Universitatea Politehnica Timişoara
Facultatea de Construcţii
Departamentul de Construcţii Metalice şi Mecanica Construcţiilor

COMPOSITE STEEL-CONCRETE STRUCTURES

- CURS 3 -

Composite Beams (2)

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§ 2.9 Plastic resistance in bending

- In the case of plastic resistance of composite beams, it is considered that there is a plastic distribution of efforts in the Ultimate Limit State.
- In consequence, the tensioned respectively compressed efforts are uniformly distributed over the height of the cross-section. This is possible by considering the following conditions (design hypothesis):
  - The resistance to vertical shear is assured only by the steel profile.
  - In the plastic design equations, it is admitted the fact that the reinforced concrete slab is full or cast on a profiled sheeting. In the latter case, the efforts present on the height of the profiled sheeting are ignored.

§ 2.9 Plastic resistance in bending

- It is accepted a plastic distribution of efforts, equal to the yield limit f_y/γ_a in the steel profile (tension or compression). In the case in which the plastic neutral axis is located in the steel profile, the section should be of class 1 or 2.
- In the compressed concrete, the stresses have constant distribution, equal to $0{,}85f_{ck}/\gamma_c$.
- The tensile strength of concrete is neglected.
- For design to sagging moment, the presence of steel reinforcement in the concrete slab will be ignored. These will be considered only in the case of hogging moment.
- The tensioned reinforcements (in hogging) are stressed to the design yield strength f_{sk}/γ_s.
- There is a full interaction between structural steel, reinforcement and concrete, so that in each element the maximum strength could be reached.

§ 2.9 Plastic resistance in bending

SECTION UNDER SAGGING MOMENT

- In case of sagging moment, the inferior fibers of beams are tensioned, while the superior fibers will be compressed.
- The plastic resistance in sagging is found in function of the position of the **Plastic Neutral Axis (PNA)**.

CASE I: PNA is located in concrete slab

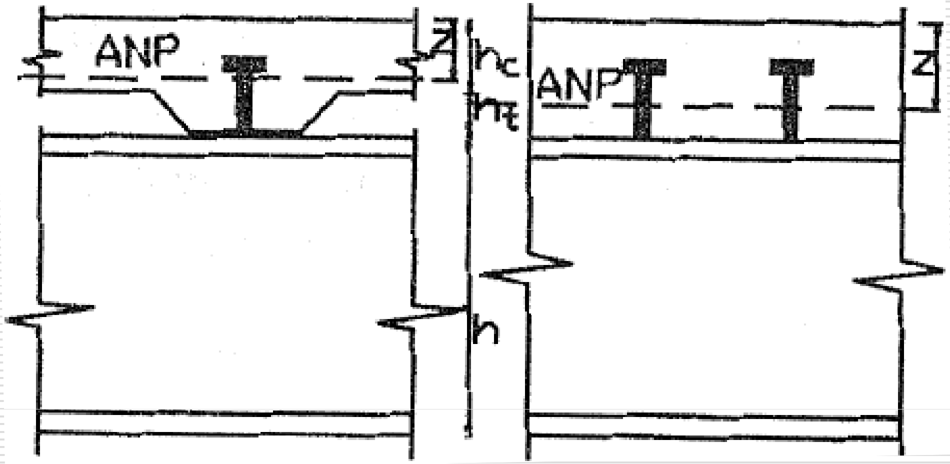
- In order to find the exact position of the PNA, first are computed the stress resultants of the compressed concrete slab - F_c and in the steel profile - F_a respectively.
- The PNA position results from direct comparison of these two values:
  - If $F_c > F_a$, PNA is located in the reinforced concrete slab;
  - If $F_c < F_a$, PNA is located in the steel profile.

§ 2.9 Plastic resistance in bending

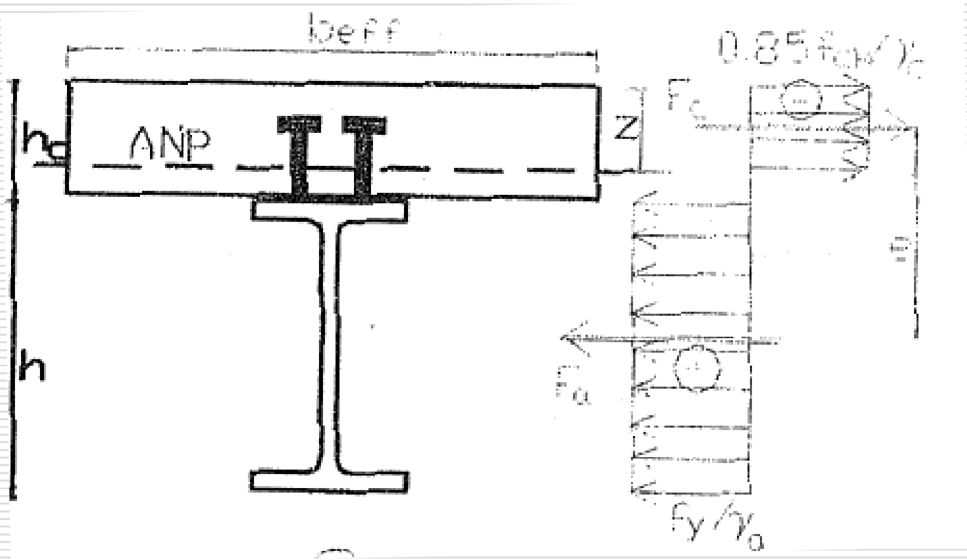
SECTION UNDER SAGGING MOMENT

CASE I: PNA is located in concrete slab

- Location of the plastic neutral axis in concrete slab: example for a composite slab and full slab respectively.



- Transversal cross-section and diagrams of normal stresses in plastic distribution.



§ 2.9 Plastic resistance in bending

SECTION UNDER SAGGING MOMENT

CASE I: PNA is located in concrete slab

- The compressive stress resultant of the concrete slab, over its height h_c is:

$$F_c = b_{eff} \cdot h_c \cdot 0.85 f_{ck} / \gamma_c$$

- The tensile stress resultant of the steel profile is:

$$F_a = A_a \cdot f_y / \gamma_a$$

where:

b_{eff} is the effective width of the concrete slab (in sagging);
h_c is the height of the concrete slab;
f_{ck} is the characteristic compressive concrete strength;
γ_a, γ_c are the partial safety factors of structural steel and concrete respectively;
A_a the cross-section area of the steel profile.

- If $F_c > F_a$, the PNA is located in the reinforced concrete slab.

§ 2.9 Plastic resistance in bending

SECTION UNDER SAGGING MOMENT

CASE I: PNA is located in concrete slab

- For finding the exact position of the PNA, by means of the distance *z* in regard to the superior fibre of the concrete slab, it will be written the equality between resultant of the compressive stresses in concrete slab (concrete considered only above the PNA) and the resultant of the tensile stresses of the steel profile:

$$b_{eff} \cdot z \cdot 0.85 f_{ck} / \gamma_c = A_a \cdot f_y / \gamma_a$$

- The only unknown in the above relation is the distance *z*, found by:

$$z = \frac{A_a \cdot f_y}{0.85 f_{ck} \cdot b_{eff}} \cdot \frac{\gamma_c}{\gamma_a} \leq h_c$$

§ 2.9 Plastic resistance in bending

SECTION UNDER SAGGING MOMENT

CASE I: PNA is located in concrete slab

- The plastic resistance is determined by the product between the resultant of the tensile stresses from steel profile F_a and the *e* distance (in fact, the moment is written about the centroid of the compressed concrete height):

$$M^{+}_{pl.Rd} = F_a \cdot e$$

- where *e* represents the distance from the centroid of the steel profile to the centroid of the compressed concrete:

$$e = \frac{h}{2} + h_c - \frac{z}{2} = \frac{1}{2}(h + 2h_c - z)$$

- The plastic resistance in sagging bending is written as:

$$M^{+}_{pl.Rd} = A_a \cdot f_y / \gamma_a \cdot \frac{1}{2}(h + 2h_c - z)$$ or: $$M^{+}_{pl.Rd} = \frac{1}{2\gamma_a} A_a \cdot f_y \cdot (h + 2h_c - z)$$

§ 2.9 Plastic resistance in bending

SECTION UNDER SAGGING MOMENT

CASE II: PNA is located in steel profile

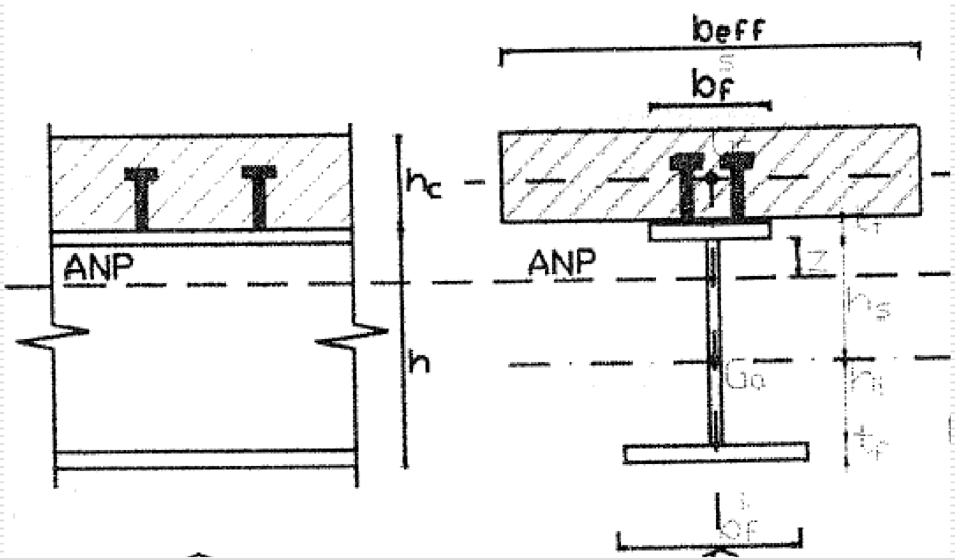
- In the case in which the tensile stress resultant of the steel profile F_a is greater than the compression stress resultant from concrete slab F_c , then the PNA is located in the steel profile.
- For the determination of the PNA position, it is considered the plastic efforts diagram presented in the below picture, in which the compression efforts exist in the concrete slab, the superior flange of the steel profile and the superior portion of the steel web (on distance z).
- The last (right) diagram of efforts is equivalent to the real diagram (left), by doubling the compressive stresses of the compressed part of the steel profile and respectively considering the entire steel profile in tension.

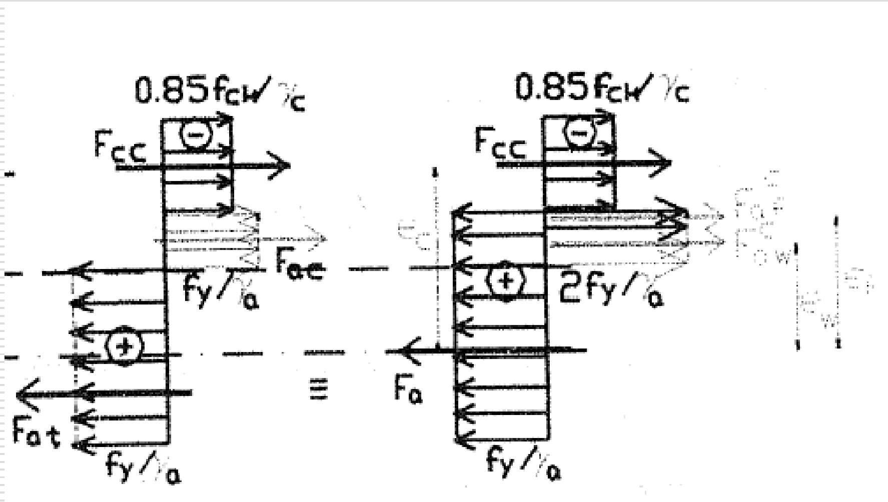
Obs: A similar design could be performed in the case in which the PNA is located in the superior flange.

§ 2.9 Plastic resistance in bending

SECTION UNDER SAGGING MOMENT

CASE II: PNA is located in steel profile





☐ Location of the PNA in the steel profile.

☐ Diagram of plastic distribution of stresses.

§ 2.9 Plastic resistance in bending

SECTION UNDER SAGGING MOMENT

CASE II: PNA is located in steel profile

- In this case, the determination of the location of PNA results from the following equation:

$$F_c = F_a \qquad \text{where:} \qquad F_c = F_{cc} + F^c_{af} + F^c_{aw}$$

where: F_c represents the compressive stress resultant;

F_a represents the tensile stress resultant of the entire steel profile;

F_{cc} represents the compressive stress resultant of the entire concrete slab;

F^c_{af} represents the compressive stress resultant of the superior flange of the steel profile;

F^c_{aw} is the compressive stress resultant of the steel web, computed on the height z of the steel web.

§ 2.9 Plastic resistance in bending

SECTION UNDER SAGGING MOMENT

CASE II: PNA is located in steel profile

- the stress resultants from the above relation are given by:

$$F_{cc} = 0.85 b_{eff} \cdot h_c \cdot f_{ck} / \gamma_c \qquad F_{aw}^{c} = z \cdot t_w \cdot 2 f_y / \gamma_a$$

$$F_{af}^{c} = b_f^s \cdot t_f \cdot 2 f_y / \gamma_a \qquad F_a = A_a \cdot f_y / \gamma_a$$

- Replacing these relations in the equation of plastic efforts it results:

$$A_a \cdot f_y / \gamma_a = 0.85 b_{eff} \cdot h_c \cdot f_{ck} / \gamma_c + b_f^s \cdot t_f \cdot 2 f_y / \gamma_a + z \cdot t_w \cdot 2 f_y / \gamma_a$$

- From which it can be derived the distance *z* from the superior fibre of the steel profile to PNA:

$$2z \cdot t_w \cdot f_y / \gamma_a = A_a \cdot f_y / \gamma_a - \left(0.85 b_{eff} \cdot h_c \cdot f_{ck} / \gamma_c + 2 b_f^s \cdot t_f \cdot f_y / \gamma_a\right)$$

from which it results:

$$z = \frac{A_a - 2 b_f^s \cdot t_f}{2 t_w} - \frac{0.85 b_{eff} \cdot h_c \cdot f_{ck} / \gamma_c}{2 t_w \cdot f_y / \gamma_a}$$

§ 2.9 Plastic resistance in bending

SECTION UNDER SAGGING MOMENT

CASE II: PNA is located in steel profile

- The plastic resistance in sagging is determined on the basis of the real (left) diagram of stresses, but is easier computed on the basis of stress distribution on the complete diagram, throughout the moment equation computed about the centroid of the concrete slab:

$$M^{+}_{pl.Rd} = F_a \cdot e_c - F^{c}_{aw}\left(e_c - e_w\right) - F^{c}_{af}\left(e_c - e_f\right)$$

- the distances e_c , e_w and e_f are determined by the following formulae (by considering an unsimetrical steel profile):

$$e_c = h_s + t_f + \frac{h_c}{2} = \frac{1}{2}\left(2h_s + 2t_f + h_c\right)$$

$$e_f = h_s + \frac{t_f}{2} = \frac{1}{2}\left(2h_s + t_f\right)$$

$$e_w = h_s - \frac{z}{2} = \frac{1}{2}\left(2h_s - z\right)$$

- where: h_s is the height of the steel profile web, above its centroid.

§ 2.9 Plastic resistance in bending

SECTION UNDER SAGGING MOMENT

CASE II: PNA is located in steel profile

z is the distance from the superior fibre of the steel web to the PNA:

$$z = \frac{A_a - 2b_f^s \cdot t_f}{2t_w} - \frac{0.85 b_{eff} \cdot h_c \cdot f_{ck} / \gamma_c}{2t_w \cdot f_y / \gamma_a}$$

- The plastic resistance under sagging moment is written as:

$$M_{pl.Rd}^{+} = A_a \cdot f_y / \gamma_a \cdot \frac{1}{2}\left(2h_s + 2t_f + h_c\right) - 2z \cdot t_w \cdot f_y / \gamma_a \left[\frac{1}{2}\left(2h_s + 2t_f + h_c\right) - \frac{1}{2}\left(2h_s - z\right)\right]$$
$$- 2b_f^s \cdot t_f \cdot f_y / \gamma_a \left[\frac{1}{2}\left(2h_s + 2t_f + h_c\right) - \frac{1}{2}\left(2h_s + t_f\right)\right]$$

or:

$$M_{pl.Rd}^{+} = \frac{1}{2}\frac{f_y}{\gamma_a}\left[A_a\left(2h_s + 2t_f + h_c\right) - 2z \cdot t_w\left(2t_f + h_c + z\right) - 2b_f^s \cdot t_f\left(t_f + h_c\right)\right]$$

§ 2.9 Plastic resistance in bending

SECTION UNDER SAGGING MOMENT

- The **verification of the cross-section** under sagging moment is performed by:

$$M^{+}_{Sd} \leq M^{+}_{pl.Rd}$$

with M^{+}_{Sd} - the sagging moment resulted from the static design

§ 2.9 Plastic resistance in bending

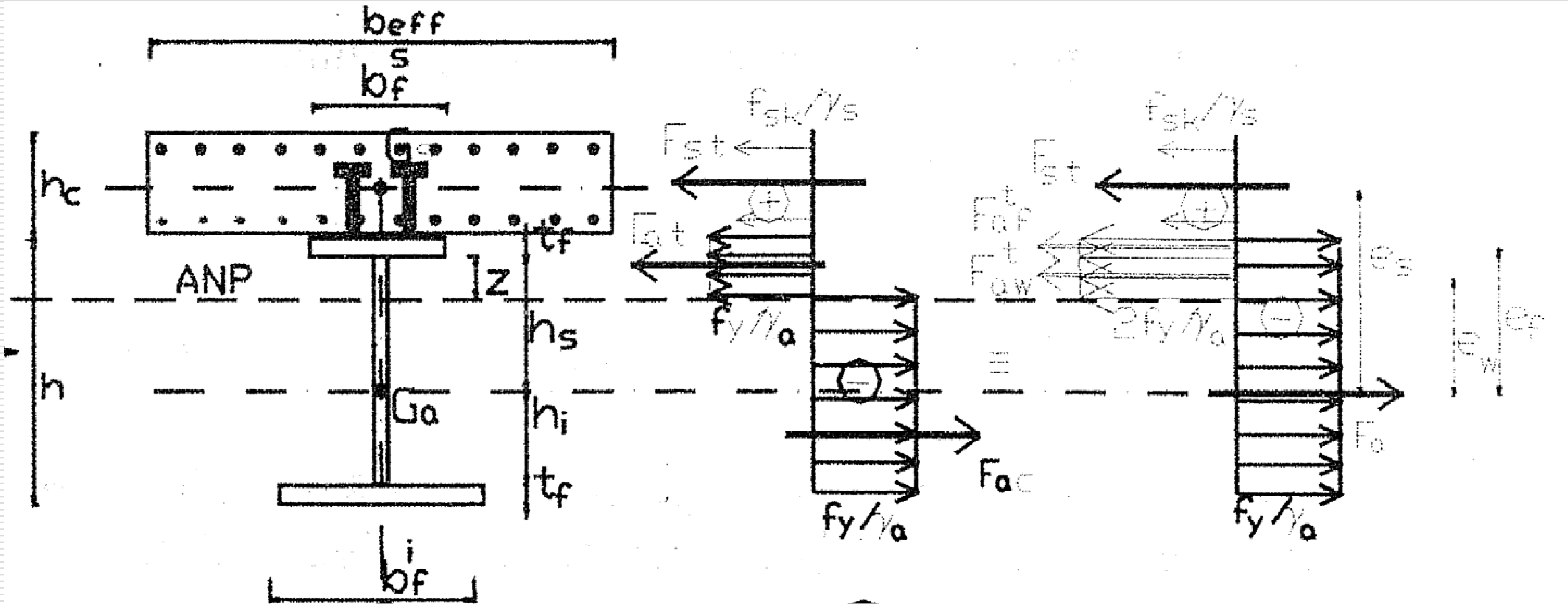
SECTION UNDER HOGGING MOMENT

- In case of hogging moment, the inferior fibers of beams are compressed, while the superior fibers will be tensioned.
- In these conditions, the tensioned concrete in the slab is neglected, according to the design assumptions.
- In consequence, in almost all the cases, the PNA will be located in the steel profile of the composite beam.

- The figure below presents the cross-section of the composite beam (the effective width b_{eff} and the stress diagrams on the height of the cross-section).

§ 2.9 Plastic resistance in bending

SECTION UNDER HOGGING MOMENT



PNA location in the steel profile (cross-section).

Plastic stresses diagram.

§ 2.9 Plastic resistance in bending

SECTION UNDER HOGGING MOMENT

- For the determination of the PNA position, it will be considered in design the plastic stresses diagram, in which tension stresses appear in the steel reinforcement (in concrete slab) and in the superior part of the steel profile (flange + web on height *z*).
- In this case the initial (real) stress diagram will be considered in an equivalent one, so that the steel profile will be considered as being compressed on its entire height at f_y/γ_a.
- In order to equilibrate these stresses, the tension stresses of the tensile part are doubled, to the value $2f_y/\gamma_a$.
- The equation that determines the PNA position is given by:

$$F_t = F_a$$

- In which: $F_t = F_{st} + F_{af}^t + F_{aw}^t$

§ 2.9 Plastic resistance in bending

SECTION UNDER HOGGING MOMENT

In the above expressions:

F_t represents the tensile stress resultant;

F_a represents the compressive stress resultant from the entire steel profile;

F_{st} is the tensile stress resultant of flexible reinforcements located in concrete slab;

F^t_{af} represents the tensile stress resultant of the top flange of steel profile;

F^t_{aw} is the tensile stress resultant on *z* web height in steel.

- The above forces are computed by:

$$F_a = A_a \cdot f_y / \gamma_a \qquad F^t_{af} = b^s_f \cdot t_f \cdot 2 f_y / \gamma_a$$

$$F_{st} = A_s \cdot f_{sk} / \gamma_s \qquad F^t_{aw} = z \cdot t_w \cdot 2 f_y / \gamma_a$$

§ 2.9 Plastic resistance in bending

SECTION UNDER HOGGING MOMENT

- Replacing these relation in the equilibrium relation it results:

$$F_t = A_s \cdot f_{sk} / \gamma_s + b_f^s \cdot t_f \cdot 2f_y / \gamma_a + z \cdot t_w \cdot 2f_y / \gamma_a$$

or, replacing F_t:

$$A_a \cdot f_y / \gamma_a = A_s \cdot f_{sk} / \gamma_s + b_f^s \cdot t_f \cdot 2f_y / \gamma_a + z \cdot t_w \cdot 2f_y / \gamma_a$$

From where we may find *z*:

$$z = \frac{A_a - 2b_f^s \cdot t_f}{2t_w} - \frac{A_s \cdot f_{sk} / \gamma_s}{2t_w \cdot f_y / \gamma_a}$$

where:

t_w is the thickness of the web of steel profile;

b^s_f and t_f are the width and the thickness of the superior flange of steel profile;

b_{eff} and h_c are the effective width and height of concrete slab;

f_{ck} and f_y are the characteristic strengths of concrete in compression and steel in tension respectively.

§ 2.9 Plastic resistance in bending

SECTION UNDER HOGGING MOMENT

- The plastic resistance in hogging is determined on the basis of the real (left) diagram of stresses, but is easier computed on the basis of stress distribution on the complete diagram, throughout the moment equation computed about the centroid of the steel reinforcements (which may or not correspond to the centroid of concrete slab):

$$M^{-}_{pl.Rd} = F_a \cdot e_s - F^{t}_{aw}\left(e_s - e_w\right) - F^{s}_{af}\left(e_s - e_f\right)$$

- where the distances e_s , e_w and e_f are determined as below (considering an unsymetric steel profile):

$$e_c = h_s + t_f + \frac{h_c}{2} = \frac{1}{2}\left(2h_s + 2t_f + h_c\right) \qquad e_f = h_s + \frac{t_f}{2} = \frac{1}{2}\left(2h_s + t_f\right)$$

$$e_w = h_s - \frac{z}{2} = \frac{1}{2}\left(2h_s - z\right)$$

- where: h_s is the height of the steel web profile, above its centroid.

§ 2.9 Plastic resistance in bending

SECTION UNDER HOGGING MOMENT

z is the distance from the superior fibre of the steel web to ANP:

$$z = \frac{A_a - 2b_f^s \cdot t_f}{2t_w} - \frac{A_s \cdot f_{sk} / \gamma_s}{2t_w \cdot f_y / \gamma_a}$$

- By these values, the plastic hogging resistance could be written as:

$$M_{pl.Rd}^- = A_a \cdot f_y / \gamma_a \cdot \frac{1}{2}\left(2h_s + 2t_f + h_c\right) - 2z \cdot t_w \cdot f_y / \gamma_a \left[\frac{1}{2}\left(2h_s + 2t_f + h_c\right) - \frac{1}{2}\left(2h_s - z\right)\right]$$
$$- 2b_f^s \cdot t_f \cdot f_y / \gamma_a \left[\frac{1}{2}\left(2h_s + 2t_f + h_c\right) - \frac{1}{2}\left(2h_s + t_f\right)\right]$$

or:

$$M_{pl.Rd}^- = \frac{1}{2}\frac{f_y}{\gamma_a}\left[A_a\left(2h_s + 2t_f + h_c\right) - 2z \cdot t_w\left(2t_f + h_c + z\right) - 2b_f^s \cdot t_f\left(t_f + h_c\right)\right]$$

§ 2.9 Plastic resistance in bending

SECTION UNDER HOGGING MOMENT

- In the case of plastic hogging design, the cross-section check is done by:

$$M^{-}_{Sd} \leq M^{-}_{pl.Rd}$$

with M^{-}_{Sd} - the hogging moment resulted from the static design

§ 2.10 Verification to vertical shear

- In case of steel-concrete composite beams, the design to vertical shear is conducted according to Eurocode 3 provisions, by considering that the entire vertical shear stress is taken by the steel profile, neglecting the presence of concrete slab.
- The checking condition to vertical shear is:

$$V_{Sd} \leq V_{pl.Rd}$$

where: V_{sd} shear force resulted from the static design;

$V_{pl,Rd}$ represents the plastic shear resistance, computed as:

$$V_{pl.Rd} = A_v \cdot \frac{f_y}{\sqrt{3}} \cdot \frac{1}{\gamma_a}$$

§ 2.10 Verification to vertical shear

- In the above formula:
  - $f_y/\sqrt{3}$ is the shear resistance of steel;
  - A_v is the shear area of the steel member, computed by:
    - For I or H welded profiles (clear web area):

$$A_v = \sum d t_w$$

    - For I or H rolled profiles: $A_v = A - 2b_f t_f + (t_w + 2r) t_f$

where:

- d and t_w are the clear distance between welds and the web thickness respectively of welded profile;
- A the area of the hot-rolled profile;
- b_f and t_f are the width and thickness respectively of the flange of steel profile;
- r is the radius of the root fillet between flange and web.

§ 2.10 Verification to vertical shear

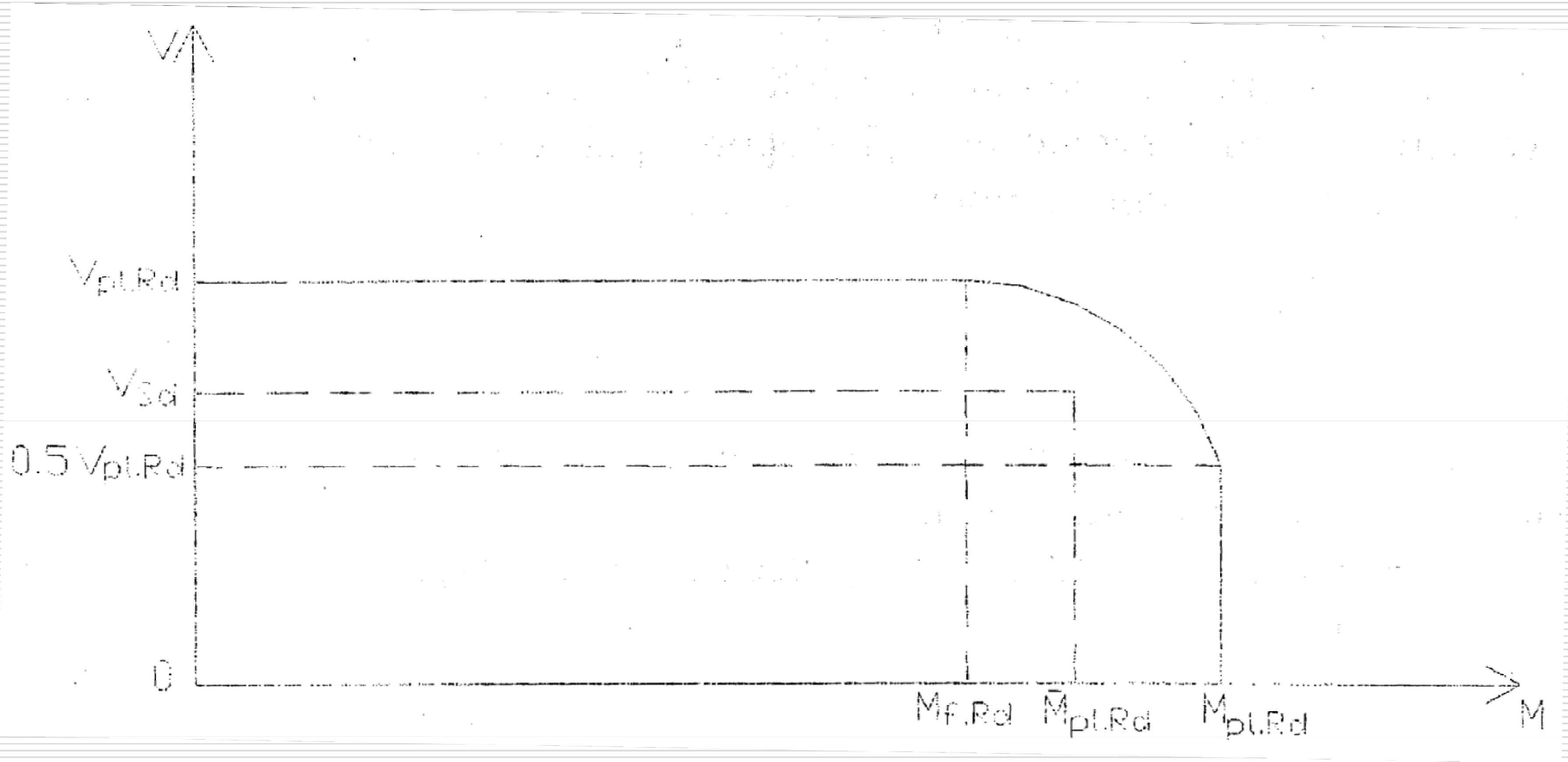
- In practice, the above inequality takes a more severe form:

$$V_{Sd} \leq 0.5 V_{pl.Rd}$$

- This formula is used in order to neglect the influence of the shear force on the plastic moment capacity $M_{pl,Rd}$.
- If the above inequality is not respected, we have to consider the interaction between the shear force and plastic moment when evaluating the plastic moment resistance.
- The interaction curve bending moment – shear force is given in the below figure.
- In this situation, the cross-section check to bending is made by the following relation:

$$M_{Sd} \leq M_{pl.Rd(redus)}$$

§ 2.10 Verification to vertical shear



Bending moment – shear force interaction curve

§ 2.10 Verification to vertical shear

In the above formula:

- M_{sd} is the moment resulted from the static design ;
- $M_{pl,Rd(red)}$ is the reduced plastic resistance, reduced by the influence of the shear force and determined by interpolation, between $M_{pl,Rd}$ and $M_{f,Rd}$ by the relations:

$$M_{pl.Rd(redus)} = M_{f.Rd} + \left(M_{pl.Rd} - M_{f.Rd}\right)\left[1 - \left(\frac{2V_{Sd}}{V_{pl.Rd}} - 1\right)^2\right]$$

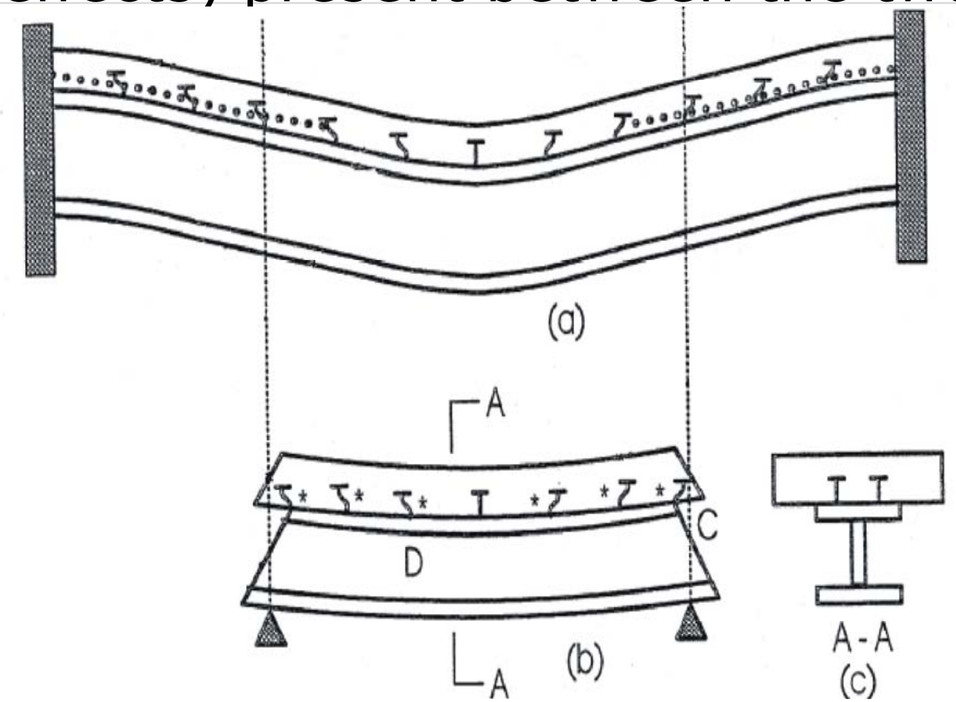
where:

- V_{sd} is the design shear force, resulted from static design;
- $V_{pl,Rd}$ is the plastic shear resistance;
- $M_{f,Rd}$ is the plastic moment of the composite section, computed considering that the structural steel beam is made only of flanges (the steel web is neglected). The effective width of this composite beam is similar to the one used for finding the plastic bending resistance $M_{pl,Rd}$.

§ 2.11 Design of shear connection

- In the case of composite steel-concrete beams, the shear connectors and transversal reinforcement from the concrete slab must be disposed on all the beam length, in order to transmit the longitudinal shear stresses between the concrete slab and the steel profile, neglecting the adherence effect (through physico-chemical effects) present between the two materials.

The deformation shape of a (fixed) composite beam.



§ 2.11 Design of shear connection

- The number of connectors results from the ratio between the longitudinal shear design resistance and the design resistance of a single connector P_{Rd}.
- Also, the connecting devices should have sufficient capacity to up-lifting forces that may exist between the concrete slab and the steel profile. In this scope, the connectors should be conceived and designed to a nominal traction stress (perpendicular on the upper steel flange) of at least 10% of their shear design.

Obs: The usual headed stud connectors have a sufficient resistance to up-lifting forces in slab.

- The number of connectors that are disposed on the steel profile could assure or not a full shear connection.
- The design of composite connection presented in this paragraph is done in the hypothesis of assuring a full shear connection between steel and concrete elements.

§ 2.11 Design of shear connection

LONGITUDINAL SHEAR FORCE

- A shear span of a composite beam presents a **full (complete) shear connection** if any increase in the number of connectors disposed on that span does not lead to any increase in the moment resistance.
- In the opposite case, the connection is partial.

- For a full shear connection, the longitudinal shear force V_l to be resisted by the shear connectors, between the point of maximum sagging moment and a simple end support (or an inflexion point) is computed by:

$$V_l = F_{cf} = \min\left(F_{cf1}, F_{cf2}\right)$$

where:

  - $F_{cf,1}$ is the axial resistance of the steel section: $F_{cf1} = A_a \dfrac{f_y}{\gamma_a}$

§ 2.11 Design of shear connection

LONGITUDINAL SHEAR FORCE

- $F_{cf,2}$ is the axial compressive resistance of concrete slab:

$$F_{cf2} = A_c \frac{0.85 f_{ck}}{\gamma_c}$$

- In the above formulae:
  - A_a cross-section area of the steel profile;
  - A_c the effective slab area on b_{eff};
- When the beam is continuous, the longitudinal shear force V_l, computed between a point of maximum bending moment and an intermediate support or a fixed end is computed by:

$$V_l = F_{cf} + A_s \frac{f_{sk}}{\gamma_s} + A_{ap} \frac{f_{yp}}{\gamma_{ap}}$$

where:

  - F_{cf} represents the compressive resistance of concrete, defined by above relations (taken as 0 in the case of cantilevers);
  - A_s represents the longitudinal reinforcing area located in the negative effective concrete width of the slab;
  - A_{ap} is the area of profiled sheeting, on negative b_{eff}.

§ 2.11 Design of shear connection

DESIGN RESISTANCE OF SHEAR CONNECTORS

- The design resistance, including their bearing capacity depends on their geometrical characteristics.

Design resistance of headed studs in concrete slabs

- The design shear resistance of a headed stud connector, welded by means of direct welding is determined as the minimum of P_{rd1} and P_{rd2} values, computed as below:

$$P_{Rd1} = 0.8 f_u \cdot \frac{\pi d^2}{4} \cdot \frac{1}{\gamma_v} \qquad P_{Rd2} = 0.29 \alpha d^2 \sqrt{f_{ck} \cdot E_{cm}} \frac{1}{\gamma_v}$$

In the above formulae:

- *d* represents the diameter of the shaft;
- f_u ultimate tensile strength of shear connector (<500N/mm²);
- f_{ck} is the characteristic compressive strength of concrete;
- E_{cm} is the secant modulus of elasticity of concrete;
- *h* total height of studs, including their head;

§ 2.11 Design of shear connection

DESIGN RESISTANCE OF SHEAR CONNECTORS

Design resistance of headed studs in concrete slabs

In the above formulae:

- γ_V – the partial safety factor for connectors =1,25;
- α – coefficient, that take the value:

$\alpha = 0.2[(h/d)+1]$	for	$3 \leq h/d \leq 4$
$\alpha = 1.0$	for	$h/d > 4$

- the above formulae are valid only for connectors with d≤22mm.
- In addition, the following requirements should be fulfilled:
  - The ring welding should have a regular form and a deposit without any defficiency;
  - The diameter of the ring weld must not be less than *1,25d*;
  - The mean height of the weld should not be less than *0,2d*, and its minimum height greater than *0,15d*.

§ 2.11 Design of shear connection

DESIGN RESISTANCE OF SHEAR CONNECTORS

Design resistance of headed studs in composite concrete slabs

- In the presence of profiled sheeting, the design shear resistance is identical to the one used for full slabs, but it is affected by a coefficient k, computed in function of the disposition of the profiled sheeting on the steel profile.
- In the case in which the ribs of the profiled sheeting are parallel to the steel web, the reduction coefficient k_l is given by the following relation:

$$k_l = 0.6 \frac{b_0}{h_p} \left[\frac{h}{h_p} - 1 \right] \leq 1.0$$

- If the ribs of the profiled sheeting are transversal to the supporting beams and the diameter of the headed stud connectors is smaller than 20mm, then the design shear resistance is affected by the reduction coefficient k_t, given by:

$$k_t = \frac{0.7}{\sqrt{N_r}} \cdot \frac{b_0}{h_p} \left[\frac{h}{h_p} - 1 \right]$$

§ 2.11 Design of shear connection

DESIGN RESISTANCE OF SHEAR CONNECTORS

Design resistance of headed studs in composite concrete slabs

In the above formulae:

- h is the total height of the connector, including its head ($h \leq h_p + 75\ mm$)
- N_r represents the number of connectors on a single rib, at its intersection to the steel beam ($N_r \leq 2$).

Obs: 1. The design resistance of the connectors depends on their geometry. The worldwide research has adopted various formulae used for the design resistance of different types of connectors.

2. For unstandardised connectors, their design resistance could be determined on the basis of push-out type tests (presented in the Annex B of Eurocode 4).

§ 2.11 Design of shear connection

REQUIRED NUMBER OF CONNECTORS

- The required number of connectors for the realisation of a full shear connection is determined on the base of the longitudinal shear force V_l and the design resistance of a single connector P_{Rd}:

$$V_l = \min\left(F_{cf1}; F_{cf2}\right) \quad \text{respectively} \quad P_{Rd} = \min\left\{P_{Rd1}; P_{Rd2}\right\}$$

(values already determined)

- The required number of connectors N required on a shear span is determined by the following relation:

$$N = \frac{V_l}{P_{Rd}}$$

Obs: The "shear span" term is used for a span between the points of maximum bending moment (sagging) and those of zero or minimum bending moment.

§ 2.11 Design of shear connection

REQUIRED NUMBER OF CONNECTORS

- The connectors are fixed on the top flange of the steel profile, on one or two rows.
- When they are disposed on two rows, they could be staggered by half-span on each row.
- The required number of connectors *N* are uniformly disposed on the shear span length.

SPACING OF CONNECTORS

- When the connectors are disposed at equal spans (usual disposing situation), the spacing between connectors results simply, by the ratio between the shear span and
  - the total number of connectors *N*, in the case in which they are disposed on a single row;
  - *N/2* in case in which the connectors are disposed on 2 rows.

Obs: an adequate space must be assured between the beam end and the first connector.

§ 2.11 Design of shear connection

SPACING OF CONNECTORS

- The spacing of connectors should take into account other specific conditions (see section 6.6.5 of Eurocode 4):
  - Where it is assumed in design that the stability of either the steel or the concrete member is ensured by the connection between the two, the spacing of the shear connectors shall be sufficiently close for this assumption to be valid;
  - Where a steel compression flange is assumed to be in Class 1 or Class 2 because of restraint from shear connectors, the centre-to-centre spacing of the shear connectors in the direction of compression should be not greater than :
    - where the slab is in contact over the full length (e.g. solid slab): $s \le 22t\sqrt{\frac{235}{f_y}}$
    - where the slab is not in contact over the full length: $s \le 15t\sqrt{\frac{235}{f_y}}$
    - the clear distance from the edge of a compression flange to the nearest line of shear connectors should be not greater than: $s_1 \le 9t\sqrt{\frac{235}{f_y}}$

§ 2.11 Design of shear connection

SPACING OF CONNECTORS

In the above formulae, *t* is the steel profile flange thickness;

- In buildings, the maximum longitudinal centre-to-centre spacing of shear connectors should be not greater than:

$$s \leq 6h_c \quad \text{şi} \quad s \leq 800mm$$

h_c is the total slab thickness.

OTHER RECOMMENDATIONS FOR CONNECTORS

- The following recommendations are valid for shear headed stud connectors located in composite beams:
  - The overall height of a stud should be not less than *3d*, where *d* is the diameter of the shank.
  - The spacing of studs in the direction of the shear force should be not less than 5d; the spacing in the direction transverse to the shear force should be not less than 2,5d in solid slabs and 4d in other cases.

§ 2.11 Design of shear connection

OTHER RECOMMENDATIONS FOR CONNECTORS

- Except when the studs are located directly over the web, the diameter of a welded stud should be not greater than 2,5 times the thickness of that part to which it is welded (flange).
- In the case in which the headed studs are used with profiled steel sheeting, the headed studs could be welded through the profiled sheeting. It is recommended that the welding of a connector will pass trough a single steel sheeting (and not in the overlapping zone). The maximum thickness of the profiled sheeting should be less than 1.25mm for galvanized sheeting and 1.5mm for normal steel sheeting. The galvanizing layer should be less than 30μm on each side of the steel sheeting.
- In the case in which the headed studs are used with profiled steel sheeting, the minimum width of the troughs that are to be filled with concrete should be not less than 50 mm.

§ 2.11 Design of shear connection

OTHER RECOMMENDATIONS FOR CONNECTORS

- In the case in which the headed studs are used with profiled steel sheeting, the total height of the connectors should pass by at least *2d* the upper face of the profiled sheeting.
- It is recommended that the profiled sheeting to be fixed in each trough, in order to behave compositely. The fixing could be done either by stud connectors or by point welding.

Obs: Other recommendations exist for other types of connectors (relatively) standardised:

- Block or bar connectors;
- Reinforcement anchor or hook connectors;
- Channel (U) or corner (L) profiles.