جَامعة المَنارة
MANARA UNIVERSITY

Steel Structures 2
Summer Sem.
2023-2024

أ.د. نايل محمد حسن

Lecture 9-10

- Flexural Members

- ✓ -I- Laterally Restrained Beams
- ✓ II- Laterally Unrestrained Beams

- Beam-Column Members

Beam-Column Members

Introduction: Beam-Column Members

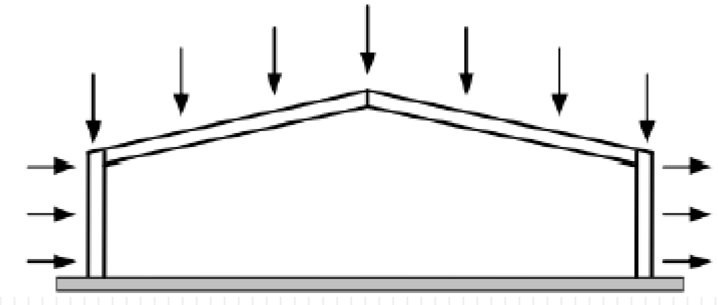
- **Axial force members are, in practice, subjected to axial load as well as bending in either or both the axis of the cross section.**

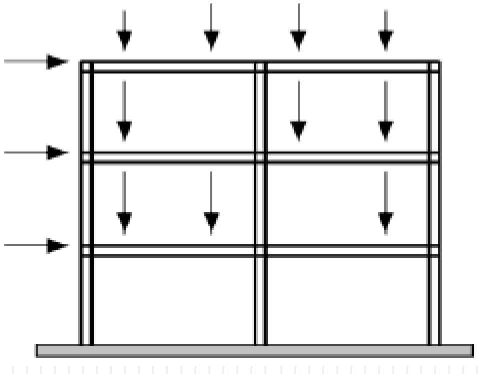
- **Similarly flexural members may also be subjected to axial load.**

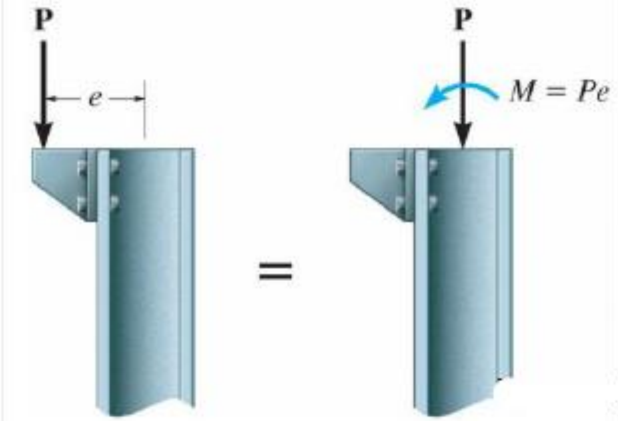
- **In either case, a member subjected to both significant axial and bending stresses is termed as Beam-Column Members.**

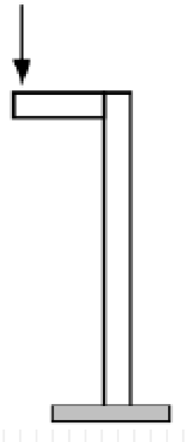
- **The behavior of such members results from the combination of both effects and varies with slenderness.**

Introduction: Beam-Column Members









A member subjected to both significant axial and bending stresses is termed as Beam-Column Members.

Introduction: Beam-Column Members

- **At low slenderness, the cross sectional resistance dominates.**
- **With increasing slenderness, pronounced second-order effects appear, significantly influenced by both geometrical imperfections and residual stresses.**
- **At high slenderness range, buckling is dominated by elastic behavior, failure tending to occur by flexural buckling (typical of members in pure compression) or by lateral-torsional buckling (typical of members in bending).**
  - **The behavior of a member under bending and axial force results from the interaction between instability and plasticity and is influenced by geometrical and material imperfections. Therefore very complex.**

Introduction: Beam-Column Members

The verification of the safety of members subject to bending and axial force is made in two steps:

- **Verification of the resistance of cross sections .**
- **Verification of the member buckling resistance (in general governed by flexural or lateral-torsional buckling).**

Cross Section Resistance : M-N interaction

Cross section resistance

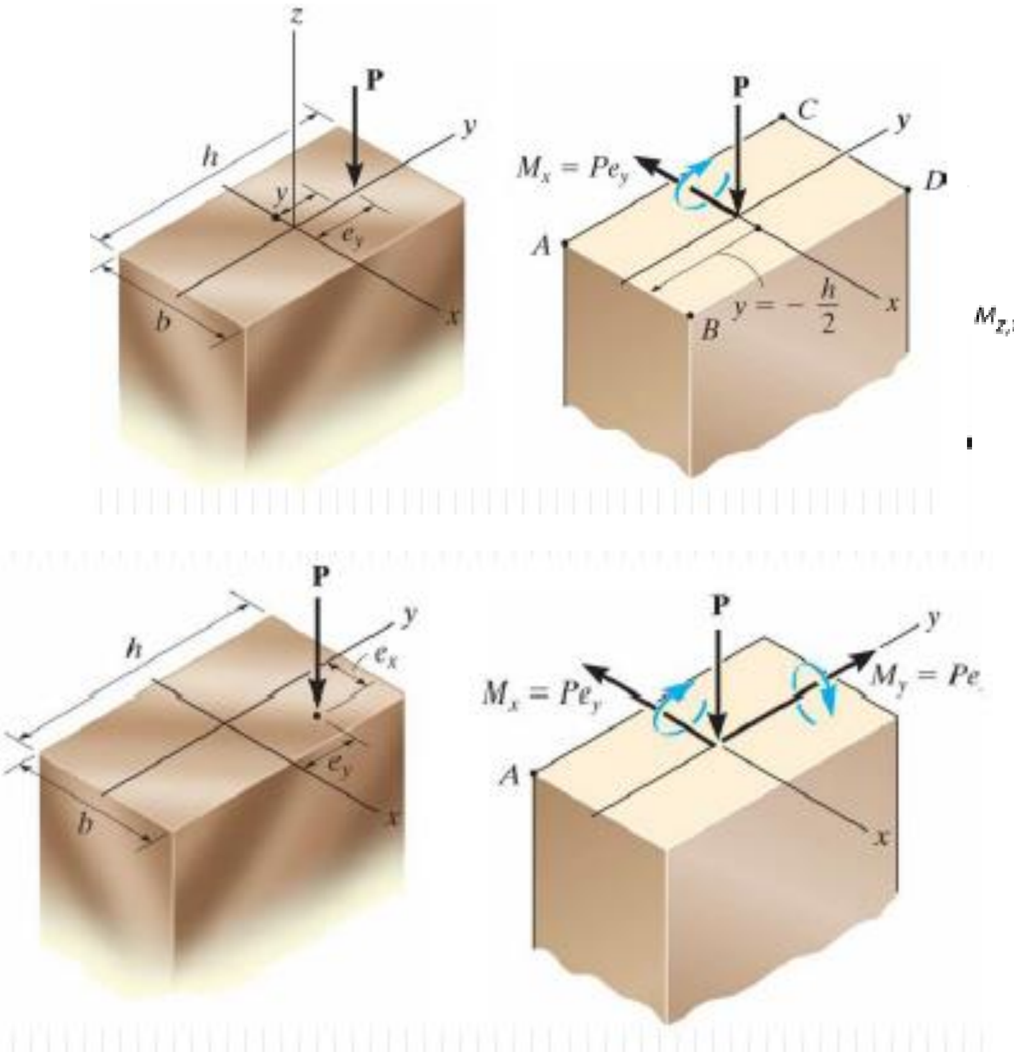
The cross section resistance is based;

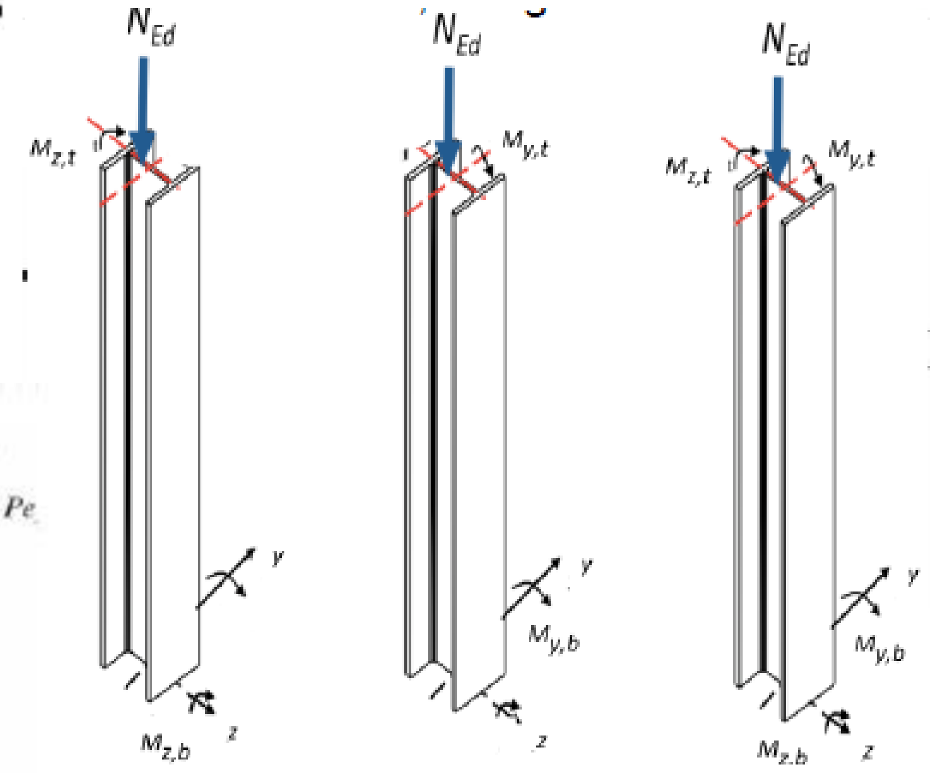
- **on its plastic capacity (class 1 or 2 sections) or**
- **on its elastic capacity (class 3 or 4 cross sections).**

When a cross section is subjected to bending moment and axial force ($N + M_y$, $N + M_z$ or even $N + M_y + M_z$),

the bending moment resistance should be reduced, using interaction formulas.

Cross Section Resistance : M-N interaction

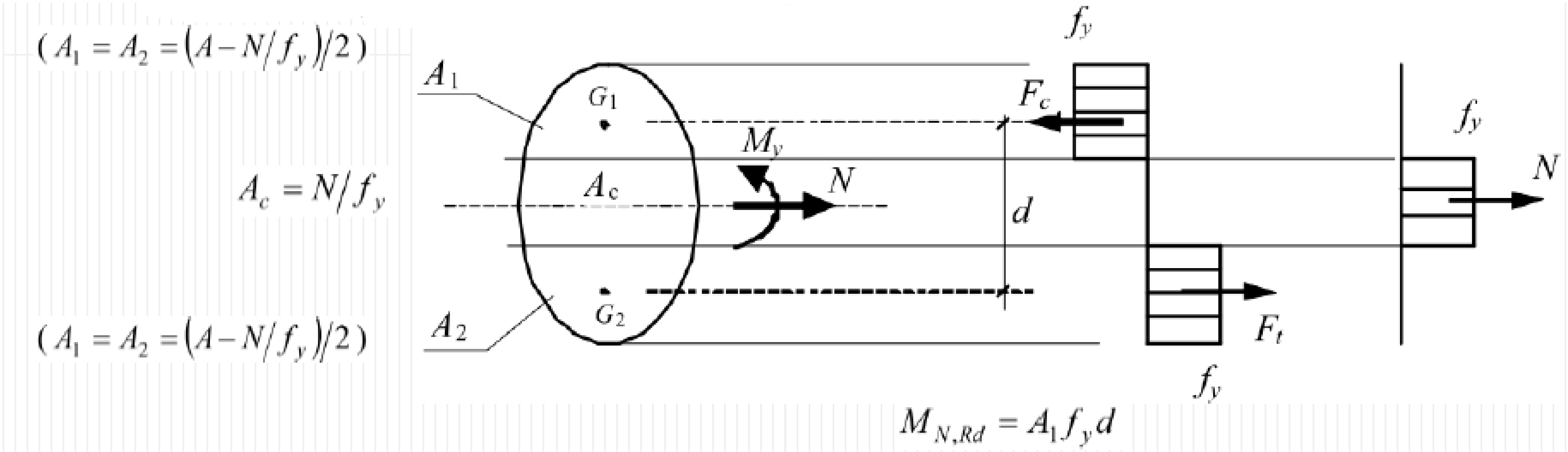




Cross Section Resistance : M-N interaction

- **The interaction formulae to evaluate the elastic cross section capacity are the well known formulae of simple beam theory, valid for any type of cross section.**
- **However, the formulae to evaluate the plastic cross section capacity are specific for each cross section shape.**
- **For a cross section subjected to N + M, a general procedure may be established to evaluate the plastic bending moment resistance $M_{N.Rd}$, reduced by the presence of an axial force N.**

Cross Section Resistance : M-N interaction



- **Although the interaction formulae are easy to obtain by applying the general method, the resulting formulae differ for each cross sectional shape and are often not straightforward to manipulate.**

Cross Section Resistance : M-N interaction

- **Historically, several approximate formulae have been developed, and, Villette (2004) proposed an accurate general formula, applicable to most standard cross sections. with an axis of symmetry with respect to the axis of bending, given by:**

$$\frac{M_{Ed}}{M_{pl,Rd}} + \left(\frac{N_{Ed}}{N_{pl,Rd}}\right)^{\alpha,plan} = 1.0 \qquad \alpha_{plan} = 1.0 + 1.82\sqrt{\left(\frac{k}{w_{pl}} - 1.01\right)\frac{k-1}{w_{pl}-1}} \; .$$

- **$w_{pl} = W_{pl}/W_e$ is the ratio between the plastic bending modulus and the elastic modulus,**
- **k=v/i is the ratio between the maximum distance v from an extreme fiber to the elastic neutral axis and the radius of gyration i of the section about the axis of bending.**

Cross Section Resistance : M-N interaction

- **For a circular hollow section, the following exact expression may be established (Lescouarc'h, 1977): :**

$$M_{N,Rd} = M_{pl,Rd} \sin \frac{\pi(1-n)}{2} \quad \text{where,} \quad n = N_{Ed} / N_{pl,Rd}$$

- **Interaction formulae for axial force and bi-axial bending have usually the following general format:**

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} = 1$$

For I or H cross sections subjected to N + My + Mz,

$$\alpha = \left(1.0 - 0.5\sqrt{n}\right)\alpha_{y,plan}$$

$$\beta = \frac{1+n}{1.0 - n^{(\alpha_{z,plan} - 0.5)}},$$

For RHS cross subjected to N + My + Mz,

$$\alpha = \beta = \frac{1.7}{1 - 1.13\,n^2} \qquad (\text{if } n < 0.8);$$

$$\alpha = \beta = 6 \qquad (\text{if } n \geq 0.8).$$

Cross Section Resistance : Design Resistance

EC1993-1-1 Provisions

Clause 6.2.9 provides several interaction formulae between bending moment and axial force, in the plastic range and in the elastic range. These are applicable to most cross sections. But in all case the following shall be satisfied;

Class 1 or 2 sections

$$M_{Ed} \leq M_{N,Rd}$$

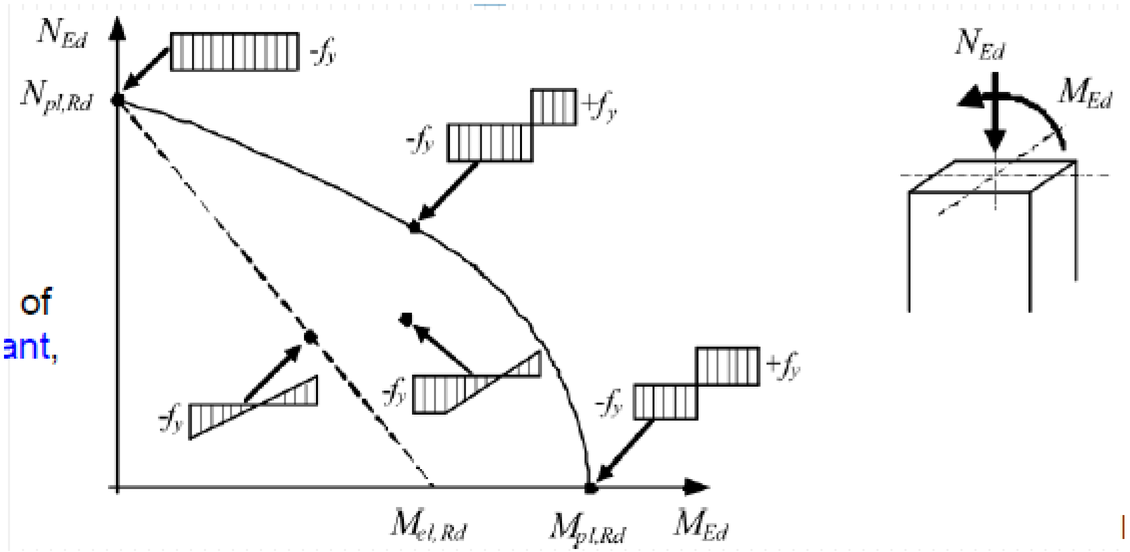
M_{Ed} is the design bending moment and $M_{N.Rd}$ represents the design plastic moment resistance reduced due to the axial force N_{Ed}

For rectangular solid sections under uni-axial bending and axial force, $M_{N,Rdis}$ given by

Cross Section Resistance : Design Resistance

For rectangular solid sections under uni-axial bending and axial force, $M_{N,Rdis}$ given by

$$M_{N,Rd} = M_{pl,Rd}\left[1-\left(\frac{N_{Ed}}{N_{pl,Rd}}\right)^2\right]$$



For low values of axial force, the reduction of the plastic moment resistance is not significant, as can be seen.

Cross Section Resistance : Design Resistance

For doubly symmetric I or H sections,

- It is not necessary to reduce the plastic moment resistance about y if the two following conditions are satisfied:

$$N_{Ed} \leq 0.25\, N_{pl,Rd} \quad \text{and} \quad N_{Ed} \leq 0.5\, h_w\, t_w\, f_y / \gamma_{M0}$$

- It is not necessary to reduce the plastic moment resistance about z if the following condition is verified:

$$N_{Ed} \leq h_w\, t_w\, f_y / \gamma_{M0}$$

For I or H sections, rolled or welded, with equal flanges and where fastener holes are not to be accounted for,

$$M_{N,y,Rd} = M_{pl,y,Rd} \frac{1-n}{1-0.5\,a} \quad \text{but} \quad M_{N,y,Rd} \leq M_{pl,y,Rd} \,;$$

$$M_{N,z,Rd} = M_{pl,z,Rd} \quad \text{if } n \leq a \,;$$

$$M_{N,z,Rd} = M_{pl,z,Rd} \left[1 - \left(\frac{n-a}{1-a} \right)^2 \right] \quad \text{if } n > a \,,$$

where, $a = (A - 2bt_f)/A$, but $a \leq 0.5$.

For circular hollow sections,

$$M_{N,Rd} = M_{pl,Rd} \left(1 - n^{1.7} \right)$$

Cross Section Resistance : Design Resistance

For RHS of uniform thickness and for welded box sections with equal flanges and equal webs and where fastener holes are not to be accounted for,

$$M_{N,y,Rd} = M_{pl,y,Rd} \frac{1-n}{1-0.5\,a_w} \quad \text{but} \quad M_{N,y,Rd} \leq M_{pl,y,Rd}$$

$$M_{N,z,Rd} = M_{pl,z,Rd} \frac{1-n}{1-0.5\,a_f} \quad \text{but} \quad M_{N,z,Rd} \leq M_{pl,z,Rd}$$

where $a_w \leq 0.5$ and $a_f \leq 0.5$ are the ratios between the area of the webs and of the flanges, respectively, and the gross area of the cross section.

In a cross section under bi-axial bending and axial force, the N + M_y + M_z interaction can be checked by the following condition:

$$\left[\frac{M_{y,Ed}}{M_{N,y,Rd}}\right]^{\alpha} + \left[\frac{M_{z,Ed}}{M_{N,z,Rd}}\right]^{\beta} \leq 1.0$$

where

α and β are parameters that depend on the shape of the cross section

I or H sections	$\alpha = 2;\ \beta = 5n$, but $\beta \geq 1$;
circular hollow sections	$\alpha = \beta = 2$;
rectangular hollow sections	$\alpha = \beta = \dfrac{1.66}{1-1.13\,n^2}$, but $\alpha = \beta \leq 6$.

Cross Section Resistance : Design Resistance

Class 3 or 4 cross sections

In class 3 or 4 cross sections, the interaction between bending and axial force requires that the following condition be checked:.

$$\sigma_{x,Ed} \leq \frac{f_y}{\gamma_{M0}}$$

where

$\sigma_{x,Ed}$ is the design value of the local longitudinal stress due to bending moment and axial force, taking into account the fastener holes where relevant. It is calculated based on the gross cross section for class 3 cross sections, and on a reduced effective cross section for class 4 sections.

Interaction of bending, axial and shear force

The interaction between bending, axial and shear force should be checked as follows :

- When $V_{Ed} \leq$ % 50 of the design plastic shear resistance $V_{Pl,Rd}$, no reduction need be made in the bendin and axial force resistances

- When $V_{Ed} >$ % 50 of the design plastic shear resistance $V_{Pl,Rd}$, then the design resistance to the combination of bending moment and axial force should be calculated using a reduced yield strength for the shea area. This reduced strength is given by $(1-\rho)f_y$, where $\rho=(2\ V_{Ed} / V_{Pl,Rd} -1)^2$

Cross Section Resistance : Design Resistance

Class 3 or 4 cross sections

In class 3 or 4 cross sections, the interaction between bending and axial force requires that the following condition be checked:.

$$\sigma_{x,Ed} \leq \frac{f_y}{\gamma_{M0}}$$

where

$\sigma_{x,Ed}$ is the design value of the local longitudinal stress due to bending moment and axial force, taking into account the fastener holes where relevant. It is calculated based on the gross cross section for class 3 cross sections, and on a reduced effective cross section for class 4 sections.

Interaction of bending, axial and shear force

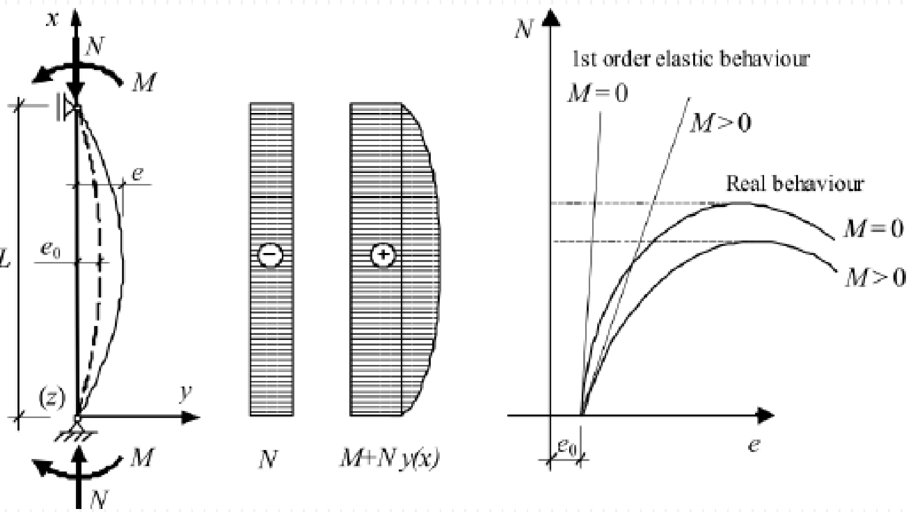
The interaction between bending, axial and shear force should be checked as follows :

- When $V_{Ed} \leq$ % 50 of the design plastic shear resistance $V_{Pl,Rd}$, no reduction need be made in the bendin and axial force resistances

- When $V_{Ed} >$ % 50 of the design plastic shear resistance $V_{Pl,Rd}$, then the design resistance to the combination of bending moment and axial force should be calculated using a reduced yield strength for the shea area. This reduced strength is given by $(1-\rho)f_y$, where $\rho=(2\ V_{Ed} / V_{Pl,Rd} - 1)^2$

Buckling Resistance: Introduction

For a member under bending and compression, besides the first-order moments and displacements (obtained based on the undeformed configuration), additional second-order moments and displacements exist ("P-δ" effects); these should be taken into account.



- In the past, various interaction formulae have been propo
to represent this situation over the full slenderness range.

- The present approach of EC3-1-1 is based on a linear-addi
interaction formula, illustrated by expression:

$$f\left(\frac{N}{N_u}, \frac{M_y}{M_{uy}}, \frac{M_z}{M_{uz}}\right) \leq 1.0$$

Where,
N, M_y and M_z are the applied forces and
N_u , M_{uy} and M_{uz} are the design resistances, that take in due acco
the associated instability phenomena.

Buckling Resistance: Design Resistance

The development of the design rules, and in particular those adopted by EC3-1-1, is quite complex, as they have to incorporate;

- two instability modes, flexural buckling and lateral-torsional buckling (or a combination of both),
- different cross sectional shapes and several shapes of bending moment diagram, among other aspects.
- several common concepts, such as that of equivalent moment, the definition of buckling length and the concept of amplification.

Several procedures provided in EC3-1-1 were described for the verification of the global stability of a steel structure, including the different ways of considering the second order effects (local P-δ effects and global P- Δ effects).

This topic is solely focused on dealing with the second order effect arising from local P-δ effects.

Buckling Resistance: Design Resistance

Local P-δ effects are generally taken into account according to the procedures given in clause 6.3 of EC3-1-1

Clause 6.3.3(1) considers two distinct situations

Members not susceptible to torsional deformation,

such as members of circular hollow section or other sections restrained from torsion.

Here, flexural buckling is the relevant instability mode.

Members that are susceptible to torsional deformations,

such as members of open section (I or H sections) that are not restrained from torsion.

Here, lateral torsional buckling tends to be the relevant instability mode.

Buckling Resistance: Design Resistance

Members which are subjected to combined bending and axial compression should satisfy the following condition given in clause 6.3.3 of EC3-1-1

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

About major axis y-y,

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

About minor axis z-z,

Where,

N_{Ed}, $M_{y,Ed}$ and $M_{z,Ed}$	are the design values of the compression force and the maximum moments about the y-y and z-z axis along the member, respectively
$\Delta M_{y,Ed}$, $\Delta M_{z,Ed}$	are the moments due to the shift of the centroidal axis on a reduced effective class 4 cross section
χ_y and χ_z	are the reduction factors due to flexural buckling
χ_{LT}	is the reduction factor due to lateral torsional buckling
k_{yy}, k_{yz}, k_{zy}, k_{zz}	are the interaction factors

Buckling Resistance: Design Resistance

Members which are subjected to combined bending and axial compression should satisfy the following condition given in clause 6.3.3 of EC3-1-1

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

About major axis y-y,

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1$$

About minor axis z-z,

Where,

Values for $N_{Rk} = f_y A_i$, $M_{i,Rk} = f_y W_i$ and $\Delta M_{i,Ed}$

Class	1	2	3	4
A_i	A	A	A	A_{eff}
W_y	$W_{pl,y}$	$W_{pl,y}$	$W_{el,y}$	$W_{eff,y}$
W_z	$W_{pl,z}$	$W_{pl,z}$	$W_{el,z}$	$W_{eff,z}$
$\Delta M_{y,Ed}$	0	0	0	$e_{N,y}\, N_{Ed}$
$\Delta M_{z,Ed}$	0	0	0	$e_{N,z}\, N_{Ed}$

Buckling Resistance: Design Resistance-interaction factors

In EC3-1-1 two methods are given for the calculation of the interaction factors k_{yy} , k_{yz} , k_{zy} and k_{zz}.

Regardless of the method to be applied;

- In members that are not susceptible to torsional deformation, it is assumed that there is no risk of lateral torsional buckling (χ_{LT} = 1.0). And calculating the interaction factors k_{yy} , k_{yz} , k_{zy} and k_{zz} for a member not susceptible to torsional deformation.

Method 1, developed by a group of French and Belgian researchers,

According to this method , a member is not susceptible to torsional deformations if

- $I_T \geq I_Y$, or
- Incase $I_T < I_Y$, but the following condition is satisfied. $\bar{\lambda}_0 \leq 0.2\sqrt{C_1}\sqrt[4]{\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\left(1-\frac{N_{Ed}}{N_{cr,T}}\right)}$,

Where,

C_1 is a coefficient that depends on the shape of the bending moment diagram between laterally braced sections

$N_{cr,z}$ and $N_{cr,T}$ represent the elastic critical loads for flexural buckling about z and for torsional buckling, respectively

λ_0 is the non dimensional slenderness coefficient for lateral torsional buckling, assessed for a situation with constant bending moment.

Buckling Resistance: Design Resistance-interaction factors

Method 1, developed by a group of French and Belgian researchers,

Annex A of EC3-1-1 presents Tables, for the calculation of the interaction factors according to Method 1

Interaction factors	Elastic sectional properties (Class 3 or 4 sections)	Plastic sectional properties (Class 1 or 2 sections)
k_{yy}	$C_{my}C_{mLT}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,y}}}$	$C_{my}C_{mLT}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,y}}}\dfrac{1}{C_{yy}}$
k_{yz}	$C_{mz}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{mz}\dfrac{\mu_y}{1-\dfrac{N_{Ed}}{N_{cr,z}}}\dfrac{1}{C_{yz}}0.6\sqrt{\dfrac{w_z}{w_y}}$
k_{zy}	$C_{my}C_{mLT}\dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,y}}}$	$C_{my}C_{mLT}\dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,y}}}\dfrac{1}{C_{zy}}0.6\sqrt{\dfrac{w_y}{w_z}}$
k_{zz}	$C_{mz}\dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{mz}\dfrac{\mu_z}{1-\dfrac{N_{Ed}}{N_{cr,z}}}\dfrac{1}{C_{zz}}$

Auxiliary terms:

$$\mu_y=\frac{1-\dfrac{N_{Ed}}{N_{cr,y}}}{1-\chi_y\dfrac{N_{Ed}}{N_{cr,y}}};\quad \mu_z=\frac{1-\dfrac{N_{Ed}}{N_{cr,z}}}{1-\chi_z\dfrac{N_{Ed}}{N_{cr,z}}};\quad w_y=\frac{W_{pl,y}}{W_{el,y}}\le 1.5;\quad w_z=\frac{W_{pl,z}}{W_{el,z}}\le 1.5$$

$n_{pl}=\dfrac{N_{Ed}}{N_{Rk}/\gamma_{M1}}$; $a_{LT}=1-\dfrac{I_T}{I_y}\ge 0$; C_{my} and C_{mz} are factors of equivalent uniform moment, determined by the table on the slide # 26,

For class 3 or 4, consider $w_y=w_z=1.0$.

Buckling Resistance: Design Resistance-interaction factors

Method 1, developed by a group of French and Belgian researchers,

Annex A of EC3-1-1 presents Tables, for the calculation of the interaction factors according to Method 1

$$C_{yy} = 1 + (w_y - 1)\left[\left(2 - \frac{1.6}{w_y} C_{my}^2 \bar{\lambda}_{max} - \frac{1.6}{w_y} C_{my}^2 \bar{\lambda}_{max}^2\right) n_{pl} - b_{LT}\right] \geq \frac{W_{el,y}}{W_{pl,y}},$$

where $b_{LT} = 0.5\, a_{LT}\, \bar{\lambda}_0^2 \dfrac{M_{y,Ed}}{\chi_{LT}\, M_{pl,y,Rd}} \dfrac{M_{z,Ed}}{M_{pl,z,Rd}}$.

$$C_{yz} = 1 + (w_z - 1)\left[\left(2 - 14\frac{C_{mz}^2 \bar{\lambda}_{max}^2}{w_z^5}\right) n_{pl} - c_{LT}\right] \geq 0.6\sqrt{\frac{w_z}{w_y}} \frac{W_{el,z}}{W_{pl,z}},$$

where $c_{LT} = 10\, a_{LT} \dfrac{\bar{\lambda}_0^2}{5 + \bar{\lambda}_z^4} \dfrac{M_{y,Ed}}{C_{my}\, \chi_{LT}\, M_{pl,y,Rd}}$.

$$C_{zy} = 1 + (w_y - 1)\left[\left(2 - 14\frac{C_{my}^2 \bar{\lambda}_{max}^2}{w_y^5}\right) n_{pl} - d_{LT}\right] \geq 0.6\sqrt{\frac{w_y}{w_z}} \frac{W_{el,y}}{W_{pl,y}},$$

where $d_{LT} = 2\, a_{LT} \dfrac{\bar{\lambda}_0}{0.1 + \bar{\lambda}_z^4} \dfrac{M_{y,Ed}}{C_{my}\, \chi_{LT}\, M_{pl,y,Rd}} \dfrac{M_{z,Ed}}{C_{mz}\, M_{pl,z,Rd}}$.

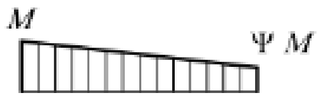
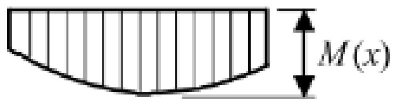
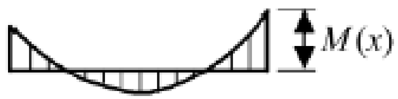
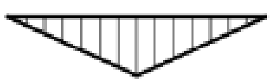
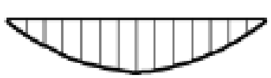
$$C_{zz} = 1 + (w_z - 1)\left[\left(2 - \frac{1.6}{w_z} C_{mz}^2 \bar{\lambda}_{max} - \frac{1.6}{w_z} C_{mz}^2 \bar{\lambda}_{max}^2\right) - e_{LT}\right] n_{pl} \geq \frac{W_{el,z}}{W_{pl,z}}, \quad 4$$

where $e_{LT} = 1.7\, a_{LT} \dfrac{\bar{\lambda}_0}{0.1 + \bar{\lambda}_z^4} \dfrac{M_{y,Ed}}{C_{my}\, \chi_{LT}\, M_{pl,y,Rd}}$.

Buckling Resistance: Design Resistance-interaction factors

Method 1, developed by a group of French and Belgian researchers,

Annex A of EC3-1-1 presents Tables, for the calculation of the interaction factors according to Method 1

Diagram of moments	$C_{mi,0}$
	$C_{mi,0} = 0.79 + 0.21\Psi_i + 0.36(\Psi_i - 0.33)\frac{N_{Ed}}{N_{cr,i}}$
 	$C_{mi,0} = 1 + \left(\frac{\pi^2 E I_i \lvert\delta_x\rvert}{L^2 \lvert M_{i,Ed}(x)\rvert} - 1\right)\frac{N_{Ed}}{N_{cr,i}}$ $M_{i,Ed}(x)$ is the maximum moment $M_{y,Ed}$ or $M_{z,Ed}$ according to the first order analyses $\lvert\delta_x\rvert$ is the maximum lateral deflection δ_z (due to $M_{y,Ed}$) or δ_y (due to $M_{z,Ed}$) along the member
	$C_{mi,0} = 1 - 0.18\frac{N_{Ed}}{N_{cr,i}}$
	$C_{mi,0} = 1 + 0.03\frac{N_{Ed}}{N_{cr,i}}$

Buckling Resistance: Design Resistance-interaction factors

Method 1, developed by a group of French and Belgian researchers,

Annex A of EC3-1-1 presents Tables, for the calculation of the interaction factors according to Method 1

Auxiliary terms (continuation):

$\bar{\lambda}_{max} = \max(\bar{\lambda}_y, \bar{\lambda}_z)$;

$\bar{\lambda}_0 =$ non dimensional slenderness for lateral torsional buckling due to uniform bending moment, that is, taking $\Psi_y = 1.0$ in Table 3.15;

$\bar{\lambda}_{LT} =$ non dimensional slenderness for lateral torsional buckling;

If $\bar{\lambda}_0 \le 0.2\sqrt{C_1}\sqrt[4]{\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\left(1-\frac{N_{Ed}}{N_{cr,T}}\right)}$: $C_{my} = C_{my,0}$; $C_{mz} = C_{mz,0}$; $C_{mLT} = 1.0$;

If $\bar{\lambda}_0 > 0.2\sqrt{C_1}\sqrt[4]{\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\left(1-\frac{N_{Ed}}{N_{cr,T}}\right)}$: $C_{my} = C_{my,0} + (1-C_{my,0})\frac{\sqrt{\varepsilon_y}\, a_{LT}}{1+\sqrt{\varepsilon_y}\, a_{LT}}$;

$$C_{mz} = C_{mz,0};\; C_{mLT} = C_{my}^2 \frac{a_{LT}}{\sqrt{\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\left(1-\frac{N_{Ed}}{N_{cr,T}}\right)}} \ge 1;$$

$\varepsilon_y = \frac{M_{y,Ed}}{N_{Ed}}\frac{A}{W_{el,y}}$ for class 1, 2 or 3 cross sections;

$\varepsilon_y = \frac{M_{y,Ed}}{N_{Ed}}\frac{A_{eff}}{W_{eff,y}}$ for class 4 cross sections;

$N_{cr,y}$ is the elastic critical load for flexural buckling about y;

$N_{cr,z}$ is the elastic critical load for flexural buckling about z;

$N_{cr,T}$ is the critical load for torsional buckling;

I_T is the constant of uniform torsion or St. Venant's torsion;

I_y is the second moment of area about y;

$C_1 = \left(\frac{1}{k_c}\right)^2$ where k_c is taken from Table 3.10.

Buckling Resistance: Design Resistance-interaction factors

Method 2, developed by a group of Austrian and German researchers,

According to Method 2, the following members may be considered as not susceptible to torsional deformation:

- members with circular hollow sections (CHS).
- members with rectangular hollow sections (RHS) (there is widly argued exception to this rule presented in (
- members with open cross section, provided that they are torsionally and laterally restrained.

Annex B of EC3-1-1 presents Tables, for the calculation of the interaction factors according to Method 2

Interaction factors k_{ij} in members not susceptible to torsional deformations according to Method 2

Interaction factors	Typeof section	Elastic sectional properties (Class 3 or 4 sections)	Plastic sectional properties (Class 1 or 2 sections)
k_{yy}	I or H sections and rectangular hollow sections	$C_{my}\left(1+0.6\bar{\lambda}_y\dfrac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{my}\left(1+0.6\dfrac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$	$C_{my}\left(1+(\bar{\lambda}_y-0.2)\dfrac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{my}\left(1+0.8\dfrac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$
k_{yz}	I or H sections and rectangular hollow sections	k_{zz}	$0.6k_{zz}$

Buckling Resistance: Design Resistance-interaction factors

Method 2, developed by a group of Austrian and German researchers,

Annex B of EC3-1-1 presents Tables, for the calculation of the interaction factors according to Method 2

Interaction factors k_{ij} in members not susceptible to torsional deformations according to Method 2

k_{zz}	I or H sections	$C_{mz}\left(1+0.6\,\bar{\lambda}_z\dfrac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$	$C_{mz}\left(1+\left(2\bar{\lambda}_z-0.6\right)\dfrac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{mz}\left(1+1.4\dfrac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
	rectangular hollow sections	$\leq C_{mz}\left(1+0.6\dfrac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$	$C_{mz}\left(1+\left(\bar{\lambda}_z-0.2\right)\dfrac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{mz}\left(1+0.8\dfrac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$

In I or H sections and rectangular hollow sections under axial compression and uniaxial bending ($M_{y,Ed}$), k_{zy} may be taken as zero.

Buckling Resistance: Design Resistance-interaction factors

Method 2, developed by a group of Austrian and German researchers,

Annex B of EC3-1-1 presents Tables, for the calculation of the interaction factors according to Method 2

Interaction factors k_{ij} in members susceptible to torsional deformations according to Method 2

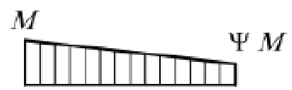
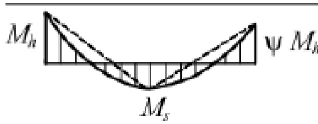
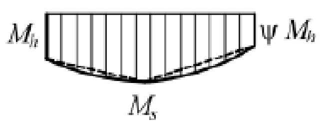
Interaction factors	Elastic sectional properties (Class 3 or 4 sections)	Plastic sectional properties (Class 1 or 2 sections)
k_{yy}	k_{yy} of Table 3.16	k_{yy} of Table 3.16
k_{yz}	k_{yz} of Table 3.16	k_{yz} of Table 3.16
k_{zy}	$\left[1-\frac{0.05\bar{\lambda}_z}{(C_{mLT}-0.25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ $\geq\left[1-\frac{0.05}{(C_{mLT}-0.25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$	$\left[1-\frac{0.1\bar{\lambda}_z}{(C_{mLT}-0.25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ $\geq\left[1-\frac{0.1}{(C_{mLT}-0.25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ for $\bar{\lambda}_z<0.4$: $k_{zy}=0.6+\bar{\lambda}_z$ $\leq 1-\frac{0.1\bar{\lambda}_z}{(C_{mLT}-0.25)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}$
k_{zz}	k_{zz} of Table 3.16	k_{zz} of Table 3.16

Buckling Resistance: Design Resistance-interaction factors

Method 2, developed by a group of Austrian and German researchers,

Annex B of EC3-1-1 presents Tables, for the calculation of the interaction factors according to Method 2

Equivalent factors of uniform moment C_{mi} according to Method 2

Diagram of moments	Range		C_{my}, C_{mz} and C_{mLT}	
			Uniform loading	Concentrated load
	$-1 \le \Psi \le 1$		$0.6+0.4\Psi \ge 0.4$	
 $\alpha_s = M_s / M_h$	$0 \le \alpha_s \le 1$	$-1 \le \Psi \le 1$	$0.2+0.8\alpha_s \ge 0.4$	$0.2+0.8\alpha_s \ge 0.4$
	$-1 \le \alpha_s < 0$	$0 \le \Psi \le 1$	$0.1-0.8\alpha_s \ge 0.4$	$-0.8\alpha_s \ge 0.4$
		$-1 \le \Psi < 0$	$0.1(1-\Psi)-0.8\alpha_s \ge 0.4$	$0.2(-\Psi)-0.8\alpha_s \ge 0.4$
 $\alpha_h = M_h / M_s$	$0 \le \alpha_h \le 1$	$-1 \le \Psi \le 1$	$0.95+0.05\alpha_h$	$0.90+0.10\alpha_h$
	$-1 \le \alpha_h < 0$	$0 \le \Psi \le 1$	$0.95+0.05\alpha_h$	$0.90+0.10\alpha_h$
		$-1 \le \Psi < 0$	$0.95+0.05\alpha_h(1+2\Psi)$	$0.90+0.10\alpha_h(1+2\Psi)$

In the calculation of α_s or α_h parameters, a hogging moment should be taken as negative and a sagging moment should be taken as positive.

Buckling Resistance: Design Resistance-interaction factors

Method 2, developed by a group of Austrian and German researchers,

Annex B of EC3-1-1 presents Tables, for the calculation of the interaction factors according to Method 2

For members with sway buckling mode, the equivalent uniform moment factor should be taken as $C_{my} = 0.9$ or $C_{mz} = 0.9$, respectively.

Factors C_{my}, C_{mz} and C_{mLT} should be obtained from the diagram of bending moments between the relevant braced sections, according to the following:

Moment factor	bending axis	points braced in direction
C_{my}	y-y	z-z
C_{mz}	z-z	y-y
C_{mLT}	y-y	y-y

Equivalent factors of uniform moment C_{mi} according to Method 2

Design According to EC3:
Section classification for sections under bending and axial force

According to EC3, the classification of a cross section is based on its maximum resistance to the type of applied internal forces, independent from their values.

- This procedure is straightforward to apply for cross sections subjected to either bending or compression.
- However, the presence of both the compression and bending moment on the cross-section member, generates a stress distribution between that related to pure compression and that associated with the presence of the sole bending moment.
- Bearing in mind this additional complexity, simplified procedures are often adopted, such as:

i. to consider the cross section subjected to compression only, being the most unfavourable situation (too conservative in some cases)

ii. to classify the cross section based on an estimate of the position of the neutral axis based on the applied internal forces.

- In the later case the neutral axis depth depends on whether the section can plastify, the bending axis, the section profile.

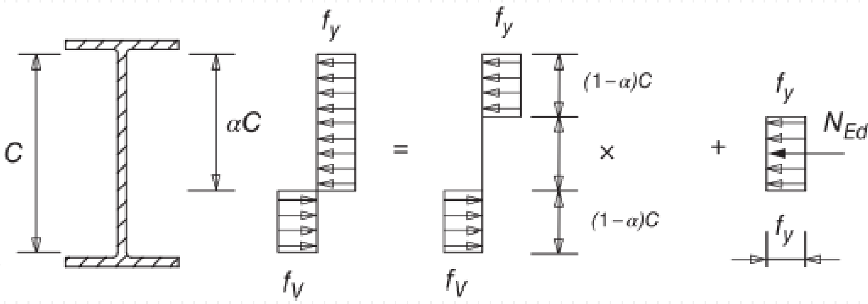
Design According to EC3:
Section classification for sections under bending and axial force

For Bending and Compression about a strong Axis (y-y).

Normal stress distribution on the web depends on the value of the design axial load by means of parameter **α** for profiles able to resist in the plastic range (classes 1 and 2).

Applying Section Equilibrium and Super positioning

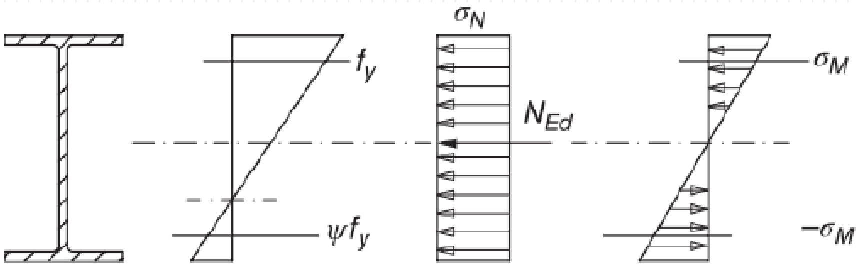
$$\alpha = \frac{1}{2}\left(1 + \frac{1}{c}\cdot\frac{N_{Ed}}{t_w f_y}\right)$$



in case of elastic normal stress distribution, reference has to be made to parameter **ψ** (classes 3 and 4).

Applying Section Equilibrium and Super positioning

$$\psi = 2\frac{N_{Ed}}{A\, f_y} - 1$$



With reference to the case of a neutral axis located in the web, α ranges between 0.5 (bending) and 1 (compression) and ψ ranges between −1 (bending) and 1 (compression).

Once the stress distribution is assumed and the values of α and ψ can be used to classify the section using tables 5.2 (sheet1 through 3)

Steel Structures 2
Summer Sem
2023-2024

أ.د. نايل محمد حسن

Lecture 11-12

- Flexural Members

- ✓ -I- Laterally Restrained Beams
- ✓ II- Laterally Unrestrained Beams

- **Beam-Column Members**
- **Beam-Column Members (problems))**

Beam-Column Members

(Problems)

Worked Problem1

Check the design of a UC 203 x 203 x 71 in steel S275 with 9 m long is used as a vertical member in a braced frame. The design loads and end moments are as shown in figure (A), The bending moments are about the axis as shown in figure (B). There is a bracing at mid height of the member from both sides as shown in the figure (A). Assume the end connections are pins.

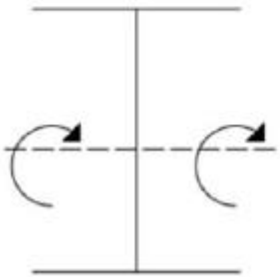
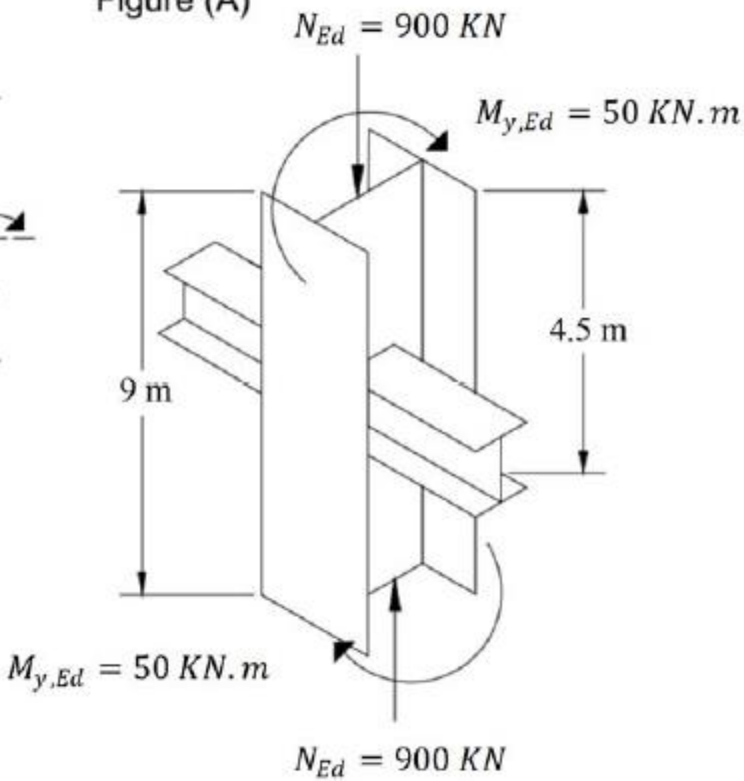


Figure (B)

Figure (A)



Worked Problem1

Solution:

1- Geometrical properties:

$h = 215.8\ mm; b_f = 206.4\ mm; t_f = 17.3\ mm; t_w = 10\ mm$

$A = 90.4\ cm^2; r = 10.2\ mm; W_{PL,y} = 799\ cm^3; W_{PL,z} = 374\ cm^3$

$I_y = 7620\ cm^4; I_z = 2540\ cm^4; i_y = 9.18\ cm; i_z = 5.3\ cm$

$I_T = 80.2 cm^4; I_W = 0.25\ dm^6; E = 210\ KN/mm^2; G = 81\ KN/mm^2$

2- Cross Section Classification:

For flange: (compression)

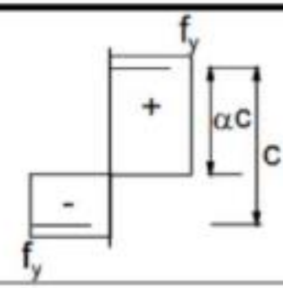
$\left(\frac{c}{t}\right) = 5.09$

$9\epsilon = 9 \times 0.92 = 8.28 > 5.09 \rightarrow\rightarrow class\ 1$

For web: (bending and compression)

$\left(\frac{c}{t}\right) = 16.1$, $\alpha = 0.5$ (symmetry stress)

$\frac{36\epsilon}{0.5} = 36 \times \frac{0.92}{0.5} = 66.24 > 16.1 \rightarrow\rightarrow class\ 1$

Part subject to bending and compression

when $\alpha > 0{,}5:\ c/t \le \frac{396\varepsilon}{13\alpha - 1}$
when $\alpha \le 0{,}5:\ c/t \le \frac{36\varepsilon}{\alpha}$

The cross section is class 1

Worked Problem1

3- Verification of $M_{N,y,Rd}$:

$$M_{N,y,Rd} = M_{pl,y,Rd}\frac{1-n}{1-0.5a}$$

$$N_{pl,Rd} = \frac{A\,f_y}{\gamma_{M0}} = 90.4 * 10^{-4} * 275 * \frac{10^3}{1} = 2486\ kN > 900\ kN\ Ok.$$

Since $N_{Ed} = 900\ kN > 0.25\ N_{pl,Rd} = 621.5\ kN$ **else No need for moment reduction.**

$$M_{pl,y,Rd} = \frac{W_{Pl,y}\,f_y}{\gamma_{M0}} = \frac{799 \times 10^{-6} \times 275 * 10^3}{1.0} = 219.73\ KN.m$$

$$a = \frac{A-2bt_f}{A} = \frac{90.4 - 2 * 20.64 * 1.73}{90.4} = 0.21 < 0.5$$

$$n = \frac{N_{Ed}}{N_{pl,Rd}} = \frac{900}{2486} = 0.362 > 0.25\ ok.$$

$$M_{N,y,Rd} = 219.73 * \frac{1-0.362}{1-0.5 * 0.21} = 156.63\ kN.m > 50\ kN.m\ Ok.$$

Woked Problem1

4- Verification of the stability of the member:

$$\frac{N_{Ed}}{\chi_y\, N_{Rk}/\gamma_{M1}} + k_{yy}\frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} \leq 1$$

$$\frac{N_{Ed}}{\chi_z\, N_{Rk}/\gamma_{M1}} + k_{zy}\frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} \leq 1$$

- **Step 1:** characteristic resistance of the cross section

$$N_{Rk} = Af_y = 90.4 * 10^{-4} * 275 * 10^3 = 2486\,kN$$

$$M_{y,Rk} = W_{Pl,y}\, f_y = 799 \times 10^{-6} \times 275 * 10^3 = 219.73\,KN$$

- **Step 2:** reduction coefficients due to flexural buckling, χ_y *and* χ_z

$\frac{h}{b} = \frac{215.8}{206.4} = 1.045 < 1.2$ and $t_f = 17.3\,mm < 100\,mm$

Flexural buckling around y -curve b ($\alpha = 0.34$)

Flexural buckling around z -curve c ($\alpha = 0.49$)

Worked Problem1

4- Verification of the stability of the member:

- **Plane x-z -** $L_{E,y} = 1 \times 4.5 = 4.5$ m

$$\lambda_1 = \pi\sqrt{\frac{210\times10^6}{275\times10^3}} = 86.81\ ;\ \lambda = \frac{L}{i} = \frac{4.5\times10^2}{9.18} = 49.02\ ;\ \bar{\lambda} = \frac{\lambda}{\lambda_1} = \frac{49.02}{86.81} = 0.564$$

$$\emptyset = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right] = 0.5[1 + 0.34 \times (0.564 - 0.2) + 0.564^2] = 0.72$$

$$\chi_y = \frac{1}{\emptyset + \sqrt{\emptyset^2 - \bar{\lambda}^2}} = \frac{1}{0.72 + \sqrt{0.72^2 - 0.564^2}} = 0.856 < 1$$

- **Plane x-y -** $L_{E,z} = 1 \times 4.5 = 4.5$ m

$$\lambda_1 = \pi\sqrt{\frac{210\times10^6}{275\times10^3}} = 86.81\ ;\ \lambda = \frac{L}{i} = \frac{4.5\times10^2}{5.3} = 84.9\ ;\ \bar{\lambda} = \frac{\lambda}{\lambda_1} = \frac{84.9}{86.81} = 0.978$$

$$\emptyset = 0.5\left[1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right] = 0.5[1 + 0.49 \times (0.978 - 0.2) + 0.978^2] = 1.16$$

$$\chi_z = \frac{1}{\emptyset + \sqrt{\emptyset^2 - \bar{\lambda}^2}} = \frac{1}{1.16 + \sqrt{1.16^2 - 0.978^2}} = 0.56 < 1$$

Worked Problem1

4- Verification of the stability of the member:

- **Step 3:** calculation of χ_{LT} using alternative method
  - The effective length of the segment is 4.5 m.
  - The ratio of the moment segment:

$$\psi = \frac{50}{-50} = -1$$

From Table (3.6):→ $C_1 = 2.6$

$$M_{cr} = C_1 \frac{\pi^2 E I_z}{(K_z L)^2}\left\{\left(\sqrt{\left(\frac{K_z}{K_w}\right)^2 . \frac{I_w}{I_z} + \frac{(K_z L)^2 G I_T}{\pi^2 E I_z} + \left(C_2 z_g - C_3 z_j\right)^2}\right) - \left(C_2 z_g - C_3 z_j\right)\right\}$$

$$M_{cr} = 2.6 \times \frac{\pi^2 \times 210\times10^3 \times 2540\times10^4}{(1\times4500)^2}\left\{\left(\sqrt{\left(\frac{1}{1}\right)^2 . \frac{0.25\times10^{12}}{2540\times10^4} + \frac{(1\times4500)^2\times81\times10^3\times80.2\times10^4}{\pi^2\times210\times10^3\times2540\times10^4} + (0)^2}\right) - 0\right\} \times 10^{-6}$$

$$M_{cr} = 1261.48\ KN.m$$

Worked Problem1

4- Verification of the stability of the member:

$$\overline{\lambda_{LT}} = \sqrt{\frac{W_{Pl}\, f_y}{M_{cr}}} = \sqrt{\frac{799 * 10^{-6} * 275 * 10^{3}}{1261.48}} = 0.417$$

$$\beta = 0.75\,;\ \overline{\lambda_{LT,0}} = 0.4$$

Check: $\overline{\lambda_{LT}} = 0.417 > \overline{\lambda_{LT,0}} = 0.4$

Or: $\frac{M_{Ed}}{M_{cr}} = \frac{50}{1261.48} = 0.039 < \overline{\lambda_{LT,0}}^{2} = 0.16$

So, we don't need to consider LTB calculation (anyway the following calculation shows the reason why)

$$\frac{h}{b} = \frac{215.8}{206.4} = 1.045 < 2 \rightarrow Buckling\ Curve\ b \rightarrow \alpha_{LT} = 0.34$$

$$\phi_{LT} = 0.5[1 + \alpha_{LT}(\overline{\lambda_{LT}} - \overline{\lambda_{LT,0}}) + \beta\overline{\lambda_{LT}}^{2}]$$

$$\phi_{LT} = 0.5[1 + 0.34(0.417 - 0.4) + 0.75 \times 0.417^{2}] = 0.568$$

Worked Problem1

4- Verification of the stability of the member:

$$\chi_{LT} = \frac{1}{\emptyset_{LT} + \sqrt{{\emptyset_{LT}}^2 - \beta \overline{\lambda_{LT}}^2}} = \frac{1}{0.568 + \sqrt{0.568^2 - 0.75 \times 0.417^2}} = 0.993$$

Check

$$\chi_{LT} = 0.993 \leq 1.0 \rightarrow O.K$$

$$\chi_{LT} = 0.993 \leq \frac{1}{\overline{\lambda_{LT}}^2} = \frac{1}{0.417^2} = 5.75 \rightarrow O.K$$

Woked Problem1

4- Verification of the stability of the member:

- Calculate $\chi_{LT,mod}$

  From Table (3.10) we calculate k_c

$$\psi = -1 \rightarrow k_C = \frac{1}{1.33 - 0.33\psi} = \frac{1}{1.33 - 0.33 \times -1} = 0.602$$

$$f = 1 - 0.5(1 - k_C)[1 - 2.0(\overline{\lambda_{LT}} - 0.8)^2]$$

$$f = 1 - 0.5(1 - 0.602)[1 - 2.0(0.417 - 0.8)^2] = 0.86 \leq 1.0$$

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} = \frac{0.993}{0.86} = 1.15 > 1 \text{ , } So\ \chi_{LT,mod} = 1$$

Worked Problem1

4- Verification of the stability of the member:

- Calculate $\chi_{LT,mod}$

  From Table (3.10) we calculate k_c

$$\psi = -1 \rightarrow k_c = \frac{1}{1.33 - 0.33\psi} = \frac{1}{1.33 - 0.33 \times -1} = 0.602$$

$$f = 1 - 0.5(1 - k_c)[1 - 2.0(\overline{\lambda_{LT}} - 0.8)^2]$$

$$f = 1 - 0.5(1 - 0.602)[1 - 2.0(0.417 - 0.8)^2] = 0.86 \leq 1.0$$

$$\chi_{LT,mod} = \frac{\chi_{LT}}{f} = \frac{0.993}{0.86} = 1.15 > 1 \text{ , So } \chi_{LT,mod} = 1$$

Woked Problem1

4- Verification of the stability of the member:

- **Step 4:** interaction factors k_{yy} and k_{zy}

$$\psi = -1 \rightarrow C_{my} = C_{mLT} = 0.6 + 0.4 * -1 = 0.2 < 0.4\ use\ 0.4$$

$$k_{yy} = C_{my}\left[1 + (\overline{\lambda_y} - 0.2)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right] = 0.4 * \left[1 + (0.564 - 0.2)\ \frac{900}{0.856 * \frac{2486}{1}}\right] = 0.46$$

$$As, k_{yy} \leq C_{my}\left[1 + 0.8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right] = 0.4 * \left[1 + 0.8\ \frac{900}{0.856 * \frac{2486}{1}}\right] = 0.535\ , use\ k_{yy} = 0.46$$

$$k_{zy} = \left[1 - \frac{0.1 * \overline{\lambda_z}}{(C_{mLT} - 0.25)}\ \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right] = \left[1 - \frac{0.1 * 0.978}{(0.4 - 0.25)}\ \frac{900}{0.56\,\frac{2486}{1}}\right] = 0.578$$

$$As, k_{zy} \geq \left[1 - \frac{0.1}{(C_{mLT} - 0.25)}\ \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right] = \left[1 - \frac{0.1}{(0.4 - 0.25)}\ \frac{900}{0.56\,\frac{2486}{1}}\right] = 0.569\ , use\ k_{zy}$$

$$= 0.578$$

Worked Problem1

4- Verification of the stability of the member:

- **Step 5:** Finally, the verification

$$\frac{N_{Ed}}{\chi_y\, N_{Rk}/\gamma_{M1}} + k_{yy}\frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} \le 1$$

$$\frac{900}{0.856 * 2486/1} + 0.46\frac{50}{1 * 219.73/1} = 0.527 < 1 \rightarrow\ Ok.$$

$$\frac{N_{Ed}}{\chi_z\, N_{Rk}/\gamma_{M1}} + k_{zy}\frac{M_{y,Ed}}{\chi_{LT} M_{y,Rk}/\gamma_{M1}} \le 1$$

$$\frac{900}{0.56 * 2486/1} + 0.578\frac{50}{1 * 219.73/1} = 0.778 < 1 \rightarrow\ Ok.$$

Steel Structures 2
Sumer Sem.
2023-2024

أ.د. نايل محمد حسن

Lecture 13-14

- Flexural Members

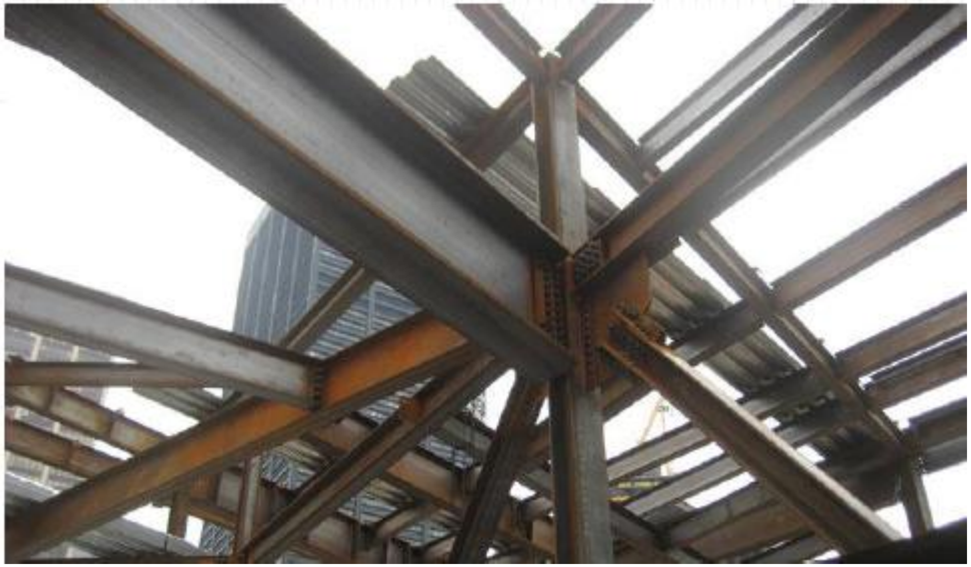
- ✓ -I- Laterally Restrained Beams
- ✓ II- Laterally Unrestrained Beams
- ✓ Beam-Column Members
- ✓ Beam-Column Members (problems))

- DESIGN OF CONNECTIONS

DESIGN OF CONNECTIONS

Introduction: Joints/connections in Steel structures

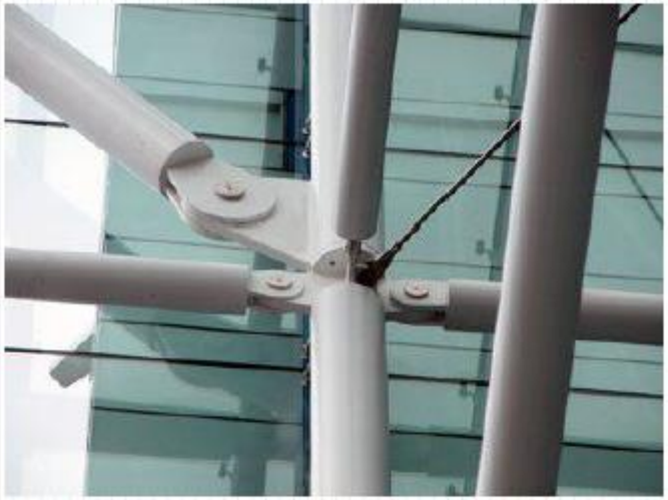
- The performance of the structural steel members is only attained as per the design if and only if the connections in steel structures are efficient !.
- Historically, most major structural failures have been due to some form of connection failure.



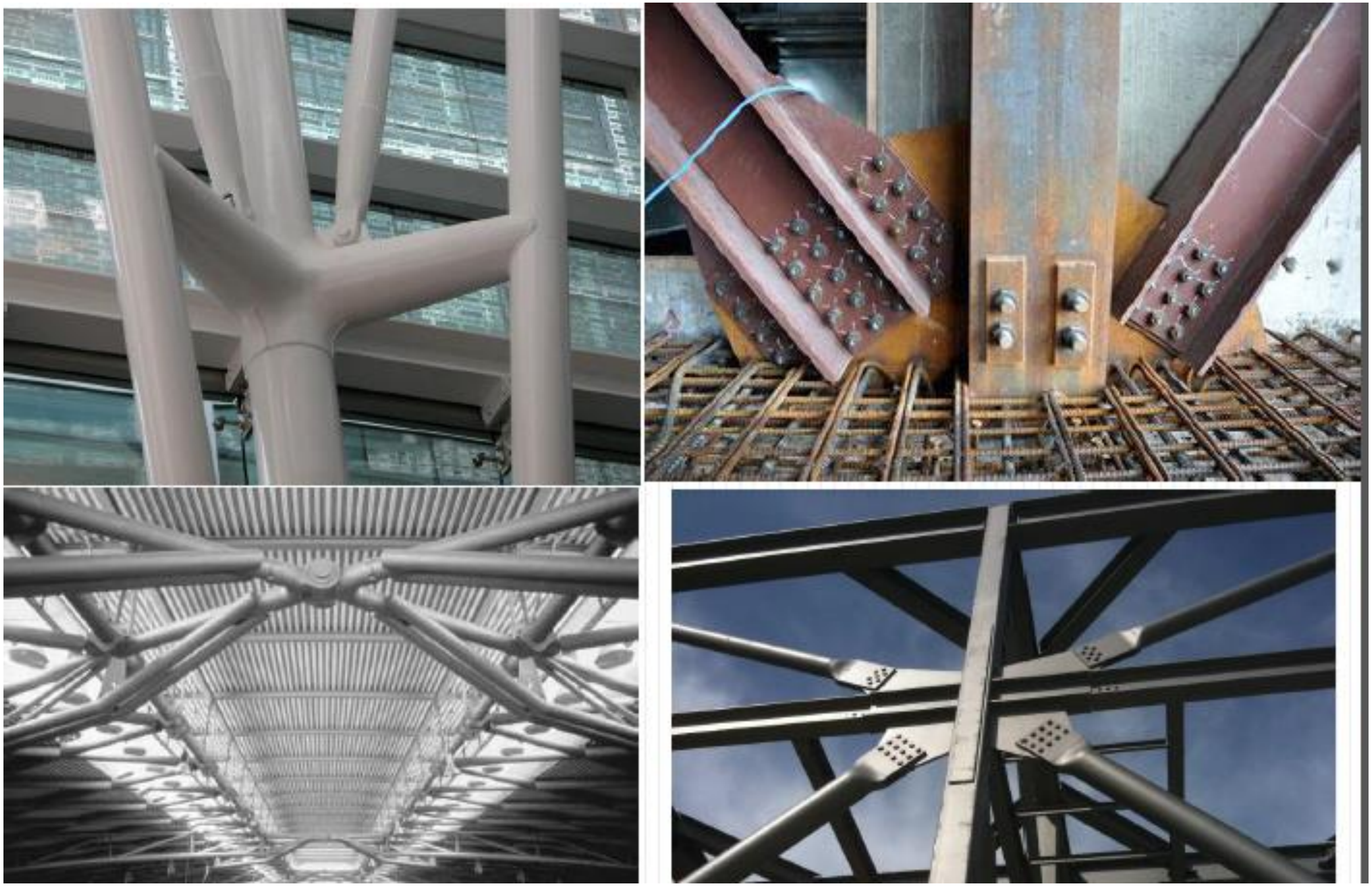
Introduction: Joints/connections in Steel structures

Connections depend on:

- **Type of loading**
- **Strength and stiffness**
- **Economy**
- **Difficulty or ease of erection**



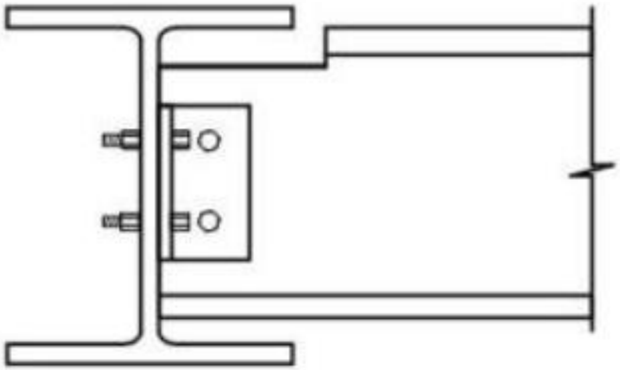
Introduction: Joints/connections in Steel structures



Introduction: Joints/connections in Steel structures

Can have various configuration depending on the structural members they connect:

- **Beam-Beam Connections**
- **Beam-Column Connections**
- **Column to Footing Connections**
- **column splices are typical cases as well as**

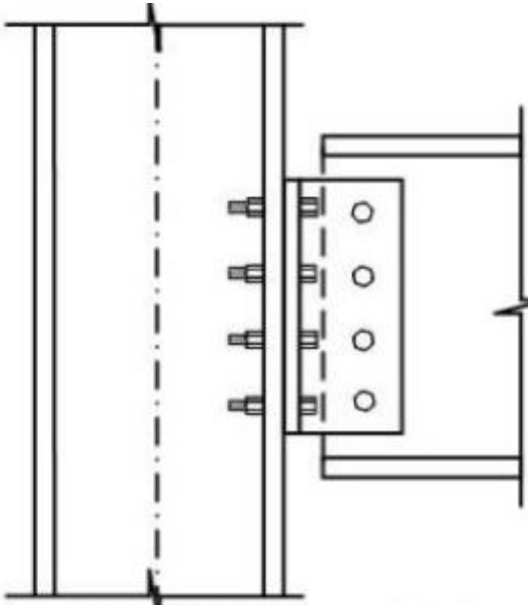


Secondary beam-main beam connection (web cleat) (assumed as pin connection)

Introduction: Joints/connections in Steel structures

Can have various configuration depending on the structural members they connect:

- **Beam-Beam Connections**
- **Beam-Column Connections**
- **Column to Footing Connections**
- **column splices are typical cases as well as**

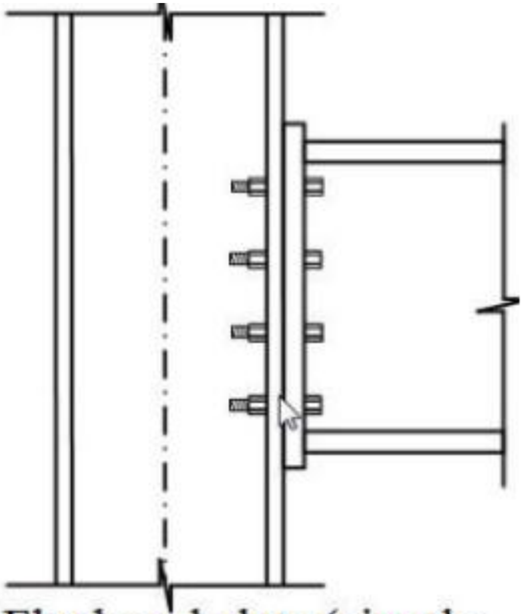


Web cleat (assumed to be simple construction)

Introduction: Joints/connections in Steel structures

Can have various configuration depending on the structural members they connect:

- **Beam-Beam Connections**
- **Beam-Column Connections**
- **Column to Footing Connections**
- **column splices are typical cases as well as**

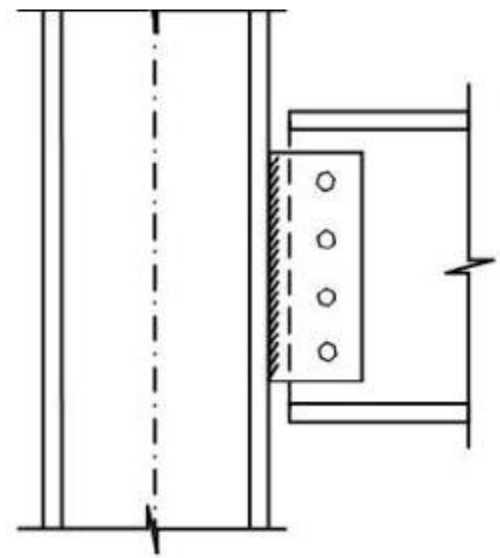


Flush end plate (simple semi-continuous)

Introduction: Joints/connections in Steel structures

Can have various configuration depending on the structural members they connect:

- **Beam-Beam Connections**
- **Beam-Column Connections**
- **Column to Footing Connections**
- **column splices are typical cases as well as**



Fin plate (assumed to be simple construction, could be semi-continuous)

Introduction: Joints/connections in Steel structures

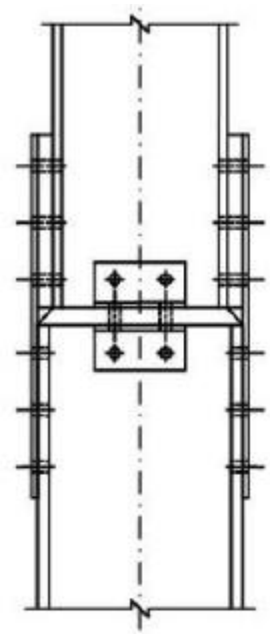
Can have various configuration depending on the structural members they connect:

- **Beam-Beam Connections**
- **Beam-Column Connections**
- **Column to Footing Connections**
- **column splices are typical cases as well as**

Introduction: Joints/connections in Steel structures

Can have various configuration depending on the structural members they connect:

- **Beam-Beam Connections**
- **Beam-Column Connections**
- **Column to Footing Connections**
- **column splices are typical cases as well as**



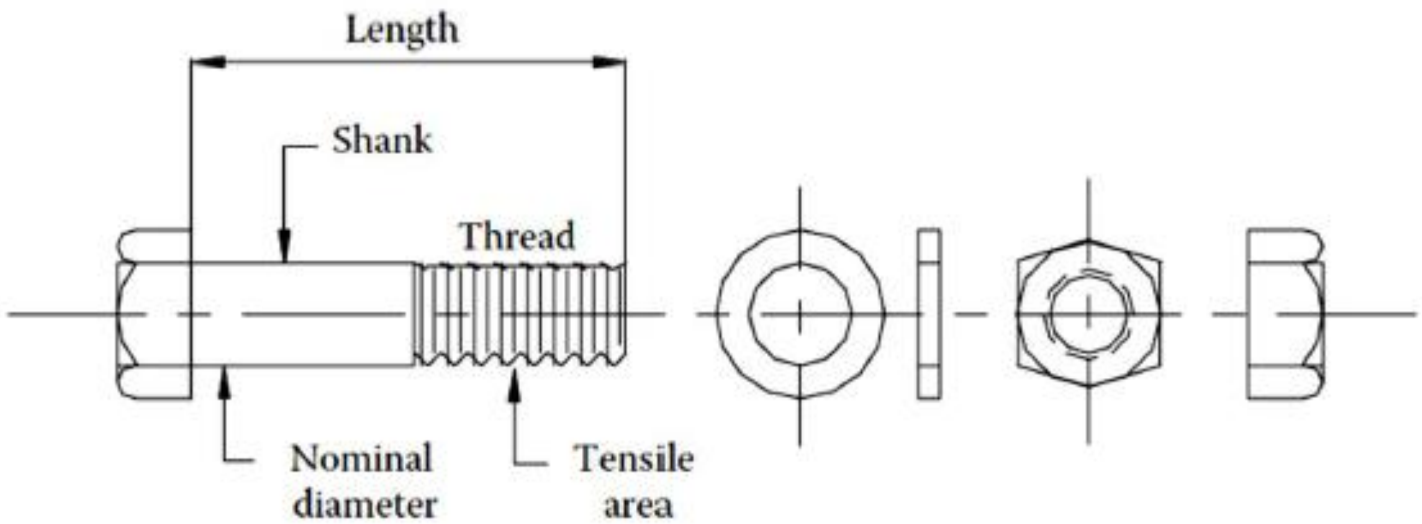
Column splice (continuous)

Introduction: Mechanical Fasteners

- Mechanical fasteners are generally realized by means of bolts, pins and rivets,
- which make possible the erection of the skeleton frame in a much reduced time frame,
- especially when compared with the one required when site welds are employed.
- They are generally composed of

Introduction: Mechanical Fasteners

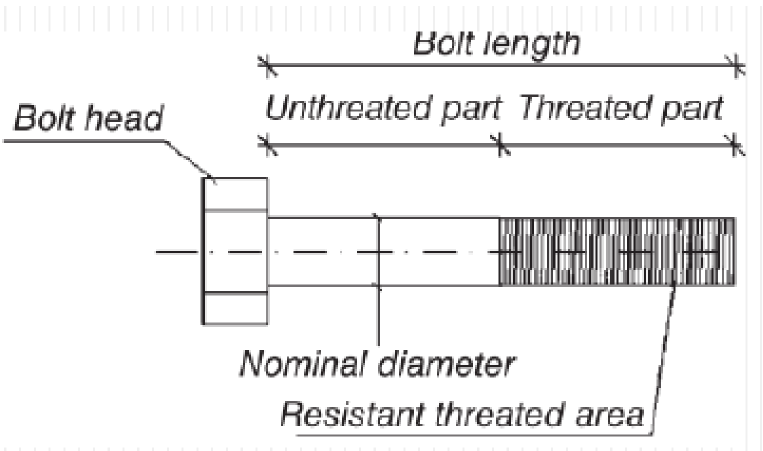
- Mechanical **fasteners** are generally realized by means of bolts, pins and rivets,
- which make possible the **erection of the skeleton** frame in a much reduced time frame,
- especially when **compared with the one** required when site welds are employed.
- They are generally composed of



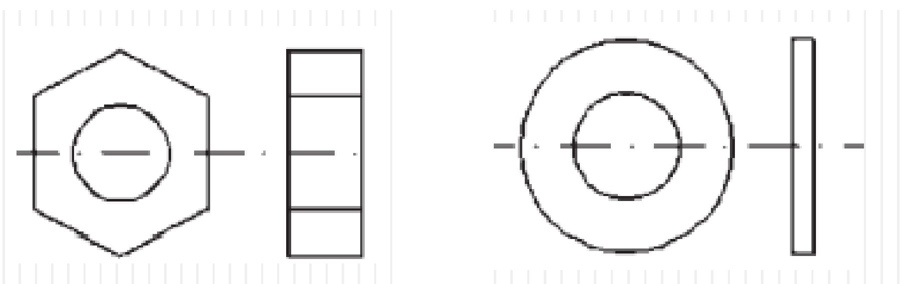
Hexagon head bolt, nut and washer.

Introduction: Mechanical Fasteners

- Mechanical fasteners are generally realized by means of bolts, pins and rivets,
- which make possible the erection of the skeleton frame in a much reduced time frame,
- especially when compared with the one required when site welds are employed.
- They are generally composed of



a bolt



a nut

one or more washers, when necessary.

Resistance of Bolted Connections: Introduction

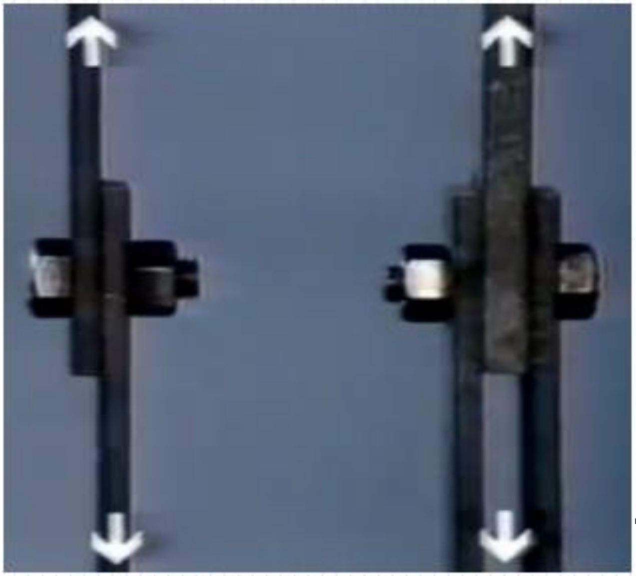
Design of connections is generally based on simplified models that require in many cases only hand written calculations.

The distribution of forces in the connection may, hence, be arbitrarily determined in whatever rational way is best, provided that:

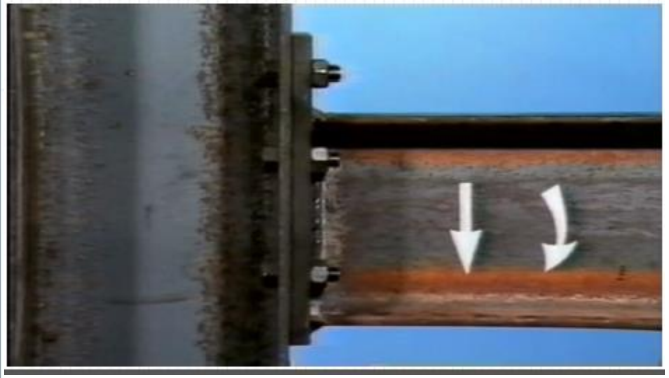
- **the assumed internal forces are balanced with the applied design forces and moments;**
- **each part of the connection is able to resist the applied forces and moments;**
- **the deformations imposed by the chosen distribution are within the deformation capacity of the fasteners, welds and the other key parts of the connection.**

Introduction: Mechanical Fasteners

Connections can be classified on the basis of the acting loads as follows:



connections in shear;

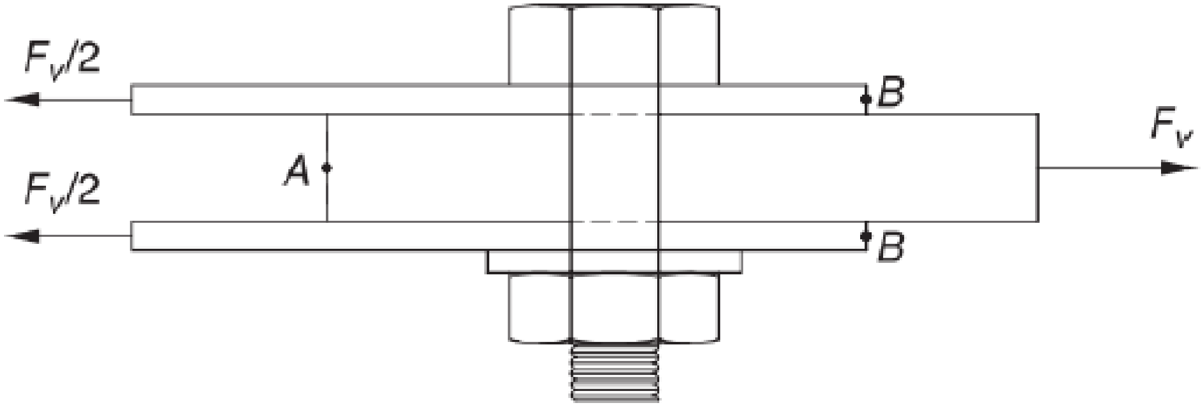


connections simultaneously in tension and shear.

connections in tension;

Introduction: Mechanical Fasteners

A connection is affected by **shear** when the plates connected via bolts are loaded by forces parallel to the contact planes.



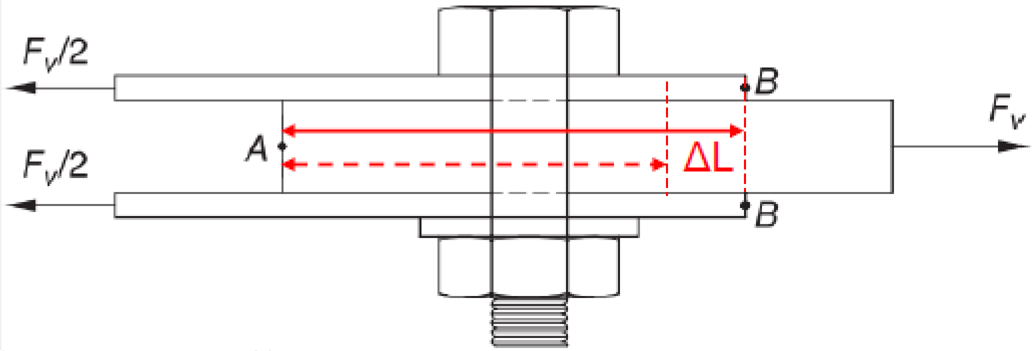
bolt can be considered to a simply supported beam loaded at its mid span.

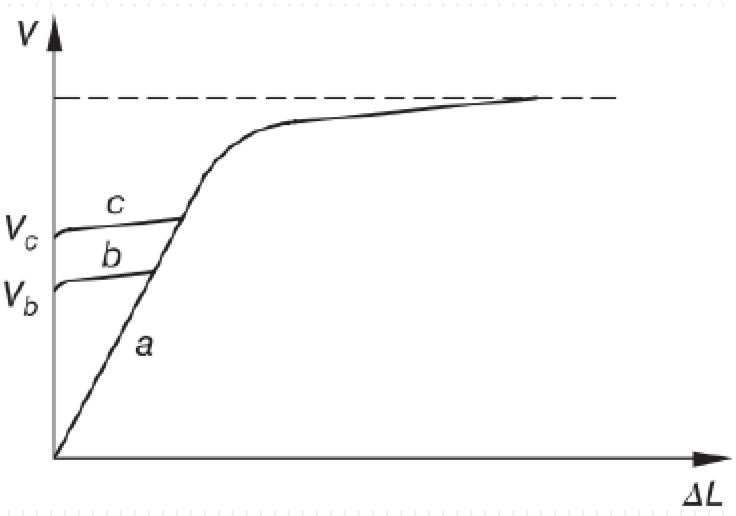
Different responses are expected, depending on two different modes to transfer the shear load, which make possible the distinction between:

- **Bearing connections and ,**
- **Slip-resistant connections.**

Resistance of Bolted Connections: connections in Shear -Bearing connections

It is required that the plates must be connected to each other achieving a firm contact and no tightening of the bolt is required.





$$\tau = \frac{V}{n \cdot A_{res}}$$

$$\tau = \frac{V}{n \cdot A}$$

Where,
A is un threaded area,
A_{res} is threaded area
V is the total shear force on the bolt and,
n is the number of shear planes

Failure of the shear connection can be due to one of the following mechanisms:

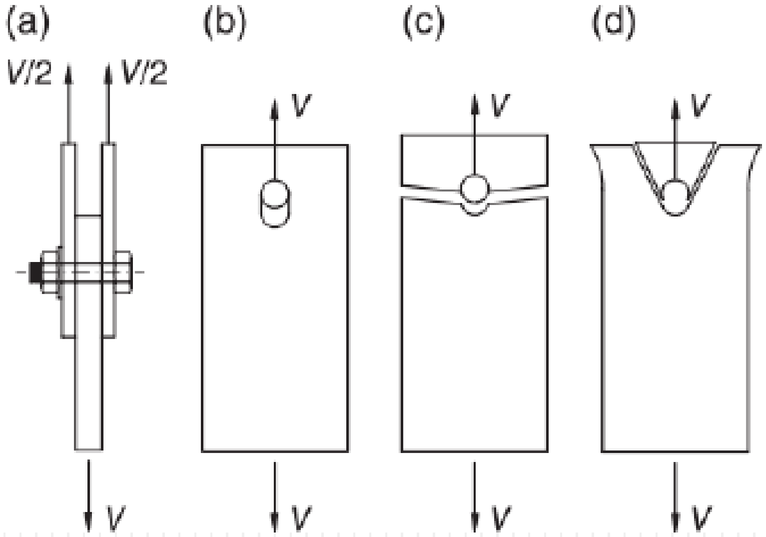
a) bolt failure;
b) plate bearing;
c) tension failure of the plate;
d) shear failure of the plate.

Resistance of Bolted Connections: connections in Shear -Bearing connections

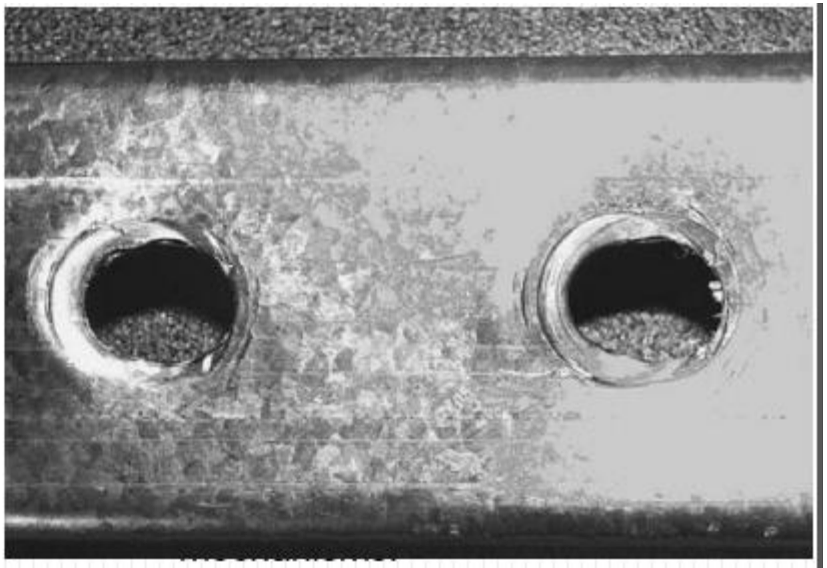
In particular, bearing pressure between bolt and plate can be approximated with reference to the mean value of the bearing stress, σ_{bear}:

$$\sigma_{bear} = \frac{V}{t \cdot d}$$

Where,
V is the acting shear force per shear plane
t is the minimum thickness of connected plates der shear plane
D is the bolt diameter.



Typical deformation holes due to a bearing.

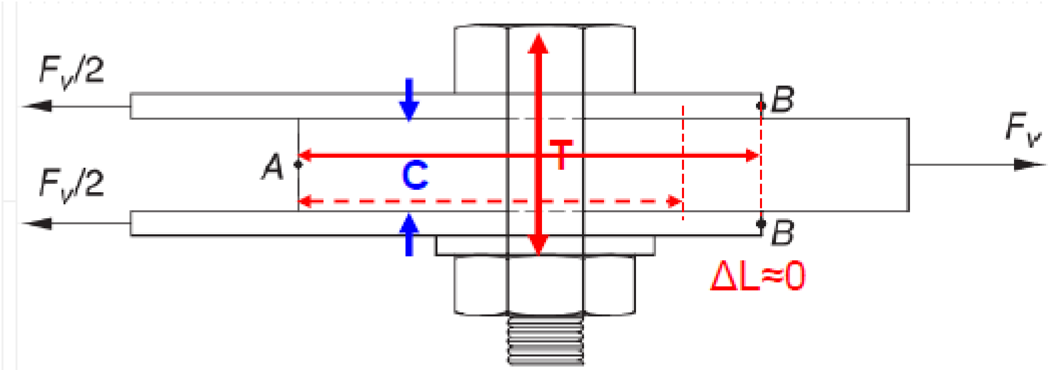


Failure of the shear connection can be due to one of the following mechanisms:

a) bolt failure;
b) plate bearing;
c) tension failure of the plate;
d) shear failure of the plate.

Resistance of Bolted Connections: Connections in Shear -Slip Resistant Connection or Connection with Pre-Loaded Joints

Pre-loading of bolts can be explicitly required for slip resistance, seismic connections, fatigue resistance, execution purposesor as a quality measure (e.g. for durability).



Thus, once the bolt is tightened, the joint is loaded by self-balanced stresses associated with the bolt in tension and the compression in the plates and with the torsion of the bolt and plate/bolt friction.

Resistance of Bolted Connections: Connections in Shear -Slip Resistant Connection or Connection with Pre-Loaded Joints

Tightening increases joint performance, mainly with reference to serviceability limit states. Furthermore, it should be noted that :

- in shear joints, tightening prevents plate slippage and, therefore, inelastic settlements in the structure;
- in tension joints, tightening prevents plate separation (reducing corrosion dangers) and significantly improves fatigue resistance.

However, tightening must not exceed a certain limit, to avoid attaining joint ultimate capacity.

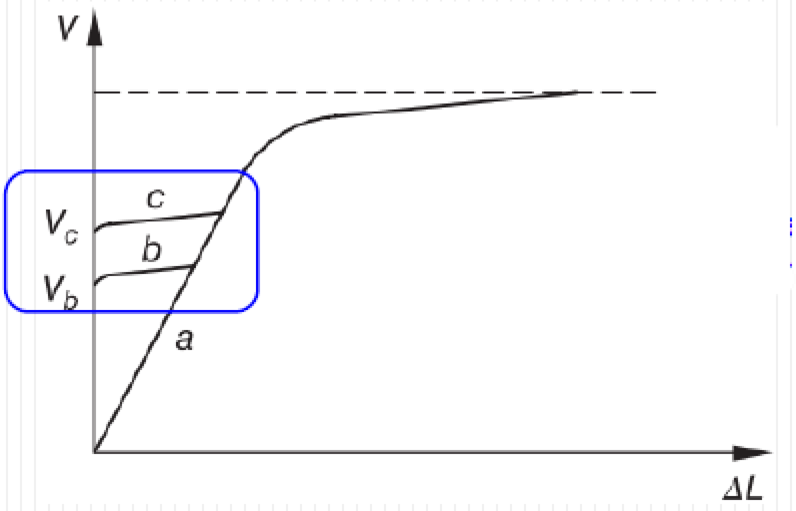
Resistance of Bolted Connections: Connections in Shear -Slip Resistant Connection or Connection with Pre-Loaded Joints

Tightening increases joint performance, mainly with reference to serviceability limit states. Furthermore, it should be noted that :

- **in shear joints, tightening prevents plate slippage and, therefore, inelastic settlements in the structure;**
- **in tension joints, tightening prevents plate separation (reducing corrosion dangers) and significantly improves fatigue resistance.**

However, tightening must not exceed a certain limit, to avoid attaining joint ultimate capacity.

The load increases from zero but no relative displacement is observed; force transmission is due to friction between the plates until friction limit of the joint is reached, which depends on the degree of preload.



Curve (c) is related to a connection with a pre-load degree greater than the one of case (b);

Resistance of Bolted Connections: Connections in Shear -Slip Resistant Connection or Connection with Pre-Loaded Joints-Stress distribution

The value of the force at which slippage occurs depends upon :.

- **bolt tightening,**
- **surface treatment, and**
- **number of surfaces in contact (n_f,).**
- **The maximum value of the force transferred by friction, F_{Lim} , can be estimated as :.**

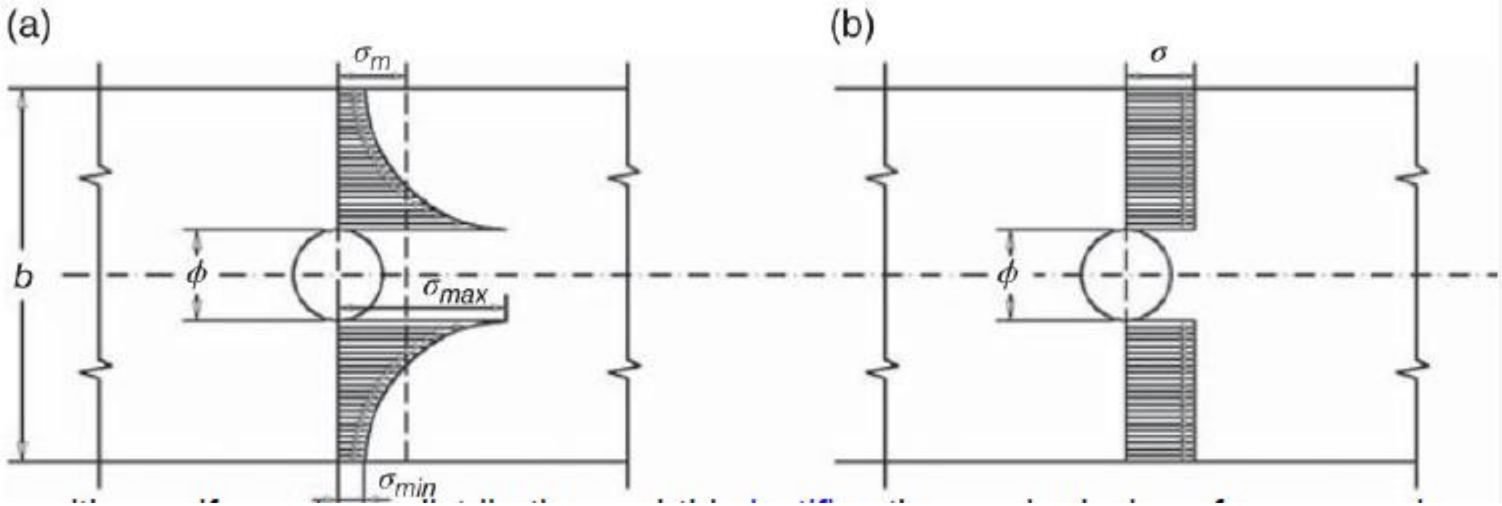
$$F_{\text{Lim}} = n_f \cdot \mu \cdot N_s$$

Where,

► μ is the friction coefficient.

Resistance of Bolted Connections: Connections in Shear -Slip Resistant Connection or Connection with Pre-Loaded Joints-Stress distribution

As the deformation capacity of plates is generally much higher than the deformation capacity of the bolts, it is strongly recommended to design the connection such that yielding of the plates in bearing occurs before yielding of the bolts in shear,in order to guarantee a ductile failure rather than a brittle failure.



Distribution of the stress in theplate of a bearing connection in elastic (a) and plastic (b) range

Resistance of Bolted Connections: Connections in Shear -Slip Resistant Connection or Connection with Pre-Loaded Joints-Stress distribution

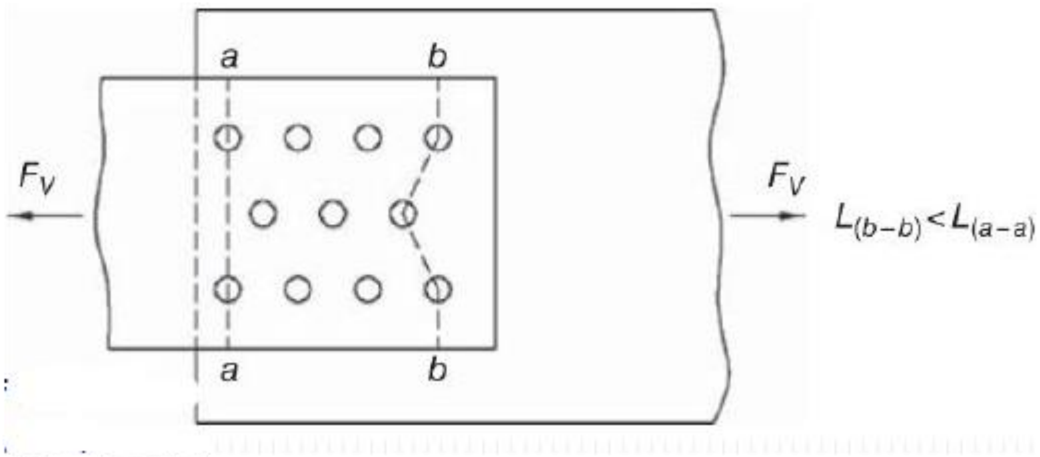
Plastic redistribution at failure occurs with a uniform stress distribution and this justifies the use in design of a mean value of stress, assumed for sake of simplicity constant in elastic range and conventionally considered equal to:

$$\sigma = \frac{V}{A_n}$$

Where,
V is the shear force.
A_n is the net area of the cross-section of the plate (i.e. gross area reduced for the presence of the hole.

Resistance of Bolted Connections: Connections in Shear -Slip Resistant Connection or Connection with Pre-Loaded Joints-Stress distribution
Note -1-

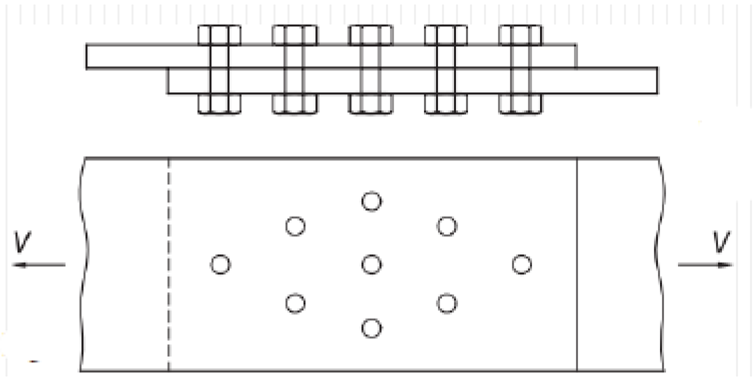
In connections with more than one bolt, a correct evaluation of the resistant area for the plates could become complex, depending on the ultimate load for tension and shear as a function of the possible failure path



the main rules for estimating an appropriate value of the reduced area have already been introduced for tension Fv member verification

Resistance of Bolted Connections: Connections in Shear -Slip Resistant Connection or Connection with Pre-Loaded Joints-Stress distribution
Note -1-

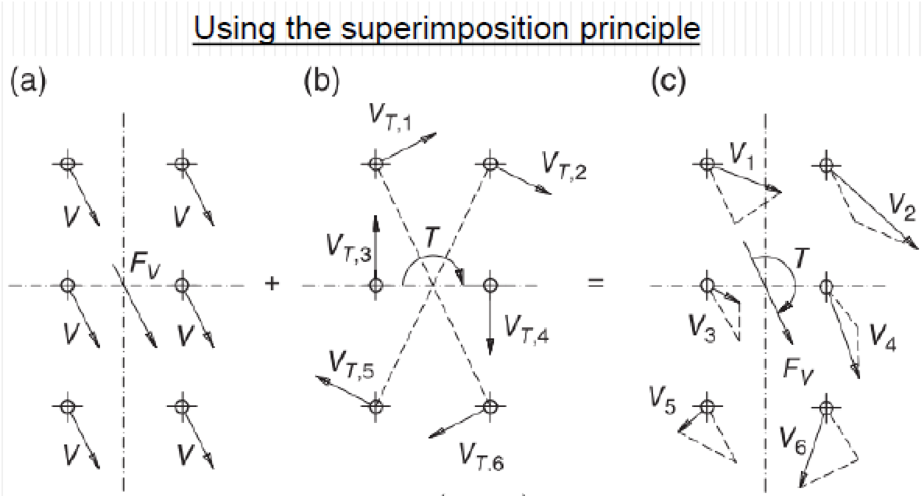
To minimize the weakness of cross-section for the presence of holes, it is possible to increase the number of the holes from the end to the center oft he connection, as shown.



It is worth noting that this causes an increase in the dimension of the joint.

Resistance of Bolted Connections: Connections in Shear -Slip Resistant Connection or Connection with Pre-Loaded Joints-Stress distribution
Note -2-

As it happens in some practical cases dealing with joints, the design load Fv can be eccentric with reference to the centroid of the fasteners, the result of this is the connection is subject to shear and torsion.



$$V = \frac{F_V}{n_f \cdot n}$$

$$V_{T,i} = \frac{(F_V \cdot e) \cdot a_i}{n_f \cdot \sum_{i=1}^{n} a_i^2}$$

$$V_i = \sqrt{V^2 + V_{T,i}^2}$$

The actual response of this connection is quite difficult to be predicted.

Where,

-n_f is the number of shear resisting plane per bolt

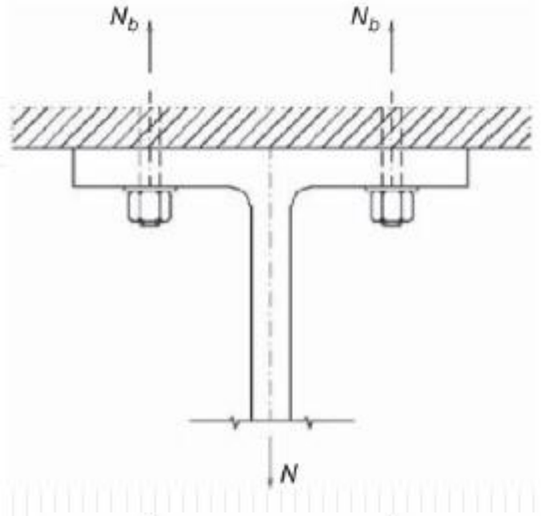
-n is the number of the bolts.

-a_i is the distance between the centroid of all thebolts and that of the single i-bolt.

Resistance of Bolted Connections: Connections in Tension

Tension occurs when the plates connected via bolts are loaded by a force normal to the contact plane; that is parallel to the bolt axis.

As in case of bearing connection, the response of a connection in tension is quite difficult to predict.

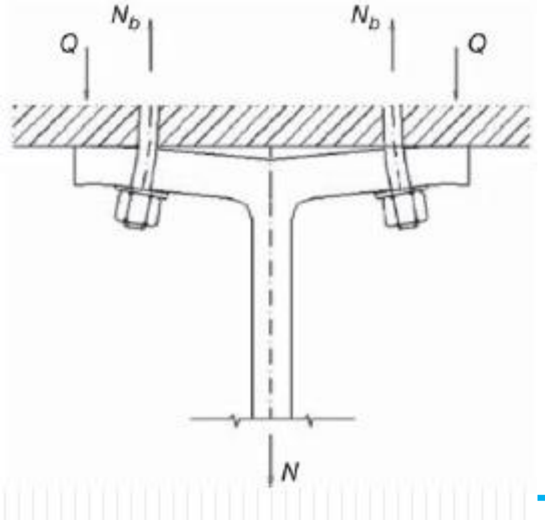


If the flange is sufficiently stiff

- its deformation can be disregarded,
- The bolts can be assumed to be in pure tension
- Connection failure is expected to be due to failure of the bolts.

If the flange is more flexible,

- the presence of prying forces, Q.
- increases the value of the axial load transferred via bolts.
- Connection failure may be due to bolts, flange or to both components.



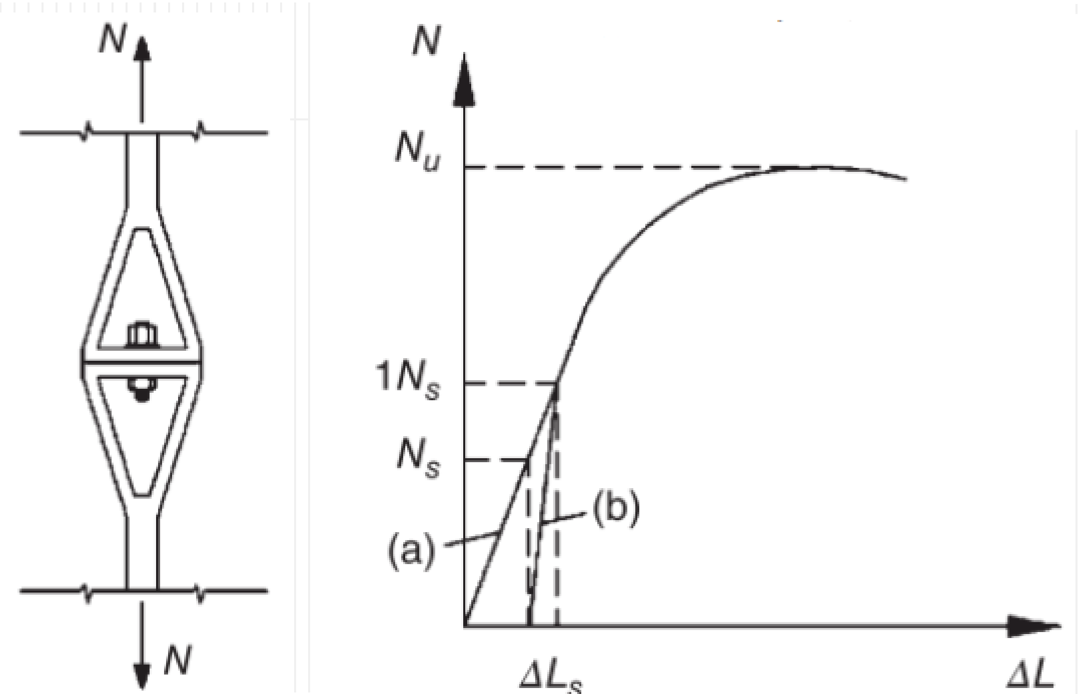
Resistance of Bolted Connections: Connections in Tension

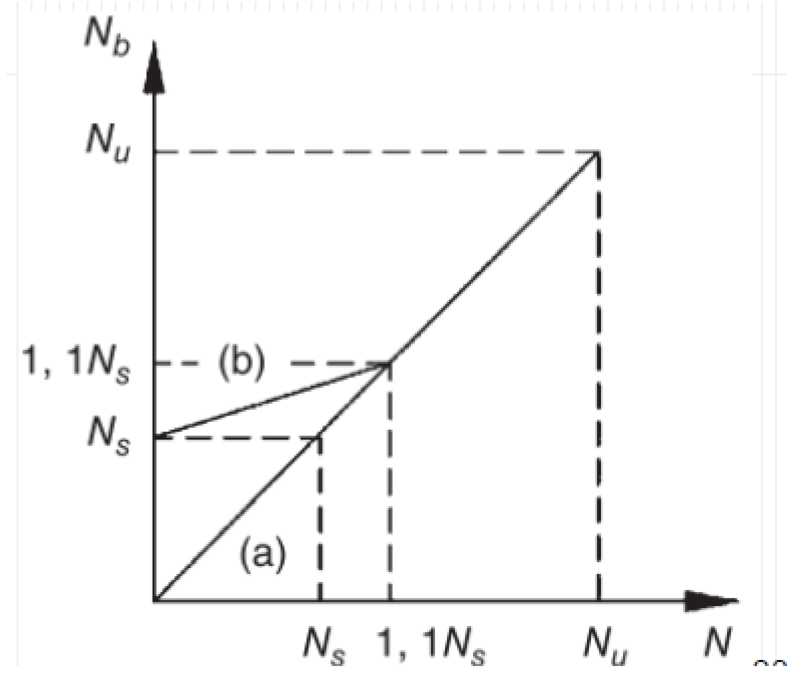
In order to better appraise the tightening effects, reference can be made to the response of the tension connection presented below, which is realized by one bolt..

Relationships between the applied external load N to the connection and bolt elongation ΔL

curve a - related to the case of non-tightened bolt
curve b - related to the case of tightened bolt

Relationships between the applied external load N is plotted versus the axial force acting in the bolt shank N_b





Resistance of Bolted Connections: Connections in Tension (Note 1)

In case of tension force applied on the centroid of the bolts, it is assumed that the design load is balanced by forces equal on each bolt. Otherwise, if a bending moment also acts, the evaluation of the bolt forces is usually based on the assumption of stiff plate.

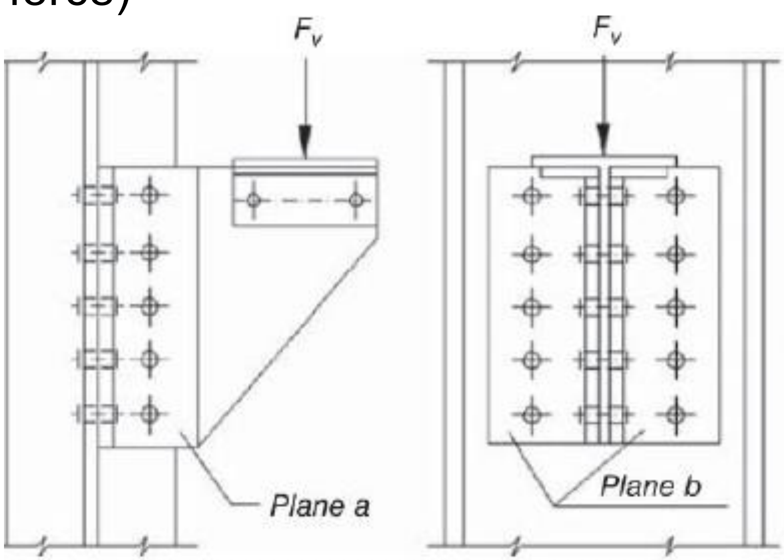
Angle legs on the plane a subjected to shear force and torsion moment

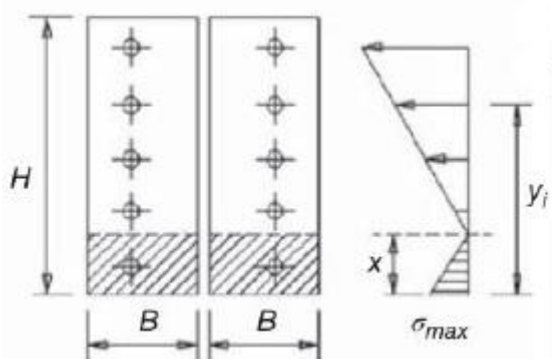
Angle legs on the plane b subjected to shear force and bending moment (tension force)

Equilibrium condition dictates

$$\frac{1}{2}\cdot\left[(2\cdot B)\cdot x^2\right] = \sum_{i=1}^{n} A_{bi}\cdot(y_i - x)$$

$$x = \frac{1}{2\cdot B}\cdot\left[-\sum_{i=1}^{n} A_{bi} + \sqrt{\left(\sum_{i=1}^{n} A_{bi}\right)^2 + (4\cdot B)\sum_{i=1}^{n}(A_{bi}\cdot y_i)}\right]$$





Resistance of Bolted Connections: Connections in Tension SUMMARY

The approaches previously introduced for the case of sole shear force and sole tension force on the connection can be combined to each other in order to be used for the more general case of shear and tension. More details about the requirements for verification are presented in the following Lectures, in accordance with European Norms.