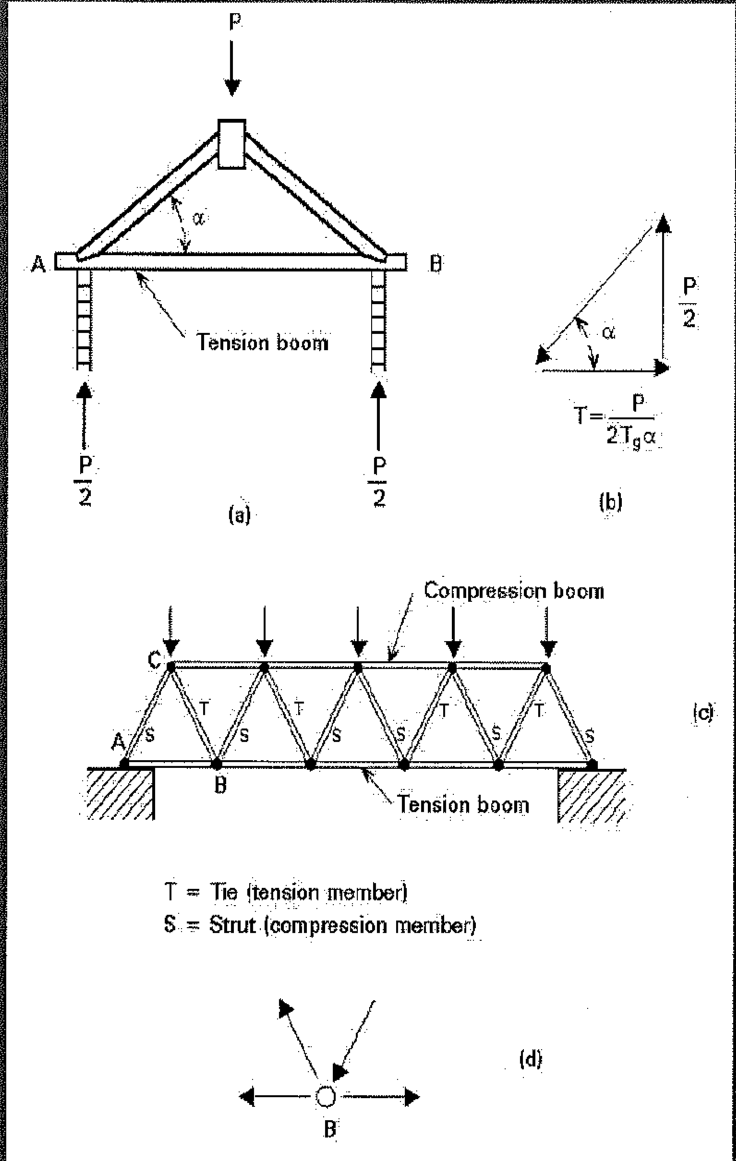
https://en.wikipedia.org/wiki/Tension_member

Tension members are the most efficient and economical of all structural elements





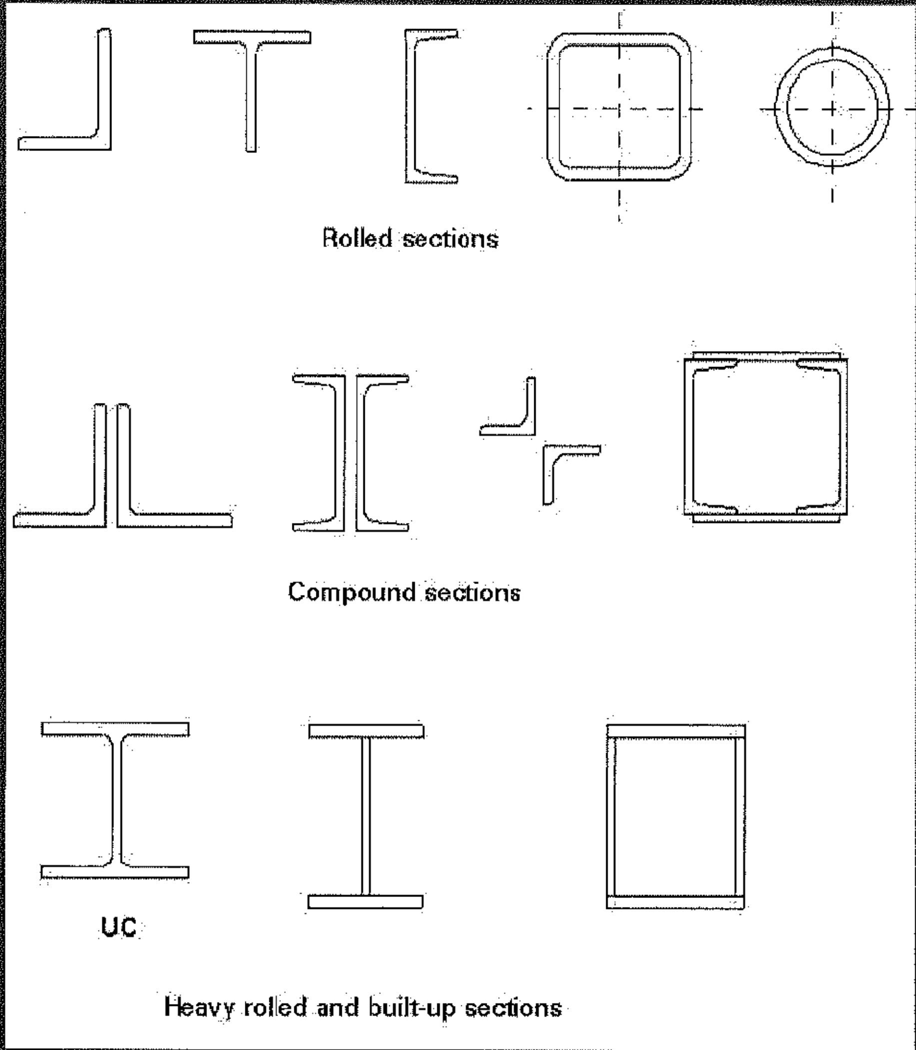
Primitive temporary bridge


P
A
B
α
Tension boom
P/2
P/2
(a)
P/2
α
T = P/(2Tgα)
(b)
Compression boom
C
A
B
T
S
Tension boom
(c)
T = Tie (tension member)
S = Strut (compression member)
B
(d)

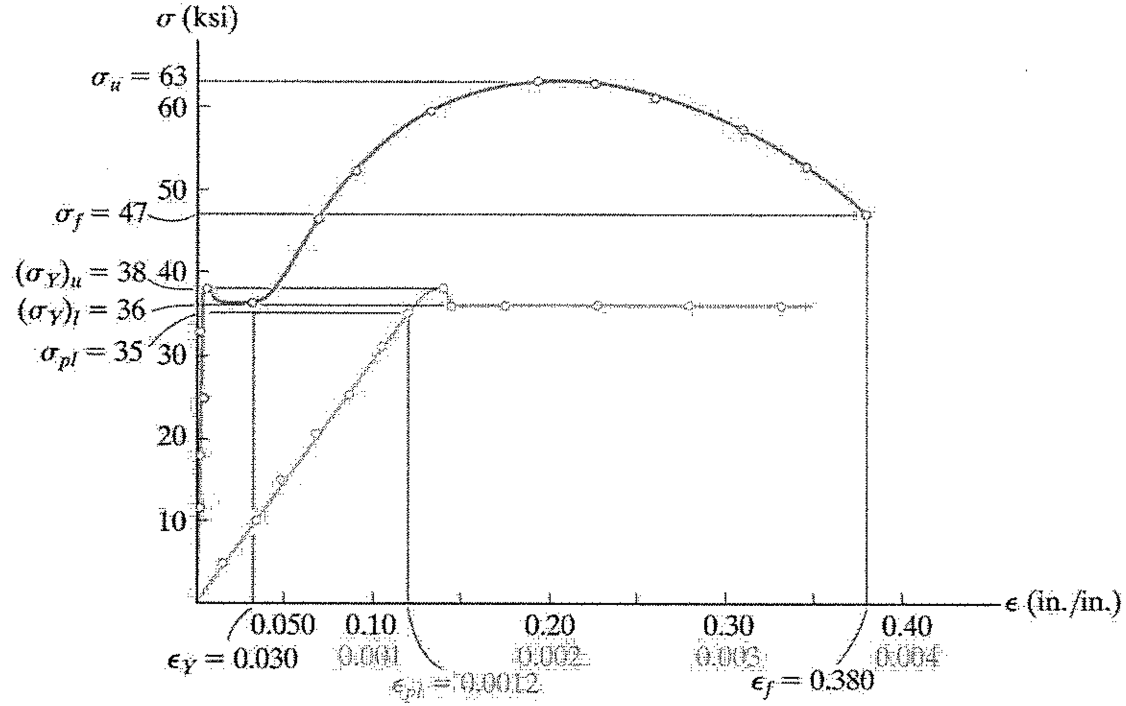
Steel and Tension

- **Steel in Tension**
- Excellent in Tension (and compression)
- Very ductile (highly desirable structural property for stress redistribution and safety)
- Connections induce loss of effective area
    - Bolt holes, etc.
    - Shear lag in outstanding parts (of both bolted and welded connections)

Typical Cross-sections of tension members



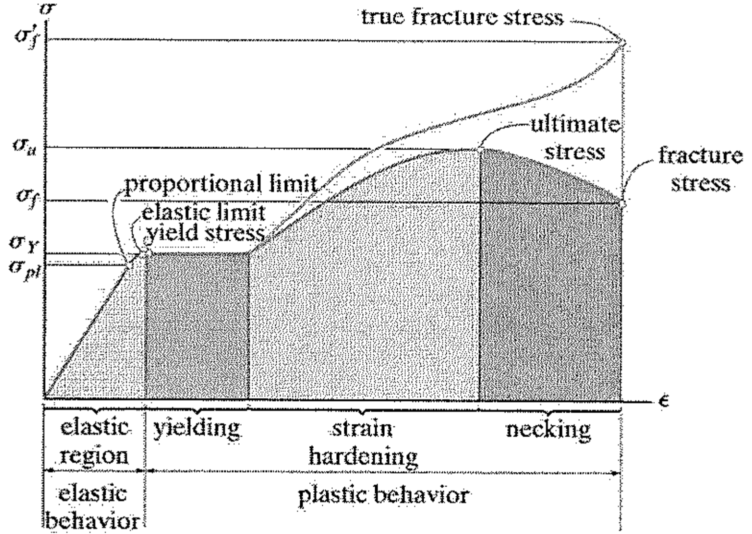
Material Behaviour



Stress-strain diagram for mild steel

Figure: 03-06

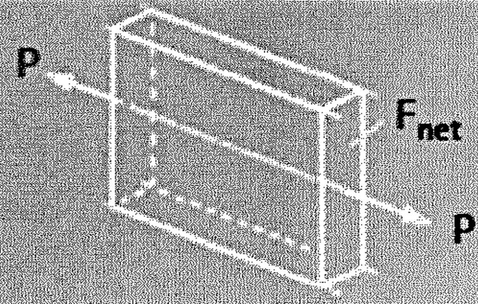
Material Behaviour

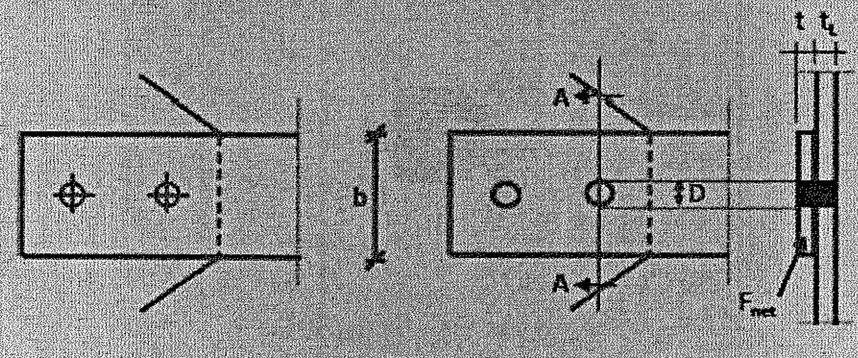


Conventional and true stress-strain diagrams for ductile material (steel) (not to scale)

Figure: 03-04

$$\sigma = \frac{P}{F_{net}} \leq \sigma_{em}$$





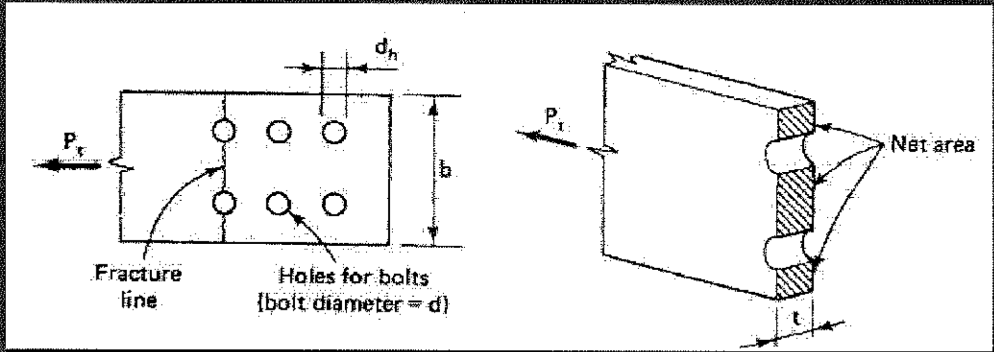
$F_{net} = b \times t - D \times t$

For bolted connections net area is the area of fracture line
Fnet = F gross – (Area of holes)

http://web.itu.edu.tr/~ustunda1/course/celiksunum.pdf

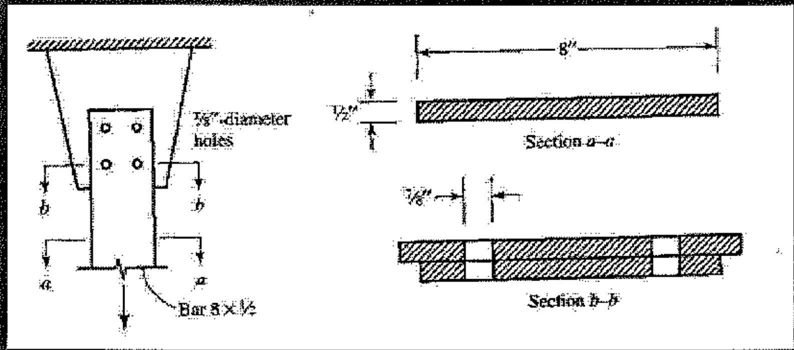
For bolted connections net area is the area of fracture line

$A_n = A_g$-(Area of holes)



A_g: Gross area

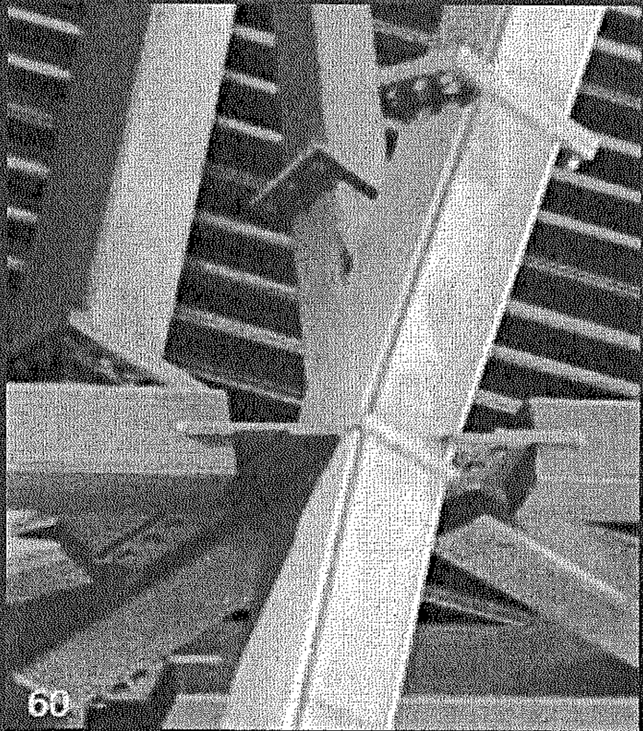
A_n : Net area

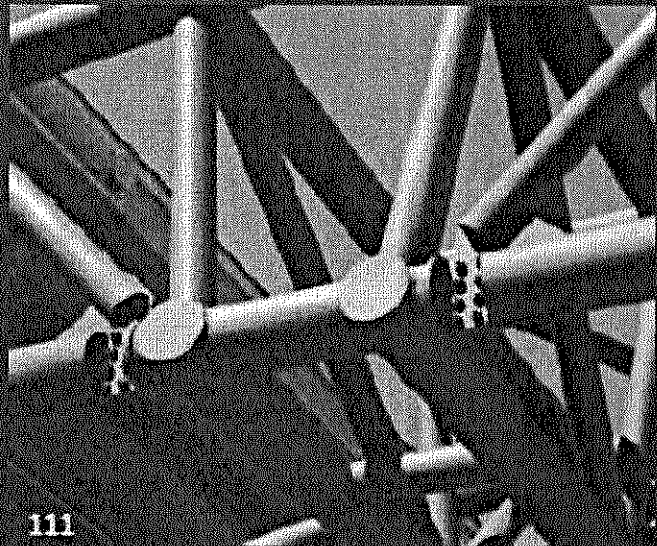


http://ce.yeditepe.edu.tr/webpages/nesrinyardimci/courses.html

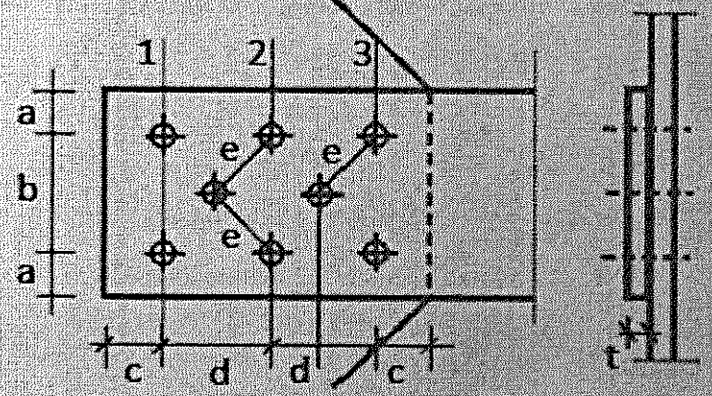
For welded connections:

$A_n = A_g$-(Area of connection cuttings)

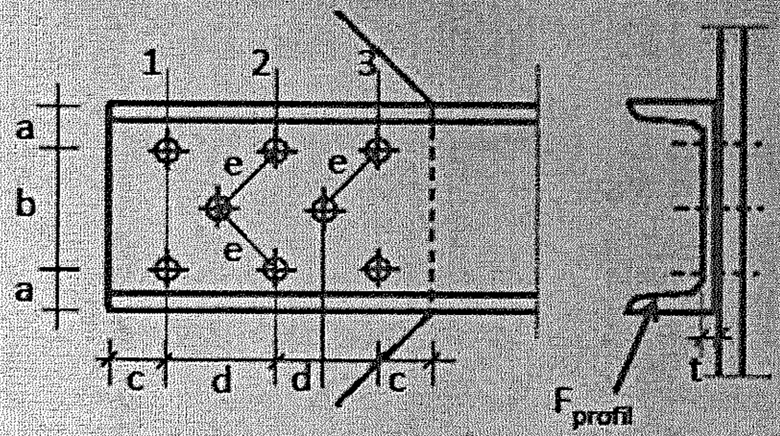




STAGGERED FASTENERS



$$F_{net} = \min \begin{cases} F_{net,1} = (b+a+a) \times t - 2 \times d \times t \\ F_{net,2} = (e+e+a+a) \times t - 3 \times d \times t \\ F_{net,3} = (e+a+a+\frac{b}{2}) \times t - 2 \times d \times t \end{cases}$$



$$F_{net} = \min \begin{cases} F_{net,1} = F_{profil} - 2 \times d \times t \\ F_{net,2} = F_{profil} - b \times t + (e+e) \times t - 3 \times d \times t \\ F_{net,3} = F_{profil} - \frac{b}{2} \times t + e \times t - 2 \times d \times t \end{cases}$$

http://web.itu.edu.tr/~ustundaı/course/celiksunum.pdf

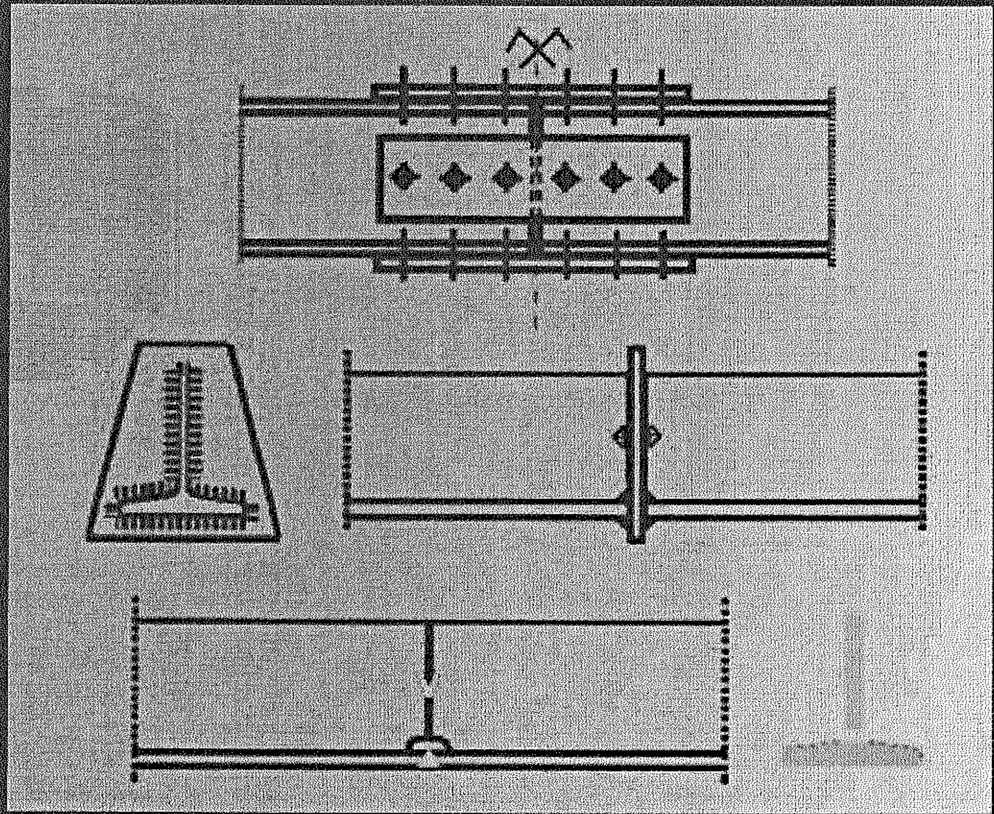
Typical tension member splices

Splices are used because of limited length of steel profiles

Bolted splice

Welded transverse splice

Groove welded splice



http://web.itu.edu.tr/~ustunda1/course/celiksunum.pdf

Design conditions:

- Symmetrical due to the splice plane.
- Calculations are done for one side.
- Total area of splice members ≥ Total area of tension member. (For bolted structures net area is used.)

$$\text{For.bolted.connections....} \sum A_{n.splice} \geq \sum A_{n.profile}$$

$$\text{For.welded.connections....} \sum A_{g.splice} \geq \sum A_{g.profile}$$

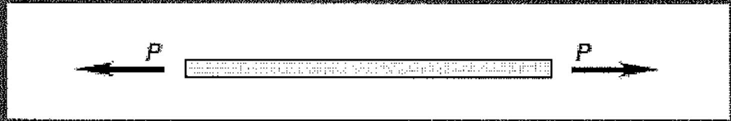
- The difference between the location of center of gravity of splice elements and the tension member should be less then 3mm. $\overline{\Delta} = \overline{X}_{profile} - \overline{X}_{splice} \leq 3mm$

- The load carried by each splice members is proportional to the gross area of that member.

  The fasteners should be control due to this load.

$$P_i \geq \frac{R_n}{\Omega} \frac{A_{gi}}{\sum A_{g.splice}}$$

- The total design load carried by all splice members should be equal to or greater than the allowable load capacity of the tension member.



SLENDERNESS LIMITATIONS

There is no maximum slenderness limit for members in tension.

User Note: For members designed on the basis of tension, the slenderness ratio L/r preferably should not exceed 300. This suggestion does not apply to rods or hangers in tension.

$$\frac{L}{r} \leq 300 \quad \text{[AISC Sec. D1]}$$

AISC 360-10

(a) For tensile yielding in the gross section:

$$P_n = F_y A_g \qquad \text{(D2-1)}$$

$\phi_t = 0.90$ (LRFD) $\quad \Omega_t = 1.67$ (ASD)

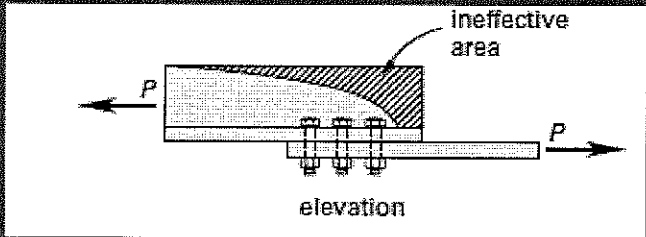
(b) For tensile rupture in the net section:

$$P_n = F_u A_e \qquad \text{(D2-2)}$$

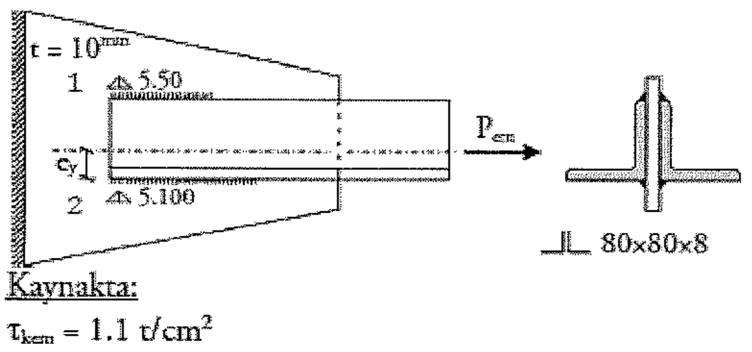
$\phi_t = 0.75$ (LRFD) $\quad \Omega_t = 2.00$ (ASD)

A_e = *effective net area*, in.^{2} (mm^2)
A_g = gross area of member, in.^{2} (mm^2)
F_y = *specified minimum yield stress*, ksi (MPa)
F_u = *specified minimum tensile strength*, ksi (MPa)

$$A_e = A_n U$$



AISC 360-10
Steel Design for the Civil PE and Structural SE Exams



Şekilde verilen düğüm noktasında, gerekli kontrolleri yaparak, çekme çubuğunun emniyetle taşıyabileceği çekme kuvvetini (P_{em}) hesaplayınız.

(YH1, Ç37) $\sigma_{em} = 1.44\ t/cm^2$

L 80.80.8

$F = 12.3\ cm^2$; $e_y = 2.26\ cm$

Make required checks and calculate P allowable for the connection shown as above

Çekme çubuğunda kontrol:

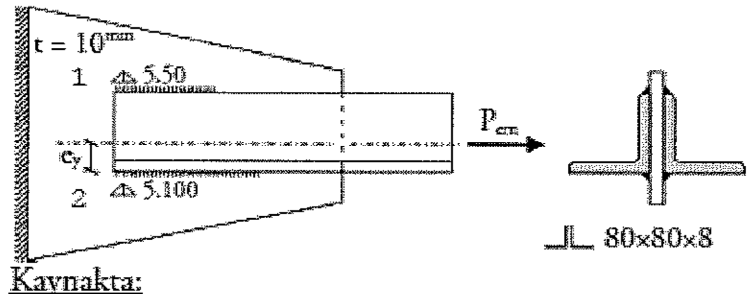
$$\sigma = \frac{P}{\Sigma F} < \sigma_{em} \Rightarrow P < \sigma_{em} \cdot \Sigma F = 1.44 \cdot 2 \cdot 12.3 = 35.42t$$

Kaynaklarda kontrol:

1 Nolu kaynakta kontrol:

$a = 5\,mm \Rightarrow 3\,mm \le a \le 0.7 t_{min} \Rightarrow 3\,mm \le 5\,mm \le 0.7 \cdot 8 = 5.6\,mm$ ✓

$l_{k1} = 50 - 2 \cdot 5 = 40\,mm \Rightarrow 15a \le l_k \le 100a \Rightarrow 15 \cdot 5 = 75\,mm > l_{k1} = 40\,mm$ ✗



Kaynakta:

$\tau_{kem} = 1.1\ t/cm^2$

Şekilde verilen düğüm noktasında, gerekli kontrolleri yaparak, çekme çubuğunun emniyetle taşıyabileceği çekme kuvvetini (P_{em}) hesaplayınız.

(YH1, Ç37) $\sigma_{em} = 1.44\ t/cm^2$

L 80.80.8

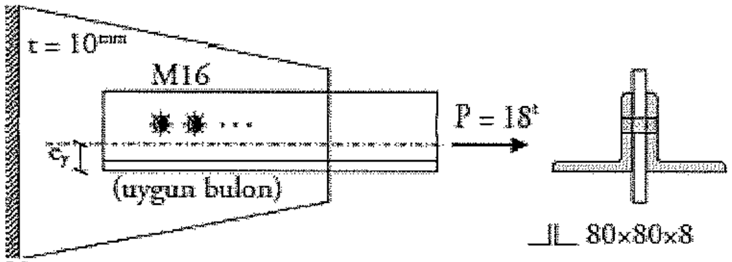
$F = 12.3\ cm^2$; $e_y = 2.26$ cm

Make required checks and calculate P allowable for the connection shown above

1 Nolu kaynak için kaynak boyu yeterli değildir ve arttırılmalıdır. Kaynak hesap boyu için minimum değer olan 75 mm alınarak hesaba devam edilmiştir.

$$P_{k1} = \frac{2.26}{8.0} \cdot P$$

$$\tau_k = \frac{P_{k1}}{\Sigma al} = \frac{2.26}{8.0 \cdot \Sigma al} \cdot P \leq \tau_{k,em} \Rightarrow P \leq \frac{\tau_{k,em} \cdot 8.0 \cdot \Sigma al}{2.26} = \frac{1.1 \cdot 8.0 \cdot (2 \cdot 0.5 \cdot 7.5)}{2.26} = 29.2\,t$$



Uygun bulonda:

$\tau_{aem} = 1.4$ t/cm^2 ; $\sigma_{lem} = 2.8$ t/cm^2

Şekilde verilen düğüm noktasında çekme çubuğuna P = 18t'luk bir çekme kuvveti etkimektedir. Tek sıra M16 uygun bulon kullanılması durumunda, minimum bulon sayısını hesaplayınız.

(YH1, Ç37) $\sigma_{em} = 1.44$ t/cm^2

L 80.80.8

F = 12.3 cm^2; e_y = 2.26 cm

calculate required number of bolt for the connection shown as above

L80.80.8	$F = 12.3$ cm^2
Uygun bulon çapı	$d = 17$ mm
Delik çapı	$D = 17$ mm
Bulon adedi	$n = ?$
Etki sayısı	$m = 2$
Çift etkili bulon	$t = \min(t_1, t_2+t_3) = \min(10, 8+8) = 10$ mm

$$\tau = \frac{P/n}{m \cdot \pi d^2/4} \le \tau_{aem} \Rightarrow n \ge \frac{P}{\tau_{aem} \cdot m \cdot \pi d^2/4} = \frac{18}{1.4 \cdot 2 \cdot \frac{\pi \cdot 1.7^2}{4}} = 2.83$$

$$\sigma = \frac{P/n}{d \cdot t} \le \sigma_{lem} \Rightarrow n \ge \frac{P}{\sigma_{lem} \cdot d \cdot t} = \frac{18}{2.8 \cdot 1.7 \cdot 1} = 3.78$$

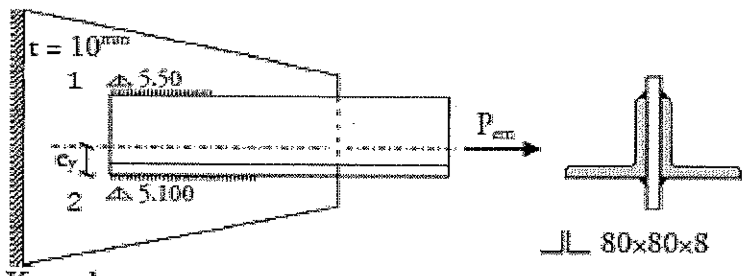
$$n = 4$$

Minimum bulon sayısı n=4'dür.

Çekme çubuğunda tahkik:

$$F_{net} = 2 \cdot (F - D \cdot t_L)$$

$$\sigma = \frac{P}{F_{net}} \le \sigma_{em} \Rightarrow P \le \sigma_{lem} \cdot F_{net}$$



Kaynakta:

τ_{kem} = 1.1 t/cm²

Şekilde verilen düğüm noktasında, gerekli kontrolleri yaparak, çekme çubuğunun emniyetle taşıyabileceği çekme kuvvetini (P_{em}) hesaplayınız.

(YH1, Ç37) $\sigma_{em} = 1.44$ t/cm²

L 80.80.8

F = 12.3 cm²; e_y = 2.26 cm

Make required checks and calculate P allowable for the connection shown above

2 nolu kaynakta kontrol:

$$a = 5\,mm \Rightarrow 3\,mm \le a \le 0.7 t_{min} \Rightarrow 3\,mm \le 5\,mm \le 0.7\cdot 8 = 5.6\,mm \quad ✓$$

$$l_{k2} = 100 - 2\cdot 5 = 90\,mm \Rightarrow 15a \le l_k \le 100a \Rightarrow 15\cdot 5 = 75\,mm \le l_{k2} = 90\,mm \le 500\,mm ✓$$

$$P_{k2} = \frac{5.74}{8.0}\cdot P$$

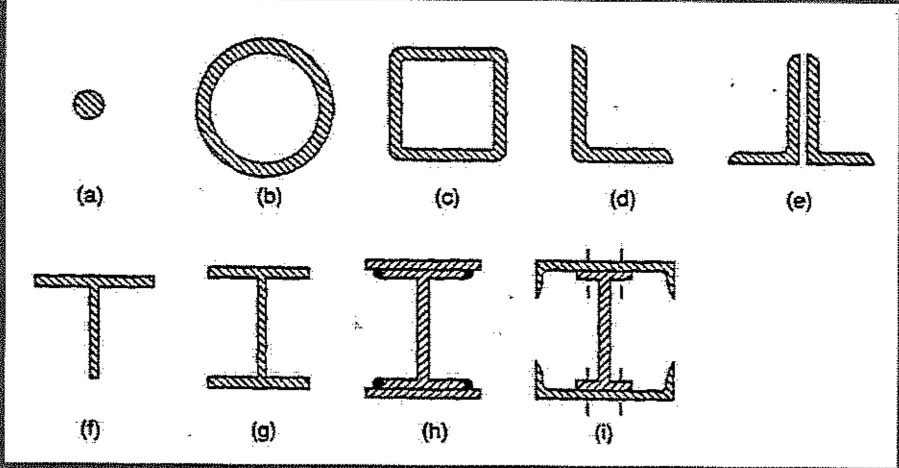
$$\tau_k = \frac{P_{k1}}{\Sigma al} = \frac{5.74}{8.0\cdot \Sigma al}\cdot P \le \tau_{k,em} \Rightarrow P \le \frac{\tau_{k,em}\cdot 8.0\cdot \Sigma al}{5.74} = \frac{1.1\cdot 8.0\cdot(2\cdot 0.5\cdot 9)}{5.74} = 13.80\,t$$

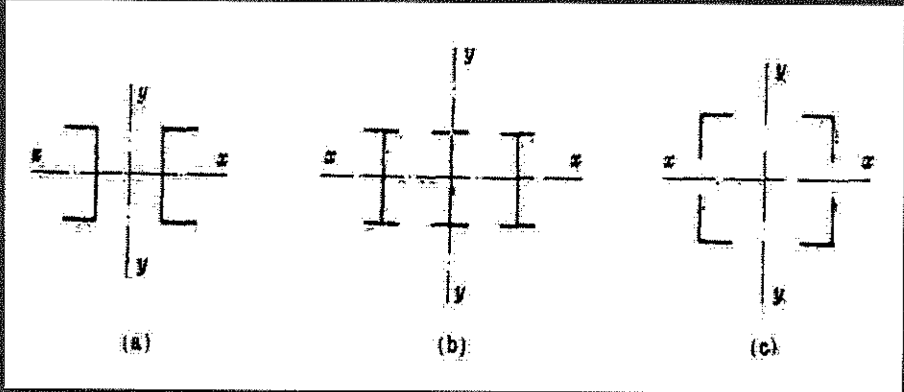
$$P_{em} = \begin{pmatrix} 35.42\,t \\ 29.20\,t \\ 13.80\,t \end{pmatrix}_{min} = 13.80\,t$$

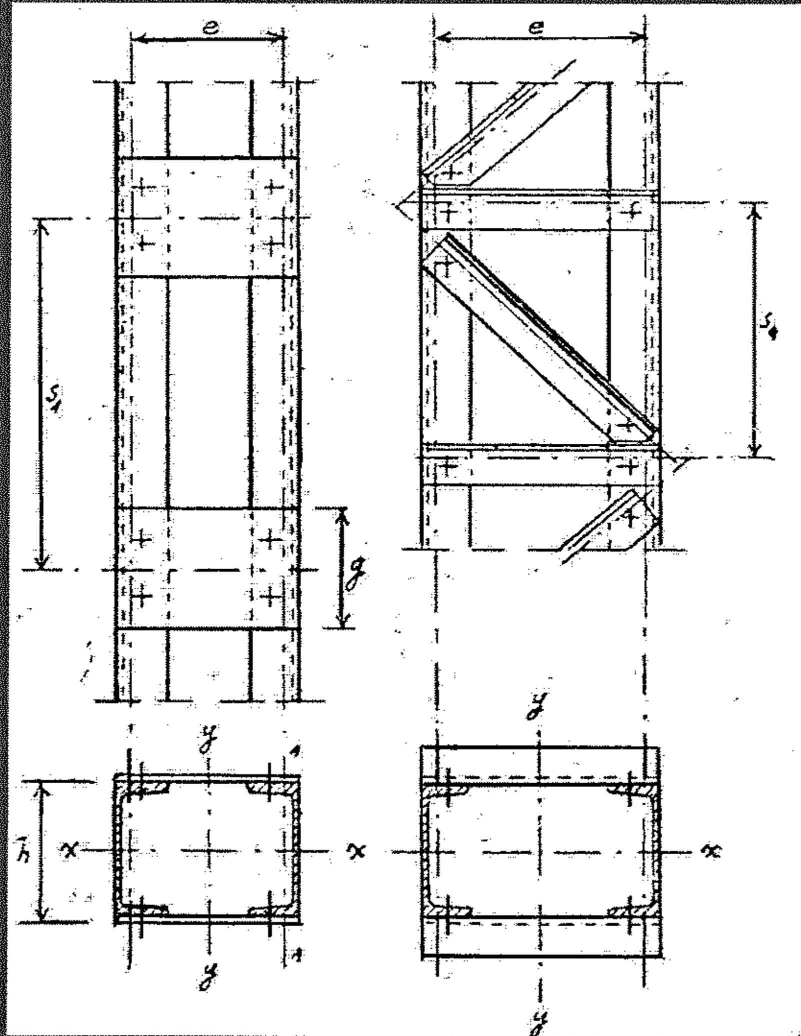
COMPRESSION MEMBERS

INTRODUCTION FOR COMPRESSION MEMBERS

- Members that are subjected to axial (concentric) loads.
- Compression members: struts (serve as bracing), posts or pillars, truss web and chord members.
- Shapes: Most of the rolled shapes.
- For larger loads built-up cross sections (allow a designer to tailor to specific needs). Lacing bars or perforated cover plates make the section as a single unit.
- The problem of stability is of great importance; they are very sensitive to factors that may tend to cause lateral displacements or buckling.







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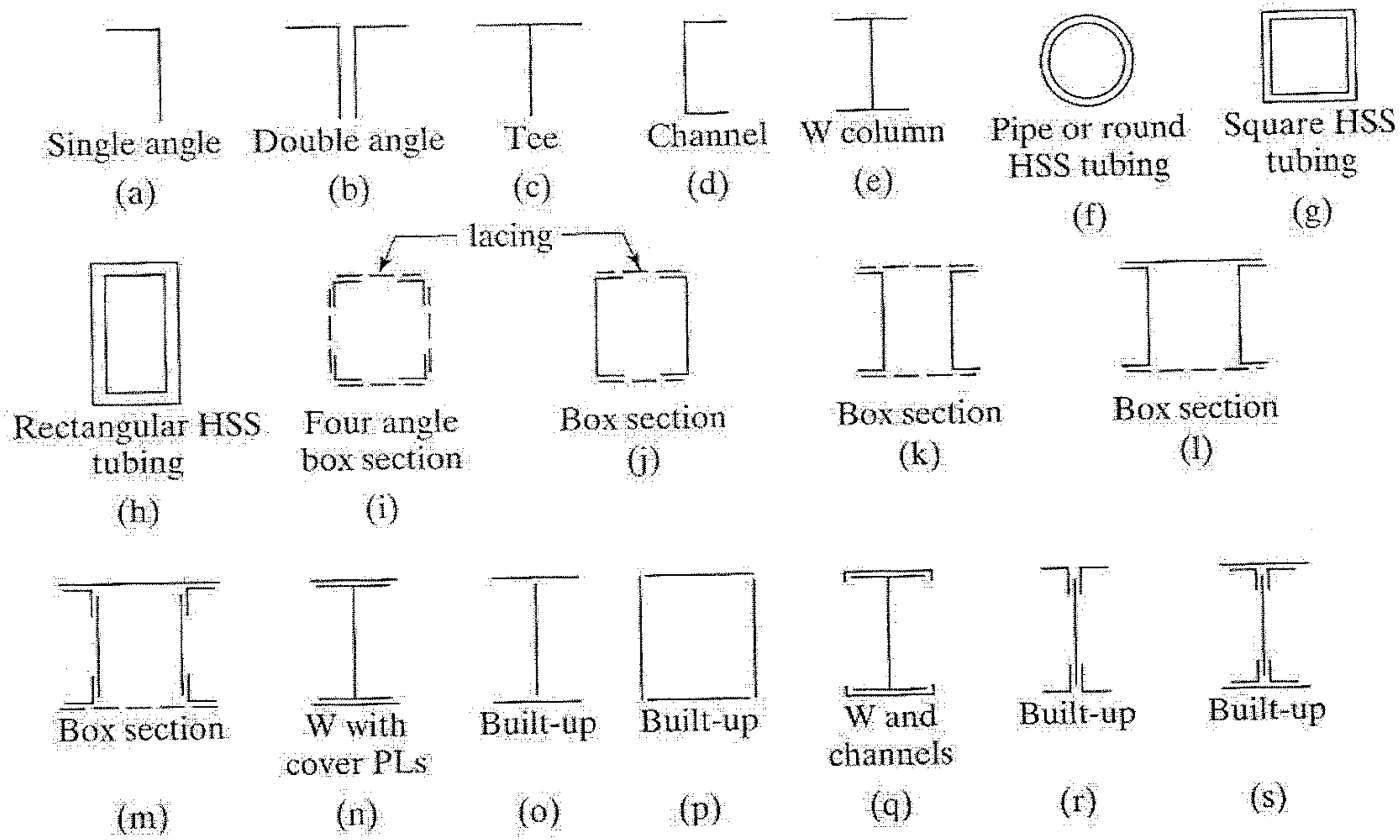

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LIMIT STATES OF BUCKLING

There are **two general modes** by which axially loaded steel columns can fail.

Local buckling: when some part or parts of the cross section of a column are so thin.

Global (member) buckling: Which are: *Flexural buckling, torsional buckling and flexural-torsional buckling.*

Flexural buckling: This type of buckling is caused by bending or flexure about the axis corresponding to the largest slenderness ratio (this is usually minor principal axis).

Torsional buckling: This type of buckling is caused by twisting about the longitudinal axis of the member. It can only occur with ***doubly symmetrical cross-sections*** with very slender cross-sectional elements. Standard hot-rolled shapes are not susceptible to torsional buckling.

Flexural-torsional buckling: This type of buckling is caused by a combination of flexural and torsional buckling. It can occur only with ***unsymmetrical cross-sections***(one axis of symmetry or no axis of symmetry).

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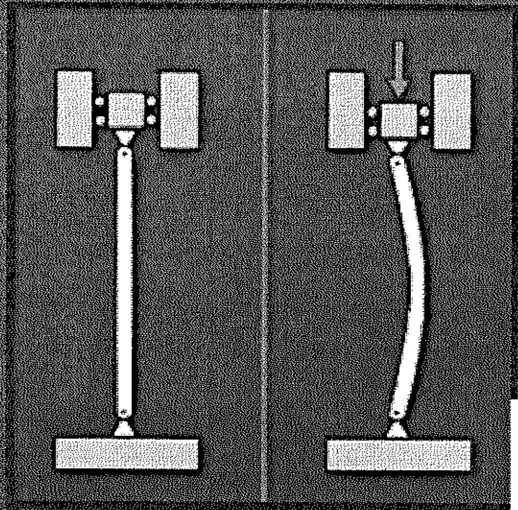


Flexural Buckling

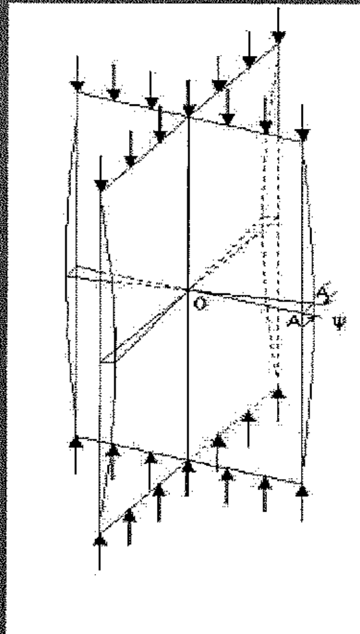
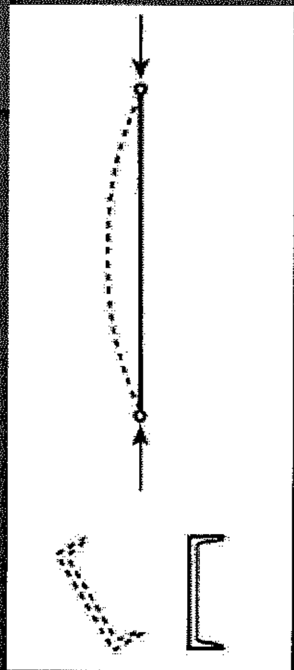


Figure 8: Torsional buckling of cruciform section

Torsional buckling



Torsional-flexural buckling

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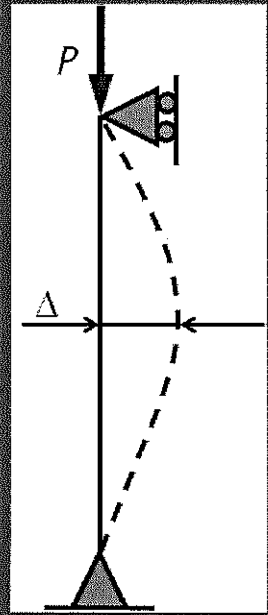
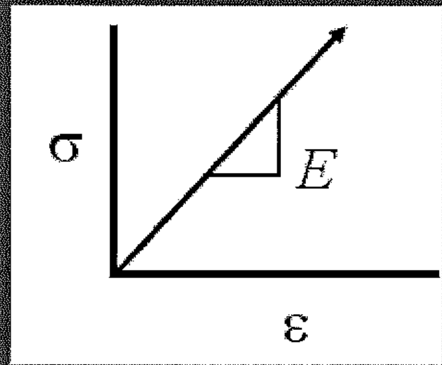
ELASTIC (EULER)BUCKLING
Flexural buckling
Column is,

- Pin-ended,
- prismatic and doubly symmetrical
- perfectly straight.
- Compressive load is applied along the centroidalaxis.
- There are no transverse loads.
- The material is homogeneous and obeys Hooke's Law.
- Plane sections before deformation remain plane after deformation.
- Deformations of the member are small,
- influence of shear on deformations is neglected and
- no twisting or distortion of the section occurs.





Critical buckling load, P_{cr}:

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

Critical buckling stress, F_{cr}:

$$F_{cr} = \frac{P_{cr}}{A} = \frac{P_E}{A} = \frac{\pi^2 EI}{AL^2} = \frac{\pi^2 E}{(L/r)^2} \qquad I = Ar^2$$

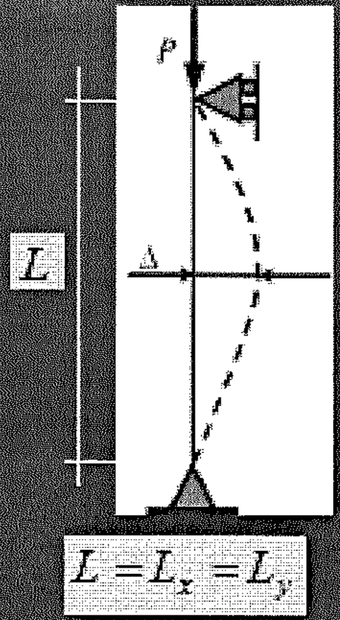
I: Moment of inertia

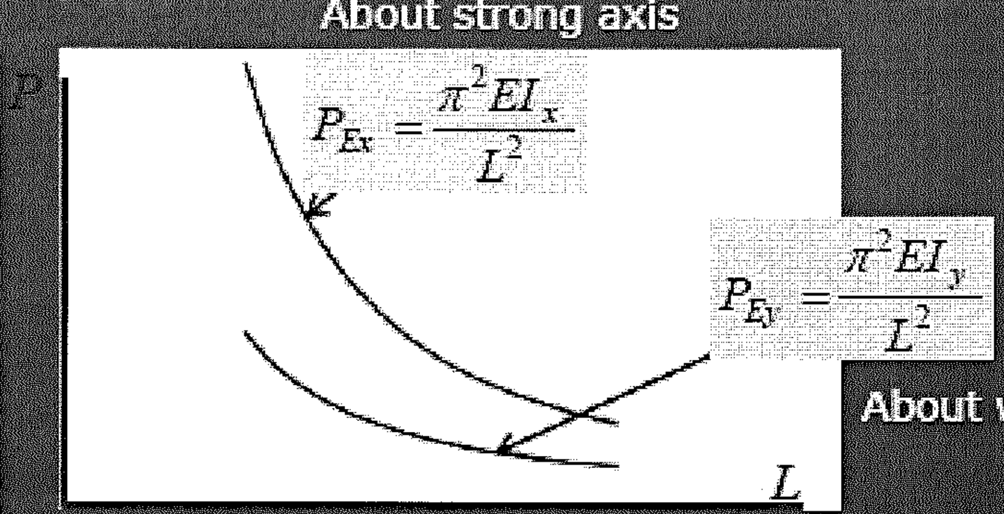
A: Cross-sectional area

r : Radius of gyration

(L / r): Slenderness ratio

Considering **typical elastic flexural buckling** of member with **I-shape, axial load – member length relationship** is shown in the following figure.





P_{Ex} and P_{Ey}	: Elastic buckling load
E	: Modulus of elasticity
I_x and I_y	: Principal moments of inertia about x and y axis
L	: Pin-ended column of length

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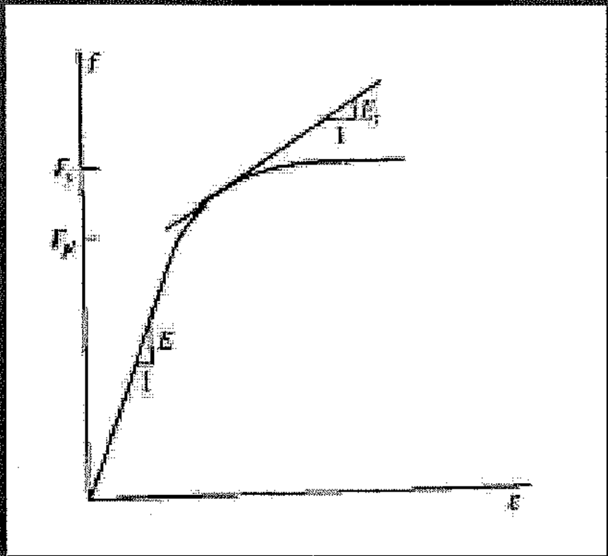
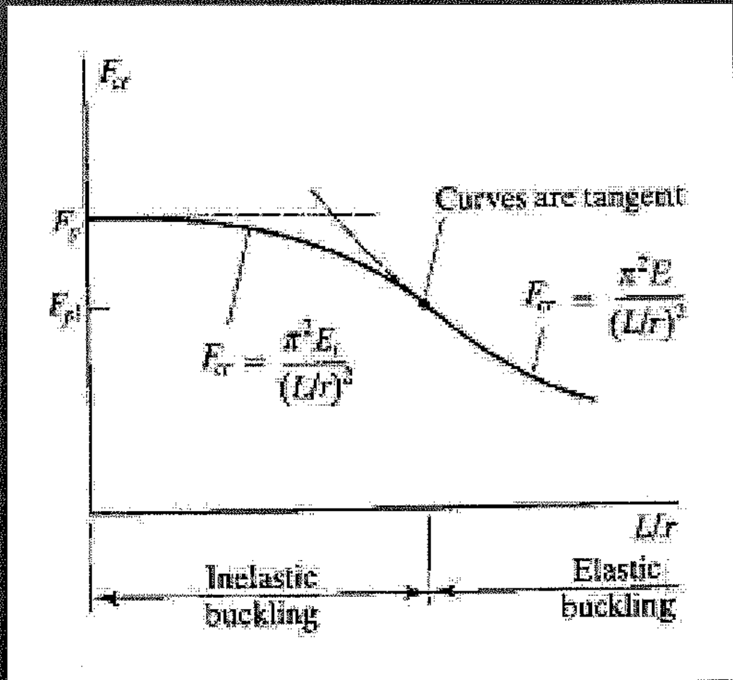
5.4. INELASTIC BUCKLING

If the stress at which buckling occurs is greater than the proportional limit of the material, the relation between stress and strain is not linear, and the modulus of elasticity E can no longer be used. Instead, variable modulus of elasticity, E_T (tangent modulus) must be used.

$$F_{cr} = \frac{\pi^2 E_t}{(L/r)^2}$$





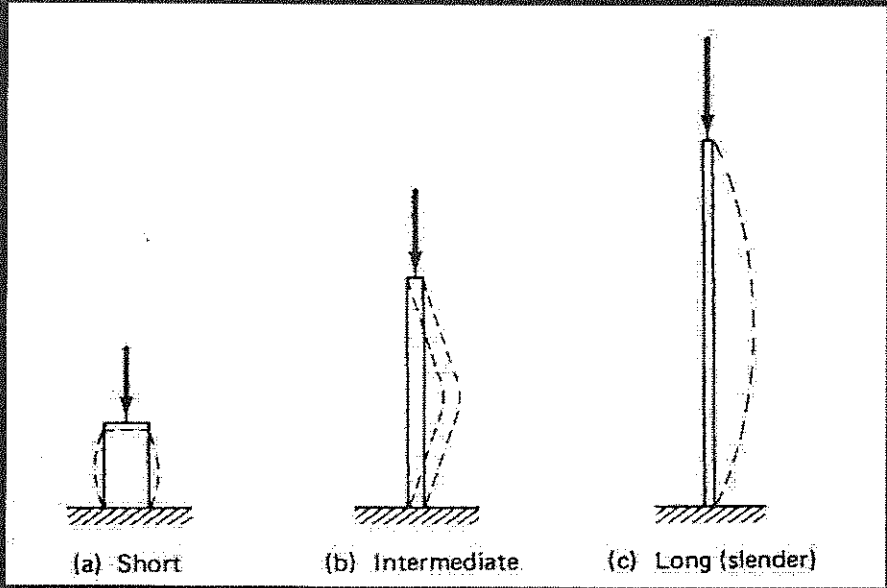
BEHAVIOUR OF COMPRESSION MEMBERS

Slender compression members (Long column):
If the axial load is slowly applied and increased, the member becomes unstable and is said to have buckled: Critical buckling load

If the member is stockier (Intermediate column), the larger load will be required to bring the member to the point of instability.

For extremely short members (Short column) failure may occure by yielding.

- ***Slender column***: It will fail by elastic buckling.
- ***Short column***: It will crush owing to yielding. Compressive stresses are in inelastic range.
- ***Intermediate column***: Falls between. It is analyzed and designed by using empirical formulas obtained by test results.



EFFECTIVE LENGTH (KL)

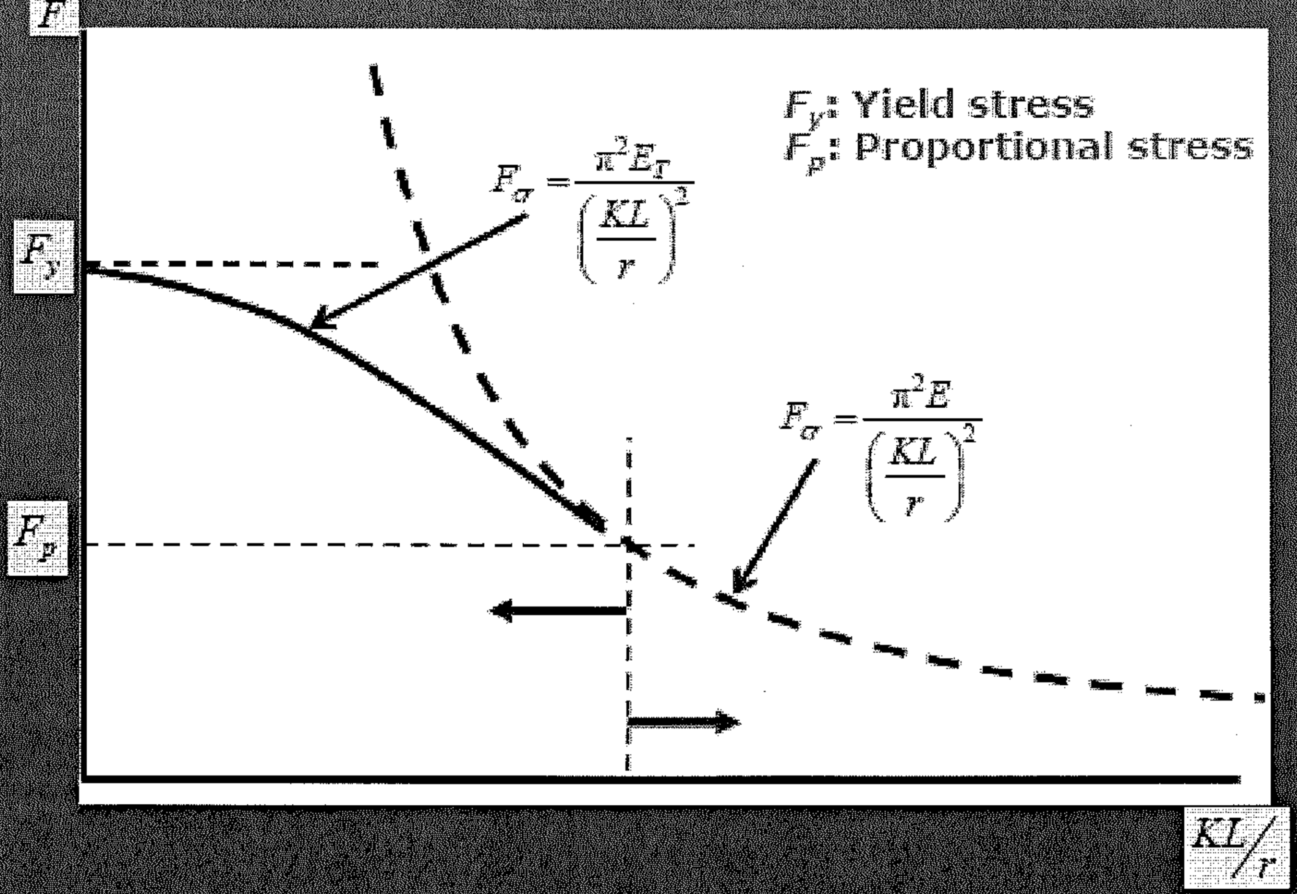
Dealing with boundary conditions the critical loads for elastic and inelastic buckling loads are:

$$P_{cr} = \frac{\pi^2 EI}{(KL)^2} \ldots \text{and} \ldots P_{cr} = \frac{\pi^2 E_t I}{(KL)^2}$$

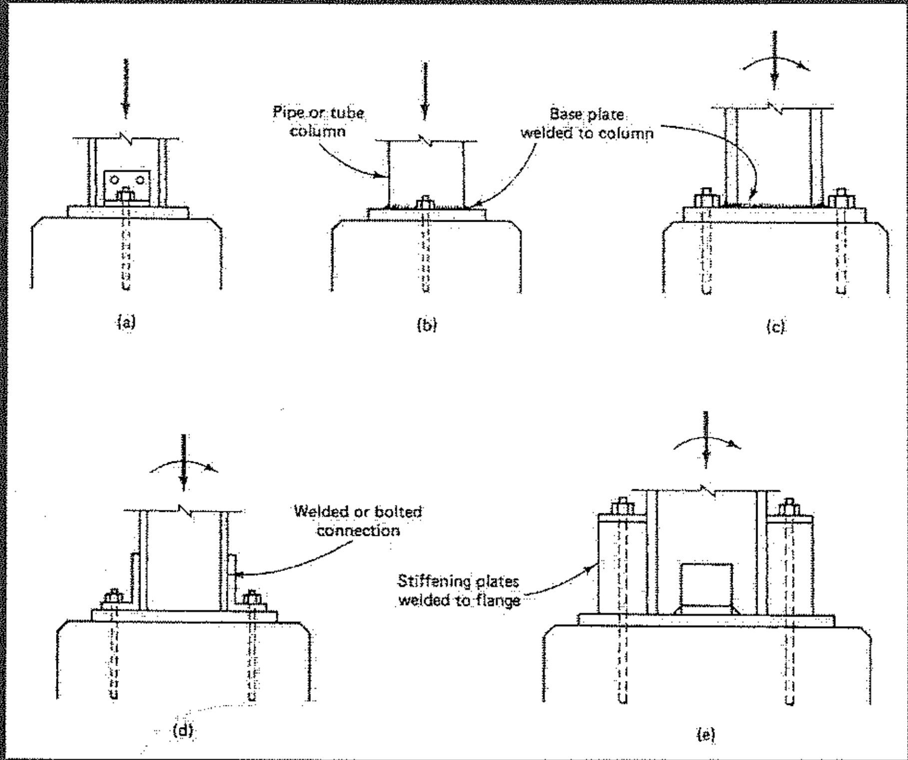
Critical buckling stresses are:

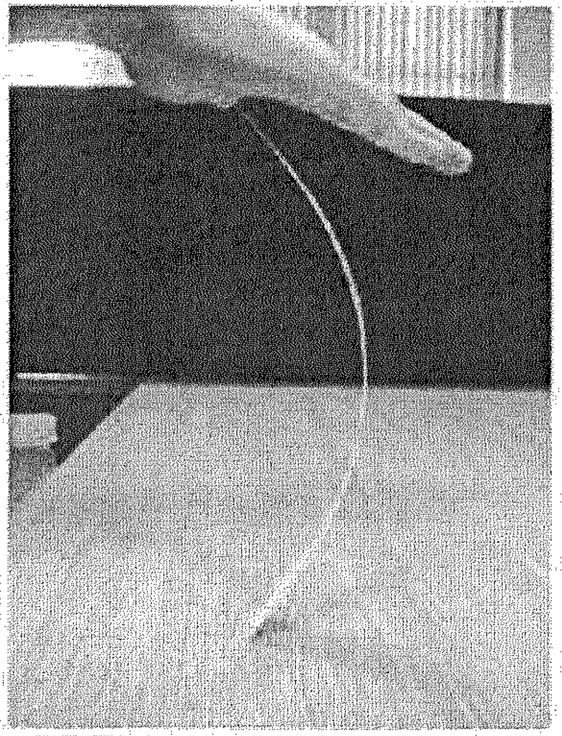
$$F_{cr} = \frac{\pi^2 E}{(KL/r)^2} \ldots \text{and} \ldots F_{cr} = \frac{\pi^2 E_t}{(KL/r)^2}$$

L : Actual length of the column
K : Effective length factor
(due to end conditions)
KL: Effective length of the column
Slenderness ratio: KL/r= λ

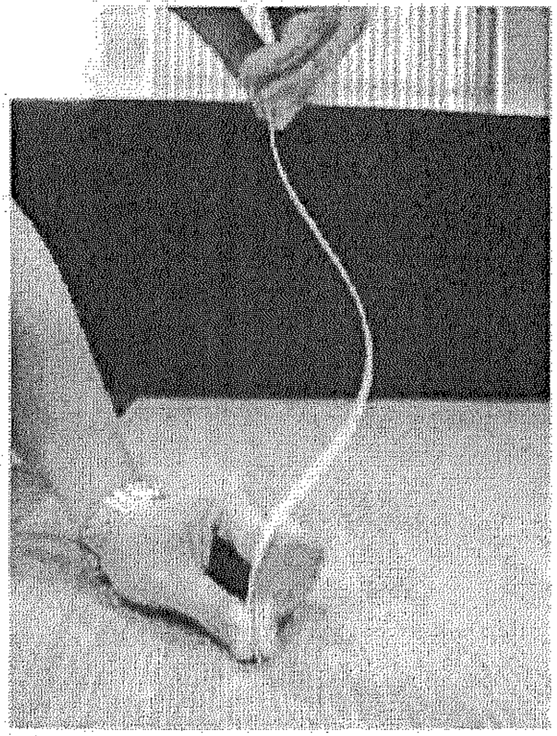

F
F_y
F_p
F_y: Yield stress
F_p: Proportional stress
$F_{cr} = \frac{\pi^2 E_T}{\left(\frac{KL}{r}\right)^2}$
$F_{cr} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$
KL/r

- (a) and (b): Very little resistance to end rotations.
- (c), (d) and (e):Sufficientresistance to end rotations.
- To apply Euler's formula to columns having other than pin ends,*effective lengths* are used.

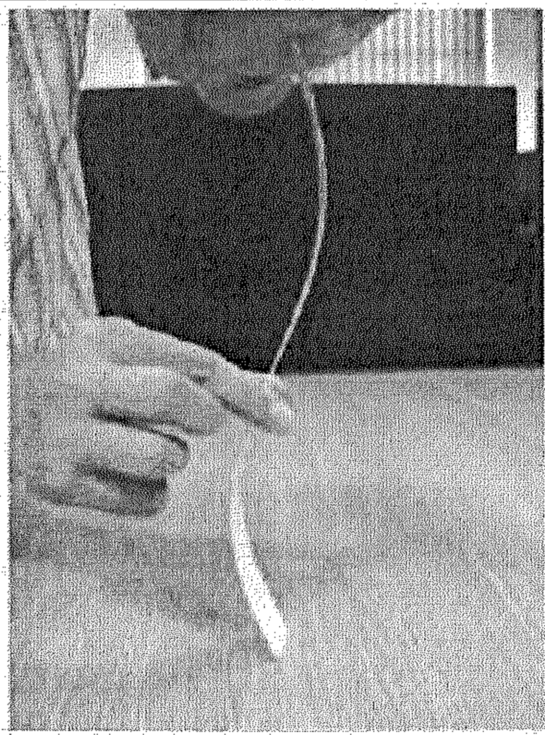




(a) Two pinned ends



(b) Two fixed ends



(c) Two pinned ends with a lateral support at the middle of the rule