



Damascus University

Faculty Of Civil Engineering

Lectures, 4th Year 2023

Dr. – Eng. Moaid Subh

PLATE GIRDER

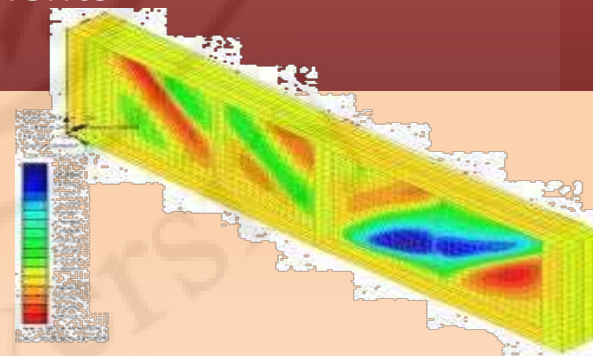
Part 1

To Eurocode 3

EN 1993-1-5 (2006)

Design of steel structures - Part 1-5

General rules Plated structural elements



1. Introduction

Modern plate girders are normally fabricated by welding together two flanges and a web plate. Such girders are capable of carrying greater loads over longer spans than is generally possible using standard rolled sections or compound girders



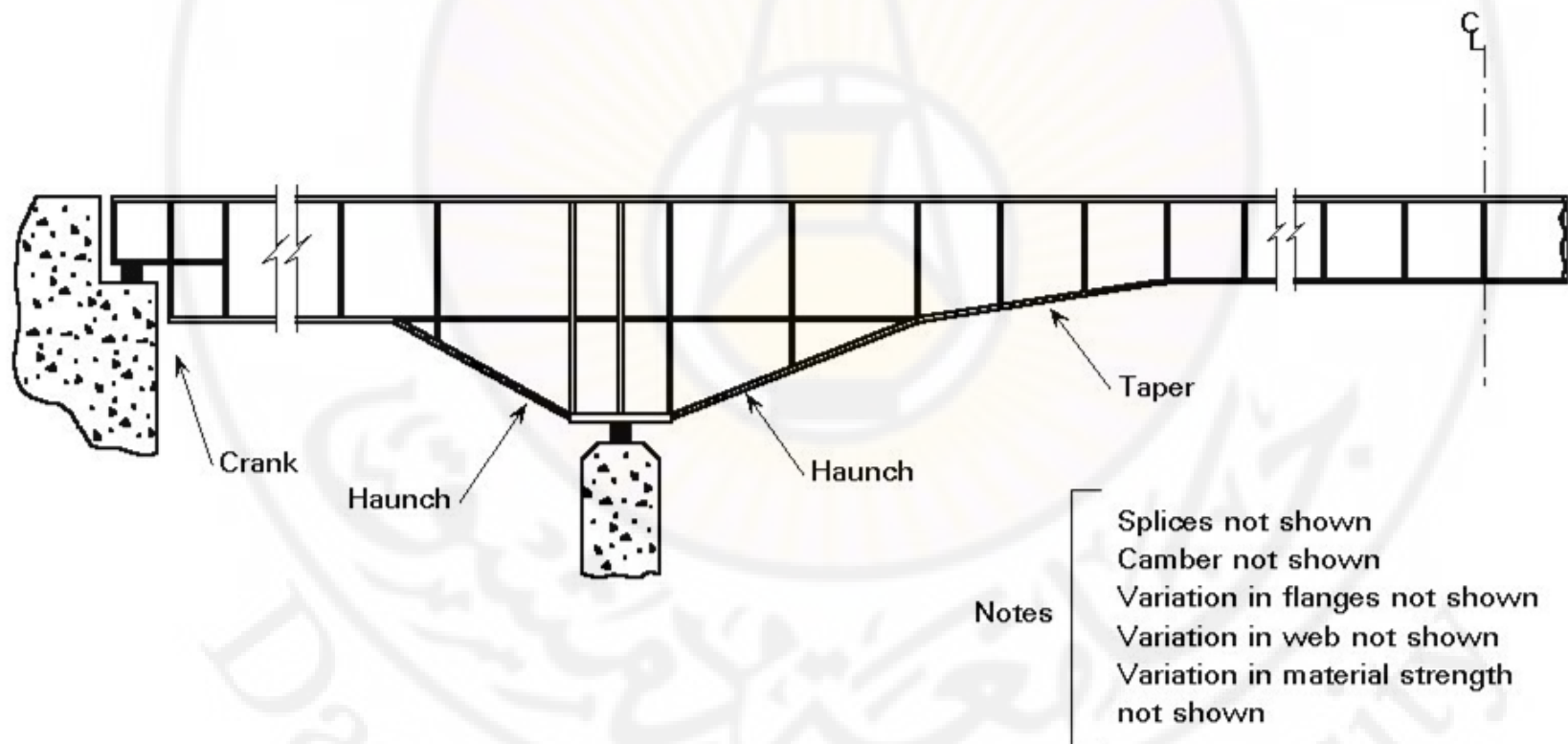
1. Introduction

Plate girders are typically used as long span **floor girders** in buildings, **as bridge girders**, and **as crane girders** in industrial structures.



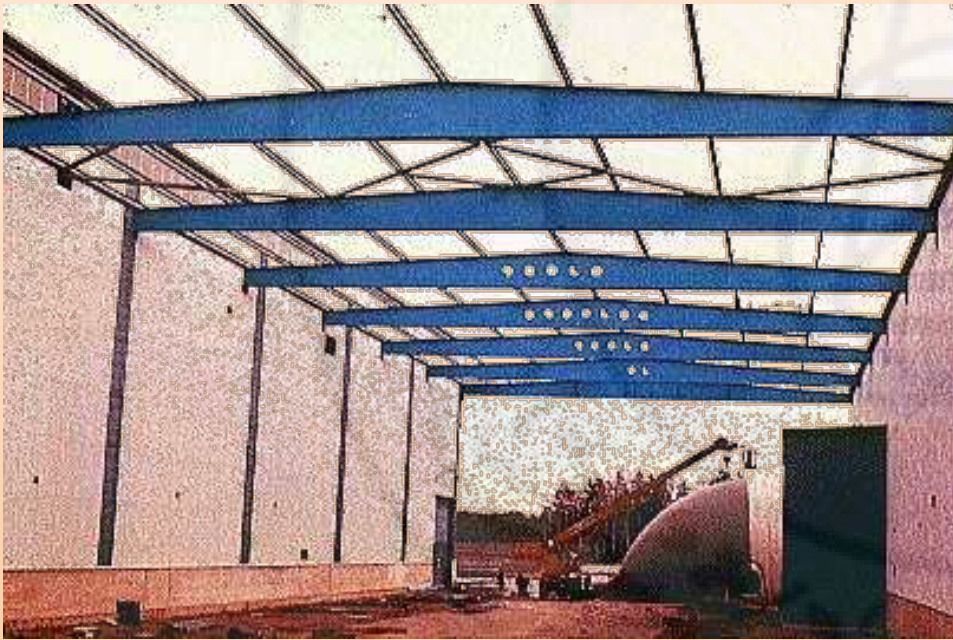
1. Introduction

Plate girders are at their most impressive in modern **bridge** construction where main spans of well over 200m are feasible, with corresponding cross-section depths, **haunched over** the supports, in the range of 5-10m. **Tapering** and **Cranking** are also common sections' variations.



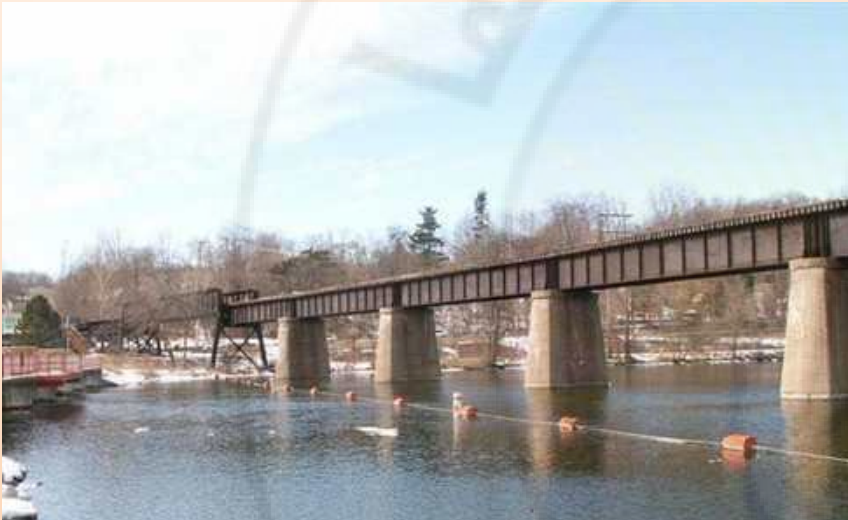
1. Introduction

Plate Girder as Roof Beam



1. Introduction

Plate Girder as Bridges



1. Introduction

Plate Girder as Crane



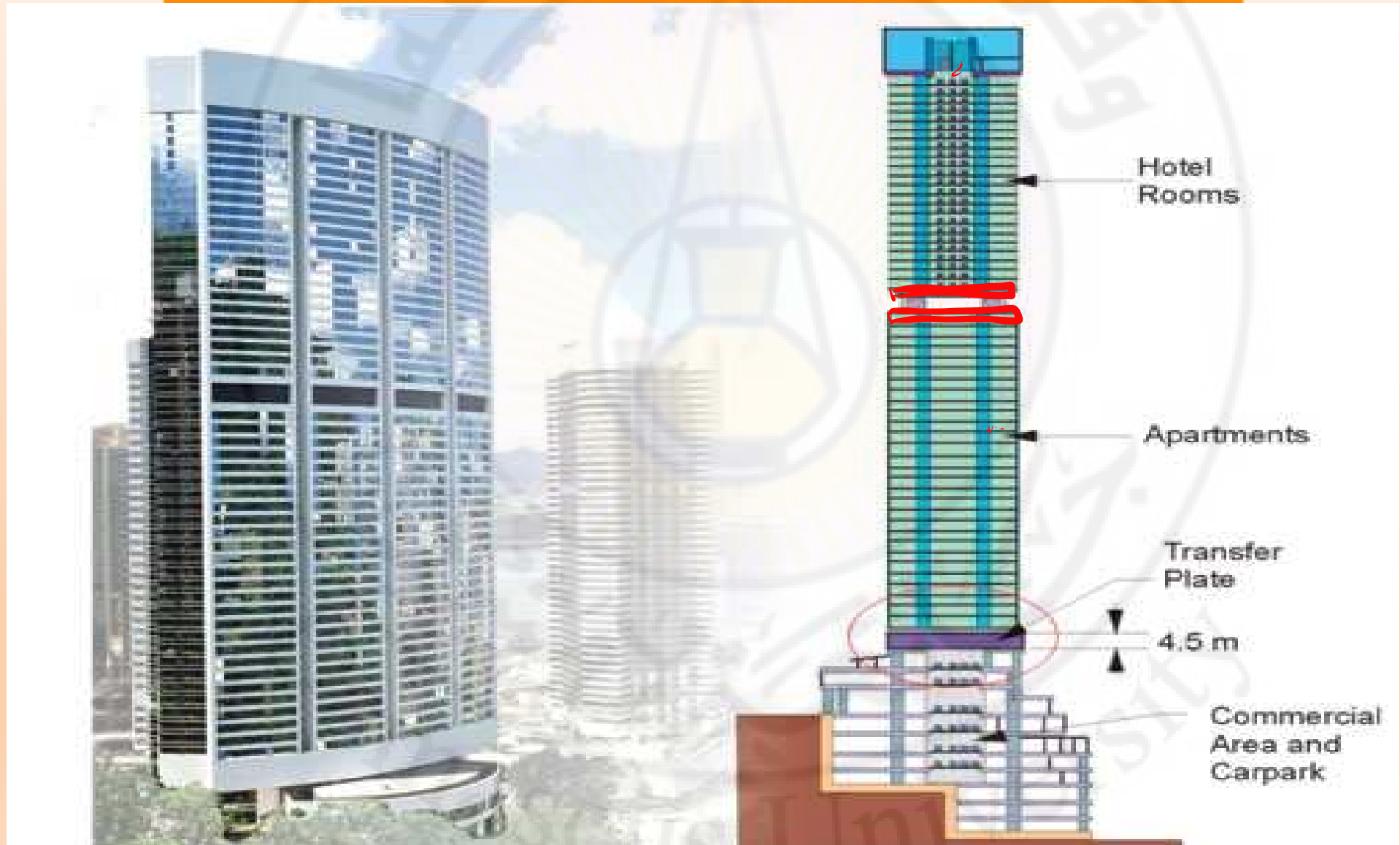
1. Introduction

Plate Girder as Transfer System



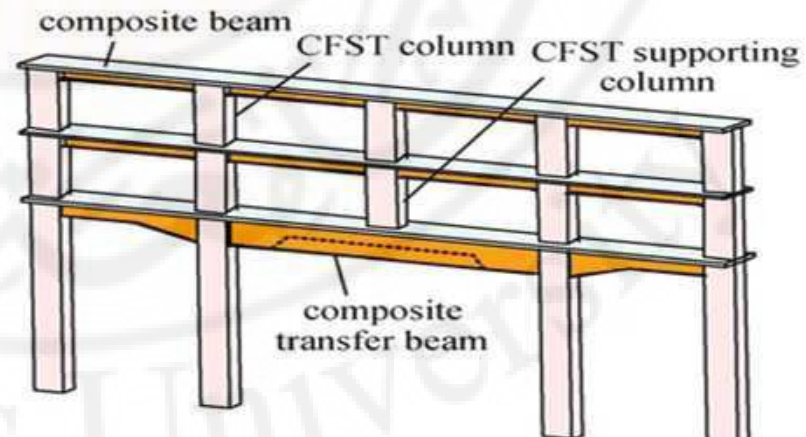
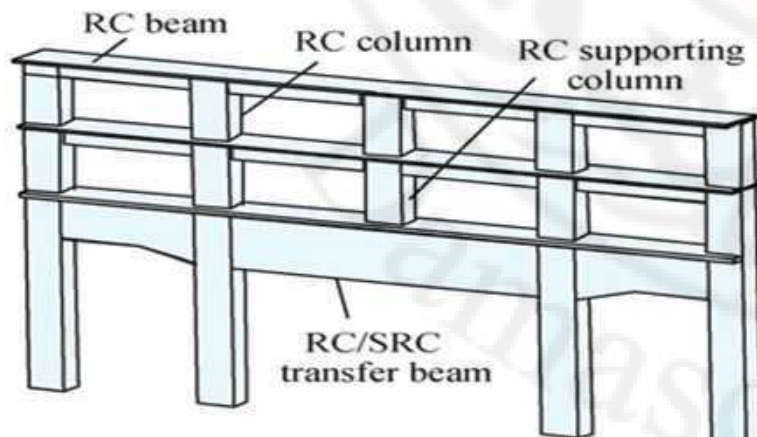
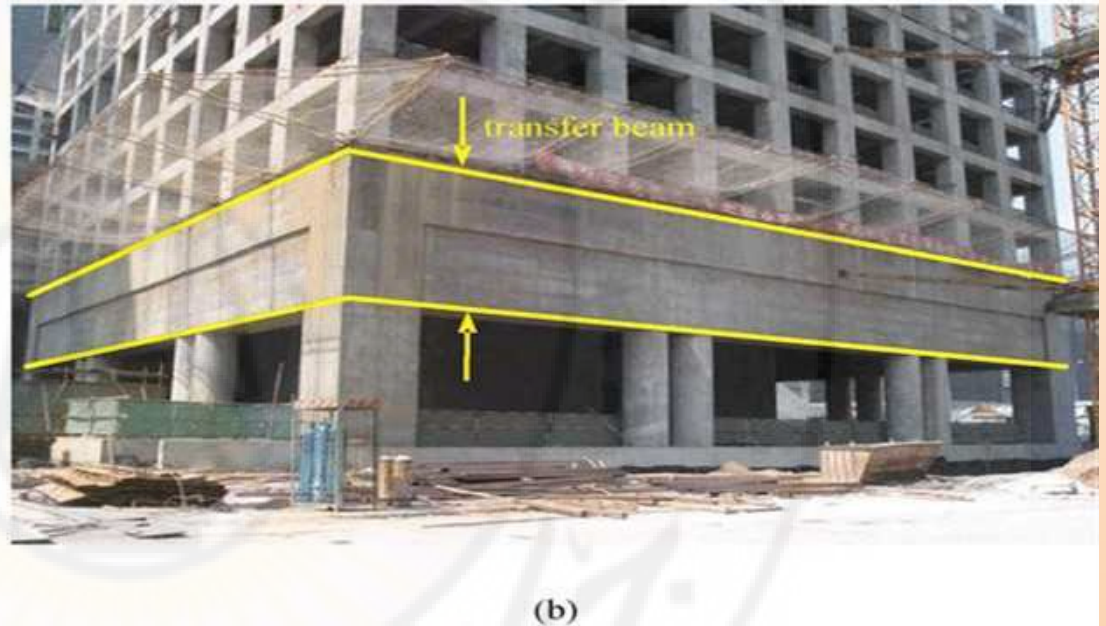
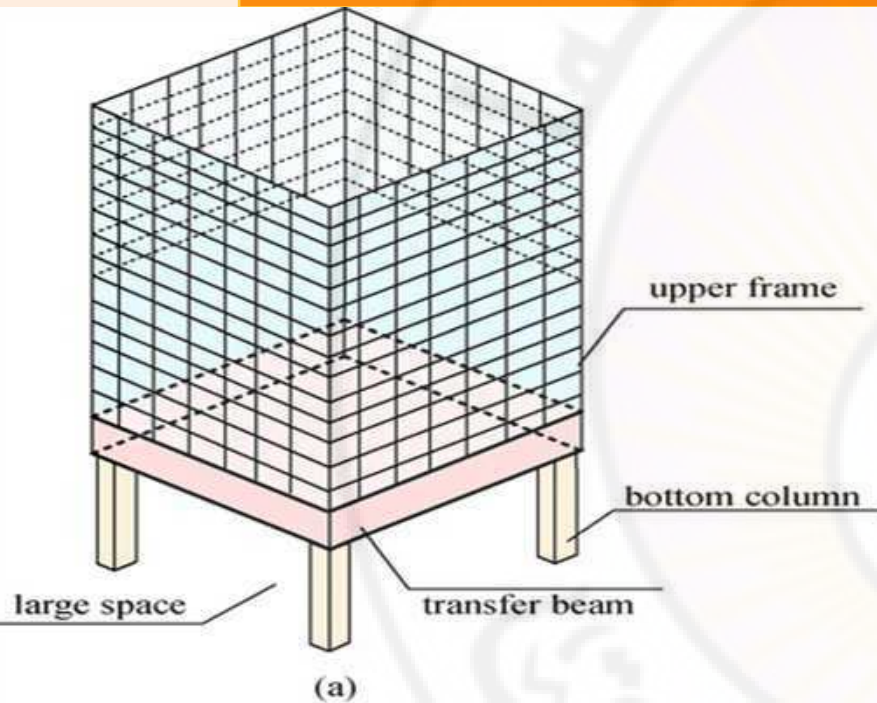
1. Introduction

Plate Girder as Transfer System



1. Introduction

Plate Girder as Transfer System

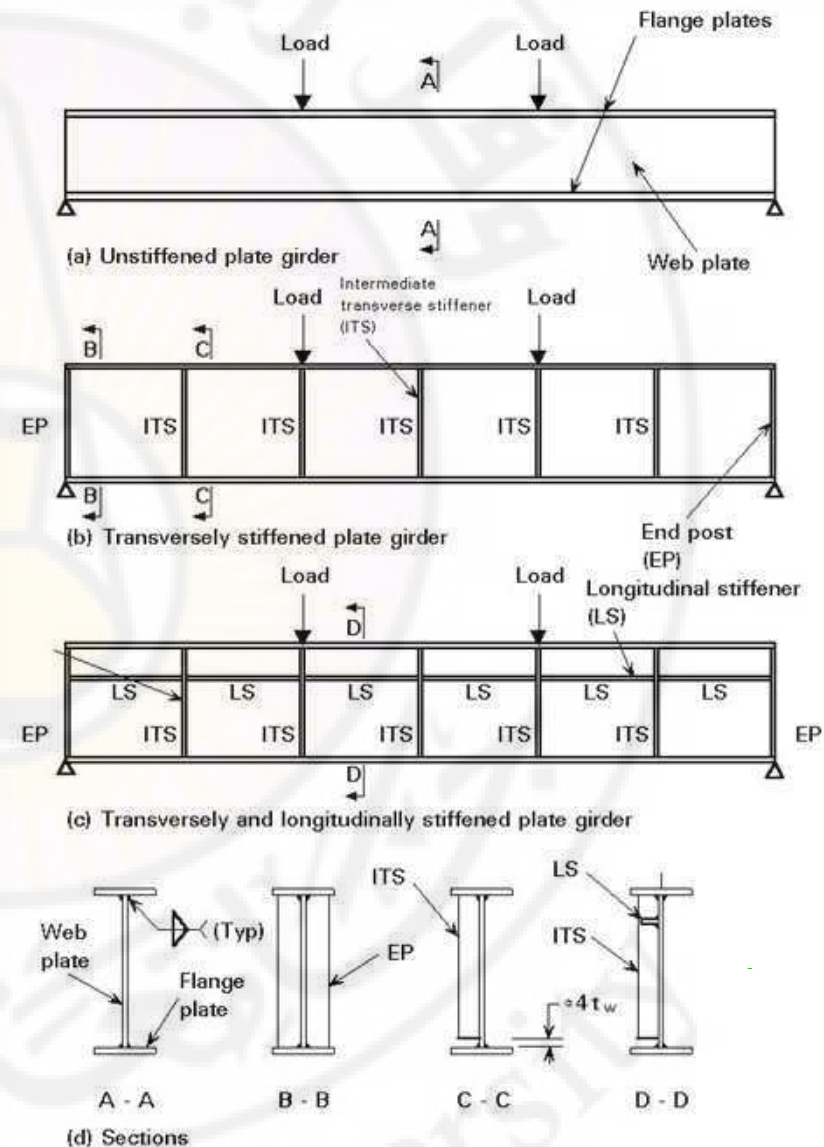


2. Plate Girder Types

There are several forms of plate girder;

1- Unstiffened 2- transversely stiffened 3- transversely and longitudinally stiffened.

The three girders shown have **bisymmetric I-profile cross-sections**, although flanges of different size are sometimes used, Other types of cross-section **are monosymmetric I-profiles**, which are popular in composite construction with the smaller flange on top or as crane girders with the larger flange on top.



2. Plate Girder Types

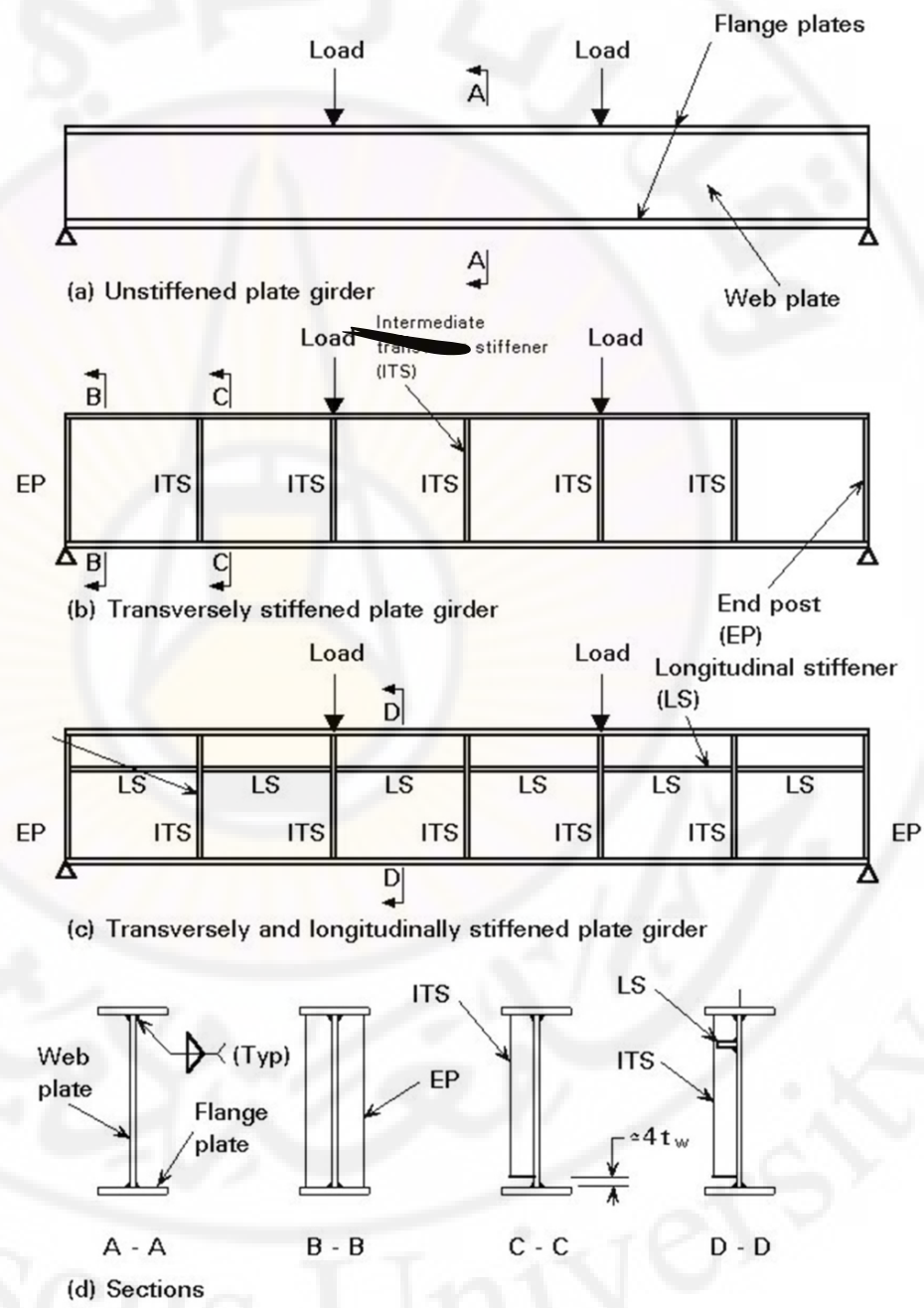
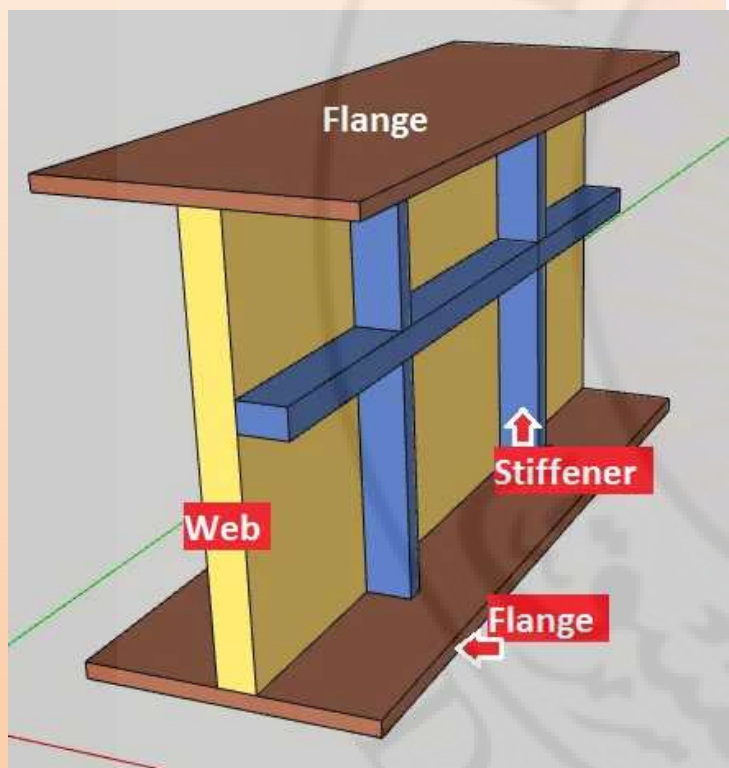
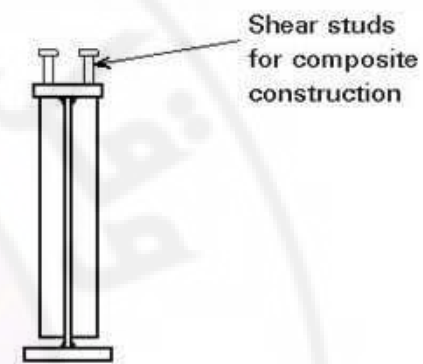


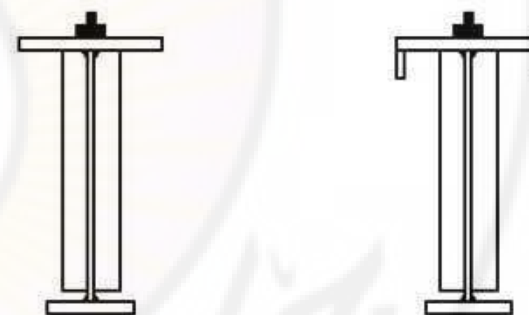
Figure 2 Stiffened and unstiffened plate girders

2. Plate Girder Types

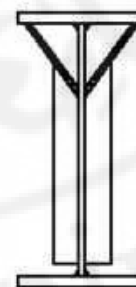
The three girders shown have **bisymmetric I-profile cross-sections**, although flanges of different size are sometimes used, Other types of cross-section **are monosymmetric I-profiles**, which are popular in composite construction with the smaller flange on top or as crane girders with the larger flange on top.



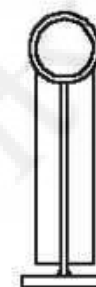
(a) Monosymmetric I-section plate girder



(b) Monosymmetric crane girders



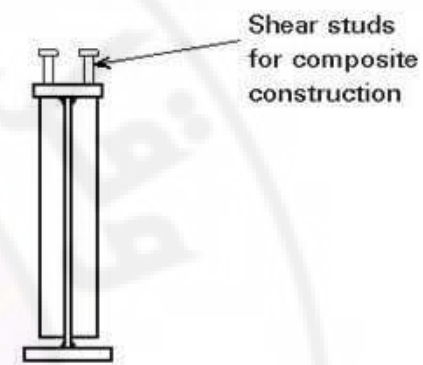
(c) Delta girder



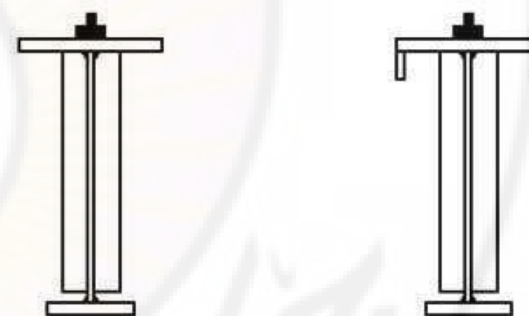
(d) Tubular top flange girder

2. Plate Girder Types

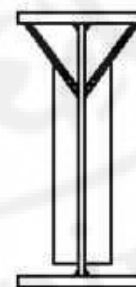
There are also two other (less common) variations - the "**delta girder**" and the tubular- **top-flange girder** - both being possible solutions in cases of long laterally-unsupported top compression flanges prone to lateral-torsional buckling.



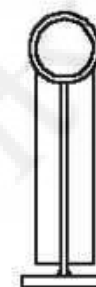
(a) Monosymmetric I-section plate girder



(b) Monosymmetric crane girders



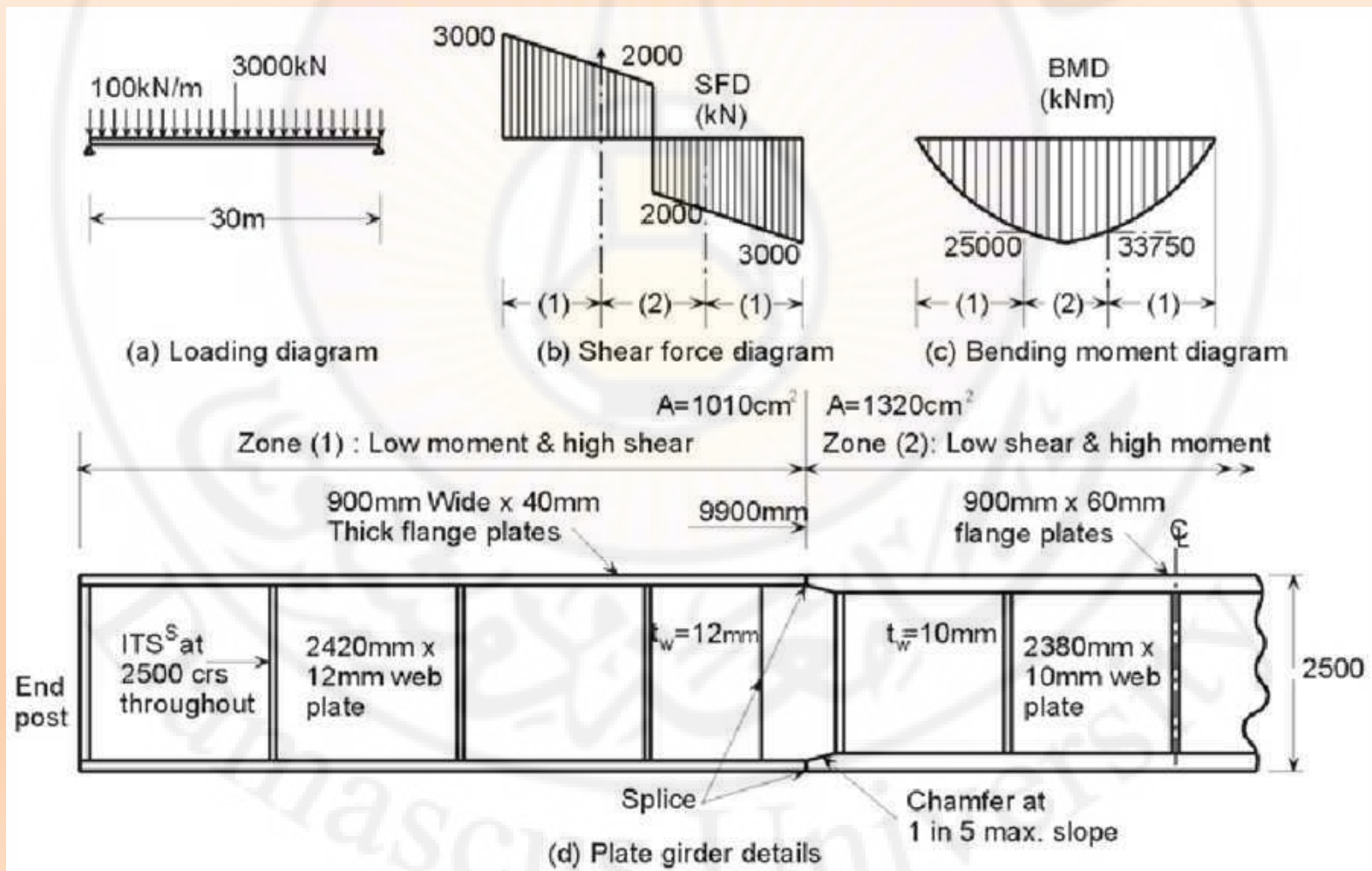
(c) Delta girder



(d) Tubular top flange girder

2. Plate Girder Types

There is also considerable scope for **variation of cross-section in the longitudinal direction**. A designer may choose to **reduce the flange thickness (or breadth) in a zone of low applied moment**, especially when a field-splice facilitates the change. Equally, **in a zone of high shear**, the designer might choose to **thicken the web plate**.



3. Plate Girder Proportions

3-1 Approximate Common Practice

Web depth

The depth, h , will usually be in the range

$$\text{Span}/12 \leq h \leq \text{Span}/8$$

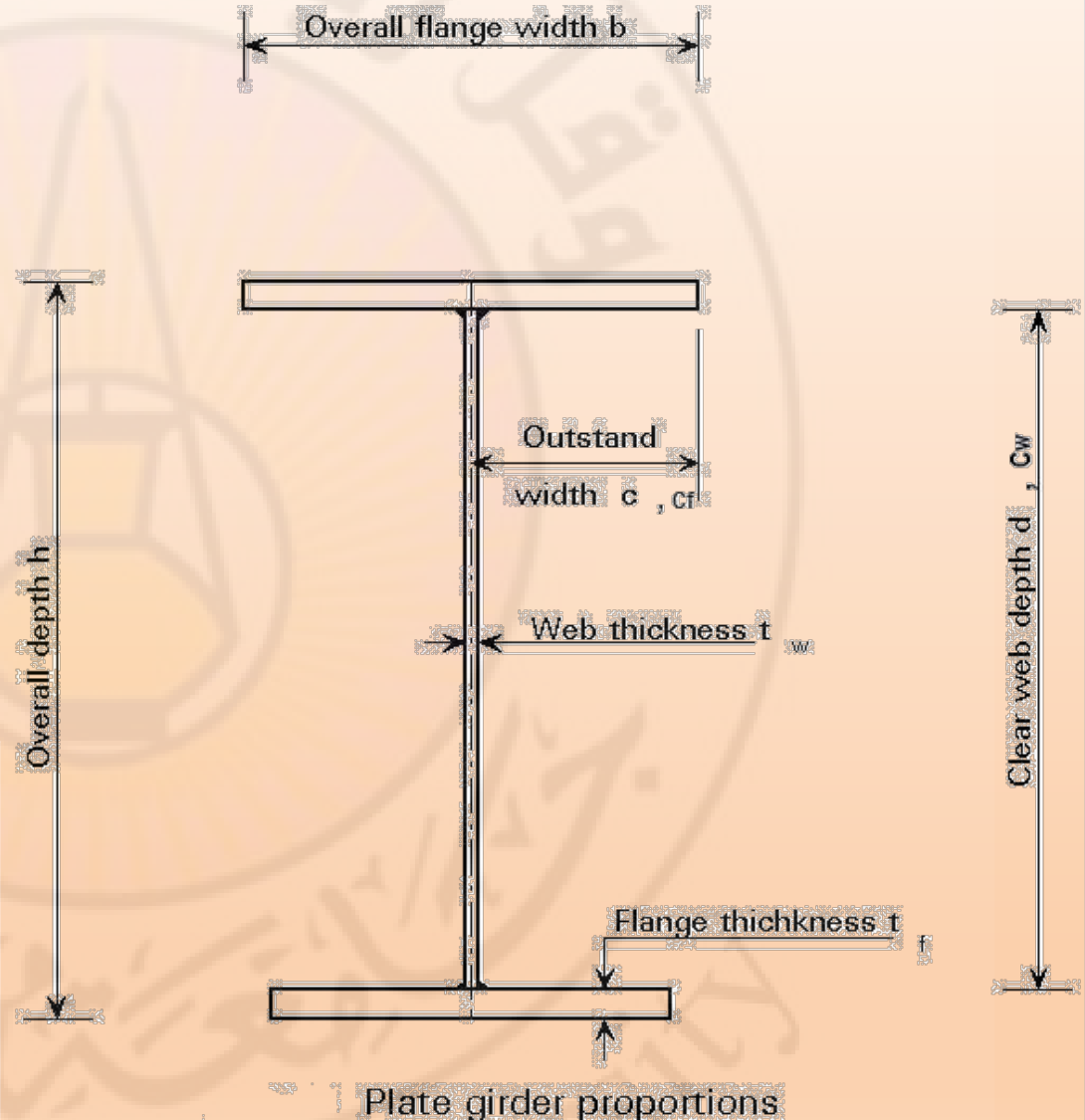
Flange breadth : The breadth, b , will usually be in the range $h/5 \leq b \leq h/3$

Flange thickness: The flange thickness, t_f , will usually at least satisfy the requirements of Eurocode 3 (Table 5.3.1) for Class 3

(semi-compact) sections, i.e. $c/t_f \leq 14 \epsilon$

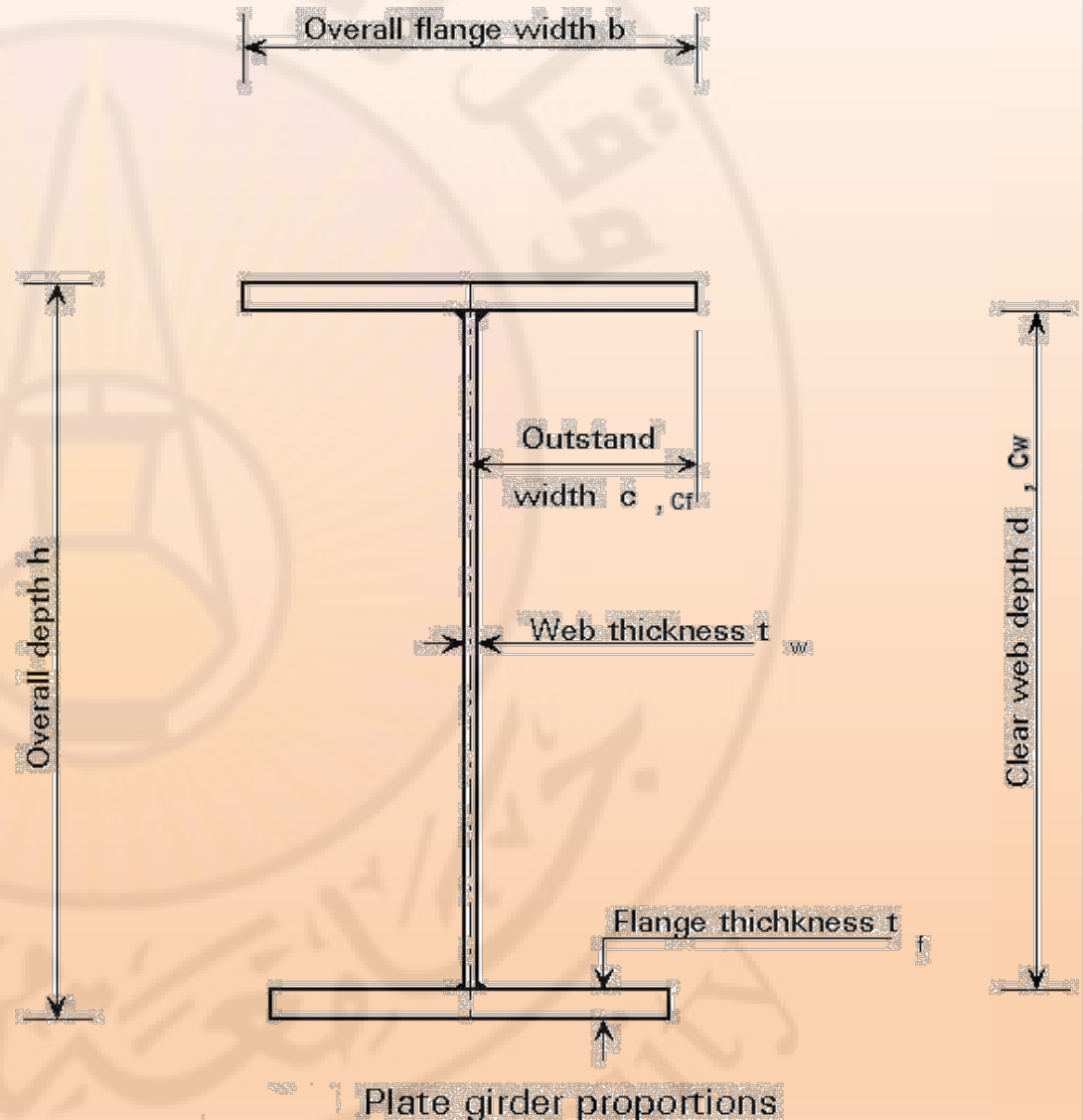
Web thickness: t_w , "as thin as possible for weight control => Class 4 section I".

$$\frac{h_w}{t_w} \approx 124 \epsilon \text{ (slender)}$$



3. Plate Girder Proportions

3-1 Approximate Common Practice



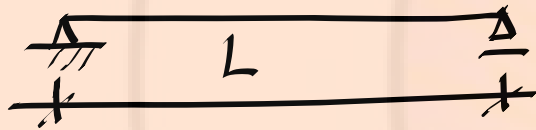
Web thickness will determine the exact basis for web design, depending on whether the web is classified with regard to shear buckling as "thick" or "thin". Thin webs will often require stiffening; this may take the form of transverse stiffeners, longitudinal stiffeners or a combination. Longitudinally stiffened girders are more likely to be found in large bridge construction where high d/t_w ratios are appropriate, e.g. **$200 \leq d/t_w \leq 500$**

3. Plate Girder Proportions

3-1 Approximate Common Practice

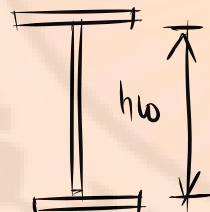
A) Element Lengths

1) Given Girder Span



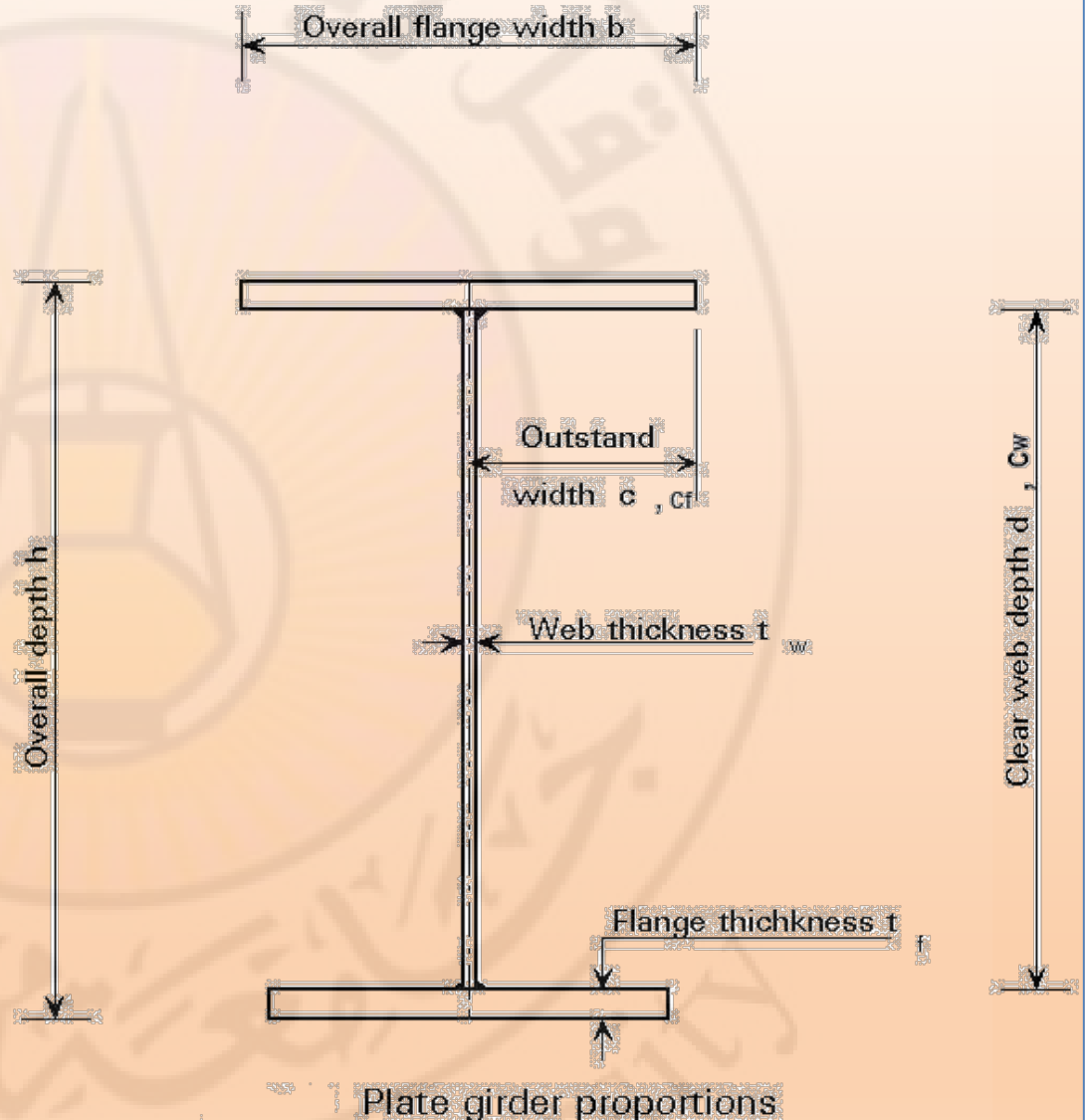
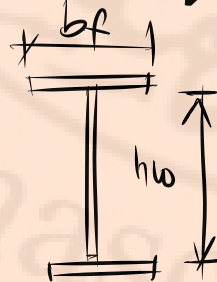
2) Estimate the Girder height

$$\frac{L}{12} < h_w < \frac{L}{8}$$



3) Estimate the flange width

$$\frac{h_w}{5} \leq b_f \leq \frac{h_w}{3}$$



3. Plate Girder Proportions

3-1 Approximate Common Practice

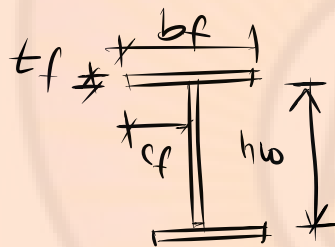
B) Thickness estimated in regard to section classification Euro code 3, Table (5.3.1)

B.1) Flang thickness t_f

minimum t_f apply class 3

$$c \leq \frac{t_f}{\epsilon}$$

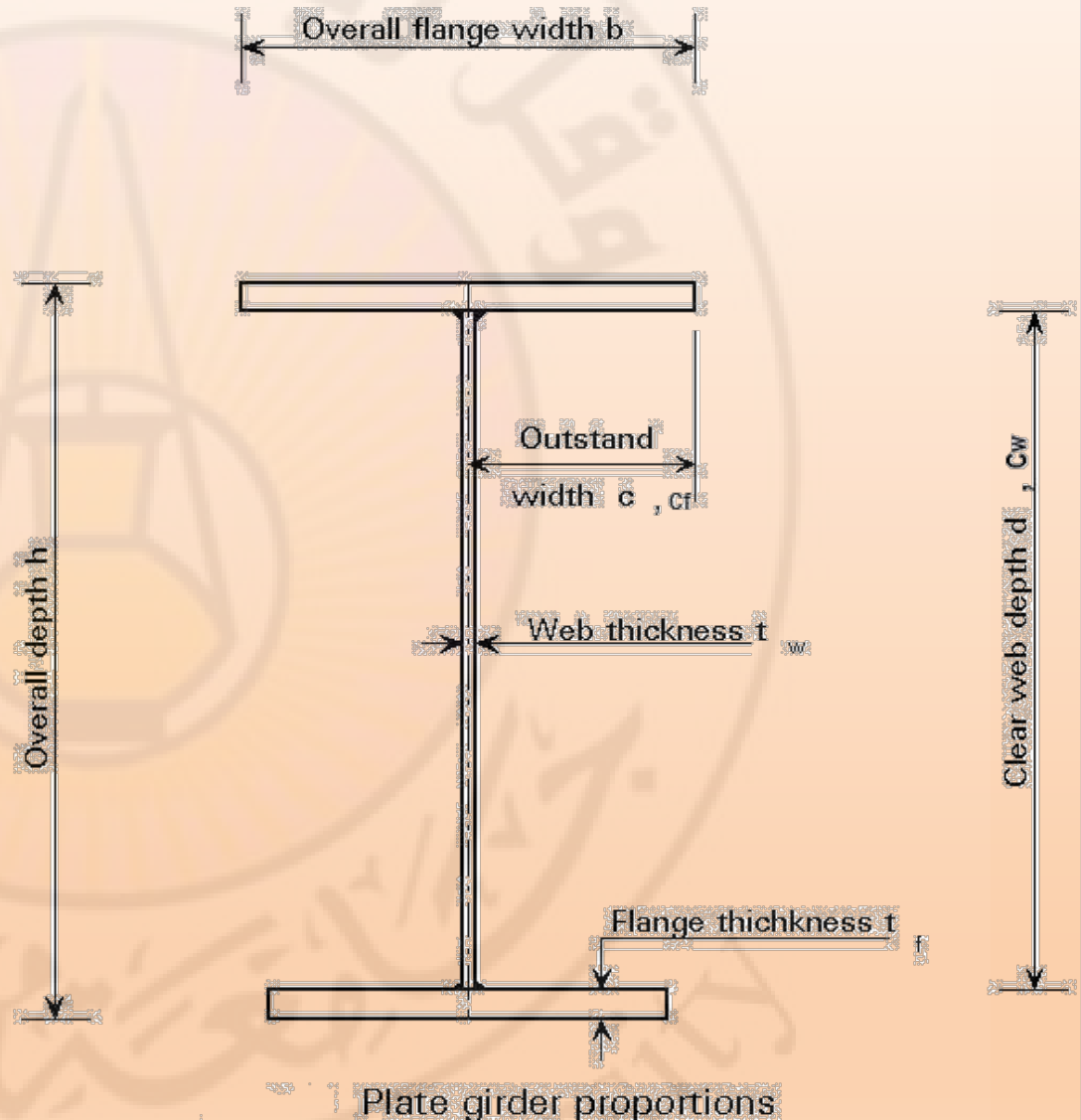
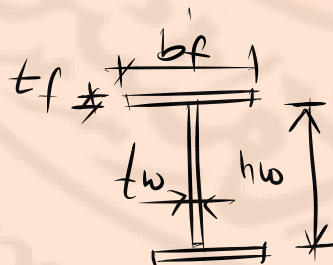
$$\frac{c}{t_f} \leq 14 \epsilon$$



B.2) Web thickness t_w

maximum t_w selected to minimize the girder weight

$$\frac{h_w}{t_w} \leq 124 \epsilon$$



3. Plate Girder Proportions

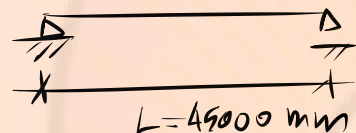
3-1 Approximate Common Practice

Example

Estimate plate girder section dimension at the span $L=45\text{m}$.

Solution

A.1) girder Height



$$\frac{L}{12} \leq h \leq \frac{L}{8}$$

$$\frac{45000}{12} \leq h \leq \frac{45000}{8}$$

$$3750 \leq h \leq 5625$$

$$\rightarrow |h = 5000| \text{ mm}$$

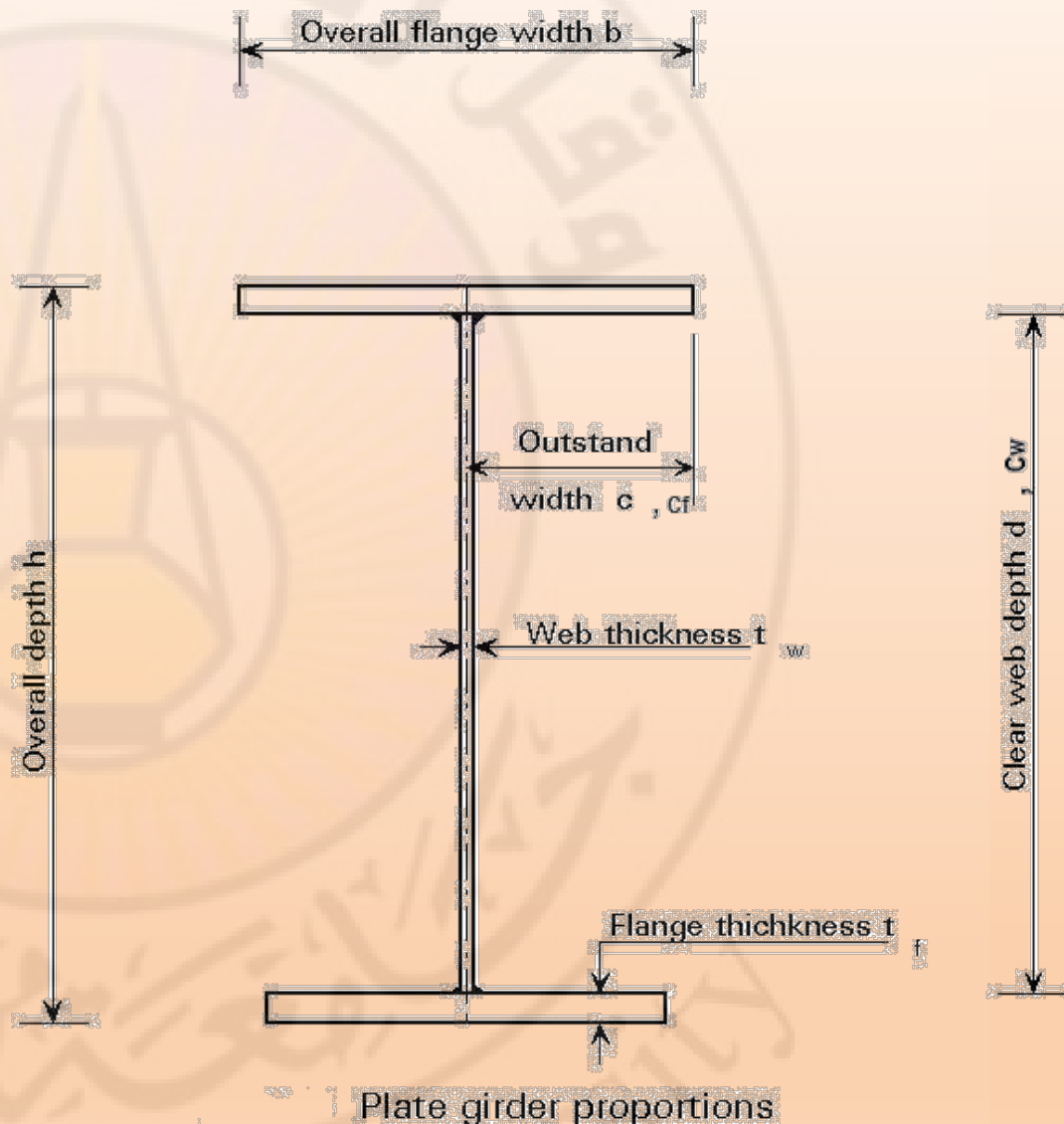
A.2) Flange width

$$\frac{h}{5} \leq b \leq \frac{h}{3}$$

$$\frac{5000}{5} \leq b \leq \frac{5000}{3}$$

$$1000 \leq b \leq 1667$$

$$\rightarrow |b = 1500| \text{ mm}$$



3. Plate Girder Proportions

3-1 Approximate Common Practice

Example

Estimate plate girder section dimension at the span $L=45\text{m}$.

Solution

B.1) Flange thickness

$$\frac{C_f}{t_f} \leq 14\epsilon$$

Suppose, $\epsilon = 1$

$$C_f \approx \frac{bf}{2} = \frac{1500}{2} = 750$$

$$\rightarrow t_f \leq \frac{C_f}{14} = \frac{750}{14} \approx 53.57$$

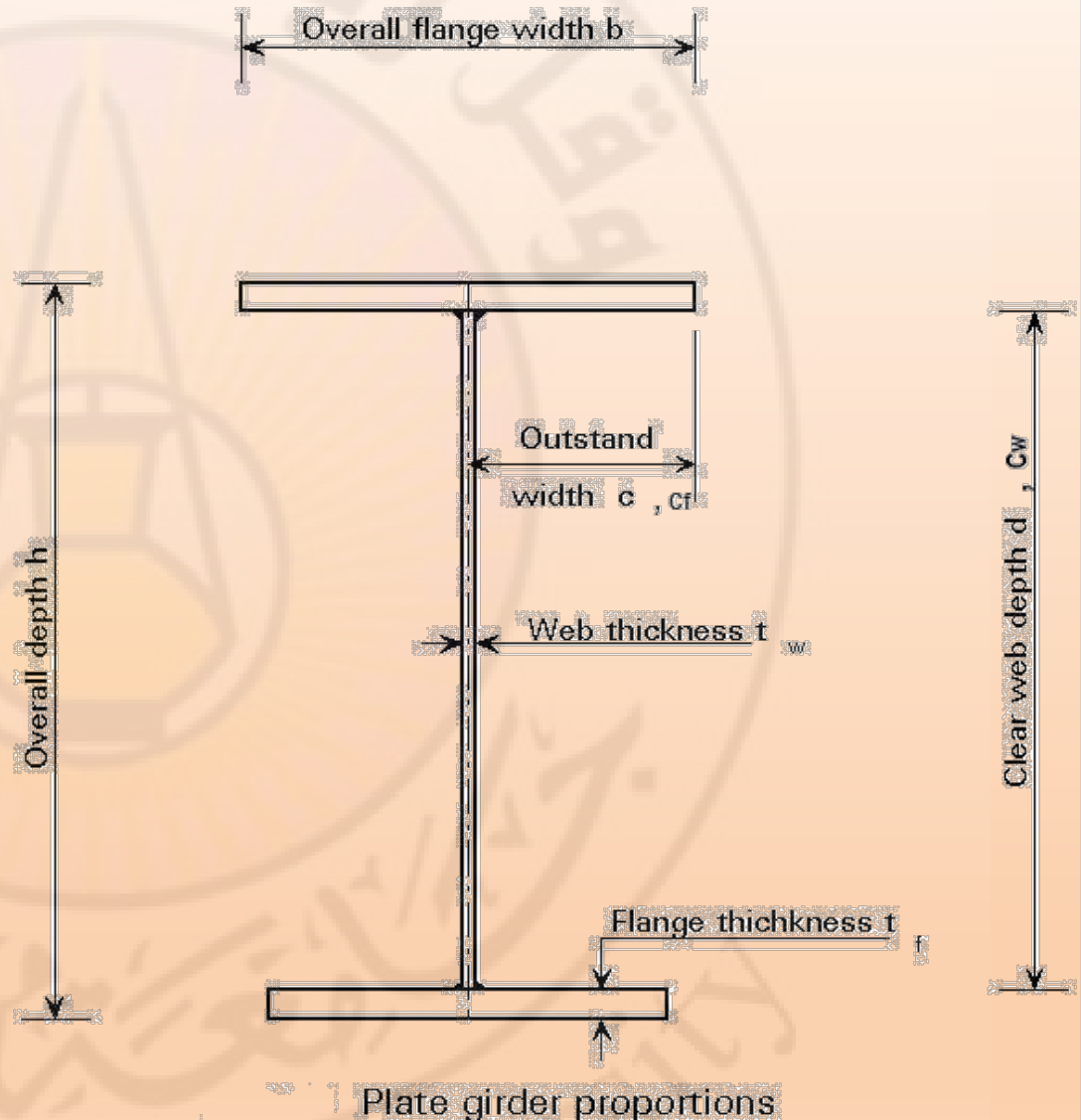
$$t_f = 50 \text{ mm}$$

B.2) Web thickness

Go to minimum, $\frac{h_w}{t_w} \geq 124\epsilon$

$$t_w \leq \frac{h_w}{124} \leq \frac{5000}{124} \leq 40 \text{ mm}$$

$\rightarrow$ take $t_w = 40 \text{ mm}$



3. Plate Girder Proportions

3-1 Approximate Common Practice

Web thickness discussion

Each 1mm of the web include the following weight,
 steel specific weight $\gamma_{\text{steel}} = 7850 \text{ kg/m}^3$

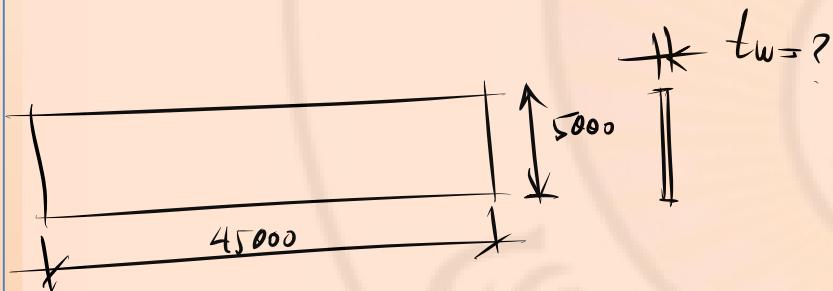
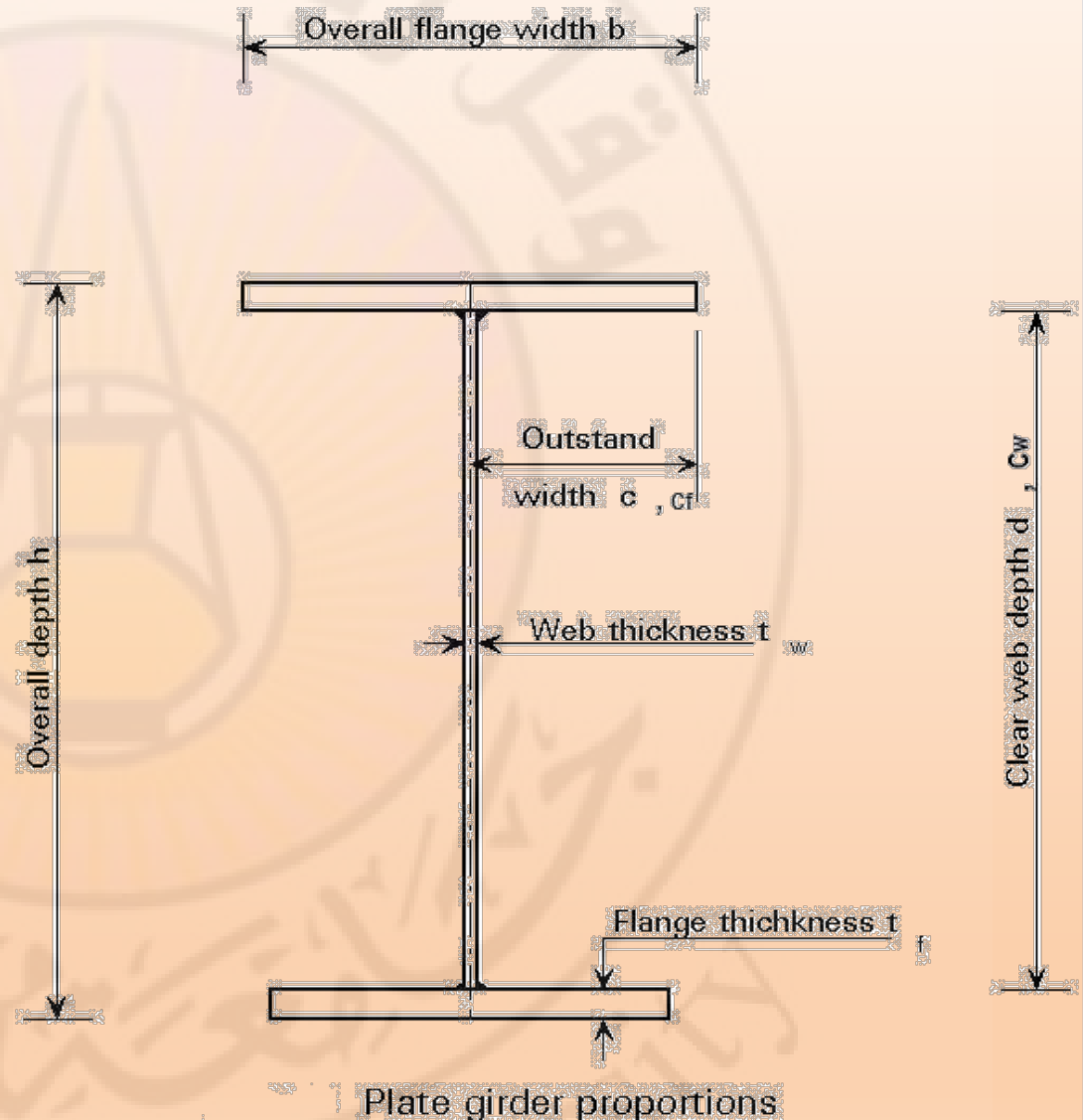


Plate steel Weight = Steel Size (m^3) * γ_{steel}

$$\frac{45(\text{m}) * 5(\text{m}) * 1(\text{mm})}{1000} * 7850 \approx 1766 \text{ kg}$$

Each (1mm) of web thickness will add $\approx 1.8 \text{ ton}$ weight



3. Plate Girder Proportions

3-1 Approximate Common Practice



3. Plate Girder Proportions

3-1 Approximate Common Practice

$$3750 \sim h_w \sim 5625$$

$$\frac{3750}{124} \sim t_w \sim \frac{5625}{124}$$

$$30 \sim c_w \sim 46$$

1)

Different in weight between the minimum and maximum choices

$$16 \text{ m} * 1.8 \text{ ton} = 24 \text{ ton}$$

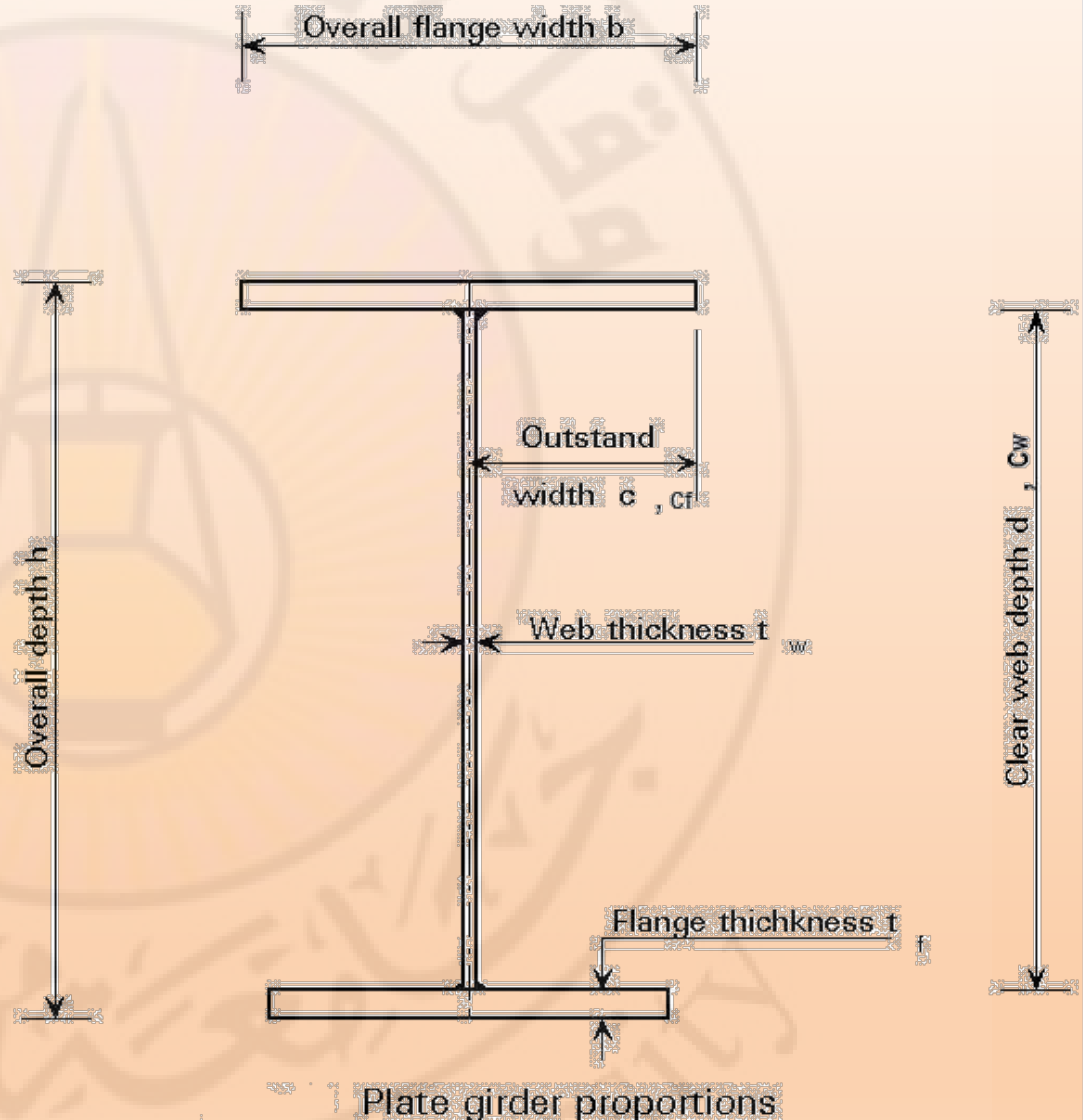
Cost, 1 kg steel will cost $\sim 10000 \text{ SYP}$

$$\text{Total cost} = 10000 * 24 * 1000 = 240000000$$

2) Reduce h_w will

① increase the demand to flange thickness

② decrease the demand to web thickness



3. Plate Girder Proportions

3-2 Practice To EC3-BS EN 1993-1-5

I-) Web depth : , h ,

Advances in fabrication methods allow the economic manufacture of plate girders of constant or variable depth.

Traditionally, constant-depth girders have been more common in buildings ; however, this may change as designers become more inclined to modify the steel structure to accommodate services.' Recommended span-to Depth ratios are given below.

Recommended span-to-depth ratios for plate girders used in buildings

Applications	Span-to-depth ratio
(1) Constant-depth beams used in simply-supported composite girders, and for simply-supported non-composite girders with concrete decking	12 to 20
(2) Constant-depth beams used in continuous non-composite girders using concrete decking (N.B. continuous composite girders are uncommon in buildings)	15 to 20
(3) Simply-supported crane girders (non-composite construction is usual)	10 to 15

3. Plate Girder Proportions

3-2 Practice To EC3-BS EN 1993-1-5

II-) Flange breadth : , (**b** , or **C_f**) , Flange thickness: , (**t_f**) ,

Web thickness: (**t_w** , or **w**) :

In general, the slenderness of the cross-sections of plate girders used in buildings should not exceed the limits specified for Class 3 cross-sections (presented in Clause 5.5 of BS EN 1993-1-13 , even though more slender cross-sections are permitted.

The choice of plate thickness is related to buckling of the cross-section. If the plates are too thin they may require stiffening to restore adequate stiffness and strength, and the extra workmanship required is expensive. In view of the above, the maximum depth-to-thickness ratio of the webs (**c_w/t_w**) of plate girders in buildings is usually limited to:

$$c_w / t_w < 124\epsilon$$

The outstand width- to-thickness ratio of the compression flange (**c_f/t_f**) is typically limited to :

$$c_f / t_f < 14\epsilon$$

c_w, and **c_f** are the flat element widths, measured from the edges of the fillet welds (or root radii for rolled sections). For initial design purposes, when the weld size may be unknown, it is conservative to ignore the weld and take $c_w = h$, (the distance between the flanges) for webs and **c_f = b/2 - t_w/2** for outstand flanges.

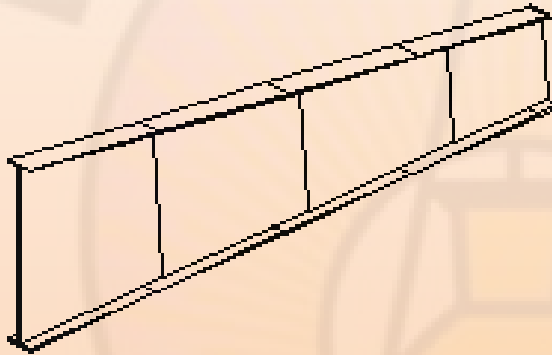
3. Plate Girder Proportions

3-2 Practice To EC3-BS EN 1993-1-5

II-) Flange breadth : , (b , or C_f) , Flange thickness: , (t_f) ,

Web thickness: (t_w , or w) :

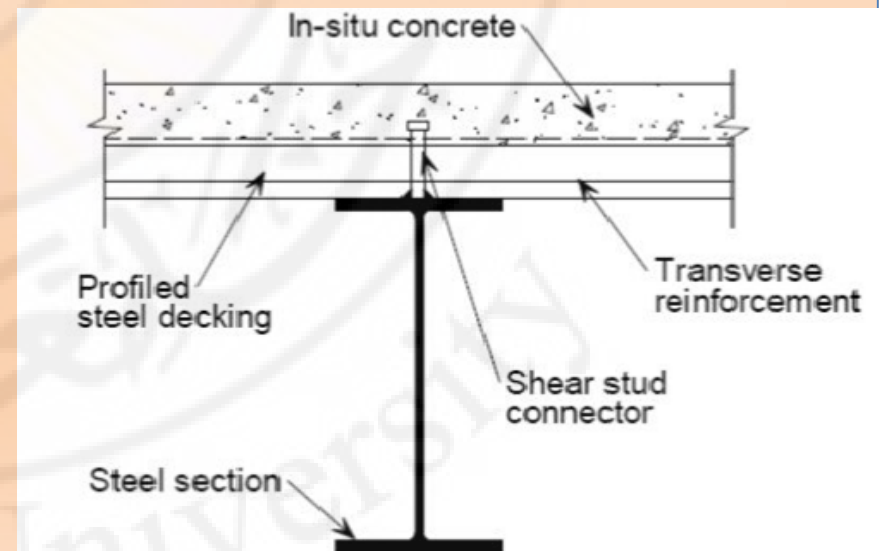
Changes in flange size along the girder are not usually worthwhile in buildings.



For non-composite girders the flange width is usually within the range 0.3-0.5 times the depth of the section (0.4 is most common).

For simply-supported composite girders these guidelines can still be employed for preliminary sizing of compression flanges.

The width of tension flanges can be increased by 30%.



3. Plate Girder Proportions

3-2 Practice To EC3-BS EN 1993-1-5

transversely and longitudinally

III-) Stiffeners

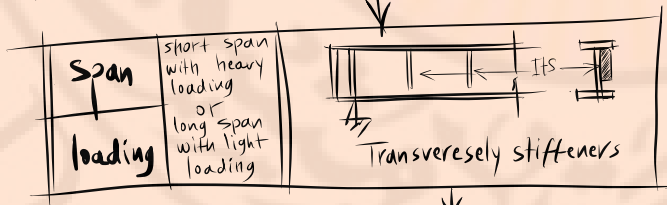
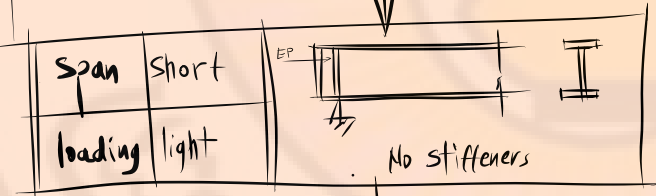
Web stiffeners location and spacing

Buildings or Bridges

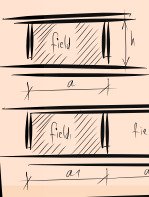
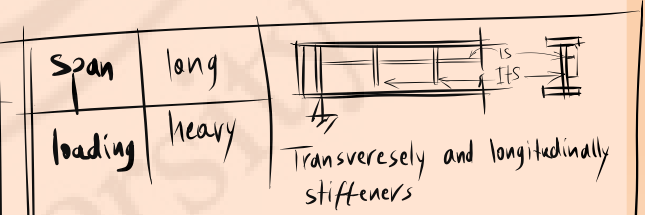
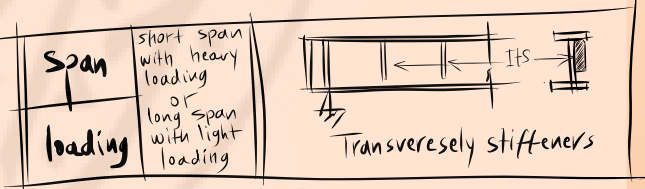
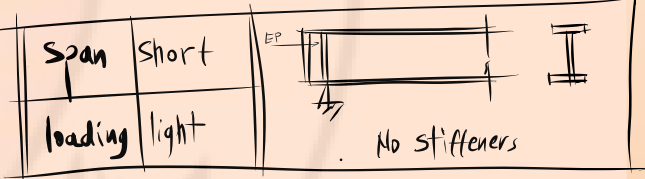
Bridges

Buildings

Procedure for transversely stiffeners may be required



Procedure for transversely and longitudinally stiffeners may be required



Known: h/w ; a may be selected first as $a = (1.1 \sim 1.5) \cdot h$

other field width may be varried during design Procedure

3. Plate Girder Proportions

3-3 Proportion to Service and Buckling Performance

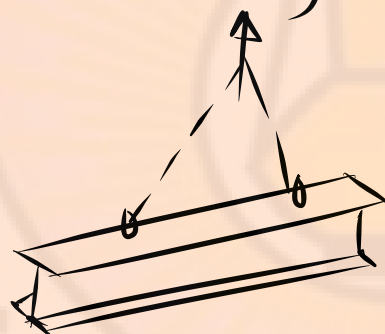
Ability

Web Minimum Thickness due to Service and Buckling Performance

Flang Buckling into Web Plane



Web Buckling during Erection



Web Breathing

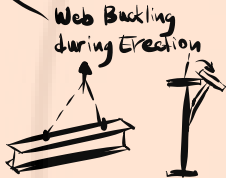


3. Plate Girder Proportions

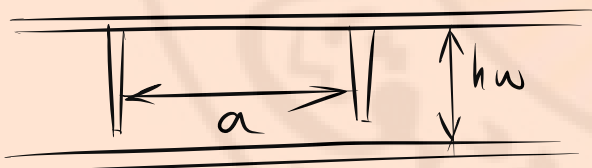
3-3 Proportion to Service and Buckling Performance

Ability

Web Minimum Thickness due to Service and Buckling Performance



2.) Stiffened Girder



1.) Unstiffened Girder



$$a > h_w; \quad t_w \geq \frac{h_w}{250}$$

$$a < h_w; \quad t_w \geq \left(\frac{h_w}{250} \right) \left(\frac{a}{h_w} \right)^{\frac{1}{2}}$$

OR:

$$t_w \geq \left(\frac{h_w}{K} \right) \left(\frac{f_f}{E} \right) \sqrt{\frac{A_f c}{A_w}}$$

$$t_w \geq \frac{h_w}{250}$$

3. Plate Girder Proportions

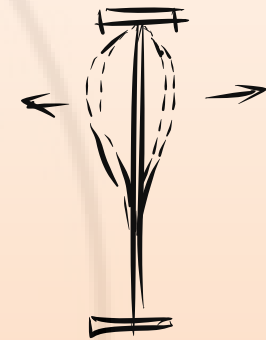
3-3 Proportion to Service and Buckling Performance

Ability

A further serviceability consideration, though one that can also contribute to **fatigue failure**, is that of web breathing. Web breathing refers to the recurring out-of-plane deflection of the web (due to plate buckling) under variable loads. It is principally associated with bridges due to the repeated nature of the loading; web breathing may be neglected provided slenderness criteria set out in Clause 7.4 of BS EN 1993-2 are met.

The following **minimum web thickness** values are prescribed in Clause 8(1) of BS EN 1993-1-5 to avoid the compression flange buckling into the web (i.e. flange induced web buckling).

Web Breathing



$$t_w \geq \left(\frac{h_w}{k} \right) \left(\frac{f_{yf}}{E} \right) \sqrt{\frac{A_{fc}}{A_w}}$$

where A_w is the cross-sectional area of the web, A_{fc} is the effective cross-sectional area of the compression flange.

3. Plate Girder Proportions

3-3 Proportion to Service and Buckling Performance

Ability

A **minimum web thickness** is needed to prevent the compression flange buckling into the web (see Clause 8(1) of BS EN 1993-1-5) and to ensure satisfactory serviceability performance including, for example, avoiding unsightly buckles developing during erection and in service.

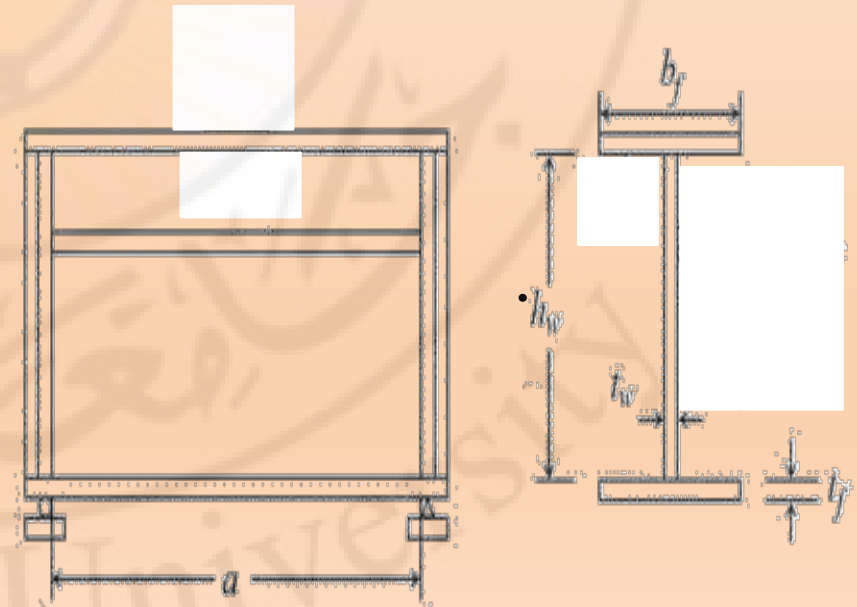
The buckling resistance of slender webs can be increased by the provision of web stiffeners. In general, the webs of plate girders used in buildings are either unstiffened or have transverse stiffeners only. Minimum web thickness values to avoid serviceability problems are not prescribed in BS EN 1993-1-5, but the following values, taken from BS 5950-1, are recommended.

For unstiffened webs, $t_w \geq \frac{h_w}{250}$

For transversely stiffened webs,

For $a > h_w$, $t_w \geq \frac{h_w}{250}$

For $a \leq h_w$, $t_w \geq \left(\frac{h_w}{250} \right) \left(\frac{a}{h_w} \right)^{1/2}$



3. Plate Girder Proportions

3-3 Proportion to Service and Buckling Performance Ability

k depends on the utilization of the section in bending:

k = 0.3 when plastic rotation is utilized (i.e. the formation and rotation of a plastic hinge in the member as part of a collapse mechanism);

k = 0.4 when the plastic moment resistance is Utilized ; and

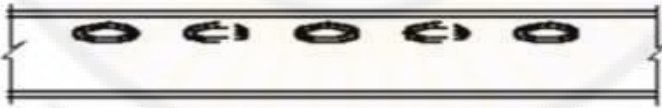
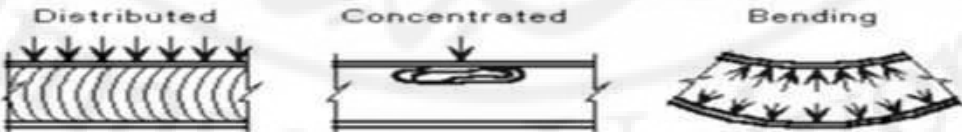
k = 0.55 when the elastic moment capacity is utilized.

Local buckling of the compression flange may also occur if the flange plate is of slender (Class 4) proportions. In general there is seldom good reason for the **cf/tf** ratio of the compression flanges of plate girders used in buildings to exceed the Class 3 limit of **14ε** set out in Clause 5.5 of BS EN 1993-1-1.



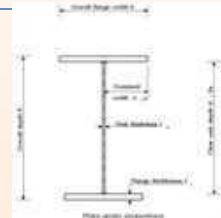
3. Plate Girder Proportions

3-3 Proportion to Service and Buckling Performance Ability

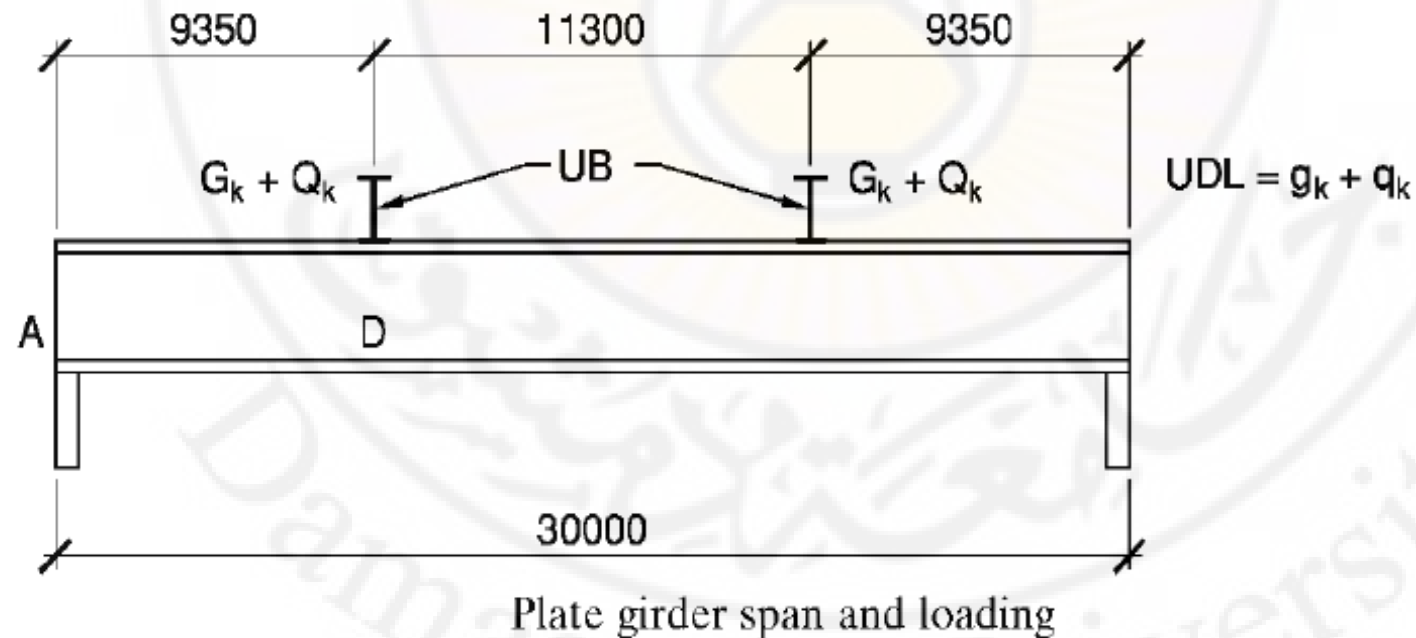
Buckling type	Illustration	[Eurocode 3 clauses] and ESDEP lectures
Shear buckling of web		[5.6] Lecture 8.4.2 & 8.4.3
Lateral-torsional buckling of girder		[5.5.2 and Annex F] Lecture 7.9
Local buckling of compression flange		[5.3 and table 5.3.1 (sheet 3)] Lecture 7.3
Compression buckling of web		[5.3 and table 5.3.1 (sheet 1)] Lecture 7.6
Flange induced buckling of the web		[5.7.7]
Local buckling of web (due to vertical load)		[5.7.4, 5.7.5, 5.3.6, 5.4.10]

3. Plate Girder Proportions

3-4 Worked Example 1

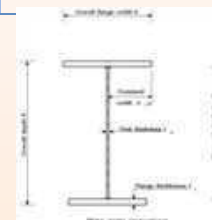


The plate girder shown below is fully laterally restrained along its length. For the design loading specified below, design a transversely stiffened plate girder in S275 steel.



3. Plate Girder Proportions

3-4 Worked Example 1



Actions (loading)

Permanent actions: UDL $g_k = 25 \text{ kN/m}$

Point loads $G_k = 200 \text{ kN}$

Variable actions: UDL $q_k = 40 \text{ kN/m}$

Point loads $Q_k = 450 \text{ kN}$

Partial factors for actions

Partial factor for permanent actions $\gamma_G = 1.35$

Partial factor for variable actions $\gamma_Q = 1.50$

Reduction factor for permanent actions $\xi = 0.925$

ULS combination of actions

Design UDL $F_d = (1.35 \times 0.925 \times 25) + (1.5 \times 40) = 91.2 \text{ kN/m}$

Design point loads $F_d = (1.35 \times 0.925 \times 200) + (1.5 \times 450) = 925 \text{ kN}$

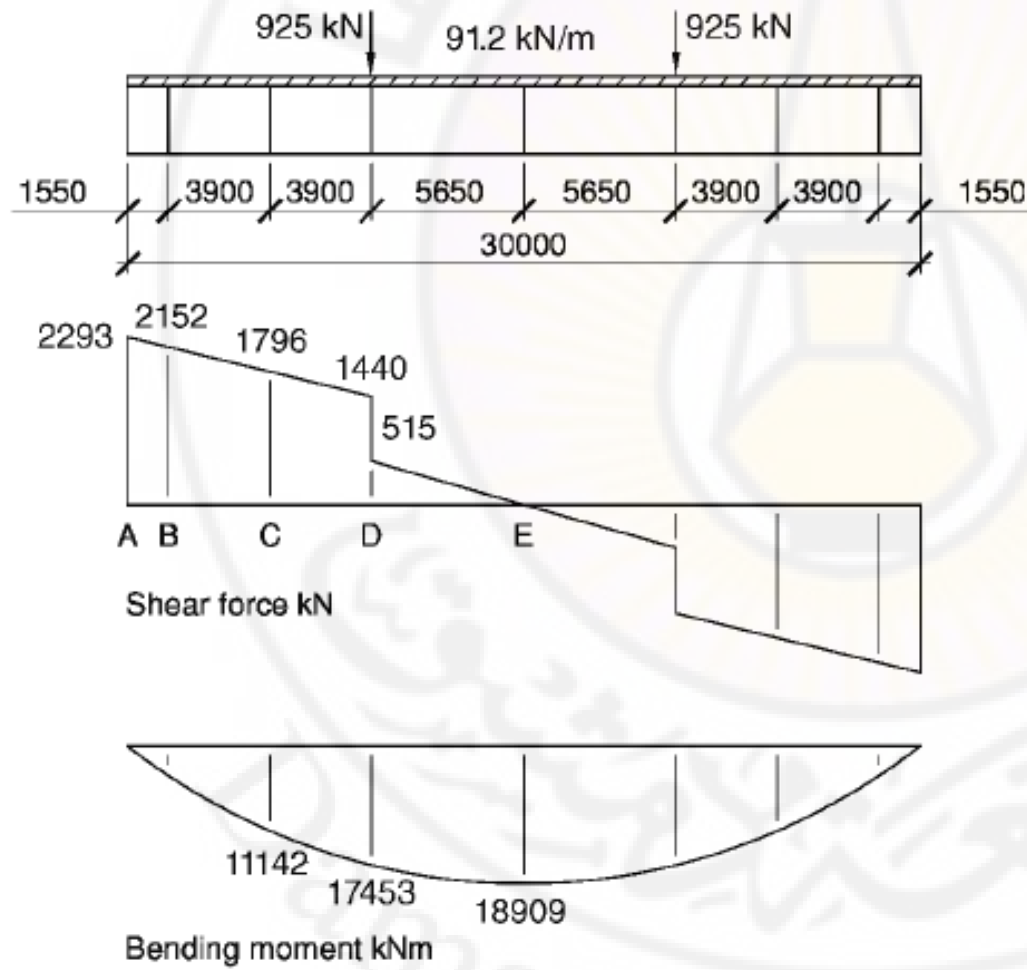
BS EN 1990

3. Plate Girder Proportions

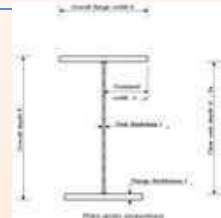
3-4 Worked Example 1

Design shear force and bending moment diagrams

The shear force and bending moment diagrams corresponding to the ULS design loads are shown below.



Design shear forces, bending moments and stiffener spacing



3. Plate Girder Proportions

3-4 Worked Example 1

Proportion

Initial sizing of plate girder

The recommended span/depth ratio for simply supported non-composite plate girders ranges between 12 for short span girders and 20 for long span girders. Herein the depth is assumed to be span/15.

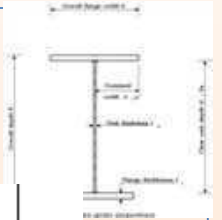
$$h_w = \frac{\text{span}}{15} = \frac{30000}{15} = 2000 \text{ mm}$$

$$\frac{h}{5} < b_f < \frac{h}{3}$$

$$\frac{2000}{5} < b_f < \frac{2000}{3}$$

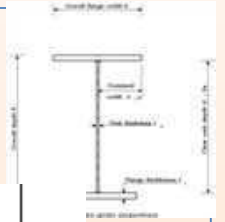
$$400 < b_f < 667$$

Choose $b_f = 500 \text{ mm}$



3. Plate Girder Proportions

3-4 Worked Example 1



$$\frac{b}{t_f} \leq 14\epsilon \Rightarrow$$

$$\rightarrow t_f \geq \frac{b}{14\epsilon} = \frac{500/2}{14} \approx 18 \text{ mm}$$

choose $t_f = 20 \text{ mm}$

$$b \times t_f = 500 \times 20 = 10\,000 \text{ mm}^2$$

Web

Provision stiffeners

Web is slender $\frac{h_w}{t_w} > 124\epsilon$

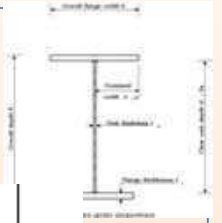
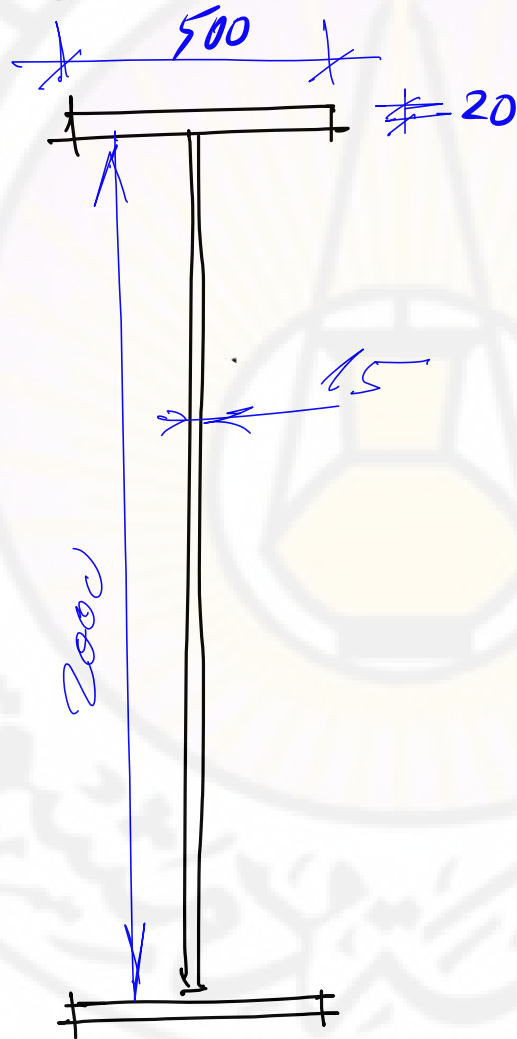
$$\epsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{255}} = 0.96$$

$$t_w < \frac{2000}{124 \times 0.96} = 16.8 \text{ mm} \quad \text{choose } t_w = 15 \text{ mm}$$

The minimum web thickness for plate girders in buildings is usually $t_w \geq h_w/124\epsilon$ to ensure a non-slender section. Assume the web thickness $t_w = 15 \text{ mm}$, slightly less than that required for a Class 3 section due to proposed transverse stiffening.

3. Plate Girder Proportions

3-4 Worked Example 1



3. Plate Girder Proportions

3-4 Worked Example 1

Web checking for service and Buckling:

1) check for criteria,

$$t_w \geq \frac{h_w}{250}$$

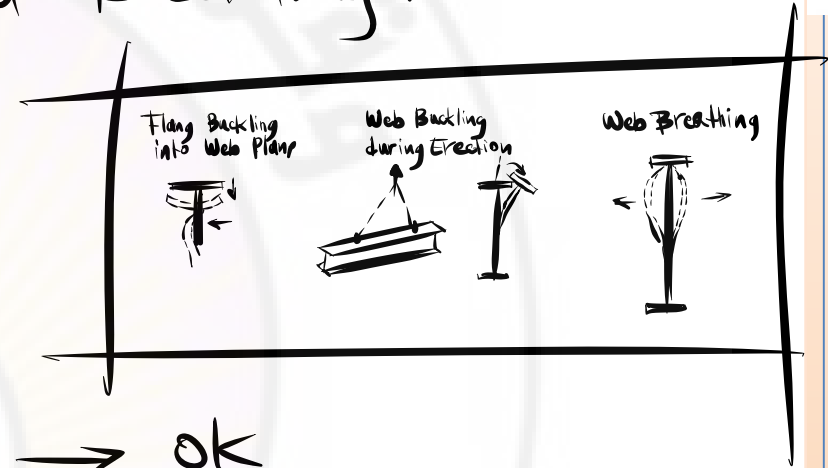
$$15; \frac{2000}{250} = 8 \text{ mm} \rightarrow \text{OK}$$

2.) check for criteria

$$t_w \geq \left(\frac{h_w}{k} \right) \left(\frac{f_{yf}}{E} \right) \sqrt{\frac{A_{fc}}{A_w}}$$

$k = 0.55$, Web is slender

$$A_{fc} = A_f \text{ (No local Buckling occurs)} = 10000 \text{ mm}^2$$



3. Plate Girder Proportions

3-4 Worked Example 1



$$A_w = 15 \times 2000 = 30\,000 \text{ mm}^2$$

$$t_w \geq \left(\frac{hw}{k} \right) \left(\frac{f_{te}}{E} \right) \sqrt{\frac{A_{fc}}{A_w}}$$

$$15 \geq \left(\frac{2000}{0.55} \right) \left(\frac{255}{210\,000} \right) \sqrt{\left(\frac{10\,000}{30\,000} \right)} = 2.6 \text{ mm}$$

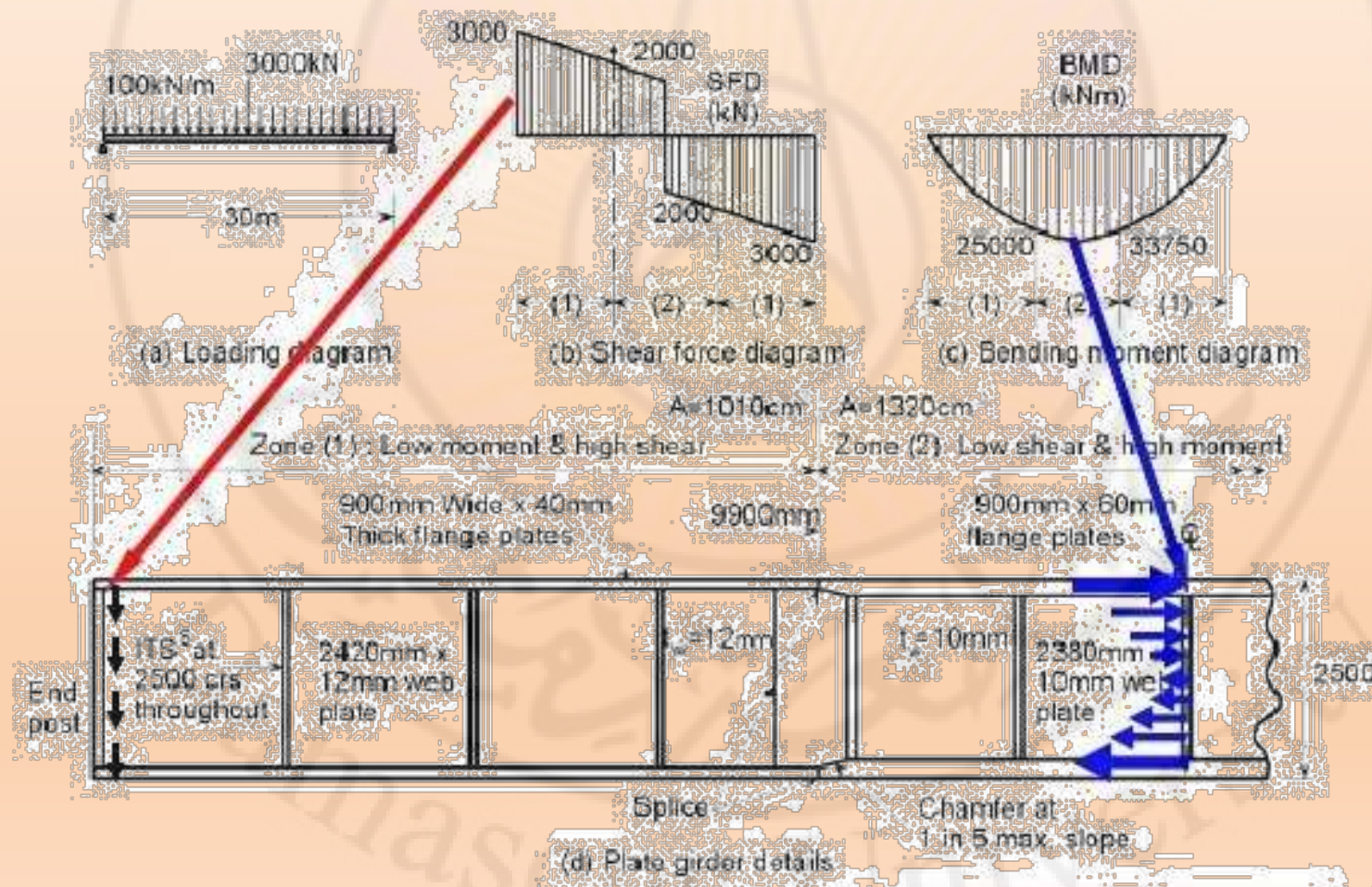
→ OK

4. Basic behaviors of plated structural elements

Bending is mainly taken up by the flanges => tensile/compressive stress => Class 1 section.

Web subjected to **direct bending stress** and **shear stress**

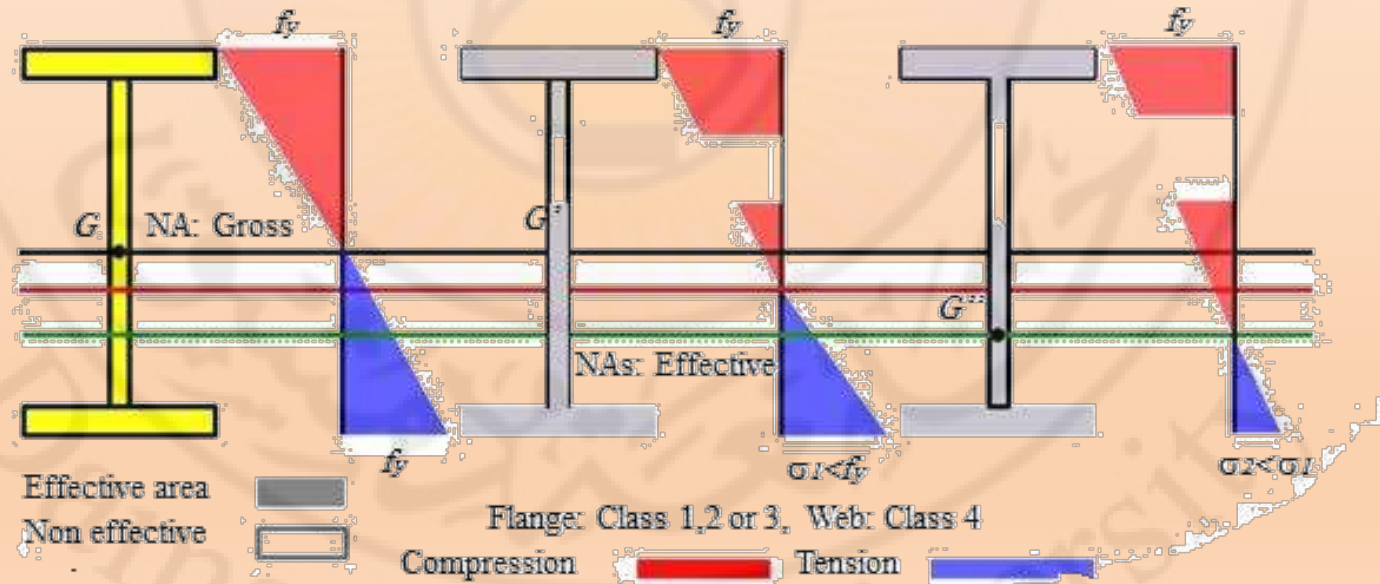
Stiffeners and end posts: To prevent buckling due to bending and shear as well as local failure under patch loads



4. Plate Girder Bending Resistance

A.) Bending resistance to EC3 -BS EN 1993-1-1

Determination of the moment resistance $M_{c,Rd}$ of laterally restrained plate girders depends upon the co-existent level of shear force. Provided that the **applied shear force is less than 50% of the plastic shear resistance $V_{pl,Rd}$** (for webs that are not susceptible to shear buckling) or 50% of the shear buckling resistance (for webs that are susceptible to shear buckling), the section is deemed to be in a state of low shear and the full in-plane bending resistance $M_{c,Rd}$ can be achieved. Cross-section bending resistance should be determined in accordance with Clause 6.2.5 of BS EN 1993-1-1, and depends on the classification of the cross-section,



4. Plate Girder Bending Resistance

A.) Bending resistance to EC3 -BS EN 1993-1-1

For Class 1 and Class 2 cross-sections:

$$M_{c,Rd} = M_{pl,Rd} = \frac{W_{pl} f_y}{\gamma_{M0}}$$

where f_y is the yield strength of the material, W_{pl} is the plastic section modulus and the partial factor for cross-section resistance is equal to 1.

4. Plate Girder Bending Resistance

A.) Bending resistance to EC3 -BS EN 1993-1-1

If the yield strength is not constant through the section (e.g. the flange plate material is of higher strength than the web), the plastic section modulus cannot be used and the cross section resistance should be determined directly from the plastic stress distribution.

For Class 3 cross-sections:

$$M_{c,Rd} = M_{el,Rd} = \frac{W_{el} f_y}{\gamma_{M0}}$$

where W_{el} is the elastic section modulus.

For Class 4 sections, local buckling occurs prior to yielding, resulting in bending resistances below the elastic moment capacity. To account for the loss of effectiveness due to local buckling, an effective section modulus should be determined and used in place of the elastic section modulus.

4. Plate Girder Bending Resistance

A.) Bending resistance to EC3 -BS EN 1993-1-1

The bending resistance of Class 4 cross sections is therefore given by:

$$M_{c,Rd} = \frac{W_{eff} f_y}{\gamma_{M0}}$$

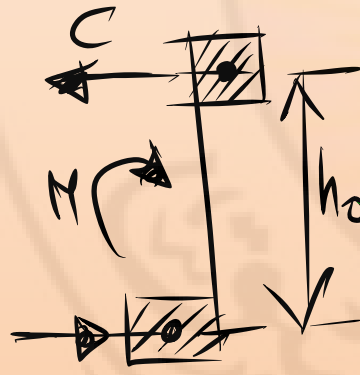
where W_{eff} is the effective section modulus of the cross-section.

4. Plate Girder Bending Resistance

B.) Bending resistance Approximate method (flange area method)

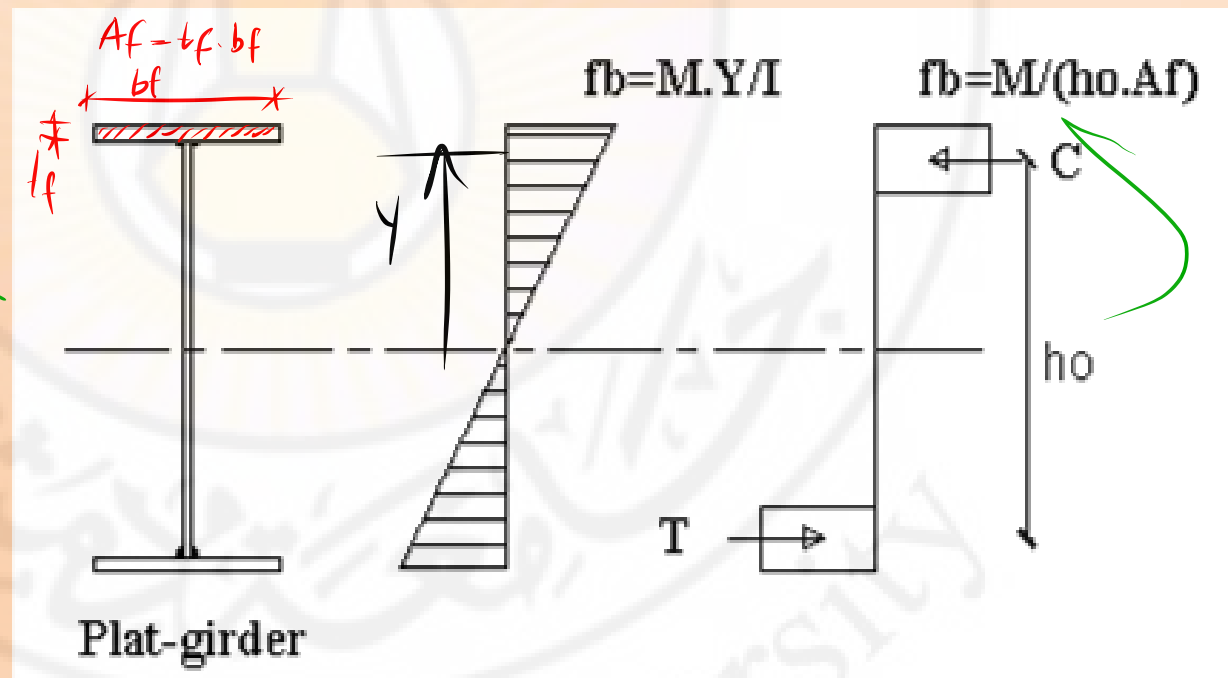
This method is used to obtain a reasonable cross section before we check with the exact method. The flange material is grouped quite closed together and nearly the moment of inertia of the flanges is equal to 85 % of the total moment of inertia of the whole section. If we assumed that the two flanges acts as the upper and lower chord of truss, i.e the flange stress is nearly constant.

Hence, the required flange area:

$$f_b = \frac{M}{I} \cdot y$$


$$C = -T = \frac{M}{h_o} = T \text{ kN (force)}$$

$$f_b = \frac{F}{A_f} = \frac{M}{h_o \cdot A}$$



4. Plate Girder Bending Resistance

B.) Bending resistance Approximate method (flange area method)

The “flanges-only” method is much more convenient in flange portioning since

$$M_{f,Rd} = 2b_f t_f (h_w/2 + t_f/2) f_y = b_f t_f (h_w + t_f) f_y = A_f (h_w + t_f) f_y$$

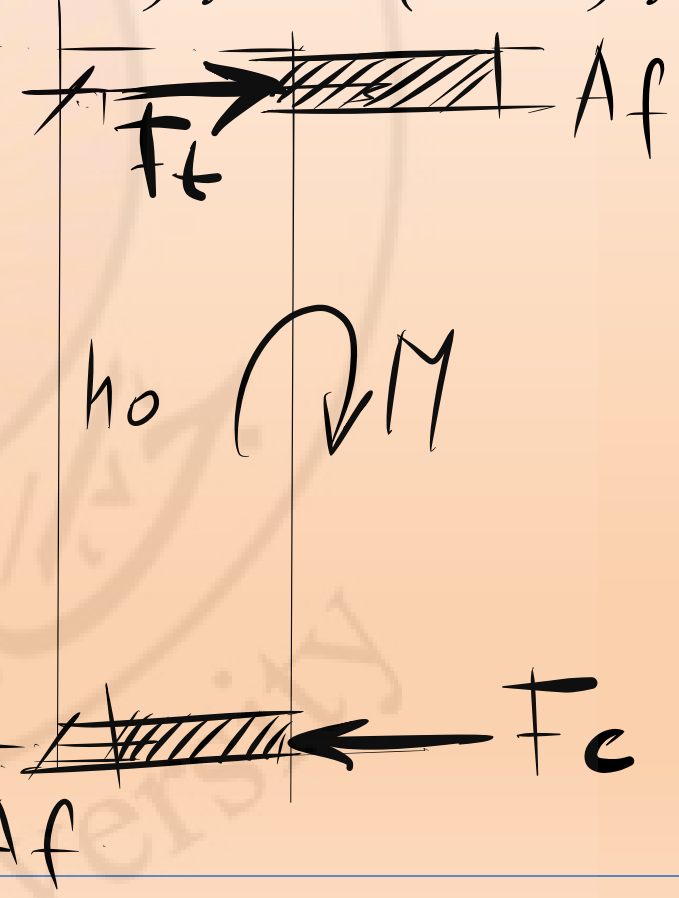
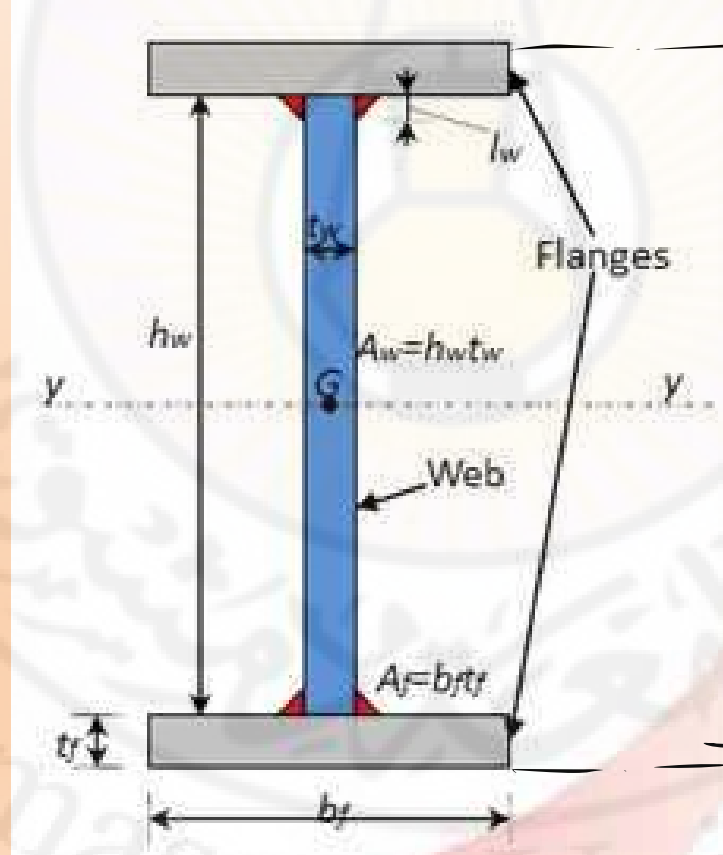
$$M = T_t \cdot h_o = -T_c \cdot h_o$$

$$T_t \cdot h_o \leq f_y \cdot A_f \cdot h_o$$

Maximum strength

$$M_{f,Rd} = f_y \cdot A_f \cdot h_o$$

$$h_o = (h_w/2 + t_f/2) \cdot 2$$



4. Plate Girder Bending Resistance

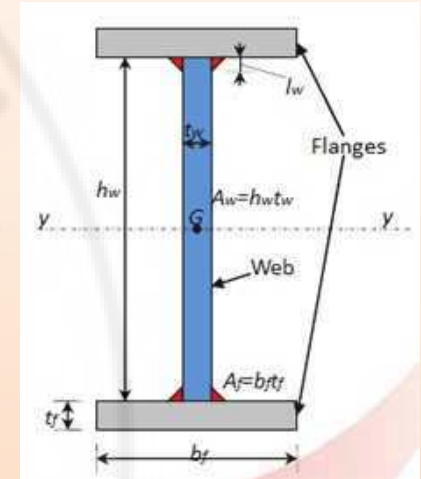
B.) Bending resistance Approximate method (flange area method)

In the “flanges only” method the flanges are full plastic at the ULS

EC3 Part 1-5 does not explicitly allow or disallow the use of “flanges-only” method

Hence, it is safe to use if we could show that for a given section

$$M_{f,Rd} \leq W_{eff} f_y \quad \text{or} \quad A_f (h_w + t_f) \leq W_{eff}$$



4. Plate Girder Bending Resistance

Research done by Lee and Chiew (NTU, CEE) shown that

whenever $A_f(h_w + t_f) \leq W_{eff}$

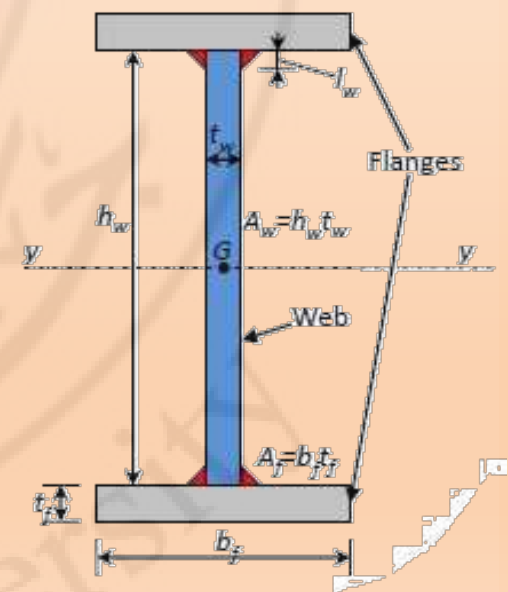
$$0 \leq A_w/A_f \leq 5 \quad 0 \leq t_f/h_w \leq 0.2 \quad h_w/(t_w \varepsilon) \leq 680 \quad \varepsilon = \sqrt{235/f_y}$$

This implies that the flanges only method is conservative within most design range

In fact, the flanges only method is “*too conservative*” in the sense that in many cases

$$R = A_f(h_w + t_f)/W_{eff} \leq 0.75$$

The modified flanges only method is suggested to improve the efficiency in design



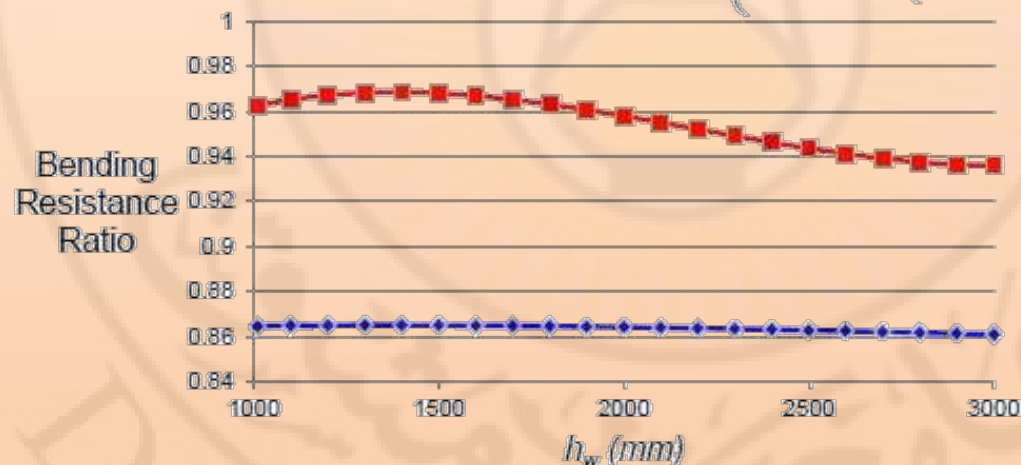
4. Plate Girder Bending Resistance

■ Bending resistance: Modified “Flanges-only” method

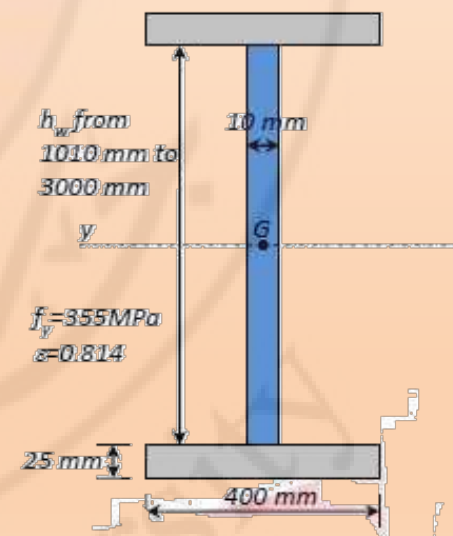
- It is found that the efficiency for bending resistance could be improved by modified the flanges only method so that

$$M_{y,Rd} = W_{eff} f_y$$

$$\approx \bar{M}_{f,Rd} = \bar{R} A_f (h_w + t_f) f_y \quad \bar{R} = 1 + \frac{A_w}{A_f} \left(0.028 + \left(\frac{400 - \frac{h_w}{t_w \varepsilon}}{950} \right)^2 \right)$$



- ◆ Flanges only resistance/ Effective modulus resistance
- Modified flanges only resistance/Effective modulus resistance



4. Plate Girder Bending Resistance

■ Bending resistance: Verification

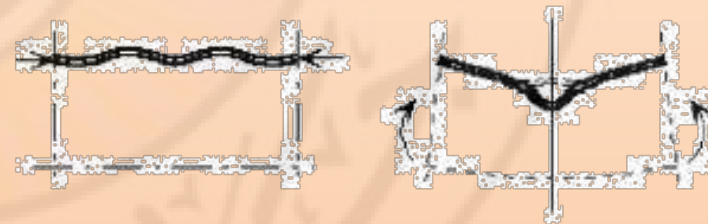
- If no axial force is present, verification should be performed as a normal section check using η_1

$$\eta_1 = \frac{M_{Ed}}{M_{y,Rd}} \leq 1.0$$

where M_{Ed} is the maximum design moment

- To prevent **flange induced buckling**, we also need

$$\frac{h_w}{t_w} \leq k \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc}}}$$



A_{fc} is the effective area of the compression flange. The factor k is taken as 0.3, 0.4 and 0.55 for Class 1, 2, 3 and 4 flanges, respectively

4. Plate Girder Bending Resistance

Bending Resistance

Stresses Verification

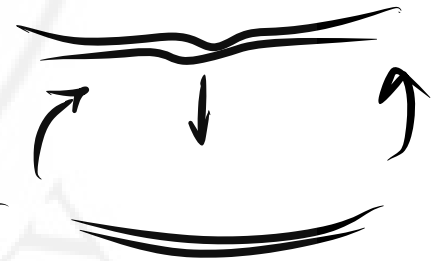
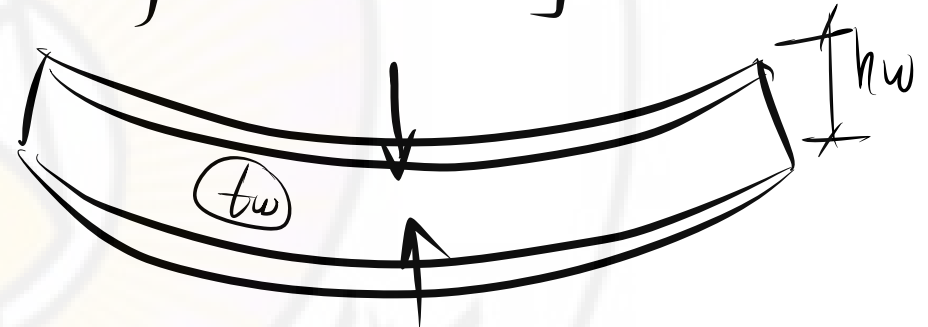
Moment only theory



$$M_{b,Rd} = A_f (h_w + t_f) f_y$$

$$\eta = \frac{M_{Ed}}{M_{b,Rd}} \leq 1$$

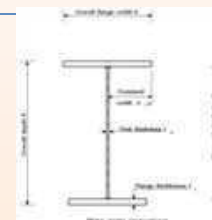
Flang Induced Buckling into Web



$$\frac{h_w}{t_w} \leq k \frac{E}{f_y f} \sqrt{\frac{A_w}{A_{fc}}}$$

4. Plate Girder Bending Resistance

Back to Worked Example 1



Cross-section classification

Initial sizing proposed a plate girder with 750 × 50 mm flanges and 2000 × 15 mm web. Check cross-section classification.

Flange: *changed to be* (750 × 50) {Training Purpose}

For $t_f = 50 \text{ mm}$, $f_{yf} = 255 \text{ N/mm}^2$

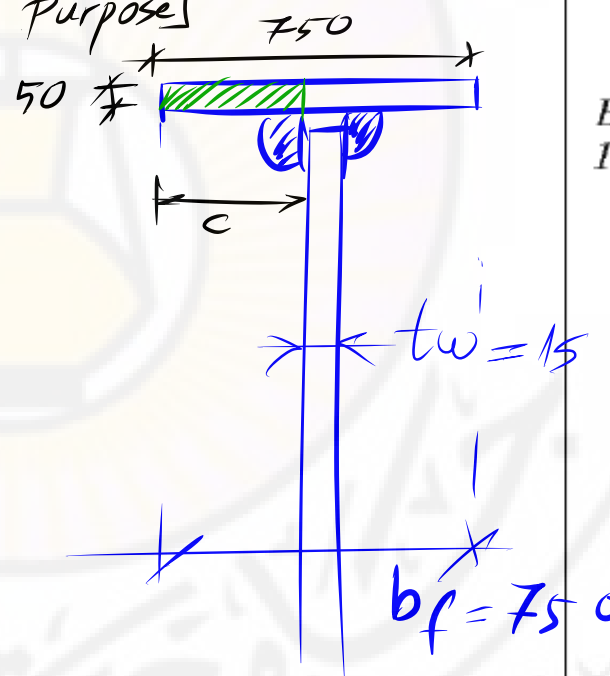
$$\varepsilon = \sqrt{\frac{235}{f_{yf}}} = \sqrt{\frac{235}{255}} = 0.96$$

Ignoring weld size in determination of plate width:

$$c_f = \frac{750 - 15}{2} = 367.5 \text{ mm}$$

$$\frac{c_f}{t_f} = \frac{367.5}{50} = 7.35$$

Limiting slenderness for Class 1 flange is $9\varepsilon = 8.64 > 7.35 \rightarrow$ Flange is class 1

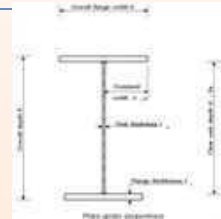


BS EN
10025-2

BS EN
1993-1-1
Table 5.2

4. Plate Girder Bending Resistance

Back to Worked Example 1



Dimensions of web and flanges

Assume stiffener spacing $a > h_w$

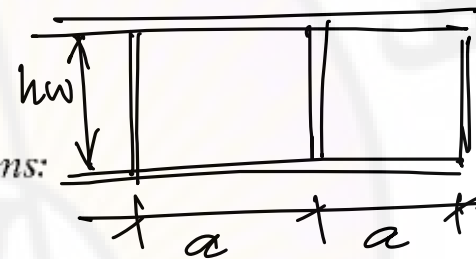
Minimum web thickness to avoid serviceability problems:

$$t_w = 15 \geq \frac{h_w}{250} = \frac{2000}{250} = 8.0 \text{ mm} \quad \text{OK}$$

To avoid the flanges buckling into the web:

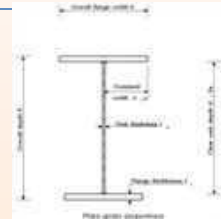
$$t_w = 15 \geq \left(\frac{h_w}{k} \right) \left(\frac{f_{yf}}{E} \right) \sqrt{\frac{A_{fc}}{A_w}} = \left(\frac{2000}{0.55} \right) \left(\frac{255}{210000} \right) \sqrt{\frac{750 \times 50}{2000 \times 15}} = 4.94 \text{ mm} \quad \text{OK}$$

BS EN
1993-1-5
Cl 8(1)



4. Plate Girder Bending Resistance

Back to : Worked Example 1



Bending moment resistance

For sections with class 1-3 flanges but a class 4 web, it may be assumed that the bending moment is resisted by the flanges alone while the web is designed to carry the shear only.

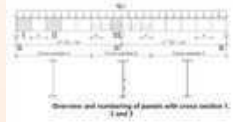
The bending resistance of the flanges alone $M_{f,Rd} = A_f f_{yf} \times (h_w + t_f)$

$$M_{f,Rd} = (750 \times 50 \times 255) \times (2000 + 50) / 10^6 = 19603 \text{ kNm}$$

$$M_{f,Rd} = 19603 > M_{max} = 18909 \text{ kNm} \quad \text{OK}$$

4. Plate Girder Bending Resistance

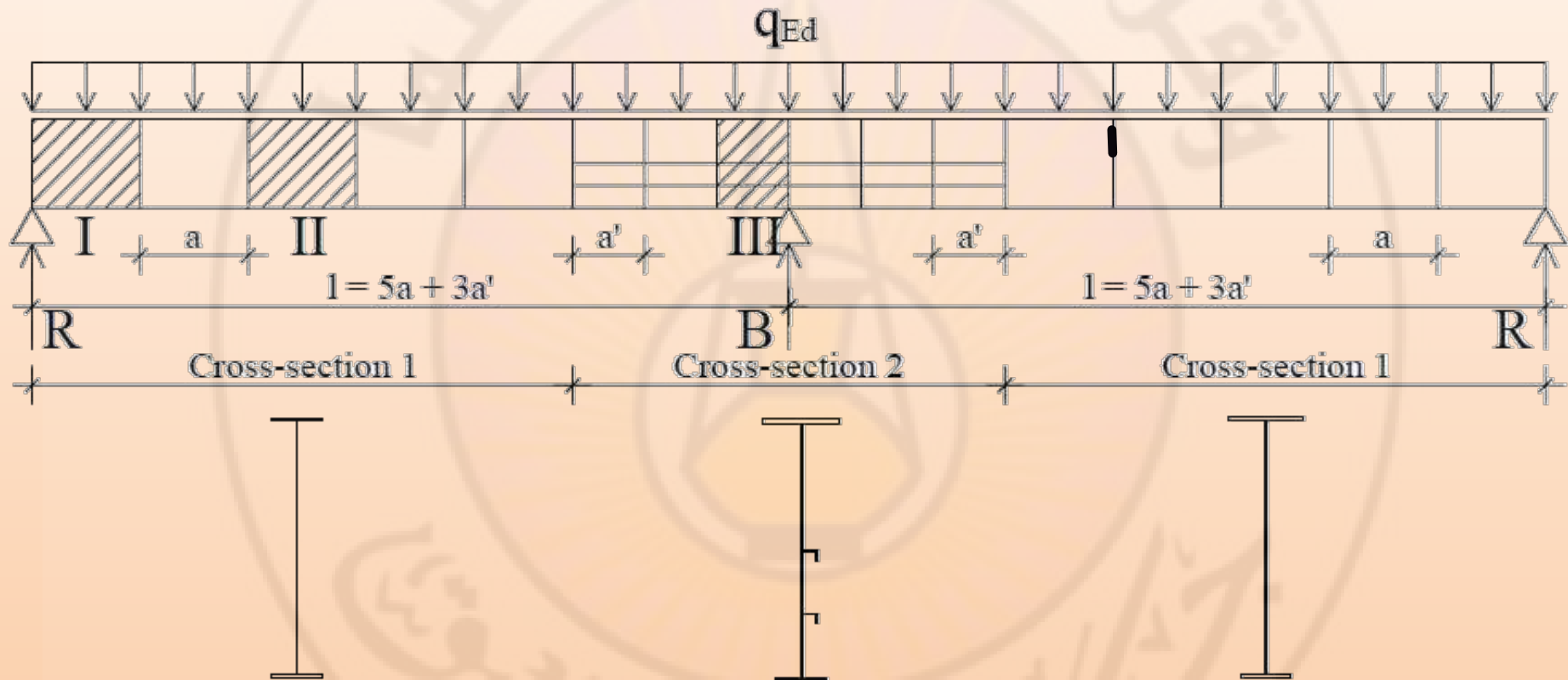
C.) Worked Example 2



A two span continuous plate girder is selected in this numerical example to enable us to illustrate the design rules for the panels at the exterior and interior support as well as in the mid span. The example is not intended to show an optimal girder design but to illustrate the application of several design rules. The design loading q_{Ed} includes self-weight of the girder and all other relevant permanent and variable loads.

4. Plate Girder Bending Resistance

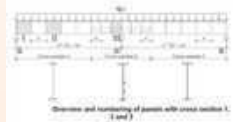
C.) Worked Example 2



Overview and numbering of panels with cross section 1, 2 and 3

4. Plate Girder Bending Resistance

C.) Worked Example 2



$$S\ 235 \Rightarrow f_y = 235\text{ MPa}$$

$$\varepsilon = \sqrt{\frac{235}{f_y [\text{N/mm}^2]}} = \sqrt{\frac{235}{235}} = 1$$

$$\gamma_{M0} = 1.0$$

$$\gamma_{M1} = 1.0$$

$$a = 3\text{ m}$$

$$a' = 2\text{ m}$$

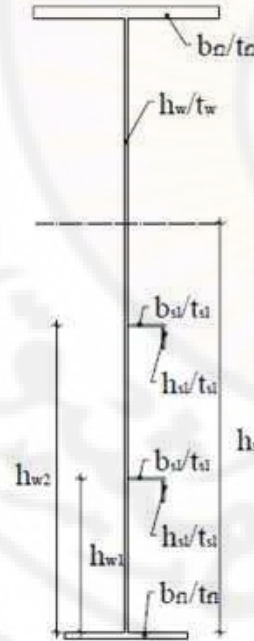
$$l = 21\text{ m}$$

$$q_{Ed} = 85\text{ kN/m}$$

Cross-section 1



Cross-section 2



$$b_{f1} / t_{f1} = 400 / 20\text{ mm}$$

$$b_{f2} / t_{f2} = 600 / 40\text{ mm}$$

$$h_w / t_w = 2000 / 8\text{ mm}$$

$$b_{sl} / t_{sl} = 120 / 8\text{ mm}$$

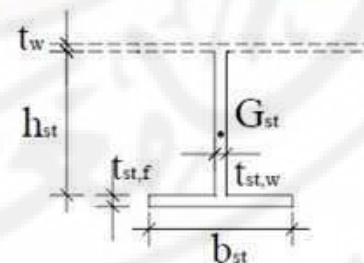
$$h_{sl} / t_{sl} = 80 / 8\text{ mm}$$

$$h_{st} / b_{st} / t_{st,w} / t_{st,f} = 180 / 180 / 10 / 20\text{ mm}$$

$$h_{w1} = 500\text{ mm}$$

$$h_{w2} = 1000\text{ mm}$$

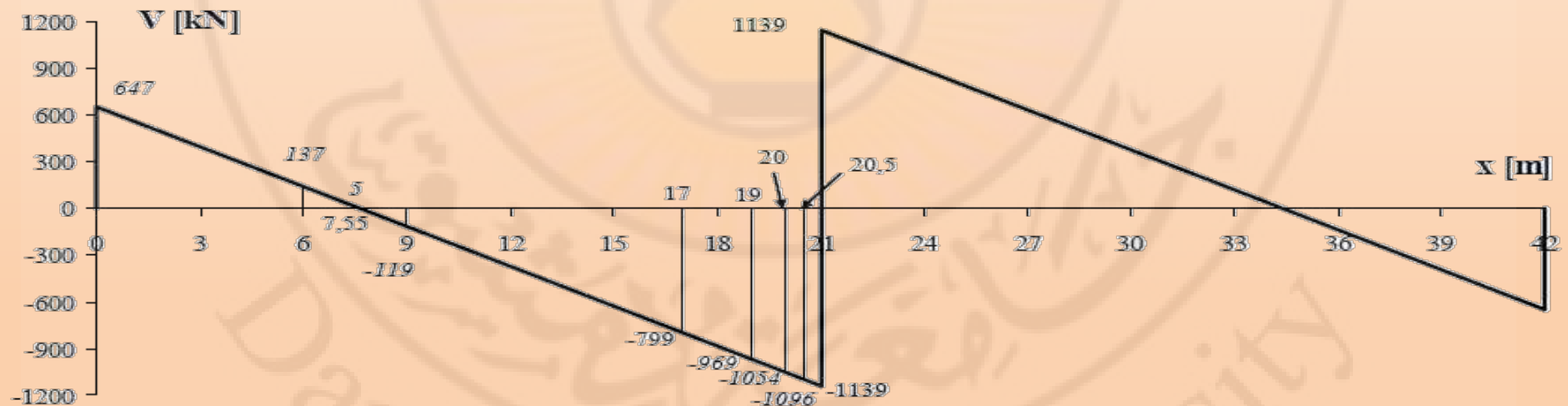
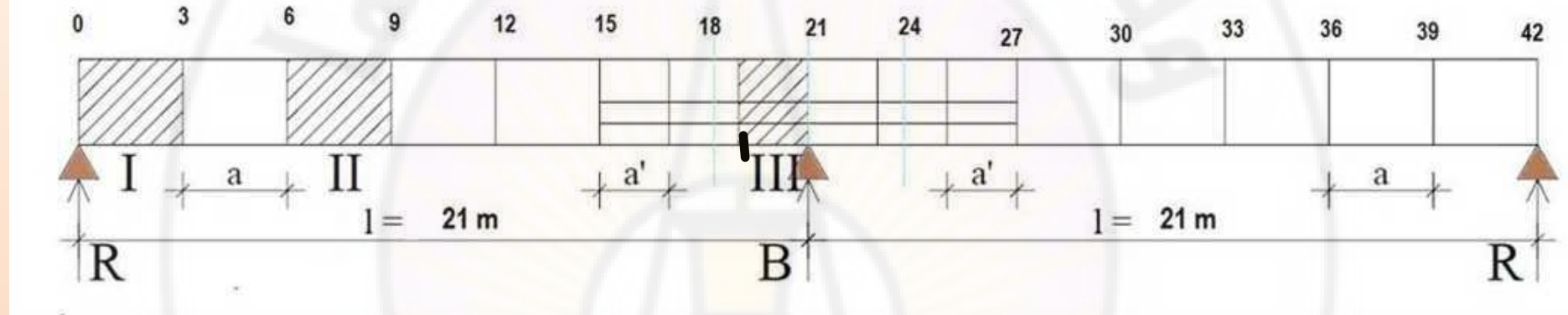
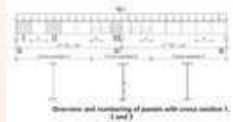
Transverse stiffener



Cross sections

4. Plate Girder Bending Resistance

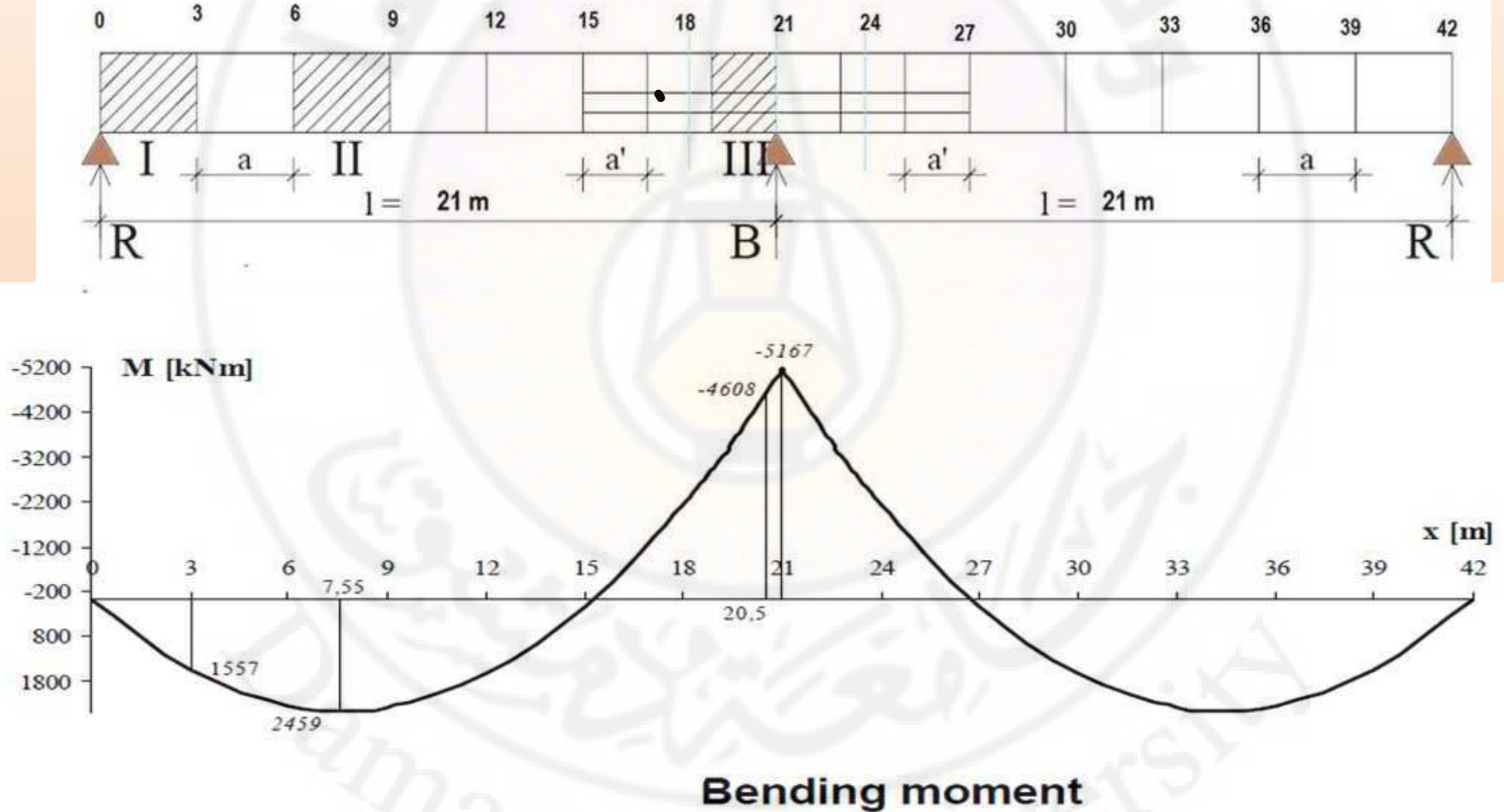
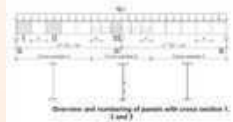
C.) Worked Example 2



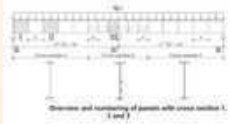
Shear force

4. Plate Girder Bending Resistance

C.) Worked Example 2



4. Plate Girder Bending Resistance



C.) Worked Example 2

1- Bending Moment Verification

1- Field 0 -3.....maxMed = 1557 KN.m

$$M_{f,Rd} = (h_w + t_{f1}) A_{f,min} \frac{f_y}{\gamma_{M0}} =$$

$$= (2,00 + 0,02) \cdot (0,40 \cdot 0,02) \cdot \frac{235 \cdot 10^6}{1,0} = 3798 \cdot 10^3 \text{ Nm}$$

$$M_{Ed} = 1557 \cdot 10^3 \text{ Nm} \leq M_{f,Rd} = 3798 \cdot 10^3 \text{ Nm} \quad \checkmark$$

The direct stresses may thus be transferred by the flanges only.

The End
of
Part I

Damascus University



Damascus University

Faculty Of Civil Engineering

Lectures, 4th Year 2023

Dr. – Eng. Moaid Subh P L A T E G I R D E R

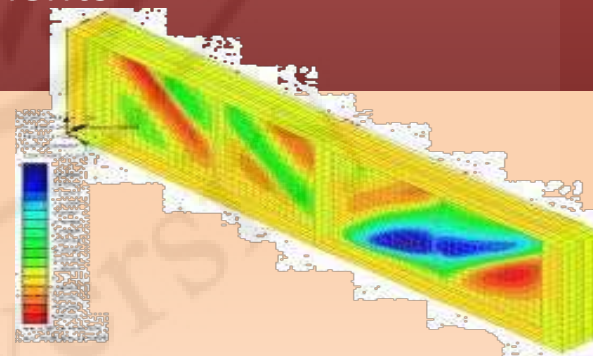
Part 2

To Eurocode 3

EN 1993-1-5 (2006)

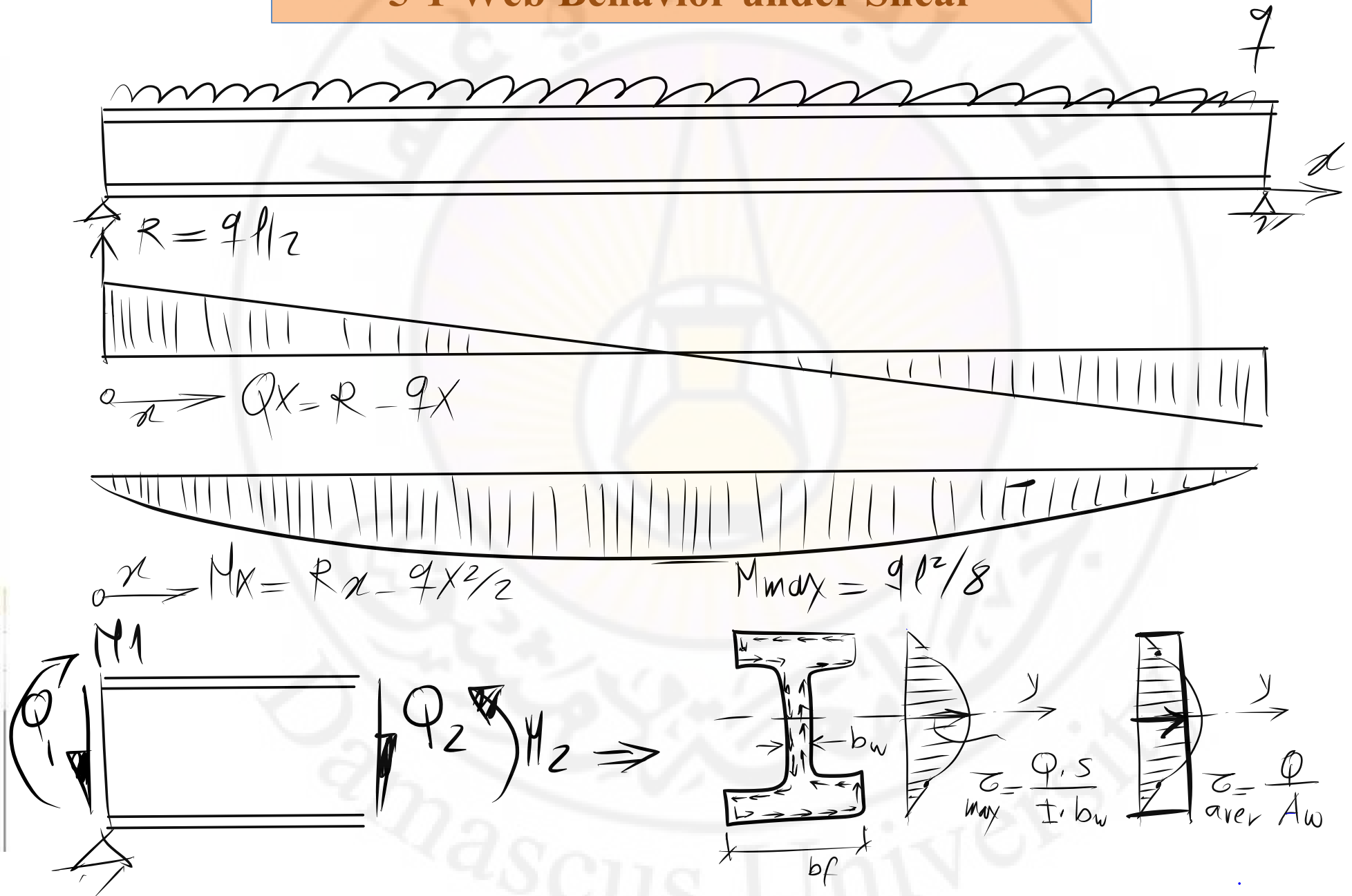
Design of steel structures - Part 1-5

General rules Plated structural elements



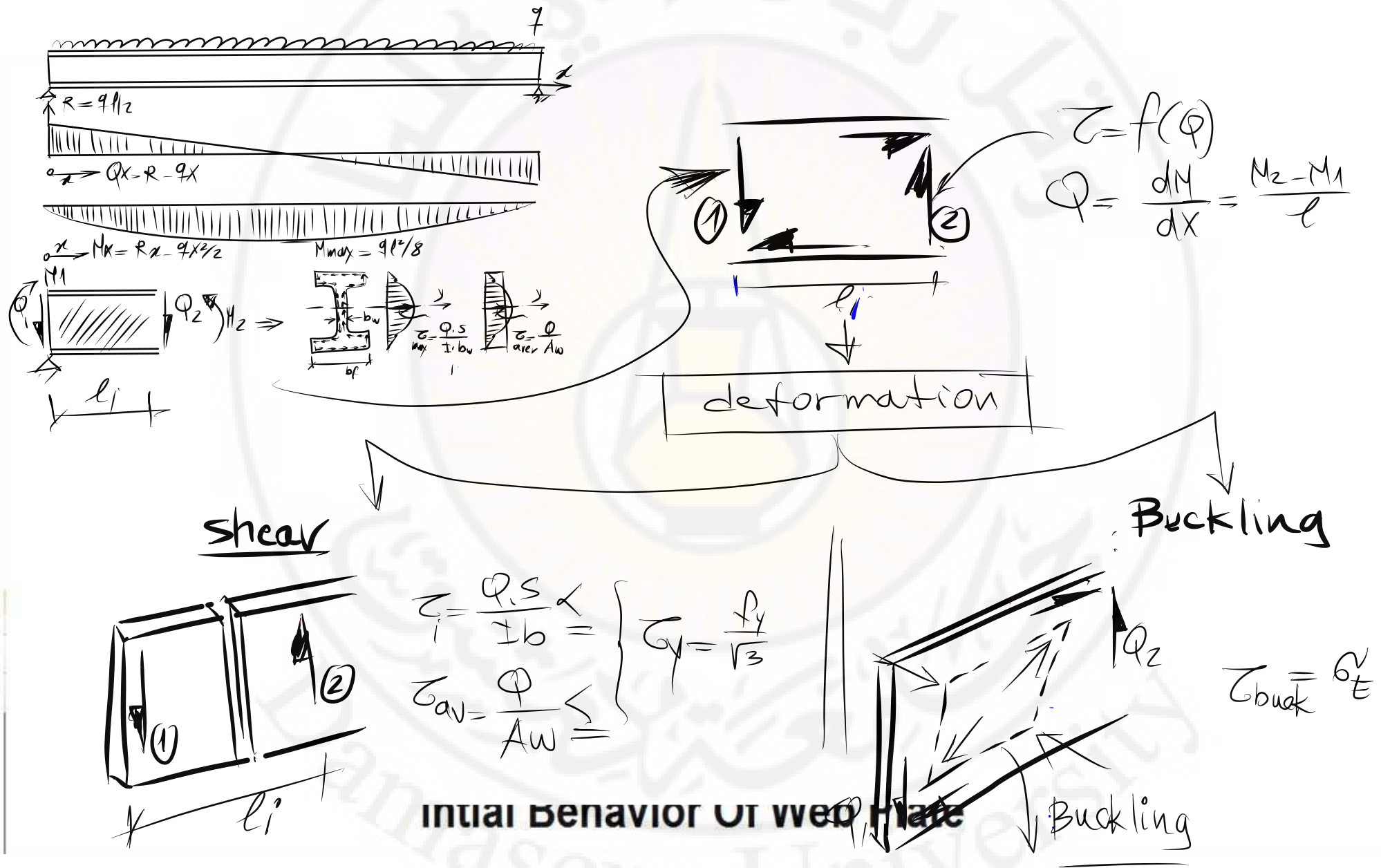
5. Plate Girder Shear Resistance

5-1 Web Behavior under Shear



5. Plate Girder Shear Resistance

5.1 Web Behavior under Shear



5. Plate Girder Shear Resistance

5-1 Web Behavior under Shear

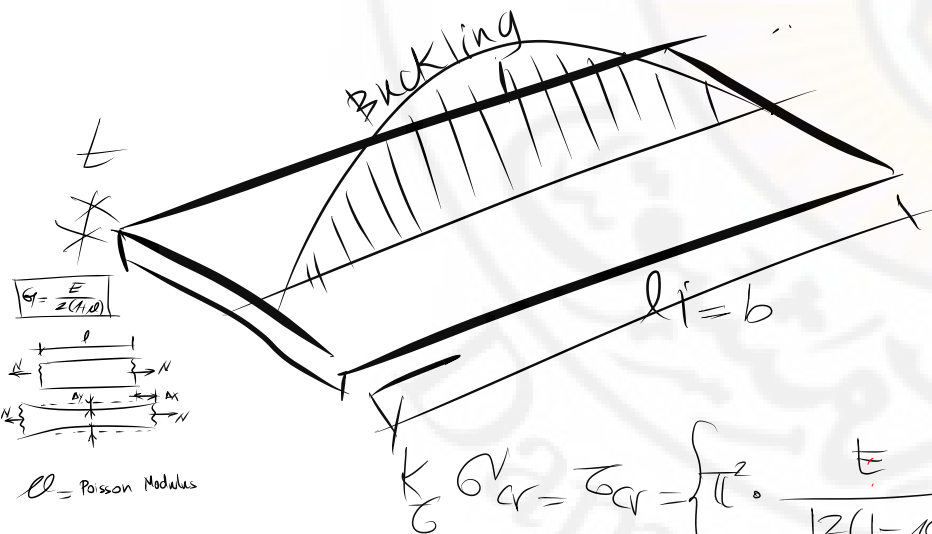
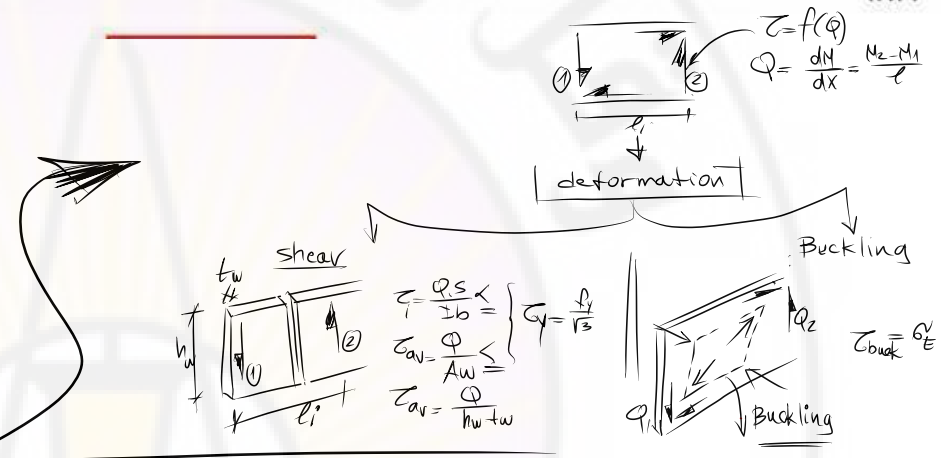
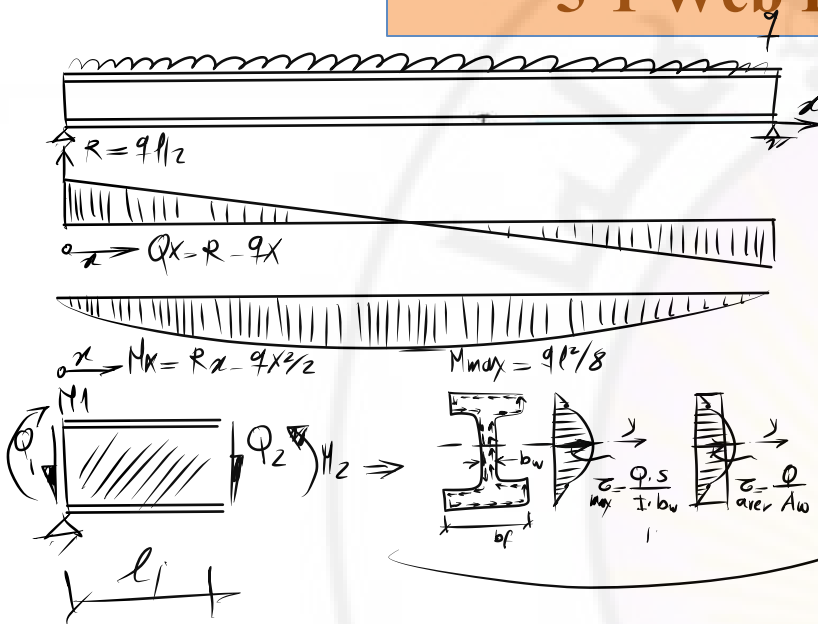
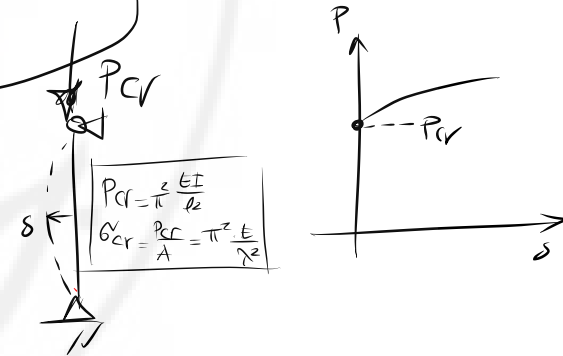


Plate Buckling
 Buckling in Column

Buckling in Plate



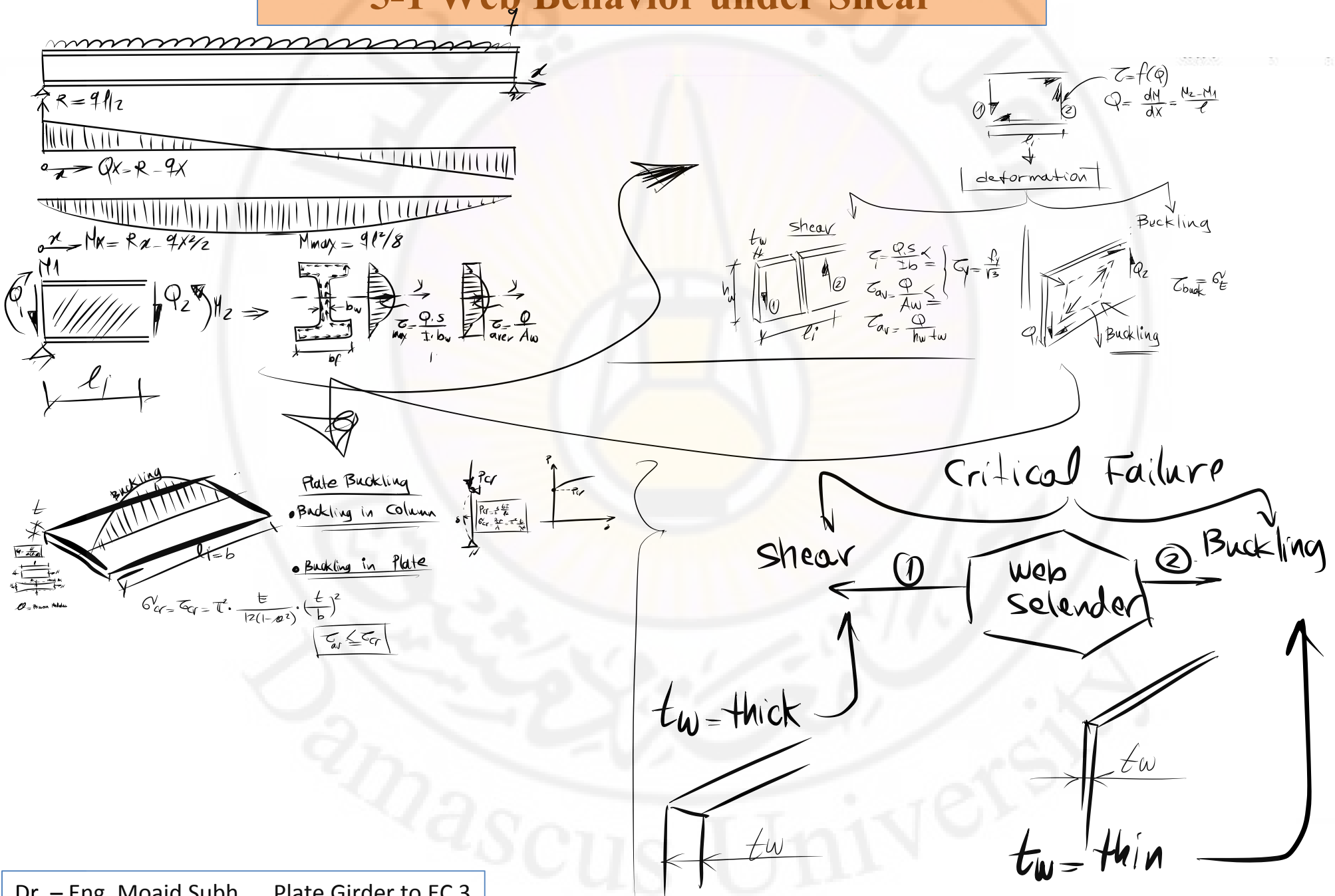
$$\frac{G}{12} \sigma_{cr} = \tau_{cr} = \left\{ \pi^2 \cdot \frac{E}{12(1-\nu^2)} \cdot \left(\frac{t}{b} \right)^2 \right\} * K_\sigma$$

$$\tau_{av} \leq \tau_{cr}$$

$$\tau_{cr} = \pi^2 \frac{E}{(1-\nu^2)} \left(\frac{t}{b} \right)^2$$

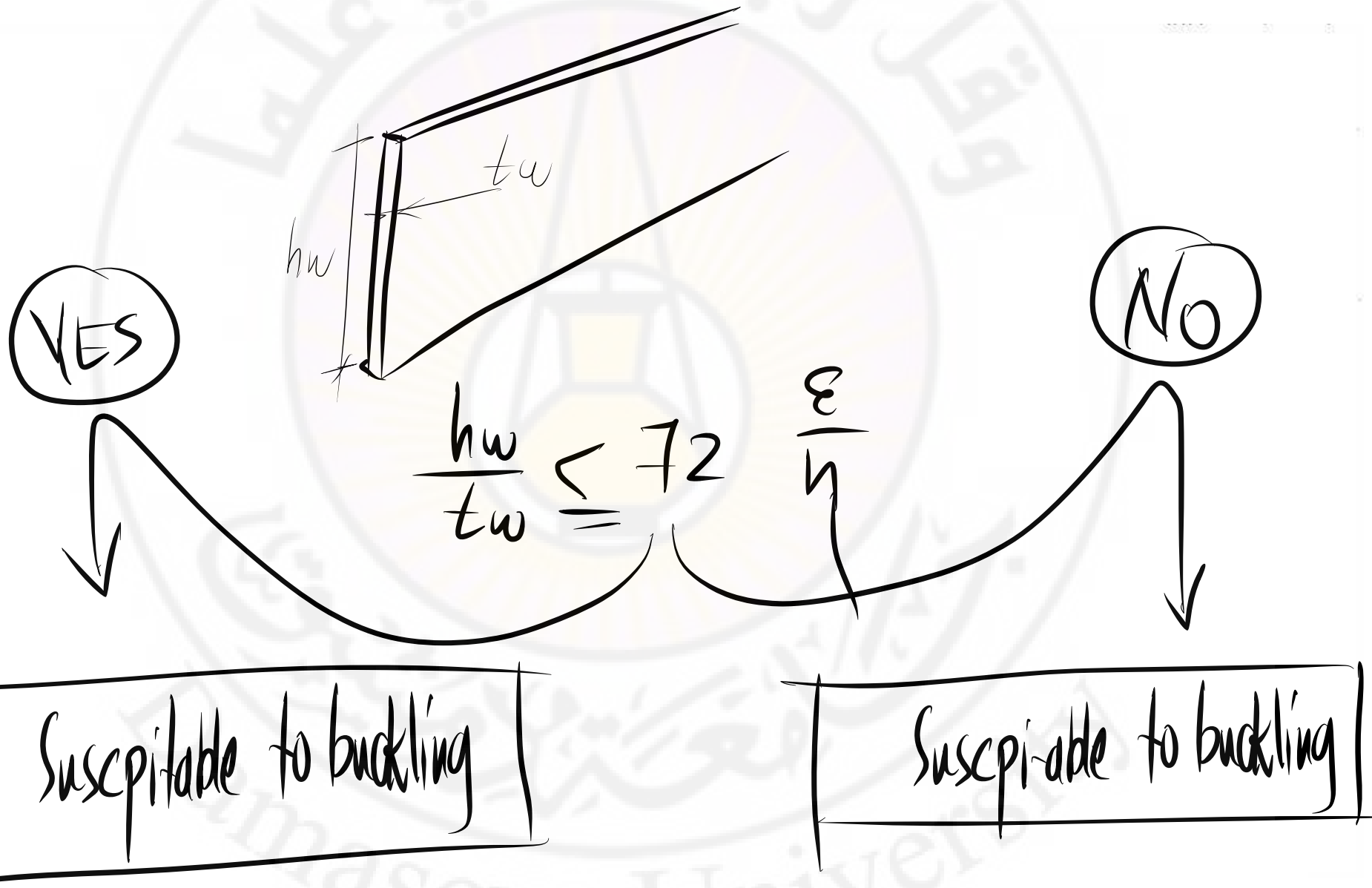
5. Plate Girder Shear Resistance

5-1 Web Behavior under Shear



5. Plate Girder Shear Resistance

5-1 Web Behavior under Shear



5. Plate Girder Shear Resistance

5-2 Shear Introduction

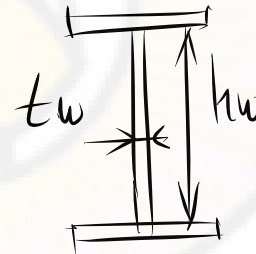
Shear resistance

A.) *Web not susceptible to shear buckling*

Susceptibility to shear buckling depends on the slenderness (height to thickness ratio) of the web. According to Clause 6.2.6(6) of BS EN 1993-1-1, for unstiffened webs, provided that:

$$\frac{h_w}{t_w} \leq 72 \frac{\epsilon}{\eta}$$

Web is class (1)



Class			
1	2	3	
$\frac{h}{t_w}$	72ϵ	83ϵ	124ϵ

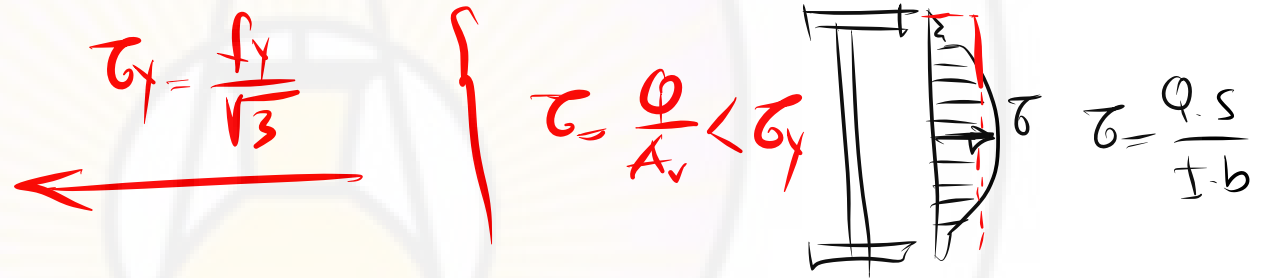
where h_w is the height of the web taken as the clear distance between the flanges, t_w is the thickness of the web, $\epsilon = (235/f_{yw})^{0.5}$ and $\eta = 1.0$ (specified for all steels in Clause NA.2.4 of the UK National Annex to BS EN 1993-1-5), the web is not susceptible to shear buckling. In such instances, the shear resistance may

5. Plate Girder Shear Resistance

5-2 Shear Introduction

be taken as the plastic shear resistance $V_{pl,Rd}$, as given in Clause 6.2.6(2) of BS EN 1993-1-1:

$$V_{pl,Rd} = \frac{A_v f_{yw}}{\sqrt{3} \gamma_{M0}}$$



where A_v is the shear area, defined in Clause 6.2.6(3) of BS EN 1993-1-1 as:

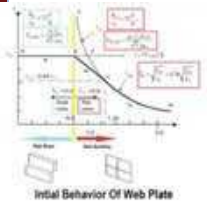
$A_v = h_w t_w$ for welded I-sections loaded parallel to the web

and

$A_v = A - h_w t_w$ for welded I-sections loaded parallel to the flanges.

5. Plate Girder Shear Resistance

5-2 Shear Introduction



Extract from table of section classification limits (BS EN 1993-1-1)

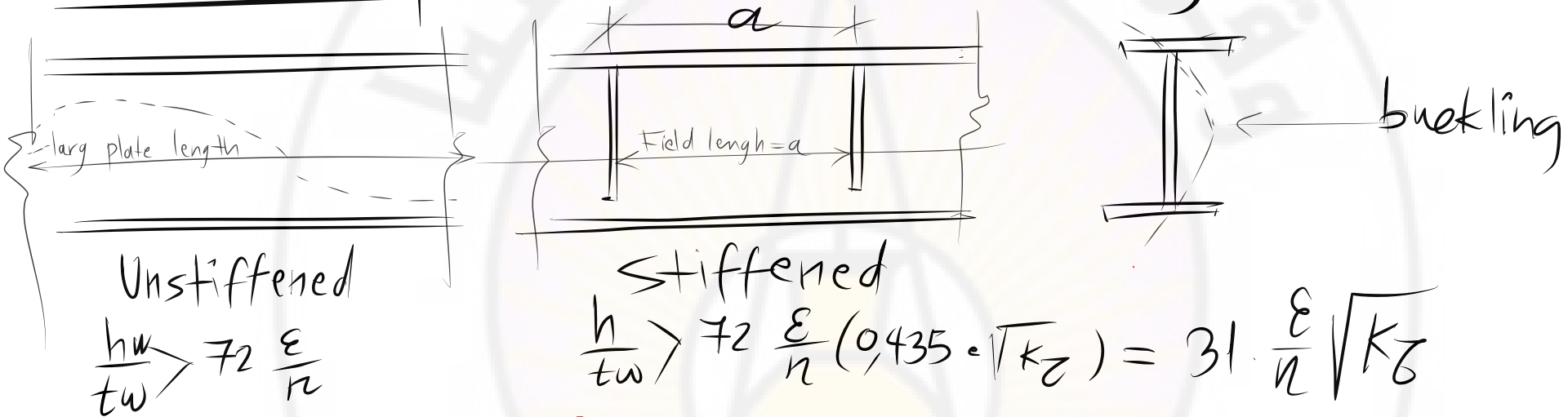
Type of element and loading	Class of section		
	Class 1 (λ_{p1})	Class 2 (λ_{p2})	Class 3 (λ_{p3})
Outstand flange in pure compression	$c_f/t_f \leq 9\epsilon$	$c_f/t_f \leq 10\epsilon$	$c_f/t_f \leq 14\epsilon$
Internal element in pure compression	$c_f/t_f \leq 33\epsilon$	$c_f/t_f \leq 38\epsilon$	$c_f/t_f \leq 42\epsilon$
Web with neutral axis at mid-depth	$c_w/t_w \leq 72\epsilon$	$c_w/t_w \leq 83\epsilon$	$c_w/t_w \leq 124\epsilon$
Web, generally	when $\alpha > 0.5$: $c_w/t_w \leq 396\epsilon/(13\alpha-1)$	when $\alpha > 0.5$: $c_w/t_w \leq 456\epsilon/(13\alpha-1)$	when $\psi > -1$: $c_w/t_w \leq 42\epsilon/(0.67+0.33\psi)$
	when $\alpha \leq 0.5$: $c_w/t_w \leq 36\epsilon/\alpha$	when $\alpha \leq 0.5$: $c_w/t_w \leq 41.5\epsilon/\alpha$	when $\psi \leq -1$: $c_w/t_w \leq 62\epsilon(1-\psi)(-\psi)^{0.5}$

where $\psi = \sigma_2/\sigma_1$ is the ratio of end stresses in the case of an elastic stress distribution, and α is the compressed portion of the web in the case of a plastic stress distribution.

5. Plate Girder Shear Resistance

5-2 Shear Introduction

Web susceptible to shear buckling



According to Clause 5.1(2) of BS EN 1993-1-5, unstiffened webs become susceptible to shear buckling when

$$\frac{h_w}{t_w} > 72 \frac{\epsilon}{\eta} \quad \left\{ \text{classes: II, III, IV} \right.$$

while stiffened webs become susceptible to shear buckling when

$$\frac{h_w}{t_w} > 31 \frac{\epsilon}{\eta} \sqrt{k_\tau} \quad \left\{ \text{classes: II, III, IV} \right.$$

5. Plate Girder Shear Resistance

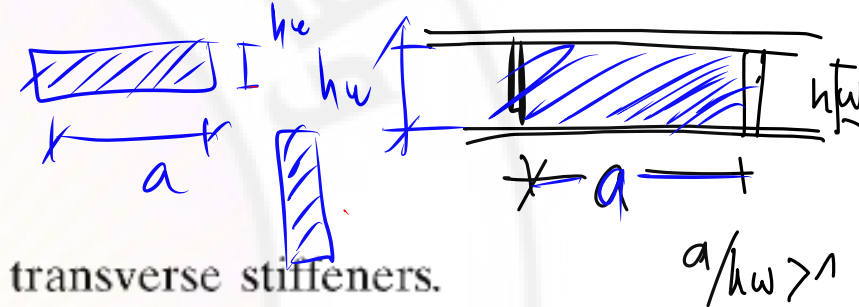
5-2 Shear Introduction

where k_τ is the shear buckling coefficient, defined in Annex A.3 of BS EN 1993-1-5 for webs without longitudinal stiffeners as:

$$k_\tau = 5.34 + 4.00(h_w/a)^2 \text{ for } a/h_w \geq 1$$

$$k_\tau = 4.00 + 5.34(h_w/a)^2 \text{ for } a/h_w < 1$$

in which a is the distance between centrelines of transverse stiffeners.



When a web is susceptible to shear buckling, transverse stiffeners should be specified at the supports to prevent the web from buckling as a column (see Clause 5.1(2) of BS EN 1993-1-5) and shear buckling resistance $V_{b,Rd}$ should be checked in accordance with Clause 5.2 of BS EN 1993-1-5. Shear buckling resistance comprises a dominant contribution from the web $V_{bw,Rd}$, and an additional contribution from the flanges $V_{bf,Rd}$, provided they are not fully utilised in resisting co-existent bending. The shear buckling resistance may not exceed the plastic resistance of the section. The design approach in BS EN 1993-1-5 is based on the rotated stress field method. For slender webs, this method allows the utilisation of considerable post-buckling strength (i.e. load-carrying capacity beyond elastic buckling).

5. Plate Girder Shear Resistance

5-2 Shear Introduction

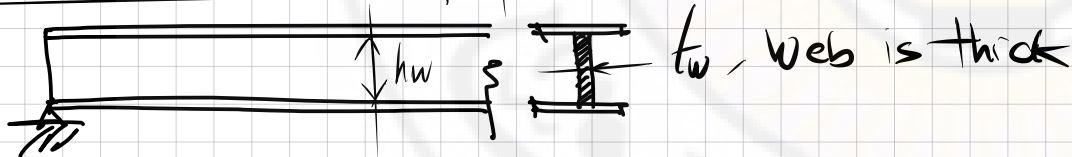
Shear Resistance

A.) Web not susceptible to shear buckling

Susceptibility to shear buckling depends on the slenderness of the web. According to Clause 6.2.6(7) of BS EN 1993-1-1

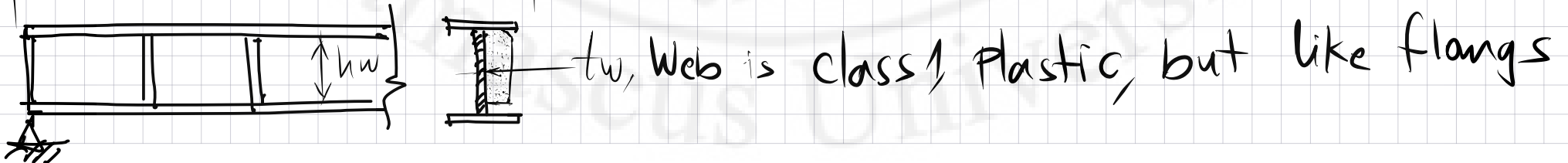
For Unstiffened Webs provided that:

$$\frac{h_w}{t_w} \leq 72 \frac{\epsilon}{\eta}, \text{ Web is class 1, Plastic}$$



For Stiffened Webs, provided that:

$$\frac{h_w}{t_w} \leq 31 \frac{\epsilon}{\eta} \sqrt{k_\sigma}, \text{ Web is class 1, treated as internal elements}$$



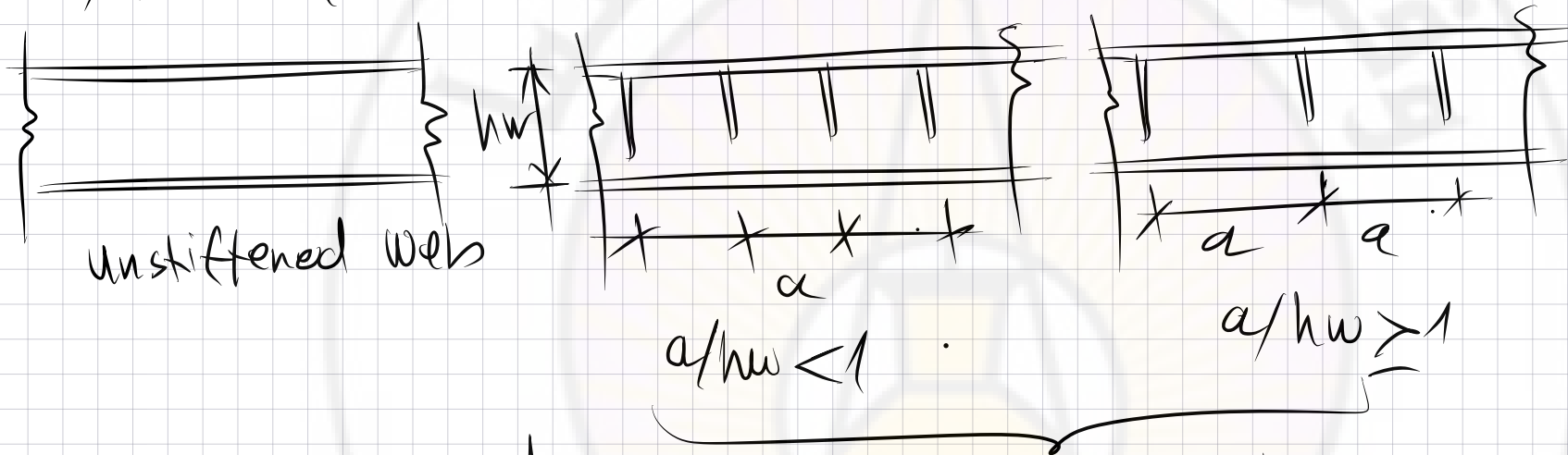
English version

Eurocode 3 : Design of steel structures

Part 1-1 : General structural rules

Example (2-1)

Explore at which height to width (a/hw) of the plate girder web, the stiffeners would be useless.



susceptible
to buckling

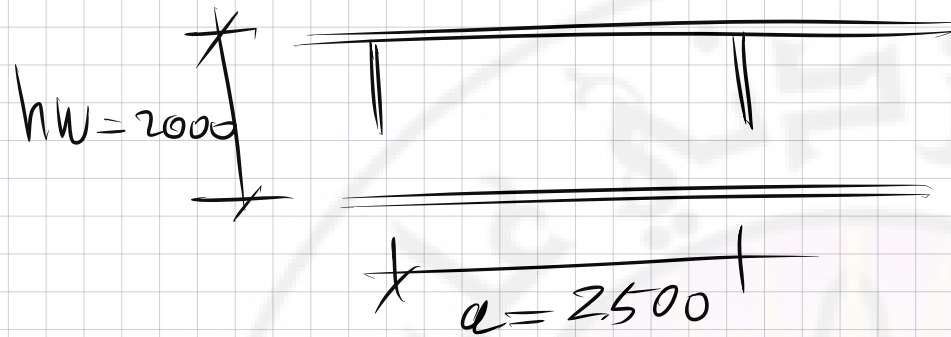
$$\frac{hw}{t} \geq 42 \frac{\epsilon}{\eta}$$

Not susceptible
to buckling

$$\frac{hw}{t} \geq 31 \frac{\epsilon}{\eta} \sqrt{K_G}$$

$$a/hw \geq 1 \rightarrow K_G = 5.34 + 4 \left(\frac{hw}{a} \right)^2$$

$$a/hw < 1 \rightarrow K_G = 4 + 5.34 \left(\frac{hw}{a} \right)^2$$



$$a/h_w = 2500/2000 = 1.25 > 1$$

$$K_\sigma = 5.34 + 4(h_w/a)^2 = 5.34 + 4\left(\frac{2000}{2500}\right)^2 = 17.9$$

$$\sqrt{K_\sigma} = \sqrt{17.9} = 4.23 \rightarrow \text{the susceptible condition will be}$$

$$\frac{h_w}{t} > 31 \frac{\sigma}{\eta} \sqrt{K_\sigma} = 87.11 \frac{\sigma}{\eta}$$

t	h_w/t	susceptible to buckling $> 72 \frac{\sigma}{\eta}$	Not susceptible to buckling $> 87.11 \frac{\sigma}{\eta}$ 60
20	100	✓	✓
25	80	✓	✗
30	66	✗	✗

Exploring the rate (a/hw)

$$\boxed{K_0 < 5.4} \rightarrow$$

$$31 \sqrt{K_0} \left(\frac{\varepsilon}{u} \right) < 42 \left(\frac{\varepsilon}{u} \right)$$

$$K_0 = \left(\frac{42}{31} \right)^2 < 5.4$$

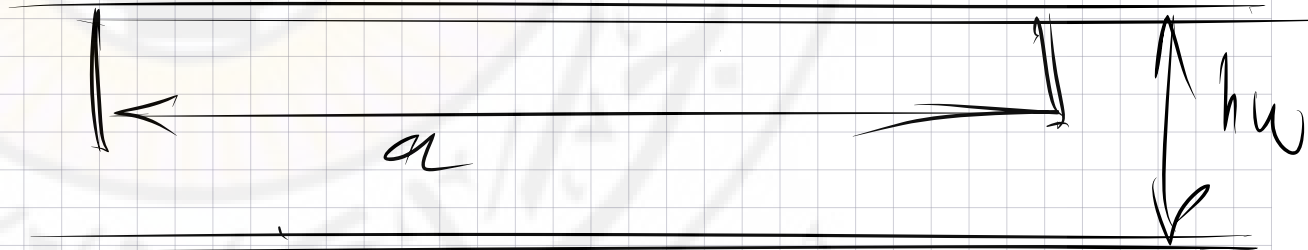
$$\boxed{\text{Case } (a/hw > 1)} \rightarrow$$

$$5.34 + 4 \left(\frac{hw}{a} \right)^2 < 5.4 \rightarrow \frac{hw}{a} < \sqrt{(5.4 - 5.34) \cdot \frac{1}{4}} < 0.123 \left(\frac{a}{hw} \approx 8 \right) > 1 \quad \checkmark$$

$$\boxed{\text{Case } (a/hw < 1)} \rightarrow$$

$$4.00 + 5.34 \left(\frac{hw}{a} \right)^2 < 5.4 \rightarrow \frac{hw}{a} < \sqrt{(5.4 - 4) \cdot \frac{1}{5.34}} < 0.512 \left(\frac{a}{hw} = 1.95 \right) > 1 \quad \times$$

$$a/hw \approx > 8 \rightarrow$$



Field is too wide

stiffeners will be useless

Example

$$h_w = 2000$$

$$a = 16400$$

$$\frac{a}{h_w} = \frac{16400}{2000} = 8.2 \rightarrow \frac{h_w}{a} = \frac{2000}{16400} = 0.122$$

$$\rightarrow K_G = 5.34 + 4\left(\frac{h_w}{a}\right)^2 = 5.83 \rightarrow \sqrt{K_G} = 2.41$$

unstiffened

$$< 72 \frac{E}{f}$$

stiffened

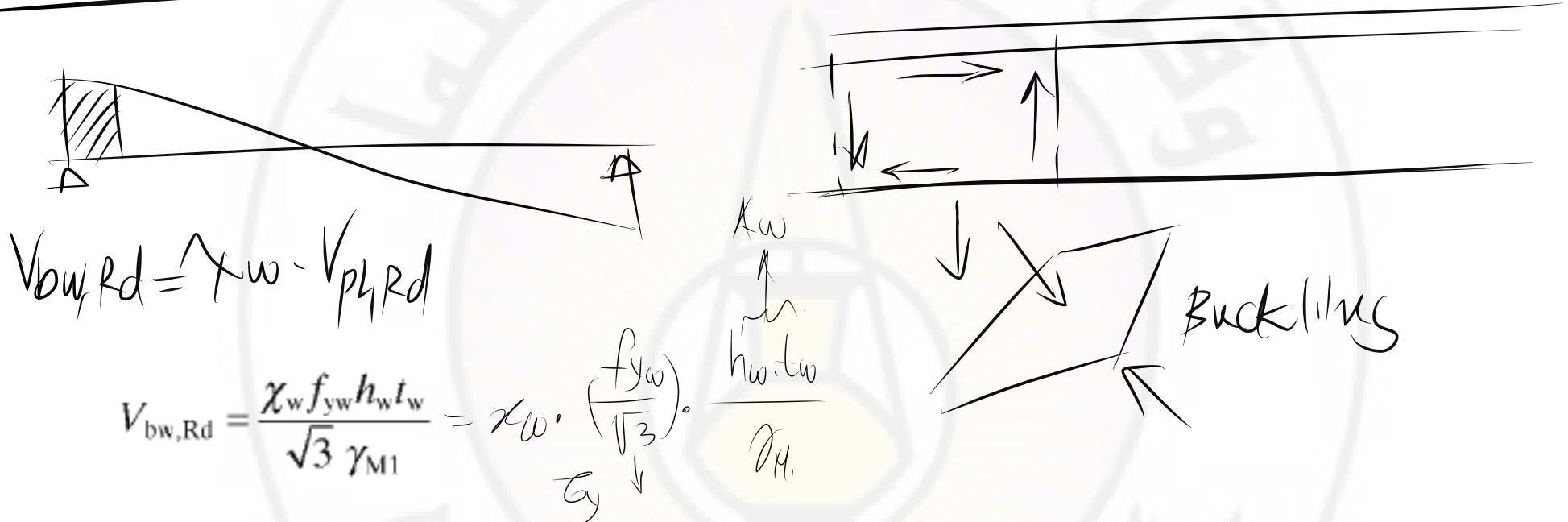
$$< 31 \frac{E}{f} \sqrt{K_G}$$

$$< 74.8$$

use of stiffeners
will be useless

5. Plate Girder Shear Resistance

B. Web is susceptible to buckling



The diagram shows a beam with a linear shear stress distribution across its depth. A detail of the web is shown with shear stress τ_{yw} acting horizontally. The web is divided into two parts: a top part of height h_w and a bottom part of height h_w . The reduction factor χ_w is indicated. The web is shown buckling, with the word "Buckling" written next to it.

$$V_{bw,Rd} = \chi_w \cdot V_{pl,Rd}$$

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} = \chi_w \cdot \left(\frac{f_{yw}}{\sqrt{3}} \right) \cdot \frac{h_w t_w}{\gamma_{M1}}$$

where χ_w is the reduction factor for shear buckling, which depends on the slenderness of the web and the rigidity of the end posts (rigid or non-rigid) at the beam supports. Web slenderness is defined in general as:

$$\chi_w = \text{function} \left(\bar{\lambda}_w = \sqrt{\frac{\tau_{yw}}{\tau_{cr}}} = 0.76 \sqrt{\frac{f_{yw}}{\tau_{cr}}} \right)$$

$$\tau = \frac{V}{t_u h_w} \leq \tau = \frac{f_y}{\sqrt{3}}$$

$$V_{pl,Rd} = \frac{f_y}{\sqrt{3}} \cdot \frac{t_u \cdot h_w}{\gamma_{M_0}}$$

5. Plate Girder Shear Resistance

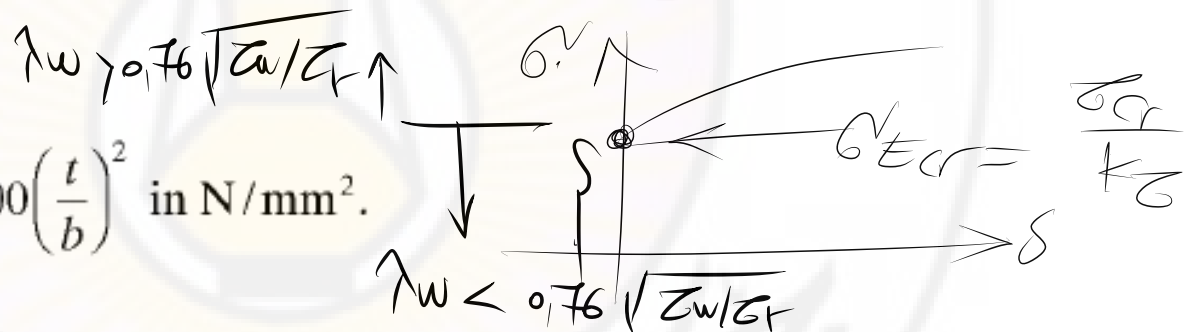
5-5 Shear Resistance of the Web According to EC3 – EN 1993-1-5

in which τ_{yw} is the yield strength of the web material in shear and τ_{cr} is the elastic buckling stress of the web in shear, which may be determined from:

$$\tau_{cr} = k_{\tau} \sigma_E$$

where k_{τ} is the shear buckling coefficient described above and σ_E is defined in Annex A.1 of BS EN 1993-1-5 as:

$$\sigma_E = \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{b} \right)^2 = 190000 \left(\frac{t}{b} \right)^2 \text{ in N/mm}^2.$$



Substituting τ_{cr} into the general expression for web slenderness given above, allows $\bar{\lambda}_w$ for webs with transverse stiffeners at the supports and intermediate transverse stiffeners to be defined directly as:

$$\bar{\lambda}_w = \frac{h_w}{37.4 t_w \epsilon \sqrt{k_{\tau}}} \quad \leftarrow \quad \left[\tau_{cr} = k_{\tau} \sigma_E \right] \quad \leftarrow \quad \lambda_w = 0.76 \cdot \sqrt{\frac{\sigma_y}{\tau_{cr}}}$$

$$\lambda_w \text{ (critical)} \rightarrow \lambda_w = 0.7 \sqrt{\sigma_w / \sigma_{cr}}$$

$$\sigma_w = \frac{\sigma_w}{A_w} = \frac{f_y}{r_z} \times \frac{1}{h_w \cdot t_w}$$

$$\sigma_{cr} = k \sigma_c \quad \sigma_{cr} = k \sigma_c \quad 190000 \left(\frac{t}{b-a} \right)^2$$

$$\lambda_w = \frac{h_w}{37.4 t_w \cdot \epsilon \sqrt{k \sigma_c}}$$

Unstiffened

$$\frac{a}{h_w} > 1 \rightarrow \text{too wide}$$

$$\frac{h_w}{a} \pi \phi \rightarrow k \sigma_c \approx 5134$$

$$\lambda_w = \frac{h_w}{86.4 t_w \cdot \epsilon}$$

Stiffened

$$k \sigma_c > 5134$$

$$\lambda_w = \frac{h_w}{37.4 t_w \cdot \epsilon \sqrt{k \sigma_c}}$$

5. Plate Girder Shear Resistance

5-5 Shear Resistance of the Web According to EC3 – EN 1993-1-5

For plate girders with transverse stiffeners at the supports only, the web is said to be 'unstiffened'. The aspect ratio a/h_w of such a web panel is high and k_τ tends to 5.34, in which case, web slenderness may be defined more simply as:

$$\bar{\lambda}_w = \frac{h_w}{86.4 t_w \epsilon}$$

{

stiffened (k_τ var) $\rightarrow \lambda_w = h_w / (37.4 * t_w * \epsilon \sqrt{k_\tau})$

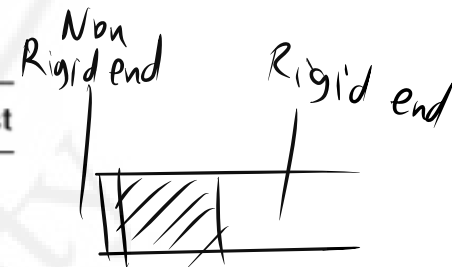
unstiffened $k_\tau \rightarrow 5.34 \rightarrow \lambda_w = h_w / (86.4 * t_w * \epsilon)$

Having established the web slenderness, shear buckling reduction factors may be determined from Table 5.1 of BS EN 1993-1-5,

For end panels, shear buckling reduction factors have been found to depend on the rigidity of the end posts, with greater rigidity offering greater anchorage to tension field action and hence higher load-carrying capacities. In BS EN 1993-1-5,

Shear buckling reduction factors for web χ_w (with $\eta = 1.0$)

Slenderness range	Rigid end post	Non-rigid end post
$\bar{\lambda}_w < 0.83$	1.0	1.0
$0.83 \leq \bar{\lambda}_w < 1.08$	$0.83 / \bar{\lambda}_w$	$0.83 / \bar{\lambda}_w$
$\bar{\lambda}_w \geq 1.08$	$1.37 / (0.7 + \bar{\lambda}_w)$	$0.83 / \bar{\lambda}_w$



5. Plate Girder Shear Resistance

5-5 Shear Resistance of the Web According to EC3 – EN 1993-1-5

Rigid and Non Rigid
End Post

Field

N.R.



Shear buckling reduction factors for web χ_w (with $\eta = 1.0$)

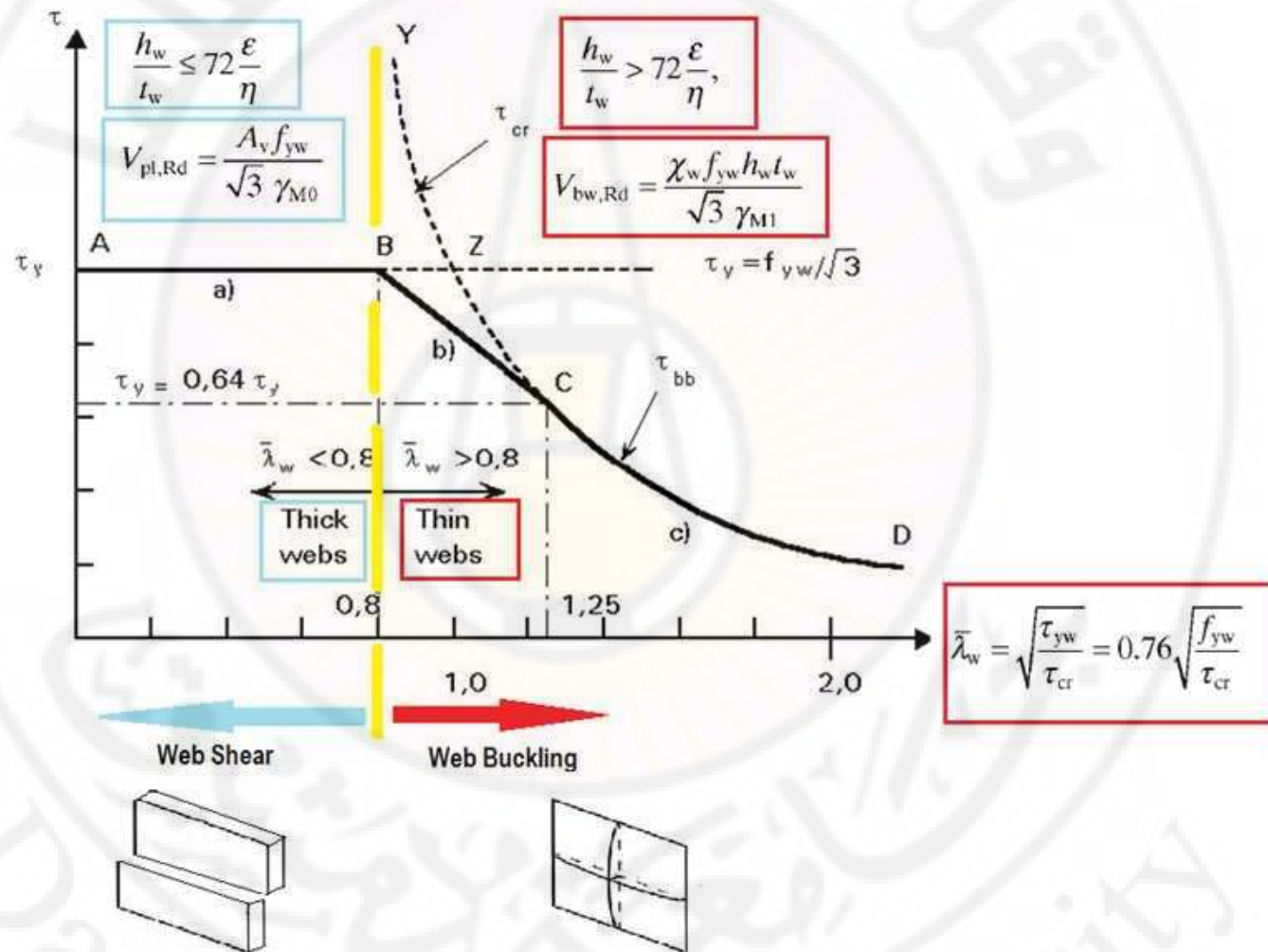
Slenderness range	Rigid end post	Non-rigid end post
$\bar{\lambda}_w < 0.83$	1.0	1.0
$0.83 \leq \bar{\lambda}_w < 1.08$	$0.83 / \bar{\lambda}_w$	$0.83 / \bar{\lambda}_w$
$\bar{\lambda}_w \geq 1.08$	$1.37 / (0.7 + \bar{\lambda}_w)$	$0.83 / \bar{\lambda}_w$

Rigid



5. Plate Girder Shear Resistance

5-1 Web Behavior under Shear



Initial Behavior Of Web Plate

5. Plate Girder Shear Resistance

5-1 Web Behavior under Shear

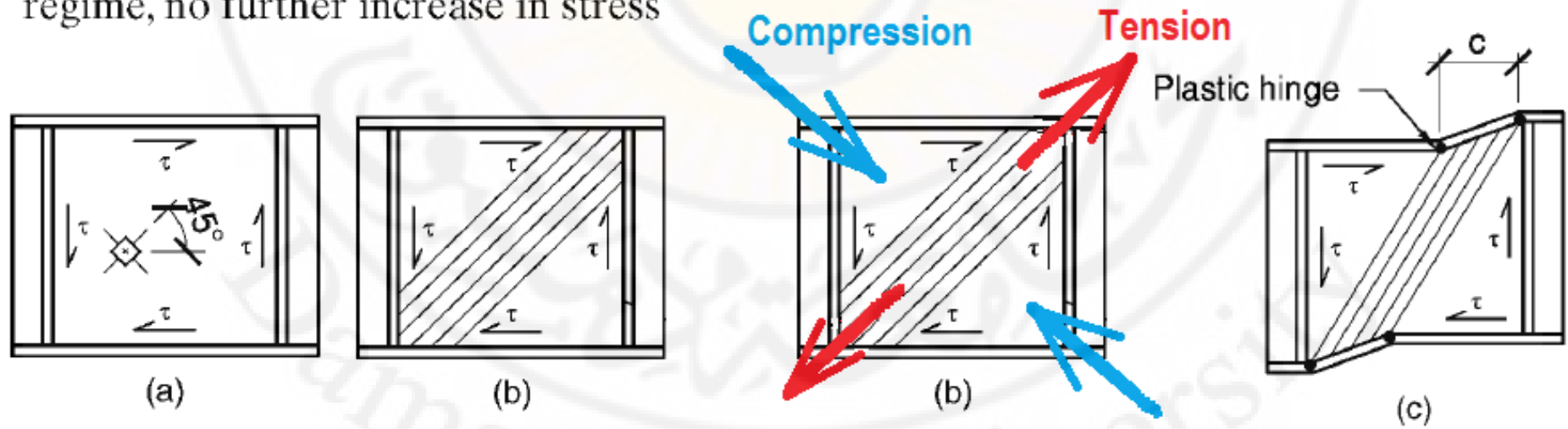
Tension Field



5. Plate Girder Shear Resistance

5-2 Shear Introduction

Webs of slender proportions (i.e. height-to-thickness ratio exceeding specified limiting values) are susceptible to shear buckling. The response of such webs from the onset of loading to collapse may be described in three successively occurring stages. In stage 1 at low load levels, the web panel is in a state of pure shear, with the principal compressive and tensile stresses orientated at 45° to the horizontal. For slender web panels, the limit of stage 1, i.e. the pure shear field, is reached when the applied shear stress τ reaches the elastic shear buckling stress τ_{cr} and buckling occurs along the direction of the compressive diagonal. In stage 2, the post-buckling regime, no further increase in stress

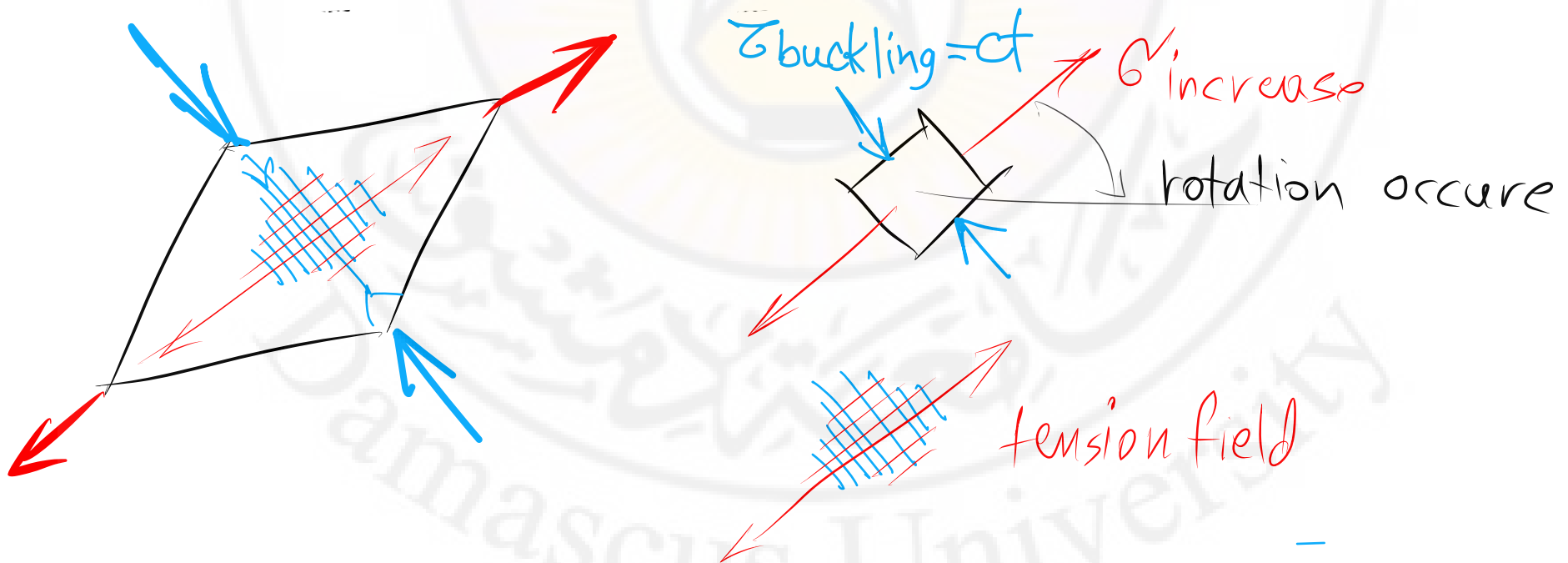


5. Plate Girder Shear Resistance

5-2 Shear Introduction

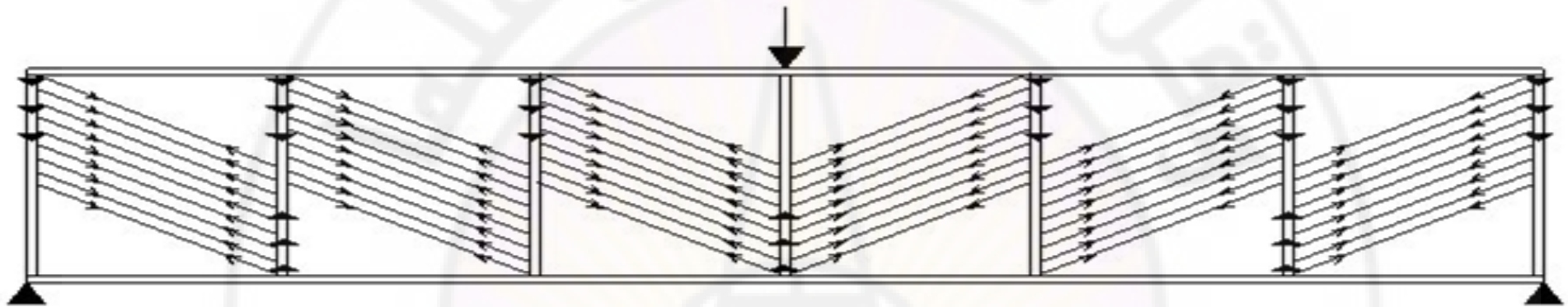
can take place along the compressive diagonal, but stresses can continue to increase along the tensile diagonal, resulting in so-called tension field action .

Under increasing load, the direction of principal tensile stress rotates towards the horizontal. These tensile stresses are resisted by the horizontal and vertical boundaries of the web panel (i.e. the flanges and the transverse stiffeners) forming a load-carrying mechanism similar to that of a Pratt truss. Ultimate shear capacity is reached in stage 3 with failure occurring due to yielding of the web panel and the formation of plastic hinges in the beam flanges.

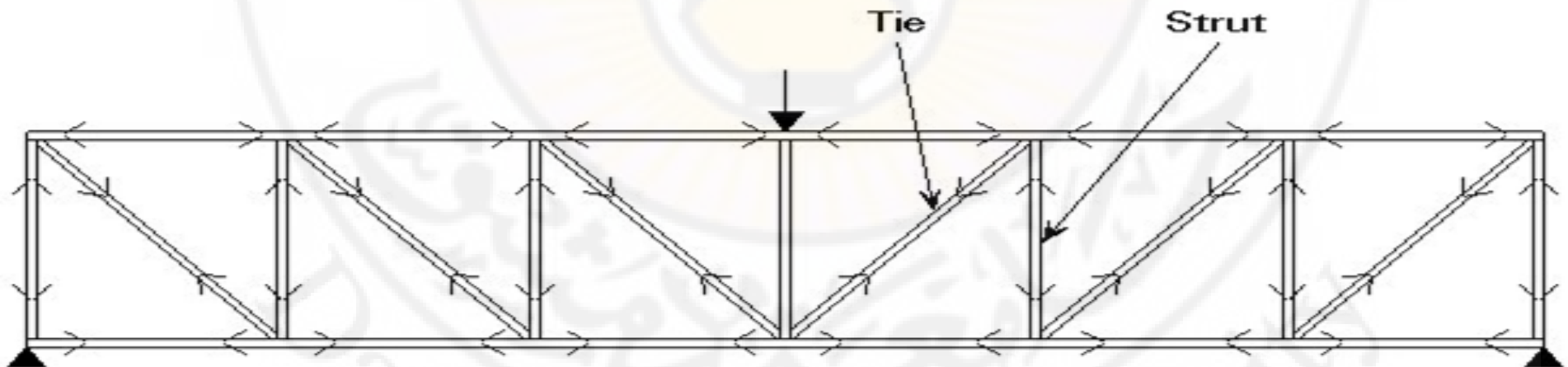


5. Plate Girder Shear Resistance

5-3 Tension Field Of The Web



(a) Tension field action



(b) Pratt or N-truss

5. Plate Girder Shear Resistance

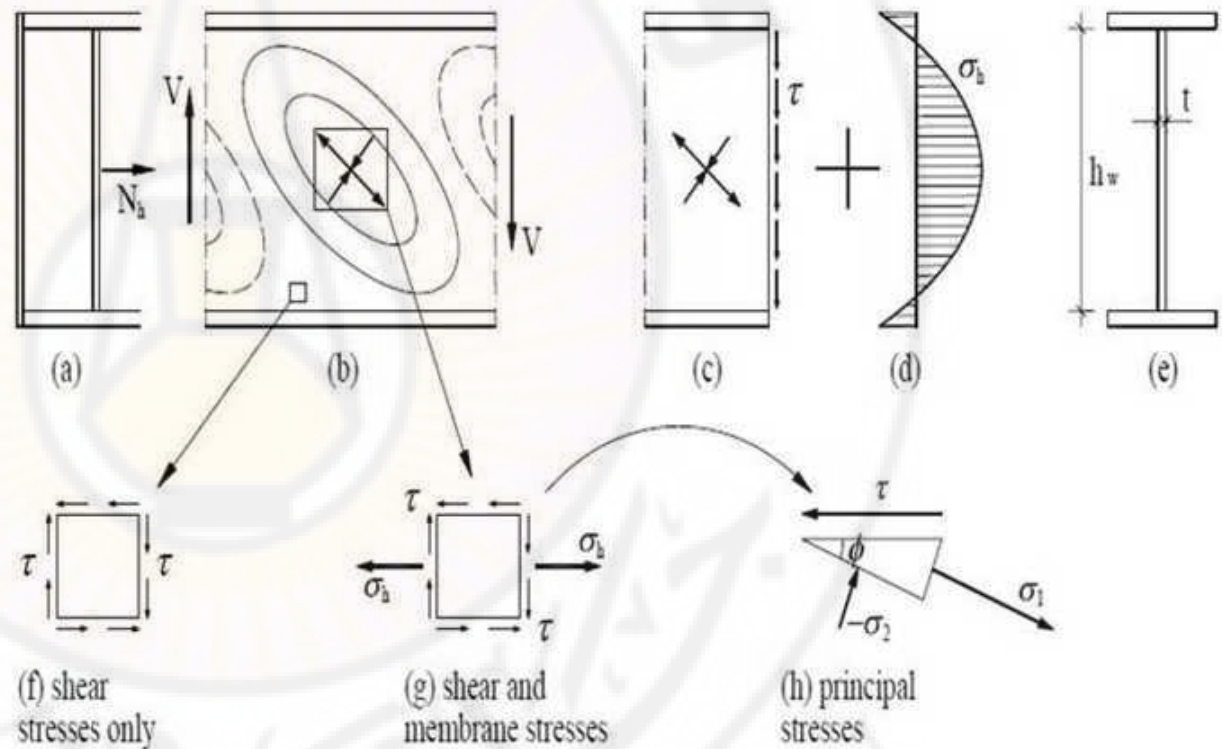
5-3 Tension Field Of The Web

Slender web panels in shear possess a significant post-buckling resistance, when a proper anchorage of tension membrane stresses that develop after buckling is assured. Many design methods based on **tension field** action have been developed so far. These theories mainly assume superposition of buckling and post-buckling shear strength and differ regarding the definition of tension field action. EN 1993-1-5 implemented the method known as "**rotated stress field**". This method was first developed for unstiffened webs with large aspect ratios where other tension field methods give very conservative results. It was used in ENV 1993-1-5 for plated steel structures and in ENV 1999-1-1 for aluminum structures, but not in ENV 1993-1-1 where the so-called Cardiff-Prague tension field method was used and the simple post-critical method was added to overcome conservative results.

5. Plate Girder Shear Resistance

5-4 The Rotated Stress Field Of The Web

The rotated stress: The basic assumption is that there are no membrane stresses in the transverse direction of the web panel. This is true for webs of long girders without transverse stiffeners other than the ones at the supports. After buckling compressive membrane stresses σ_2 cannot increase any more, but tension membrane stresses σ_1 may still increase until the ultimate resistance is reached. Under such conditions the equilibrium requires rotation of the stress field.

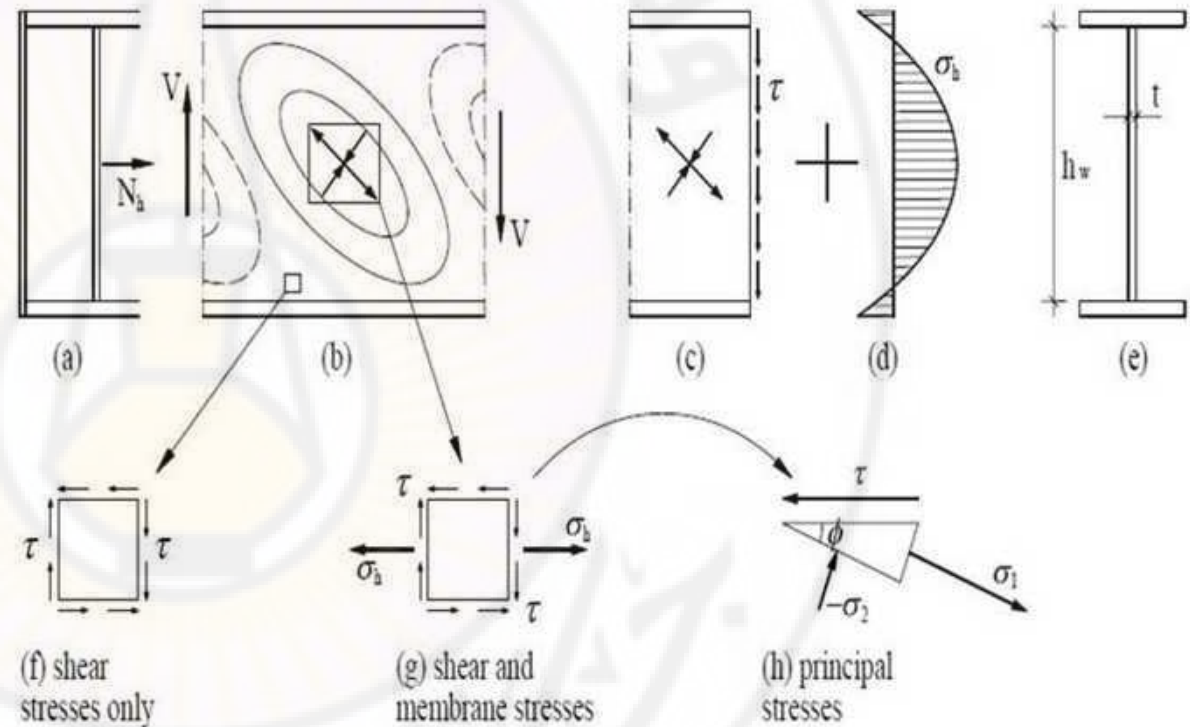


Mechanical model of the rotated stress field

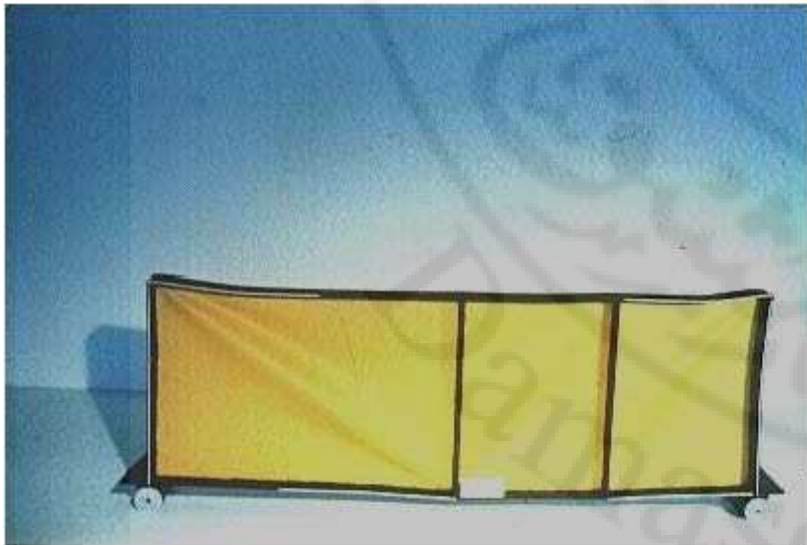
5. Plate Girder Shear Resistance

5-4 The Rotated Stress Field Of The Web

When principal tension stresses increase, the angle ϕ must decrease because of equilibrium reasons, and N_h is the total longitudinal force existing in the web at the postcritical stage to be anchored onto the end post, and σ_h is the corresponding normal stress in the longitudinal direction.

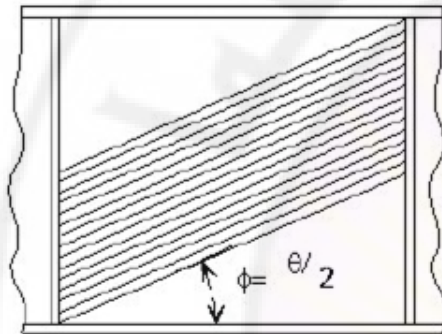


Mechanical model of the rotated stress field

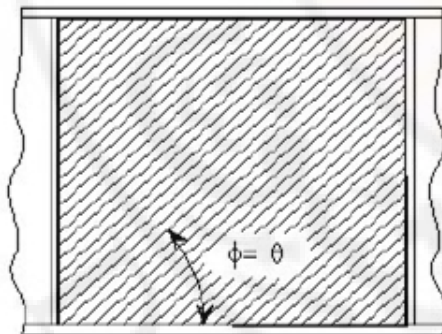


5. Plate Girder Shear Resistance

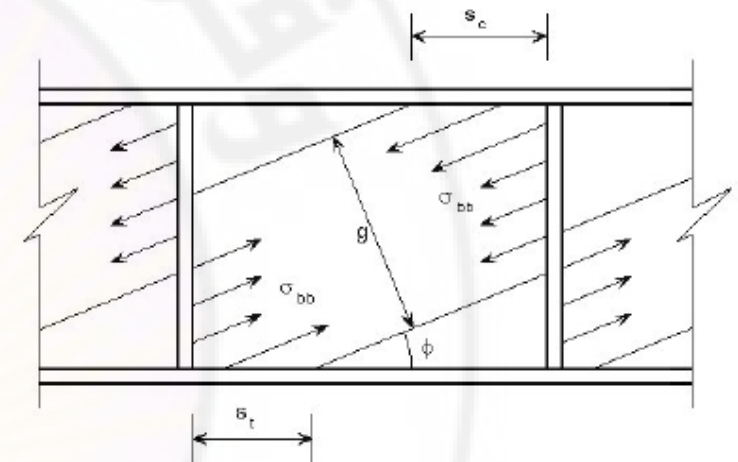
5-4 The Rotated Stress Field Of The Web



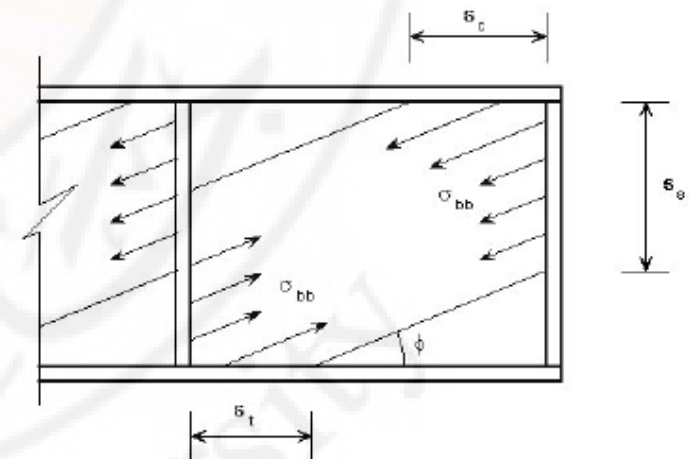
(a) "Web only"
tension field action



(b) Complete
tension field action



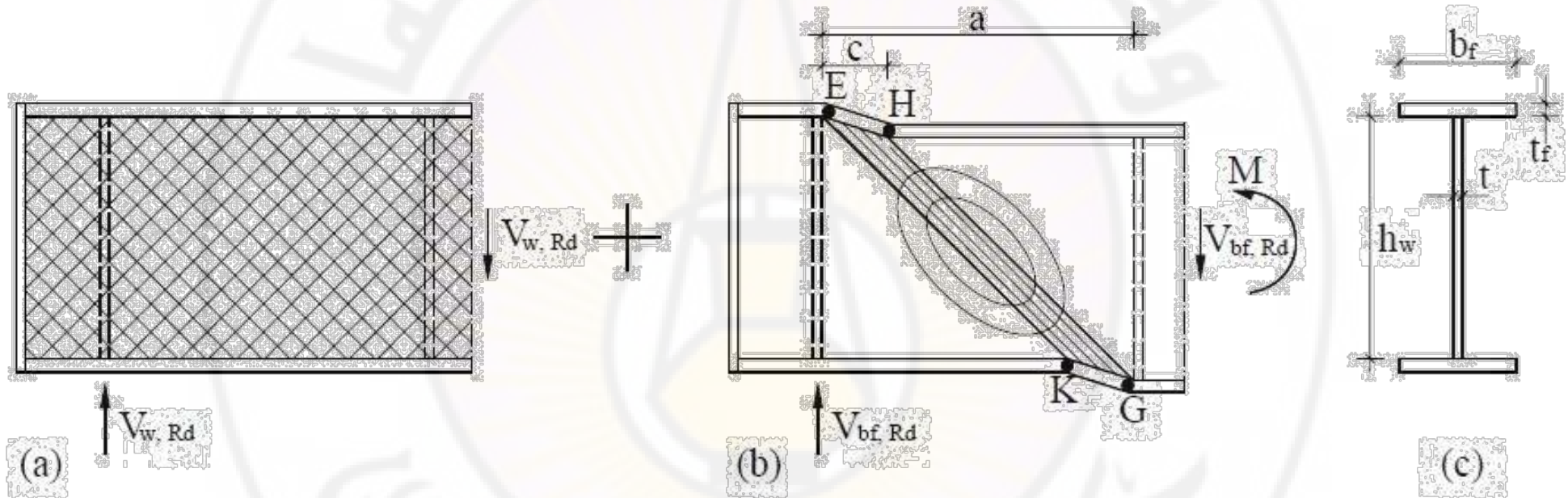
(a) Internal panel



(b) End panel

5. Plate Girder Shear Resistance

5-4 The Rotated Stress Field Of The Web



Tension field carried by bending resistance of flanges

5. Plate Girder Shear Resistance

5-5 Shear Resistance of the Web According to EC3 – EN 1993-1-5

Shear buckling resistance is defined in Clause 5.2(1) of BS EN 1993-1-5 as:

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}}$$

in which $\gamma_{M1} = 1.0$ is the partial factor for instability. The contribution of the web $V_{bw,Rd}$ is given by:

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}}$$

5. Plate Girder Shear Resistance

5-5 Shear Resistance of the Web According to EC3 – EN 1993-1-5

The flange contribution to shear buckling resistance $V_{bf,Rd}$ is defined in Clause 5.4(1) of BS EN 1993-1-5 as:

$$V_{bf,Rd} = \frac{b_f t_f^2 f_{yf}}{c \gamma_{M1}} \left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}} \right)^2 \right)$$

where b_f is the total flange width, but should be taken as no larger than $15\epsilon t_f$ either side of the web, $M_{f,Rd}$ is the moment resistance of the flanges alone calculated on the basis of effective section properties if the compression flange is Class 4, and c is the width of the tension band :

$$c = a \left(0.25 + \frac{1.6 b_f t_f^2 f_{yf}}{t_w h_w^2 f_{yw}} \right)$$

$$V_{n,Rz} = V_{b,w,Rd} + V_{b',Rd} \leq \frac{\eta f_{yw} I_p t_w}{\sqrt{3} \gamma_{M1}}$$

Total Girder Resistance



5. Plate Girder Shear Resistance

5-6 - Summary

Shear Resistance of the Web According to EC3 – EN 1993-1-5

- Unless the web is Class 1, we need to check whether the web is stocky or not for shear buckling
- For unstiffened web

$$\frac{h_w}{t_w} \leq 72 \frac{\varepsilon}{\eta}$$

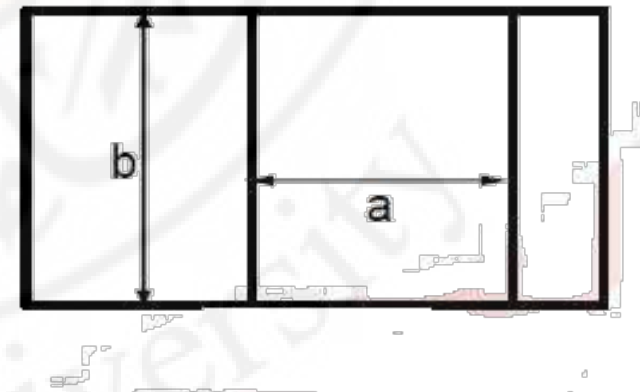
- For stiffened web (either transverse or longitudinal or both)

$$\frac{h_w}{t_w} \leq 31 \frac{\varepsilon}{\eta} \sqrt{k_\tau}$$

where $\eta=1.0$, and for plate girder without longitudinal stiffener

$$k_\tau = 5.34 + 4.00 \left(\frac{h_w}{a} \right)^2 \quad \text{for } a/h_w \geq 1$$

$$k_\tau = 4.00 + 5.34 \left(\frac{h_w}{a} \right)^2 \quad \text{for } a/h_w < 1$$



5. Plate Girder Shear Resistance

5-6 - Summary

Shear Resistance of the Web According to EC3 – EN 1993-1-5

- If the web is stocky, no shear buckling of web shall occur and the shear strength of the web is given by EC3 Part 1-1

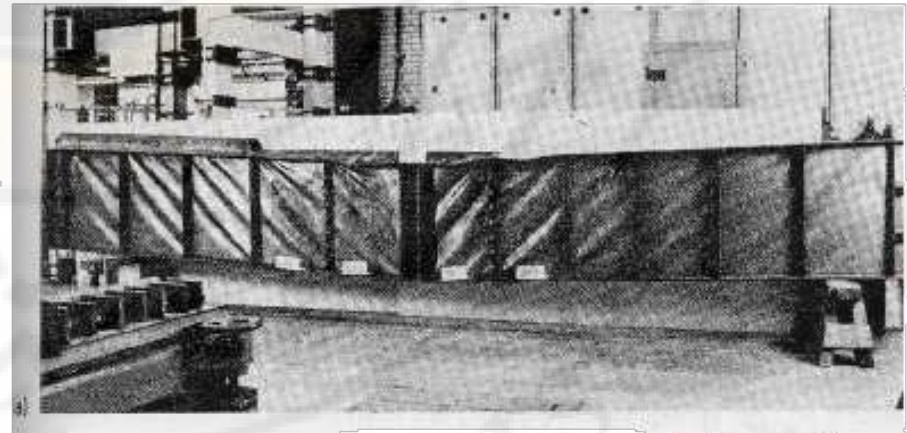
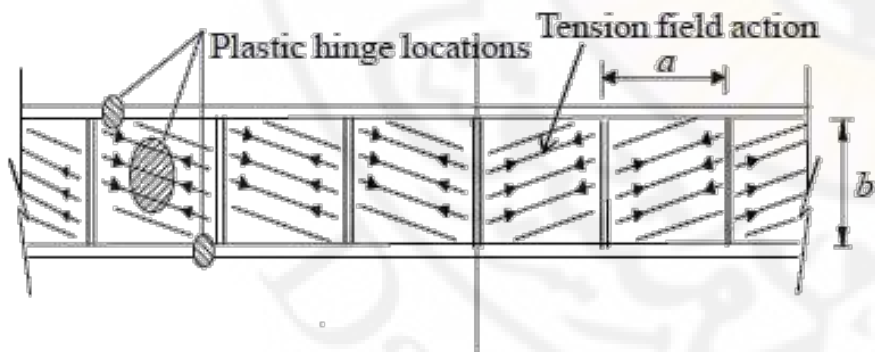
$$V_{pl,Rd} = \frac{A_v (f_y / \sqrt{3})}{\gamma_{M0}}$$

- If the web is NOT stocky, shear buckling governs the failure

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}}$$

Web contribution (TFA)

Flange contribution (PH formation)



5. Plate Girder Shear Resistance

5-6 - Summary

Shear Resistance of the Web According to EC3 – EN 1993-1-5

- For the contribution from the web $V_{bw,Rd}$

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}}$$

χ_w is the shear reduction factor which depends on the end post conditions and the modified web slenderness $\bar{\lambda}_w$, $\eta=1$ for all steel.

Table 5.1: Contribution from the web χ_w to shear buckling resistance

	Rigid end post	Non-rigid end post
$\bar{\lambda}_w < 0,83/\eta$	η	η
$0,83/\eta \leq \bar{\lambda}_w < 1,08$	$0,83/\bar{\lambda}_w$	$0,83/\bar{\lambda}_w$
$\bar{\lambda}_w \geq 1,08$	$1,37/(0,7 + \bar{\lambda}_w)$	$0,83/\bar{\lambda}_w$

For transverse stiffeners at supports only (e.g. end post only)

$$\bar{\lambda}_w = \frac{h_w}{86,4 t_w \varepsilon}$$

For the presence transverse and intermediate transverse stiffener

$$\bar{\lambda}_w = \frac{h_w}{37,4 t_w \varepsilon \sqrt{k_\tau}}$$

5. Plate Girder Shear Resistance

5-6 - Summary

Shear Resistance of the Web According to EC3 – EN 1993-1-5

- For the contribution from the flange $V_{bf,Rd}$

$$V_{bf,Rd} = \frac{b_f t_f^2 f_{yf}}{c \gamma_{M1}} \left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}} \right)^2 \right)$$

where $M_{f,Rd}$ is the moment of resistance of effective area of the flanges only (as in the “flanges only” method) and

$$c = a \left(0.25 + \frac{1.6 b_f t_f^2 f_{yf}}{t_w h_w^2 f_{yw}} \right)$$

- Verification of shear resistance** is checked by calculating η_3

$$\eta_3 = \frac{V_{Ed}}{V_{b,Rd}} \leq 1.0$$

5. Plate Girder Shear Resistance

5-7 - Interaction of bending and shear

Resistance to combined bending and shear

Web not susceptible to shear buckling

For webs not susceptible to shear buckling, resistance to combined bending and shear is covered in Clause 6.2.8 of BS EN 1993-1-1. In this Clause it is stated that, provided the design shear force V_{Ed} is less than 50% of the plastic shear resistance $V_{pl,Rd}$ of the section, the interaction between bending and shear can be ignored and the full bending moment resistance can be attained. Where the design shear force exceeds 50% of the plastic shear resistance, the bending resistance of the cross-section must be reduced; this is achieved by applying a reduced yield strength to the shear area of the section. The reduced yield strength f_{yr} is defined on the basis of the level of applied shear force as follows:

$$f_{yr} = (1 - \rho)f_y$$

in which f_y is the basic material yield strength and ρ is determined as:

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1 \right)^2$$

5. Plate Girder Shear Resistance

5-7 - Interaction of bending and shear

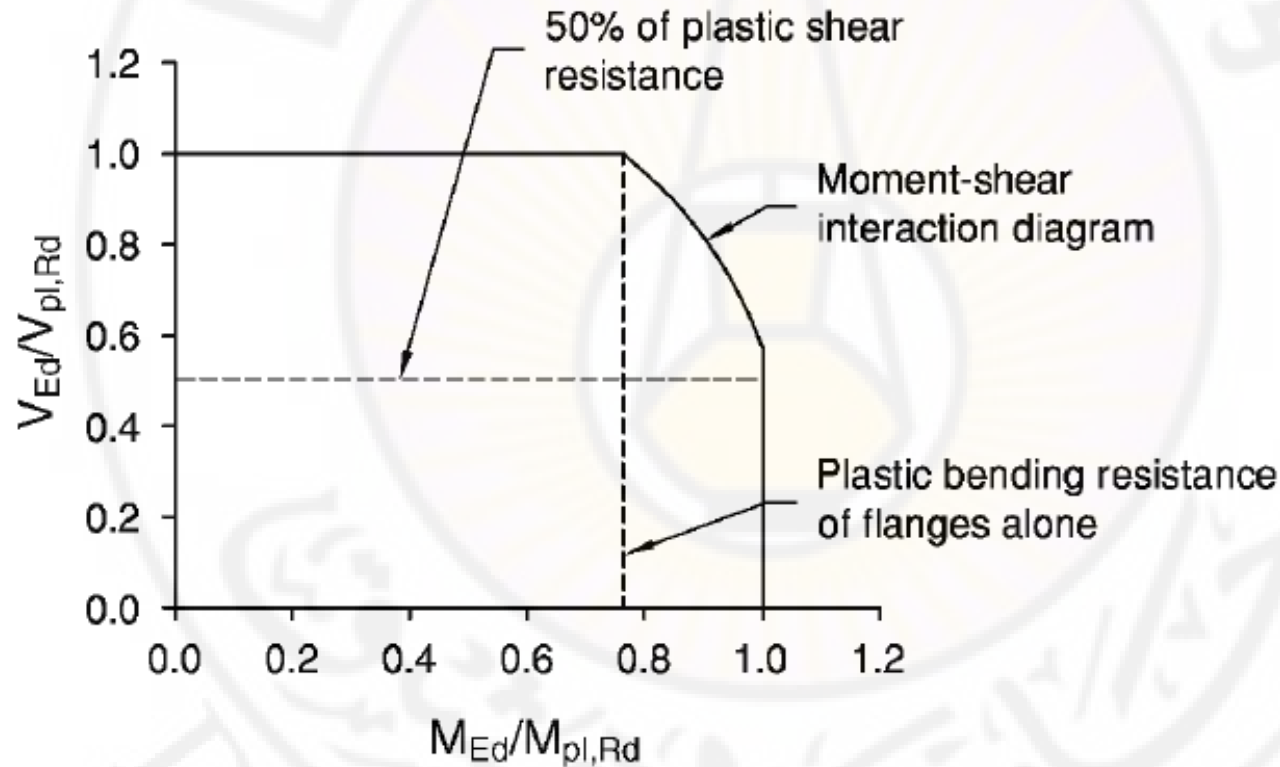
On the above basis, the reduced major axis bending moment resistance $M_{y,V,Rd}$ of a Class 1 or 2 I-section with equal flanges and uniform yield strength, in the presence of a shear force greater than 50% of the plastic shear resistance, may be determined directly from:

$$M_{y,V,Rd} = \frac{\left[W_{pl,y} - \frac{\rho A_w^2}{4t_w} \right] f_y}{\gamma_{M0}} \leq M_{y,c,Rd}$$

where $A_w = h_w t_w$ is the area of the web and $M_{y,c,Rd}$ is the full in-plane bending resistance of the cross-section, as defined in Clause 6.2.5(2) of BS EN 1993-1-1.

5. Plate Girder Shear Resistance

5-7 - Interaction of bending and shear



Shear-moment interaction for Class 1 and 2 cross-sections where the web is not susceptible to shear buckling

5. Plate Girder Shear Resistance

5-7 - Interaction of bending and shear

Web susceptible to shear buckling

For webs susceptible to shear buckling, the interaction between moment and shear is effectively covered in Clause 5 of BS EN 1993-1-5, when the flanges are not fully utilised in resisting bending ($M_{Ed} < M_{f,Rd}$), and in Clause 7.1 of BS EN 1993-1-5, when the flanges are fully utilised in resisting bending ($M_{Ed} \geq M_{f,Rd}$). The former case is discussed in Section 18.4.4.2 while, for the latter case, the following interaction expression, set out in Clause 7.1(1) of BS EN 1993-1-5, applies:

$$\frac{M_{Ed}}{M_{pl,Rd}} + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}} \right) \left(\frac{2V_{Ed}}{V_{bw,Rd}} - 1 \right)^2 \leq 1.0$$

The key features of the full bending-shear interaction diagram for Class 1 and 2 cross-sections are as follows:

5. Plate Girder Shear Resistance

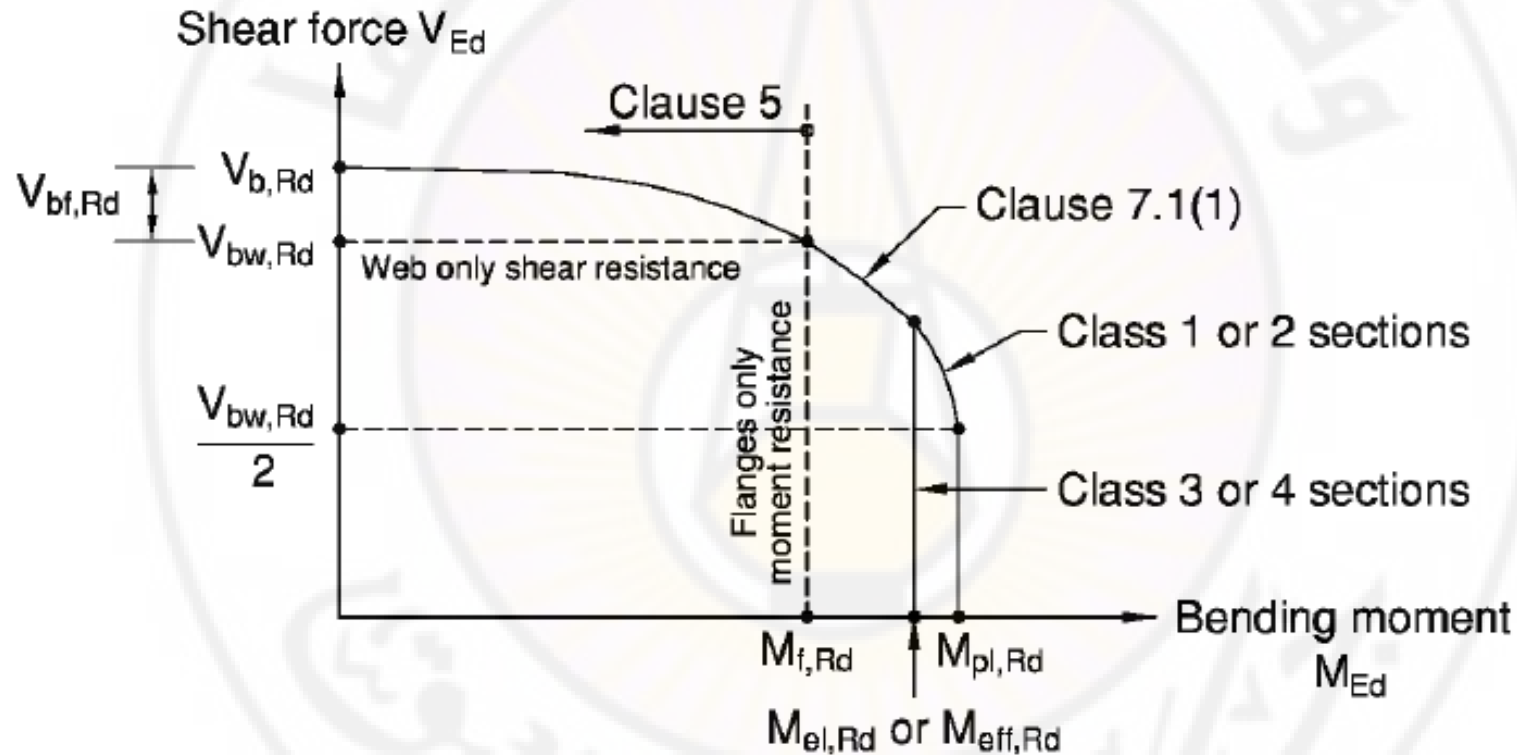
5-7 - Interaction of bending and shear

The key features of the full bending-shear interaction diagram for Class 1 and 2 cross-sections are as follows:

1. Provided the design shear force V_{Ed} is less than or equal to half of the shear buckling resistance of the web $V_{bw,Rd}$, then the full plastic moment capacity $M_{pl,Rd}$ of the cross-section can be achieved.
2. The shear capacity of the web alone $V_{bw,Rd}$ and the bending moment capacity of the flanges alone $M_{f,Rd}$ can be mobilised simultaneously.
3. The maximum shear capacity ($V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd}$), including the full contribution of the flanges, is only achieved in the presence of zero co-existent bending moment.
4. As the level of the co-existent bending moment increases, the flange contribution to shear resistance reduces by the factor $(1 - M_{Ed}/M_{f,Rd})^2$ until $M_{Ed} = M_{f,Rd}$, at which point the flanges are fully utilised in resisting bending and make no contribution to the shear resistance. This interaction is set out in Clause 5 of BS EN 1993-1-5 and discussed in Section 18.4.4.2.
5. For $M_{Ed} > M_{f,Rd}$, the interaction between bending and shear is set out in Clause 7.1(1) of BS EN 1993-1-5, as described above.

5. Plate Girder Shear Resistance

5-7 - Interaction of bending and shear



Shear-moment interaction for cross-sections where the web is susceptible to shear buckling

5. Plate Girder Shear Resistance

5-8 – Summary

Interaction of bending and shear

- If $\bar{\eta}_3 = V_{Ed} / V_{bw,Rd} \leq 0.5$ and $M_{Ed} < M_{f,Rd}$, the design resistance to bending moment need not be reduced to allow for the shear force
- Otherwise, additional verifications are needed, it is necessary that

$$\bar{\eta}_1 + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}} \right) (2\bar{\eta}_3 - 1)^2 \leq 1.0 \quad \bar{\eta}_1 = \frac{M_{Ed}}{M_{pl,Rd}}$$

where $M_{pl,Rd}$ is the design plastic resistance of the cross section consisting of the effective area of the flanges and the fully effective web irrespective of its class

- The above interaction check should be done at all sections **other than** those located at a distance less than **$h_w/2$ from a support with vertical stiffener**, as normally the shear force is overestimated there.

$$\eta_3 = \frac{V_{Ed}}{V_{b,Rd}} \leq 1.0$$

4. Plate Girder Bending Resistance

Back to Worked Example 1

Shear buckling resistance of end (anchor) panel AB

The shear resistance of panel AB will be calculated assuming a non-rigid end post, with the reduced capacity compensated for by closer stiffener spacing. The contribution of the flanges will be ignored, as described above.

$$h_w = 2000 \text{ mm}; a = 1550 \text{ mm}; f_{yw} = 275 \text{ N/mm}^2; V_{Ed} = 2293 \text{ kN}$$

$$\text{Web aspect ratio } a/h_w = 1550/2000 = 0.78$$

Buckling coefficient (for $a/h_w < 1$):

$$k_\tau = 4.00 + 5.34(h_w/a)^2 = 4.00 + 5.34(2000/1550)^2 = 12.89$$

Web slenderness:

$$\bar{\lambda}_{wv} = 0.76 \sqrt{\frac{f_{yw}}{\tau_{cr}}} = \frac{h_w}{37.4 t_w \epsilon \sqrt{k_\tau}} = \frac{2000}{37.4 \times 15 \times 0.92 \sqrt{12.89}} = 1.07$$

Web buckling reduction factor (for non-rigid end post):

$$\bar{\lambda}_{wv} < 1.08 \quad \therefore \chi_w = \frac{0.83}{\bar{\lambda}_{wv}} = \frac{0.83}{1.07} = 0.77$$

Non-rigid Post

Shear resistance of web $V_{bw,Rd}$:

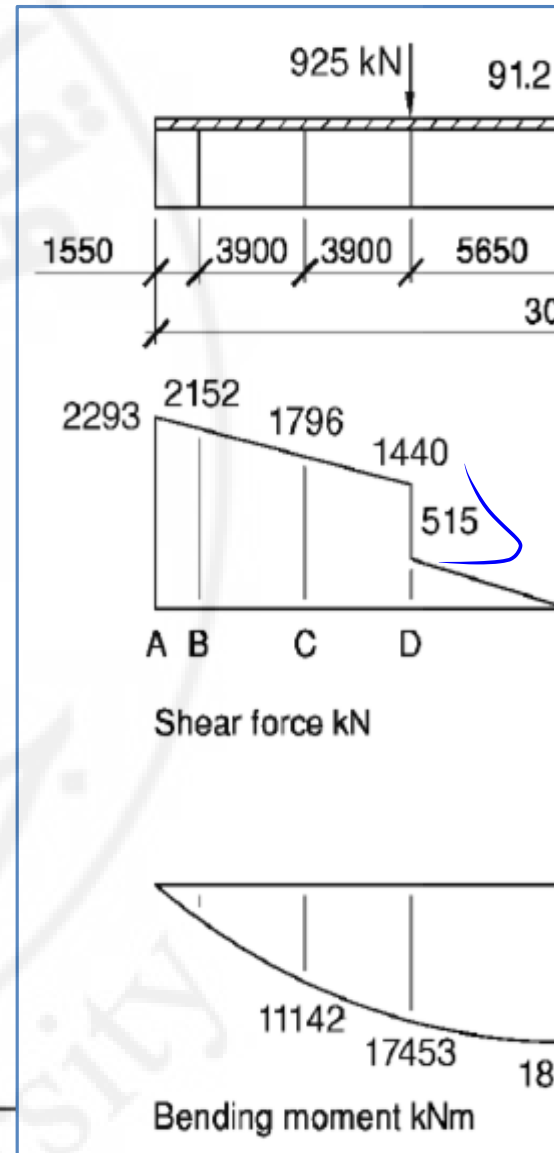
$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} = \frac{0.77 \times 275 \times 2000 \times 15}{\sqrt{3} \times 1.0 \times 10^3} = 3681 \text{ kN} > 2293 \text{ kN} = V_{Ed} \quad \text{OK}$$

BS EN
1993-1-5
Cl A.3(1)

BS EN
1993-1-5
Cl 5.3(3)

BS EN
1993-1-5
Cl 5.3(1)

BS EN
1993-1-5
Cl 5.2(1)



4. Plate Girder Bending Resistance

Back to Worked Example 1

Shear buckling resistance of panel BC

The shear panel resistance of the end panel BC is calculated assuming a rigid end post but ignoring the contribution of the flange.

$$h_w = 2000 \text{ mm}; a = 3900 \text{ mm}; f_{yw} = 275 \text{ N/mm}^2; V_{Ed} = 2152 \text{ kN}$$

$$\text{Web aspect ratio } a/h_w = 3900/2000 = 1.95$$

Buckling coefficient (for $a/h_w \geq 1$):

$$k_\tau = 5.34 + 4.00(h_w/a)^2 = 5.34 + 4.00(2000/3900)^2 = 6.39$$

Web slenderness:

$$\bar{\lambda}_w = 0.76 \sqrt{\frac{f_{yw}}{\tau_{cr}}} = \frac{h_w}{37.4 t_w \epsilon \sqrt{k_\tau}} = \frac{2000}{37.4 \times 15 \times 0.92 \sqrt{6.39}} = 1.53$$

Web buckling reduction factor (for rigid end post):

$$\bar{\lambda}_w \geq 1.08 \quad \therefore \chi_w = \frac{1.37}{(0.7 + \bar{\lambda}_w)} = \frac{1.37}{(0.7 + 1.53)} = 0.62$$

Rigid End

Shear resistance of web $V_{bw,Rd}$:

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} = \frac{0.62 \times 275 \times 2000 \times 15}{\sqrt{3} \times 1.0 \times 10^3} = 2932 \text{ kN} > 2152 \text{ kN} = V_{Ed} \quad \text{OK}$$

BS EN
1993-1-5
Cl A.3(1)

BS EN
1993-1-5
Cl 5.3(3)

BS EN
1993-1-5
Cl 5.3(1)

BS EN
1993-1-5
Cl 5.2(1)

4. Plate Girder Bending Resistance

Back to Worked Example 1

Shear buckling resistance of panel DE

The shear resistance of panel DE is calculated assuming a rigid end post but ignoring the contribution of the flange.

$$h_w = 2000 \text{ mm}; a = 3900 \text{ mm}; f_{yw} = 275 \text{ N/mm}^2; V_{Ed} = 515 \text{ kN}$$

$$\text{Web aspect ratio } a/h_w = 5650/2000 = 2.83$$

Buckling coefficient (for $a/h_w \geq 1$):

$$k_\tau = 5.34 + 4.00(h_w/a)^2 = 5.34 + 4.00(2000/5650)^2 = 5.84$$

Web slenderness:

$$\bar{\lambda}_w = 0.76 \sqrt{\frac{f_{yw}}{\tau_{cr}}} = \frac{h_w}{37.4 t_w \varepsilon \sqrt{k_\tau}} = \frac{2000}{37.4 \times 15 \times 0.92 \sqrt{5.84}} = 1.60$$

Web buckling reduction factor (for rigid end post):

$$\bar{\lambda}_w \geq 1.08 \quad \therefore \chi_w = \frac{1.37}{(0.7 + \bar{\lambda}_w)} = \frac{1.37}{(0.7 + 1.60)} = 0.60 \quad \text{Rigid End}$$

Shear resistance of web $V_{bw,Rd}$:

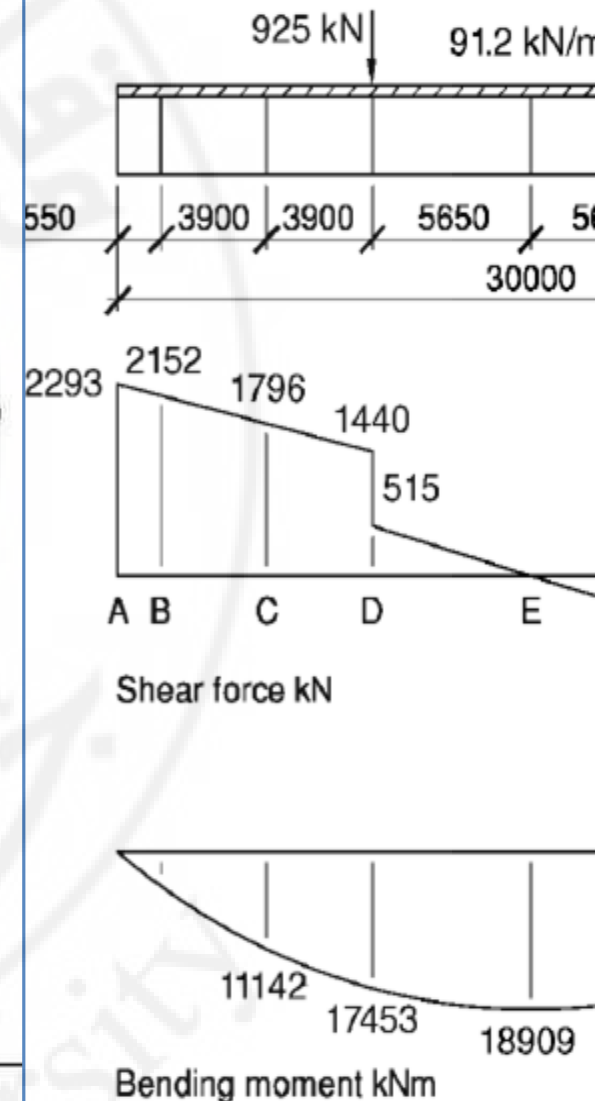
$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t_w}{\sqrt{3} \gamma_{M1}} = \frac{0.60 \times 275 \times 2000 \times 15}{\sqrt{3} \times 1.0 \times 10^3} = 2843 \text{ kN} > 515 \text{ kN} = V_{Ed} \text{ OK}$$

BS EN
1993-1-5
Cl A.3(1)

BS EN
1993-1-5
Cl 5.3(3)

BS EN
1993-1-5
Cl 5.3(1)

BS EN
1993-1-5
Cl 5.2(1)



The End of Part II

Damascus University

Damascus University

Faculty Of Civil Engineering

4th Year Lectures 2023

Dr. – Eng. Moaid Subh

PLATE GIRDER

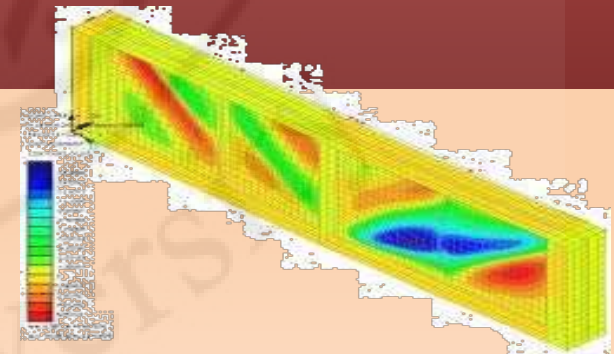
Part 3

To Eurocode 3

EN 1993-1-5 (2006)

Design of steel structures - Part 1-5

General rules Plated structural elements



6. Stiffeners and end post

6-1 – Introduction

General

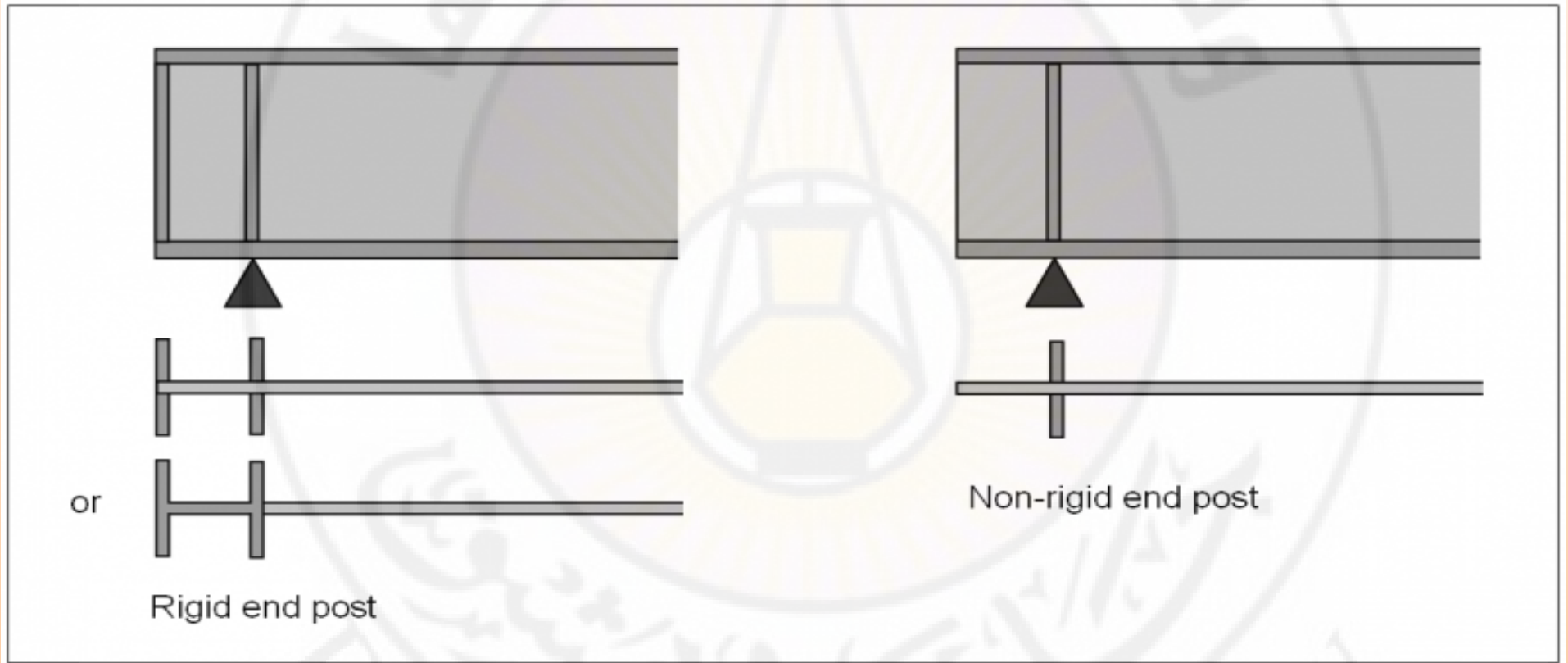
When a web is susceptible to shear buckling, some form of end anchorage is required at the supports to resist the post-buckling membrane stresses (tension field) that develop in a buckled web. The situation at the end of the plate girder is clearly more onerous than that at internal locations where support is present from adjacent panels. End anchorage need not be provided if the plastic shear resistance $V_{pl,Rd}$ rather than the shear buckling resistance $V_{b,Rd}$ is the governing design criterion (i.e. when the web is not susceptible to shear buckling).

End posts are classified in BS EN 1993-1-5 as either rigid or non-rigid. For web slendernesses λ_w of less than 1.08, shear buckling resistance is unaffected by this distinction, but for higher slendernesses, the greater anchorage afforded by a rigid end post results in improved shear buckling resistance. End posts also act as bearing stiffeners resisting the reaction from the supports.

Rigid end posts should therefore be designed to resist the horizontal component of the tension field together with compressive forces due to the support reactions. Non-rigid end posts need only be designed as bearing stiffeners to resist the compressive forces due to the support reactions.

6. Stiffeners and end post

6-2 - End post Stiffener



6. Stiffeners and end post

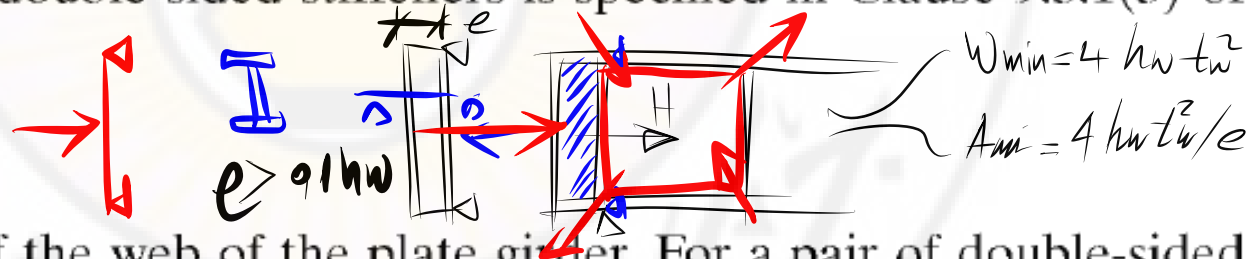
6-2 - End post Stiffener

Rigid end posts

A rigid end post, as defined in Clause 9.3.1(2) of BS EN 1993-1-5, should comprise **two double-sided transverse stiffeners** or a **rolled section** connected to the end of the web plate. In both cases, the end post acts as a short beam spanning vertically between the flanges and resisting the longitudinal membrane force arising from tension field action.

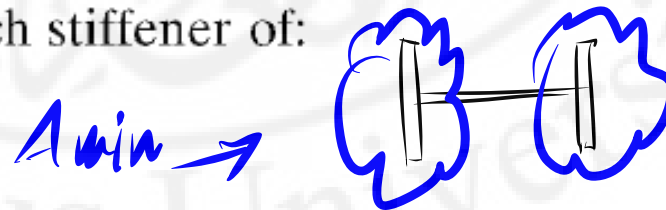
The minimum section modulus $W_{\min}$ for a rigid end post comprising either a rolled section or a pair of double-sided stiffeners is specified in Clause 9.3.1(3) of BS EN 1993-1-5 as:

$$W_{\min} = 4h_w t_w^2$$



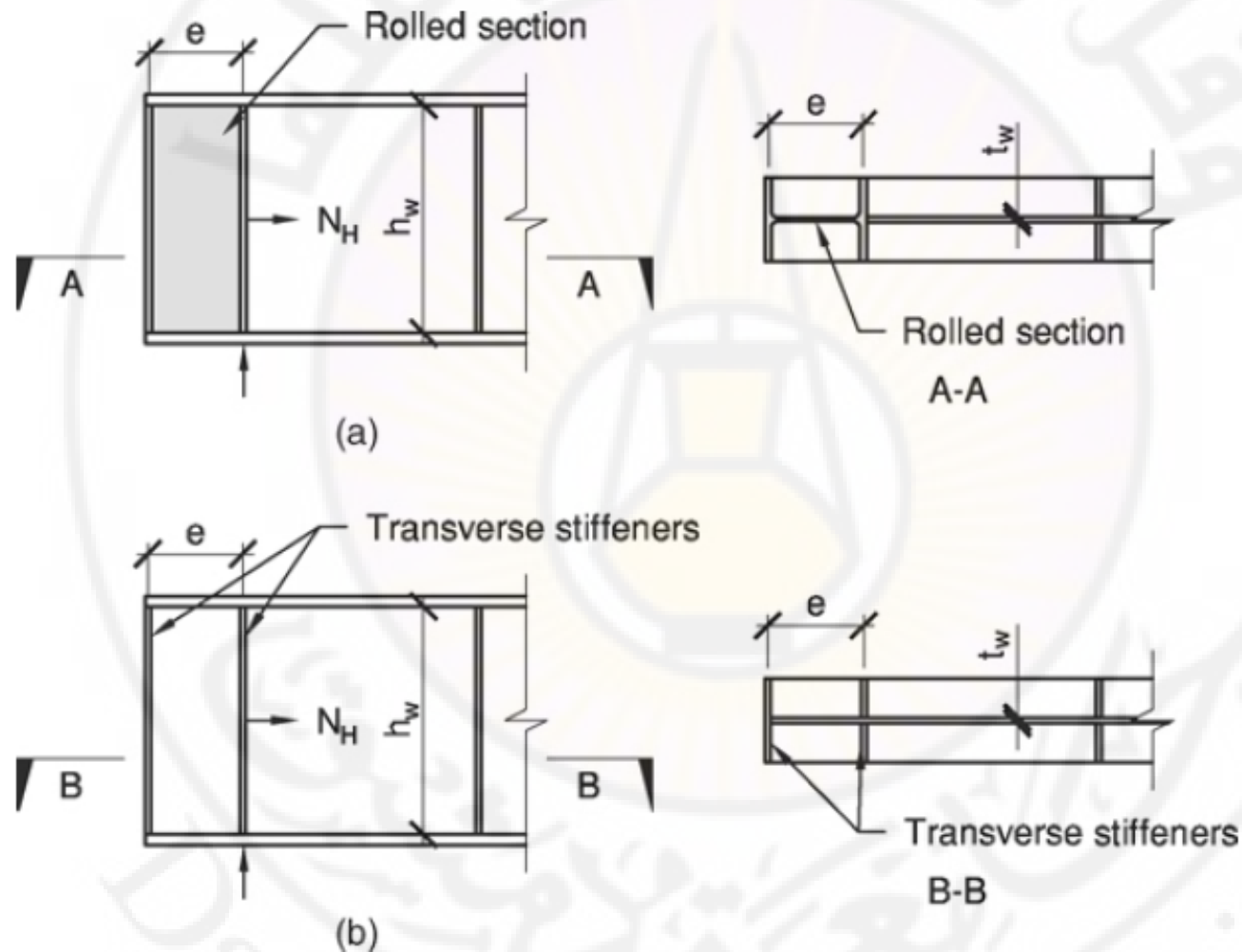
where t_w is the thickness of the web of the plate girder. For a pair of double-sided flat stiffeners separated by a distance e (which must be at least $0.1h_w$, according to Clause 9.3.1(3) of BS EN 1993-1-5), this corresponds (ignoring the web contribution) to a minimum area $A_{\min}$ for each stiffener of:

$$A_{\min} = 4h_w t_w^2 / e$$



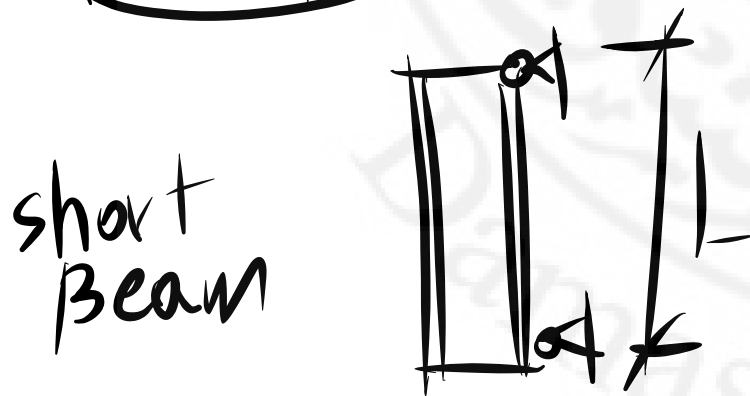
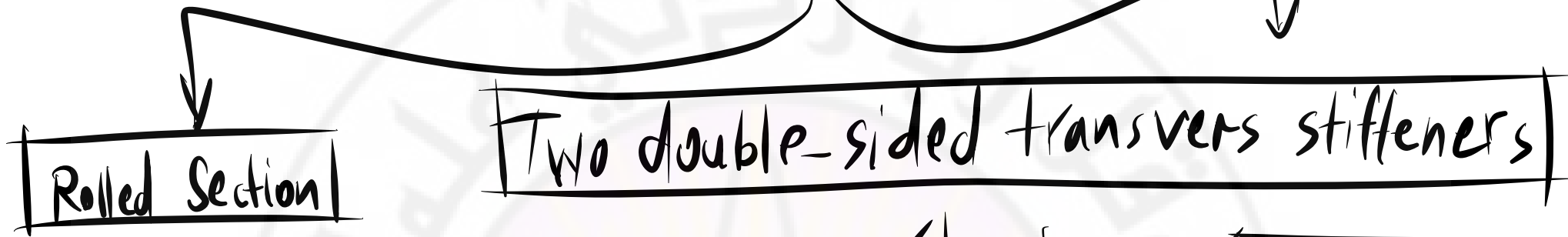
6. Stiffeners and end post

6-2 - End post Stiffener



Rigid end post: (a) formed by a rolled section, and (b) formed by a pair of double-sided stiffeners

Rigid End Posts



Rigid End Posts



Tension Field

longitudinal membrane forces arising from Tension field action

as per [clause 9.3.1 (3) of BS EN 1993-1-5] as:

1- Case of REP-R

$$N_{min} = 4 h w t^2$$

2- Case of REP-S

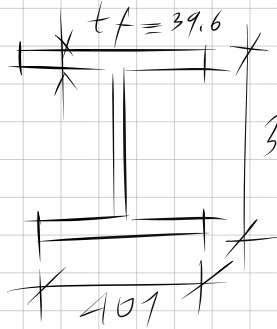
$$A_{min} = 4 h w t^2 / e$$

Training: (T1)

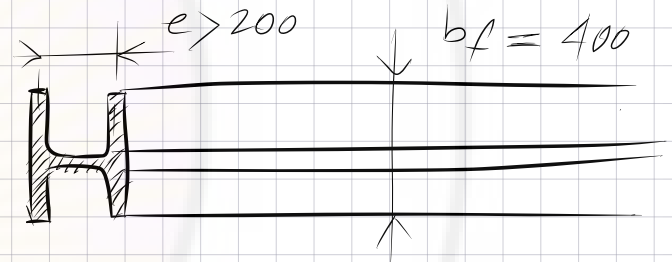
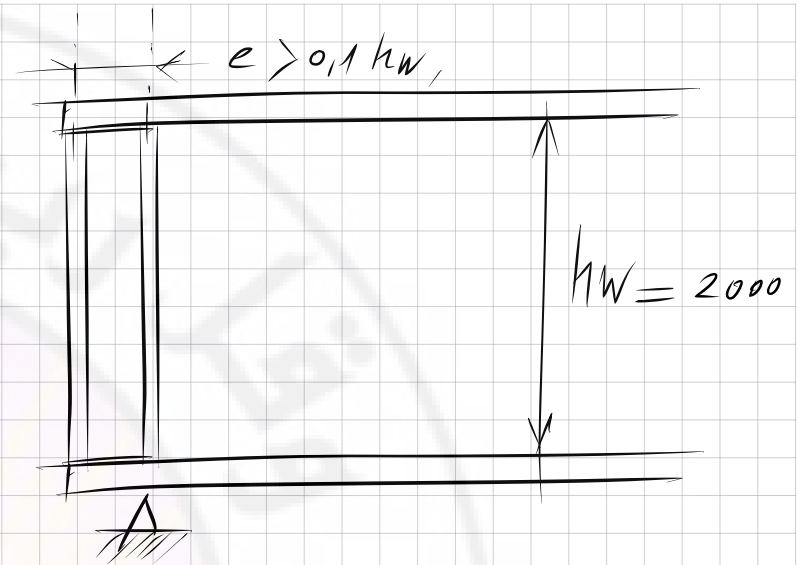
1.) check hot rolled requirement to be rigid end post - case **REP-R**

Select hot rolled with $b_f \approx 400$, $D > 200$

choose Wide Flange column HD 400x314



$$399, \text{ HD } 400 \times 314, W_{el} = 5525 \text{ cm}^3 \\ = 5525000 \text{ mm}^3$$



check for e ; $e = 399 - 39.6 = 359.7 > 0.1 h_w = 200 \text{ mm}$

check for stiffness as per [clause 9.3.1 (3) BS EN 1993-1-5]

$$W_{min} = 4 h_w t_w^2 = 4 (2000) (8)^2 = 512,000 \text{ mm}^2$$

$$\left\{ W_{el} = 5,525 \times 10^6 > W_{min} = 0,512 \times 10^6 \right\} \quad \text{OK}$$

Training: (T2)

1.) check Stiffness requirement to be rigid end post - Case **REPS**

$$\begin{aligned} A_{end} &= ① + ② + ③ \\ &= 160 \times 15 + 160 \times 15 + 8 \times 15 \\ &= 4920 \text{ mm}^2 \end{aligned}$$

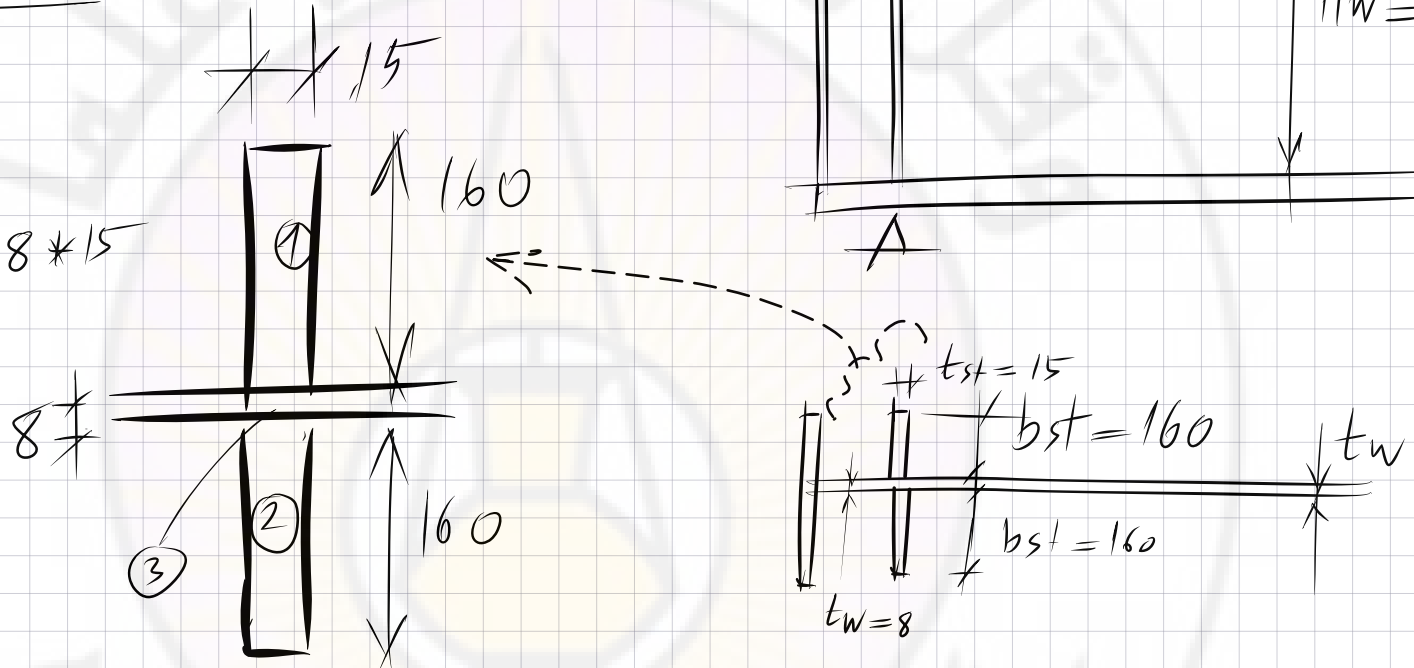
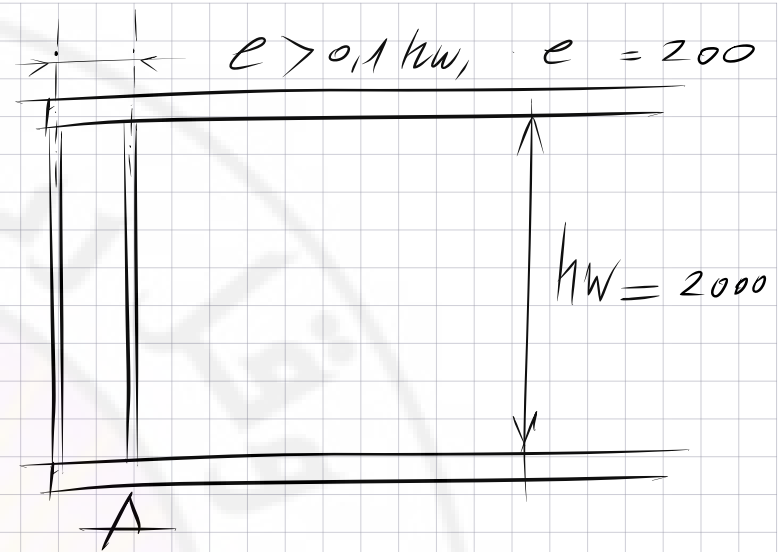
A_{end} must be greater than

$$A_{min} = 4 h_w t_w^2 / e$$

[clause 9.3.1 (3) BS EN 1993-1-5]

$$A_{min} = 4 h_w t_w^2 / e = 4 \times 2000 (8)^2 / 200 = 2560 \text{ mm}^2$$

$$\{ A_{end} = 4920 > A_{min} = 2560 \} \text{ OK}$$



6. Stiffeners and end post

6-2 - End post Stiffener

The magnitude of the longitudinal membrane force N_H to be resisted by a rigid end post is not specified in BS EN 1993-1-5, but has been derived in many References for a perfect plate and modified for design to give:

$$N_H = h_w t_w \left(\frac{\tau_{Ed}^2}{\tau_{cr} / 1.2} - \tau_{cr} / 1.2 \right) \geq 0$$

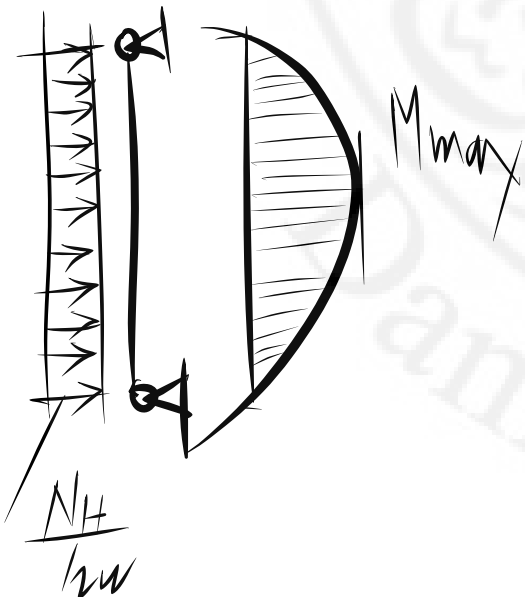
in which τ_{Ed} may be taken as $V_{Ed} / h_w t_w$ and τ_{cr} is the elastic shear buckling stress of the web. This membrane force may be assumed to act on the end post as a uniformly distributed load, which gives a maximum moment at mid-height of $N_H h_w / 8$.

Anchorage Beam



$$NH = h_w \cdot t_w \left(\frac{\tau_{Ed}^2}{\tau_{cr/1.2}} - \tau_{cr/1.2} \right) \geq 0 \quad (\text{KN})$$

$$M_{max} = NH \cdot h_w / 8 \quad (\text{KN-m})$$



$$\tau_{Ed} = \frac{V_{Ed}}{h_w t_w}, \quad \tau_{cr} = k_{\tau} \cdot G_E, \quad G_E = 190000 \left(\frac{t}{b} \right)^2 \text{ N/mm}^2$$

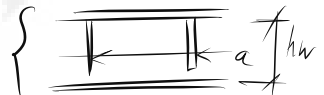
Stiffened

Unstiffened

$$k_{\tau} = 5.34 + 4.00 (h_w/a)^2 \rightarrow a/h_w > 1$$

$$k_{\tau} = 4.00 + 5.34 (a/h_w)^2 \rightarrow a/h_w < 1$$

$$k_{\tau} = 5.34$$



Training: (T3)

check the rigid end post beam
against membrane force arising
from tension field action on ①

$$V_{Ed} = 647 \times 10^3 \text{ N}, h_w = 2000 \text{ mm}, t_w = 8 \text{ mm}$$

$$b = a = 3000 \text{ mm}, S235$$

$$a/h_w = 3000/2000 = 1.5 > 1$$

$$K_\tau = 5.34 + 4.00 (h_w/a)^2$$

$$= 5.34 + 4.00 (2000/3000)^2 = 7.1$$

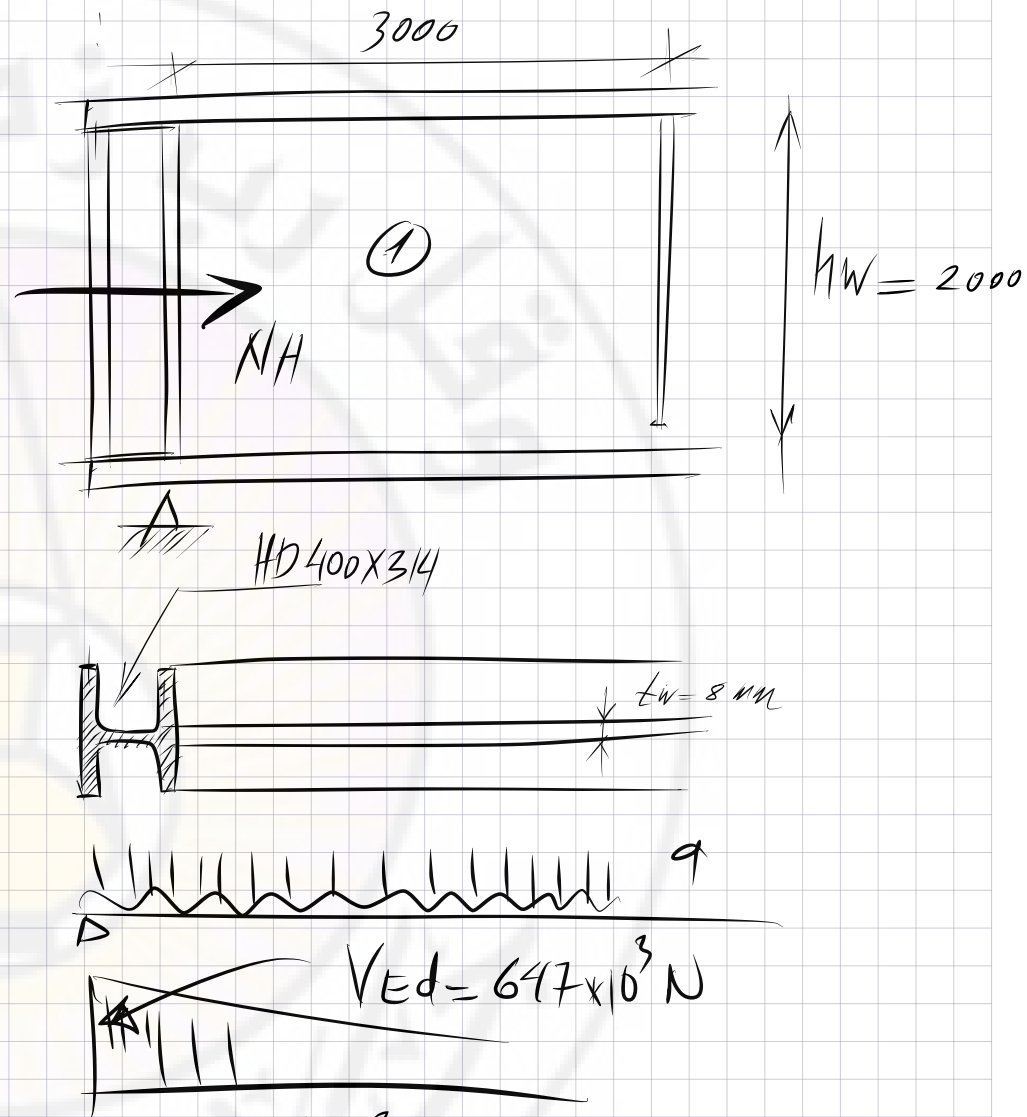
$$\sigma_E = 190000 \left(\frac{t_w}{b}\right)^2 = 190000 \left(\frac{8}{3000}\right)^2 = 1.351 \text{ N/mm}^2$$

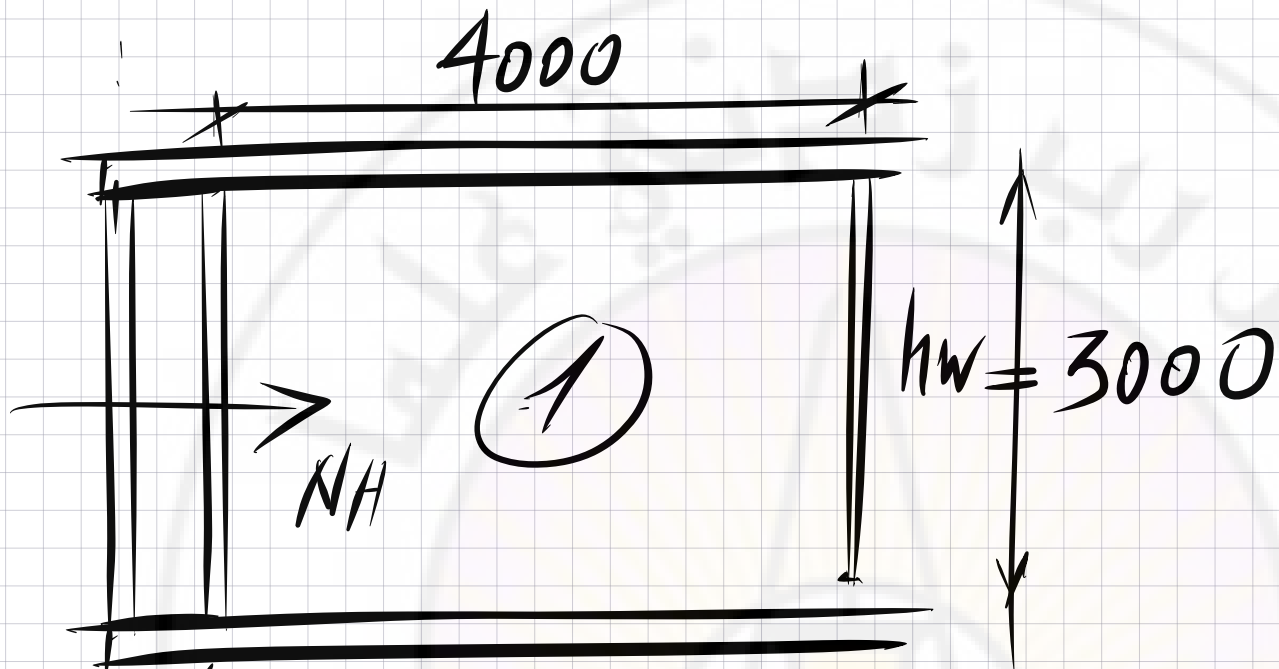
$$\tau_{cr} = K_\tau \cdot \sigma_E = 7.1 \times 1.351 = 9.64 \text{ N/mm}^2$$

$$\tau_{Ed} = V_{Ed} / h_w \cdot t_w = 647 \times 10^3 / (2000 \times 8) = 40.44 \text{ N/mm}^2$$

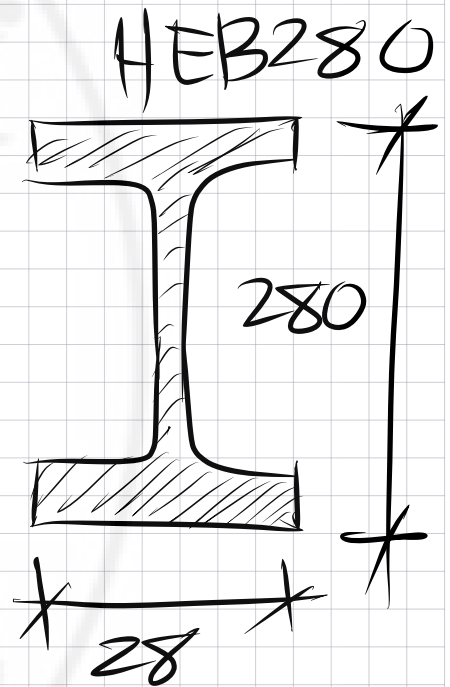
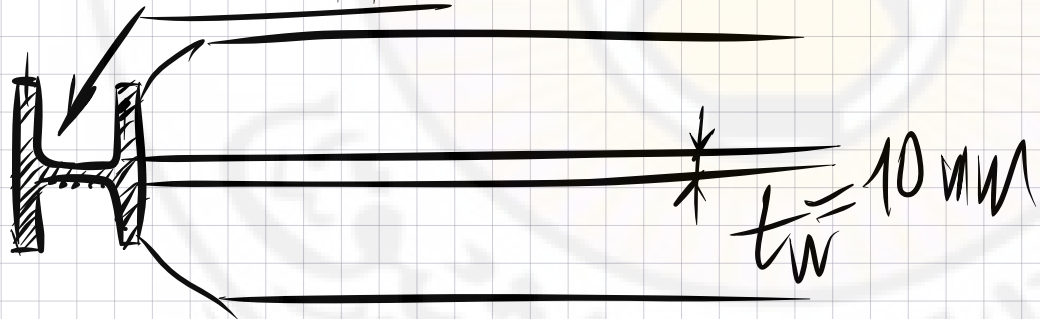
$$N_H = h_w \cdot t_w \left(\frac{\tau_{Ed}^2}{\tau_{cr}/1.2} - \tau_{cr}/1.2 \right) = 2000 \times 8 \left[\frac{(40.44)^2}{9.64/1.2} - 9.64/1.2 \right]$$

$$= 313 \times 10^6 \text{ N}$$





HEB280



$V_{Ed} = 700 \times 10^3 \text{ N}$

$A = 131.4 \text{ cm}^2$
 $I_x = 19270 \text{ cm}^4$
 $W_y = 1376 \text{ cm}^3$

Training: (T3)

$$\frac{NH}{h_w} = \frac{31,3 \times 10^6}{2000}$$

$$= 15,65 \times 10^3 \text{ N/m}$$

$$M_{\max} = \frac{NH}{h_w} \times \frac{h_w^2}{8}$$

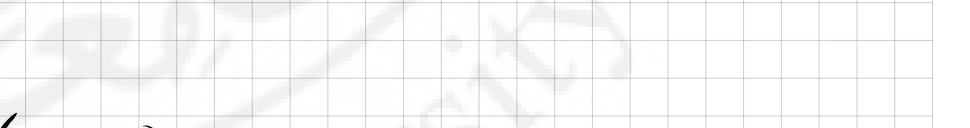
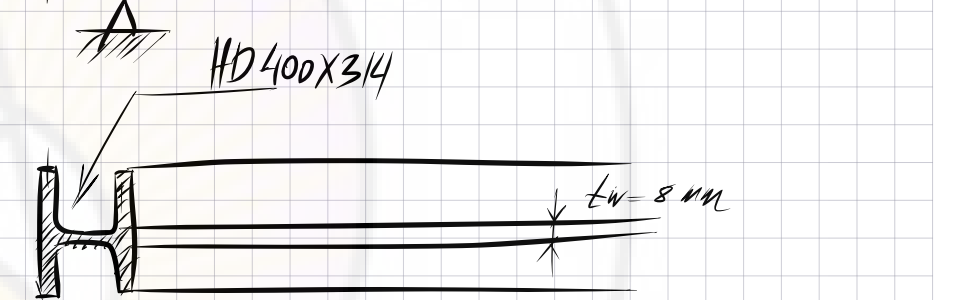
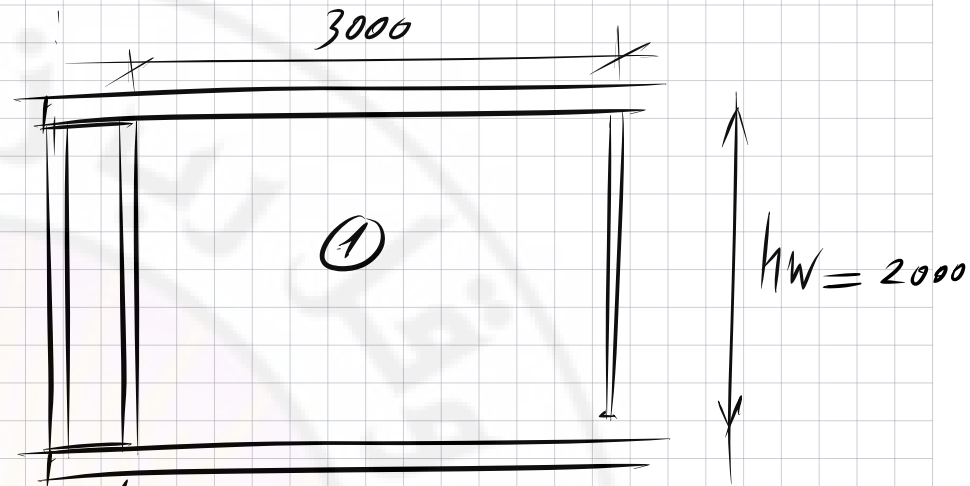
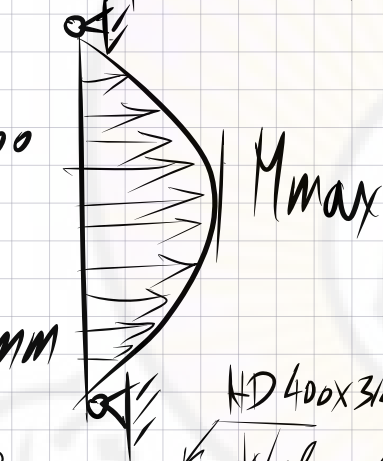
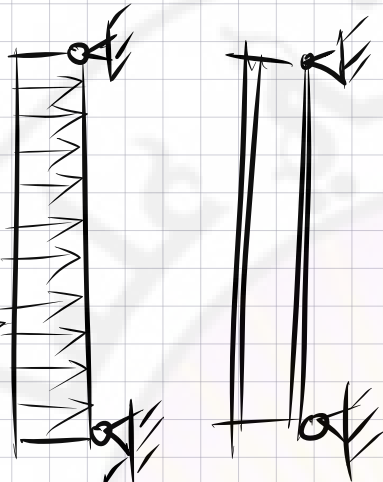
$$= \frac{15,65 \times 10^3 \times 2000}{8}$$

$$= 3912,5 \times 10^3 \text{ N}\cdot\text{mm}$$

$$f_b = \frac{M}{W_{el}}$$

$$= \frac{3912,5 \times 10^3}{5525000}$$

$$= 0,71 \text{ N/mm}^2 < f_y = 235 \text{ N/mm}^2 \text{ (S235)} \rightarrow \text{OK}$$



Training: (T4)

check the rigid end Post Plates
against membrane force arising
from tension field action on ①

$$V_{Ed} = 647 \times 10^3 \text{ N}, h_w = 2000 \text{ mm}, t_w = 8 \text{ mm}$$

$$b = a = 3000 \text{ mm}, S235$$

$$a/h_w = 3000/2000 = 1.5 > 1$$

$$K_z = 5.34 + 4.00 (h_w/a)^2$$

$$= 5.34 + 4.00 (2000/3000)^2 = 7.1$$

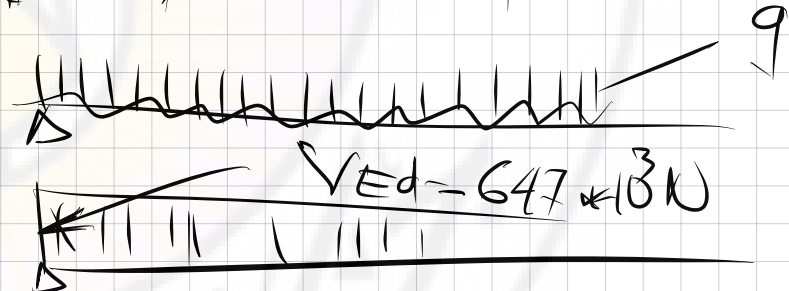
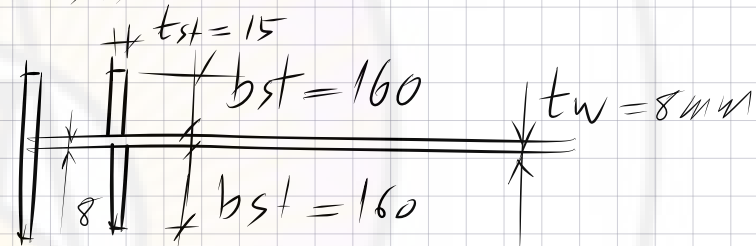
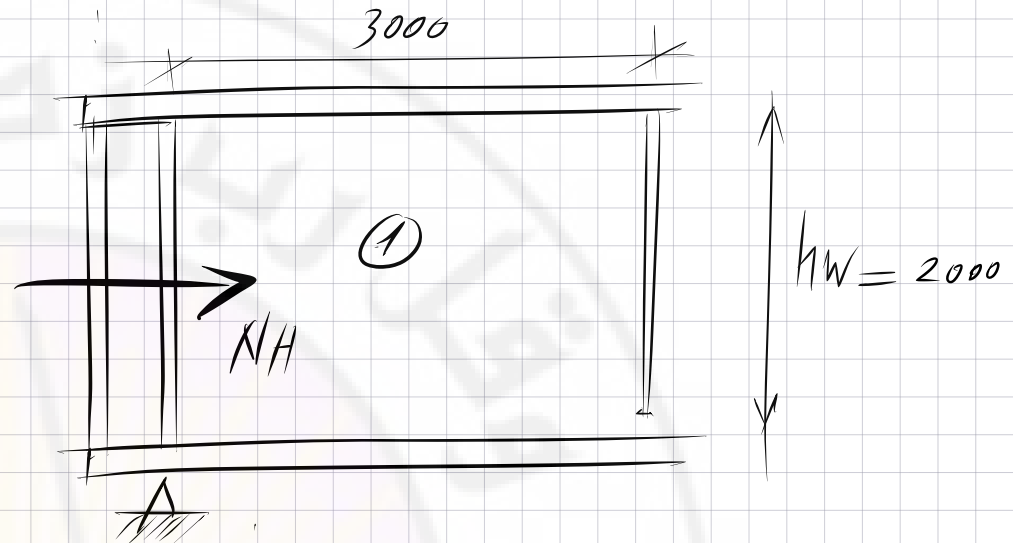
$$\sigma_E = 190\,000 \left(\frac{t_w}{b} \right)^2 = 190\,000 \left(\frac{8}{3000} \right)^2 = 1.351 \text{ N/mm}^2$$

$$\tau_{cr} = K_z \sigma_E = 7.1 \times 1.351 = 9.64 \text{ N/mm}^2$$

$$\tau_{Ed} = V_{Ed} / h_w \cdot t_w = 647 \times 10^3 / (2000 \times 8) = 40.44 \text{ N/mm}^2$$

$$N_H = h_w \cdot t_w \left(\frac{\tau_{Ed}^2}{\tau_{cr}/1.2} - \tau_{cr}/1.2 \right) = 2000 \times 8 \left[\frac{(40.44)^2}{9.64/1.2} - 9.64/1.2 \right]$$

$$= 313 \times 10^6 \text{ N}$$



Training: (T4)

$$\frac{NH}{hw} = \frac{31.3 \times 10^6}{2000}$$

$$= 15.65 \times 10^3 \text{ N/m}$$

$$M_{\max} = \frac{NH}{hw} \times \frac{hw}{8}$$

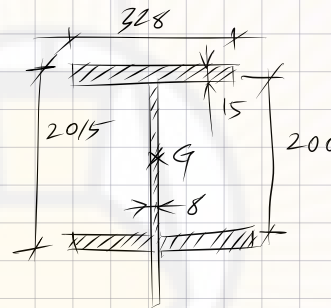
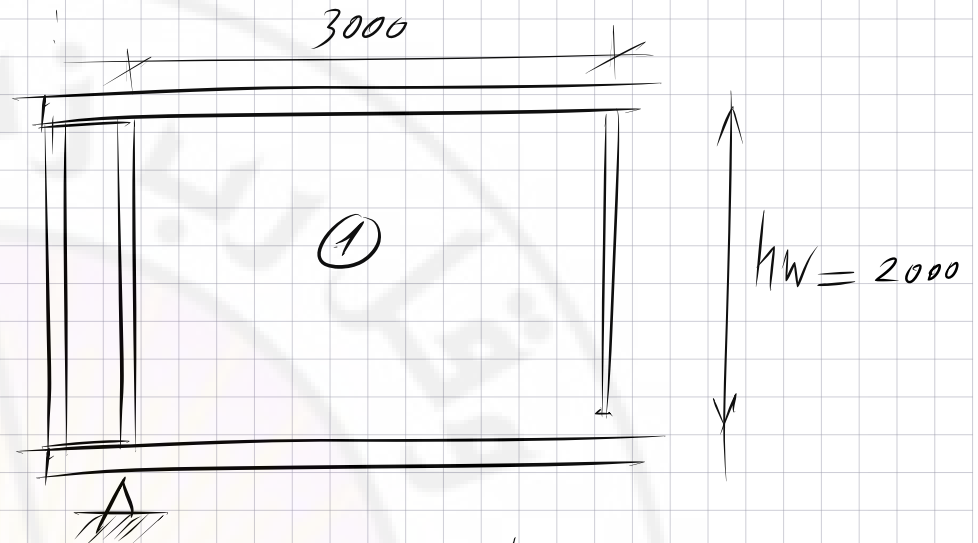
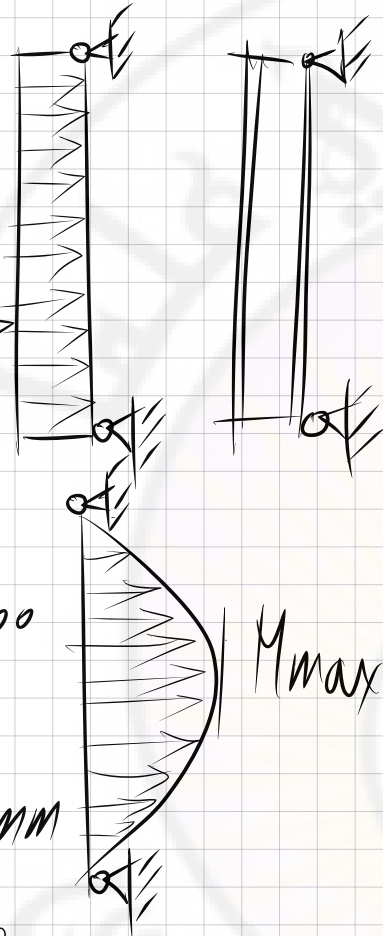
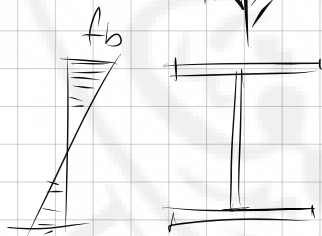
$$= \frac{15.65 \times 10^3 \times 2000}{8}$$

$$= 3912.5 \times 10^3 \text{ N}\cdot\text{mm}$$

$$f_b = \frac{M}{W_{el}}$$

$$= \frac{3912.5 \times 10^3}{856 \times 10^3}$$

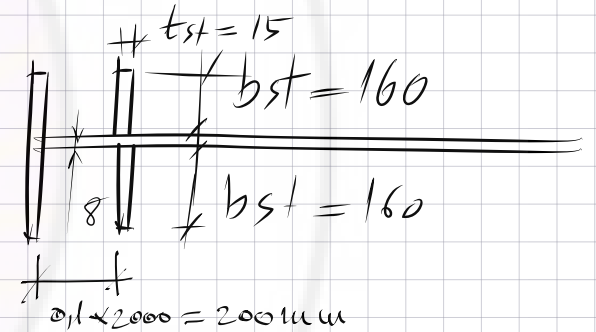
$$= 4.57 \text{ N/mm}^2 \ll f_y = 235 \text{ N/mm}^2 \quad (S235)$$



$$I \approx 328 \times 15 \times \left(\frac{200}{2}\right)^2 \times 2$$

$$= 984 \times 10^6 \text{ mm}^4$$

$$W_{el} = \frac{I}{y/2} = \frac{984 \times 10^6}{230/2} = 856 \times 10^3 \text{ mm}^3$$



6. Stiffeners and end post

6-2 - End post Stiffener

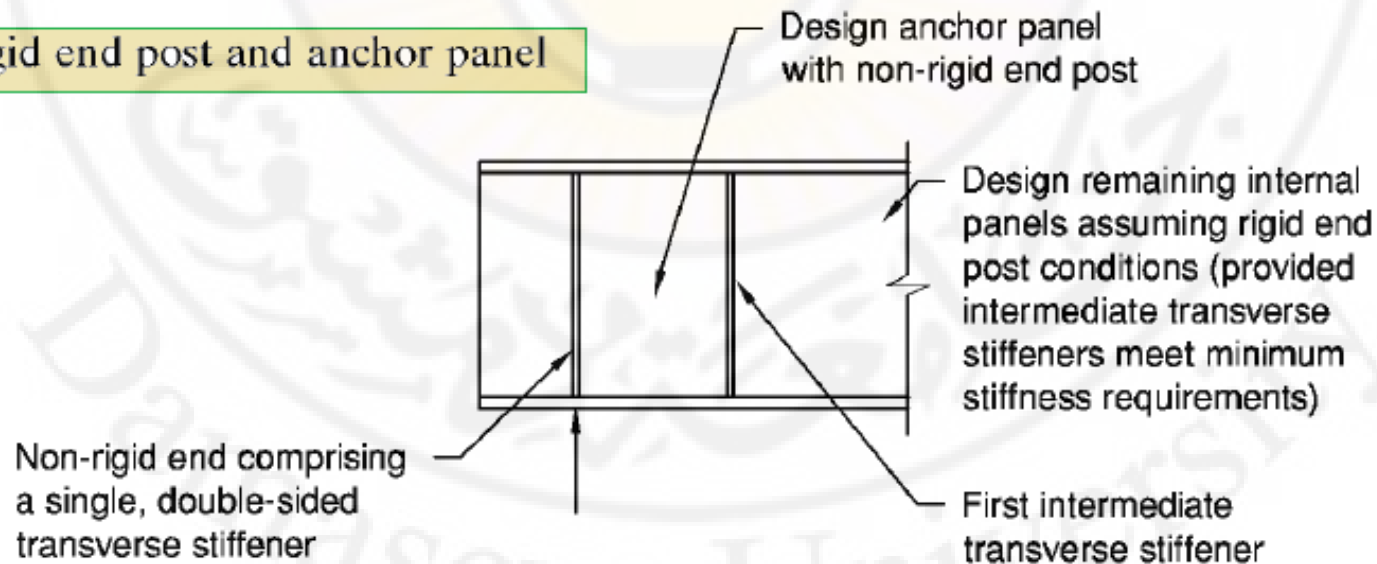
Non-rigid end posts

A non-rigid end post comprises only a single double-sided stiffener. The reduced anchorage, compared with a rigid end post, afforded to the adjacent web panel, results in lower shear buckling capacity for slender webs ($\bar{\lambda}_w > 1.08$)

Non-rigid end posts should be designed as bearing stiffeners,

They do not need to be designed to carry any tensile anchorage force, since this is assumed to be resisted by the web itself.

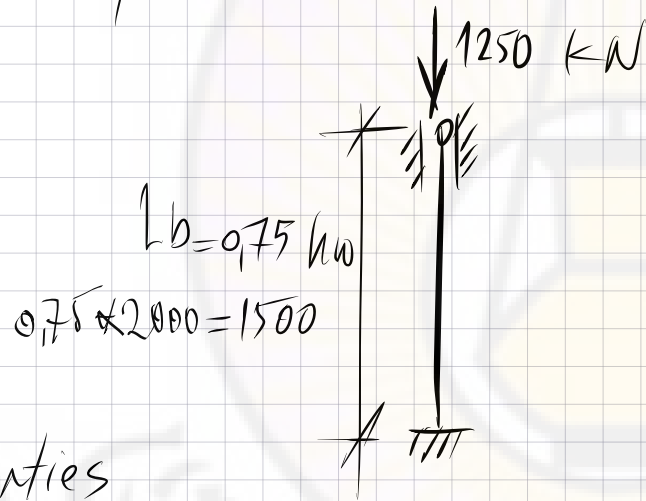
Non-rigid end post and anchor panel



Training: (T5)

check the non rigid end
stiffener and the reaction force
 $V_{Ed} = 2500 \text{ kN}$, S235

The stiffener and part of web act
as column,

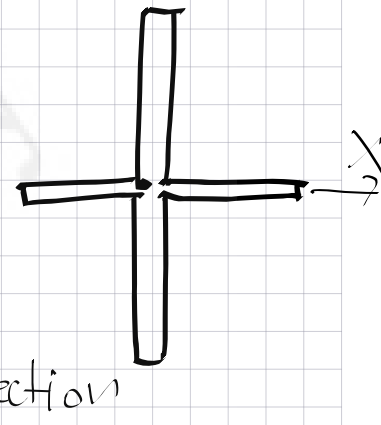
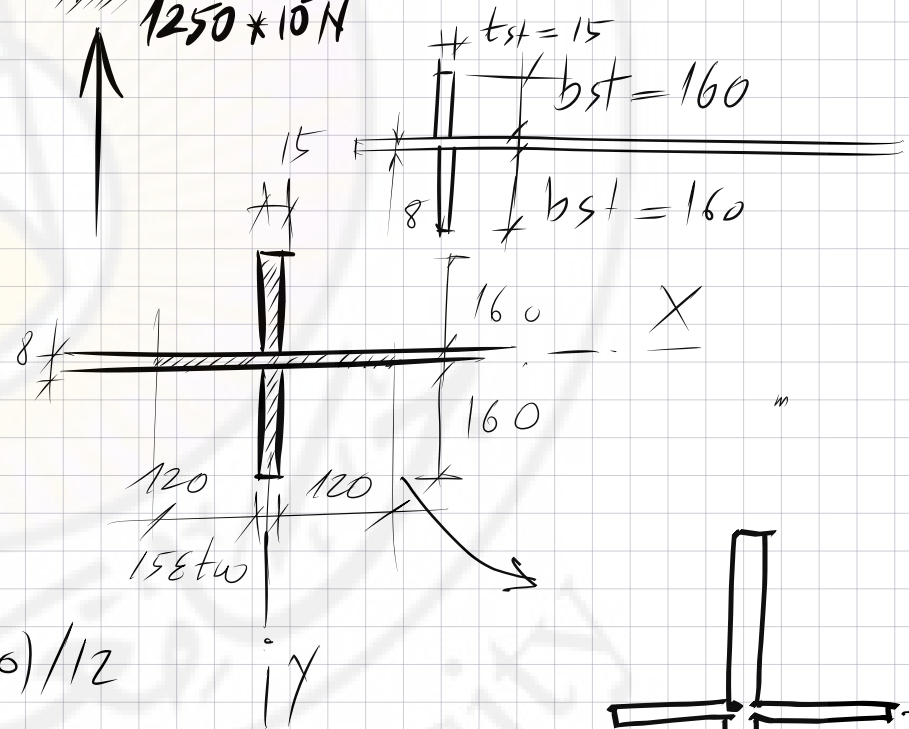
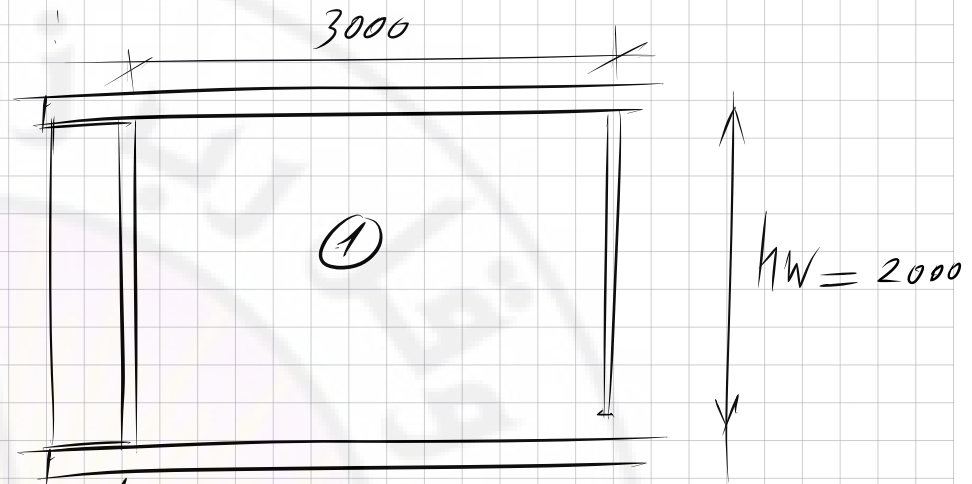


Section Properties

$$A_{st} = 2(160 \times 15 + 120 \times 8) = 6720 \text{ mm}^2$$

$$I_{stx} = (160 + 8 + 160) \cdot 15^3 / 12 + 8^3 \cdot (120 + 15 + 120) / 12$$

$$= 44120 \times 10^3 \text{ mm}^4$$



+ section

Training: (T5)

$$A_{st} = 6720 \text{ mm}^2, I_{st y} = 44120 \times 10^3 \text{ mm}^4$$

$$i_{st} = \sqrt{I_{st y} / A_{st}} = \sqrt{44120 \times 10^3 / 6720} \\ = 81 \text{ mm}$$

$$\lambda = \frac{L_b}{i_{st}} = \frac{1500}{81} = 18,52$$

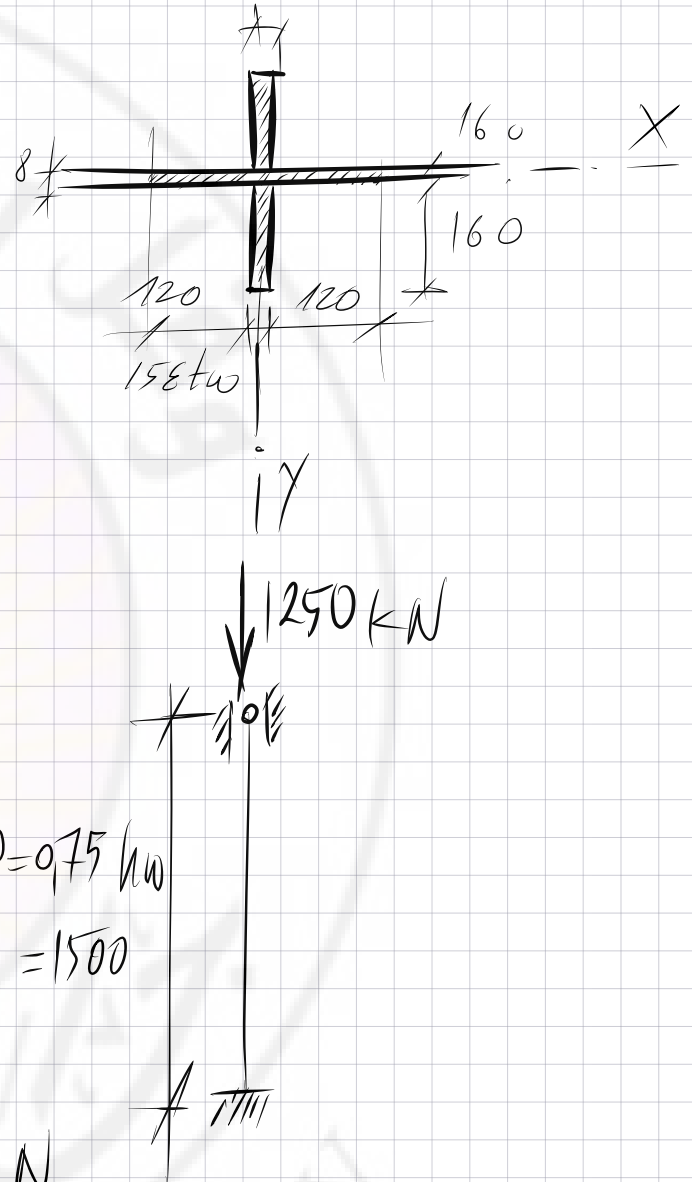
$$\lambda_1 = 93,8 = 93$$

$$\bar{\lambda} = \frac{18,52}{93} = 0,2 = 0,2$$

$$\rightarrow \chi = 1$$

$$N_{b,Rd} = \frac{\chi A_{st} f_y}{\gamma_{M1}} = \frac{6720 \times 235}{1,1} = 1435,6 \times 10^3 \text{ N}$$

$$N_{b,Rd} > N_{Ed} = 1250 \times 10^3 \text{ N} \rightarrow \text{ok}$$



6. Stiffeners and end post

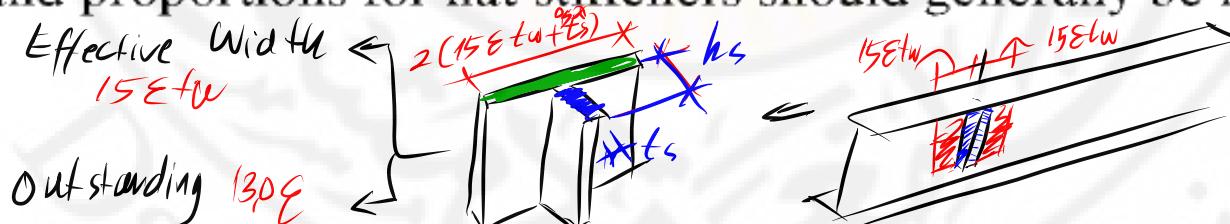
6-3 - Web stiffeners

Types of stiffeners

Transverse stiffeners are generally required to ensure the satisfactory performance of the web panels of slender plate girders. The two most common types of stiffeners are (1) intermediate transverse stiffeners to increase the shear capacity of the web, and (2) bearing or load-bearing stiffeners to prevent local failure of the web under concentrated loads or reactions applied through the flange. A particular stiffener may serve more than one function, in which case it should be designed for the combined effects: e.g. an intermediate transverse stiffener may also be load-bearing. The effective cross-section of a stiffener (used both for verifying minimum stiffness and for buckling checks) is defined in Clause 9.1(2) as the stiffener(s) plus a width of web equal to $15\epsilon t_w$, but clearly not greater than the actual material available.

The outstand proportions for flat stiffeners should generally be limited to:

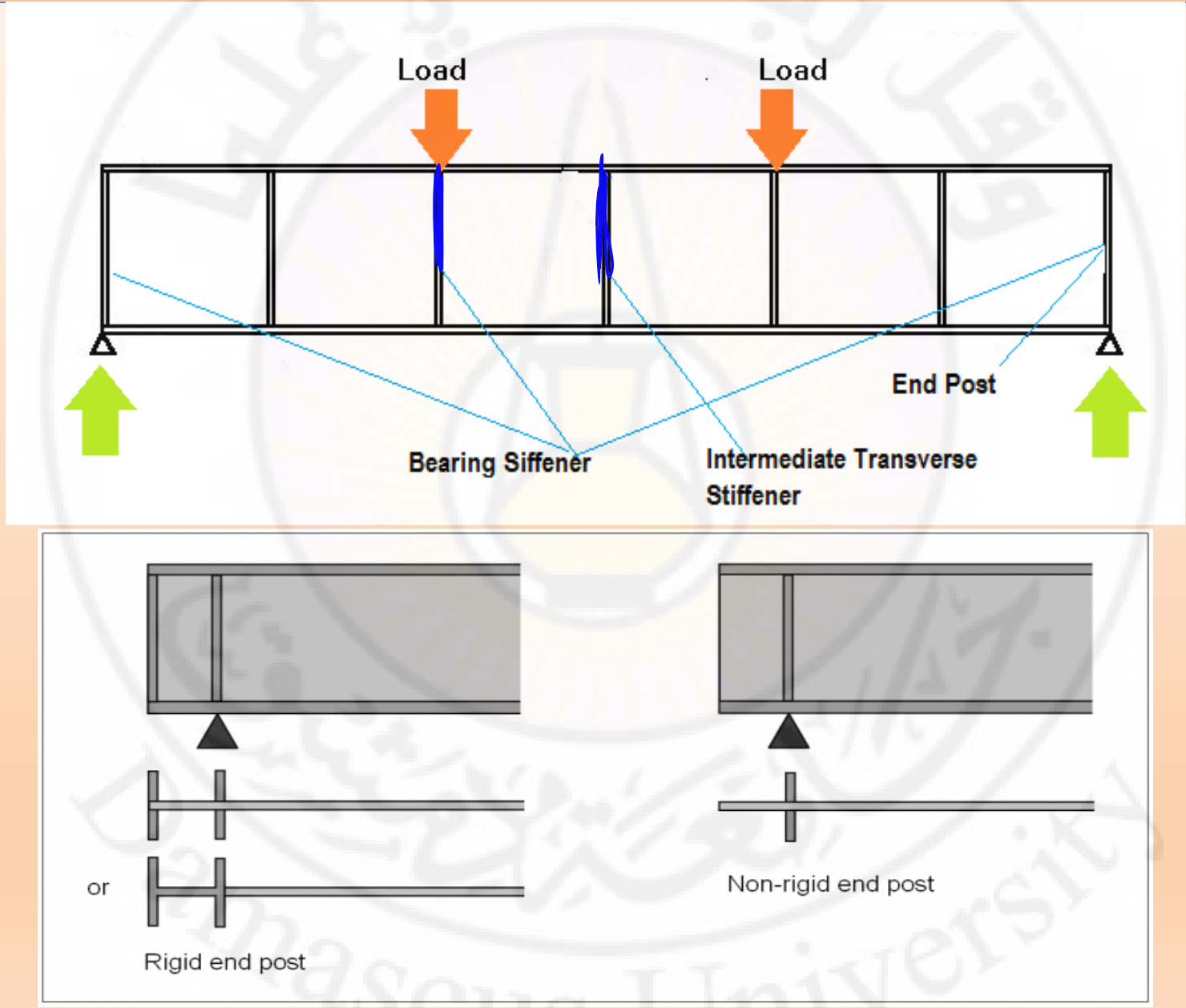
$$\frac{h_s}{t_s} \leq 13.0\epsilon$$



in order to avoid torsional buckling, where h_s and t_s are the outstand width and thickness of the stiffener, respectively. This requirement may be derived from that set out in Clause 9.2(8) of BS EN 1993-1-5.

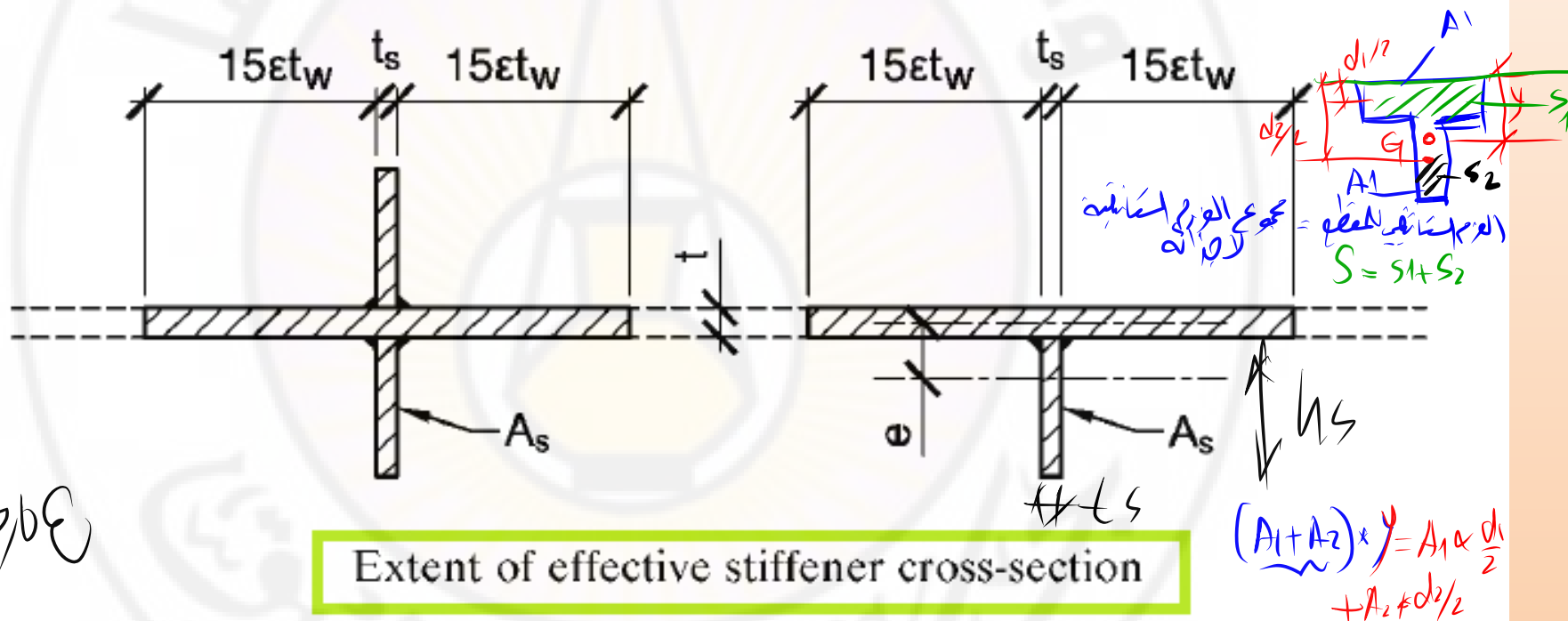
6. Stiffeners and end post

6-4 – Bearing stiffeners



6. Stiffeners and end post

6-3 - Web stiffeners



6. Stiffeners and end post

6-3 - Web stiffeners

Intermediate transverse web stiffeners

Intermediate transverse web stiffeners are used to increase both the elastic shear buckling resistance and ultimate shear buckling resistance $V_{b,Rd}$, of slender web panels. They are usually placed on one side of the web only, and are designed for minimum stiffness and buckling resistance, as specified in Clause 9.3.3(1) of BS EN 1993-1-5.

Intermediate transverse web stiffeners should have the following minimum second moment of area I_{st} , (see Clause 9.3.3(3) of BS EN 1993-1-5) in order to control the lateral deflection of the web at their locations and to be considered as rigid restraints:

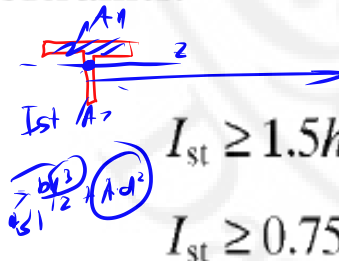
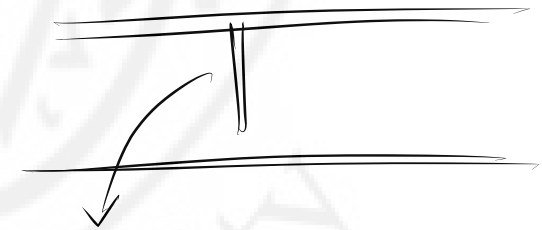


Diagram of a stiffener section showing dimensions and area components. The diagram includes a cross-section of a stiffener with a central web and two flanges. The web has a thickness t_w and the flanges have a width b_f and thickness t_f . The total width of the stiffener is a . The second moment of area I_{st} is calculated as the sum of the second moments of the flanges and the web. The diagram also shows the area A_1 and A_2 of the flanges and the web, and the distance z from the neutral axis to the outer edge of the flange.

$$I_{st} \geq 1.5 h_w^3 t_w^3 / a^2 \quad \text{for } a/h_w < \sqrt{2}$$

$$I_{st} \geq 0.75 h_w t_w^3 \quad \text{for } a/h_w \geq \sqrt{2}$$



stiffener section I_{st}

6. Stiffeners and end post

6-3 - Web stiffeners

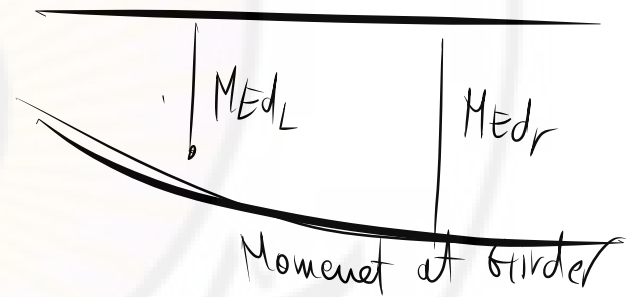
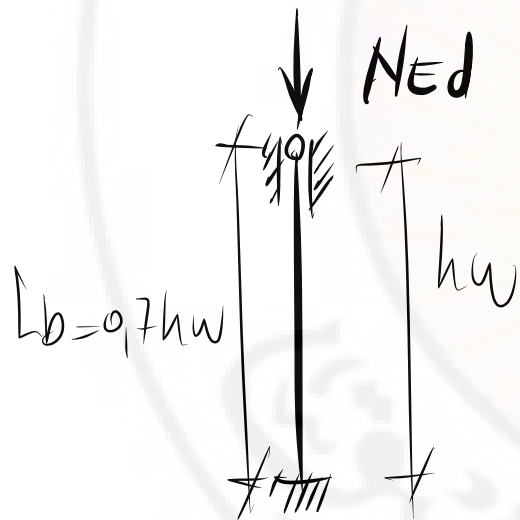
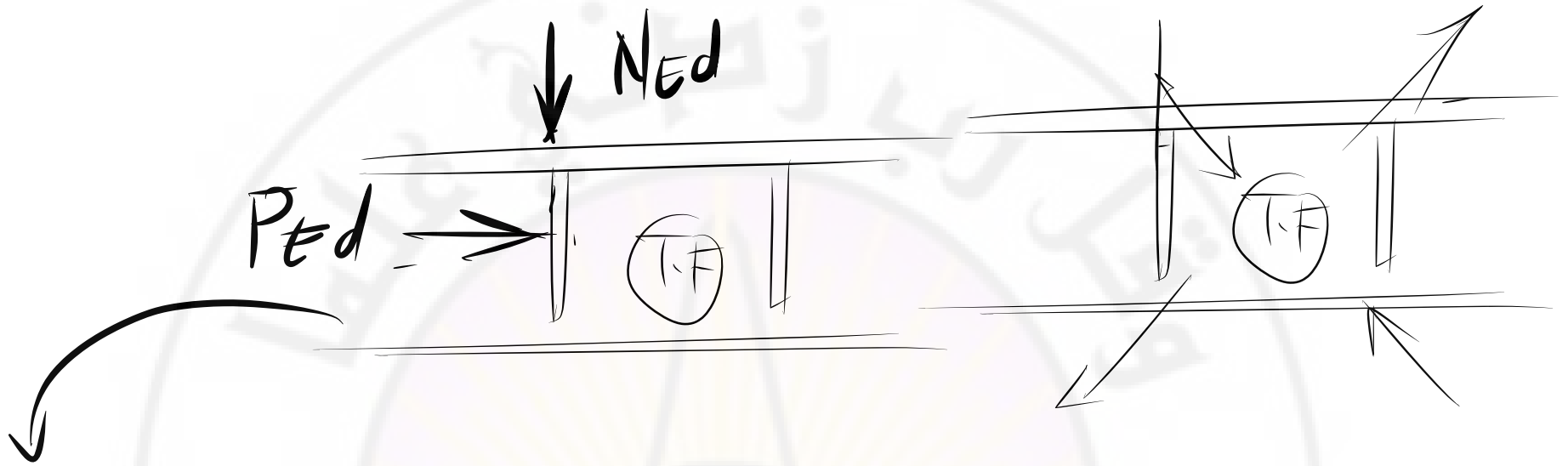
The buckling resistance of intermediate transverse web stiffeners should also be checked. The effective stiffener cross-section should be designed to resist a compressive axial force, P_{Ed} , arising from the shear-induced tension field, plus any externally applied loads and moments. P_{Ed} acts at the mid-plane of the web and therefore causes both axial and bending stresses in the effective stiffener sections. P_{Ed} is defined in Clause NA.2.7 of the UK National Annex to BS EN 1993-1-5 as:

$$P_{Ed} = V_{Ed} - 0.8\tau_{cr}h_w t_w \sqrt{1 - \frac{\sigma_{x,Ed}}{0.8\sigma_{cr,x}}} \quad \text{for } a \geq h_w$$

$$P_{Ed} = \left[V_{Ed} - 0.8\tau_{cr} h_w t_w \sqrt{1 - \frac{\sigma_{x,Ed}}{0.8\sigma_{cr,x}}} \right] \frac{a}{h_w} \quad \text{for } a < h_w$$



in which $\sigma_{cr,x}$ is the elastic plate buckling stress for the web under direct longitudinal compression, τ_{cr} is the elastic shear buckling stress of the web and $\sigma_{x,Ed}$ is longitudinal direct stress in the web, taken as zero for symmetric sections and the algebraic mean of the stresses at the top and bottom of the web in the case of asymmetric sections.



$$\frac{N_{Ed}}{\chi_y \cdot A \cdot f_y} + \frac{1}{(1 - N_{Ed}/N_{cr,y})} \cdot \frac{M_{y,Ed}}{W_{el,y} \cdot f_y} + \frac{M_{z,Ed}}{W_{el,z} \cdot f_y} \leq 1$$

(Buckling Curve c)

5. Plate Girder Shear Resistance

- **Design of intermediate transverse stiffener**

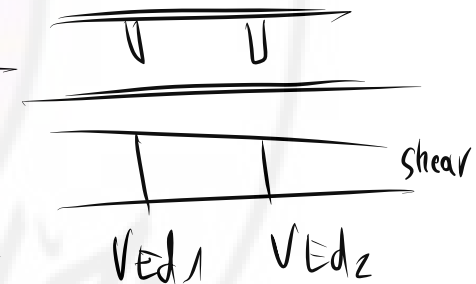
- **Verification of shear stress:** Axial force generated by TFA

It can be shown that when shear force is present, under the action of TFA, an axial force $N_{st,ten}$ will be imposed on the intermediate transverse stiffener such that

In General $\rightarrow$
$$N_{Ed} = N_{st,ten} = V_{Ed} - \frac{1}{(\bar{\lambda}_w)^2} \frac{h_w f_{yw} t_w}{\sqrt{3} \gamma_{M1}} \geq 0$$

V_{Ed} is design shear force in the adjacent panels.

- For the case of variable shear force, the value at the distance of $0.5h_w$ from the edge of the panel with the **larger** force should be taken

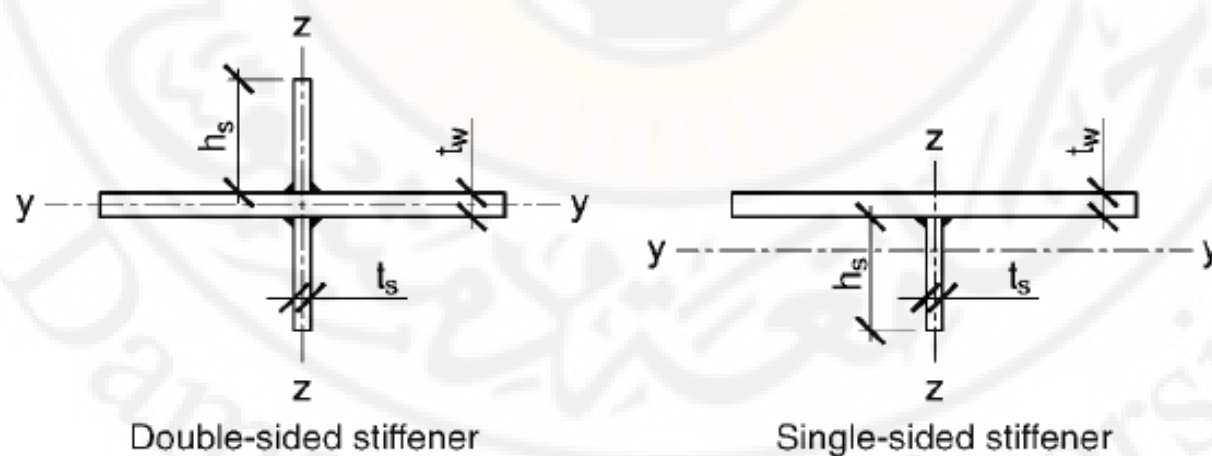


6-3 - Web stiffeners

The buckling resistance of the effective stiffener section under axial compression may be evaluated as for a conventional compression member in accordance with Clause 6.3.1 of BS EN 1993-1-1, using buckling curve c. The effective buckling length of the stiffener may be taken as $0.75h_w$ if both ends are assumed to be fixed laterally. If there is less end restraint, the effective length should be taken equal to the actual length. The resistance of the effective stiffener section under combined axial load and bending may be evaluated in accordance with Clause 6.3.3 of BS EN 1993-1-1, though a simplified interaction expression, proposed in Reference 4, that assumes no lateral torsional buckling of the stiffener, is also suitable:

$$\frac{N_{\text{Ed}}}{\chi_v A f_v} + \frac{1}{1 - (N_{\text{Ed}} / N_{\text{cr},v})} \frac{M_{y,\text{Ed}}}{W_{\text{el},v} f_v} + \frac{M_{z,\text{Ed}}}{W_{\text{el},z} f_v} \leq 1.0$$

with the axis convention $\phi = 0$ at the left end. Note that no amplifier is applied to the M_{zEd} moment since the web prevents buckling in this plane.



Axis convention for effective stiffener cross-section

6. Stiffeners and end post

6-4 – Bearing stiffeners

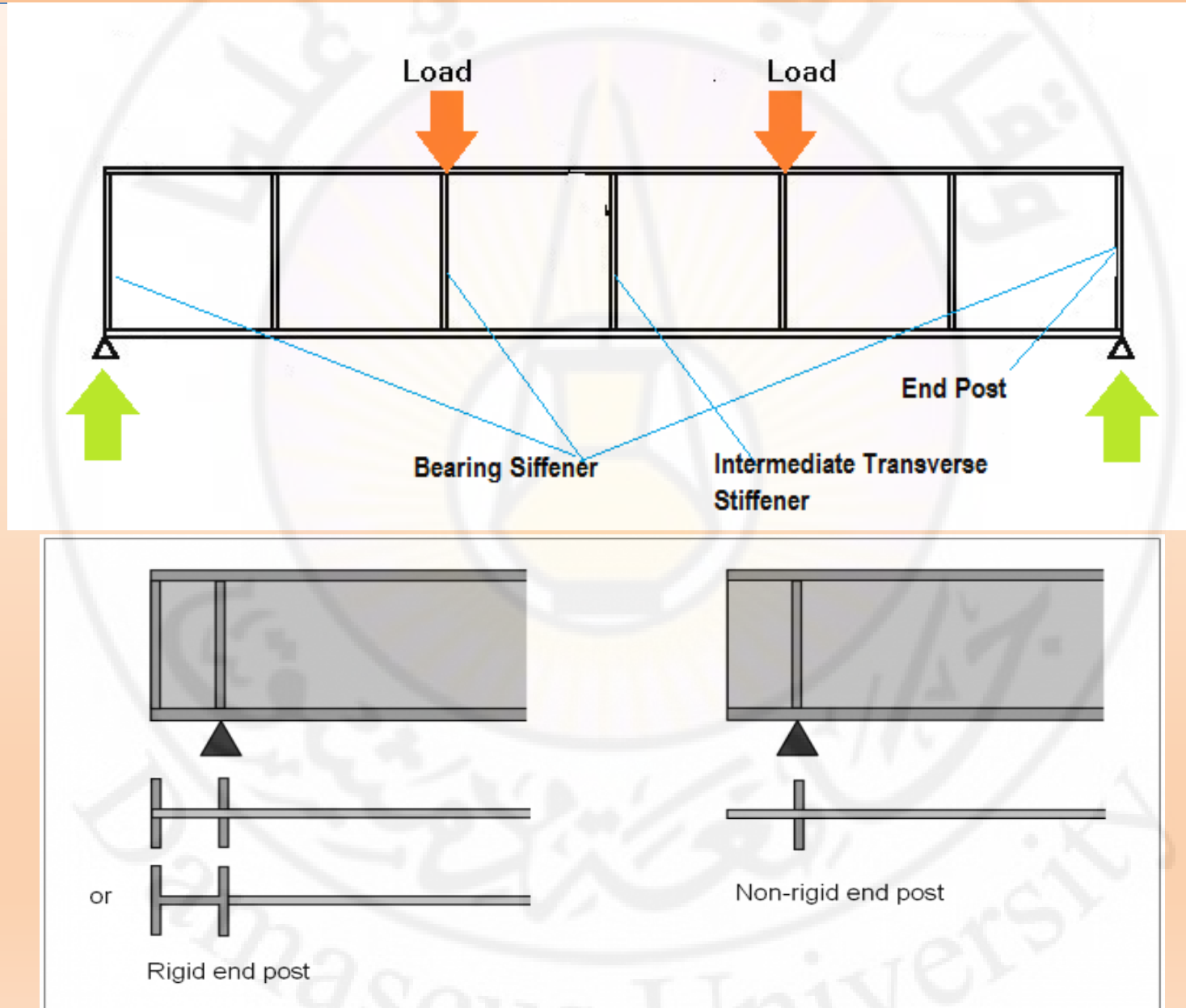
Bearing or load-bearing stiffeners

Bearing or load-bearing stiffeners are employed to prevent local failure of the web under concentrated loads or reactions applied through the flange. Clause 5.1(2) of BS EN 1993-1-5 requires such stiffeners to be provided at the supports of beams in which the web slenderness h_w/t_w exceeds $72\varepsilon/\eta$. Bearing stiffeners should generally be placed symmetrically about the web centre-line, else the resulting eccentricity should be allowed for in their design (see Clause 9.4(3) of BS EN 1993-1-5). Other than at the supports, load-bearing stiffeners are required at the location of any concentrated loads, if the resistance of the unstiffened web, evaluated in accordance with Clause 6 of BS EN 1993-1-5, would otherwise be exceeded (see Clause 9.4(1) of BS EN 1993-1-5). Clause 6 of BS EN 1993-1-5 also applies to patch loading, when concentrated loads act between stiffener locations; this is common, for example, in the case of travelling loads.

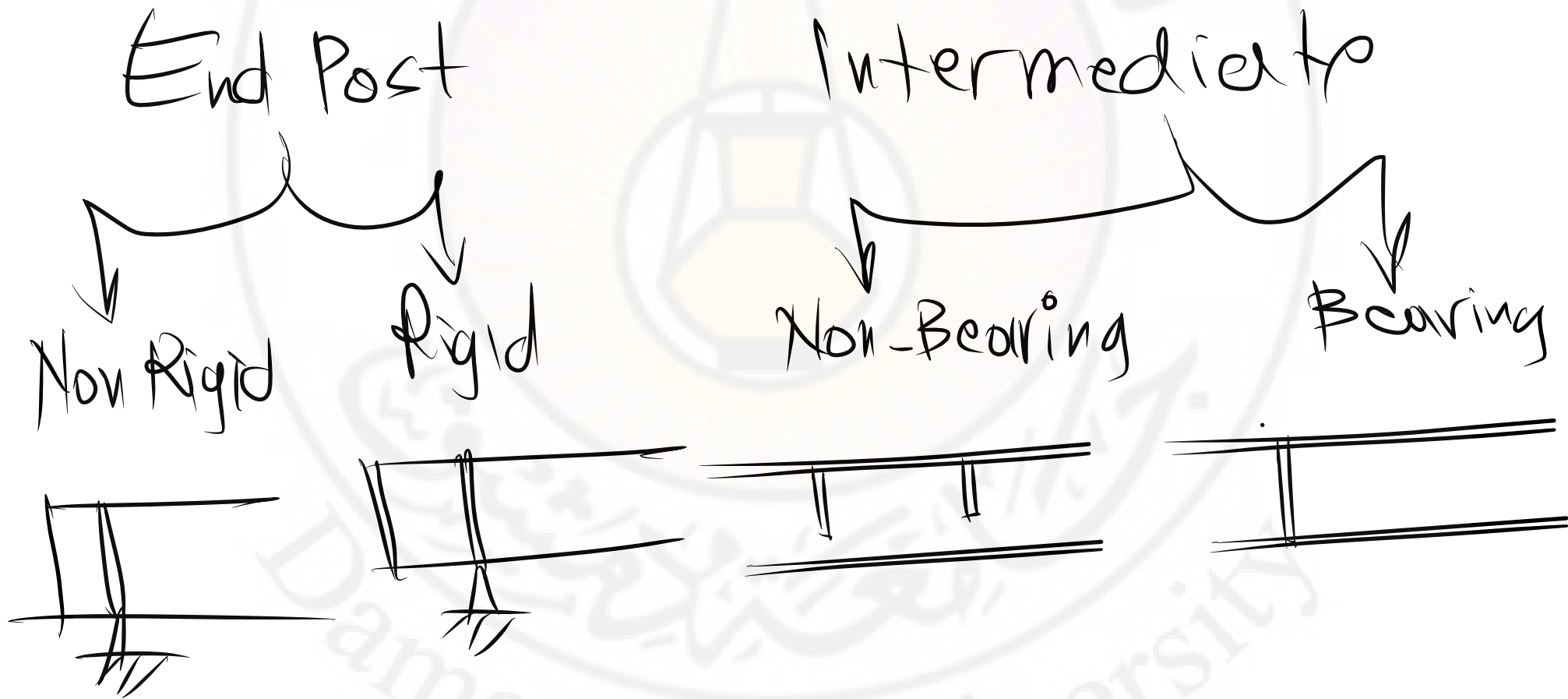
Checks for cross-section resistance and buckling resistance under combined loading are required as described above. The simplified interaction expression may also be applied to load-bearing stiffeners. Note that, for bearing stiffeners that also act as rigid end posts, the additional bending moments associated with anchoring of the tension field must be considered in conjunction with the support reaction and that, different cross sections are defined when resisting the different loading components.

6. Stiffeners and end post

6-4 – Bearing stiffeners



Summary for All Stiffeners



6. Stiffeners and end post

6-5 - Summary - Stiffeners and end post design

• Intermediate transverse stiffener

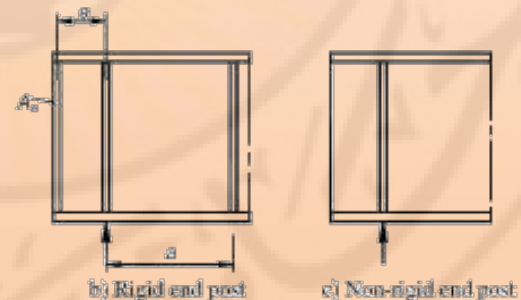
- Takes no direct external loading but subjected to internal direct and shear force
- Creates compartments to increase web buckling resistance
- Provides **rigid support** for TFA and flange resistance (PH) for web buckling

• Load bearing stiffener

- Prevent yielding, crushing, local and global failures at where heavy patch loads are applied

• End Post

- A special form of load bearing stiffener
- As “anchor” supports for TFA, flange resistance (PH) and reaction forces
- Rigid and non-rigid end posts are possible



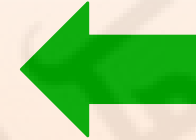
TFA : Tension Field Action , PH : Plastic Hinge

5. Plate Girder Shear Resistance

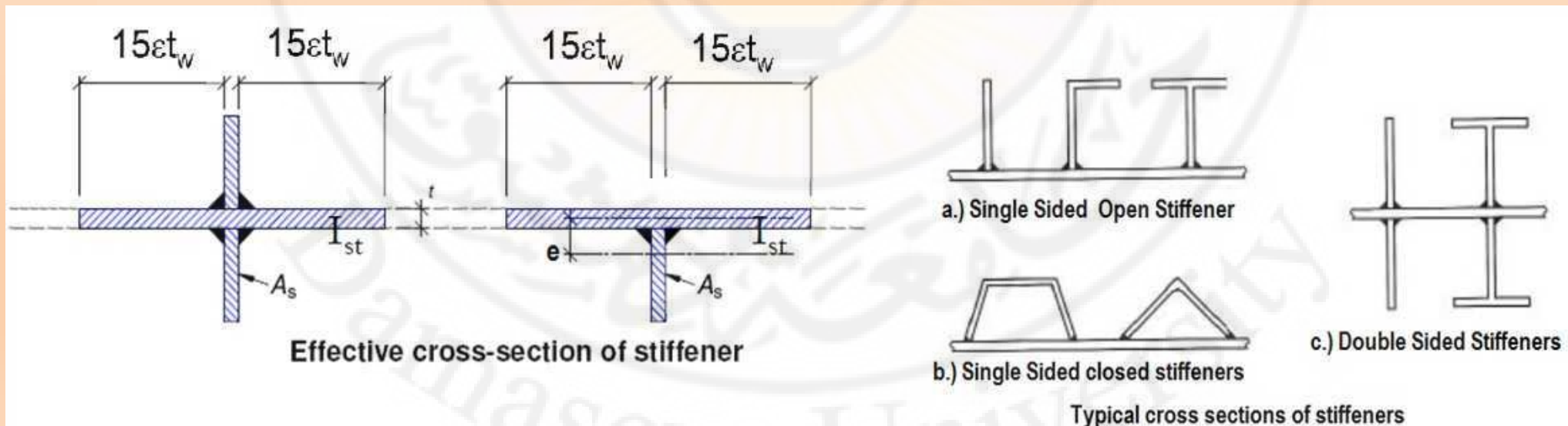
6-5 - Summary

- Stiffeners and end post design

- **General requirements of all stiffeners**



- Independent design check needed
- Cross section of the stiffener consists of
 - (1) gross area of the stiffener itself, and
 - (2) contribution width of $15\epsilon t_w$ on each side (but no overlapping)
- Normally, thin-walled open section (at least Class 2) are used

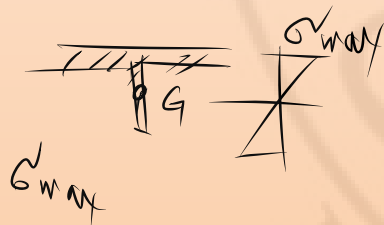


5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design

- **General requirements of all stiffeners**
 - Two **general performance requirements** for **ALL** transverse stiffeners (intermediate, load bearing and end post) at section where $M_{Ed} \neq 0$
 - **Requirement (A) Stress and deflection limits:** To verify using a **second order elastic analysis** that at the ultimate limit state



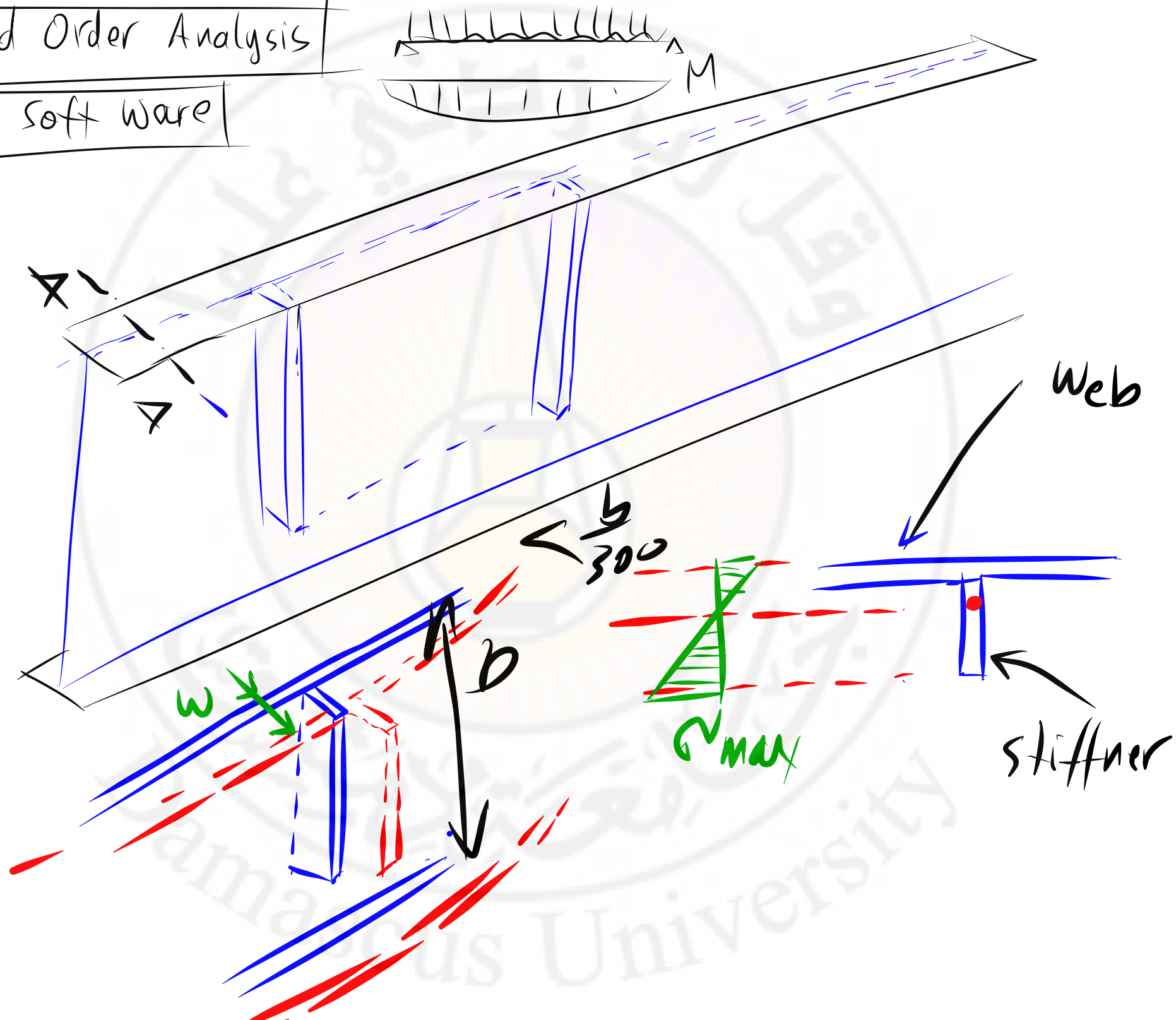
$$\sigma_{max} \leq \frac{f_y}{\gamma_{M1}} \quad \text{and} \quad w \leq \frac{b}{300}$$



σ_{max} is the ultimate stress (elastic) and w is the ultimate lateral deflection while b is the panel height (or web height)

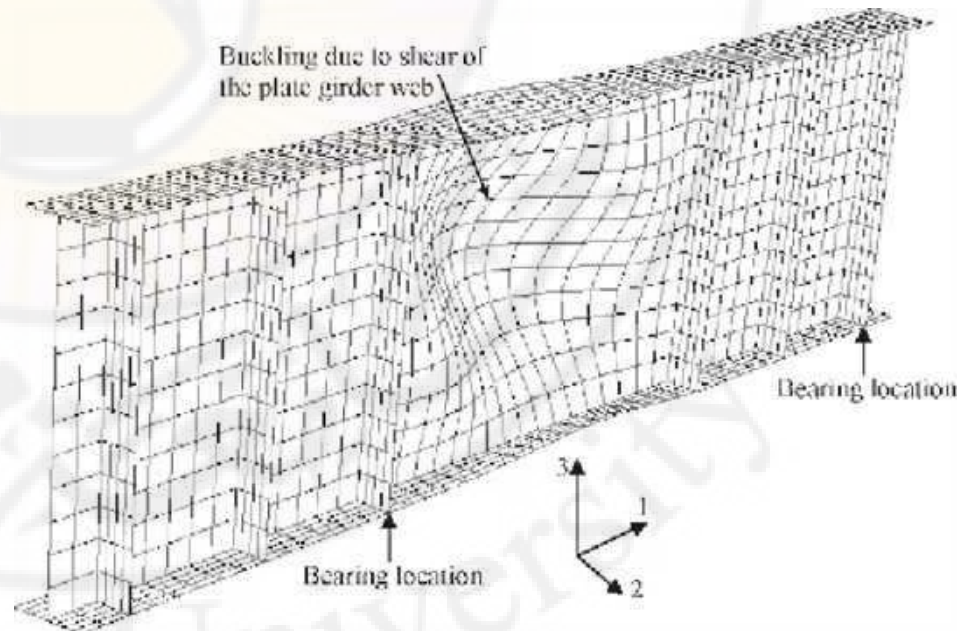
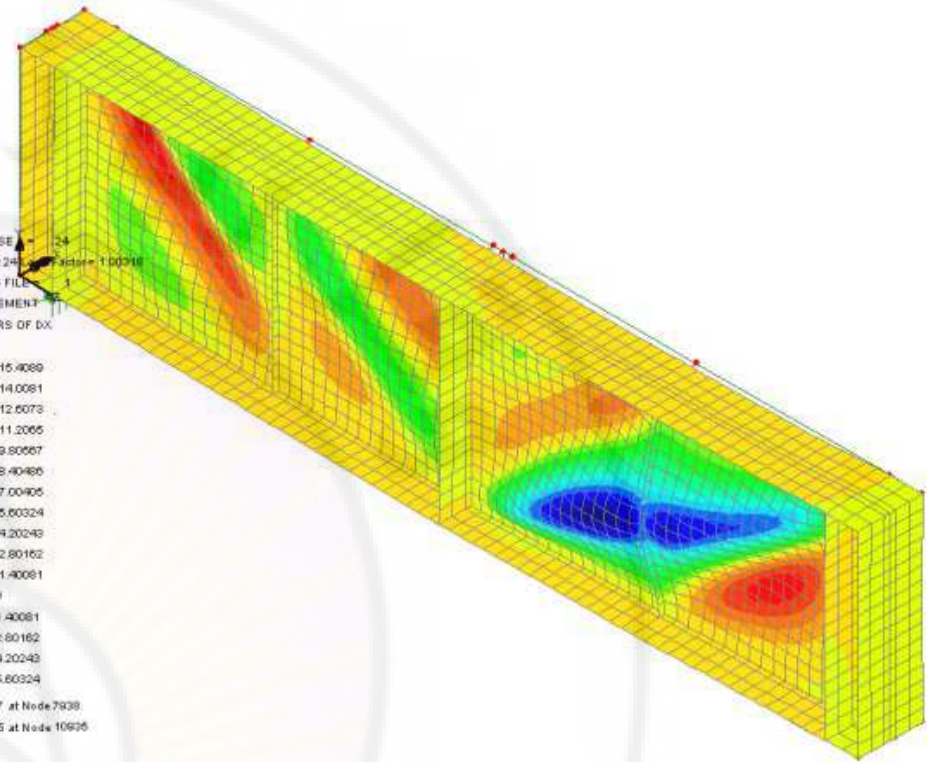
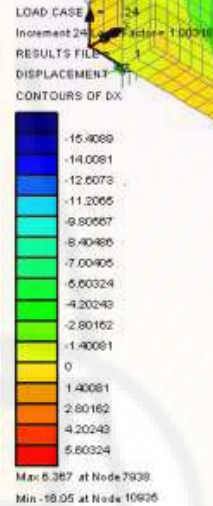
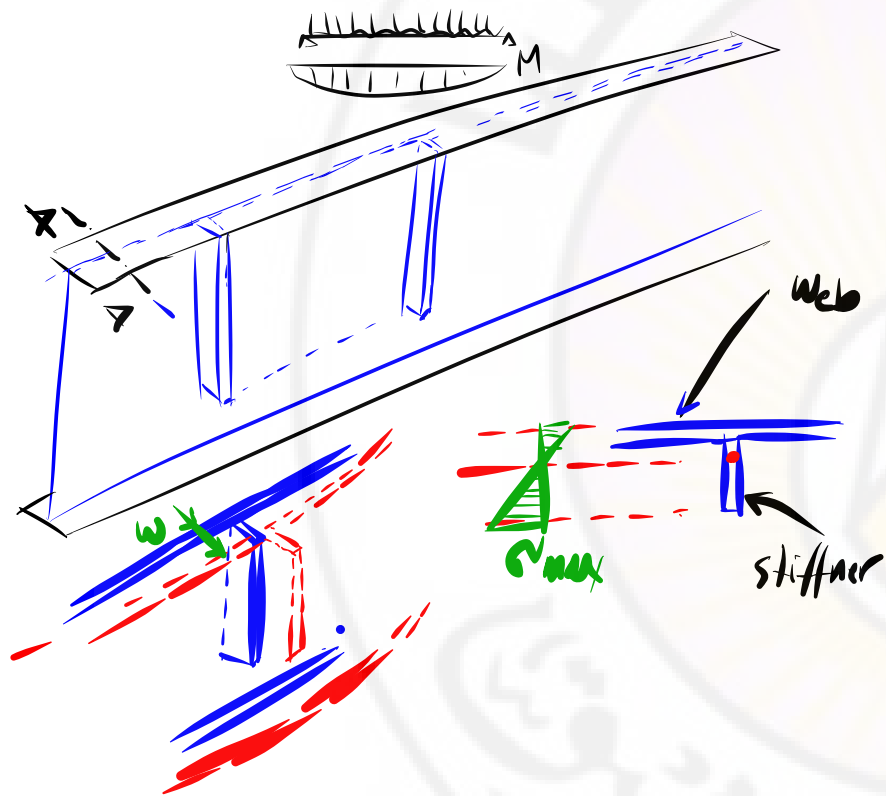
Second Order Analysis

Using Software



Second Order Analysis

Using Soft Ware



5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design



• General requirements of all stiffeners

- **Requirement (B) Torsional buckling:** The stiffener will not fail by torsional buckling which requires **either**

$$\textcircled{1} \quad \frac{I_T}{I_P} \geq 5.3 \frac{f_y}{E}$$

where

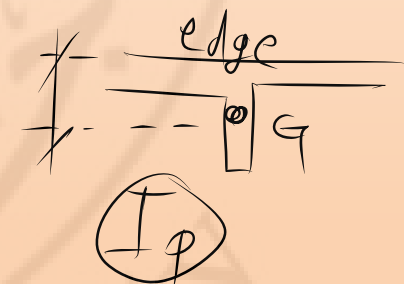
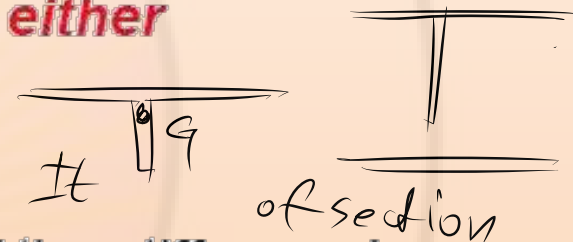
I_T is the St. Venant torsional constant of the stiffener alone

I_P is the polar second moment of area of the stiffener alone around the edge fixed to the plate

Or when warping stiffness is considered

$$\textcircled{2} \quad \sigma_{cr} \geq 2f_y$$

where σ_{cr} is elastic critical stress for torsional buckling



5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design

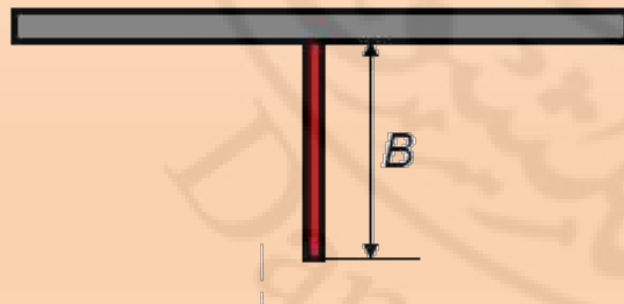


• General requirements of all stiffeners

- For a stiffener consist of flat plates, the requirement for torsional buckling could be simplified to limit the width (B) to thickness (t) ratio of the plates to those values shown below

Steel grade	235	275	355	420	460
$B/t \leq$	13.0	12.0	10.6	9.7	9.3

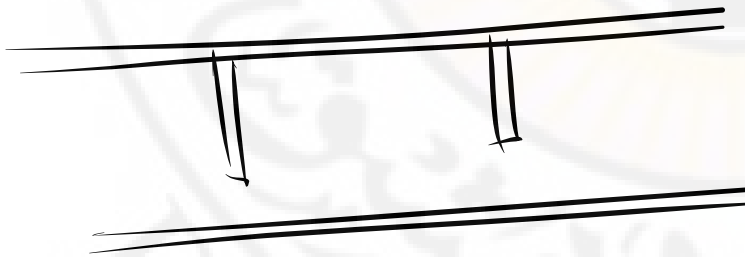
Limiting B/t values to prevent torsional buckling



- Note: if $B/t >$ the above suggested values, normally is it very hard to satisfy the requirement unless more advance analysis (e.g. FEA) is used.

Intermediates

Non-Bearing



5. Plate Girder Shear Resistance

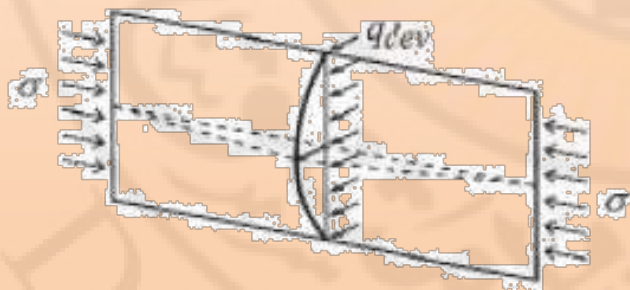
6-5 - Summary

- Stiffeners and end post design

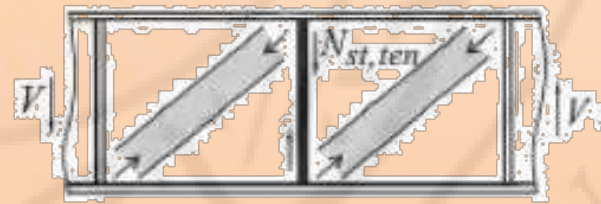


• Design of intermediate transverse stiffener

- The main function of intermediate transverse stiffener is to provide rigid boundary supports for the panel and prevent web buckling due to direct stress (plate-like buckling) and to shear stress (axial force from TFA and PH formations)
- Both **Requirements A and B** should be verified for both direct stress and shear stress



Direct stress (σ)



Shear stress ($N_{st,ten}$)

5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design

II.2

- Design of intermediate transverse stiffener
- Verification for direct stress (f_{dev})
 - For intermediate stiffeners that is not subjected to external axial load, **Requirement (B)** can be ensured by simply limiting the B/t ratio
 - In general, verification for direct stress for **Requirement (A)** needs a second order elastic analysis but it is not convenient in practice.
 - Hence, EC3 allows two simplified methods for direct stress verification for intermediate transverse stiffener in the absent of external axial force

5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design

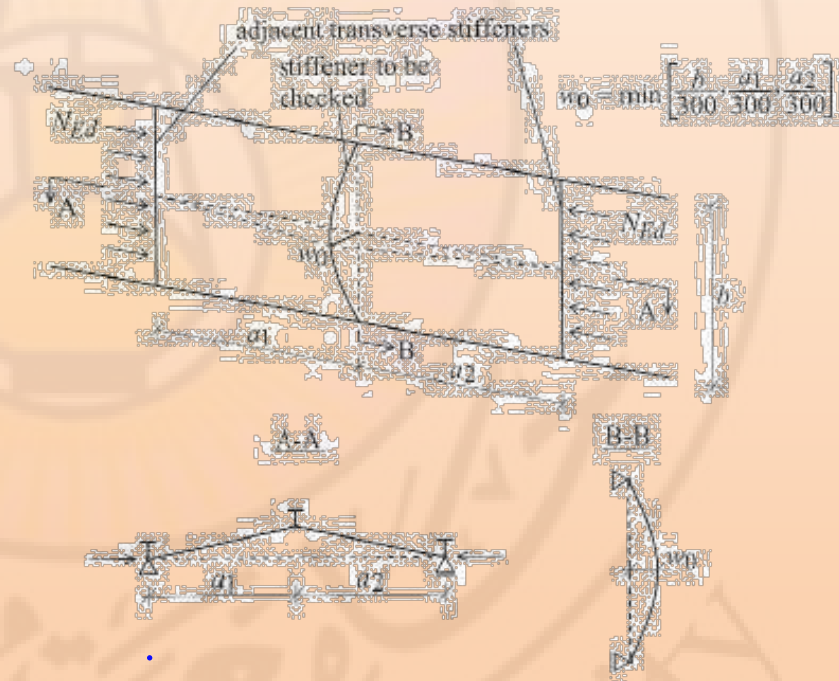
- Design of intermediate transverse stiffener
- Verification of direct stress: **Method I:** Refined I_{st} requirement **Requirements (A)** is considered to be satisfied if I_{st} is greater than

$$I_{st,req} = \frac{\sigma_m}{E} \left(\frac{b}{\pi} \right)^4 \left(1 + w_0 \frac{300}{b} u \right)$$

where

$$\sigma_m = \frac{\sigma_{cr,c}}{\sigma_{cr,p}} \left(\frac{N_{Ed}}{b} \right) \left(\frac{1}{a_1} + \frac{1}{a_2} \right)$$

$$u = \frac{\pi^2 E e_{max}}{300 f_y b \gamma_{M1}} \geq 1.0$$



5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design

II.2.1

- Design of intermediate transverse stiffener

➤ Verification of direct stress: Method I: Refined I_{st} requirement

$$w_0 = \min \left[\frac{b}{300}, \frac{a_1}{300}, \frac{a_2}{300} \right]$$

The elastic critical stresses for column and plate like buckling

$$\sigma_{cr,c} = \frac{\pi^2 E t_w^2}{12(1-\nu^2) a^2} \quad \sigma_{cr,p} = k_\sigma \frac{\pi^2 E t_w^2}{12(1-\nu^2) b^2}$$

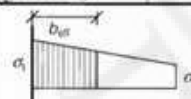
and limiting values of their ratio: $0.5 \leq \frac{\sigma_{cr,c}}{\sigma_{cr,p}} \leq 1.0$

k_σ is the plate buckling coefficient and dependent on the stress ratio ψ and can be calculation from Table 4.1 of EC3 Part 1-5, Section 4.4

Table 4.1: Internal compression elements

Stress distribution (compression positive)	Effective ^b width b_{eff}
	$\psi = 1$: $b_{eff} = \rho \cdot \bar{b}$ $b_{e1} = 0,5 b_{eff}$ $b_{e2} = 0,5 b_{eff}$
	$1 > \psi > 0$: $b_{eff} = \rho \cdot \bar{b}$ $b_{e1} = \frac{2}{5 - \psi} b_{eff}$ $b_{e2} = b_{eff} - b_{e1}$
	$\psi < 0$: $b_{eff} = \rho b_e = \rho \bar{b} / (1 - \psi)$ $b_{e1} = 0,4 b_{eff}$ $b_{e2} = 0,6 b_{eff}$
$\psi = \sigma_2 / \sigma_1$ 1 $1 > \psi > 0$ 0 $0 > \psi > -1$ -1 $(\frac{b_{e1}}{b_{eff}}) \cdot 1 > \psi \geq -3 (\frac{b_{e2}}{b_{eff}})$	
Buckling factor k_{σ}	4,0 $8,2 / (1,05 + \psi)$ 7,81 $7,81 - 6,29\psi + 9,78\psi^2$ 23,9 $5,98 (1 - \psi)^2$

Table 4.2: Outstand compression elements

Stress distribution (compression positive)	Effective ^b width b_{eff}
	$1 > \psi > 0$: $b_{eff} = \rho c$
	$\psi < 0$: $b_{eff} = \rho b_e = \rho c / (1 - \psi)$
$\psi = \sigma_2 / \sigma_1$ 1 $1 > \psi > 0$ 0 $0 > \psi > -1$ -1 $1 \geq \psi \geq -3$	
Buckling factor k_{σ}	0,43 0,57 0,85 $0,57 - 0,21\psi + 0,07\psi^2$
	$1 > \psi > 0$: $b_{eff} = \rho c$
	$\psi < 0$: $b_{eff} = \rho b_e = \rho c / (1 - \psi)$
$\psi = \sigma_2 / \sigma_1$ 1 $1 > \psi > 0$ 0 $0 > \psi > -1$ -1	
Buckling factor k_{σ}	0,43 $0,578 / (\psi + 0,34)$ 1,70 $1,7 - 5\psi + 17,1\psi^2$ 23,8

K_B

5. Plate Girder Shear Resistance

6-5 - Summary

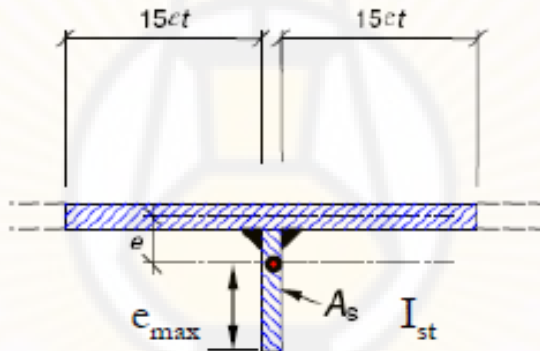
- Stiffeners and end post design

II.2.1

- Design of intermediate transverse stiffener

- Verification of direct stress: Method I: Refined I_{st} requirement

e_{max} is the maximum distance from the edge of the stiffener to the centroid of the stiffener



N_{Ed} is the maximum force of both adjacent panels and should be **at least** equal to the maximum compressive stress at the edge of the panel times half of the effective compressive area of the panel.

In checking, calculate u first, if $u < 1.0$ displacement check is decisive and set $u = 1.0$. Otherwise, a strength check governs.

5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design

II 2.2

- **Design of intermediate transverse stiffener**

- **Verification of direct stress:** Method II: Equivalent uniformly lateral load

In this method, a first order elastic analysis is carried out on the stiffener which is loaded laterally with an equivalent u.d.l force $q_{dev,Ed}$ such that

$$q_{dev,Ed} = \frac{\pi}{4} \sigma_m (w_0 + w_{el})$$

w_{el} is the elastic deflection of the stiffener (which could only be determined iteratively). In practice, it could be taken as the maximum deflection = $b/300$

All other terms are defined in Method I

5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design

II 2.2

- **Design of intermediate transverse stiffener**
 - **Verification of shear stress**
 - When shear force is present in a section, in addition to direct stress, intermediate transverse stiffener also act as rigid support for the interior panel.
 - EC3 1-5 also requires that verification for shear stress for **Requirement (A)** should be carried out using second order elastic analysis.
 - If both direct stress and shear stress are present at the same section (i.e. both M_{Ed} and V_{Ed} are not zero), then the effects of direct stress, which tends to produce additional deviation forces and lateral deflection, must be taken into account in the analysis

M_{Ed} → direct stress
 V_{Ed} → shear stress

5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design

II 2.2

- **Design of intermediate transverse stiffener**

- **Verification of shear stress:** Minimum stiffness requirement

For intermediate transverse stiffener acts as rigid support for panel, on top of the general **Requirements (A) and (B)**, the following minimum stiffness is also required for its second moment of area for the axis parallel to the web plate, I_{st}

$$I_{st} \geq \frac{1.5h_w^3 t_w^3}{a^2} \quad \text{if } a/h_w < \sqrt{2}$$

$$I_{st} \geq 0.75h_w t_w^3 \quad \text{if } a/h_w \geq \sqrt{2}$$

However, the above conditions does not demand very strong stiffener and **does not** ensure that **Requirements (A) or (B)** could be met.

5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design

II 2.2

- **Design of intermediate transverse stiffener**

- **Verification of shear stress:** Axial force generated by TFA

It can be shown that when shear force is present, under the action of TFA, an axial force $N_{st,ten}$ will be imposed on the intermediate transverse stiffener such that

$$N_{st,ten} = V_{Ed} - \frac{1}{(\bar{\lambda}_w)^2} \frac{h_w f_{yw} t_w}{\sqrt{3} \gamma_{M1}} \geq 0$$

V_{Ed} is design shear force in the adjacent panels.

- For the case of variable shear force, the value at the distance of $0.5h_w$ from the edge of the panel with the **larger** force should be taken

5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design

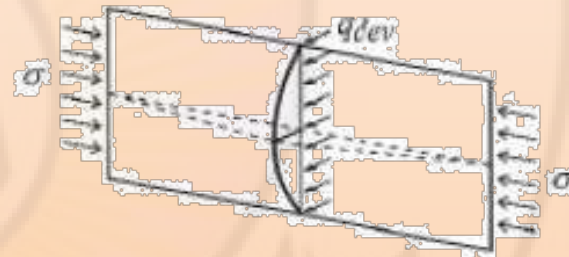
II 2.2

- **Design of intermediate transverse stiffener**

- **Verification of shear stress:** Interaction with direct stress

If direct stress is present (i.e. $M_{Ed} \neq 0$), a small lateral deflection will be induced and the deviation forces can be transformed into additional axial force in the stiffener:

$$\Delta N_{st,ten} = \frac{\sigma_m h_w^2}{\pi^2}$$



- After both $N_{st,ten}$ and $\Delta N_{st,ten}$ are calculated, an appropriate second order elastic model should be employed to check for the performance of the stiffener for **Requirement (A)**

- However, EC3 1-5 does not suggest any simplified model for the verification of **Requirement (A)**

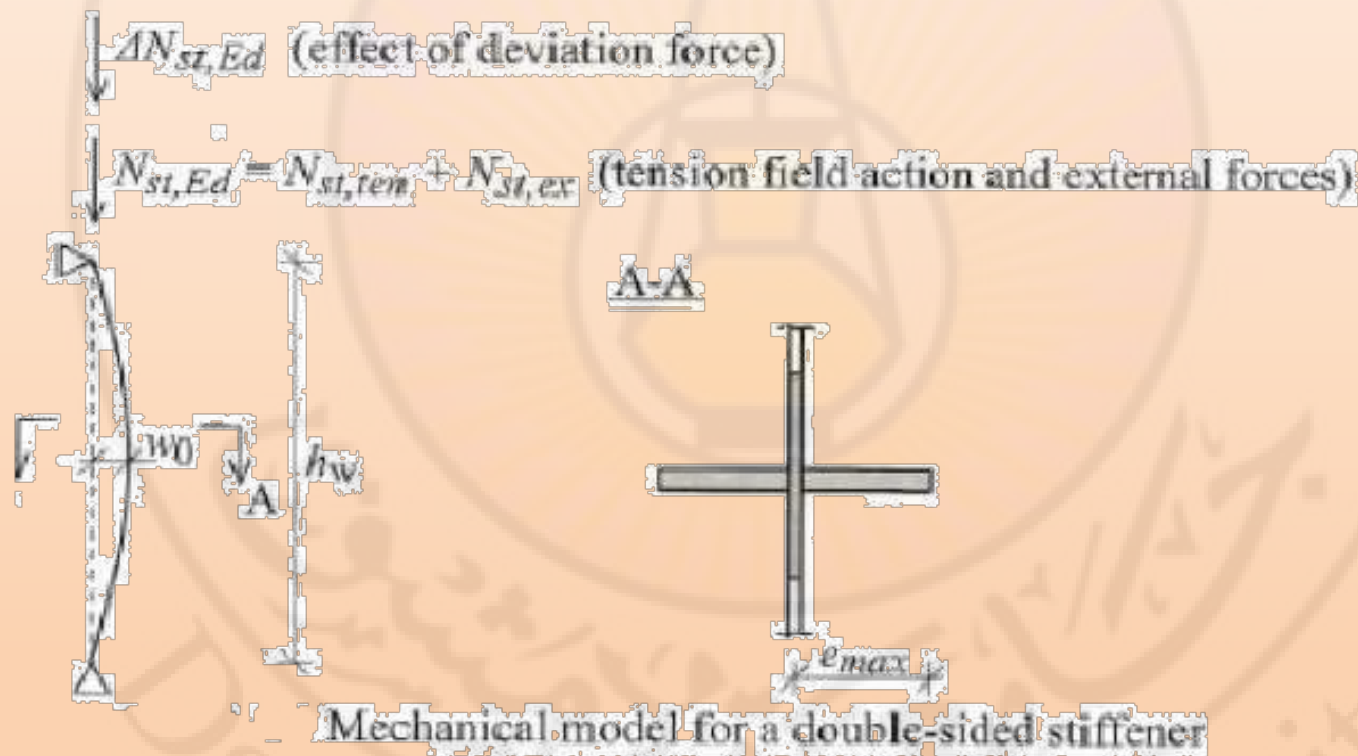
5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design



- Design of intermediate transverse stiffener
- Model for double-sided stiffener



5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design



- Design of intermediate transverse stiffener

➤ Model for double-sided stiffener

By using this model, the lateral deflection and maximum stress can be calculated as

$$w = w_0 \frac{1}{\left(\frac{N_{cr,st}}{N_{st,Ed} + \Delta N_{st,Ed}} - 1 \right)} \leq \frac{h_w}{300}$$

$$N_{cr,st} = \frac{\pi^2 EI_{st}}{h_w^2} \quad \text{Requirement A}$$

elastic buckling of the stiffener

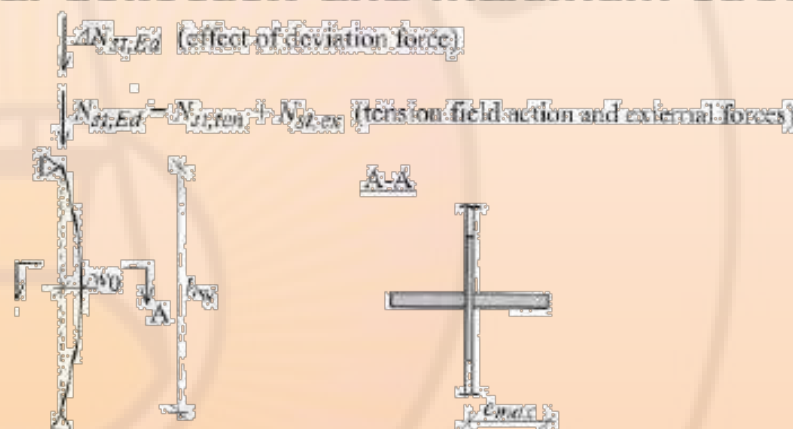


Fig. 2.69: Mechanical model for a double-sided stiffener

$$\sigma_{\max} = \frac{N_{st,Ed}}{A_{st}} + \left(\frac{(N_{st,Ed} + \Delta N_{st,Ed}) e_{\max}}{I_{st}} \right) w_0 \left(\frac{1}{1 - \frac{(N_{st,Ed} + \Delta N_{st,Ed})}{N_{cr,st}}} \right) \leq \frac{f_y}{\gamma_{M1}}$$

End Post

Non Rigid Rigid



5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design



• Design of end post

- Two types of end posts: Rigid and Non-Rigid
- To provide support for TFA and support reaction
- Rigid end post should comprises of two double-sided transverse stiffener which forms the flange of a short column of length h_w and the following two requirements should be met

$$e \geq 0.1h_w$$

$$\min(A_e, A_u) \geq \frac{4h_w t_w^2}{e}$$



• Rolled section forming an end-post

5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design



- **Design of end post**

- **Requirements (A) and (B)** should still be satisfied for the end post and the double-sided model mentioned could be used.
- As end post often subjected to large reaction force, satisfaction of **Requirement (A)** which limits the maximum stress to f_y may not be able to prevent the buckling of the end post as a strut
- EC3 requires separate checking of the end post as a column with buckling length not less than $0.75h_w$.
- **Curve c** from EC3 Part 1-1 should be used for buckling check
- The total force acting on the end post should equal to the sum of reaction, force due to TFA and interaction with direct stress
- If both ends of the end post are fixed laterally, the buckling length could be taken as $0.75h_w$. Otherwise, a larger value should be used.

5. Plate Girder Shear Resistance

6-5 - Summary

- Stiffeners and end post design

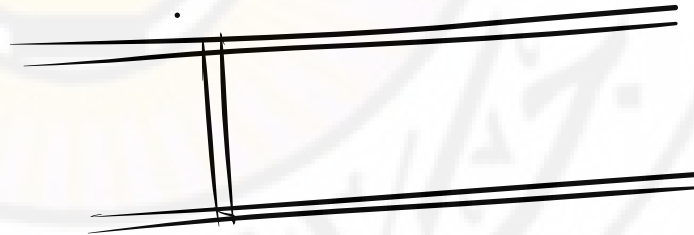


- **Design of load bearing stiffeners**

- Patch loads and concentrated loads are common in plate girder
- Sufficient bearing resistance is needed to resist transverse force acting on the flange plate
- In case that the bearing strength provided by the web plate is insufficient, load bearing stiffener should be constructed.
- In general, **Requirements (A) and (B)** are also needed to be satisfied for loading bearing stiffeners.
- Again, the total force acting on a load bearing stiffener should equal to the sum of reaction, force due to TFA and interaction with direct stress
- Similar to end post, their design is often government by the buckling resistance as a strut (with buckling length $\geq 0.75h_w$) but without the compulsory use of two double-side plates

Intermediates

↓
Bearing



5. Plate Girder Shear Resistance

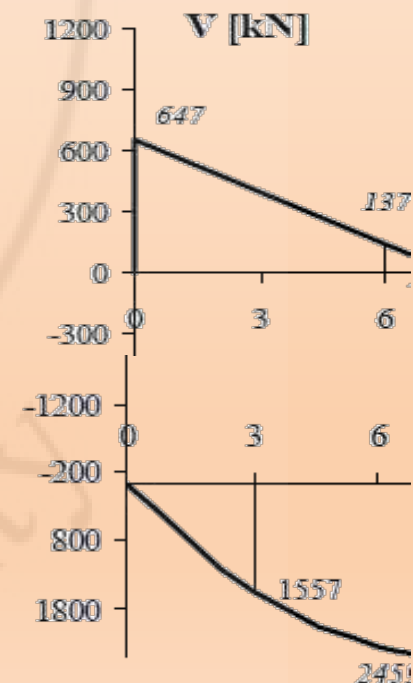
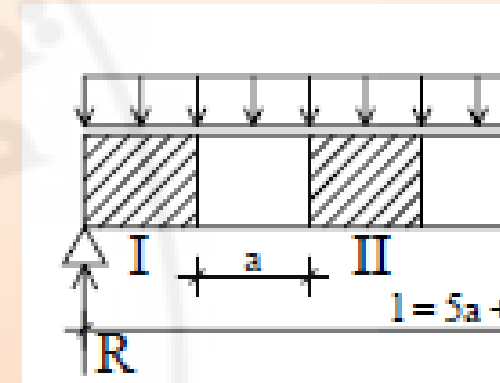
Back to Worked Example 2

Panel I (at the exterior support)

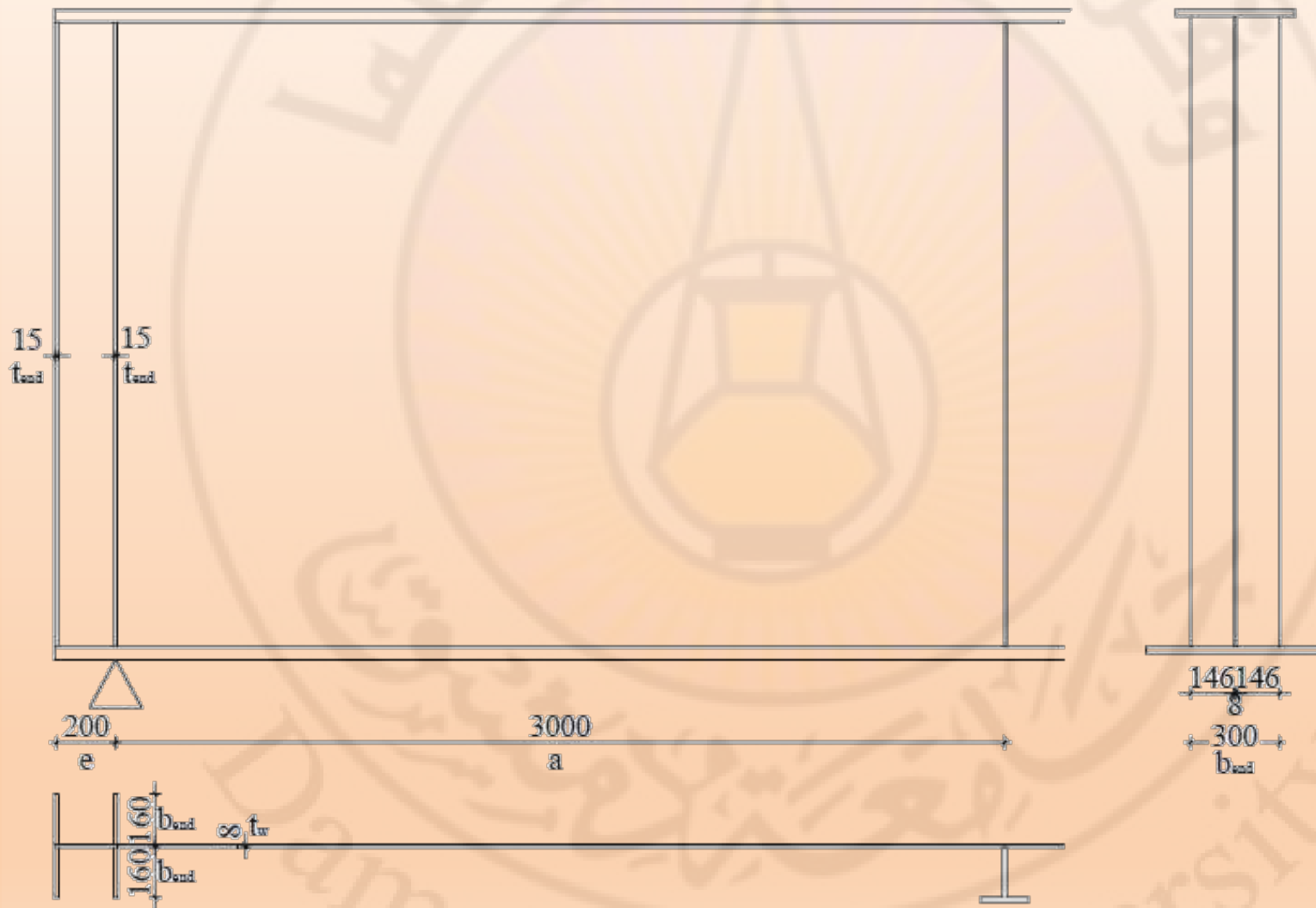
$$V_{Ed} = 647 \cdot 10^3 \text{ N (at the support),}$$

$$M_{Ed} = 1557 \cdot 10^3 \text{ Nm (at the end of panel I – } x = 3 \text{ m)}$$

Interaction M-V is checked with maximum values of shear force and bending moment existing in the panel, but not located in the same cross section (a conservative approach).



Back to Worked Example 2



5. Plate Girder Shear Resistance

Back to Worked Example 2

Rigid end post

The end post is designed to be rigid. Therefore it must fulfil the following criteria:

$$e \geq 0,1h_w = 0,1 \cdot 2,0 = 0,20 \text{ m} \quad \text{chosen: } e = 0,20 \text{ m} \quad \text{Slide 111,83}$$

(B/t) for all Stiffener ≤ 13 (Steel 235)

$$B < 13 \times 15 = 195 \dots \text{Select } B = 160 \text{ mm} \quad \text{Slide 98}$$

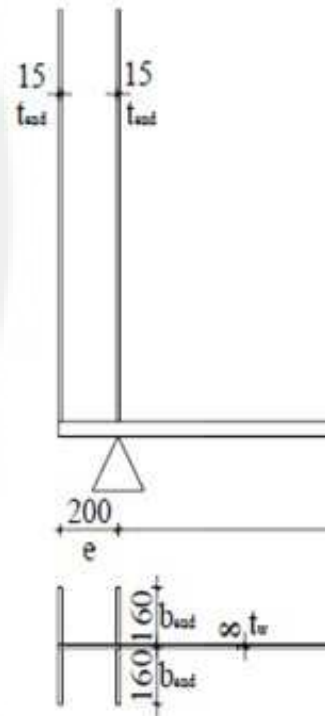
$$B_{\text{end}} = 160 \times 2 = 320 \text{ mm}$$

$$\text{end plate dimensions: } b_{\text{end}} / t_{\text{end}} = 320 / 15 \text{ double sided}$$

$$A_{\text{end}} = 2 \cdot 0,16 \cdot 0,015 + 0,008 \cdot 0,015 = 4,92 \cdot 10^{-3} \text{ m}^2 \geq 4 \frac{h_w t_w^2}{e} = 4 \frac{2,00 \cdot 0,008^2}{0,20}$$

$$A_{\text{end}} = 4,92 \cdot 10^{-3} \text{ m}^2 \geq 2,56 \cdot 10^{-3} \text{ m}^2 \quad \text{Slide 111,83}$$

$$\frac{b_{\text{end}}}{t_{\text{end}}} = \frac{0,160}{0,015} = 10,7 < 14\varepsilon = 14 - \text{Class 3, therefore fully effective.}$$



5. Plate Girder Shear Resistance

Back to Worked Example 2

Shear resistance

Contribution from web

$k_{t,st} = 0$ – no longitudinal stiffener, $\eta = 1,2$ (steel grade S235)

$$\frac{a}{h_w} = \frac{3,00}{2,00} = 1,5 \Rightarrow$$

$$k_t = 5,34 + 4 \left(\frac{h_w}{a} \right)^2 + k_{t,st} = 5,34 + 4 \cdot \left(\frac{2,00}{3,00} \right)^2 + 0 = 7,118$$

$$\frac{h_w}{t_w} = \frac{2,00}{0,008} = 250 \geq \frac{31}{\eta} \varepsilon \sqrt{k_t} = \frac{31}{1,2} \cdot 1 \cdot \sqrt{7,118} = 69 \Rightarrow \text{verification of shear}$$

buckling is necessary

$$\bar{\lambda}_w = \frac{h_w}{37,4 t_w \varepsilon \sqrt{k_t}}$$

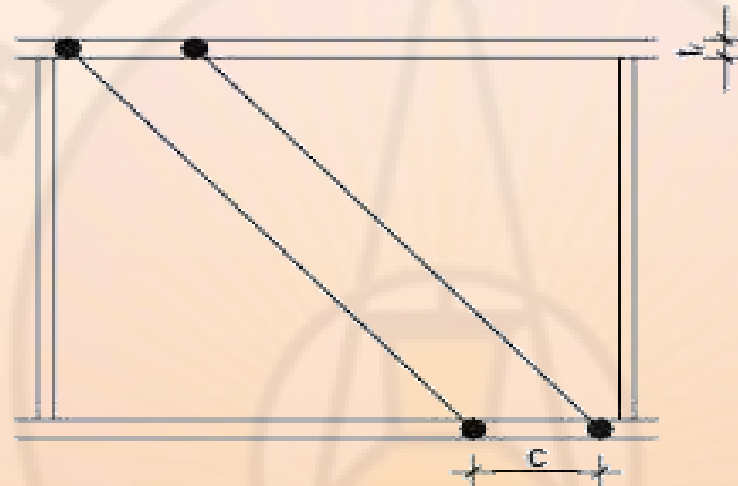
$$\bar{\lambda}_w = \frac{2,00}{37,4 \cdot 0,008 \cdot 1 \cdot \sqrt{7,118}} = 2,505 \geq 1,08 \Rightarrow \chi_w = \frac{1,37}{0,7 + \bar{\lambda}_w} = \frac{1,37}{0,7 + 2,505} = 0,427$$

$$V_{bw,Rd} = \frac{\chi_w f_{yw} h_w t_w}{\gamma_{M1} \sqrt{3}} = \frac{0,427 \cdot 235 \cdot 10^6 \cdot 2,00 \cdot 0,008}{1,1 \cdot \sqrt{3}} = 843 \cdot 10^3 \text{ N}$$

5. Plate Girder Shear Resistance

Back to Worked Example 2

Contribution from flanges



Panel I – tension band

$$c = a \left(0,25 + \frac{1,6 \cdot b_{fl} \cdot t_{fl}^2 \cdot f_{yf}}{t_w \cdot h_w^2 \cdot f_{yw}} \right) = 3,00 \left(0,25 + \frac{1,6 \cdot 0,40 \cdot 0,02^2 \cdot 235 \cdot 10^6}{0,008 \cdot 2,00^2 \cdot 235 \cdot 10^6} \right) = 0,774 \text{ m}$$

$$b_f = 0,40 \text{ m} \leq 2 \cdot 15 t_f + t = 2 \cdot 15 \cdot 1 \cdot 0,02 + 0,008 = 0,608 \text{ m}$$

$$M_{f,Rd} = (h_w + t_{fl}) A_{f,min} \frac{f_y}{\gamma_{M0}} = (2,00 + 0,02) \cdot (0,40 \cdot 0,02) \cdot \frac{235 \cdot 10^6}{1,0} = 3798 \cdot 10^3 \text{ Nm}$$

$$V_{bf,Rd} = \frac{b_{fl} \cdot t_{fl}^2 \cdot f_{yf}}{c \cdot \gamma_{M1}} \left(1 - \left(\frac{M_{Ed}}{M_{f,Rd}} \right)^2 \right) = \frac{0,40 \cdot 0,02^2 \cdot 235 \cdot 10^6}{0,774 \cdot 1,1} \left(1 - \left(\frac{1557 \cdot 10^3}{3798 \cdot 10^3} \right)^2 \right) = 37 \cdot 10^3 \text{ N}$$

Flanges do not significantly contribute to the shear resistance; indeed $V_{bf,Rd}$ is substantially smaller than $V_{bw,Rd}$.

5. Plate Girder Shear Resistance

Back to Worked Example 2

Verification for shear resistance

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{\eta f_{yw} h_w t_w}{\gamma_{M1} \sqrt{3}}$$

$$V_{b,Rd} = (843 + 37) \cdot 10^3 = 880 \cdot 10^3 \text{ N} \leq \frac{1,2 \cdot 235 \cdot 2 \cdot 0,008}{1,1 \cdot \sqrt{3}} \cdot 10^6 = 2368 \cdot 10^3 \text{ N} \quad \checkmark$$

$$V_{Ed} = 647 \cdot 10^3 \text{ N} \leq V_{b,Rd} = 880 \cdot 10^3 \text{ N} \quad \checkmark$$

Verification of bending resistance

$$M_{Ed} = 1557 \cdot 10^3 \text{ Nm} \leq M_{f,Rd} = 3798 \cdot 10^3 \text{ Nm} \quad \checkmark$$

The direct stresses may thus be transferred by the flanges only.

Verification of interaction M-V

Flanges take bending moment and the web takes shear force. There is no need for a verification of M-V-interaction.

5. Plate Girder Shear Resistance

Back to Worked Example 2

Panel II (at midspan)

Cross-section of panel II is the same as panel I.

$$V_{Ed} = 137 \cdot 10^3 \text{ N (at the edge of panel II – } x = 6 \text{ m),}$$

$$M_{Ed} = 2459 \cdot 10^3 \text{ Nm (maximum value within the panel, at 7.55 m from the left support)}$$

Verification of shear resistance

Panel II is the same as panel I and has also the same shear resistance of the web. Neglecting the contribution from the flanges because of high moments gives:

$$V_{Ed} = 137 \cdot 10^3 \text{ N} \leq V_{b,Rd} = 880 \cdot 10^3 \text{ N}$$

$V_{b,Rd}$ is calculated in Section 17.3.3.

Verification of bending resistance

$$M_{Ed} = 2459 \cdot 10^3 \text{ Nm} \leq M_{f,Rd} = 3798 \cdot 10^3 \text{ Nm} \quad \checkmark$$

$M_{f,Rd}$ is calculated in Section 0. The direct stresses may thus be transferred by the flanges only.

The End
Part 3

Damascus University

EUROPEAN STANDARD NORME EUROPÉENNE EUROPÄISCHE NORM

ICS 91.010.30; 91.080.10

Eurocode

Eurocode 3 - Calcul des structures
Plaques planes

This European Standard was approved

CEN members are bound to comply with
Standard the status of a national standard
standards may be obtained on application

This European Standard exists in three
under the responsibility of a CEN member
versions.

CEN members are the national standards
Germany, Greece, Hungary, Iceland, Ireland,
Slovakia, Slovenia, Spain, Sweden, Switzerland

Content

1 Introduction

- 1.1 Scope
- 1.2 Normative references
- 1.3 Terms and definitions
- 1.4 Symbols

2 Basis of design and model

- 2.1 General
- 2.2 Effective width model
- 2.3 Plate buckling effects
- 2.4 Reduced stress method
- 2.5 Non uniform members
- 2.6 Members with corrugated

3 Shear lag in member design

- 3.1 General
- 3.2 Effective^s width for el
- 3.3 Shear lag at the ultimate

4 Plate buckling effects due

- 4.1 General
- 4.2 Resistance to direct st
- 4.3 Effective cross section
- 4.4 Plate elements without
- 4.5 Stiffened plate elemen
- 4.6 Verification

5 Resistance to shear

- 5.1 Basis
- 5.2 Design resistance
- 5.3 Contribution from the
- 5.4 Contribution from flar
- 5.5 Verification

6 Resistance to transverse f

- 6.1 Basis
- 6.2 Design resistance
- 6.3 Length of stiff bearing
- 6.4 Reduction factor χ_F fo
- 6.5 Effective loaded lengt
- 6.6 Verification

7 Interaction

- 7.1 Interaction between st
- 7.2 Interaction between tr

8 Flange induced buckling

9 Stiffeners and detailing

- 9.1 General
- 9.2 Direct stresses
- 9.3 Shear
- 9.4 Transverse loads

10 Reduced stress method

Annex A (informative) Calcu

Annex B (informative) Non un

Annex C (informative) Finite

Annex D (informative) Plate g

Annex E (normative) Alterna



Foreword

This European Standard EN 1993-1-5, which gives the design rules for elements, has been prepared by Technical Group 5 of CEN, of which is held by BSI. CEN/TC250.

This European Standard shall be given the status of a national standard by text or by endorsement, at the latest by March 2010, and shall be withdrawn at latest by March 2010.

This Eurocode supersedes ENV 1993-1-5.

According to the CEN-CENELEC Internal Rules, the following countries are bound to implement this European Standard: Austria, Belgium, Bulgaria, Cyprus, Czech Republic, Denmark, Estonia, Finland, France, Germany, Greece, Hungary, Iceland, Ireland, Italy, Lithuania, Luxembourg, Malta, Netherlands, Norway, Poland, Portugal, Romania, Slovakia, Slovenia, Sweden, Switzerland and United Kingdom.

National annex for EN 1993-1-5

This standard gives alternative procedures. Some choices may have to be made. The National Annex containing all Nationally Determined Parameters shall be constructed in the relevant country.

National choice is allowed in EN 1993-1-5:

- 2.2(5)
- 3.3(1)
- 4.3(6)
- 5.1(2)
- 6.4(2)
- 8(2)
- 9.1(1)
- 9.2.1(9)
- 10(1)
- 10(5)
- C.2(1)
- C.5(2)
- C.8(1)
- C.9(3)
- D.2.2(2)

1 Introduction

1.1 Scope

(1) EN 1993-1-5 gives design rules for shear lag effects due to in-plane forces.

(2) Effects due to shear lag in girders are covered. Also covered are plate girders and silos. The effects of out-of-plane

NOTE 1: The rules in this part of EN 1993-1-5 are

NOTE 2: For the design of structures subject to fatigue due to out-of-plane

NOTE 3: For the effects of loading effects see EN 1993-1-5

NOTE 4: Single plate elements

$$r \geq \frac{a^2}{t}$$

where a is the panel width and t is the plate thickness

1.2 Normative references

(1) This European Standard refers to the following publications. These normative references are listed hereafter. For dated references, only the edition cited applies. For undated references, the latest edition of the standard referred to applies.

EN 1993-1-1 *Eurocode 3: Design of steel structures – Part 1-1: General rules and*

1.3 Terms and definitions

For the purpose of this standard, the following terms and definitions apply.

1.3.1

elastic critical stress

stress in a component at which buckling occurs in a perfect structure

1.3.2

membrane stress

stress at mid-plane of the plate

1.3.3

gross cross-section

the total cross-sectional area of the component

1.3.4

effective cross-section and effective width

the gross cross-section or width of the component, reduced to account for the effects of the word “effective” denotes effects of

“effective” denotes effects of shear lag;
 “effective” denotes effects of plate buckling.

1.3.5

plated structure

a structure built up from nominally flat unstiffened plates

1.3.6

stiffener

a plate or section attached to a plate to stiffen it
 – longitudinal if its direction is parallel to the main direction of the plate;
 – transverse if its direction is perpendicular to the main direction of the plate.

1.3.7

stiffened plate

plate with transverse or longitudinal stiffeners

1.3.8

subpanel

unstiffened plate portion surrounded by stiffeners

1.3.9

hybrid girder

girder with flanges and web made of different materials or with flanges compared to webs

1.3.10

sign convention

unless otherwise stated compression is positive

1.4 Symbols

(1)	In addition to those given in EN 1993-1-1, the following symbols apply:
A_{st}	total area of all the longitudinal stiffeners
A_{st}	gross cross sectional area of one longitudinal stiffener
A_{eff}	effective cross sectional area;
$A_{c,eff}$	effective ^p cross sectional area;
$A_{c,eff,loc}$	effective ^p cross sectional area of one stiffener
a	length of a stiffened or unstiffened plate
b	width of a stiffened or unstiffened plate
b_w	clear width between welds for longitudinal stiffeners
b_{eff}	effective ^s width for elastic shear buckling
F_{Ed}	design transverse force;
h_w	clear web depth between flanges
L_{eff}	effective length for resistance to buckling
$M_{t,Rd}$	design plastic moment of resistance
$M_{pl,Rd}$	design plastic moment of resistance
M_{Ed}	design bending moment;
N_{Ed}	design axial force;
t	thickness of the plate;

V_{Ed}	design shear force incl
W_{eff}	effective elastic section
β	effective ⁿ width factor

(2) Additional symbols are

2 Basis of design and

2.1 General

(1)P The effects of shear lag and fatigue limit states.

NOTE: Partial factors γ_{M1} and γ_{M2} are given in Annexes of EN 1993-1 to 1.

2.2 Effective width models

(1)P The effects of shear lag and account in the global analysis.

(2) The effects of shear lag and effectiveⁿ width. For simplicity, span.

(3) For each span of a member width and $L/8$ per side of the web cantilever.

(4) The effects of plate buckling on sectional areas of the elements.

(5) For global analysis the cross-sectional area of an element the same element.

NOTE 1: The parameter ρ is

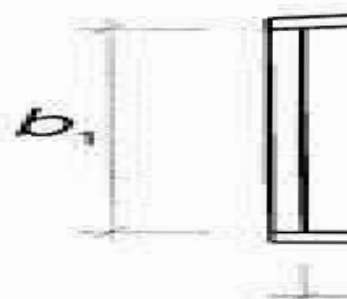
NOTE 2: For determining

2.3 Plate buckling effects

(1) Effectiveⁿ width models for transverse loads as well as in members at the ultimate limit state

- panels are rectangular and f
- the diameter of any unstiffened panel.

NOTE: The rules may apply to angles with α_{lim} not exceeding 10 degrees. If α_{lim} exceeds larger of b_1 and b_2 of the panel.



Figure

(2) For the calculation of stresses σ used AC_1 if the condition in 2.2(5) is fulfilled. β should be used with β replaced by β_{ult} .

2.4 Reduced stress method

(1) As an alternative to the use of the cross sections may be assumed to exceed the limits specified in section 1.

NOTE: The reduced stress method elements. However, in verifying the elements of the cross section.

2.5 Non uniform members

(1) Non uniform members (e.g. hat irregular large openings may be analysed.

NOTE 1: See Annex B for non uniform members.

NOTE 2: For FE-calculations see Annex C.

2.6 Members with corrugate

(1) For members with corrugated webs should be considered to transfer.

NOTE: For AC_1 text deleted AC_1 see Annex D.

3 Shear lag in members

3.1 General

(1) Shear lag in flanges may be neglected if the width of an internal element and the distance from the web to the edge of an external element are both less than the limit given in 3.3.

(2) Where the above limit for shear lag is exceeded, the effective width and the stress distribution according to 3.3 may be used.

(3) Stresses due to patch loads may be determined using the effective width.

3.2 Effective^s width for shear lag

3.2.1 [AC1] Effective^s width [AC1]

(1) The effective^s width b_{eff} is defined by:

$$b_{eff} = \beta b_0$$

where the effective^s factor β is given by:

[AC1] This effective^s width may [AC1] be used for:

(2) Provided adjacent spans are not less than the effective length, the effective length may be taken as the distance between adjacent spans.

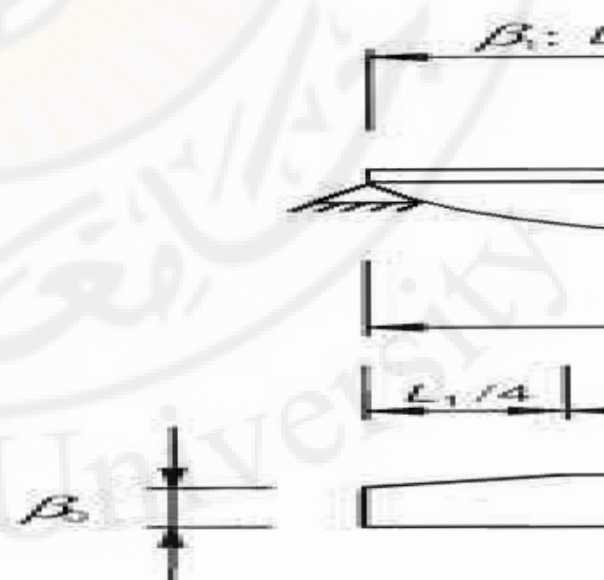
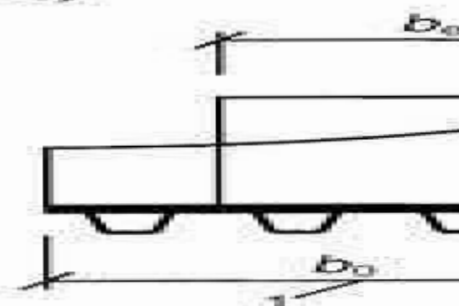


Figure 3.1: Effective width



- 1 for flange
- 2 for internal stiffener
- 3 plate thickness
- 4 stiffener spacing

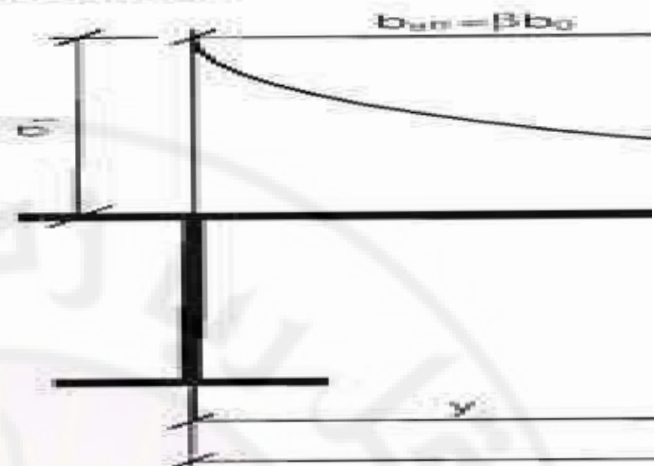
Figure 1

Table 1

κ	Verification
$\kappa \leq 0,02$	
$0,02 < \kappa \leq 0,70$	sagging beam
	hogging beam
$> 0,70$	sagging beam
	hogging beam
all κ	end support
all κ	Cantilever
$\kappa = \alpha_0 b_0 / L_e$ with $\alpha_0 = \sqrt{\frac{E_s}{E}}$ in which A_{st} is the area of stiffeners symbols are as defined in Figure 1	

3.2.2 Stress distribution

(1) The distribution of longitudinal stress from Figure 3.3.



$\beta > 0,20 :$

$\sigma_2 = 1,25 (\beta$

$\sigma(y) = \sigma_2 +$

Figure 3.3

3.2.3 In-plane load effects

(1) The elastic stress distribution due to in-plane forces (patch loads), see Figure 3.4.

$$\sigma_{z,Ed} = \frac{F_{Ed}}{b_{eff} (t_w + [AC1] a_{scl})}$$

with: $b_{eff} = s_e \sqrt{1 + \left(\frac{z}{s_e n} \right)^2}$

$$n = 0,636 \sqrt{1 + \frac{0,878}{t_w}}$$

$$s_e = s_x + 2 t_f$$

where a_{scl} is the gross cross-section area of the steel reinforcement.

[AC1] This may be taken as 1,0.

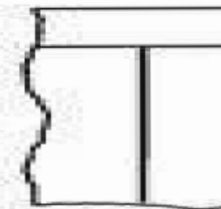
t_w is the web thickness.

z is the distance from the neutral axis to the top or bottom edge of the beam.

[AC1] s_e is the length of the effective stress distribution.

s_w is the spacing of the reinforcement.

NOTE: The equation (3.10) is only valid for beams with a constant cross-section. For beams with a varying cross-section, the equation (3.10) should be neglected.



1
2
3

Fi

NOTE: The above stress d

3.3 Shear lag at the ultimate

- (1) At the ultimate limit state
- a) elastic shear lag effects as de
 - b) combined effects of shear lag
 - c) elastic-plastic shear lag effect

NOTE 1: The National Annex to EN 1993-6, the method in

NOTE 2: The combined ef by:

$$A_{eff} = A_{c,eff} \beta_{ult}$$

where $A_{c,eff}$ is the effective area
 β_{ult} is the effect taken as β d

$$\alpha_0 = \sqrt{\frac{A_c}{b_0}}$$

t_f is the flange

NOTE 3: Elastic-plastic A_{eff} as follows:

$$A_{eff} = A_{e,eff} \beta^{\kappa} \geq A_{e,eff}$$

where β and κ are taken

The expressions in NOTE replaced by the gross area

4 Plate buckling effects

4.1 General

(1) This section gives rules state when the following criteria

- a) The panels are rectangular;
- b) Stiffeners, if any, are provided;
- c) Open holes and cut outs are avoided;
- d) Members are of uniform cross-section;
- e) No flange induced web buckling.

NOTE 1: For compression

NOTE 2: For stiffeners and

4.2 Resistance to direct

(1) The resistance of plated members in compression for class 4 sections and member verifications for compression

(2) Effective^p areas should be based on the limit of yield strain in the mid plane

4.3 Effective cross section

(1) In calculating longitudinal resistance, plate buckling using the effective cross-section

(2) The effective cross section should be determined for compression elements and on the tension elements

(3) The effective area A_{eff} should be determined due to uniform axial compression. The effective area A_{eff} relative to the gross area A_g should be determined for additional moment which should be considered

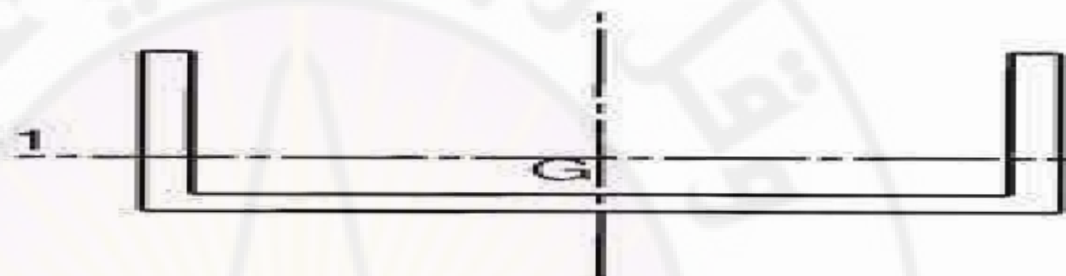
(4) The effective section modulus W_{eff} should be determined for bending stresses, see Figure 4.1. The effective section modulus W_{eff} should be determined for both main axes.

NOTE: As an alternative to the above, the effects of

- (5) The stress in a flange shot plane of the flange.
- (6) Hybrid girders may have
- a) the increase of flange stresses in the web to f_{yw} ;
 - b) f_{yr} ~~AC1~~ text deleted ~~AC1~~ is used

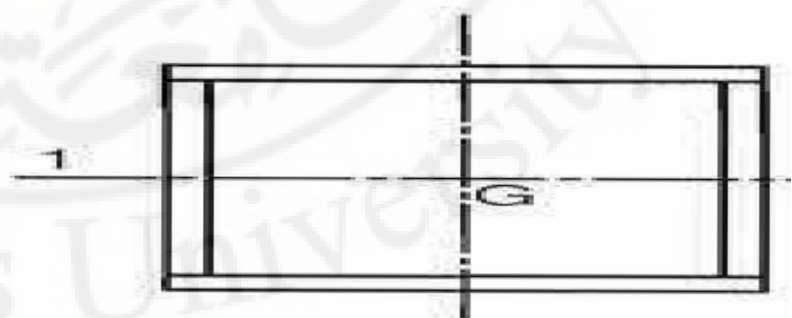
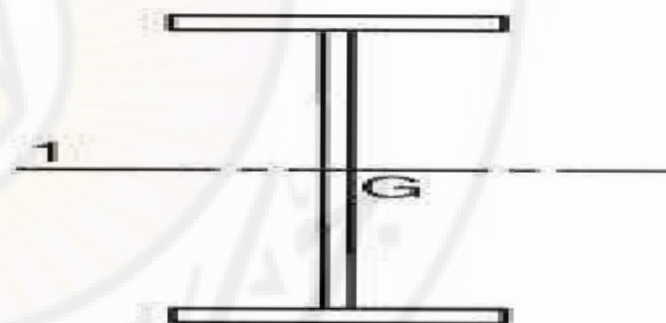
NOTE: The National Annex

- (7) The increase of deformation for hybrid girders complying with
- (8) For hybrid girders complying



Gross cross section

Figure



Gross cross section

Figure 4.2

4.4 Plate elements with

(1) The effective^p areas of elements and Table 4.2 for outstand gross cross-sectional area A_c shall be

$$A_{c,eff} = \rho A_c$$

where ρ is the reduction factor

(2) The reduction factor ρ may be determined as follows:

– internal compression elements

$$\rho = 1,0$$

$$\rho = \frac{\bar{\lambda}_p - 0,055 (3 + \psi)}{\bar{\lambda}_p^2}$$

– outstand compression elements

$$\rho = 1,0$$

$$\rho = \frac{\bar{\lambda}_p - 0,188}{\bar{\lambda}_p^2} \leq 1,0$$

where $\bar{\lambda}_p = \sqrt{\frac{f_y}{\sigma_{cr}}} = \frac{\bar{b}/t}{28,4 \epsilon \sqrt{k_\sigma}}$

ψ is the stress ratio determined as follows:

$\bar{b}$ is the appropriate width to be used:

b_w for webs;

b for internal flange elements;

$b - 3 t$ for flanges of RHS;

c for outstand flange elements;

h for equal-leg angles;

h for unequal-leg angles.

k_σ is the buckling factor corresponding to the end conditions, given in Table 4.1 or Table 4.2;

t is the thickness;

σ_{cr} is the elastic critical plate stress, given in Table 4.2;

$$\epsilon = \sqrt{\frac{235}{f_y [N/mm^2]}}$$

(3) For flange elements of I-sections, the effective area should be based on the properties of the flanges if relevant. For web elements, the effective area should be based on the effective distribution based on the effective area.

NOTE: If the stress distribution is not uniform, the effective area should be based on the stresses from the various stress components.

gross web and these stresses are a section that can be used for all stag

- (4) Except as given in 4.4(5), the pl

$$\bar{\lambda}_{p,red} = \bar{\lambda}_p \sqrt{\frac{\sigma_{com,Ed}}{f_y / \gamma_{M0}}}$$

where $\sigma_{com,Ed}$ is the maximum design area of the section cau

NOTE 1: The above procedure is (see Table 4.1 and Table 4.2) is c section defined at the end of the pro

NOTE 2: See also alternative proc

- (5) For the verification of the design EN 1993-1-1, either the plate slender global imperfections should be used.

- (6) For aspect ratios $a/b < 1$ a col according to 4.5.4 using the reduction

NOTE: This applies e.g. for flat e like and require a reduction factor γ longitudinal stiffeners column type



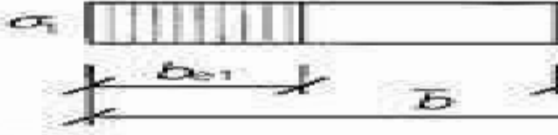
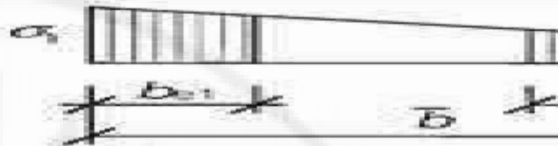
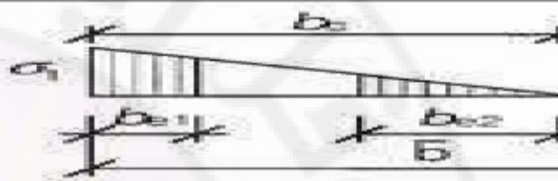
a) column-like beh of plates without longitudinal sup



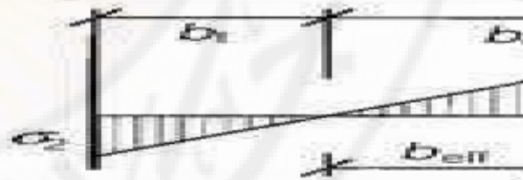
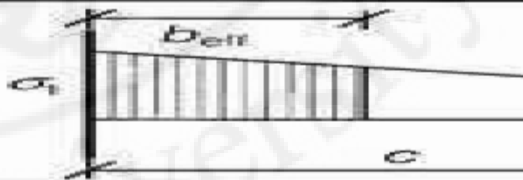
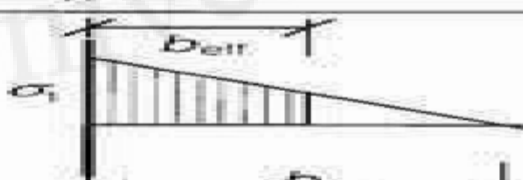
c) col stif

Figure

Tab

Stress distribution (comp)		
		
		
		
$\psi = \sigma_2 / \sigma_1$	I	
Buckling factor k_σ	4,0	8

Tab

Stress distribution (comp)		
		
		
$\psi = \sigma_2 / \sigma_1$		
Buckling factor k_σ		
		
		
$\psi = \sigma_2 / \sigma_1$		
Buckling factor k_σ		0,4

4.5 Stiffened plate ele

4.5.1 General

(1) For plates with longitudinal stiffeners, the effective width between the stiffeners and the reduction factor should be accounted for.

(2) The effective^p section area $A_{c,eff}$ with 4.4 to account for local p should be checked for global reduction factor ρ_c . $A_{c,eff}$ sho

(3) The effective^p area of the

$$A_{c,eff} = \rho_c A_{c,eff,loc} + \sum_c A_{st,eff}$$

where $A_{c,eff,loc}$ is the effective^p area in the compression zone except $b_{edge,eff}$, see example in Figure 4.4

(4) The area $A_{c,eff,loc}$ should

$$A_{c,eff,loc} = A_{st,eff} + \sum_c A_{c,eff}$$

where $\sum_c$ applies to the plates

see Figure 4.4;

$A_{st,eff}$ is the sum of the effective area A_{st} located

$b_{c,loc}$ is the width of the

ρ_{loc} is the reduction factor

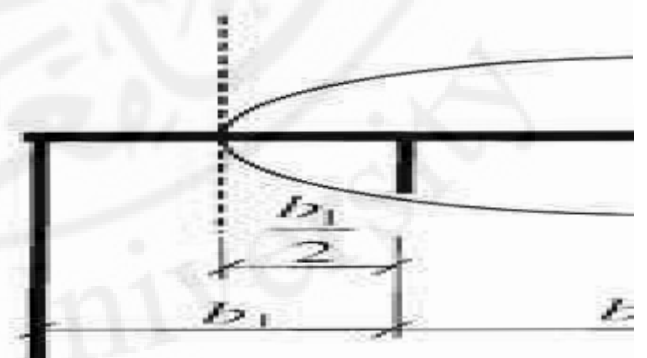


Figure 4.4

NOTE: For non-uniform c

(5) In determining the reduced buckling, which is more severe than

(6) Interpolation should be carried out between the reduced buckling and the reduction factor

(7) The reduction of the compression stress should be applied to the whole cross section.

(8) If shear lag is relevant (see 4.5.2), the effective cross-section of a stiffened plate should then be taken into account for shear lag effects.

(9) The effective cross-section area of the tension zone reduced

(10) The effective section modulus of the tension zone divided by the distance from the neutral axis to the

4.5.2 Plate type behaviour

(1) The relative plate slenderness $\bar{\lambda}_p$ is defined as

$$\bar{\lambda}_p = \sqrt{\frac{\beta_{A,c} f_c}{\sigma_{cr,p}}}$$

with
$$\beta_{A,c} = \frac{A_{c,eff,loc}}{A_c}$$

where A_c is the gross area of the plate supported by an edge where shear lag is relevant

$A_{c,eff,loc}$ is the effective area of the plate due allowance made for shear lag

(2) The reduction factor ρ for the effective area should be calculated from equation (4.7).

NOTE: For calculation of $\sigma_{cr,p}$ see 4.5.2.

4.5.3 Column type buckling

(1) The elastic critical column stress $\sigma_{cr,c}$ should be taken as the buckling stress

(2) For an unstiffened plate the elastic critical column stress $\sigma_{cr,c}$ is given by

$$\sigma_{cr,c} = \frac{\pi^2 E t^2}{12 (1 - \nu^2) a^2}$$

(3) For a stiffened plate $\sigma_{cr,c}$ may be calculated from the stiffener closest to the panel edge

$$\sigma_{cr,c} = \frac{\pi^2 E I_{s,c,1}}{A_{s,c,1} a^2}$$

where $I_{xc,1}$ is the second moment of area of the plate, relative to the centroidal axis x_c ; $A_{xc,1}$ is the gross cross-section area of the plate (Figure A.1).

NOTE: $\sigma_{cc,c}$ may be obtained from the critical stress of the unstiffened plate, and, $\overline{[AC_1]} b_{xc,1} \overline{[AC_1]}$ and b_c are the dimensions of the plate (Figure A.1).

(4) The relative column slenderness $\bar{\lambda}_c$ is defined as follows:

$$\bar{\lambda}_c = \sqrt{\frac{f_y}{\sigma_{cc,c}}} \quad \text{for unstiffened columns}$$

$$\bar{\lambda}_c = \sqrt{\frac{\beta_{A,c} f_y}{\sigma_{cc,c}}} \quad \text{for stiffened columns}$$

with $\beta_{A,c} = \frac{A_{xc,1,eff}}{A_{xc,1}}$;

$A_{xc,1}$ is defined in 4.5.3(3);

$A_{xc,1,eff}$ is the effective cross-section area of the plate, due allowance for plate buckling being made.

(5) The reduction factor χ_c should be determined from the curve $\alpha = 0,21$ corresponding to buckling curve class b, or to:

$$\alpha_c = \alpha + \frac{0,09}{i/e}$$

with $i = \sqrt{\frac{I_{xc,1}}{A_{xc,1}}}$

$e = \max(e_1, e_2)$ is the largest distance from the centroidal axis of the effective column to the stiffener (or of the centroidal axis of the effective column, if no stiffener is present);

$\alpha = 0,34$ (curve b) for closed sections;

$\alpha = 0,49$ (curve c) for open sections.

4.5.4 Interaction between plate and column

(1) The final reduction factor ρ_c should be determined from:

$$\rho_c = (\rho - \chi_c) \xi (2 - \xi) + \chi_c$$

where $\xi = \frac{\sigma_{cc,p}}{\sigma_{cc,c}} - 1$ but $0 \leq \xi \leq 1$

$\sigma_{cc,p}$ is the elastic critical plate stress;

$\sigma_{cc,c}$ is the elastic critical column stress.

χ_c is the reduction factor

ρ is the reduction factor

4.6 Verification

(1) Member verification [AC1]

$$\eta_1 = \frac{\frac{N_{Ed}}{f_y A_{eff}} + \frac{M_{Ed} + N_{Ed} e_N}{f_y W_{eff}}}{\gamma_{M0}}$$

where A_{eff} is the effective cross-sectional area

e_N is the shift in the neutral axis

M_{Ed} is the design bending moment

N_{Ed} is the design axial force

W_{eff} is the effective elastic section modulus

γ_{M0} is the partial factor

NOTE: For members subjected to shear, the following applies:

$$\eta_1 = \frac{\frac{N_{Ed}}{f_y A_{eff}} + \frac{M_{y,Ed} + N_{Ed} e_{y,N}}{f_y W_{y,eff}} + \frac{M_{z,Ed} + N_{Ed} e_{z,N}}{f_y W_{z,eff}}}{\gamma_{M0}}$$

$M_{y,Ed}$, $M_{z,Ed}$ are the design bending moments

[AC1] $e_{y,N}$, $e_{z,N}$ [AC1] are the eccentricities

(2) Action effects M_{Ed} and N_{Ed}

(3) The plate buckling verification 0.4a or 0.5b, whichever is the stronger, for the gross sectional resistance needs to be checked.

5 Resistance to shear

5.1 Basis

(1) This section gives rules for the design of welded joints in the state where the following criteria are satisfied:

- a) the panels are rectangular with constant thickness
- b) stiffeners, if any, are provided perpendicular to the direction of shear
- c) all holes and cut outs are small
- d) members are of uniform cross-section

(2) Plates with h_w/t greater than ε are to be checked for resistance to shear

where $\varepsilon = \sqrt{\frac{235}{f_y [N/mm^2]}}$

NOTE 1: h_w see Figure 5

NOTE 2: The National including S460. For highe

5.2 Design resistance

(1) For unstiffened or stiffe

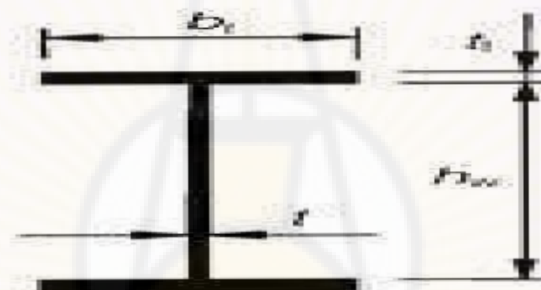
$$V_{b,Rd} = V_{bwc,Rd} + V_{bf,Rd} \geq$$

in which the contribution from

$$V_{bwc,Rd} = \frac{\chi_w f_{yw} h_w t}{\sqrt{3} \gamma_{M1}}$$

and the contribution from the

(2) Stiffeners should compl
9.3.5.



Cross section notations

a

5.3 Contribution from

(1) For webs with transvers
stiffeners or longitudinal stiffe
resistance should be obtained f

Table 5.1: Contr

$\bar{\lambda}_w < 0,83 / \eta$
$0,83 / \eta \leq \bar{\lambda}_w < 1,08$
$\bar{\lambda}_w \geq 1,08$

NOTE: See 6.2.6 in EN 1

- (2) Figure 5.1 shows various end conditions:
- No end post, see 6.1 (2), type 1
 - Rigid end posts, see 9.3.1; continuous girder;
 - Non rigid end posts see 9.3.2

(3) The ACI modified slenderness ratio is:

$$\bar{\lambda}_{wv} = 0,76 \sqrt{\frac{f_{yw}}{\tau_{cr}}}$$

where $\tau_{cr} = k_{\tau} \sigma_E$

NOTE 1: Values for σ_E and $\sigma_{E, \text{min}}$ are given in Table 5.1.

NOTE 2: The ACI modified slenderness ratio is:

a) transverse stiffeners at supports:

$$\bar{\lambda}_{wv} = \frac{h_{wv}}{86,4 t \epsilon}$$

b) transverse stiffeners at supports and between supports:

$$\bar{\lambda}_{wv} = \frac{h_{wv}}{37,4 t \epsilon \sqrt{k_{\tau}}}$$

in which k_{τ} is the minimum slenderness ratio.

NOTE 3: Where non-rigid transverse stiffeners are used, k_{τ} shall be taken as the minimum of the value k_{τ} for the stiffeners at supports (h_{wv}) and that between two rigid transverse stiffeners (h_w).

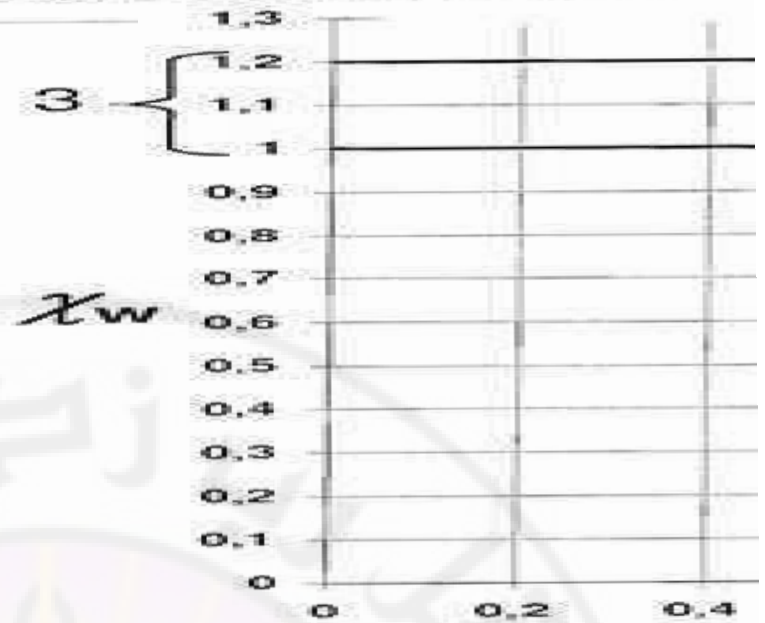
NOTE 4: Rigid boundaries for the web buckling analysis can be assumed (see Figure 5.3).

NOTE 5: For non-rigid transverse stiffeners, the value of k_{τ} shall be determined as follows:

- a combination of two adjacent stiffeners;
- a combination of three adjacent stiffeners.

For procedure to determine k_{τ} see Figure 5.3.

(4) The second moment of area I_w shall be taken as the minimum value when calculating k_{τ} . Formulae for I_w are given in Table 5.2.

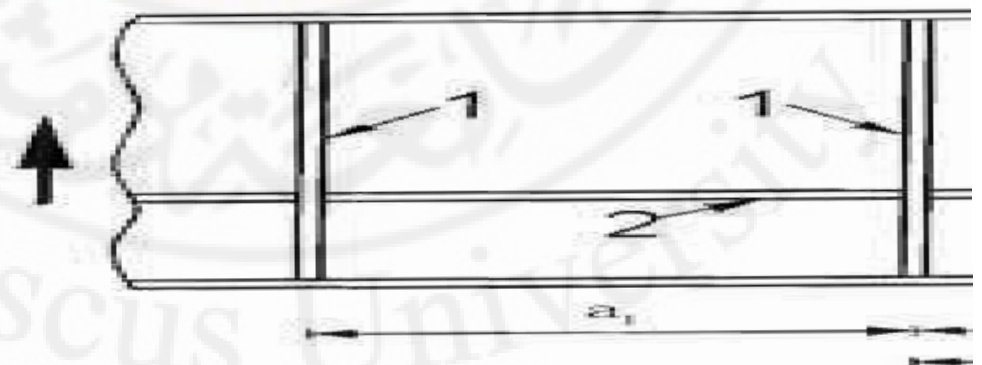


(5) For webs with longitudinal stiffeners as less than

$$\bar{\lambda}_w = \frac{h_{wt}}{37,4 t \epsilon \sqrt{k_{\sigma}}}$$

where h_{wt} and k_{σ} refer to the stiffeners within the web panel under consideration.

NOTE: To calculate k_{σ} the



(a)

Figure 5.3:

5.4 Contribution from

(1) When the flange resists the contribution from the flange

$$V_{bf,Rd} = \frac{b_f t_f^2 f_{yf}}{c \gamma_{M1}} \left(1 - \right.$$

b_f and t_f are taken for the flange
 b_f being taken as not less than

$M_{f,Rd} = \frac{M_{f,k}}{\gamma_{M0}}$ is the moment
 flanges only,

$$c = a \left(0,25 + \frac{1,6 b_f t_f^2 f_{yf}}{t h_w^2 f_{yw}} \right)$$

(2) When an axial force N is present, the following factor:

$$\left(1 - \frac{N_{Ed}}{(A_{f1} + A_{f2}) f_{yf}} \right) \gamma_{M0}$$

where A_{f1} and A_{f2} are the areas

5.5 Verification

(1) The verification should be

$$\eta_3 = \frac{V_{Ed}}{V_{bf,Rd}} \leq 1,0$$

where V_{Ed} is the design shear

6 Resistance to transverse

6.1 Basis

(1) The design resistance is given by 6.2, provided

(2) The load is applied as follows:

- a) through the flange and resists
- b) through one flange and transverse
- c) through one flange adjacent

- (3) For box girders with inclined web internal forces to be taken into account for each flange respectively.
- (4) The interaction of the transverse

Type (a)

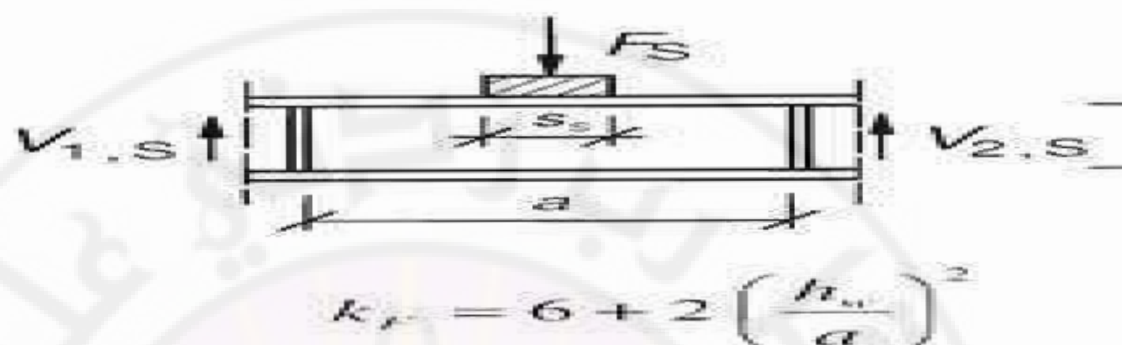


Figure 6.1: Buckling coefficient

6.2 Design resistance

- (1) For unstiffened or stiffened web the design resistance F_{Rd} should be taken as

$$F_{Rd} = \frac{f_{yw} L_{eff} t_w}{\gamma_{M1}}$$

- where t_w is the thickness of the web;
 f_{yw} is the yield strength of the web;
 L_{eff} is the effective length for shear

$$L_{eff} = \chi_F \ell_y$$

- where ℓ_y is the effective loaded length;
 χ_F is the reduction factor due to shear

6.3 Length of stiff bearing

- (1) The length of stiff bearing s_s on which the load F_S is effectively distributed at a slope of 45° should be taken as
- (2) If several concentrated forces are applied, the length s_s should be taken as for the total load with

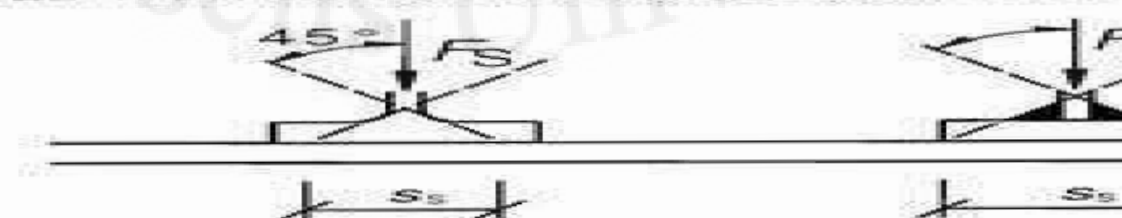


Figure 6.2: Length of stiff bearing

- (3) If the bearing surface of the flange should be taken as zero.

6.4 Reduction factor χ_F for flanges

- (1) The reduction factor χ_F should be determined as follows:

$$\chi_F = \frac{0,5}{\bar{\lambda}_F} \leq 1,0$$

where $\bar{\lambda}_F = \sqrt{\frac{\ell_x t_w f_{yw}}{F_{cr}}}$

$$F_{cr} = 0,9 k_F E \frac{t_w^3}{h_w}$$

- (2) For webs without longitudinal stiffeners:

NOTE: For webs with longitudinal stiffeners, the following rules are recommended:

For webs with longitudinal stiffeners:

$$k_F = 6 + 2 \left[\frac{h_w}{a} \right]^2 + \left[5,44 \frac{b_1}{h_w} \right]$$

where b_1 is the depth of the stiffener

$$\gamma_s = 10,9 \frac{I_{s,c,1}}{h_w t_w^3} \leq 13 \left[\frac{a}{h_w} \right]$$

where $I_{s,c,1}$ is the second moment of area of the web according to Figure 6.1

Equation (6.6) is valid for $0,6 \leq \gamma_s \leq 13$

- (3) ℓ_y should be obtained from Equation (6.5)

6.5 Effective loaded length ℓ_x

- (1) The effective loaded length ℓ_x should be determined as follows:

$$m_1 = \frac{f_{yf} b_f}{f_{yw} t_w}$$

$$m_2 = 0,02 \left(\frac{h_w}{t_f} \right)^2 \quad \text{if } \bar{\lambda}_F \leq 0,5$$

$$m_2 = 0 \quad \text{if } \bar{\lambda}_F > 0,5$$

For box girders, b_f in equation (6.7) should be taken as the flange width.

- (2) For types a) and b) in Figure 6.2, ℓ_x should be determined as follows:

$$\ell_x = s_y + 2 t_f \left(1 + \sqrt{m_1 + m_2} \right)$$

(3) For type c) ℓ_y should be taken as

$$\ell_y = \ell_c + t_f \sqrt{\frac{m_1}{2} + \left(\frac{\ell_c}{t_f} \right)^2}$$

$$\ell_y = \ell_c + t_f \sqrt{m_1 + m_2}$$

where $\ell_c = \frac{k_f E t_w^2}{2 f_{vw} h_w} \leq s_x + c$

6.6 Verification

(1) The verification should be

$$\eta_2 = \frac{F_{Ed}}{f_{vw} L_{eff} t_w} \leq 1,0$$

where F_{Ed} is the design transverse force

L_{eff} is the effective length

t_w is the thickness of the web

7 Interaction

7.1 Interaction between shear and bending

(1) Provided that $\bar{\eta}_3$ (see below) is not less than 0,5, the design shear force need not be reduced to allow for the interaction with bending and shear in the web of a

$$\bar{\eta}_1 + \left(1 - \frac{M_{f,Rd}}{M_{pl,Rd}} \right) (2\bar{\eta}_3 - 1) \leq 1,0$$

where $M_{f,Rd}$ is the design plastic moment resistance of the flanges;

$M_{pl,Rd}$ is the design plastic moment resistance of the flanges and the full section;

$$\bar{\eta}_1 = \frac{M_{Ed}}{M_{pl,Rd}}$$

$$\bar{\eta}_3 = \frac{V_{Ed}}{V_{tw,Rd}} \quad \text{[AC1] for } V_{Ed} \leq V_{tw,Rd}$$

In addition the requirements in section 6.2.2 should be satisfied.

Action effects should include global effects.

(2) The criterion given in (1) should not be applied to regions more than $h_w/2$ from a support with vertical shear.

(3) The plastic moment of resistance of the flange with the small

(4) If an axial force N_{Ed} is EN 1993-1-1 and 5.4(2) respectively 7.1(5) should be applied.

(5) A flange in a box girder shear stress in the flange which taken as η_1 according to 4.6(1) within the subpanel and χ_w de longitudinal stiffeners to be rigid

7.2 Interaction between

(1) If the girder is subjected conjunction with bending and interaction expression:

$$\eta_2 + 0,8 \eta_1 \leq 1,4$$

(2) If the concentrated load section 6. Additionally 6.2.1(5)

8 Flange induced bu

(1) To prevent the compress met:

$$\frac{h_w}{t_w} \leq k \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc}}}$$

where A_w is the cross section A_{fc} is the effective cro h_w is the depth of the t_w is the thickness of

The value of the factor k should

- plastic rotation utilized
- plastic moment resistance M_{pl}
- elastic moment resistance M_{el}

(2) When the girder is curved criterion should be met:

$$\frac{h_w}{t_w} \leq \frac{k \frac{E}{f_{yf}} \sqrt{\frac{A_w}{A_{fc}}}}{\sqrt{1 + \frac{h_w E}{3 r f_{yf}}}}$$

r is the radius of curvature of th

NOTE: The National Annex

9 Stiffeners and details

9.1 General

(1) This section gives design rules specified in sections 4 to 6.

NOTE: The National Annex may specify additional rules.

(2) When checking the buckling resistance of a stiffener, the stiffener plus a portion of the plate on each side of the stiffener axis should be considered.

(3) The axial force in a stiffener should be taken as the sum of the axial force in the plate (see 9.3.3(3)) and any external force.

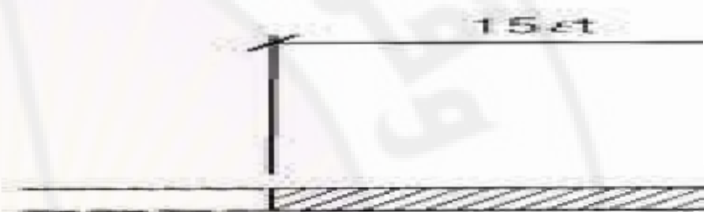


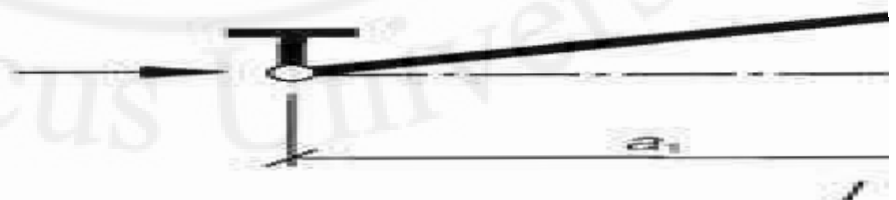
Figure 9.1

9.2 Direct stresses

9.2.1 Minimum requirements

(1) In order to provide a minimum resistance to buckling, transverse stiffeners should satisfy the following requirements:

(2) The transverse stiffener should be designed for an initial sinusoidal imperfection w_0 where a_1 and a_2 are the lengths of the stiffener between the height of the stiffener and the height between the centres of adjacent stiffeners, respectively, accounted for.



(3) The transverse stiffener should be designed for the assumption that both adjacent plates are subjected to the same axial force.

and axial force according to the NEd should be considered to be simply supported.

(4) It should be verified that $\sigma_{m,cr}$ is satisfied at the ultimate limit state:

- that the maximum stress in the stiffener is
- that the additional deflection w_{ed} is

(5) In the absence of an axial force N_{Ed} , $\sigma_{m,cr}$ is assumed to be satisfied provided that u is greater than:

$$I_{cr} = \frac{\sigma_{m,cr}}{E} \left(\frac{b}{\pi} \right)^4 \left(1 + w_0 \frac{300}{b} \right)$$

where
$$\sigma_{m,cr} = \frac{\sigma_{cr,c} + \sigma_{cr,p}}{2} \frac{N_{Ed}}{b} \left(\frac{1}{a_1} + \frac{1}{a_2} \right)$$

$$u = \frac{\pi^2 E e_{max}}{f_y 300 b} \geq 1,0$$

e_{max} is the maximum displacement

N_{Ed} is the maximum compressive stress

$\sigma_{cr,c} + \sigma_{cr,p}$ are defined in EN 1993-1-7.

NOTE: Where out of plane load N_{Ed} is defined in EN 1993-2 and EN 1993-1-7.

(6) If the stiffener carries axial force, account for deviation forces. The uniform stresses from axial load in the stiffener is

(7) As a simplification the required order elastic analysis taking account q acting on the length b :

$$q = \frac{\pi}{4} \sigma_{m,cr} (w_0 + w_{ed})$$

where $\sigma_{m,cr}$ is defined in (5) above

w_0 is defined in Figure 5

w_{ed} is the elastic deformation additional deflection

(8) Unless a more advanced analysis is used for stiffeners with open cross-section:

$$\frac{I_T}{I_p} \geq 5,3 \frac{f_y}{E}$$

where I_p is the polar second moment of area

I_T is the St. Venant torsion constant

(9) Where warping stiffness is considered:

$$\sigma_{cr} \geq \theta f_y$$

where σ_{cr} is the elastic critical stress of the plate;

θ is a parameter to ensure ductility.

NOTE: The parameter θ may be determined by the following rules.

9.2.2 Minimum requirements for stiffeners

(1) The requirements concerning stiffeners.

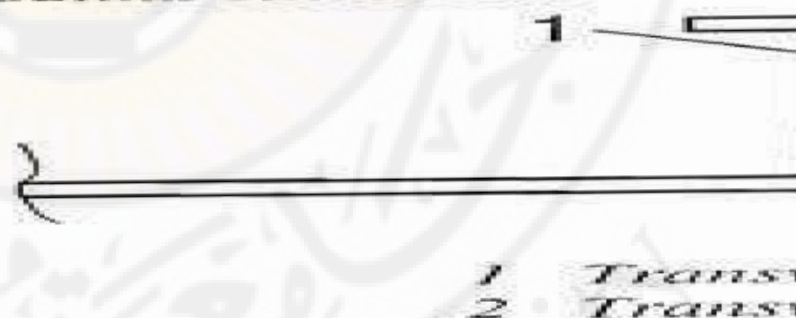
(2) Discontinuous longitudinal stiffeners or are not connected to either

- used only for webs (i.e. not allowed for flanges);
- neglected in global analysis;
- neglected in the calculation of stress in the flanges;
- considered in the calculation of the moment of resistance;
- considered in the calculation of the moment of resistance.

(3) Strength assessments for stiffeners.

9.2.3 Welded plates

(1) Plates with changes in plate thickness. The effects of eccentricity need to be considered if the eccentricity of the welded junction exceeds $b_0/2$ or 200 mm, whichever is smaller, between longitudinal stiffeners.



F

9.2.4 Cut outs in stiffeners

(1) The dimensions of cut out

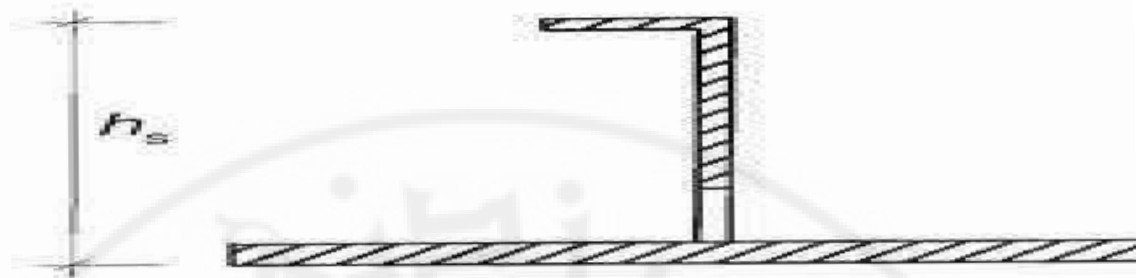


Figure 9.5

(2) The length ℓ should not

$\ell \leq 6 t_{\min}$ for flat stiffener

$\ell \leq 8 t_{\min}$ for other stiffen

$\ell \leq 15 t_{\min}$ for stiffeners wi

where $t_{\min}$ is the lesser of the pla

(3) The limiting values ℓ is

$\sigma_{x,Ed} \leq \sigma_{x,Rd}$ and $\ell \leq 15 t_{\min}$

$\sigma_{x,Ed}$ is the compression stress

(4) The dimensions of cut out

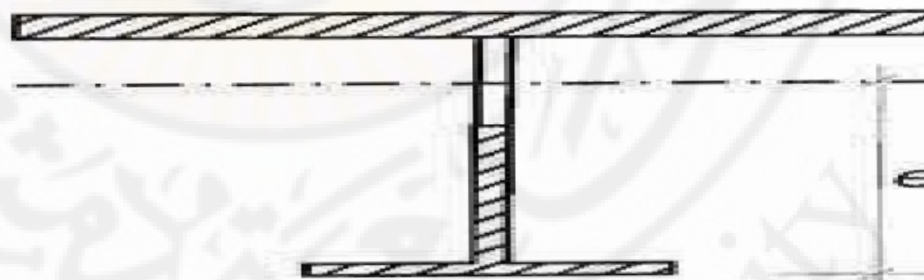


Figure 9.5

(5) The gross web adjacent to

$$V_{Ed} = \frac{I_{net}}{e} \frac{f_{yk}}{\gamma_{M0}} \frac{\pi}{b_G}$$

I_{net} is the second mome

e is the maximum dis
see Figure 9.5;

b_G is the length of the t

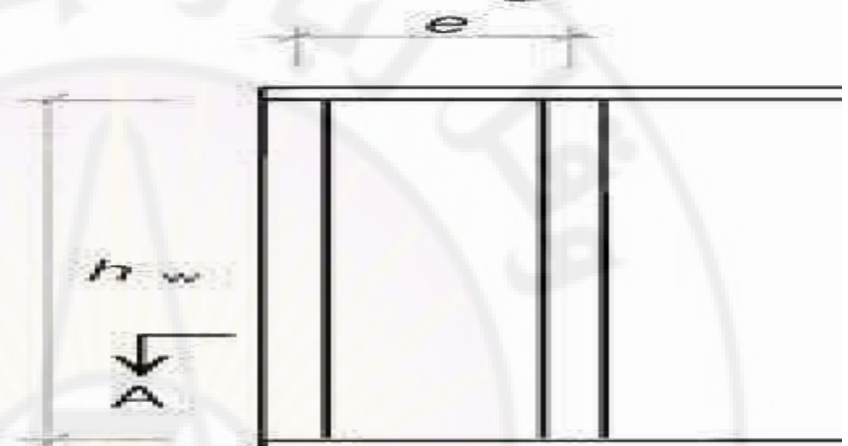
9.3 Shear

9.3.1 Rigid end post

(1) The rigid end post (see Figure 5.3(1)), and should be designed in the plane of the web.

NOTE: For the effects of eccentricity, see 5.3(2).

(2) A rigid end post should comprise a short beam of length h_w , see Figure 9.6. Alternatively, a rigid end post may be provided in the web plate as shown in Figure 9.6.



1 Insert

Figure 9.6:

(3) Each double sided stiffener should be designed as a beam of length h_w , where e is the centre to centre distance between stiffeners. If a rolled section other than flats is used, the stiffener should be designed for bending around a horizontal axis perpendicular to the web.

(4) As an alternative the girder end stiffener adjacent to the support so that the rigid end post.

9.3.2 Stiffeners acting as non-rigid end post

(1) A non-rigid end post may be a bearing stiffener resisting the reaction force.

9.3.3 Intermediate transverse stiffeners

(1) Intermediate stiffeners that act as bearing stiffeners for strength and stiffness.

(2) When flexible intermediate transverse stiffeners are used, the calculation of k_s in 5.3(5).

(3) The effective section of minimum second moment of

$$\text{if } a/h_w < \sqrt{2}: I_{se} \geq$$

$$\text{if } a/h_w \geq \sqrt{2}: I_{se} \geq$$

NOTE: Intermediate

$$\left[V_{Ed} - \frac{1}{\lambda_w^2} f_{yw} h_w t / (\sqrt{3} \gamma_{M1}) \right]$$

performed for the shear for

9.3.4 Longitudinal stiffener

(1) If longitudinal stiffener stresses for the cross sectional

9.3.5 Welds

(1) The web to flange weld

$\chi_w f_{yw} h_w t / (\sqrt{3} \gamma_{M1})$. For large shear flow $\eta f_{yw} t / (\sqrt{3} \gamma_{M1})$.

(2) In all other cases weld sections taking into account a

9.4 Transverse loads

(1) If the design resistance

(2) The out-of-plane buckling (see 9.3.3(3)) should be determined. If both ends are assumed to be fixed, the value of ℓ should be used for the loaded end, the cross sectional

(3) Where single sided or double sided stiffeners are allowed for using 6.3.3 or 6.3.4, the compression flange they resist torsional buckling.

10 Reduced stress method

(1) The reduced stress method applies to unstiffened and stiffened plates.

NOTE 1: This method is applicable to the following:

- $\sigma_{x,Ed}$, $\sigma_{z,Ed}$ and τ_{Ed} are
- the stress limits of the

NOTE 2: The stress limits $\sigma_{x,Ed}$, $\sigma_{z,Ed}$ and τ_{Ed} give limits of application for the reduced stress method.

(2) For unstiffened or stiffened plates, the properties may be assumed, with ρ and χ_{M1} given by:

$$\frac{\rho \alpha_{ult,k}}{\chi_{M1}} \geq 1$$

where $\alpha_{ult,k}$ is the minimum resistance of the plate under the complete design load.

ρ is the reduction factor for buckling, see (5).

χ_{M1} is the partial factor for material resistance.

(3) The α_{cr} modified plate resistance is given by:

$$\bar{\lambda}_p = \sqrt{\frac{\alpha_{ult,k}}{\alpha_{cr}}}$$

where α_{cr} is the minimum resistance of the plate under the complete design load.

NOTE 1: For calculating α_{cr} , see section 4 and 5 without reduction factor ρ .

NOTE 2: When α_{cr} cannot be determined for a subpanel and the full panel resistance is used, the reduction factor ρ is given by:

(4) In determining $\alpha_{ult,k}$ the following formula is used:

$$\frac{1}{\alpha_{ult,k}^2} = \left(\frac{\sigma_{x,Ed}}{f_y} \right)^2 + \left(\frac{\sigma_{z,Ed}}{f_y} \right)^2 + \left(\frac{\tau_{Ed}}{f_v} \right)^2$$

where $\sigma_{x,Ed}$, $\sigma_{z,Ed}$ and τ_{Ed} are the design stresses.

NOTE: By using the equivalent stress σ_{eq} for plate buckling.

(5) The reduction factor ρ is given by:

a) the minimum value of the following:

ρ_x for longitudinal stresses

ρ_z for transverse stresses

χ_w for shear stresses from

each calculated for the **[AC1]**

NOTE: This method leads

$$\left(\frac{\sigma_{x,Ed}}{f_y / \gamma_{M1}} \right)^2 + \left(\frac{\sigma_{z,Ed}}{f_y / \gamma_{M1}} \right)^2$$

NOTE: For determining γ to σ_z in the z-direction. For

- b) a value interpolated between as interpolation function

NOTE: This method leads

$$\left(\frac{\sigma_{x,Ed}}{\rho_x f_y / \gamma_{M1}} \right)^2 + \left(\frac{\sigma_{z,Ed}}{\rho_z f_y / \gamma_{M1}} \right)^2$$

NOTE 1: Since verification bending moment, axial force

NOTE 2: The National A of panels with tension and compressive parts.

- (6) Where α_{cr} values for the components of the stress field

$$\frac{1}{\alpha_{cr}} = \frac{1 + \psi_x}{4 \alpha_{cr,x}} + \frac{1 + \psi_z}{4 \alpha_{cr,z}}$$

where $\alpha_{cr,x} = \frac{\sigma_{cr,x}}{\sigma_{x,Ed}}$

$$\alpha_{cr,z} = \frac{\sigma_{cr,z}}{\sigma_{z,Ed}}$$

$$\text{[AC1]} \quad \alpha_{cr,t} = \frac{\tau_{cr}}{\tau_{Ed}} \quad \text{[AC1]}$$

and $\sigma_{cr,x}$, $\sigma_{cr,z}$, τ_{cr} , ψ_x and ψ_z are

- (7) Stiffeners and detailing c

Annex A [informative] – plates

A.1 Equivalent orthotropic

(1) Plates with at least three longitudinal stiffeners;

(2) The elastic critical plate buckling stress is not less than

$$\sigma_{\alpha,p} = k_{\alpha,p} \sigma_E$$

where $\sigma_E = \frac{\pi^2 E t^2}{12 (1 - \nu^2) b^2} = 190000 \frac{t^2}{b^2}$

$k_{\alpha,p}$ is the buckling coefficient of the plate;

b is defined in Figure A.1;

t is the thickness of the plate.

NOTE 1: The buckling coefficient $k_{\alpha,p}$ can be determined by means of relevant computer simulations; alternative methods for determining $k_{\alpha,p}$ are not recommended; alternative buckling in the subpanels can be ignored.

NOTE 2: $\sigma_{E,p}$ is the elastic critical stress at which global buckling occurs, see Figure A.1.

NOTE 3: Where a web is of concealed fasteners, the stiffeners are not effective.

NOTE 4: For stiffened plates with stiffeners of different stiffness, the coefficient $k_{\alpha,p}$ (global buckling of the plate) is given by:

$$k_{\alpha,p} = \frac{2 \left((1 + \alpha^2)^2 + \gamma - 1 \right)}{\alpha^2 (\psi + 1) (1 + \delta)} \quad \text{if } \alpha \geq 0.5$$

$$k_{\alpha,p} = \frac{4 (1 + \sqrt{\gamma})}{(\psi + 1) (1 + \delta)} \quad \text{if } \alpha < 0.5$$

with: $\psi = \frac{\sigma_2}{\sigma_1} \geq 0.5$

$$\gamma = \frac{I_{st}}{I_p}$$

$$\delta = \frac{A_{st}}{A_p} \quad \text{[AC1]}$$

$$\alpha = \frac{a}{b} \geq 0.5$$

where: I_{st} is the second moment of area of the stiffeners;

I_p is the second moment of area of the plate;

A_{st} is the sum of the cross-sectional areas of the stiffeners;

A_p is the gross area of the plate;

σ_1 is the larger edge stress;

σ_2 is the smaller edge stress.

a , b and t are :

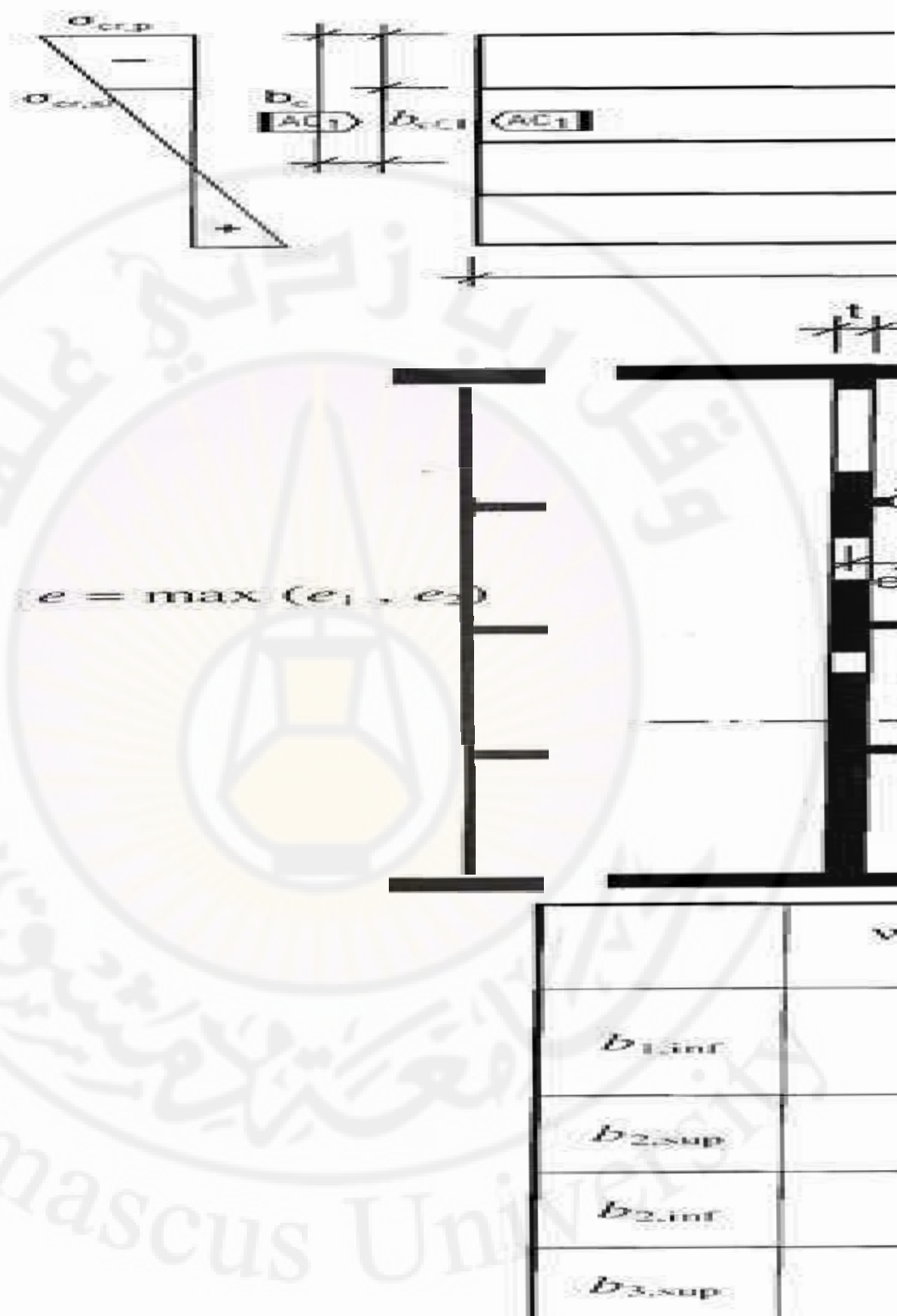


Figure A.1:

A.2 Critical plate buckling compression zone

A.2.1 General procedure

(1) If the stiffened plate has only one compression zone, the procedure may be simplified by a fictitious isolation of the direction perpendicular to this strain.

(2) For calculation of $A_{s,1}$ and $I_{s,1}$, the area of the stiffener and adjacent compression zone, a portion $(3-\psi)/(5-\psi)$ at the edge with the highest subpanel, a portion $0,4$ of the width b of the column, see Figure A.2 and also Table A.1.

(3) The effective^p cross-sectional area of the stiffener and the adjacent effective elements in the column may be determined from the section of the plate.

(4) If $\rho_c f_y / \chi_{N1}$, with ρ_c determined from (4.6), no further reduction of the effective area $A_{s,1}$ in (4.6) should be modified as follows:

$$A_{s,eff,dec} = \frac{\rho_c f_y A_{s,1}}{\sigma_{com,Ed} \chi_{N1}}$$

(5) The reduction mentioned in A.2.1(4) need be applied to other compressed parts of the plate.

(6) As an alternative to using an effective area $A_{s,eff,dec}$ determined from A.2.1(5) to (7) and c

NOTE: The method outlined in (6) from the plate is neglected, that is:

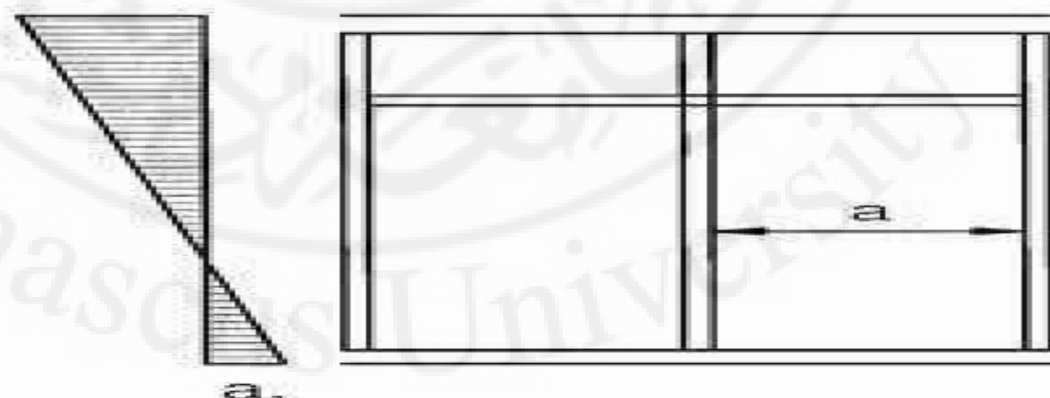


Figure A.2: Notations for critical plate buckling compression zone

(7) If the stiffened plate has two compression zones, the procedure described in A.2.1(1) may be applied to each zone.

- buckles while the other one is accounted for by considering a
- a) its cross-sectional area and its individual stiffeners
 - b) it is positioned at the location of the stiffeners

For each of these situations illustrate the following:

- $b_1 = b_1^*$ and $b_2 = b_2^*$ and $B^* =$



Cross-sectional area $A_{sc,i}$

Second moment of area $I_{sc,i}$

Figure A.3: Notation

A.2.2 Simplified model for stiffened plates

(1) In the case of a stiffened plate, the elastic critical buckling stress $\sigma_{cr,el}$ is given by the following zone:

$$\sigma_{cr,el} = \frac{1.05 E \sqrt{I_{sc,i}} t^3}{A_{sc,i} b_1 b_2}$$

$$\sigma_{cr,el} = \frac{\pi^2 E I_{sc,i}}{A_{sc,i} a^2} + \frac{\pi^2 E I_{sc,i}}{4 b^2}$$

with $a_c = 4.33 \sqrt{\frac{I_{sc,i} b_1^2 b_2^2}{t^3 b}}$

where $A_{sc,i}$ is the gross area of the stiffener

$I_{sc,i}$ is the second moment of area about an axis through the centroid of the stiffener

b_1, b_2 are the distances from the centroid of the stiffener to the edges of the plate

[AC1] note deleted [AC1]

(2) In the case of a stiffened plate, the elastic critical plate buckling stress $\sigma_{cr,el}$ is given by the following zone:

equation (A.4) with $b_1 = b_1^*$, . . . calculation.

A.3 Shear buckling

(1) For plates with rigid transverse longitudinal stiffeners, the shear

$$k_{\tau} = 5,34 + 4,00 \left(\frac{h_w}{t} \right)^2$$

$$k_{\tau} = 4,00 + 5,34 \left(\frac{h_w}{t} \right)^2$$

where $k_{\tau w} = 9 \left(\frac{h_w}{a} \right)^2 \sqrt[4]{\left(\frac{I_{yw}}{t^3} \right)}$

a is the distance between

I_{yw} is the second mo

⌈AC1⌋ For webs with ⌈A

the stiffness of the

NOTE: No intermediate

(2) The equation (A.5) also

$\alpha = \frac{a}{h_w}$ satisfies $\alpha \geq 3$. For

shear buckling coefficient sho

$$k_{\tau} = 4,1 + \frac{6,3 + 0,18 \frac{t}{h_w}}{\alpha^2}$$

Annex B [informative]

B.1 General

(1) The rules in section 10 apply to flanges and to webs with regular stiffeners.

(2) α_{wh} and α_{cef} may be obtained from Table B.1.

(3) The reduction factors ρ for stiffeners are given by the design curves, see sections 4 and 5.

NOTE: The reduction factor ρ is defined as

$$\rho = \frac{1}{\phi_p + \sqrt{\phi_p^2 - \bar{\lambda}_p}}$$

where $\phi_p = \frac{1}{2} \left(1 + \alpha_p \left(\frac{\bar{\lambda}_p}{\bar{\lambda}_{p0}} \right)^2 \right)$

and $\bar{\lambda}_p = \sqrt{\frac{\alpha_{wh,k}}{\alpha_{cef}}}$

This procedure applies to flanges and webs. The values have been calibrated for flanges and webs in correlation to the equivalent stress σ_{eq} .

$$e_0 = \alpha_p \left(\bar{\lambda}_p - \bar{\lambda}_{p0} \right) \frac{t}{6} \frac{1}{\rho}$$

Proc
hot rolled
welded cold formed

B.2 Interaction of plate buc

(1) The method given in B.1 may be used for the calculation of torsional buckling of members by calculating α_{min} as the minimum load amplifier for the most critical cross section, neglecting the effect of plate buckling.

α_{min} is the minimum load amplifier for the most critical cross section, neglecting the effect of plate buckling.

α_{cr} is the minimum load amplifier for the most critical member including plate buckling.

(2) When α_{cr} contains lateral torsional buckling, the minimum of the reduction factor χ according to 6.3.3 of EN 1993-1-1 shall be used.

Annex C [informative]

C.1 General

(1) Annex C gives guidance for fatigue verifications of plated structures.

NOTE 1: For FE-calculations.

NOTE 2: This guidance is for design.

(2) The choice of the FE-model depends on the following assumptions:

Table C.1

No	Material behaviour	Geometrical behaviour
1	linear	linear
2	non linear	linear
3	linear	non linear
4	linear	non linear
5	non linear	non linear

C.2 Use

- (1) In using FEM for design, the following aspects should be considered:
- the modelling of the structure;
 - the choice of software and of the mesh;
 - the use of imperfections;
 - the modelling of material properties;
 - the modelling of loads;
 - the modelling of limit state design;
 - the partial factors to be applied.

NOTE: The National Annex may specify additional aspects.

C.3 Modelling

(1) The choice of FE-model should be based on the required accuracy of results. For validation, a higher accuracy is required.

- (2) The FE-modelling may be based on the following assumptions:
- the component as a whole or
 - a substructure as a part of the structure.

NOTE: An example for a substructure is a region of an intermediate stiffener or a subpanel of a bottom plate.

(3) The boundary conditions should be chosen so that the results obtained are conservative.

- (4) Geometric properties should be
- (5) All imperfections should be bas
- (6) Material properties should conf

C.4 Choice of software and

- (1) The software should be suitable

NOTE: Reliability can be proven

- (2) The mesh size, loading, bound documented in a way that they can be

C.5 Use of imperfections

- (1) Where imperfections need to b geometric and structural imperfections

- (2) Unless a more refined analysi carried out, equivalent geometric impe

NOTE 1: Geometric imperfectio amplitudes given in the National A

NOTE 2: Structural imperfection: fabrication process with amplitudes

- (3) The direction of the applied imp
- (4) For applying equivalent geomet

Table C.2:

Type of imperfection	
global	memb
global	longitudinal
local	panel or subpa
local	stiffener or

Type of imperfection
global member with length ℓ
global longitudinal stiffener with length a
local panel or subpanel
local stiffener or flange subject to twist

Figure C.1: M

(5) In combining imperfections may have their

NOTE 1: Any type of in the accompanying imperfi

NOTE 2: Equivalent ge the member.

C.6 Material propert

(1) Material properties sho

(2) Depending on the ac assumptions for the material b

a) elastic-plastic without strai

b) elastic-plastic with a nomin

c) elastic-plastic with linear s

d) true stress-strain curve mod

$$\sigma_{true} = \sigma (1 + \epsilon)$$

$$\epsilon_{true} = \ln (1 + \epsilon)$$



Figur

NOTE: For the elastic model

C.7 Loads

(1) The loads applied to the structure.
For simplicity a single load must be used.

C.8 Limit state criteria

(1) The ultimate limit state criteria are:
1. for structures susceptible to buckling, attainment of the maximum load;
2. for regions subjected to tensile forces, attainment of a limiting value of the load.

NOTE 1: The National Annex should recommend a value for the load.

NOTE 2: Other criteria should be used for the yielding zone.

C.9 Partial factors

(1) The load magnification factor α_1 for reliability.

(2) The magnification factor α_2 for:
1. α_1 to cover the model of test calibrations, see EN 1993-1-1;
2. α_2 to cover the scatter of the material properties and γ_{M2} if fracture is considered.

(3) It should be verified that:

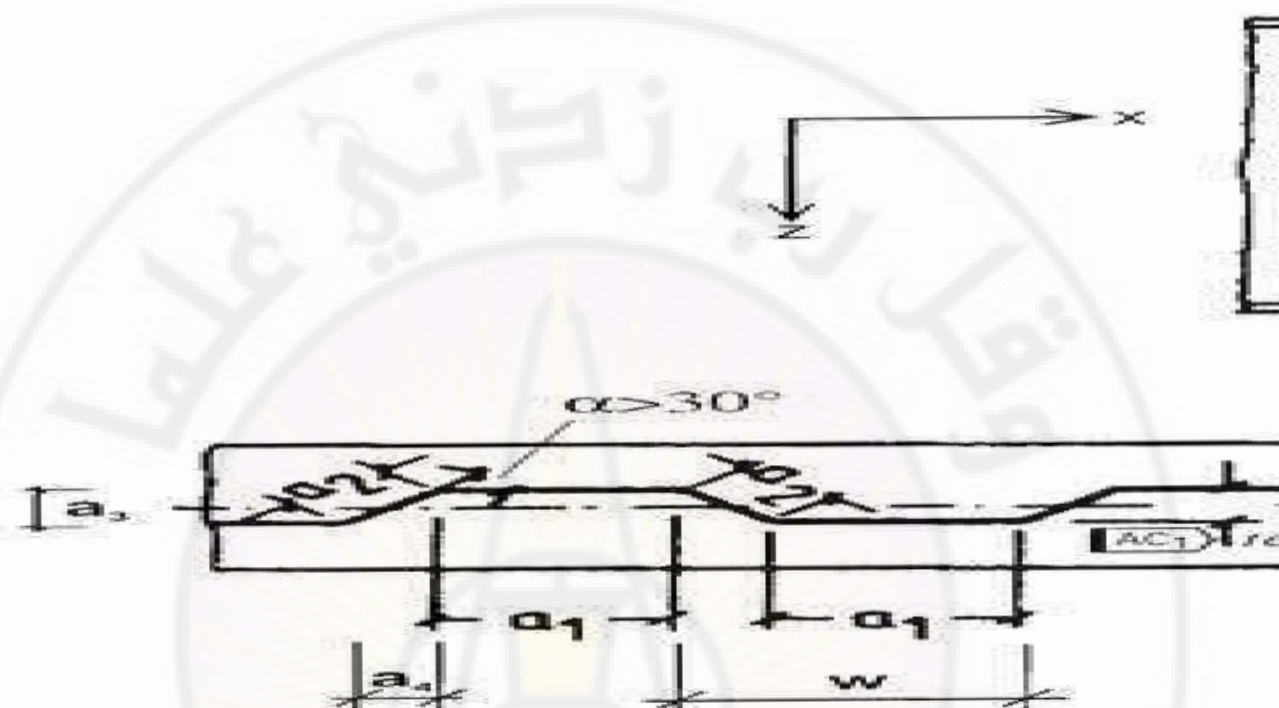
$$\alpha_{eff} > \alpha_1 \cdot \alpha_2$$

NOTE: The National Annex should specify the relevant parts of EN 1993-1-1.

Annex D [informative] –

D.1 General

(1) Annex D covers design rules D.1.



Fig

D.2 Ultimate limit state

D.2.1 Moment of resistance

(1) The moment of resistance $[AC_1]$ $M_{y,Rd}$ is

$$[AC_1] M_{y,Rd} [AC_1] = \min \left\{ \underbrace{\frac{b_2 t_2 f_{yT,r}}{\gamma_{M0}} \left(h_w + \frac{t}{2} \right)}_{\text{tension flange}} \right\}$$

where $f_{yT,r}$ is the value of yield stress

$$f_{yT,r} = f_{yT} f_r$$

$$f_r = 1 - 0,4 \sqrt{\frac{\sigma_x(M_z)}{f_{yT}}} \frac{1}{\gamma_{M0}}$$

$\sigma_x(M_z)$ is the stress due to the

χ is the reduction factor

NOTE 1: The transverse moment

NOTE 2: For sinusoidally corrug

Figure D.2: Transverse

- (2) The effective^p area of the of the slenderness parameter $\bar{\lambda}$ a) and b): AC1

a)
$$k_{\sigma} = 0,43 + \left(\frac{b}{a} \right)^2$$

where b is the maximum width
 $a = a_1 + 2a_2$

b)
$$k_{\sigma} = 0,60$$

AC1 text deleted AC1

D.2.2 Shear resistance

- (1) The shear resistance AC1

$$\text{[AC1]} V_{bw,Rd} \text{ [AC1]} = \chi_c \frac{f_{yw}}{\gamma_{M1} \sqrt{3}} h_w t_w$$

where χ_c is the lesser of the obtained from (2):

- (2) The reduction factor $\chi_{c,t}$

$$\chi_{c,t} = \frac{1,15}{0,9 + \bar{\lambda}_{c,t}} \leq 1,0$$

AC1 where $\bar{\lambda}_{c,t} = \sqrt{\frac{f_{yw}}{\tau_{cr,t} \sqrt{3}}} \text{ [AC1]}$

$$\tau_{cr,t} = 4,83 E \left[\frac{t_w}{a_{\max}} \right]^2$$

$a_{\max}$ should be taken as the

NOTE: For sinusoidally corrugated webs, $\chi_{cr,y}$ and $\chi_{cr,z}$ are:

The use of the following equation:

$$\tau_{cr,y} = (5,34 + \frac{\alpha_3 s}{h_w t_w}) \frac{\pi^2 E}{12(1-\nu^2)}$$

where w is the length of one half wave and s is the unfolded length.

(3) The reduction factor $\chi_{c,y}$ for

$$\chi_{c,y} = \frac{1,5}{0,5 + \bar{\lambda}_{c,y}^2} \leq 1,0$$

[AC1] where $\bar{\lambda}_{c,y} = \sqrt{\frac{f_{yw}}{\tau_{cr,y} \sqrt{3}}}$ **[AC1]**

$$\tau_{cr,y} = \frac{32,4}{t_w^2 h_w^2} \sqrt{D_x D_z^3}$$

$$D_x = \frac{E t_w^3}{12(1-\nu^2)} \frac{w}{s}$$

$$D_z = \frac{E I_z}{w}$$

I_z second moment of area of the web

NOTE 1: s and I_z are related to

NOTE 2: Equation (D.10) is valid for

D.2.3 Requirements for end

(1) Bearing stiffeners should be

Annex E [normative cross sections]

E.1 Effective areas for

(1) As an alternative to the effective areas at stress levels 1

a) for internal compression ele

$$\rho = \frac{1 - 0,055(3 + \psi) / \bar{\lambda}}{\lambda_{p,red}}$$

b) for outstand compression el

$$\rho = \frac{1 - 0,188 / \bar{\lambda}_{p,red}}{\lambda_{p,red}} + 0$$

For notations see 4.4(2) and 4.4

E.2 Effective areas for

(1) For the calculation of ef be calculated from:

$$\bar{\lambda}_{p,ser} = \bar{\lambda}_p \sqrt{\frac{\sigma_{com,Ed,ser}}{f_y}}$$

where $\sigma_{com,Ed,ser}$ is defined as (section) in the

(2) The second moment of effective cross section for the p

$$I_{eff} = I_{gr} - \frac{\sigma_{gr}}{\sigma_{com,Ed,ser}} (I_{gr} - I_{eff}(\sigma_{com,Ed,ser}))$$

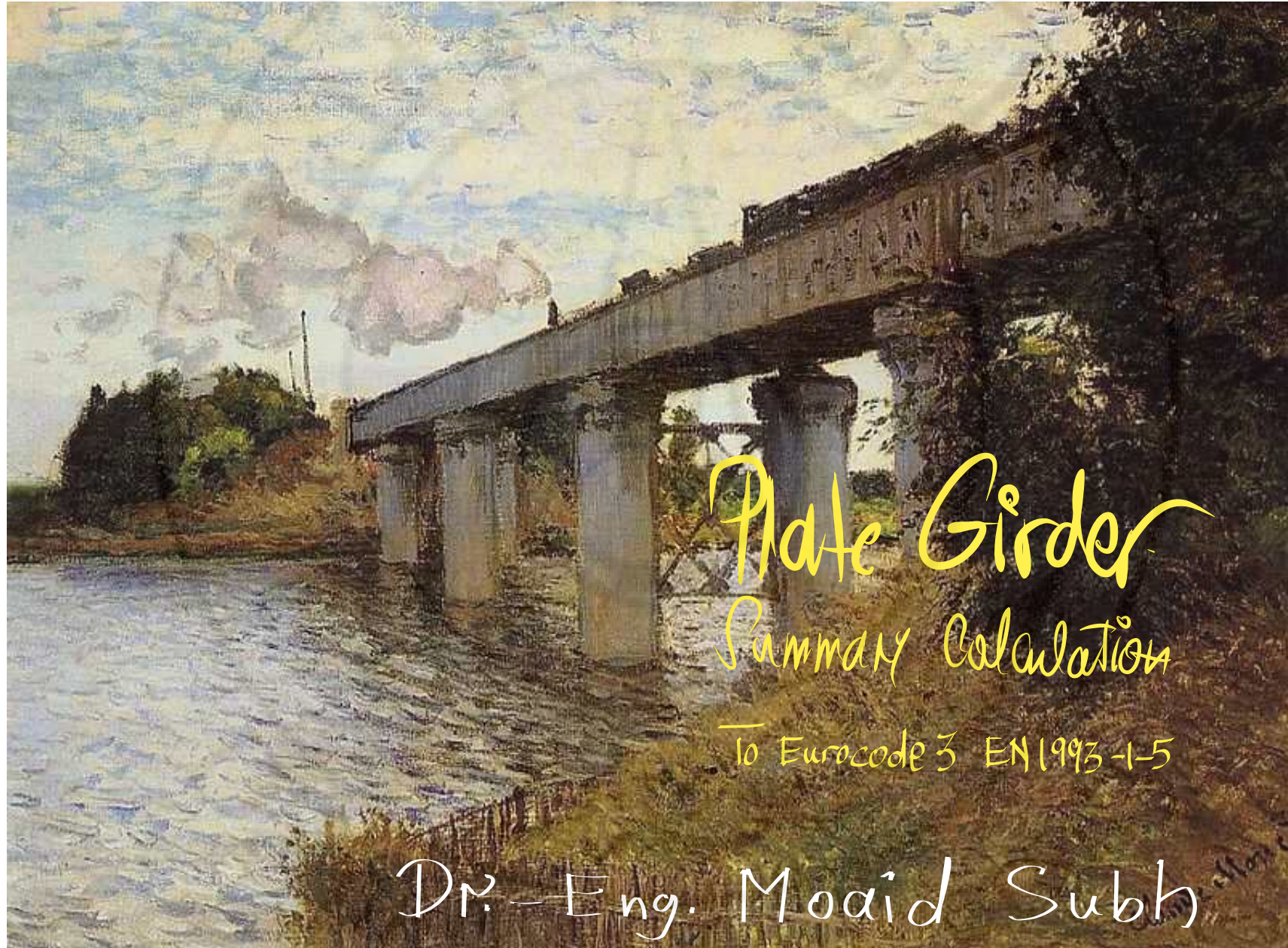
where I_{gr} is the second mo

σ_{gr} is the maximum

$I_{eff}(\sigma_{com,Ed,ser})$ is the se
buckling
span leng

(3) The effective second mo severe locations. Alternatively moment under serviceability lo

(4) The calculations require single calculation at a stress lev



The Railway Bridge at Argentueil, Painting Art by Claude Monet

preface

This summary calculation is a hand manuscript to generalized the subject plate girder on ec3 in a short amount. Understanding this summary is closely related to the detailed lectures. The pdf files of these lectures are included the following links :

1 - Plate Girder - 4th Year -2021- to EC3 - part 1.pdf

https://drive.google.com/file/d/1TBxShMWVxrX8J6HGnz_sVJEbA7yMUESa/view?usp=sharing

2- Plate Girder - 4th Year -2021- to EC3 - part 2.pdf

https://drive.google.com/file/d/1C9R_pe_kvz6j1-7SId3TptFkddyAGmA/view?usp=sharing

3- Plate Girder - 4th Year - 2021 - to EC3 - part 3.pdf

<https://drive.google.com/file/d/1NgCs0JcH1-90LfSs6creZU0-Zt6vArVK/view?usp=sharing>

4- EC3- en.1993.1.5.2006.pdf

<https://drive.google.com/file/d/1ooV3UIGj01rndULIWzSJoddKqj9VfuGF/view?usp=sharing>

The Contents

- 1- Chapter I : The Proportions
- 2- Chapter II : Bending & shear calculations
- 3- Chapter III : The Stiffeners

The References

- [1] - Euro Code 3 - Design of steel structures - Part 1-5: plated structural elements
- [2] B. Johansson, R. Maquoi, G. Sedlacek, C. Müller, D. Beg
Commentary and worked Examples to EN1993- 1-5- plated structural Elements, ECCS, 2007
- [3] Darko Beg, Ulrike Kuhlmann, Laurence Davaine, Benjamin Braun, Design of Plated Structures, ECCS, CECH, EKS, Ernst & Sohn; 2010
- [4] Modid Subh, Plate Girder to Euro Code 3, EN1993-1-5 (2000) Part 1, 2, 3
Lectures Manuscript - 2021.

Chapter I

The Proportions

① Plate Girder Proportions

Approximate Common Practice

- $h: \left| \frac{\text{Span}}{12} \leq h \leq \frac{\text{Span}}{8} \right|$
- $b: \left| \frac{h}{5} \leq b \leq \frac{h}{3} \right|$
- t_f : minimum satisfy the requirements of EC3 (Table 5-B.1) for class 3

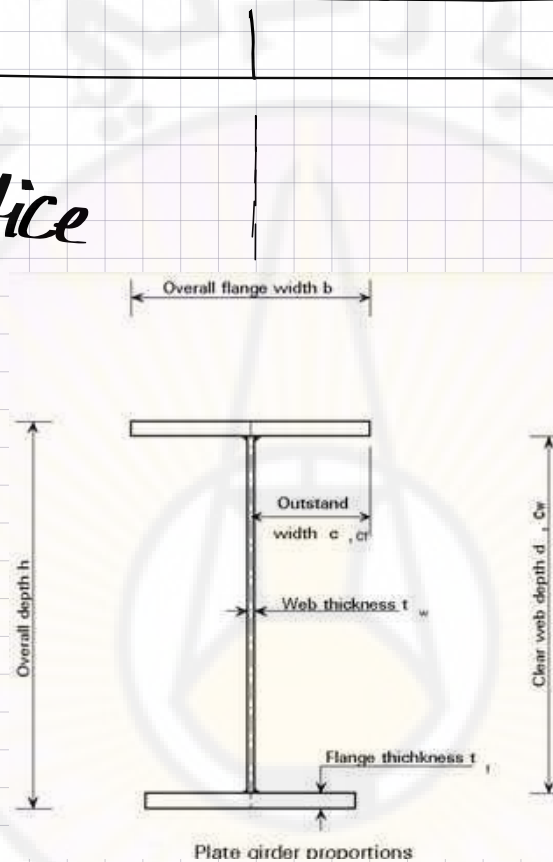
$$\left| c/t_f \leq 14 \epsilon \right|$$

- t_w : As thin as possible, maximum reach to class 4.

$$\left| C_w/t_w > 124 \epsilon \right|$$

- In case of bridge construction even:

$$\left| 200 \leq h/t_w < 500 \right|$$



Practice To EC3-BSEN 1993-1-5

Accommodation the Services is the criteria for "Recommended span-to-depth ratio".

Recommended span-to-depth ratios for plate girders used in buildings:

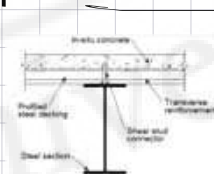
Applications	Span-to-depth ratio
(1) Constant-depth beams used in simply-supported composite girders, and for simply-supported non-composite girders with concrete decking	12 to 20
(2) Constant-depth beams used in continuous non-composite girders using concrete decking (N.B. continuous composite girders are uncommon in buildings)	15 to 20
(3) Simply-supported crane girders (non-composite construction is usual)	10 to 15

• b_f :

Non-Composite girders:

$$\left| 0,3 h_w < b_f < 0,5 h_w \right|$$

Simply-supported composite girder



Compression flanges

$$\left| 0,3 h_w < b_{fc} < 0,5 h_w \right|$$

Tension flanges

$$b_{ft} = b_{fc} * 1,30$$

① Plate Girder Proportions

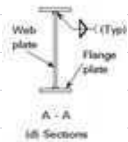
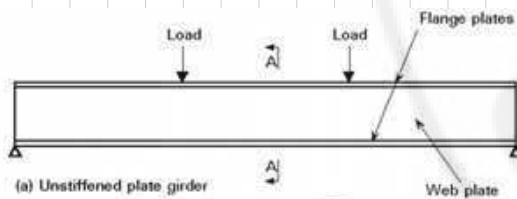
Approximate Common Practice

Practice To EC3-BS EN 1993-1-5

Stiffeners

1 of 3-decisions to make

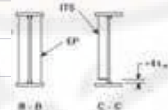
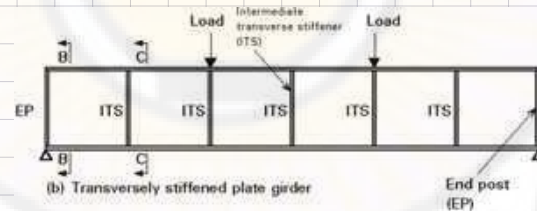
Unstiffened



In Case of:

- medium span ($12 < L < 20$ m)
- Light Weight (As final Roof Girder)
- or Small span in Building's floor

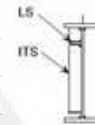
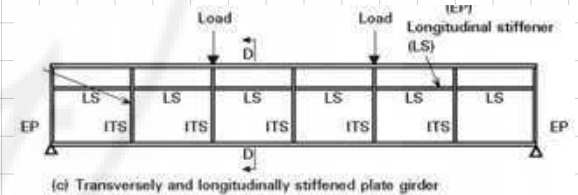
Transversed Stiffened



In case of:

- Long span ($20 < L < 150$ m)
- static load
- Low dynamic loads
- use in Buildings, Hall, Bridge
- Start with $a = h_w$, then change

Transversed and longitudinal Stiffened



- In case of:
- mega span ($L > 100$ m)
- Heavy static or Heavy dynamic load
- used in Bridges

① Plate Girder Proportions

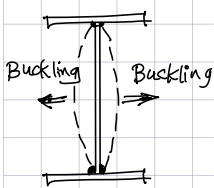
Approximate Common Practice

Practice To EC3-BS EN 1993-1-5

Service and Buckling Performance Ability

Web Breathing

Contribute to fatigue failure



Web breathing refers to recurring out of plane deflection of the web (due to buckling) under variable loads.

It is principally associated with bridges due to the

repeated nature of the loading. Web breathing may be neglected provided slenderness criteria set out in clause 7.4 of BS 1993-2 are met.

Eurocode 3 - Design of steel structures - Part 2: Steel Bridges

Figure 3.1 - Detail of a stiffener at a web-to-flange connection

Figure 3.2 - Forming a stiffener with stiffeners

7.4 Limitation of web breathing

(1) The slenderness of web plates should be limited to avoid excessive breathing that might result in fatigue at or adjacent to the web-to-flange connections.

NOTE: The National Annex may define cases where web breathing checks are not necessary.

(2) Web breathing may be neglected for web panels without longitudinal stiffeners or for subpanels of stiffened webs, where the following criteria are met:

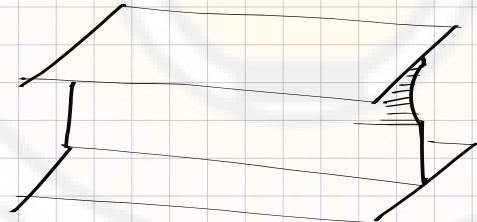
$h/t_w \leq 30 + 40 L$ ≤ 300 for road bridges (7.5)

$h/t_w \leq 55 + 3.3 L$ ≤ 250 for railway bridges (7.6)

where L is the span length in m, but not less than 20 m.

Unstiffly Web buckling

- 1- during erection
- 2- during service



Minimum Web thickness Value to avoid service ability problems are not prescribed in BS 1993-1-5, but taken from BS 5400-1.

$t_w \geq h_w/250$ for unstiffened webs

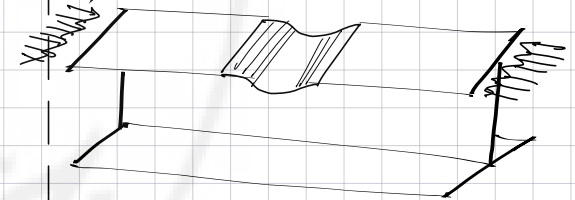
$t_w \geq h_w/250$

$t_w \geq \left(\frac{h_w}{250}\right) \left(\frac{a}{h_w}\right)^{1/2}$

$a > h_w$
 $a \leq h_w$

stiffened webs

Compression flange Buckling into Web



Clause 8.6(1) of BS EN 1993-1-5 is prescribe the minimum required web thickness.

$$t_w \geq \left(\frac{h_w}{k}\right) \left(\frac{f_{yf}}{E}\right) \sqrt{\frac{A_{fc}}{A_w}}$$

Chapter II

Bending & shear calculations

II Bending

Exact Method (EM)

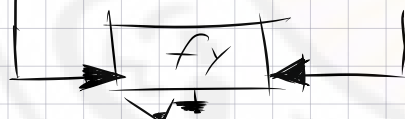
BS EN 1993-1-1, clause 6.2.5

Bending affected by shear
(M) → function of (Q)

↓
 $V_{Ed} < 0.5 \cdot V_{pl,Rd}$

No shear will affect the Moment Resistance

$$f_y = f_y$$



Class 1 & 2
 $M_{y,Rd} = W_{pl} \cdot f_y / \gamma_{M0}$

Class 3
 $M_{y,Rd} = W_{eff,min} \cdot f_y / \gamma_{M0}$

Class 4
 $M_{y,Rd} = W_{eff,min} \cdot f_y / \gamma_{M0}$

↓
 $V_{Ed} \geq 0.5 \cdot V_{pl,Rd}$

Reduction shall be applied at Moment Resistance

$$f_{yk} = (1 - \rho) f_y$$

$$\rho = \left[\frac{2V_{Ed}}{V_{pl,Rd}} - 1 \right]^2$$

Approximate Method (AM)

Flang only Method

No code supported

$$M_{f,Rd} = A_f (h_w + t_f) f_y$$

Condition of use :

1. Flanges are class 1 or 2
2. $A_f (h_w + t_f) \leq W_{el}$

(II) Shear

Web is not susceptible to shear buckling

Unstiffened Web		Stiffened
$\frac{hw}{tw} > 72 \frac{\epsilon}{\rho}$		$\frac{hw}{tw} > 31 \frac{\epsilon}{\rho} \sqrt{K_\sigma}$

Web is Stocky

Use:

BS EN 1993-1-1

$$V_{pl,Rd} = \frac{A_v (f_y / \sqrt{3})}{\gamma_{M0}}$$

Web is susceptible to shear buckling

Unstiffened Web		Stiffened
$\frac{hw}{tw} < 72 \frac{\epsilon}{\rho}$		$\frac{hw}{tw} < 31 \frac{\epsilon}{\rho} \sqrt{K_\sigma}$

Web is not Stocky

Use:

BS EN 1993-1-5

$$V_{b,Rd} = V_{bw,R} + V_{bf,Rd}$$

$$V_{b,Rd} \leq \frac{\eta f_{yw} \cdot h_w t_w}{\sqrt{3} \cdot \gamma_{M1}}$$

II Bending and Shear

(B1)

No Tension Field

Web is not susceptible to shear
unstiff $\rightarrow h/t_w \leq 72 \frac{E}{f_y}$, stiff $\rightarrow h/t_w \leq 31 \frac{E}{f_y} \sqrt{k_\sigma}$

BS EN 1993-1-1, clause 6.2.8

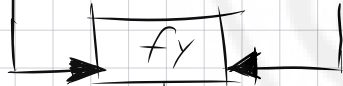
Bending affected by shear
(M) $\rightarrow$ function of (Q)

(B1-I)

$$V_{Ed} < 0.5 V_{pl,Rd}$$

No shear will affect the Moment Resistance

$$f_y = f_y$$



class 1, 2
 $M_{y,Rd} = W_{pl} \cdot f_y / \gamma_{M0}$

class 3
 $M_{y,Rd} = W_{el,min} \cdot f_y / \gamma_{M0}$

class 4
 $M_{y,Rd} = W_{eff,min} \cdot f_y / \gamma_{M0}$

(B1-II)

$$V_{Ed} \geq 0.5 V_{pl,Rd}$$

Reduction shall be applied at Moment Resistance

$$f_{yk} = (1 - \rho) f_y$$

$$\rho = \left[\frac{2 V_{Ed}}{V_{pl,Rd}} - 1 \right]^2$$

(B2)

Tension Field

Web is susceptible to shear

unstiff $\rightarrow h/t_w > 72 \frac{E}{f_y}$, stiff $h/t_w > 31 \frac{E}{f_y} \sqrt{k_\sigma}$

(B3)

No Tension Field

BS EN 1993-1-5, clause 5

Flanges are not fully utilised to resisting bending

$$M_{Ed} < M_{f,Rd}$$

$$\bar{V}_3 = \frac{V_{Ed}}{V_{w,Rd}} \leq 0.5$$

$$\bar{V}_3 = \frac{V_{Ed}}{V_{w,Rd}} > 0.5$$

$$V_{b,Rd} = V_{w,Rd} + V_{bf,Rd}$$

BS EN 1993-1-5, clause 7.1

Flanges are fully utilised in resisting bending

$$M_{Ed} \geq M_{f,Rd}$$

$$M_{Ed} = M_{f,Rd}$$

$$M_{Ed} > M_{f,Rd}$$

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

Moment is not contribute on to shear resistance

(II)

Bending (M) Shear (V) Interaction

(B1) No Tension Field

Web is not susceptible to shear
 unstiff $\rightarrow h_w/t_w \leq 72 \frac{E}{f_y}$, stiff $\rightarrow h_w/t_w \leq 31 \frac{E}{f_y} \sqrt{k_\sigma}$

BS EN 1993-1-1, clause 6.2.8

Bending affected by shear
 (M) $\rightarrow$ function of (V)

(B1-I)

$$V_{Ed} < 0.5 V_{pl,Rd}$$

Interaction MBQ not Required

No shear will affect the Moment Resistance

$$f_y = f_y$$

$$\text{Class 1, 2} \\ M_{y,Rd} = W_{pl,y} \cdot f_y / \gamma_{M0}$$

$$\text{Class 3} \\ M_{y,Rd} = W_{el,y,min} \cdot f_y / \gamma_{M0}$$

$$\text{Class 4} \\ M_{y,Rd} = W_{eff,y,min} \cdot f_y / \gamma_{M0}$$

(B1-II)

$$V_{Ed} \geq 0.5 V_{pl,Rd}$$

Interaction, see figure (B1)

Reduction shall be applied to Moment Resistance

$$f_{yR} = (1 - \rho) f_y \\ \rho = \left[\frac{2 V_{Ed}}{V_{pl,Rd}} - 1 \right]^2$$

The interaction is representing by ρ

(B2) Tension Field

BS EN 1993-1-5, clause 5

Flanges are not fully utilised to resisting bending

$$M_{Ed} < M_{f,Rd}$$

$$\bar{\eta}_3 = \frac{V_{Ed}}{V_{w,Rd}} \leq 0.5$$

$$\bar{\eta}_3 = \frac{V_{Ed}}{V_{w,Rd}} > 0.5$$

Interaction is representing by introducing $V_{bf,Rd}$

$$V_{b,Rd} = V_{w,Rd} + V_{bf,Rd}$$

Web is susceptible to shear

unstiff $\rightarrow h_w/t_w > 72 \frac{E}{f_y}$, stiff $\rightarrow h_w/t_w > 31 \frac{E}{f_y} \sqrt{k_\sigma}$

(B3)

BS EN 1993-1-5, clause 7.1

No Tension Field

Flanges are fully utilised in resisting bending

$$M_{Ed} \geq M_{f,Rd}$$

$$M_{Ed} = M_{f,Rd}$$

$$M_{Ed} > M_{f,Rd}$$

$$M_{Ed} < M_{y,Rd}$$

Moment is not Contribution to shear resistance

$$V_{b,Rd} = V_{w,Rd}$$

$$V_{b,Rd} = V_{w,Rd}$$

$$V_{b,Rd} < 0.5 V_{w,Rd}$$

No Interaction

$$V_{b,Rd} \geq 0.5 V_{w,Rd}$$

Interaction (MBV)

Formula (7.1)

$$\bar{\eta}_1 + \left[1 - \frac{M_{f,Rd}}{M_{pl,Rd}} \right] (\bar{\eta}_3 - 1) < 1.0$$

for $\bar{\eta}_1 \geq \frac{M_{Ed}}{M_{pl,Rd}}$

$$\bar{\eta}_1 = \frac{M_{Ed}}{M_{pl,Rd}}, \bar{\eta}_3 = \frac{V_{Ed}}{V_{w,Rd}}$$

II Bending (M) Shear (V) Interaction

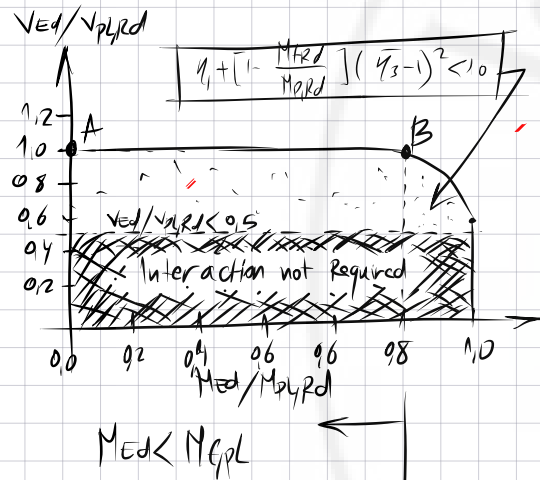


Plate Not Susceptible to buckling

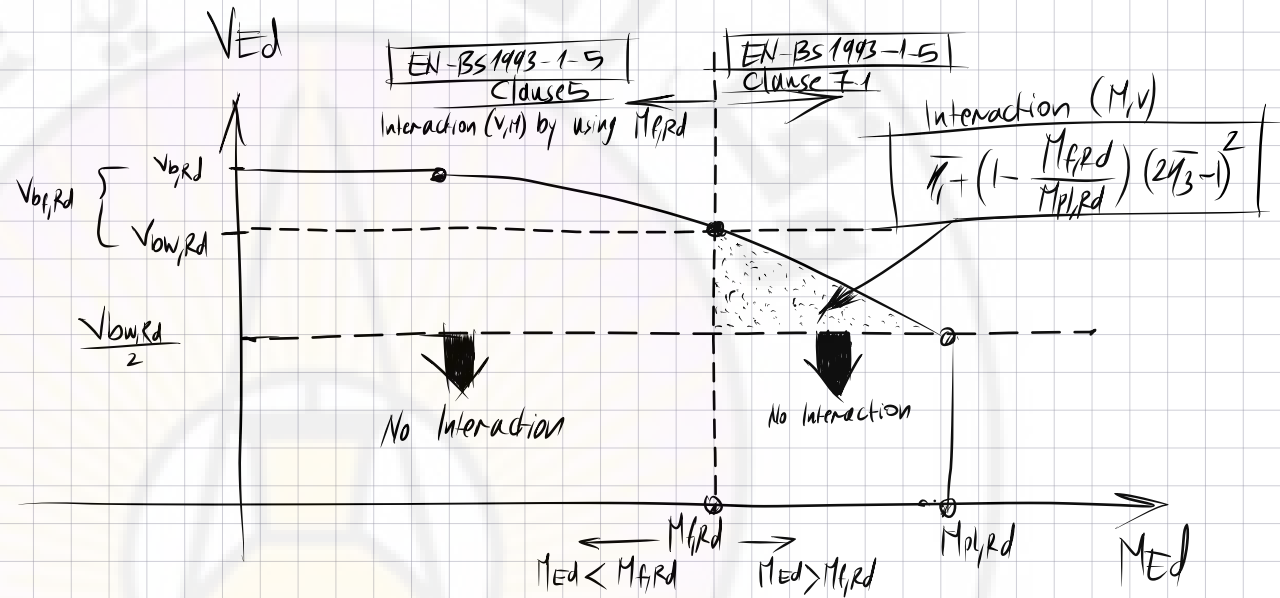


Plate Susceptible to buckling

Chapter III

The Stiffeners

III Introduction

III-1. General

Stiffeners are supplied to form fields required to emerge tension field

Stiffeners expose to Post-buckling membrane stresses.

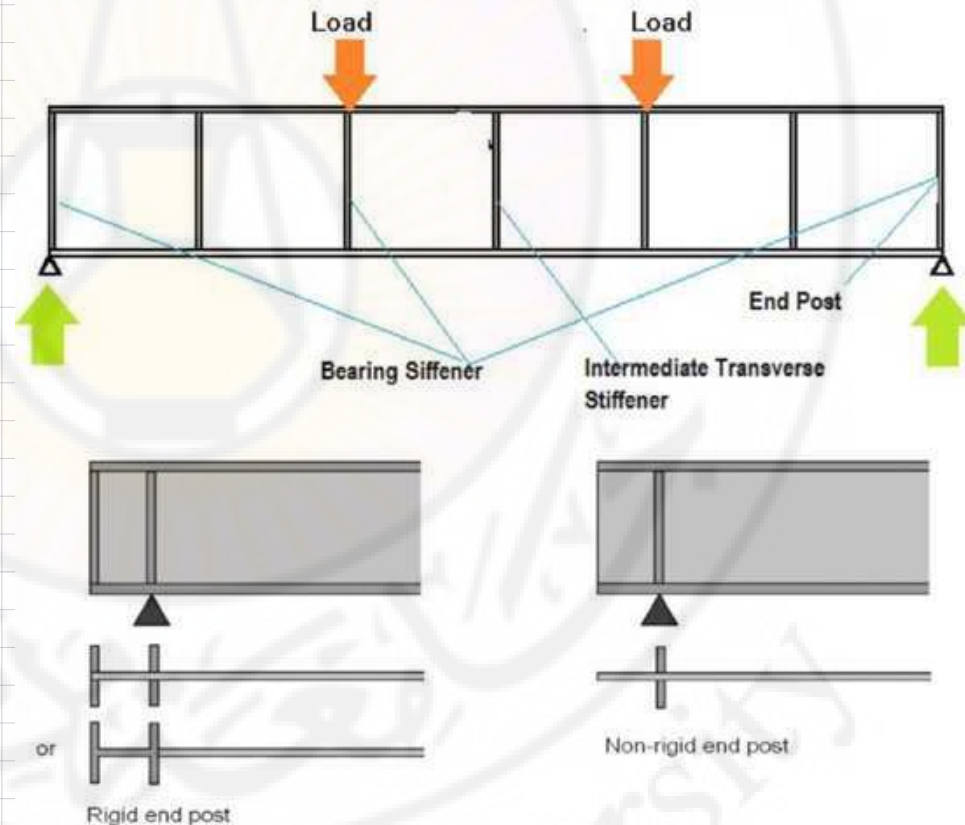
According to their locations and functions stiffeners are classified to :

1. End Post Stiffeners (EP)

2. Intermediate Transverse Stiffeners (ITS)

3. Bearing Stiffeners (BS)

Web Stiff



III-2 | General Requirements Of All Stiffeners |

GRS

Two general performance requirements for all stiffeners:

↓
Requirement (A)

2th Order Analysis

Verify using second order Analysis

↓
Requirement (B)

Torsional buckling

The stiffeners will not fail by torsional buckling

III-2 | General Requirements Of All Stiffeners |

GRS

Two general performance requirements for all stiffeners:

Requirement (A)

stress limits

$$\sigma_{max} \leq \frac{f_y}{\gamma_{M1}}$$

deflection limits

$$w \leq \frac{b}{300}$$

Requirement (B)

Three Method may be used to Verify:

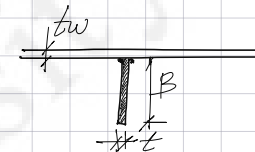
1) $\frac{I_T}{I_p} \geq 5.3 \frac{f_y}{E}$

2) or: when warping stiffness is considered
 $\sigma_{cr} \geq 2f_y$

3) In case of flat plates stiffeners, if B/t satisfy the table value, Torsional Buckling Verifying is not required

Steel grade	235	275	355	420	460
$B/t \leq$	13.0	12.0	10.6	9.7	9.3

Limiting B/t values to prevent torsional buckling



III-2 | General Requirements Of All Stiffeners |

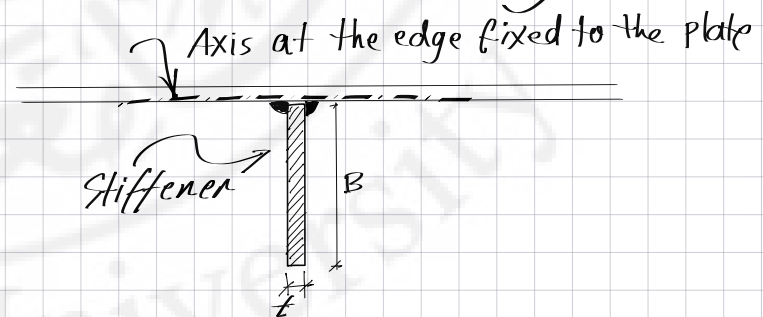
GRS

Requirement (A)

σ_{max} : Ultimate stress (elastic)
 w : ultimate lateral deflection
 b : Panel height (or Web height)

Requirement (B)

I_t : St. Venant torsional Constant of the stiffener alone
 I_p : Polar second moment of area of the stiffener alone around the edge of the plate
 σ_{cr} : Elastic critical stress for torsional buckling



III-3 | End Post Stiffeners | EP

III-3-1. Introduction

End Post are classified in BS EN 1993-1-5 as either:

① Rigid or ② Non-Rigid

End Post provide anchorage for buckled field,
So no difference between (① and ②) if field's $\bar{\lambda}_w < 1,08$

Table 5.1: Contribution from the web χ_w to shear buckling resistance

	Rigid end post	Non-rigid end post
$\bar{\lambda}_w < 0,83/\eta$	η	η
$0,83/\eta \leq \bar{\lambda}_w < 1,08$	$0,83/\bar{\lambda}_w$	$0,83/\bar{\lambda}_w$
$\bar{\lambda}_w \geq 1,08$	$1,37/(0,7 + \bar{\lambda}_w)$	$0,83/\bar{\lambda}_w$

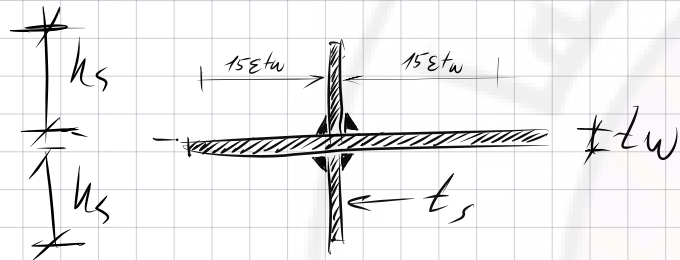
End Posts act as bearing stiffeners resisting from supports.

Rigid end Post should resist also horizontal component of the tension field together with compressive forces

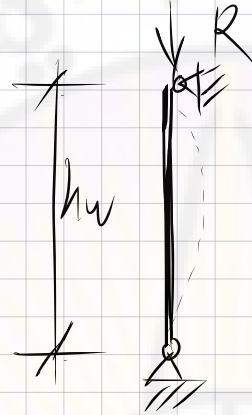
due to the support reactions.

∞. Non-rigid end Post need only be designed as bearing stiffeners.

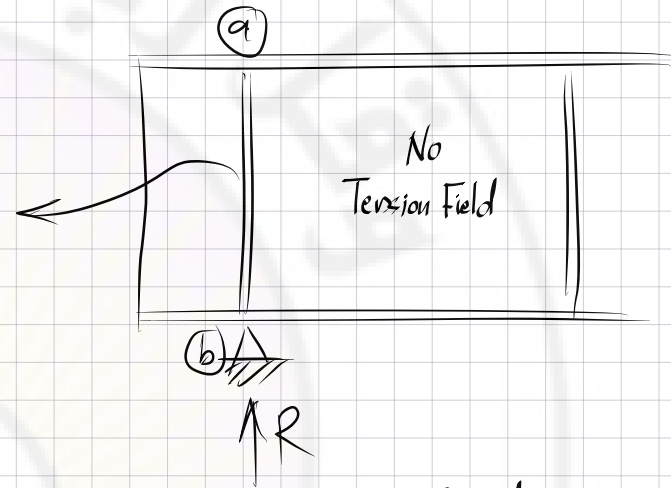
Buckling $\rightarrow (L_b = 0.7h_w, \text{Curve} = c)$



End Post section



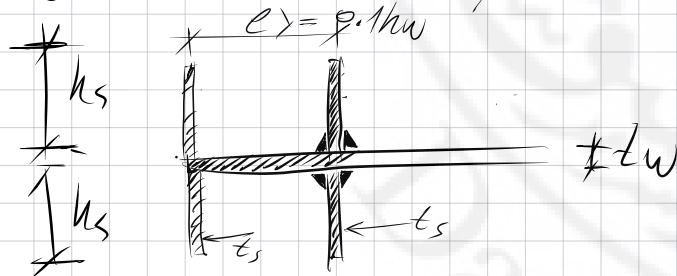
Column



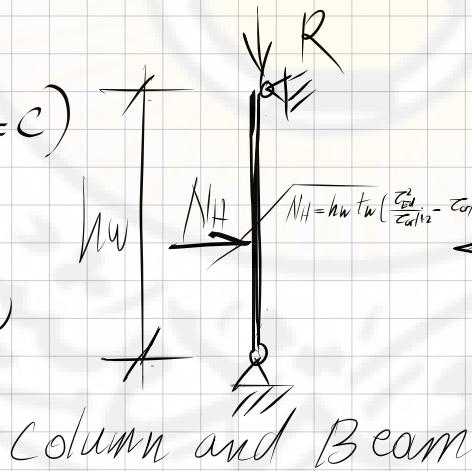
Non-Rigid

Beam (Moment, $M = N_H \cdot h_w / 8$)

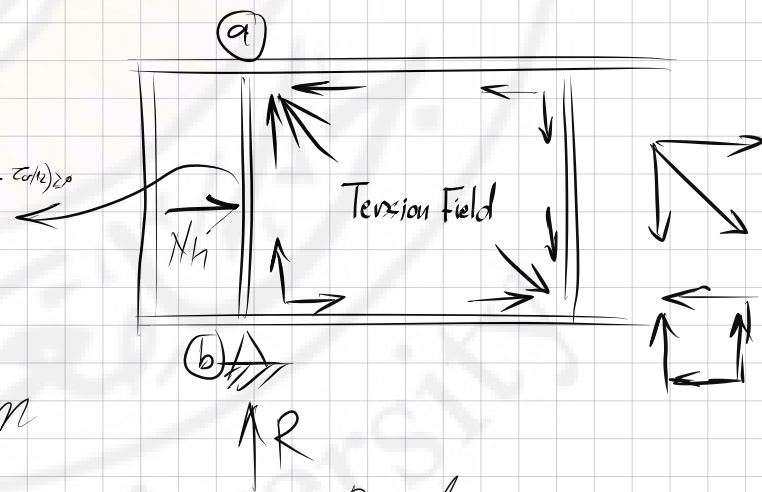
Buckling $\rightarrow (L_b = 0.7h_w, \text{Curve} = c)$



End Post section



Column and Beam

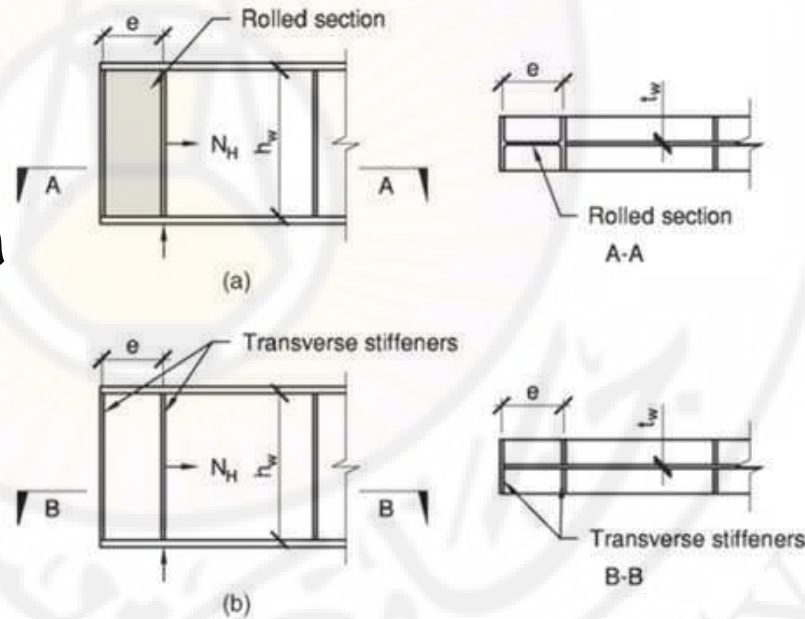


Rigid

III-3-2 | Rigid End Post |

❧ Rigid end Post as defined in Clause 9.3.1 (2) of BS EN1993-1-5, should comprise two double-sided transverse stiffeners or a rolled section

❧ In both cases, the end Post act as Beam spanning vertically between the flanges and resisting the longitudinal membrane forces arising from tension field action



Rigid end post: (a) formed by a rolled section, and (b) formed by a pair of double-sided stiffeners

- The minimum section modulus W_{min} for this Beam

$$W_{min} = 4 h w t_w^2$$

- In case of pair of double-sided flat stiffeners separated by a distance (e) according to BS EN 1993-1-5, clause 9.3.1 (3), should be

$$e = 0.1 h w$$

With minimum Area of

$$A_{min} = 4 h w t_w^2 / e$$

- The magnitude of longitudinal membrane force N_H is not specified in the code but in many references as: $N_H = h w t_w \left(\frac{\tau_{Ed}}{\tau_{cr/1.2}} - \tau_{cr/1.2} \right) \geq 0$

$\tau_{Ed} = V_{Ed} / h w t_w$, τ_{cr} : critical shear buckling

- The force N_H assumed to act as uniformly distributed load as $(N_H = N_H / h w)$, its effected moment

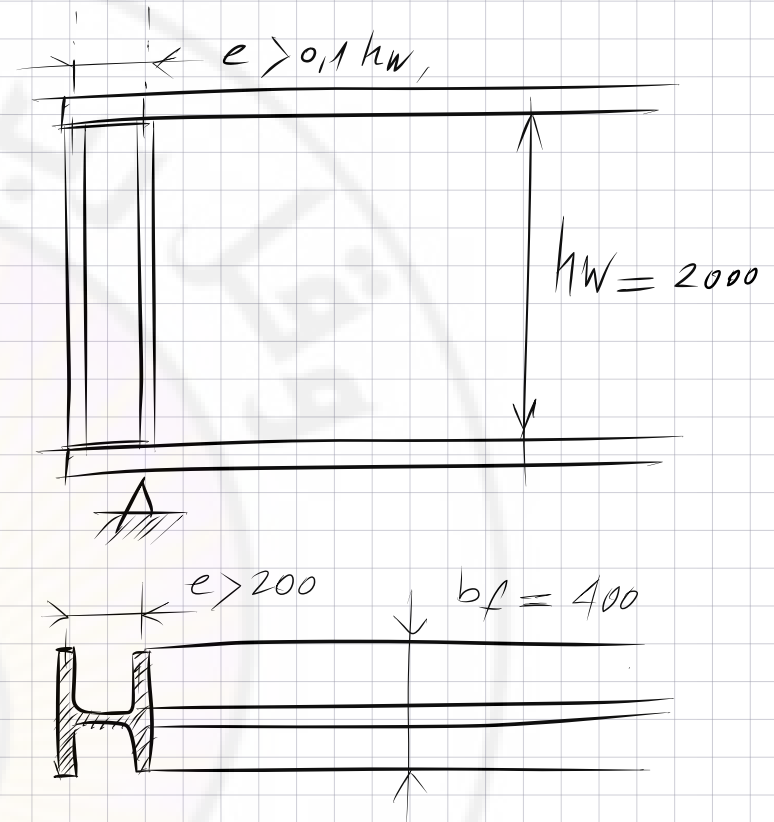
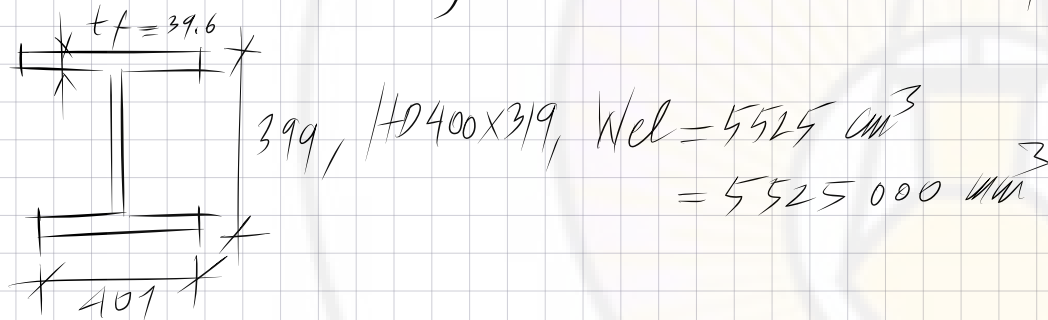
$$M = N_H \cdot h w / 8$$

Training: (T1)

1.) check hot rolled requirement to be rigid end post - case (REP-R)

Select hot rolled with $b_f \approx 400$, $D > 200$

choose Wide Flange Column HD 400x314



check for e ; $e = 399 - 39.6 = 359.7 > 0.1 h_W = 200 \text{ mm}$

check for stiffness as per [clause 9.3.1 (3) BS EN 1993-1-5]

$$W_{min} = 4 h_W t_w^2 = 4 (2000) (8)^2 = 512,000 \text{ mm}^2$$

$$\left\{ W_{el} = 5525 \times 10^6 > W_{min} = 0.512 \times 10^6 \right\} \quad \text{OK}$$

Training: (T2)

1.) check Stiffness requirement to be rigid end post - case (REPS)

$$\begin{aligned} A_{end} &= ① + ② + ③ \\ &= 160 \times 15 + 160 \times 15 + 8 \times 15 \\ &= 4920 \text{ mm}^2 \end{aligned}$$

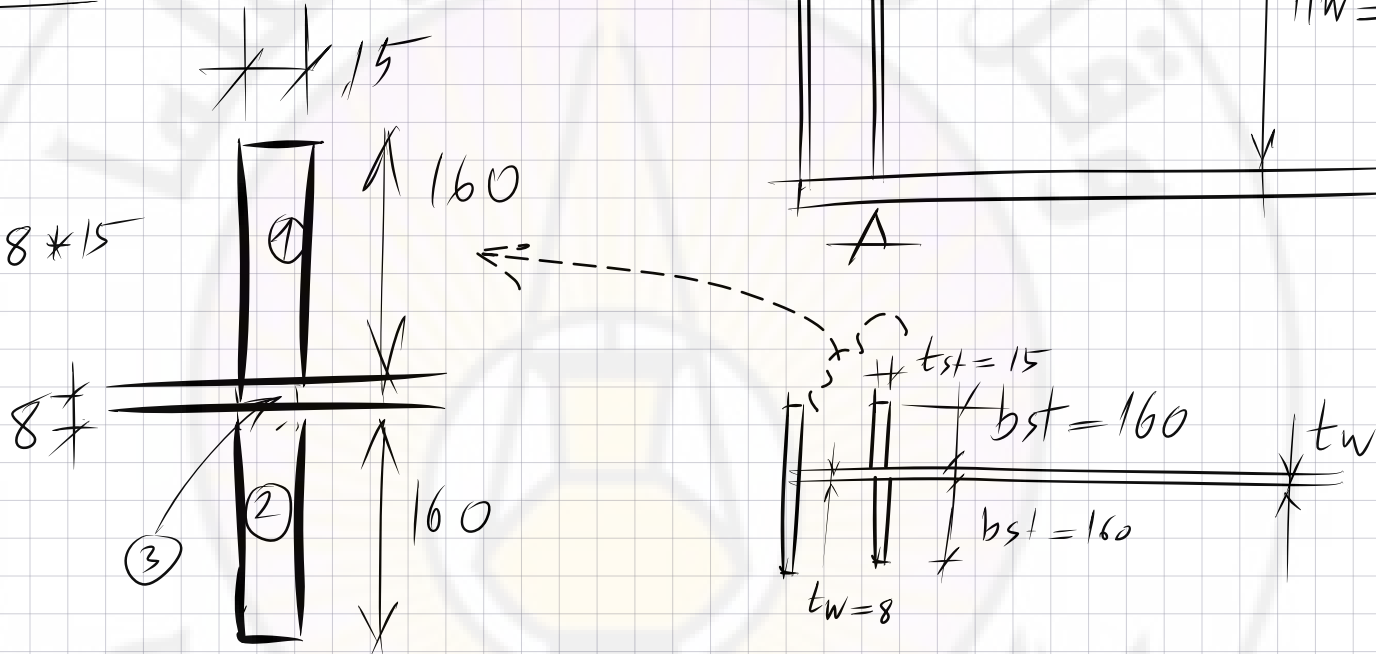
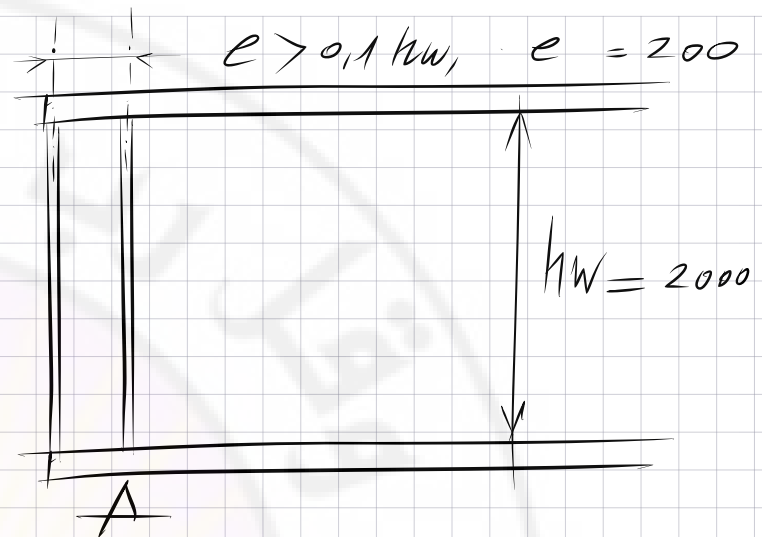
A_{end} must be greater than

$$A_{min} = 4 h_w t_w^2 / e$$

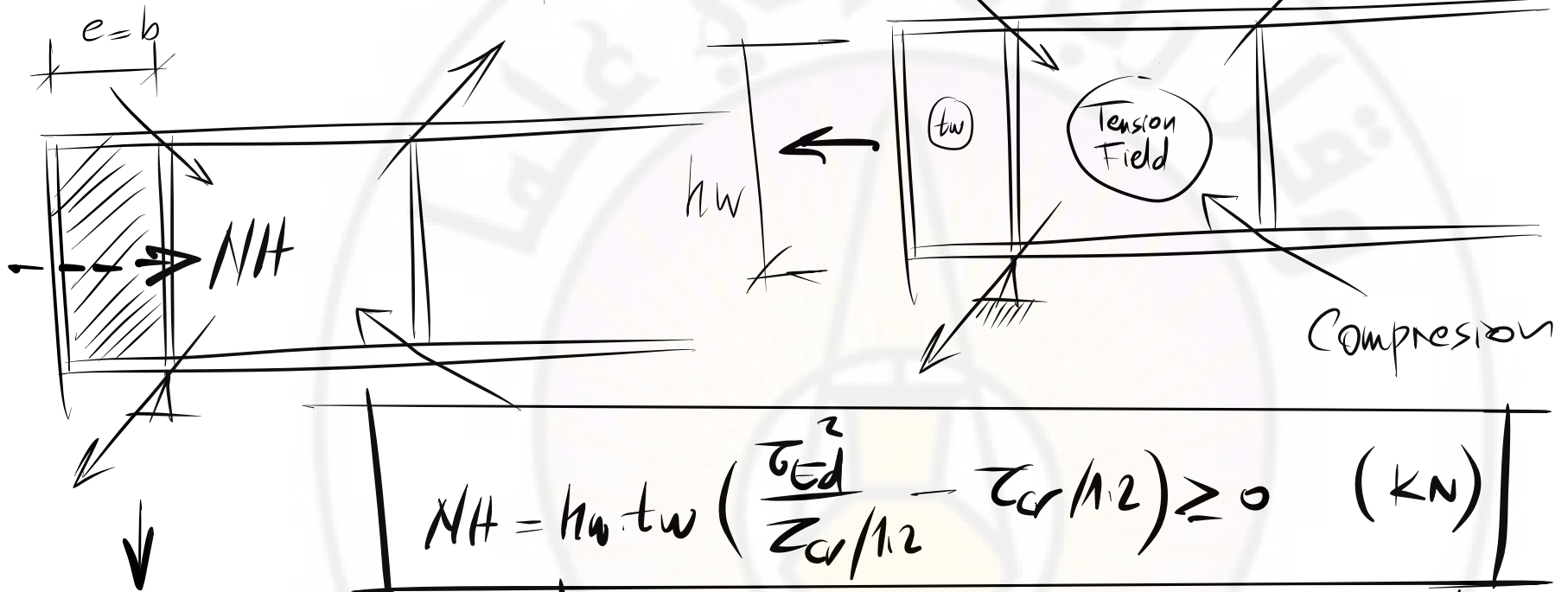
[clause 9.3.1 (3) BS EN 1993-1-5]

$$A_{min} = 4 h_w t_w^2 / e = 4 \times 2000 (8)^2 / 200 = 2560 \text{ mm}^2$$

$$\{ A_{end} = 4920 > A_{min} = 2560 \} \text{ OK}$$

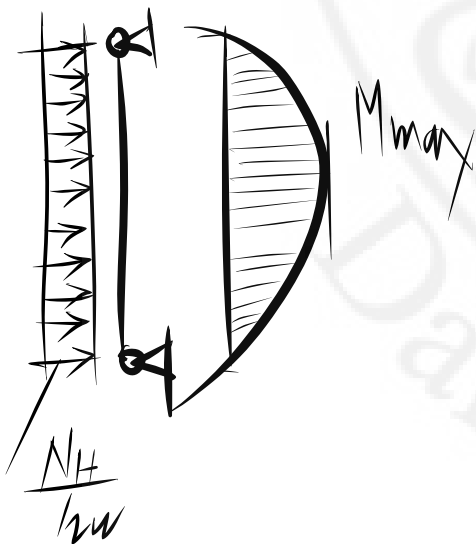


Anchorage Beam



$$NH = h_w \cdot t_w \left(\frac{\tau_{Ed}^2}{\tau_{cr/1.2}} - \tau_{cr/1.2} \right) \geq 0 \quad (\text{KN})$$

$$M_{max} = NH \cdot h_w / 8 \quad (\text{KN}\cdot\text{m})$$



$$\tau_E = \frac{V_{Ed}}{h_w t_w}, \quad \tau_{cr} = k_z \cdot G_E, \quad G_E = 190000 \left(\frac{t}{b} \right)^2 \text{ N/mm}^2$$

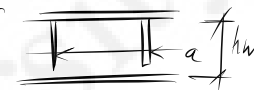
Stiffened

$$k_z = 5.34 + 4.00 (h_w/a)^2 \rightarrow a/h_w > 1$$

Unstiffened

$$k_z = 4.00 + 5.34 (a/h_w)^2 \rightarrow a/h_w < 1$$

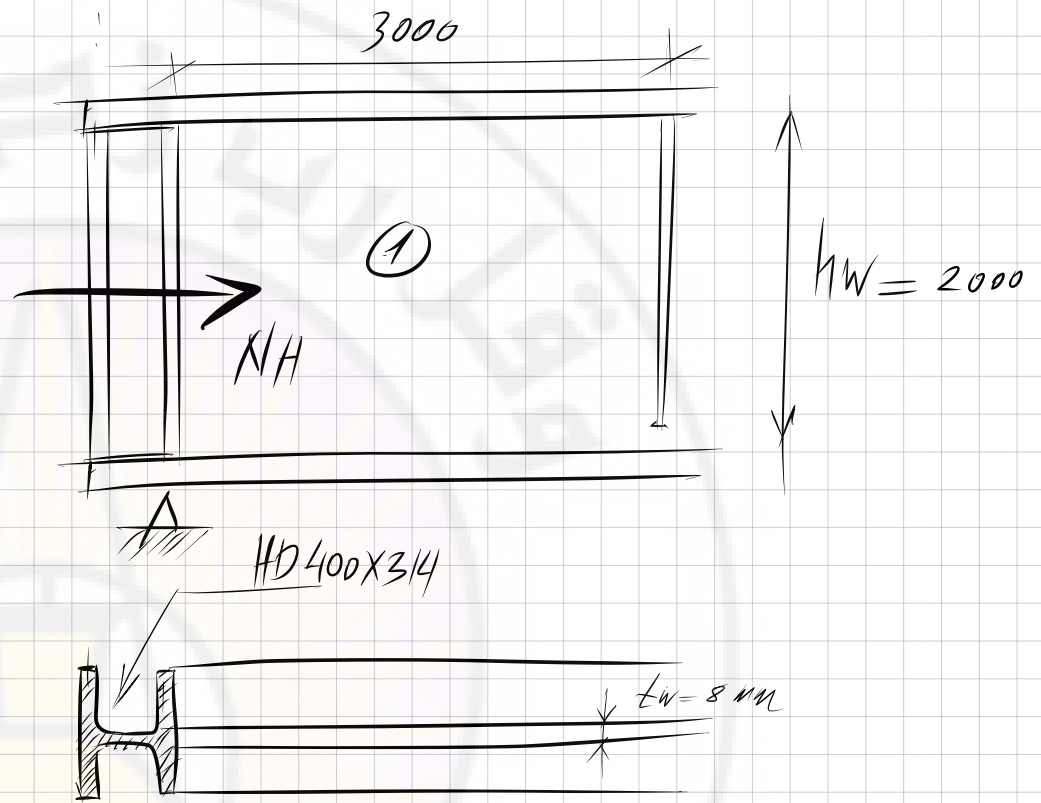
$$k_z = 5.34$$



Training: (T3)

check the rigid end Post beam
against membrane force arising
from tension field action on ①

$$V_{Ed} = 647 \times 10^3 \text{ N}, h_w = 2000 \text{ mm}, t_w = 8 \text{ mm} \\ b = a = 3000 \text{ mm}, S235$$



$$a/h_w = 3000/2000 = 1.5 > 1$$

$$K_\tau = 5.34 + 4.00 (h_w/a)^2 \\ = 5.34 + 4.00 (2000/3000)^2 = 7.1$$

$$\sigma_E = 190000 \left(\frac{t_w}{b} \right)^2 = 190000 \left(\frac{8}{3000} \right)^2 = 1.351 \text{ N/mm}^2$$

$$\tau_{cr} = K_\tau \cdot \sigma_E = 7.1 \times 1.351 = 9.64 \text{ N/mm}^2$$

$$\tau_{Ed} = V_{Ed} / h_w \cdot t_w = 647 \times 10^3 / (2000 \times 8) = 40.44 \text{ N/mm}^2$$

$$NH = h_w \cdot t_w \left(\frac{\tau_{Ed}^2}{\tau_{cr}/1.2} - \tau_{cr}/1.2 \right) = 2000 \times 8 \left[\frac{(40.44)^2}{9.64/1.2} - 9.64/1.2 \right] \\ = 313 \times 10^6 \text{ N}$$

Training: (T3)

$$\frac{NH}{hw} = \frac{31.3 \times 10^6}{2000}$$

$$= 15.65 \times 10^3 \text{ N/m}$$

$$M_{\max} = \frac{NH}{hw} \times \frac{hw^3}{8}$$

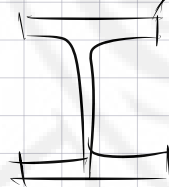
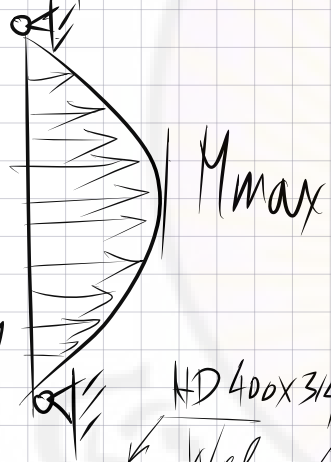
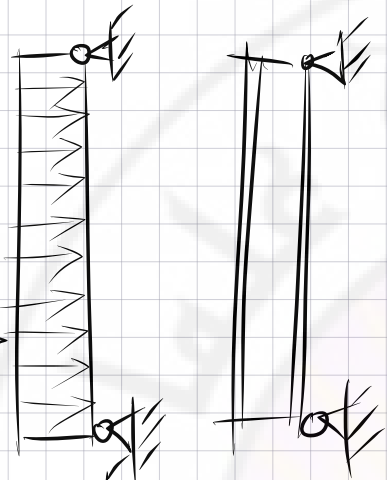
$$= \frac{15.65 \times 10^3 \times 2000^3}{8}$$

$$= 3912.5 \times 10^3 \text{ N}\cdot\text{mm}$$

$$f_b = \frac{M}{W_{el}}$$

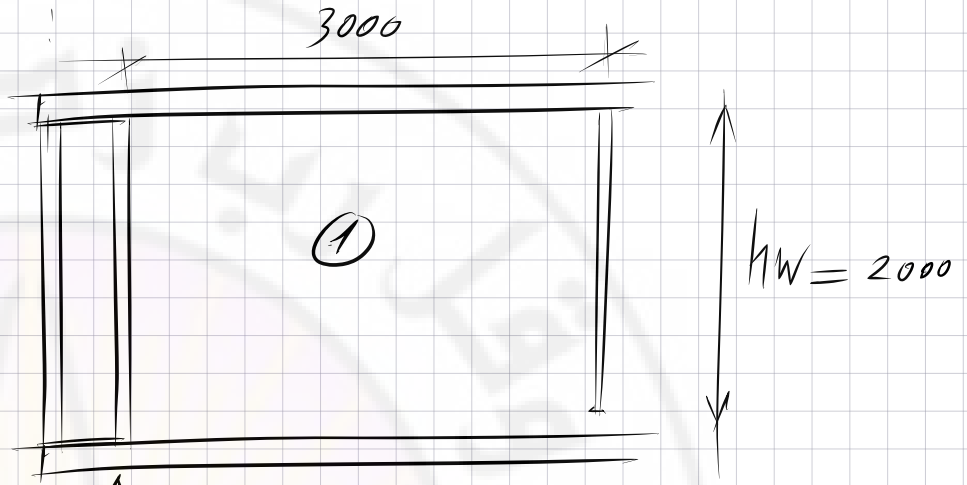
$$= \frac{3912.5 \times 10^3}{5525000}$$

$$= 0.71 \text{ N/mm}^2 < f_y = 235 \text{ N/mm}^2 \quad (S235)$$



HD 400x314

$W_{el} = 5525000 \text{ mm}^3$



HD 400x314

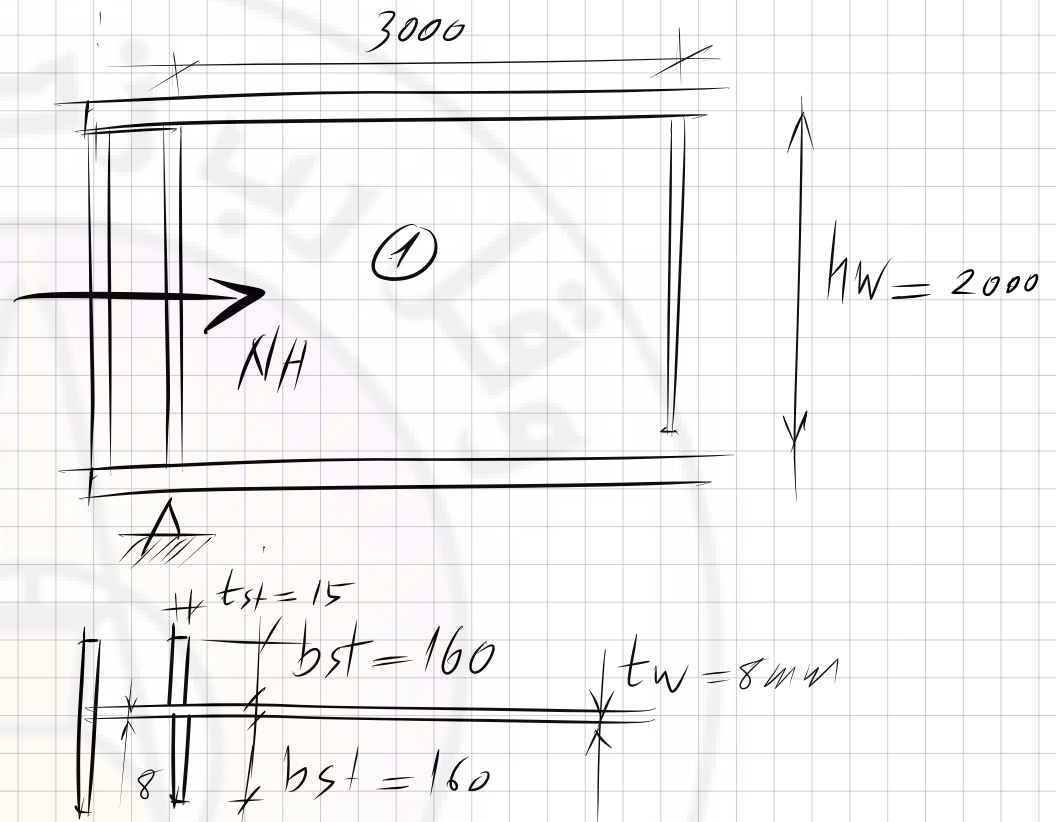


$t_w = 8 \text{ mm}$

Training: (T4)

check the rigid end Post Plates
against membrane force arising
from tension field action on ①

$$V_{Ed} = 647 \times 10^3 \text{ N}, h_w = 2000 \text{ mm}, t_w = 8 \text{ mm} \\ b = a = 3000 \text{ mm}, S235$$



$$a/h_w = 3000/2000 = 1.5 > 1$$

$$K_\sigma = 5.34 + 4.00 (h_w/a)^2 \\ = 5.34 + 4.00 (2000/3000)^2 = 7.1$$

$$\sigma_E = 190000 \left(\frac{t_w}{b}\right)^2 = 190000 \left(\frac{8}{3000}\right)^2 = 1.351 \text{ N/mm}^2$$

$$\tau_{cr} = K_\sigma \cdot \sigma_E = 7.1 \times 1.351 = 9.64 \text{ N/mm}^2$$

$$\tau_{Ed} = V_{Ed} / h_w \cdot t_w = 647 \times 10^3 / (2000 \times 8) = 40.44 \text{ N/mm}^2$$

$$N_H = h_w \cdot t_w \left(\frac{\tau_{Ed}^2}{\tau_{cr}^{1.2}} - \tau_{cr} / 1.2 \right) = 2000 \times 8 \left[\frac{(40.44)^2}{9.64 / 1.2} - 9.64 / 1.2 \right] \\ = 313 \times 10^6 \text{ N}$$

Training: (T4)

$$\frac{NH}{hw} = \frac{31,3 \times 10^6}{2000}$$

$$= 15,65 \times 10^3 \text{ N/m}$$

$$M_{\max} = \frac{NH}{hw} \times \frac{hw^3}{8}$$

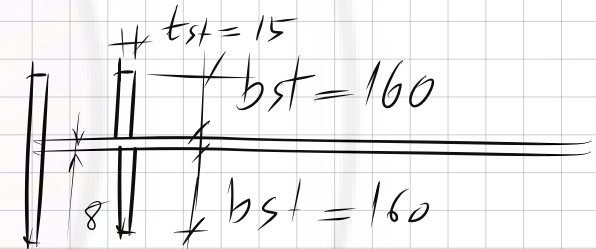
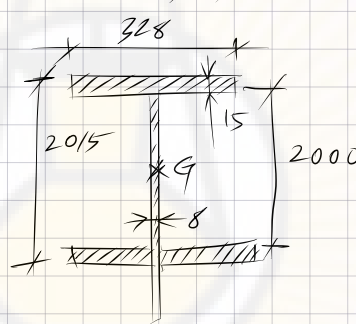
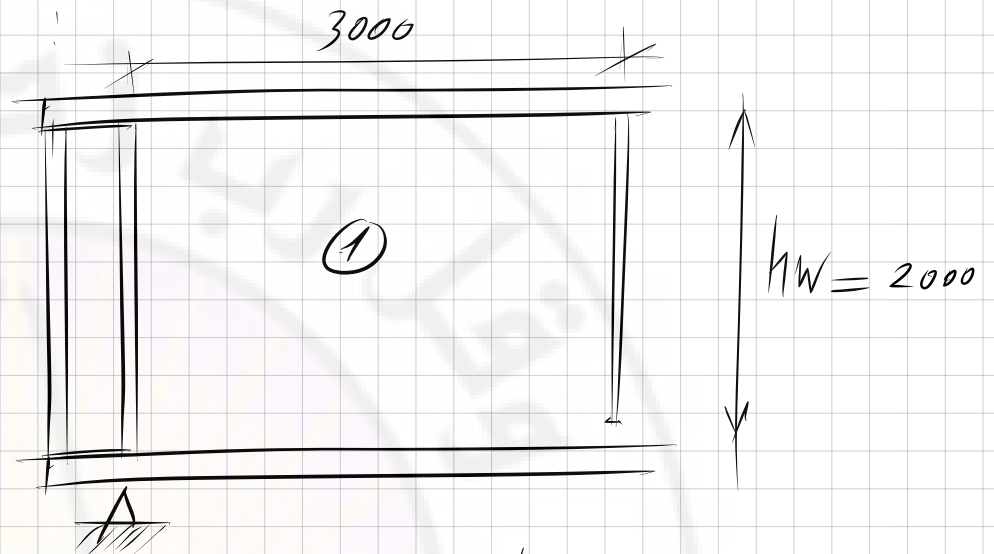
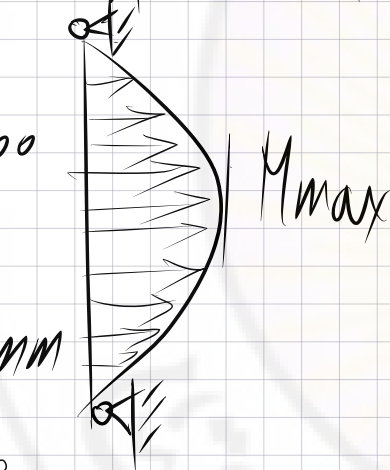
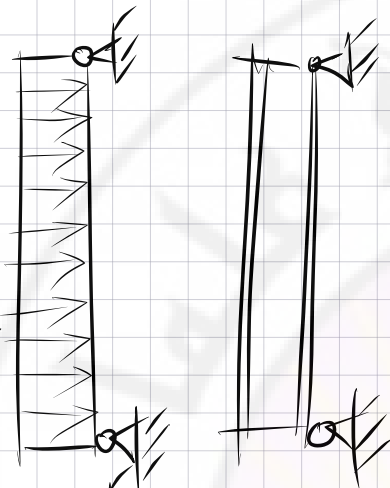
$$= \frac{15,65 \times 10^3 \times 2000^3}{8}$$

$$= 3912,5 \times 10^3 \text{ N}\cdot\text{mm}$$

$$f_b = \frac{M}{W_{el}}$$

$$= \frac{3912,5 \times 10^3}{9767 \times 10^3}$$

$$= 0,4 \text{ N/mm}^2 < f_y = 235 \text{ N/mm}^2 \quad (S235)$$

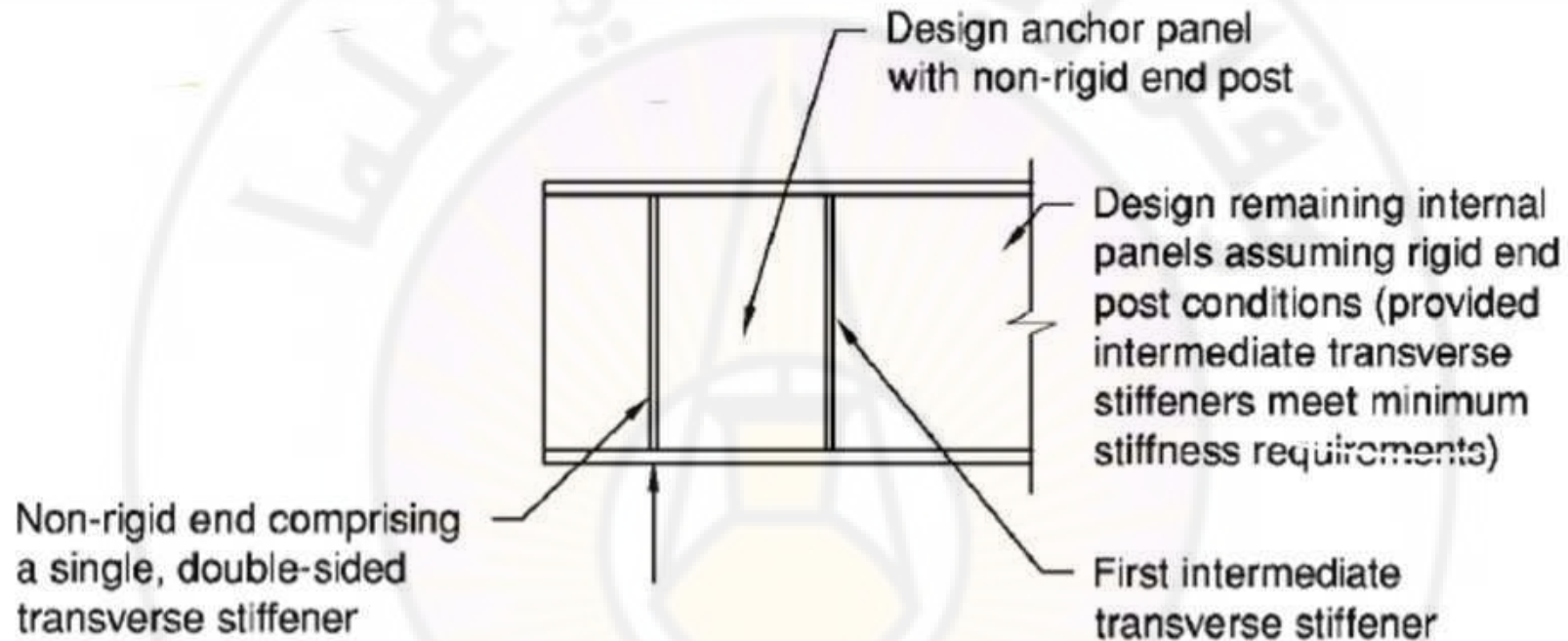


$$I \approx 328 \times 15 \times \left(\frac{2000}{2}\right)^2 \times 2$$

$$= 9840 \times 10^6 \text{ mm}^4$$

$$W_{el} = \frac{I}{D/2} = \frac{9840 \times 10^6}{2015/2} = 9767 \times 10^3 \text{ mm}^3$$

III-3-3 Non-rigid end Post

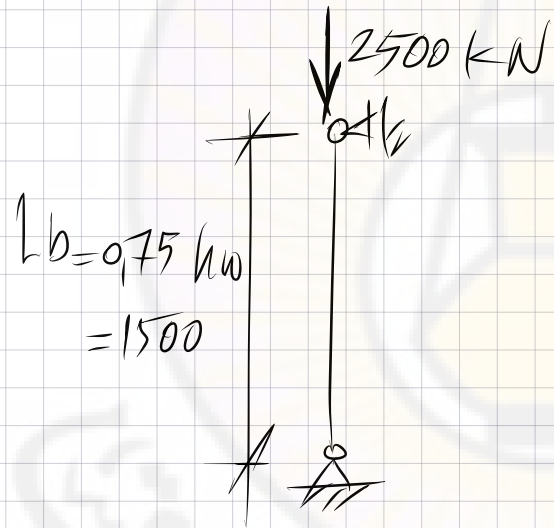


- ❖ A non-rigid end Post comprises only a single double-stiffeners
- ❖ Non-rigid end Post should be designed as bearing stiffeners. They do not need to be designed to carry any tensile anchorage.

Training: (T5)

check the non rigid end
stiffener and the reaction force
 $V_{Ed} = 2500 \text{ kN}$, S235

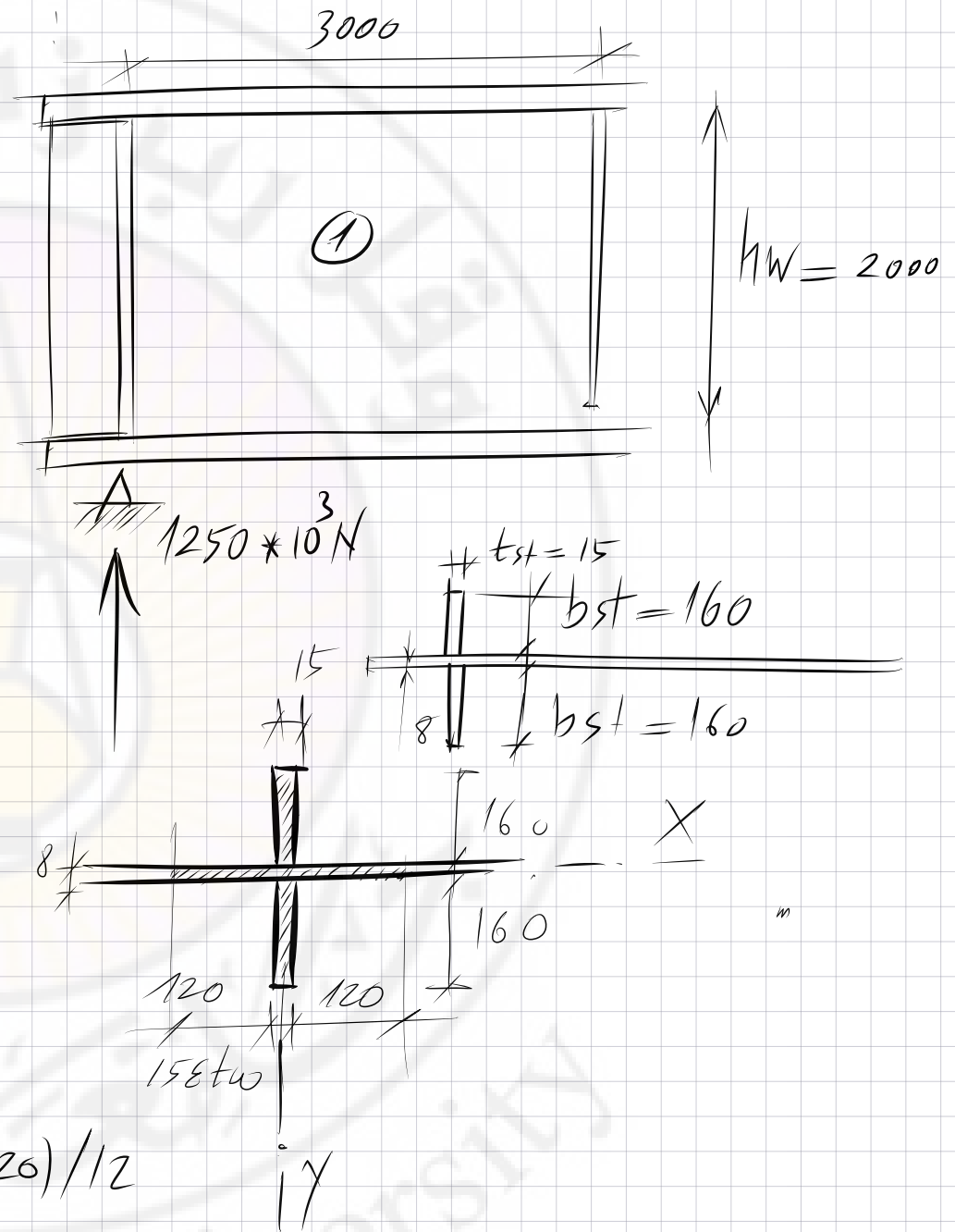
The stiffener and part of web act
as column,



Section Properties

$$A_{st} = 2(160 \times 15 + 120 \times 8) = 6720 \text{ mm}^2$$

$$\begin{aligned} I_{stx} &= (160 + 8 + 160) \cdot \frac{15^3}{12} + 8^3 \cdot \frac{(120 + 15 + 120)}{12} \\ &= 44120 \times 10^3 \text{ mm}^4 \end{aligned}$$



Training: (T5)

$$A_{st} = 6720 \text{ mm}^2, I_{st} = 44120 \times 10^3 \text{ mm}^4$$

$$i_{st} = \sqrt{I_{st} / A_{st}} = \sqrt{44120 \times 10^3 / 6720} \\ = 81 \text{ mm}$$

$$\lambda = \frac{L_0}{i_{st}} = \frac{1500}{81} = 18,52$$

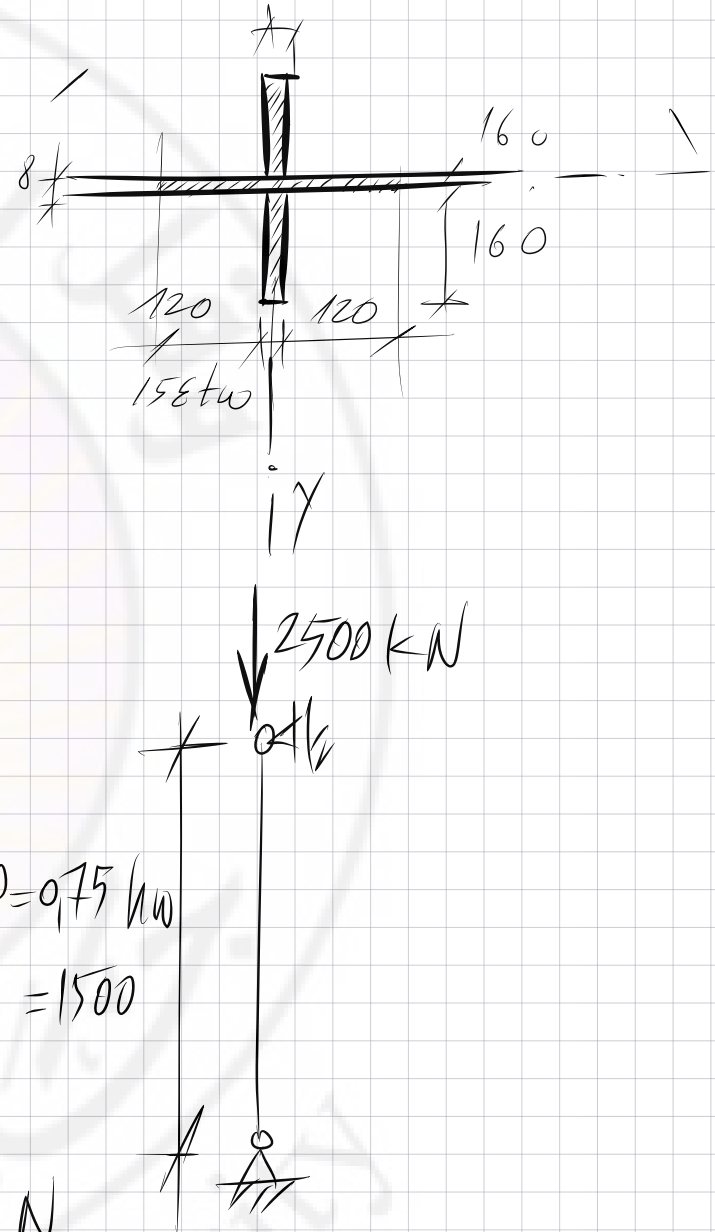
$$\lambda_1 = 93,8 = 93$$

$$\bar{\lambda} = \frac{18,52}{93} = 0,2 = 0,2$$

$$\rightarrow \chi = 1$$

$$N_{b,Rd} = \frac{\chi A_{st} f_y}{\gamma_{M1}} = \frac{6720 \times 235}{1,1} = 1435,6 \times 10^3 \text{ N}$$

$$N_{b,Rd} > N_{Ed} = 2500 \times 10^3 \text{ N} \rightarrow \text{ok}$$



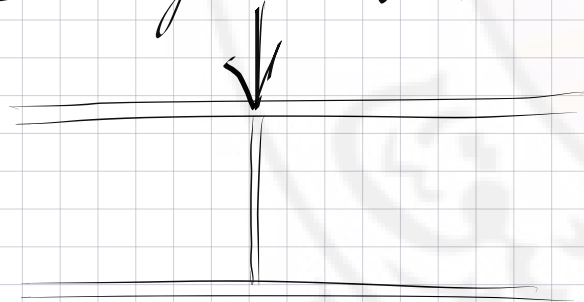
III-3-4 Intermediate Transverse Stiffeners ITS

III-3-4.1. Introduction

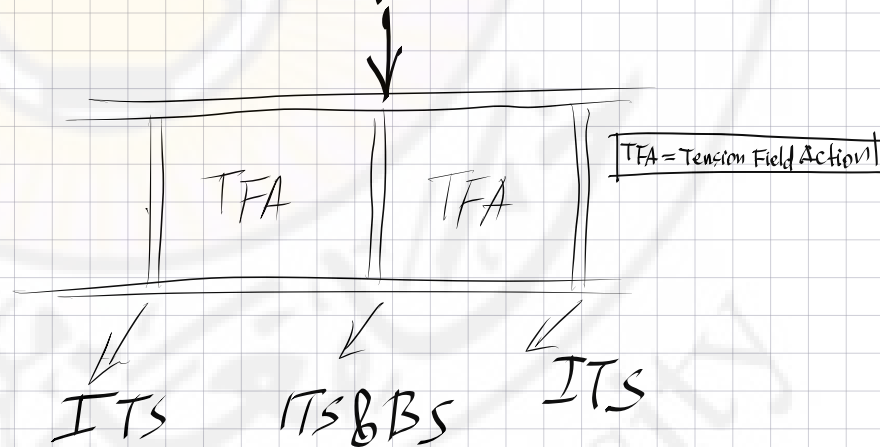
ITS and BS stiffeners are usually called:

Web Stiffeners

A particular stiffeners may serve more than one function, in which case it should be designed for the combined effect



Normal Bearing stiff.



Combined Action (ITS & BS)

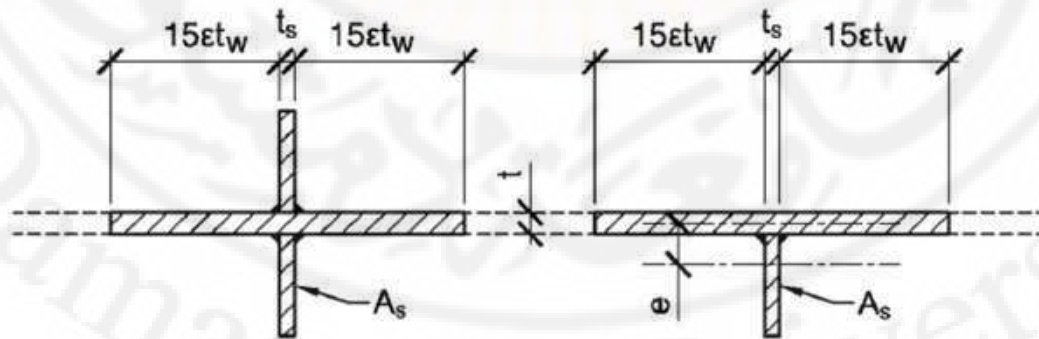
✎ The effective cross-section of a stiffener (used both for verifying minimum stiffness and for buckling effect) is defined in Clause 9.1(2) as the stiffener(s) plus a width of web equal to :

$$15 \epsilon t_w$$

✎ The outstand proportion for a flat stiffeners should generally be limited to :

$$h_s/t_s \leq 13,0 \epsilon$$

In order to avoid torsional buckling as per clause 9.2(8) of BS EN 1993 1-5



✶ ITS are used to:

- ① increase both the elastic shear buckling resistance and
- ② Ultimate shear buckling resistance $V_{b,Rd}$ of slender Web Panel.

✶ ITS are usually on one side of the Web only.

✶ ITS are designed for minimum stiffness and buckling resistance, as specified in clause 9.3.3 (1) of BS EN 1993-1-5.

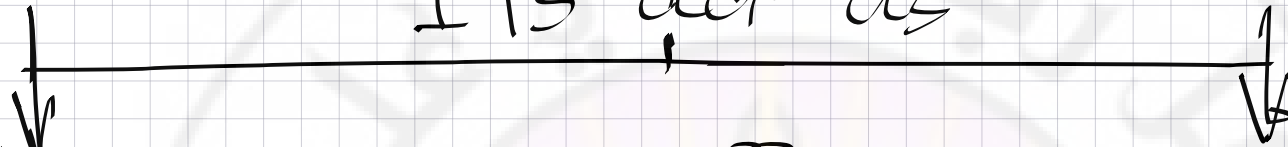
✶ According to clause 9.3.3 (3) of BS EN 1993-1-5
The minimum I_{st} required to prevent lateral deflection

$$\begin{array}{l} I_{st} \geq 1.5 h_w^3 t_w^3 / a^2 \rightarrow a/h_w < \sqrt{2} \\ I_{st} \geq 0.75 h_w \cdot t_w^3 \rightarrow a/h_w \geq \sqrt{2} \end{array}$$

III-3-4-2

ITS Design

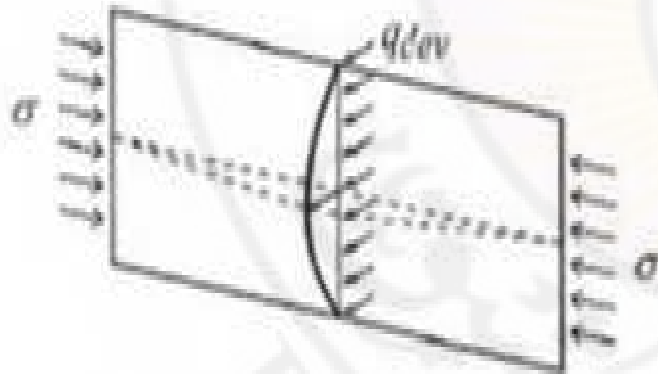
ITS act as



Ⓘ Rigid boundary support for Panel to Prevent Web buckling due to direct stress

Ⓢ Resist the shear stress effect arise from TFA and PH, which resulting axial force

TFA = Tension Field Action
PH = Plastic hinge



Direct stress

Bending $M_{Ed} \neq 0$



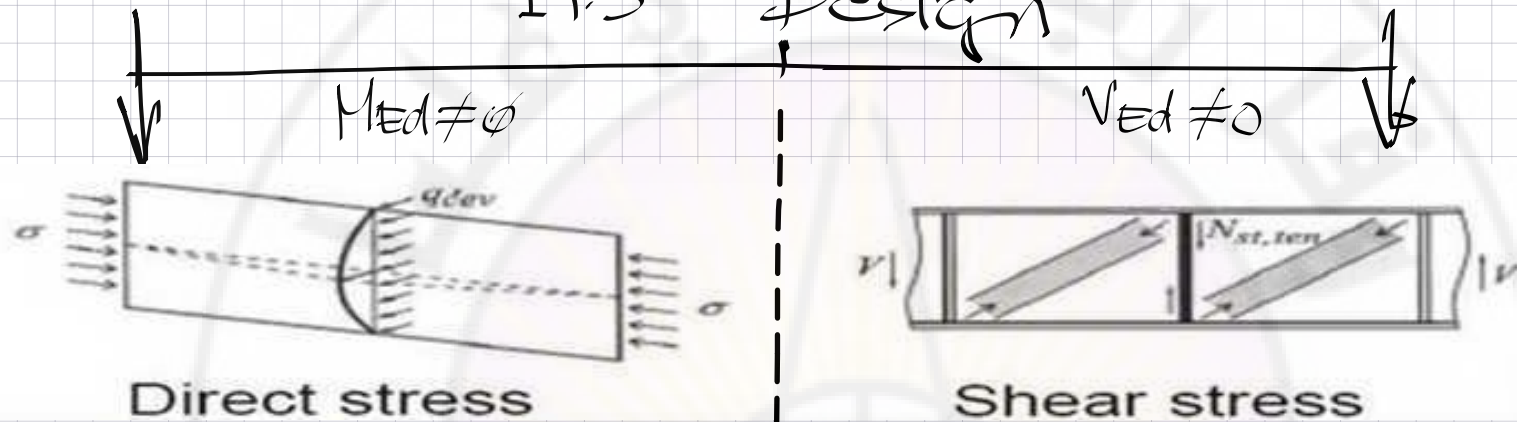
Shear stress

Shear $V_{Ed} \neq 0$

III-3-4-2

ITS Design

ITS Design



Resistance will be applied via Requirements

Requirements (A) : Verifying using second order analysis:
set limits for stress (σ_{max} and deflection w)

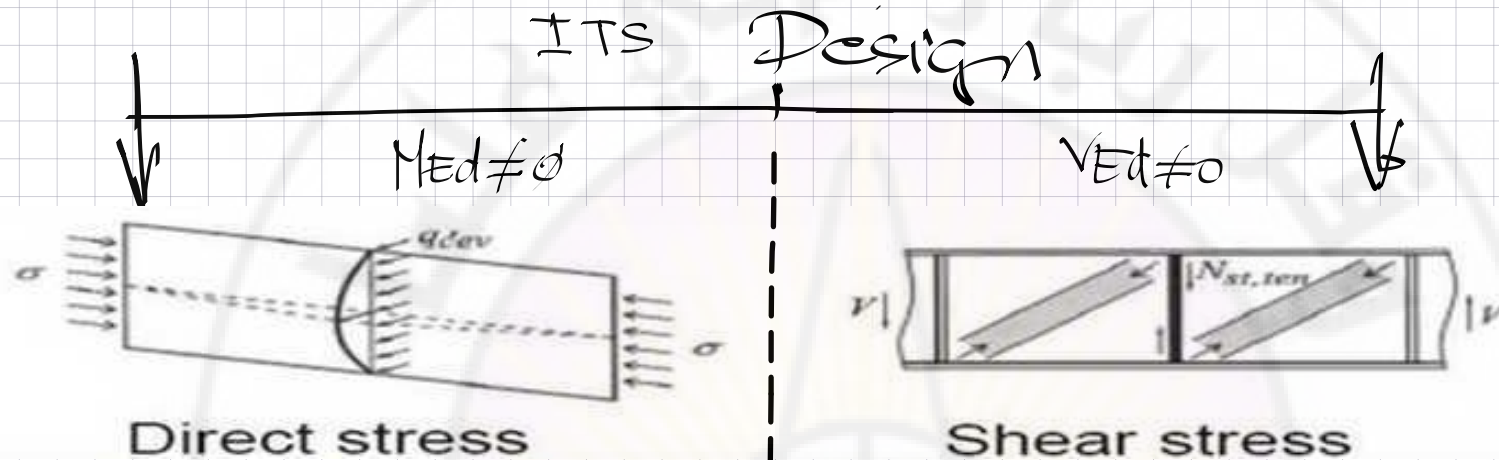
$$\sigma_{max} \leq f_y / \gamma_{M1} \quad , \quad w \leq b / 300$$

σ_{max} : Ultimate stress (elastic)

w : Ultimate lateral deflection

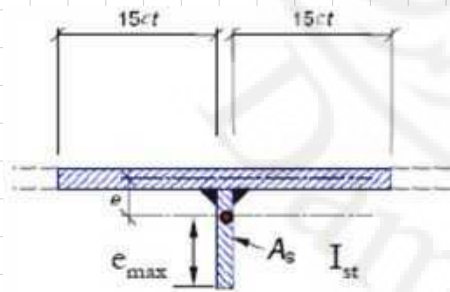
b : Panel height, or web height (h_w)

III-3-4-2 ITS Design



Resistance will be applied via Requirements

Requirements (A): Satisfied if:

$$I_{st} > I_{st,req}$$


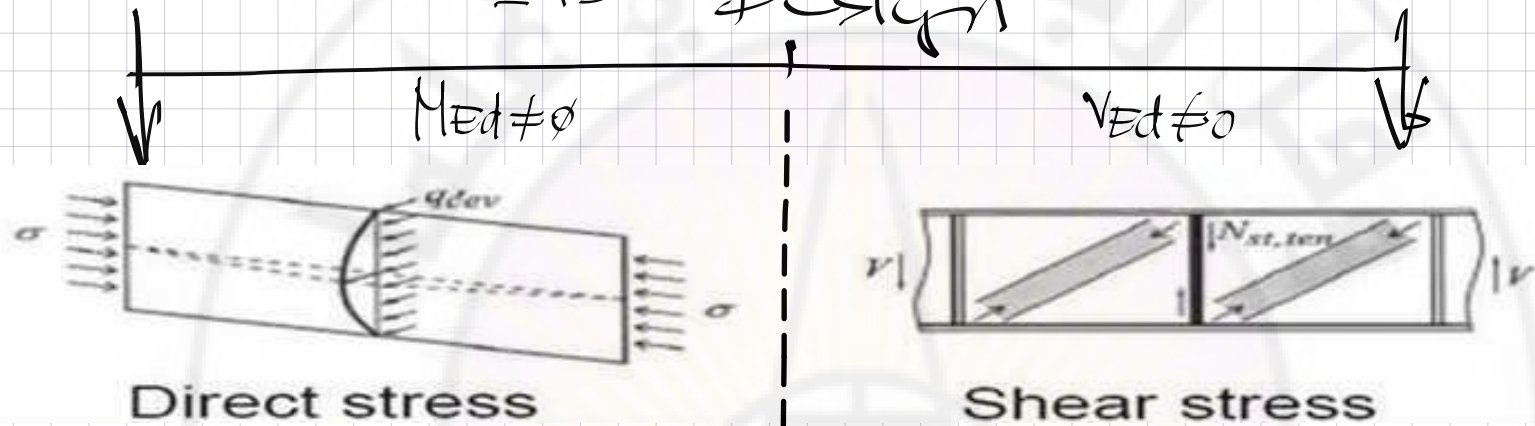
Requirements (A): Satisfied if:

$$I_{st} \geq \frac{1.5 h_w^3 \cdot t_w^3}{\alpha^2} \quad \text{if } \frac{q}{h_w} < \sqrt{2}$$

$$I_{st} \geq 0.75 h_w t_w^3 \quad \text{if } \frac{q}{h_w} \geq \sqrt{2}$$

III-3-4-2 ITS Design

ITS Design



Resistance will be applied via Requirements

Requirements (A): Satisfied if:

$$I_{st, req} = \frac{\sigma_{cr,c}}{E} \left(\frac{b}{\pi} \right)^4 \left(1 + w_0 \frac{300}{b} \cdot u \right)$$

$$\sigma_{cr,c} = \frac{\sigma_{cr,c}}{\sigma_{cr,p}} \left(\frac{N_{Ed}}{b} \right) \left(\frac{1}{a_1} + \frac{1}{a_2} \right)$$

$$u = \pi^2 \cdot E \cdot z_{max} / (300 f_y b / 10 H_1) \geq 1$$

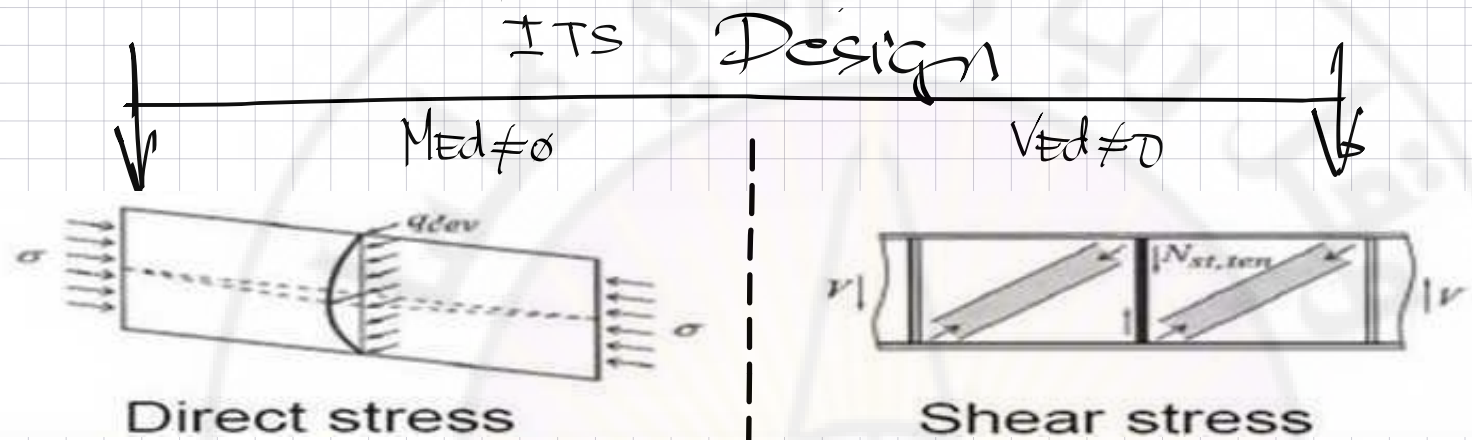
$$w_0 = \min \left[\frac{b}{300}, \frac{a_1}{300}, \frac{a_2}{300} \right]$$

Verification of
shear stress

forces

Axial forces generated by TFA can be shown that when shear force is present, under the action of TFA, an axial

III-3-4-2 ITS Design



Resistance will be applied via Requirements

Requirements (A):

elastic critical stress for column

$$\sigma_{cr,c} = \frac{\pi^2 E t_w^2}{12(1-\nu^2)a^2}$$

elastic critical stress for Panel

$$\sigma_{cr,p} = k_0 \frac{\pi^2 E t_w^2}{12(1-\nu^2)b^2}$$

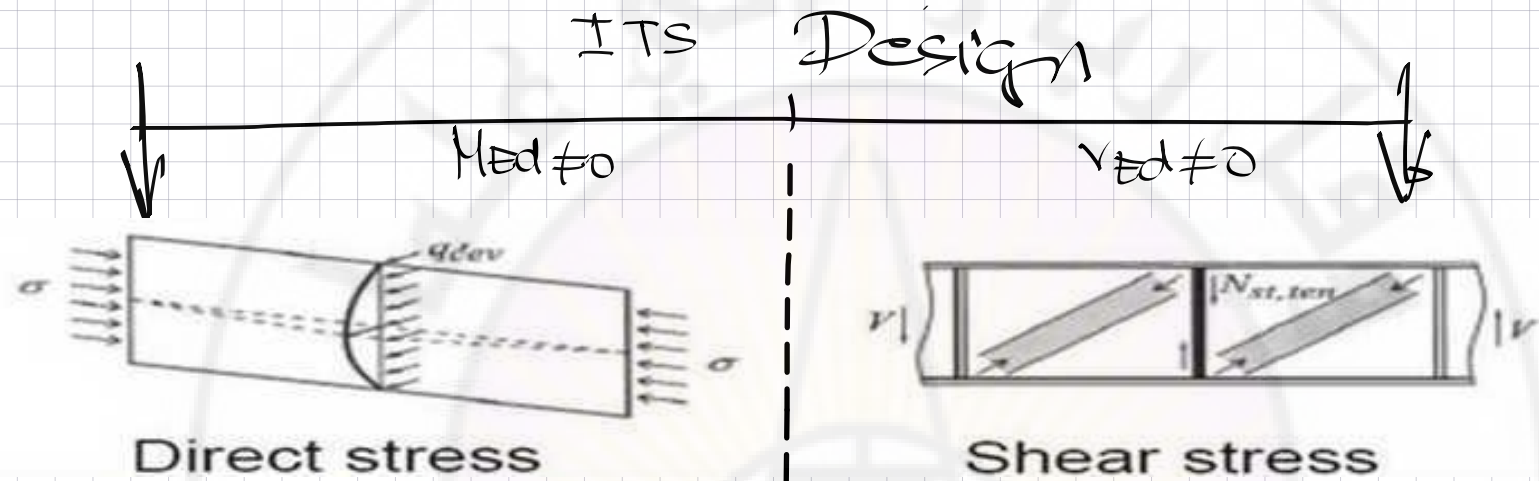
with limiting value of their ratio $0.5 < \frac{\sigma_{cr,c}}{\sigma_{cr,p}} \leq 1.0$

force $N_{st,ten}$ will be imposed on the intermediate transverse stiffener such that

$$N_{st,ten} = V_{Ed} \tau \frac{h_w f_{yw} t_w}{(\lambda_w)^2 \sqrt{3} \gamma_{M1}} \geq 1.0$$

V_{Ed} : design shear force from the adjacent panel

III-3-4-2 ITS Design



Resistance will be applied via Requirements

Requirements (A):

K_{σ} is the Plate buckling Coefficient, depend on ψ . Use Table 4.1 of EC3 part 1-5 section 4.4

For the case of Variable shear force, the value at the distance of a_{shw} from the edge of the Panel with the larger force should be taken

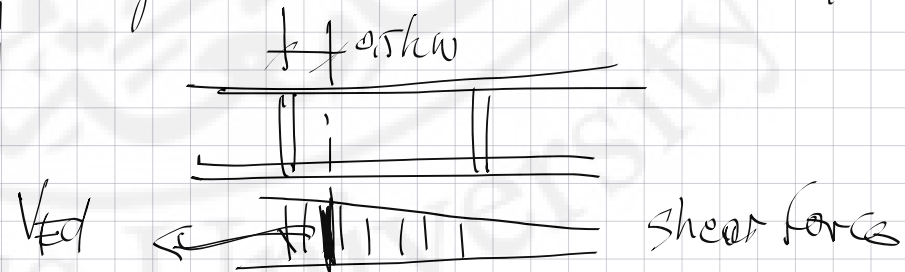


Table 4.1: Internal compression elements

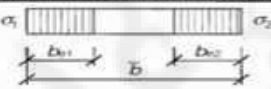
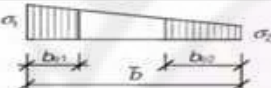
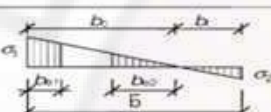
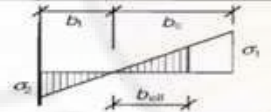
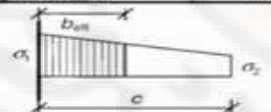
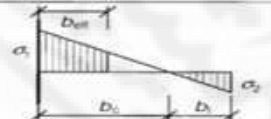
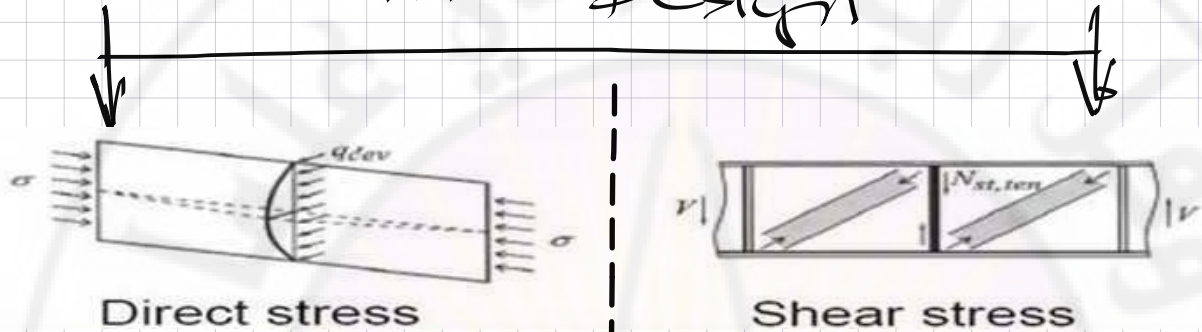
Stress distribution (compression positive)				Effective ^p width b_{eff}	
				$\psi = 1$: $b_{eff} = \rho \cdot b$ $b_{e1} = 0,5 b_{eff}$ $b_{e2} = 0,5 b_{eff}$	
				$1 > \psi \geq 0$: $b_{eff} = \rho \cdot b$ $b_{e1} = \frac{2}{5 - \psi} b_{eff}$ $b_{e2} = b_{eff} - b_{e1}$	
				$\psi < 0$: $b_{eff} = \rho \cdot b_c = \rho \cdot b / (1 - \psi)$ $b_{e1} = 0,4 b_{eff}$ $b_{e2} = 0,6 b_{eff}$	
$\psi = \sigma_2 / \sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_{σ}	4,0	$8,2 / (1,05 + \psi)$	7,81	$7,81 - 6,29\psi + 9,78\psi^2$	$23,9 - 5,98(1 - \psi)^2$

Table 4.2: Outstand compression elements

Stress distribution (compression positive)				Effective ^p width b_{eff}	
				$1 > \psi \geq 0$: $b_{eff} = \rho \cdot c$	
				$\psi < 0$: $b_{eff} = \rho \cdot b_c = \rho \cdot c / (1 - \psi)$	
$\psi = \sigma_2 / \sigma_1$	1	0	-1	$1 \geq \psi \geq -3$	
Buckling factor k_{σ}	0,43	0,57	0,85	$0,57 - 0,21\psi + 0,07\psi^2$	
				$1 > \psi \geq 0$: $b_{eff} = \rho \cdot c$	
				$\psi < 0$: $b_{eff} = \rho \cdot b_c = \rho \cdot c / (1 - \psi)$	
$\psi = \sigma_2 / \sigma_1$	1	$1 > \psi > 0$	0	$0 > \psi > -1$	-1
Buckling factor k_{σ}	0,43	$0,578 / (\psi + 0,34)$	1,70	$1,7 - 5\psi + 17,4\psi^2$	23,8

III-3-4-2 ITS Design

ITS Design



Resistance will be applied via Requirements

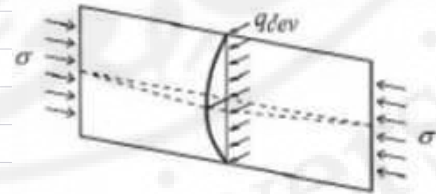
Requirements (A):

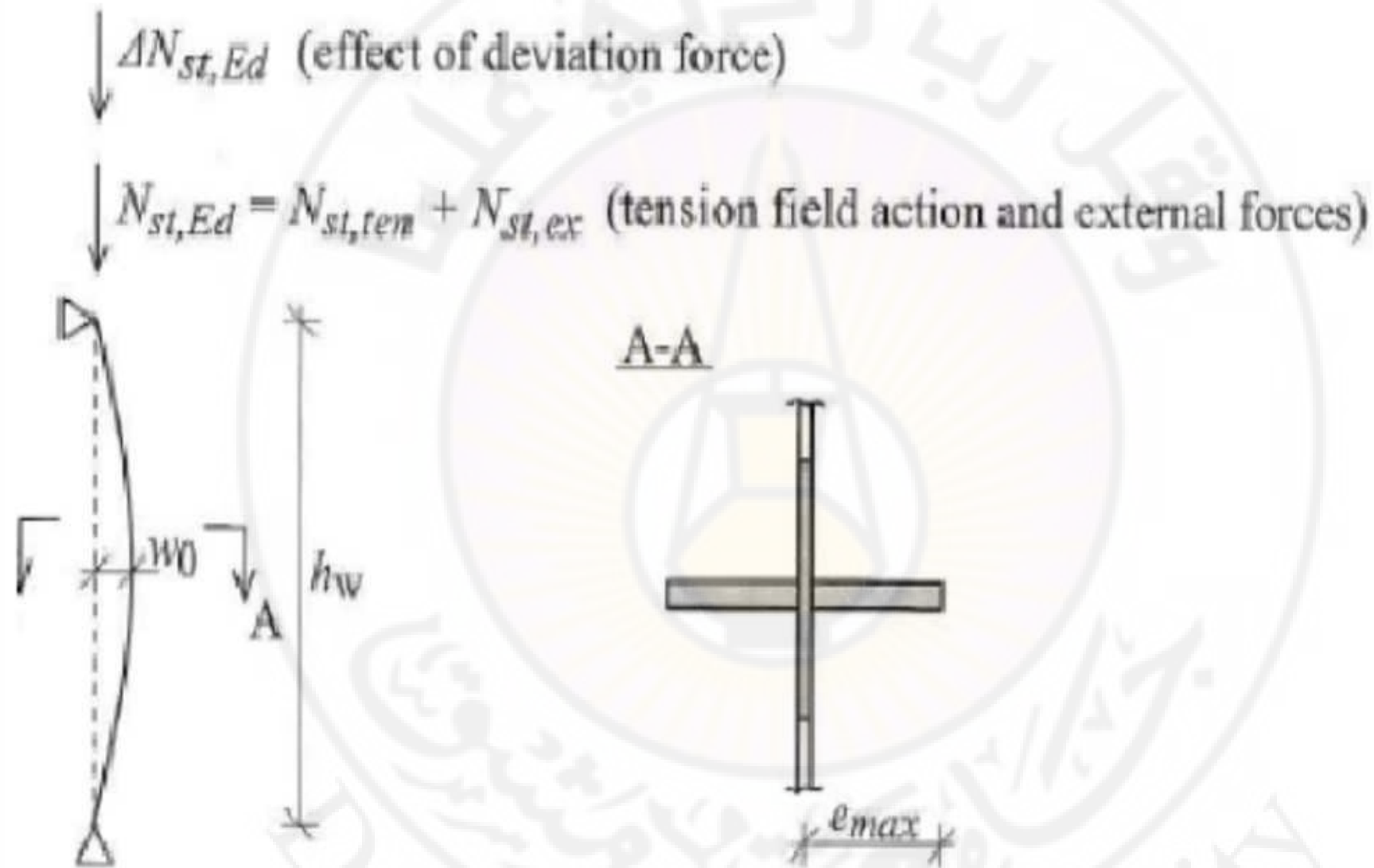
N_{Ed} : The maximum force of both adjacent panels.

Interaction with direct stress
if direct stress is present (i.e. $M_{Ed} \neq 0$), a small lateral deflection will be induced:

$$\Delta N_{st,ten} = \frac{\sigma_m h w^2}{\pi^2}$$

$$N_{Ed} = \sigma_m \cdot h w \cdot t w / 4$$

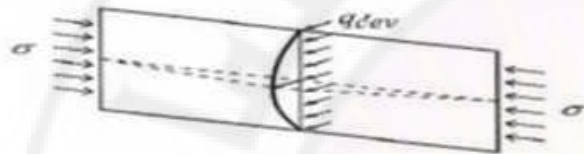




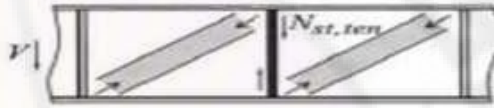
Mechanical model for a double-sided stiffener

III-3-4-2 ITS Design

ITS Design



Direct stress



Shear stress

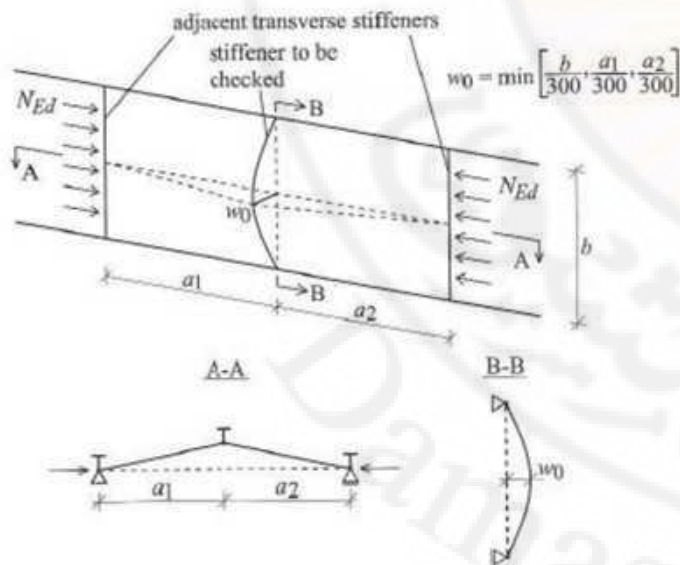
Resistance will be applied via Requirements

Requirements (A) :

Deflection

Deflection should be calculated as :

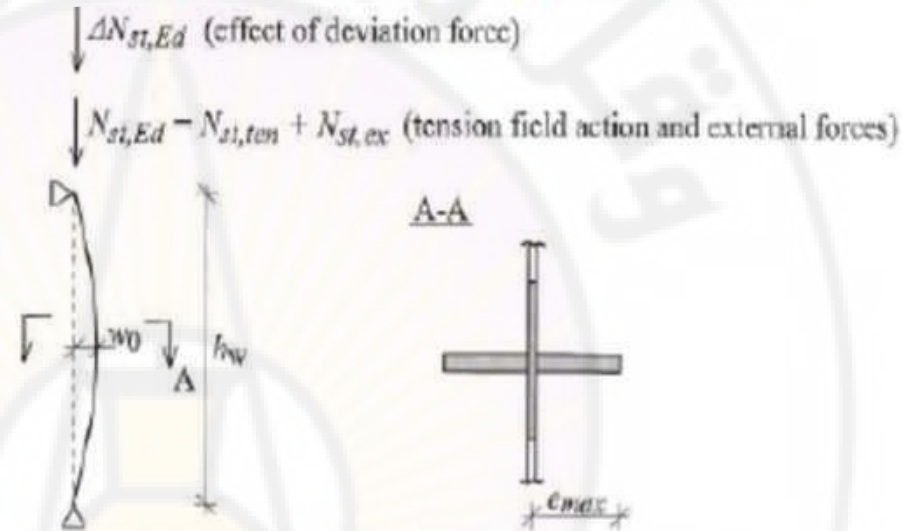
$$w \leq \frac{lw}{300}$$



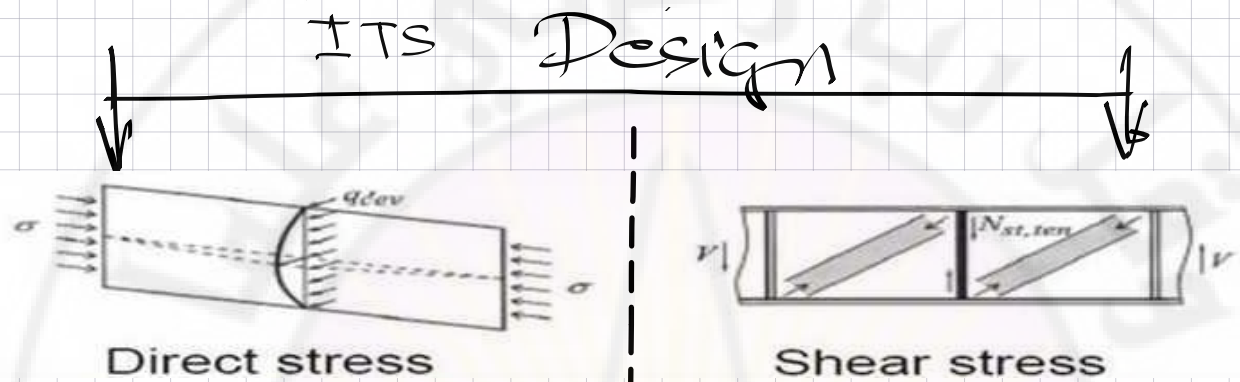
$$w = w_0 \frac{1}{\left(\frac{N_{cr,st}}{N_{st,Ed} + \Delta N_{st,Ed}} - 1 \right)} \leq \frac{h_w}{300}$$

$$N_{cr,st} = \frac{\pi^2 EI_{st}}{h_w^2}$$

$$\sigma_{\max} = \frac{N_{st,Ed}}{A_{st}} + \left(\frac{(N_{st,Ed} + \Delta N_{st,Ed}) e_{\max}}{I_{st}} \right) w_0 \left(\frac{1}{\left(1 - \frac{(N_{st,Ed} + \Delta N_{st,Ed})}{N_{cr,st}} \right)} \right) \leq \frac{f_y}{\gamma_{M1}}$$



III-3-4-2 ITS Design



Resistance will be applied via Requirements

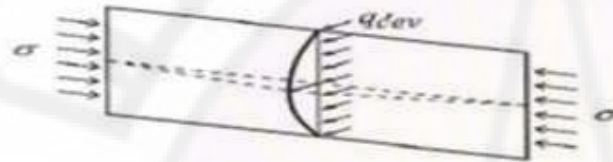
Requirements (A) :

In checking

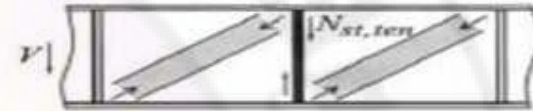
- 1) Calculate (u) first
- 2) if $u < 1.0 \rightarrow$
displacement check is
decisive:
 $u = 1$

III-3-4-2 ITS Design

ITS Design



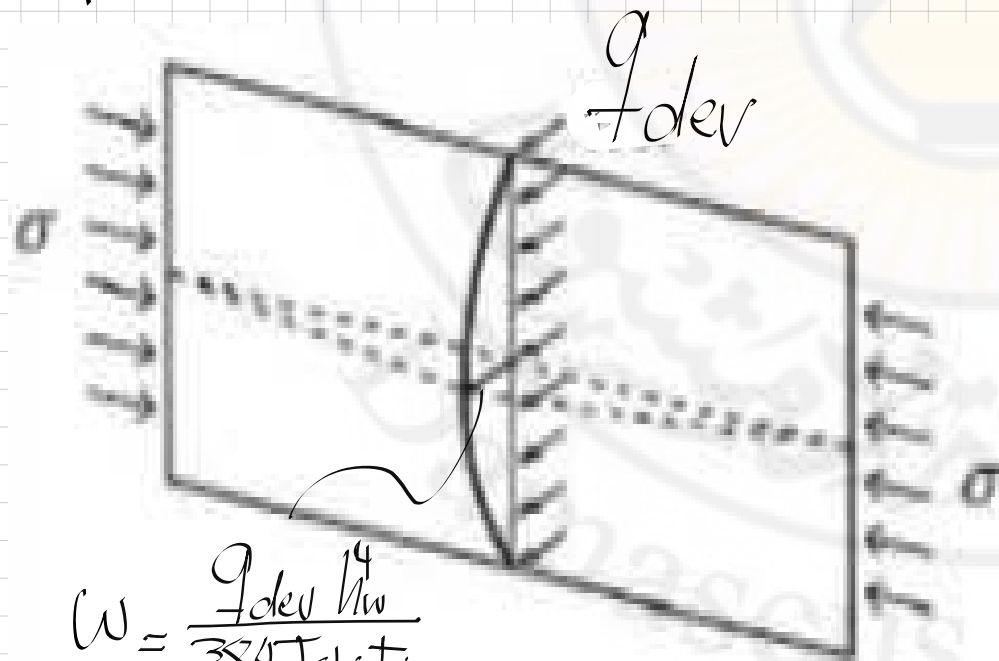
Direct stress



Shear stress

Resistance will be applied via Requirements

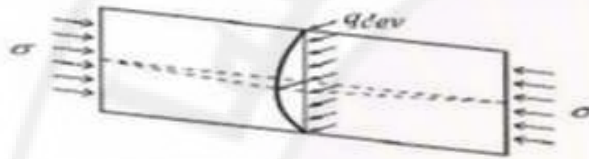
Requirements (A):



$$W = \frac{q_{dev} h_w^4}{384 I_{st} \cdot E}$$

III-3-4-2 ITS Design

ITS Design



Direct stress



Shear stress

Resistance will be applied via Requirements

Requirements (A):

Calculate the deflection as displacement through

$$f_{day, Ed} = \frac{\pi}{4} \sigma_m (w_0 + w_{el})$$

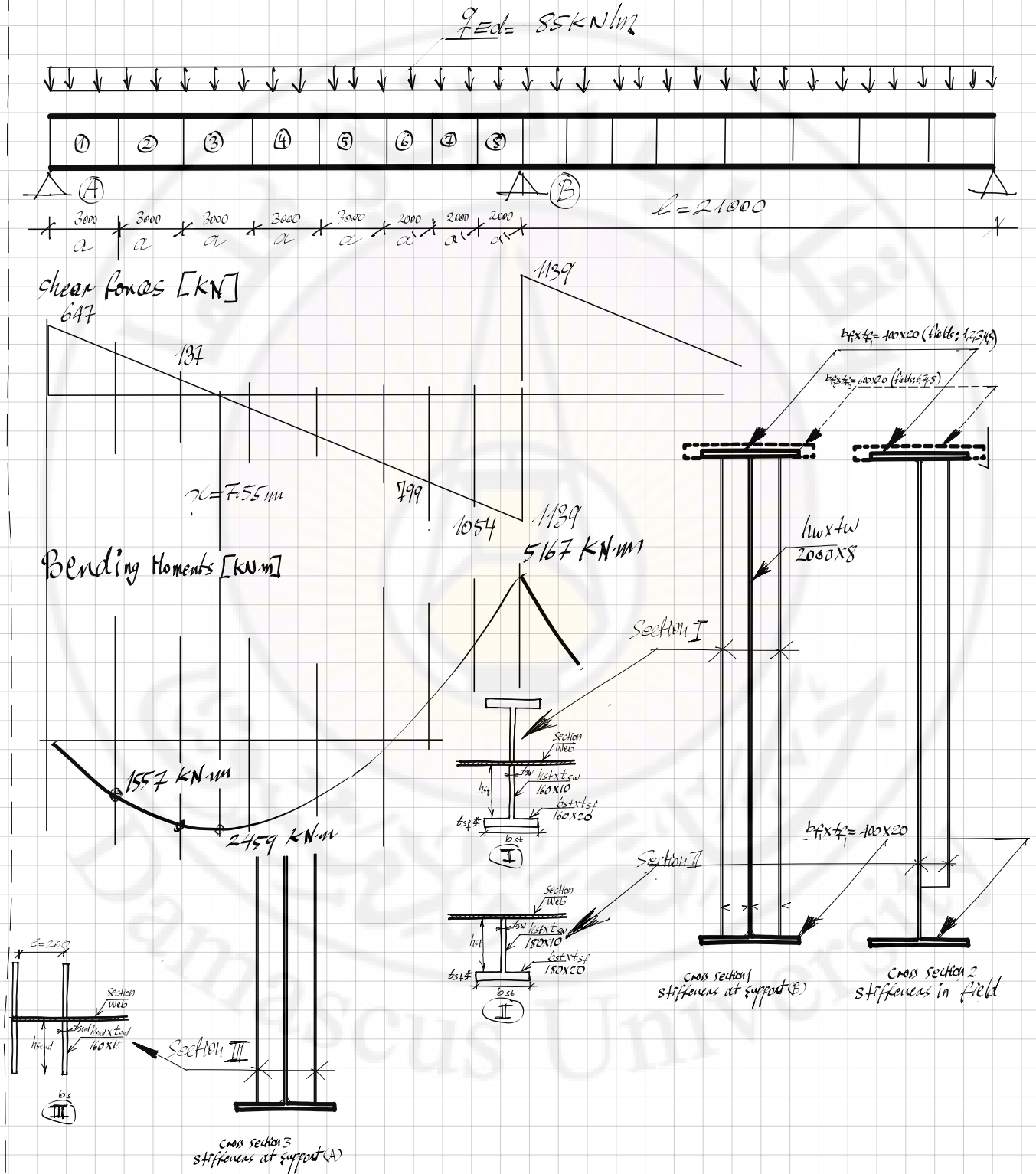
$$w_{el} = \text{max deflection} = \frac{b}{300}$$

$$w = \frac{f_{day, Ed} \cdot h_w^4}{384 I_{st} \cdot E}$$

$$w \leq b/300$$

Training ⑥

check the Intermediate Transfer stiffener (ITS)
at the field ③,



Field (3)

at the mid span

$$V_{Ed} = 137 \cdot 10^3 \text{ N}$$

max(3)

$$M_{Ed} = 2459 \cdot 10^3 \text{ N.m}$$

max(3)

③ Stiffeners Verification

3.1 Intermediate transverse Stiffeners [Fields (7), (8)]

The transverse stiffeners are checked for the most severe conditions at the location close to the interior support. It can be easily shown that the same cross section fulfils the relevant requirements also in midspan region. ②

One-sided T-stiffeners

T-section of stiffener (Section A)

$h_{st}/b_{st}/t_{stw}/t_{stf} = 180/180/10/20$
is class (3).

$$A_{st} = h_{st} t_{stw} + b_{st} t_{stf}$$

$$= 0,18 \times 0,01 + 0,18 \times 0,02 = 5,40 \cdot 10^{-3} \text{ m}^2$$

$$e_1 = \frac{\frac{h_{st}^2}{2} \cdot t_{stw} + b_{st} \cdot t_{stf} \left(h_{st} + \frac{t_{stf}}{2} \right)}{A_{st}} = \frac{\frac{0,18^2}{2} \times 0,01 + 0,18 \times 0,02 \left(0,18 + \frac{0,02}{2} \right)}{5,40 \times 10^{-3}} = 156,7 \cdot 10^{-3} \text{ m}$$

$$e_2 = h_{st} + t_{stf} - e_1 = 0,18 + 0,02 - 156,7 \cdot 10^{-3} = 43,3 \cdot 10^{-3} \text{ m}$$

Double T-section

of stiffeners (Section A')

$$A'_{st} = A_{st} + (2 \times 15 \varepsilon t_w + t_{stw}) t_w$$

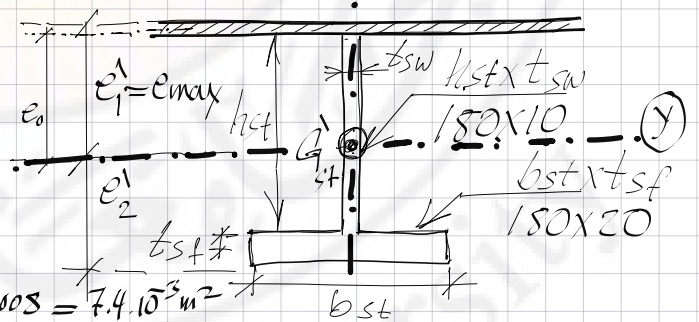
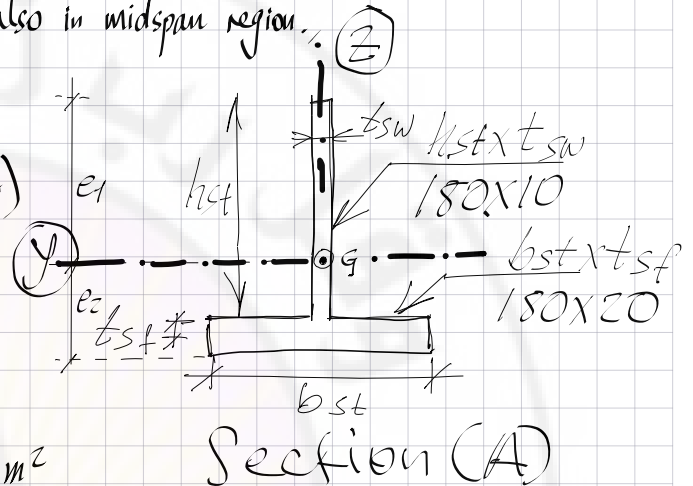
$$= 5,4 \cdot 10^{-3} + (2 \times 15 \times 1,0 \times 0,008 + 0,01) \times 0,008 = 7,4 \cdot 10^{-3} \text{ m}^2$$

$$e'_1 = \frac{1}{A'_{st}} \left[(2 \times 15 \varepsilon t_w + t_{stw}) \frac{t_w^3}{2} + h_{st} \cdot t_{stw} \left(t_w + \frac{h_{st}}{2} \right) + b_{st} t_{stf} \left(t_w + h_{st} + \frac{t_{stf}}{2} \right) \right]$$

$$= \frac{1}{7,4 \times 10^{-3}} \left[(2 \times 15 \times 1 \times 0,008 + 0,01) \frac{0,008^2}{2} + 0,18 \times 0,01 \left(0,008 + \frac{0,18}{2} \right) + 0,18 \times 0,02 \left(0,008 + 0,18 + \frac{0,02}{2} \right) \right]$$

$$e'_1 = 121,2 \cdot 10^{-3} \text{ m}$$

$$e'_2 = t_w + h_{st} + t_{stf} = 0,008 + 0,18 + 0,02 - 121,2 \cdot 10^{-3} = 86,8 \cdot 10^{-3} \text{ m}$$



$$I_{st} = \frac{(2 \times 15 \times t_w + t_{st,w}) t_w^3}{12} + \frac{h_{st}^3 t_{st,w}}{12} + \frac{b_{st,w} t_{st,f}^3}{12} + (2 \times 15 \times t_w + t_{st,w}) t_w \left(e_1 - \frac{t_w}{2} \right)^2 + h_{st} t_{st,w} \left(e_1 - \frac{h_{st}}{2} - t_w \right)^2 + b_{st} t_{st,f} \left(e_2 - \frac{t_{st,f}}{2} \right)^2$$

$$I_{st} = \frac{(2 \times 15 \times 1 \times 0,008 + 0,01) \times 0,008^3}{12} + \frac{0,18^3 \times 0,01}{12} + \frac{0,18 \times 0,02^3}{12} + (2 \times 15 \times 1 \times 0,008 + 0,01) \times 0,008 \left(12,2 \times 10^{-3} - \frac{0,008}{2} \right)^2 + 0,18 \times 0,01 \left(12,2 \times 10^{-3} - \frac{0,18}{2} - 0,008 \right)^2 + 0,18 \times 0,02 \left(86,8 \times 10^{-3} - \frac{0,02}{2} \right)^2$$

$$I_{st} = 54,66 \cdot 10^{-6} \text{ m}^4$$

Torsion Buckling

$$\sigma_{cr} = \frac{1}{I_p} \left(G I_t + \frac{\pi^2 E I_w}{l_w^2} \right) \geq 6 \cdot f_y$$

$$I_p = I_y + I_z$$

Second moments of areas around the point of stiffener to Web junction

$$I_y = \frac{h_{st}^3 t_{st,w}}{12} + \frac{b_{st} t_{st,f}^3}{12} + h_{st} t_{st,w} \left(e_1 - \frac{h_{st}}{2} \right)^2 + b_{st} t_{st,f} \left(e_2 - \frac{t_{st,f}}{2} \right)^2 + A_{st} \cdot e_1^2$$

$$I_y = \frac{0,18^3 \times 0,01}{12} + \frac{0,18 \times 0,02^3}{12} + 0,18 \times 0,01 \left(156,7 \times 10^{-3} - \frac{0,18}{2} \right)^2 + 0,18 \times 0,02 \left(43,3 \times 10^{-3} - \frac{0,02}{2} \right)^2 + 540 \times 10^{-3} \times \left(156,7 \times 10^{-3} \right)^2 = 149,58 \cdot 10^{-6} \text{ m}^4$$

$$I_z = \frac{h_{st} \cdot t_{st,w}^3}{12} + \frac{b_{st}^3 h_{st,f}}{12} = \frac{0,18 \times 0,01^3}{12} + \frac{0,18 \times 0,02}{12} = 9,74 \cdot 10^{-6} \text{ m}^4$$

$$I_p = I_y + I_z = 149,58 \cdot 10^{-6} + 9,74 \cdot 10^{-6} = 159,32 \cdot 10^{-6} \text{ m}^4$$

$$I_w = \frac{b_{st}^3 \cdot t_{st,f}}{12} \left(h_{st} + \frac{t_{st,f}}{2} \right) = \frac{0,18^3 \times 0,02}{12} \left(0,18 + \frac{0,02}{2} \right)^2 = 350,892 \cdot 10^{-9} \text{ m}^6$$

$$I_t = \frac{1}{3} (h_{st} \cdot t_{st,w}^3 + b_{st} \cdot t_{st,f}^3) = \frac{1}{3} (0,18 \times 0,01^3 + 0,18 \times 0,02^3) = 540 \cdot 10^{-9} \text{ m}^4$$

$$\sigma_{cr} = \frac{1}{159,32 \cdot 10^{-6}} \left(80,77 \cdot 10^9 \times 540 \times 10^{-9} + \frac{\pi^2 \times 210 \cdot 10^9 \times 350,892 \times 10^{-9}}{(200)^2} \right) = 1415 \cdot 10^6 \text{ N/m}^2$$

$$\sigma_{cr} = 1415 \cdot 10^6 \text{ N/m}^2 > 6 \times 235 \cdot 10^6 \text{ N/m}^2 \dots \text{OK}$$

Minimum requirements for transverse stiffeners (shear)

$$\frac{a'}{h_w} = \frac{200}{200} = 1 < \sqrt{2} \Rightarrow$$

$$I_{st} = 54,66 \times 10^{-6} \text{ m}^4 \geq \frac{1,5 h_w^3 t_w^3}{a'^2} = \frac{1,5 \times 200^3 \times 0,008^3}{200^2} = 1,536 \cdot 10^{-6} \text{ m}^4$$

Transverse stiffeners act as rigid supports for web panel.

Additional compression axial force in transverse stiffener from the tension field action.

$V_{Ed} = 1054 \cdot 10^3 \text{ N}$ (0,5 h_w from the edge of the panel)
 $a' = 200 \text{ mm}$

slenderness $\bar{\lambda}_w$ for shear

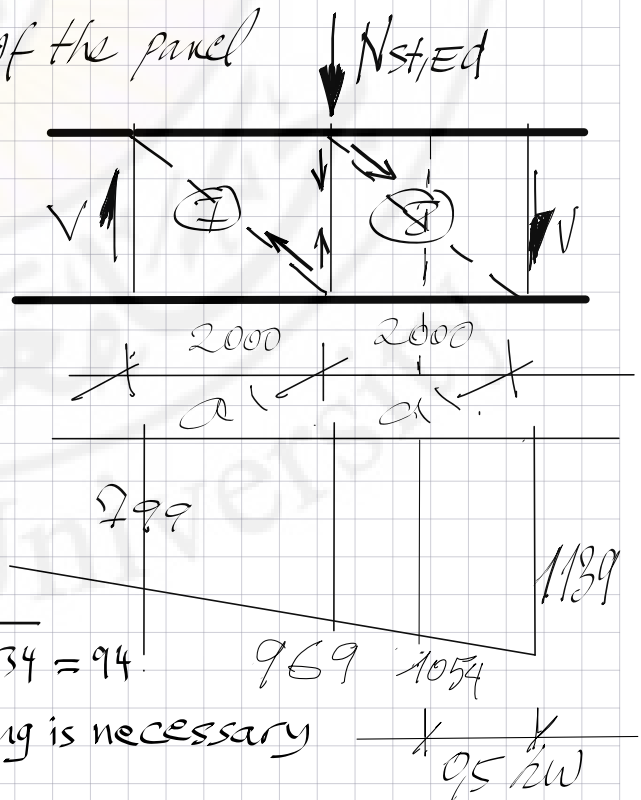
$$\bar{\lambda}_w = \frac{h_w}{37,4 t_w \cdot \varepsilon \sqrt{k_\tau}}$$

$$\frac{a}{h_w} = \frac{200}{200} = 1 \rightarrow k_\tau = 5,34 + 4 \left(\frac{h_w}{a} \right)^2 = 5,34 + 4 \left(\frac{200}{200} \right)^2 = 9,34$$

$$\frac{h_w}{t_w} = \frac{200}{0,008} = 250 > \frac{31}{\eta} \varepsilon \sqrt{k_\tau} = \frac{31}{1} \cdot 1 \cdot \sqrt{9,34} = 94$$

Verification of shear buckling is necessary

$$\bar{\lambda}_w = \frac{h_w}{37,4 \cdot t_w \cdot \sqrt{k_\tau}} = \frac{200}{37,4 \times 0,008 \cdot \sqrt{9,34}} = 2,2$$



Intermediate rigid stiffeners may be designed for an axial force equal to $N_{st,Ed}$ according to EN 1993-1-5, 9.3.3 (3).

In the case of variable shear force the check is performed for the distance $0,9 h_w$ from the edge of the panel with the largest shear force.

$$N_{st,Ed} = V_{Ed} - \frac{h_w \cdot t_w \cdot f_{yw}}{\tau_w^2 \cdot \alpha_{M1} \cdot \sqrt{3}}$$

$$= 1054 \times 10^3 - \frac{2,00 \times 0,008 \times 235 \times 10^6}{2,2^2 \times 1,1 \times \sqrt{3}} =$$

$$N_{st,Ed} = 1054 \times 10^3 - 408 \times 10^3 = 646 \times 10^3 \text{ N}$$

Axial force from the deviation forces:

$$\Delta N_{st,Ed} = \frac{\sigma_m h_w^3}{\tau^2}$$

$$\sigma_m = \frac{\sigma_{cr,c}}{\sigma_{cr,p}} \cdot \frac{N_{Ed}}{h_w} \left(\frac{1}{a_1} + \frac{1}{a_2} \right)$$

$$\text{suppose } \frac{\sigma_{cr,c}}{\sigma_{cr,p}} = 1$$

$$\sigma_{c,max} = \sigma_{Ed,max} = \eta_1 \frac{f_y}{\alpha_{M0}}$$

$$\eta_1 = \frac{M_{Ed}}{M_{Rd}}$$

$$M_{Rd} = \frac{f_y}{\alpha_{M0}} \cdot W_{eff}$$

$$\text{suppose } W_{eff} = \alpha \cdot W_{elastic} = 24185 \cdot 10^6 \text{ mm}^3$$

$$M_{Rd} = 235 \times 10^6 / 1,0 \times 24185 \times 10^6 = 5683 \cdot 10^3 \text{ N}\cdot\text{m}$$

$$\eta_1 = \frac{5167 \cdot 10^3}{5683 \cdot 10^3} = 0,909 < 1 \rightarrow \text{ok}$$

$$\sigma_{Ed, \max} = \eta_1 \frac{f_y}{\gamma_{M0}} = 0,909 \frac{235 \cdot 10^6}{1,0} = 214 \cdot 10^6 \text{ N/m}^2$$

$$\sigma_{c, \max} = \sigma_{Ed, \max} = 214 \cdot 10^6 \text{ N/m}^2$$

$$N_{Ed} = \sigma_{c, \max} \cdot \frac{A_{eff}}{2}$$

$$\text{suppose } A_{eff} = 11.561 \cdot 10^3 \text{ N}$$

$$N_{Ed} = 214 \cdot 10^6 \frac{11.561 \cdot 10^3}{2} = 1237 \cdot 10^3 \text{ N}$$

$$\sigma_m = 1 \cdot \frac{1237 \cdot 10^3}{200} \left(\frac{1}{20} + \frac{1}{20} \right) = 617,5 \cdot 10^3 \text{ N/m}^2$$

$$\Delta N_{st, Ed} = \frac{\sigma_m \cdot h_w^2}{\pi^2} = \frac{617,5 \cdot 10^3 \cdot 200}{\pi^2}$$

$$= 250,3 \cdot 10^3 \text{ N}$$

Strength and stiffness check of the stiffener at ULS

$$\Sigma N_{st, Ed} = N_{st, Ed} + \Delta N_{st, Ed}$$

$$= 646 \cdot 10^3 + 250,3 \cdot 10^3$$

$$= 896,3 \cdot 10^3 \text{ N}$$

Single sided transverse stiffeners are not treated explicitly in the EN1993-1-5. In this numerical example relevant stress and $\rightarrow$

displacement checks.

$$\sigma_{max} \leq f_y$$

$$w \leq kw/300$$

are performed according to the interaction formula given in 9.2.1 of the Commentary to EN 1993-1-5. This interaction formula is based on the second order elastic analysis and takes account of the stiffener eccentricity and simultaneous action of the deviation forces and the axial forces from the tension field action.

$$N_{cr,st} = \frac{\pi^2 EI_{st}}{k_w^2} = \frac{\pi^2 \cdot 210 \cdot 10^9 \cdot 54.66 \cdot 10^6}{(200)^2} = 28322 \cdot 10^3 \text{ N}$$

$$e_{max} = e'_1 = 121.2 \cdot 10^{-3} \text{ m}$$

$$e_0 = e_{max} - \frac{t_w}{2} = 121.2 \cdot 10^{-3} - \frac{9.008}{2} = 117.2 \cdot 10^{-3} \text{ m}$$

$$w_0 = \min\left(\frac{a_1}{300}, \frac{a_2}{300}, \frac{k_w}{300}\right) \\ = \min\left(\frac{200}{300}, \frac{200}{300}, \frac{200}{300}\right) = 67 \cdot 10^{-3} \text{ m}$$

$$\eta_m = \frac{N_{st,Ed} \cdot e_0}{\sum N_{st,Ed} \cdot w_0} = \frac{646 \cdot 10^3 \cdot 117.2 \cdot 10^{-3}}{896.3 \cdot 10^3 \cdot 67 \cdot 10^{-3}} = 12.6$$

$$\sigma_{max} = \frac{N_{st,Ed}}{A_{st}} + \frac{\sum N_{st,Ed} \cdot e_{max} \cdot w_0}{I_{st}} \cdot \frac{1}{1 - \frac{\sum N_{st,Ed}}{N_{cr,st}}} (1 + 1.1 \eta_m)$$

$$\sigma_{max} = \frac{646 \cdot 10^3}{7.40 \cdot 10^{-3}} + \frac{896.3 \cdot 10^3 \cdot 117.2 \cdot 10^{-3} \cdot 67 \cdot 10^{-3}}{54.66 \cdot 10^6} \cdot \frac{1}{1 - \frac{896.3 \cdot 10^3}{28322 \cdot 10^3}} (1 + 1.1 \cdot 12.6)$$

$$\sigma_{max} = 87.3 \cdot 10^6 + 12876155 \cdot (1.03)(14) = 273 \cdot 10^6 \text{ N/m}^2 > \frac{235}{1.1} = 213 \cdot 10^6 \text{ N/m}^2$$

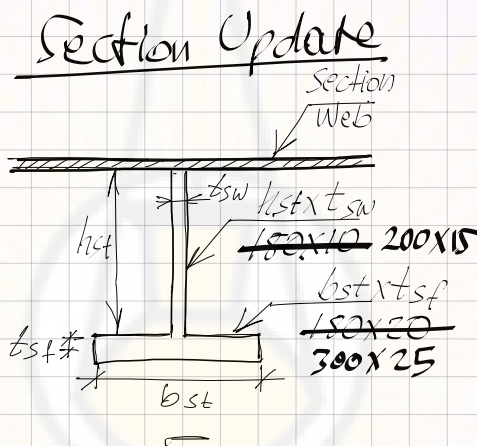
Fail.

Many Alternatives are available to overcome this failure.

The most effective solution is to increase $b_{st}, h_{st}, t_{sfw}, t_{sf}$ from $180 \times 180 \times 10 \times 20$ to $(200 \times 300 \times 15 \times 25)$

$$I_{st} = \text{Change from } 54,66 \cdot 10^6 \text{ m}^4 \rightarrow I_s = 80,3 \cdot 10^6 \text{ m}^4$$

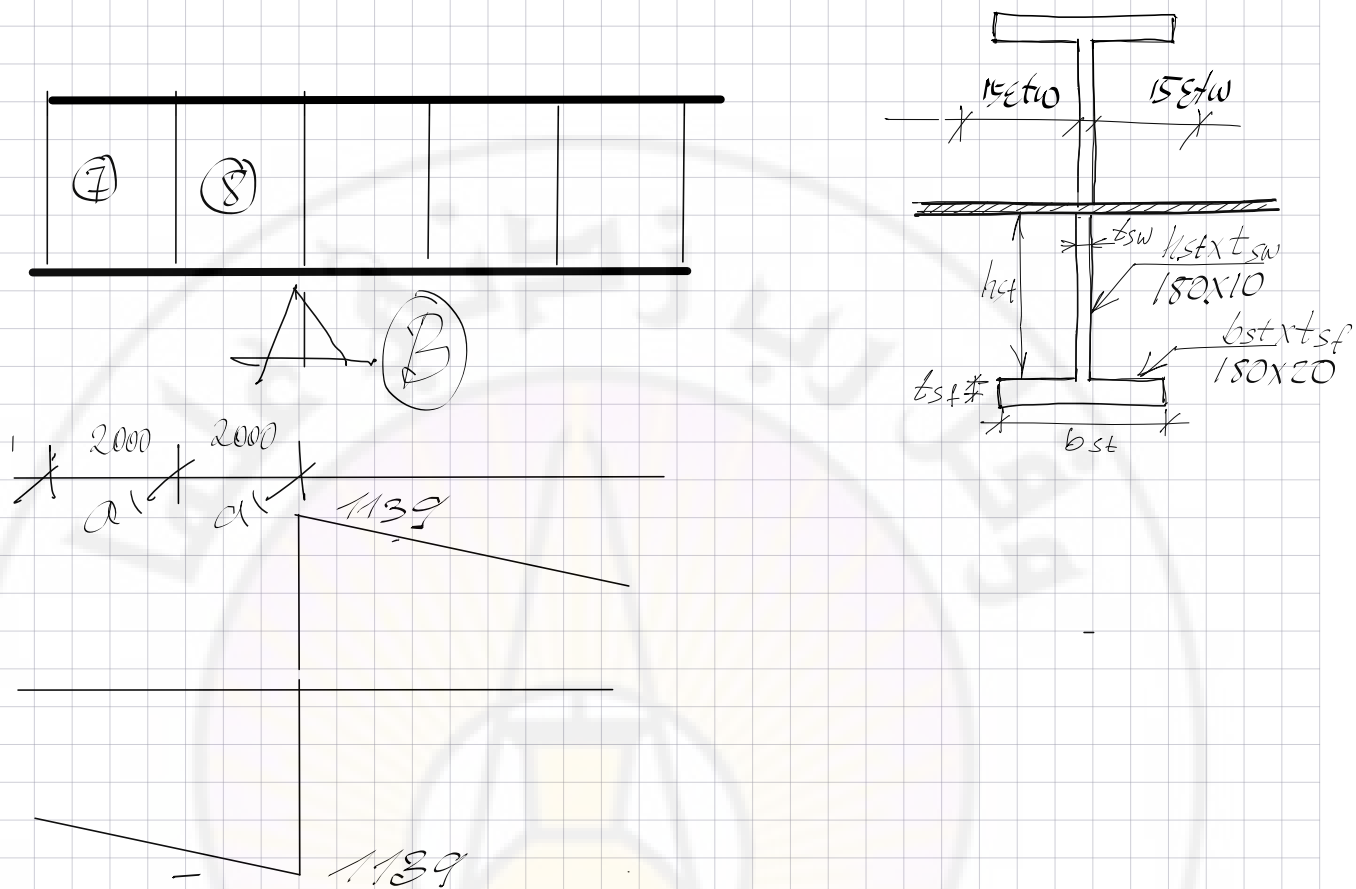
$$\sigma_{\max} = 87,3 \cdot 10^6 + 125,6 \cdot 10^6 = \left[\cdot 10^6 \text{ N/m}^2 > \frac{235}{1,1} = 213 \cdot 10^6 \text{ N/m}^2 \right]$$



$$w = w_0 \frac{1}{\frac{N_{cr,st}}{\sum N_{st,Ed}} - 1} (1 + 1,25 \eta_w) \leq \frac{h_w}{300}$$

$$= 6,7 \cdot 10^{-3} \frac{1}{\frac{28322 \cdot 10^3}{896,3 \cdot 10^3} - 1} (1 + 1,25 \cdot 12,6) = 3,66 \cdot 10^{-3} \text{ m} \leq \frac{200}{300} = 6,7 \cdot 10^{-3} \text{ m}$$

Vertical Stiffener above the interior support



$$B_{Ed} = 2 * 1139 = 2278 \cdot 10^3 \text{ N}$$

$$A_{stB} = (30 * t_w + t_w) + 2(t_{sth} b_{st} + t_{sth} h_{ct})$$

$$= (30 * 1 * 0.008 + 0.01) 0.008 + 2 * (0.02 * 0.18 + 0.01 * 0.2)$$

$$A_{stB} = 12.8 \cdot 10^{-3} \text{ m}^2$$

$$\sigma' = \frac{B_{Ed}}{A_{stB}} = \frac{2278 * 10^3}{12.8 * 10^{-3}} = 175 * 10^6 \text{ N/m}^2 < \frac{f_y}{\gamma_{M0}} = \frac{235 * 10^6}{1.0} = 235 \cdot 10^6 \text{ N/m}^2$$

Buckling length of the stiffener:

$$l_a = 0.75 l_w = 0.75 * 200 = 1.5 \text{ m}$$

Verification of buckling design resistance of column.

$$I_{st,B} = \frac{b_{st}(2h_{st} + 2t_{st} + t_w)^3}{12} - \frac{(b_{st} - t_{st,w})(2h_{st} + t_w)^3}{12}$$

$$I_{st,B} = \frac{0,18(2 \times 0,18 + 2 \times 0,02 + 0,008)^3}{12} - \frac{(0,18 - 0,01)(2 \times 0,18 + 0,008)^3}{12}$$
$$= 31275 \times 10^{-6} \text{ m}^4$$

$$i_{st,B} = \sqrt{\frac{I_{st,B}}{A_{st,B}}} = \sqrt{\frac{31275 \times 10^{-6}}{12,8 \times 10^{-3}}} = 1563 \cdot 10^{-3} \text{ m}$$

$$\lambda = \frac{l_u}{i_{st}} = \frac{1,50}{0,1563} = 9,597$$

$$\lambda_1 = 93,92 = 93,9$$

$$\bar{\lambda} = \frac{\lambda}{\lambda_1} = \frac{9,597}{93,9} = 0,102 < 0,2$$
$$\Rightarrow \chi = 1$$

$$N_{b,Rd} = \chi A_{st,B} \cdot \frac{f_y}{\gamma_{M1}}$$

$$= 1 \times 12,8 \times 10^{-3} \cdot \frac{235 \times 10^6}{1,1}$$
$$= 2734 \times 10^3 \text{ N}$$

$$N_{Ed} = B_{Ed} = 2278 \cdot 10^3 \text{ N} \leq N_{b,Rd} = 2734 \cdot 10^3 \text{ N}$$

The background of the slide features a large, faint watermark of the Damascus University logo. The logo is circular, with the university's name in Arabic at the top and 'Damascus University' in English at the bottom. In the center is a stylized emblem with a yellow and orange sun-like shape and a green triangle.

The End

Dr.-Eng. Moaid Subh
Email: Subheducation@gmail.com

Plate Girder Working Example

According to: Eurocode 3 Design of Steel Structures, Part 4.5 Plated Structural Elements



native piling, by: KARE DESIGN

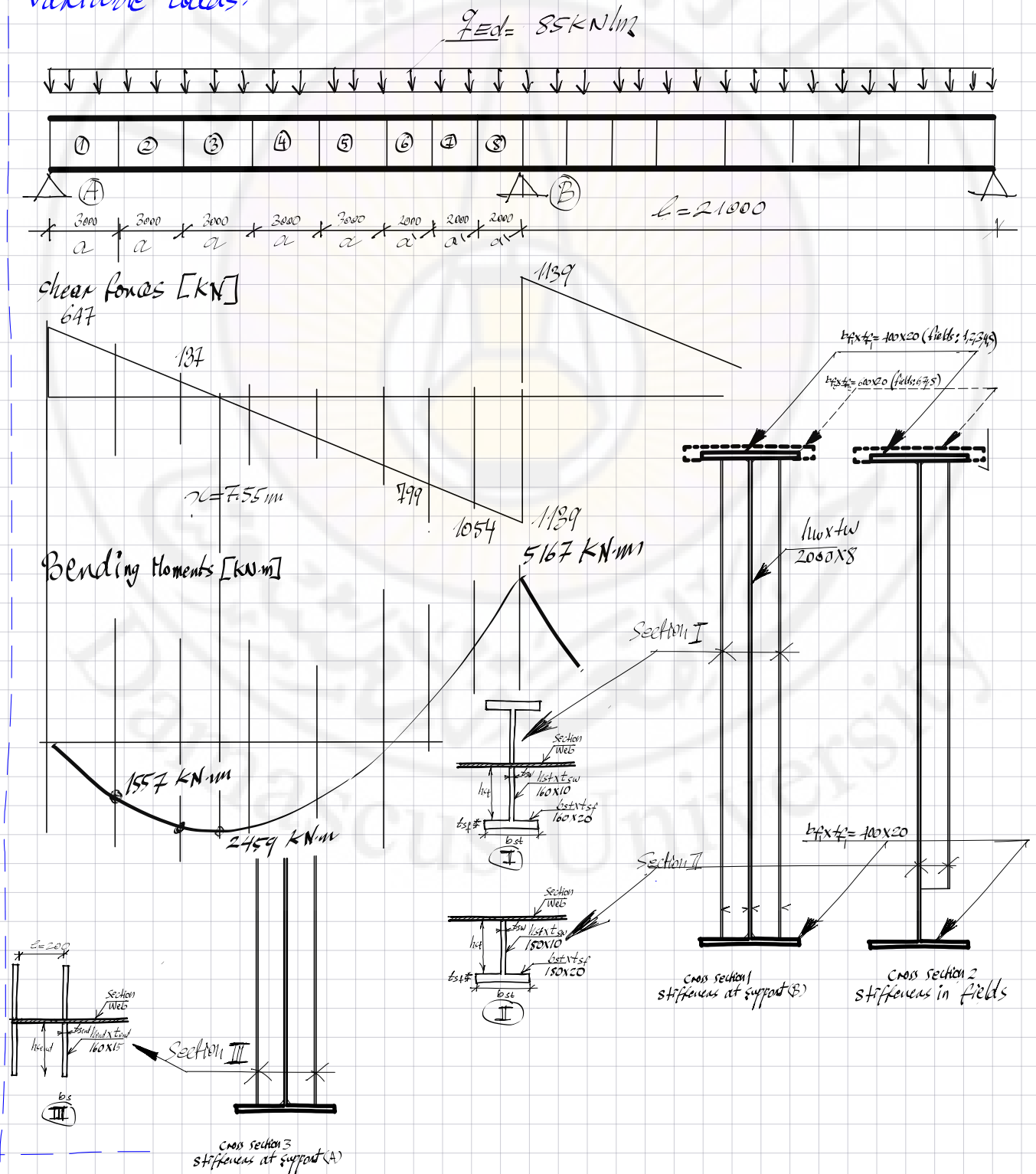


References:

- [1] Eurocode 3 EN 1993-1-5: Design of steel structures, Part 1.5 Plated structural elements,
- [2] Eurocode EN 1990:2003: Basic of structural Design
- [3] Eurocode 3 EN 1991-1:2004: Design of steel structures, Part 1.1 General rules and rules for buildings.
- [4] Eurocode 3 EN 1993-2-2004: Design of steel structures, Part 2 Bridges.
- [5] Johansson, Magno, R., Sedlacek G., New design rules for Plated structures in Eurocode 3, Journal of Constructional Steel Research 57, 2001,
- [5] B. Johansson, R. Magno, G. Sedlacek, C. Müller, D. Beg
Commentary and worked Examples To EN 1993-1-5 "Plated structural Elements. ECCS Publications, EUR 22898 EN 2007.
- [9] Danko Beg et. al. "Design of Plated structures Eurocode 3: Design of steel structures: Part 1-5. Design of plated structures", ECCS and Ernst & Sohn, 2010.

Problem 2

A two span continuous Plate girder Example is simplified to illustrate to explain the design rules for Vertical Stiffeners only [9]. The example is not intended to show an optimal girder design but to illustrate the application of Vertical stiffener design rules. The design loading q_{Ed} include self-weight of the girder and all other relevant permanent and variable loads.



The Design

① Field (1) $V_{ed} = 647 \cdot 10^3 \text{ N}$, $M_{ed, \max(1)} = 1557 \cdot 10^3 \text{ N.m}$

Interaction M-V will be checked with maximum value for shear and bending moment existing in the panel, but not located in the same cross section (this is a conservative approach).

1.1: Bending Moment Verification

$$M_{f, Rd} = (h_w + t_{f1}) A_{f, min} \cdot \frac{f_y}{\gamma_{M0}}$$

$$= (200 + 9.02) (940 \times 9.02) \frac{235 \times 10^6}{10}$$

$$= 3798 \cdot 10^3 \text{ N.m}$$

$$M_{ed} = 1557 \cdot 10^3 \text{ N.m} \leq M_{f, Rd} = 3798 \cdot 10^3 \text{ N.m}$$

The direct stresses may thus be transferred by the flanges only

1.2 Shear Resistance

The end Post is designed to be rigid. Therefore it must fulfil the following criteria:

$$e \geq 0.1 h_w = 0.1 \times 20 = 2 \text{ m}$$

choose: $e = 20 \text{ cm}$

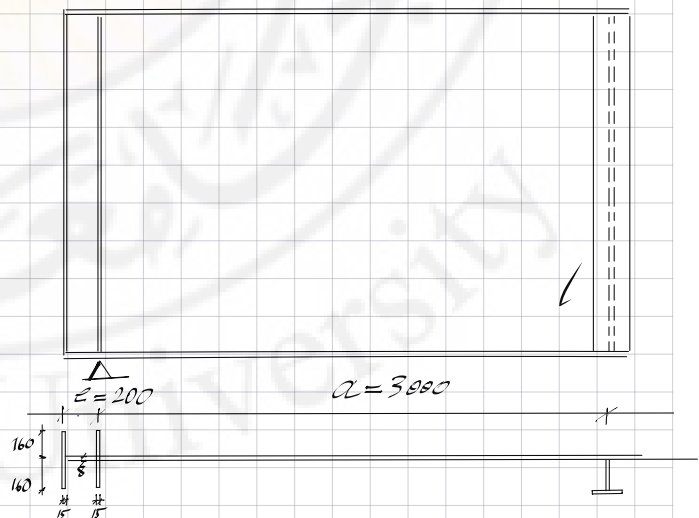
end Plate dimension

$$\frac{b_{end}}{t_{end}} = \frac{320}{15} \quad \text{double sided}$$

$$A_{end} = 2 \times 0.16 \times 0.015 + 0.008 \times 12$$

$$= 4.92 \cdot 10^{-3} \text{ m}^2 > 4 \frac{h_w t_w^2}{e} = 4 \frac{200 \times 0.008^2}{9.2} = 2.56 \cdot 10^{-3} \text{ m}^2 \quad \dots \quad \text{OK}$$

$$\frac{b_{end}}{t_{end}} = \frac{0.160}{0.015} = 10.7 < 14 \varepsilon = 14 \sqrt{\frac{235}{235}} = 14 \dots \text{class 3, therefore fully effective.}$$



Susceptibility to buckling

Stiffened Webs: susceptible to buckling

$$\frac{h_w}{t_w} > 31 \frac{\epsilon}{\gamma} \sqrt{k_\tau}$$

$$\frac{\alpha}{h_w} = \frac{300}{200} = 1.5 \Rightarrow k_\tau = 5.34 + 4 \left(\frac{h_w}{\alpha} \right)^2$$

$$= 5.34 + 4 \left(\frac{200}{300} \right)^2 = 7.118$$

$$\frac{h_w}{t_w} = \frac{200}{0.008} = 250 > \frac{31}{1.2} \cdot 1 \cdot \sqrt{7.118} = 69$$

Verification of shear buckling is necessary.

$$\bar{\lambda}_w = \frac{h_w}{37.4 t_w - \epsilon \cdot \sqrt{k_\tau}} = \frac{200}{37.4 \times 0.008 \times 1 \cdot \sqrt{7.118}} = 2.505 > 1.08$$

$$\Rightarrow \lambda = \frac{1.37}{0.7 + \bar{\lambda}_w} = \frac{1.37}{0.7 + 2.505} = 0.427$$

Shear Resistance: Contribution from Web:

$$V_{bw,Rd} = \frac{\tau_w \cdot f_{yw} \cdot h_w \cdot t_w}{\gamma_{M1} \cdot \sqrt{3}} = \frac{0.427 \times 235 \times 10^6 \times 200 \times 0.008}{1.1 \times \sqrt{3}}$$

$$= 843 \cdot 10^3 \text{ N}$$

Shear Resistance: Contribution from flanges:

$$C = \alpha \left(0.25 + \frac{1.6 b_{f1} \cdot t_{f1}^2 \cdot f_{yf}}{t_w \cdot h_w^2 \cdot f_{yw}} \right)$$

$$= 300 \left(0.25 + \frac{1.6 \times 0.40 \times 0.02^2 \times 235 \times 10^6}{0.008 \times 200^2 \times 235 \times 10^6} \right)$$

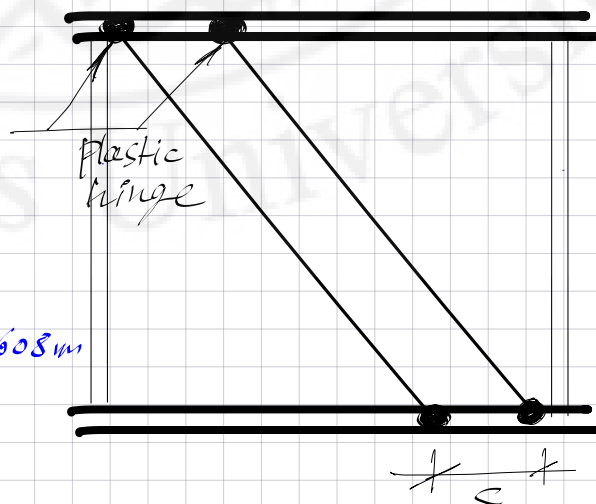
$$= 0.774$$

$$b_f = 0.40 \text{ m} \leq 2 \times 15 \times t_f + t_w$$

$$< 2 \times 15 \times 1 \times 0.02 + 0.008 = 0.608 \text{ m}$$

$$V_{bf,Rd} = \frac{b_{f1} \cdot t_{f1}^2 \cdot f_{yf}}{C \cdot \gamma_{M1}} \left[1 - \left(\frac{M_{Ed}}{M_{f,Rd}} \right)^2 \right]$$

$$V_{bf,Rd} = \frac{0.40 \times 0.02^2 \times 235 \times 10^6}{0.774 \times 1.1} \left[1 - \left(\frac{1557 \cdot 10^3}{3798 \cdot 10^3} \right)^2 \right] = 37 \cdot 10^3 \text{ N}$$



Flanges do not significantly contribute to the shear resistance; indeed $V_{bf,Rd}$ is substantially smaller than $V_{bw,Rd}$.

Verification for shear resistance

$$V_{b,Rd} = V_{bw,Rd} + V_{bf,Rd} \leq \frac{1}{\gamma_{M1}} \frac{f_{yw} \cdot h_w \cdot t_w}{\sqrt{3}}$$
$$= (843 + 37) \cdot 10^3 = 880 \cdot 10^3 \text{ N} \leq \frac{1.2 \cdot 235 \cdot 24 \cdot 0.005}{1.1 \cdot \sqrt{3}} \cdot 10^6 = 2365 \cdot 10^3 \text{ N}$$
$$V_{Ed} = 647 \cdot 10^3 \text{ N} \leq V_{b,Rd} = 880 \cdot 10^3 \text{ N}$$

✓ Verification of Interaction M-V

Flanges take bending moment and the Web takes shear forces. Interaction was guaranteed via contribution the flanges is shear resistance

② Field (3) $V_{Ed} = 137 \cdot 10^3 \text{ N}$, $M_{Ed, \max(3)} = 2459 \cdot 10^3 \text{ N.m}$
at the mid span

2.1: Verification of bending resistance:

$$M_{f,Rd} = 3798 \cdot 10^3 \text{ N}, \text{ is calculated in field (1)}$$

$$M_{Ed} = 2459 \cdot 10^3 \text{ N.m} \leq M_{f,Rd} = 3798 \cdot 10^3 \text{ N.m} \rightarrow \text{OK}$$

The direct stresses may thus be transferred by the flanges only.

2.2: Verification of shear resistance

Panel (3) is the same as panel (1) and has the same shear resistance of the Web. Neglecting the contribution from flanges because of high moments gives:

$$V_{Ed} = 137 \cdot 10^3 \text{ N} \leq V_{bw,Rd} = 843 \cdot 10^3 \text{ N} \rightarrow \text{OK}$$

2.3: Verification of Interaction

Flanges take bending moment and the Web take shear force. There is no need for a verification of the M-V Interaction

③ Stiffeners Verification

3.1 Intermediate transverse Stiffeners [Fields (7), (8)]

The transverse stiffeners are checked for the most severe conditions at the location close to the interior support. It can be easily shown that the same cross section fulfils the relevant requirements also in midspan region.

One-sided T-stiffeners

T-section of stiffener (Section A)

$h_{st}/b_{st}/t_{stw}/t_{stf} = 180/180/10/20$
is class (3).

$$A_{st} = h_{st} t_{stw} + b_{st} t_{stf}$$

$$= 0,18 \times 0,01 + 0,18 \times 0,02 = 5,40 \cdot 10^{-3} \text{ m}^2$$

$$e_1 = \frac{\frac{h_{st}^2}{2} \cdot t_{stw} + b_{st} \cdot t_{stf} \left(h_{st} + \frac{t_{stf}}{2} \right)}{A_{st}} = \frac{\frac{0,18^2}{2} \times 0,01 + 0,18 \times 0,02 \left(0,18 + \frac{0,02}{2} \right)}{5,40 \times 10^{-3}} = 156,7 \cdot 10^{-3} \text{ m}$$

$$e_2 = h_{st} + t_{stf} - e_1 = 0,18 + 0,02 - 156,7 \cdot 10^{-3} = 43,3 \cdot 10^{-3} \text{ m}$$

Double T-section
of stiffeners (Section A')

$$A_{st}' = A_{st} + (2 \times 15 \varepsilon t_w + t_{stw}) t_w$$

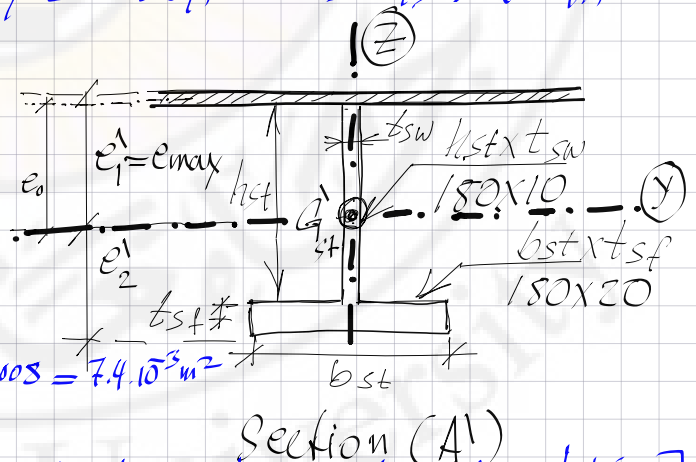
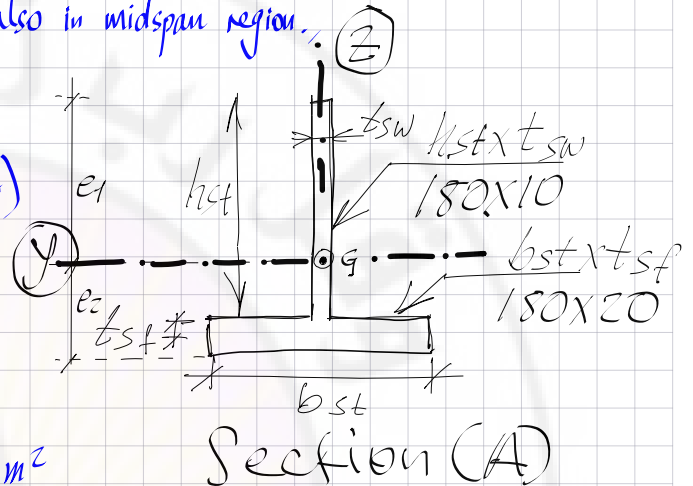
$$= 5,4 \cdot 10^{-3} + (2 \times 15 \times 1,0 \times 0,008 + 0,01) \times 0,005 = 7,4 \cdot 10^{-3} \text{ m}^2$$

$$e_1' = \frac{1}{A_{st}'} \left[(2 \times 15 \varepsilon t_w + t_{stw}) \frac{t_w^3}{2} + h_{st} \cdot t_{stw} \left(t_w + \frac{h_{st}}{2} \right) + b_{st} t_{stf} \left(t_w + h_{st} + \frac{t_{stf}}{2} \right) \right]$$

$$= \frac{1}{7,4 \times 10^{-3}} \left[(2 \times 15 \times 1 \times 0,005 + 0,01) \frac{0,008^2}{2} + 0,18 \times 0,01 \left(0,008 + \frac{0,18}{2} \right) + 0,18 \times 0,02 \left(0,005 + 0,18 + \frac{0,02}{2} \right) \right]$$

$$e_1' = 121,2 \cdot 10^{-3} \text{ m}$$

$$e_2' = t_w + h_{st} + t_{stf} = 0,005 + 0,18 + 0,02 - 121,2 \cdot 10^{-3} = 86,8 \cdot 10^{-3} \text{ m}$$



$$I_{st} = \frac{(2 \times 15 \times t_w + t_{st,w}) t_w^3}{12} + \frac{h_{st}^3 t_{st,w}}{12} + \frac{b_{st,w} t_{st,f}^3}{12} + (2 \times 15 \times t_w + t_{st,w}) t_w \left(e_1 - \frac{t_w}{2} \right)^2 + h_{st} t_{st,w} \left(e_1 - \frac{h_{st}}{2} - t_w \right)^2 + b_{st} t_{st,f} \left(e_2 - \frac{t_{st,f}}{2} \right)^2$$

$$I_{st} = \frac{(2 \times 15 \times 1 \times 0,008 + 0,01) \times 0,008^3}{12} + \frac{0,18^3 \times 0,01}{12} + \frac{0,18 \times 0,02^3}{12} + (2 \times 15 \times 1 \times 0,008 + 0,01) \times 0,008 \left(12,2 \times 10^{-3} - \frac{0,008}{2} \right)^2 + 0,18 \times 0,01 \left(12,2 \times 10^{-3} - \frac{0,18}{2} - 0,008 \right)^2 + 0,18 \times 0,02 \left(86,8 \times 10^{-3} - \frac{0,02}{2} \right)^2$$

$$I_{st} = 54,66 \cdot 10^{-6} \text{ m}^4$$

Torsion Buckling

$$\sigma_{cr} = \frac{1}{I_p} \left(G I_t + \frac{\pi^2 E I_w}{l_w^2} \right) \geq 6 \cdot f_y$$

$$I_p = I_y + I_z$$

Second moments of areas around the point of stiffener to Web junction

$$I_y = \frac{h_{st}^3 t_{st,w}}{12} + \frac{b_{st} t_{st,f}^3}{12} + h_{st} t_{st,w} \left(e_1 - \frac{h_{st}}{2} \right)^2 + b_{st} t_{st,f} \left(e_2 - \frac{t_{st,f}}{2} \right)^2 + A_{st} \cdot e_1^2$$

$$I_y = \frac{0,18^3 \times 0,01}{12} + \frac{0,18 \times 0,02^3}{12} + 0,18 \times 0,01 \left(156,7 \times 10^{-3} - \frac{0,18}{2} \right)^2 + 0,18 \times 0,02 \left(43,3 \times 10^{-3} - \frac{0,02}{2} \right)^2 + 5,40 \times 10^{-3} \times (156,7 \times 10^{-3})^2 = 149,58 \cdot 10^{-6} \text{ m}^4$$

$$I_z = \frac{h_{st} \cdot t_{st,w}^3}{12} + \frac{b_{st}^3 \cdot h_{st,f}}{12} = \frac{0,18 \times 0,01^3}{12} + \frac{0,18 \times 0,02}{12} = 9,74 \cdot 10^{-6} \text{ m}^4$$

$$I_p = I_y + I_z = 149,58 \cdot 10^{-6} + 9,74 \cdot 10^{-6} = 159,32 \cdot 10^{-6} \text{ m}^4$$

$$I_w = \frac{b_{st}^3 \cdot t_{st,f}}{12} \left(h_{st} + \frac{t_{st,f}}{2} \right) = \frac{0,18^3 \times 0,02}{12} \left(0,18 + \frac{0,02}{2} \right) = 350,892 \cdot 10^{-9} \text{ m}^6$$

$$I_t = \frac{1}{3} (h_{st} \cdot t_{st,w}^3 + b_{st} \cdot t_{st,f}^3) = \frac{1}{3} (0,18 \times 0,01^3 + 0,18 \times 0,02^3) = 540 \cdot 10^{-9} \text{ m}^4$$

$$\sigma_{cr} = \frac{1}{159,32 \cdot 10^{-6}} \left(80,77 \cdot 10^9 \times 540 \times 10^{-9} + \frac{\pi^2 \times 210 \cdot 10^9 \times 350,892 \times 10^9}{(200)^2} \right) = 1415 \cdot 10^6 \text{ N/m}^2$$

$$\sigma_{cr} = 1415 \cdot 10^6 \text{ N/m}^2 > 6 \times 235 \cdot 10^6 \text{ N/m}^2 \text{ ---- OK}$$

The check of torsional buckling is decisive in this case (see the subsequent checks). However, axial forces in transverse stiffener are usually small and the check $\sigma_{cr} > 6 f_y$ is usually very conservative.

To overcome this problem the limiting slenderness may be calculated with the maximum design compressive stress $\sigma_{com,Ed}$ in the stiffener $\rightarrow$

replacing f_y . In this case the torsional buckling check change to:

$$\sigma_{cr} \geq 6 \cdot \sigma_{comp,Ed} \cdot \sigma_{M1}$$

Minimum requirements for transverse stiffeners (shear)

$$\frac{a'}{h_w} = \frac{200}{200} = 1 < \sqrt{2} \Rightarrow$$

$$I_{st} = 54,66 \times 10^{-6} \text{ m}^4 \geq \frac{1,5 h_w^3 t_w^3}{a'^2} = \frac{1,5 \times 200^3 \times 0,008^3}{200^2} = 1,536 \cdot 10^{-6} \text{ m}^4$$

Transverse stiffeners act as rigid supports for web panel.

Additional compression axial force in transverse stiffener from the tension field action.

$V_{Ed} = 1054 \cdot 10^3 \text{ N}$ (0,5 h_w from the edge of the panel
 $a' \approx 200 \text{ mm}$)

slenderness $\bar{\lambda}_w$ for shear

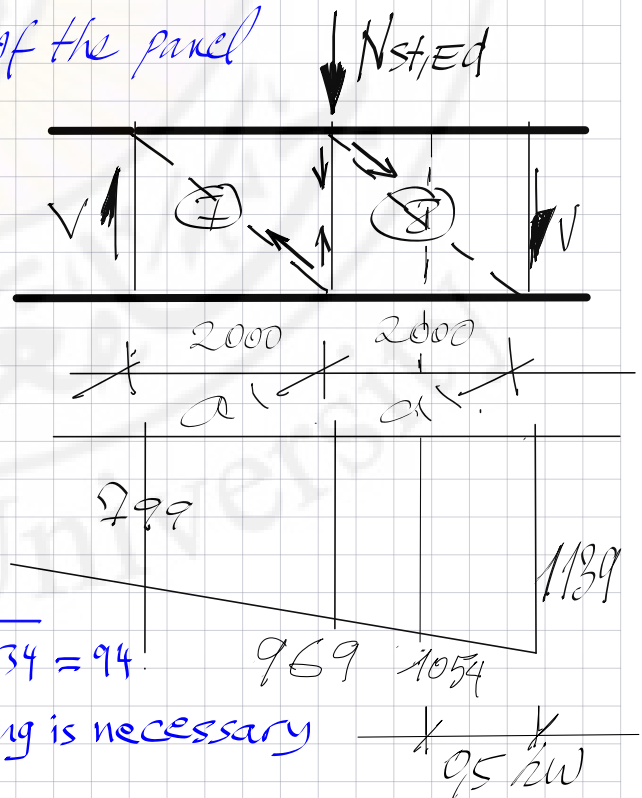
$$\bar{\lambda}_w = \frac{h_w}{37,4 t_w \cdot \varepsilon \sqrt{k_\tau}}$$

$$\frac{a}{h_w} = \frac{200}{200} = 1 \rightarrow k_\tau = 5,34 + 4 \left(\frac{h_w}{a} \right)^2 = 5,34 + 4 \left(\frac{200}{200} \right)^2 = 9,34$$

$$\frac{h_w}{t_w} = \frac{200}{0,008} = 250 > \frac{31}{1} \varepsilon \sqrt{k_\tau} = \frac{31}{1} \cdot 1 \cdot \sqrt{9,34} = 94$$

Verification of shear buckling is necessary

$$\bar{\lambda}_w = \frac{h_w}{37,4 t_w \cdot \varepsilon \sqrt{k_\tau}} = \frac{200}{37,4 \times 0,008 \cdot \sqrt{9,34}} = 2,2$$



Intermediate rigid stiffeners may be designed for an axial force equal to $N_{st,Ed}$ according to EN 1993-1-5, 9.3.3 (3).

In the case of variable shear force the check is performed for the distance $0,9 h_w$ from the edge of the panel with the largest shear force.

$$N_{st,Ed} = V_{Ed} - \frac{h_w \cdot t_w \cdot f_{yw}}{\tau_w^2 \cdot \alpha_{M1} \cdot \sqrt{3}}$$

$$= 1054 \times 10^3 - \frac{2100 \times 9,008 \times 235 \times 10^6}{21,2^2 \times 1,1 \times \sqrt{3}} =$$

$$N_{st,Ed} = 1054 \times 10^3 - 408 \times 10^3 = 646 \times 10^3 \text{ N}$$

Axial force from the deviation forces:

$$\Delta N_{st,Ed} = \frac{\sigma_m h_w^3}{\tau^2}$$

$$\sigma_m = \frac{\sigma_{cr,c}}{\sigma_{cr,p}} \cdot \frac{N_{Ed}}{h_w} \left(\frac{1}{a_1} + \frac{1}{a_2} \right)$$

suppose $\frac{\sigma_{cr,c}}{\sigma_{cr,p}} = 1$

$$\sigma_{c,max} = \sigma_{Ed,max} = \eta_1 \frac{f_y}{\alpha_{M0}}$$

$$\eta_1 = \frac{M_{Ed}}{M_{Rd}}$$

$$M_{Rd} = \frac{f_y}{\alpha_{M0}} \cdot W_{eff}$$

suppose $W_{eff} = \alpha \cdot W_{elastic} = 24185 \cdot 10^6 \text{ mm}^3$

$$M_{Rd} = 235 \times 10^6 / 1,0 \times 24185 \times 10^6 = 5683 \cdot 10^3 \text{ N}\cdot\text{m}$$

$$\eta_1 = \frac{5167 \cdot 10^3}{5683 \cdot 10^3} = 0,909 < 1 \rightarrow \text{ok}$$

$$\sigma_{Ed, \max} = \eta_1 \frac{f_y}{\gamma_{M0}} = 0,909 \frac{235 \cdot 10^6}{1,0} = 214 \cdot 10^6 \text{ N/m}^2$$

$$\sigma_{c, \max} = \sigma_{Ed, \max} = 214 \cdot 10^6 \text{ N/m}^2$$

$$N_{Ed} = \sigma_{c, \max} \cdot \frac{A_{eff}}{2}$$

$$\text{suppose } A_{eff} = 11,561 \cdot 10^3 \text{ N}$$

$$N_{Ed} = 214 \cdot 10^6 \frac{11,561 \cdot 10^3}{2} = 1237 \cdot 10^3 \text{ N}$$

$$\sigma_m = 1 \cdot \frac{1237 \cdot 10^3}{200} \left(\frac{1}{20} + \frac{1}{20} \right) = 617,5 \cdot 10^3 \text{ N/m}^2$$

$$\Delta N_{st, Ed} = \frac{\sigma_m \cdot h_w^2}{\pi^2} = \frac{617,5 \cdot 10^3 \cdot 200}{\pi^2}$$

$$= 250,3 \cdot 10^3 \text{ N}$$

Strength and stiffness check of the stiffener at ULS

$$\Sigma N_{st, Ed} = N_{st, Ed} + \Delta N_{st, Ed}$$

$$= 646 \cdot 10^3 + 250,3 \cdot 10^3$$

$$= 896,3 \cdot 10^3 \text{ N}$$

Single sided transverse stiffeners are not treated explicitly in the EN1993-1-5. In this numerical example relevant stress and $\rightarrow$

displacement checks.

$$\sigma_{\max} \leq f_y$$

$$w \leq h_w/300$$

are performed according to the interaction formula given in 9.2.1 of the Commentary to EN1993-1-5. This interaction formula is based on the second order elastic analysis and takes account of the stiffener eccentricity and simultaneous action of the deviation forces and the axial forces from the tension field action.

$$N_{cr,st} = \frac{\pi^2 \cdot E I_{st}}{h_w^2} = \frac{\pi^2 \cdot 210 \cdot 10^9 \cdot 54.66 \cdot 10^6}{(200)^2} = 28322 \cdot 10^3 \text{ N}$$

$$e_{\max} = e'_1 = 121.2 \cdot 10^{-3} \text{ m}$$

$$e_0 = e_{\max} - \frac{t_w}{2} = 121.2 \cdot 10^{-3} - \frac{9.008}{2} = 117.2 \cdot 10^{-3} \text{ m}$$

$$w_0 = \min\left(\frac{a_1}{300}, \frac{a_2}{300}, \frac{h_w}{300}\right) \\ = \min\left(\frac{200}{300}, \frac{200}{300}, \frac{200}{300}\right) = 67 \cdot 10^{-3} \text{ m}$$

$$\eta_m = \frac{N_{st,Ed} \cdot e_0}{\sum N_{st,Ed} \cdot w_0} = \frac{646 \cdot 10^3 \cdot 117.2 \cdot 10^{-3}}{896.3 \cdot 10^3 \cdot 67 \cdot 10^{-3}} = 12.6$$

$$\sigma_{\max} = \frac{N_{st,Ed}}{A_{st}} + \frac{\sum N_{st,Ed} \cdot e_{\max} \cdot w_0}{I_{st}} \cdot \frac{1}{1 - \frac{\sum N_{st,Ed}}{N_{cr,st}}} (1 + 1.1 \eta_m)$$

$$\sigma_{\max} = \frac{646 \cdot 10^3}{7.40 \cdot 10^{-3}} + \frac{896.3 \cdot 10^3 \cdot 117.2 \cdot 10^{-3} \cdot 67 \cdot 10^{-3}}{54.66 \cdot 10^6} \cdot \frac{1}{1 - \frac{896.3 \cdot 10^3}{28322 \cdot 10^3}} (1 + 1.1 \cdot 12.6)$$

$$\sigma_{\max} = 87.3 \cdot 10^6 + 12876155 \cdot (1.03)(14) = 273 \cdot 10^6 \text{ N/m}^2 > \frac{235}{1.1} = 213 \cdot 10^6 \text{ N/m}^2$$

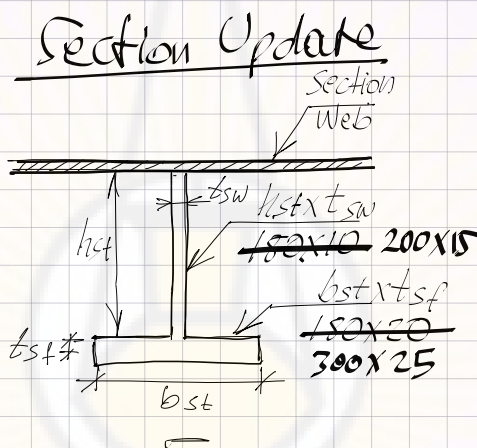
Fail.

Many Alternatives are available to overcome this failure.

The most effective solution is to increase b_{st} , h_{st} , $t_{st,w}$, $t_{st,f}$ from $180 \times 180 \times 10 \times 20$ to $(200 \times 300 \times 15 \times 25)$

$$I_{st} = \text{Change from } 54,66 \cdot 10^6 \text{ m}^4 \rightarrow I_s = 80,3 \cdot 10^6 \text{ m}^4$$

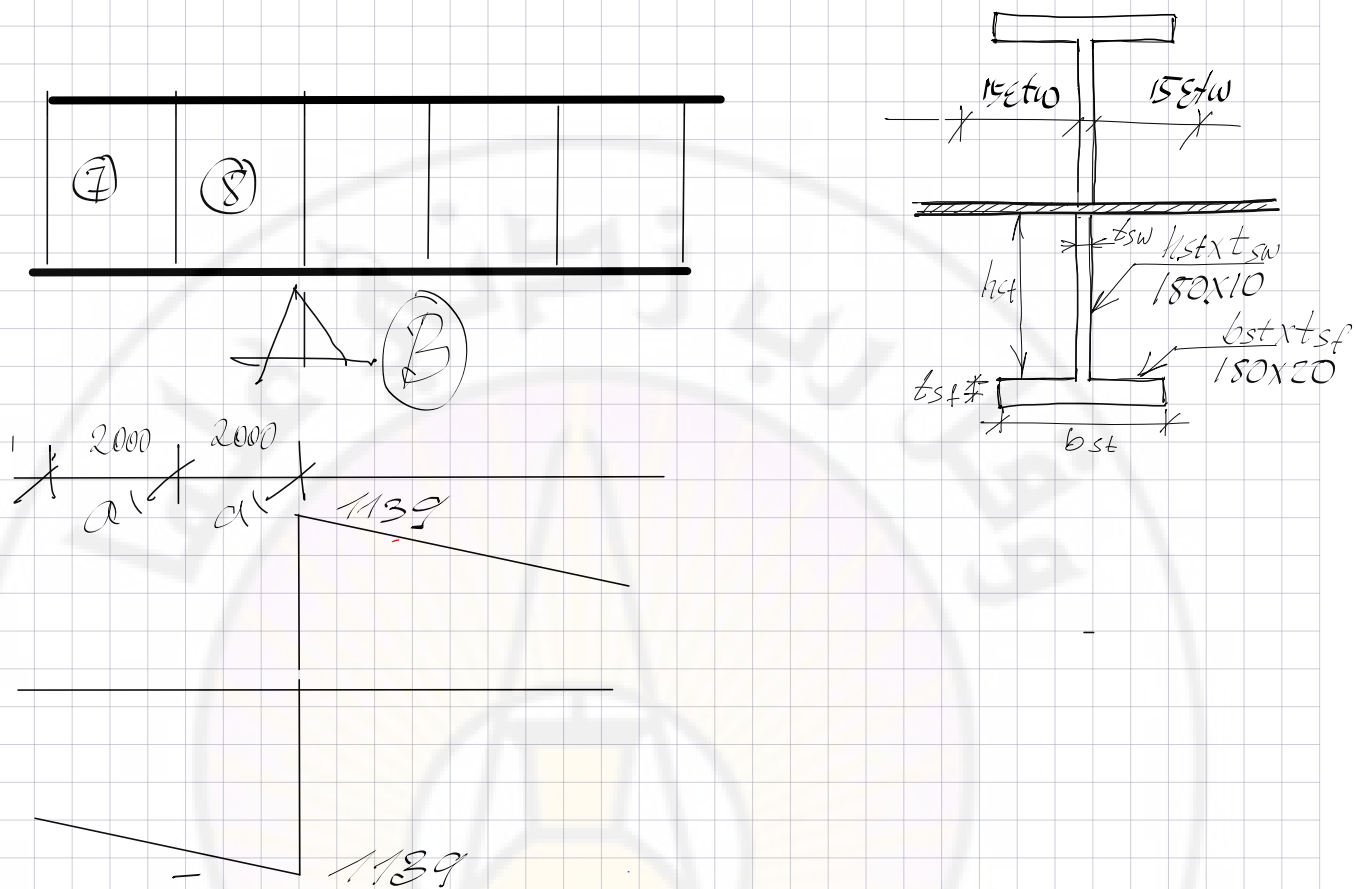
$$\sigma_{\max} = 87,3 \cdot 10^6 + 125,6 \cdot 10^6 = \boxed{213 \cdot 10^6 \text{ N/m}^2 > \frac{235}{1,1} = 213 \cdot 10^6 \text{ N/m}^2}$$



$$w = w_0 \frac{1}{\frac{N_{cr,st}}{\sum N_{st,Ed}} - 1} (1 + 1,25 \eta_w) \leq \frac{h_w}{300}$$

$$= 6,7 \cdot 10^{-3} \frac{1}{\frac{28320 \cdot 10^3}{896,3 \cdot 10^3} - 1} (1 + 1,25 \cdot 12,6) = 3,66 \cdot 10^{-3} \text{ m} \leq \frac{200}{300} = 6,7 \cdot 10^{-3} \text{ m}$$

Vertical stiffener above the interior support



$$B_{Ed} = 2 \times 1139 = 2278 \cdot 10^3 \text{ N}$$

$$A_{stB} = (30 \cdot t_w + t_w) + 2(t_{stf} b_{st} + t_{stw} h_{st})$$

$$= (30 \times 1 \times 0.008 + 0.01) 0.008 + 2 \cdot (0.02 \times 0.18 + 0.01 \times 0.2)$$

$$A_{stB} = 12.8 \cdot 10^{-3} \text{ m}^2$$

$$\sigma' = \frac{B_{Ed}}{A_{stB}} = \frac{2278 \cdot 10^3}{12.8 \cdot 10^{-3}} = 175 \cdot 10^6 \text{ N/m}^2 < \frac{f_y}{\gamma_{M0}} = \frac{235 \cdot 10^6}{1.0} = 235 \cdot 10^6 \text{ N/m}^2$$

Buckling length of the stiffener:

$$l_a = 0.75 l_w = 0.75 \times 200 = 1.5 \text{ m}$$

Verification of buckling design resistance of column.

$$I_{st,B} = \frac{b_{st}(2h_{st} + 2t_{st} + t_w)^3}{12} - \frac{(b_{st} - t_{st,w})(2h_{st} + t_w)^3}{12}$$

$$I_{st,B} = \frac{0,18(2 \cdot 0,18 + 2 \cdot 0,02 + 0,008)^3}{12} - \frac{(0,18 - 0,01)(2 \cdot 0,18 + 0,008)^3}{12}$$
$$= 31275 \cdot 10^{-6} \text{ m}^4$$

$$i_{st,B} = \sqrt{\frac{I_{st,B}}{A_{st,B}}} = \sqrt{\frac{31275 \cdot 10^{-6}}{12,8 \cdot 10^{-3}}} = 1563 \cdot 10^{-3} \text{ m}$$

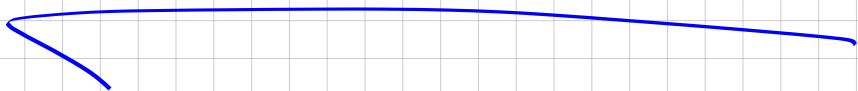
$$\lambda = \frac{l_u}{i_{st}} = \frac{1,50}{0,1563} = 9597$$

$$\lambda_1 = 93,92 = 93,9$$

$$\bar{\lambda} = \frac{\lambda}{\lambda_1} = \frac{9597}{939} = 0,102 < 0,2$$
$$\Rightarrow \chi = 1$$

$$N_{b,Rd} = \chi A_{st,B} \cdot \frac{f_y}{\gamma_{M1}}$$

$$= 1 \cdot 12,8 \cdot 10^{-3} \cdot \frac{235 \cdot 10^6}{1,1}$$
$$= 2734 \cdot 10^3 \text{ N}$$

$$N_{Ed} = B_{Ed} = 2278 \cdot 10^3 \text{ N} \leq N_{b,Rd} = 2734 \cdot 10^3 \text{ N}$$


Problem 3

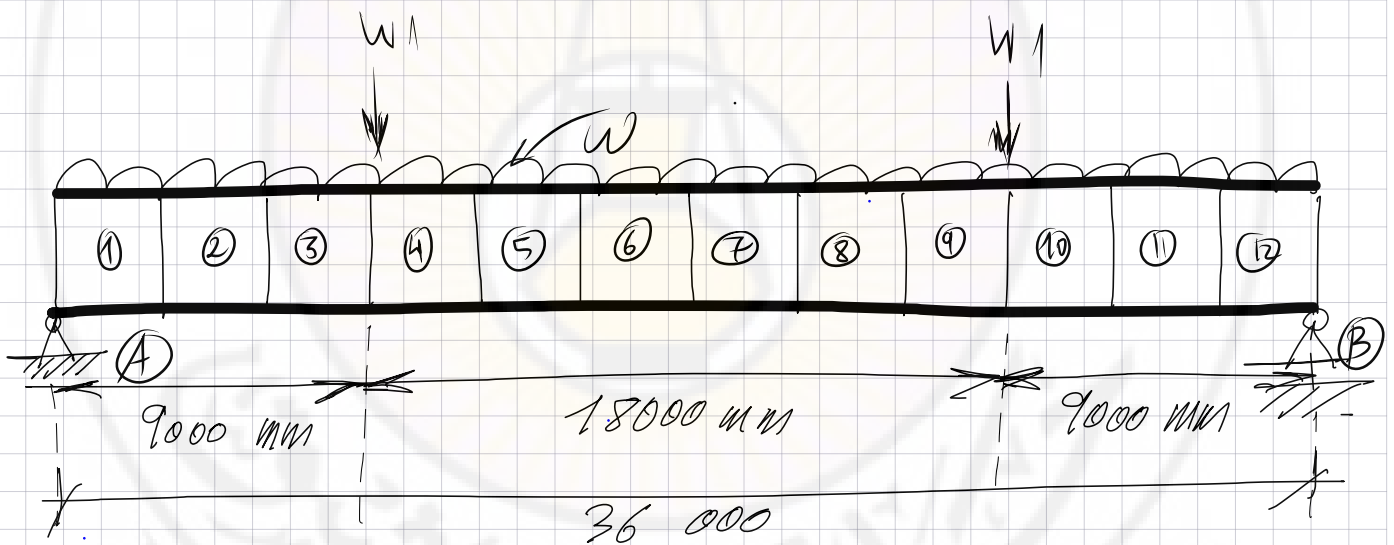
Design a plate girder, shown below, with a span of 36m. The girder is fully restrained against lateral buckling throughout its span and carrying two concentrated loads.

Steel grade: S275

Material factor for steel, $\gamma_m = 1.15$

Dead load factor, $\gamma_{fd} = 1.35$

Imposed load factor, $\gamma_{fi} = 1.5$



1) Loading

Dead load

Uniformly distributed load; (UDL)
including self-weight

$$W_d = 20 \text{ kN/m}$$

Concentrated load 1,

$$W_{1d} = 200 \text{ kN}$$

Concentrated load 2,

$$W_{2d} = 200 \text{ kN}$$

Factored load

$$W_1 = W_d \times \gamma_{fd} + W_{1d} \times \gamma_{fi} = 20 \times 1.35 + 200 \times 1.5 = 329 \text{ kN/m}$$

$$W_1 = W_2 = 200 \times 1.35 + 400 \times 1.5 = 870 \text{ kN}$$

2. Bending Moment and Shear Forces

Reactions

$$R_A = R_B = 870 + 79.5 \times 36/2 = 2301 \text{ kN}$$

shear forces equation:

$$Q_x = 2301 - w \cdot x$$

$$= 2301 - w \cdot x - W_1 \quad \begin{matrix} 0 \leq x < 9.0 \text{ m} \\ 9.0 \leq x < 27 \text{ m} \end{matrix}$$

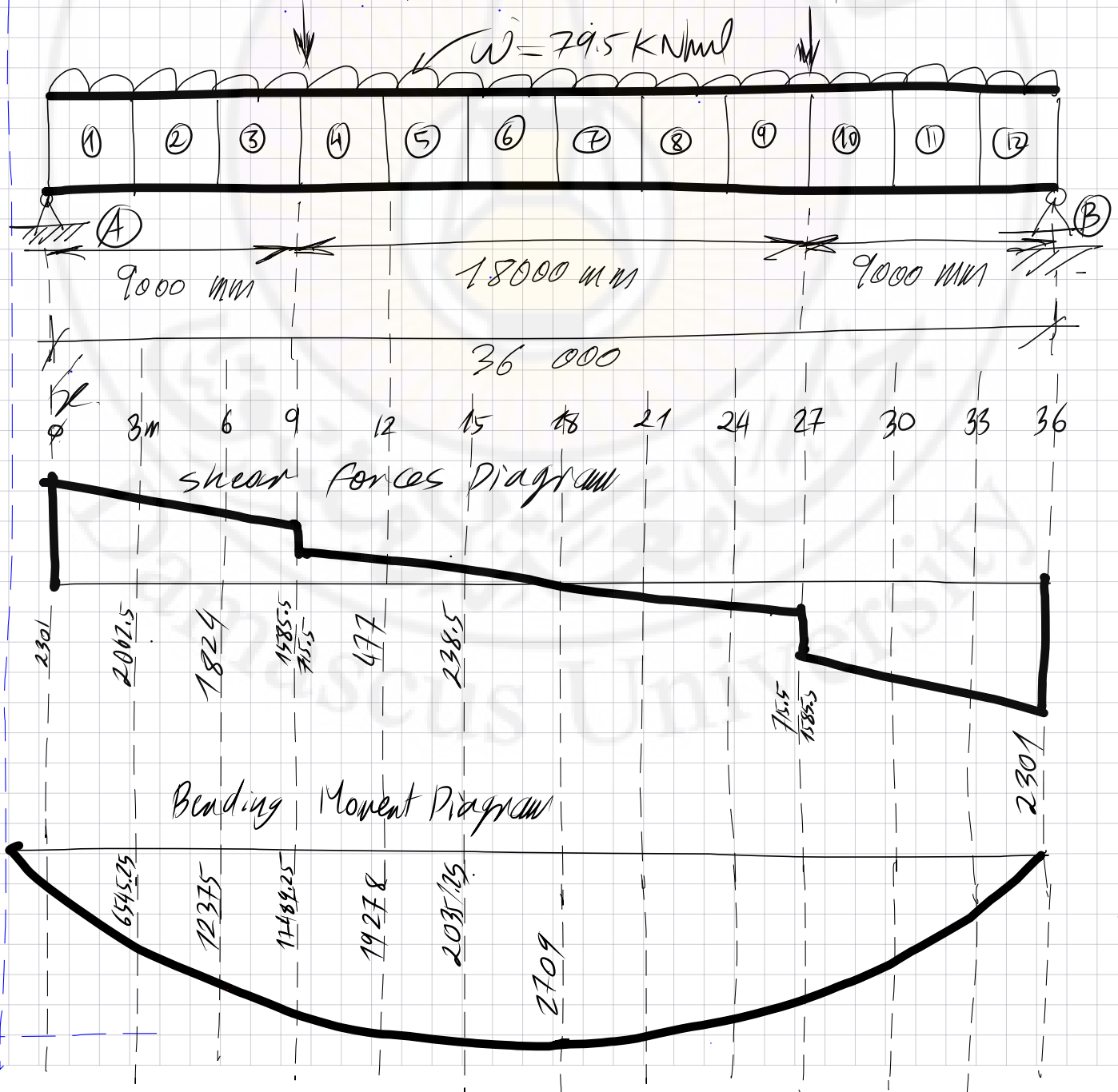
Bending Moment equation:

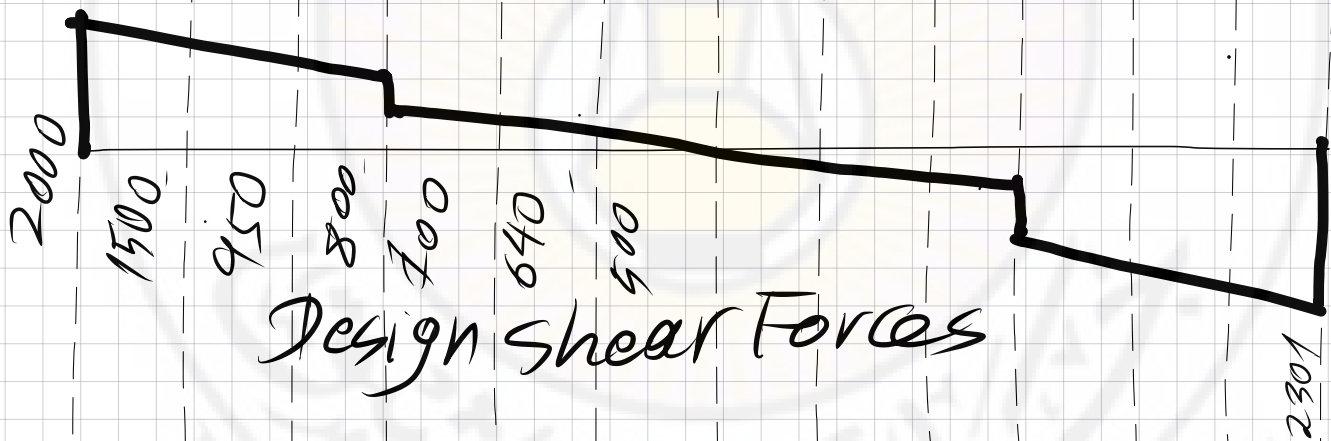
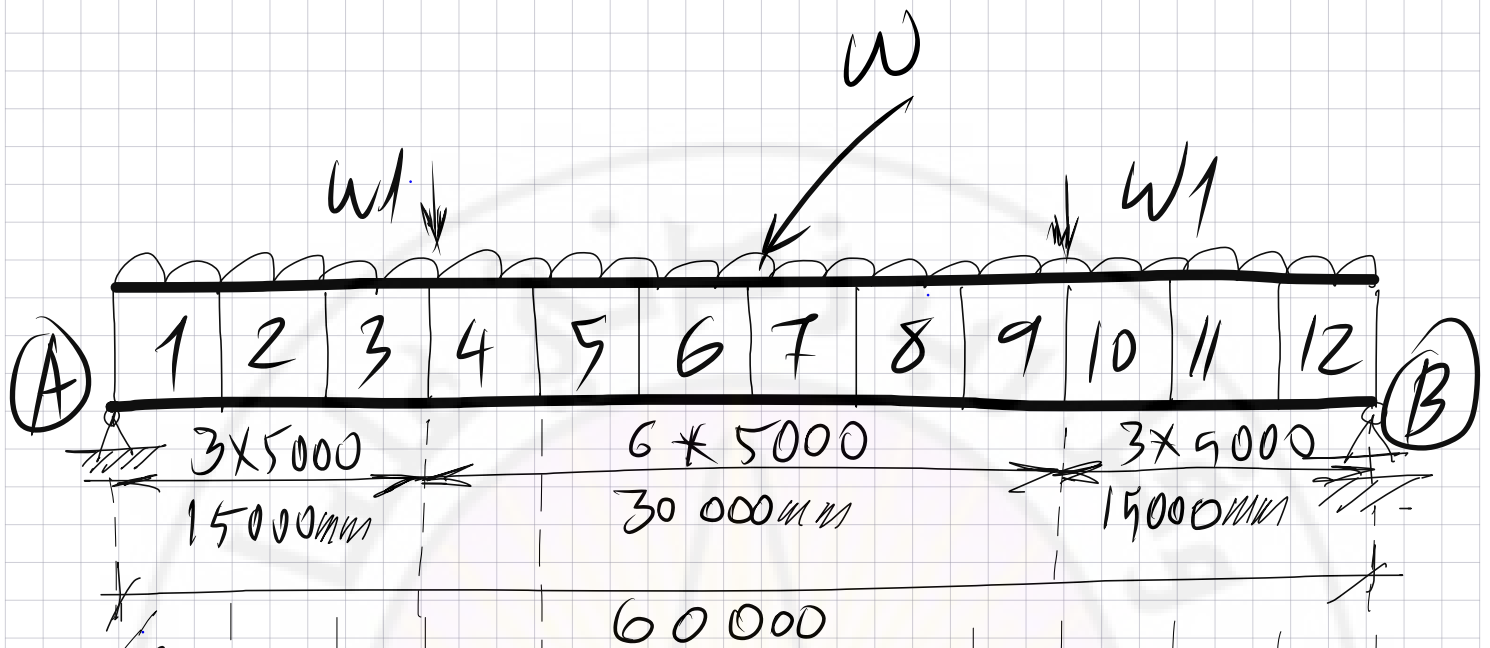
$$M_x = 2301 \cdot x - w \cdot x^2/2 \quad 0 \leq x < 9.0$$

$$M_x = 2301 \cdot x - w \cdot x^2/2 - W_1(x-9) \quad 9.0 \leq x < 27$$

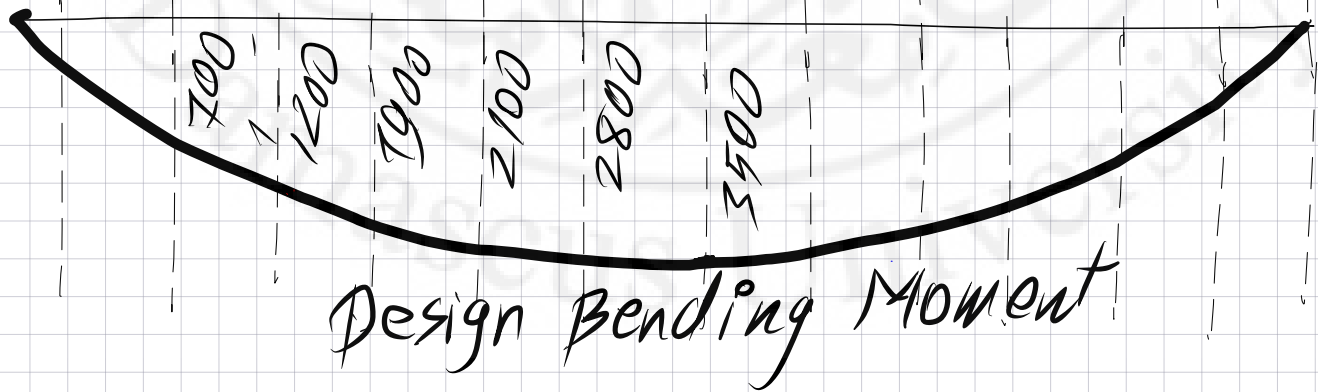
$$W_1 = 870 \text{ kN}$$

$$W_1 = 870 \text{ kN}$$





Design Shear Forces



Design Bending Moment

3. Section Proportion

Depth of girder: By using EC3 rules

$$h_w \approx \frac{1}{10} \sim \frac{1}{15} \cdot l$$

$$= \frac{36000}{10} \sim \frac{36000}{15} = 3600 \sim 2400 \xrightarrow{\text{choose}} \boxed{h_w = 2400 \text{ mm}}$$

Flang:

Use Common Practice

$$h_w/5 < b_f < h_w/3$$

$$2400/5 < b_f < 2400/3$$

$$480 < b_f < 800 \xrightarrow{\text{choose}} b_f = 720 \text{ mm}$$

Area of flang by Bending Moment:

$$M_{\max} = A_f \cdot d \cdot p_y / \sigma_M$$

$$A_f = \frac{M_{\max}}{d \cdot (p_y / \sigma_M)}$$

assume $t_f = 40 \text{ mm}$

$$p_y (S275) = 255 \text{ N/mm}^2 \quad \left\{ \rightarrow A_f = \frac{20709 \times 10^6}{2400 \times (255 / 1.15)} = 3894 \text{ mm}^2 \right.$$

$$t_f = \frac{3894}{720} = 54 \text{ mm} \xrightarrow{\text{choose}} \boxed{t_f = 60 \text{ mm}}$$

Web:

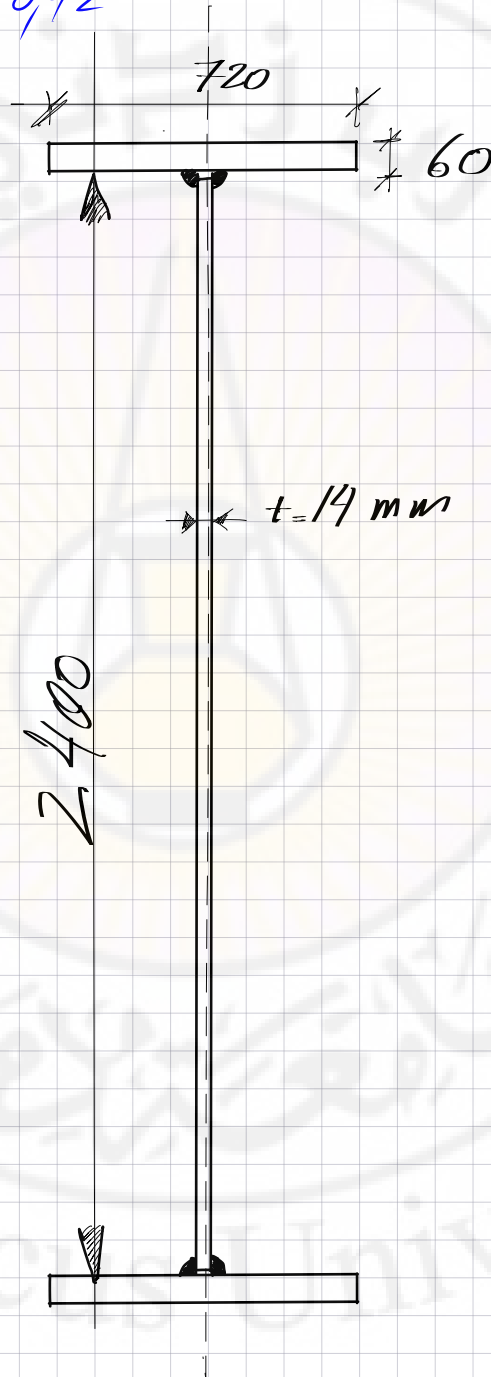
Use Web with as much minimum as could
try to be slender

$$\frac{c_w}{t_w} > 124 \varepsilon$$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{275}} = 0,92$$

$$\frac{C_w}{124} > t_w$$

$$t_w < \frac{2400}{124 \times 0,92} = 21,1 \text{ mm try} \rightarrow \boxed{t_w = 14 \text{ mm}}$$



4. Section classification

∞ Flange:

$$t = 60 \text{ mm} \rightarrow f_y = 255 \text{ N/mm}^2 \quad \varepsilon = \sqrt{\frac{235}{255}} = 0,96$$

$$b = \frac{B - t}{2} = \frac{720 - 14}{2} = 353 \text{ mm}$$

$$\frac{b}{t} = \frac{353}{60} = 5,9 < \eta \varepsilon = 9 \times 0,96 = 8,64$$

Hence, Flange is Plastic (Class 1)

∞ Web

$$t = 14 \rightarrow f_y = 275 \quad \varepsilon = \sqrt{\frac{235}{275}} = 0,92$$

$$\frac{d}{t} = \frac{2400}{14} = 171,4 > 124 \varepsilon = 124 \times 0,92 = 114,1$$

5. Checks

∞ Avoid serviceability problems

Assuming Stiffener spacing $a > h_w$

$$t_w \geq \frac{h_w}{250} = \frac{2400}{250} = 9,6 \text{ mm} \quad \text{OK}$$

∞ Avoid compression flange buckling to Web

$$t_w \geq \left(\frac{h_w}{k} \right) \left(\frac{f_y}{E} \right) \left(\sqrt{A f_c / A_w} \right)$$

Flange is class 1 $\rightarrow k = 0,55$

$$t_w \geq \left(\frac{2400}{0,55} \right) \left(\frac{255}{210000} \right) \left(\sqrt{\frac{720 \times 60}{2400 \times 14}} \right) = 6 \text{ mm} \quad \text{OK}$$

check for moment carrying capacity of the flanges:

The moment is assume to be resisted by alone and the web resists shear only:

Distance between centroid of flanges:

$$h_s = d + T = 2400 + 60 = 2460 \text{ mm}$$

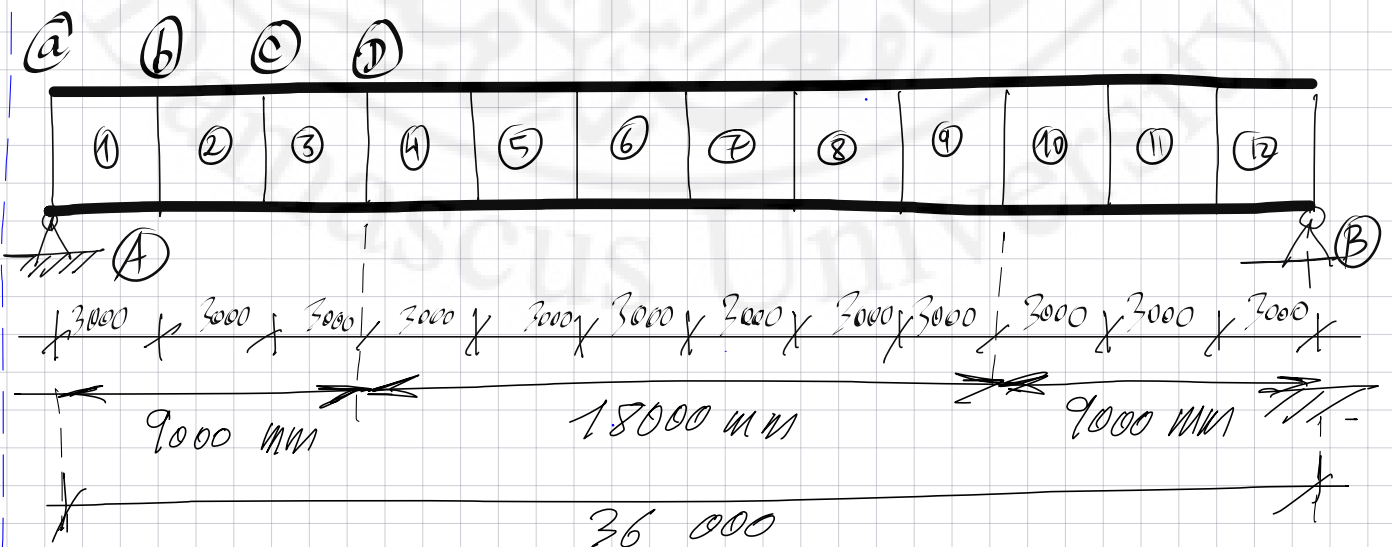
$$A_f = B \times T = 720 \times 60 = 43200 \text{ mm}^2$$

$$\begin{aligned} M_c &= P_{yf} \times A_f \times h_s \\ &= (255 \times 43200 \times 2460) \times 10^{-6} \\ &= 27100 \text{ kN}\cdot\text{m} > 20709 \text{ k}\cdot\text{m} \end{aligned}$$

Hence, the section is adequate for carrying moment and web is designed for shear

6. Web Design

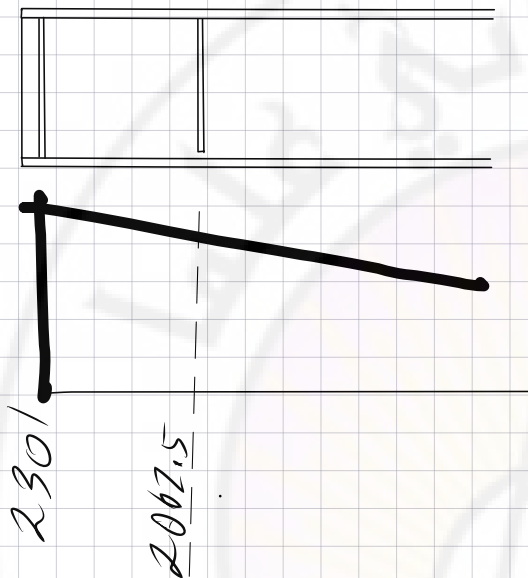
The Stiffeners are spaced as 3000 mm. changing spacing is possible during the progress of the design.



End panel Calc) design:

$$\left. \begin{array}{l} h_w = 2400 \text{ mm} \\ t_w = 14 \text{ mm} \end{array} \right\} \rightarrow \frac{a}{h_w} = \frac{3000}{2400} = 1.25, \quad \frac{h_w}{t_w} = \frac{2400}{14} = 171.4$$

$$\frac{h_w}{\alpha} = \frac{2400}{3000} = 0.8$$



Non - Rigid End post.

Shear (V_{Ed})

susceptible to shear :

$$\frac{h_w}{t_w} \leq 31 \frac{\varepsilon}{\eta} \sqrt{k_\sigma}$$

$$\varepsilon = 0.92, \quad \eta = 1$$

$$a/h_w = 1.25 > 1 \rightarrow k_\sigma = 5.34 + 4.00 \left(\frac{h_w}{\alpha} \right)^2$$

$$k_\sigma = 5.34 + 4.00 (0.8)^2 = 7.9$$

$$\text{check } 171.4 < 31 \times \sqrt{7.9} = 87.13$$

Panel is susceptible to buckling:

Web stiffeners, Non-rigid Post

$$\bar{\lambda}_w = 0.76 \sqrt{\frac{F_{yw}}{E_s}} = \frac{h_w}{37.4 t_w \varepsilon \sqrt{k_\sigma}} = \frac{2400}{37.4 \times 14 \times 0.92 \times \sqrt{7.9}} = 1.77$$

$$\lambda_w = 1.77 > 1.08 \rightarrow \eta_{w0} = \frac{0.83}{\lambda_w} = \frac{0.83}{1.77} = 0.47$$

$$= 2507 \text{ kN} > 2301 \text{ kN} \dots \text{OK}$$

Reduce the thickness to 8 mm, lets study?

$$\frac{h_w}{\alpha} = \frac{2400}{2760} = 0,87$$



Shear (V_{Ed})

$$\frac{1\omega}{\pm\omega} \leq 31 \frac{\varepsilon}{\mu} \sqrt{k_0}$$

$$\alpha/\hbar\omega = 1.25 > 1 \rightarrow K_G = 5.34 + 4.00 \left(\frac{\hbar\omega}{\alpha} \right)^2$$

check $300 < 31 + \sqrt{837} = 90$

Web stiffeners, Rigid End Post

$$\pi_{\omega} = 3p > 1.08 \rightarrow \kappa_{\omega} = \frac{0.83}{\pi_{\omega}} = \frac{0.83}{3.0} = 0.28$$

$$= 553.6 \text{ kN} < 2301 \text{ kN} \dots \text{fail} \rightarrow \text{Tension field needed.}$$

∞ Contribution from flanges:

Reserve on flanges:

$$\frac{M_{ed}}{M_{f,rd}} = \frac{6545,25}{27100} = 0,24$$

only 24% of flange capacity is used to resist the bending
Capacity available for tension field

$$M_{reserve} = 1 - 0,24 = 0,76 \dots$$

Shear resistance available from tension field

$$V_{bf,rd} = \frac{b_f \cdot t_f^2 \cdot f_{yf}}{c \cdot \sigma_{H1}} \left[1 - \left(\frac{M_{ed}}{M_{f,rd}} \right)^2 \right]$$

$$c = a \left(0,25 + \frac{1,6 b_f \cdot t_f^2 \cdot f_{yf}}{b_w h_w^2 f_{yw}} \right)$$

$$= 2760 \left(0,25 + \frac{1,6 \times 720 \times (60)^2 \times 255}{8 \times (2400)^2 \times 275} \right) = 920 \text{ mm}$$

$$V_{bf,rd} = \frac{720 \times (60)^2 \times 255}{920 \times 1,1} (1 - (0,24)^2) \times 10^{-3} = 653 \text{ kN}$$

$$V_{b,R} = V_{bw,rd} + V_{bf,rd}$$

$$= 853,6 + 653$$

$$= 1507 \text{ kN} < 2301 \text{ kN} \dots \text{fail}$$

The increase of the tension field contribution
there are several choices:

① Reduce the Panel Width

② Increase the flange thickness.

$$f_{ny}: \quad a = 1800 - 240 = 1560 \text{ mm}$$

$$t_f = 75 \text{ mm}$$

$$M_{f,rd} = 27100 \times 75 / 60 = 33875 \text{ kN} \cdot \text{mm}$$

$$c = 1560 \left(0,25 + \frac{1,6 \times 720 \times (75)^2 \times 255}{8 \times (2400)^2 \times 275} \right) = 593 \text{ mm}$$

$$V_{bf,rd} = \frac{720 \times (75)^2 \times 255}{479 \times 1,1} \left(1 - \left(\frac{6545,5^2}{33875} \right) \right) \times 10^{-3} = 1525,0 \text{ kN}$$

$$V_{b,R} = V_{bw,rd} + V_{bf,rd}$$

$$= 853,6 + 1525$$

$$= 2378,6 \text{ kN} > 2301 \text{ kN} \dots \text{OK}$$

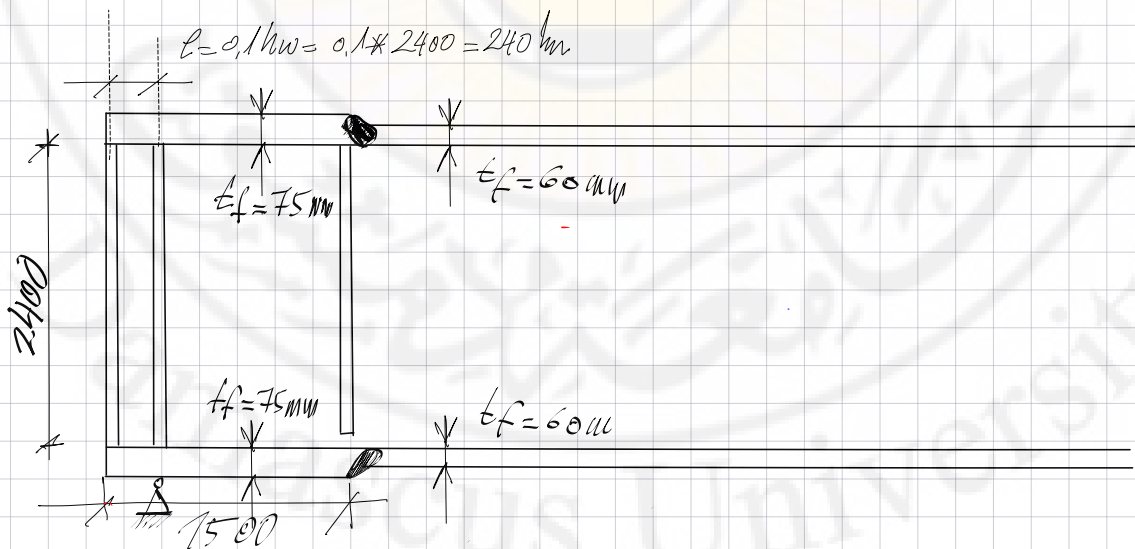
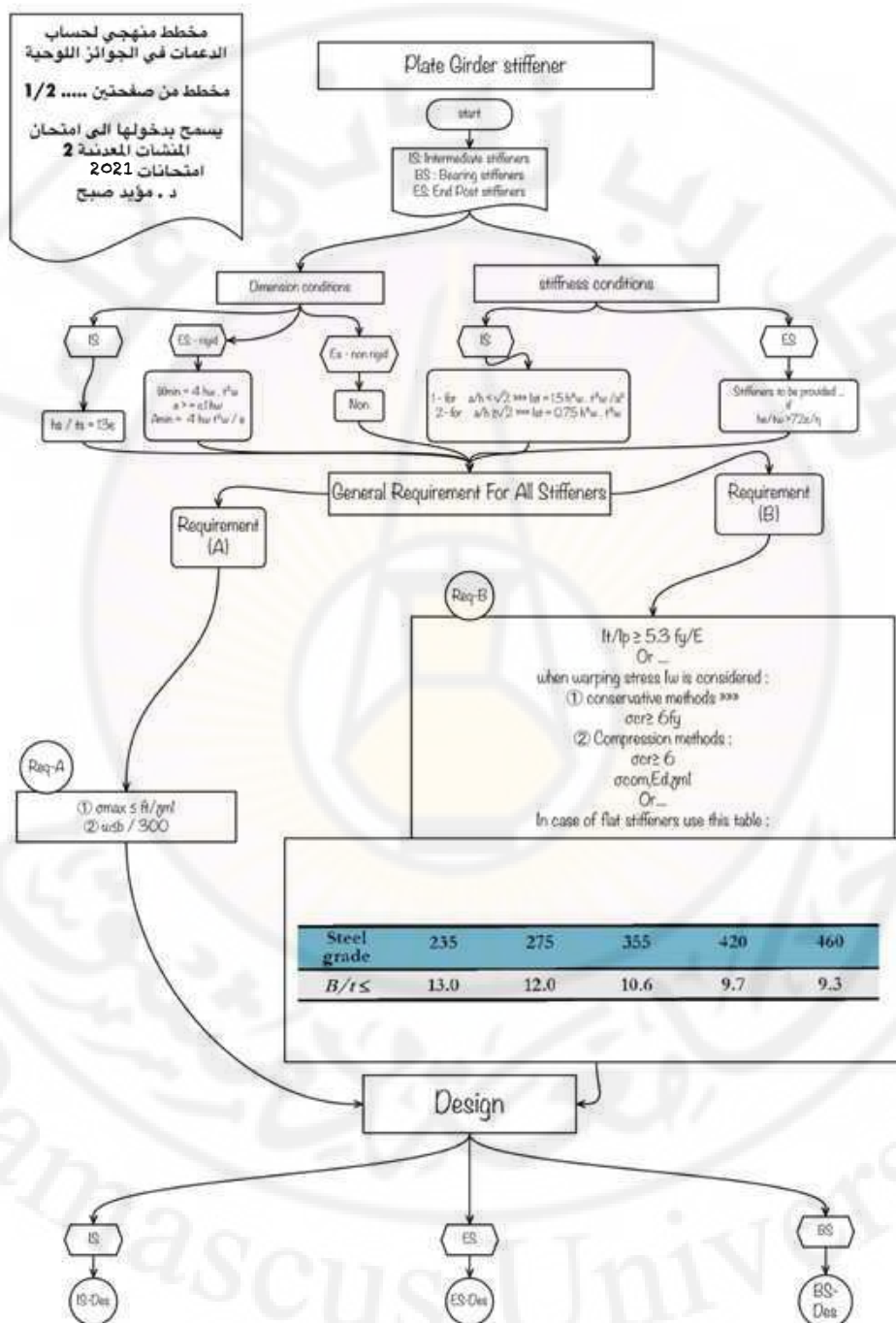


Plate Girder Stiffener Calculation Flowchart



مخطط منهجي لحساب الدعائم في الجوائز اللوحية

مخطط من صفتين 2/2

يسمح بدخولها الى امتحان المنشآت المعدنية 2021
د . مؤيد صبح

Plate Girder stiffener

Design

