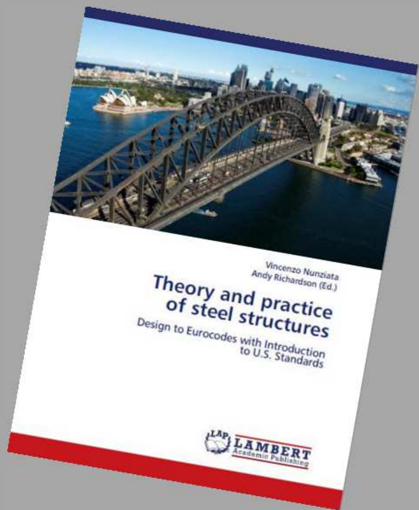
October 11, 2013

Theory and Practice of Steel Structures

Design to Eurocodes with Introduction to U.S. Standards

… and some structural applications



Vincenzo Nunziata

“In a globalized world where information travels fast and where science is rapidly evolving, it is no longer thinkable that codes and standards are unique to any particular country”

The National Standard Organizations of the following countries are bound to implement European Standard:

Austria, Belgium, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Latvia, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Slovakia, Slovenia, Spain, Sweden, Switzerland and United Kingdom.

Eurocodes + National Annex

=

National Standard Eurocodes (e.g. BS EN 1993-1-1)

The Structural Eurocodes program comprises the following standards generally consisting of a number of Parts:

EN 1990 Eurocode: Basis of structural design
EN 1991 Eurocode 1: Actions on structures
EN 1992 Eurocode 2: Design of concrete structures
EN 1993 Eurocode 3: Design of steel structures
EN 1994 Eurocode 4: Design of composite steel and concrete structures
EN 1995 Eurocode 5: Design of timber structures
EN 1996 Eurocode 6: Design of masonry structures
EN 1997 Eurocode 7: Geotechnical design
EN 1998 Eurocode 8: Design of structures for earthquake resistance
EN 1999 Eurocode 9: Design of aluminum structures.

Eurocodes → Parts + Sub-Parts = 59 → ~ 3500 Pages !

For example

BS EN 1993-1-1

Sub-Part 1: General Rules and Rules for Buildings

Part 1: Design of steel structures

Eurocode 3

European **N**orm

British **S**tandard

1. MATERIALS

Table 1.1 *Nominal values of yield strength fy and ultimate tensile strength fu for hot rolled structural steel.*

Standard and steel grade	Nominal thickness of the element t [mm]			
	t ≤ 40mm		40mm ≤ t ≤ 80mm	
	f_y [N/mm^2]	f_u [N/mm^2]	f_y [N/mm^2]	f_u [N/mm^2]
EN 10025-2				
S 235	235	360	215	360
S 275	275	430	255	410
S 355	355	510	335	470
S 450	440	550	410	550
EN 10025-3				
S 275 N/NL	275	390	255	370
S 355 N/NL	355	490	335	470
S 420 N/NL	420	520	390	520
S 460 N/NL	460	540	430	540
EN 10025-4				
S 275 M/ML	275	370	255	360
S 355 M/ML	355	470	335	450
S 420 M/ML	420	520	390	500
S 460 M/ML	460	540	430	530
EN 10025-5				
S 235 W	235	360	215	340
S 355 W	355	510	335	490

Table 1.1 UK *Values for yield strength fy and ultimate tensile strength fu from product standards (EN 10025-2 and EN 10210-1)*

Steel grade	Thickness range (mm)	Yeld strength, f_y (N/mm^2)	Thickness range (mm)	Ultimate tensile strength, f_u (N/mm^2)
S 235				
	t ≤16mm	235	t <3mm	360
	16< t ≤40mm	225		
	40< t ≤63mm	215	3< t ≤100mm	360
	63< t ≤80mm	215		
	80< t ≤100mm	215		
S 275				
	t ≤16mm	275	t <3mm	430
	16< t ≤40mm	265		
	40< t ≤63mm	255	3< t ≤100mm	410
	63< t ≤80mm	245		
	80< t ≤100mm	235		
S 355				
	t ≤16mm	355	t <3mm	510
	16< t ≤40mm	345		
	40< t ≤63mm	335	3< t ≤100mm	470
	63< t ≤80mm	325		
	80< t ≤100mm	315		

Table 1.3 *Yield Strength and Ultimate Tensile Strength for steel grades used in the U.S. for some typical steel sections.*

Standard and Grade	Available Shapes	f_y [N/mm²]	f_u [N/mm²]
ASTM A992 High-strength low-alloy	Preferred grade for W-Shapes Also used for M, S, C, MC, and L shapes as well as plate if specified.	345	448
ASTM A36 Carbon Steel	Preferred grade for M, S, C, MC, and L-shapes as well as plate. Can also be used for W and HP Shapes.	248	400
ASTM A572 High-strength low-alloy	Preferred grade for HP Shapes. Also used for M, S, C, MC, and L-shapes as well as plate and W shapes.	345	448
ASTM A500 Grade B	Used for Rectangular Hollow Structural Sections (HSS)	317	400
ASTM A500 Grade B	Used for Round Hollow Structural Sections (HSS)	290	400
Note: AISC table 2-3 and 2-4 provides further information about grades of steel, and specifications. Only the preferred grades are listed here. For design in the US, stress is expressed in kips/ in² (1 kip is 1000 lbs). To convert from N/mm² to ksi, multiply the values in N/mm² by 0.145.			

S 355 J0 G3

Finishing status, G3 or G4 at discretion of the manufacturer.

Indicates the temperature of the resilience test:

R = temperature of 23 °C ± 5 °C
0 = temperature of 0 °C
2 = temperature of - 20 °C

Resilience; J or K; for 27 and 40 joules, respectively .

Indication of the minimum yield strength required for thicknesses ≤ 40mm, expressed in N/mm^2.

Symbol S: Structural Steel.

Classification of Cross-Sections

EUROCODE		**BS 5950**
Class 1	→	Plastic
Class 2	→	Compact
Class 3	→	Semi-Compact
Class 4	→	Slender

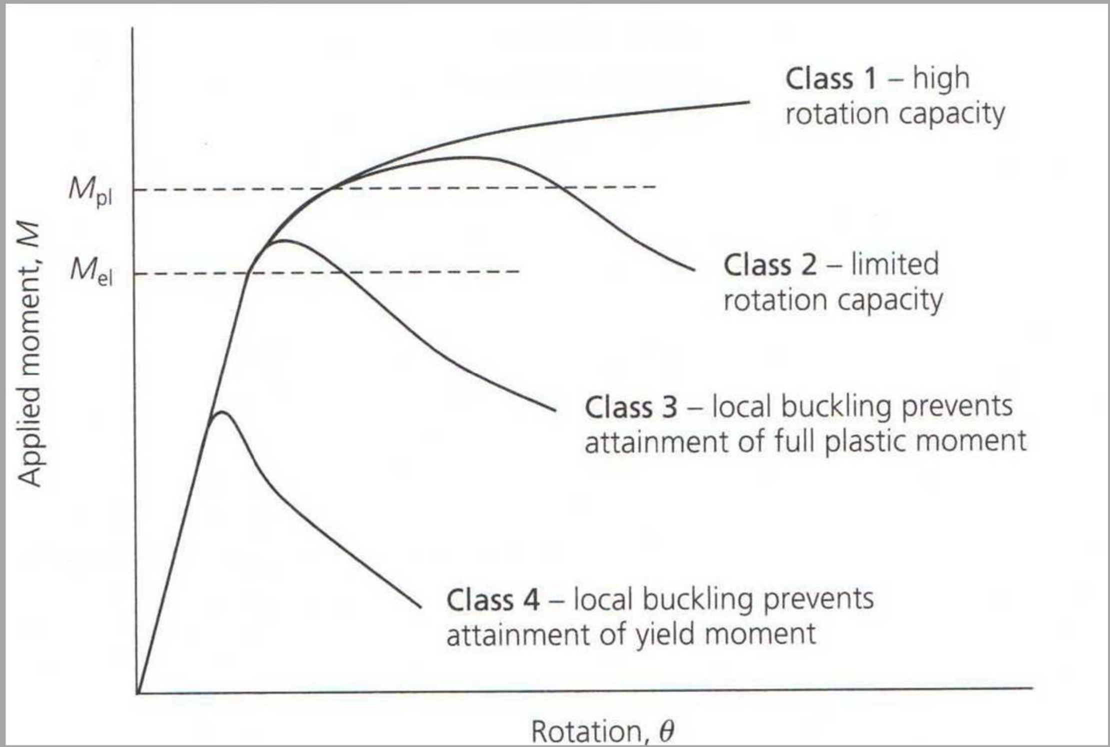


Table for classification of cross - sections

Table F1 – IPE Series Profiles

Profile	Compression			Bending		
	S235	S275	S355	S235	S275	S355
IPE 80	1	1	1	1	1	1
IPE 100	1	1	1	1	1	1
IPE 120	1	1	1	1	1	1
IPE 140	1	1	1	1	1	1
IPE 160	1	1	1	1	1	1
IPE 180	1	1	2	1	1	1
IPE 200	1	1	2	1	1	1
IPE 220	1	1	2	1	1	1
IPE 240	1	2	2	1	1	1
IPE 270	2	2	3	1	1	1
IPE 300	2	2	4	1	1	1
IPE 330	2	3	4	1	1	1
450						
IPE 500	3	4	4	1	1	1
IPE 550	4	4	4	1	1	1
IPE 600	4	4	4	1	1	1

Tabble F2 – HEA Series Profiles

Profile	Compression			Bending		
	S235	S275	S355	S235	S275	S355
HEA 100	1	1	1	1	1	1
HEA 120	1	1	1	1	1	1
HEA 140	1	1	1	1	1	1
HEA 160	1	1	1	1	1	1
HEA 180	1	1	2	1	1	2
HEA 200	1	1	2	1	1	2
HEA 220	1	1	2	1	1	2
HEA						

2. LIMIT STATE DESIGN

LIMIT STATES

ULTIMATE LIMIT STATES

are those associated with collapse, or other forms of structural failure that may endanger the safety of persons.

SERVICEABILITY LIMIT STATES

are those related to the functioning and use of the structure beyond which there is no guarantee that the structure will perform as intended

ULTIMATE LIMIT STATES – EC3

The limit state of ***equilibrium***, **EQU**, which considers the structure, ground or the whole soil-structure as rigid bodies. An example where this limit state would apply would be the overturning of retaining walls.

The limit state of ***resistance of the structure*, STR**, which also covers the foundation elements and ground support. This limit state will apply when designing structural members, and is used almost exclusively.

The limit state of ***resistance of the ground***, **GEO**, applies when sizing of foundations and retaining walls, checking with structures that interact directly with the ground, and when checking the global stability of the whole structure.

Design Loads (or actions) for Ultimate Limit States

$$F_d = \gamma_G \cdot \sum_{j \geq 1} G_{k,j} + \gamma_p \cdot P + \gamma_{Q,1} \cdot Q_{k,1} + \gamma_Q \cdot \sum_{i>1} \psi_{0,i} \cdot Q_{k,i}$$

Where:

F_d = design value of an action (Design Load)

$G_{k,j}$ = characteristic value of permanent action j

P = relevant representative value of a prestressing action

$Q_{k,1}$ = characteristic (service) load value of the leading variable action 1

$Q_{k,i}$ = characteristic (service) load value of the accompanying variable action i

γ_G = partial factor for permanent action (see Table 2.1)

γ_p = partial factor for prestressing actions (see Table 2.1)

γ_Q = partial factor for variable actions (see Table 2.1)

$\psi_{0,i}$ = factor for combination value of a variable action (see Table 2.2)

Table 2.1 *Design values of actions*

		Partial factor	EQU	**STR**	GEO
Permanent action	Favorable	γ_G	0,9	**1,0**	1,0
	Unfavorable		1,1	**1,35**	1,0
Variable action	Favorable	γ_Q	0,0	**0,0**	0,0
	Unfavorable		1,5	**1,5**	1,3

Table 2.2 Recommended values of ψ factors

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A: domestic, residential areas	0.7	0.5	0.3
Category B: office areas	0.7	0.5	0.3
Category C: congregation areas	0.7	0.7	0.6
Category D: shopping areas	0.7	0.7	0.6
Category E: storage areas	1.0	0.9	0.8
Category F: traffic area, vehicle weight ≤ 30kN	0.7	0.7	0.6
Category G: traffic area, 30kN < vehicle weight ≤ 160kN	0.7	0.5	0.3
Category H: roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0.7	0.5	0.2
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0.7	0.5	0.2

The check is summarized by the following relationship:

$$E_d \leq R_d$$

Where:

E_d is the effect of the design action (moment, shear, tension, etc.).

R_d is the corresponding design resistance.

Steel Design in the US – Load Combinations

<u>LRFD Design (Strength Design) Load Combinations For 2009 IBC:</u>

1,4 D

1,2 D + 1,6 L + 0,5 (Lr or S or R)

1,2D + 1,6 (Lr or S or R) + (0,5L* or .8W)

1,2D + 1,6W + 0,5 L* + 0,5 (Lr or S or R)

.9D ± 1.6W

.9D ± 1.6E

This check is summarized by the following relationship:

$$\phi \bullet R_n > R_u$$

With

$\phi \bullet R_n$ the capacity, which is the material strength multiplied by a material strength reduction factor, phi.

R_u the Factored Load (also referred to as the Required Strength, or 'Ultimate' Load)

For design of steel beams according to AISC, the following limit state applies for LRFD:

$$\phi \bullet M_n > M_u$$

where

M_u is the Factored Load (required strength) based on load combinations.

For a dead and live load combination, the ultimate load on the beam would be

$$M_u = 1{,}2 \bullet M_D + 1{,}6 \bullet M_L$$

with

M_D = moment due to dead (permanent) loads

M_L = the moment due to live (imposed) loads

$\phi \bullet M_n$ is the Beam Capacity

with

ϕ = the capacity reduction factor.

For bending, $\phi = 0{,}9$ (AISC Specification F1)

M_n = Nominal resistance (as denoted by the subscript 'n') = $F_y \bullet Z$ for laterally braced compact sections.

F_y = yield strength of the steel

Z = is the plastic section modulus (i.e. $W_{y,pl}$ in EC)

SERVICEABILITY LIMIT STATES

Characteristic combination:

$$F_d = \sum_{j\geq 1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \psi_{0,i} \cdot Q_{k,i}$$

Frequent combination:

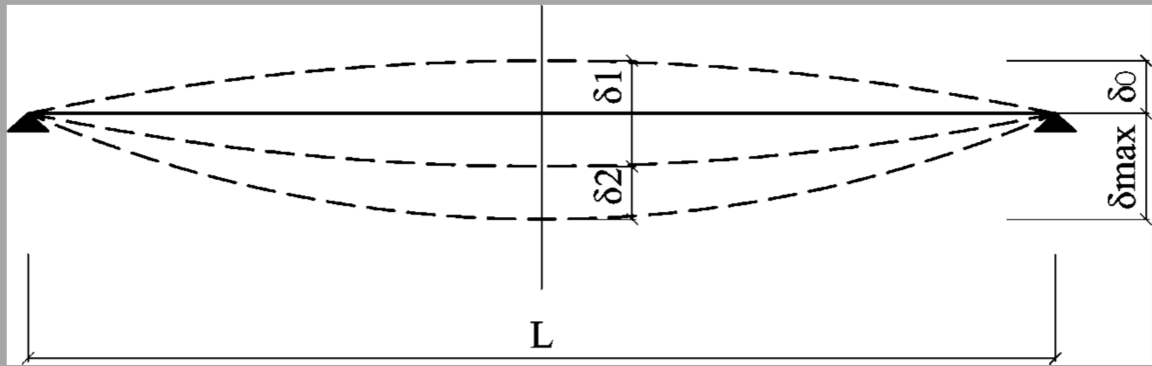
$$F_d = \sum_{j\geq 1} G_{k,j} + P + \psi_{1,1} \cdot Q_{k,1} + \sum_{i>1} \psi_{2,i} \cdot Q_{k,i}$$

Quasi-permanent combination:

$$F_d = \sum_{j\geq 1} G_{k,j} + P + \sum_{i\geq 1} \psi_{2,i} \cdot Q_{k,i}$$

For the simply supported beam in Figure, the following applies:

$$\delta_{max} = \delta_1 + \delta_2 - \delta_0$$



where

δ_{max} The remaining total deflection less the camber

δ_0 The camber in the unloaded structural member

δ_1 The initial part of the deflection under permanent loads of the relevant combination of actions

δ_2 The additional part of the deflection due to the variable actions of the relevant combination of actions and eventually more long-term part of the deflection under permanent loads.

Table 2.4 Recommended limit values for the vertical displacements

Condition	Limit	
	δ_{max}	δ_2
Roofs not accessible except for normal maintenance and repair.	L/200	L/250
Roofs accessible with occupancy	L/250	L/400
Floors	L/250	L/400
Beams loaded directly or indirectly from walls & columns.	L/500	-

Table 2.4.1 US – Limits on vertical deflections

Condition	Limit	
	Δ_L	Δ_{D+L}
Floor members	L/360	L/240
Roof members supporting plaster ceiling	L/360	L/240
Roofs members supporting non-plaster ceiling	L/240	L/180
Roof members not supporting a ceiling	L/180	L/120

Resistance of cross-sections

Elastic Verification of cross-sections

$$\sigma_{x,Ed}^{\ 2} + \sigma_{z,Ed}^{\ 2} - \sigma_{z,Ed} \cdot \sigma_{x,Ed} + 3 \cdot \tau_{Ed}^{\ 2} \leq \left(\frac{f_y}{\gamma_{M0}} \right)^2$$

where

$\sigma_{x,Ed}$ is the design value of the local longitudinal stress at the point of consideration

$\sigma_{z,Ed}$ is the design value of the local transverse stress at the point of consideration

τ_{Ed} is the design value of the local shear stress at the point of consideration.

$\gamma_{M0} = 1$ is the partial factor for design stress

Checks based on the elastic resistance of cross sections are more conservative than the checks based on the plastic resistance.

Exercise 6

Design the steel slab shown in Figure 2.2 part of a building for residential use, applying the limit states. The beams steel grade is S235; the lateral buckling of beams is prevented by the slab.

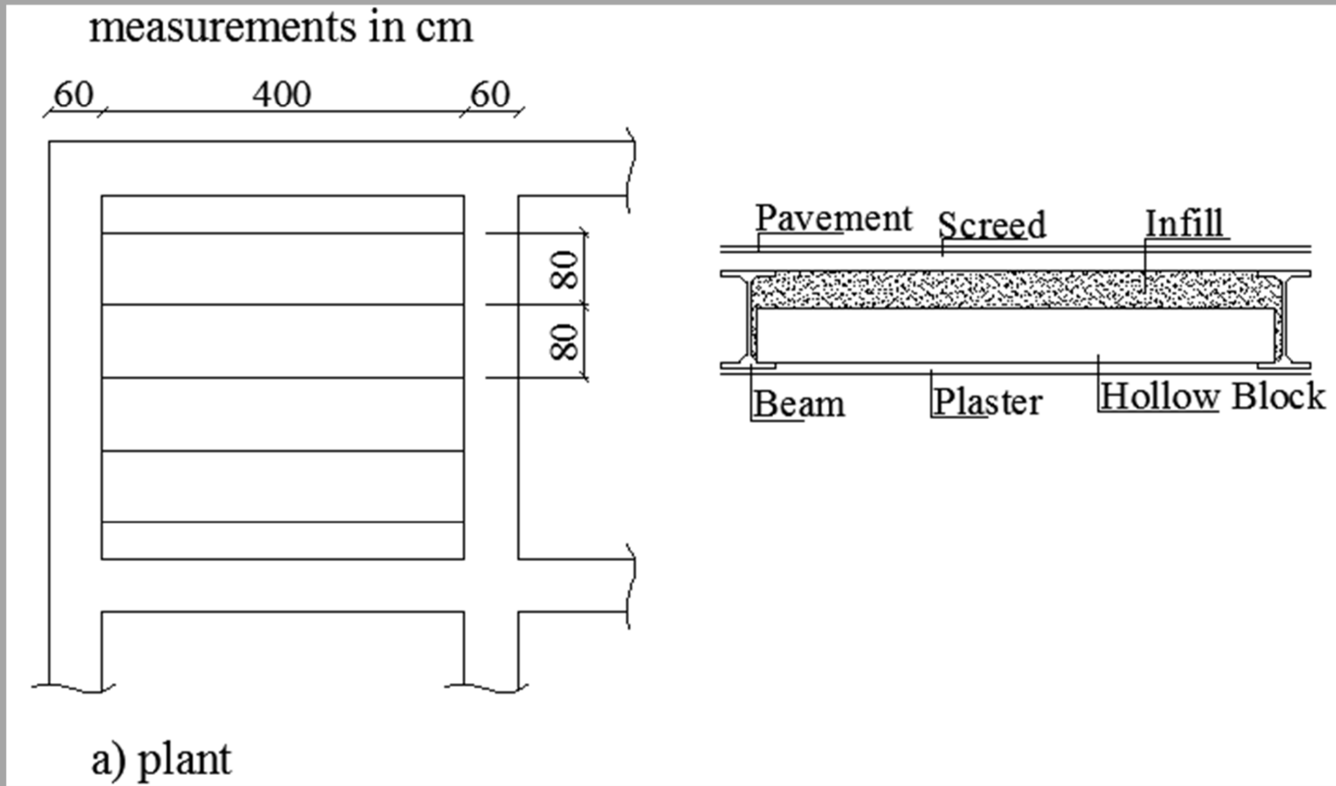


Figure 2.2 Steel slab

<u>*SOLUTION*</u>

Load analysis.

- Permanent loads:

Pavement + screed	$1,20 \cdot 0,80 = 0,96 \ kN/m$
Infill	$1,00 \cdot 0,80 = 0,80 \ kN/m$
Hollow Block	$0,42 \cdot 0,80 = 0,34 \ kN/m$
Plaster ceiling	$0,30 \cdot 0,80 = 0,24 \ kN/m$
Self-weight beam IPE 140 (initial assumption)	$0,13 \ kN/m$
Sum:	$2,47 \ kN/m$

Imposed loads.

(category A: floors)

- refer to EN 1991 Table 6.2 and 6.3: $2 \cdot 0{,}8 \;\; kN/m = 1{,}6 \;\; kN/m$

Beam Properties

IPE 140; Per the Appendix B
Second Moment of Area (y-y axis) = $I_y = 541\ cm^4$
Elastic section modulus = $W_{el,y} = 77{,}3\ cm^3$
Plastic section modulus = $W_{pl,y} = 88{,}3\ cm^3$

Serviceability limit states

The verification of this limit state is mainly based on the calculation of displacements to be lower than the limit allowed (section 2.7).

Design load = F_d = G + Q = 2,47 + 1,6 = 4.07 kN/m

Take the simple beam span to be 5% greater than the clear span:

$$L \cong 1{,}05 \cdot 4 = 4{,}20m$$

$$\delta_{\max} = \frac{5}{384} \cdot \frac{F_d \cdot L^4}{E \cdot I} = \frac{5}{384} \cdot \frac{4,07 \cdot 4200^4}{210000 \cdot \left(541 \cdot 10^4\right)} = 14,5mm \leq \frac{L}{250} = 16,8mm$$

$$\delta_2 = \frac{5}{384} \cdot \frac{Q \cdot L^4}{E \cdot I} = \frac{5}{384} \cdot \frac{1,6 \cdot 4200^4}{210000 \cdot \left(541 \cdot 10^4\right)} = 5,7mm < \frac{L}{400} = 10,5mm$$

The deflection checks are satisfied.

Elastic Verification

Design loads: $F_d = \gamma_G \cdot G + \gamma_Q \cdot Q = 1,35 \cdot 2,47 + 1,5 \cdot 1,6 = 5,73\ kN/m$

$$M_{\max} = \frac{F_d \cdot L^2}{8} = \frac{5,73 \cdot 4,2^2}{8} = 12,63\ kNm$$

$$\sigma_{\max} = \frac{M_{\max}}{W_{el,y}} = \frac{12,63 \cdot 10^6}{77,3 \cdot 10^3} = 163\ \mathrm{N/mm^2} < \frac{f_y}{\gamma_{M0}} = \frac{235}{1,00} = 235\ N/mm^2$$

Hence, the elastic verification checks are satisfied.

Plastic Verification

Cross section classification

$$\varepsilon = \sqrt{235/f_y} = 1$$

Outstand flanges in compression: c/t = 27,1/6,9 = 3,93 ≤ 9 Class 1

Web – internal part in bending: c/t =112,2/4,7=23,8 ≤ 72 Class 1

Therefore, the IPE 140 Section, grade S235 steel is Class 1 (*Also confirmed in the appendix F table F1*).

Design actions:

Shear:

$$V_{\max} = \frac{F_d \cdot L}{2} = \frac{5,73 \cdot 4,20}{2} = 12,03\,kN$$

Bending moment (as before):

$$M_{\max} = 12,63\,kNm$$

Plastic shear resistance of the cross section

$$V_{pl,Rd} = A_v \cdot \frac{f_y / \sqrt{3}}{\gamma_{M0}} = \left[A - 2bt_f + (t_w + 2r) \cdot t_f \right] \cdot \frac{f_y / \sqrt{3}}{\gamma_{M0}} =$$

$$\left[1640 - 2 \cdot 73 \cdot 6,9 + (4,4 + 2 \cdot 7) \cdot 6,9 \right] \cdot \frac{235}{1,00 \cdot \sqrt{3}} = 103114\ N = 103,1\ kN$$

$$V_{\max} \leq \frac{1}{2} V_{pl,Rd}$$

Since the maximum applied shear force, V_{max} is less than 50% of $V_{pl,\ Rd}$, there is no need to take a reduction in the plastic moment resistance of the beam due to the simultaneous presence of shear.

Plastic moment resistance

$$M_{pl,Rd} = \frac{W_{pl} \cdot f_y}{\gamma_{M0}} = \frac{88,3 \cdot 235 \cdot 10^{-3}}{1,00} = 20,75\ kNm$$

As $M_{max} < M_{pl,\ Rd}$, the check is satisfied

Comment

The elastic moment resistance ($M_{el,Rd} = W_y \cdot f_y$) is about 8% less than the plastic moment resistance, which provides a slight safety advantage. In addition to providing a conservative check, the elastic moment resistance is also less tedious to compute, which is another advantage. <u>For the majority of steel beams, the size will be governed by the deflection limitations</u> ..

– For AISC (US Standards)

$$\Delta_L = \frac{5}{384} \cdot \frac{w_L \cdot L^4}{E \cdot I} = \frac{5}{384} \cdot \frac{1{,}6 \cdot 4200^4}{210000 \cdot \left(541 \cdot 10^4\right)} = 5{,}7mm < \frac{L}{360} = 11{,}7mm$$

The beam meets the requirement for live load deflection. Also check the total load deflection due to dead plus live load (though the IBC states the dead load may be taken as zero in this case).

$$\Delta_{\text{total}} = \frac{5}{384} \cdot \frac{w_{tot} \cdot L^4}{E \cdot I} = \frac{5}{384} \cdot \frac{4{,}07 \cdot 4200^4}{210000 \cdot \left(541 \cdot 10^4\right)} = 14{,}5mm \leq \frac{L}{240} = 17{,}5mm$$

The beam meets the serviceability requirements. Check for flexural and shear strength. Since the shape is not a standard shape in the AISC manual, determine if the section is compact. Sections are compact if the following conditions 2 and 9 of T B4.1 of the AISC specification are met:

$$a)\ \frac{b_f}{2 \cdot t_f} < 0{,}38 \cdot \sqrt{\frac{E}{F_y}} \Rightarrow \frac{73}{2 \cdot 6{,}9} < 0{,}38 \cdot \sqrt{\frac{210000}{235}} \Rightarrow 5{,}29 < 29{,}9\ (OK)$$

$$b)\ \frac{h}{t_w} < 3{,}76 \cdot \sqrt{\frac{E}{Fy}} \Rightarrow 140 / 4{,}7 < 3{,}76 \cdot \sqrt{\frac{210000}{235}} \Rightarrow 29{,}8 < 112\ (OK)$$

Hence, the section is compact, and laterally braced. Therefore the limit state for bending is:

$$\phi M_n > M_u$$

$$w_u = 1{,}2 w_d + 1{,}6 w_l = 1{,}2 \cdot 2{,}46 + 1{,}6 \cdot 1{,}6 = 5{,}5\ kN/m$$

$$M_u = \frac{w_u \cdot l^2}{8} = \frac{5{,}5 \cdot 4{,}2^2}{8} = 12{,}1\ kN \cdot m$$

$$\phi M_n = \phi \cdot F_y \cdot Z = 0{,}9 \cdot (235 \cdot 10^{-3} \frac{kN}{mm^2}) \cdot (88{,}3\ mm^3) = 18{,}7\ kN \cdot m$$

Hence the check for bending; $\phi M_n > M_u$ strength is met

The check for shear strength is:

$$\phi V_n > V_u$$

$$V_u = \frac{w_u \cdot l}{2} = \frac{5{,}5 \cdot 4{,}2}{2} = 11{,}6\ kN$$

Determine C_v and ϕ:

$$\frac{h}{t_w} < 2{,}24 \sqrt{\frac{E}{F_y}} \Rightarrow \frac{140}{4{,}7} < 2{,}24 \sqrt{\frac{210000}{235}} \Rightarrow 29{,}8 < 67 \text{ (OK)}$$

Hence, $C_v = 1{,}0$ and $\phi_v = 1{,}0$

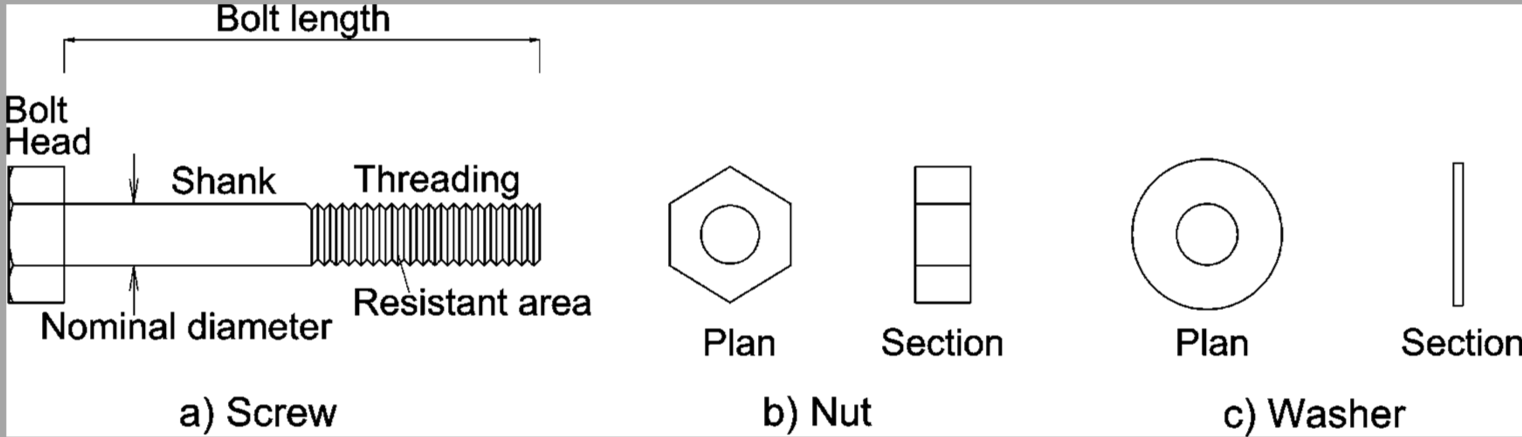
$$\phi V_n = \phi_v \cdot 0{,}6 \cdot F_y \cdot A_w \cdot C_v = \phi_v \cdot F_y \cdot (h \cdot t_w) \cdot C_v$$

$$= 1{,}0 \cdot 0{,}6 \cdot 235 \cdot 10^{-3} \cdot (140 \cdot 4{,}7) \cdot 1{,}0 = 92{,}8\ kN$$

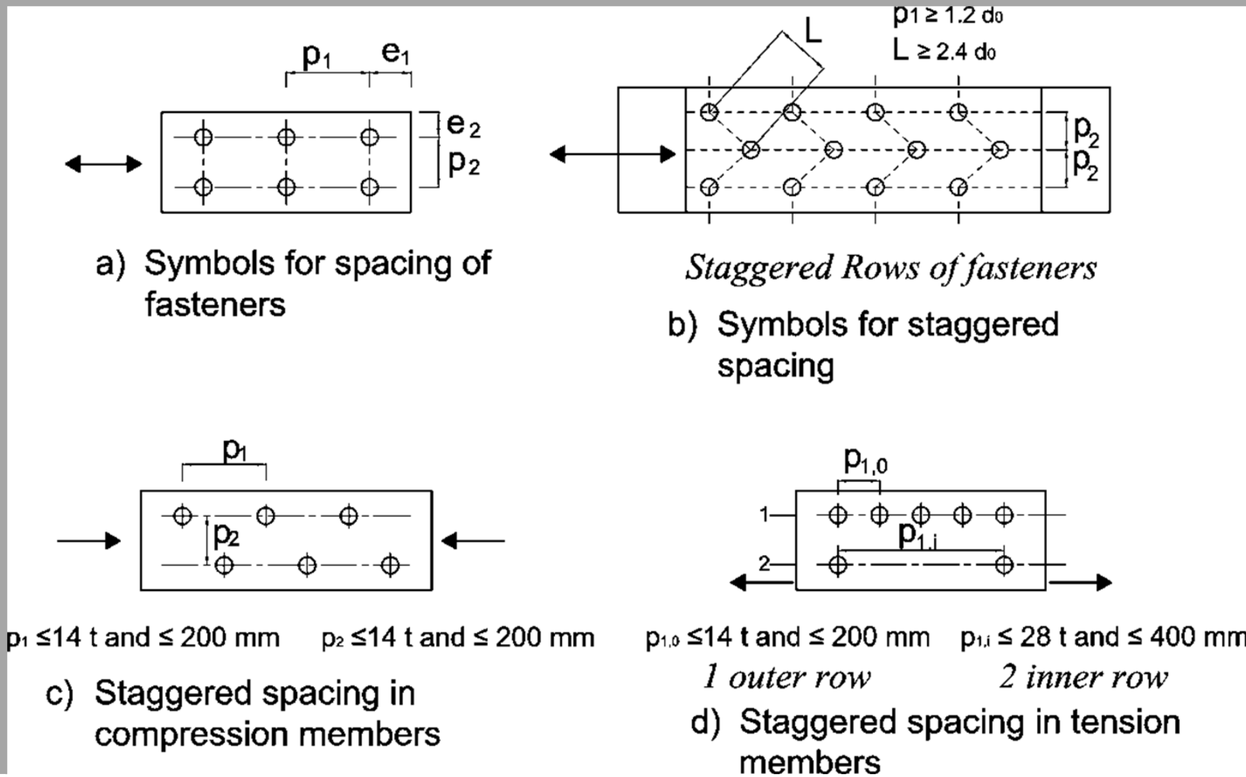
Hence the check for shear strength; $\phi V_n > V_u$ is met

3. CONNECTIONS

Bolted connections



Disposition of holes
in a bolted connection



Minimum and maximum spacing, end and edge distances

Distances and spacing (fig. 3.4)	Minimum	Maximum		
		Steel exposed to the weather or other corrosive influences	Steel not exposed to the weather or other corrosive influences	Steel used unprotected (EN 10025-5)
e_1	$1{,}2\ d_0$	4t + 40mm	-	max(8t; 125mm)
e_2	$1{,}2\ d_0$	4t + 40mm	-	max(8t; 125mm)
p_1	$2{,}2\ d_0$	min(14t; 200mm)	min(14t; 200mm)	min(14t; 175mm)
$p_{1,0}$	-	min(14t; 200mm)	-	-
$p_{1,i}$	-	min(28t; 400mm)	-	-
p_2	$2{,}4\ d_0$	min(14t; 200mm)	min(14t; 200mm)	min(14t;175mm)

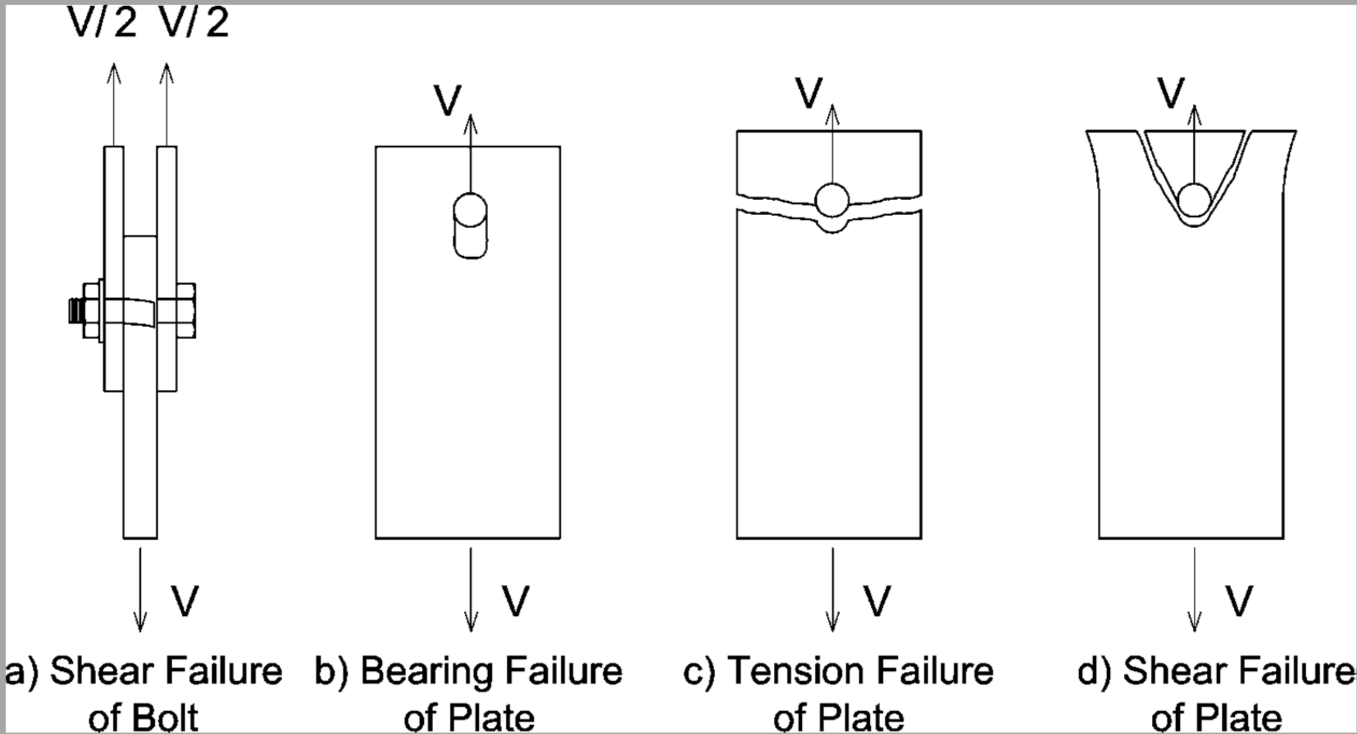
Suggested values for spacing and distances of bolted connections

$$10 \geq p_1/d \geq 3$$

$$3 \geq e_1/d \geq 1{,}5$$

$$3 \geq e_2/d \geq 1{,}5$$

Collapse of connection



Bolted Tension Connection Design

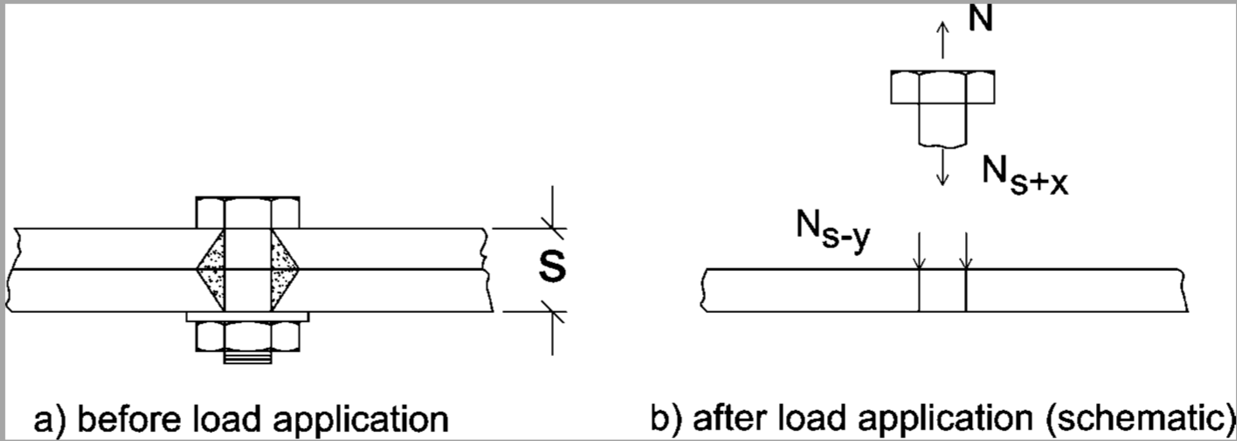


Figure 3.18 Basic tension connection

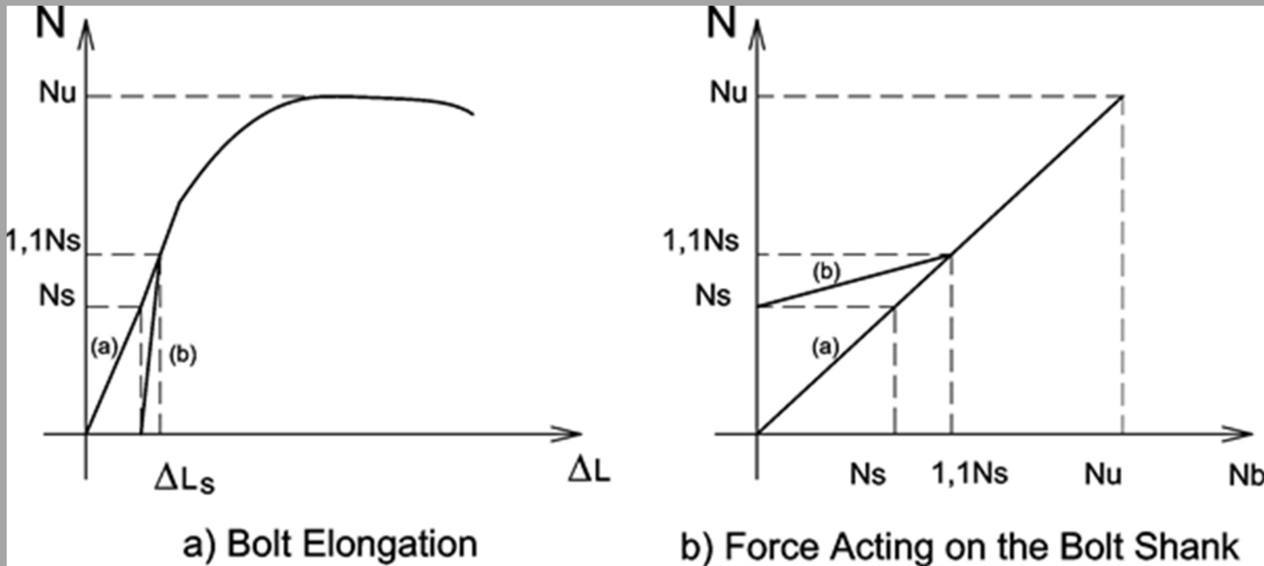


Figure 3.19 External Loads on the Bolt versus

Tension Connections Stressed by Eccentric Forces

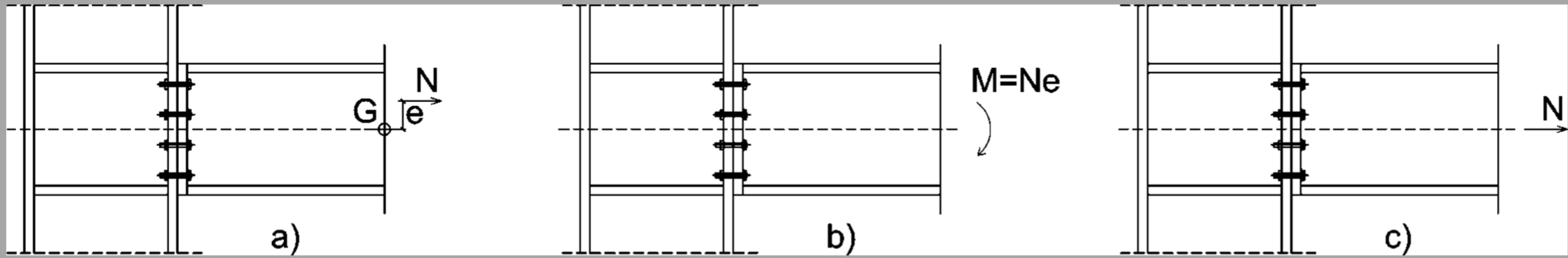
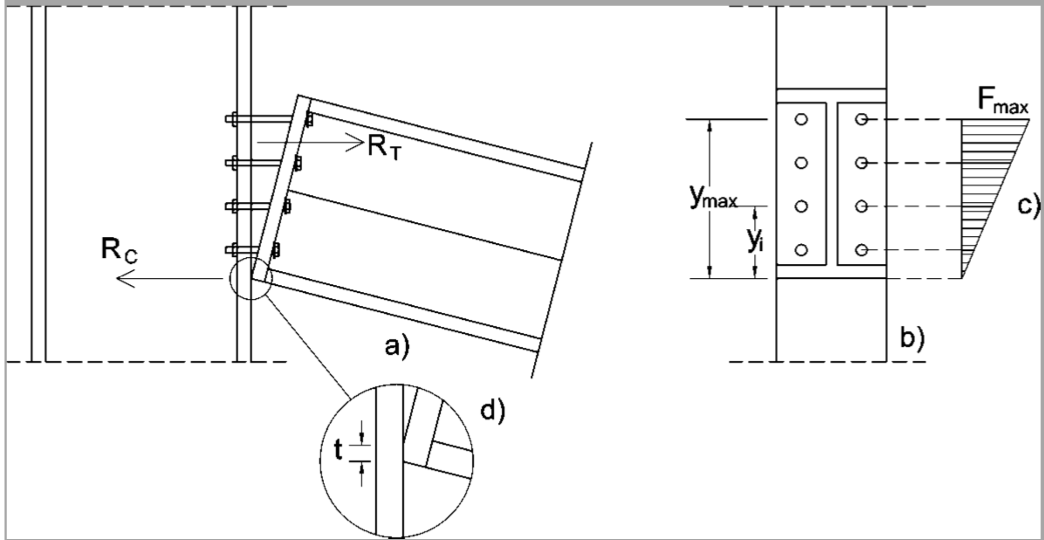


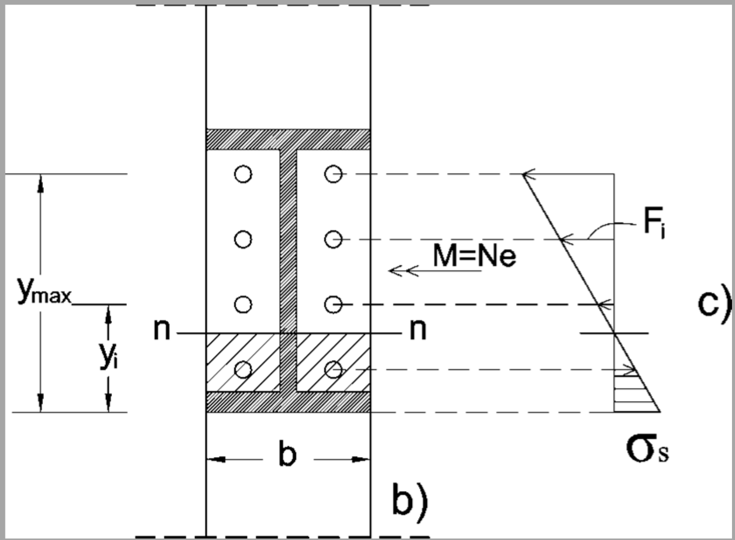
Figure 3.20 Joint Model

TENSION DUE TO ECCENTRICITY
- RIGID FLANGE ASSUMPTION



$$F_{max} = \frac{M}{\Sigma y_i^2} y_{max}$$

TENSION DUE TO ECCENTRICITY
– HOOKE'S LAW APPROACH



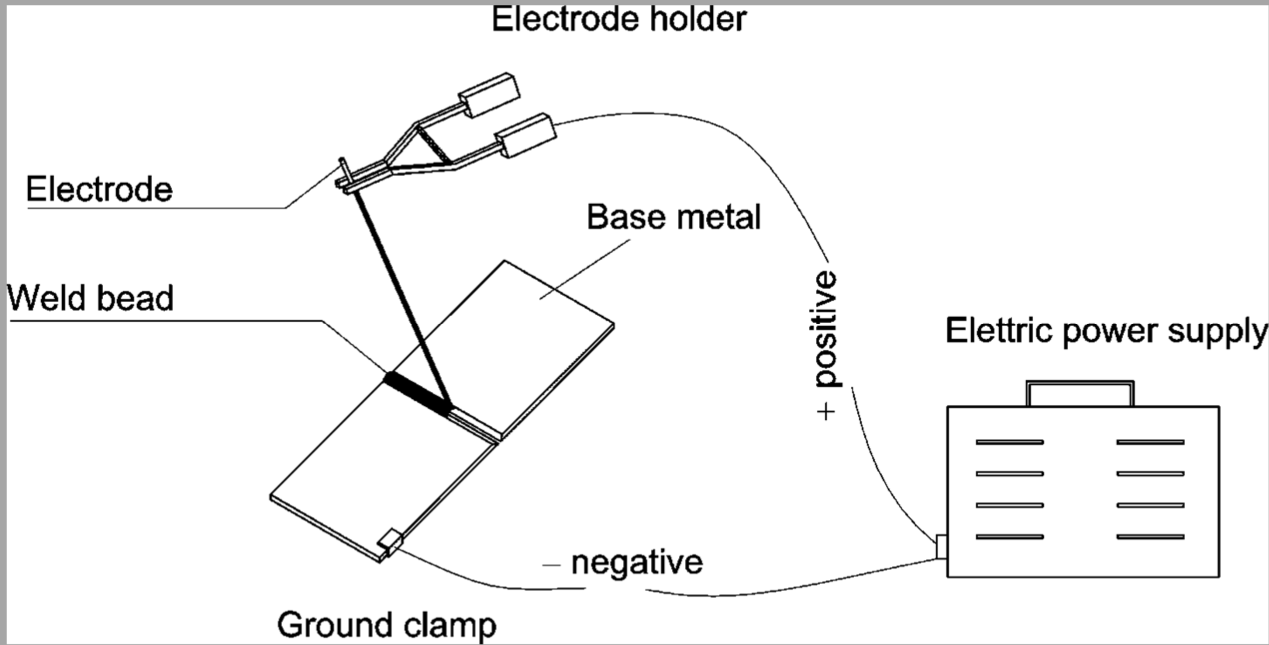
$$F_{max} = \frac{M \cdot A_{bi} \left(y_{max} - x \right)}{I}$$

Welded Connections

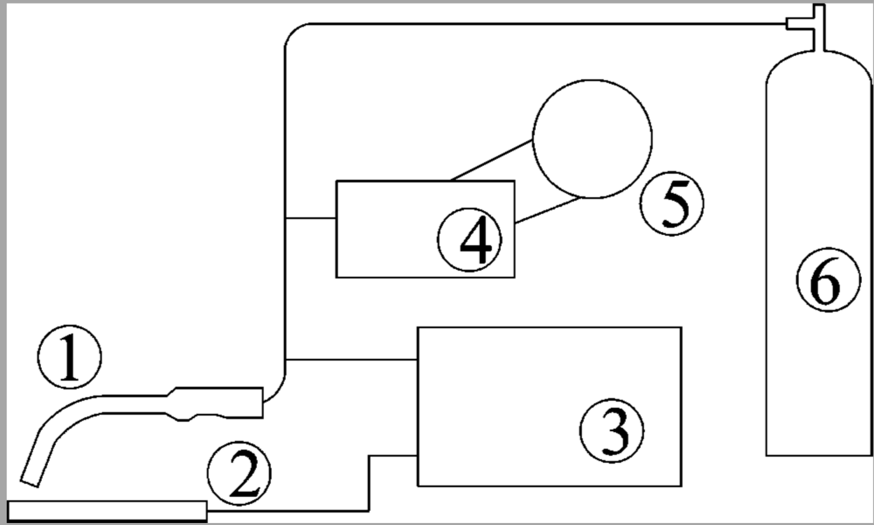
Types of welding processes:

- Oxy-acetylene welding;
- Shielded metal-arc welding (SMAW);
- Submerged arc welding (SAW);
- Gas-shielded metal arc welding (GMAW)
- Tungsten inert gas welding (TIG).

SHIELDED METAL ARC WELDING (SMAW)

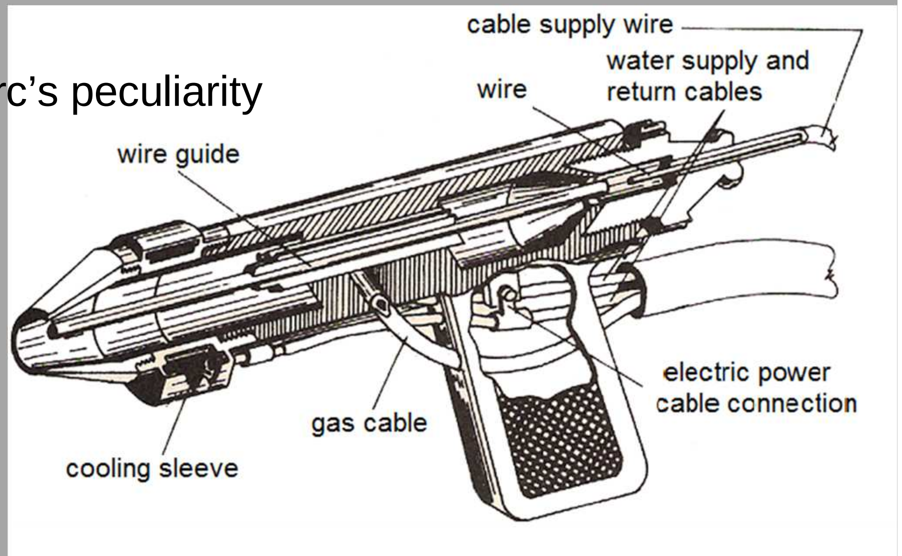


GAS METAL ARC WELDING (GMAW)



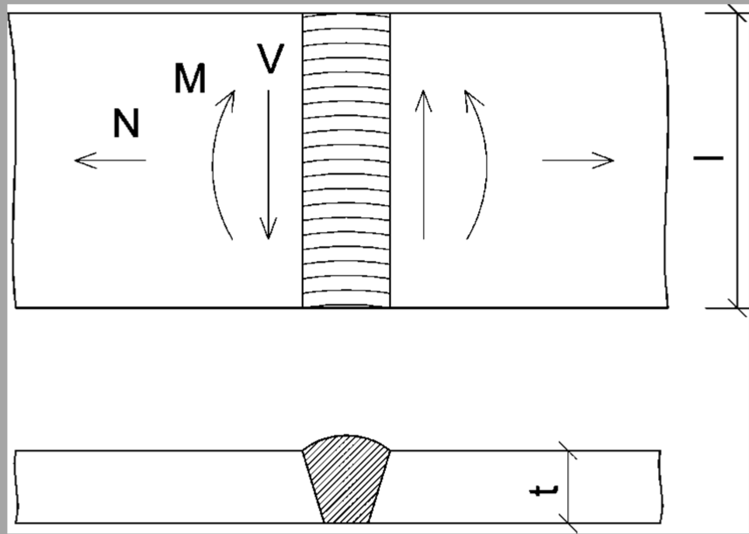
1. Torch with dual function: that of shooting the arc between the wire and the workpiece and that one of bringing the shielding gas on weld pool.
2. base metal
3. power supply (in modern machines the control of arc's peculiarity is electronically carried out)
4. wire feed mechanism and control
5. wire winding reel
6. cylinder of shielding gas

Welding torch gun

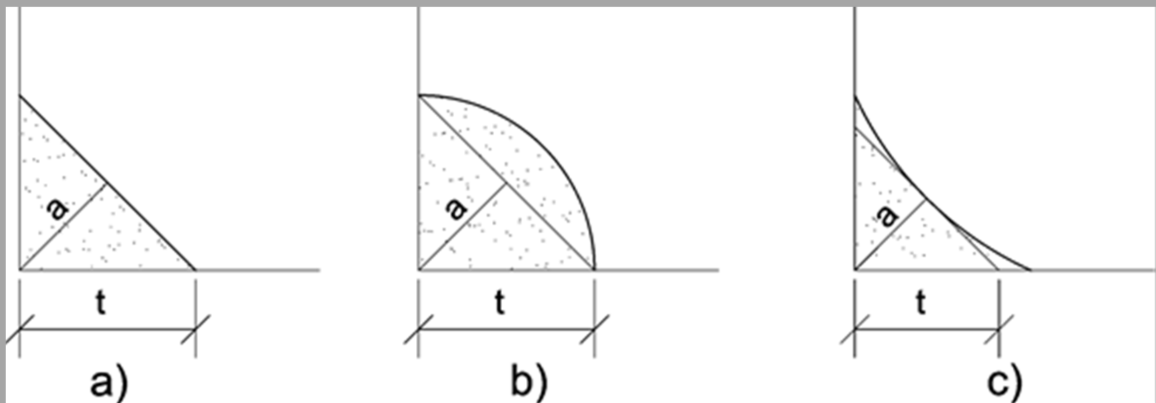


Resistance weld checks

Butt welds

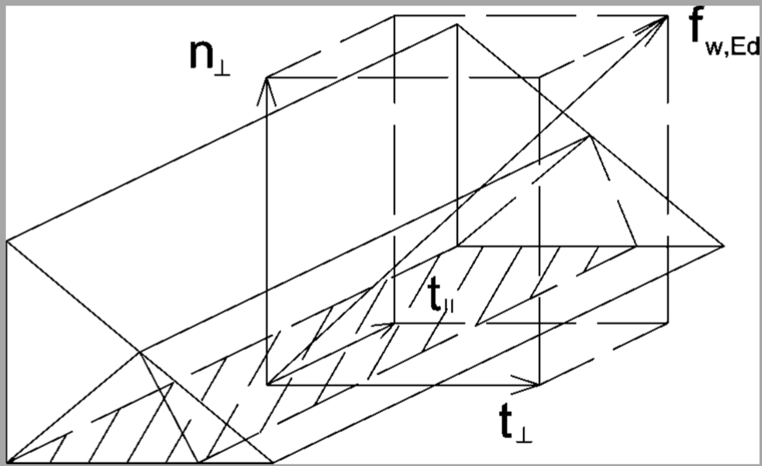


Fillet weld



$$a \cong \frac{t}{\sqrt{2}} = 0,70 \cdot t$$

Resultant of stresses acting on the fillet weld



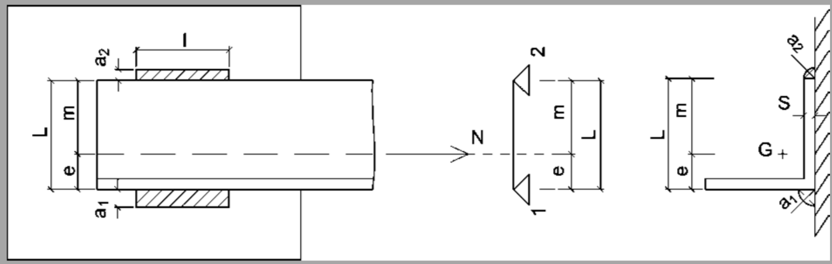
$$f_{w,Ed} = \sqrt{n_\perp^2 + t_\perp^2 + t_{//}^2} = \sqrt{\sigma_\perp^2 + \tau_\perp^2 + \tau_{//}^2}$$

$$f_{w,Ed} \le f_{w,Rd}$$

SIMPLIFIED METHOD

$$f_{w,Ed} \le \begin{cases} 0.88\, f_d & \text{to steel grade S235} \\ 0.85\, f_d & \text{to steel grade S275} \\ 0.74\, f_d & \text{to steel grade S355} \end{cases} = \eta_w \cdot f_d$$

Fillet weld to the member resistance



Angle	Simplified Method				
L x *s* (*mm*)	a_1 (*mm*)	a_2 (*mm*)	l (*mm*)	t_1 (*mm*)	t_2 (*mm*)
40 x 4	7,2	2,8	35	10,3	4
40 x 5	8,5	3,5	36	12,1	5
50 x 5	9	3,5	44	12,9	5
50 x 6	10,2	4,2	45	14,6	6
60 x 6	10,7	4,2	53	8,6	6
60 x 8	13,3	5,6	54	19	8
70 x 7	12,5	4,9	62	17,9	7
70 x 9	15,2	6,3	63	21,7	9
80 x 8	14,2	5,6	70	20,3	8
80 x 10	16,9	7	72	24,1	10
90 x 9	16	6,3	79	22,9	9
90 x 11	18,7	7,7	81	26,7	11
100 x 10	17,8	7	88	25,4	10
100 x 12	20,5	8,4	90	29,3	12

Exercise 39

For the two nodes shown in Figure 3.62, design the welds based on the resistance of the webs.

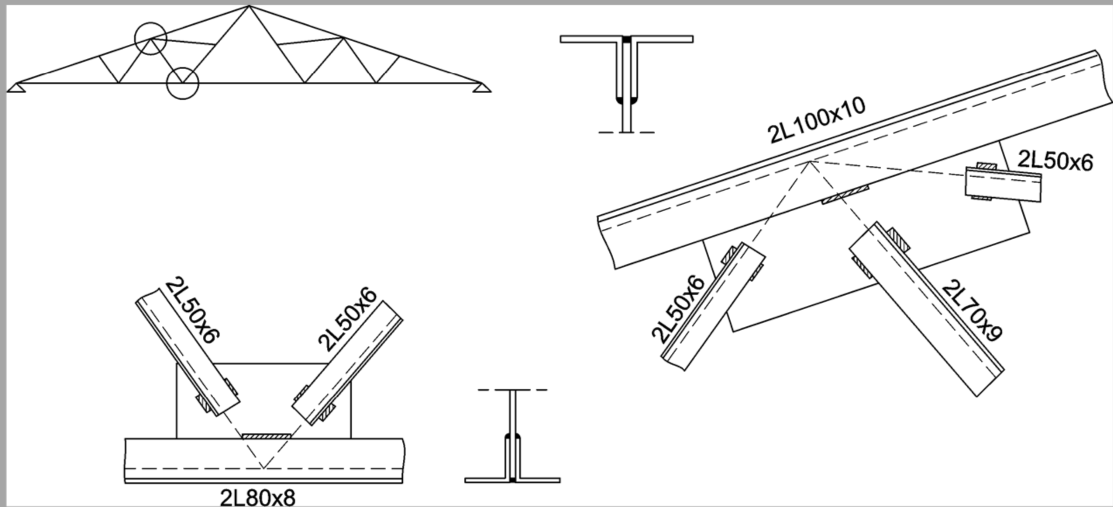


Figure 3.62 Welds truss nodes details

SOLUTION

The following is obtained from Table 3.10:

L 50 x 6

l = 45 mm; a_1 = 10,2 mm; a_2 = 4,2 mm

L 70x9

l = 63 mm; a_1 = 15,2 mm; a_2 = 6,3 mm

L 80x8

l = 70 mm; a_1 = 14,2 mm; a_2 = 5,6 mm

L 100x10

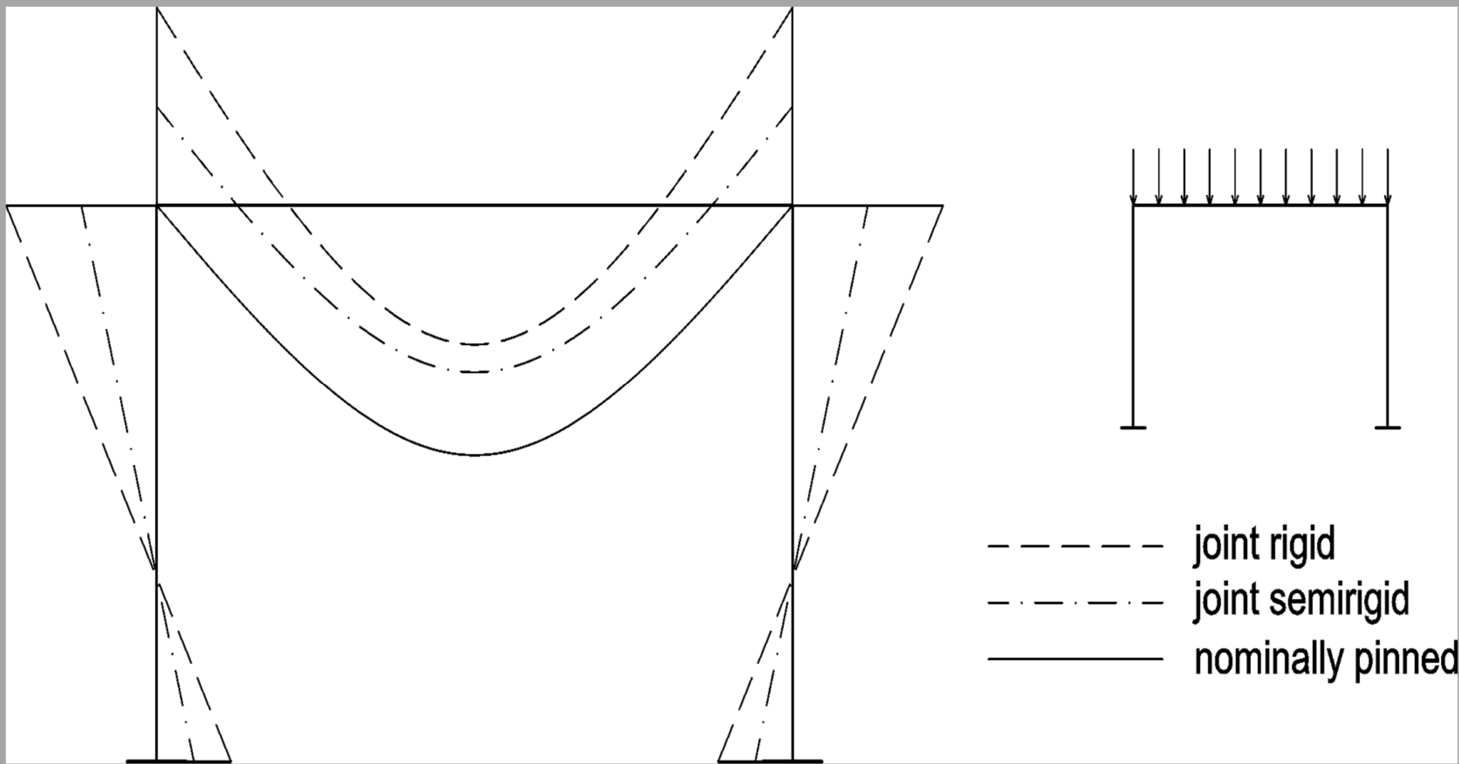
l = 88 mm; a_1 = 17,8 mm; a_2 = 7 mm

The structural design of steel structures typically falls into one of the simplified models:

- **Simple frame**, in which the joint may be assumed not to transmit bending moments (pinned joints)

- **Continuous frame**, in which the behavior of the joint may be assumed to have no effect on the analysis (rigid joints)

Effect of Joint Continuity on Bending



4. TRUSSES AND BRACINGS

Types of trusses

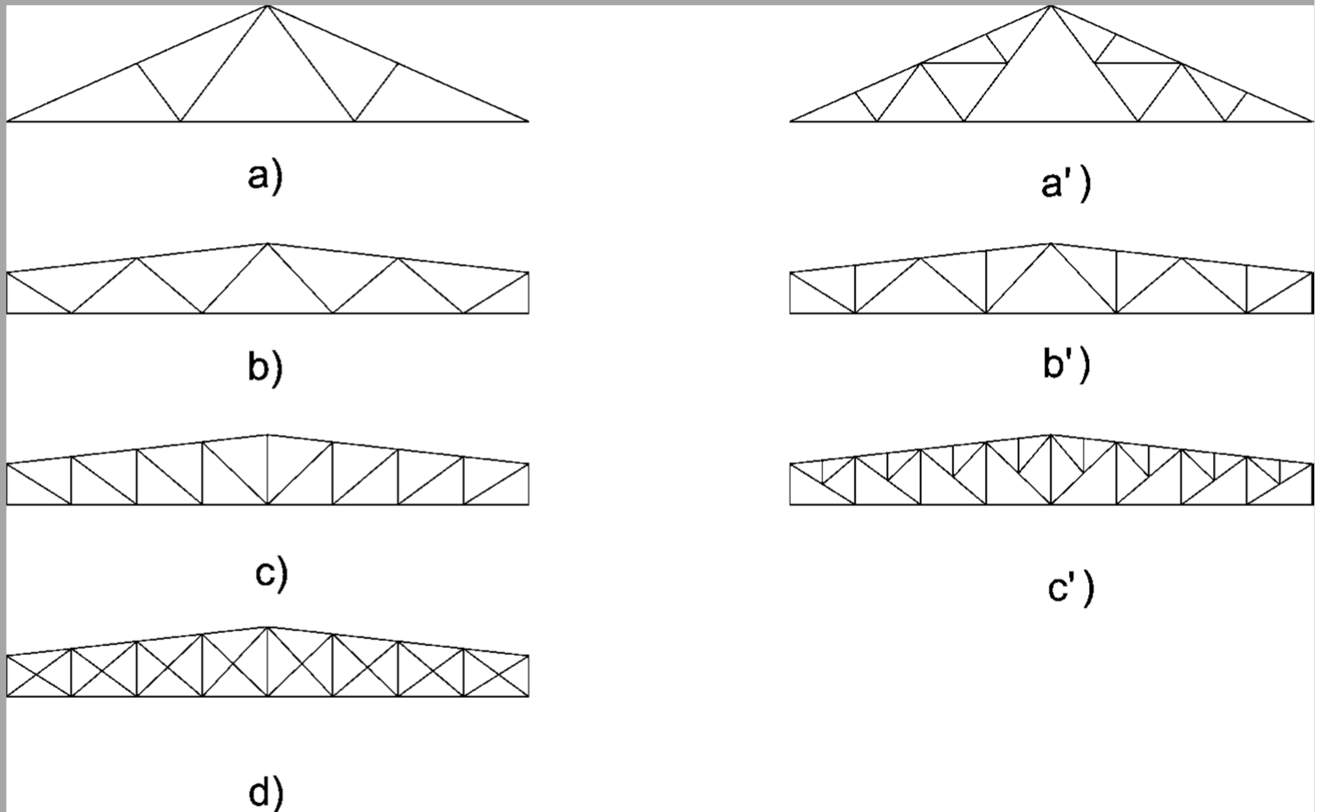
The main types of trusses are shown in Figure:

a) Fink or Polonceau

b) Warren

c) Pratt or Mohniè

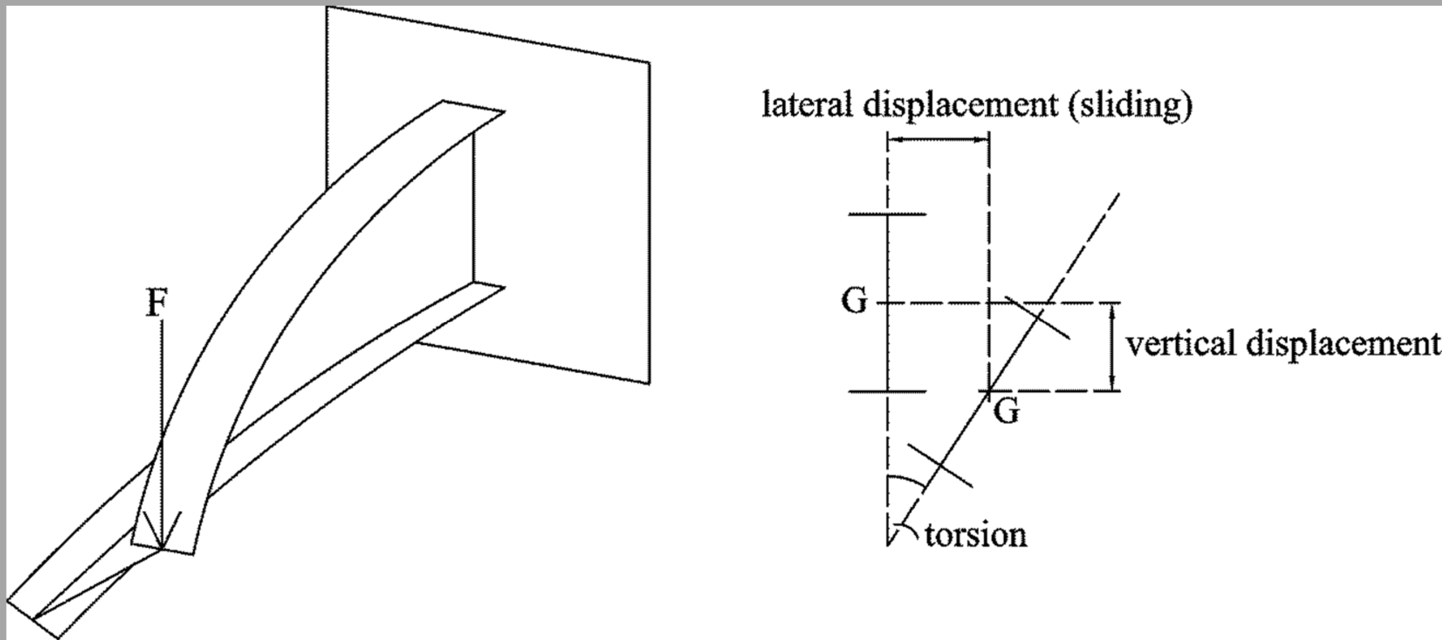
d) Double diagonal



5. BEAMS

Lateral torsional buckling of beams

Lateral buckling of a cantilever



- Prestressed Steel Structures

The technology for the great works and the Bridges

- Composite steel-concrete lattice beams: HEC system

The quick and economic solution for large spans and structures

- Prestressed steel structures design : a new frontier for structural engineering.

Prestressed Steel Structures ? !

What is This ?

Analysis of Prestressed Steel Structures

<u>A prestressed system consist in</u> :

“ Subjecting a structure to loads that produce opposing stresses to those when it is in service”

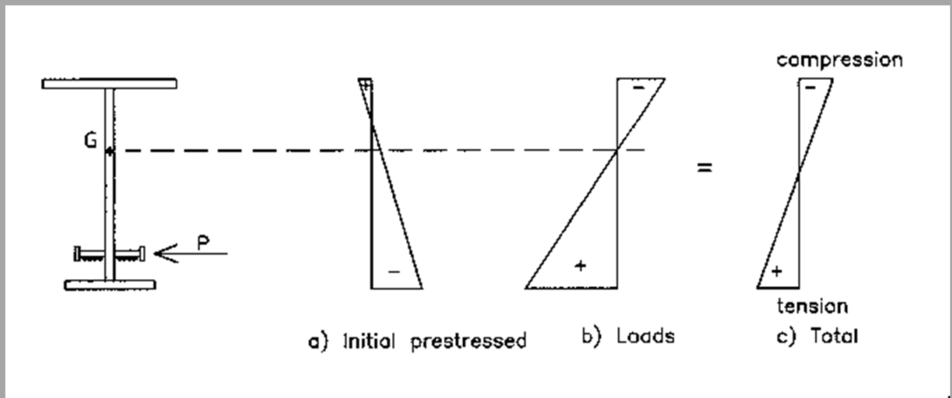


Figure 1 – Prestressed steel

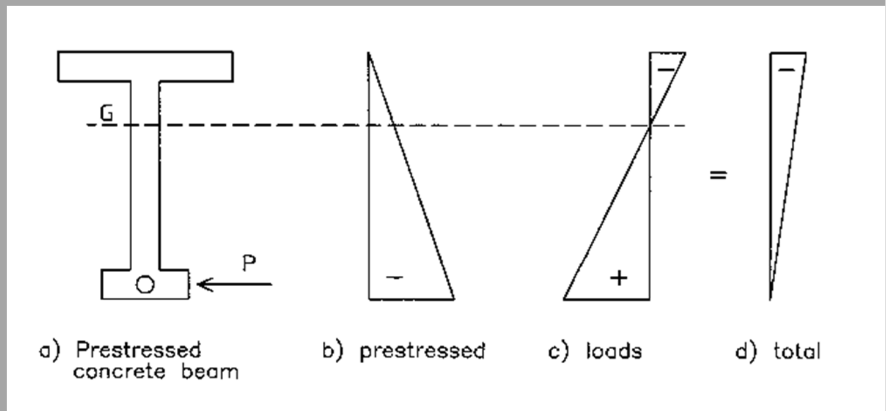


Figure 2 – Prestressed concrete

***<u>The methods used for analyzing the section are</u>* <u>:</u>**

1. ***The static method***, which in each section considers the effect of prestressing as an eccentric pressure (traditional method).

2. ***The equivalent loads method***, in which the effect of prestressing is analyzed through the introduction of a system of equivalent forces of external provenance which exert pressure upon the girder and are called “equivalent loads”.

Static Analysis of the section

- **Bending**

In reference to figure 3, it may be verified, <u>at trasfer</u> :

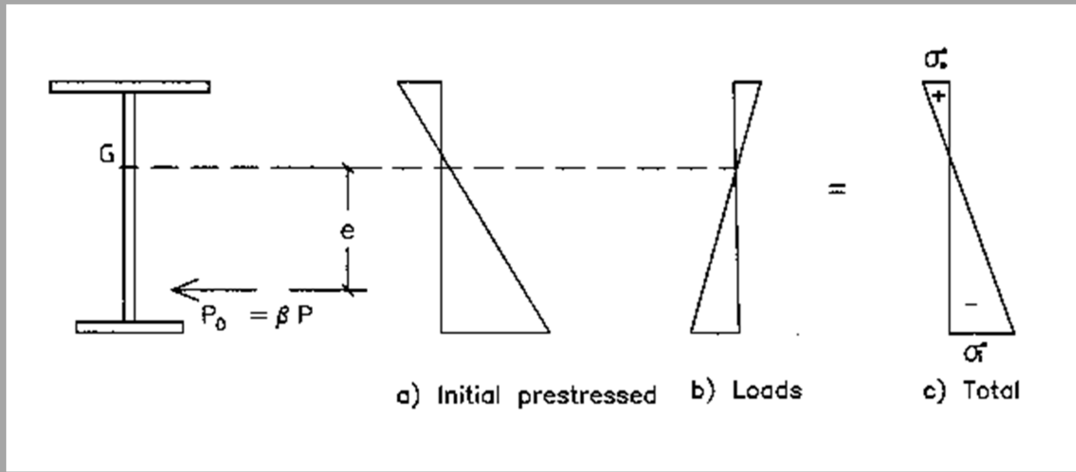


Figure 3 – Condition at Trasfer

$$\sigma_i^0 = -\frac{\gamma_p \beta P}{A} - \frac{\gamma_p \beta P e}{W_i} + \frac{M_{\min}}{W_i} \leq f_d$$

Placing $\sigma_i^0 = f_d$ we will obtain **P**

At service, figure 4, will verify :

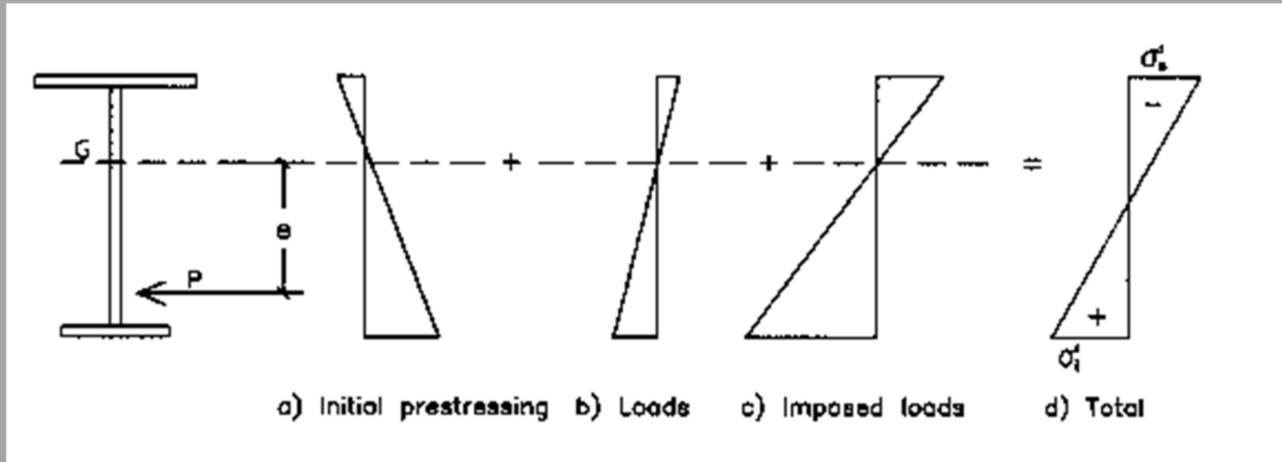


Figure 4 – Condition at service

$$\begin{cases} \sigma_s^1 = -\dfrac{\gamma_p P}{A} + \dfrac{\gamma_p P e}{W_s} - \dfrac{M_{\max}}{W_s} \leq f_d \\ \sigma_i^1 = -\dfrac{\gamma_p P}{A} - \dfrac{\gamma_p P e}{W_i} + \dfrac{M_{\max}}{W_i} \leq f_d \end{cases}$$

The intervention of an external moment causes the prestressing forces (or rather, the center of pressure) to shift:

$$\delta_0 = \frac{M_{\min}}{\beta \cdot P^*} \qquad \delta_1 = \frac{M_{\max}}{P^*} \qquad \text{where} \qquad P^* = \gamma_{\mathrm{p}} \cdot P$$

In reference to figure 5, we have :

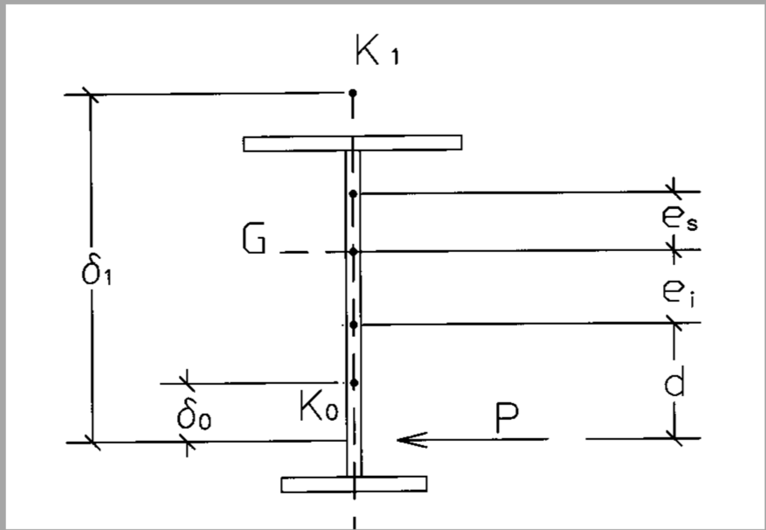


Figure 5 – Representation of kern points e_i and e_s

$$M_u = M_{\max} - M_{\min} = P^*\delta_1 - \beta P^*\delta_0;$$

$$M_u = P^*(\delta_1 - \beta\delta_0) = P^* K_a$$

For prestressed concrete the section is proportioned in such a way as to obtain (Figure 6) :

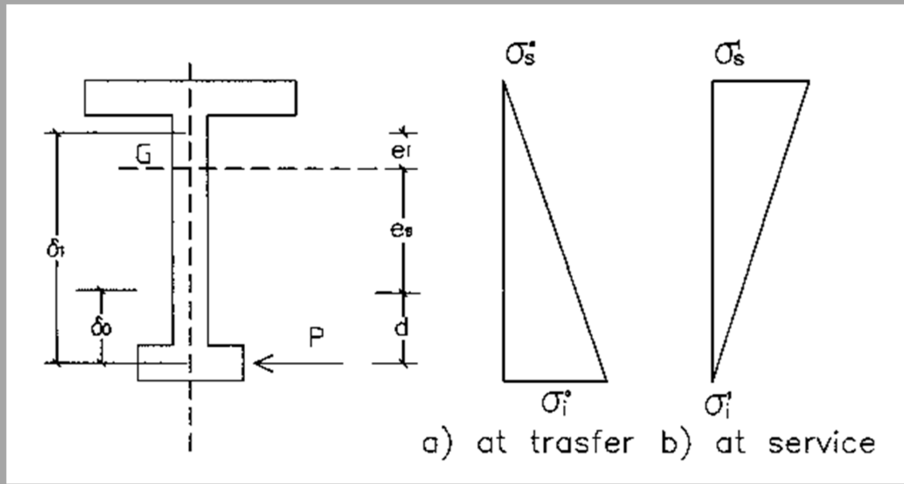


Figure 6 – Prestressed concrete girder

$$\sigma_s^0 = 0 \qquad \sigma_i^1 = 0$$

In this case we obtain :

$$M_u = P^*(\delta_1 - \beta\delta_0) = P^*\left[(d + e_s + e_i) - \beta d\right] = P^* K_c$$

Observation :

1) $K_a \geq K_c$

2) Prestressed concrete beam will have to remain compressed throughout its entire working life.

The system of equivalent loads for prestressing.

The effect of prestressing on a steel girder may be analyzed by the introduction of a system of external loads or equivalent loads.

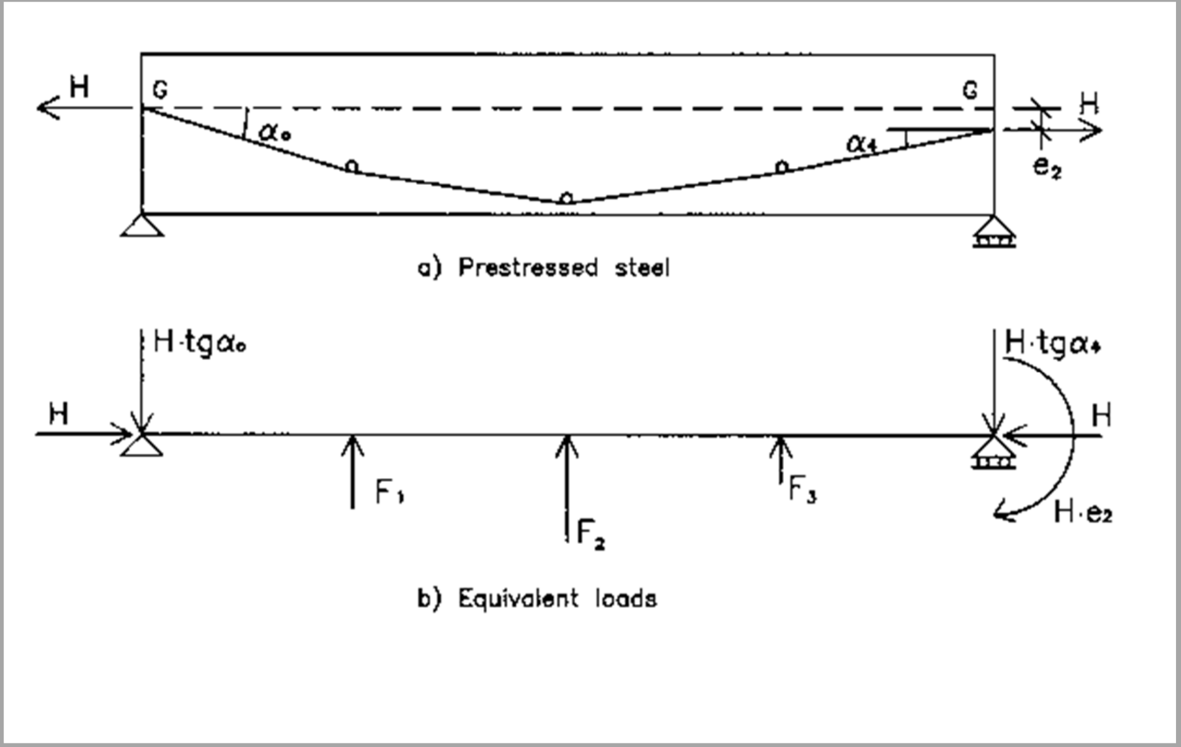


Fig. 10 – Equivalent loads

Prestressed composite steel-concrete girders

The system of prestressing is particularly advantageous for composite steel-concrete sections.

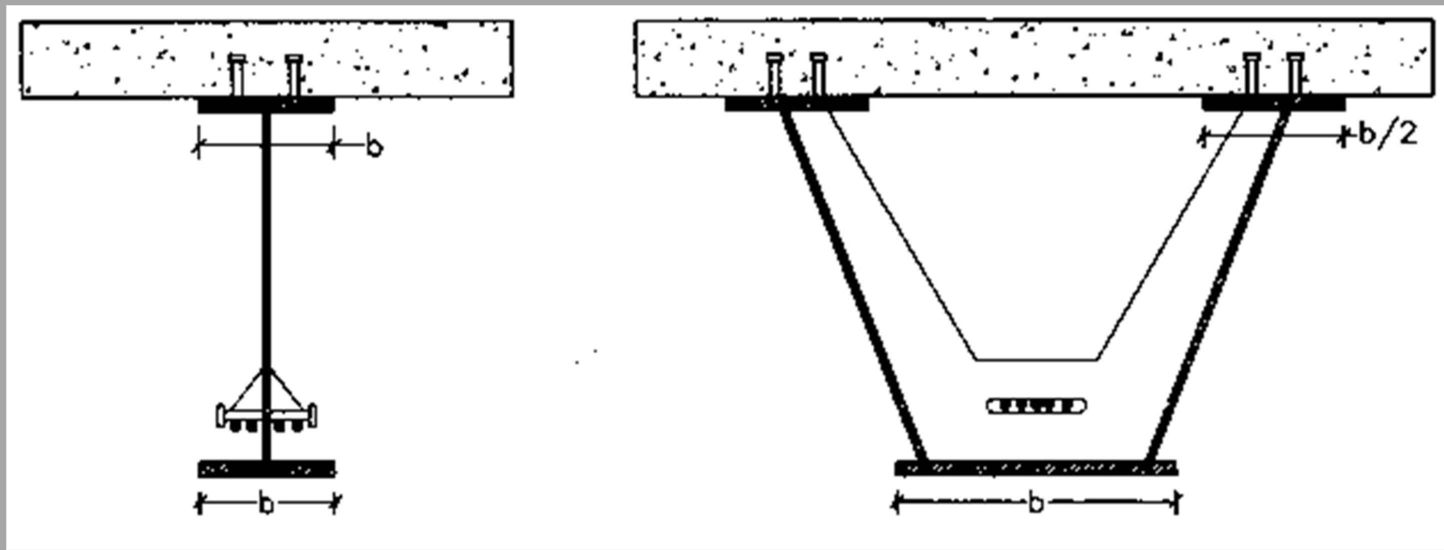


Figure 12 – Composite sections.

The calculation of tensions, in reference to simply supported beams, is carried out by using the *“homogenized section method”.*

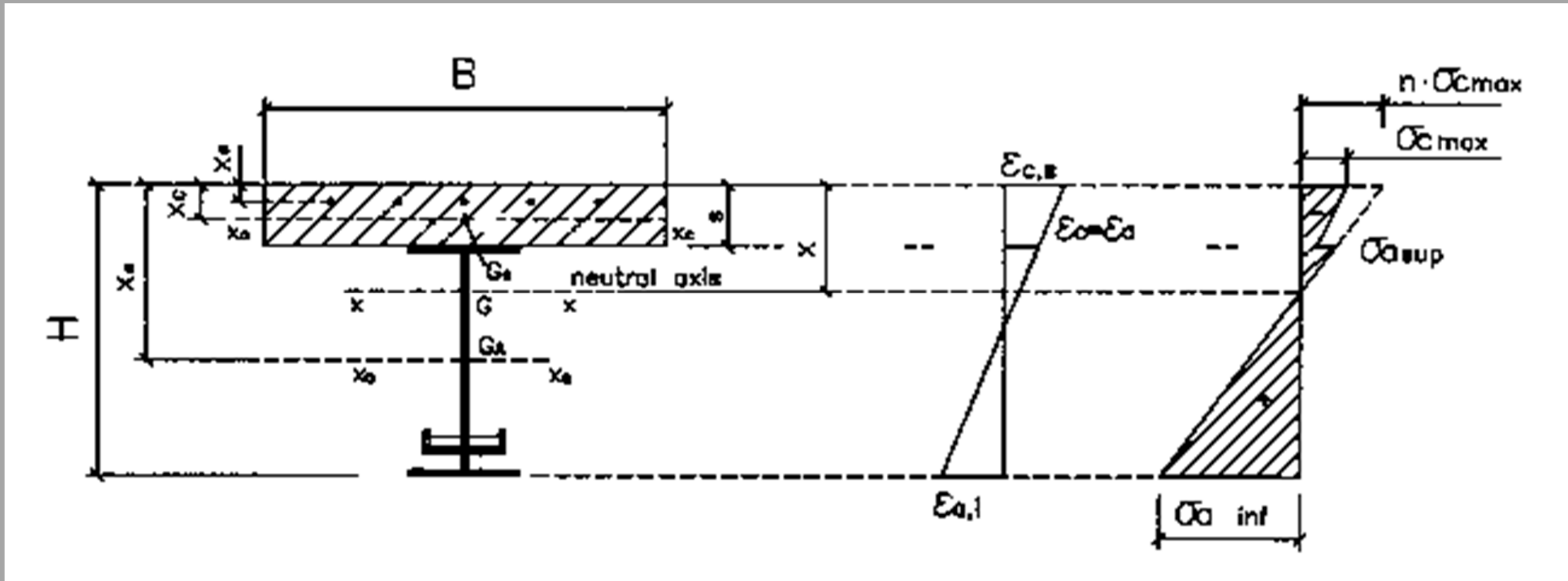


Figure 13 – Prestressed composite girder.

Testing on simply supported prestressed steel beam: L = 21,60 m - q = 25 kN / m

V. Nunziata, 1999

Figure1 - Beam with full load

Figure 2 - Sliding bearing

Figure 3 – Strands anchor

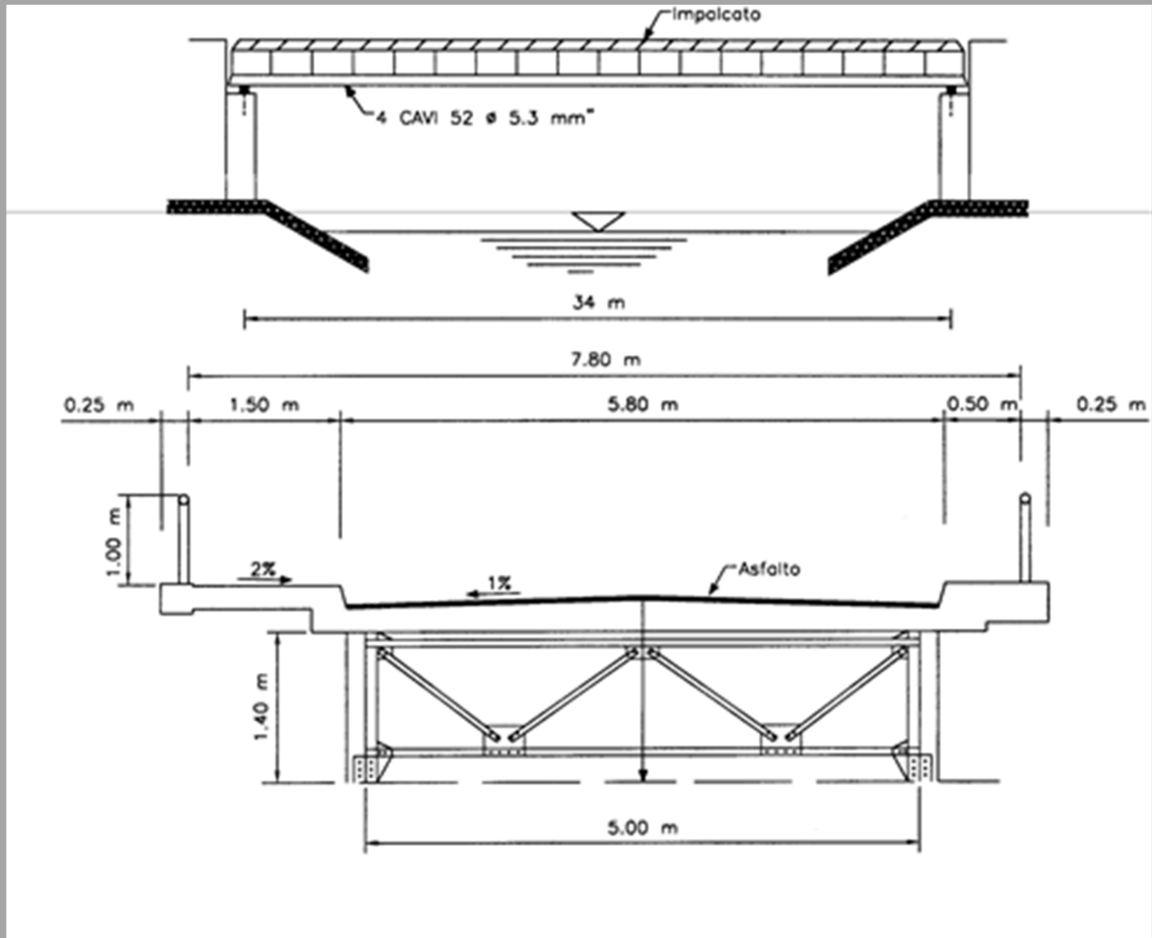
a) At trasfer

b) at full load

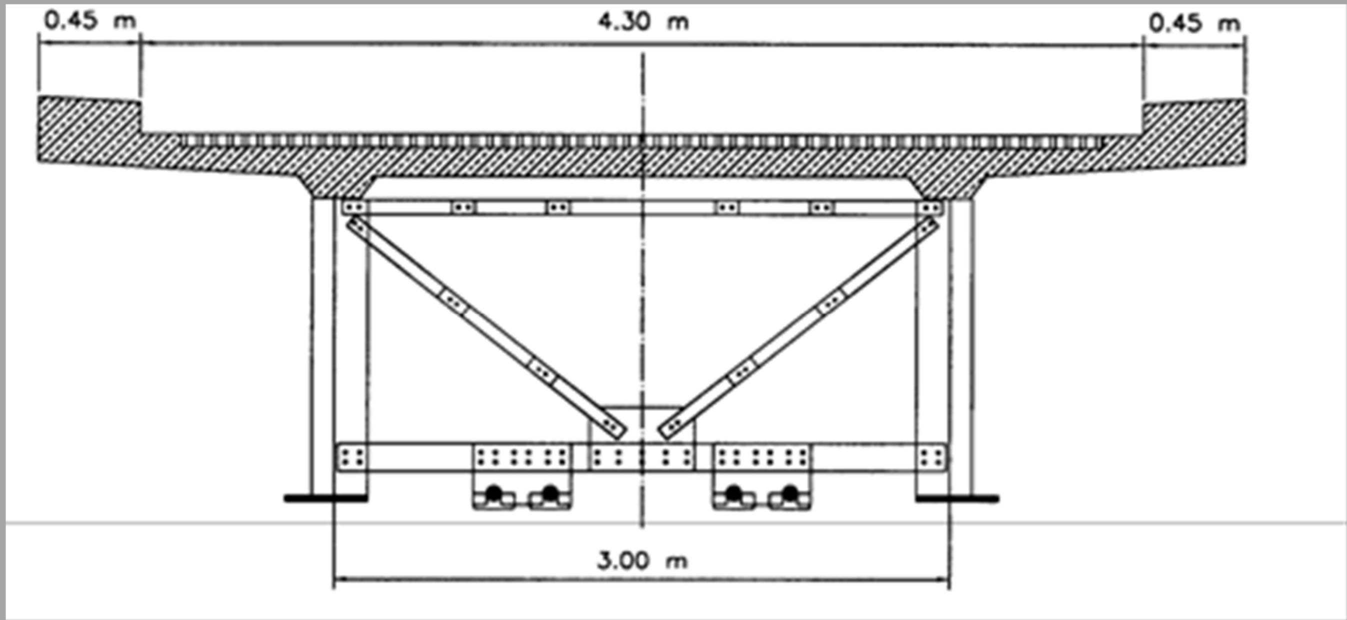
Figure 4 - Variation shift at midspan

Realizations

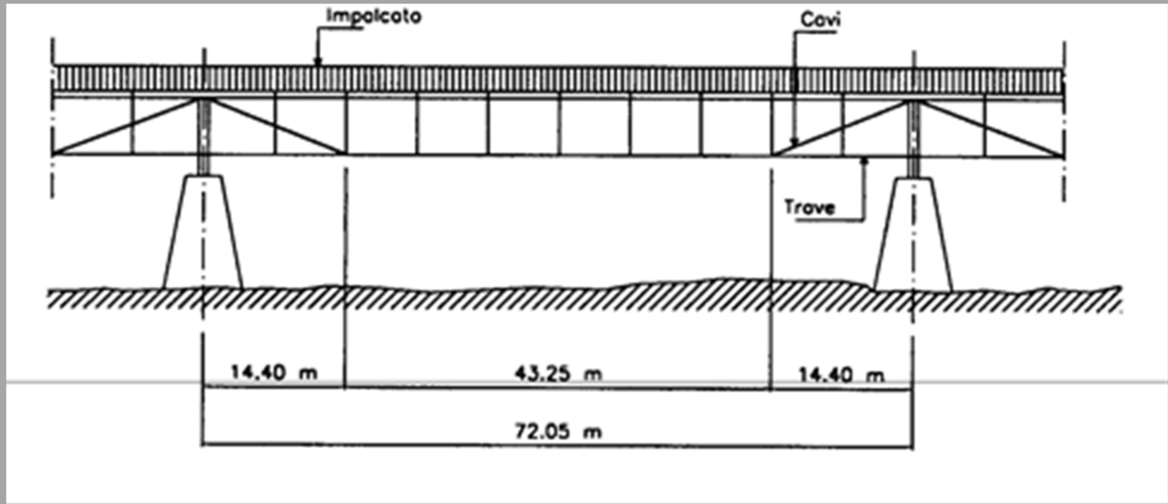
Lauffen bridge, Germany.

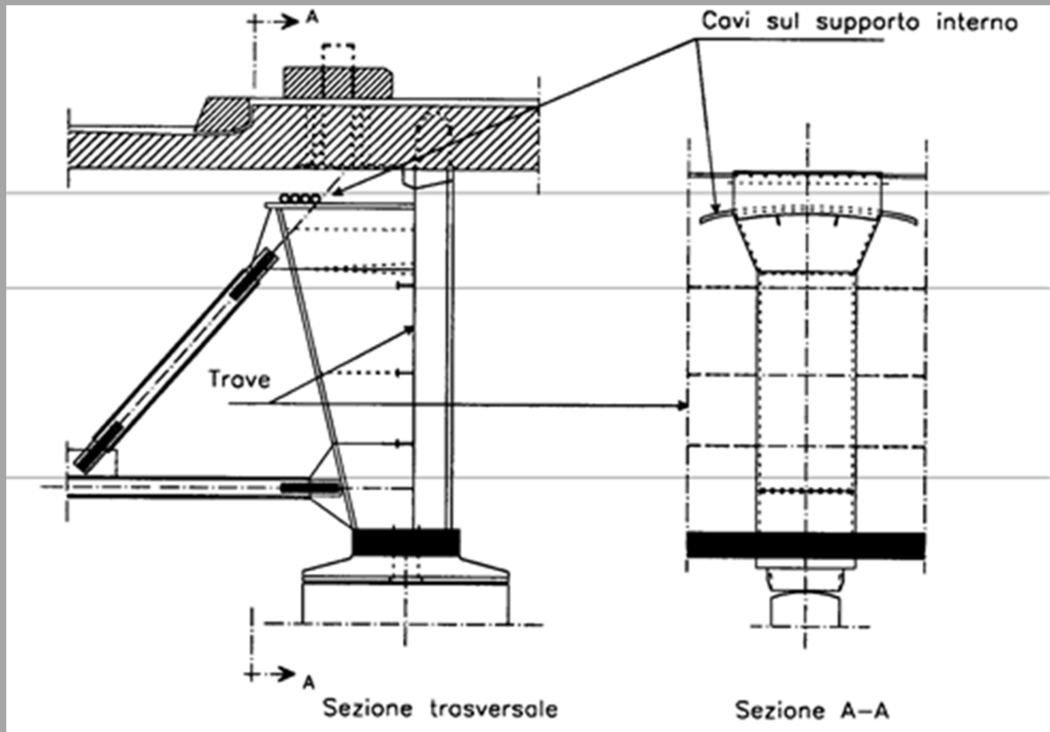


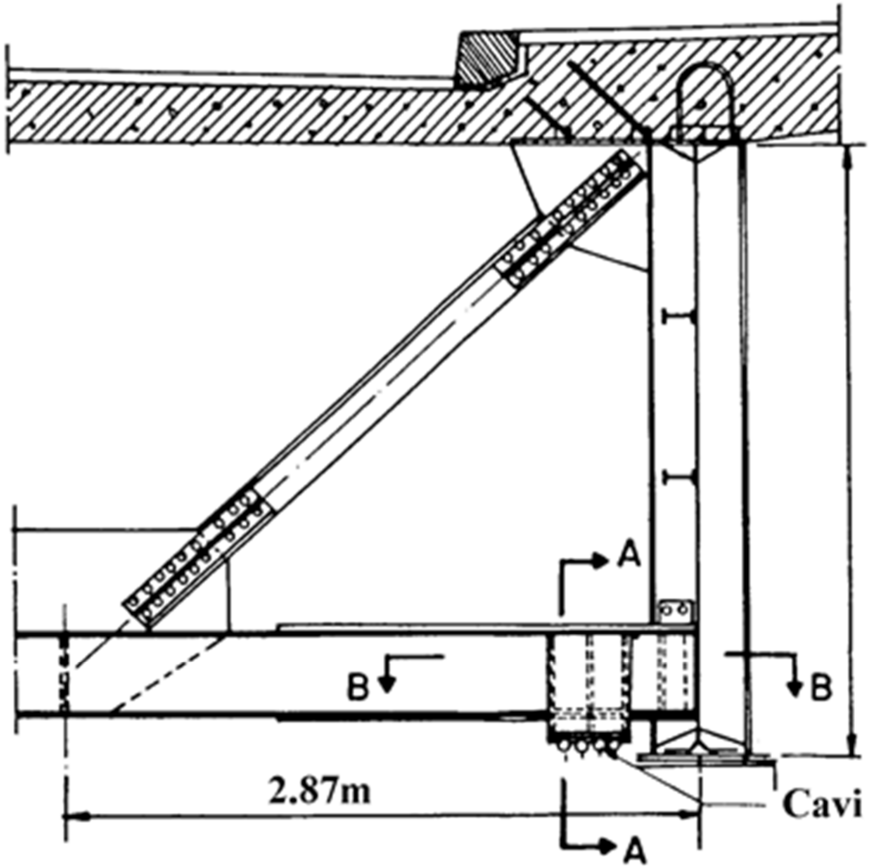
***Ischl bridge, Austria*.**



North Bridge Approaches at Dusseldorf, Germany.

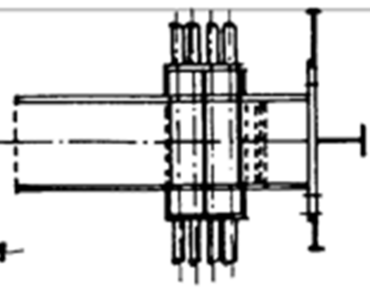






Sezione trasversale

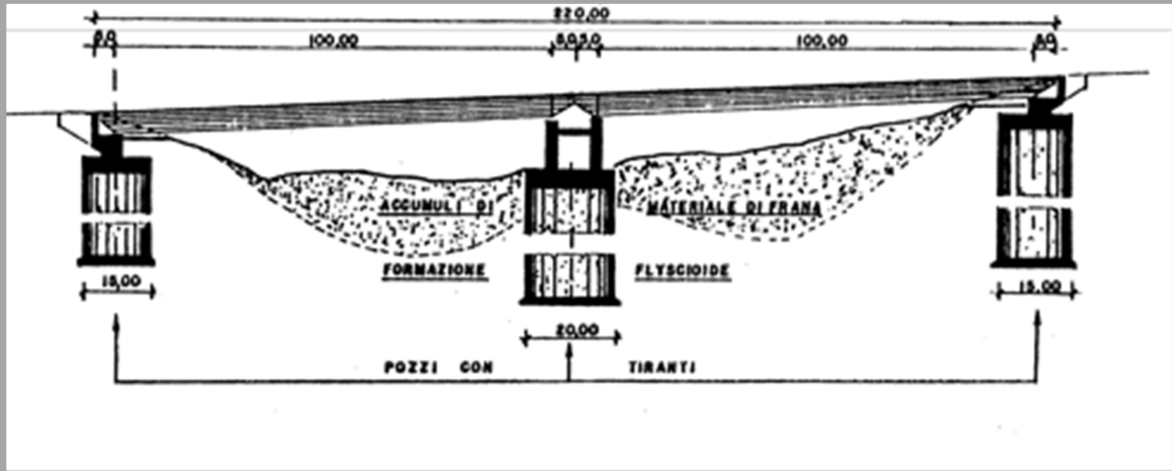
Sezione A-A

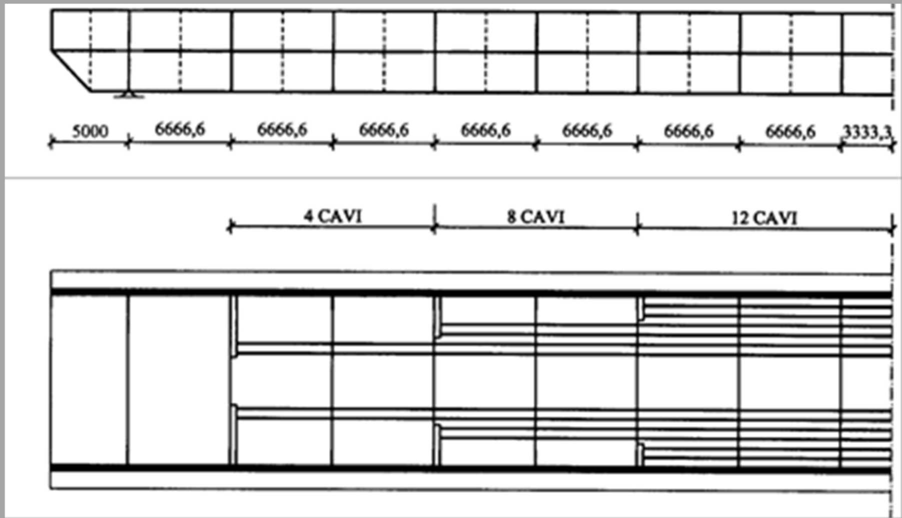


Sezione B-B

Bois de Rosset Viaduct, Switzerland

Viaduct No. 10 of the S.S.18 Road, Italy





5000
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6666,6
3333,3
4 CAVI
8 CAVI
12 CAVI

Observation

The main obstacle in widespread use of prestressed steel structu-
res is their scarce knowledge.
When this gap will be overcome the prestressed steel will beca-
me the real new frontier for structural engineering.

- Composite steel-concrete lattice beams: HEC system

patented by V. Nunziata

ACHIEVEMENTS

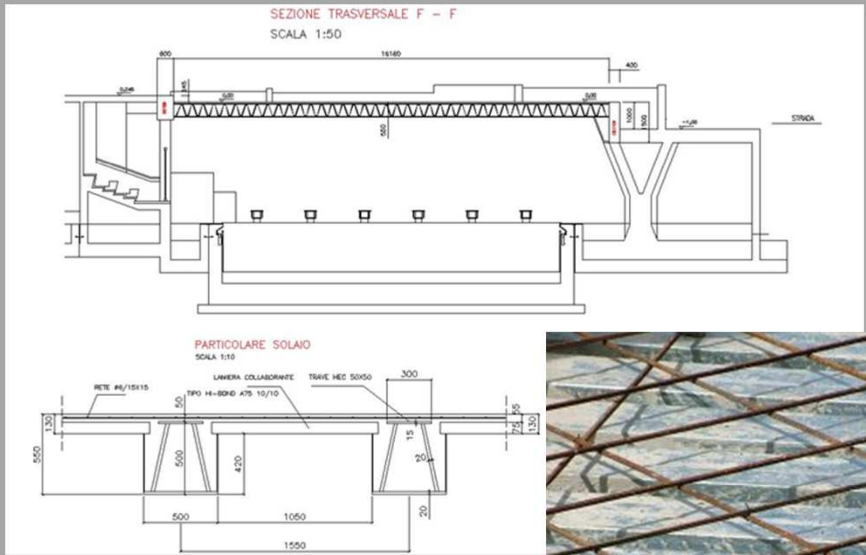
HEC Beam can be used in diferent types of costruction as well as for the seismic Retrofitting and structural restoration.

ADVANTAGES

- ✓ Self-suppoting
- ✓ Easily building
- ✓ Lower height than reinforced concrete beams or prestressed concrete beams
- ✓ Easily to joint
- ✓ Possibility to be used as normal profile
- ✓ Possibility to be curved for roofs or large spans
- ✓ Lower costs

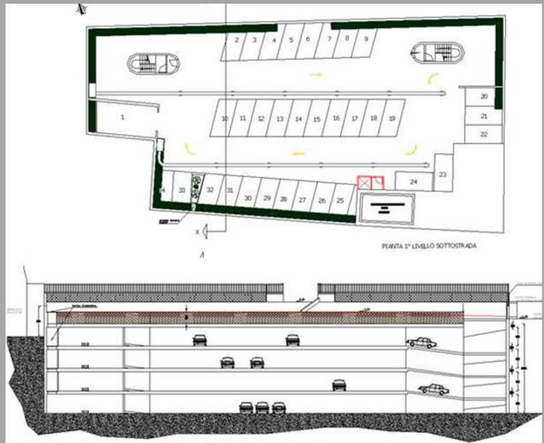
Realizations

Pool roof, Salerno - IT




Slab in an existing building with r.c. beams and columns – Naples , IT

Underground parking, Naples - IT



Office building, Naples - IT

THANK YOU