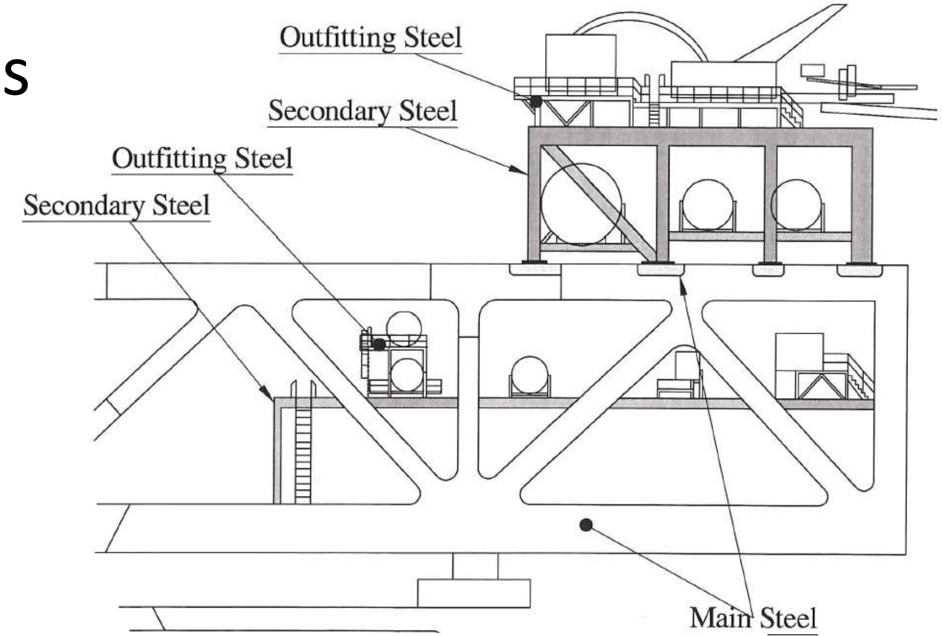
TBYG3018 Design of Offshore Structures

Modules in "Design of Offshore Structures"

Structural Disiplines

- Main steel
- Secondary steel
- Outfitting steel



The definition of main steel as used in this book is the structure that is designed to withstand the global load, both static gravity loads, dynamic loads from environmental forces and dynamic loads from accelleration of the masses.

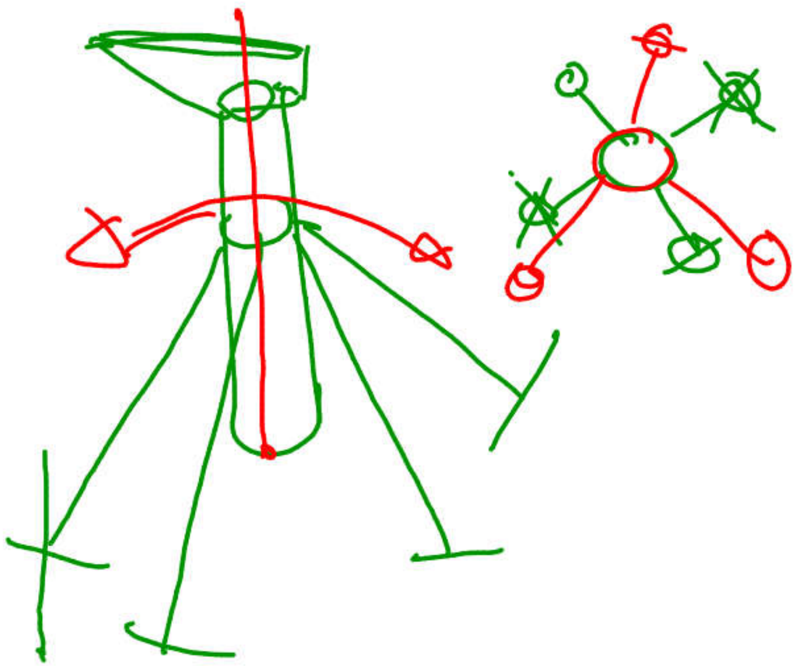
Secondary steel is made to transfer loads from different areas and onto the main steel. Secondary steel capacities are not included in the main strength analysis of the platform.

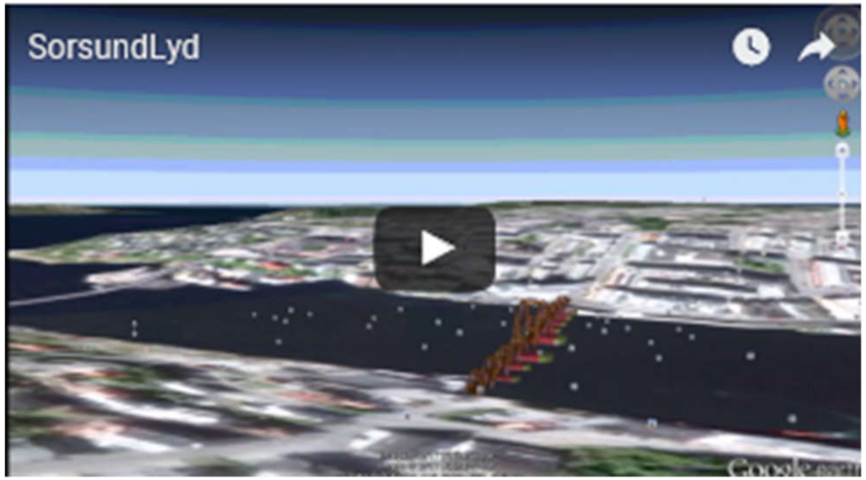
Outfitting steel has some of the functions of secondary steel, but is specialised (seatings for equipment etc, or ladders, handrails or other specialities)

Subsea Structural and Mechanical tasks

Structural/marine	(Mechanical) Design	Piping
•Structural design basis	•Functional Design Specification (FDS)	•Piping design basis /specification
•Geotechnical design basis	Mechnical layout	•Standard details/solutions
•Material selection	•Standard details/solutions	•Piping layout
•Standard structure/marine details/solutions	Interfaceing products	•Piping Documentation
•Foundation design	•MCS products (includes connection +++)	Piping costumer drawings
•Protection structures	•Customer models/drawings	Piping fabrication drawings/models
•Transport equipment (slings, padyes, heavy lift methods)	•Machining drawings and tolerances	MTO
•Geotechnical	•Hydraulics, control systems etc.	•Welding register
•Marine and dynamic evaluations	•Valves	
•Structural, Marine and Geotechnical Documentation	•Tools	
•Welding inspection category	•ROV access intervention report	
•Modelling and drawing production and method	•Customer drawings	
•Structural steel MTO and weight	•FAT procedures	
	•User manuals	

Statically defined or undefined?
What are the advatages?

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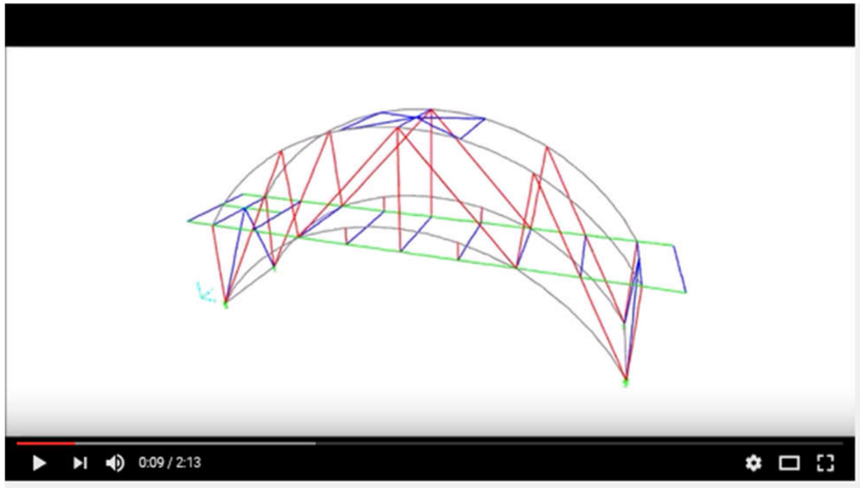
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Module 7 Common structural shapes and Trusses and Frames

Jomar Tørset, Høgskolelektor

3 Principles of Load Carrying Structures

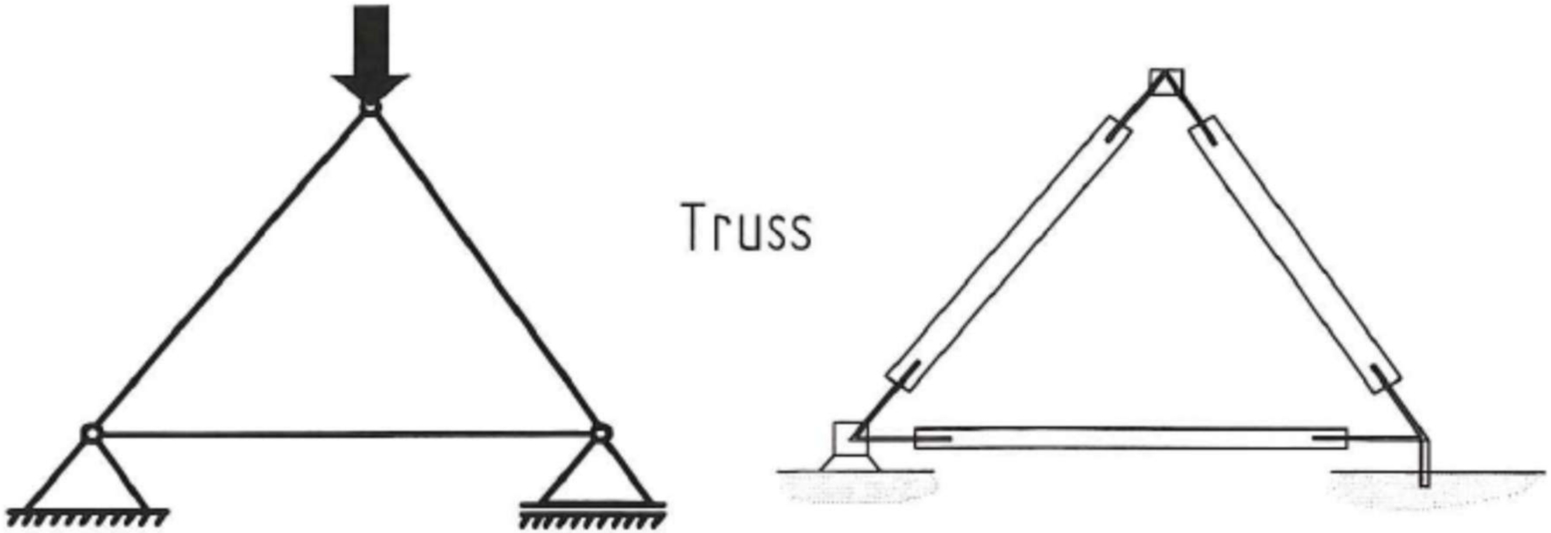
3.1 Structure types

In offshore engineering, we mainly use three basic types of structures:

1. Trusses
2. Frames
3. Plates

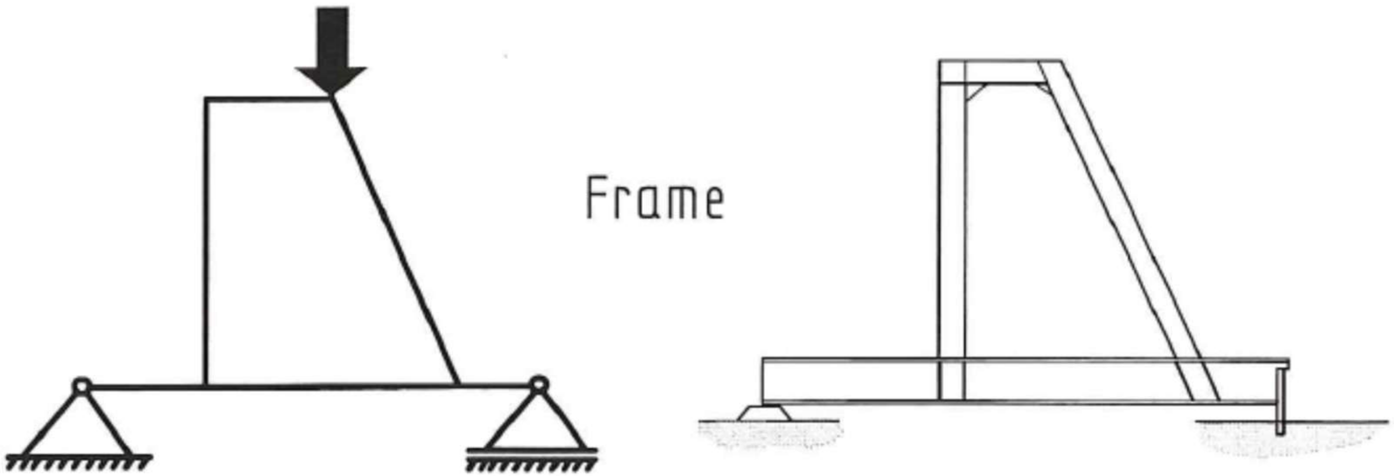
3.1.1 Trusses

Trusses consist of structural beams that carry axial load only. These truss members are tied together in joints that do not transfer moments. These joints are hinges, and stiffness has to be achieved by introducing truss members in all directions that is necessary to constrict the movements of the joints in the truss.



3.1.2 Frames

Frames are made up from beams and columns that are connected in joints that can and do take moment. The beams in a frame can transfer load transverse to their axial directions. In addition, the beams in a frame can transfer moments. Geometrical freedom of the design is increased compared to a truss design.



In general, a frame structure is most efficient when it has a structural layout like a truss, utilising the axial stiffness of the beams.

When the structural layout of a frame differ from that of a truss, increased section dimensions has to be used for that location. This means that all joints where the centerlines of the beams do not meet, additional material has to be added in order to obtain sufficient resistance to take shear and moment.

Beware also of deformation controlled bending in a truss-like frame structure due to the axial deformation of the members.

3.1.3 Section properties recommendations

Since frames has the capabilities to take moment and shear, we can customise the beam types of the frames according to the loading.

As a general rule: we should opt for as much material as practically possible in the fluxline of the primary force that the section has to take:

A beam where the shear force is dominant is typically made as an open section with a high web.

A beam where the bending moment is dominant is typically made as an open section with voluminous flanges. The higher section we make, the more bending moment resistance we get.

A beam that has high torsional loading should be made as a hollow section, so that a continous shear flow is possible around the section. If this section is welded, the smallest weld should at least have a throat thickness equal to the thinnest plate in order to fully utilise the steel of the section.

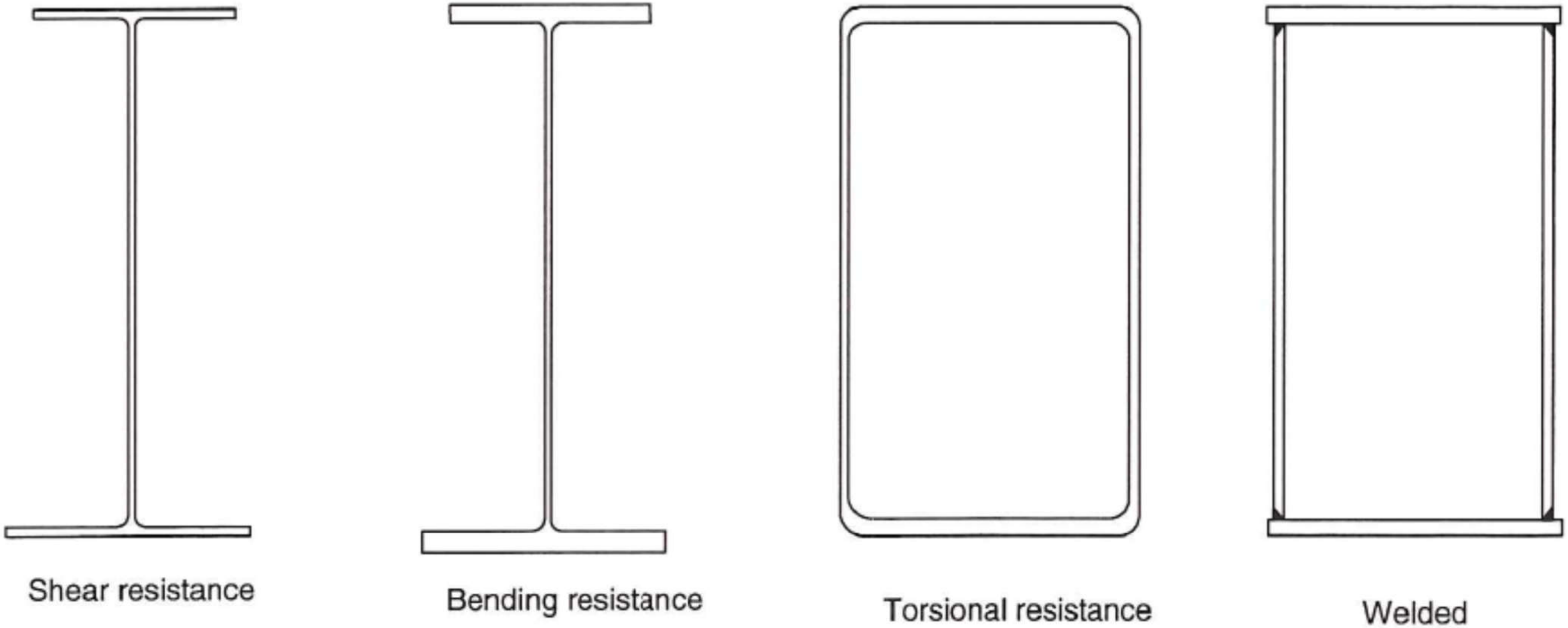
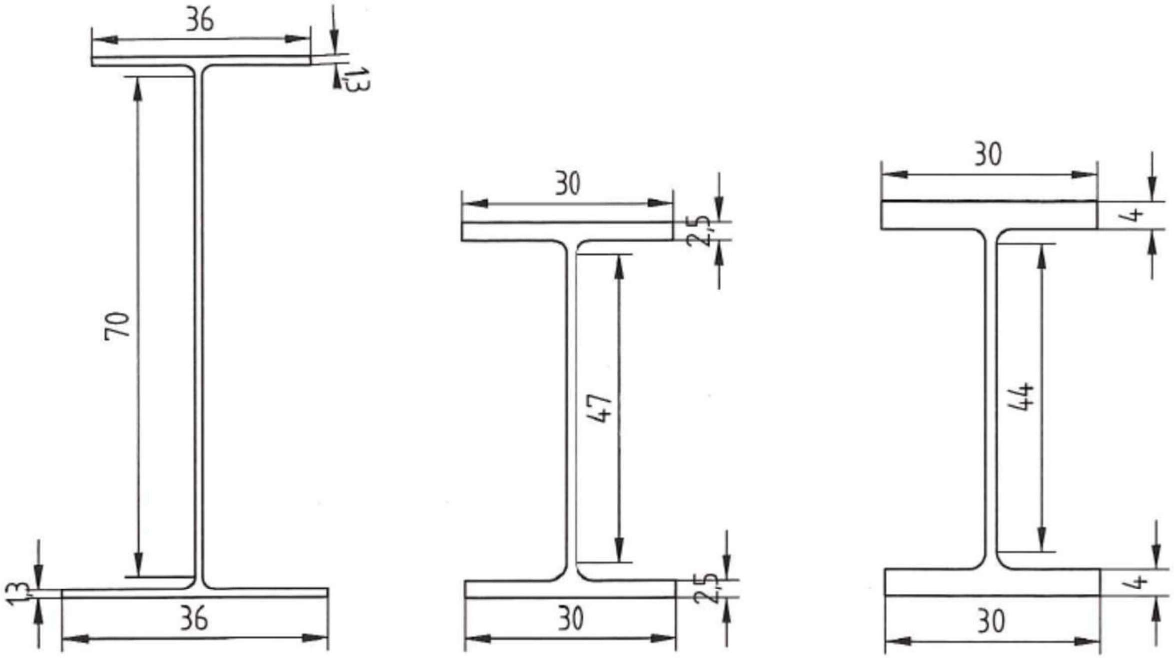


Fig. 3.1.3.1, Types of sections and their characteristic qualities



Stiffness 160000
Bending stress 100

Stiffness 114000
Bending stress 100

stiffness 160000
bending stress 71

The high and slender beam will need stiffeners along the web in order to be fully utilised.

The section to the left is mainly suitable for shear loading. Its slenderness makes it necessary to stiffen it locally in order to keep the web and flanges from buckling locally. The section to the right has the same stiffness as the slender section. The compactness makes it less weight efficient for all applications exept of predominantly axial loading.

3.1.4 Arches and other specialised frame types

Arches are suitable for situations where the loading is dominated by hydrostatic pressure. This means that the loading along the arched beam is normal to the longitudinal axis of the beam, and parallel to the strong axis of the beam. Arched beams are not very suitable for point loads.

The same is typical for the traditional barrel band, which is actually a circumferal arch going all the way around a barrel. The idealised barrel band need only axial rigidity, since it has already reached its most stable form statically.

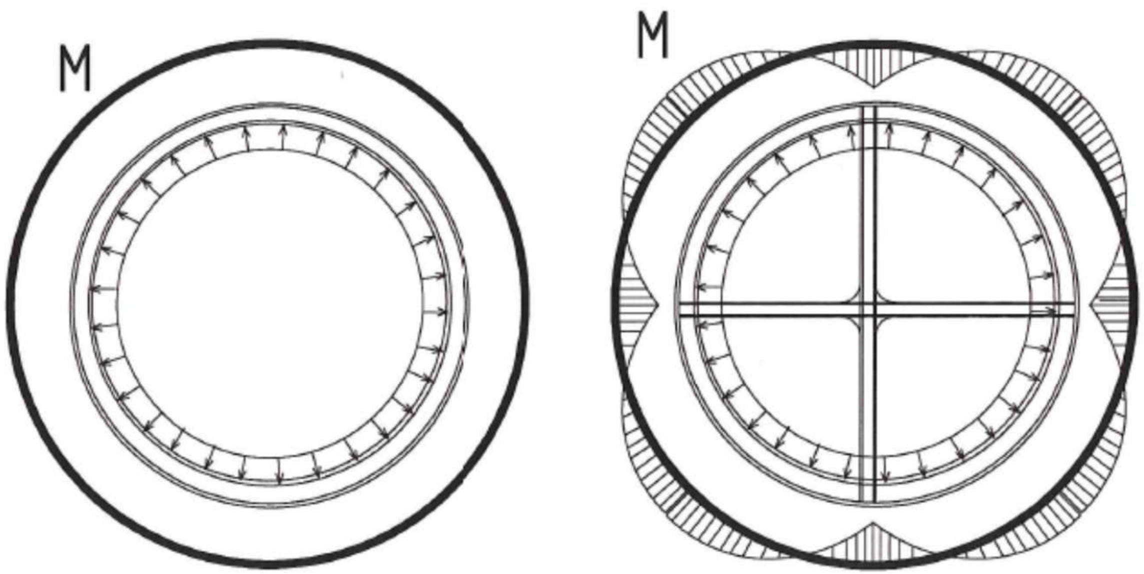
If we introduce a rigid structure into a barrel, fixing the barrel bands at certain locations, then the barrel band will be exposed to shell moments upon loading. This is because the radial deformation of the barrel band is restricted at the fixations. We then have a situation similar to what we see in a column stringer (transverse beam around the circumference of a steel column in for instance the column of a semi submersible.

As an example, we can take a barrel with internal pressure. At first, we have a conventional barrel, where the barrel skin and the barrel bands are in equilibrium with the internal pressure.

The total reaction force of the pressure sums up to 0, and the shape of the barrel is restricted only by the barrel band or barrel skin taking axial load.

If we introduce internal stiffening to this situation, the forces in the barrel band will change dramatically, by the introduction of shear forces and moments.

We can conclude that the arch, or barrel, is most suitable for evenly distributed loadings. In columns of offshore structures, we have no choice but to have internal restrictions like tank division bulkheads. The round outer shape is to minimise wave loading. The column stringers (barrel bands of the column) then has to be made continous with stringers in these internal bulkheads. It is however wise to keep in mind that our most intense design effort has to focus on the detailing around where the arched stringers are connected to the stringers of the internal bulkheads. At these points, both shear forces and moments are at the highest. At the same spots we have welded connections we have to consider for fatigue loadings.



Introducing moments in a barrel band subjected to internal pressure by implementing internal stiffening.

Fig. 3.1.4, The effect of internal stiffening of a barrel subjected to evenly distributed load

The internal pressure may be pressures in a process tank or pipe. The pressure may be internal or external, the point is that the arch is ideal for evenly distributed loading, not pointloads.

3.1.5 Plates

Plates carry loads by in-plane stress and plate bending. The primary stiffness of a steel plated structure comes from in-plane stress stiffness. The strength/stiffness ratio of steel (σ_y/E) usually results in very thin plates that between girders act more like a membrane structure. The transverse stiffness of a stiffened steel plate field comes from the stiffeners/girders with the plate contributing as effective flange. The plate itself can carry transverse loads only in the short span between stiffeners.

For other materials like sandwich plates of reinforced plastics, or reinforced concrete, the stiffness/strength ratio is different and the plates carry also bending and transverse shear without stiffeners due to a higher thickness/length ratio.

Curving plates also increases the shell-like load carrying, i.e. can take certain loads on shell bending without transverse stiffening.

Plated structures gives us lots of freedom in the design. But it is worthwile to keep in mind that the flow of forces in plated structures has many similarities with that of trusses:

Shearloaded bulkhead principal stresses, max compressive (right) and tensile (left).

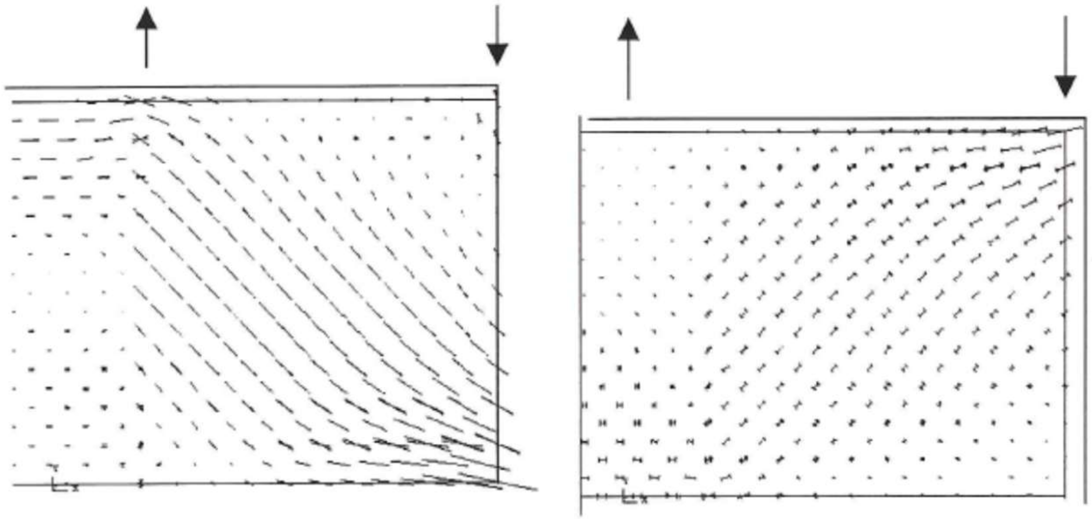


Fig. 3.1.5, Principal stresses of shear loaded bulkhead

The bulkhead shown is a typical example of a bulkhead that carries predominant shear loads. Differing boundary conditions and loads generate different patterns of principal stresses. The stress pattern does however often correspond to that of a truss.

After having clarified the dominant flow of stresses through a bulkhead, the results should be considered when reinforcing, or when positioning penetrations etc.

3.2 Basic load types of integrated structures

An integrated structure are basically loaded by two types of loading: Force driven and deformation driven.

Force driven loading is the type of load that is normally considered. It consists of forces external to the structure, forces that can be isolated and treated in a traditional manner. In general, structural rigidity gives resistance to isolated forces.

Deformation driven loads are more complex to design for. A deformation driven load occurs when our structure is squeezed, stretched or bended by deformations of its foundations. Remark: For an integrated structure, the structure surrounding it, on which it is seated, is ofcourse its foundations. So when this "mother structure" deforms, deformation driven loads can be imposed on the integrated structure:

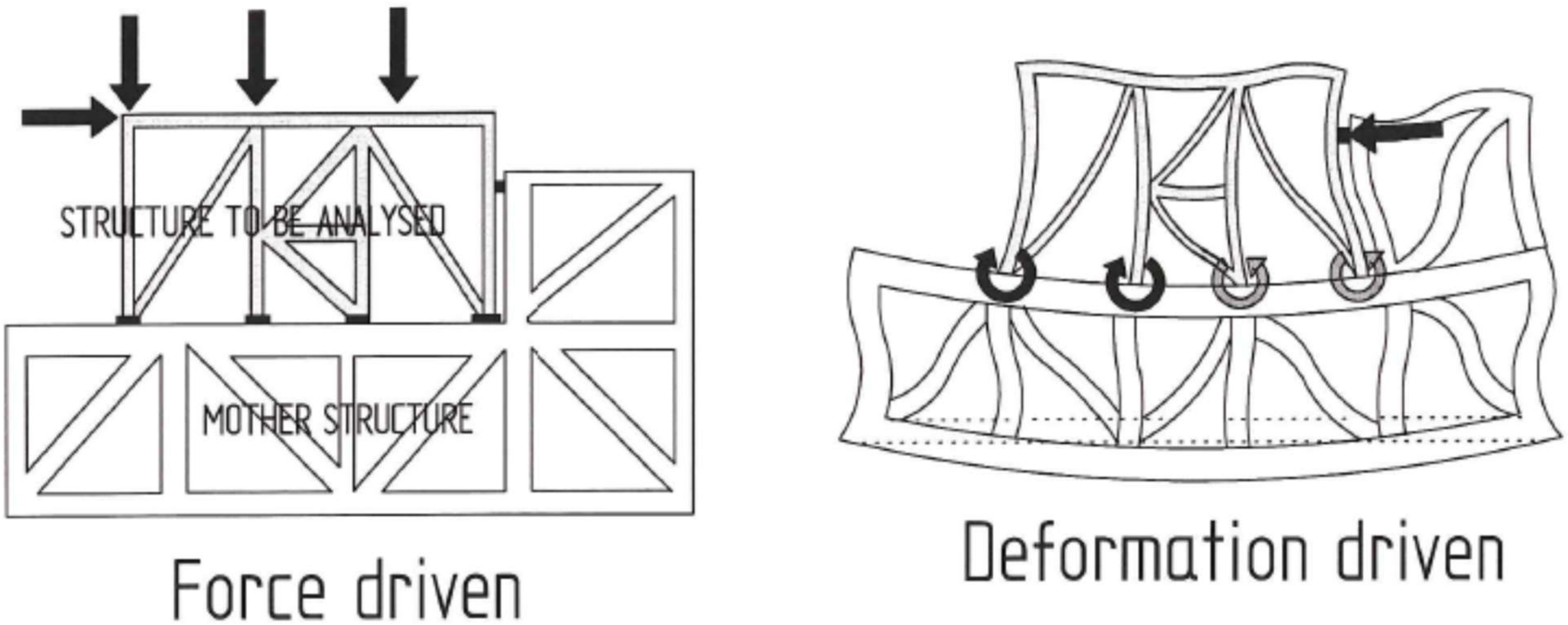


Fig. 3.2.1: Force driven and deformation driven situation:

When designing for deformation driven loads, we aim at reducing the amount of force that flow through the structure we are designing, letting the mother structure deform as unrestricted as possible.

This might lead to the avoidance of bracings, and to the establishment of hinges to an extent where the structure to be analysed is still stable. An example of how this can be done is shown on in fig. 3.5.2

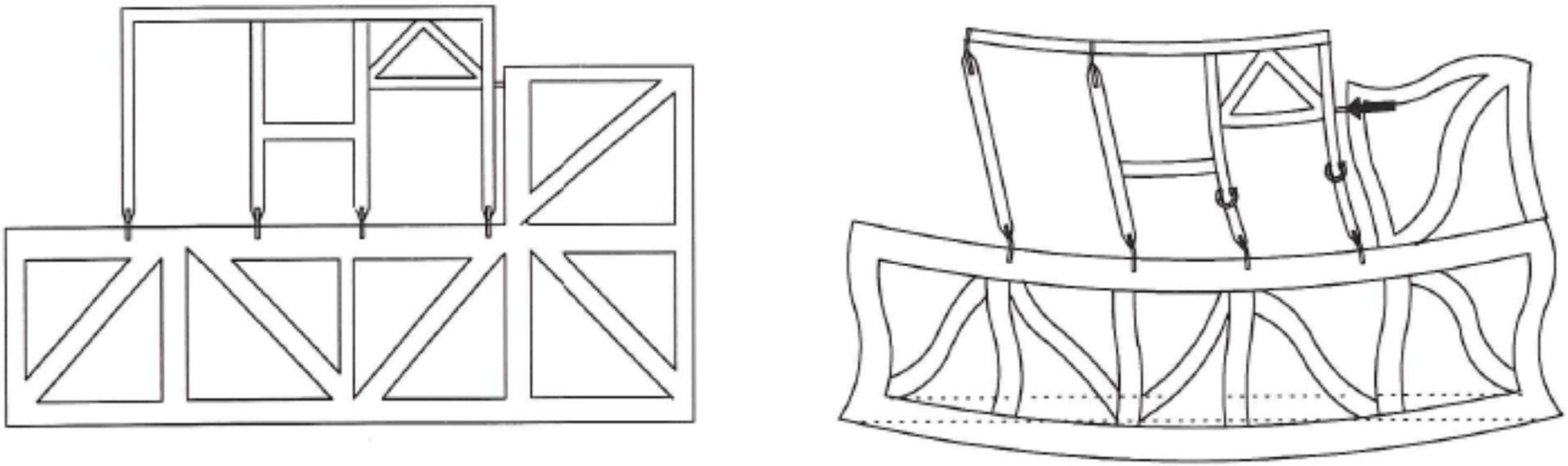


Fig. 3.2.2: Softening up a deformation driven situation:

As can be seen in this example, the softening up of joints and the omittance of braces has led to a considerable reduction in the forces that was transferred through the structure that is to be designed.

A further reduction of the stiffness of the structure to be designed will lead to a further reduction of the forces it has to transfer. We will however have to keep in mind that the structure to be analysed has to have structural integrity on its own, and that we have to balance its stiffness also with that issue in mind.

In this example, the top section of the structure to be designed is held by a separate A-brace tied to the upper part of the mother structure.

Note the bracket end connection that replaces a stiff joint. This type of joint may have stiffness in one direction at the same time as it operates as a hinge in the other direction as shown here.

For a given angular forced deformation, a low section gets lower stresses than a high section. This is practical to see by visualising fundamental elastic theories to the situation, and the reason is similar to the effect we get when we soften up the structure to be analysed above:

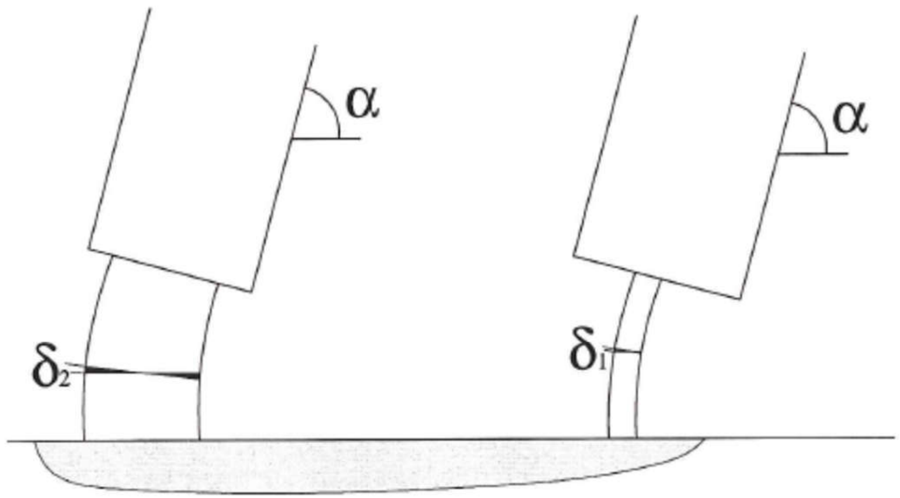


Fig. 3.2.3: *$\delta 2 > \delta 1$* ***for similar angular deformation that has to be taken over a similar length***

We therefore see that we sometimes achieve a dramatic reduction of stress due to a reduction of structural dimension. This should be well known basics of structural design, but we all too often experience that the only medicine that is tried is to increase sectional dimension.

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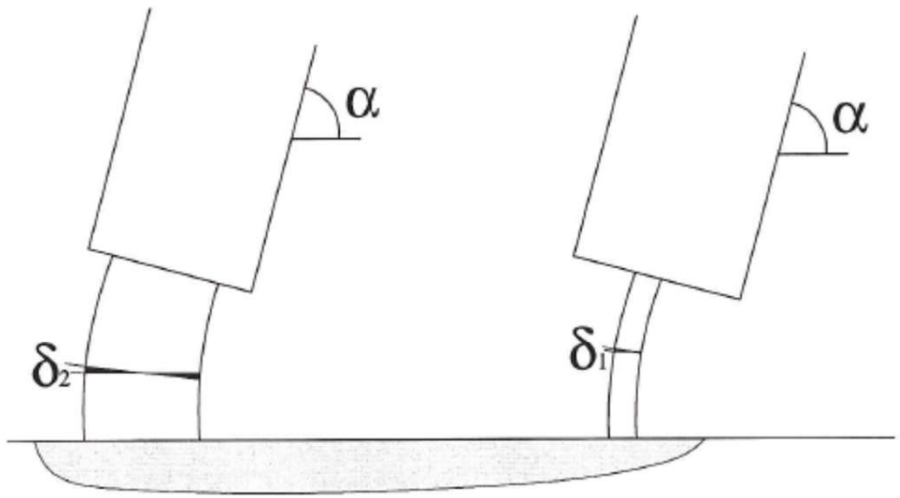

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3.3 *Keeping structures stable*

We have now seen that sometimes we can benefit from softening up structures that are engulfed in a mother structure in order to reduce forced deformations and all the problems that are associated with those.

This can of course be done only if we keep our structure stable, so that it works well for the masses that it supports.

We seek the most rational way of holding our structure, and at the same time we try to minimise co-axial stiffnesses to the mother structure in such a way that the stiffness of the structure we design is low in the directions between its supports in the mother structure

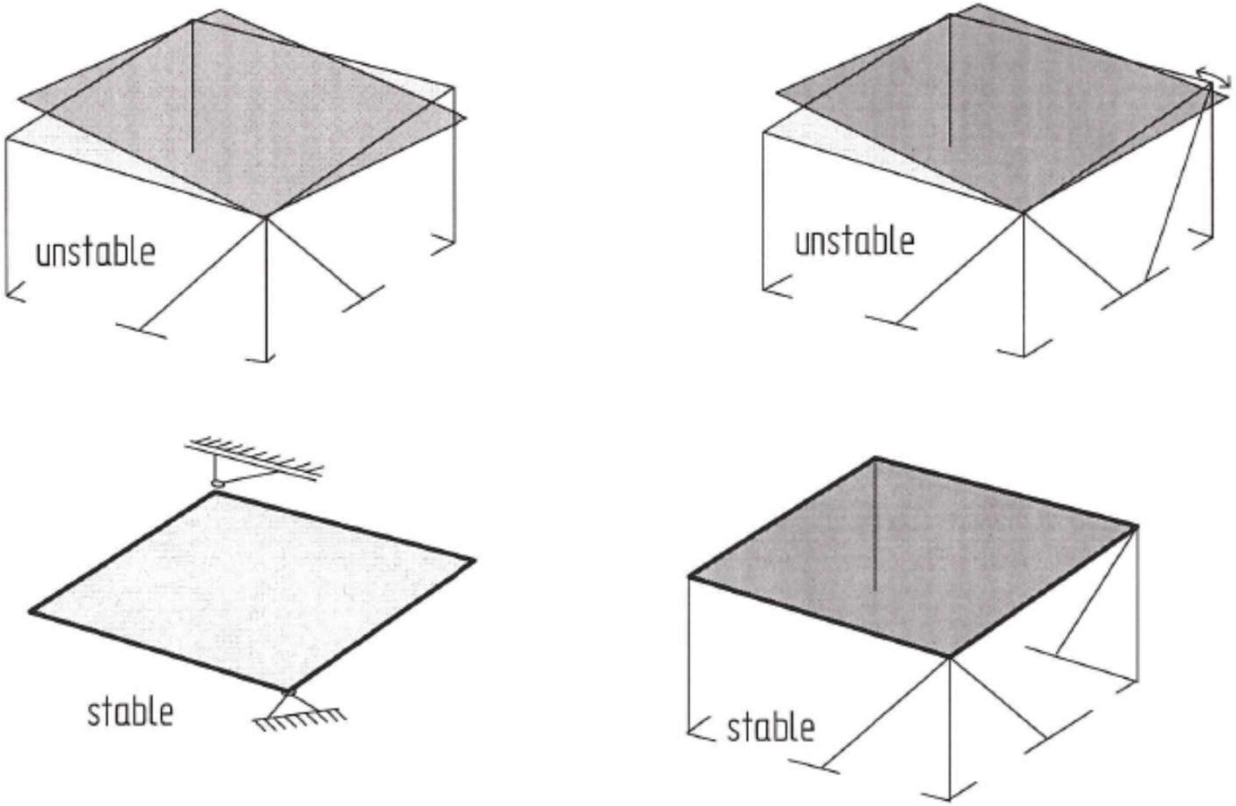


Fig. 3.3.1 stabilising a small frame structure

The key is to ensure that the deck of the frame can not rotate about its primary support (where it is held against translations in all directions). The stable situation can be obtained in several ways. Common for all of them is that the deck/plate is restricted along a line that does not go through the point of its primary support (the near-end corner in the figure above).

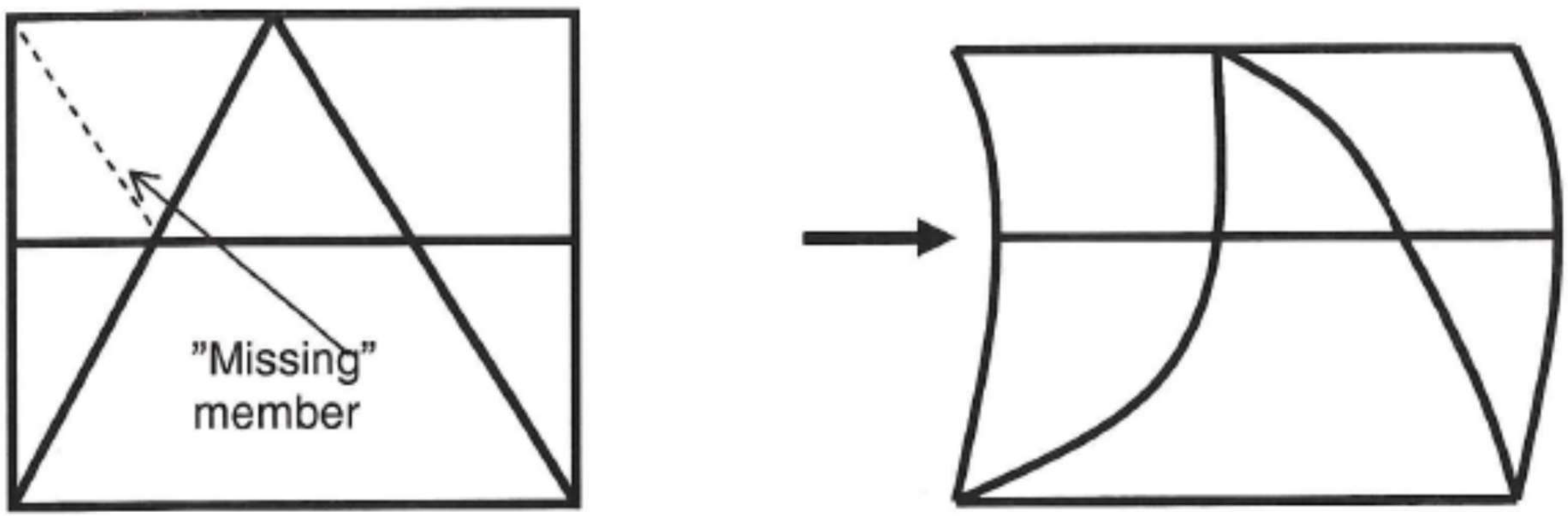
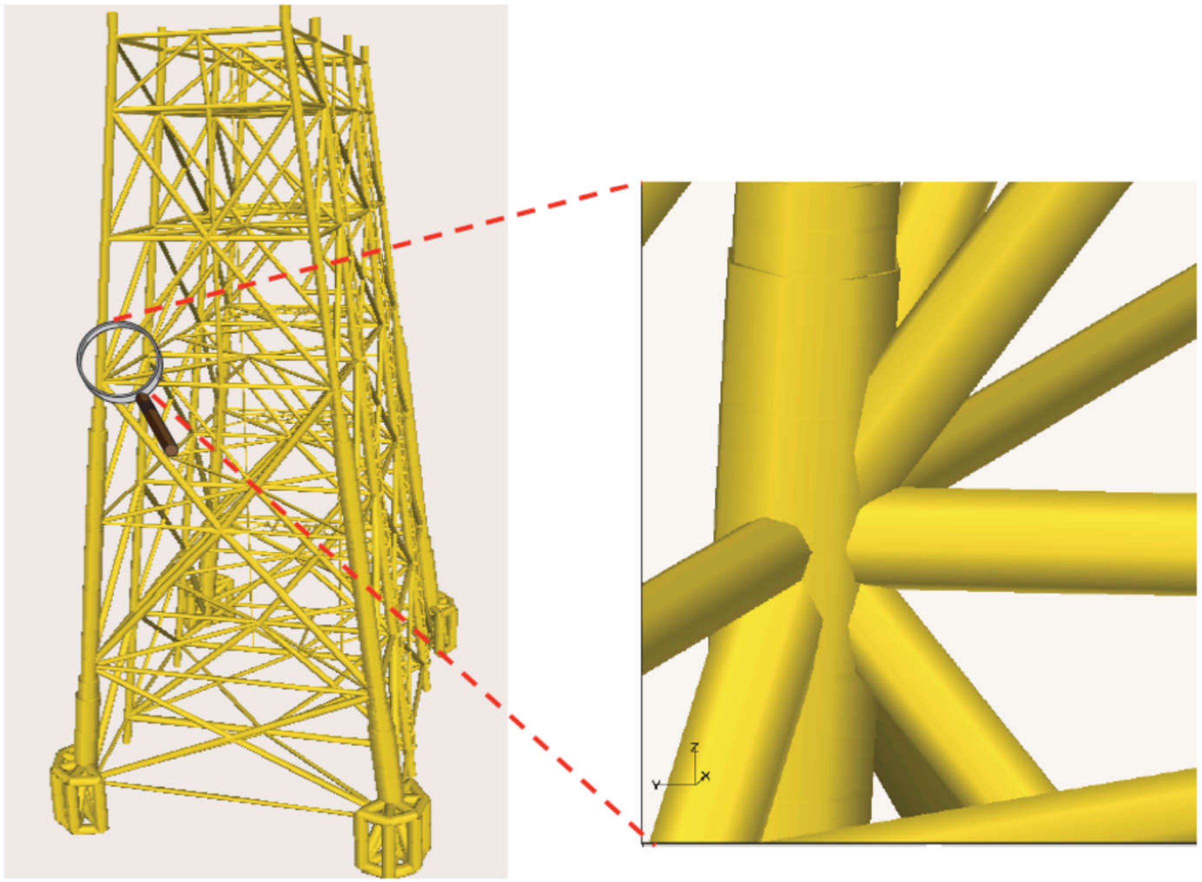
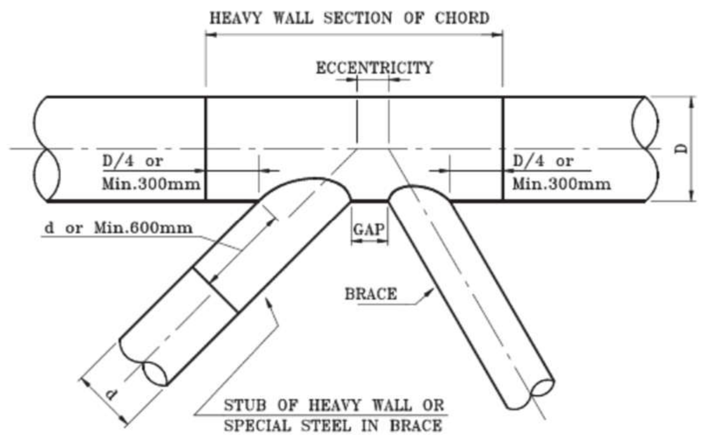


Figure 3.3.2 common “mistake”, the tween deck is supported sideways by bending and shear of the members only. This can cause high stresses during transport or on topsides with significant lateral motions in the in-place conditions.

Typical joint in offshore jacket



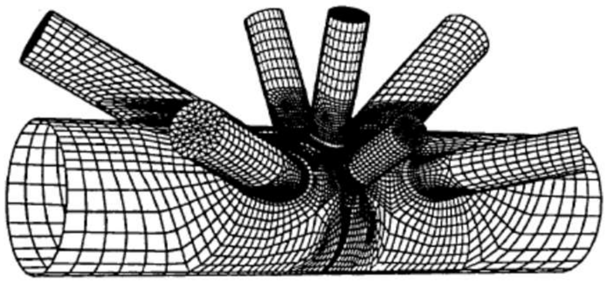
The tasks of the joint



- Transmit axial forces from one brace to another.
- This is done by a complicated composite action between shell bending, membrane forces and shear stresses in the chord.
- A shell analysis will often result in local stresses at the welds (hot-spot stresses) which is larger than f_y.
- Normally not so critical because yielding stops after some time due to elastic redistribution of stresses (shakedown).

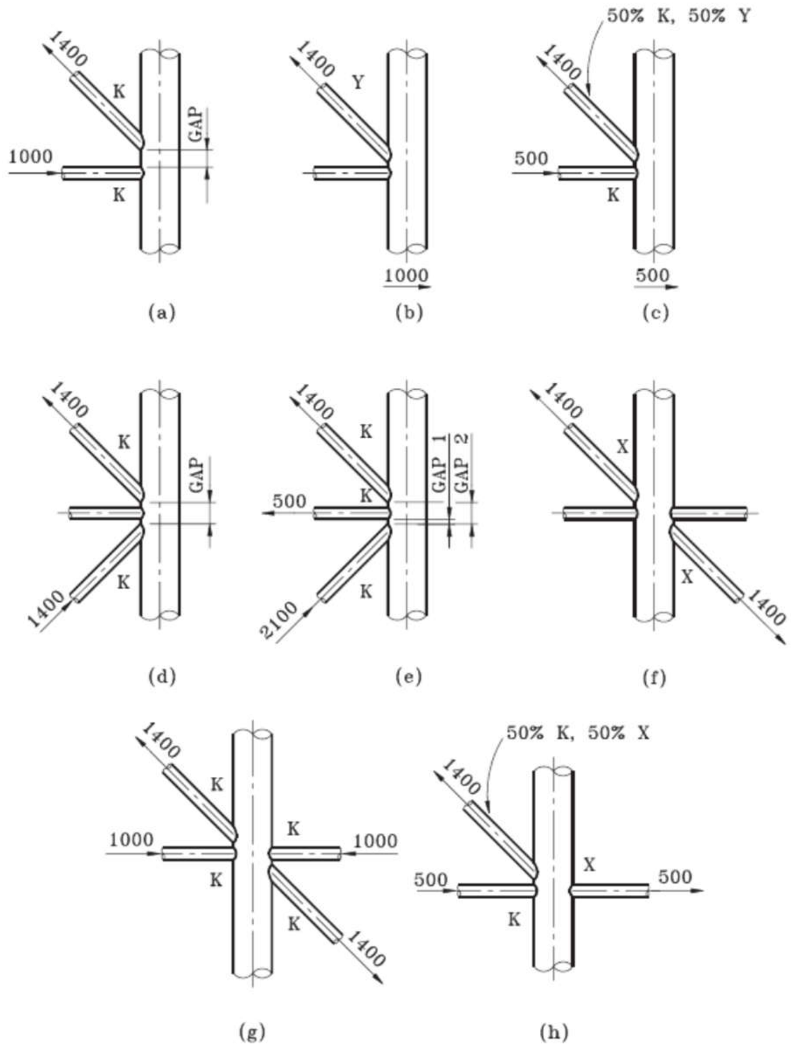
Resistance of joints

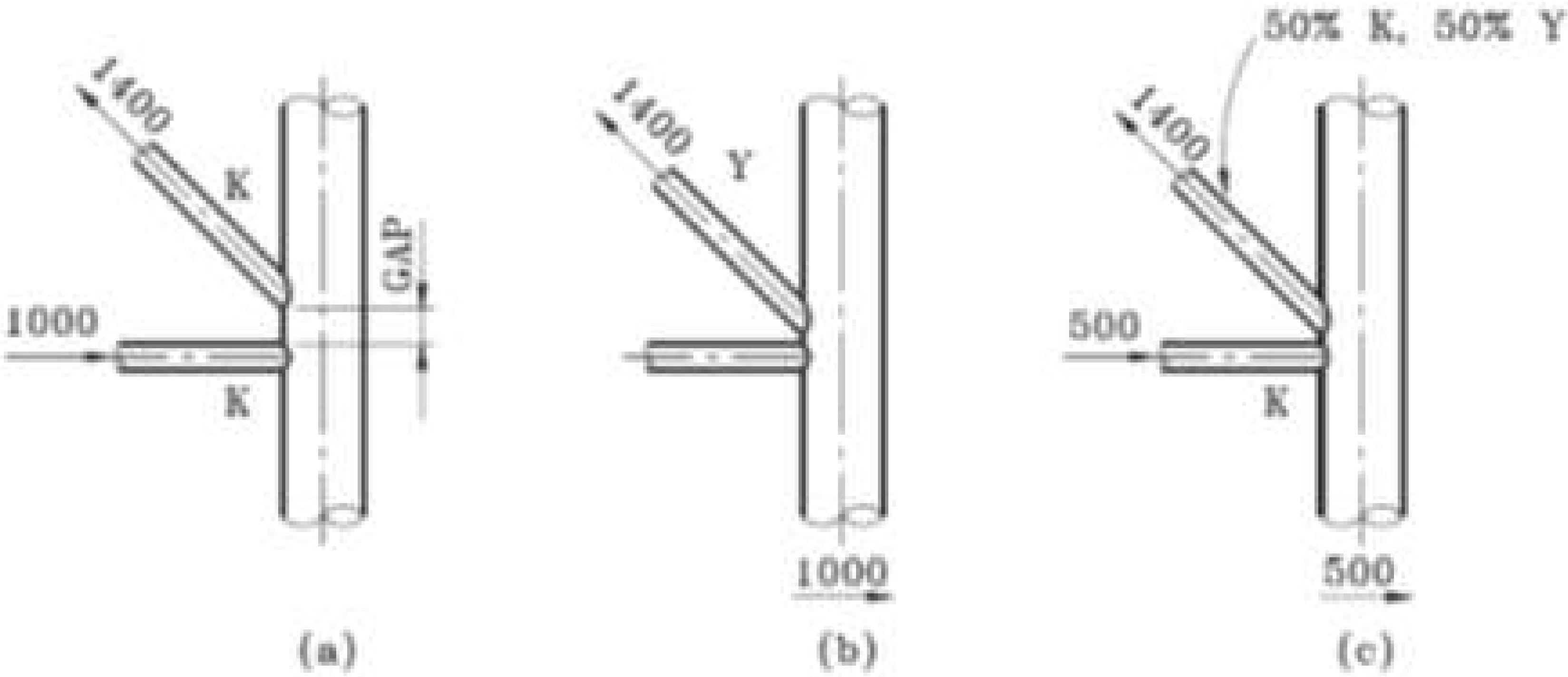
- Traditionally based on experimental data (relatively large uncertainties).
- Alternatively, non-linear finite-element analysis (FEA) when the empirical code formulas are insufficient.

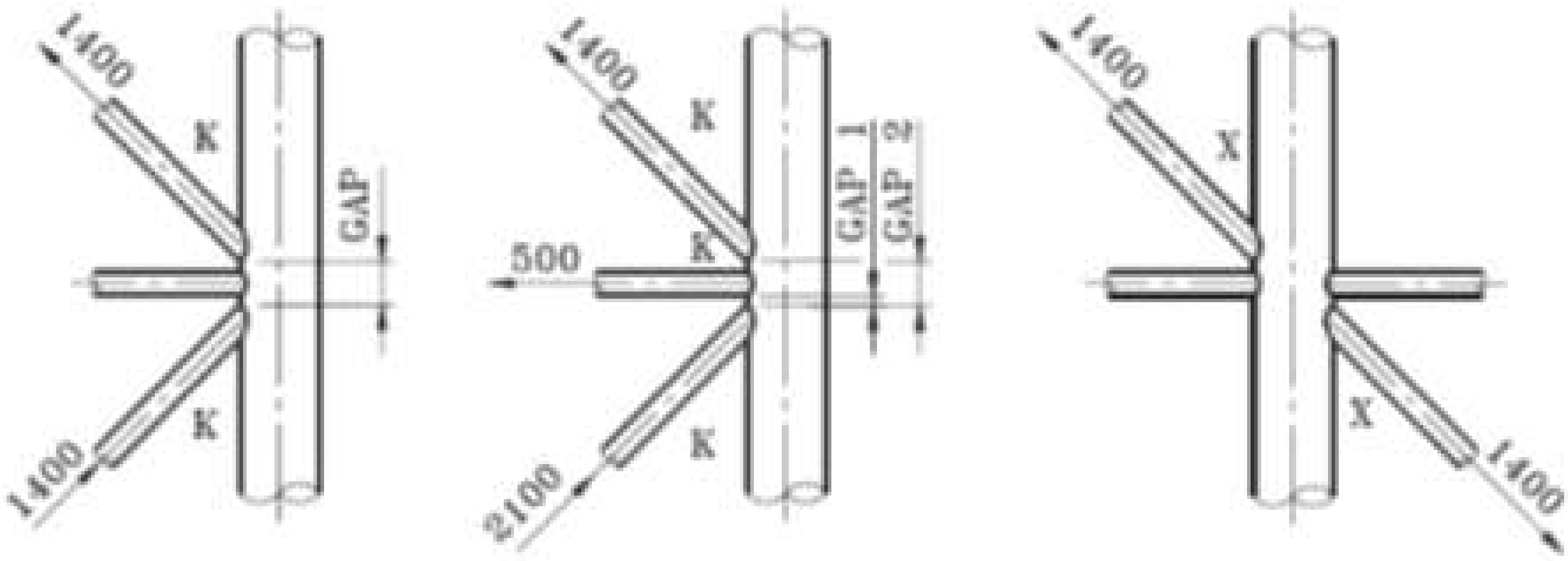


- The code formulas are based on experimental data for planar joints even though most joints are multiplanar.
- This is solved by looking at the plane for which the forces are transmitted for the given load case.
- This is normally conservative since braces in other directions most often will be positive for the resistance.

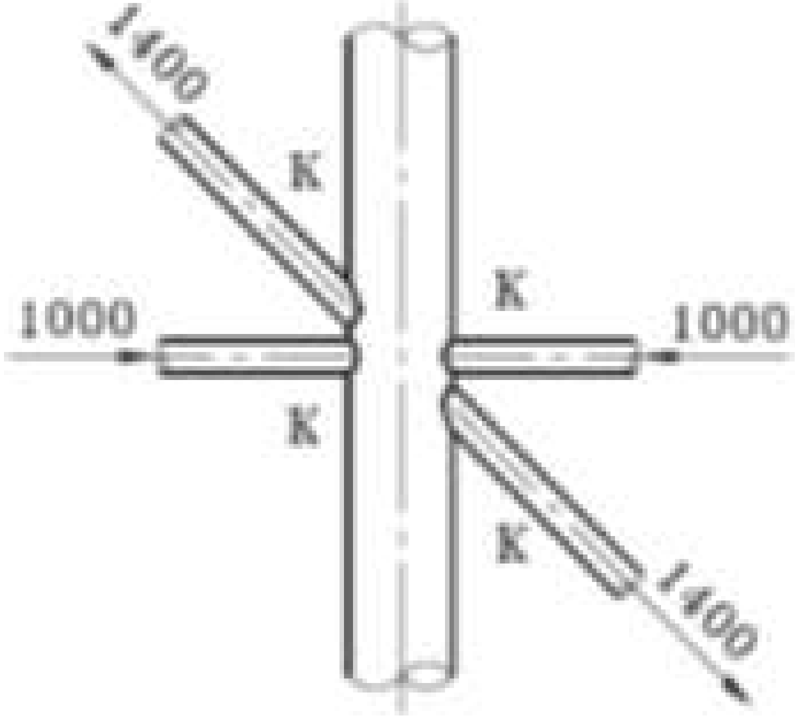
Classification of joints



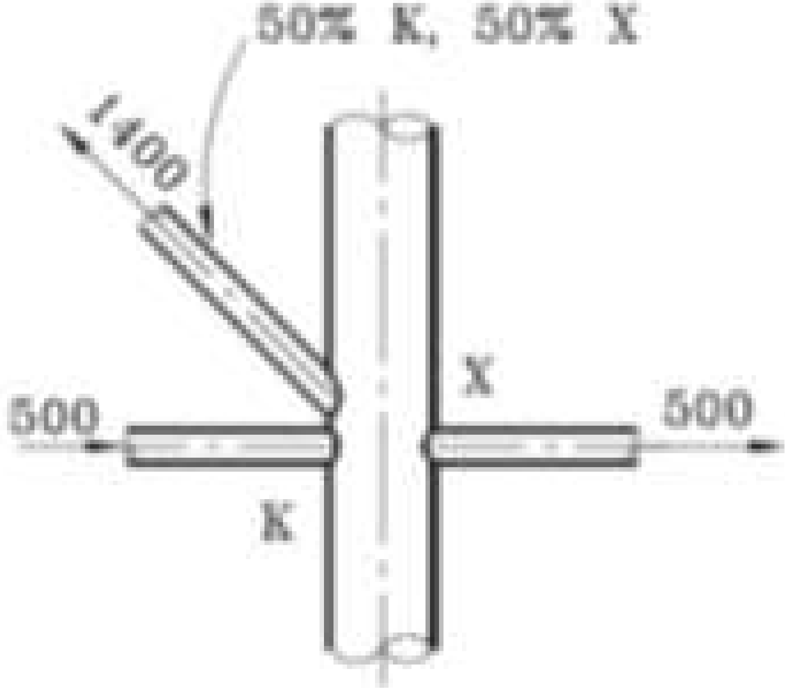

1400
K
GAP
1000
K
(a)
1400
Y
1000
(b)
50% K, 50% Y
1400
500
K
500
(c)



(d) (e) (f)

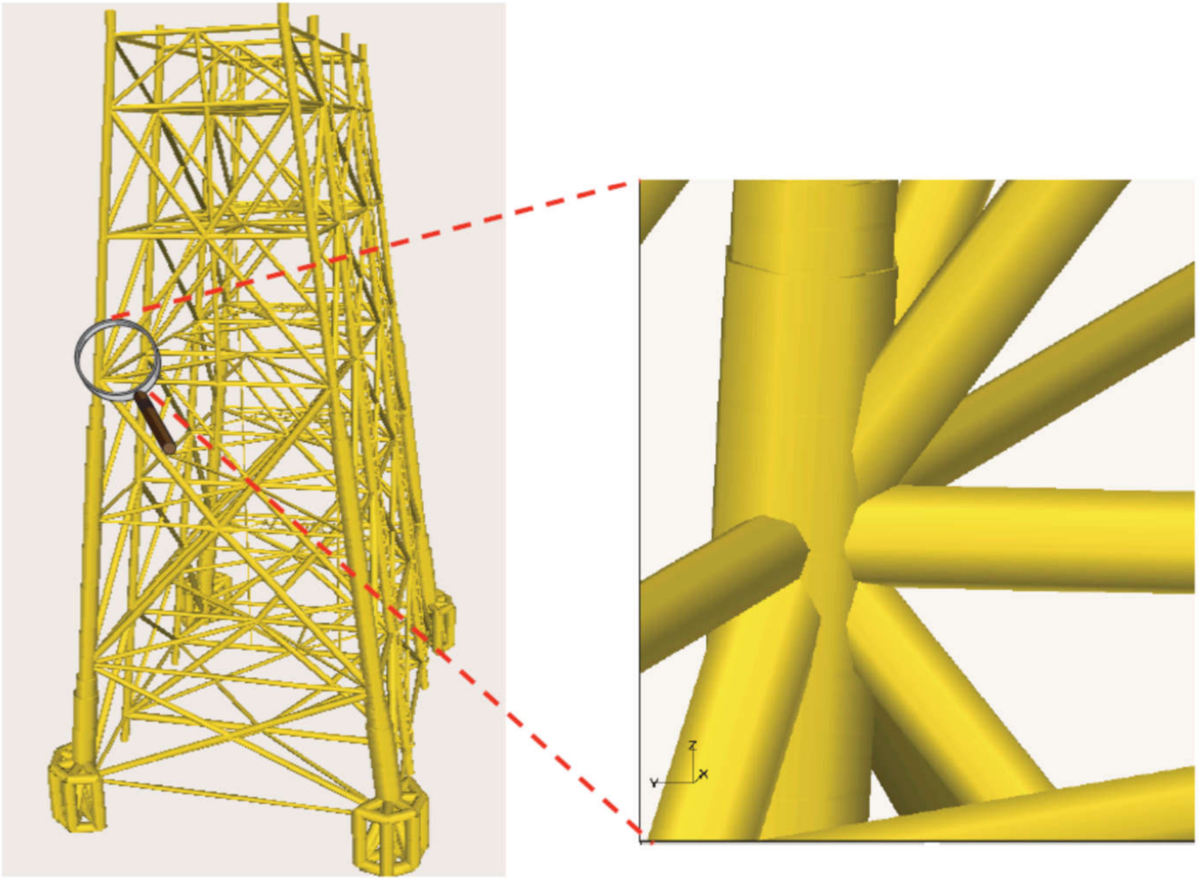


(g)



(h)

Typical joint in offshore jacket



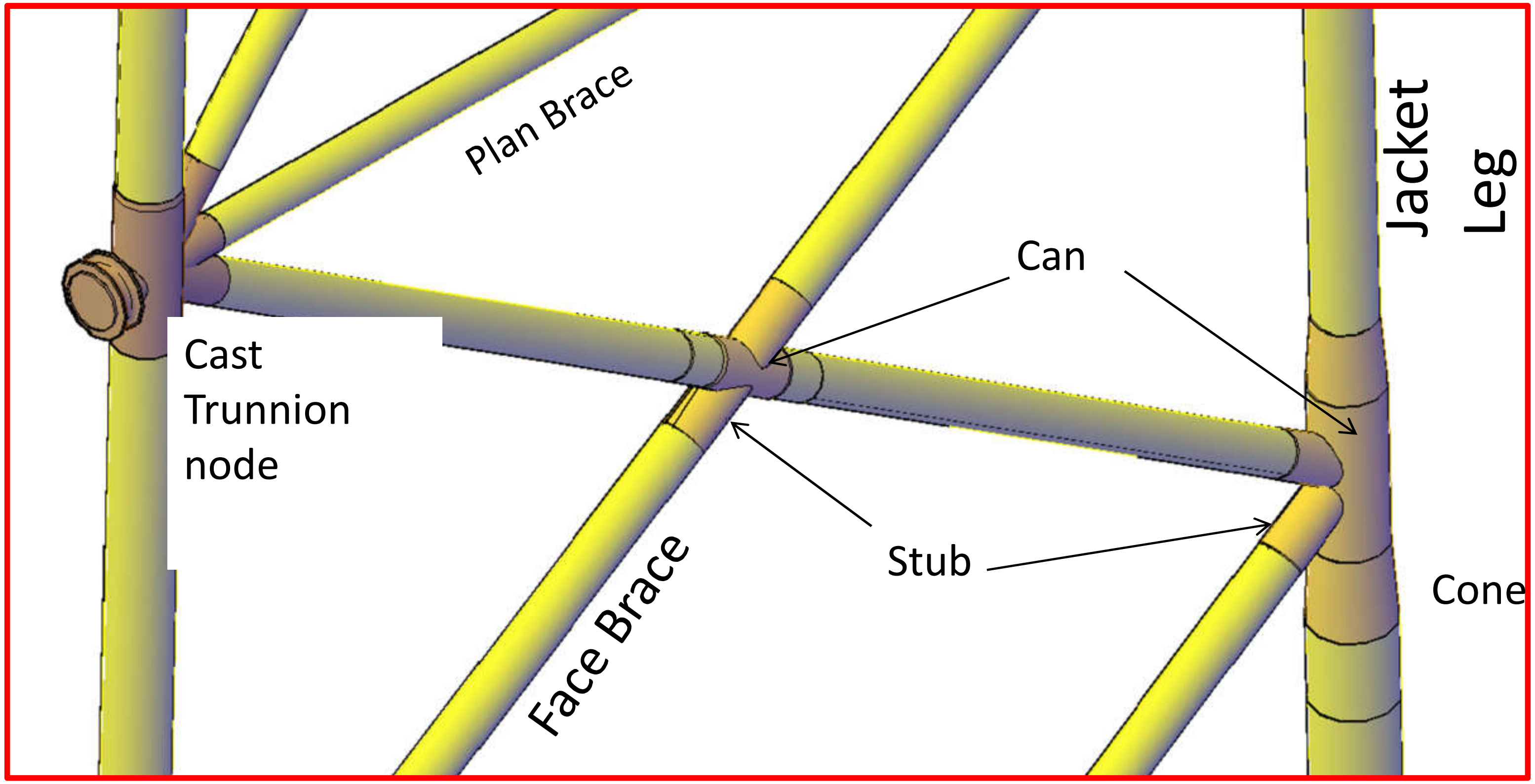
Jacket Komponenter

Stub

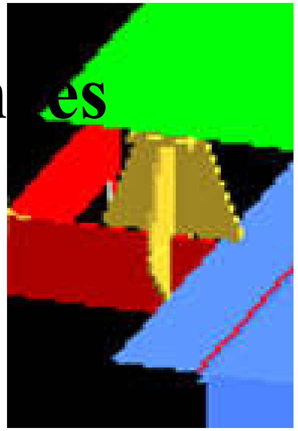
Kort (minimum 600 mm lang) del av røret som sveises mot can'en. Brukes for å få mulighet til å sveise på innsiden av røret.

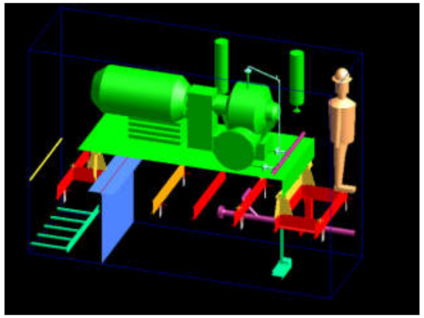
Can

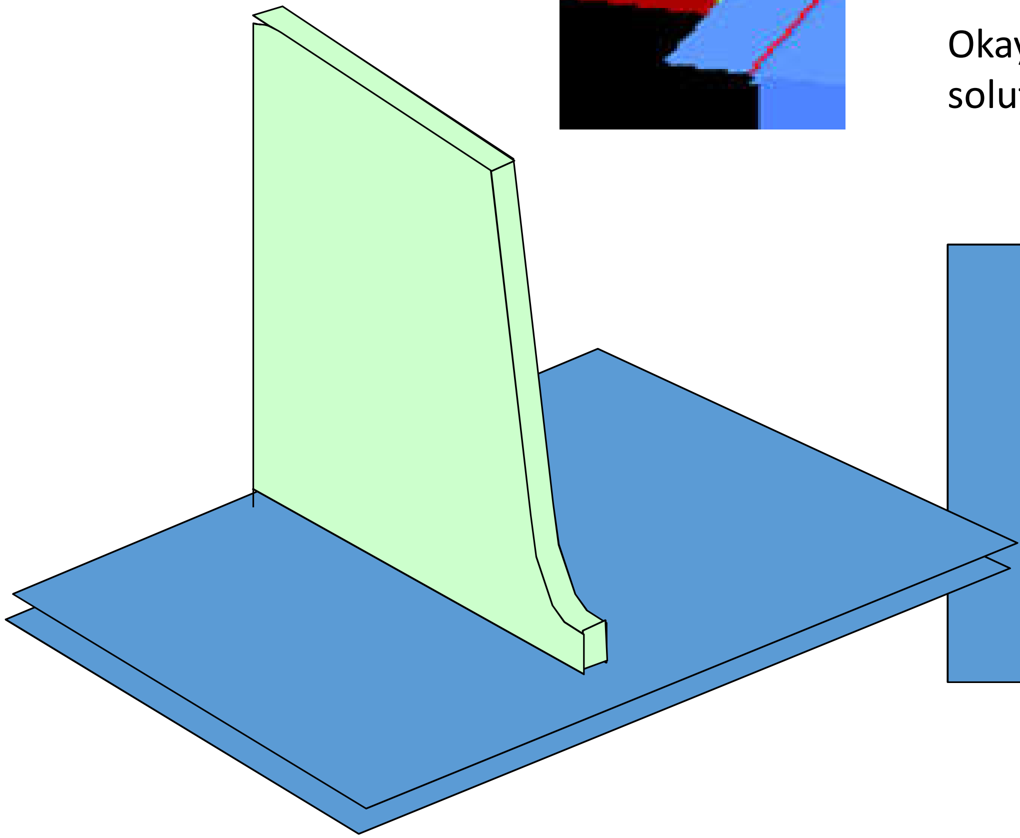
Lokal forsterkning av et rør i et knutepunkt.



Some examples of design challenges



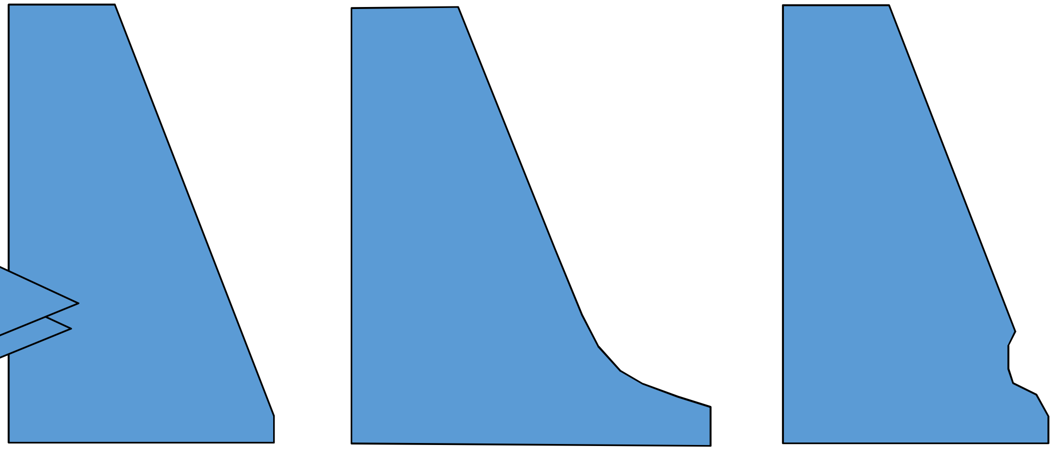




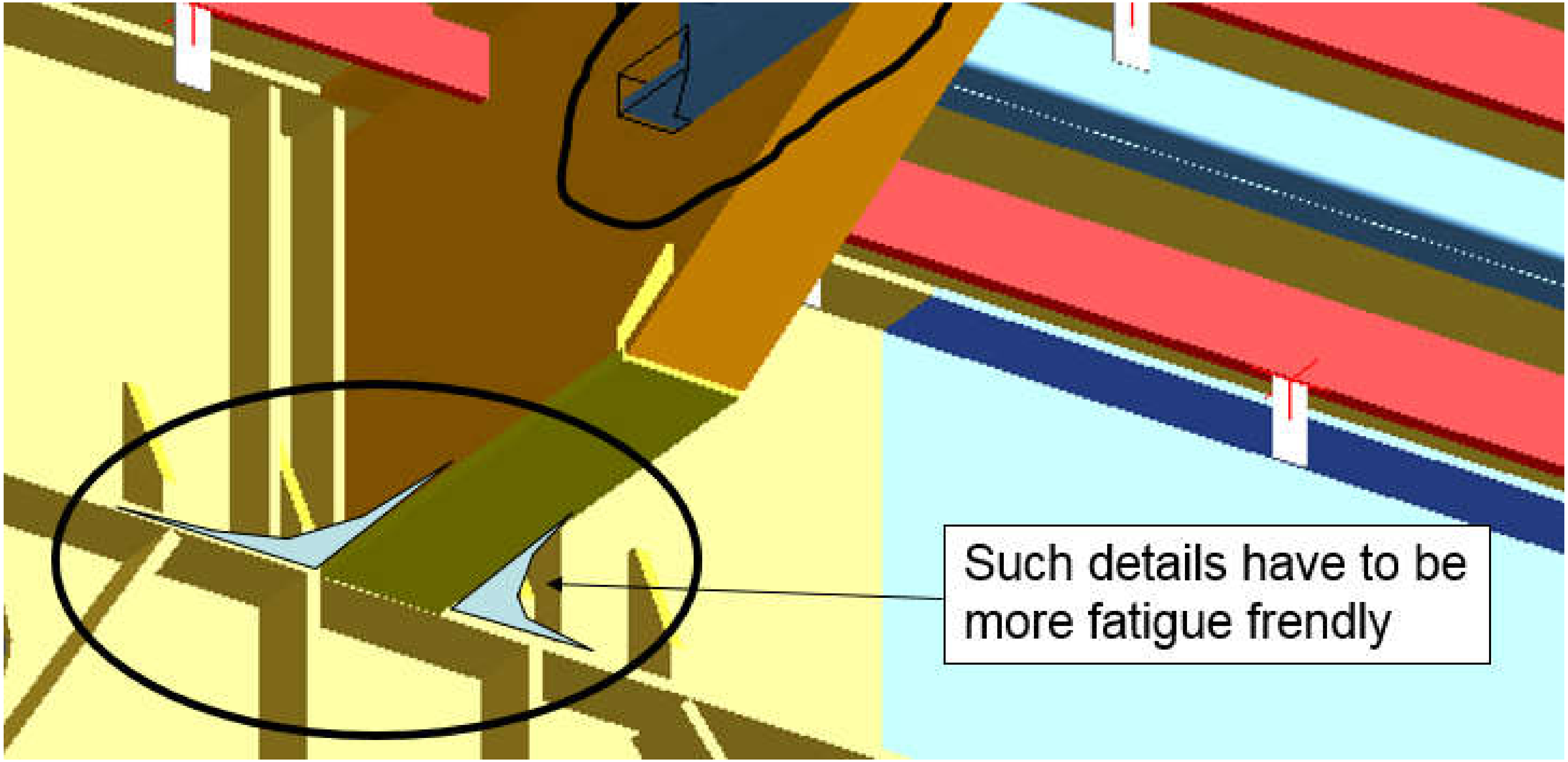
Okay solution

Preferable solution for fatigue

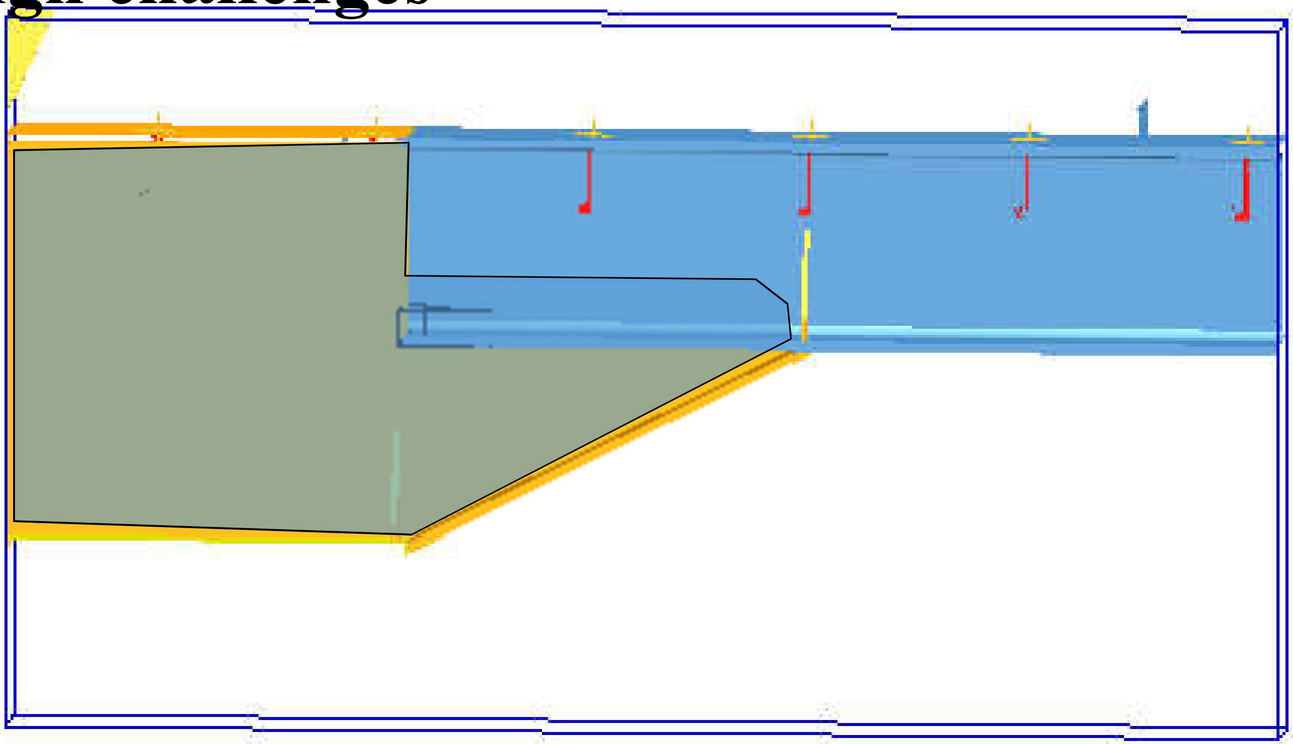
If detail critical and already build solution



Some examples
of design challenges

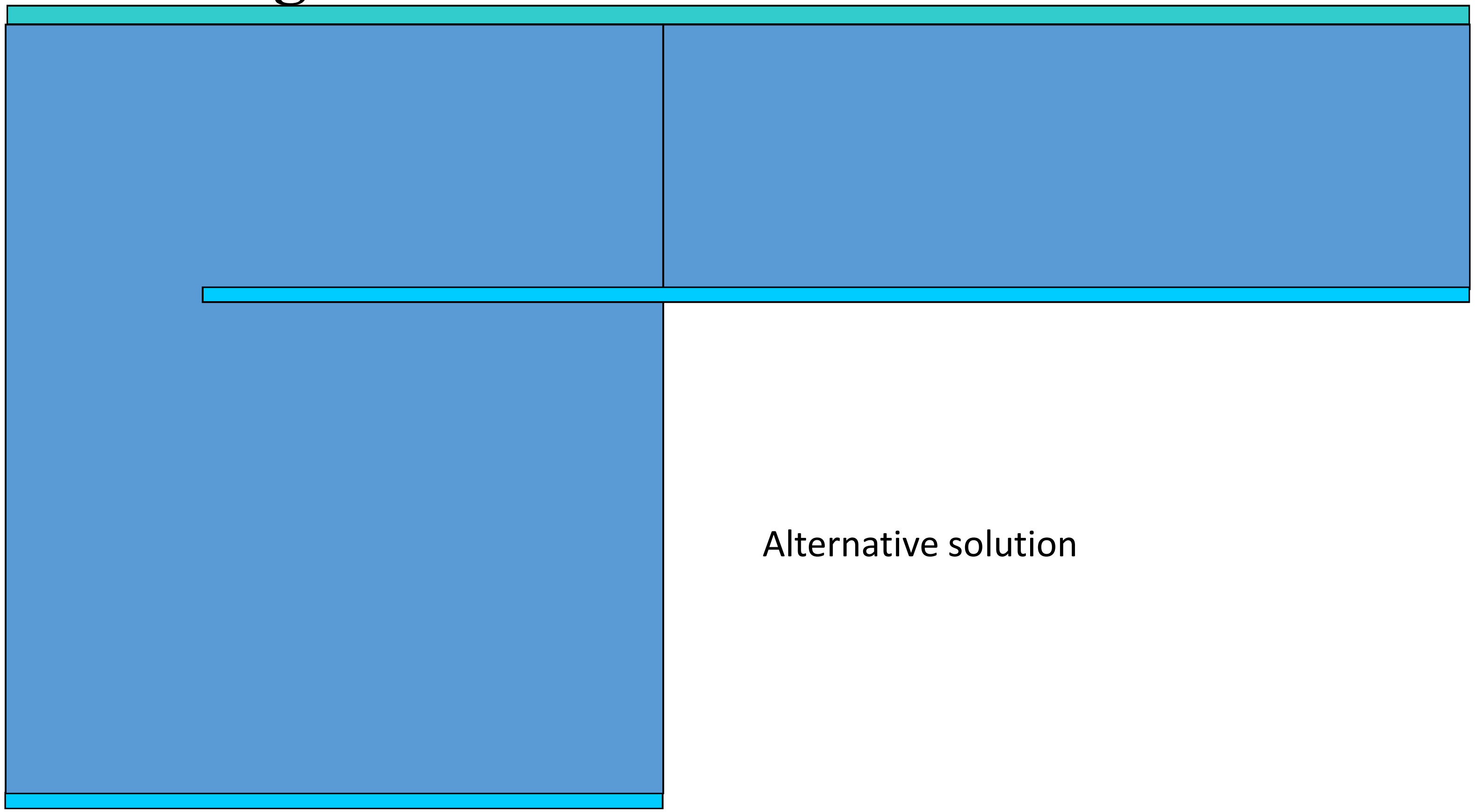


Some examples of design challenges



To avoid weld on weld

Some examples
of design challenges



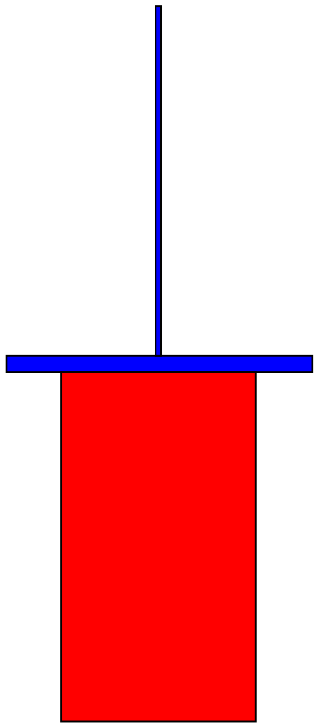
Alternative solution

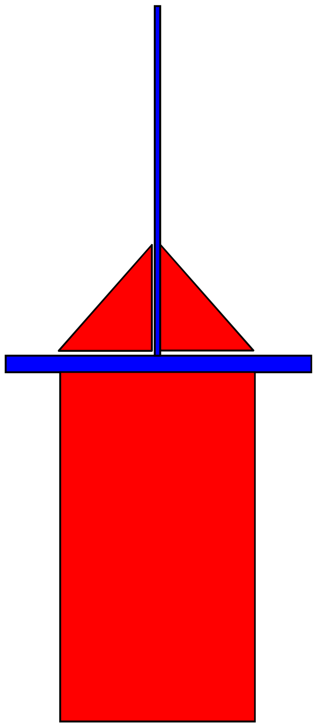
Some examples of design challenges

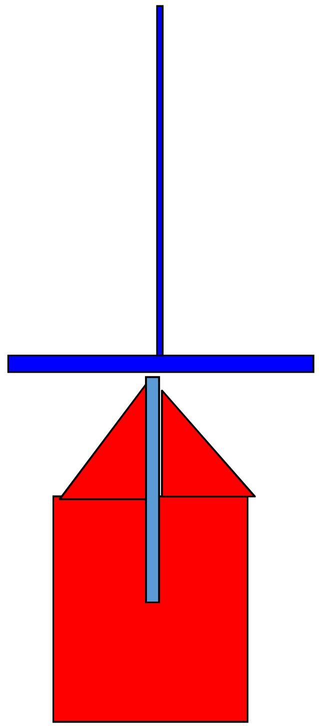
Can you check this critical detail?

Implementation Alternative 1 SHS Infill?

Or implement a gusset plate solution?







Some examples
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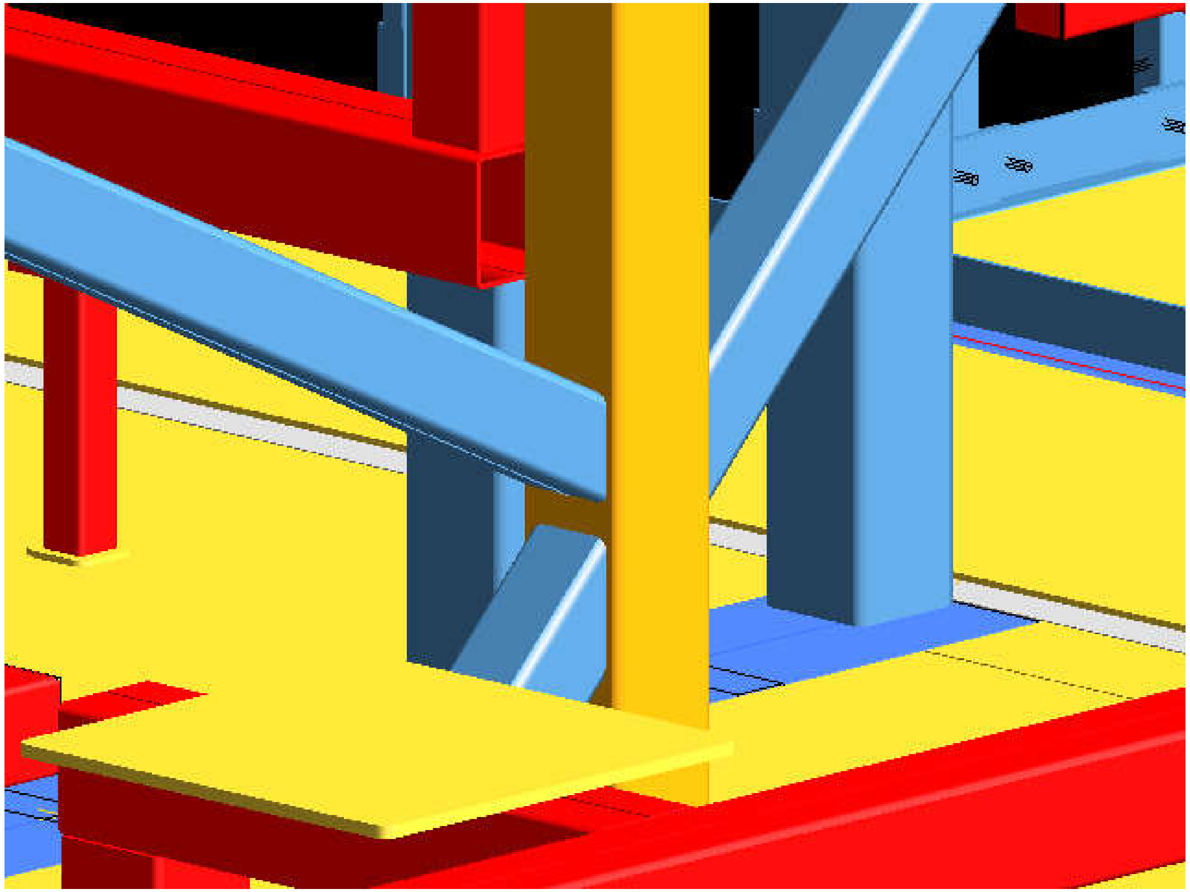

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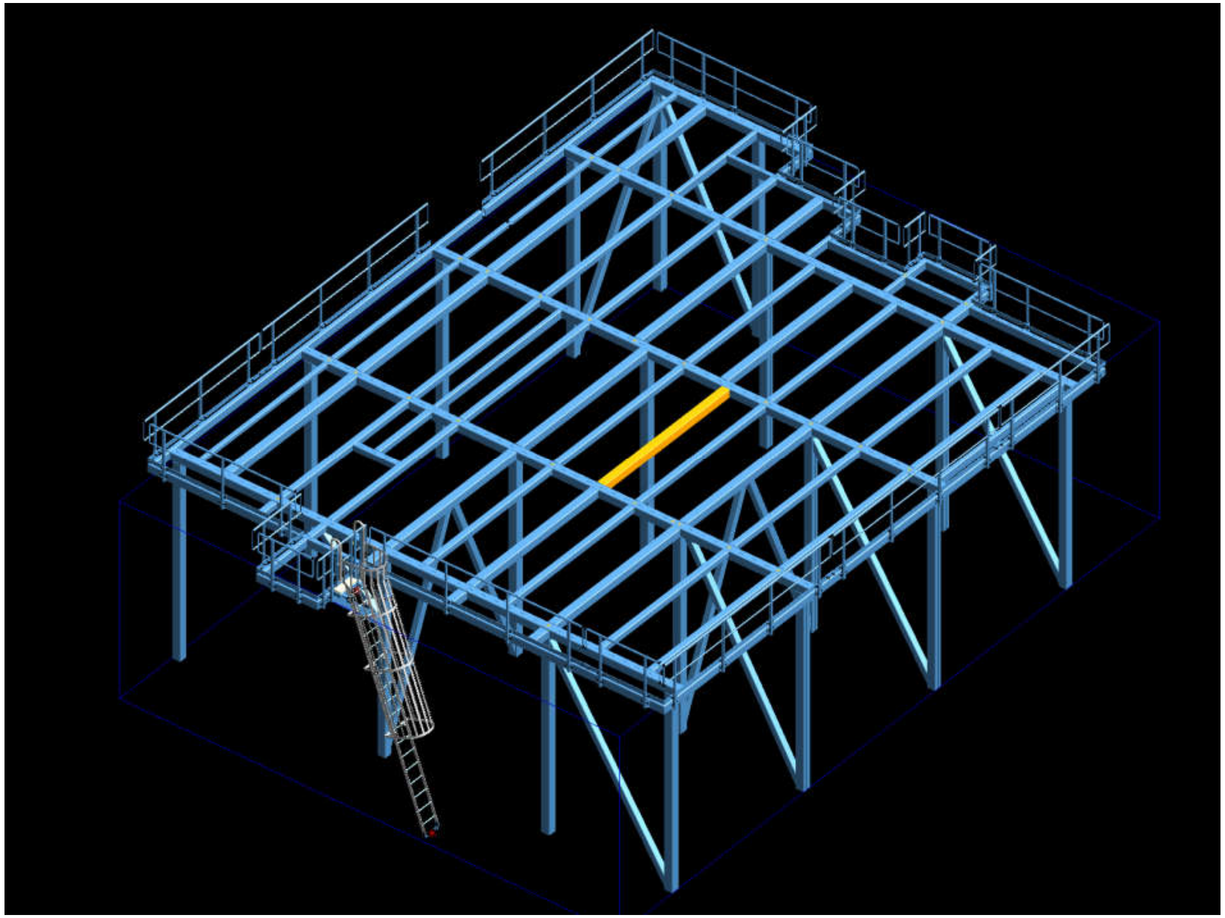
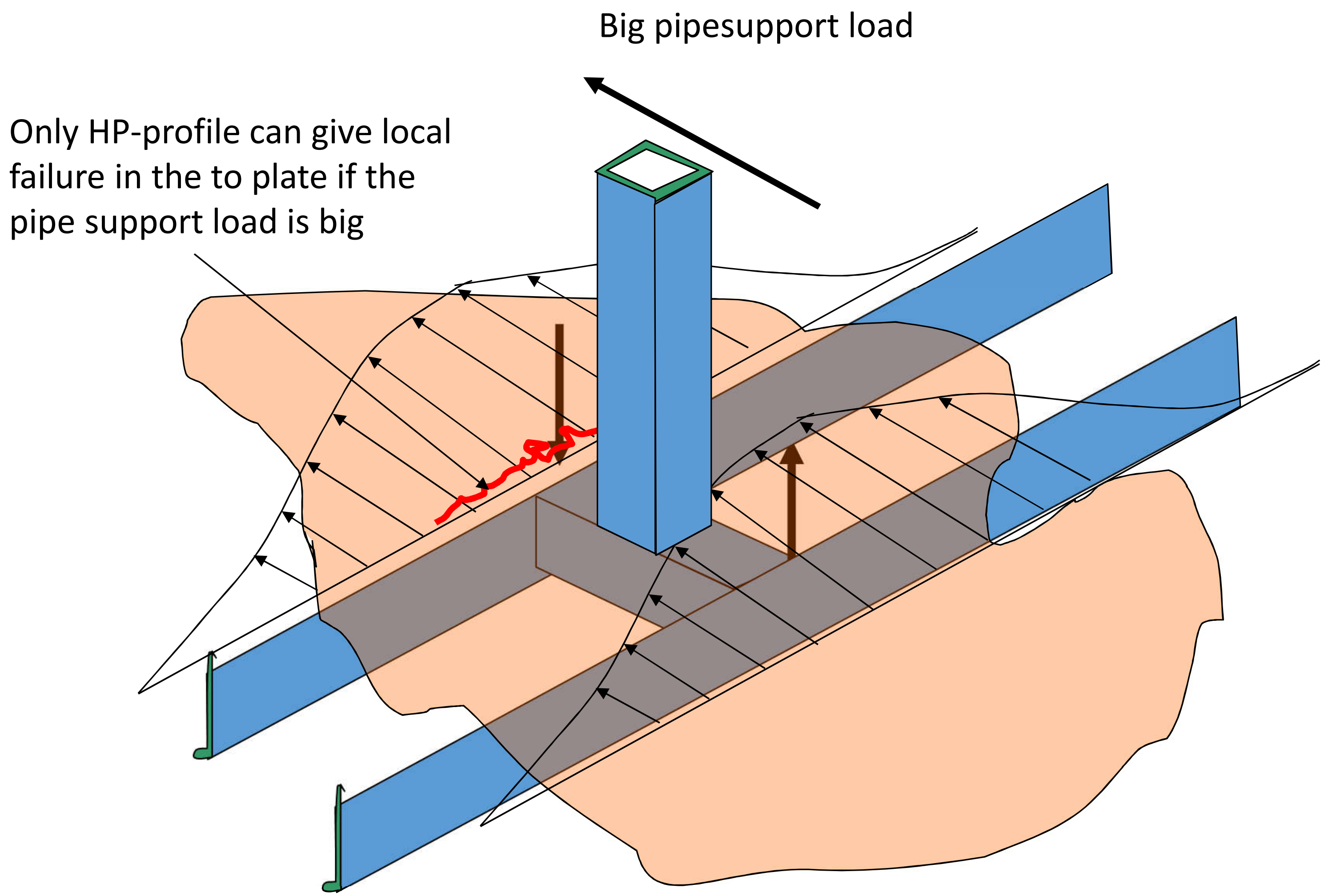
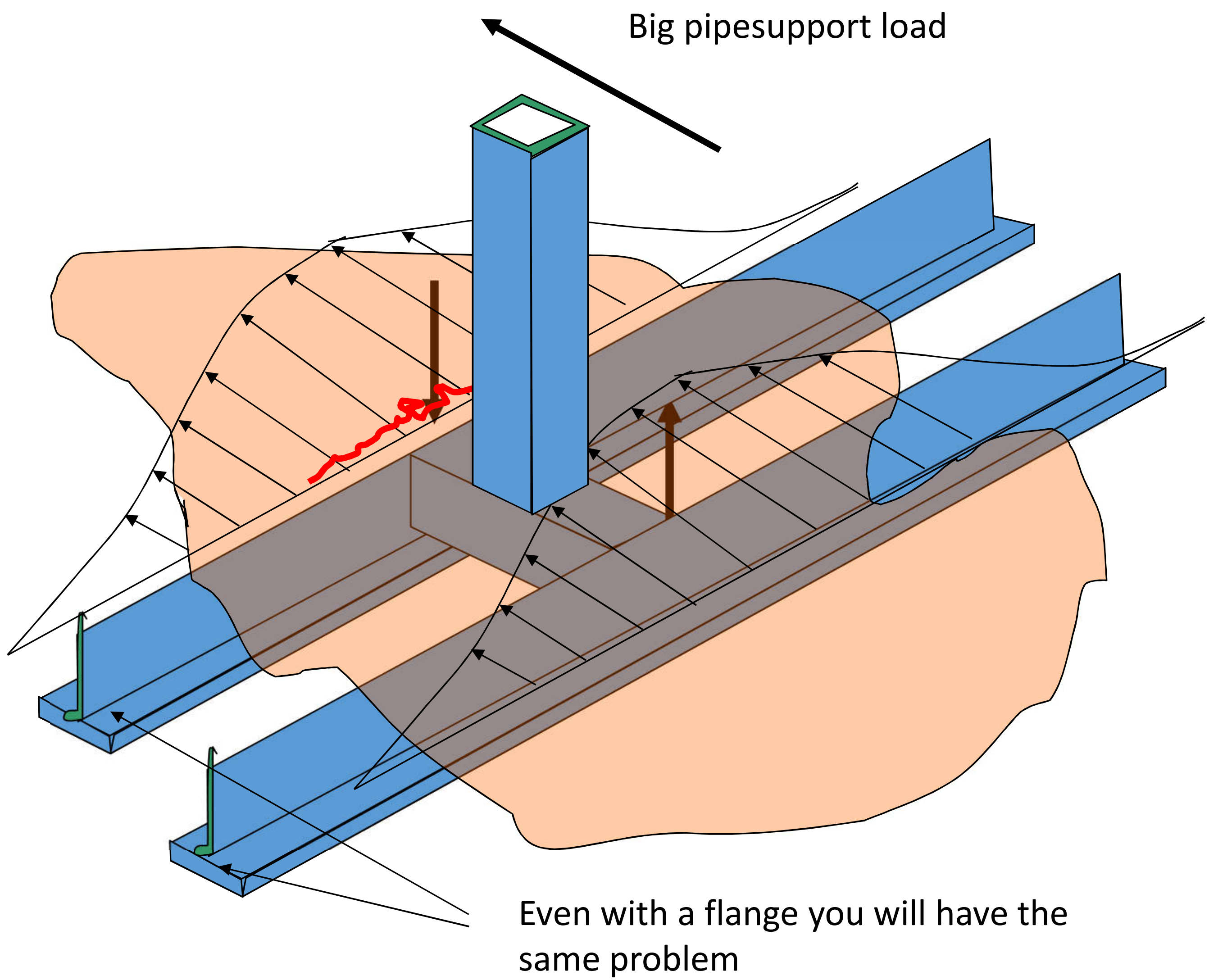
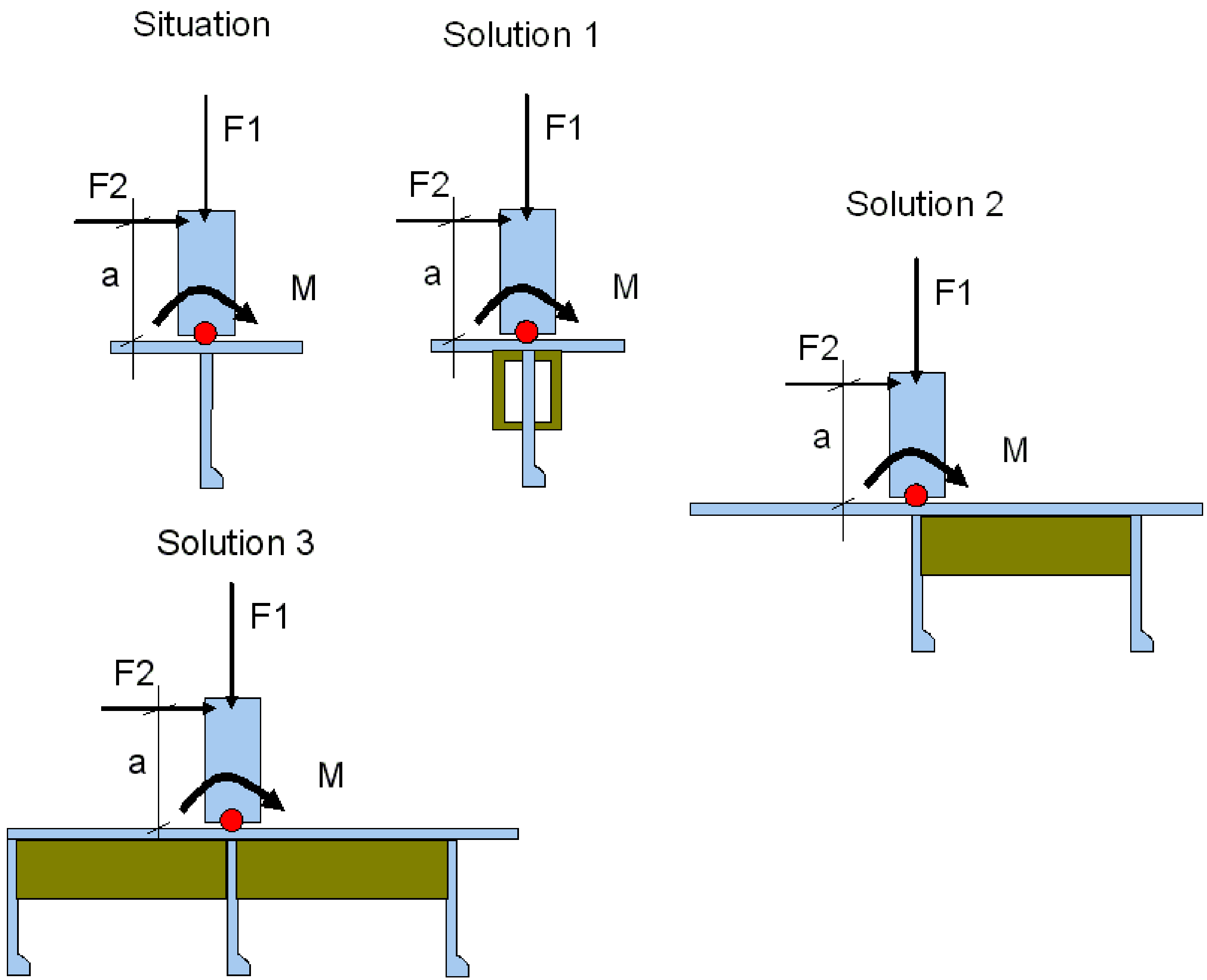


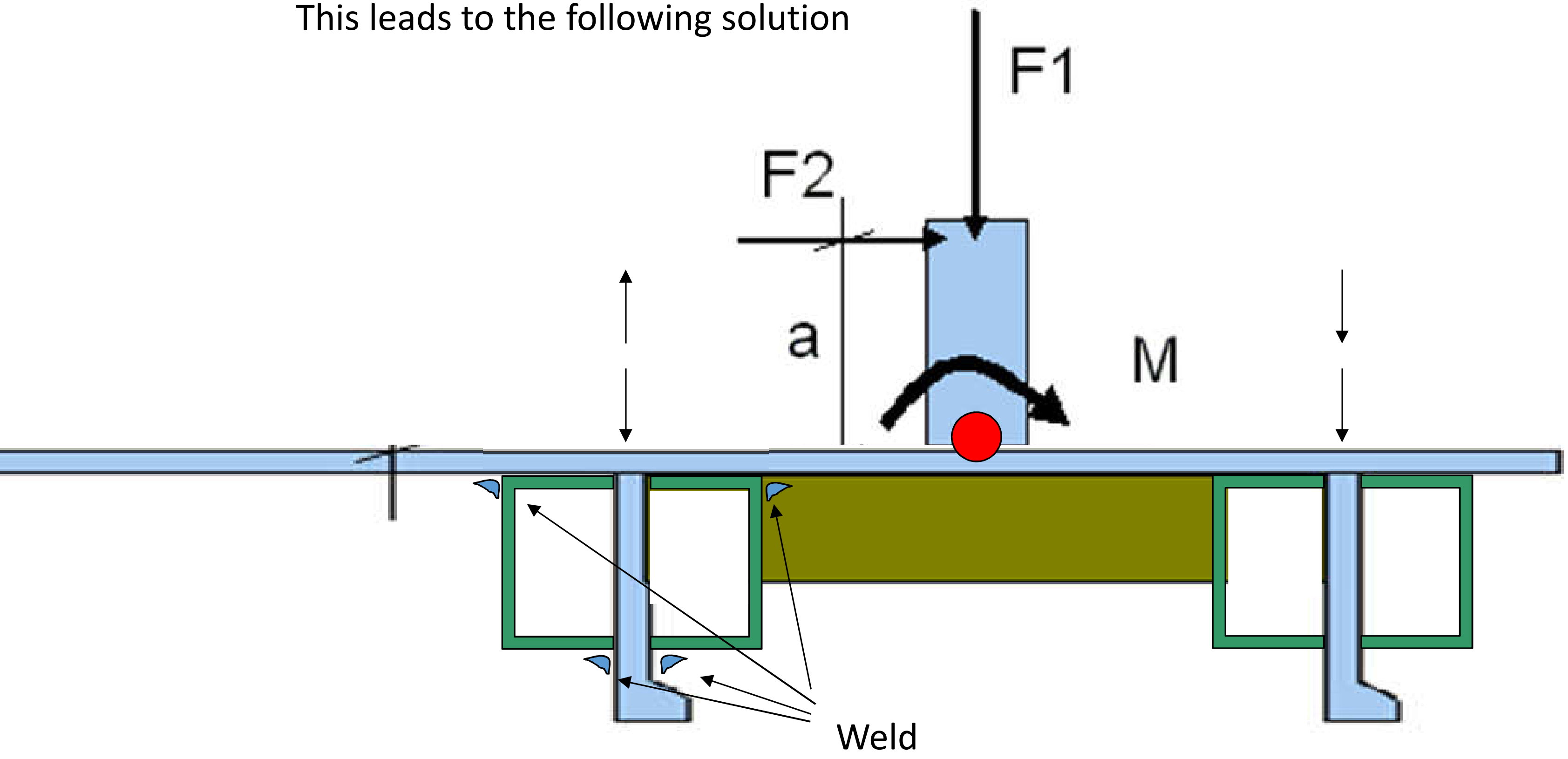

Big pipesupport load
Only HP-profile can give local failure in the to plate if the pipe support load is big

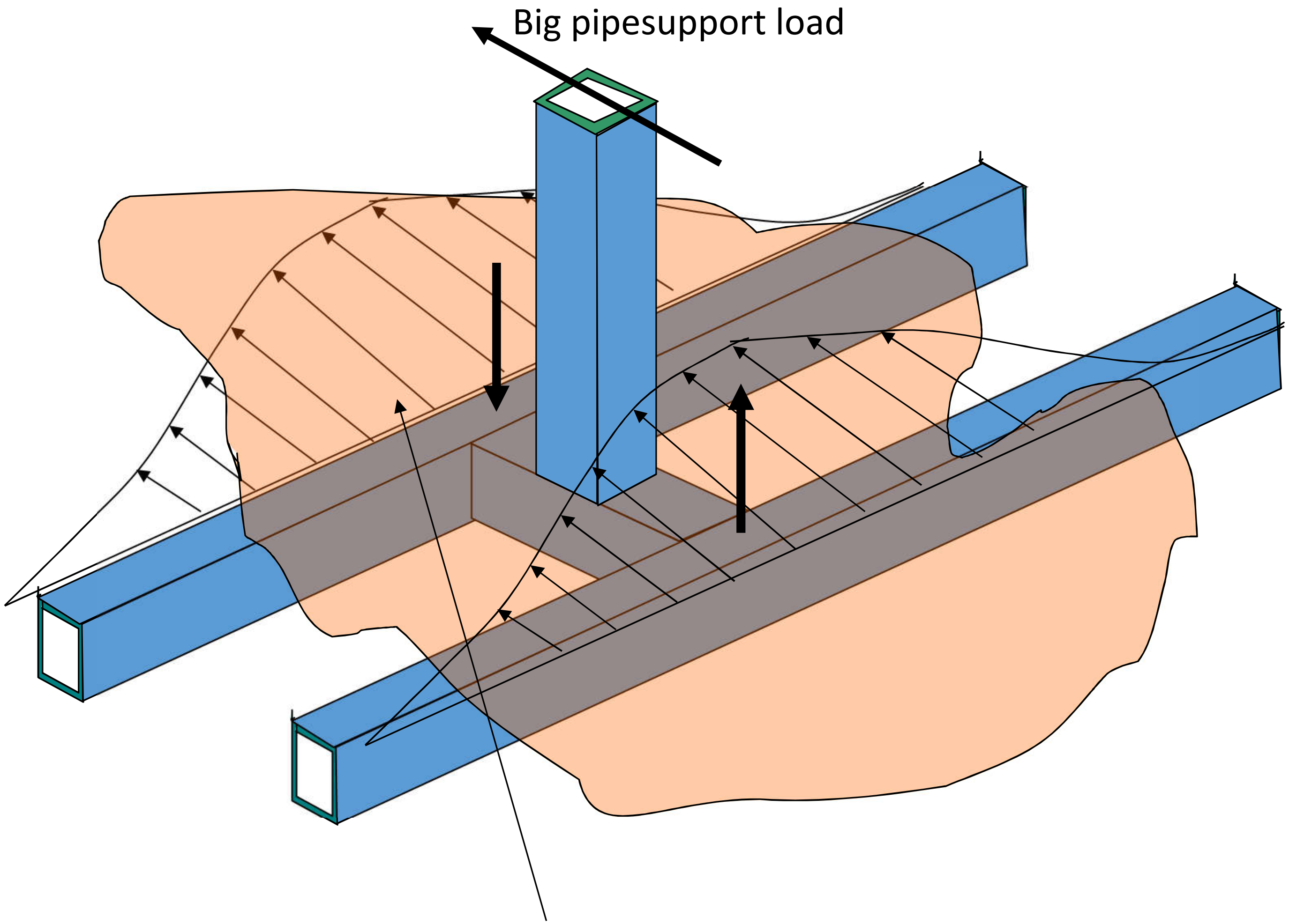

Big pipesupport load
Even with a flange you will have the same problem



Situation
F1
F2
a
M
Solution 1
F1
F2
a
M
Solution 2
F1
F2
a
M
Solution 3
F1
F2
a
M

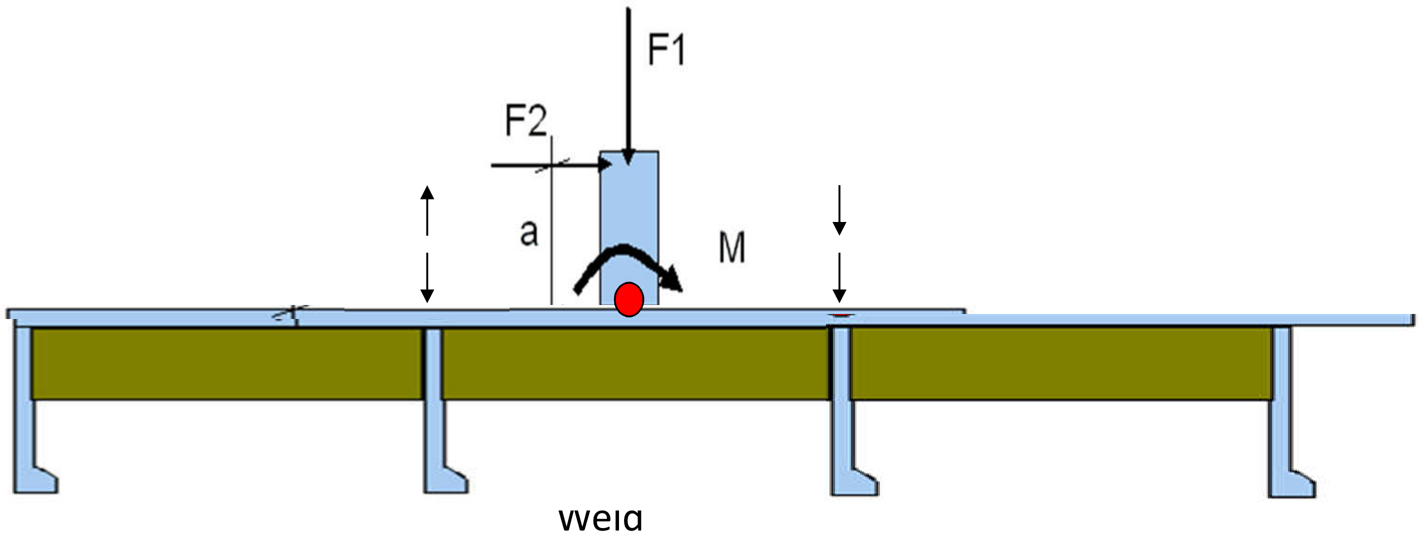
This leads to the following solution



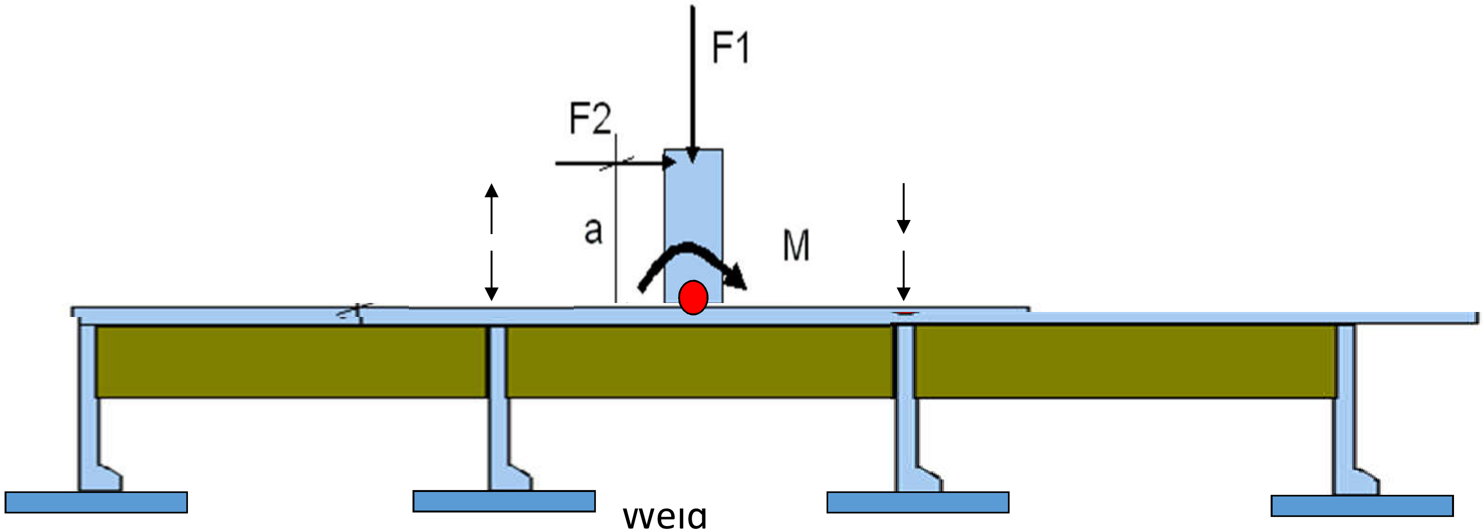


Hollow section is better in order to
to take the horizontal shear force and moments
at the horizontal direction.

But this can be replace with the following (see sketch below) which efficent with respect of less welding and less complex end solutions



If the hp stiffener is not strong enough the most efficent is to strengthen the profile as shown below:



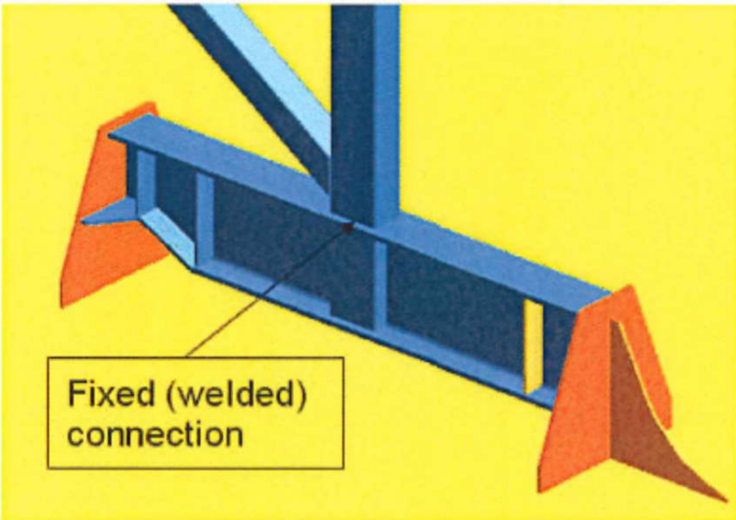


Figure: Fixed piperack support

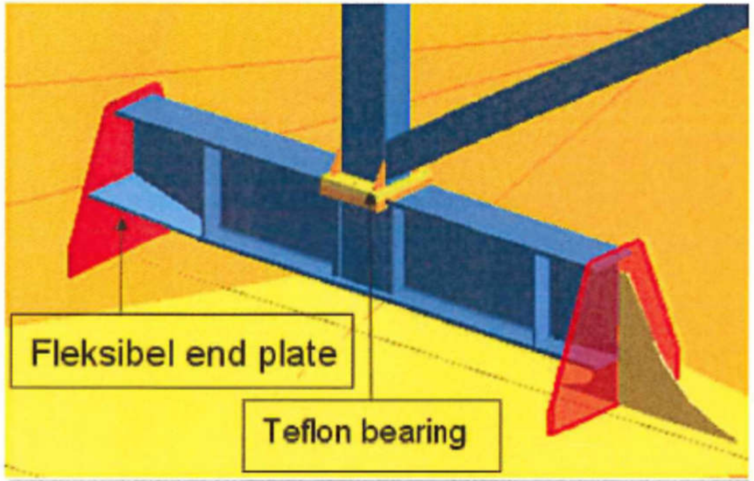
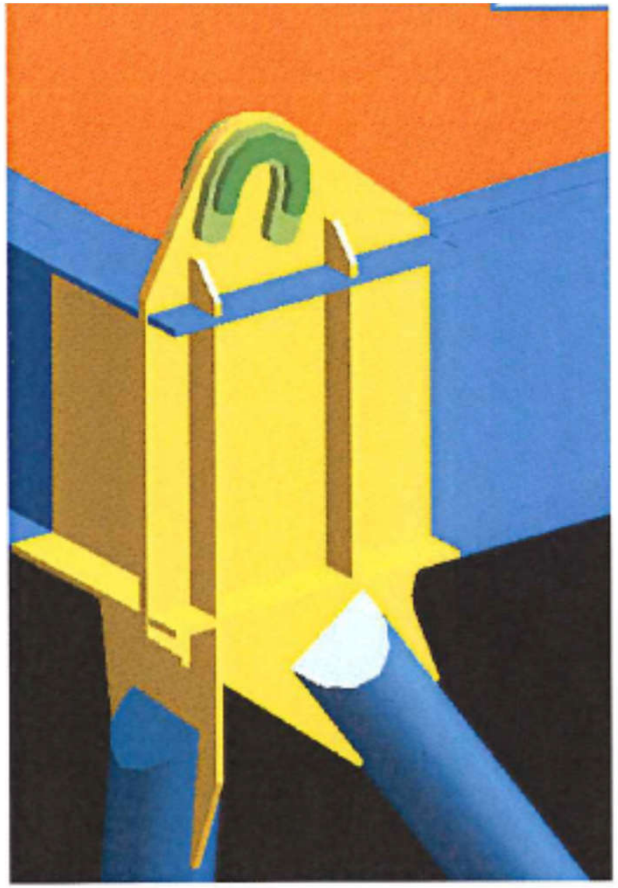


Figure: Sliding piperack support



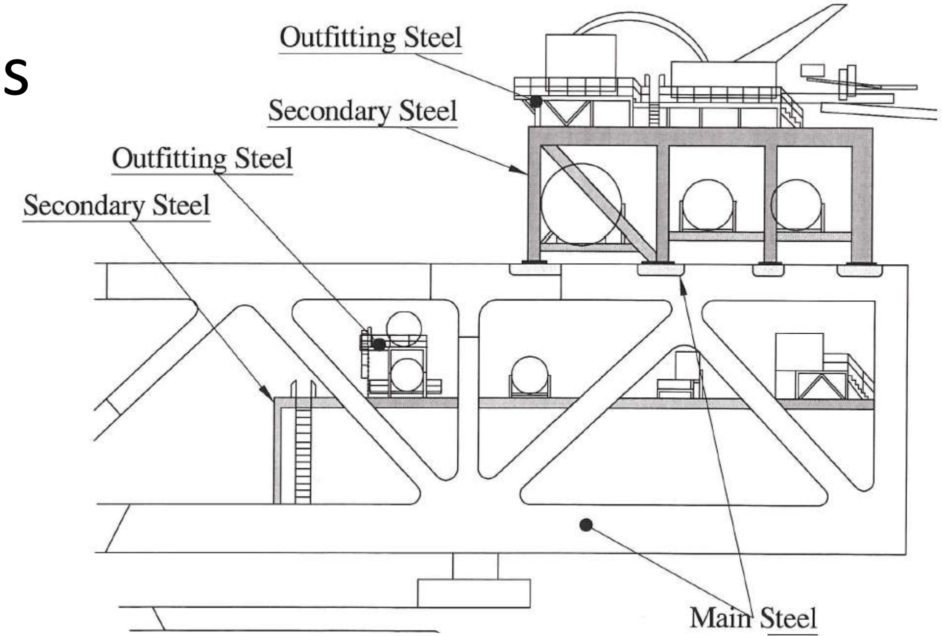
TBYG3018 Design of Offshore Structures

Modules in "Design of Offshore Structures"

Typical Topics «Design Basis Offshore Steel Structure»

Structural Disiplines

- Main steel
- Secondary steel
- Outfitting steel

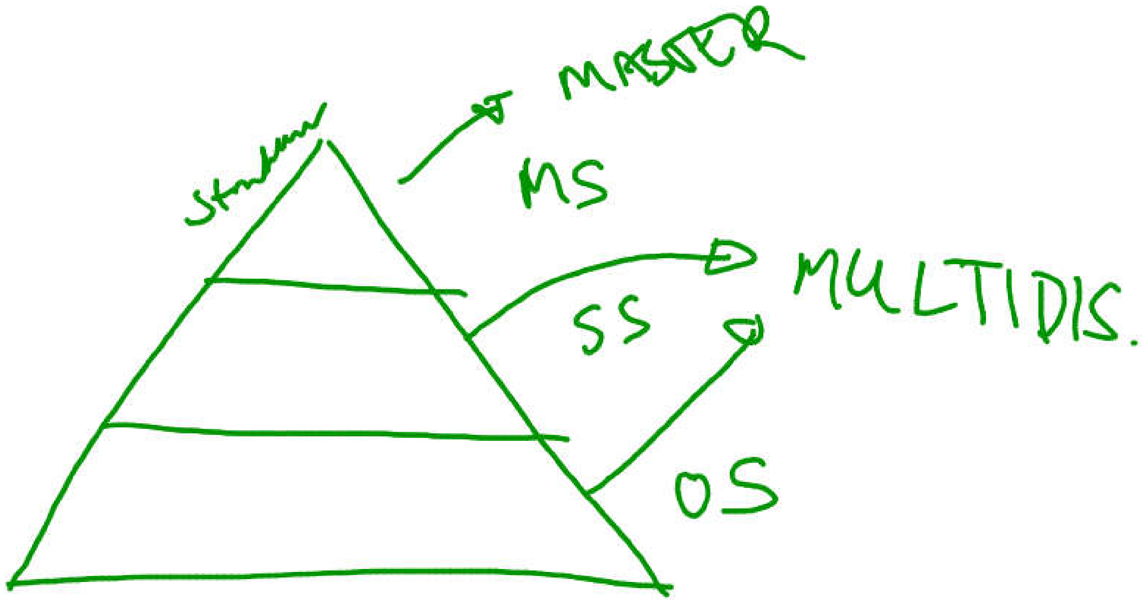


The definition of main steel as used in this book is the structure that is designed to withstand the global load, both static gravity loads, dynamic loads from environmental forces and dynamic loads from accelleration of the masses.

Secondary steel is made to transfer loads from different areas and onto the main steel. Secondary steel capacities are not included in the main strength analysis of the platform.

Outfitting steel has some of the functions of secondary steel, but is specialised (seatings for equipment etc, or ladders, handrails or other specialities)

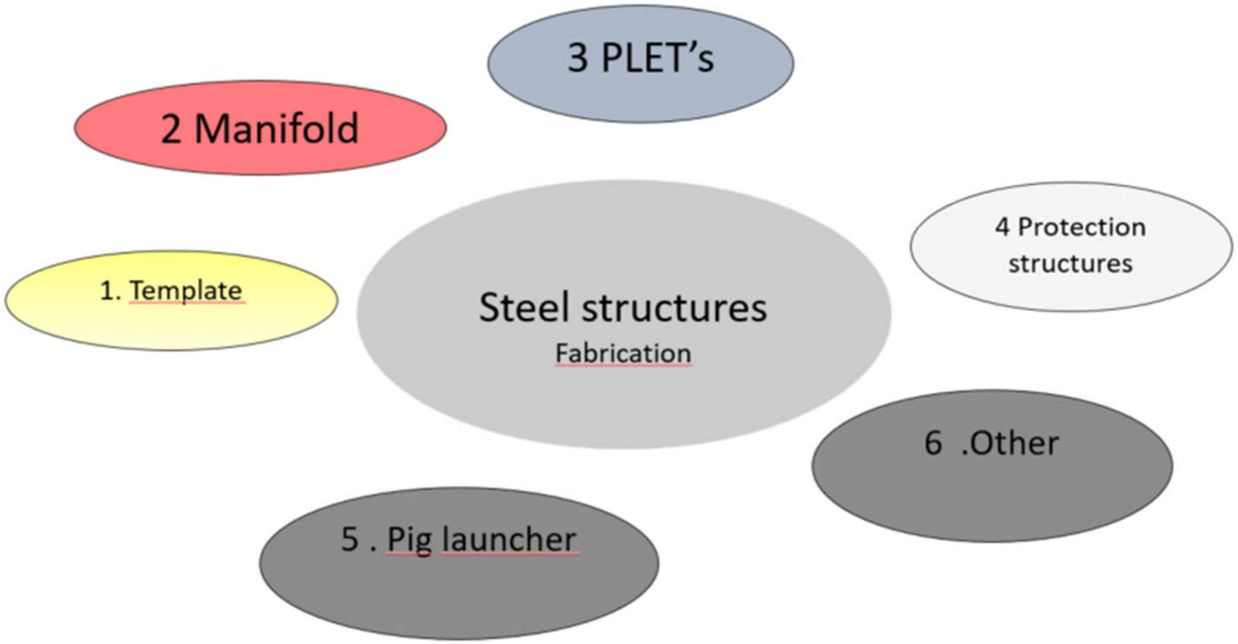
- Main steel
- Secondary steel
- Outfitting steel – more multidisipline. Smaller structures.



Oil&Gas MARKET SEGMENTATION

A	Subsea facilities	Subsea Foundations	Subsea Structures	Umbilicals, Riser, Flowlines	Offshore power cables	Pipelines	
C	Fixed steel platforms	Living Quarters	Utilities, Power & Process Modules	Deck Structures	Flares & drilling modules	Jackets Jack-ups	Foundations & Monotower
D	Floating steel units	Living Quarters	Utilities, Power & Process Modules	Deck Structures	Flares & drilling modules	Hulls	Mooring & Anchors

Subsea Structural and mechanical tasks



Codes / Design Basis and Design Briefs / Certifier

The design basis for the project consists of Parts A, B and C:

The Design Basis Part A includes:

- Part A.1: General design requirements
- Part A.2: Hydrodynamic and morphological design basis
- Part A.3: Geophysical and geotechnical factual data reports
- Part A.4: Site specific wind data

The Design Basis Part B consists of:

- Part B: Northwind OWF

The Design Basis Part C consists of:

- Part C: Integrated data for detailed design

INTERFACE REPORTS:
to WTG SUPPLIER

The Design Briefs are:

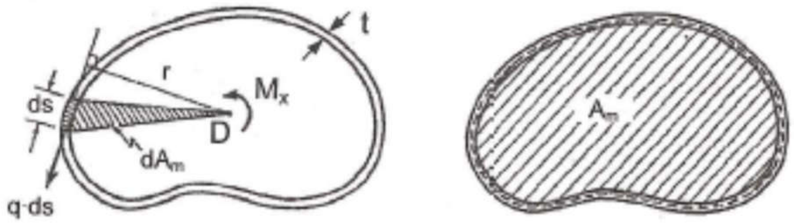
- 3-1 Design Brief - Geotechnical Data Interpretation
- 3-2 Design Brief - Extreme Operational Event
- 3-3 Design Brief - Fatigue Analysis
- 3-4 Design Brief - Natural Frequency
- 3-5 Design Brief - Grouted Connection
- 3-6 Design Brief - Ship Impact
- 3-7 Design Brief - Transportation
- 3-8 Design Brief - Installation
- 3-9 Design Brief - Dismantling
- 3-10 Design Brief - Design Primary Structures
- 3-11 Design Brief - Design Secondary Structures
- 3-12 Design Brief - Design Provisional Structures
- 3-13 Design Brief - Design Elastomeric Bearings
- 3-14 Design Brief - Corrosion and Cathodic Protection
- 3-15 Design Brief - Scour Protection
- 3-16 Design Brief - Fabrication
- 3-17 Design Brief - Operation & Maintenance
- 3-18 Design Brief - Quality Control (Fabrication and installation)
- 3-19 Design Brief - Hydrodynamic Coefficients
- 3-20 Design Brief - Driveability and Driving-Induced Fatigue Analysis
- 3-21 Design Brief - Damping Ratio

LICENGINEERING A/S

TBYG3018 Design of Offshore Structures

Module 4 – Design of Steel Structures according to NORSOK

Jomar Tørset, Assistant professor



Figur 4.11 Definisjonsskisse for tynnvegget rørtverrsnitt

Figur 4.11 viser at momentbidraget fra skjærstrømmen q på et infinitesimalt element ds om et vilkårlig punkt D er

$$dM_x = q\,ds \cdot r$$

hvor r er avstanden fra D til elementet. Arealinkrementet $dA_m = ds \cdot r/2$, og integrasjon langs tverrsnittets periferi gir for et tverrsnitt med vilkårlig form og veggtykkelse

$$M_x = \oint q ds \cdot r = q \oint r \cdot ds = q \cdot 2A_m \qquad (4.13)$$

Her er A_m arealet omsluttet av tverrsnittets senterlinje. Ligning (4.14) gir *Bredts 1.formel*:

$$q = \tau \cdot t = \frac{M_x}{2A_m} \qquad (4.14)$$

Dersom veggtykkelsen varierer er den maksimale skjærspenning gitt ved

$$\tau_{maks} = \frac{q}{t_{\min}} = \frac{M_x}{2A_m t_{min}} \tag{4.15}$$

Torsjonskonstanten I_T finnes ved en energibetraktning hvor arbeidet av det ytre moment M_x settes lik det indre arbeid av skjærstrømmen q. Den tangentielle forskyvning av et infinitesimalt element $dx \cdot ds$ er gitt ved $dv = \gamma \cdot dx$, og man får dermed

$$\frac{1}{2} M_x \cdot d\varphi = \frac{1}{2} \oint q ds \cdot dv = \frac{1}{2} \oint (\tau t \cdot ds)(\gamma \cdot dx) = \frac{1}{2} \oint \frac{\tau^2 t^2}{Gt} ds \cdot dx = \frac{1}{2} \frac{q^2}{G} \oint \frac{ds}{t} dx$$

eller

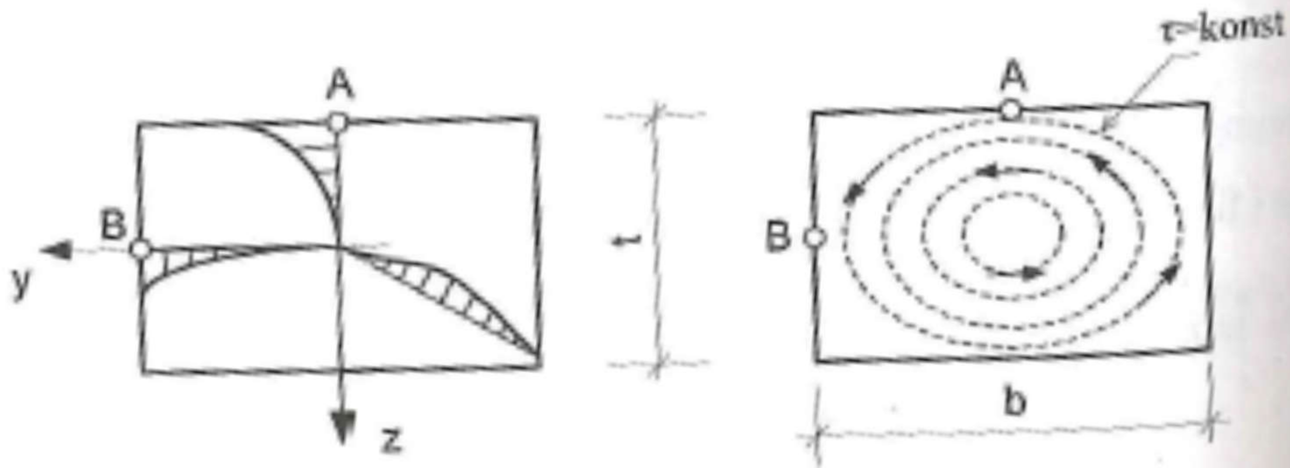
$$\theta = \frac{d\varphi}{dx} = \frac{1}{M_x} \frac{q^2}{G} \oint \frac{ds}{t} = \frac{M_x}{GI_T} \tag{4.16}$$

Dette gir *Bredts 2. formel*:

$$I_T = \frac{4A_m^2}{\oint \frac{ds}{t}} \tag{4.17}$$

begge sideflatene som ...

hjørnet heller ikke ha noen komponent i tverrsnittets plan. Som ...



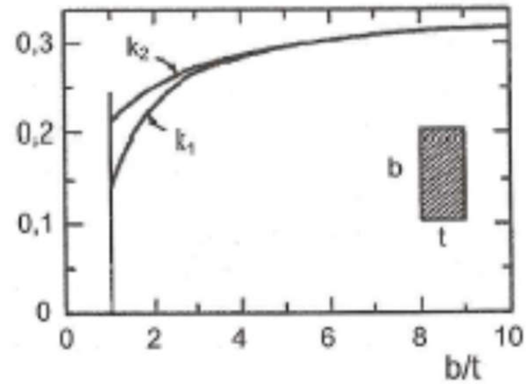
Figur 4.4 Skjærspenningens fordeling og retning i rektangulært tverrsnitt

Den maksimale skjærspenning τ_{maks} og St Venants torsjonskonstant I_T kan bestemmes numerisk eller ved hjelp av elastisitetsteorien [4.2], og er tilnærmet gitt ved

$$\tau_{maks} = \frac{M_x}{k_2 \cdot bt^2} \quad (b > t) \tag{4.6}$$

$$I_T = k_1 \cdot bt^3 \quad (b > t) \tag{4.7}$$

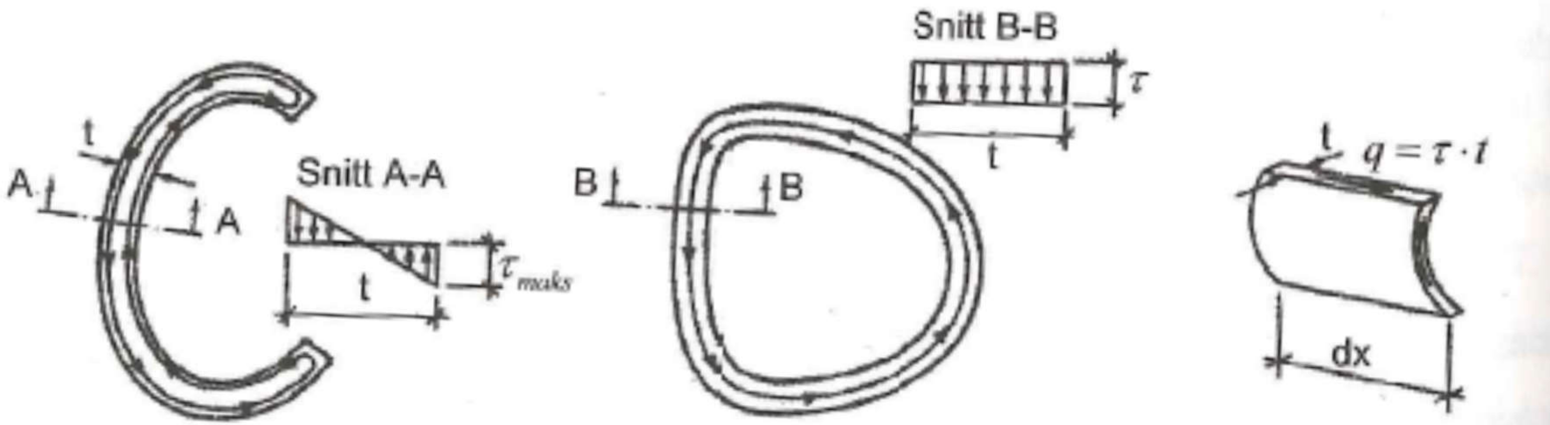
Koeffisientene k_1 og k_2 er gitt i figur 4.5, som viser at koeffisientene → 1/3 når b/t→ ∞, og at denne verdien i praksis kan benyttes når b>>t.



Figur 4.5 Torsjonskoeffisienter for rektangulære tverrsnitt

De vanligste profiler i stålkonstruksjoner er sammensatt av en rekke rette eller krumme, tynnveggede plateelementer, figur 4.6. Torsjonsegenskapene av slike tverrsnitt bestemmes ved at man antar at tverrsnittsdel nr "i" opptar andelen $M_{x,i}$ av M_x. Da tverrsnittsformen opprettholdes under deformasjonen får samtlige plateelementer samme rotasjon $\theta_i = \theta$. Hvis alle elementene også har samme skjærmodul G får man fra ligning (4.8)

$$\theta_i = \frac{M_{x,i}}{(G I_T)_i} = \theta \tag{4.9}$$

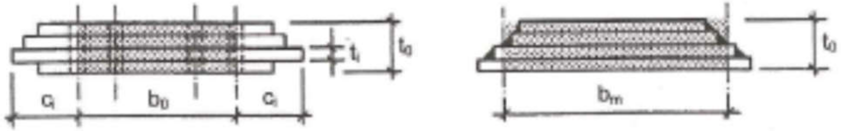


Figur 4.10 Skjærspenningsfordeling i åpent og lukket tverrsnitt

For et lukket, tynnvegget tverrsnitt defineres *skjærspenningsstrømmen* q ved

$$q = \tau \cdot t \tag{4.12}$$

Torsjonskonstanten for en stav som består av to plater med tverrsnittsdimensjoner b og t lagt flate mot flate er lik summen av delplatenes bidrag, dvs $I_T=2\cdot bt^3/3$. Samvirke mellom platene kan oppnås dersom kontaktflaten mellom platene kan overføre skjærspenninger, for eksempel ved hjelp av friksjon. I dette tilfellet er $I_T=2\cdot b(2t)^3/3=16bt^3/3$. Samvirke kan oppnås ved hjelp av forspente skruer eller ved sveising, som vist i figur 4.8.



Figur 4.8 Samvirke mellom plateelementer

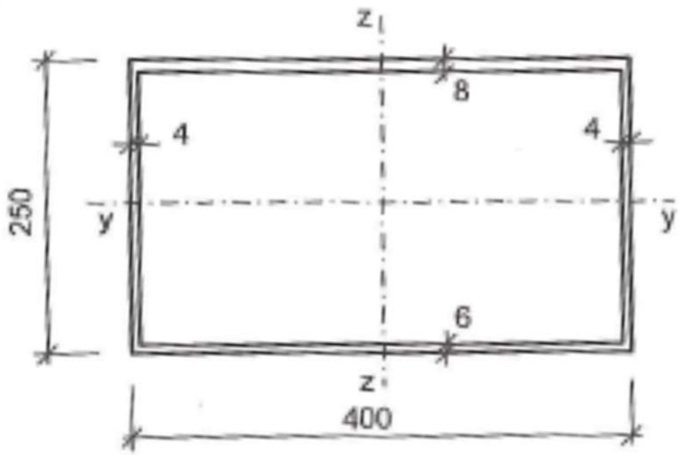
Dersom forspente skruer benyttes for å skape samvirke er det vanlig å anta at den delen av platene som ligger mellom første og siste skrue virker som en homogen plate, mens de utstikkende deler må antas å virke uavhengig av hverandre.

$$I_T \approx \frac{1}{3} b_0 t_0^3 + \frac{1}{3} \sum c_i t_i^3$$

Dersom platene er sveist til hverandre er det vanlig å betrakte platene som en homogen plate med midlere bredde b_m og tykkelse lik summen av platene [4.6]. Her overfører sveisene skjærkraften mellom platedelene.

$$I_T \approx \frac{1}{3} b_m t_0^3$$

Eksempel 4.2 Torsjon av rektangeltverrsnitt



Figur 4.12 Tynnvegget kassetverrsnitt

Det tynnveggete kassetverrsnitt i figur 4.12 er belastet med et torsjonsmoment $M_x = 120$ kNm. Da veggtykkelsen er konstant langs hver av de fire sidekantene er linjeintegralet i Bredt's formel enkel å utføre.

$$A_m = \frac{1}{2}\oint r \cdot ds = 396 \cdot 243 = 96{,}2 \cdot 10^3 \text{ mm}^2$$

Ligning (4.16) gir

$$\tau_{maks} = \frac{1}{t_{min}} \frac{M_x}{2A_m} = \frac{1}{4} \frac{120 \cdot 10^6}{2 \cdot 96,2 \cdot 10^3} = 156 \text{ N/mm}^2$$

Videre has

$$\oint \frac{ds}{t} = \frac{396}{6} + \frac{396}{8} + 2\frac{243}{4} = 237$$

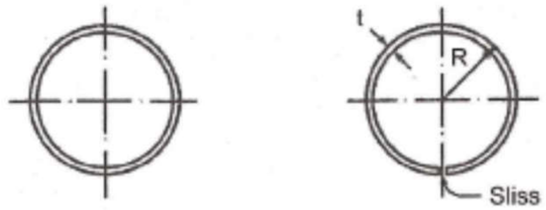
og torsjonskonstanten er dermed

$$I_T = \frac{4 \cdot A_m^2}{\oint \frac{ds}{t}} = \frac{4 \cdot 96,2^2 \cdot 10^6}{237} = 156 \cdot 10^6 \text{ mm}^4$$

Med G = $0.8 \cdot 10^5$ N/mm^2 er torsjonsrotasjon pr lengdeenhet

$$\theta = \frac{M_x}{GI_T} = \frac{120 \cdot 10^6}{0,8 \cdot 10^5 \cdot 156 \cdot 10^6} = 0,96 \cdot 10^{-5} \text{ rad/mm}$$

Lukkede tverrsnitt har vesentlig bedre torsjonsegenskaper enn åpne tverrsnitt. Dette illustreres ved å sammenligne et tynnvegget sirkulært rør med et identisk rør med en langsgående sliss, figur 4.13.



Figur 4.13   Åpent og lukket sirkulært rørtverrsnitt

$$\left(I_T\right)_{åpent} = \frac{1}{3}\left(2\pi R\right)t^3 = \frac{2}{3}\pi R t^3$$

$$\left(I_T\right)_{lukket} = \frac{4 \cdot A_m^2}{\oint \dfrac{ds}{t}} = \frac{4\left(\pi R^2\right)^2}{\dfrac{2\pi R}{t}} = 2\pi R^3 t$$

Eller

$$\frac{\left(I_T\right)_{lukket}}{\left(I_T\right)_{åpent}} = 3\left(\frac{R}{t}\right)^2$$

Med $R/t = 10$ gir dette et forhold på 300 i torsjonsstivhet. Forholdet mellom τ_{maks} for de to tverrsnittene bestemmes fra ligningene (4.12) og (4.16)

$$\frac{\left(\tau_{maks}\right)_{lukket}}{\left(\tau_{maks}\right)_{åpent}} = \frac{\dfrac{M_x}{2\pi R^2 t}}{\dfrac{3M_x}{2\pi R t^2}} = \frac{1}{3}\frac{t}{R} = \frac{1}{30}$$

Ved full plastifisering vil volumet under membranen være lik det volumet vi får når man drysser tørr sand med friksjonsvinkel τ_y på en flate lik stavtverrsnittet. Dette er bakgrunnen for sandhauganalogien for beregning av plastisk torsjonskapasitet /5.5/. Ved denne analogien er fortsatt skjærspenningen rettet tangensielt til nivålinjene, og den plastiske torsjonskapasiteten er lik volumet av sandhaugen multiplisert med 2.0.

Eksempel 5.5

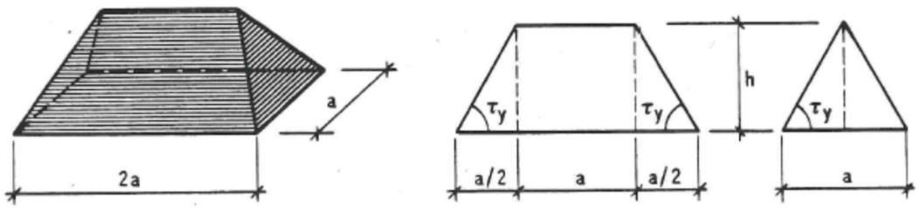


Fig 5.21 - Sandhauganalogien for rekangulært tverrsnitt

Sandhauganalogien skal benyttes for å bestemme den plastiske torsjonskapasiteten av et rektangulært tverrsnitt med sidekanter 2a og a. Fig 5.21 viser formen på sandhaugen, samt snitt gjennom de to symmetriaksene i tverrsnittet. Høyden på haugen er bestemt ved:

$$\tan \tau_y = \frac{h}{a/2}$$

eller

$$h = \frac{1}{2}\,\tau_y \cdot a$$

Momentkapasiteten er lik to ganger volumet av sandhaugen.

$$M_x = 2[a \cdot a \cdot \frac{1}{3} h + \frac{1}{2} a \cdot h \cdot a] = \frac{5}{3} ha^2$$

eller innsatt for h:

$$M_x = \frac{5}{3} \frac{1}{2} \tau_y a \cdot a^2 = \frac{5}{6} \tau_y \cdot a^3 \underline{= 0.83 \tau_y a^3}$$

Den elastiske kapasiteten er gitt ved $\tau_{maks} = \tau_y$ i det kritiske punkt i tverrsnittet. Fra lign (5.4) fås:

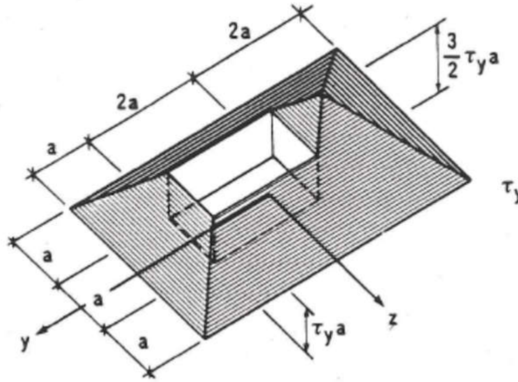
$$M_x = k_2 \cdot (2a) \cdot a^2 \cdot \tau_{maks} \approx 0.25 \cdot 2a^3 \cdot \tau_y \underline{= 0.50 \tau_y a^3}$$

For dette tverrsnittet gir altså en plastisk beregning ca 66% større kapasitet enn en elastisk analyse.

Ved anvendelse på hule tverrsnitt må sandhauganalogien modifiseres, slik at grunnflaten for sandhaugen (dvs stavens tverrsnitt) utstyres med et rør som føres gjennom flaten. Røret gis samme tverrsnittsform som det indre hulrom og en høyde lik $\tau_y \cdot t$ over grunnflaten, hvor t er den minste avstand mellom indre og ytre periferi for tverrsnittet. Den plastiske torsjonskapasiteten er lik to ganger volumet av sandhaugen pluss et volum

$$\Delta V = A_i \cdot (\tau_y \cdot t)$$

hvor A_i er tverrsnittsarealet av røret.



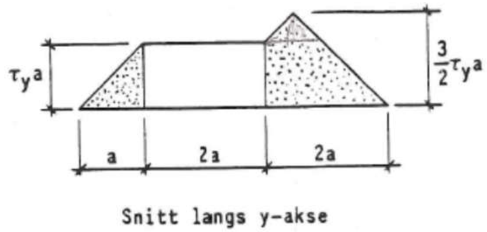


Fig 5.22 - Sandhaug-analogien for rektangeltverrsnitt med hull

Volum av sandhaug:

$$V_s = 2a \cdot 2a \cdot \frac{1}{3} \cdot \tau_y \cdot a + \frac{1}{2} \cdot 2a \cdot \tau_y \cdot a(3a + a) + a \cdot a \cdot \tau_y \cdot a + a \cdot a \cdot \frac{1}{2} \tau_y a$$

$$= \frac{29}{6} \tau_y \cdot a^3$$

Volum av rør:

$$V_r = 2a \cdot a \cdot \tau_y \cdot a = 2\tau_y a^3$$

Eurokode 3: Prosjektering av stålkonstruksjoner
Del 1-8: Knutepunkter og forbindelser

Eurocode 3: Design of steel structures
Part 1-8: Design of joints

1. siffer = $f_{u\ nom}/100$ (N/mm^2)

2. siffer = $(f_{y\ nom}/f_{u\ nom}) \cdot 10$

Produktet av de to sifre gir 1/10 av nominell flytespenning.

Tabell 11.1 - Fasthetsdata for skruemateriale

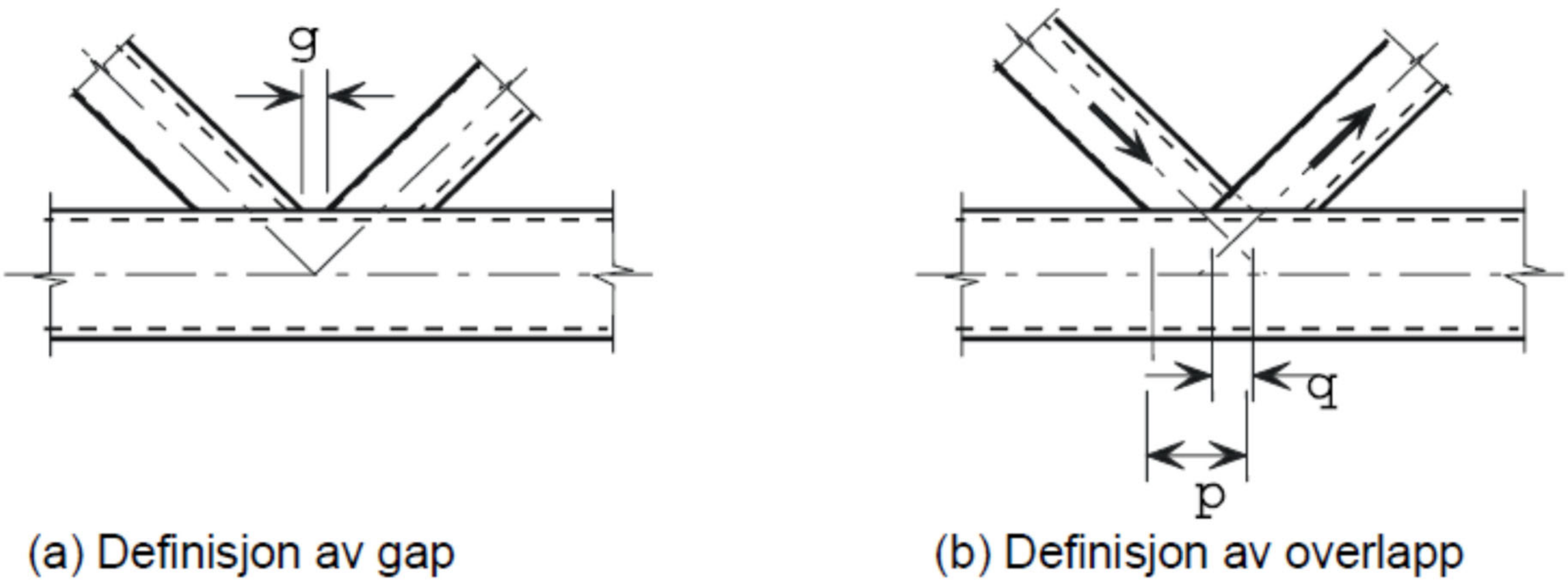
		Fasthetsklasse		
		4.6	8.8	10.9
f_u (N/mm^2)	Nominell	400	800	1000
	Min	400	830	1040
f_y eller $f_{0.2}$ (N/mm^2)	Nominell	240	640	900
	Min	240	660	940
δ_5 (%)		25	12	9

(3) Flytegrensen f_{yb} og strekkfastheten f_{ub} for fasthetsklassene 4.6, 4.8, 5.6, 5.8, 6.8, 8.8 og 10.9 er gitt i tabell 3.1. Disse verdiene bør brukes som karakteristiske verdier ved dimensjoneringen.

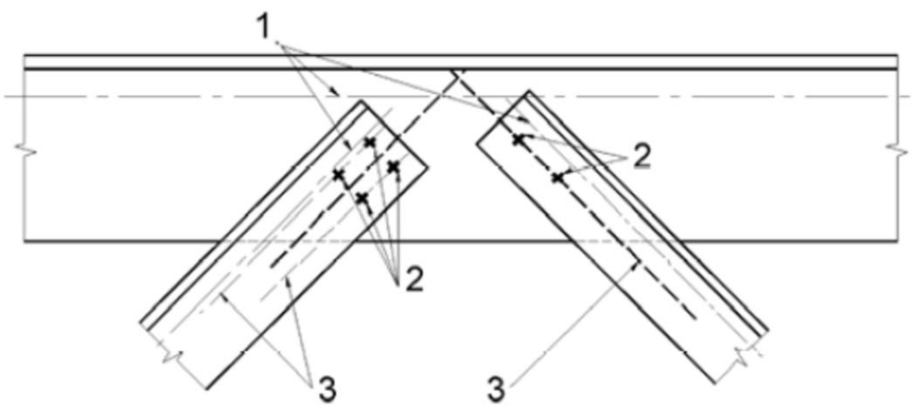
Tabell 3.1 – Nominelle verdier av flytegrensen f_{yb} og strekkfastheten f_{ub} for skruer

Fasthetsklasser for skruer	**4.6**	**4.8**	**5.6**	**5.8**	**6.8**	**8.8**	**10.9**
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1000

MERKNAD Det nasjonale tillegget kan utelukke visse fasthetsklasser for skruer.



Figur 1.3 – Knutepunkter med gap og med overlapp



1 Tyngdepunktakser
2 Festemidler
3 Referanseakser

Figur 2.1 – Definisjon av akser

3.1.2 Forspente skruer

(1) Bare skrueforbindelser av fasthetsklasse 8.8 og 10.9, som er i samsvar med kravene gitt i 1.2.4, Referansestandarder, gruppe 4, kan brukes som forspente skruer til konstruksjonsformål. Det forutsettes kontrollert tiltrekking i samsvar med kravene i 1.2.7, Referansestandarder, gruppe 7.

1.2.7 Referansestandarder, gruppe 7: Utførelse av stålkonstruksjoner

NS-EN 1090-2	Utførelse av stålkonstruksjoner og aluminiumkonstruksjoner Tekniske krav for stålkonstruksjoner

Tabell 2.1 – Partialfaktorer for knutepunkter

Kapasitet for konstruksjonsdeler og tverrsnitt	γ_{M0} , γ_{M1} og γ_{M2}, se NS-EN 1993-1-1
Kapasitet for skruer	γ_{M2}
Kapasitet for nagler	
Kapasitet for bolter i bolteledd	
Kapasitet for sveis	
Kapasitet for hullkanttrykk	
Friksjonskapasitet: - i bruddgrensetilstanden (kategori C) - i bruksgrensetilstanden (kategori B)	 γ_{M3} $\gamma_{M3,ser}$
Kapasitet for hullkanttrykk for en injeksjonsskrue	γ_{M4}
Kapasitet for knutepunkter i en fagverkskonstruksjon av hulprofiler	γ_{M5}
Kapasitet for bolter i bruksgrensetilstanden	$\gamma_{M6,ser}$
Forspenning av høyfaste skruer	γ_{M7}
Kapasitet for betong	γ_c, se NS-EN 1992

MERKNAD Numeriske verdier av γ_M kan gis i det nasjonale tillegget. Følgende verdier anbefales: γ_{M2} = 1,25; γ_{M3} = 1,25 og $\gamma_{M3,ser}$ = 1,1; γ_{M4} = 1,0 ; γ_{M5} = 1,0; $\gamma_{M6,ser}$ = 1,0; γ_{M7} = 1,1 .

(3)P Knutepunkter som utsettes for utmatting, skal også oppfylle kravene gitt i NS-EN 1993-1-9.

1)	Største verdier for hullavstand, kantavstander og endeavstander er ubegrenset, bortsett fra i følgende tilfelle: - for trykkdeler for å unngå lokal knekking og hindre korrosjon i eksponerte konstruksjonsdeler (største verdier er gitt i tabellen) og; - for eksponerte strekkdeler for å unngå korrosjon (største verdier er gitt i tabellen).
2)	Lokal knekkapasitet for en plate under trykk mellom festemidlene bør være beregnet etter NS-EN 1993-1-1, der 0,6 p_1 bør brukes som knekklengde. Det er ikke nødvendig å påvise for lokal knekking mellom festemidlene hvis p_1/t er mindre enn 9 $\boldsymbol{\varepsilon}$. Kantavstanden normalt på kraftretningen for en trykkpåkjent konstruksjonsdel bør ikke overskride kravene mot lokal knekking for en ensidig innfestet flens, se NS-EN 1993-1-1. Endeavstanden berøres ikke av dette kravet.
3)	t er tykkelsen til den tynneste ytre konstruksjonsdelen som festes.
4)	Grenseverdiene for avlange hull er gitt i 1.2.7, Referansestandarder, gruppe 7.
5)	For skruerader som er innbyrdes forskjøvet, kan minste hullavstand reduseres til $p_2 = 1{,}2d_0$, forutsatt at minste avstand L mellom to festemidler er større enn eller lik $2{,}4d_0$, se figur 3.1b).

2.6 Skjærpåkjente knutepunkter utsatt for støt, vibrasjoner og/eller lastreversering

(1) Hvis et skjærpåkjent knutepunkt utsettes for støt eller betydelige vibrasjoner, bør én av følgende festemidler brukes:

– sveiser;
– skruer med låseinnretninger;
– forspente skruer;
– injeksjonsskruer;
– andre typer skruer som effektivt hindrer bevegelse av de delene som forbindes;
– nagler.

(2) Hvis det ikke er akseptabelt med glidning i et knutepunkt (for eksempel fordi det utsettes for laster som reverserer), bør enten forspente skruer av kategori B eller C (se 3.4), passkruer (se 3.6.1), nagler eller sveis brukes.

(3) I vindavstivninger- og/eller andre avstivningskonstruksjoner kan skruer i kategori A brukes, se 3.4.

3.4.1 Skjærforbindelser

(1) Skrueforbindelser påkjent av skjær bør dimensjoneres som én av følgende kategorier:

a) **Kategori A: Forbindelser med avskjæring/hullkanttrykk**
Denne kategorien omfatter skruer fra fasthetsklasse 4.6 til og med fasthetsklasse 10.9. Det kreves ingen forspenning eller spesielle tiltak for kontaktflatene. Dimensjonerende skjærkraft bør ikke overskride dimensjonerende avskjæringskapasitet beregnet etter 3.6, eller dimensjonerende hullkanttrykk beregnet etter 3.6 og 3.7.

b) **Kategori B: Forbindelser med glidningsforhindring i bruksgrensetilstanden**
I denne kategorien bør forspente skruer i samsvar med 3.1.2(1) brukes. Glidning bør ikke forekomme i bruksgrensetilstanden. Dimensjonerende skjærkraft i bruksgrensetilstanden bør ikke overskride dimensjonerende friksjonskapasitet beregnet etter 3.9. Dimensjonerende skjærkraft i bruddgrensetilstanden bør ikke overskride dimensjonerende avskjæringskapasitet beregnet etter 3.6, eller dimensjonerende hullkanttrykk beregnet etter 3.6 og 3.7.

c) **Kategori C: Forbindelser med glidningsforhindring i bruddgrensetilstanden**
I denne kategorien bør forspente skruer i samsvar med 3.1.2(1) brukes. Glidning bør ikke forekomme i bruddgrensetilstanden. Dimensjonerende skjærkraft i bruddgrensetilstanden bør ikke overskride dimensjonerende friksjonskapasitet beregnet etter 3.9, eller dimensjonerende hullkanttrykk beregnet etter 3.6 og 3.7. For en forbindelse med strekk bør i tillegg dimensjonerende plastisk kapasitet for netto tverrsnitt ved skruehullene, $N_{net,Rd}$, (se 6.2 i NS-EN 1993-1-1) påvises i bruddgrensetilstanden.

De nødvendige påvisningene for disse forbindelsene er sammenfattet i tabell 3.2.

3.4.2 Strekkforbindelser

(1) Strekkpåkjente skrueforbindelser bør dimensjoneres som én av følgende kategorier:

a) **Kategori D: ikke forspent**
I denne kategorien benyttes skruer fra fasthetsklasse 4.6 til og med fasthetsklasse 10.9. Det kreves ingen forspenning. Kategorien bør ikke brukes der forbindelsene ofte utstettes for varierende strekkpåkjenning. Kategorien kan imidlertid brukes i forbindelser dimensjonert for normale vindlaster.

b) **Kategori E: forspent**
I denne kategorien benyttes forspente skruer fra 8.8 og 10.9 med kontrollert tiltrekking i samsvar med 1.2.7, Referansestandarder, gruppe 7.

De nødvendige påvisningene for disse forbindelsene er sammenfattet i tabell 3.2.

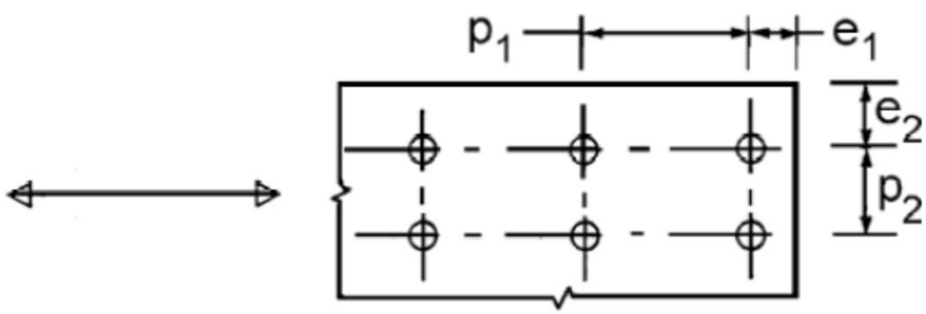
Tabell 3.2 – Kategorier av skrueforbindelser

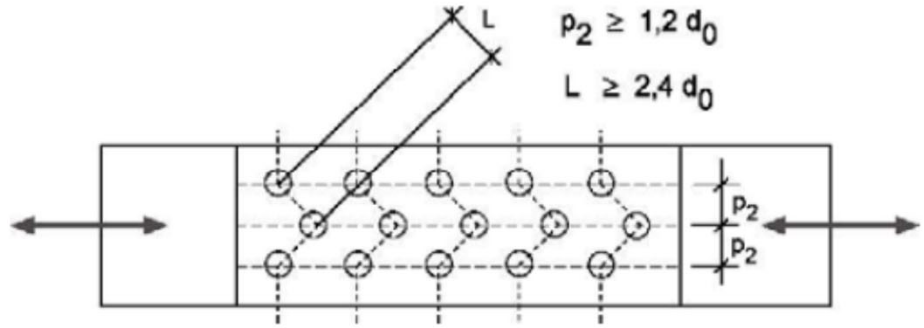
Kategori	Kriterier	Merknader
Skjærforbindelser		
A Avskjæring/hullkanttrykk	$F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Det kreves ingen forspenning. Fasthetsklasser fra 4.6 til 10.9 kan brukes.
B Glidningsforhindret i bruksgrensetilstanden	$F_{v,Ed.ser} \leq F_{s,Rd,ser}$ $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Forspente skruer fra 8.8 eller 10.9 bør brukes. For friksjonskapasitet i bruksgrensetilstanden, se 3.9.
C Glidningsforhindret i bruddgrensetilstanden	$F_{v,Ed} \leq F_{s,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$ $\Sigma F_{v,Ed} \leq N_{net,Rd}$	Forspente skruer fra 8.8 eller 10.9 bør brukes. For friksjonskapasitet i bruddgrensetilstanden, se 3.9. $N_{net,Rd}$, se 3.4.1(1) c).
Strekkforbindelser		
D Ikke forspent	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	Det kreves ingen forspenning. Fasthetsklasser fra 4.6 til 10.9 kan brukes. $B_{p,Rd}$, se tabell 3.4.
E Forspent	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	Forspente skruer i klasse 8.8 eller 10.9 bør brukes. $B_{p,Rd}$, se tabell 3.4.
Dimensjonerende strekkraft $F_{t,Ed}$ bør inkludere krefter fra hevarmvirkning, se 3.11. For skruer som er påkjent av både skjærkraft og strekkraft, gjelder også kriteriene i tabell 3.4.		

MERKNAD Forspenning kan brukes av utførelsesmessige grunner eller som kvalitetstiltak, f.eks. for å oppnå bedre bestandighet. Det nasjonalle tillegget kan gi regler med krav til forspenningsnivået for slike tilfeller.

Failure mode	Bolts	Rivets
Shear resistance per shear plane	$F_{\mathrm{v,Rd}} = \dfrac{\alpha_v \; f_{ub} \; A}{\gamma_{M2}}$ - where the shear plane passes through the threaded portion of the bolt (A is the tensile stress area of the bolt A_s): - for classes 4.6, 5.6 and 8.8: $\alpha_v = 0{,}6$ - for classes 4.8, 5.8, 6.8 and 10.9: $\alpha_v = 0{,}5$ - where the shear plane passes through the unthreaded portion of the bolt (A is the gross cross section of the bolt): $\alpha_v = 0{,}6$	$F_{\mathrm{v,Rd}} = \dfrac{0{,}6 \; f_{ur} \; A_0}{\gamma_{M2}}$

	section of the bolt): $\alpha_v = 0{,}6$
Bearing resistance [1), 2), 3)]	$F_{b,Rd} = \dfrac{k_1\, a_b\, f_u\, d\, t}{\gamma_{M2}}$ where α_b is the smallest of α_d ; $\dfrac{f_{ub}}{f_u}$ or 1,0; in the direction of load transfer: - for end bolts: $\alpha_d = \dfrac{e_1}{3d_0}$; for inner bolts: $\alpha_d = \dfrac{p_1}{3d_0} - \dfrac{1}{4}$ perpendicular to the direction of load transfer: - for edge bolts: k_1 is the smallest of $2{,}8\dfrac{e_2}{d_0} - 1{,}7$ or 2,5 - for inner bolts: k_1 is the smallest of $1{,}4\dfrac{p_2}{d_0} - 1{,}7$ or 2,5

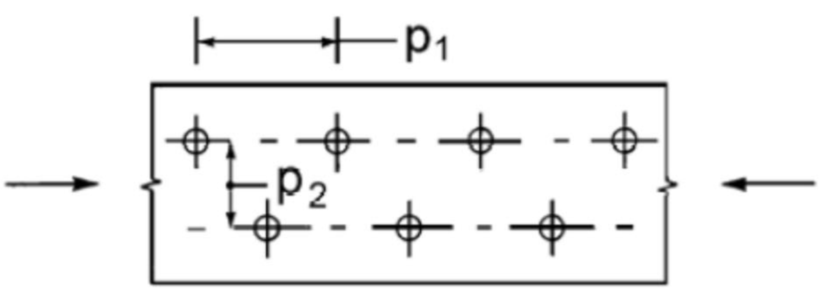


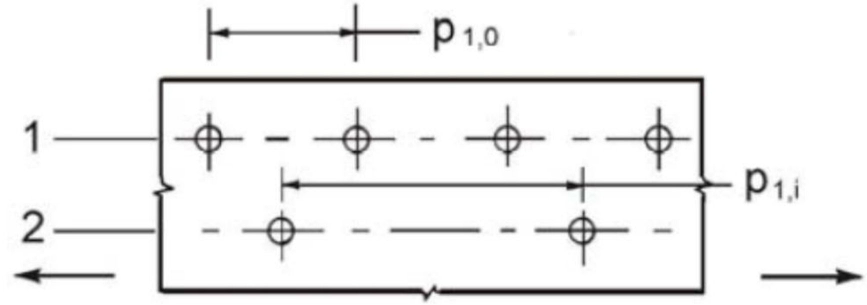


Forskjøvede hullrader

a) Symboler for hullavstander

b) Symboler for hullavstander der hullradene er forskjøvet





1) The bearing resistance $F_{b,Rd}$ for bolts

- in oversized holes is 0,8 times the bearing resistance for bolts in normal holes.
- in slotted holes, where the longitudinal axis of the slotted hole is perpendicular to the direction of the force transfer, is 0,6 times the bearing resistance for bolts in round, normal holes.

2) For countersunk bolt:

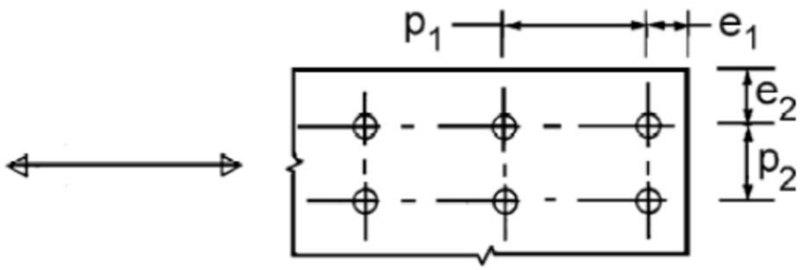
- the bearing resistance $F_{b,Rd}$ should be based on a plate thickness t equal to the thickness of the connected plate minus half the depth of the countersinking.
- for the determination of the tension resistance $F_{t,Rd}$ the angle and depth of countersinking should conform with 1.2.4 Reference Standards: Group 4, otherwise the tension resistance $F_{t,Rd}$ should be adjusted accordingly.

3) When the load on a bolt is not parallel to the edge, the bearing resistance may be verified separately for the bolt load components parallel and normal to the end.

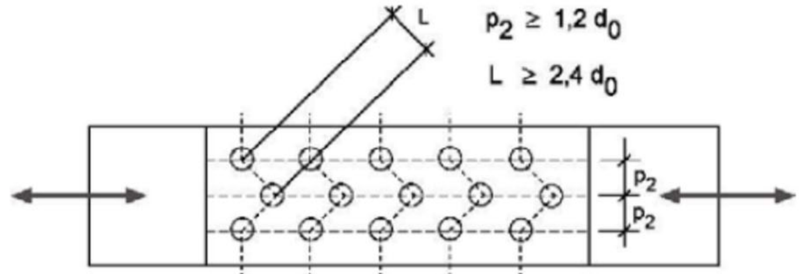
Tension resistance [2)]	$F_{t,Rd} = \dfrac{k_2 \; f_{ub} \; A_s}{\gamma_{M2}}$ where $k_2 = 0{,}63$ for countersunk bolt, otherwise $k_2 = 0{,}9$.	$F_{t,Rd} = \dfrac{0{,}6 \; f_{ur} \; A_0}{\gamma_{M2}}$
Punching shear resistance	$B_{p,Rd} = 0{,}6 \, \pi \, d_m \, t_p f_u / \gamma_{M2}$	No check needed
Combined shear and tension	$\dfrac{F_{v,Ed}}{F_{v,Rd}} + \dfrac{F_{t,Ed}}{1{,}4 \, F_{t,Rd}} \le 1{,}0$	

Tabell 3.3 – Minste og største hull-, ende- og kantavstander

Ende-, kant- og hullavstander, se figur 3.1	Minste	Største[1) 2) 3)]		
		Stålkonstruksjoner av stålsorter i samsvar med NS-EN 10025, unntatt stål etter NS-EN 10025-5		Stålkonstruksjoner av stål i samsvar med NS-EN 10025-5
		Stål som utsettes for klimatiske påvirkninger eller andre korrosive påvirkninger	Stål som ikke utsettes for klimatiske påvirkninger eller andre korrosive påvirkninger	Stål som brukes ubeskyttet
Endeavstand e_1	$1{,}2d_0$	$4t$ + 40 mm		Den største verdien av $8t$ eller 125 mm
Kantavstand e_2	$1{,}2d_0$	$4t$ + 40 mm		Den største verdien av $8t$ eller 125 mm
Avstand e_3 i avlange hull	$1{,}5d_0$ [4)]			
Endeavstand e_4 i avlange hull	$1{,}5d_0$ [4)]			
Hullavstand p_1	$2{,}2d_0$	Den minste verdien av $14t$ eller 200 mm	Den minste verdien av $14t$ eller 200 mm	Den minste verdien av $14t_{min}$ eller 175 mm
Hullavstand $p_{1,0}$		Den minste verdien av $14t$ eller 200 mm		
Hullavstand $p_{1,i}$		Den minste verdien av $28t$ eller 400 mm		
Hullavstand p_2 [5)]	$2{,}4d_0$	Den minste verdien av $14t$ eller 200 mm	Den minste verdien av $14t$ eller 200 mm	Den minste verdien av $14t_{min}$ eller 175 mm

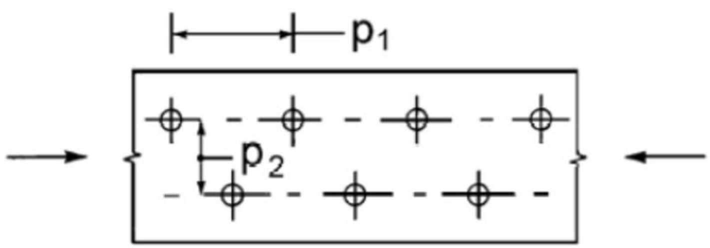


a) Symboler for hullavstander



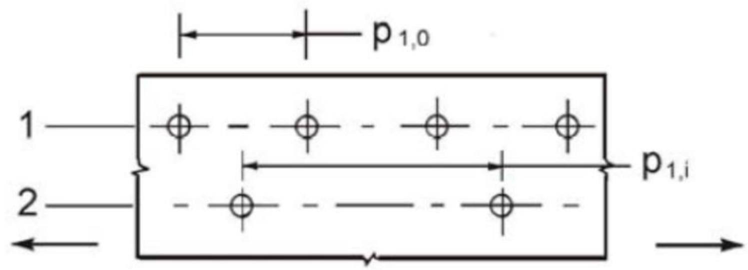
Forskjøvede hullrader

b) Symboler for hullavstander der hullradene er forskjøvet



$p_1 \leq 14\ t$ og ≤ 200 mm $\qquad p_2 \leq 14\ t$ og ≤ 200 mm

c) Hullavstander i trykkpåkjente deler med forskjøvede hullrader



$p_{1,0} \leq 14\ t$ og ≤ 200 mm $\qquad p_{1,i} \leq 28\ t$ og ≤ 400 mm

1 ytre rad $\qquad$ *2 indre rad*

d) Hullavstander i strekkpåkjente deler med forskjøvede hullrader

Eurokode 3: Prosjektering av stålkonstruksjoner
Del 1-8: Knutepunkter og forbindelser

Eurocode 3: Design of steel structures
Part 1-8: Design of joints

(3) Flytegrensen f_{yb} og strekkfastheten f_{ub} for fasthetsklassene 4.6, 4.8, 5.6, 5.8, 6.8, 8.8 og 10.9 er gitt i tabell 3.1. Disse verdiene bør brukes som karakteristiske verdier ved dimensjoneringen.

Tabell 3.1 – Nominelle verdier av flytegrensen f_{yb} og strekkfastheten f_{ub} for skruer

Fasthetsklasser for skruer	**4.6**	**4.8**	**5.6**	**5.8**	**6.8**	**8.8**	**10.9**
f_{yb} (N/mm^2)	240	320	300	400	480	640	900
f_{ub} (N/mm^2)	400	400	500	500	600	800	1000

MERKNAD Det nasjonale tillegget kan utelukke visse fasthetsklasser for skruer.

1.2.7 Referansestandarder, gruppe 7: Utførelse av stålkonstruksjoner

NS-EN 1090-2	Utførelse av stålkonstruksjoner og aluminiumkonstruksjoner Tekniske krav for stålkonstruksjoner

Tabell 2.1 – Partialfaktorer for knutepunkter

Kapasitet for konstruksjonsdeler og tverrsnitt	γ_{M0} , γ_{M1} og γ_{M2}, se NS-EN 1993-1-1
Kapasitet for skruer	γ_{M2}
Kapasitet for nagler	
Kapasitet for bolter i bolteledd	
Kapasitet for sveis	
Kapasitet for hullkanttrykk	
Friksjonskapasitet: - i bruddgrensetilstanden (kategori C) - i bruksgrensetilstanden (kategori B)	 γ_{M3} $\gamma_{M3,ser}$
Kapasitet for hullkanttrykk for en injeksjonsskrue	γ_{M4}
Kapasitet for knutepunkter i en fagverkskonstruksjon av hulprofiler	γ_{M5}
Kapasitet for bolter i bruksgrensetilstanden	$\gamma_{M6,ser}$
Forspenning av høyfaste skruer	γ_{M7}
Kapasitet for betong	γ_c, se NS-EN 1992

MERKNAD Numeriske verdier av γ_M kan gis i det nasjonale tillegget. Følgende verdier anbefales: γ_{M2} = 1,25; γ_{M3} = 1,25 og $\gamma_{M3,ser}$ = 1,1; γ_{M4} = 1,0 ; γ_{M5} = 1,0; $\gamma_{M6,ser}$ = 1,0; γ_{M7} = 1,1 .

(3)P Knutepunkter som utsettes for utmatting, skal også oppfylle kravene gitt i NS-EN 1993-1-9.

3.4.1 Skjærforbindelser

(1) Skrueforbindelser påkjent av skjær bør dimensjoneres som én av følgende kategorier:

a) **Kategori A: Forbindelser med avskjæring/hullkanttrykk**
Denne kategorien omfatter skruer fra fasthetsklasse 4.6 til og med fasthetsklasse 10.9. Det kreves ingen forspenning eller spesielle tiltak for kontaktflatene. Dimensjonerende skjærkraft bør ikke overskride dimensjonerende avskjæringskapasitet beregnet etter 3.6, eller dimensjonerende hullkanttrykk beregnet etter 3.6 og 3.7.

b) **Kategori B: Forbindelser med glidningsforhindring i bruksgrensetilstanden**
I denne kategorien bør forspente skruer i samsvar med 3.1.2(1) brukes. Glidning bør ikke forekomme i bruksgrensetilstanden. Dimensjonerende skjærkraft i bruksgrensetilstanden bør ikke overskride dimensjonerende friksjonskapasitet beregnet etter 3.9. Dimensjonerende skjærkraft i bruddgrensetilstanden bør ikke overskride dimensjonerende avskjæringskapasitet beregnet etter 3.6, eller dimensjonerende hullkanttrykk beregnet etter 3.6 og 3.7.

c) **Kategori C: Forbindelser med glidningsforhindring i bruddgrensetilstanden**
I denne kategorien bør forspente skruer i samsvar med 3.1.2(1) brukes. Glidning bør ikke forekomme i bruddgrensetilstanden. Dimensjonerende skjærkraft i bruddgrensetilstanden bør ikke overskride dimensjonerende friksjonskapasitet beregnet etter 3.9, eller dimensjonerende hullkanttrykk beregnet etter 3.6 og 3.7. For en forbindelse med strekk bør i tillegg dimensjonerende plastisk kapasitet for netto tverrsnitt ved skruehullene, $N_{net,Rd}$, (se 6.2 i NS-EN 1993-1-1) påvises i bruddgrensetilstanden.

De nødvendige påvisningene for disse forbindelsene er sammenfattet i tabell 3.2.

3.4.2 Strekkforbindelser

(1) Strekkpåkjente skrueforbindelser bør dimensjoneres som én av følgende kategorier:

a) **Kategori D: ikke forspent**
I denne kategorien benyttes skruer fra fasthetsklasse 4.6 til og med fasthetsklasse 10.9. Det kreves ingen forspenning. Kategorien bør ikke brukes der forbindelsene ofte utstettes for varierende strekkpåkjenning. Kategorien kan imidlertid brukes i forbindelser dimensjonert for normale vindlaster.

b) **Kategori E: forspent**
I denne kategorien benyttes forspente skruer fra 8.8 og 10.9 med kontrollert tiltrekking i samsvar med 1.2.7, Referansestandarder, gruppe 7.

De nødvendige påvisningene for disse forbindelsene er sammenfattet i tabell 3.2.

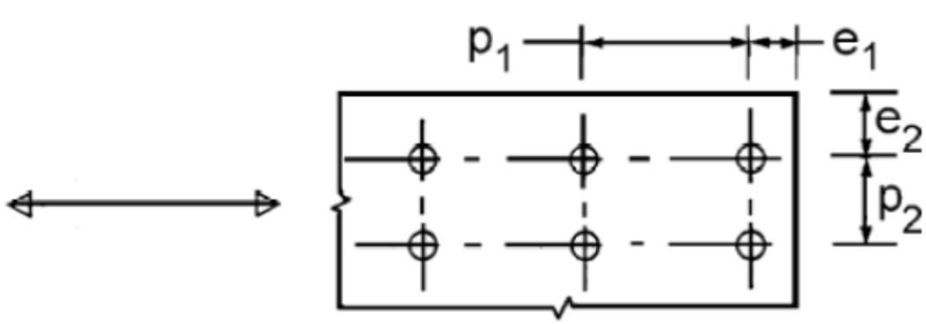
Tabell 3.2 – Kategorier av skrueforbindelser

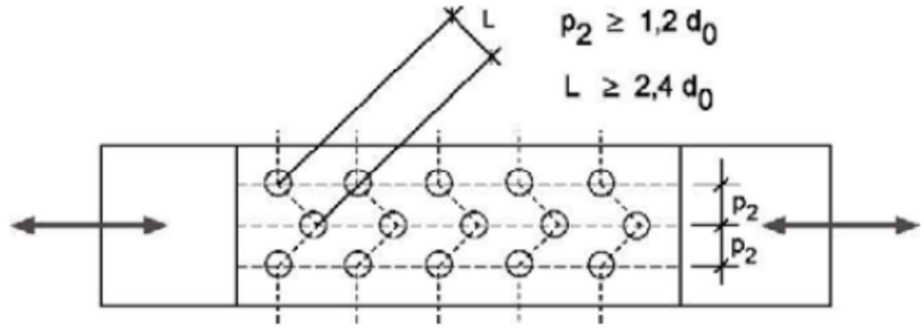
Kategori	Kriterier	Merknader
Skjærforbindelser		
A Avskjæring/hullkanttrykk	$F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Det kreves ingen forspenning. Fasthetsklasser fra 4.6 til 10.9 kan brukes.
B Glidningsforhindret i bruksgrensetilstanden	$F_{v,Ed.ser} \leq F_{s,Rd,ser}$ $F_{v,Ed} \leq F_{v,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$	Forspente skruer fra 8.8 eller 10.9 bør brukes. For friksjonskapasitet i bruksgrensetilstanden, se 3.9.
C Glidningsforhindret i bruddgrensetilstanden	$F_{v,Ed} \leq F_{s,Rd}$ $F_{v,Ed} \leq F_{b,Rd}$ $\Sigma F_{v,Ed} \leq N_{net,Rd}$	Forspente skruer fra 8.8 eller 10.9 bør brukes. For friksjonskapasitet i bruddgrensetilstanden, se 3.9. $N_{net,Rd}$, se 3.4.1(1) c).
Strekkforbindelser		
D Ikke forspent	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	Det kreves ingen forspenning. Fasthetsklasser fra 4.6 til 10.9 kan brukes. $B_{p,Rd}$, se tabell 3.4.
E Forspent	$F_{t,Ed} \leq F_{t,Rd}$ $F_{t,Ed} \leq B_{p,Rd}$	Forspente skruer i klasse 8.8 eller 10.9 bør brukes. $B_{p,Rd}$, se tabell 3.4.
Dimensjonerende strekkraft $F_{t,Ed}$ bør inkludere krefter fra hevarmvirkning, se 3.11. For skruer som er påkjent av både skjærkraft og strekkraft, gjelder også kriteriene i tabell 3.4.		

MERKNAD Forspenning kan brukes av utførelsesmessige grunner eller som kvalitetstiltak, f.eks. for å oppnå bedre bestandighet. Det nasjonalle tillegget kan gi regler med krav til forspenningsnivået for slike tilfeller.

Failure mode	Bolts	Rivets
Shear resistance per shear plane	$F_{v,Rd} = \dfrac{\alpha_v \; f_{ub} \; A}{\gamma_{M2}}$ - where the shear plane passes through the threaded portion of the bolt (A is the tensile stress area of the bolt A_s): - for classes 4.6, 5.6 and 8.8: $\alpha_v = 0,6$ - for classes 4.8, 5.8, 6.8 and 10.9: $\alpha_v = 0,5$ - where the shear plane passes through the unthreaded portion of the bolt (A is the gross cross section of the bolt): $\alpha_v = 0,6$	$F_{v,Rd} = \dfrac{0,6 \; f_{ur} \; A_0}{\gamma_{M2}}$

	section of the bolt): $\alpha_v = 0{,}6$
Bearing resistance [1), 2), 3)]	$F_{b,Rd} = \dfrac{k_1\, a_b\, f_u\, d\, t}{\gamma_{M2}}$ where α_b is the smallest of α_d ; $\dfrac{f_{ub}}{f_u}$ or 1,0; in the direction of load transfer: - for end bolts: $\alpha_d = \dfrac{e_1}{3d_0}$; for inner bolts: $\alpha_d = \dfrac{p_1}{3d_0} - \dfrac{1}{4}$ perpendicular to the direction of load transfer: - for edge bolts: k_1 is the smallest of $2{,}8\dfrac{e_2}{d_0} - 1{,}7$ or 2,5 - for inner bolts: k_1 is the smallest of $1{,}4\dfrac{p_2}{d_0} - 1{,}7$ or 2,5

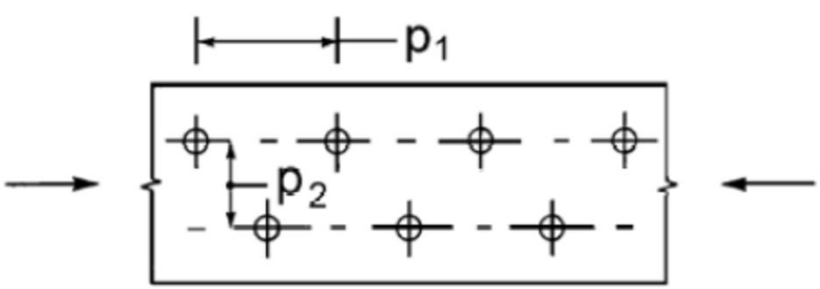


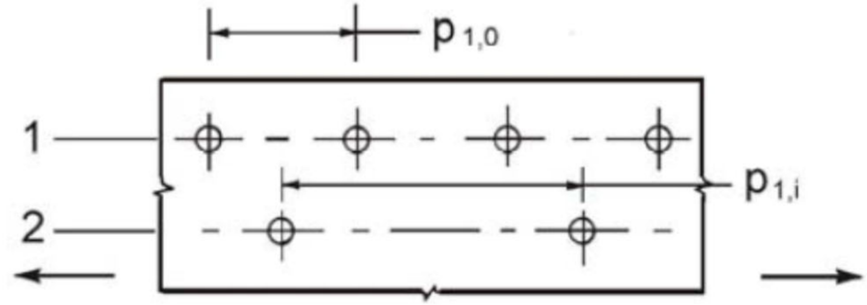


Forskjøvede hullrader

a) Symboler for hullavstander

b) Symboler for hullavstander der hullradene er forskjøvet

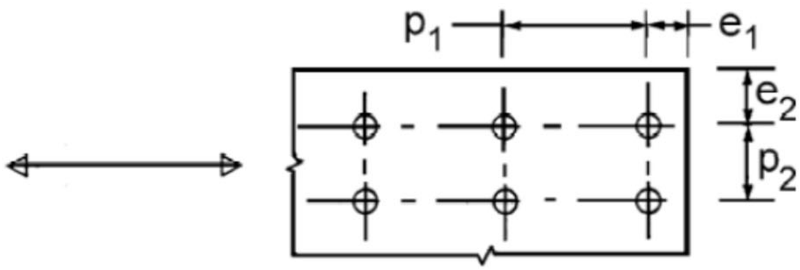




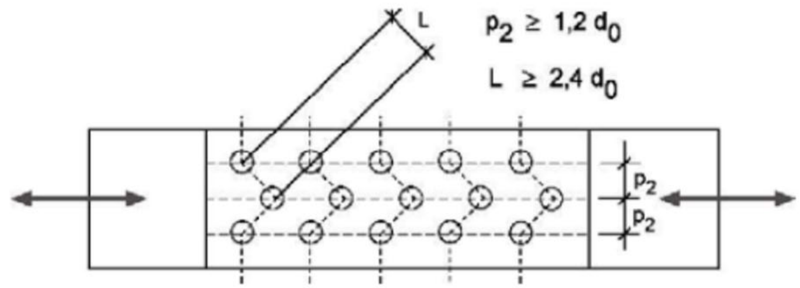
Tension resistance [2)]	$F_{\mathrm{t,Rd}} = \dfrac{k_2 \; f_{ub} \; A_s}{\gamma_{M2}}$ where $k_2 = 0{,}63$ for countersunk bolt, otherwise $k_2 = 0{,}9$.	$F_{\mathrm{t,Rd}} = \dfrac{0{,}6 \; f_{ur} \; A_0}{\gamma_{M2}}$
Punching shear resistance	$B_{\mathrm{p,Rd}} = 0{,}6 \, \pi \, d_{\mathrm{m}} \, t_{\mathrm{p}} f_{\mathrm{u}} / \gamma_{\mathrm{M2}}$	No check needed
Combined shear and tension	$\dfrac{F_{v,Ed}}{F_{v,Rd}} + \dfrac{F_{t,Ed}}{1{,}4 \, F_{t,Rd}} \le 1{,}0$	

Tabell 3.3 – Minste og største hull-, ende- og kantavstander

Ende-, kant- og hullavstander, se figur 3.1	Minste	Største[1) 2) 3)]		
		Stålkonstruksjoner av stålsorter i samsvar med NS-EN 10025, unntatt stål etter NS-EN 10025-5		Stålkonstruksjoner av stål i samsvar med NS-EN 10025-5
		Stål som utsettes for klimatiske påvirkninger eller andre korrosive påvirkninger	Stål som ikke utsettes for klimatiske påvirkninger eller andre korrosive påvirkninger	Stål som brukes ubeskyttet
Endeavstand e_1	$1{,}2d_0$	$4t$ + 40 mm		Den største verdien av $8t$ eller 125 mm
Kantavstand e_2	$1{,}2d_0$	$4t$ + 40 mm		Den største verdien av $8t$ eller 125 mm
Avstand e_3 i avlange hull	$1{,}5d_0$ [4)]			
Endeavstand e_4 i avlange hull	$1{,}5d_0$ [4)]			
Hullavstand p_1	$2{,}2d_0$	Den minste verdien av $14t$ eller 200 mm	Den minste verdien av $14t$ eller 200 mm	Den minste verdien av $14t_{min}$ eller 175 mm
Hullavstand $p_{1,0}$		Den minste verdien av $14t$ eller 200 mm		
Hullavstand $p_{1,i}$		Den minste verdien av $28t$ eller 400 mm		
Hullavstand p_2 [5)]	$2{,}4d_0$	Den minste verdien av $14t$ eller 200 mm	Den minste verdien av $14t$ eller 200 mm	Den minste verdien av $14t_{min}$ eller 175 mm

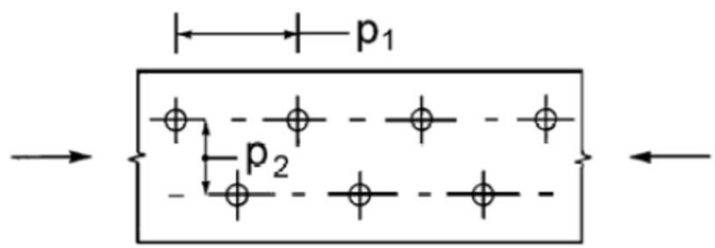


a) Symboler for hullavstander



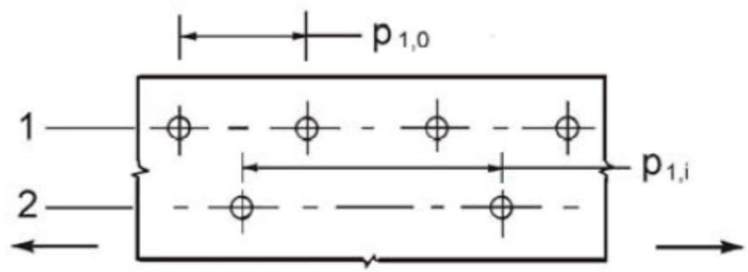
Forskjøvede hullrader

b) Symboler for hullavstander der hullradene er forskjøvet



$p_1 \leq 14\ t$ og ≤ 200 mm $\qquad p_2 \leq 14\ t$ og ≤ 200 mm

c) Hullavstander i trykkpåkjente deler med forskjøvede hullrader



$p_{1,0} \leq 14\ t$ og ≤ 200 mm $\qquad p_{1,i} \leq 28\ t$ og ≤ 400 mm

1 ytre rad $\qquad$ *2 indre rad*

d) Hullavstander i strekkpåkjente deler med forskjøvede hullrader

Tabell 3.4 – Dimensjonerende kapasitet for individuelle festemidler påkjent av avskjæring og/eller strekk

<table>
<tr><th>Bruddform</th><th>Skruer</th><th>Nagler</th></tr>
<tr><td>Avskjæringskapasitet per snitt</td><td>$F_{v,Rd} = \dfrac{\alpha_v \; f_{ub} \; A}{\gamma_{M2}}$
- dersom avskjæringssnittet går gjennom den gjengede delen av skruen (A settes lik spenningsarealet for skruen, A_s):
- for fasthetsklasse 4.6, 5.6 og 8.8:
$\alpha_v = 0{,}6$
- for fasthetsklasse 4.8, 5.8, 6.8 og 10.9:
$\alpha_v = 0{,}5$
- dersom avskjæringssnittet går gjennom den ugjengede delen av skruen (A er skruens brutto tverrsnitt): $\alpha_v = 0{,}6$</td><td>$F_{v,Rd} = \dfrac{0{,}6 \; f_{ur} \; A_0}{\gamma_{M2}}$</td></tr>
<tr><td>Kapasitet for hullkanttrykk [1), 2),3)]</td><td colspan="2">$F_{b,Rd} = \dfrac{k_1 \alpha_b f_u \, d \, t}{\gamma_{M2}}$
der α_b er den minste av α_d, $\dfrac{f_{ub}}{f_u}$ eller 1,0
i kraftretningen:
- for endeskruer: $\alpha_d = \dfrac{e_1}{3d_0}$;
- for innvendige skruer: $\alpha_d = \dfrac{p_1}{3d_0} - \dfrac{1}{4}$
normalt på kraftretningen:
- for skruer langs randen: k_1 er den minste av $2{,}8\dfrac{e_2}{d_0} - 1{,}7$, $1{,}4\dfrac{p_2}{d_0} - 1{,}7$, eller 2,5
- for innvendige skruer: k_1 er den minste av $1{,}4\dfrac{p_2}{d_0} - 1{,}7$ eller 2,5</td></tr>
</table>

Strekkapasitet [2)]	$F_{t,Rd} = \dfrac{k_2 \ f_{ub} \ \ A_s}{\gamma_{M2}}$ der k_2 = 0,63 for senkskrue, ellers k_2 = 0,9.	$F_{t,Rd} = \dfrac{0{,}6 \ f_{ur} \ \ A_0}{\gamma_{M2}}$
Gjennomlokking	$B_{p,Rd} \quad = \quad 0{,}6 \ \pi \ d_m \ t_p \ f_u \ / \ \gamma_{M2}$	Påvisning ikke nødvendig
Kombinert avskjæring og strekk	$\dfrac{F_{v,Ed}}{F_{v,Rd}} + \dfrac{F_{t,Ed}}{1{,}4 \, F_{t,Rd}} \leq 1{,}0$	

[1)] Kapasitet for hullkanttrykk $F_{b,Rd}$ for skruer

- i overstore hull er 0,8 ganger kapasiteten for skruer i normale hull.

- i avlange hull, der hullets lengdeakse er normalt på kraftretningen, er 0,6 ganger kapasiteten for skruer i runde, normale hull;

[2)] For senkskruer:

- beregnes kapasiteten for hullkanttrykk $F_{b,Rd}$ på grunnlag av tykkelsen av den innfestede platen t minus halve dybden av forsenkningen;

- for bestemmelse av strekkapasiteten $F_{t,Rd}$ bør vinkelen og dybden av forsenkningen være i samsvar med 1.2.4 Referansestandarder, gruppe 4 I motsatt fall bør strekkapasiteten $F_{t,Rd}$ justeres tilsvarende.

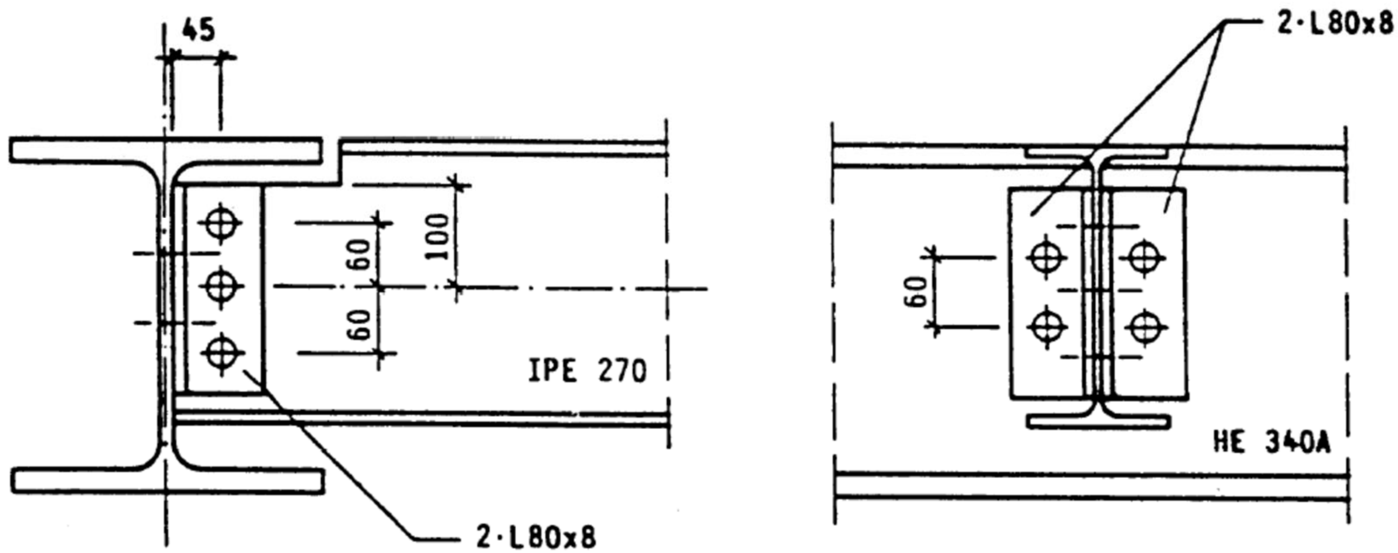
[3)] Hvis lasten på en skrue ikke er parallell med randen, kan kapasiteten for hullkanttrykk påvises separat for skruens lastkomponenter parallelt med og vinkelrett på randen.

Secondary IPE270 beam connected to main HE340A beam

The secondary beam is assumed to be simply supported to the main beam, i.e. no moment is to be transmitted. According to the action effect analysis a shear force $V_{Ed} = 200\,kN$ is to be transmitted by the connection.

M20-8.8 bolts

S355 steel $\qquad d_0 = 20 + 2 = 22\,mm$

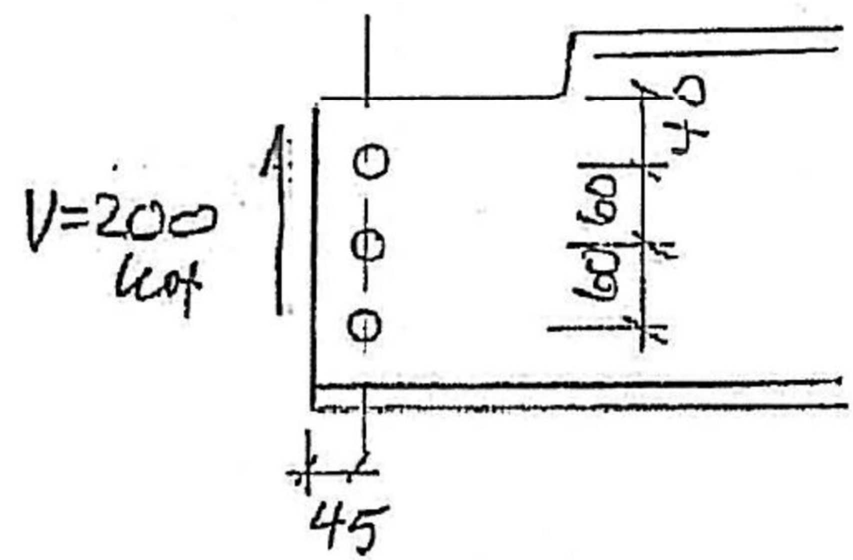


Necessary checks

1. Resistance of bolts IPE270
   - Shear resistance of bolts (two shear planes)
   - Bearing resistance of bolts
2. Resistance of bolts HE340A
   - Shear resistance of bolts (one shear plane)
   - Bearing resistance of bolts
3. Resistance of reduced section IPE270
   - General yield
4. Block tearing of bolt group IPE270
5. Resistance of weld (not considered)

Resistance of bolts in IPE270

Distributes the shear force equally on all three bolts:

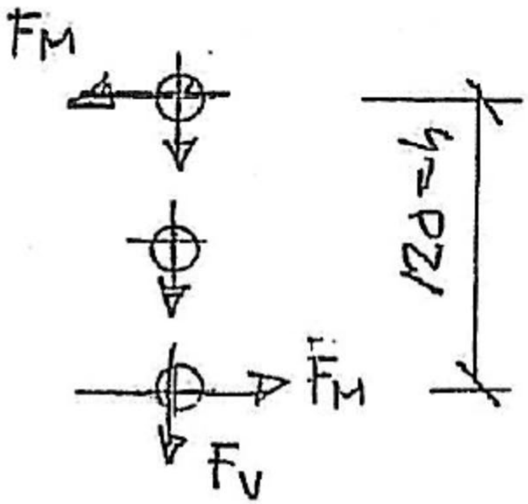


$$F_V = \frac{200}{3} = \underline{66.7\,kN}$$

The shear force is eccentric with respect to the system line of the bolt group. This results in an eccentricity moment:

$$M = V \cdot e = 200 \cdot 0.045 = \underline{9.0\,kNm}$$

The eccentricity moment is balanced by a horizontal force couple in the top and bottom bolt:



$$F_M = \frac{9.0}{0.120} = \underline{75\,kN}$$

The resulting force in the top and bottom bolt is:

$$F_{v,Ed} = \sqrt{F_V^2 + F_M^2} = \sqrt{66.7^2 + 75^2} = \boxed{100\,kN}$$

Shear resistance of M20-8.8 bolt with two shear planes:

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} \cdot 2 = \frac{0.6 \cdot 800 \cdot 245}{1.3} \cdot 2 = \boxed{181\,kN}$$

The edge and end distances are different for F_V and F_M. We calculate k_1 and α_b for both and use the smallest value:

F_V

$$\alpha_b = \min\left(\frac{40}{3\cdot 22}, \frac{60}{3\cdot 22} - \frac{1}{4}, \frac{800}{510}, 1.0\right) = \underline{0.61}$$

$$k_1 = \min\left(2.8\cdot\frac{45}{22} - 1.7, 2.5\right) = \underline{2.5}$$

F_M

$$\alpha_b = \min\left(\frac{45}{3\cdot 22}, \frac{800}{510}, 1.0\right) = \underline{0.68}$$

$$k_1 = \min\left(2.8\cdot\frac{40}{22} - 1.7, 2.5\right) = \underline{2.5}$$

Bearing resistance of bolt ($t_{web} = 6.6\,mm < \sum t_L = 2\times 8 = 16\,mm$):

$$F_{b,Rd} = \frac{2.5\cdot 0.61\cdot 510\cdot 20\cdot 6.6}{1.3} = \boxed{79.0\,kN}$$

We have a bearing type shear connection (i.e. Category A):

$$F_{v,Ed} = 100\,kN < F_{v,Rd} = 181\,kN$$

$$F_{v,Ed} = 100\,kN > F_{b,Rd} = 79.0\,kN \;\rightarrow \boxed{\textbf{not OK!}}$$

Resistance of bolts in HE340A

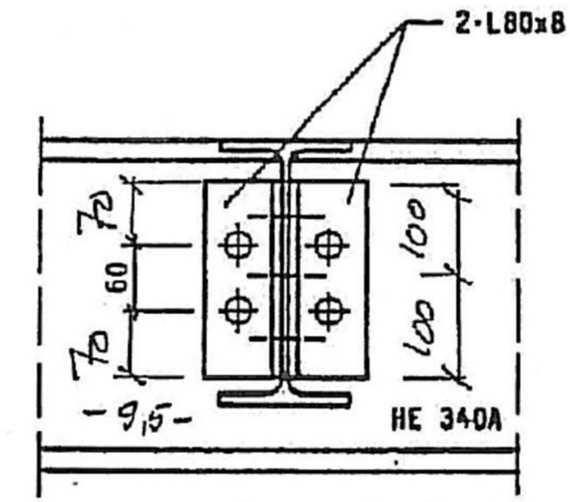
Distributes the shear force equally on all four bolts:

$$F_{v,Ed} = \frac{200}{4} = \boxed{50.0\,kN}$$

Shear resistance of M20-8.8 bolt with one shear plane:

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} = \frac{0.6 \cdot 800 \cdot 245}{1.3} = \boxed{90.5\,kN}$$

We observe that all distances except p_1 are above the 'optimal' distances



$$e_1 = 70\,mm > 3\,d_0$$
$$p_1 = 60\,mm < 3.75\,d_0$$
$$e_2 = 36\,mm > 1.5\,d_0$$
$$p_2 = 95\,mm > 3\,d_0$$

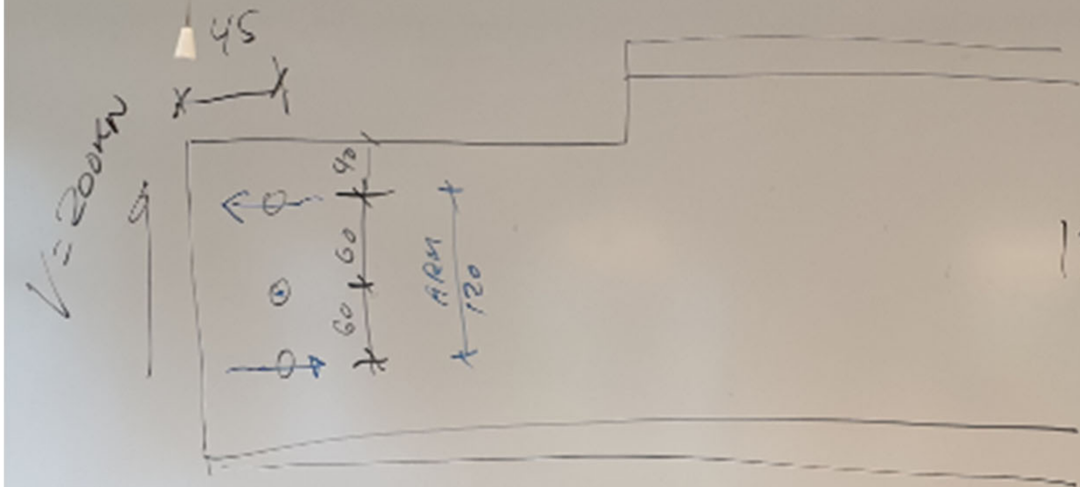
$$\alpha_b = \min\left(\frac{60}{3\cdot 22} - \frac{1}{4}, \frac{800}{510}, 1.0\right) = \underline{0.61}$$
$$k_1 = 2.5$$

Bearing resistance of bolt ($t_L = 8.0\,mm < t_{web} = 9.5\,mm >$):

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}} = \frac{2.5\cdot 0.66\cdot 510\cdot 20\cdot 8.0}{1.3} = \boxed{103\,kN}$$

NS-EN 1993-1-5

NS-EN 1993-1-8)



IPE 270

BOLTER:

$$F_V = \frac{200}{3} = 66.7\,kN$$

$$F_M = \frac{M}{ARM} = \frac{200\ 0{,}45}{0{,}120}$$

$$= \frac{9{,}0\ kNm}{0{,}120m} = \underline{75\ kN}$$

$$F_{V,Ed} = \sqrt{F_V^2 + F_M^2}$$

$$= \sqrt{66{,}7^2 + 75^2} = \underline{100 kN}$$

HULLKANTTRYKK:

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}}$$

NS-EN 1993-1-1

NS-EN 1993-1-5

NS-EN 1993-1-8

$$\alpha_b = \min\left(1{,}0,\ \frac{f_{ub}}{f_u},\ \frac{e_1}{3 \cdot d_0},\ \frac{p_1}{3 \cdot d_0} - \frac{1}{4}\right) = \underline{0{,}6}$$

ϕ

HULLKANTTRYKK:

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}}$$

NS-EN 1993-1-

NS-EN 1993-1-

NS-EN 1993-1-

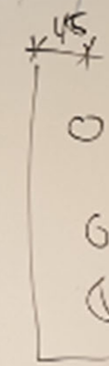
$$\alpha_b = \min\left(1{,}0,\ \frac{f_{ub}}{f_u},\ \frac{e_1}{3 \cdot d_0},\ \frac{p_1}{3 \cdot d_0} - \frac{1}{4}\right) = 0{,}6$$

ϕ

ENDE $$k_1 = \min\left(2{,}8\frac{e_2}{d_0} - 1{,}7,\ 2{,}5\right)$$

$$\left(2{,}8 \cdot \frac{45}{22} - 1{,}5,\ 2{,}5\right)$$

$$\left(4{,}0,\ 2{,}5\right) \quad k_1 = 2{,}5$$



HULLKANTTRYKK:

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \, f_u \cdot d \cdot t}{\gamma_{M2}}$$

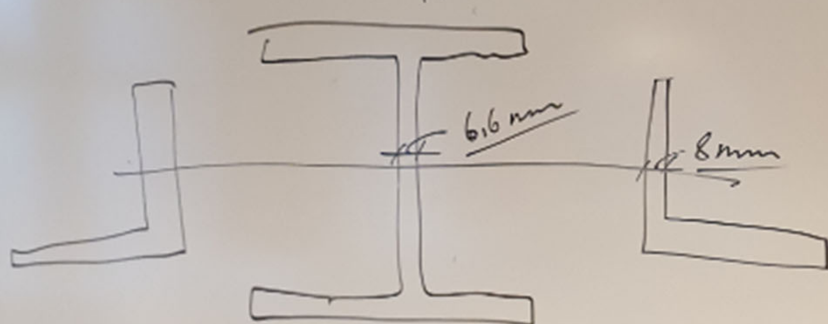
$k_1 = 2{,}5$

$\alpha_1 = 0{,}6$

NS-EN 1993-1-1

NS-EN 1993-1-5

NS-EN 1993-1-8



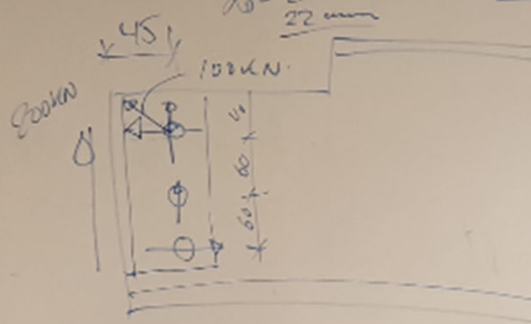
$t_{web} = 6{,}6 < \sum t_L = 2 \times 8mm = \underline{16mm}$

$$F_{b,Rd} = \frac{2{,}5 \cdot 0{,}6 \cdot 510 \cdot 20 \cdot 6{,}6}{1{,}25}$$

$$= \underline{79kN}$$

$d = 20$ mm bolt

$d_0 = 20 + 2$ mm $=$ Hull.
22 mm



$$F_V = \frac{200}{3} = \underline{66{,}7\text{kN}}.$$

$$H = 0{,}045 \cdot 200 = \underline{9\text{kN}}$$

BOLTER: KATEGORI A

ANSKJÆRING HULLKANTTRYKK.

IPE270:

ANSKJÆRING:

$$F_{v,Rd} = 181\text{kN}$$

HULLKANTRYKK:

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M1}}$$

$$k_1 = \min\left(2{,}8 \cdot \frac{45}{22} - 1{,}7,\ 2{,}5\right) \qquad \alpha_b = \min\left(1{,}0,\ \frac{800}{510},\ \frac{45}{3 \cdot 22}\right) = 0{,}68.$$

$\underline{k_1 = 2{,}5}$

$$F_{b,Rd} = \frac{2{,}5 \cdot 0{,}61 \cdot 800 \cdot 6{,}6 \cdot}{13} = \underline{79\text{kN}}$$

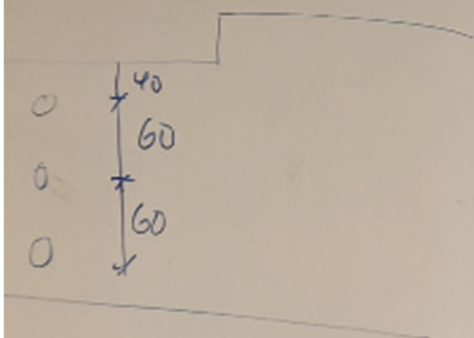
1. SKJÆRSPENNINGER
2. TORSJONSPENNINGER
3. NEDBØYNING / BRUKSKRAV.

$$\frac{f_u}{f_y}\ \frac{800}{510} \approx \underline{0{,}64}$$

8.8 bolt

$f_u = 800$ MPa

$f_y = 80\%\ 800 = 640$ MPa



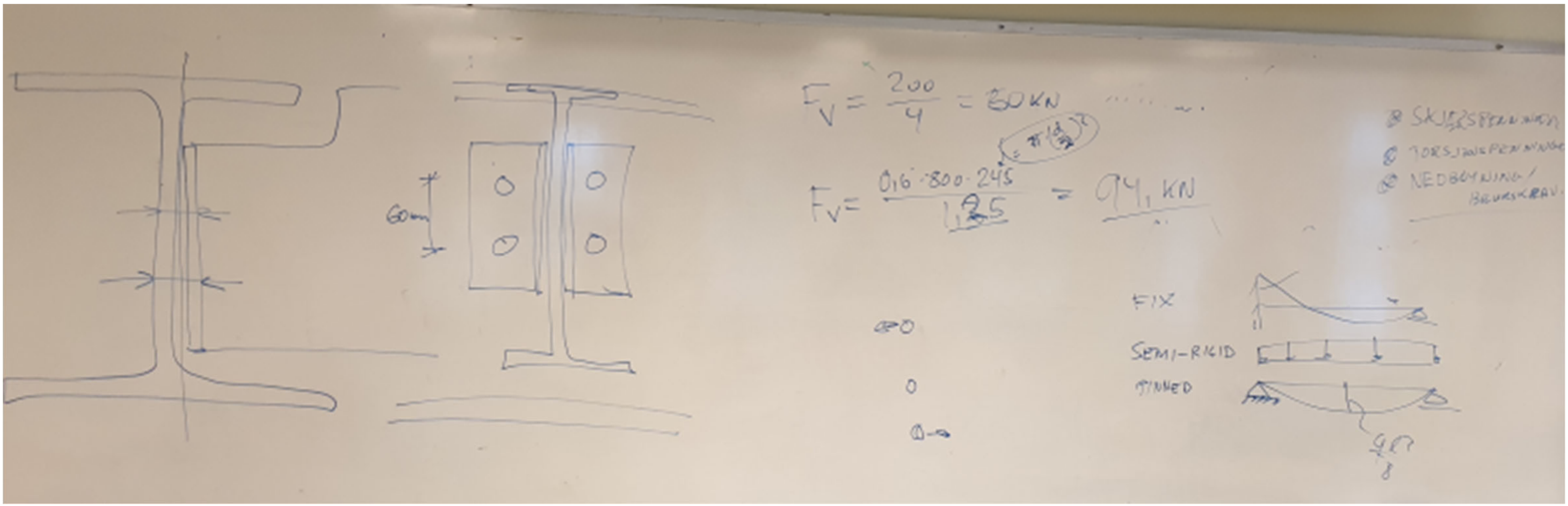
$d_0 = 22$ mm

Sjekk endeavstand: $e_1 \geq 1{,}2 \cdot d_0 = 1{,}2 \cdot 22 = 26{,}4$ mm.

Kantavstand: $e_2 \geq 1{,}2 \cdot d_0 = \underline{26{,}4 \text{ mm}}$

Avstand mellom hull i lastretning: $2{,}2 \cdot d_0 = 48{,}4$ mm.

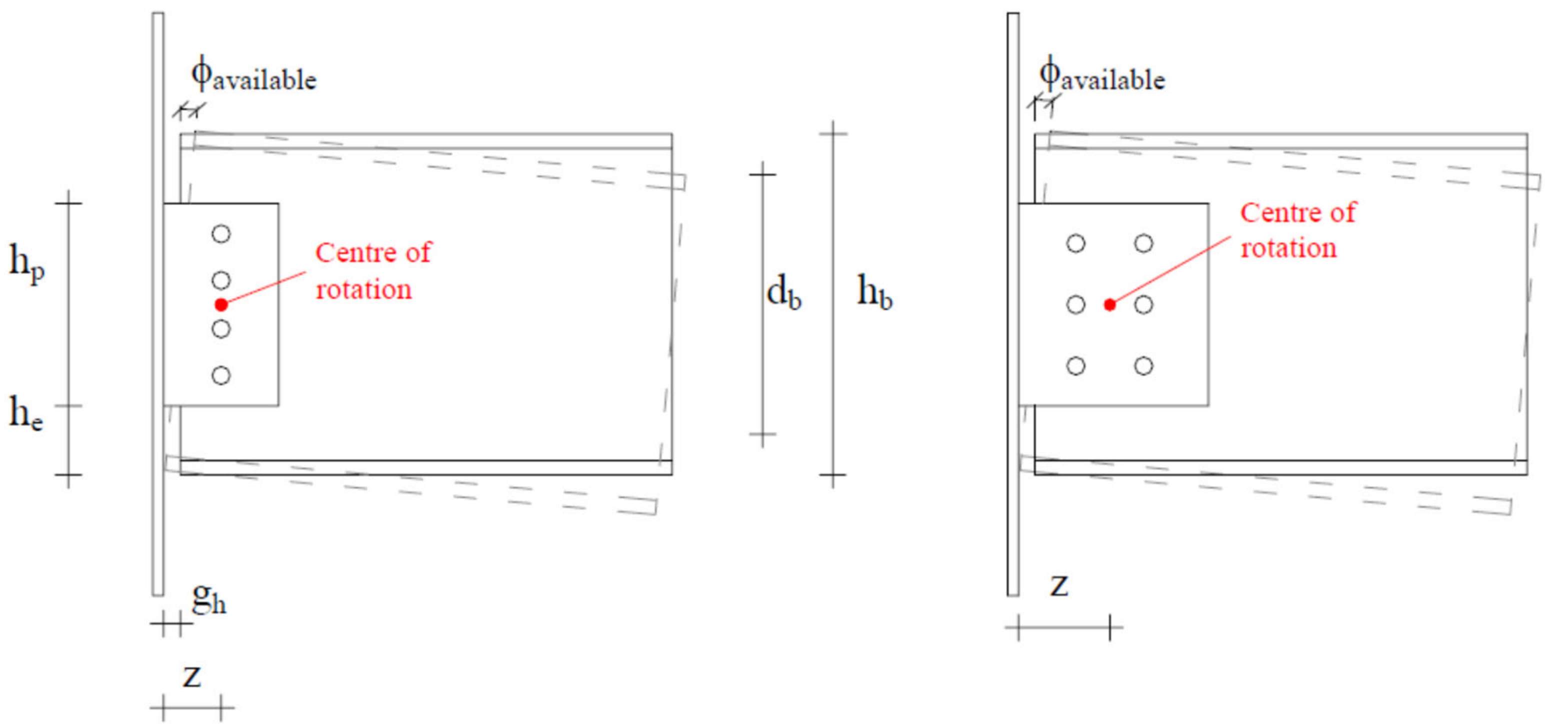
på tvers av lastretning: $2{,}4 \cdot d_0 = 52$ mm.



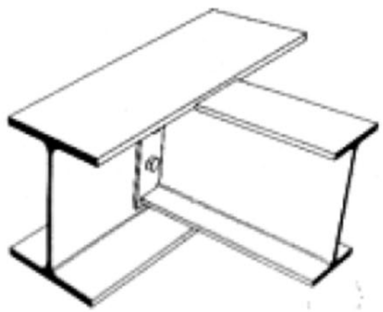
60mm
Fv = 200/4 = 50 kN
Fv = 0,6 · 800 · 245 / 1,25 = 94,1 kN
FIX
SEMI-RIGID
PINNED

Fin plate connection

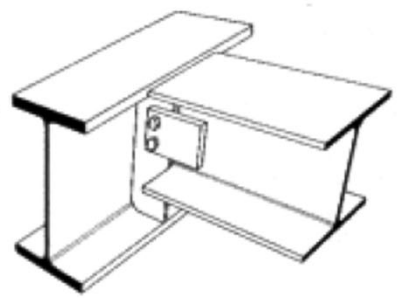
Design requirements for sufficient rotation capacity



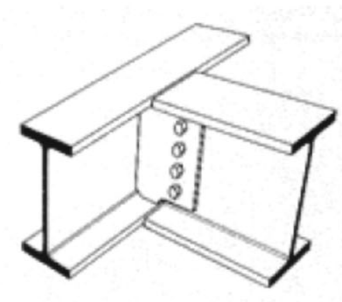
Single sided beam-beam connections



BB1



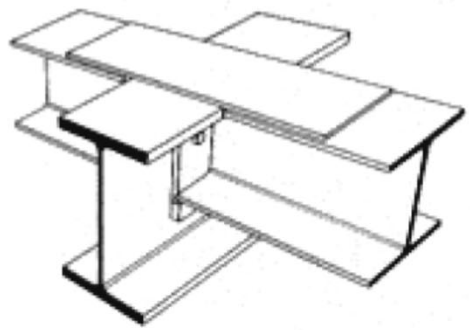
BB2



BB3

- Free in major axis bending since the main girder do not have torsional stiffness enough to provide fixation.
- The connections do not have rigid end plates that can prevent warping and are not rigid enough to withstand minor axis bending.

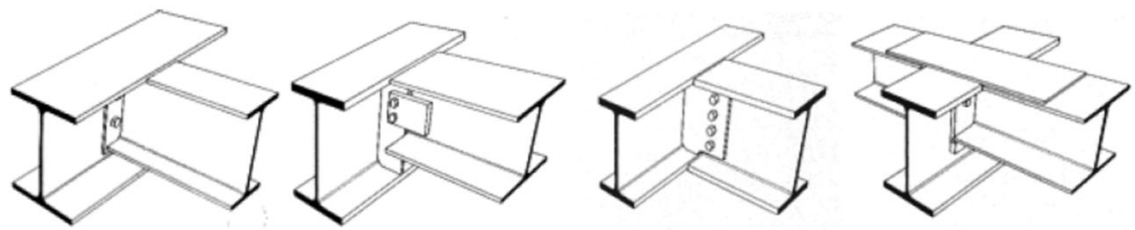
Continuous beam-beam connection



BB4

- Can transfer moment about the major axis with help of the splice plate welded to the beams top flanges.
- The bottom flanges of the secondary beams are not fixed to the main girder. The resistance to lateral bending is therefore limited and a conservative assumption of a free minor axis bending is reasonable.

Rigidity of common beam/beam connections

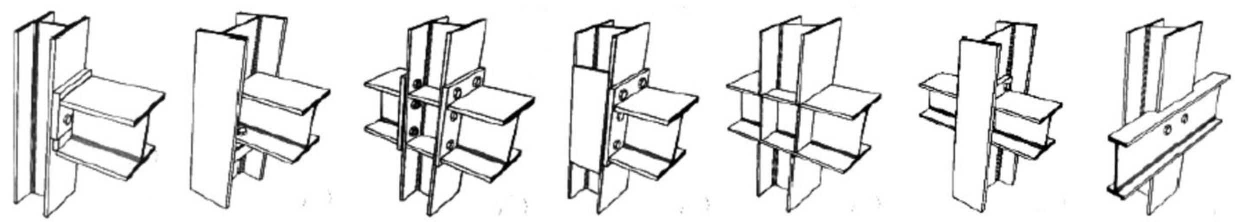


BB1 BB2 BB3 BB4

Boundary conditions	*BB1-BB3*	*BB4*
Major axis bending	Free	Fixed
Minor axis bending (k_z)	Free	Free
Warping (k_w)	Free	Free/Fixed[1]

[1]Dependent on the rigidity of the connecting structure and the end plates

Rigidity of common beam/column connections



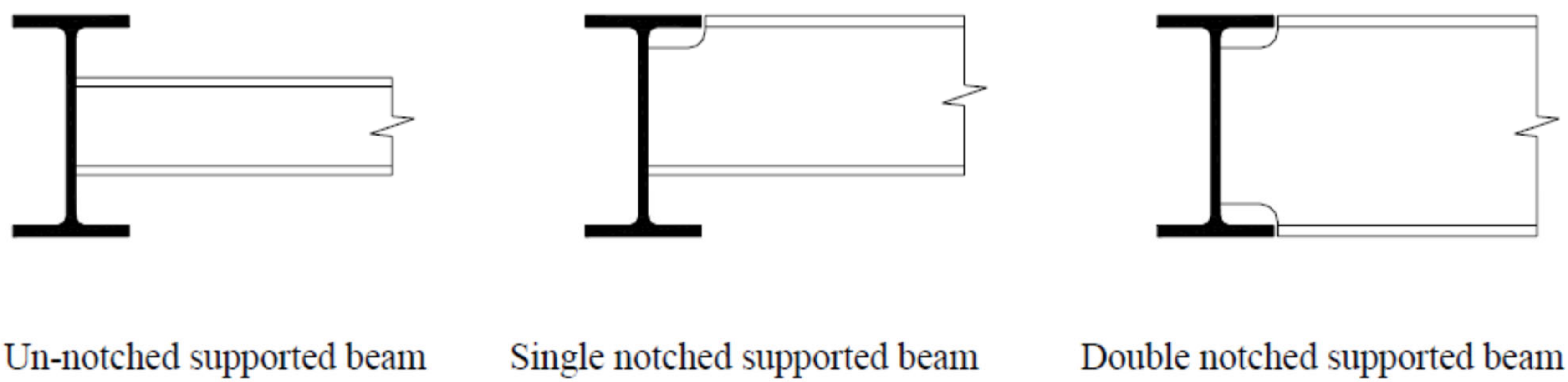
BC1 BC2 BC3 BC4 BC5 BC6 BC7

Boundary conditions	*BC1*	*BC2*	*BC3 & BC4*	*BC5*	*BC6*	*BC7*
Major axis bending	Free	Free	Fixed	Fixed	Fixed	Fixed
Minor axis bending (k_z)	Free	Free	Fixed	Fixed	Free	Fixed
Warping (k_w)	Free/Fixed[1]	Free	Free/Fixed[1]	Fixed	Free/Fixed[1]	Free

[1]Dependent on the rigidity of the connecting structure and the end plates

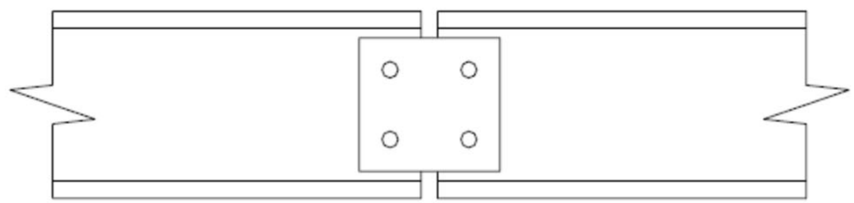
Possible joint configurations

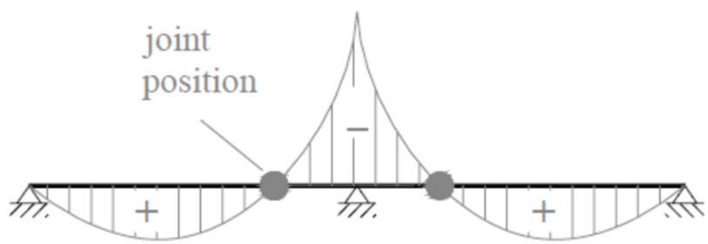
Beam-to-beam

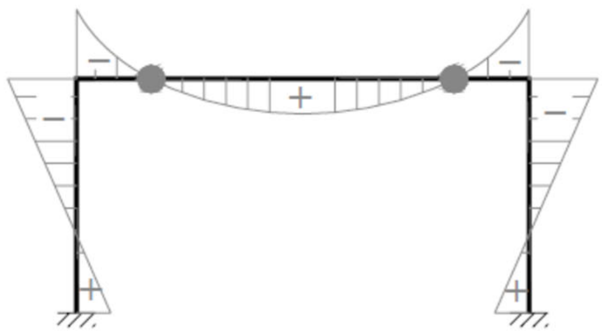


Possible joint configurations

Beam splice

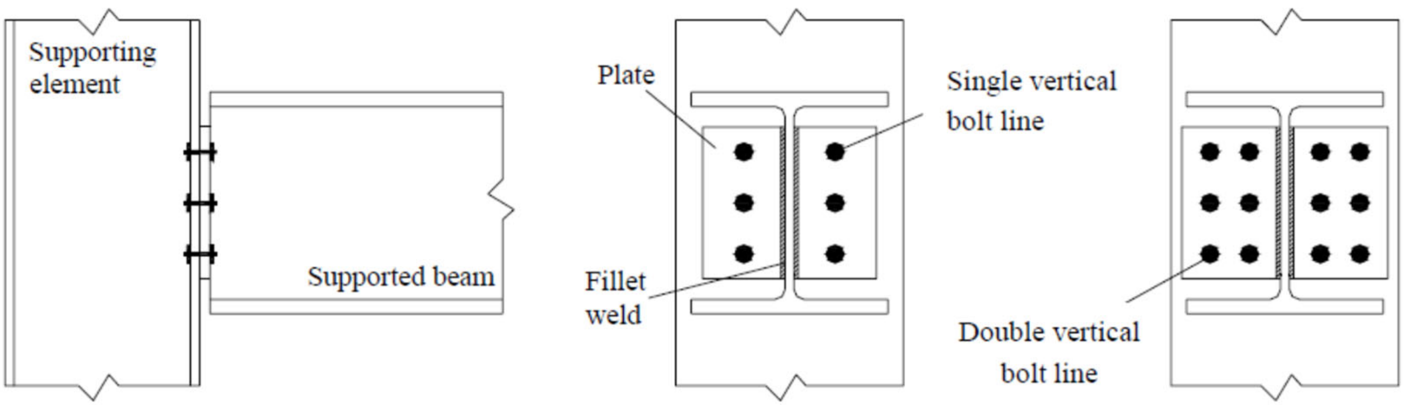






Beam-to-column connections

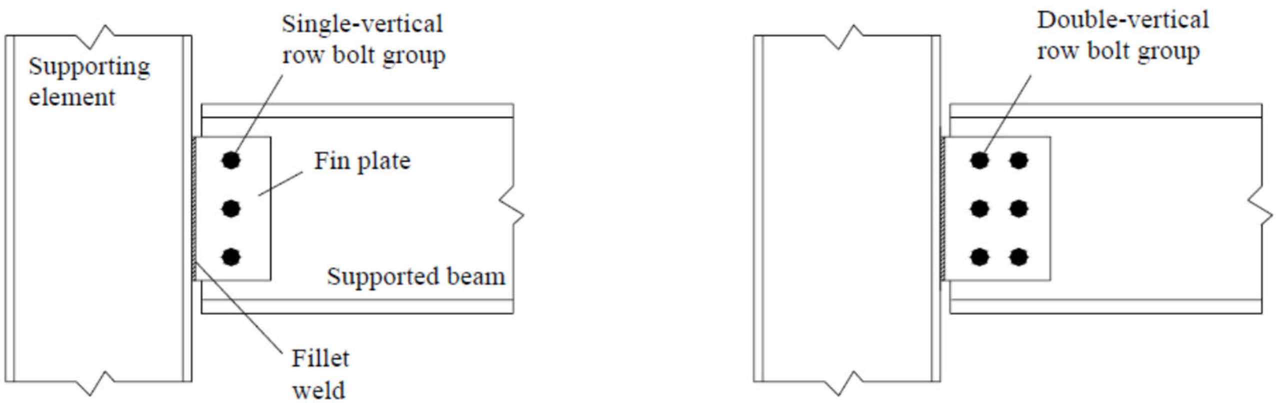
Header plate



- A header plate is welded to the supported member in the shop and bolted to the supporting member on site.
- The height of the header plate should not exceed the clear depth of the supported beam.

Beam-to-column connections

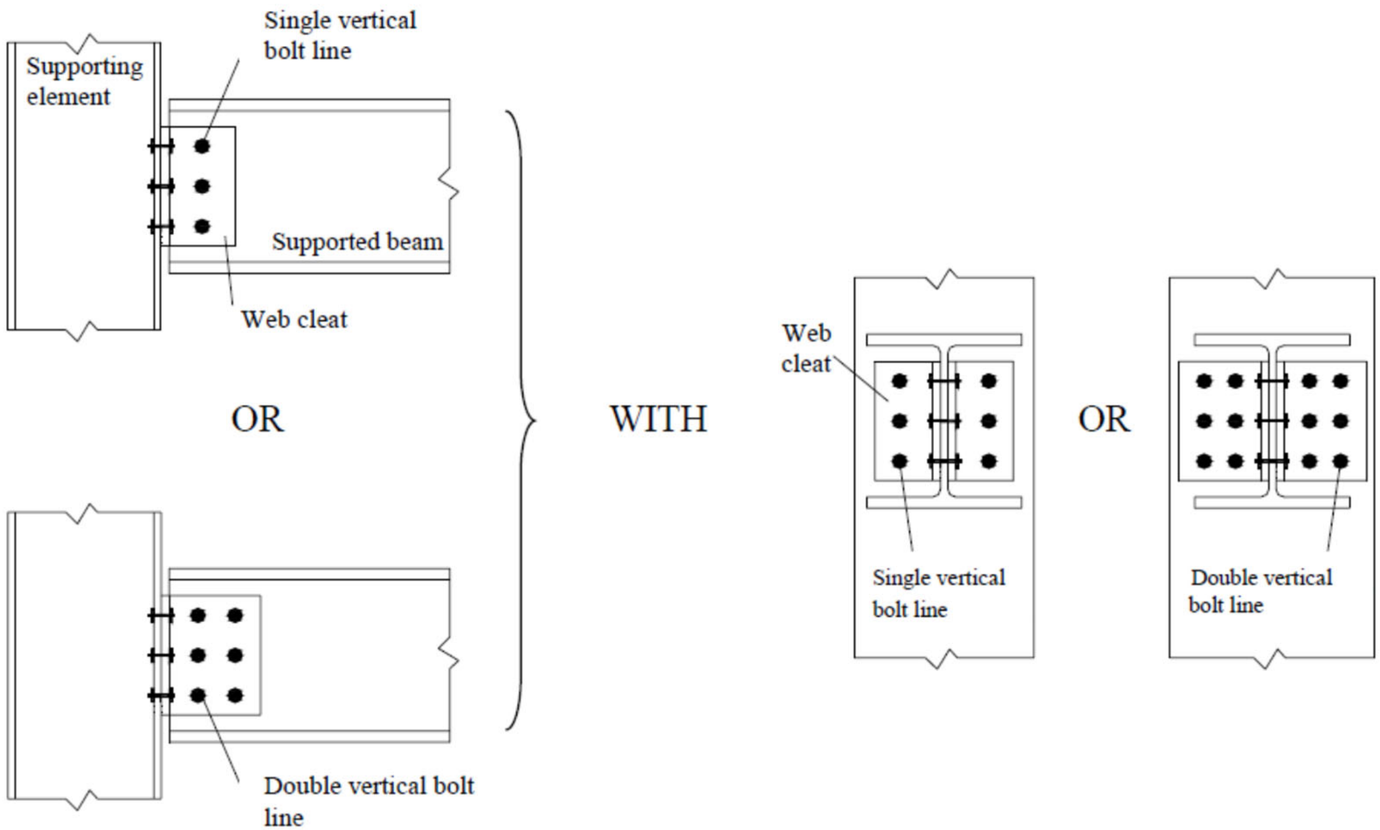
Fin plate



- A fin plate is welded to the supporting member in the shop and bolted to the web of the supported member on site.
- The fillet weld should be laid on both sides of the fin plate.

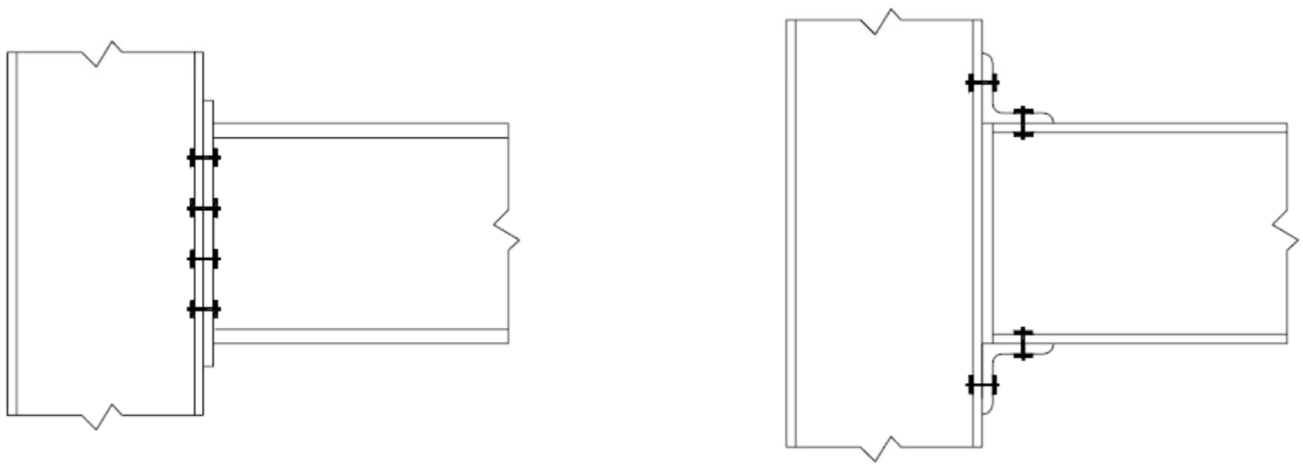
Beam-to-column connections

Web cleats



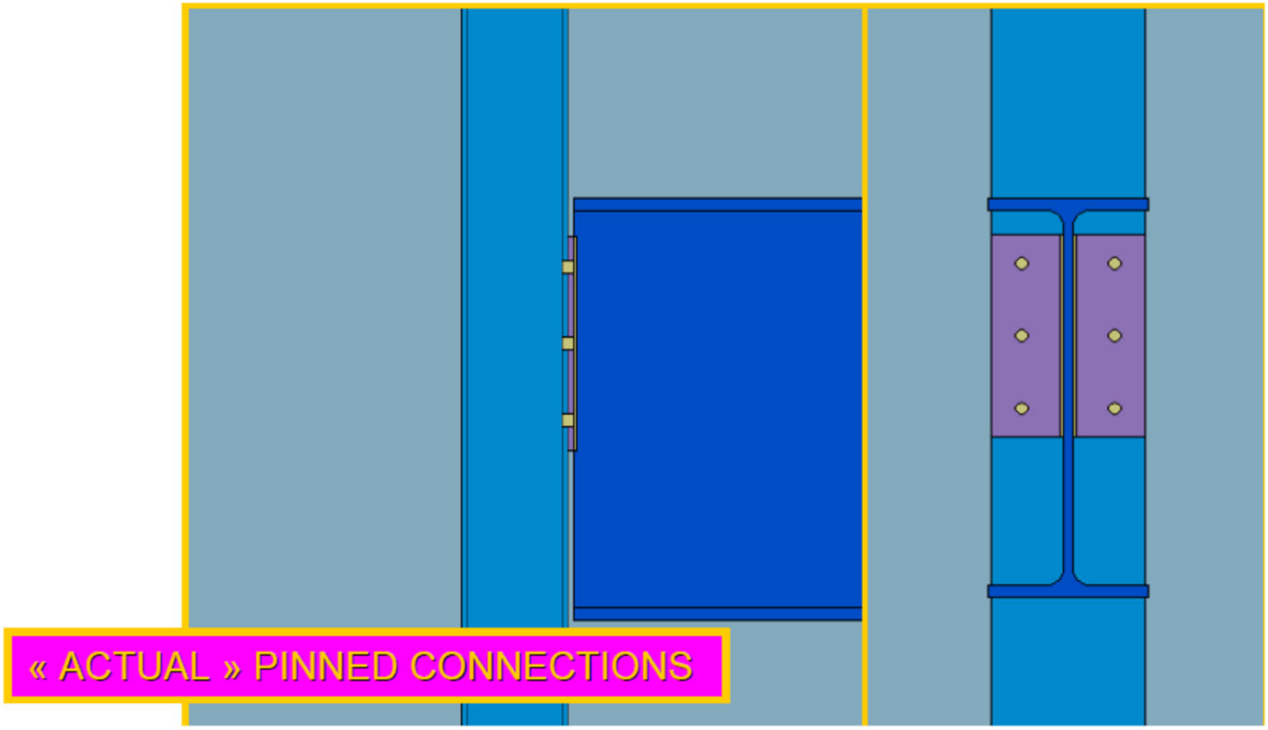
Beam-to-column connections

Semi-rigid connections

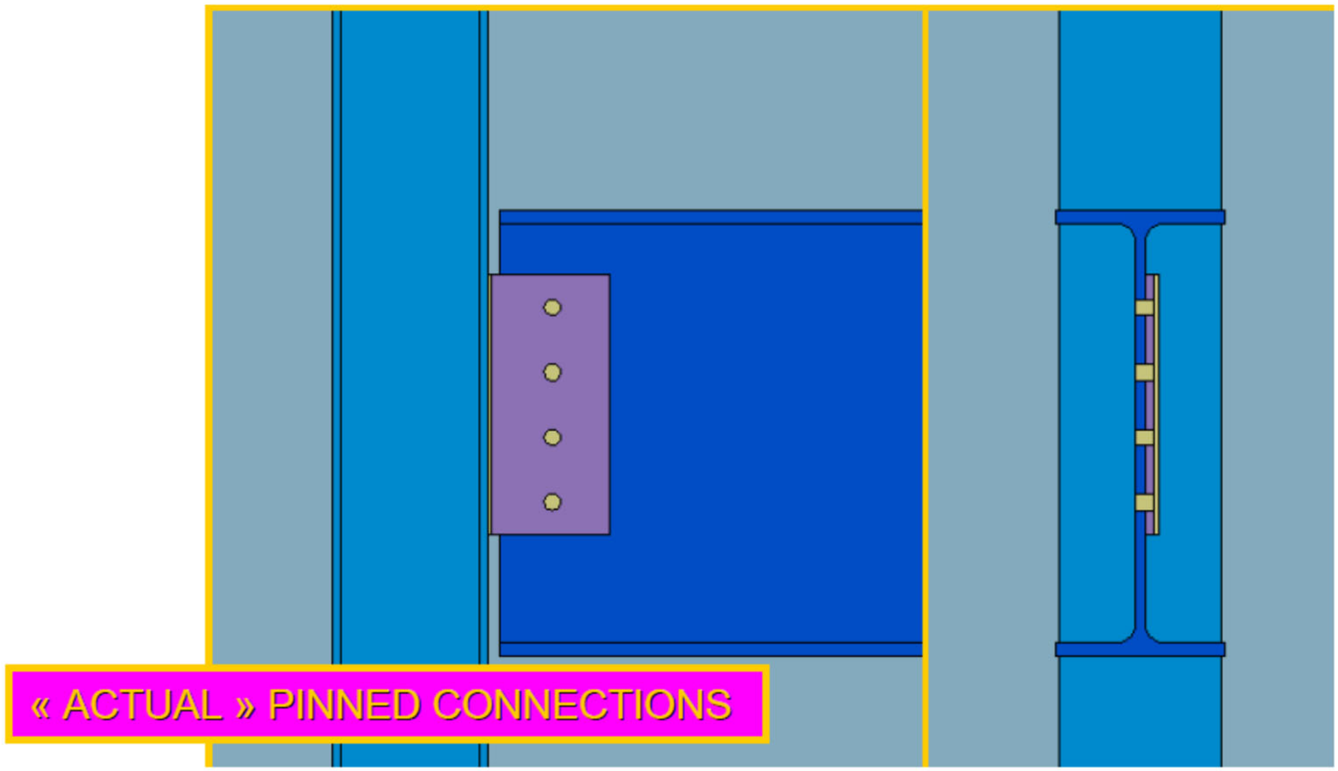


- The connections above used to be considered as hinges, but nowadays Eurocode 3 Part 1-8 classifies them as semi-rigid.

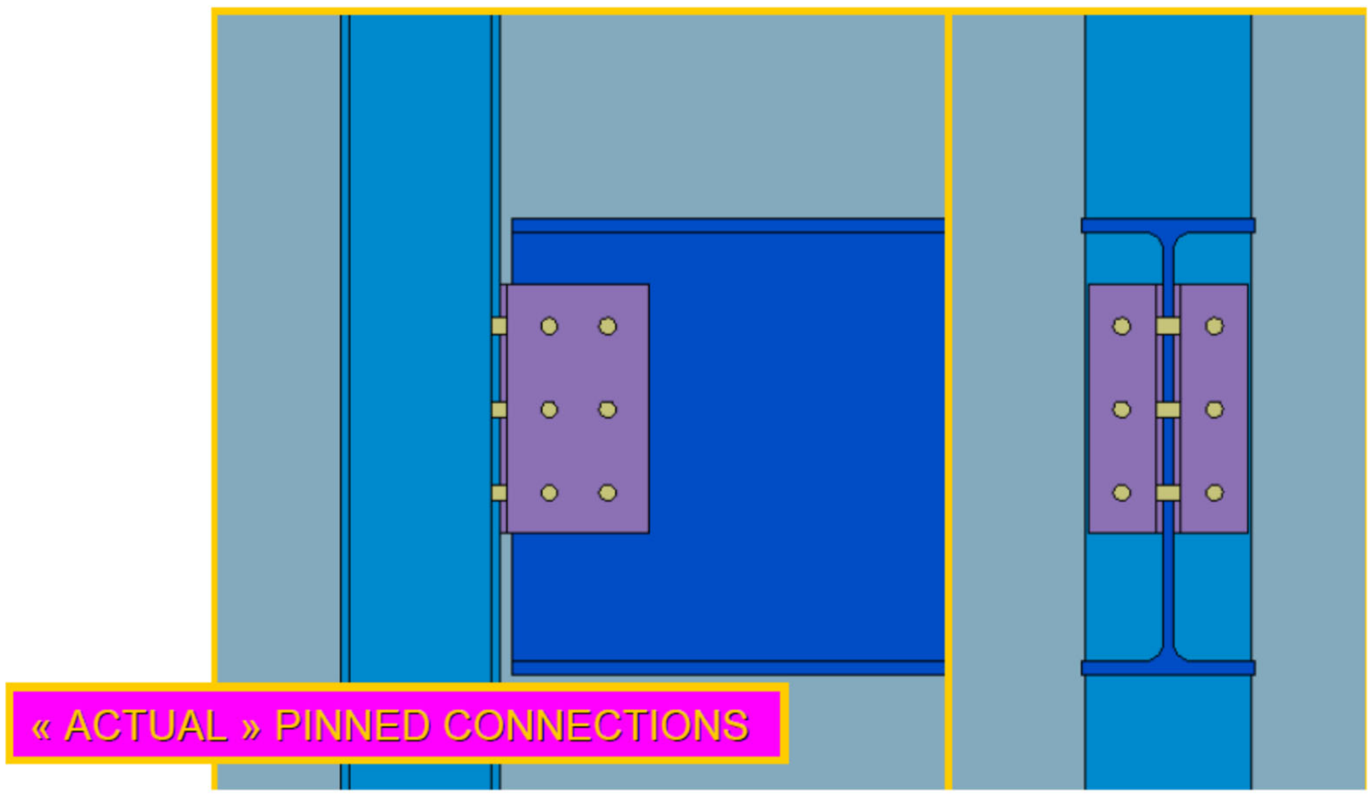
Header plate connection



Fin plate connection



Web cleat connection

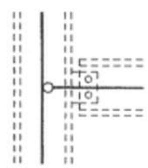
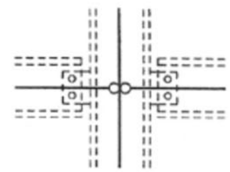
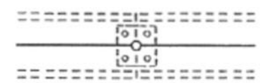
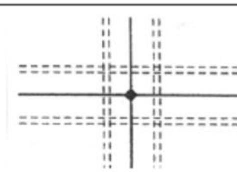
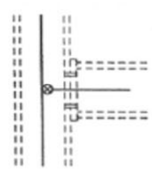
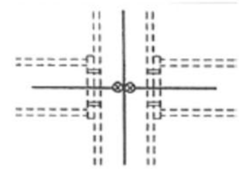
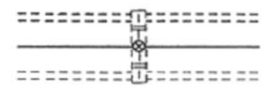


Types of joint modelling

To identify whether the effects of joint behaviour on the analysis need be taken into account, a distinction should be made between the three following types of joint modelling:

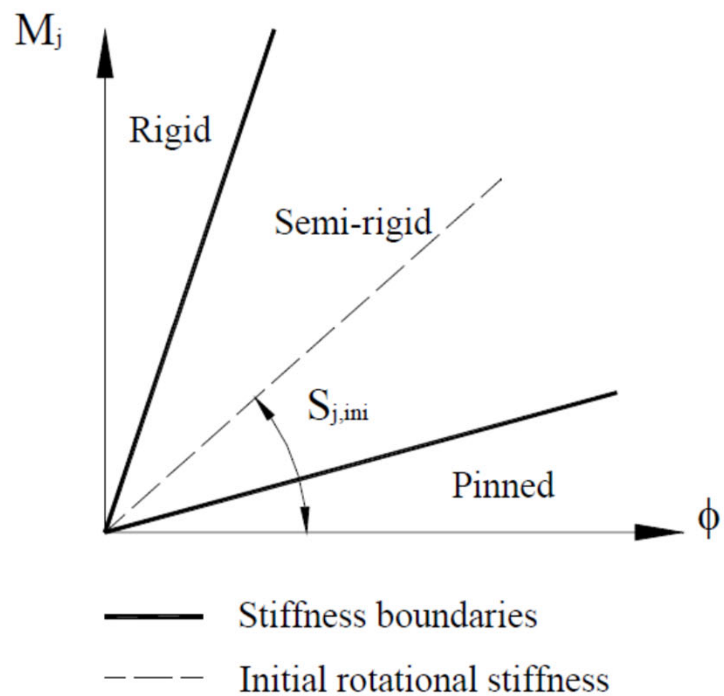
- **Simple** - the joint may be assumed to not transfer bending moments $\rightarrow$ hinge support.
- **Continuous** - the behaviour of the joint may be assumed to have no effect on the analysis.
- **Semi-continuous** - the actual behaviour of the joints needs to be taken into account $\rightarrow$ rotational spring support.

Local joint modelling

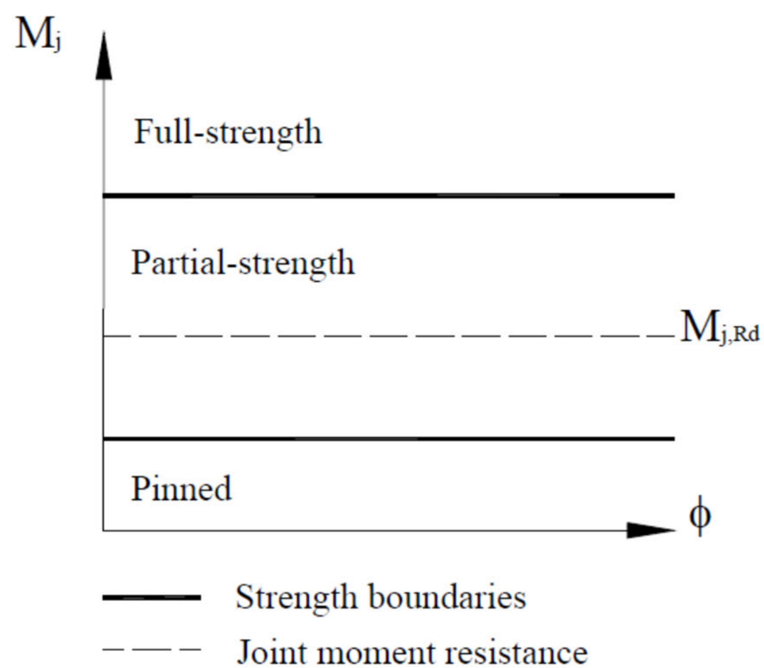
TYPE OF JOINT MODEL	SINGLE-SIDED CONFIGURATION	DOUBLE-SIDED CONFIGURATION	BEAM SPLICE
Simple			
Continuous			
Semi-continuous			

In the global structural analysis, the hinge or spring which models the joint is assumed to be located at the intersection of the axes of the connected elements.

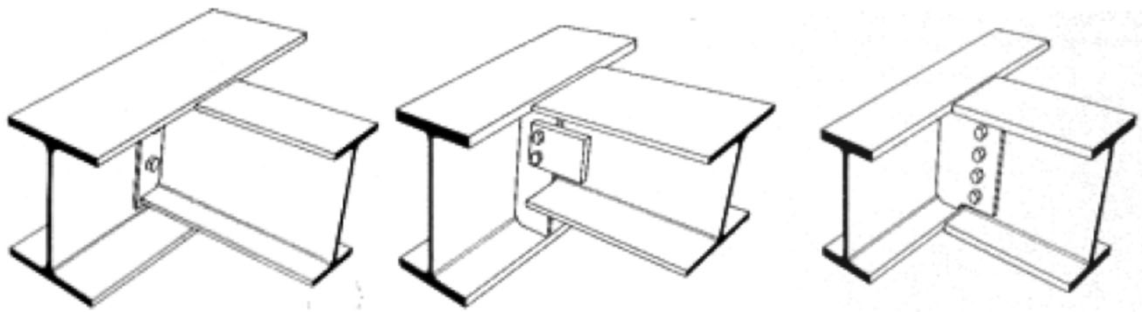
Boundaries for stiffness classification of joints



Boundaries for strength classification of joints



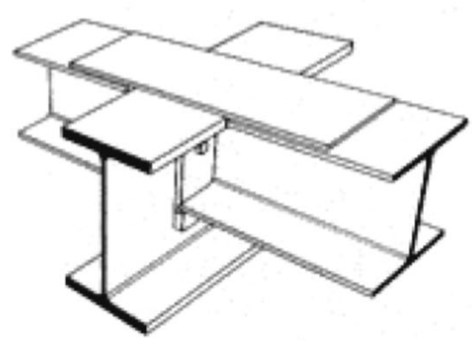
Single sided beam-beam connections



BB1 BB2 BB3

- Free in major axis bending since the main girder do not have torsional stiffness enough to provide fixation.
- The connections do not have rigid end plates that can prevent warping and are not rigid enough to withstand minor axis bending.

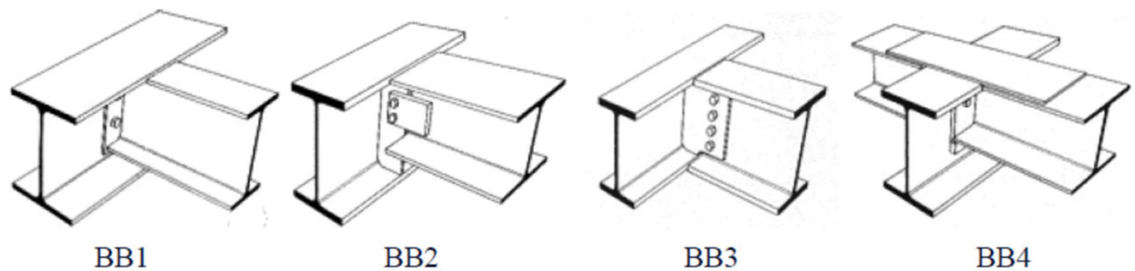
Continuous beam-beam connection



BB4

- Can transfer moment about the major axis with help of the splice plate welded to the beams top flanges.
- The bottom flanges of the secondary beams are not fixed to the main girder. The resistance to lateral bending is therefore limited and a conservative assumption of a free minor axis bending is reasonable.

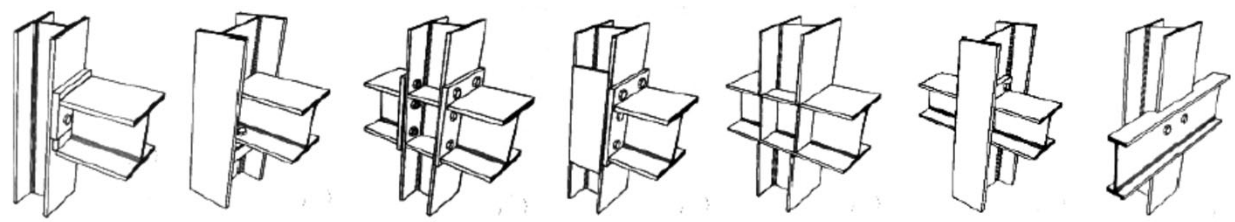
Rigidity of common beam/beam connections



Boundary conditions	*BB1-BB3*	*BB4*
Major axis bending	Free	Fixed
Minor axis bending (k_z)	Free	Free
Warping (k_w)	Free	Free/Fixed[1]

[1]Dependent on the rigidity of the connecting structure and the end plates

Rigidity of common beam/column connections



BC1 BC2 BC3 BC4 BC5 BC6 BC7

Boundary conditions	*BC1*	*BC2*	*BC3 & BC4*	*BC5*	*BC6*	*BC7*
Major axis bending	Free	Free	Fixed	Fixed	Fixed	Fixed
Minor axis bending (k_z)	Free	Free	Fixed	Fixed	Free	Fixed
Warping (k_w)	Free/Fixed[1]	Free	Free/Fixed[1]	Fixed	Free/Fixed[1]	Free

[1]Dependent on the rigidity of the connecting structure and the end plates

Eurokode 3: Prosjektering av stålkonstruksjoner
Del 1-8: Knutepunkter og forbindelser

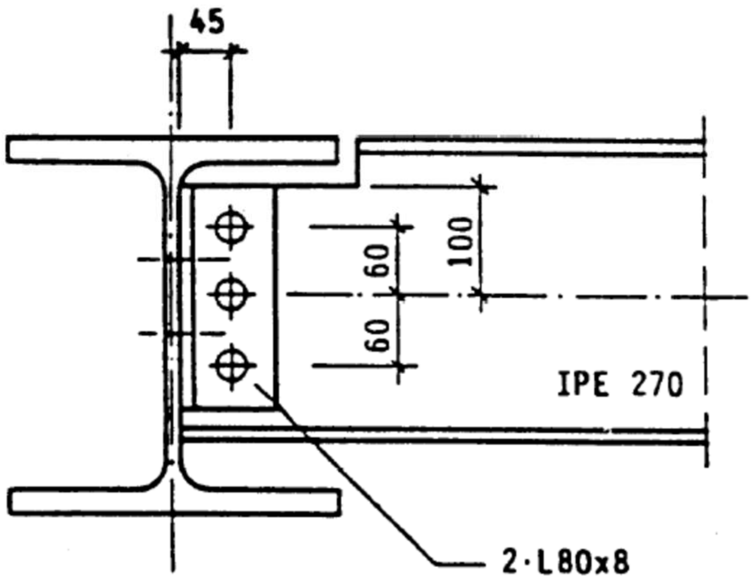
Eurocode 3: Design of steel structures
Part 1-8: Design of joints

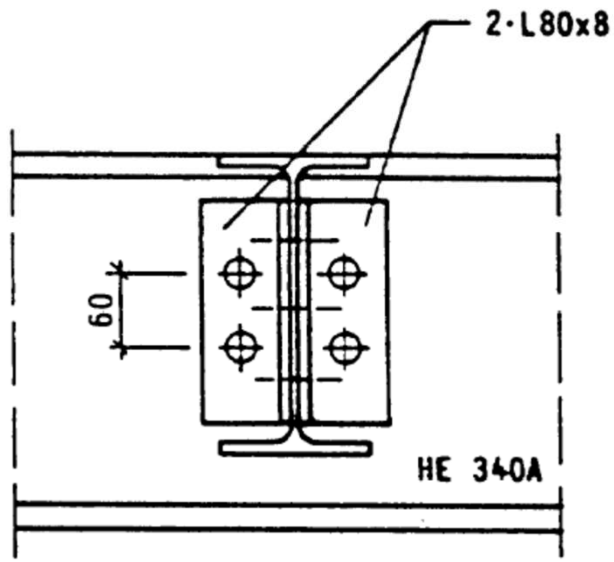
Secondary IPE270 beam connected to main HE340A beam

The secondary beam is assumed to be simply supported to the main beam, i.e. no moment is to be transmitted. According to the action effect analysis a shear force $V_{Ed} = 200\,kN$ is to be transmitted by the connection.

M20-8.8 bolts

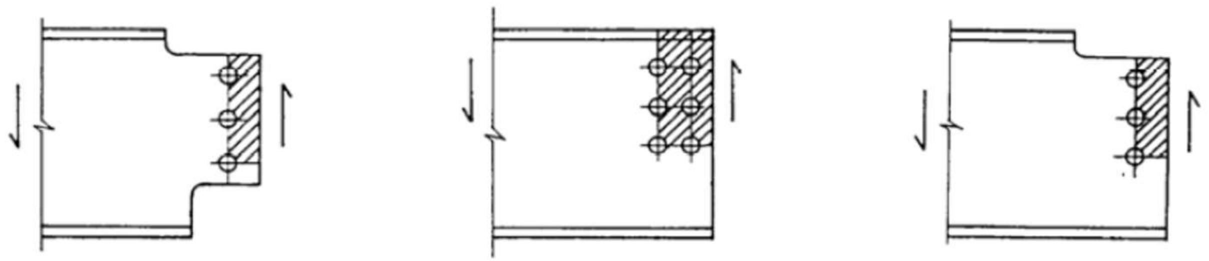
S355 steel $\qquad d_0 = 20 + 2 = 22\,mm$

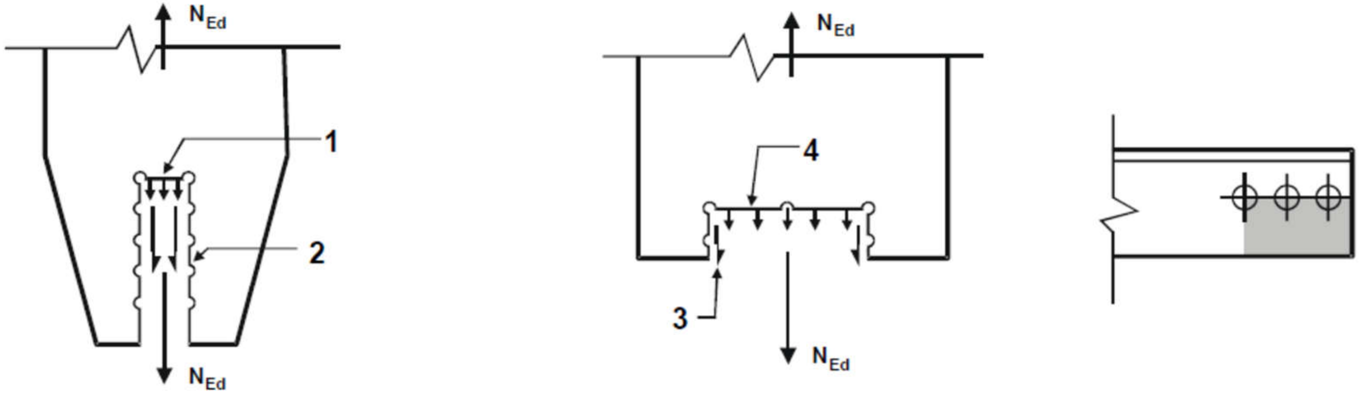




(3) For en skruegruppe påkjent av en eksentrisk kraft er dimensjonerende kapasitet mot blokkutriving $V_{eff,2,Rd}$ gitt ved:

$$V_{eff,2,Rd} = 0{,}5\, f_u\, A_{nt} / \gamma_{M2} + (1 / \sqrt{3})\, f_y\, A_{nv} / \gamma_{M0} \qquad (3.$$





1 liten strekkraft

2 stor skjærkraft

3 liten skjærkraft

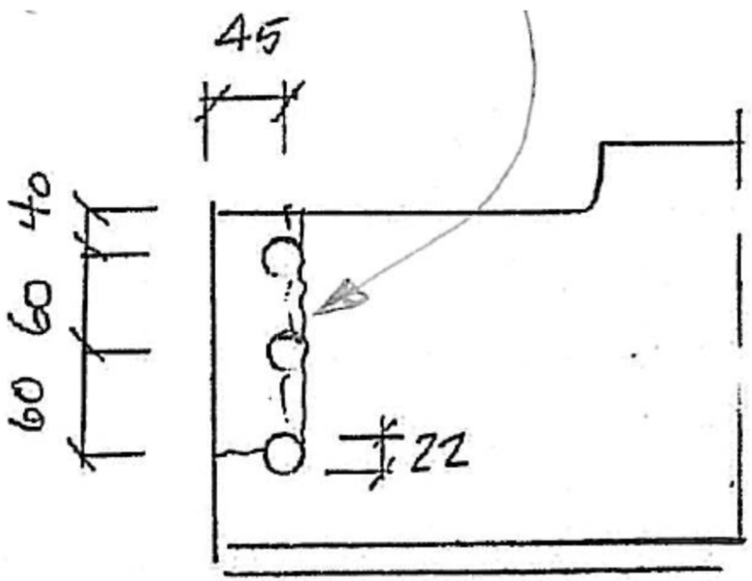
4 stor strekkraft

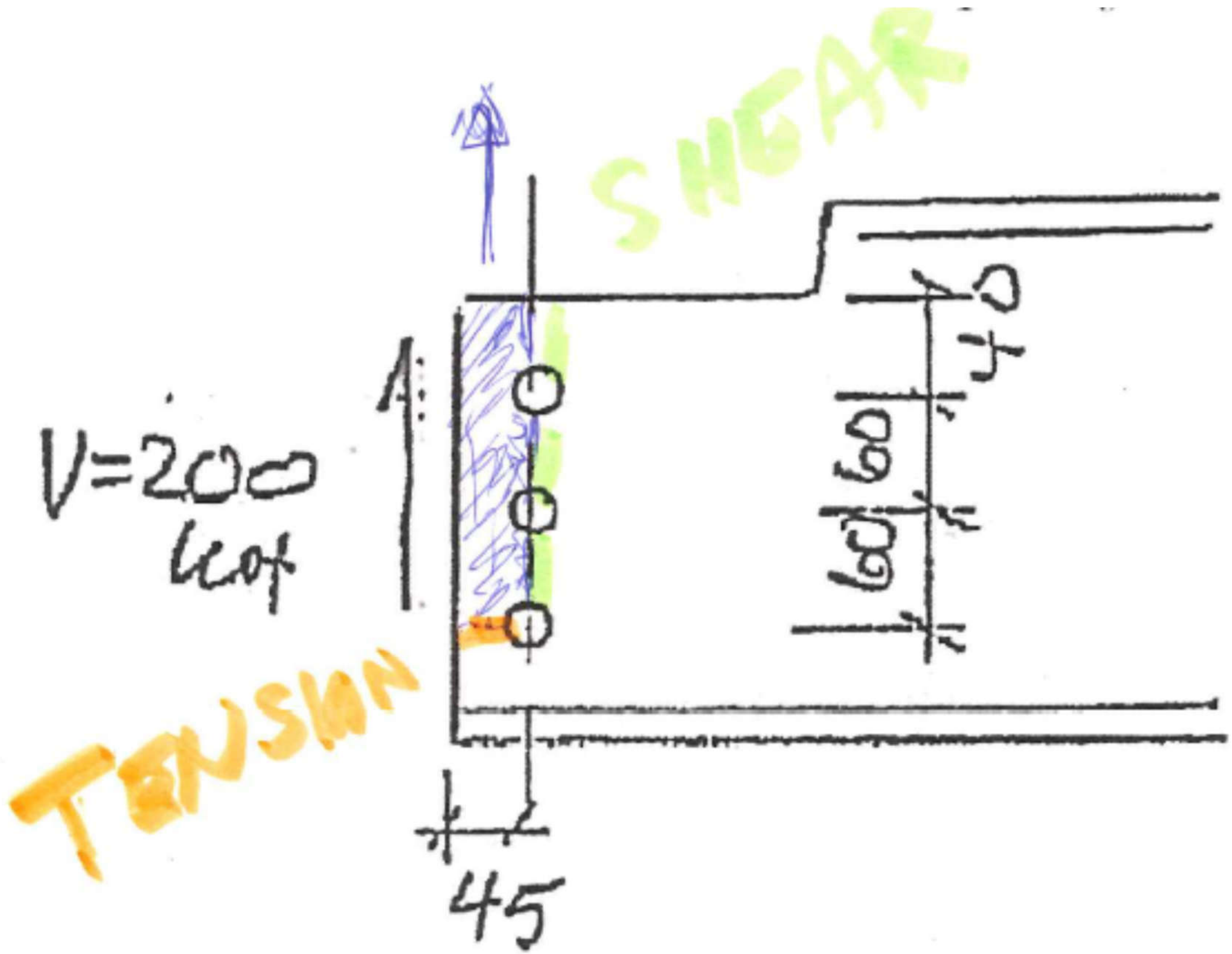
Block tearing of bolt group IPE270

According to NS-EN 1993-1-8 3.10.2(3), for a bolt group subject to eccentric loading, the design block shear tearing resistance is:

$$V_{eff,2,Rd} = 0.5 \cdot \frac{f_u A_{nt}}{\gamma_{M2}} + \frac{f_y A_{nv}}{\sqrt{3}\gamma_{M0}}$$

Block tearing consists of failure in shear at the row of bolts along the shear face of the hole group accompanied by tensile rupture along the line of bolt holes on the tension face of the bolt group.




SHEAR
V=200
kN
TENSION
45
40
60
60

The net area subject to tension is:

$$A_{nt} = (45 - 11) \cdot 6.6 = \underline{224\,mm^2}$$

The net area subject to shear is:

$$A_{nv} = (2 \cdot (60 - 22) + (40 - 11)) \cdot 6.6 = \underline{693\,mm}$$

Design load corresponding to tearing of bolt group:

$$V_{eff,2,Rd} = 0.5 \cdot \frac{510 \cdot 224}{1.3} + \frac{355 \cdot 693}{\sqrt{3} \cdot 1.15} = \underline{167\,kN} < V_{Ed} \;\rightarrow \boxed{\textbf{not OK!}}$$

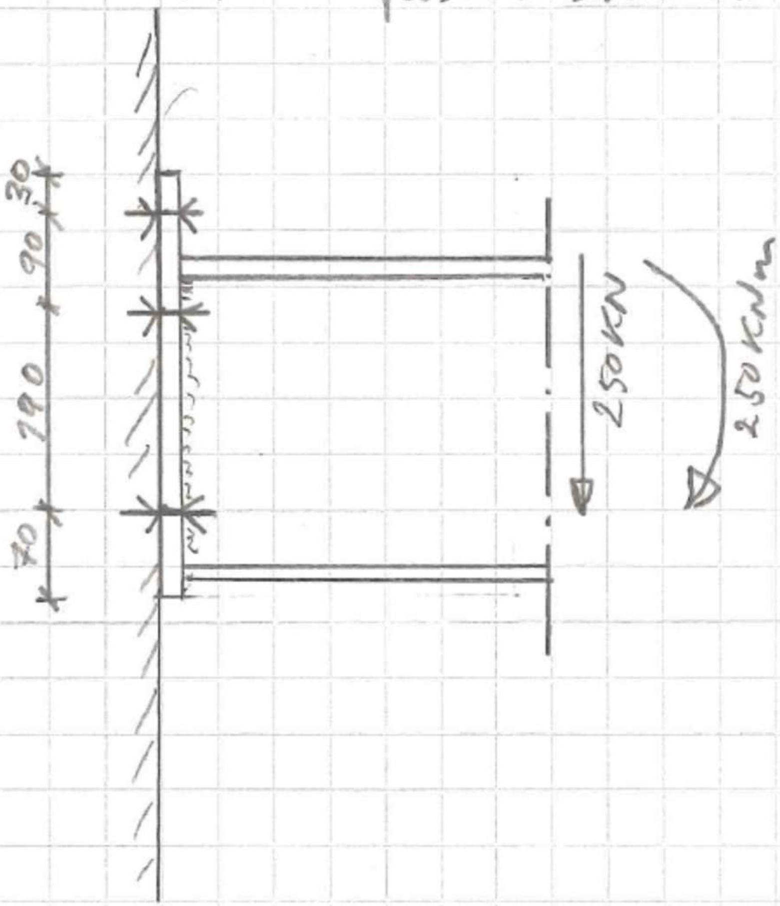
Conclusion

The bearing resistance of the bolts in the IPE270 is too small,, the stress in the reduced section is above the yield strength, and block tearing of bolt group is critical.

Kontroll bolter:

M20 fasthetsklasse 8.8. $\Rightarrow$ $f_u = 800\,N/mm^2$

$f_y = 640\,N/mm^2$



11.4.3 Strekkforbindelser med bøyedeformasjoner

(Hevarm prinsippet)

① Meget stiv flens: Brudd i skruer.

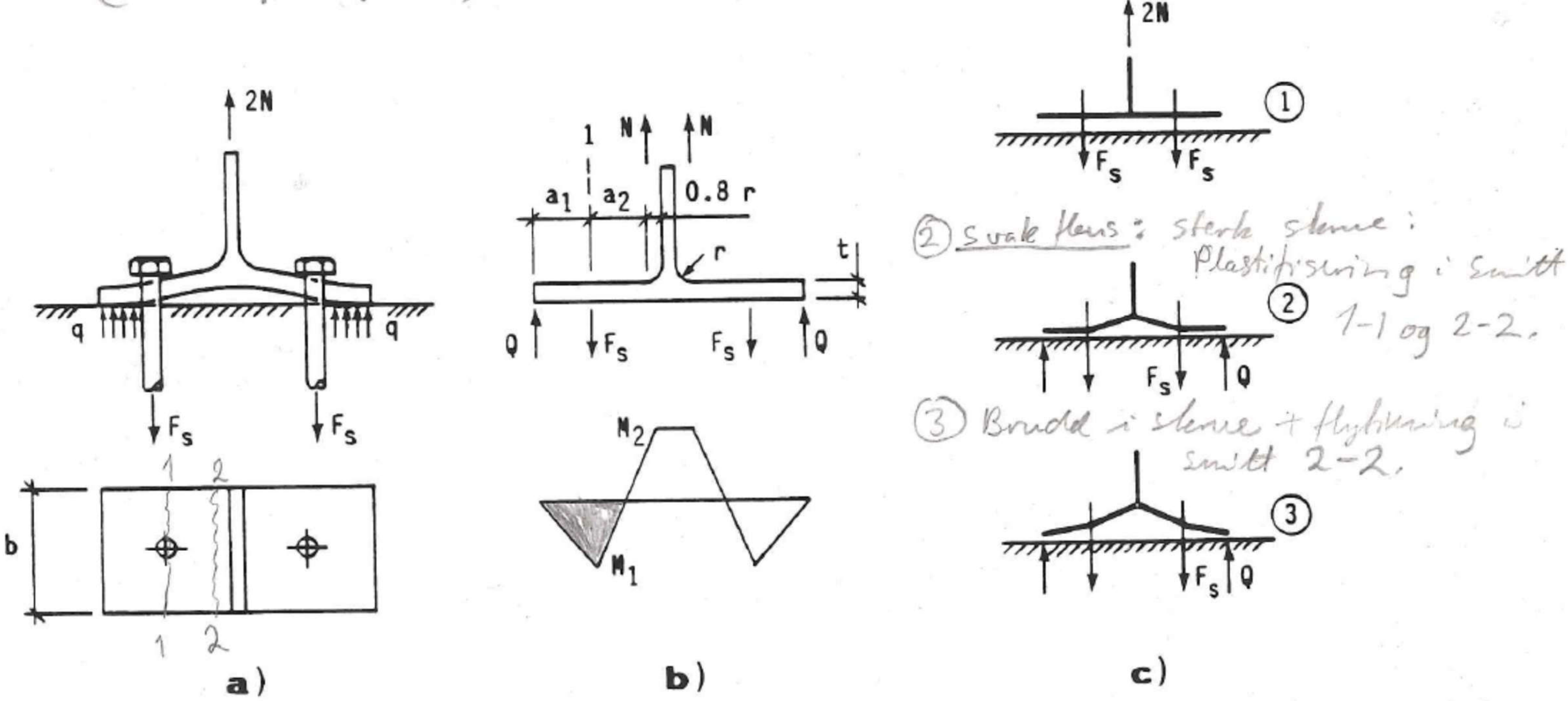
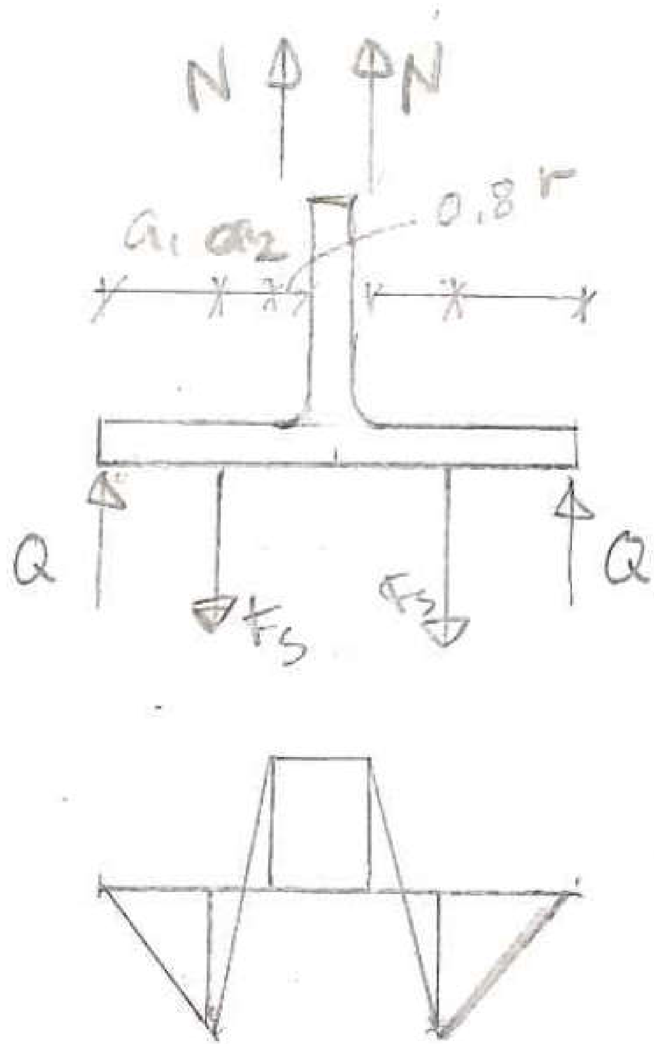


Fig 11.34 - Deformasjoner og krefter i T-forbindelse

Hevarm:



1). <u>Meget stiv flens</u>:

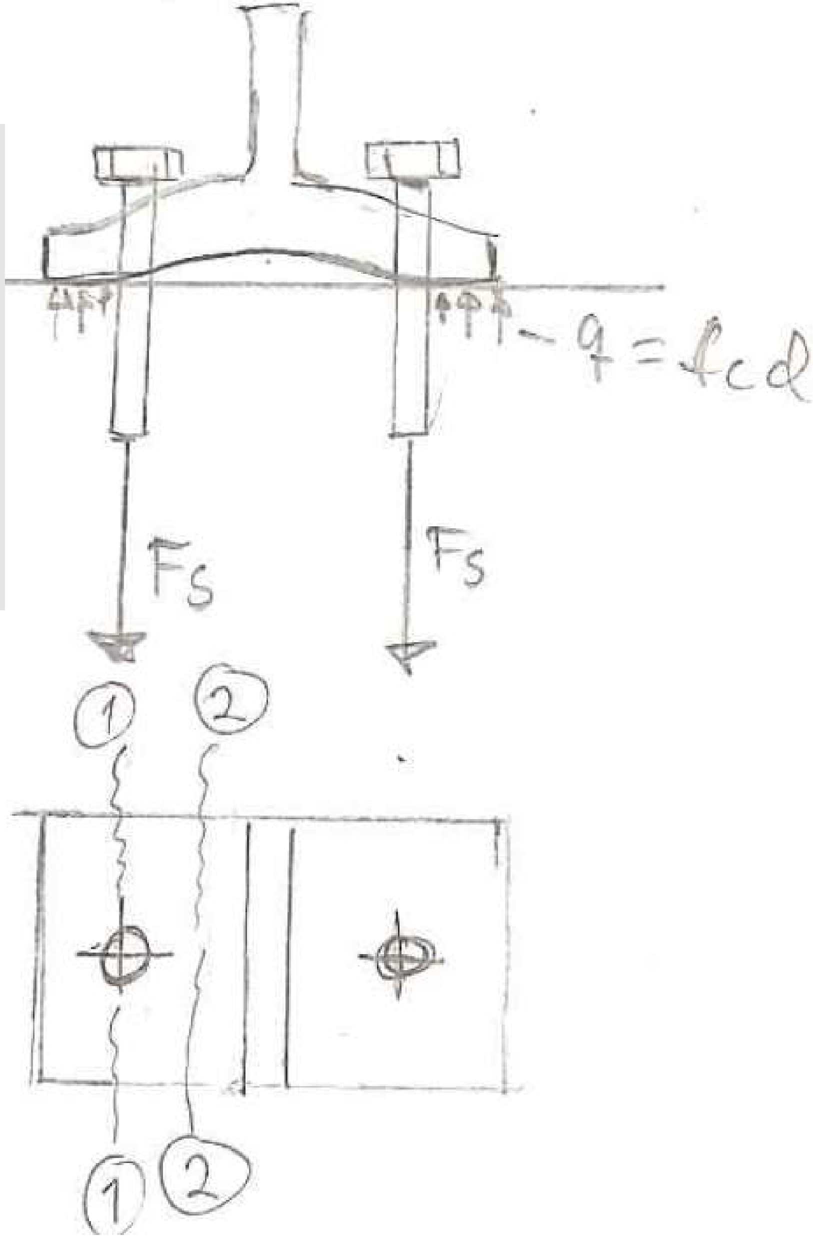
Kun brudd i skruer.

2). <u>Svak flens, sterk skrue</u>:

Plastifisering i snitt 1-1 og 2-2.

3). Brudd i skrue + flytning i 2-2:

$$F_S = N + Q$$



4.5.3.2 Retningsmetode

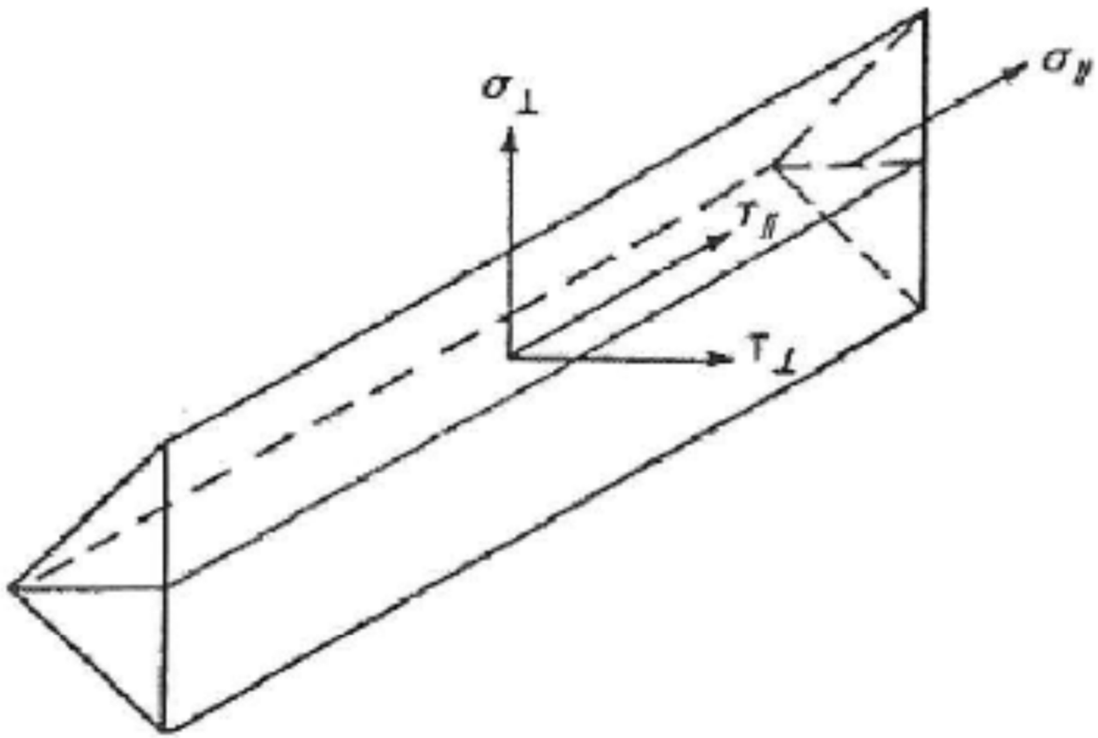
(1) I denne metoden dekomponeres kreftene som overføres per lengdeenhet av sveisen opp i komponenter parallelt med og på tvers av sveisens lengdeakse, og normalt på og på tvers av sveisens kilsveisareal.

(2) Dimensjonerende areal av sveisesnittet, A_w, settes lik: $A_w = \sum a\, \ell_{eff}$.

(3) Dimensjonerende sveisesnitt forutsettes å være konsentrert i rotpunktet.

(4) Spenningen forutsettes jevnt fordelt over sveisesnittet. Dette gir følgende normal- og skjærspenninger vist på figur 4.5:

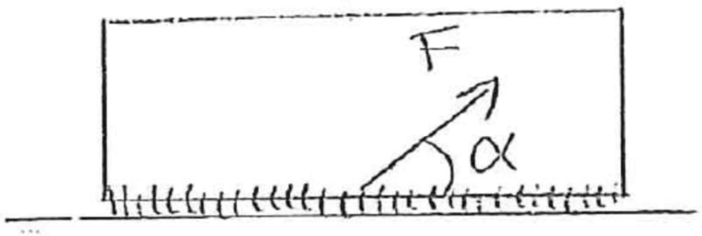
- $\sigma_\perp$ er normalspenning normalt på sveisesnittet;
- $\sigma_\parallel$ er normalspenning parallelt med sveisens akse;
- $\tau_\perp$ er skjærspenning (i sveisesnittets plan) normalt på sveisens lengdeakse;
- $\tau_\parallel$ er skjærspenning (i sveisesnittets plan) parallelt med sveisens lengdeakse.



Figur 4.5 – Spenninger i sveisesnittets plan

b) Why simplified method results in larger a

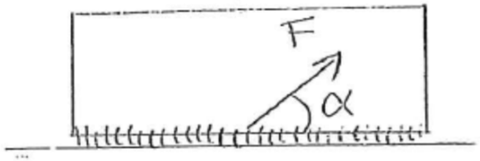
The strength of a weld depends on the angle between the direction of the forces and the orientation of the weld.



$$\alpha = 0^\circ \text{ gives } \tau_{\|} \qquad \rightarrow \text{strength } \frac{1}{\sqrt{3}} \cdot \frac{f_u}{\beta_w \gamma_{M2}}$$

$$\alpha = 90^\circ \text{ gives } \sigma_{\perp} \text{ and } \tau_{\perp} \quad \rightarrow \text{strength } \frac{1}{\sqrt{2}} \cdot \frac{f_u}{\beta_w \gamma_{M2}}$$

Simplified method uses $\frac{1}{\sqrt{3}} \cdot \frac{f_u}{\beta_w \gamma_{M2}}$ irrespective of the direction of the forces, i.e. a conservative strength value. In question a) the stresses $\sigma_{\perp}$ and $\tau_{\perp}$ totally dominates and the directional method results in about $\frac{\sqrt{3}}{\sqrt{2}}$ lower resistance.



b. Fastheten til en sveis er avhengig av vinkelen mellom kraftens angrepsretning og sveisens orientering:

* $\alpha = 0°$ gir $\tau_{\parallel}$ ⇒ Fasthet $\frac{1}{\sqrt{3}} \cdot \frac{f_u}{\beta_w \gamma_{M2}}$
* $\alpha = 90°$ gir $\sigma_{\perp}$ og $\tau_{\perp}$ ⇒ Fasthet $\frac{1}{\sqrt{2}} \frac{f_u}{\beta_w \gamma_{M2}}$

Forenklet metode bruker konsekvent $\frac{1}{\sqrt{3}} \cdot \frac{f_u}{\beta_w \gamma_{M2}}$, dvs. konservativ verdi for fastheten. I sp.m. a. er $\sigma_{\perp}$ og $\tau_{\perp}$ fullstendig dominerende, og dermed gir retningsmetoden ca faktor $\frac{\sqrt{3}}{\sqrt{2}} = 1.22$ lavere kapasitet

(5) Ved fastsettelse sveisens kapasitet ses det bort fra normalspenningen $\sigma_{\parallel}$ parallelt med aksen.

(6) Dimensjonerende kapasitet for en kilsveis er tilfredsstillende hvis begge følgende betingelser er oppfylt:

$$[\sigma_{\perp}^2 + 3\,(\tau_{\perp}^2 + \tau_{\parallel}^2)]^{0,5} \leq f_u \,/\, (\beta_w\, \gamma_{M2}), \text{ og } \sigma_{\perp} \leq 0,9\, f_u \,/\, \gamma_{M2} \qquad (4.1)$$

der

f_u er nominell strekkfasthet i den svakeste delen i forbindelsen;

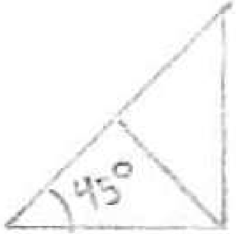
β_w er en korrelasjonsfaktor fra tabell 4.1.

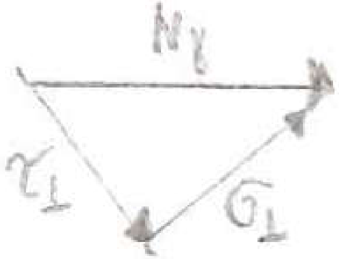
(7) Sveiser mellom deler med ulike stålsorter bør dimensjoneres på grunnlag av egenskapene til godset med den laveste fastheten.

Tabell 4.1 – Korrelasjonsfaktor β_w for kilsveis

Standard og stålsort			Korrelasjonsfaktor β_w
NS-EN 10025	**NS-EN 10210**	**NS-EN 10219**	
S 235 S 235 W	S 235 H	S 235 H	0,8
S 275 S 275 N/NL S 275 M/ML	S 275 H S 275 NH/NLH	S 275 H S 275 NH/NLH S 275 MH/MLH	0,85
S 355 S 355 N/NL S 355 M/ML S 355 W	S 355 H S 355 NH/NLH	S 355 H S 355 NH/NLH S 355 MH/MLH	0,9
S 420 N/NL S 420 M/ML		S 420 MH/MLH	1,0
S 460 N/NL S 460 M/ML S 460 Q/QL/QL1	S 460 NH/NLH	S 460 NH/NLH S 460 MH/MLH	1,0

Sveiser:

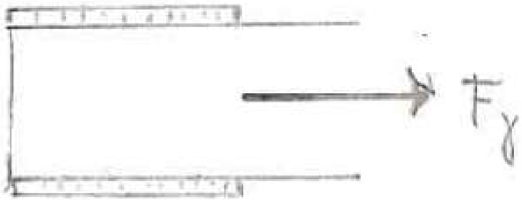




Minste sveiselengde og a-mål (s 44)

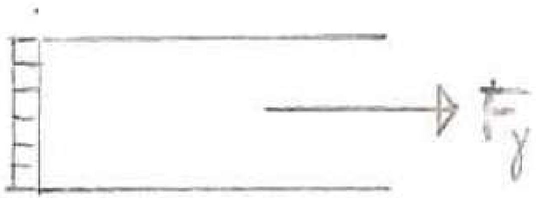
$\tau_\perp = \sigma_\perp$ ved $45°$

1



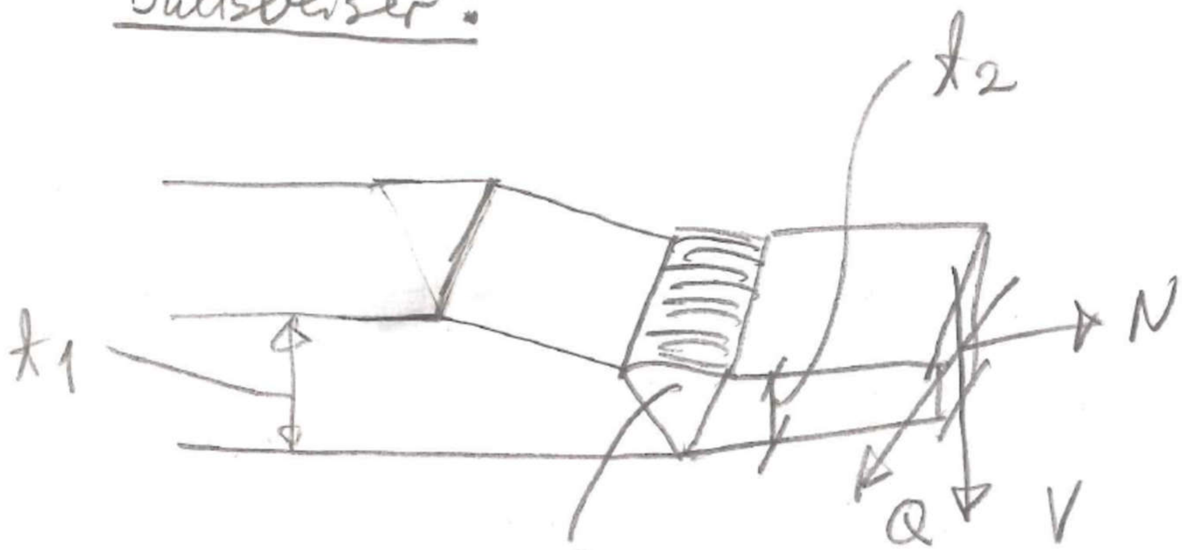
$F_d = 355 \cdot a_e \cdot b / \gamma_m \cdot \sqrt{3}$

2



$F_d = 355 a_e \cdot l_e / \gamma_m \cdot \sqrt{2}$

Buttsveiser:



$f_y = 355\ N/mm^2$

- Buttsveisers kapasitet vil være større/lik kapasiteten av den svakeste av platedelene.
- Vi behøver ikke å beregne kapasiteten av sveisen.

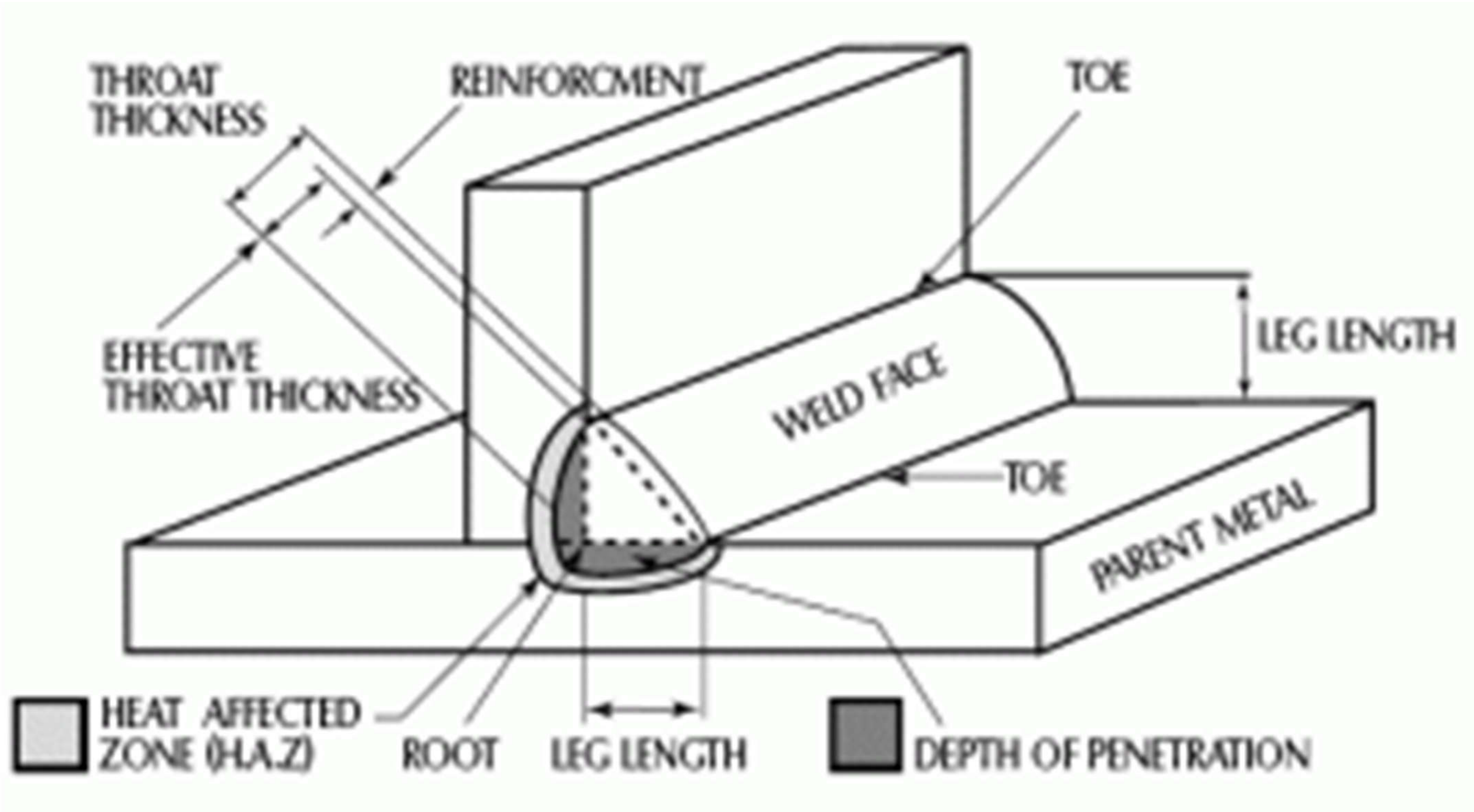
T-FORBINDELSER :

Førstevalg : Kilsveis,
(se sveis nr. 502 på side 6)

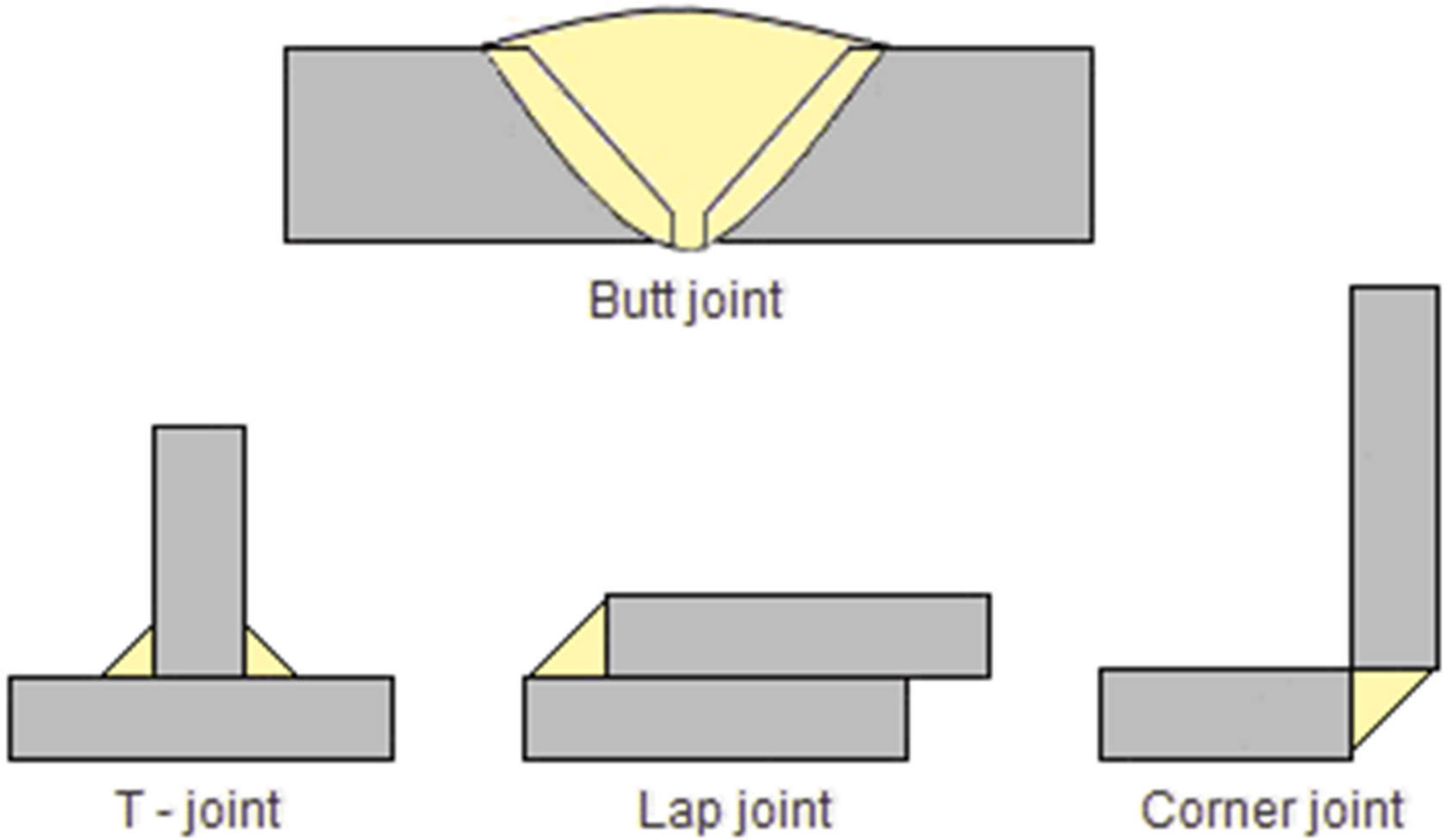
Andrevalg : Delvis innbrenning dersom A-målet overstiger 10mm. (se sveis nr. 422 på side 6)

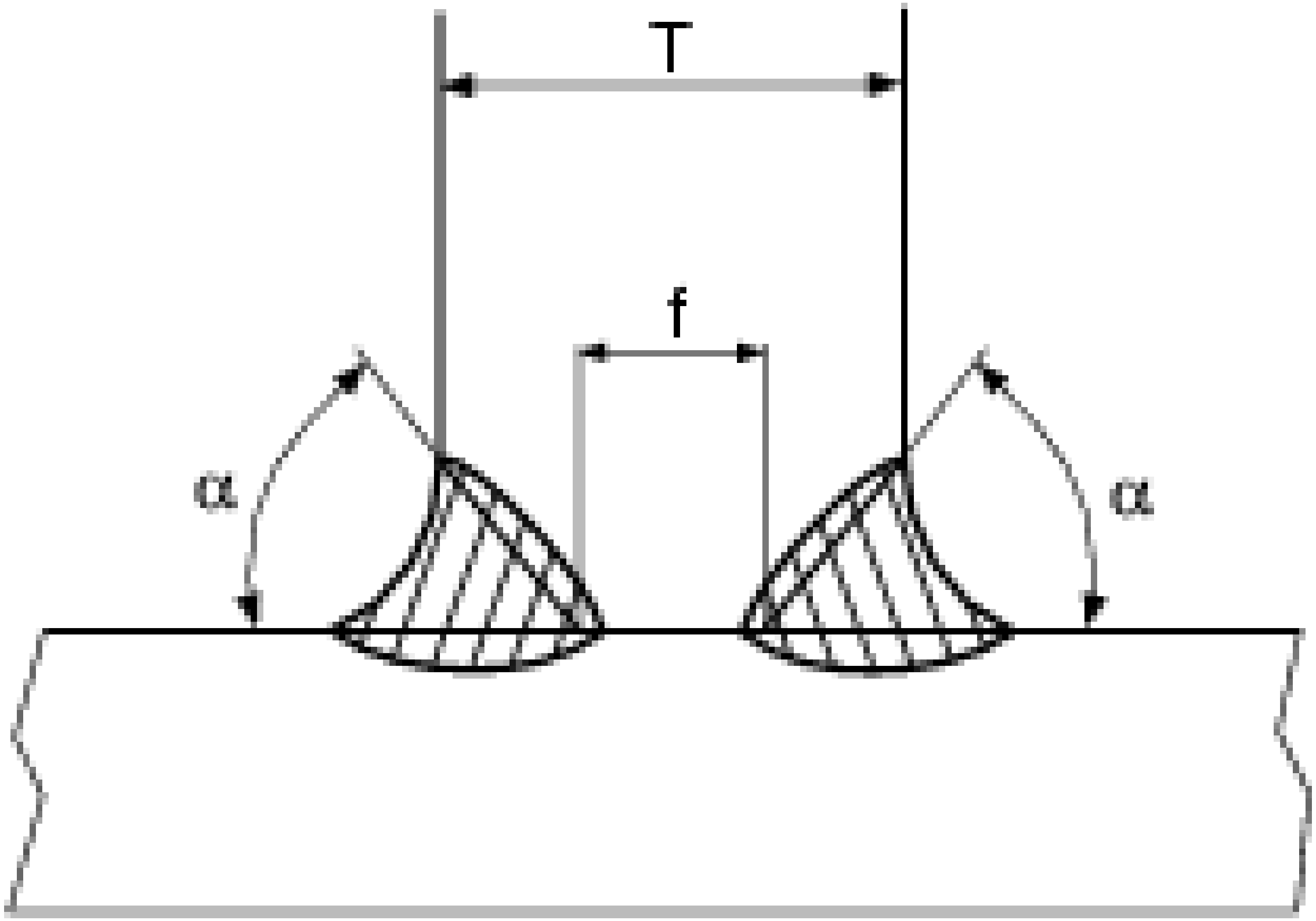
Tredjevalg : Fuger med full gjennombrenning.
a. Platetykkelse ≤ 17,9 mm = ½ V-fuge
(se sveis nr. 312/313 på side 6)
b. Platetykkelse ≥ 18 mm = K-fuge
(se sveis nr. 412 på side 6)

Fillet weld (kilsveis)



Throat thichness = a-mål (NO)


Butt joint
T - joint
Lap joint
Corner joint

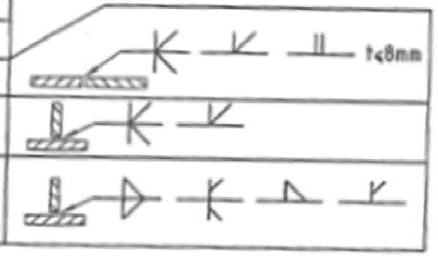


T
f
α
α

WELD NUMBERING

THE FOLLOWING NUMBERSYSTEM IS USED ON TROLL-A PRECOMPRESSION PROJECT

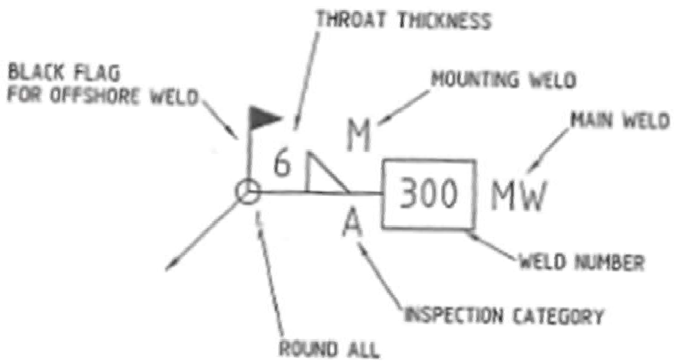
	INSPECTION CATEGORIES				
	A	B	C	D	E
BUTT WELD	100-199	500-599	800-829	996	999
CROSS & T-BUTT WELD, FULL PENETRATION	200-299	600-699	830-869	997	999
CROSS & T-JOINT WELD, PART PENETRATION AND FILLETWELD	300-399	700-799	870-899	998	999



WELDS IN INSPECTION CATEGORI A & B SHALL HAVE INDIVIDUAL WELDNUMBERS.
THE NUMBERING STARTS OVER AGAIN ON EACH DRAWING.
THE LAST NUMBER USED ON THE DRAWING IS TO BE SHOWN IN THE "NOTES" AREA.

MAIN WELDS SHALL BE IDENTIFIED ON THE DRAWING WITH "MW"

WELDS THAT ARE PERFORMED DURING MOUNTING OF SECTIONS SHALL BE IDENTIFIED ON THE DRAWING WITH "M"

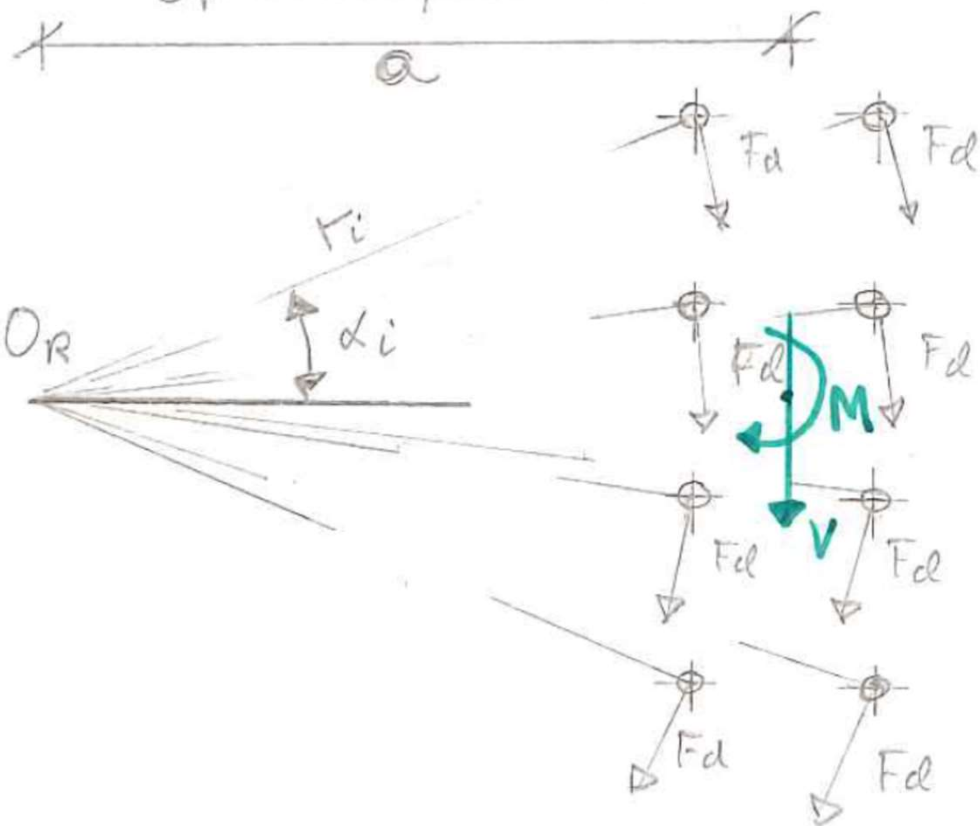


Eurokode 3: Prosjektering av stålkonstruksjoner
Del 1-8: Knutepunkter og forbindelser

Eurocode 3: Design of steel structures
Part 1-8: Design of joints

Skjærkraft + Moment:

1). Plastisk: O_r = rotasjonssenter.



Moment likevekt:

$$\sum_{n=1}^{\Theta} Fd \cdot r_i = [M + V \cdot a]$$

Vertikal likevekt:

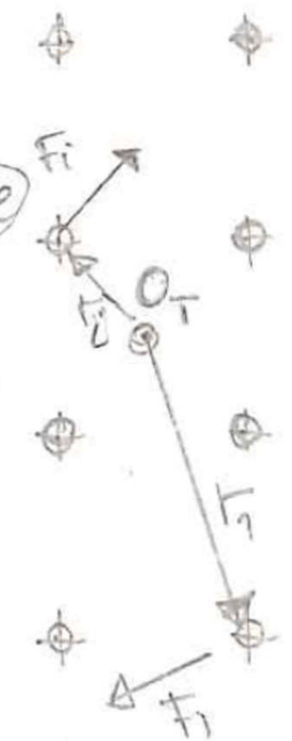
$$\sum_{N=1}^{\infty} Fd \cos \alpha_i = V$$

2). Elastisk;

Moment:

Kraft i skrue "i": $V \cdot e$

$$F_i = \frac{M}{\Sigma r_i^2} \cdot r_i = \frac{M}{I_p} \cdot r_i$$

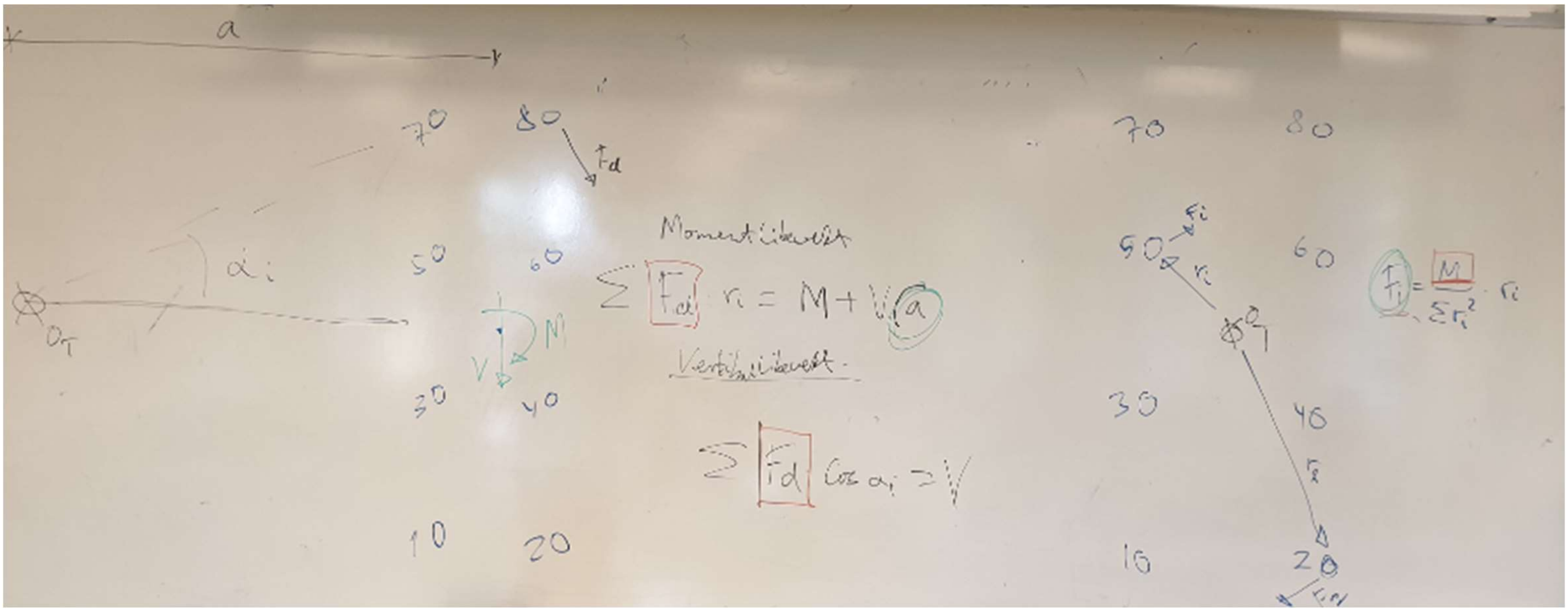


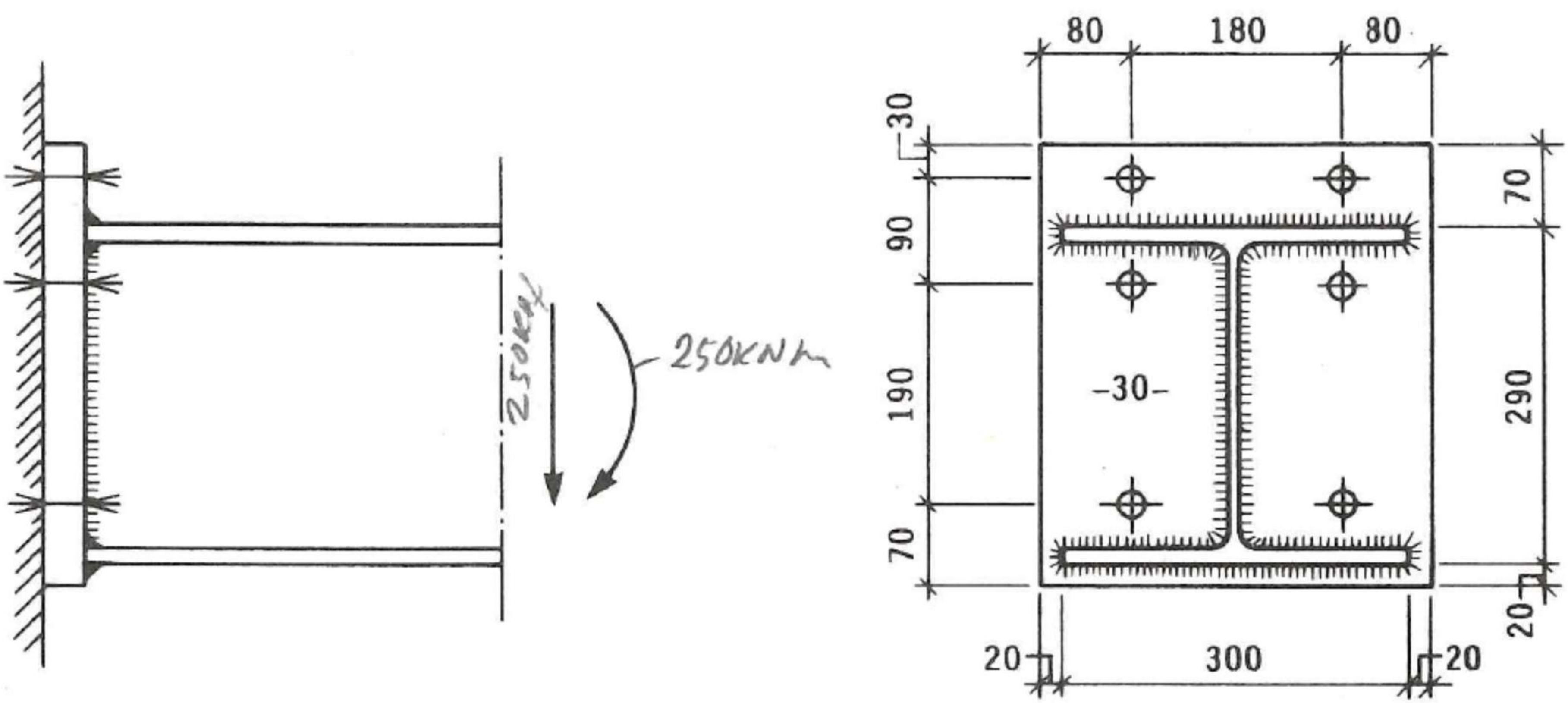
Skjærkraft:

$$F_i = \frac{V}{n}$$

(ytterste bolt mest påkjent; sjekker kun den).

$$\boxed{F_a^2 = V_y^2\left(\frac{V_y \cdot e}{I_p} \cdot r_i\right)^2 + V_y^2\left(\frac{1}{n}\right)^2}$$


a
70
80
F_d
50
60
α_i
O_T
M
V
30
40
10
20
$\sum \boxed{F_d}\, r_i = M + V a$
$\sum \boxed{F_d} \cos \alpha_i = V$
70
80
F_i
50
r_i
60
O_T
$F_i = \frac{\boxed{M}}{\sum r_i^2} \cdot r_i$
30
40
r_2
10
20


250kN
250KNm
80
180
80
30
90
190
70
-30-
70
290
20
20
300
20

I figurene er vist en bjelke av profil HE 300A festet til en stiv vegg ved bruk av en plate med tykkelse t = 30 mm og seks bolter M20 av fasthetsklasse 8.8. Profilet er sveist til platen med dobbeltsidig kilsveis både langs flensene og steget. Det effektive rotmål for sveisene langs flensene er a_{ef} = 5 mm og langs steget a_{es} = 3 mm.

Materialet i bjelken og platen er av stålfasthetsklasse med f_y = 225 N/mm^2 og f_u = 400 N/mm^2.

Bjelken er belastet med skjærkraft og bøyningsmoment med retninger som vist i figuren. I bruddgrensetilstanden er størrelsen av kreftene i innspenningen: Skjærkraft V = 250 kN, bøyningsmoment M = 250 kNm.

Kontroller om alle deler av forbindelsen, inkludert bjelken, har tilstrekkelig kapasitet.

Om noen del av forbindelsen har for liten kapasitet, endres utførelsen slik at fornyet beregning viser at kapasiteten er tilstrekkelig. Endring kan ikke foretas av bjelkeprofil eller av stålfasthetsklasse for materialet i bjelken og platen.

Boltene regnes som avskjæringsforbindelse.

Det benyttes materialfaktor γ_m = 1.1.

KONTROLL AV BOLTER

Den gunstigste kraftfordelingen vil være å la de to nederste bolter ta skjærkraften, og de fire øverste ta strekket fra momentet.

I tillegg til strekkraft fra ytre moment vil boltene også måtte kontrolleres for tillegget i strekkraft pga. hevarmeffekt

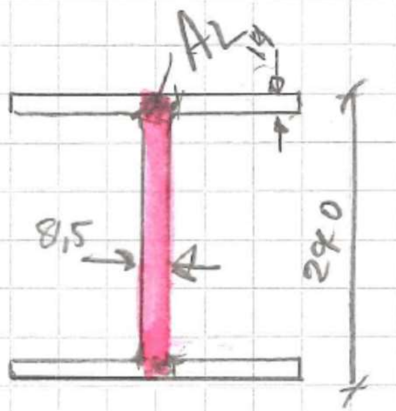
Kontroll bjelke! HE 300 A

Moment: $\sigma = \frac{M}{W} = \frac{250 \cdot 10^6}{1260 \cdot 10^3} = 198{,}4 \, N/mm^2 < f_y$

$\Rightarrow$ O.K

Pkt. A5.1.b).

Skjærkraft: $V_d = A_L \, \tau_d = A_L \cdot \frac{f_y}{\gamma_m} \frac{1}{\sqrt{3}}$



$A_L = (290 - 2 \cdot 14) \cdot 8{,}5 = 2227 \, mm^2$

$V_d = 2227 \, mm^2 \cdot \frac{1}{\sqrt{3}} \cdot \frac{225}{1{,}1}$

$= 263 \, kN$

$\frac{V}{V_d} = \frac{250}{263} > 0{,}9 \Rightarrow$ OK

PLASTISK / ELASTISK
BOLTEFORBINDELSE

$$\sigma = \frac{M}{W} = \frac{250 \cdot 10^6\ \text{kNm}}{1260 \cdot 10^3\ \text{m}^3}$$

$$= \underline{198{,}4}\ \text{MPa}$$

$$\sigma = \frac{F_{flens}}{A_{flens}} = \frac{\frac{M}{a_{cm}} = \frac{250\ \text{kNm}}{0{,}276\ \text{m}} = 905\ \text{kN}}{0{,}3\text{m} \cdot 0{,}014\ \text{m} \quad 4200\ \text{mm}^2}$$

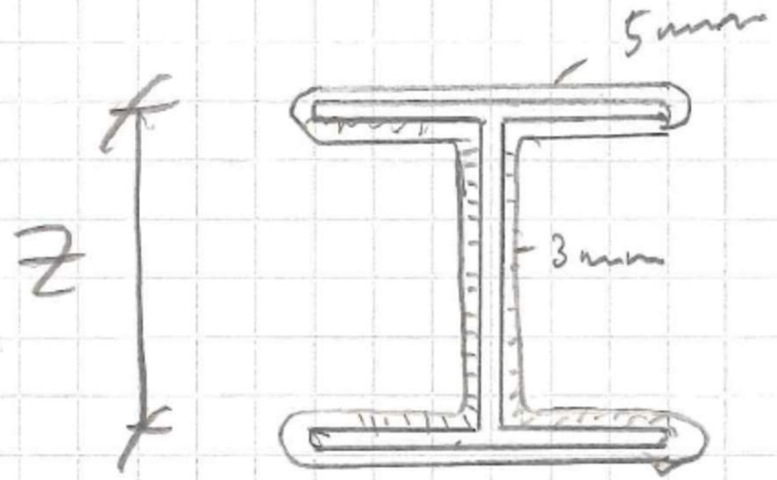
$$= \frac{905\,000}{4200} = \underline{215\ \text{MPa}}$$

$$V_d = A_c \cdot \tau_d \qquad A_c = (290\ \text{mm} - 2 \cdot 14\ \text{mm}) \cdot 8{,}5\ \text{mm}$$

$$= 2227\ \text{mm} \cdot 118\ \text{MPa} \qquad = 2227\ \text{mm}^2$$

OK $= \underline{263\ \text{kN}}$

$$\tau_d = \frac{f_y}{\gamma_m} \frac{1}{\sqrt{3}} = \frac{225}{1{,}1} \frac{1}{\sqrt{3}} = \underline{118\ \text{MPa}}$$



Kapasitet sveis steg:

$$l_e = 290 - 2 \cdot 14 = 262 \text{ mm}^2$$

$$f_{sd} = \frac{355}{\sqrt{3} \cdot 1{,}1} = 186{,}3 \text{ N/mm}^2$$

$$F_{steg} = f_{sd} \cdot A = 186{,}3 \cdot 2 \cdot 3 \cdot 262$$

$$= \underline{293 \text{ kN}} > V = 250 \text{ kN}$$

$\underline{\underline{\text{OK}}}$

KAPASITET SVEIS PÅ STEG!

$l_e = 290mm - 2 \cdot 14 = \underline{262mm}$

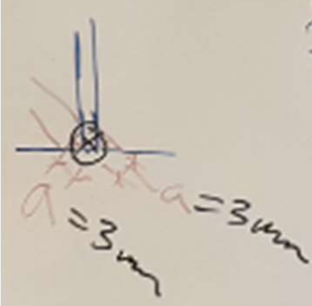
$$f_{sd} = \frac{225\ NPa}{\sqrt{3} \cdot 1,1} = \underline{118,1\ MPa}$$

$$F_{Rd,steg} = f_{sd} \cdot A_{sveis} = 118MPa \cdot 2 \cdot 3mm \cdot 262mm.$$

$$= \qquad = 185\ kN$$

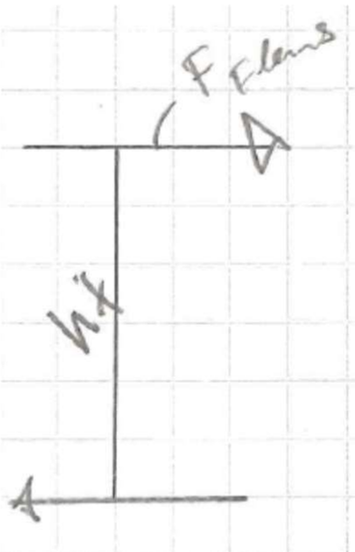
SVEIS ØKES TIL

<u>4mm</u>



PLASISK/ELASTISK
BOLTEFORBINDELSE

TVERRSNITTSKONTROLL
ELASTISK.



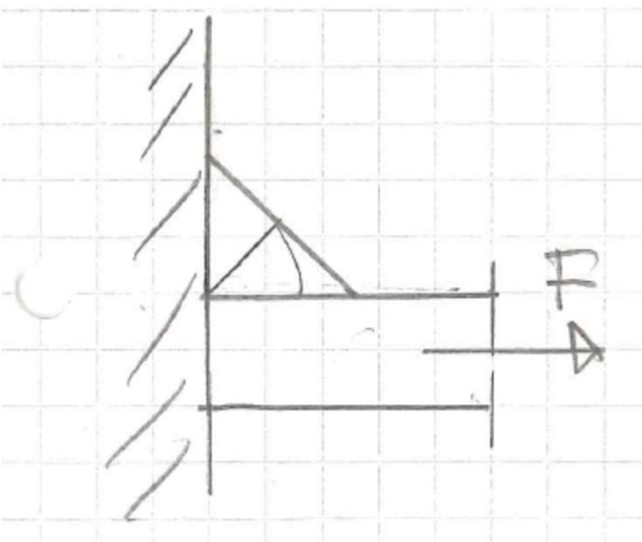
Flens!

$$h_f = 290 - 14 = 276 \text{ mm}$$

$$F_{flens} = \frac{M}{h_f} = A_{flens} \cdot f_{sd} =$$

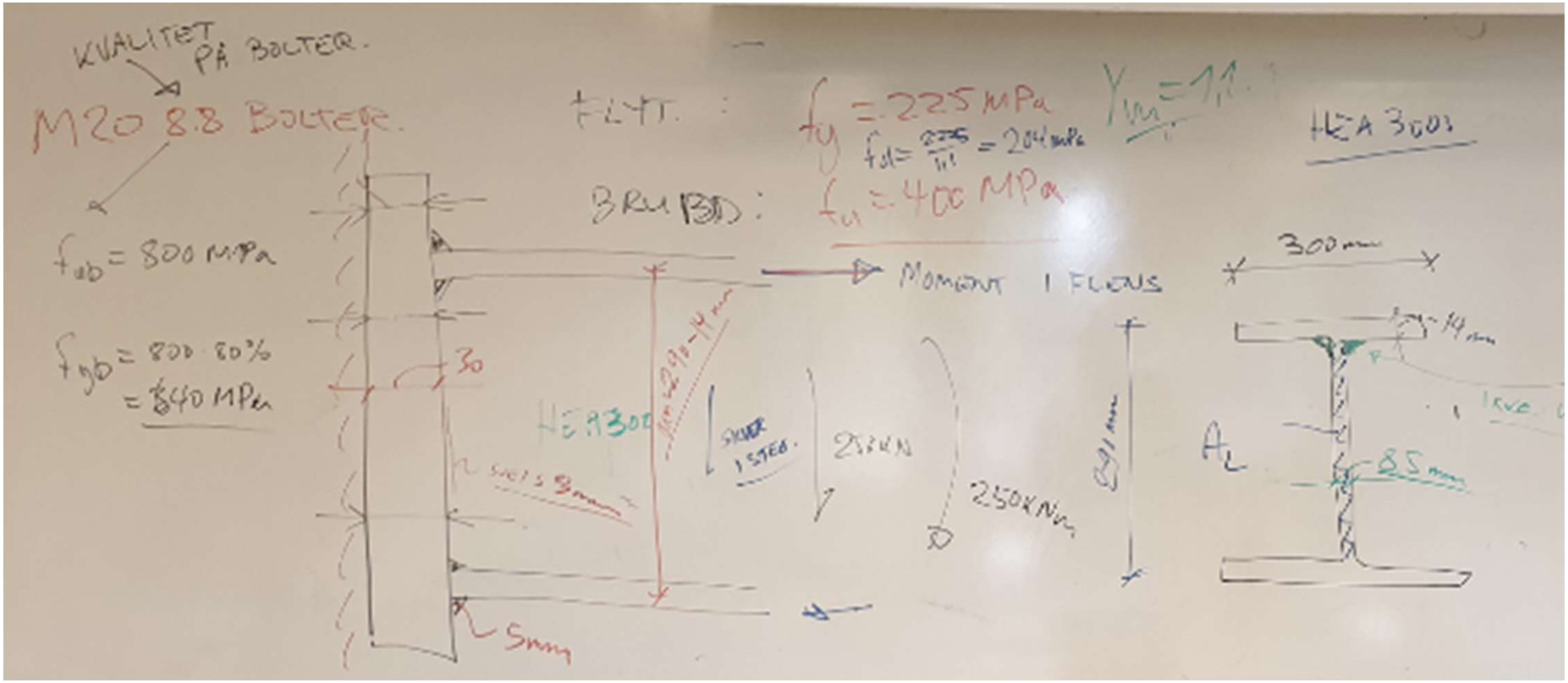
$$f_{sd} = \frac{355}{\sqrt{2} \cdot 1,1} = 228 \text{ N/mm}^2$$

$$A_{flens} = a_{nødv}\,(300 + 14 \cdot 2 + 300 - 27 \cdot 2 - 8,5)$$



$$a_{nødv} = \frac{250 \cdot 10^6}{228 \cdot 565,5 \cdot 276} = \underline{7,0 \text{ mm}}$$

Nødvendig sveis i flens $\underline{\underline{a = 7,0 \text{ mm}}}$

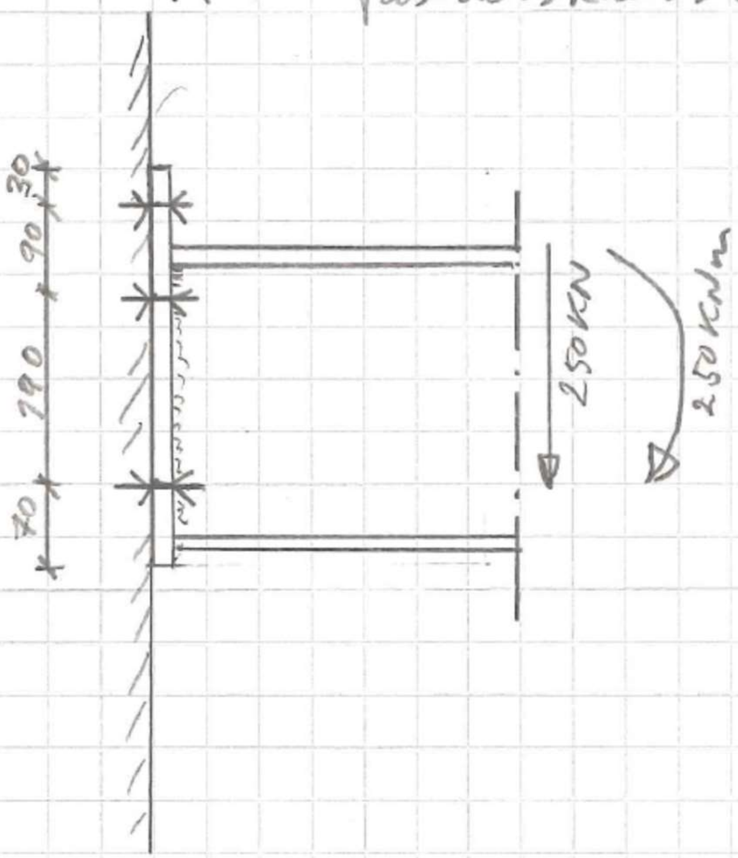


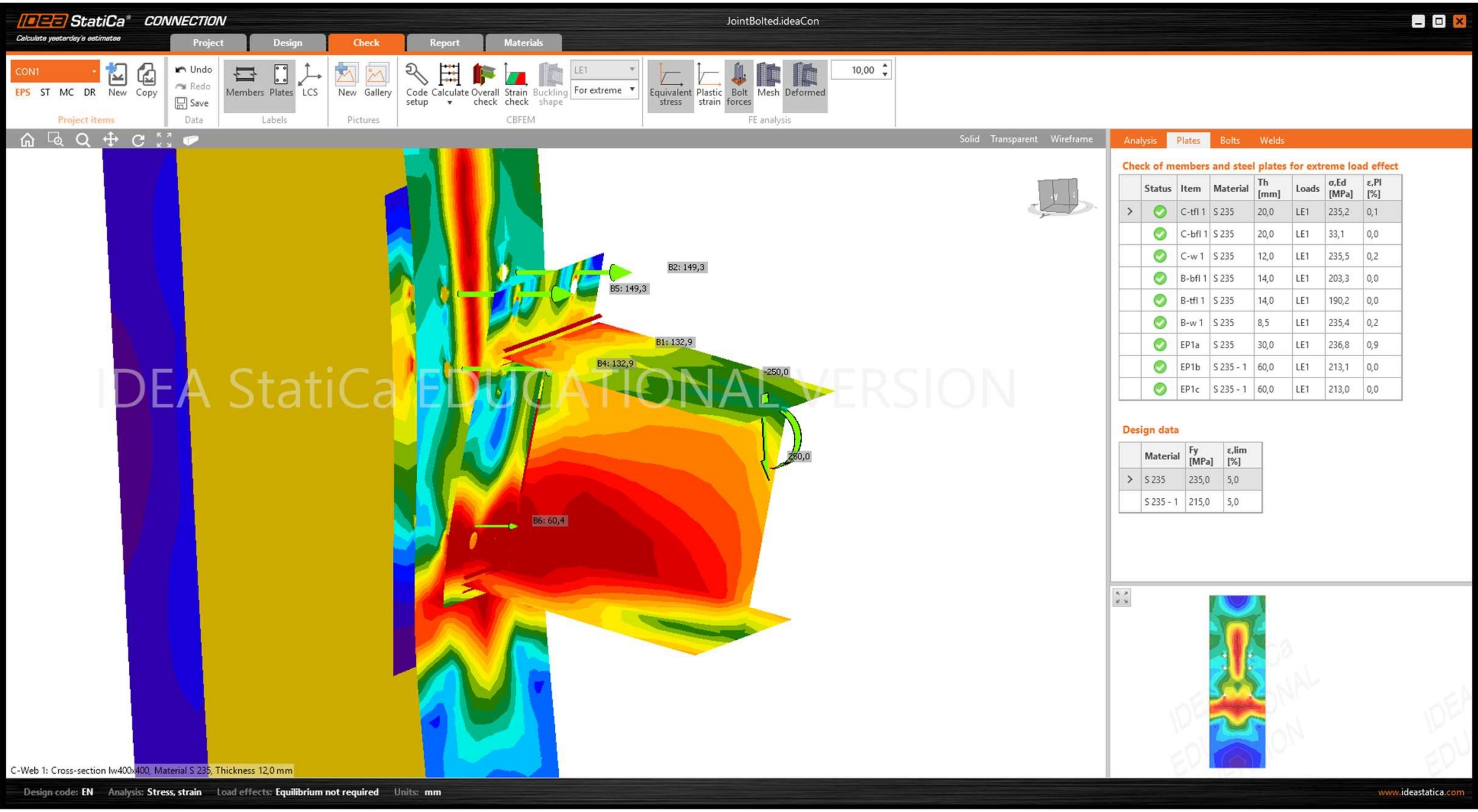
KVALITET PÅ BOLTER.
M20 8.8 BOLTER.
fub = 800 MPa
fyb = 800 · 80%
= 640 MPa
30
FLYT. :
fy = 225 MPa
γM = 1,1
BRUDD :
fu = 400 MPa
HEA 300
MOMENT I FLENS
300 mm
14 mm
HEA300
250 kNm
8,5 mm
5mm

Kontroll bolter:

M20 fasthetsklasse 8.8. $\Rightarrow$ $f_u = 800\,N/mm^2$

$f_y = 640\,N/mm^2$

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Project
Design
Check
Report
Materials
CON1
EPS ST MC DR
New
Copy
Project items
Undo
Redo
Save
Data
Members
Plates
LCS
Labels
New
Gallery
Pictures
Code setup
Calculate
Overall check
Strain check
Buckling shape
LE1
For extreme
CBFEM
Equivalent stress
Plastic strain
Bolt forces
Mesh
Deformed
FE analysis
10,00
Solid
Transparent
Wireframe
B2: 149,3
B5: 149,3
B1: 132,9
B4: 132,9
-250,0
280,0
B6: 60,4
IDEA StatiCa EDUCATIONAL VERSION
C-Web 1: Cross-section Iw400x400, Material S 235, Thickness 12,0 mm
Analysis
Plates
Bolts
Welds
Check of members and steel plates for extreme load effect
Status Item Material Th [mm] Loads σ,Ed [MPa] ε,Pl [%]
C-tfl 1 S 235 20,0 LE1 235,2 0,1
C-bfl 1 S 235 20,0 LE1 33,1 0,0
C-w 1 S 235 12,0 LE1 235,5 0,2
B-bfl 1 S 235 14,0 LE1 203,3 0,0
B-tfl 1 S 235 14,0 LE1 190,2 0,0
B-w 1 S 235 8,5 LE1 235,4 0,2
EP1a S 235 30,0 LE1 236,8 0,9
EP1b S 235 - 1 60,0 LE1 213,1 0,0
EP1c S 235 - 1 60,0 LE1 213,0 0,0
Design data
Material Fy [MPa] ε,lim [%]
S 235 235,0 5,0
S 235 - 1 215,0 5,0
Design code: EN
Analysis: Stress, strain
Load effects: Equilibrium not required
Units: mm
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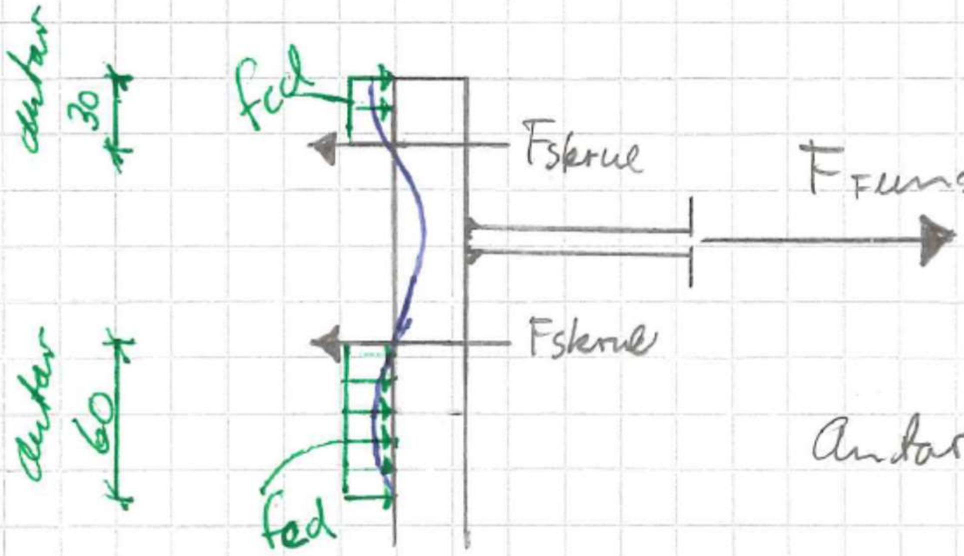


$$F_{Flens} = \frac{250\ kNm}{(290-14)\,mm} = 905{,}8\,kN$$

Antar $f_{cd} = 16\ N/mm^2$ (Vanlig betong)

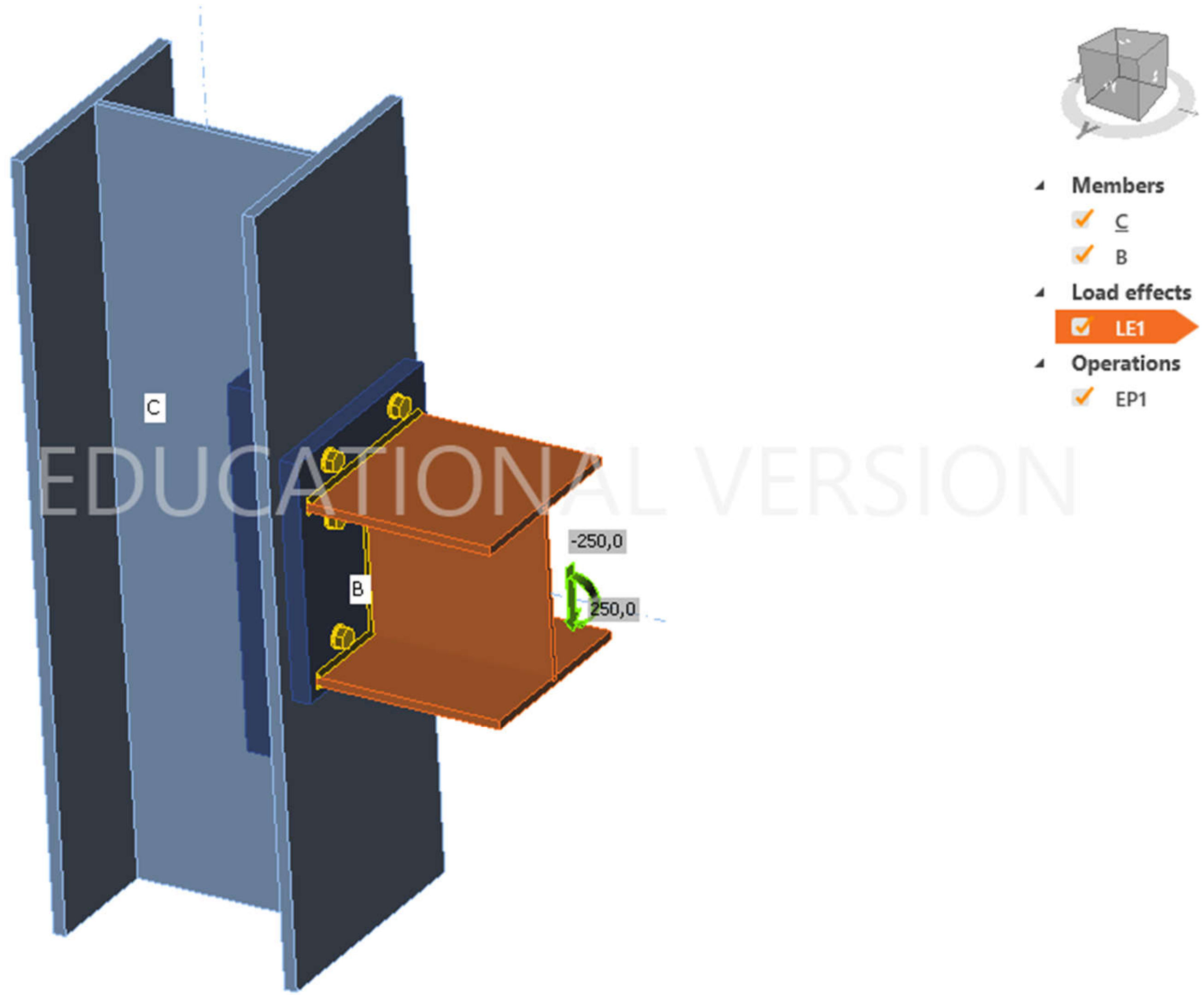
$$2\,F_{skruer} = F_{Flens} + f_{cd}\,(60+30)\,320$$

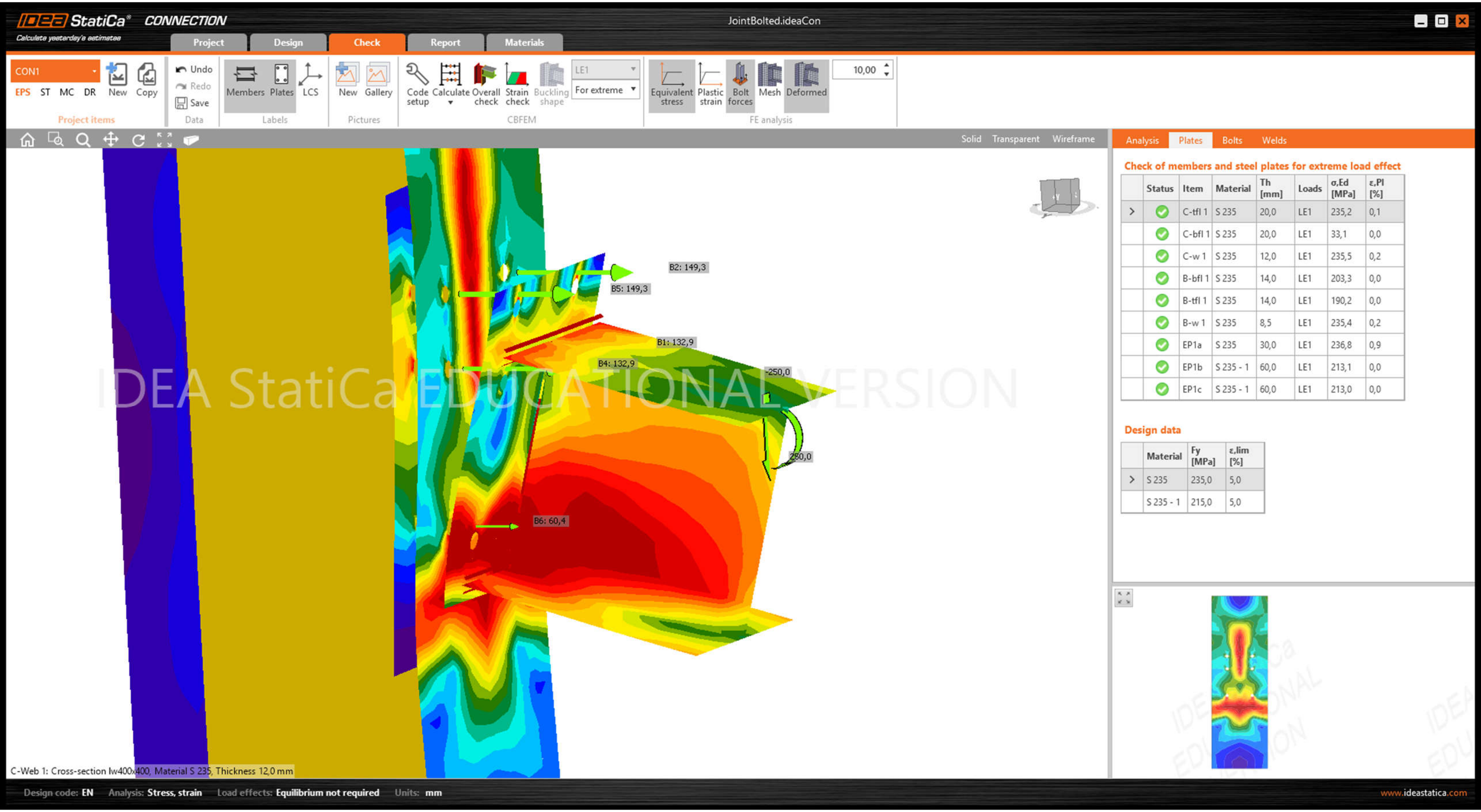
$$F_{skruer} = \left(905{,}8\ kN + (16\ N/mm^2 \cdot 90 \cdot 320)\cdot 10^{-3}\right)\cdot\frac{1}{2}$$

$$= \underline{683{,}3\ kN}$$

Eurokode 3: Prosjektering av stålkonstruksjoner
Del 1-8: Knutepunkter og forbindelser

Eurocode 3: Design of steel structures
Part 1-8: Design of joints


C
B
-250,0
250,0
Members
C
B
Load effects
LE1
Operations
EP1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Check
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Materials
CON1
EPS ST MC DR
New Copy
Project items
Undo
Redo
Save
Data
Members Plates LCS
Labels
New Gallery
Pictures
Code setup
Calculate
Overall check
Strain check
Buckling shape
LE1
For extreme
CBFEM
Equivalent stress
Plastic strain
Bolt forces
Mesh
Deformed
10,00
FE analysis
Solid Transparent Wireframe
B2: 149,3
B5: 149,3
B1: 132,9
B4: 132,9
-250,0
250,0
B6: 60,4
IDEA StatiCa EDUCATIONAL VERSION
C-Web 1: Cross-section Iw400x400, Material S 235, Thickness 12,0 mm
Analysis Plates Bolts Welds
Check of members and steel plates for extreme load effect
Status | Item | Material | Th [mm] | Loads | σ,Ed [MPa] | ε,Pl [%]
C-tfl 1 | S 235 | 20,0 | LE1 | 235,2 | 0,1
C-bfl 1 | S 235 | 20,0 | LE1 | 33,1 | 0,0
C-w 1 | S 235 | 12,0 | LE1 | 235,5 | 0,2
B-bfl 1 | S 235 | 14,0 | LE1 | 203,3 | 0,0
B-tfl 1 | S 235 | 14,0 | LE1 | 190,2 | 0,0
B-w 1 | S 235 | 8,5 | LE1 | 235,4 | 0,2
EP1a | S 235 | 30,0 | LE1 | 236,8 | 0,9
EP1b | S 235 - 1 | 60,0 | LE1 | 213,1 | 0,0
EP1c | S 235 - 1 | 60,0 | LE1 | 213,0 | 0,0
Design data
Material | Fy [MPa] | ε,lim [%]
S 235 | 235,0 | 5,0
S 235 - 1 | 215,0 | 5,0
Design code: EN Analysis: Stress, strain Load effects: Equilibrium not required Units: mm
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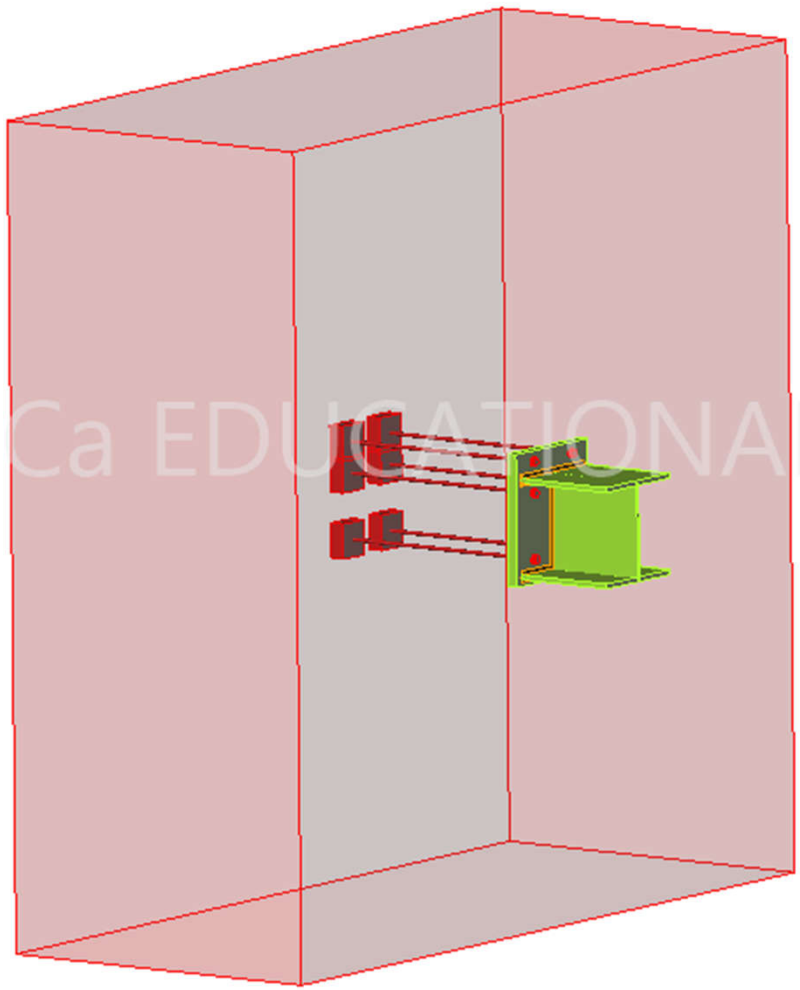
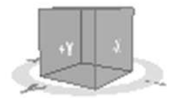
Analysis	✓	100,0%
Plates	✓	0,4 < 5%
Anchors	✗	2098,0 > 100%
Welds	✓	98,5 < 100%
Concrete block	✗	119,9 > 100%
Buckling		Not calculated

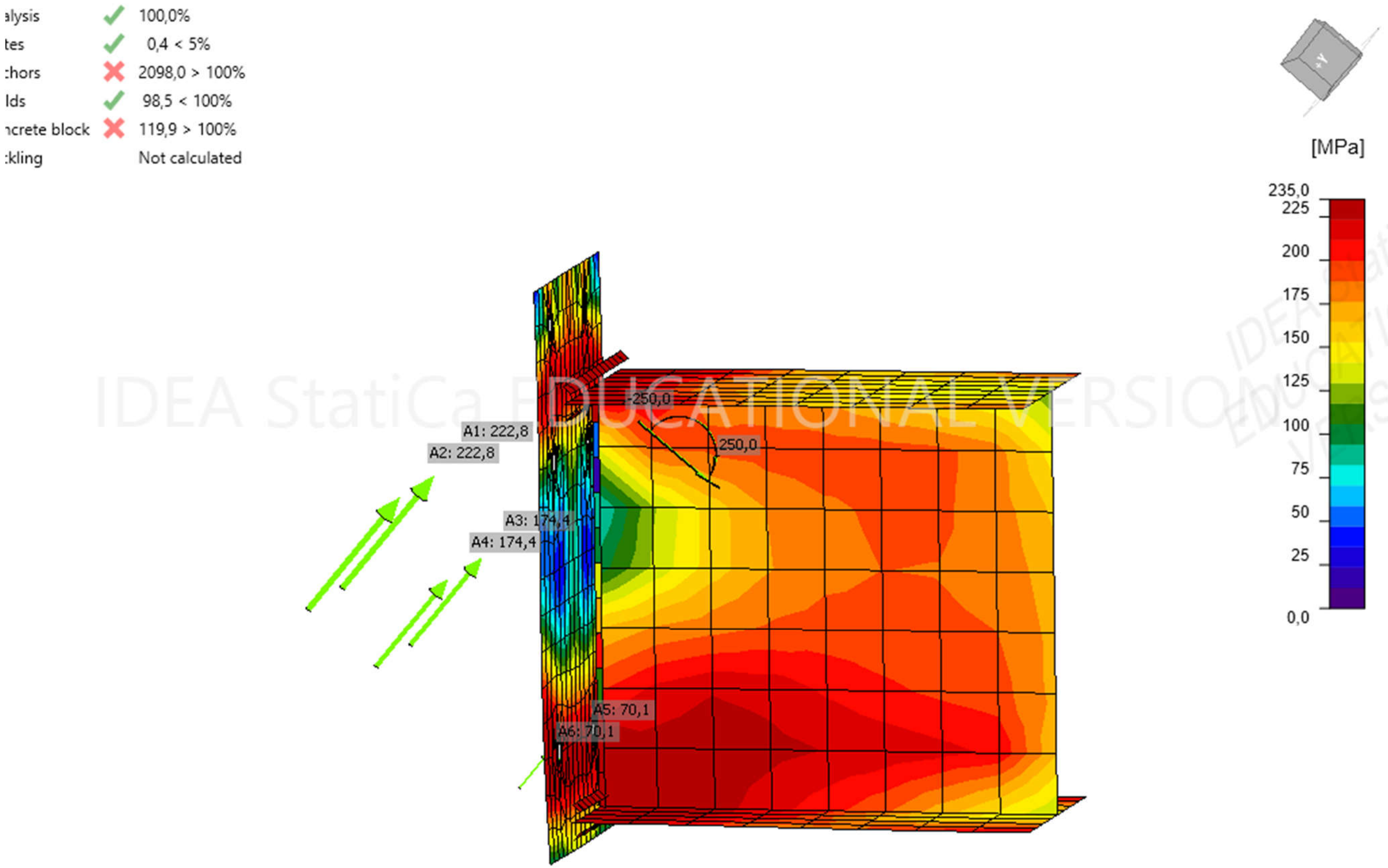




Members
- COL

Load effects
- LE1

Operations
- BP1



alysis 100,0%
tes 0,4 < 5%
chors 2098,0 > 100%
lds 98,5 < 100%
ncrete block 119,9 > 100%
kling Not calculated
[MPa]
235,0
225
200
175
150
125
100
75
50
25
0,0
A1: 222,8
A2: 222,8
A3: 174,4
A4: 174,4
A5: 70,1
A6: 70,1
-250,0
250,0
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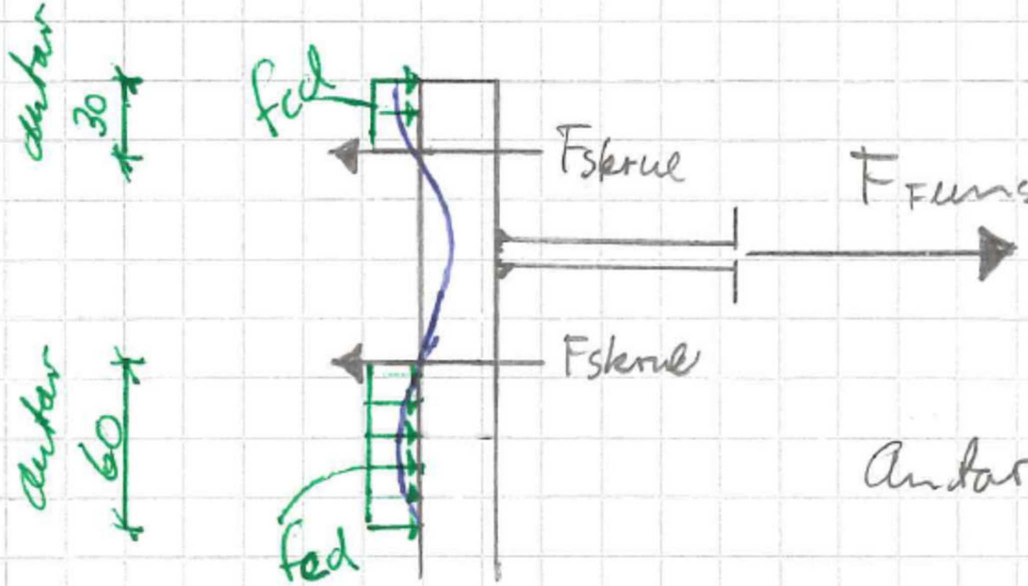


$$F_{Flens} = \frac{250\ KNm}{(290-14)mm} = 905{,}8\,KN$$

antar $f_{cd} = 16\ N/mm^2$ (Vanlig betong)

$$2\,F_{skruer} = F_{Flens} + f_{cd}\,(60+30)\,320$$

$$F_{skruer} = \left(905{,}8\ KN + (16\ N/mm^2 \cdot 90 \cdot 320) \cdot 10^{-3}\right)\frac{1}{2}$$

$$= \underline{683{,}3\ KN}$$

Strekk kraft pr. skrue:

$$\underline{F_{skrue}} = 683{,}3/2 = \underline{341{,}7\,KN} = F_s$$

Skjærkraft pr. skrue:

$$\underline{F_{skjær}} = \frac{250\,KN}{6} = \underline{41{,}7\,KN} = F_a$$

Strekkkapasitet pr. bolt:

$$\gamma_{M2} := 1.1 \qquad k_2 := 0.9 \qquad f_{ub} := 800 \cdot MPa \qquad A_s := 245 \cdot mm^2$$

$$F_{t.Rd} := \frac{k_2 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} \qquad F_{t.Rd} = 160.364\,kN$$

Kapasitet må økes.

Kombinert avskjæring og strekk	$\dfrac{F_{v,Ed}}{F_{v,Rd}} + \dfrac{F_{t,Ed}}{1{,}4\,F_{t,Rd}} \leq 1{,}0$

$$F_{t.Ed} := 341.7 \cdot kN$$

$$F_{v.Ed} := 41.7kN$$

$$k_2 := 0.9$$

$$f_{ub} := 1000 \cdot MPa$$

$$A_s := 459 \cdot mm^2$$

$$\gamma_{M2} := 1.1$$

$$F_{t.Rd} := \frac{k_2 \cdot f_{ub} \cdot A_s}{\gamma_{M2}}$$

$$F_{t.Rd} = 375.545\,kN$$

$$\alpha_v := 0.6$$

$$A_{skrue} := 353 \cdot mm^2$$

$$F_{v.Rd} := \frac{\left(\alpha_v \cdot f_{ub} \cdot A_{skrue}\right)}{\gamma_{M2}}$$

$$F_{v.Rd} = 192.545\,kN$$

$$UF := \left(\frac{F_{v.Ed}}{F_{v.Rd}}\right) + \left(\frac{F_{t.Ed}}{1.4F_{t.Rd}}\right)$$

$$UF = 0.866$$

$$UF := \left(\frac{F_{v.Ed}}{F_{v.Rd}}\right) + \left(\frac{F_{t.Ed}}{1.4F_{t.Rd}}\right) \qquad UF = 0.816$$

Punching

Gjennomlokking	$B_{p,Rd} \quad = \quad 0{,}6\, \pi\, d_m\, t_p\, f_u \,/\, \gamma_{M2}$

d_m er middelverdien av avstanden mellom hjørner (e) og avstanden mellom de rette sideflatene (nøkkelvidden s) av skruehodet eller mutteren, der den minste av de to middelverdiene (for henholdsvis skruehode og mutter) benyttes;

t_p er underlagsplatens tykkelse (under skruen eller mutteren);

$B_{p,Rd}$ er skruehodets eller mutterens dimensjonerende kapasitet mot gjennomlokking;

$t := 30 \cdot mm$

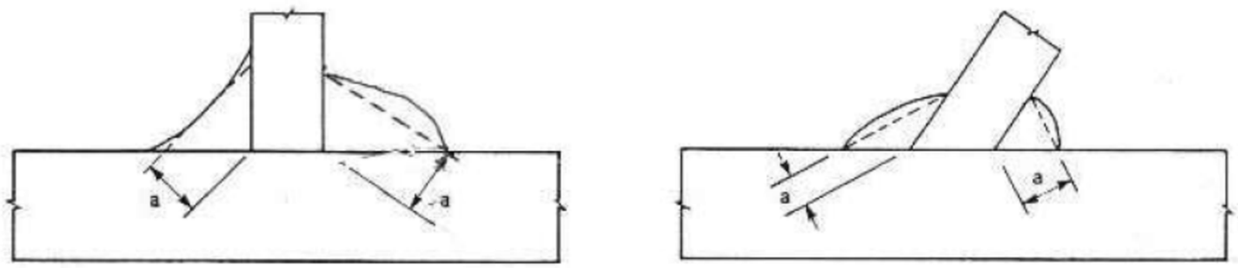
$f_u := 400 \cdot MPa$

$$d_m := \frac{F_{t.Rd} \cdot \gamma_{M2}}{0.6 \cdot \pi \cdot t \cdot f_u}$$

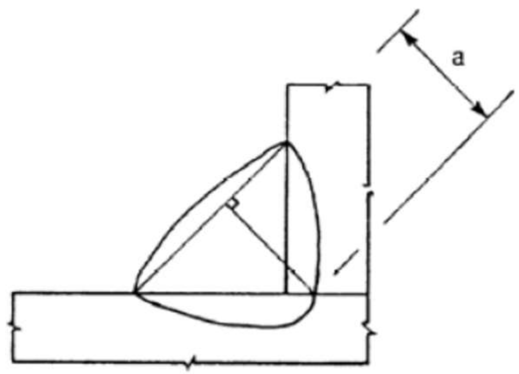
$d_m = 18.263\, mm$

Eurokode 3: Prosjektering av stålkonstruksjoner
Del 1-8: Knutepunkter og forbindelser

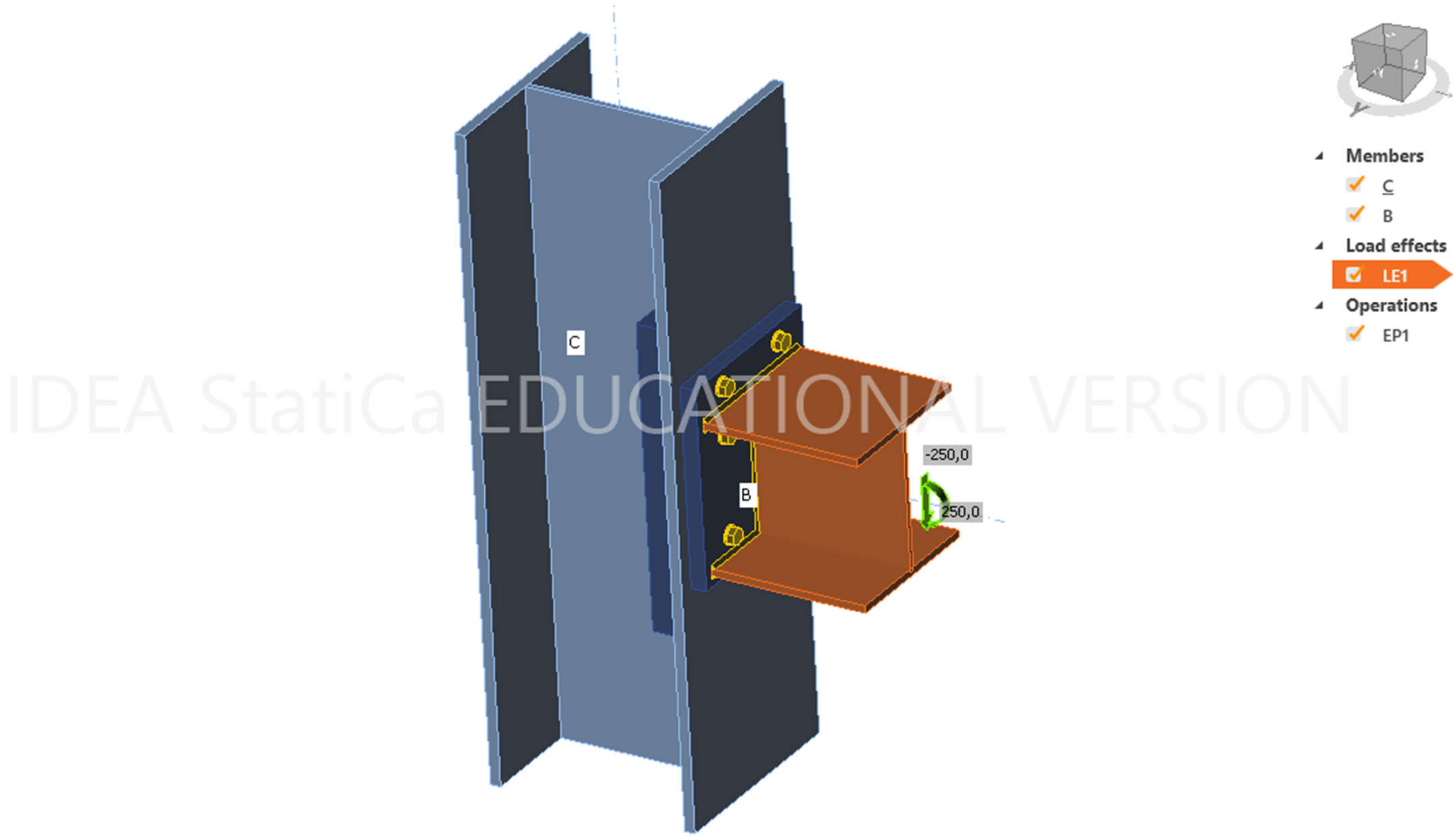
Eurocode 3: Design of steel structures
Part 1-8: Design of joints



Figur 4.3 – Effektivt halsmål a for en kilsveis



Figur 4.4 – Effektivt halsmål a for en kilsveis med dyp innsmelting



C
B
-250,0
250,0
Members
C
B
Load effects
LE1
Operations
EP1
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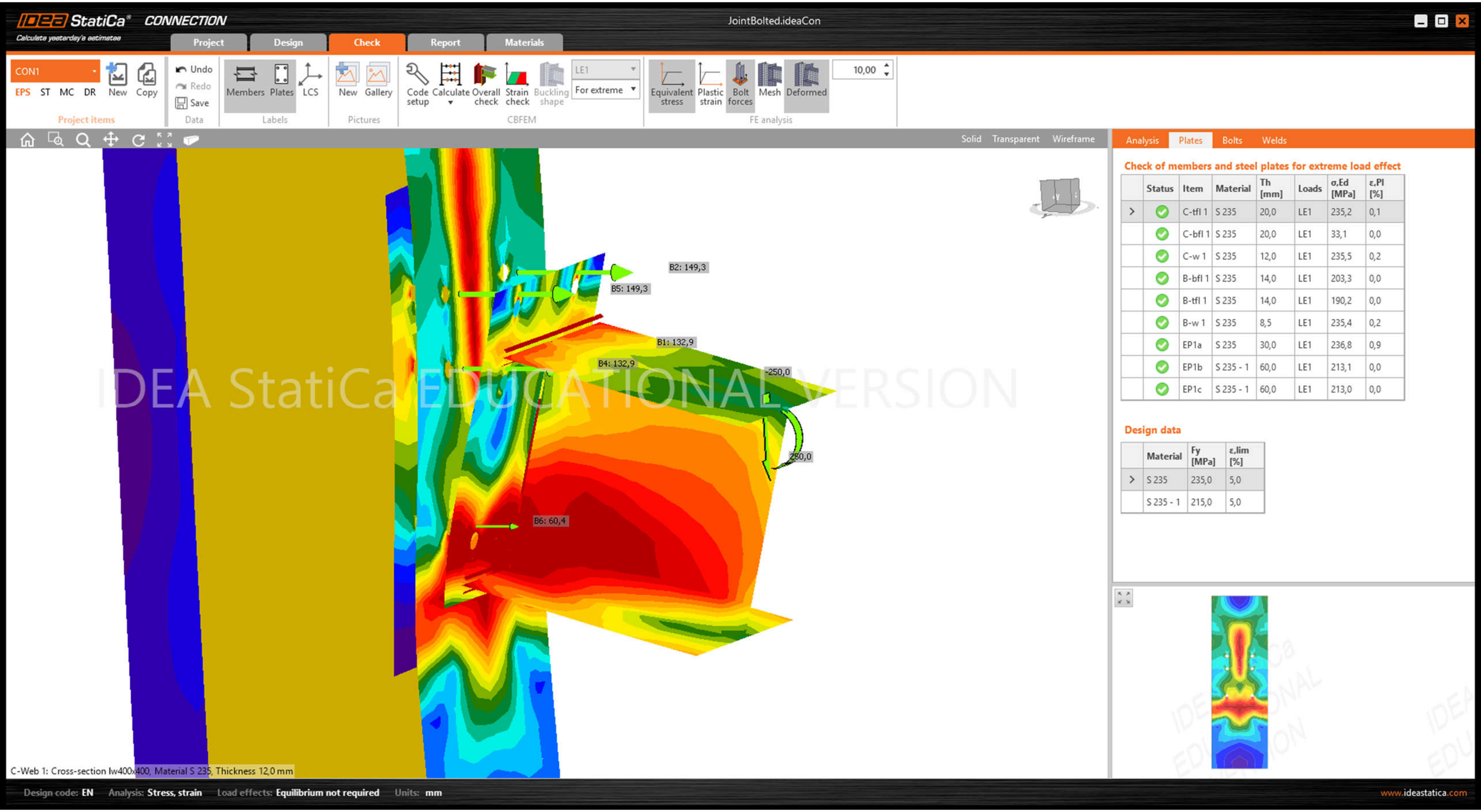
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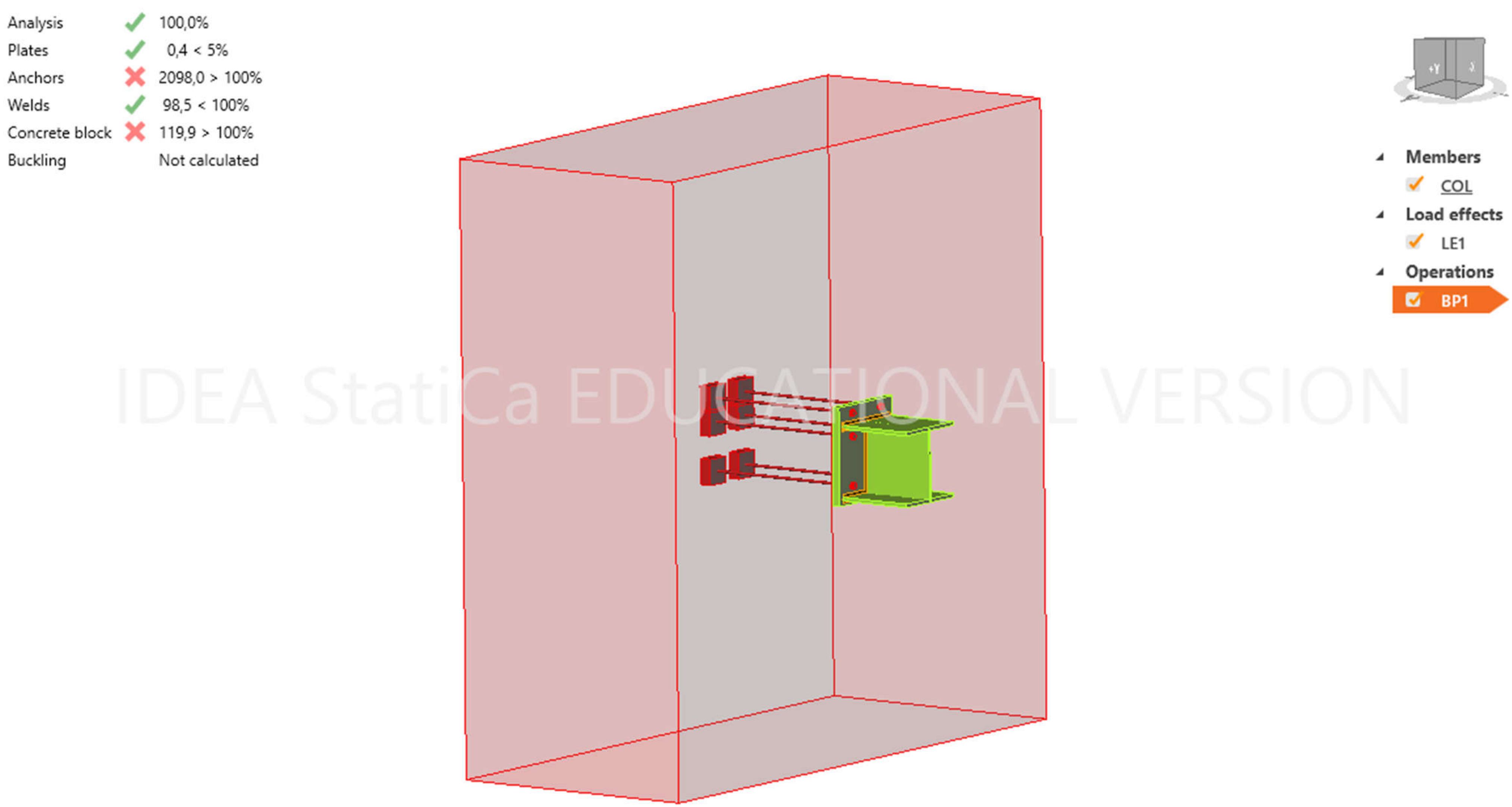
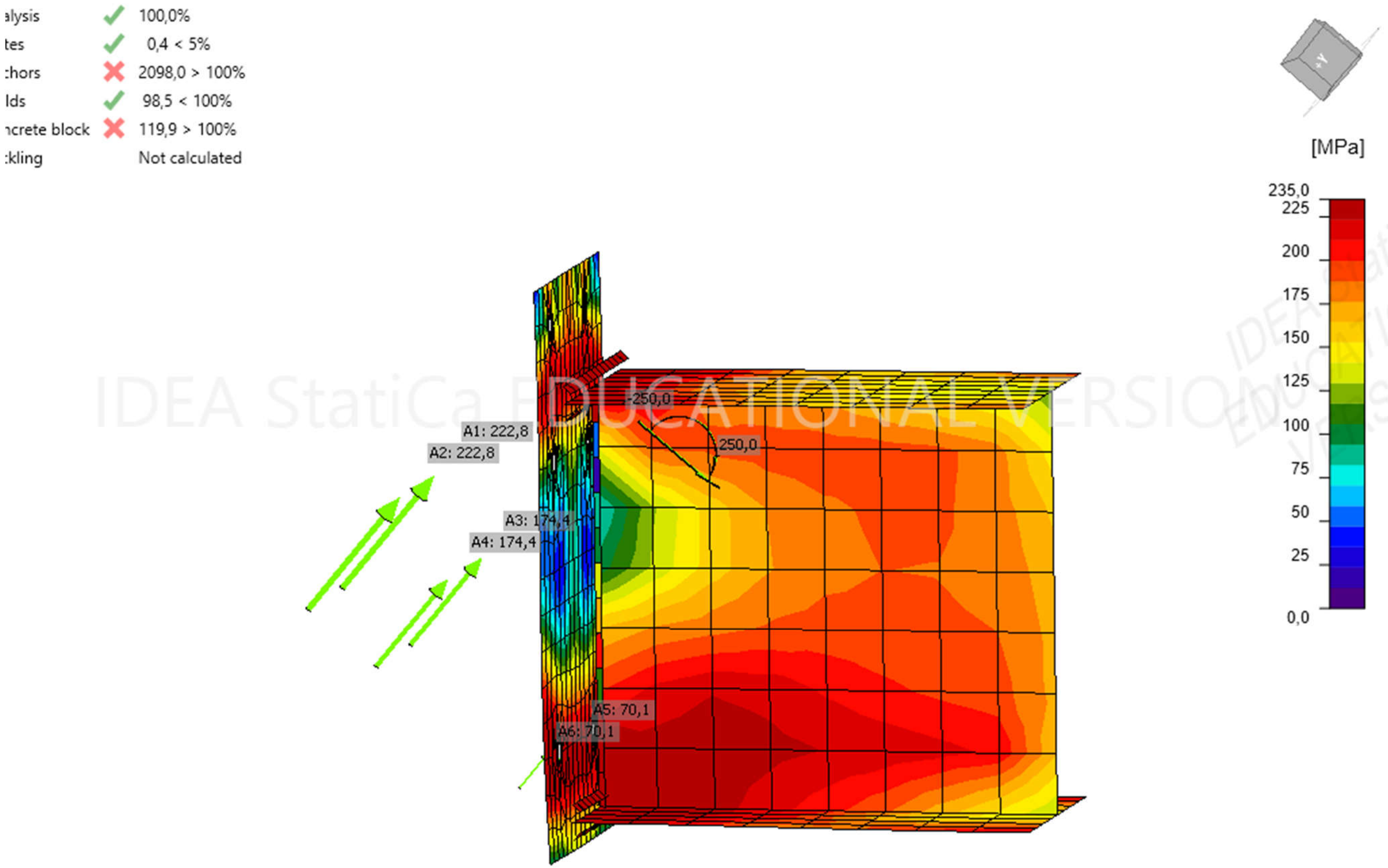
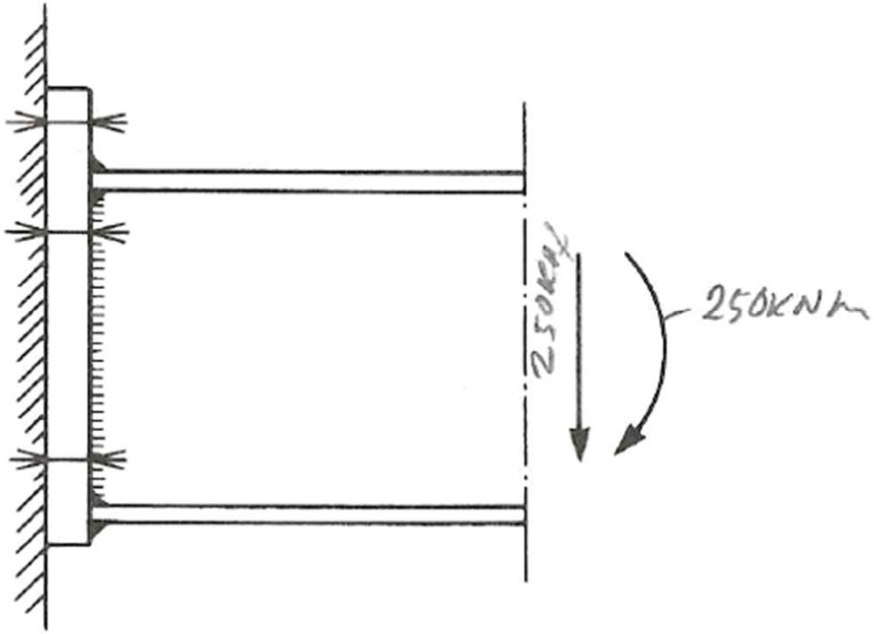
Design
Check
Report
Materials
CON1
EPS ST MC DR
New
Copy
Project items
Undo
Redo
Save
Data
Members
Plates
LCS
Labels
New
Gallery
Pictures
Code setup
Calculate
Overall check
Strain check
Buckling shape
LE1
For extreme
CBFEM
Equivalent stress
Plastic strain
Bolt forces
Mesh
Deformed
FE analysis
10,00
Solid
Transparent
Wireframe
B2: 149,3
B5: 149,3
B1: 132,9
B4: 132,9
-250,0
250,0
B6: 60,4
IDEA StatiCa EDUCATIONAL VERSION
C-Web 1: Cross-section Iw400x400, Material S 235, Thickness 12,0 mm
Analysis
Plates
Bolts
Welds
Check of members and steel plates for extreme load effect
Status Item Material Th [mm] Loads σ,Ed [MPa] ε,Pl [%]
C-tfl 1 S 235 20,0 LE1 235,2 0,1
C-bfl 1 S 235 20,0 LE1 33,1 0,0
C-w 1 S 235 12,0 LE1 235,5 0,2
B-bfl 1 S 235 14,0 LE1 203,3 0,0
B-tfl 1 S 235 14,0 LE1 190,2 0,0
B-w 1 S 235 8,5 LE1 235,4 0,2
EP1a S 235 30,0 LE1 236,8 0,9
EP1b S 235 - 1 60,0 LE1 213,1 0,0
EP1c S 235 - 1 60,0 LE1 213,0 0,0
Design data
Material Fy [MPa] ε,lim [%]
S 235 235,0 5,0
S 235 - 1 215,0 5,0
Design code: EN
Analysis: Stress, strain
Load effects: Equilibrium not required
Units: mm
www.ideastatica.com


Analysis 100,0%
Plates 0,4 < 5%
Anchors 2098,0 > 100%
Welds 98,5 < 100%
Concrete block 119,9 > 100%
Buckling Not calculated
IDEA StatiCa EDUCATIONAL VERSION
Members
COL
Load effects
LE1
Operations
BP1



alysis 100,0%
tes 0,4 < 5%
:hors 2098,0 > 100%
lds 98,5 < 100%
ncrete block 119,9 > 100%
:kling Not calculated
[MPa]
235,0
225
200
175
150
125
100
75
50
25
0,0
A1: 222,8
A2: 222,8
A3: 174,4
A4: 174,4
A5: 70,1
A6: 70,1
-250,0
250,0
IDEA StatiCa EDUCATIONAL VERSION



Kraft i flens fra moment (da flense.wn tar momentet):

$$F_{Ed.Flens} := \frac{M_{Rd}}{\left(H - t_{flens}\right)}$$

$$F_{Ed.Flens} = 905.797 \cdot kN$$

Tabell 4.1 – Korrelasjonsfaktor β_w for kilsveis

Standard og stålsort			Korrelasjonsfaktor β_w
NS-EN 10025	NS-EN 10210	NS-EN 10219	
S 235 S 235 W	S 235 H	S 235 H	0,8
S 275 S 275 N/NL S 275 M/ML	S 275 H S 275 NH/NLH	S 275 H S 275 NH/NLH S 275 MH/MLH	0,85
S 355 S 355 N/NL S 355 M/ML S 355 W	S 355 H S 355 NH/NLH	S 355 H S 355 NH/NLH S 355 MH/MLH	0,9
S 420 N/NL S 420 M/ML		S 420 MH/MLH	1,0
S 460 N/NL S 460 M/ML S 460 Q/QL/QL1	S 460 NH/NLH	S 460 NH/NLH S 460 MH/MLH	1,0

Spenninger på flenssveis:

$$n_{vinkelrettpaa} := \frac{F_{Ed.Flens}}{2 \cdot B \cdot a_{maal}}$$

$$n_{vinkelrettpaa} = 251.61 \cdot MPa$$

$$\sigma_{vinkelrettpaa} := \frac{n_{vinkelrettpaa}}{\sqrt{2}}$$

$$\sigma_{vinkelrettpaa} = 177.915 \cdot MPa$$

$$\tau_{vinkelrettpaa} := \frac{n_{vinkelrettpaa}}{\sqrt{2}}$$

$$\tau_{vinkelrettpaa} = 177.915 \cdot MPa$$

(5) Ved fastsettelse sveisens kapasitet ses det bort fra normalspenningen $\sigma_\parallel$ parallelt med aksen.

(6) Dimensjonerende kapasitet for en kilsveis er tilfredsstillende hvis begge følgende betingelser er oppfylt:

$$[\sigma_\perp^2 + 3\,(\tau_\perp^2 + \tau_\parallel^2)]^{0,5} \leq f_u \,/\, (\beta_w\, \gamma_{M2}), \text{ og } \sigma_\perp \leq 0{,}9\, f_u \,/\, \gamma_{M2} \tag{4.1}$$

der

f_u er nominell strekkfasthet i den svakeste delen i forbindelsen;

β_w er en korrelasjonsfaktor fra tabell 4.1.

(7) Sveiser mellom deler med ulike stålsorter bør dimensjoneres på grunnlag av egenskapene til godset med den laveste fastheten.

Forenklet metode:

$$\sigma_{sveis} := \sqrt{\sigma_{vinkelrettpaa}^{2} + 3\tau_{vinkelrettpaa}^{2}}$$

$$\sigma_{sveis} = 355.831 \cdot \mathrm{MPa}$$

$$\sigma_{Rd} := \frac{f_u}{\beta_w \cdot \gamma_{m2}} \qquad \beta_w = 0.8$$

$$\sigma_{Rd} = 400 \cdot \mathrm{MPa}$$

$$\sigma_{normaltpaa.Rd} := \frac{0.9 \cdot f_u}{\gamma_{m2}}$$

$$\sigma_{normaltpaa.Rd} = 288 \cdot \mathrm{MPa}$$

$$\sigma_{vinkelrettpaa} = 177.915 \cdot \mathrm{MPa}$$

Retningsavhengig utnyttelse (kan utnytte denne sveisen mer):

Øket a-maal:

$$a_{maal} := 5 \cdot mm$$

Spenninger på flenssveis:

$$n_{vinkelrettpaa} := \frac{F_{Ed.Flens}}{2 \cdot B \cdot a_{maal}}$$

$$n_{vinkelrettpaa} = 301.932 \cdot MPa$$

$$\sigma_{vinkelrettpaa} := \frac{n_{vinkelrettpaa}}{\sqrt{2}}$$

$$\sigma_{vinkelrettpaa} = 213.498 \cdot MPa$$

$$\tau_{vinkelrettpaa} := \frac{n_{vinkelrettpaa}}{\sqrt{2}}$$

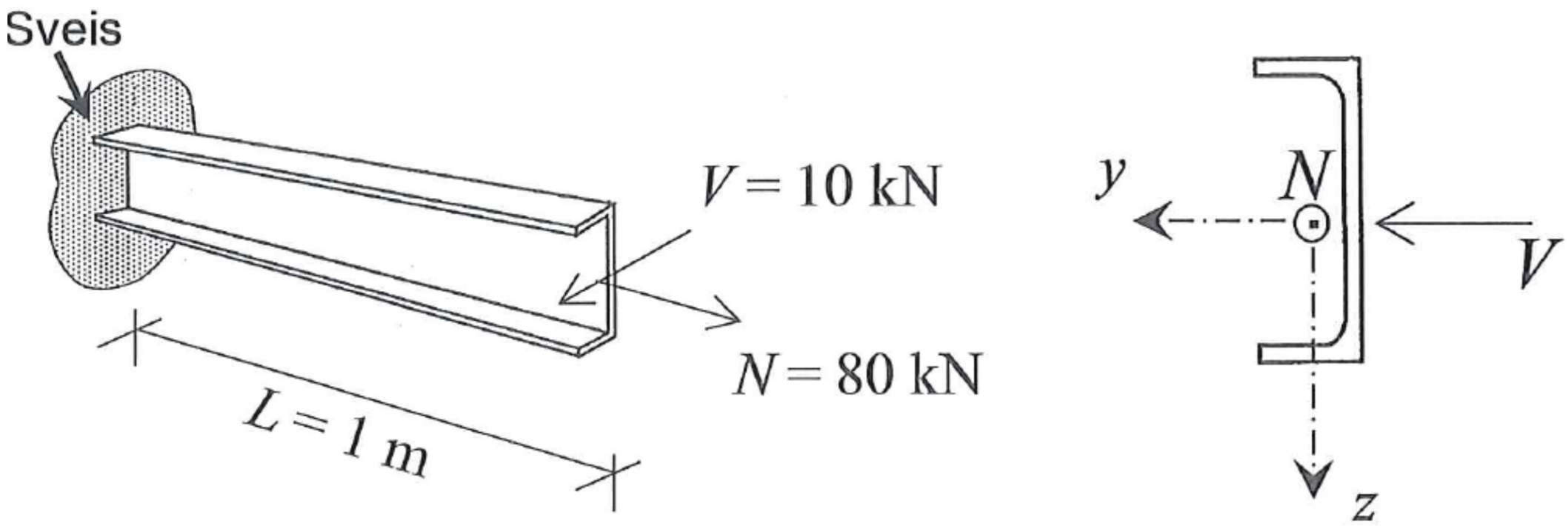
$$\tau_{vinkelrettpaa} = 213.498 \cdot MPa$$

$$\sigma_{sveis} := \sqrt{\sigma_{vinkelrettpaa}^{2} + 2\tau_{vinkelrettpaa}^{2}}$$

$$\sigma_{sveis} = 369.79 \cdot MPa$$

Cantilevered beam welded to stiff wall

A cantilevered beam with USP200 profile is fixed to a stiff wall by a continuous weld along the entire cross section. The actions act through the centre of area of the cross section. The steel is S355.

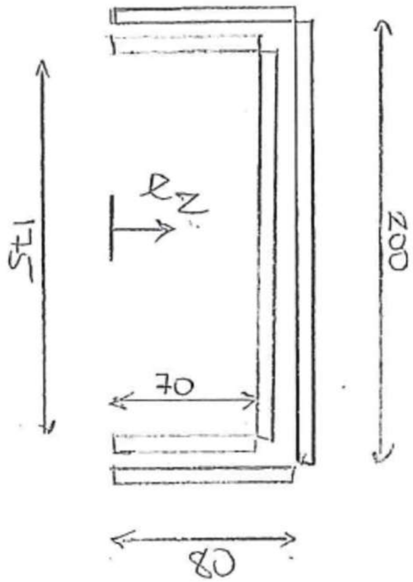


a) Necessary throat thickness?

The action effects at the fixed end are:

$$N = 80\,kN$$
$$V_y = 10\,kN$$
$$M_z = 10\,kNm$$

The design throat area is approximately:



Total area:

$$A = ((70 + 80) \cdot 2 + 175 + 200) \cdot a = \underline{675\, a}$$

Centre of area:

$$y_{NA} = \frac{(70 \cdot 35 + \cdot 80 \cdot 40) \cdot 2 + 175 \cdot 70 + 200 \cdot 80}{675} = \underline{59\, mm}$$

Second moment of area:

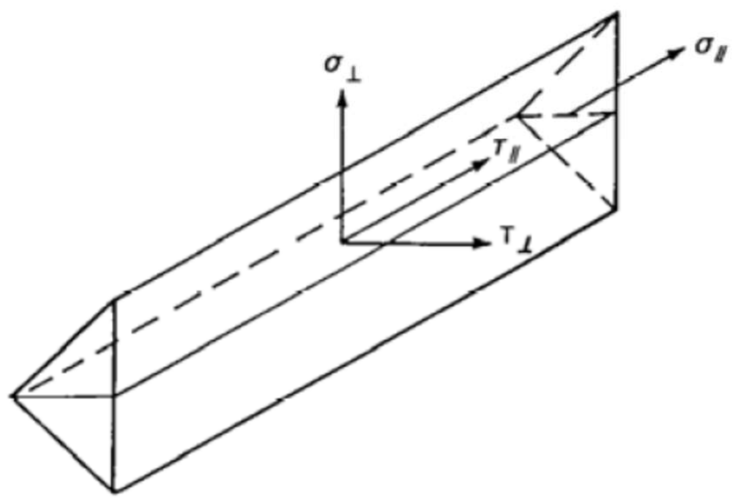
$$\begin{aligned} I_z = &\left(\frac{a \cdot 70^3}{12} + a \cdot 70 \cdot (59 - 35)^2\right) \cdot 2 \\ &+ \left(\frac{a \cdot 80^3}{12} + a \cdot 80 \cdot (59 - 40)^2\right) \cdot 2 \\ &+ 175 \cdot a \cdot (70 - 59)^2 + 200 \cdot a \cdot (80 - 59)^2 = \underline{390 \cdot 10^3\, a} \end{aligned}$$

The stresses acting on the design throat section are:

$$n_{\perp}^{N} = \frac{N}{A} = \frac{80 \cdot 10^3}{675\,a} = \frac{119}{a}$$

$$\tau_{\parallel}^{V_y} = \frac{V_y}{A_v} = \frac{10 \cdot 10^3}{(70+80) \cdot 2a} = \frac{33.3}{a}$$

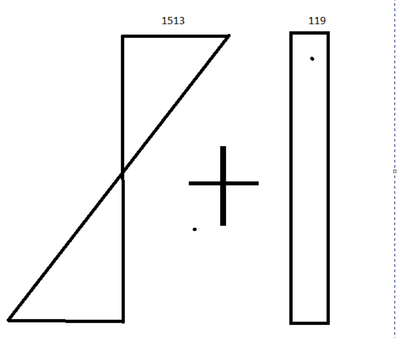
$$n_{\perp}^{M_z} = \frac{M_z}{I_z} \cdot y_{max} = \frac{10 \cdot 10^6}{390 \cdot 10^3\,a} \cdot 59 = \frac{1513}{a}$$



The stresses acting on the throat section are:

$$\tau_{\perp} = n_{\perp} = \frac{119 + 1513}{\sqrt{2}} = \frac{1154}{a}$$

$$\tau_{\parallel} = \frac{33.3}{a}$$



These stresses are inserted into the interaction formula:

$$\sqrt{\sigma_{\perp}^2 + 3(\tau_{\perp}^2 + \tau_{\parallel}^2)} \leq \frac{f_u}{\beta_w \gamma_{M2}}$$

$\tau_{\parallel}$ is small and may be neglected such that:

$$\sqrt{\left(\frac{1154}{a}\right)^2 + 3 \cdot \left(\frac{1154}{a}\right)^2} \leq \frac{510}{0.9 \cdot 1.3}$$

The equation may be rewritten as:

$$2 \cdot \frac{1154}{a} = 436$$

The throat thickness is then given by:

$$a = 2 \cdot \frac{1154}{436} = \underline{5.3\,mm} \ \rightarrow \text{choose} \ \boxed{a = 6\,mm}$$

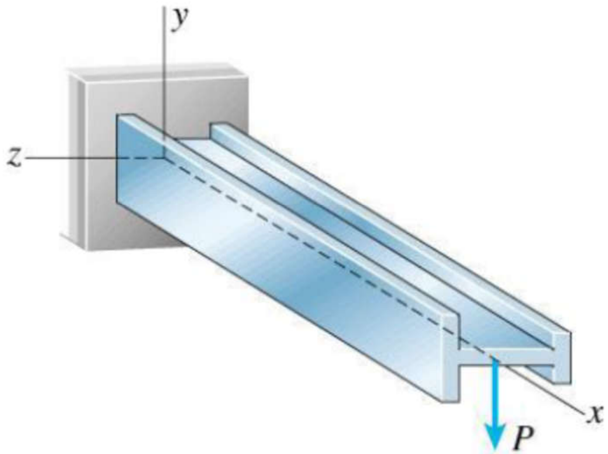
TBYG3018 Design of Offshore Structures

Module 4 – Design of Steel Structures according to NORSOK

Jomar Tørset, Assistant professor

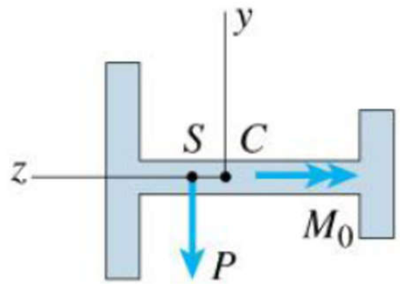
<u>Skjærsenter - Hovedpunkter</u>

Utkragerbjelken på figuren til høyre er utsatt for en punktlast P ytterst.



Det viser seg at **lasten P må angripe et spesielt punkt i tverrsnittet, nemlig punktet S, *skjærsenteret*, hvis bjelken skal bøye seg uten samtidig å *vris* (få torsjon) om lengdeaksen x.**

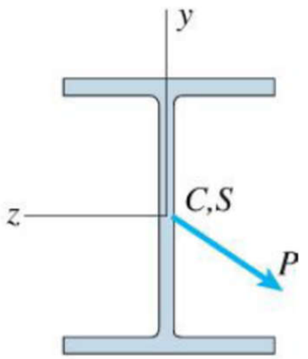
Angriper lasten P i punktet S, blir det altså kun skjærkraft (V = P) og vanlig *bøye*moment (kalt M_0 i figuren t.h.), *ikke* noe torsjonsmoment.



Hvordan finne hvor skjærsenteret er?

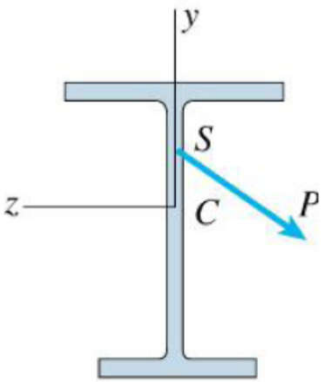
Dobbeltsymmetriske tverrsnitt

Skjærsenteret S må ligge i arealsenteret C.



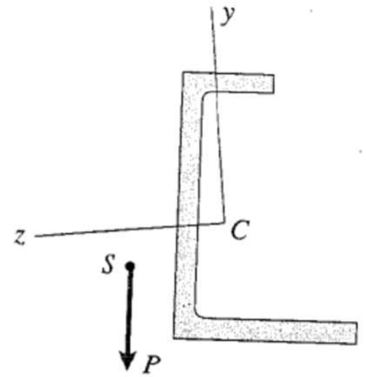
Enkeltsymmetriske tverrsnitt

Skjærsenteret må ligge på den ene symmetriaksen vi har, dvs. y-aksen i tilfellet under.

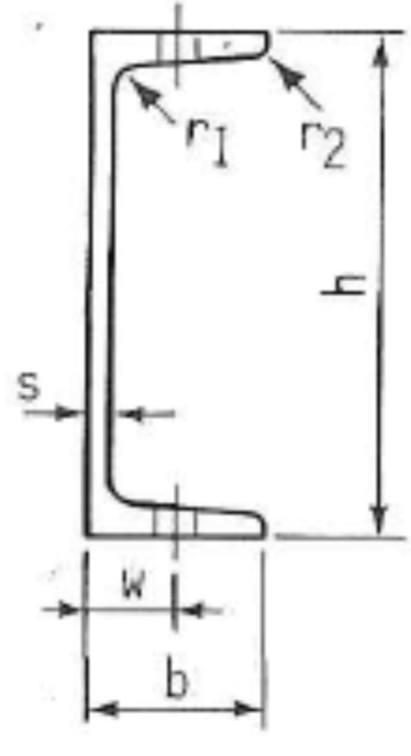


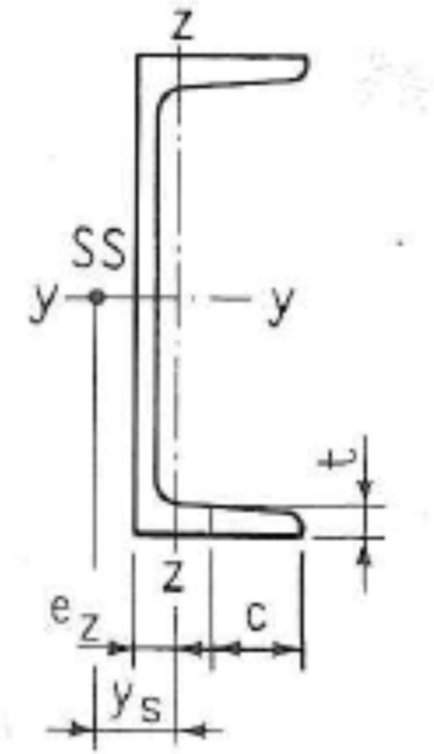
Vi kommer tilbake til hvor på symmetrisaksen S ligger.

Usymmetrisk tverrsnitt



Det er komplisert å finne skjærsenteret S for slike tverrsnitt, og





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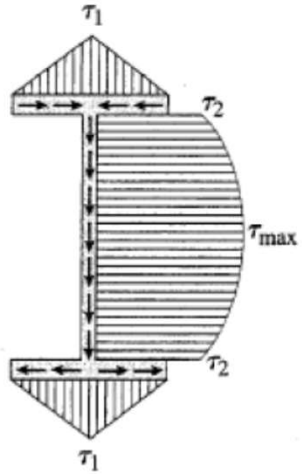
For **tynnveggede tverrsnitt** vil skjærspenningene vil ha samme retning som tverrsnittsdelene. Skjærsenteret S må være plassert slik at det totale torsjonsmomentet om S fra alle skjærspenningene blir lik null.

Vi har sett i eksempler at τ varierer *parabelformet* (altså som en 2.gradsfunksjon) over tverrsnittsdeler som har lengderetning parallelt med V. (Dette ser vi også av å sette opp uttrykk for S som funksjon av y.) τ vil altså variere parabelformet på steget i et typisk ståltverrsnitt (som på figuren til høyre) – og variere *lineært* på flensene.



Vi skal nå se mer på hvordan vi finner

skjærsenteret for tynnveggede, enkeltsymmetriske tverrsnitt (kap. 6.9 i Gere/Goodno).

Metode:

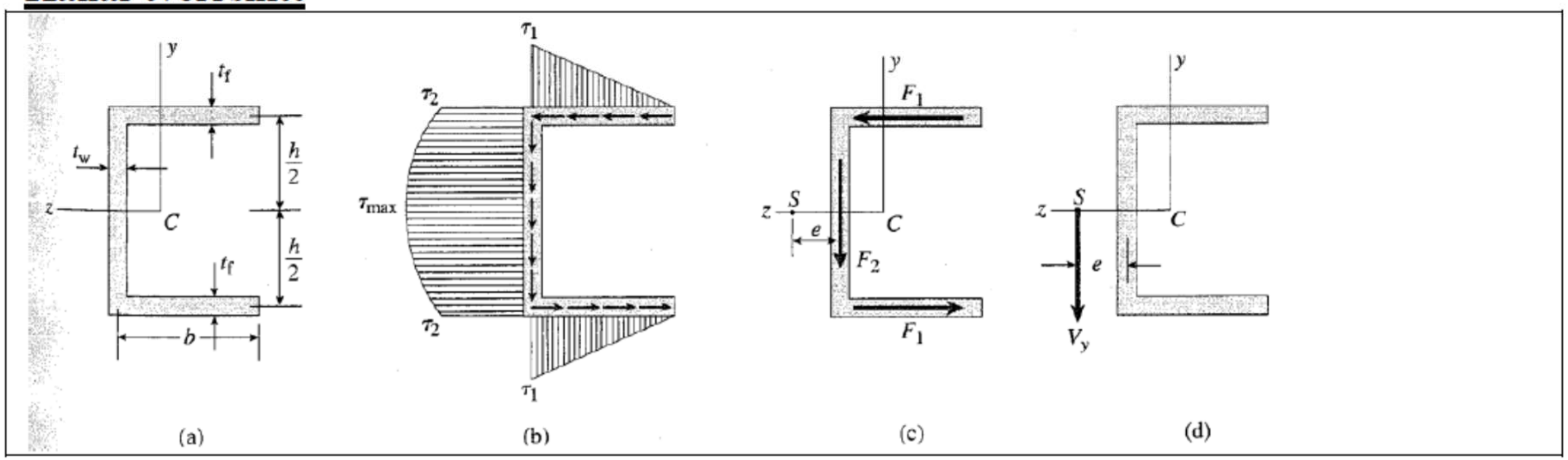
1) Beregn skjærspenningene som oppstår ved bøyning om symmetriaksen,

   vha. formelen $\boxed{\tau = \dfrac{V \cdot S}{I \cdot t}}$

   (Statisk moment S skrives Q i Gere/Goodno.)

2) Beregn hvor resultanten (V) av disse skjærspenningene ligger.
3) Skjærsenteret S ligger på angrepslinjen til resultanten av skjærspenningene.

Kanal-tverrsnitt



Symmetri $\Rightarrow$ S er på z-aksen.

1) Spenningene τ_1, τ_2 og τ_{max} beregnes vha. formelen $\boxed{\tau = \frac{V \cdot S}{I \cdot t}}$

2) Av dette kan vi beregne F_1 og F_2, men uten å regne forstår vi uansett at $F_2 = V$

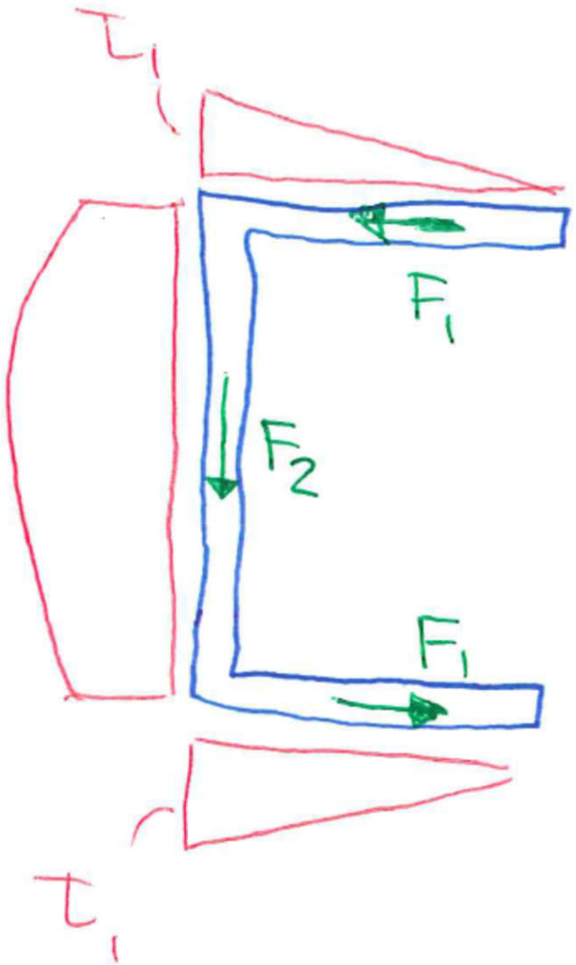
Derfor er det egentlig ikke nødvendig å beregne τ_2 og τ_{max} hvis vi bare er ute etter å lokalisere skjærsenteret.

De to kreftene F_1 og kraften F_2 danner til sammen en resultant lik V.

3) $F_1 \cdot h - F_2 \cdot e = 0 \Rightarrow \boxed{e = \frac{b^2 h^2 t_f}{4 I_z}}$

Arealmomentet I_z er oppgitt i likning (6-62) s. 499 i Gere/Goodno. For øvrig pleier det å være oppgitt i profiltabeller (tverrsnittstabeller).

SKJÆRSENTER:



$$F_1 = \frac{\tau_1 \cdot t_f \cdot b}{2}$$

$$= \frac{V \cdot S_{hjørne}}{I_z \cdot t_f} \cdot \frac{1}{2} t_f b$$

$$= \frac{V \cdot b \cdot t_f \cdot \frac{h}{2}}{I_z \cdot t_f} \cdot \frac{1}{2} t_f b$$

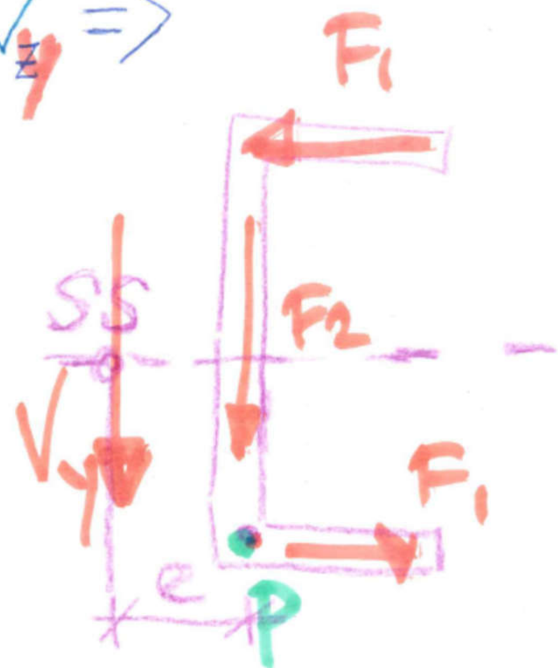
$$= \frac{1}{4} V \cdot \frac{b^2 t_f h}{I_z}$$

MOMENT OM P:

$F_2 = V_y \Rightarrow$

$$F_1 \cdot h - \cancel{F_2} \cdot e = 0$$

$$\frac{1}{4} V \frac{b^2 t_f h}{I_z} \cdot h - V \cdot e = 0$$



$$\boxed{e = \frac{b^2 h^2 t_f}{4 I_z}}$$

TBYG3018 Design of Offshore Structures

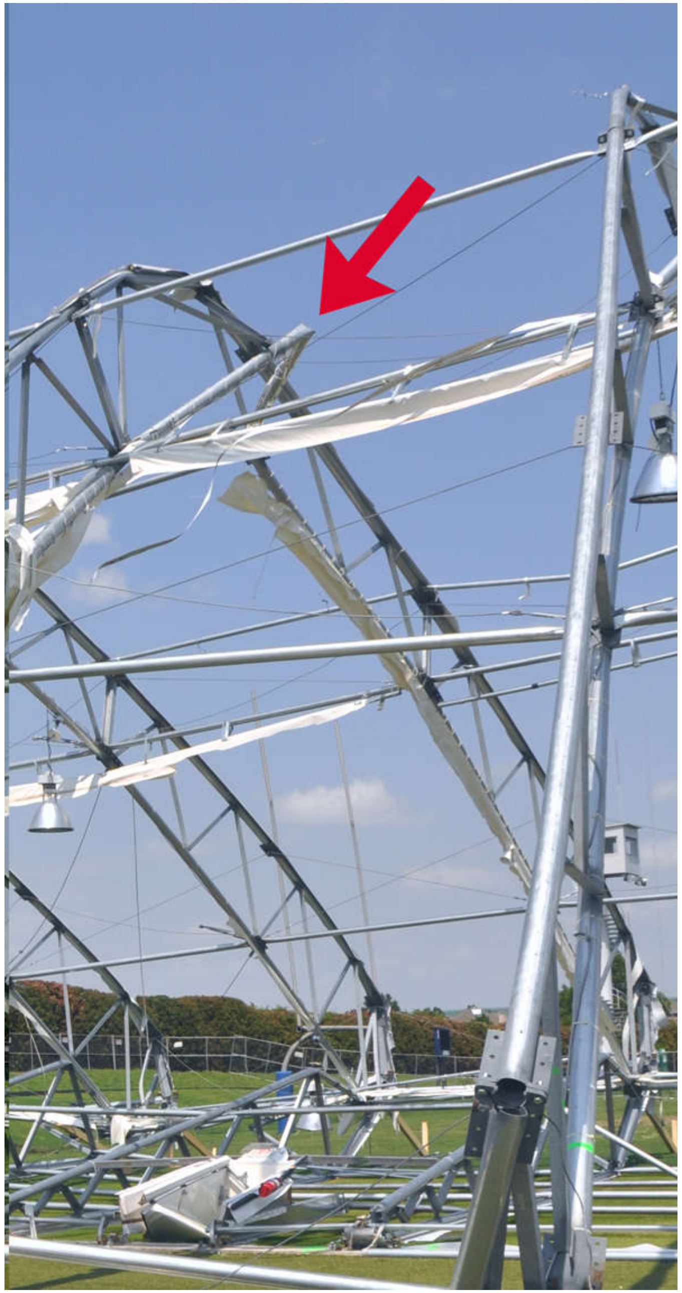
Module 3 – Introduction to Design of Offshore Structures

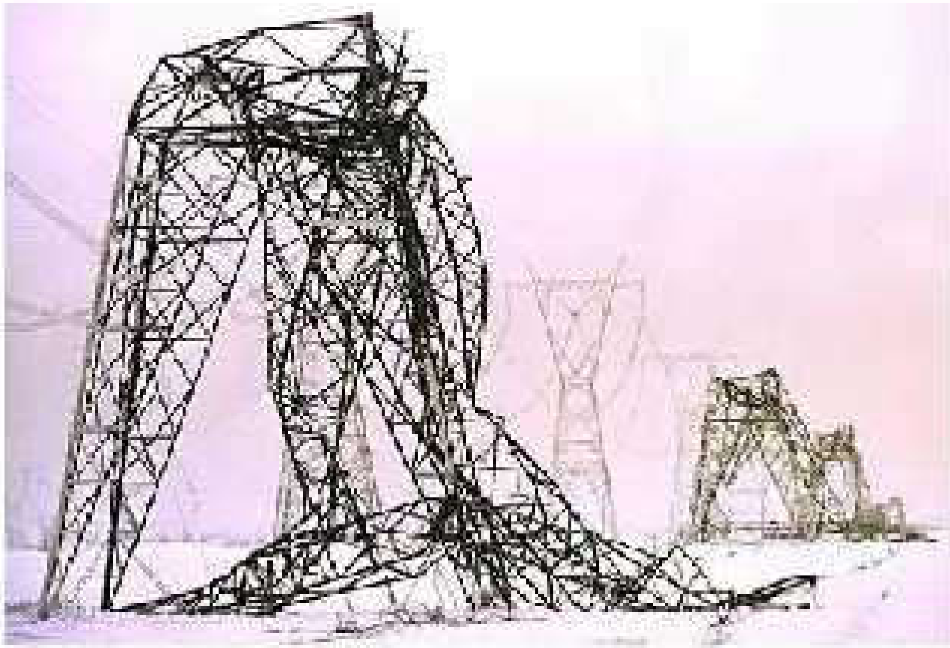
Jomar Tørset, Assistant professor

Norwegian University of Science and Technology

Buckling

(www.eqclearinghouse.org/2011-03-11-sendai/2011/08/03/eeri-steel-structures-reconnaissance-group/dsc_0399/)





(www.students.uwosh.edu/~piehld88/ndcweb/ndcproj1.htm)

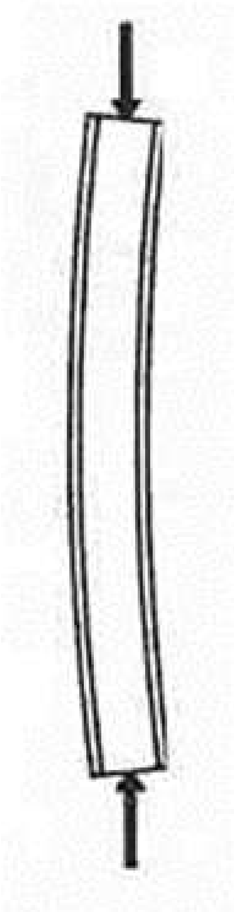
(www.newswise.com/images/uploads/2009/10/6/buckling.jpg)

(http://en.wikipedia.org/wiki/Sun_kink)

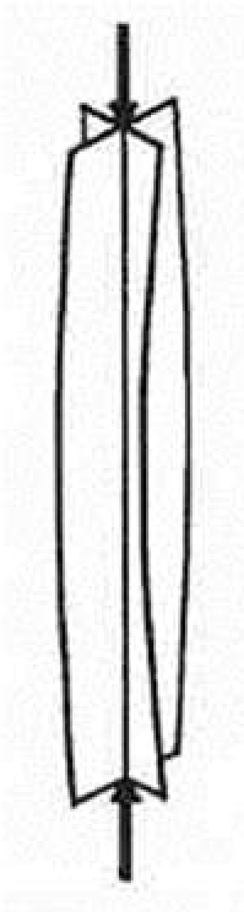
(www.startribune.com/blogs/125895788.html)

(http://research.ucc.ie/boolean/2010/00/dePaor/11/en)

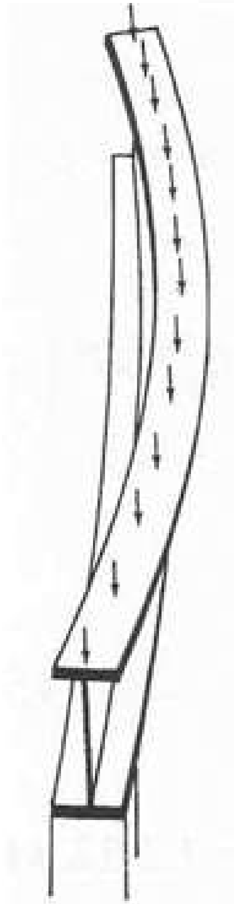
Stabilitetsfenomener



Knekking

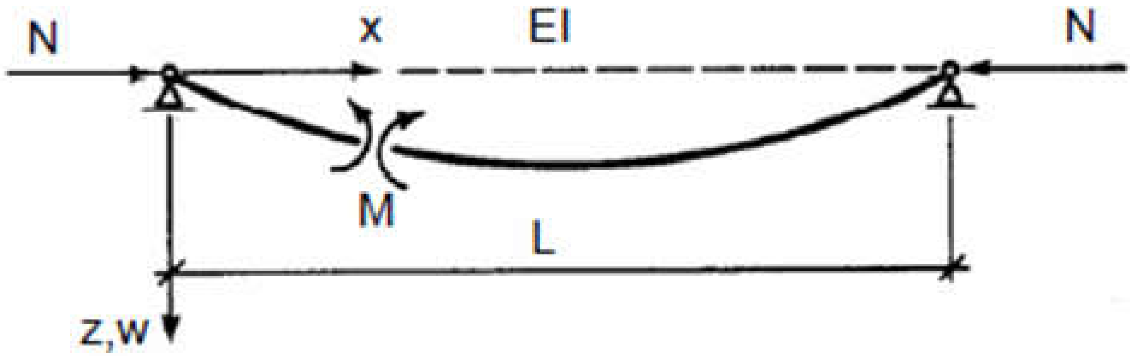


Torsjonsknekking



Vipping

Eulerstaven



Diff.ligning: $$w_{,xx} + \frac{N}{EI} w = 0$$

Eulerlast:

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

Forutsetninger:

① Rett stav uten formfeil

② Ledd i begge ender

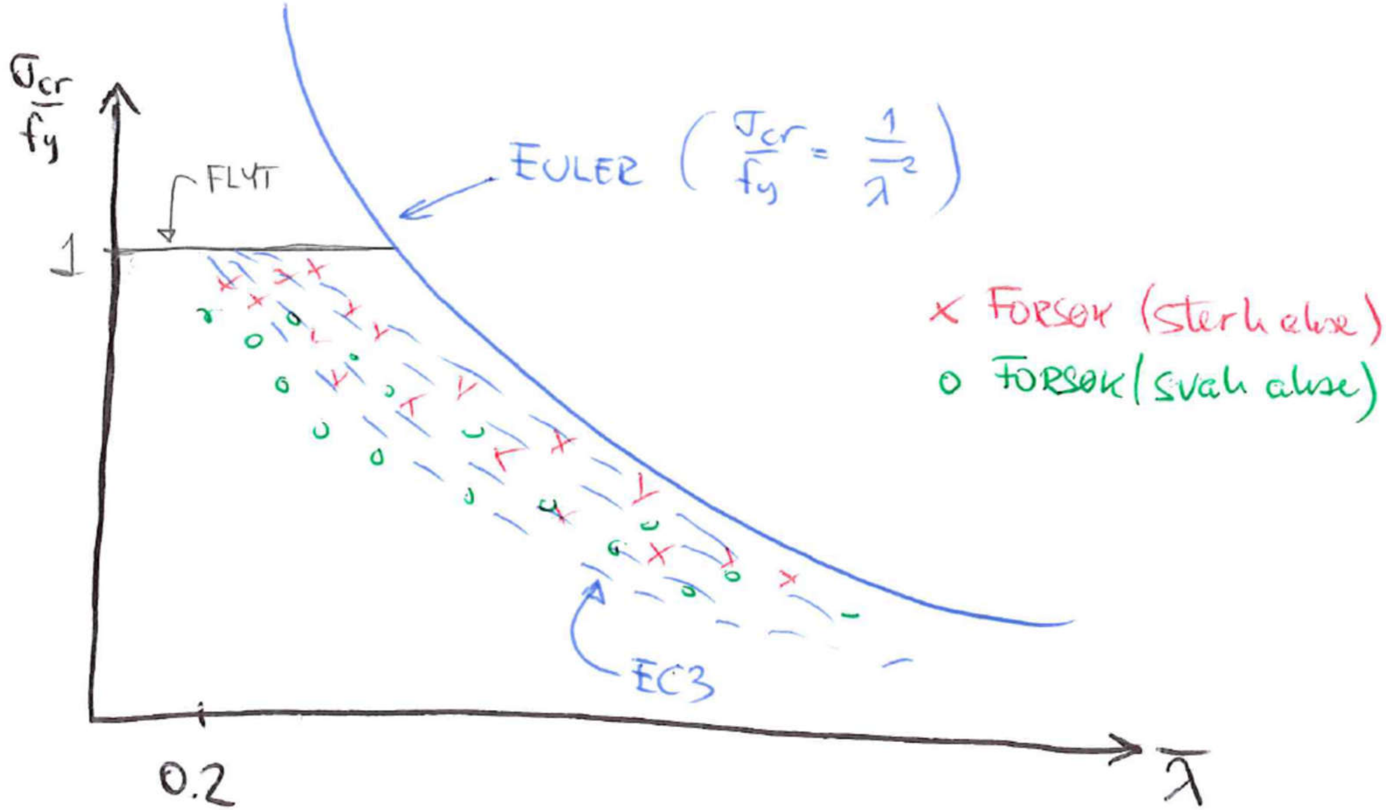
③ Lineært elastisk materiale (Hooks lov)

④ Sentrisk last

⑤ Små forskyvninger

⑥ Konstant tverrsnitt (EI)

Grafisk:



$\frac{\sigma_{cr}}{f_y}$
FLYT
1
EULER $\left(\frac{\sigma_{cr}}{f_y} = \frac{1}{\bar{\lambda}^2}\right)$
× FORSØK (sterk akse)
o FORSØK (svak akse)
EC3
0.2
$\bar{\lambda}$

Table 6.1: Imperfection factors for buckling curves

Buckling curve	a_0	a	b	c	d
Imperfection factor α	0,13	0,21	0,34	0,49	0,76

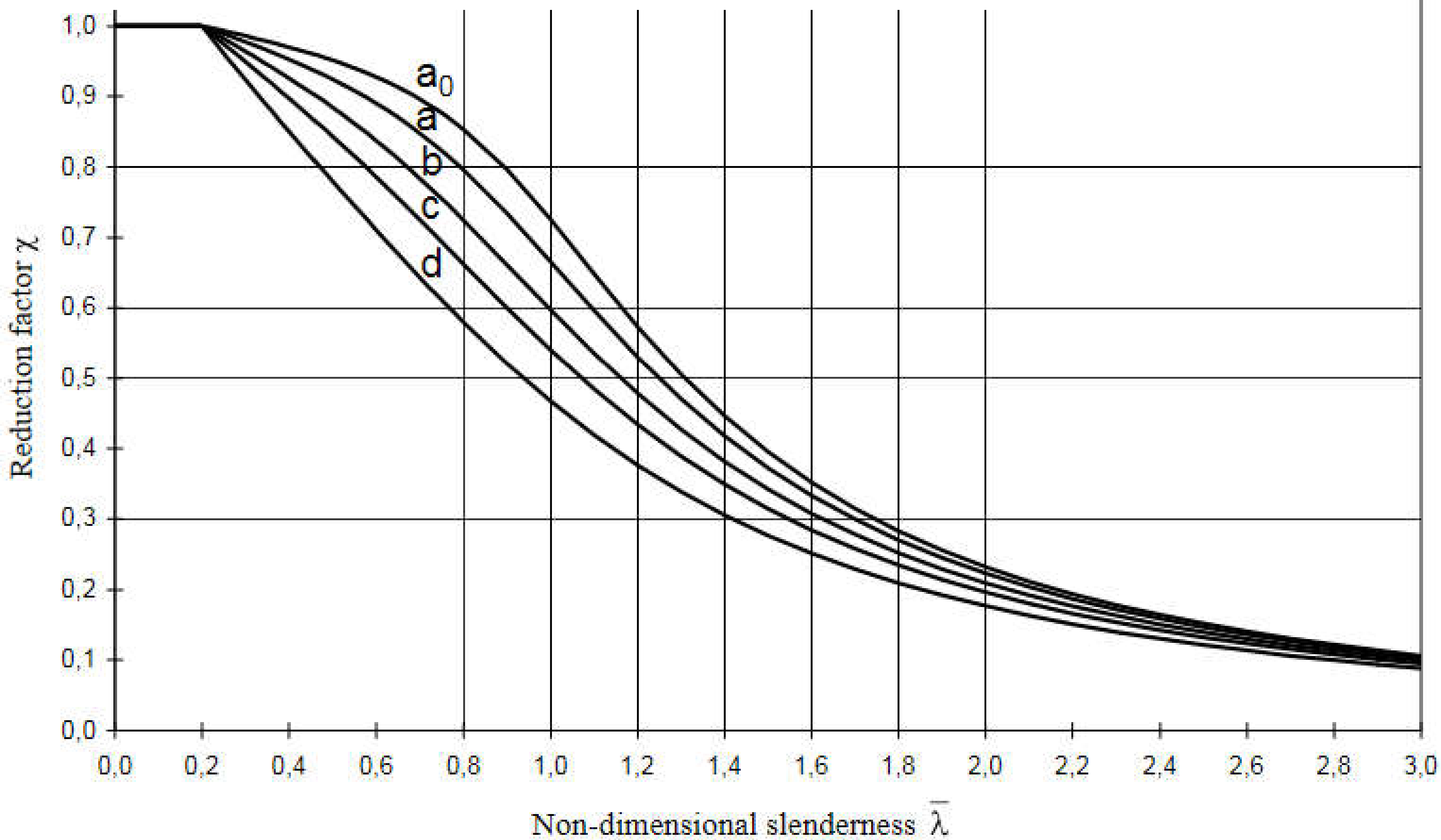
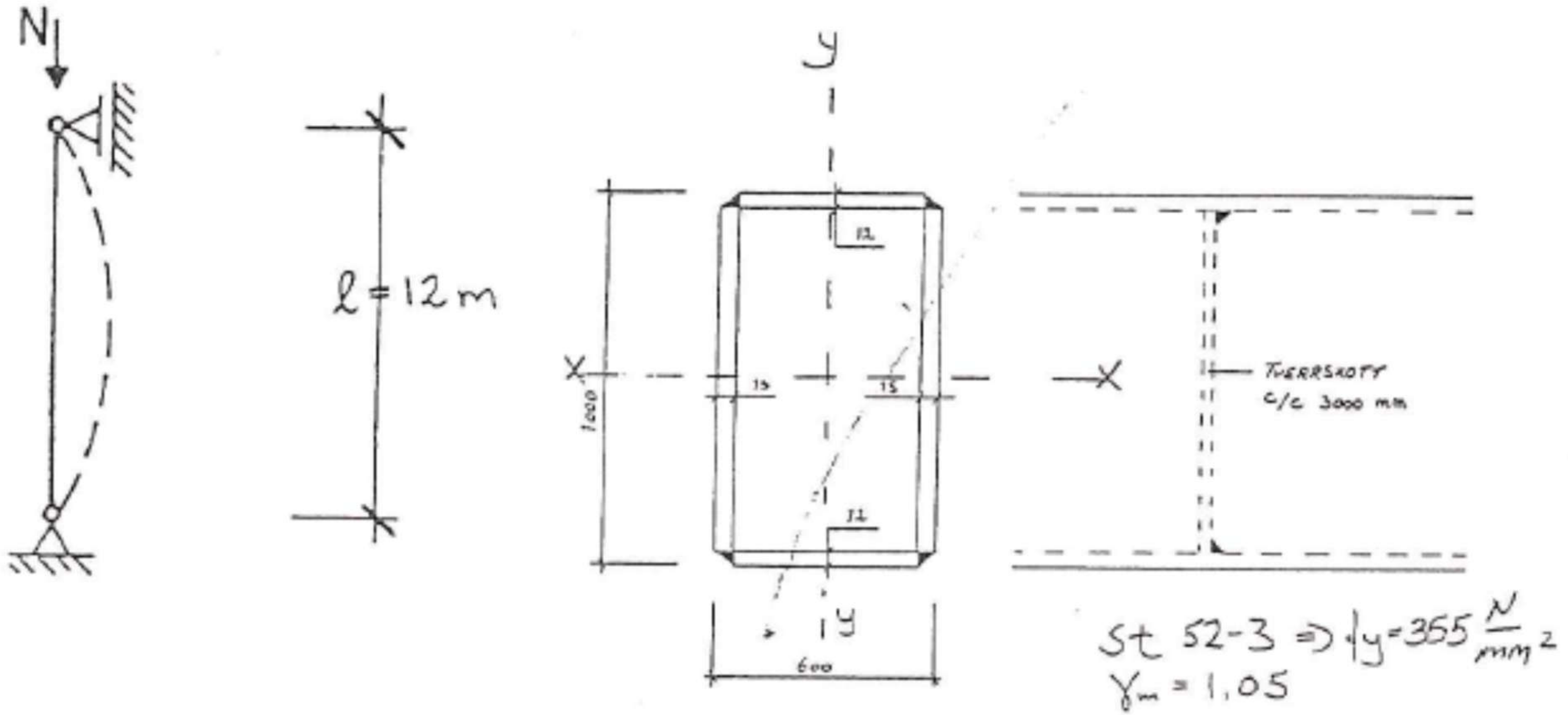


Figure 6.4: Buckling curves

Eksempel



N
ℓ = 12 m
y
x
12
15
15
12
1000
600
TVERRSKOTT
c/c 3000 mm
St 52-3 ⇒ fy = 355 N/mm²
γm = 1,05

6.3.1 Uniform members in compression

6.3.1.1 Buckling resistance

(1) A compression member should be verified against buckling as follows:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1{,}0 \tag{6.46}$$

where N_{Ed} is the design value of the compression force;

$N_{b,Rd}$ is the design buckling resistance of the compression member.

(2) For members with non-symmetric Class 4 sections allowance should be made for the additional moment ΔM_{Ed} due to the eccentricity of the centroidal axis of the effective section, see also 6.2.2.5(4), and the interaction should be carried out to 6.3.4 or 6.3.3.

(3) The design buckling resistance of a compression member should be taken as:

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \quad \text{for Class 1, 2 and 3 cross-sections} \tag{6.47}$$

$$N_{b,Rd} = \frac{\chi A_{eff} f_y}{\gamma_{M1}} \quad \text{for Class 4 cross-sections} \tag{6.48}$$

where χ is the reduction factor for the relevant buckling mode.

NOTE For determining the buckling resistance of members with tapered sections along the member or for non-uniform distribution of the compression force second order analysis according to 5.3.4(2) may be performed. For out-of-plane buckling see also 6.3.4.

(4) In determining A and A_{eff} holes for fasteners at the column ends need not to be taken into account.

6.3.1.2 Buckling curves

(1) For axial compression in members the value of χ for the appropriate non-dimensional slenderness $\bar{\lambda}$ should be determined from the relevant buckling curve according to:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \leq 1{,}0 \tag{6.49}$$

where $\Phi = 0{,}5\left[1 + \alpha\left(\bar{\lambda} - 0{,}2\right) + \bar{\lambda}^2\right]$

$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}} \quad \text{for Class 1, 2 and 3 cross-sections}$$

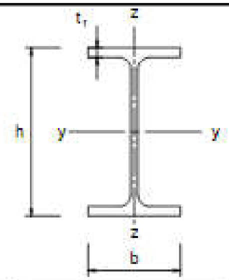
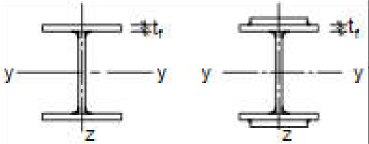
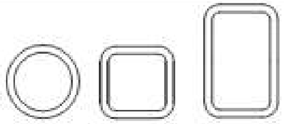
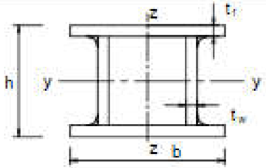
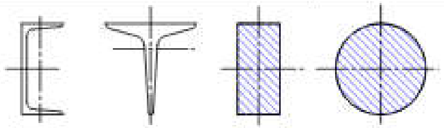
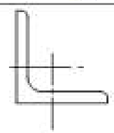
$$\bar{\lambda} = \sqrt{\frac{A_{eff} f_y}{N_{cr}}} \quad \text{for Class 4 cross-sections}$$

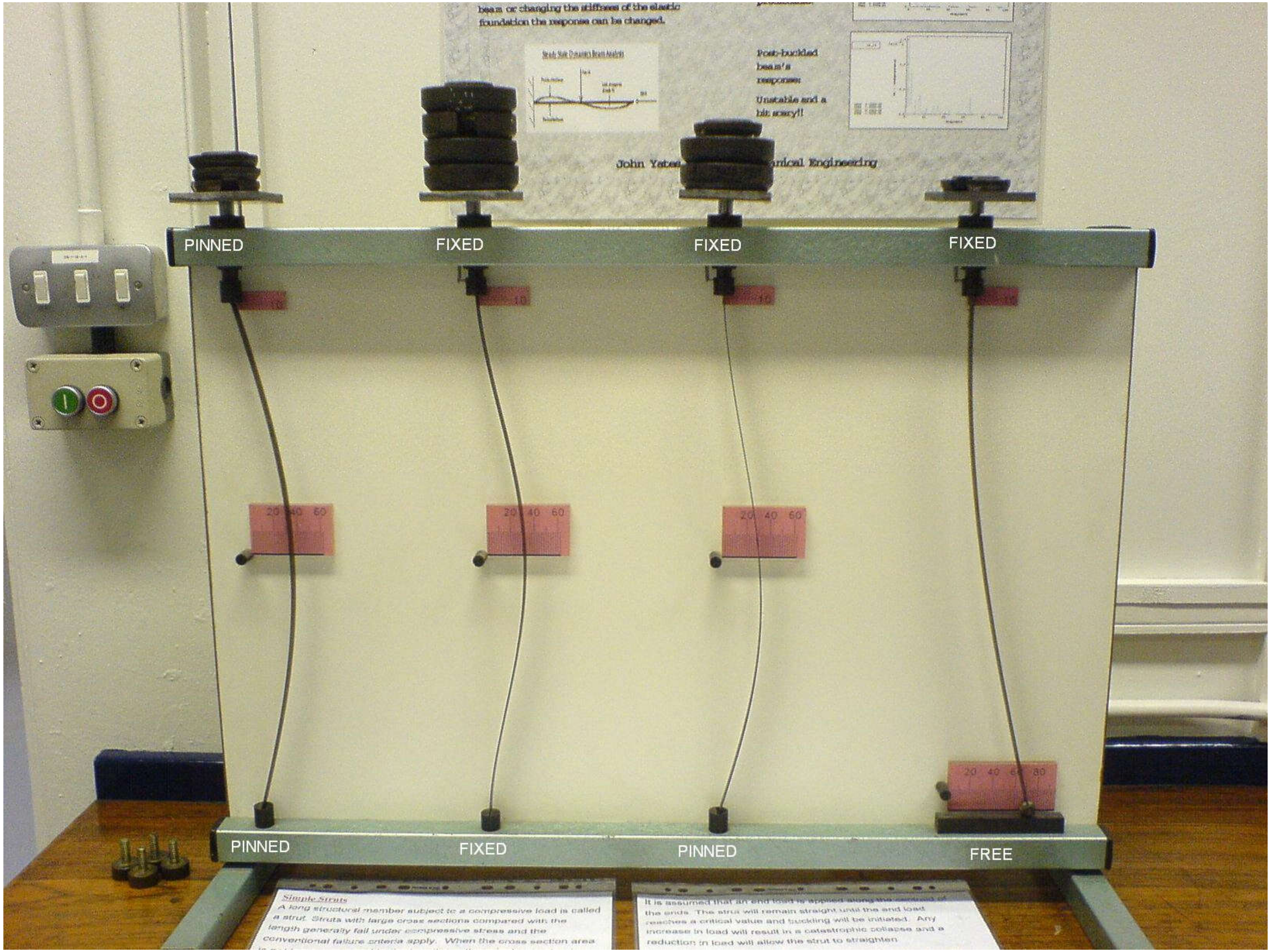
α is an imperfection factor

N_{cr} is the elastic critical force for the relevant buckling mode based on the gross cross sectional properties.

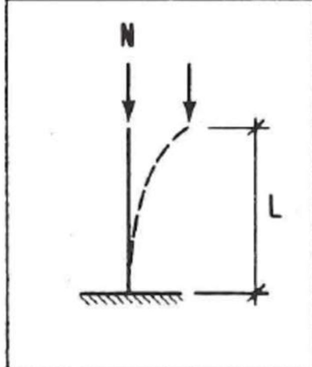
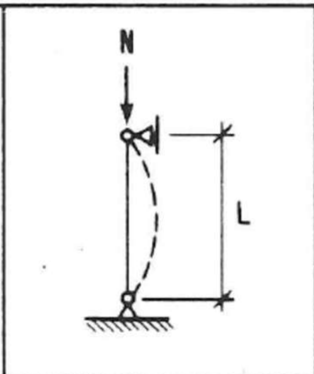
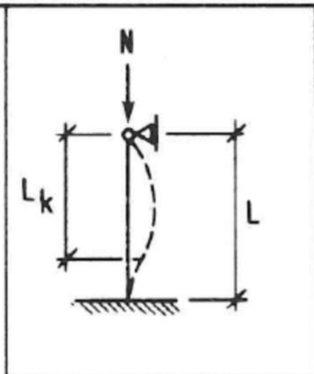
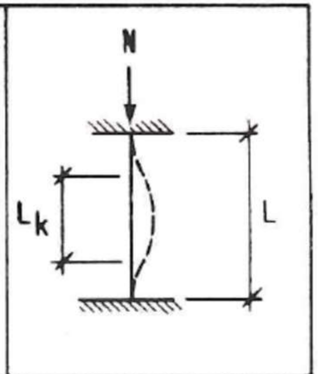
(2) The imperfection factor α corresponding to the appropriate buckling curve should be obtained from Table 6.1 and Table 6.2.

Table 6.2: Selection of buckling curve for a cross-section

Cross section		Limits		Buckling about axis	Buckling curve S 235 S 275 S 355 S 420	Buckling curve S 460
Rolled sections		$h/b > 1{,}2$	$t_f \le 40$ mm	y – y z – z	a b	a_0 a_0
			40 mm < $t_f \le 100$	y – y z – z	b c	a a
		$h/b \le 1{,}2$	$t_f \le 100$ mm	y – y z – z	b c	a a
			$t_f > 100$ mm	y – y z – z	d d	c c
Welded I-sections		$t_f \le 40$ mm		y – y z – z	b c	b c
		$t_f > 40$ mm		y – y z – z	c d	c d
Hollow sections		hot finished		any	a	a_0
		cold formed		any	c	c
Welded box sections		generally (except as below)		any	b	b
		thick welds: $a > 0{,}5t_f$ $b/t_f < 30$ $h/t_w < 30$		any	c	c
U-, T- and solid sections				any	c	c
L-sections				any	b	b


PINNED
FIXED
FIXED
FIXED
PINNED
FIXED
PINNED
FREE

Tabell 6.1 - Basistilfeller for stavknekking

				
Knekkings-betingelse	$\cos kL = 0$	$\sin kL = 0$	$\frac{kL}{\mathrm{tg}\, kL} = 1$	$\cos kL = 1$
Laveste egenverdi	$kL = \frac{\pi}{2}$	$kL = \pi$	$kL = 4.493$	$kL = 2\pi$
Knekklengde	$L_k = 2.0\ L$	$L_k = L$	$L_k \approx 0.7\ L$	$L_k = 0.5\ L$

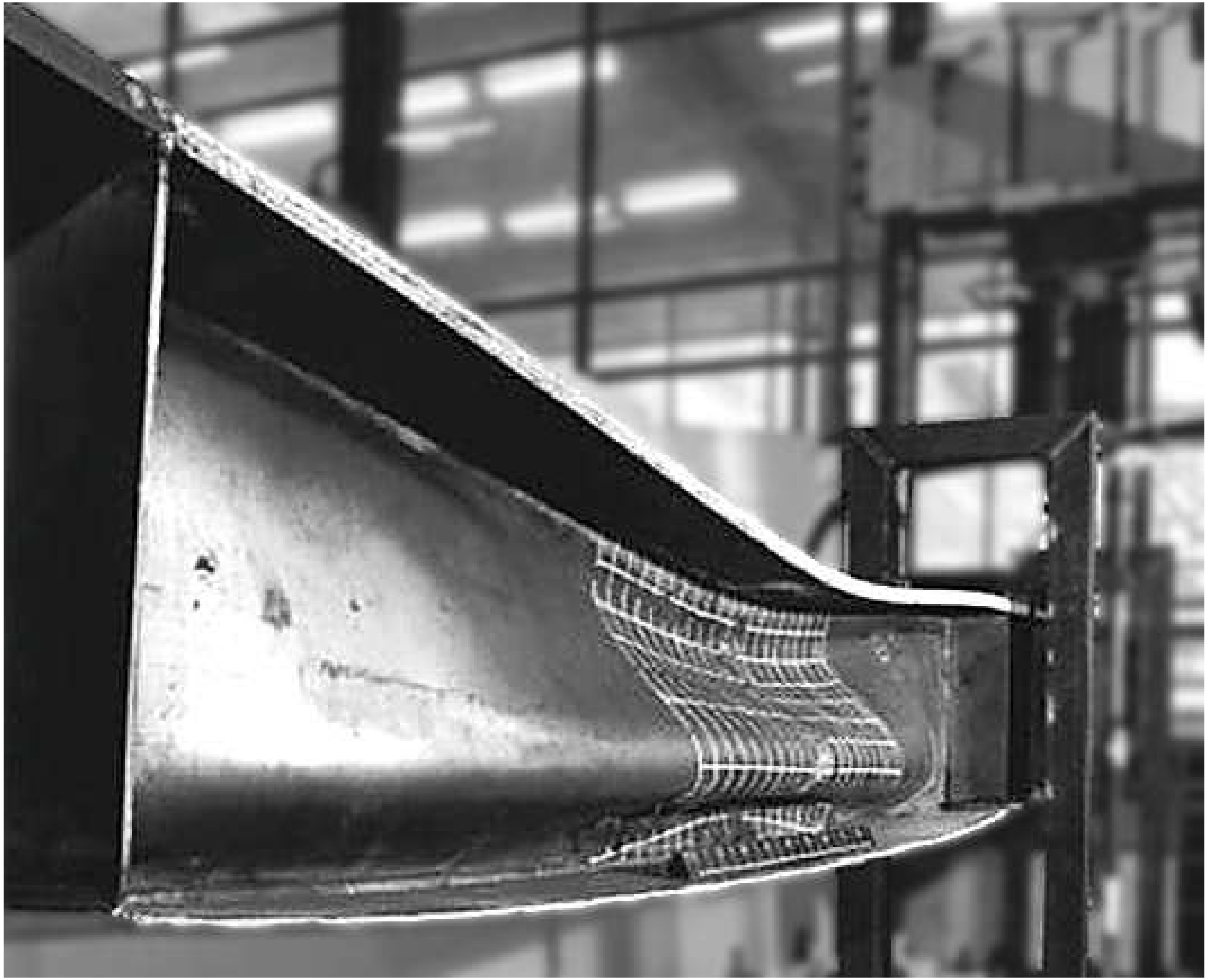
Knekklengden L_k for en stav eller et stavsystem er definert slik at Eulerlasten for en leddlagret stav med lengde L_k er lik Eulerlasten N_E for den gitte stav.

NB! Det finnes datamaskin programmer

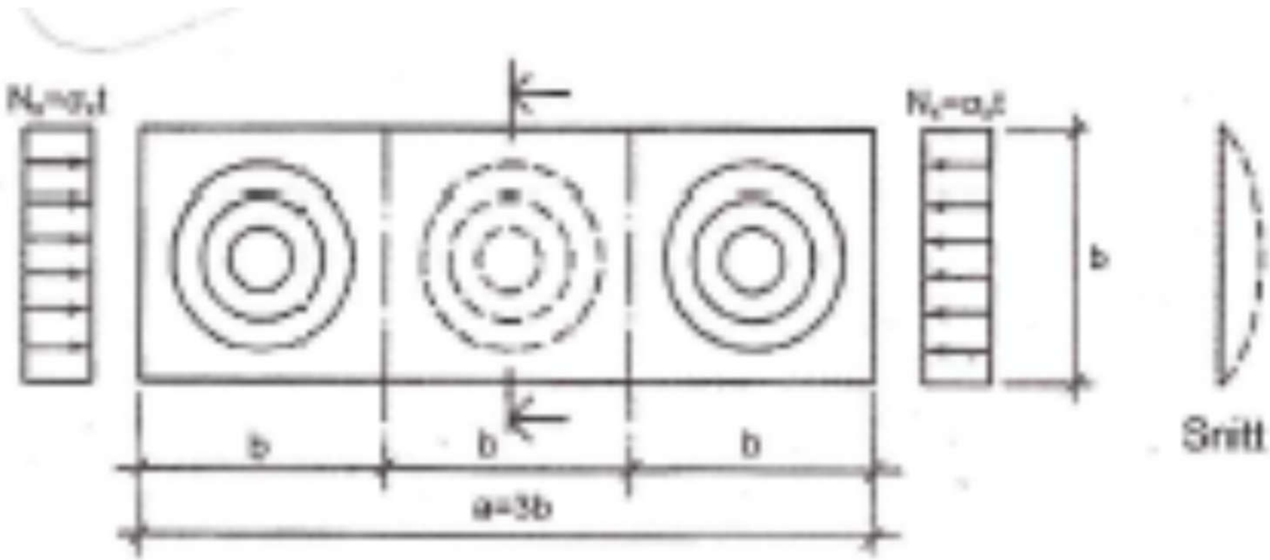
som regner ut N_E

NS 2472 spør om L_K

$$L_K = \pi \sqrt{\frac{EI}{N_E}}$$



(www.laufsed.com/teaching_engineering.html)



a) Knekkform for plate med a=3b under konstant aksialkraft

Eurokode 3: Prosjektering av stålkonstruksjoner
Del 1-5: Plater påkjent i plateplanet

TBYG3018 Design of Ocean Space Structures

Module 3 – Introduction to design of steel structures

Jomar Tørset, Assistant professor

Classification of beam sections

NS-EN 1993-1-1 Clause 5.5

Class 1: Can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance.

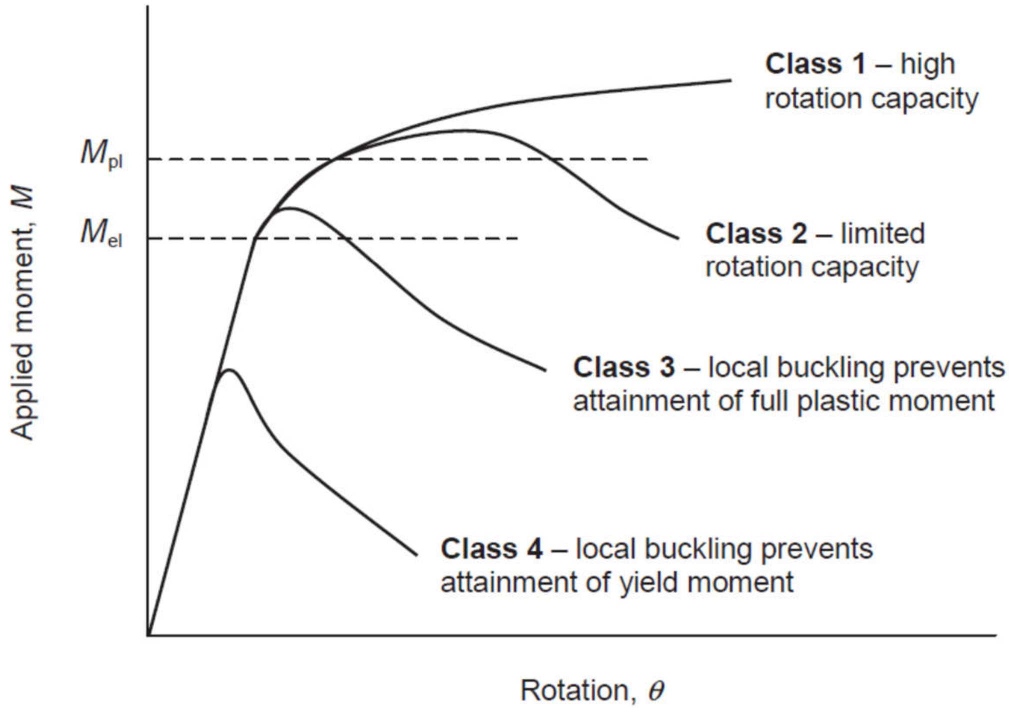
Class 2: Can develop their plastic moment resistance, but have limited rotation capacity because of local buckling..

Class 3: Can reach the yield strength in the extreme compression fibre, but local buckling is liable to prevent the development of the plastic moment resistance.

Class 4: Local buckling will occur before the attainment of the yield stress in one or more parts of the cross-section.

Classification of beam sections

NS-EN 1993-1-1 Clause 5.5



6.1.4.2 Classification

(1) Four classes of cross-sections are defined, as follows:

- Class 1 cross-sections are those that can form a plastic hinge with the rotation capacity required for plastic analysis without reduction of the resistance.

NOTE Further information on class 1 cross-sections is given in Annex G.

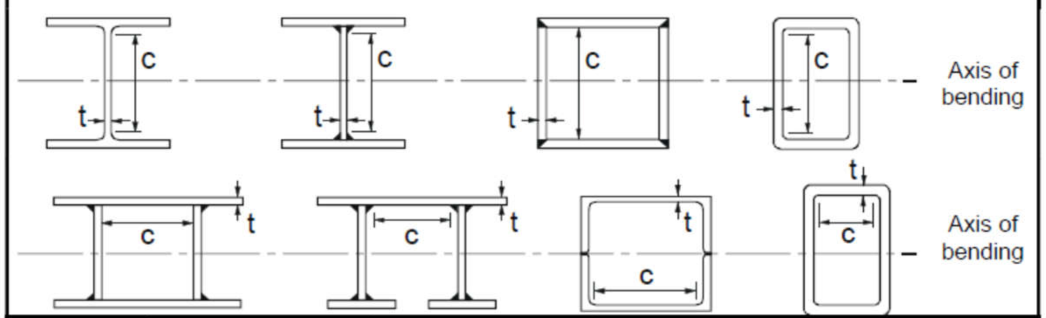
- Class 2 cross-sections are those that can develop their plastic moment resistance, but have limited rotation capacity because of local buckling.

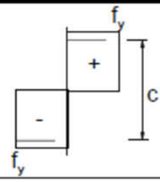
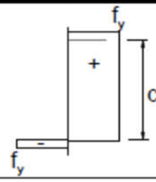
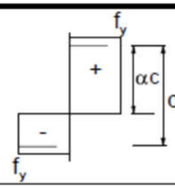
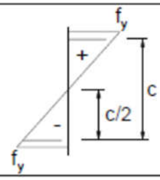
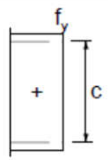
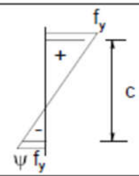
- Class 3 cross-sections are those in which the calculated stress in the extreme compression fibre of the aluminium member can reach its proof strength, but local buckling is liable to prevent development of the full plastic moment resistance.

- Class 4 cross-sections are those in which local buckling will occur before the attainment of proof stress in one or more parts of the cross-section.

Table 5.2 (sheet 1 of 3): Maximum width-to-thickness ratios for compression parts

Internal compression parts



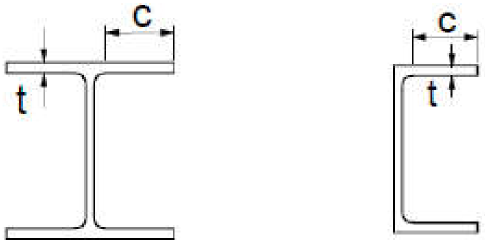
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \le -1$*): $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

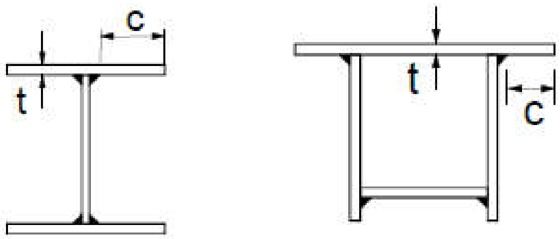
*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_y$ or the tensile strain $\varepsilon_y > f_y/E$

Table 5.2 (sheet 2 of 3): Maximum width-to-thickness ratios for compression parts

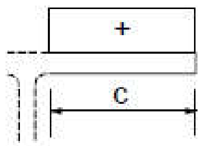
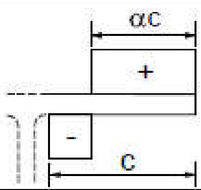
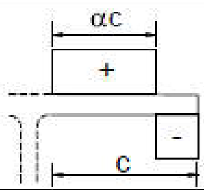
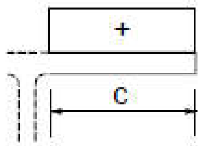
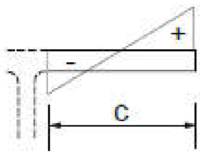
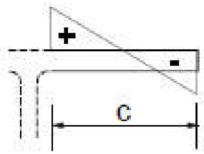
Outstand flanges



Rolled sections



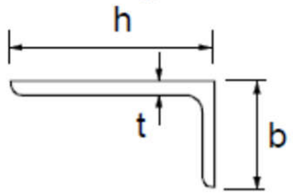
Welded sections

Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \leq 9\varepsilon$	$c/t \leq \dfrac{9\varepsilon}{\alpha}$	$c/t \leq \dfrac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \leq 10\varepsilon$	$c/t \leq \dfrac{10\varepsilon}{\alpha}$	$c/t \leq \dfrac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 14\varepsilon$	$c/t \leq 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

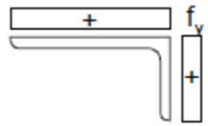
Table 5.2 (sheet 3 of 3): Maximum width-to-thickness ratios for compression parts

Angles

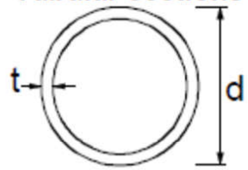


Refer also to "Outstand flanges" (see sheet 2 of 3)

Does not apply to angles in continuous contact with other components

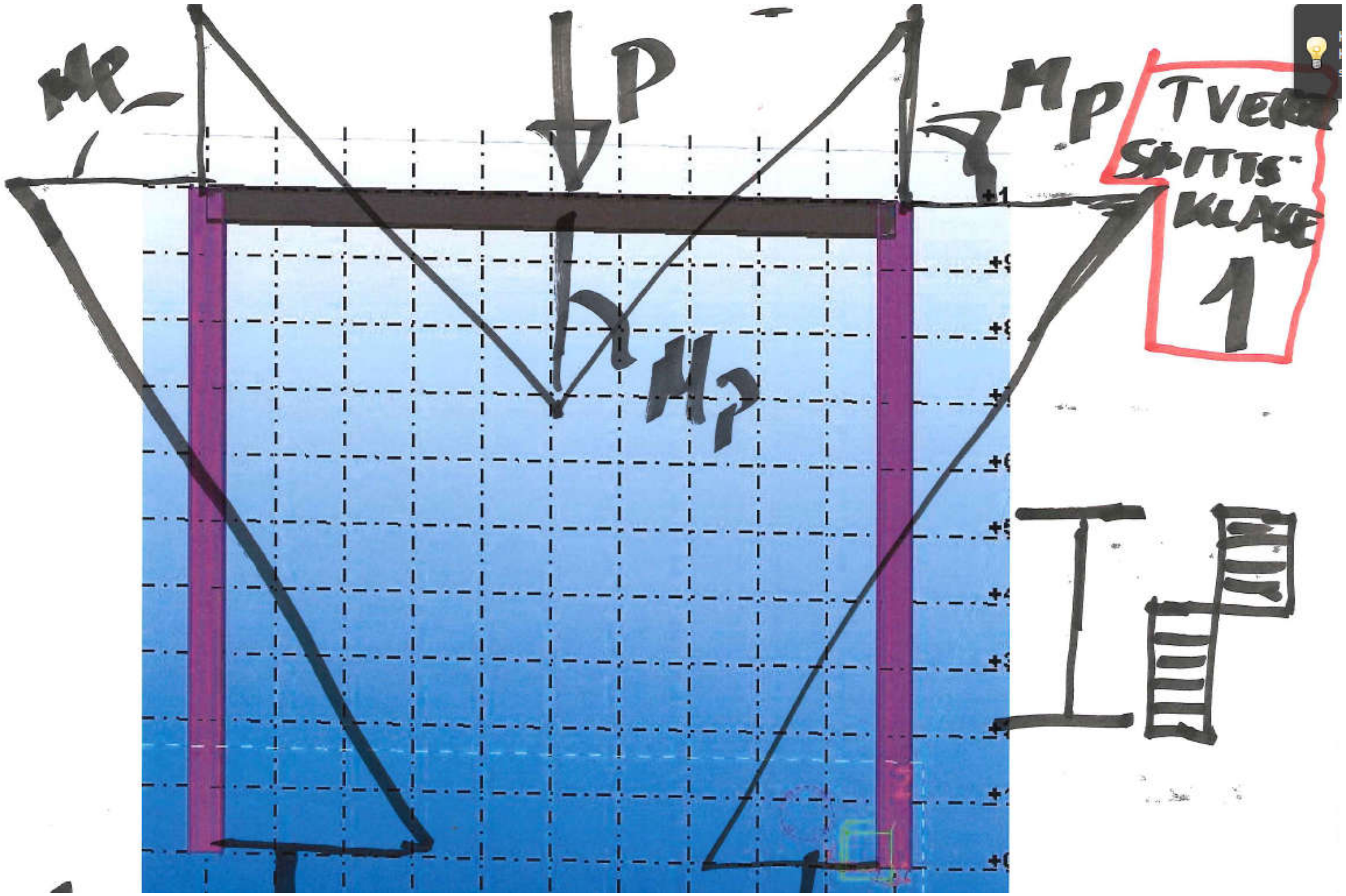
Class	Section in compression
Stress distribution across section (compression positive)	
3	$h/t \leq 15\varepsilon : \dfrac{b+h}{2t} \leq 11{,}5\varepsilon$

Tubular sections



Class	Section in bending and/or compression
1	$d/t \leq 50\varepsilon^2$
2	$d/t \leq 70\varepsilon^2$
3	$d/t \leq 90\varepsilon^2$ **NOTE** For $d/t > 90\varepsilon^2$ see EN 1993-1-6.

$\varepsilon = \sqrt{235/f_y}$						
	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71
	ε^2	1,00	0,85	0,66	0,56	0,51



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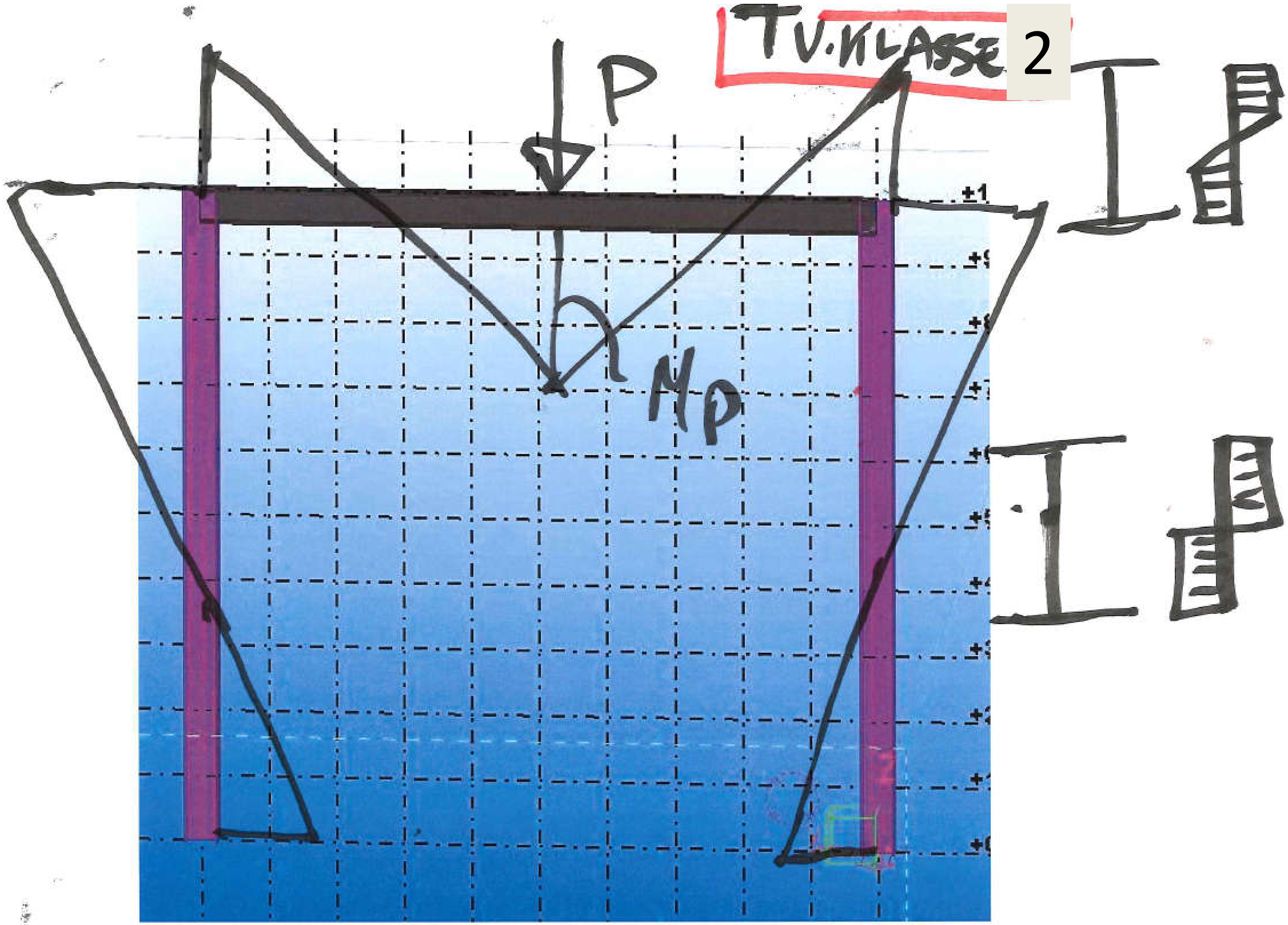
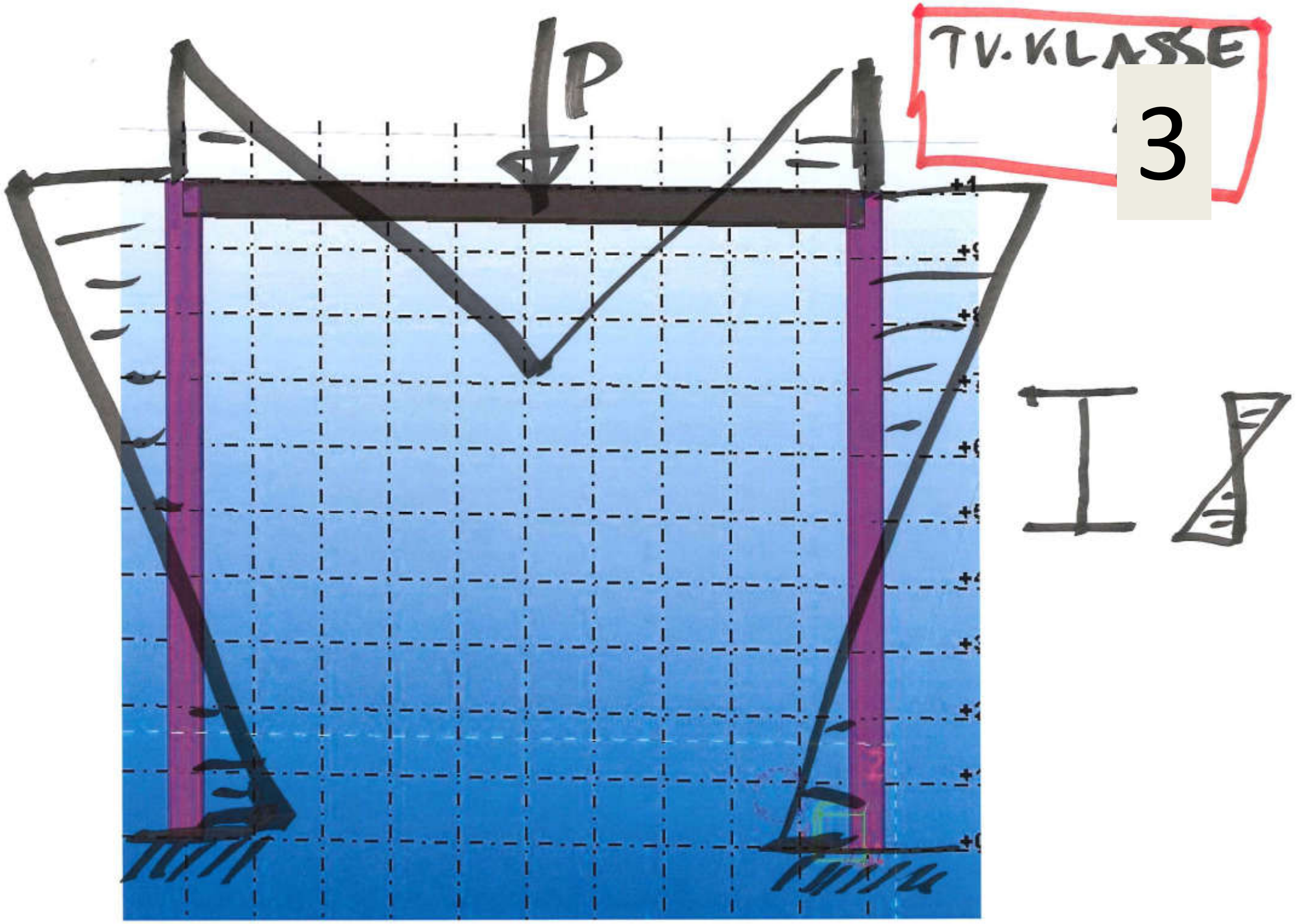
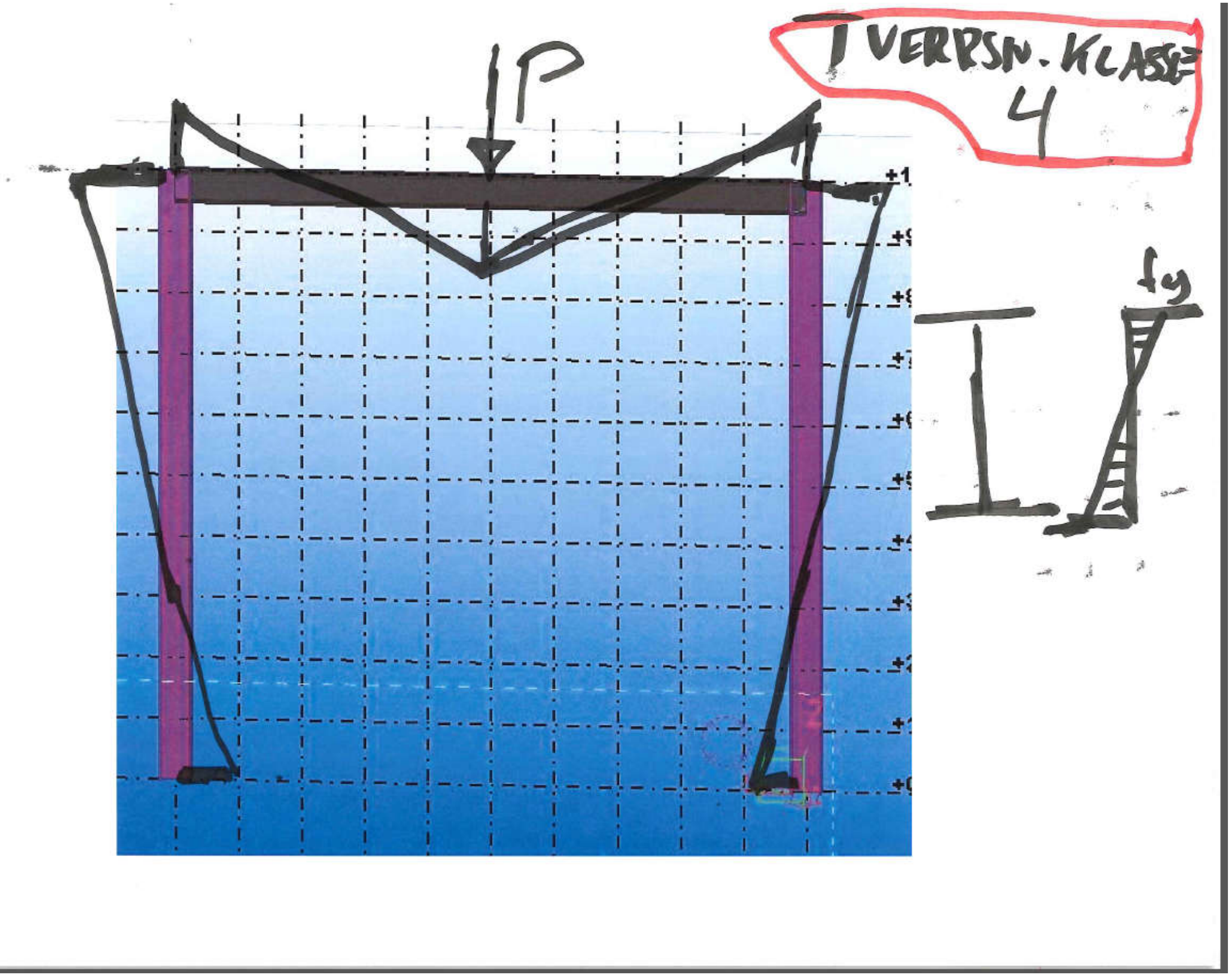

TV.KLASSE 2
P
Mp


TV. KLASSE
3
P



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Von Mises yield criterion

- Von Mises yield criterion is commonly used to establish the elastic resistance of metallic structures. For a two-dimensional stress state it may be written as:

$$\sqrt{\sigma_x^2 + \sigma_y^2 - \sigma_x \sigma_y + 3\tau_{xy}^2} = f_d = \frac{f_y}{\gamma_{M0}}$$

or in principal stresses:

$$\sqrt{\sigma_1^2 + \sigma_2^2 - \sigma_1 \sigma_2} = f_d = \frac{f_y}{\gamma_{M0}}$$

- The material factor γ_{M0} is taken as

  1.05 for buildings (NS-EN 1993-1-1 NA.6.1)
  1.10 for bridges (NS-EN 1993-2 NA.6.1)
  1.15 for offshore structures (NORSOK N-004 6.1)

Von Mises yield criterion

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$$\sqrt{\sigma_x^2 + \sigma_y^2 - \sigma_x \sigma_y + 3\tau_{xy}^2} = f_d = \frac{f_y}{\gamma_{M0}}$$

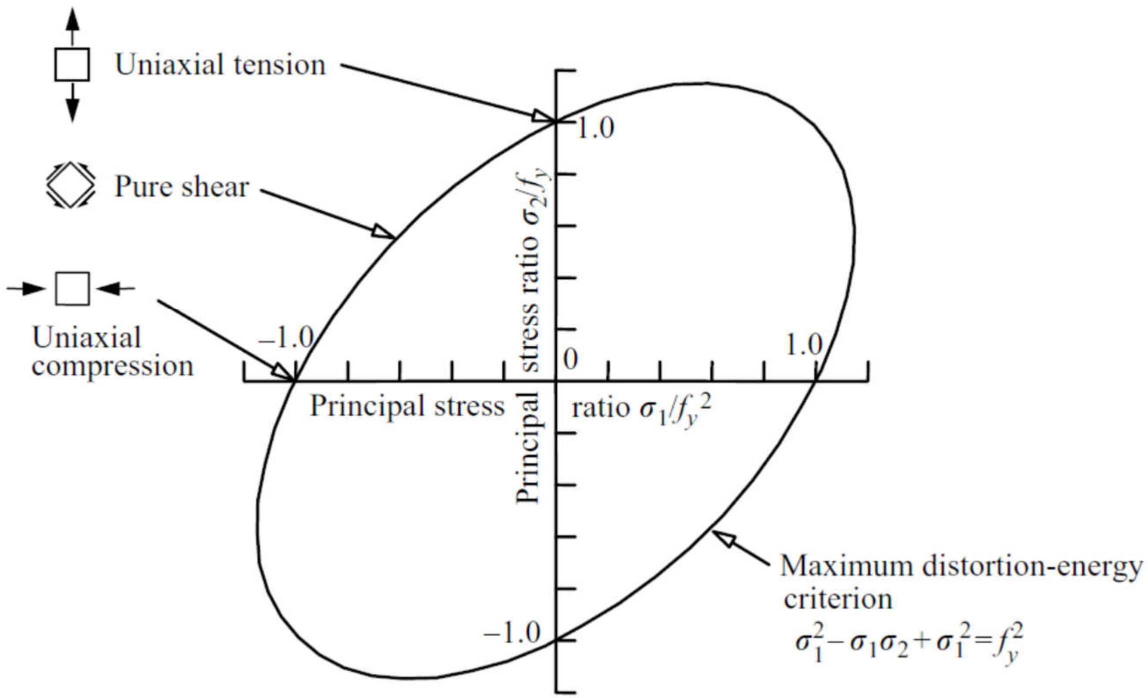
  or in principal stresses:

$$\sqrt{\sigma_1^2 + \sigma_2^2 - \sigma_1 \sigma_2} = f_d = \frac{f_y}{\gamma_{M0}}$$

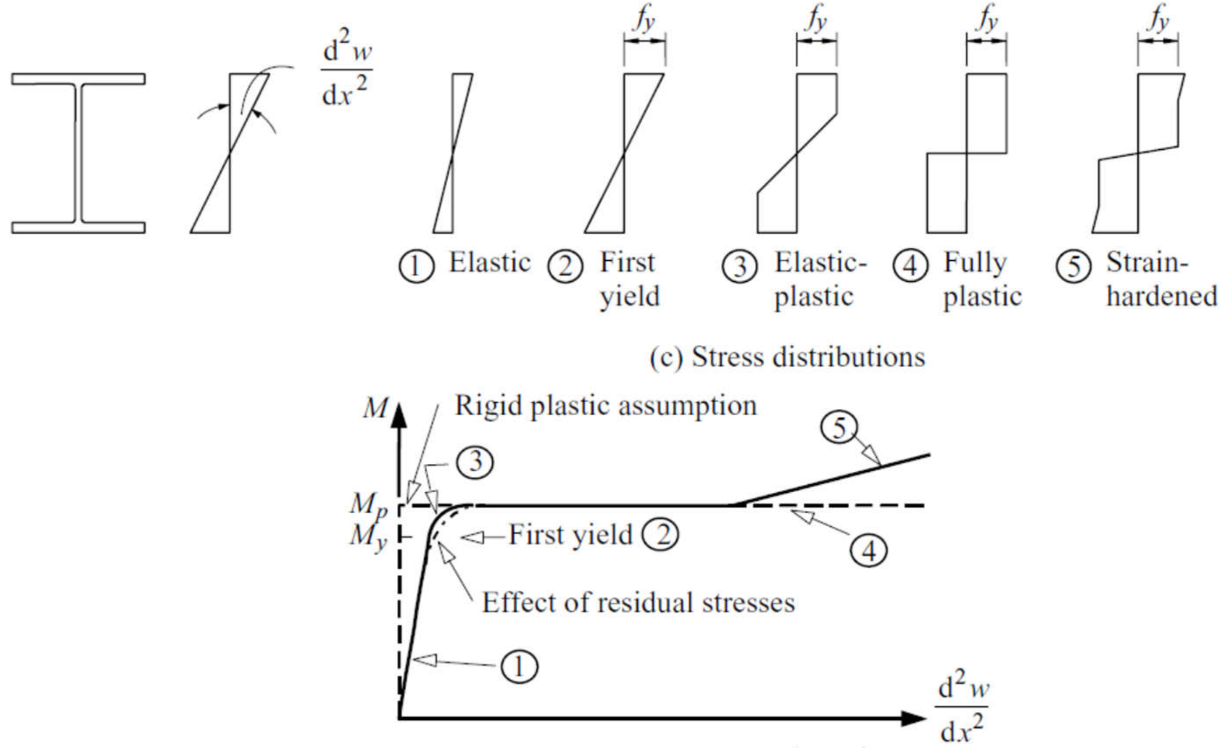
- The material factor γ_{M0} is taken as

  1.05 for buildings (NS-EN 1993-1-1 NA.6.1)
  1.10 for bridges (NS-EN 1993-2 NA.6.1)
  1.15 for offshore structures (NORSOK N-004 6.1)

Yielding under biaxial stresses



Moment-curvature relationships for steel beams



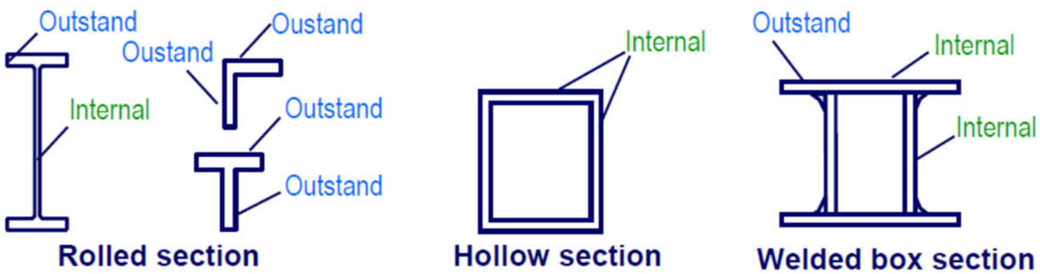
Lower-bound theorem

A chosen (assumed) distribution of stresses in a structure which satisfies equilibrium between internal and external forces and nowhere exceeds the plastic resistance, results in a resistance less or equal to the correct value.

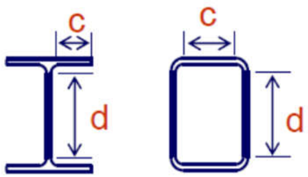
Basis of section classification

- Some parts are outstand:
  - Flanges of I beams
  - Legs of angles and tees
  - Flange part of welded sections

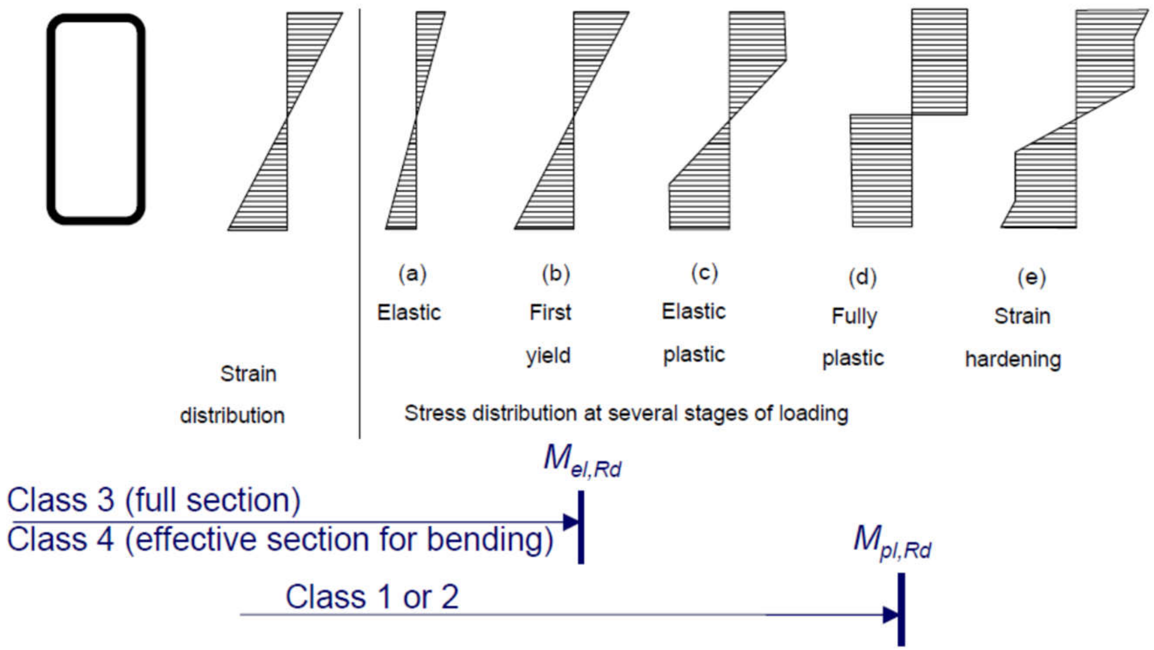
- Some parts are internal:
  - Webs of I beams
  - Walls of hollow sections
  - Flange part/web of welded box sections



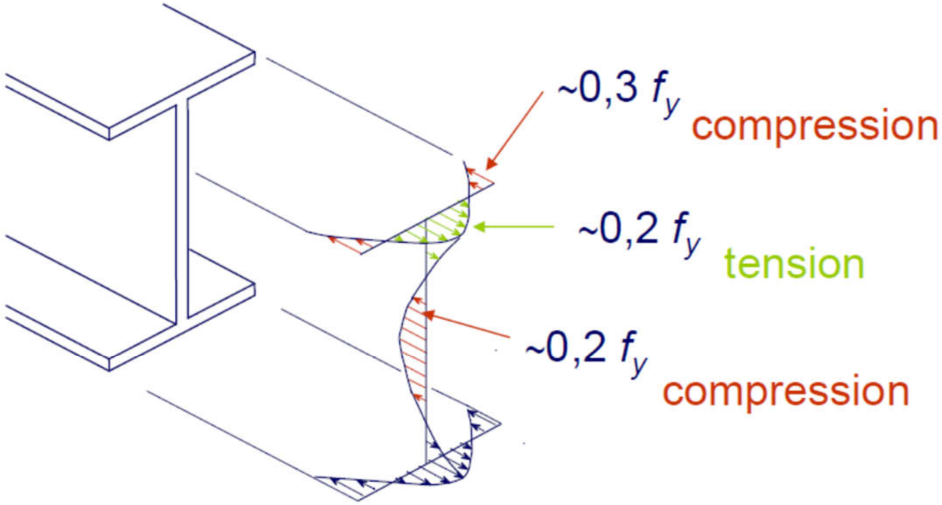
Section classification

Element 	Class 1	Class 2	Class 3
Flange	$c/t_f \leq 9\varepsilon$	$c/t_f \leq 10\varepsilon$	$c/t_f \leq 14\varepsilon$
Web subject to bending	$d/t_W \leq 72\varepsilon$	$d/t_W \leq 83\varepsilon$	$d/t_W \leq 124\varepsilon$
Web subject to compression	$d/t_W \leq 33\varepsilon$	$d/t_W \leq 38\varepsilon$	$d/t_W \leq 42\varepsilon$

Evolution of the direct stress distribution

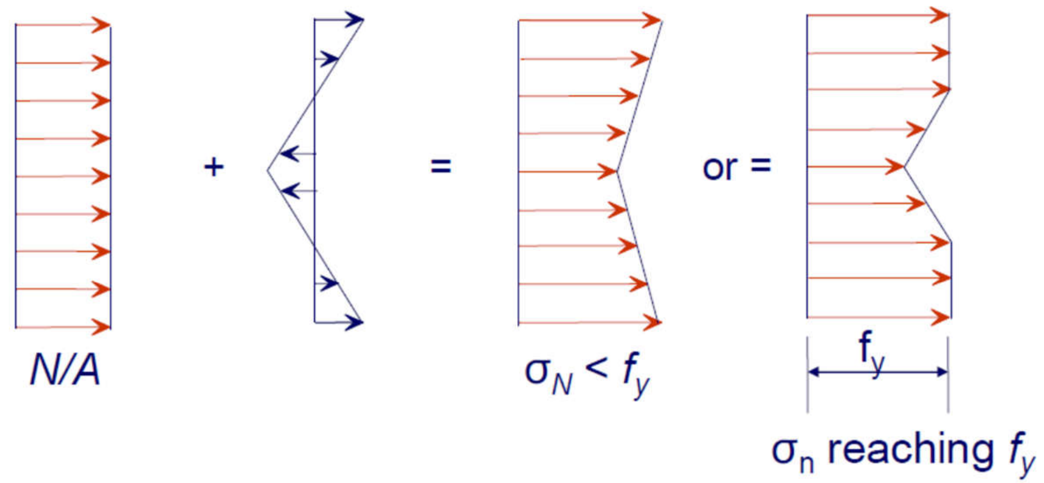


Residual stresses



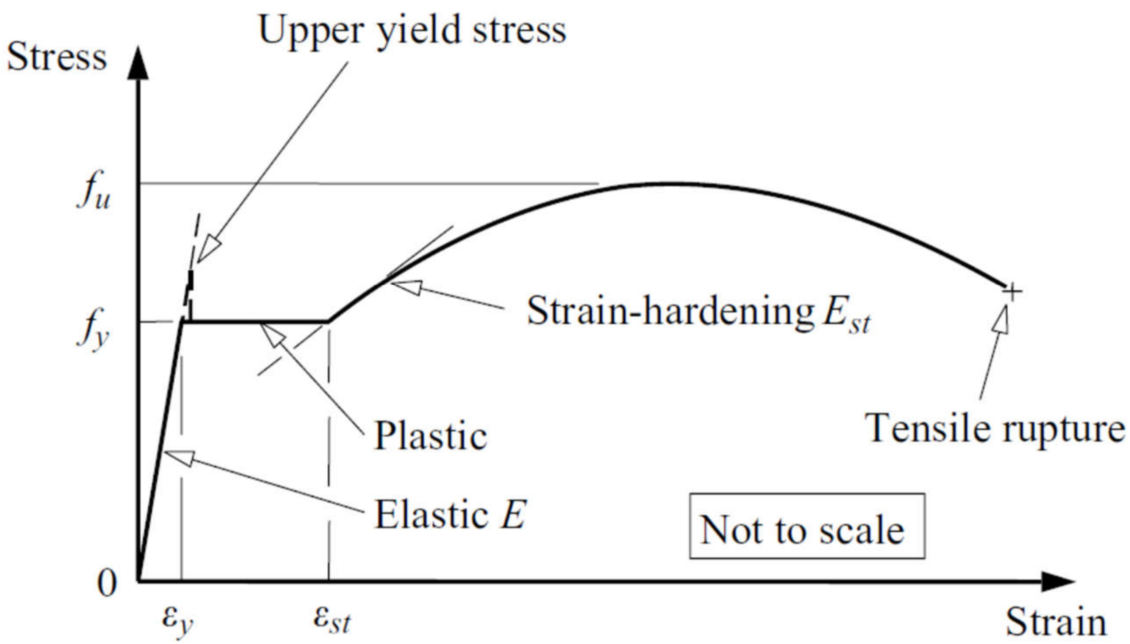
- Due to differential cooling during hot-rolling or welding.
- Above distribution is typical for a hot-rolled section.
- Peak residual stresses are larger (approaching f_y) for welded sections.

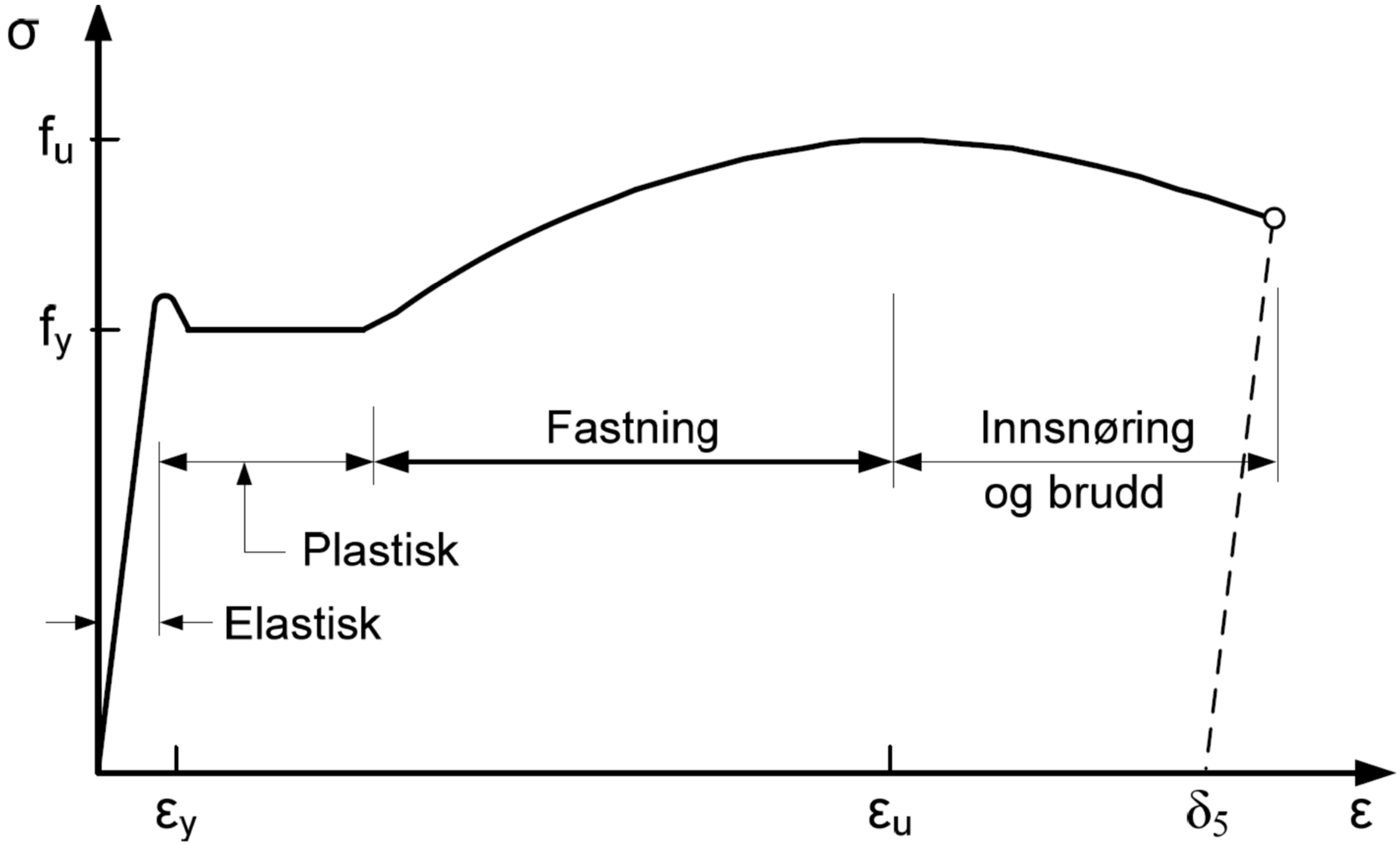
Effect of residual stresses



- Cause early yielding when combined with axial stresses.
- Reduces the flexural stiffness.

Idealised stress-strain behaviour




σ
f_u
f_y
Fastning
Innsnøring
og brudd
Plastisk
Elastisk
ε_y
ε_u
δ_5
ε

Moment resistance

The design value of the bending moment M_{Ed} at each cross-section shall satisfy

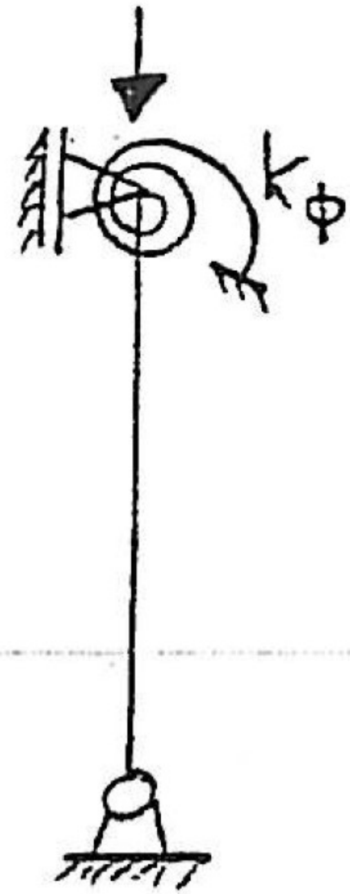
$$M_{Ed} \leq M_{c,Rd} = \begin{array}{ll} M_{pl,Rd} = \dfrac{W_{pl} f_y}{\gamma_{M0}} & \text{Class 1 and 2} \\[2ex] M_{el,Rd} = \dfrac{W_{el,min} f_y}{\gamma_{M0}} & \text{Class 3} \\[2ex] M_{o,Rd} = \dfrac{W_{eff,min} f_y}{\gamma_{M0}} & \text{Class 4} \end{array}$$

TBYG3018 Design of Offshore Structures

Module 4 - Design of offshore structures
according to NORSOK and Eurocode

Jomar Tørset, Assistant professor

Norwegian University of Science and Technology



STØRRE k_ϕ

$\Downarrow$

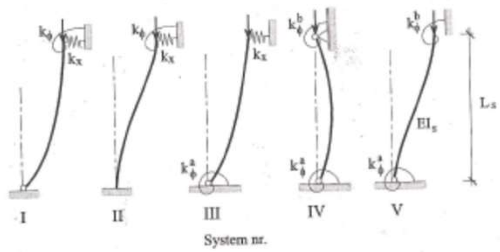
STØRRE GRAD
AV INNSPENNIN

$\Downarrow$

REDUSERT
KNEKKTENDENS

TABELL 4.1 SYSTEMDEFINISJON AV ELASTISK INNSPENTE STAVER

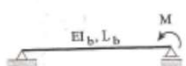
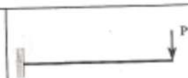
Basissystemer:



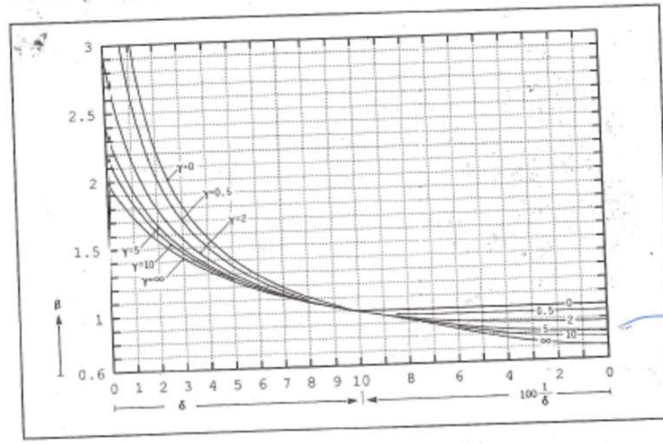
Dimensjonsløse parametre: $\delta = \dfrac{k_x L_s^3}{E I_s}$ $\qquad \gamma = \dfrac{k_\phi L_s}{E I_s}$

(For k_ϕ og γ: Indeks a og b indikerer stavenden)

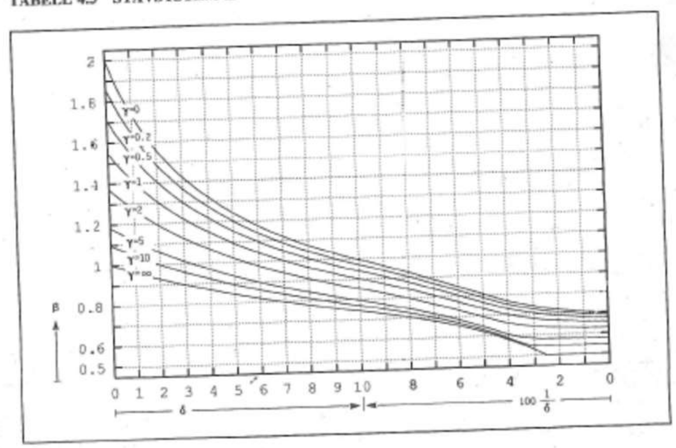
Elastisk knekklast (Eulerlast): $N_E = \dfrac{\pi^2 E I_s}{L_k^2} = \dfrac{\pi^2 E I_s}{(\beta L_s)^2}$

 $k_\phi = \dfrac{3 EI_b}{L_b}$	 $k_\phi = \dfrac{6 EI_b}{L_b}$	 $k_x = \dfrac{3 EI_b}{L_b^3}$
 $k_\gamma = \dfrac{2 EI}{L_b}$	 $k_\phi = \dfrac{4 EI}{L_b}$	 $k_\phi = \dfrac{GI_T}{L_b}$

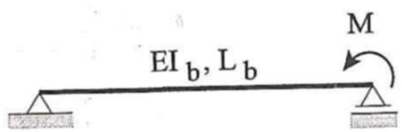
TABELL 4.2 STAVSYSTEM I OG III



TABELL 4.3 STAVSYSTEM II

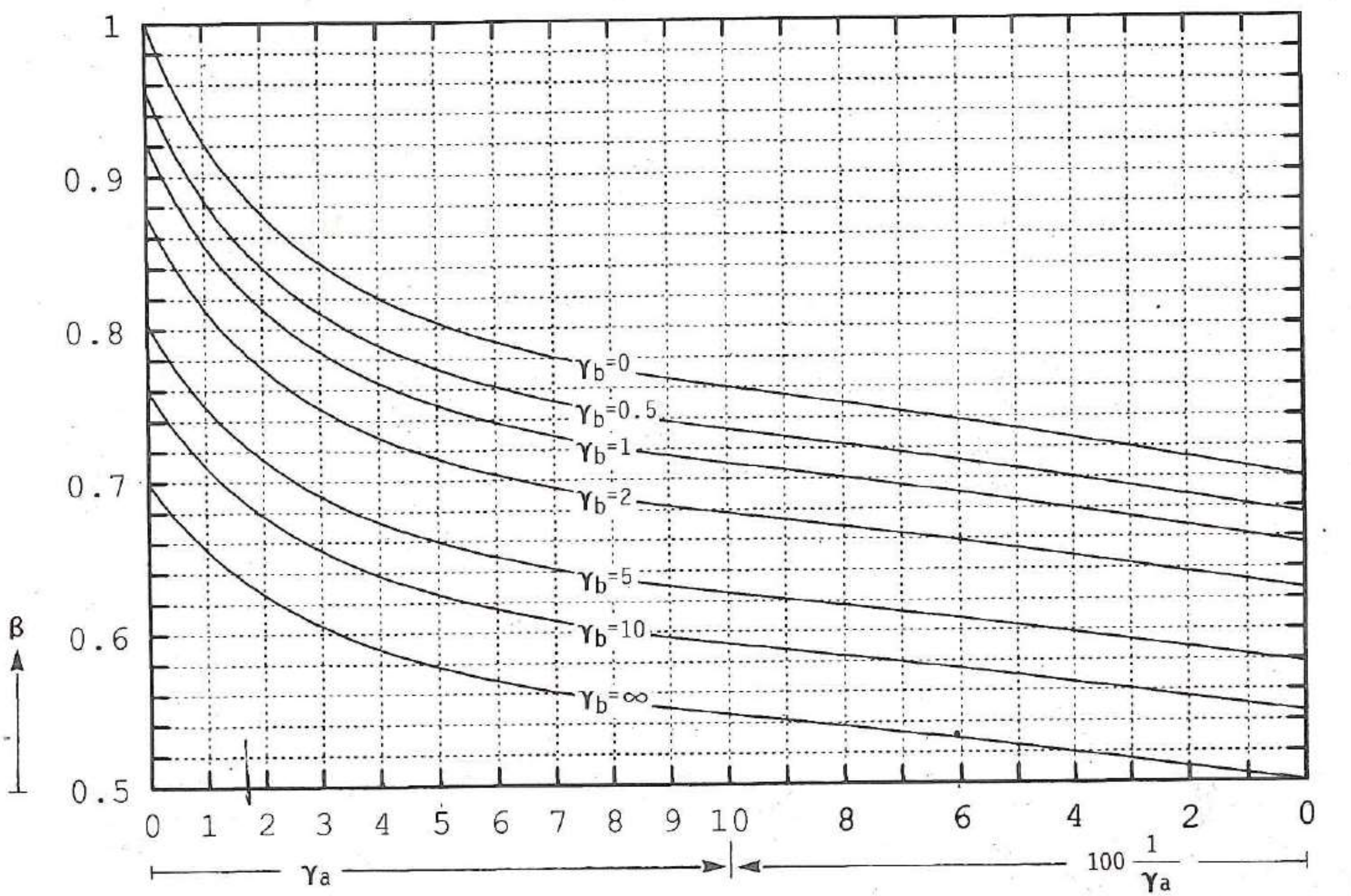


Elastisk knekklast (Eulerlast): $N_E = \dfrac{\pi^2 E I_s}{L_k^2} = \dfrac{\pi^2 E I_s}{(\beta L_s)^2}$

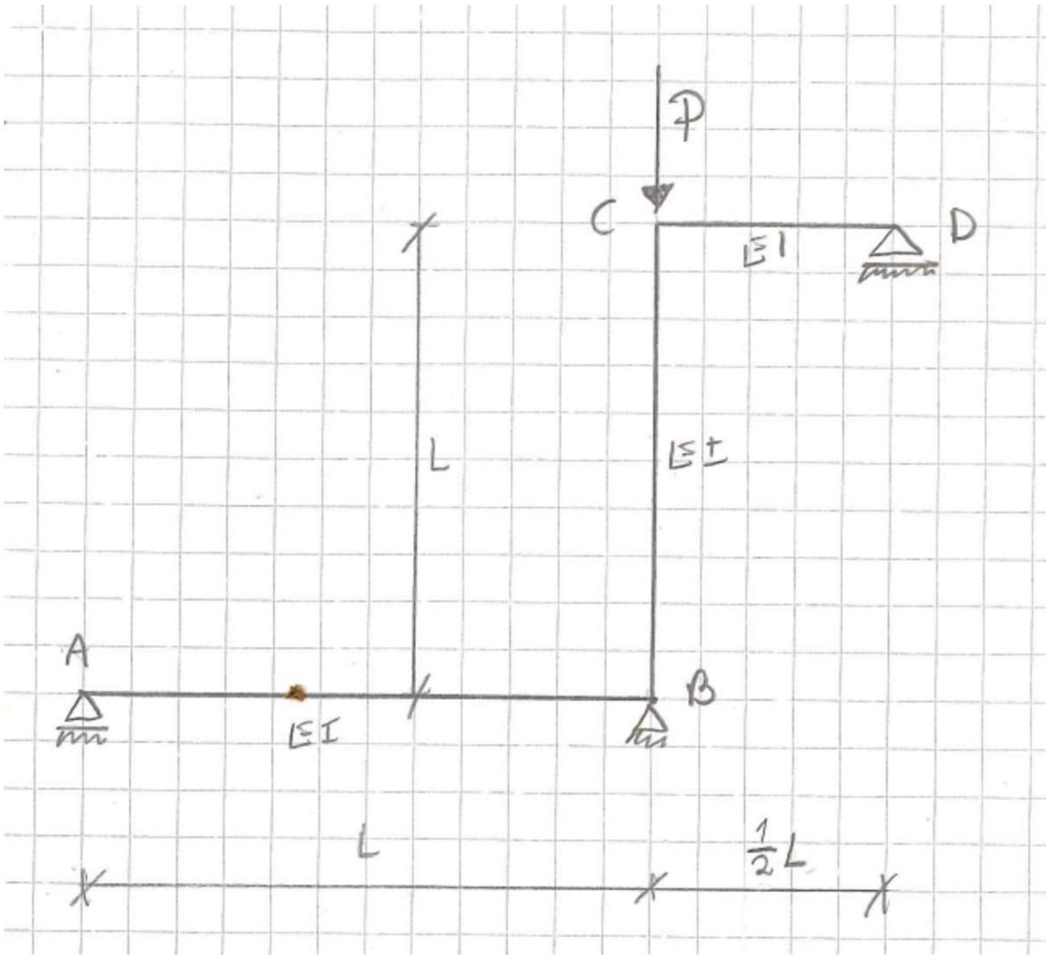
		
$k_\phi = \dfrac{3\,EI_b}{L_b}$	$k_\phi = \dfrac{6\,EI_b}{L_b}$	$k_x = \dfrac{3\,EI_b}{L_b^3}$
		
$k_r = \dfrac{2\,EI}{L_b}$	$k_\phi = \dfrac{4\,EI}{L_b}$	$k_\phi = \dfrac{GI_T}{L_b}$

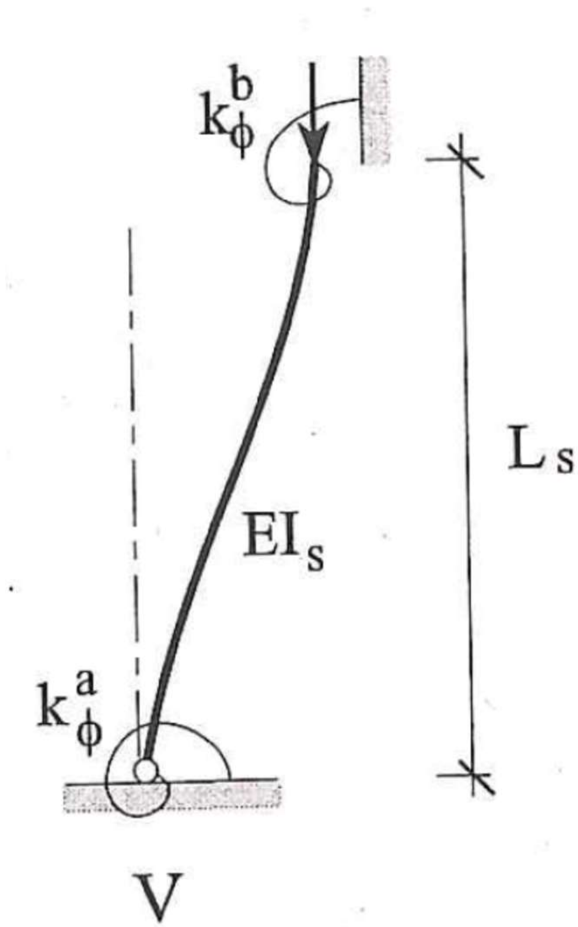
Dimensjonsløse parametre: $\delta = \frac{k_x L_s^3}{E I_s}$ $\gamma = \frac{k_\phi L_s}{E I_s}$

(For k_ϕ og γ: Indeks a og b indikerer stavenden)



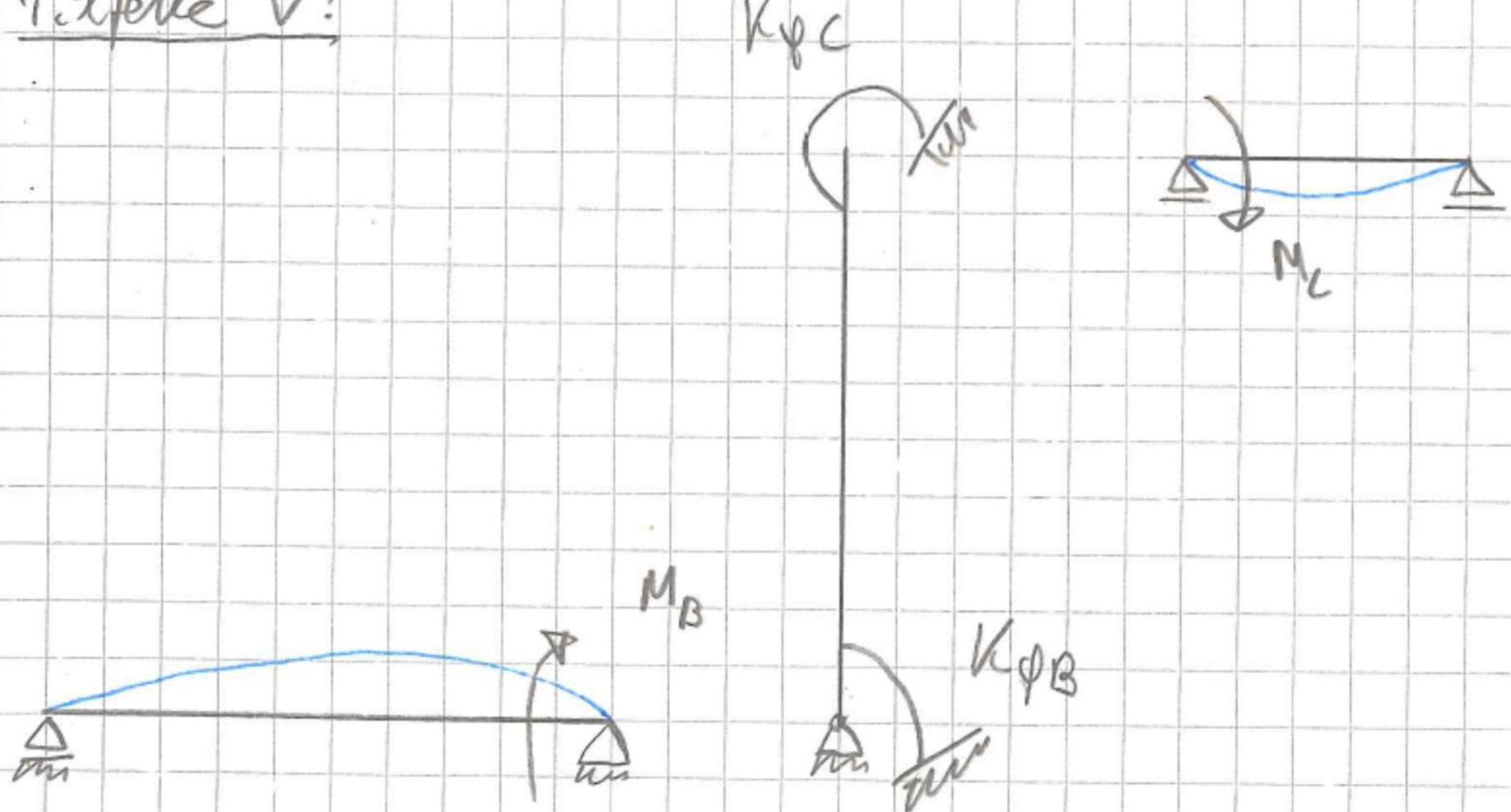
1
0.9
0.8
0.7
0.6
0.5
β
γb=0
γb=0.5
γb=1
γb=2
γb=5
γb=10
γb=∞
0 1 2 3 4 5 6 7 8 9 10
8 6 4 2 0
γa
100 1/γa

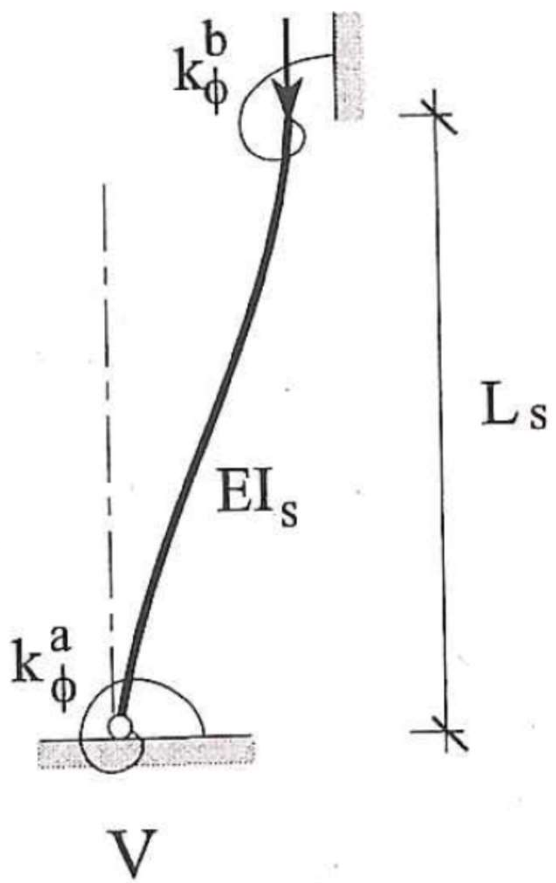

P
C
D
EI
L
EI
A
B
EI
L
1/2 L



k_ϕ^b
EI_s
L_s
k_ϕ^a
V

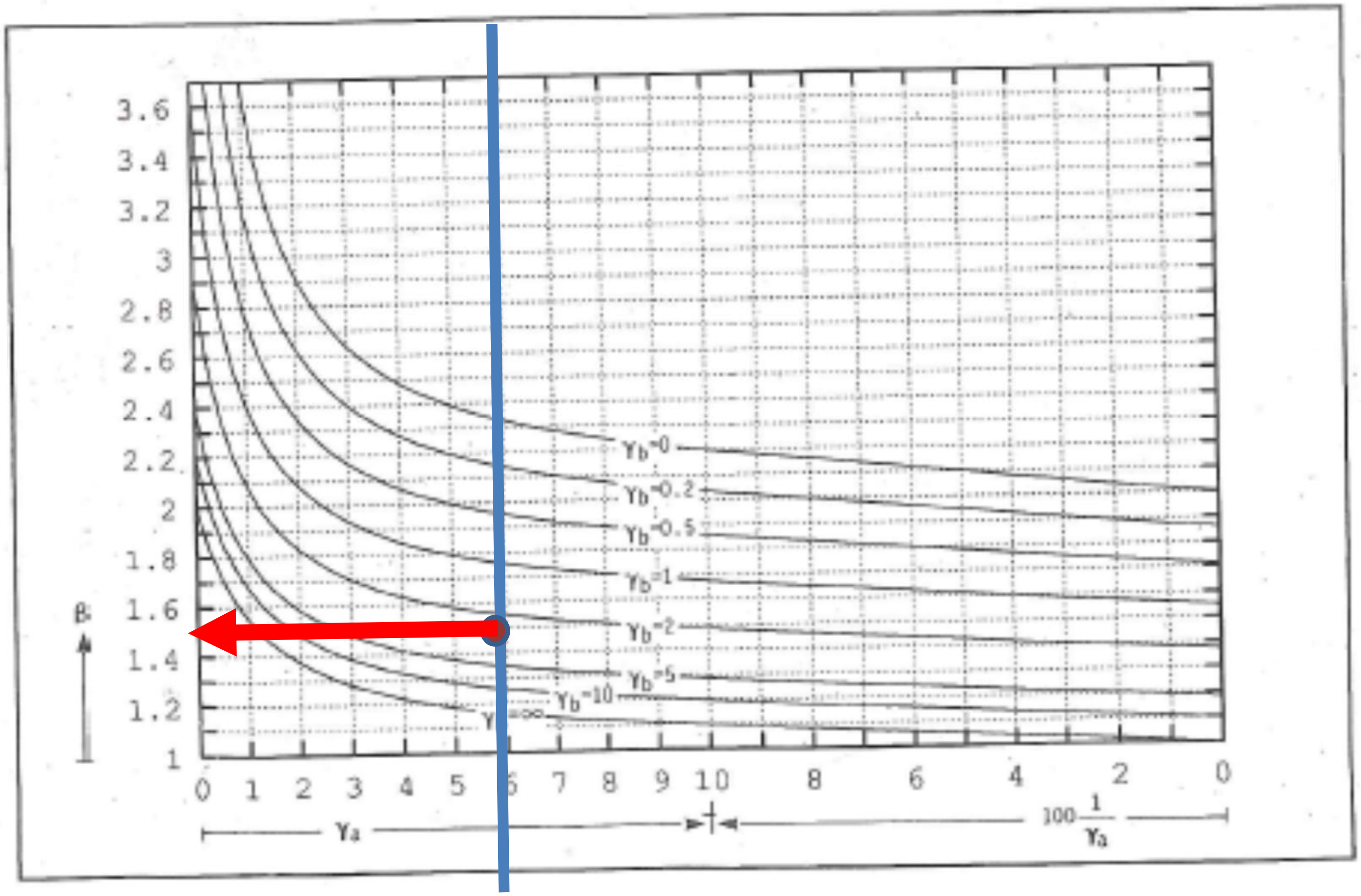
Tilfelle V:





$$\left.\begin{array}{l} C_b = \dfrac{3EI}{l/2} \Rightarrow \gamma_b = 6 \\[2em] C_a = \dfrac{3EI}{l} \Rightarrow \gamma_a = 3 \end{array}\right\} \Rightarrow \beta \simeq 1.4$$

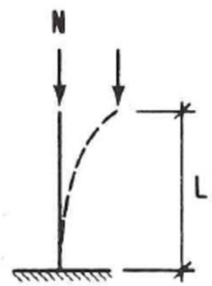
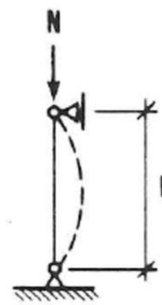
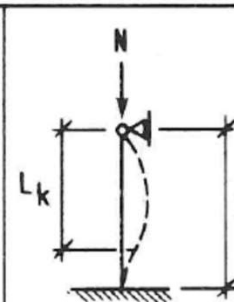
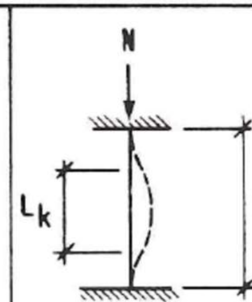
TABELL 4.5 STAVSYSTEM V

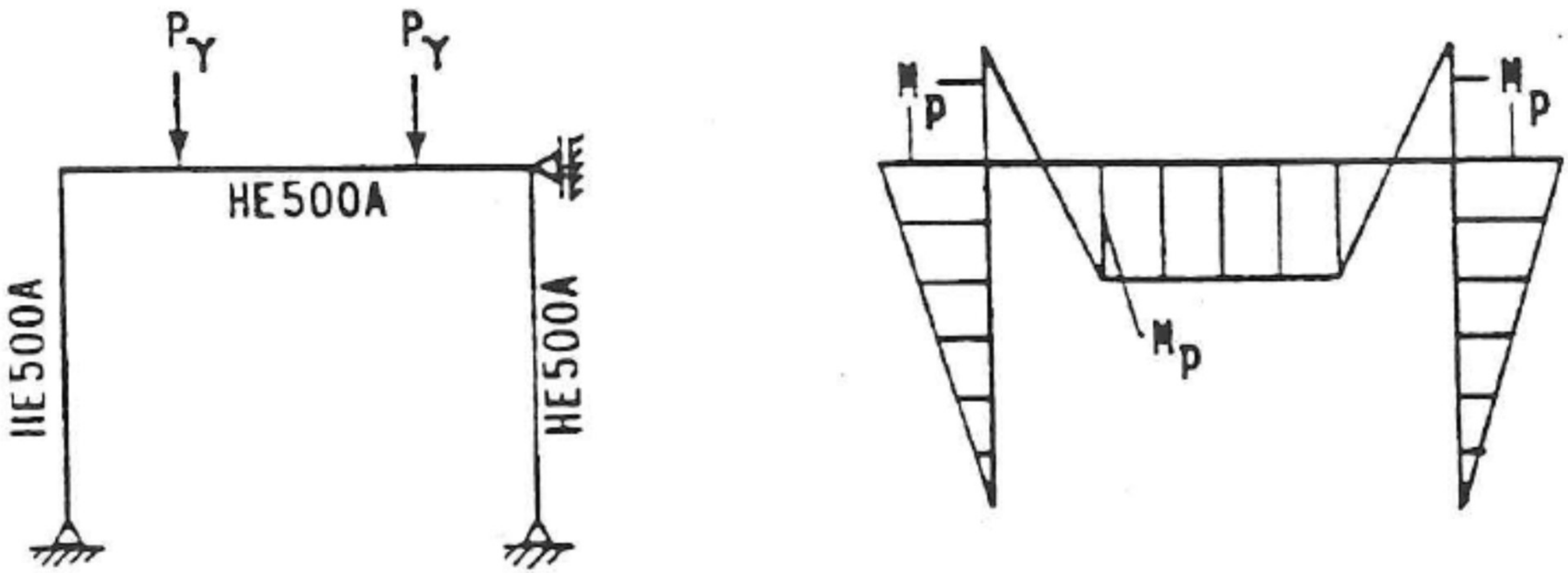


$$\beta := 1.5$$

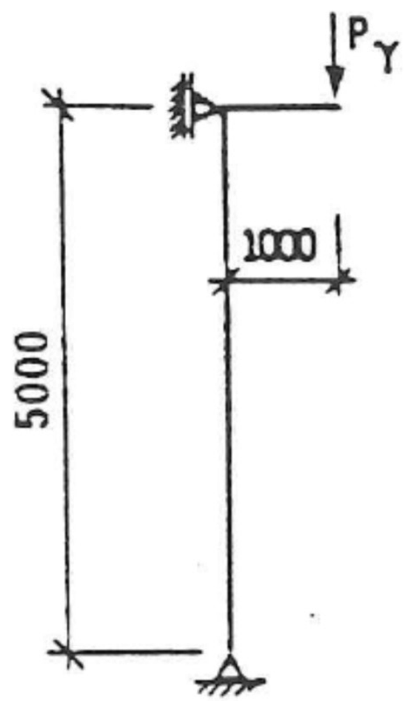
$$N_{kr} := \frac{\pi^2 \cdot E \cdot I}{(1.5 \cdot L)^2}$$

Tabell 6.1 - Basistilfeller for stavknekking

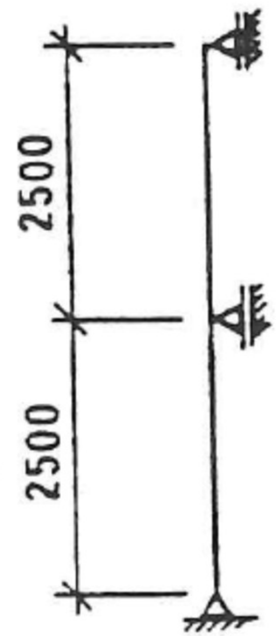
				
Knekkings–betingelse	$\cos kL = 0$	$\sin kL = 0$	$\frac{kL}{\operatorname{tg} kL} = 1$	$\cos kL = 1$
Laveste egenverdi	$kL = \frac{\pi}{2}$	$kL = \pi$	$kL = 4.493$	$kL = 2\pi$
Knekklengde	$L_k = 2.0\ L$	$L_k = L$	$L_k \approx 0.7\ L$	$L_k = 0.5\ L$



Figur 1



Figur 2a



Figur 2b

Lecture notes

TKT 4230 Steel structures 2

Buckling load for common plates

Ole Andre Øiseth

Department of Structural Engineering

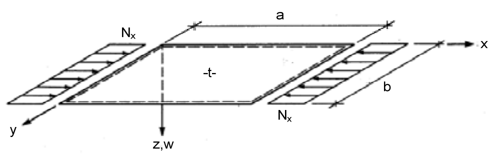
Faculty of Engineering Science and Technology

Norwegian University of Science and Technology

1

Normal stresses

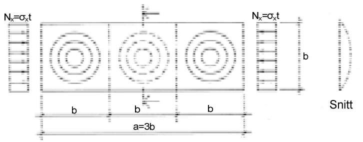
- Simply supported plate with uniform stress in x direction



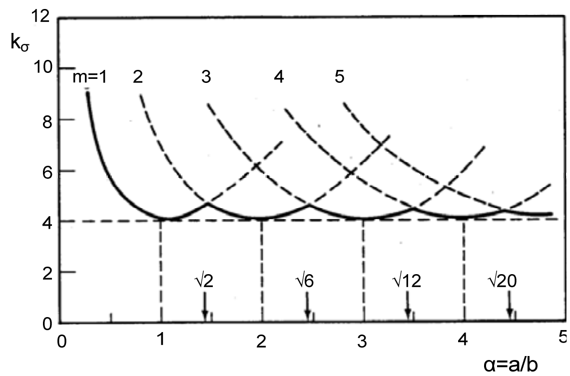
2

Normal stresses

- The buckling factor



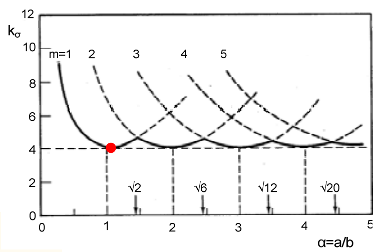
a) Knekkform for plate med a=3b under konstant aksialkraft

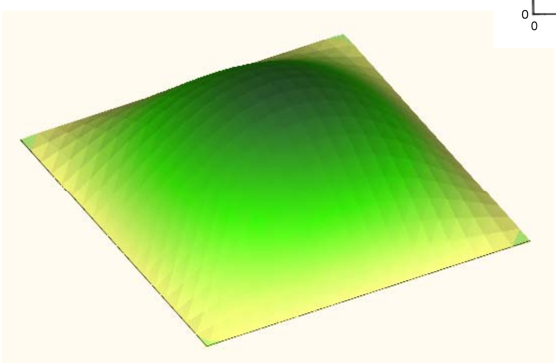


Normal stresses

$$\sigma_e = \frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

$\alpha = 1$

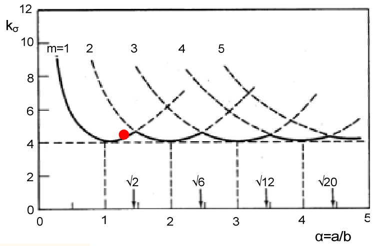


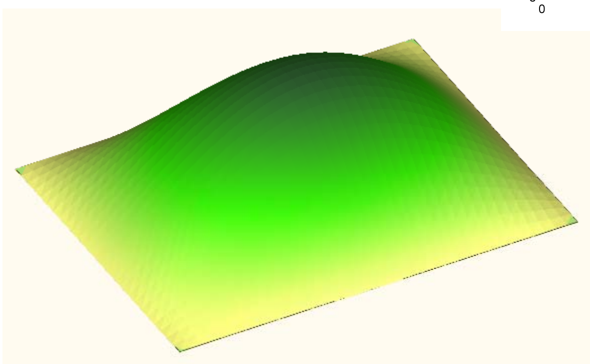


Normal stresses

$$\sigma_e = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

$$\alpha = 1.4$$



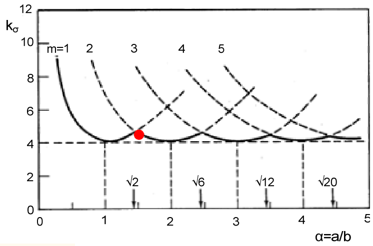


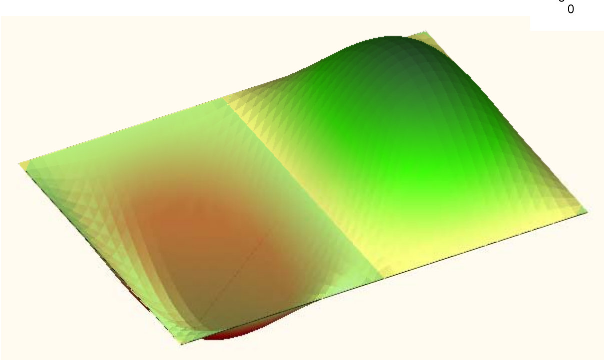
5

Normal stresses

$$\sigma_e = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

$$\alpha = 1.45$$



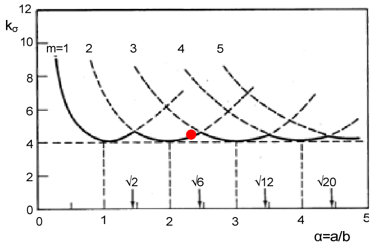


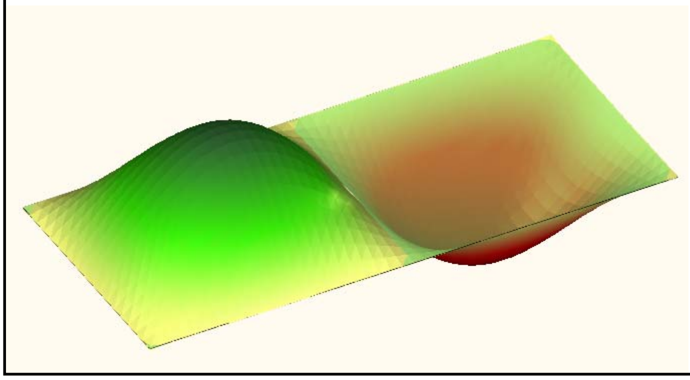
6

Normal stresses

$$\sigma_e = \frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

$\alpha = 2.4$



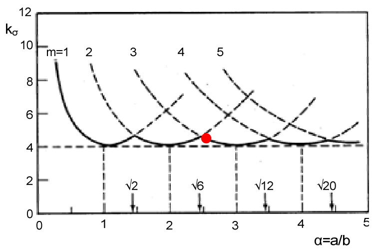


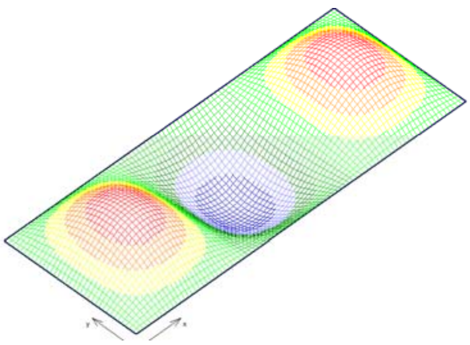
7

Normal stresses

$$\sigma_e = \frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

$\alpha = 2.5$



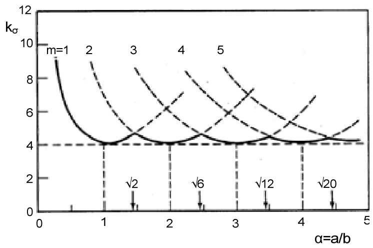


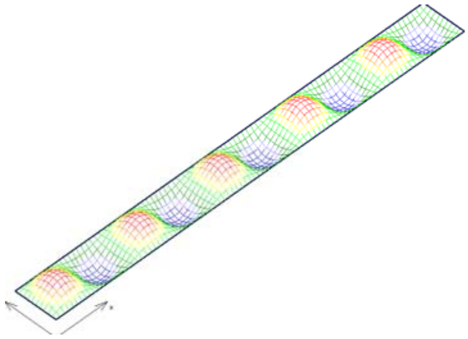
8

Normal stresses

$$\sigma_e = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

$\alpha = 10$

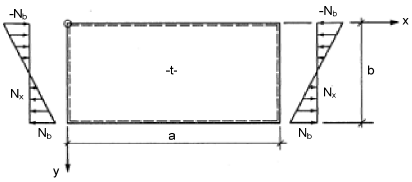




9

Normal stresses

- Simply supported plate with linear stress distribution

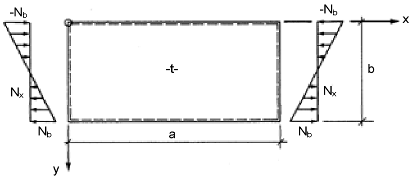


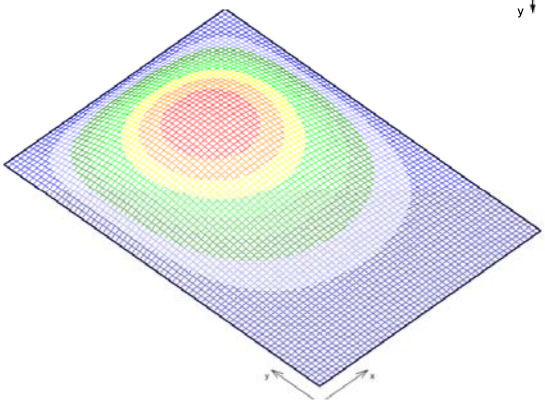
10

Normal stresses

$$\sigma_e = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

$\alpha = 0.7$



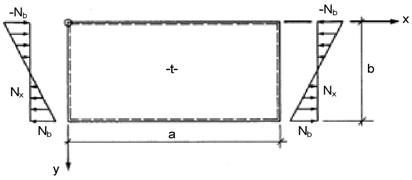


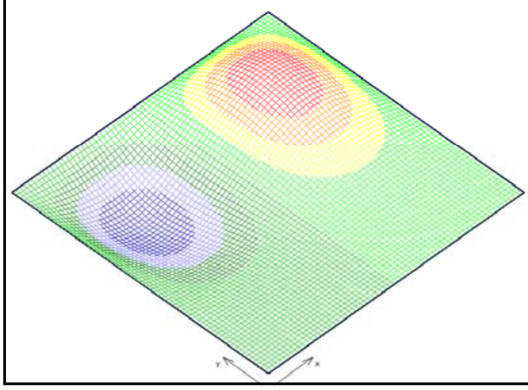
11

Normal stresses

$$\sigma_e = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

$\alpha = 1$

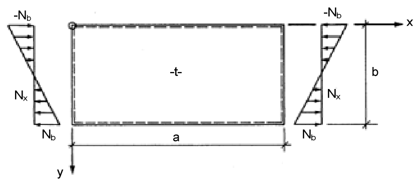


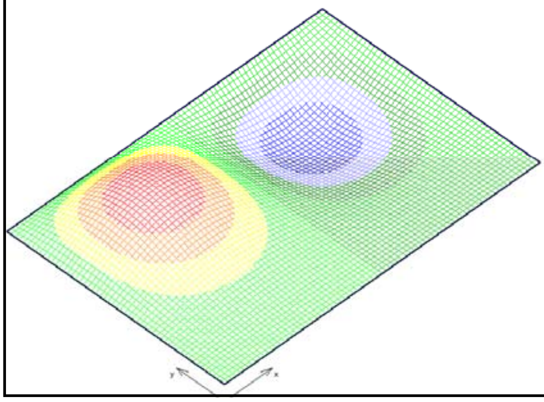


12

$$\sigma_e = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

$\alpha = 1.45$



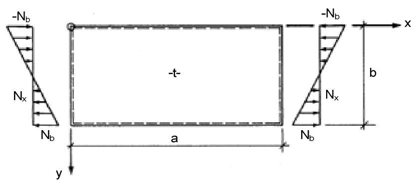


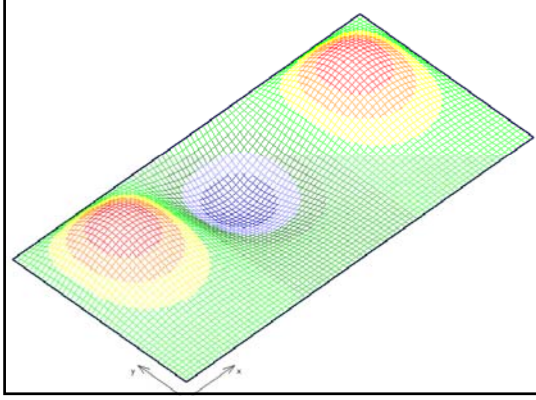
13

Normal stresses

$$\sigma_e = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

$\alpha = 2$



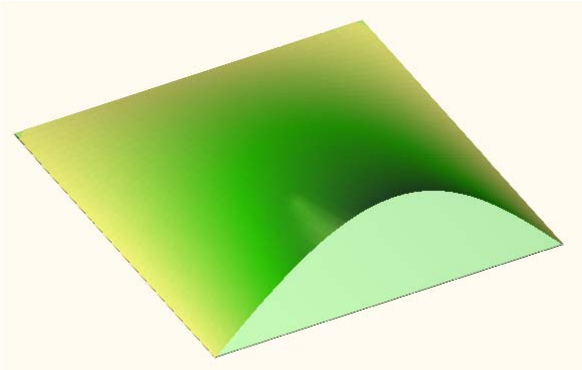


14

Normal stresses

$$\sigma_e = \frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

$\alpha = 1$

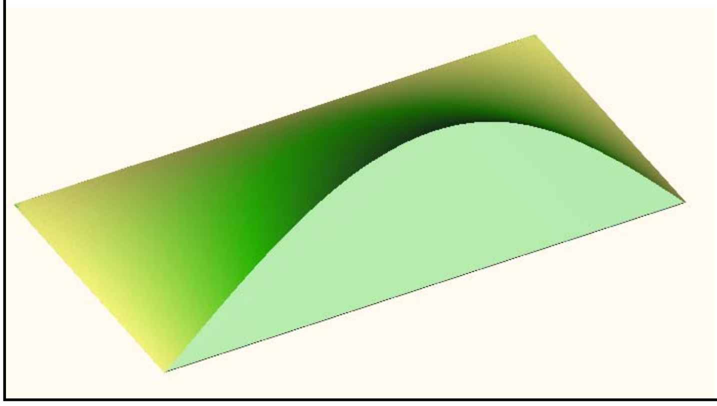


15

Normal stresses

$$\sigma_e = \frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

$\alpha = 2$

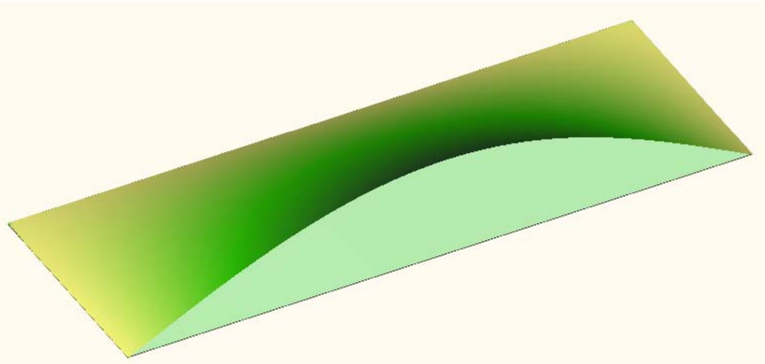


16

Normal stresses

$$\sigma_e = \frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

$\alpha = 3$

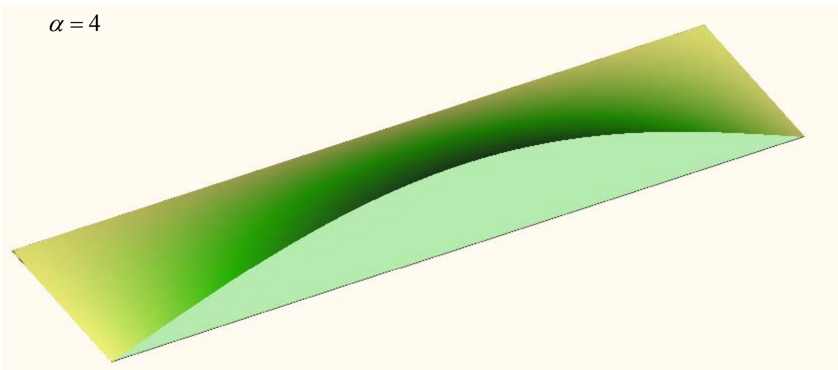


17

Normal stresses

$$\sigma_e = \frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

$\alpha = 4$

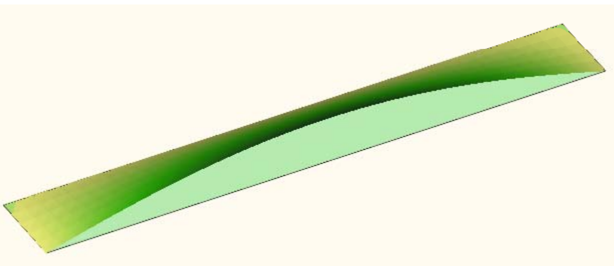


18

Normal stresses

$$\sigma_e = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2, \qquad \sigma_{cr} = k_\sigma \cdot \sigma_e$$

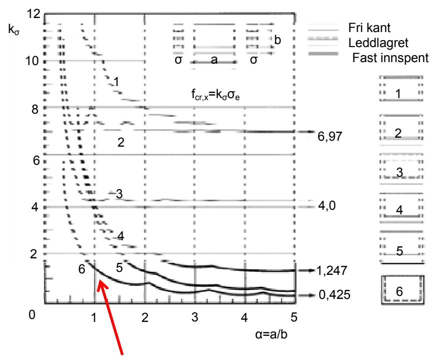
$$\alpha = 7.5$$



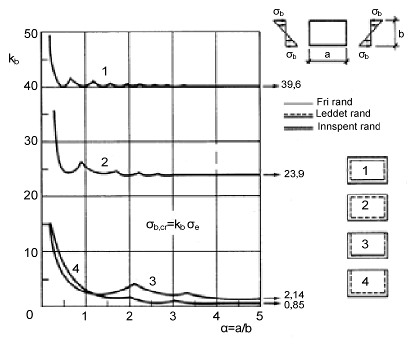
19

Normal stresses

- Summary
  - The asymptotes are used in design



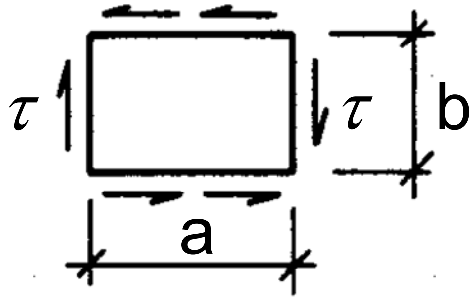
This curve is not correct!!



20

Shear

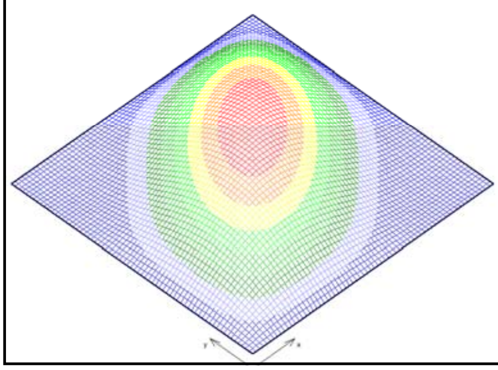
- Simply supported plate subjected to uniform shear

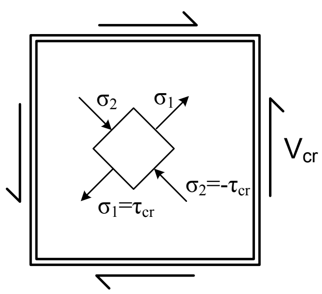


Shear

$$\sigma_e = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2, \qquad \tau_{cr} = k_\tau \cdot \sigma_e$$

$$\alpha = 1$$

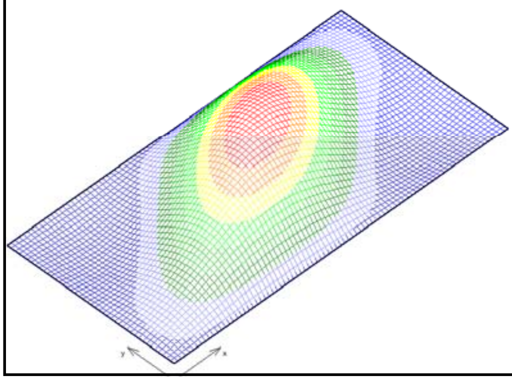


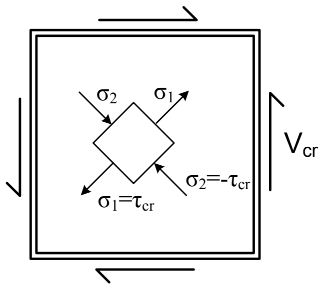


Shear

$$\sigma_e = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2, \qquad \tau_{cr} = k_\tau \cdot \sigma_e$$

$\alpha = 2$



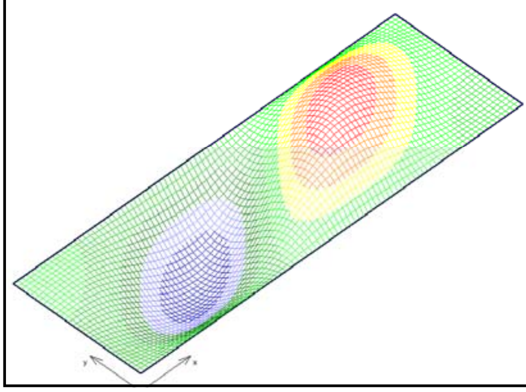


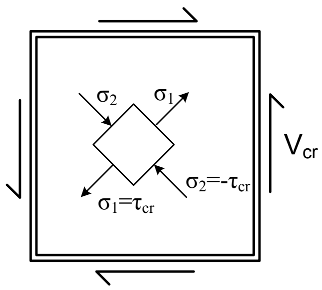
23

Shear

$$\sigma_e = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2, \qquad \tau_{cr} = k_\tau \cdot \sigma_e$$

$\alpha = 3$



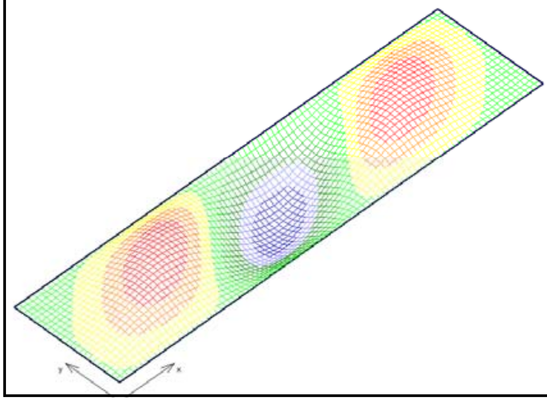


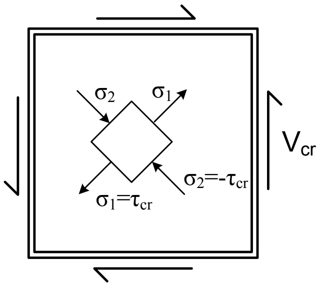
24

Shear

$$\sigma_e = \frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2, \qquad \tau_{cr} = k_\tau \cdot \sigma_e$$

$\alpha = 4$

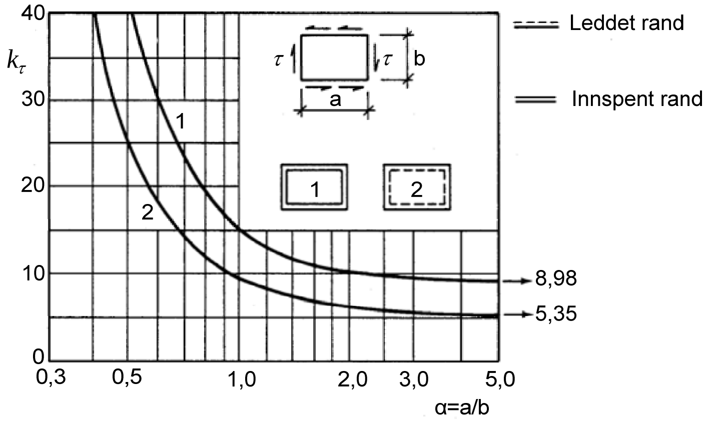




25

Shear

- Summary



26

TBYG3018 Design of Offshore Structures

Module 4 Design of offshore structures according til NORSOK and Eurocode

Jomar Tørset, Assistant professor

Norwegian University of Science and Technology

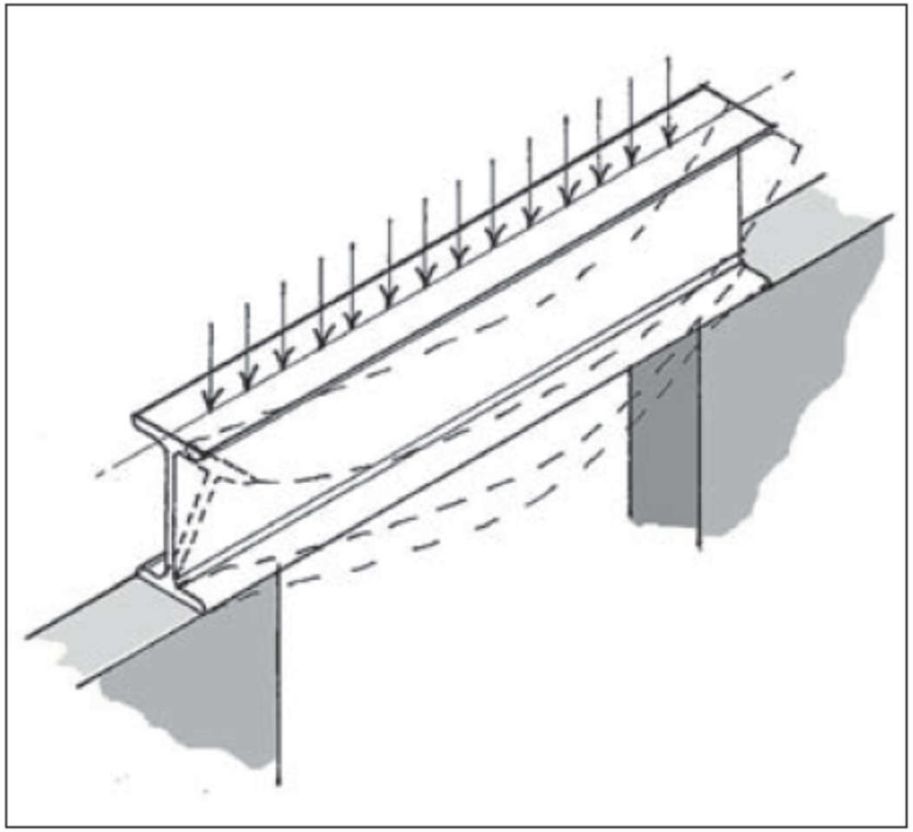
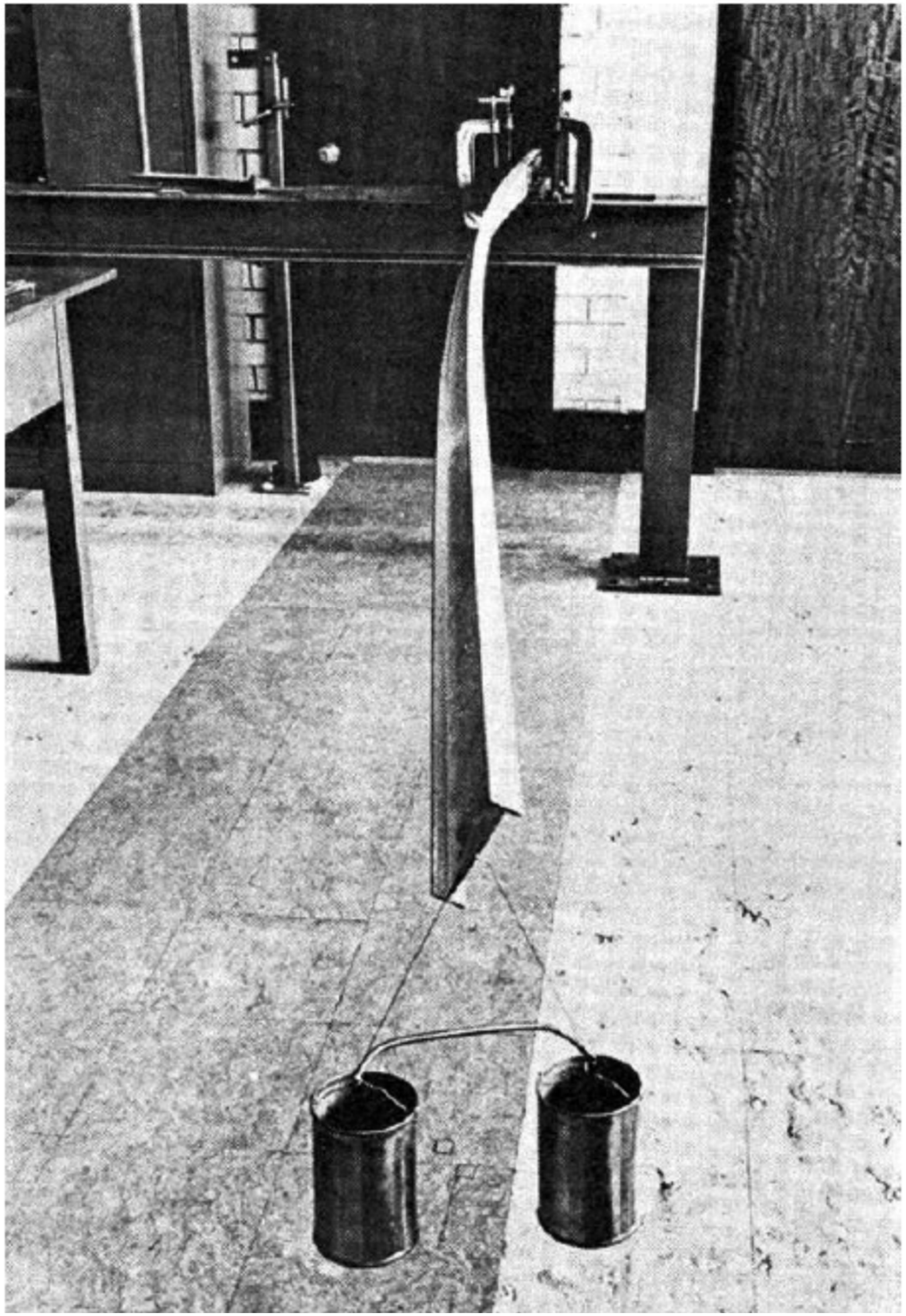
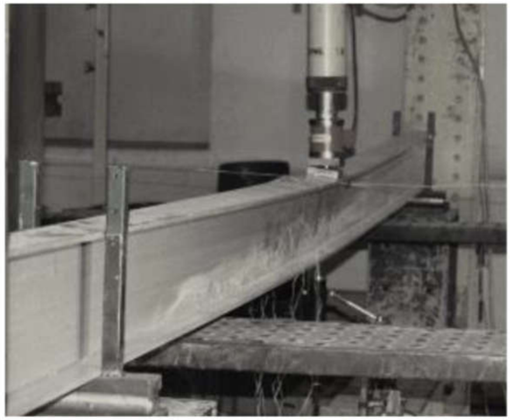


Figure 1
Lateral torsional buckling of an open section steel beam



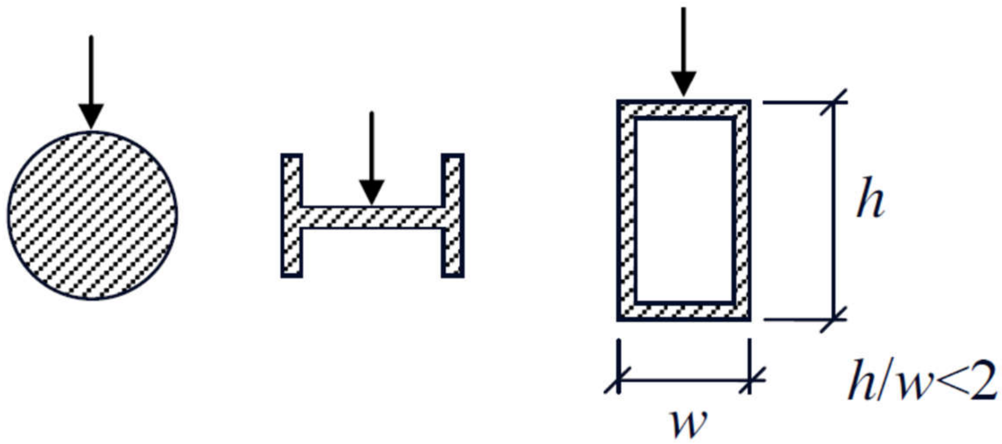
LTB sensitive beams

- Low flexural stiffness about minor axis (EI_z).
- Low torsional stiffness (GI_T).
- Low warping stiffness (EI_w).
- High point of load application.
- Long unrestrained spans.

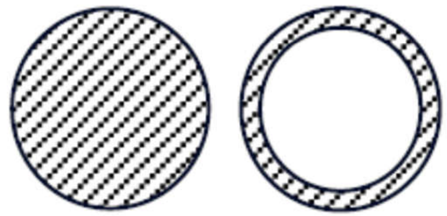
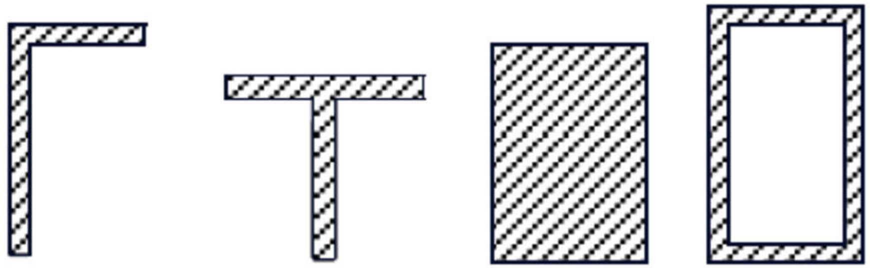
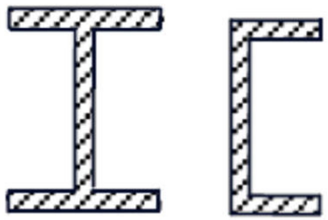


Influence of cross section

- Lateral torsional buckling is only possible in major axis bending.
- If the flexural stiffness is high enough about the weak axis or if the stiffness is equal about both axes, LTB will not occur.
- The figure show sections that are safe with regard to lateral torsional buckling.

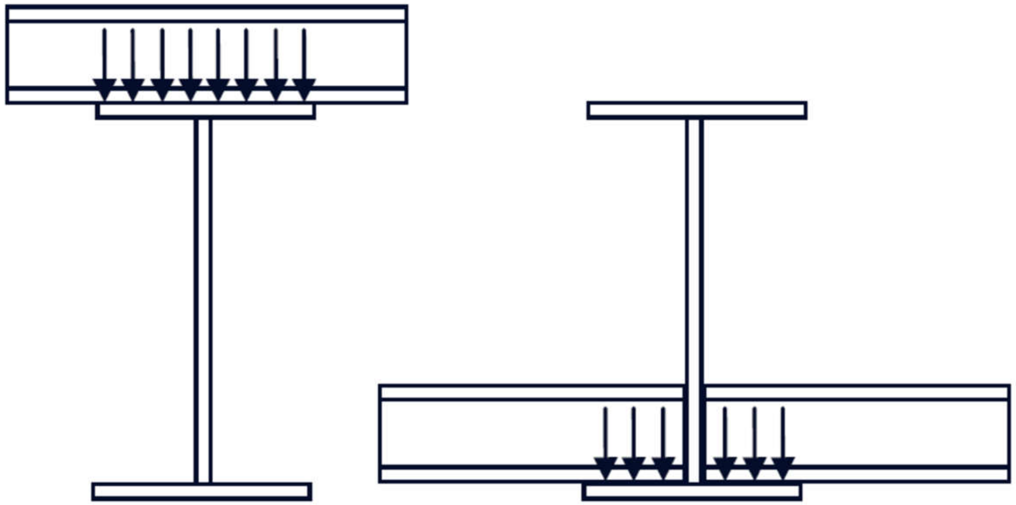


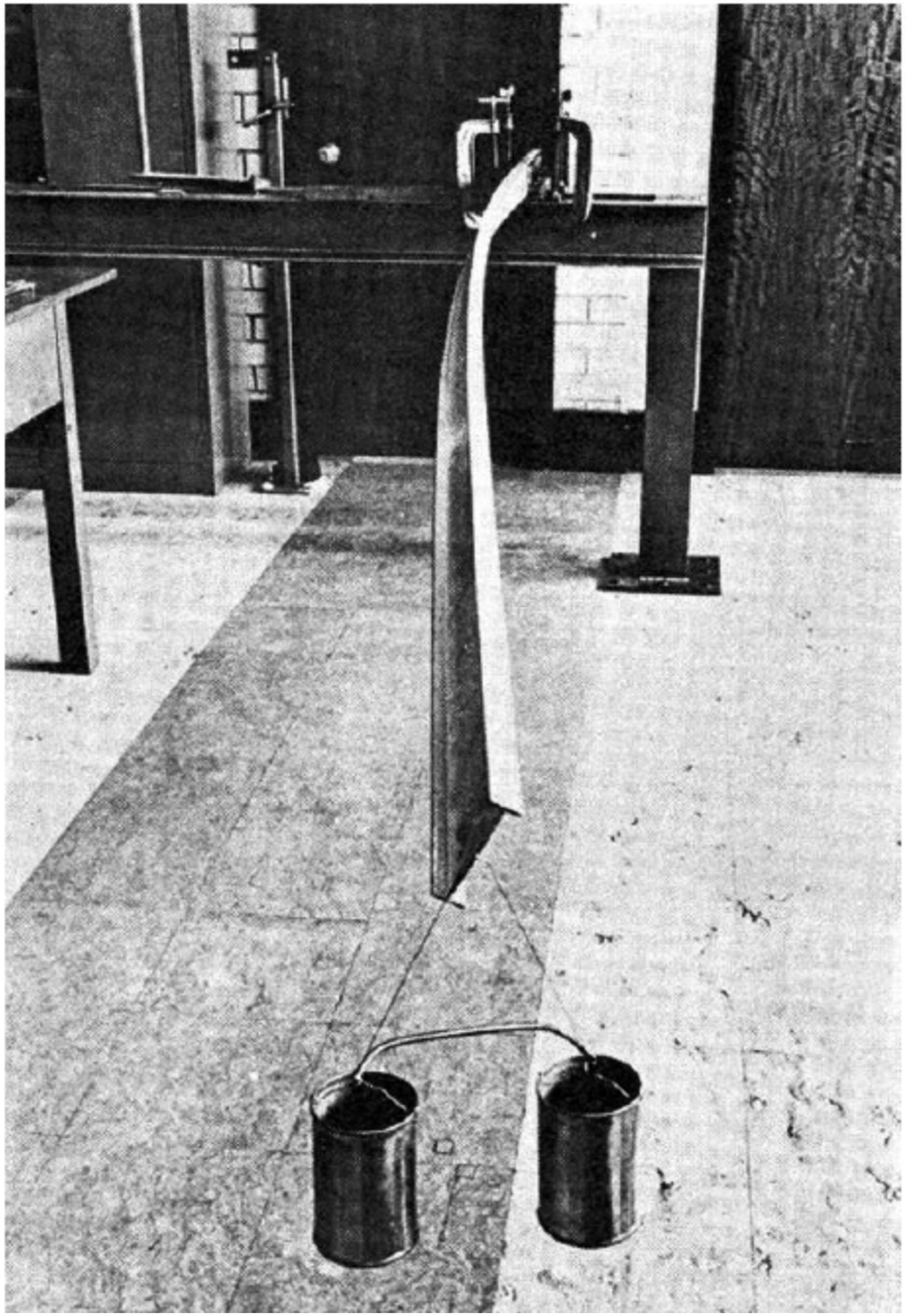
rping sensitivity of sections

(1) Warping free sections	
(2) Semi warping free sections	
(3) Warping sections	

Influence of point of load application

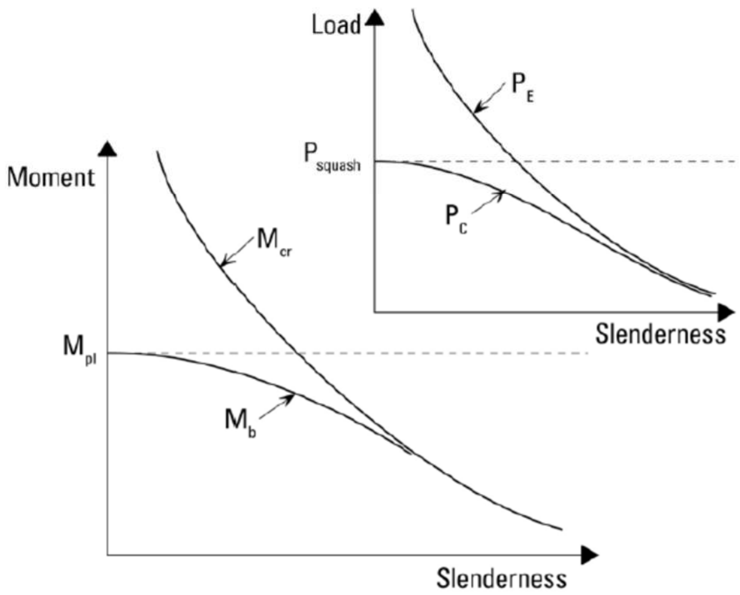
- If a low point of load application is used the load helps to stabilise the beam.
- A high point of load application contributes to twisting moments and makes the beam less stable.





Section slenderness

The elastic critical moment M_{cr} is used as basis for determining the slenderness (similar to the Euler load for flexural buckling).



3-factor formula

Expands the reference case to be valid for single-symmetric cross sections with arbitrary moment distributions by introducing correction factors C_1, C_2 and C_3.

$$M_{\mathrm{cr}} = \mu_{\mathrm{cr}} \frac{\pi\sqrt{EI_{\mathrm{z}}GI_{\mathrm{t}}}}{L}$$

where relative non-dimensional critical moment μ_{cr} is

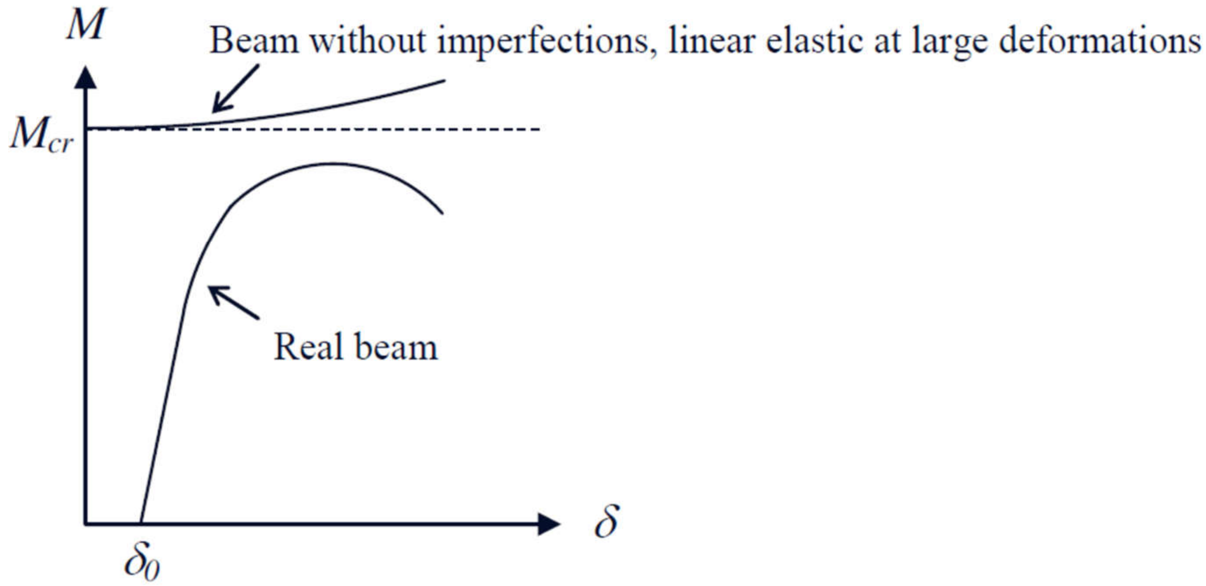
$$\mu_{\mathrm{cr}} = \frac{C_1}{k_{\mathrm{z}}}\left[\sqrt{1+\kappa_{\mathrm{wt}}^2+(C_2\zeta_{\mathrm{g}}-C_3\zeta_{\mathrm{j}})^2}-(C_2\zeta_{\mathrm{g}}-C_3\zeta_{\mathrm{j}})\right],$$

non-dimensional torsion parameter is $\kappa_{\mathrm{wt}} = \dfrac{\pi}{k_{\mathrm{w}}L}\sqrt{\dfrac{EI_{\mathrm{w}}}{GI_{\mathrm{t}}}}$

Factors that decrease the capacity of real beams

- Non-linear material response (gradual plastification).
- Initial out-of-straightness.
- Residual stresses from manufacturing.
- Local buckling of beam sections in class 4.
- Piercings, asymmetry and defects.

Factors that decrease the capacity of real beams



Reference case (NS-EN 1999-1-1 Annex I)

The elastic critical moment of a beam of uniform symmetrical cross section with equal flanges, under standard conditions of restraint at each end, subject to uniform moment in plane going through the shear centre is given by:

$$M_{cr} = \frac{\pi^2 EI_z}{L^2}\sqrt{\frac{L^2 GI_t}{\pi^2 EI_z} + \frac{I_w}{I_z}} = \frac{\pi\sqrt{EI_z GI_t}}{L}\sqrt{1 + \frac{\pi^2 EI_w}{L^2 GI_t}}$$

where:

$$G = \frac{E}{2(1+\nu)}$$

I_t is the torsion constant

I_w is the warping constant

I_z is the second moment of area about the minor axis

L is the length of the beam between points that have lateral restraint

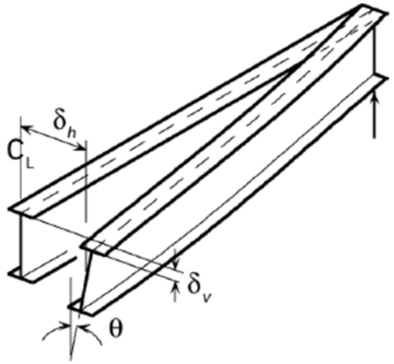
ν is the Poisson ratio

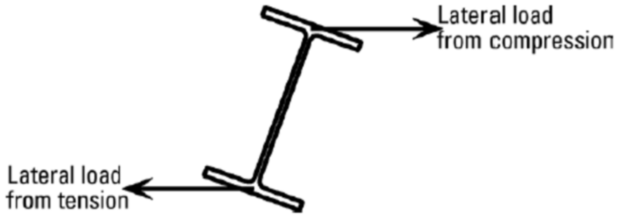
Correction factors

- k_z is related to restraint against lateral bending.
- k_w is related to restraint against warping.
- ζ_g is related to point of load application.
- ζ_j is related to section symmetry.
- C_1 account for the shape of the moment diagram.
- C_2 account for the point of load application.
- C_3 account for the asymmetry about the y-axis.

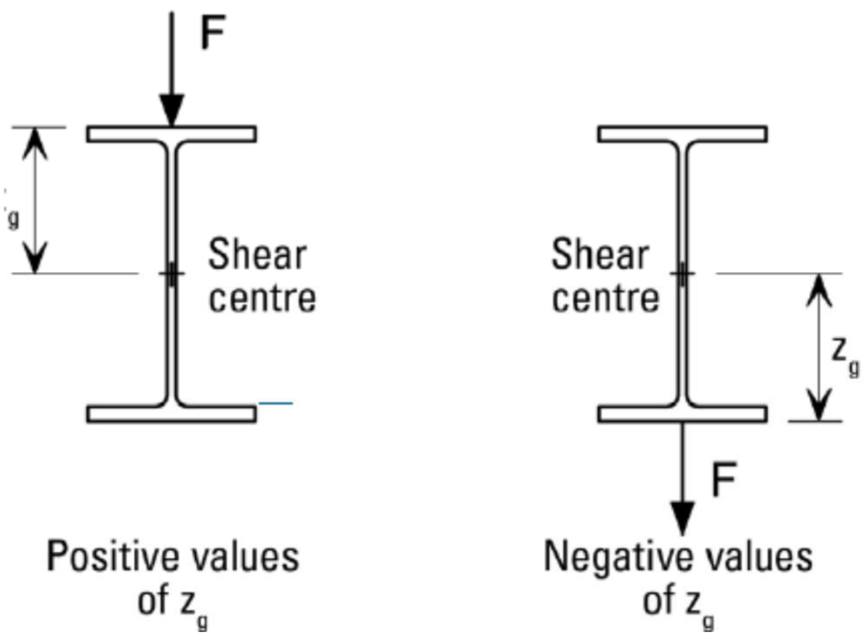
What is lateral torsional buckling?

- LTB may occur in an unrestrained beam, i.e when its compression flange is free to displace laterally and rotate.
- The compression flange tries to buckle laterally, whereas the tension flange tries to keep the member straight.
- This causes both lateral displacement and twisting of the member, i.e. lateral torsional buckling.



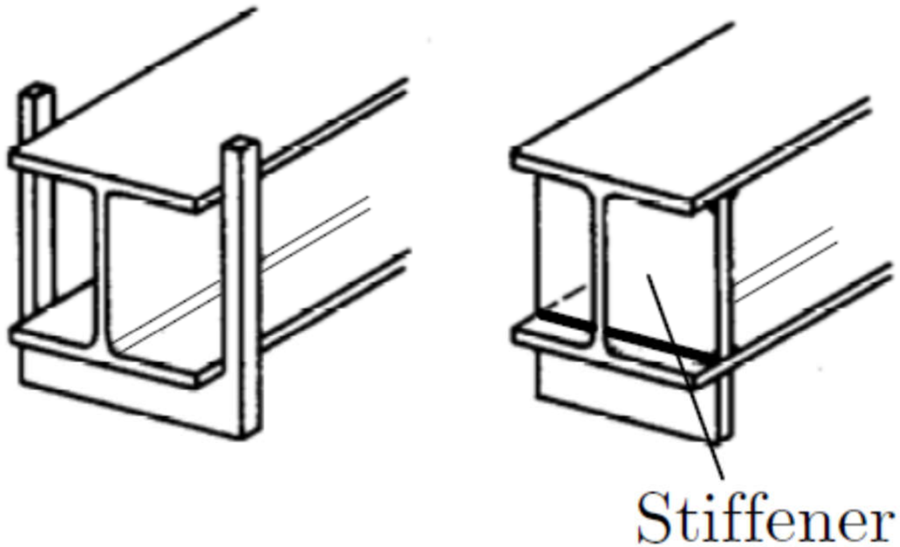


- **Location of the applied load:** load applied above the shear centre destabilises the beam; load applied below the shear centre has a stabilising effect on the beam.
- **End support conditions:** more restraint than fork support increases the resistance, less restraint has the opposite effect.
- **Shape of the moment diagram:** uniform bending moment distribution result in the smallest resistance.



A fork support have the following boundary conditions:

- Translation in x, y, and z (fixed)
- Rotation about x-axis (fixed)
- Rotation about y-axis (free)
- Rotation about z-axis (free)
- Warping (free)



When considering ordinary beams on two supports, translation in y- and

TBYG3018 Design of Offshore Structures

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according to NORSOK and Eurocode

Jomar Tørset, Assistant professor

Norwegian University of Science and Technology

STØRRE kφ

⇓

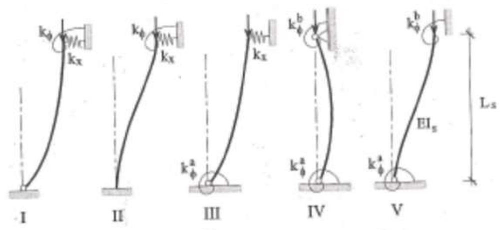
STØRRE GRAD
AV INNSPENNIN

⇓

REDUSERT
KNEKKTENDENS

TABELL 4.1 SYSTEMDEFINISJON AV ELASTISK INNSPENTE STAVER

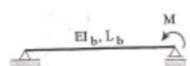
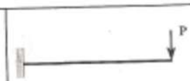
Basissystemer:



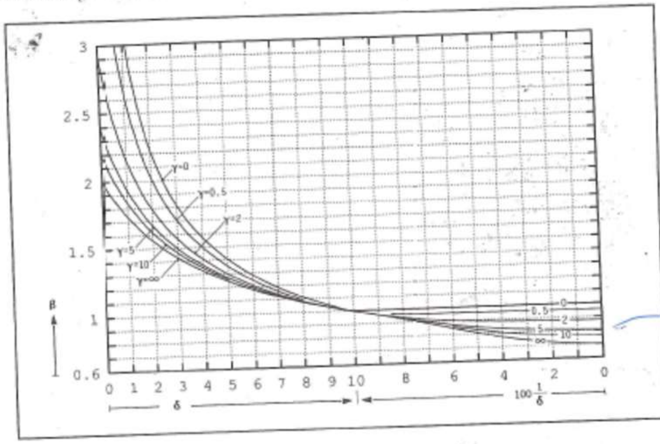
Dimensjonsløse parametre: $\delta = \frac{k_x L_s^3}{EI_s}$ $\gamma = \frac{k_\phi L_s}{EI_s}$

(For k_ϕ og γ: Indeks a og b indikerer stavenden)

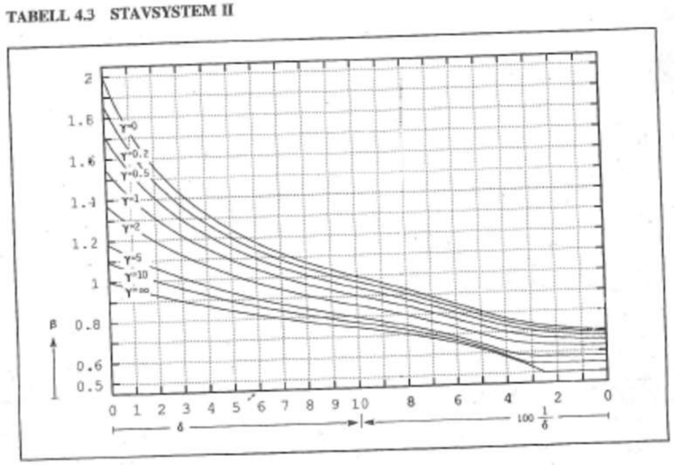
Elastisk knekklast (Eulerlast): $N_E = \frac{\pi^2 EI_s}{L_k^2} = \frac{\pi^2 EI_s}{(\beta L_s)^2}$

 $k_\phi = \frac{3EI_b}{L_b}$	 $k_\phi = \frac{6EI_b}{L_b}$	 $k_x = \frac{3EI_b}{L_b^3}$
 $k_\gamma = \frac{2EI}{L_b}$	 $k_\phi = \frac{4EI}{L_b}$	 $k_\phi = \frac{GI_T}{L_b}$

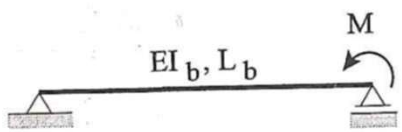
TABELL 4.2 STAVSYSTEM I OG III



TABELL 4.3 STAVSYSTEM II

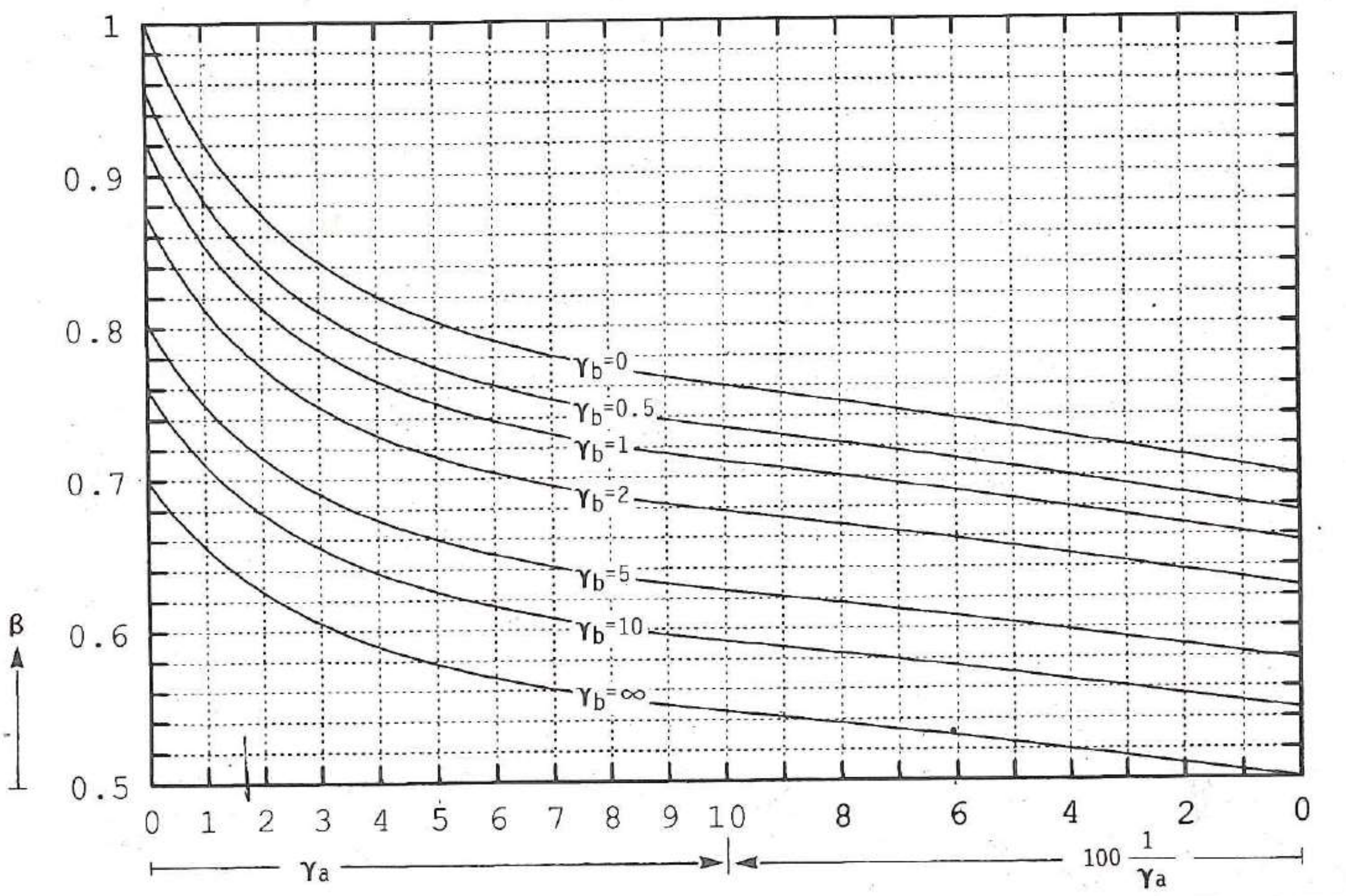


Elastisk knekklast (Eulerlast): $N_E = \frac{\pi^2 E I_s}{L_k^2} = \frac{\pi^2 E I_s}{(\beta L_s)^2}$

 $$k_{\phi} = \frac{3\,EI_b}{L_b}$$	 $$k_{\phi} = \frac{6\,EI_b}{L_b}$$	 $$k_x = \frac{3\,EI_b}{L_b^3}$$
 $$k_{r} = \frac{2\,EI}{L_b}$$	 $$k_{\phi} = \frac{4\,EI}{L_b}$$	 $$k_{\phi} = \frac{GI_T}{L_b}$$

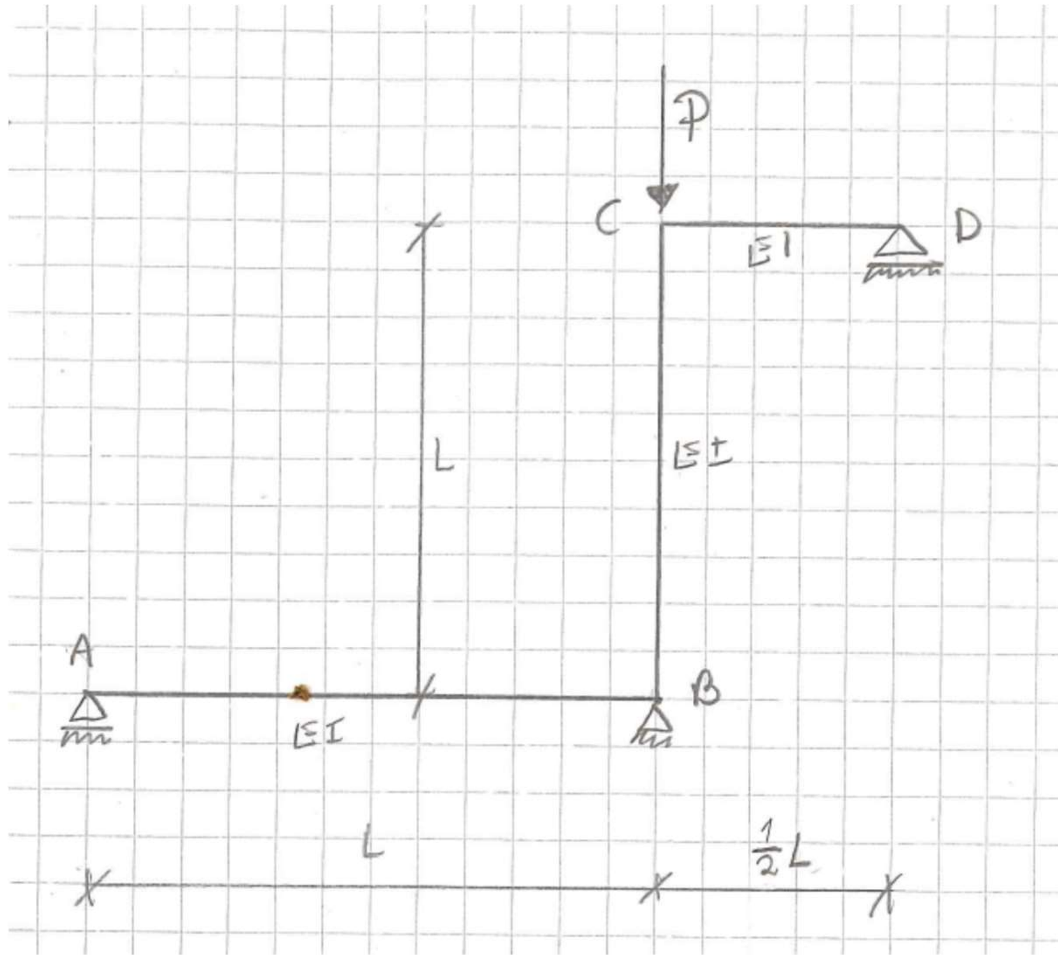
Dimensjonsløse parametre: $\delta = \frac{k_x L_s^3}{E I_s}$ $\gamma = \frac{k_\phi L_s}{E I_s}$

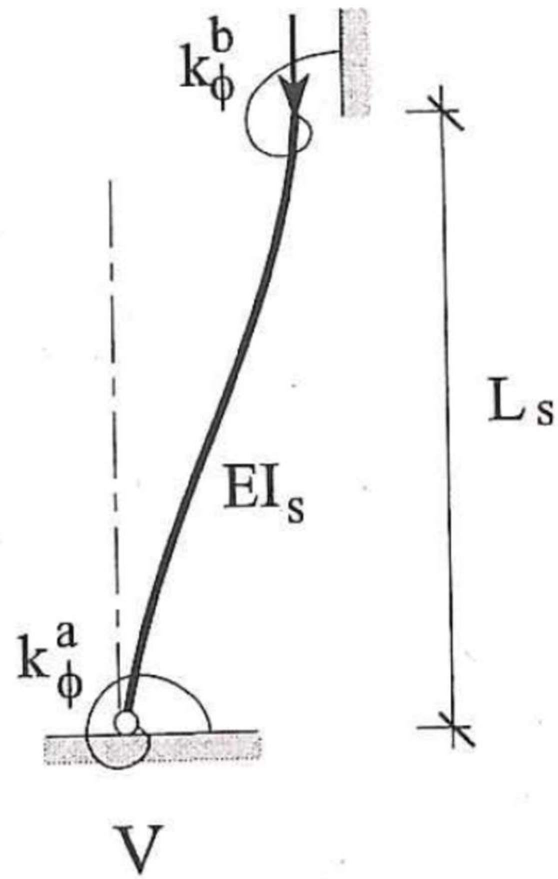
(For k_ϕ og γ: Indeks a og b indikerer stavenden)



1
0.9
0.8
0.7
0.6
0.5
β
γb=0
γb=0.5
γb=1
γb=2
γb=5
γb=10
γb=∞
0 1 2 3 4 5 6 7 8 9 10
8 6 4 2 0
γa
100 1/γa

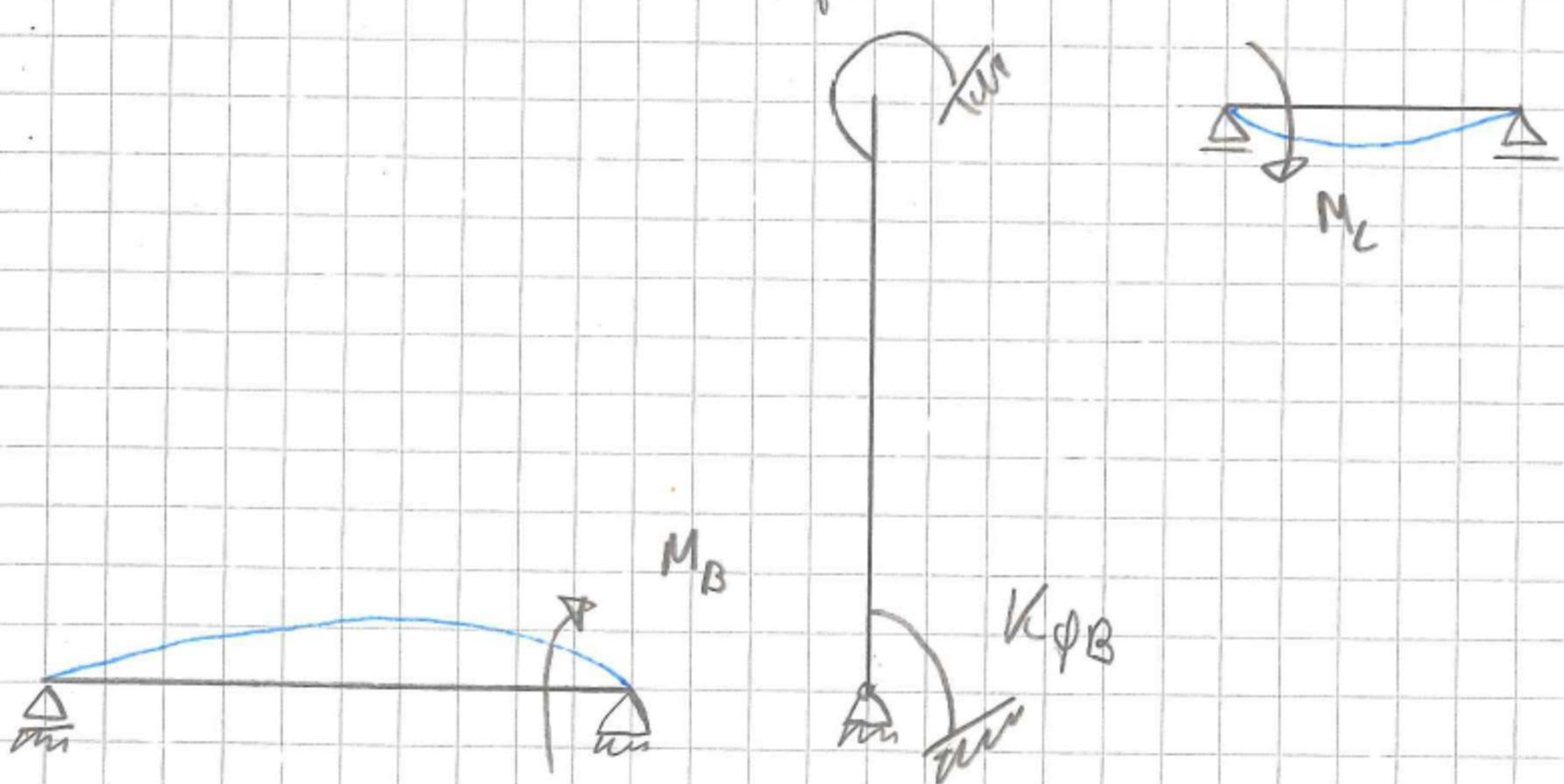
NTNU

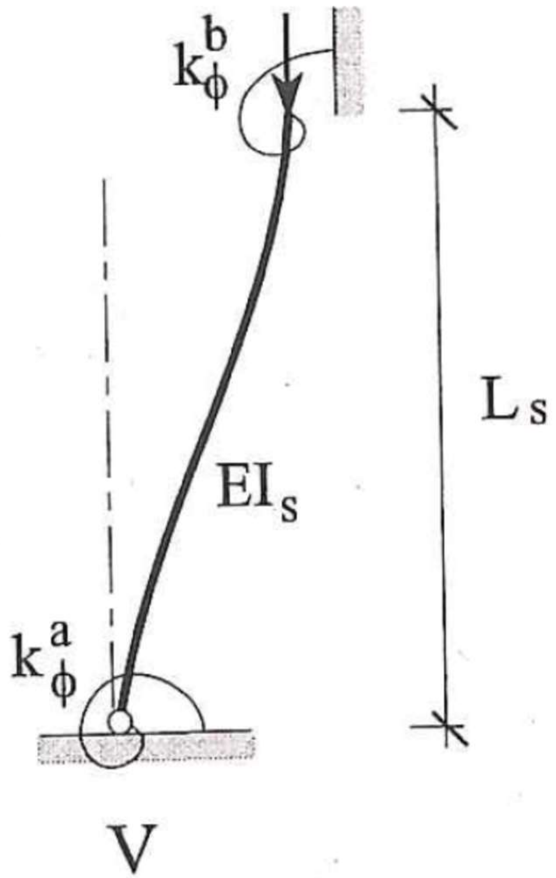

P
C
D
EI
L
EI
A
B
EI
L
1/2 L



k_ϕ^b
L_s
EI_s
k_ϕ^a
V

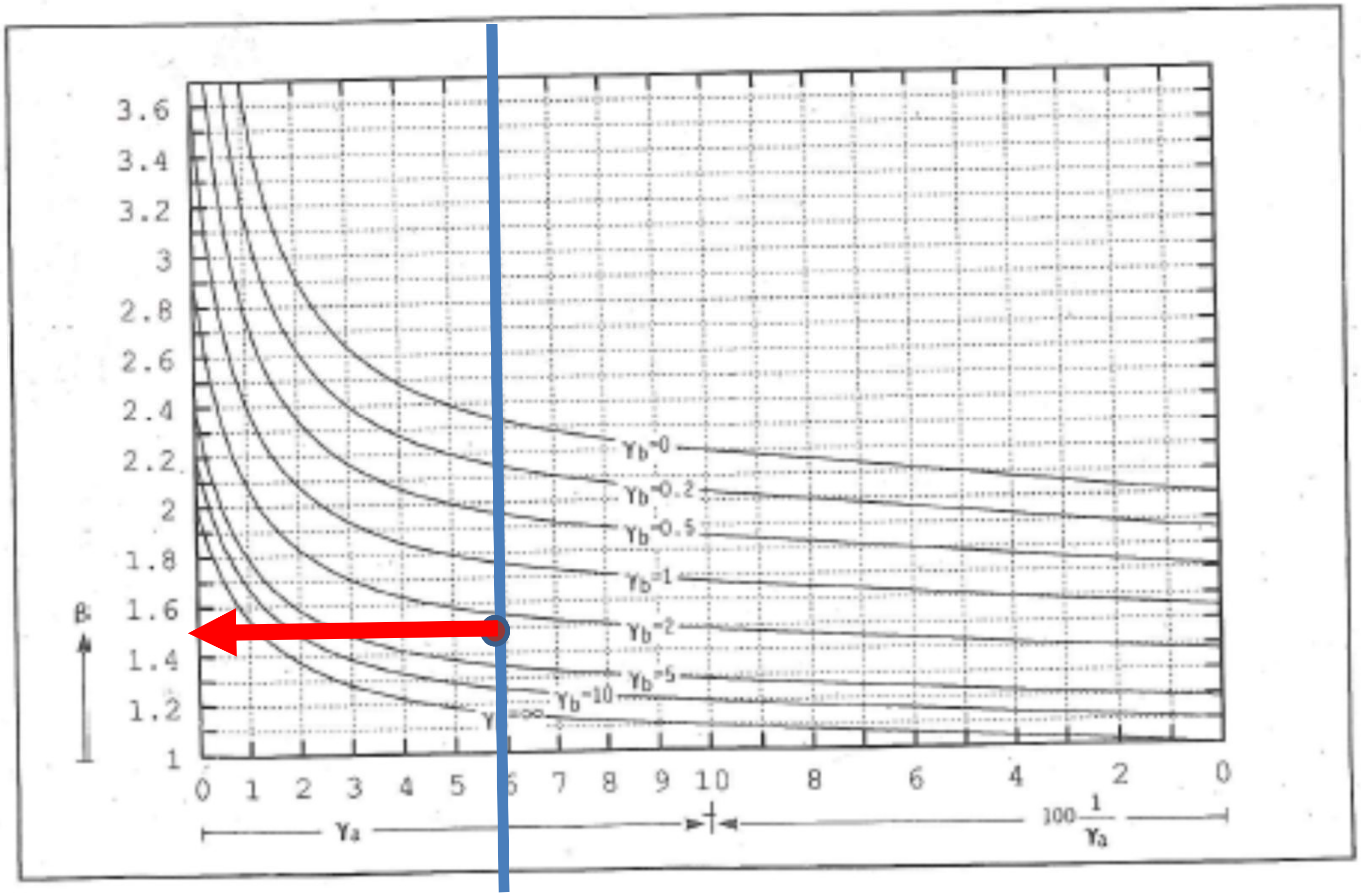
Tilfelle V:





$$\left.\begin{array}{l} C_b = \dfrac{3EI}{\ell/2} \Rightarrow \gamma_b = 6 \\[3em] C_a = \dfrac{3EI}{\ell} \Rightarrow \gamma_a = 3 \end{array}\right\} \Rightarrow \beta \simeq 1.4$$

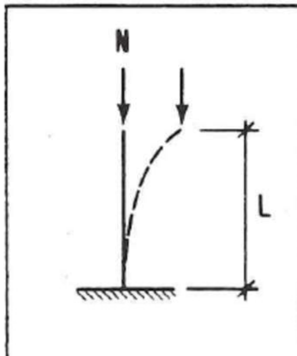
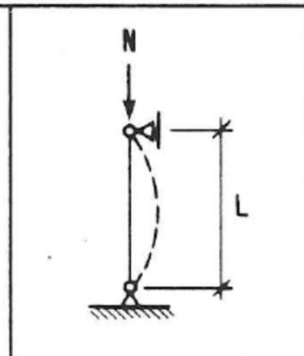
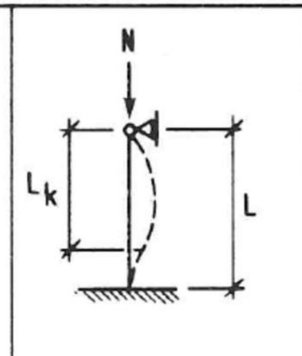
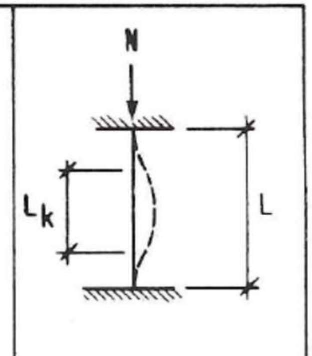
TABELL 4.5 STAVSYSTEM V

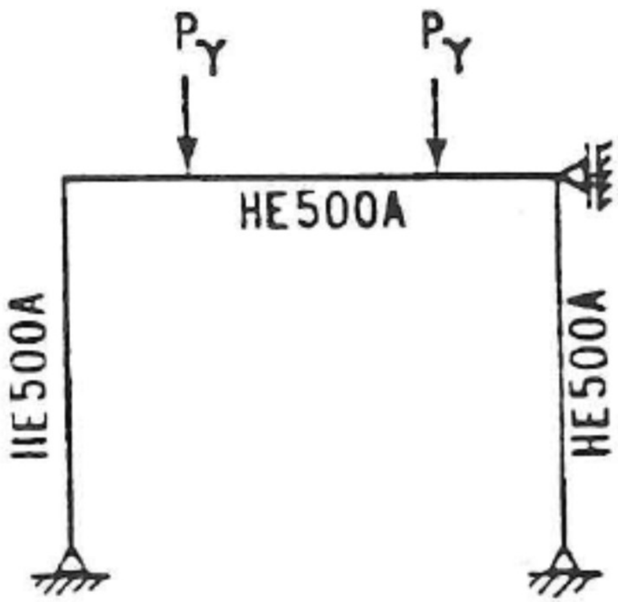


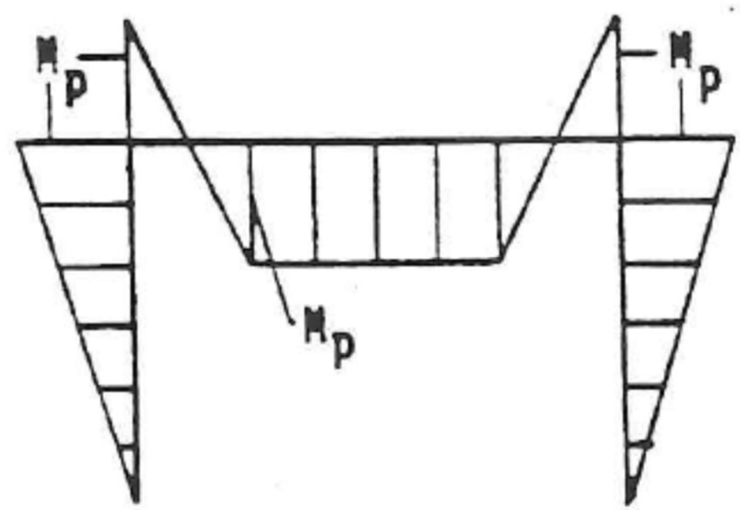
$$\beta := 1.5$$

$$N_{kr} := \frac{\pi^2 \cdot E \cdot I}{(1.5 \cdot L)^2}$$

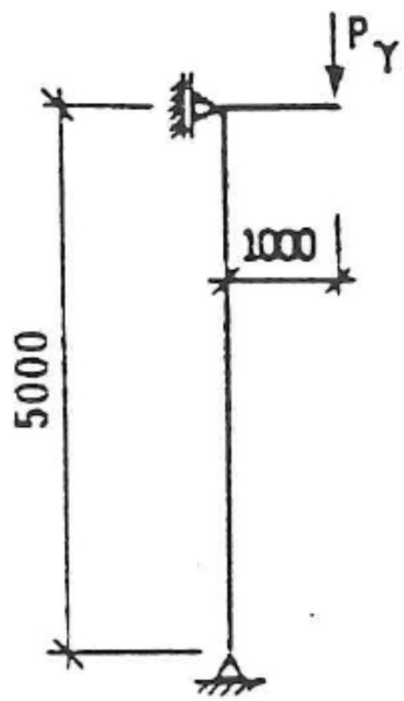
Tabell 6.1 - Basistilfeller for stavknekking

				
Knekkings-betingelse	$\cos kL = 0$	$\sin kL = 0$	$\frac{kL}{\mathrm{tg}kL} = 1$	$\cos kL = 1$
Laveste egenverdi	$kL = \frac{\pi}{2}$	$kL = \pi$	$kL = 4.493$	$kL = 2\pi$
Knekklengde	$L_k = 2.0\ L$	$L_k = L$	$L_k \approx 0.7\ L$	$L_k = 0.5\ L$

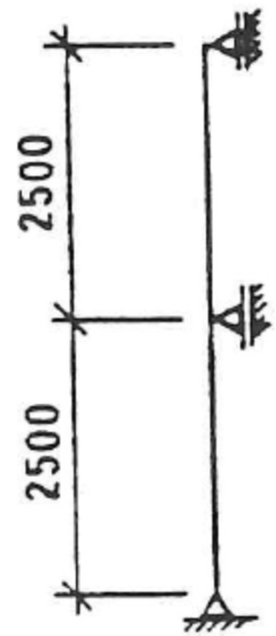




Figur 1



Figur 2a



Figur 2b

NOTE 2 In case the conditions of application expressed in (1) and (2) are not fulfilled, see 6.3.4.

(3) For members of structural systems the resistance check may be carried out on the basis of the individual single span members regarded as cut out of the system. Second order effects of the sway system (P-Δ-effects) have to be taken into account, either by the end moments of the member or by means of appropriate buckling lengths respectively, see 5.2.2(3)c) and 5.2.2(8).

(4) Members which are subjected to combined bending and axial compression should satisfy:

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \qquad (6.61)$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}} \leq 1 \qquad (6.62)$$

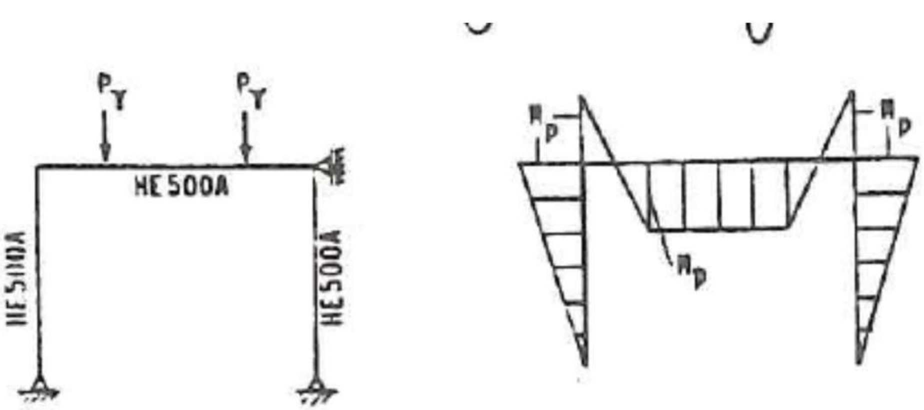
Figur 1 viser en portalramme belastet med to punktlaster samt momentdiagrammet som fremkommer ved en flyteleddsberegning. Med utgangspunkt i denne beregningen velger man å dimensjonere søylen iht beregningsmodellen i fig. 2. Ved bøyning om sterk akse, benyttes systemet i fig. 2a, mens bøyning om svak akse utføres iht systemet i fig. 2b. (Merk at det er innført en ekstra avstivning for bøyning om svak akse.)

Bestem den maksimale last P_γ systemet kan bære.

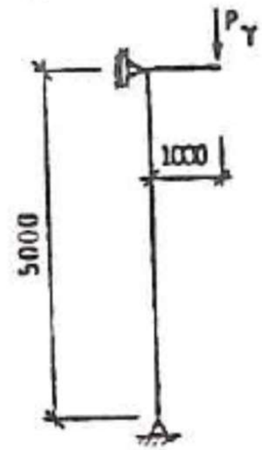
Materiale: $f_y = 240$ MPa

$\gamma_m = 1.0$

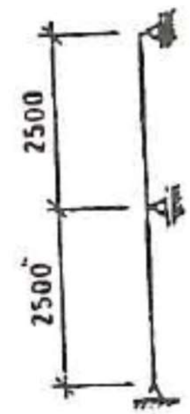
where N_{Ed}, $M_{y,Ed}$ and $M_{z,Ed}$	are the design values of the compression force and the maximum moments about the y-y and z-z axis along the member, respectively
$\Delta M_{y,Ed}$, $\Delta M_{z,Ed}$	are the moments due to the shift of the centroidal axis according to 6.2.9.3 for class 4 sections, see Table 6.7,
χ_y and χ_z	are the reduction factors due to flexural buckling from 6.3.1
χ_{LT}	is the reduction factor due to lateral torsional buckling from 6.3.2
k_{yy}, k_{yz}, k_{zy}, k_{zz}	are the interaction factors



Figur 1



Figur 2a



Figur 2b

Figur 1 viser rammen og plastisk moment diagram.
Figur 2a og 2b viser forenklet system. om
sterk- og svak- akse.

Lasten P er i figur 2a plassert i moment nullpunktet på rigelen.

Profil data HE 500A:

$A = 19.8 \cdot 10^3 \, mm^2$

$I_x = 869.7 \cdot 10^6 \, mm^4$ $\qquad$ $I_y = 103,7 \cdot 10^6 \, mm^4$

$W_{px} = 2 \cdot S_x = 3940 \cdot 10^3 \, mm^3$ $\qquad$ $i_y = 72.4 \, mm$

$i_x = 210 \, mm$

Materialdata: $\left.\begin{array}{l} f_y = 240 \ N/mm^2 \\ \gamma_m = 1.0 \end{array}\right\} \Rightarrow f_d = 240 \ N/mm^2$

- Ser på plan knekning om x- og y-aksen
- Lokal knekning ⊃: tverrsnitts klassifisering
- Plastisk kapasitet.

Kritisk last finnes vha. en av disse tilfellene.

-1- Tverrsnitts klassifisering

Det første flyteleddet dannes på rigelen ⇒ tverrsnitts klasse 1 (5.2.2.2).

fig 5.2.2a - flens: $\frac{b}{t_{max}} \leq 0.30\sqrt{\frac{E}{f_y}} = 8.9$

$\frac{b}{t} = 6.3 < 8.9$ ⊃: OK

- steg: $\frac{h}{t_{max}} \leq 2.0\sqrt{\frac{E}{f_y}} = 59.2$

$\frac{h}{t} = 32.5 < 59.2$ ⊃: OK

⊃: HE 500A er ikke utsatt for lokal knekking før full plastisk kapasitet er mobilisert.

-2- Global knekningskontroll

Merk: neglisjerer et fenomen som kalles vipping,

Annex A [informative] – Method 1: Interaction factors k_{ij} for interaction formula in 6.3.3(4)

Table A.1: Interaction factors k_{ij} (6.3.3(4))

Interaction factors	Design assumptions	
	elastic cross-sectional properties class 3, class 4	plastic cross-sectional properties class 1, class 2
k_{yy}	$C_{my} C_{mLT} \dfrac{\mu_y}{1 - \dfrac{N_{Ed}}{N_{cr,y}}}$	$C_{my} C_{mLT} \dfrac{\mu_y}{1 - \dfrac{N_{Ed}}{N_{cr,y}}} \dfrac{1}{C_{yy}}$
k_{yz}	$C_{mz} \dfrac{\mu_y}{1 - \dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{mz} \dfrac{\mu_y}{1 - \dfrac{N_{Ed}}{N_{cr,z}}} \dfrac{1}{C_{yz}} 0{,}6 \sqrt{\dfrac{w_z}{w_y}}$
k_{zy}	$C_{my} C_{mLT} \dfrac{\mu_z}{1 - \dfrac{N_{Ed}}{N_{cr,y}}}$	$C_{my} C_{mLT} \dfrac{\mu_z}{1 - \dfrac{N_{Ed}}{N_{cr,y}}} \dfrac{1}{C_{zy}} 0{,}6 \sqrt{\dfrac{w_y}{w_z}}$
k_{zz}	$C_{mz} \dfrac{\mu_z}{1 - \dfrac{N_{Ed}}{N_{cr,z}}}$	$C_{mz} \dfrac{\mu_z}{1 - \dfrac{N_{Ed}}{N_{cr,z}}} \dfrac{1}{C_{zz}}$

Auxiliary terms:

$$\mu_y = \frac{1 - \frac{N_{Ed}}{N_{cr,y}}}{1 - \chi_y \frac{N_{Ed}}{N_{cr,y}}}$$

$$\mu_z = \frac{1 - \frac{N_{Ed}}{N_{cr,z}}}{1 - \chi_z \frac{N_{Ed}}{N_{cr,z}}}$$

$$w_y = \frac{W_{pl,y}}{W_{el,y}} \leq 1{,}5$$

$$w_z = \frac{W_{pl,z}}{W_{el,z}} \leq 1{,}5$$

$$n_{pl} = \frac{N_{Ed}}{N_{Rk} / \gamma_{M1}}$$

C_{my} see Table A.2

$$a_{LT} = 1 - \frac{I_T}{I_y} \geq 0$$

$$C_{yy} = 1 + (w_y - 1)\left[\left(2 - \frac{1{,}6}{w_y} C_{my}^2 \bar{\lambda}_{max} - \frac{1{,}6}{w_y} C_{my}^2 \bar{\lambda}_{max}^2\right) n_{pl} - b_{LT}\right] \geq \frac{W_{el,y}}{W_{pl,y}}$$

with $b_{LT} = 0{,}5\, a_{LT}\, \bar{\lambda}_0^2 \dfrac{M_{y,Ed}}{\chi_{LT} M_{pl,y,Rd}} \dfrac{M_{z,Ed}}{M_{pl,z,Rd}}$

$$C_{yz} = 1 + (w_z - 1)\left[\left(2 - 14 \frac{C_{mz}^2 \bar{\lambda}_{max}^2}{w_z^5}\right) n_{pl} - c_{LT}\right] \geq 0{,}6 \sqrt{\frac{w_z}{w_y}} \frac{W_{el,z}}{W_{pl,z}}$$

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

$$C_{zy} = 1 + (w_y - 1)\left[\left(2 - 14 \frac{C_{my}^2 \bar{\lambda}_{max}^2}{w_y^5}\right) n_{pl} - d_{LT}\right] \geq 0{,}6 \sqrt{\frac{w_y}{w_z}} \frac{W_{el,y}}{W_{pl,y}}$$

with $d_{LT} = 2\, a_{LT} \dfrac{\bar{\lambda}_0}{0{,}1 + \bar{\lambda}_z^4} \dfrac{M_{y,Ed}}{C_{my}\, \chi_{LT}\, M_{pl,y,Rd}} \dfrac{M_{z,Ed}}{C_{mz}\, M_{pl,z,Rd}}$

$$C_{zz} = 1 + (w_z - 1)\left[\left(2 - \frac{1{,}6}{w_z} C_{mz}^2 \bar{\lambda}_{max} - \frac{1{,}6}{w_z} C_{mz}^2 \bar{\lambda}_{max}^2\right) n_{pl} - e_{LT}\right] \geq \frac{W_{el,z}}{W_{pl,z}}$$

with $e_{LT} = 1{,}7\, a_{LT} \dfrac{\bar{\lambda}_0}{0{,}1 + \bar{\lambda}_z^4} \dfrac{M_{y,Ed}}{C_{my}\, \chi_{LT}\, M_{pl,y,Rd}}$

$$\varepsilon_y = \frac{M_{y,Ed}}{N_{Ed}} \frac{A}{W_{el,y}} \quad \text{for class 1, 2 and 3 cross-sections}$$

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

$N_{cr,y}$ = elastic flexural buckling force about the y-y axis

$N_{cr,z}$ = elastic flexural buckling force about the z-z axis

$N_{cr,T}$ = elastic torsional buckling force

I_T = St. Venant torsional constant

I_y = second moment of area about y-y axis

Table 6.7: Values for $N_{Rk} = f_y A_i$, $M_{i,Rk} = f_y W_i$ and $\Delta M_{i,Ed}$

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

NOTE For members not susceptible to torsional deformation χ_{LT} would be $\chi_{LT} = 1,0$.

NOTE For members not susceptible to torsional deformation χ_{LT} would be $\chi_{LT} = 1,0$.

(5) The interaction factors k_{yy} , k_{yz} , k_{zy} , k_{zz} depend on the method which is chosen.

NOTE 1 The interaction factors k_{yy}, k_{yz}, k_{zy} and k_{zz} have been derived from two alternative approaches. Values of these factors may be obtained from Annex A (alternative method 1) or from Annex B (alternative method 2).

NOTE 2 The National Annex may give a choice from alternative method 1 or alternative method 2.

NOTE 3 For simplicity verifications may be performed in the elastic range only.

Table A.1 (continued)

$$\bar{\lambda}_{max} = \max\begin{cases}\bar{\lambda}_y \\ \bar{\lambda}_z\end{cases}$$

$\bar{\lambda}_0$ = non-dimensional slenderness for lateral-torsional buckling due to uniform bending moment, i.e. $\psi_y = 1{,}0$ in Table A.2

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

If $\bar{\lambda}_0 \leq 0{,}2\sqrt{C_1}\sqrt[4]{\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\left(1-\frac{N_{Ed}}{N_{cr,TF}}\right)}$:

$$C_{my} = C_{my,0}$$

$$C_{mz} = C_{mz,0}$$

$$C_{mLT} = 1{,}0$$

If $\bar{\lambda}_0 > 0{,}2\sqrt{C_1}\sqrt[4]{\left(1-\frac{N_{Ed}}{N_{cr,z}}\right)\left(1-\frac{N_{Ed}}{N_{cr,TF}}\right)}$:

$$C_{my} = C_{my,0} + (1 - C_{my,0})\frac{\sqrt{\varepsilon_y}a_{LT}}{1+\sqrt{\varepsilon_y}a_{LT}}$$

$$C_{mz} = C_{mz,0}$$

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

$$\varepsilon_y = \frac{M_{y,Ed}}{N_{Ed}}\frac{A}{W_{el,y}} \quad \text{for class 1, 2 and 3 cross-sections}$$

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

$N_{cr,y}$ = elastic flexural buckling force about the y-y axis

$N_{cr,z}$ = elastic flexural buckling force about the z-z axis

$N_{cr,T}$ = elastic torsional buckling force

I_T = St. Venant torsional constant

I_y = second moment of area about y-y axis

$$\bar{\lambda}_{max} = \max\begin{cases}\bar{\lambda}_y \\ \bar{\lambda}_z\end{cases}$$

$\bar{\lambda}_0$ = non-dimensional slenderness for lateral-torsional buckling due to uniform bending moment, i.e. $\psi_y = 1{,}0$ in Table A.2

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

If $\bar{\lambda}_0 \le 0{,}2\sqrt{C_1}\sqrt[4]{\left(1-\dfrac{N_{Ed}}{N_{cr,z}}\right)\left(1-\dfrac{N_{Ed}}{N_{cr,TF}}\right)}$:

$$C_{my} = C_{my,0}$$

$$C_{mz} = C_{mz,0}$$

$$C_{mLT} = 1{,}0$$

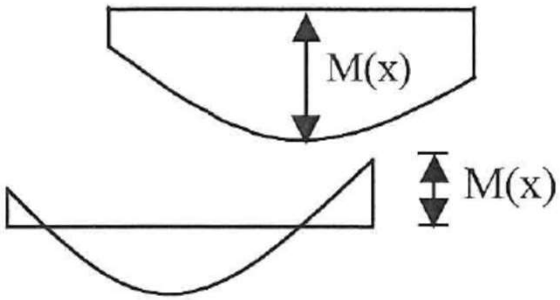
If $\bar{\lambda}_0 > 0{,}2\sqrt{C_1}\sqrt[4]{\left(1-\dfrac{N_{Ed}}{N_{cr,z}}\right)\left(1-\dfrac{N_{Ed}}{N_{cr,TF}}\right)}$:

$$C_{my} = C_{my,0} + \left(1-C_{my,0}\right)\frac{\sqrt{\varepsilon_y}\,a_{LT}}{1+\sqrt{\varepsilon_y}\,a_{LT}}$$

$$C_{mz} = C_{mz,0}$$

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

Table A.2: Equivalent uniform moment factors $C_{mi,0}$

Moment diagram	$C_{mi,0}$
 $-1 \le \psi \le 1$	$C_{mi,0} = 0{,}79 + 0{,}21\psi_i + 0{,}36(\psi_i - 0{,}33)\dfrac{N_{Ed}}{N_{cr,i}}$
	$C_{mi,0} = 1 + \left(\dfrac{\pi^2 EI_i \lvert\delta_x\rvert}{L^2 \lvert M_{i,Ed}(x)\rvert} - 1\right)\dfrac{N_{Ed}}{N_{cr,i}}$ $M_{i,Ed}(x)$ is the maximum moment $M_{y,Ed}$ or $M_{z,Ed}$ $\lvert\delta_x\rvert$ is the maximum member displacement along the member
	$C_{mi,0} = 1 - 0{,}18\dfrac{N_{Ed}}{N_{cr,i}}$
	$C_{mi,0} = 1 + 0{,}03\dfrac{N_{Ed}}{N_{cr,i}}$

Annex B [informative] – Method 2: Interaction factors k_{ij} for interaction formula in 6.3.3(4)

Table B.1: Interaction factors k_{ij} for members not susceptible to torsional deformations

Interaction factors	Type of sections	Design assumptions: elastic cross-sectional properties class 3, class 4	Design assumptions: plastic cross-sectional properties class 1, class 2
k_{yy}	I-sections RHS-sections	$C_{my}\left(1+0{,}6\bar{\lambda}_y\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{my}\left(1+0{,}6\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$	$C_{my}\left(1+\left(\bar{\lambda}_y-0{,}2\right)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{my}\left(1+0{,}8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$
k_{yz}	I-sections RHS-sections	k_{zz}	$0{,}6\ k_{zz}$
k_{zy}	I-sections RHS-sections	$0{,}8\ k_{yy}$	$0{,}6\ k_{yy}$
k_{zz}	I-sections	$C_{mz}\left(1+0{,}6\bar{\lambda}_z\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{mz}\left(1+0{,}6\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$	$C_{mz}\left(1+\left(2\bar{\lambda}_z-0{,}6\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{mz}\left(1+1{,}4\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
	RHS-sections		$C_{mz}\left(1+\left(\bar{\lambda}_z-0{,}2\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$ $\leq C_{mz}\left(1+0{,}8\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$

For I- and H-sections and rectangular hollow sections under axial compression and uniaxial bending $M_{y,Ed}$ the coefficient k_{zy} may be $k_{zy} = 0$.

Table B.2: Interaction factors k_{ij} for members susceptible to torsional deformations

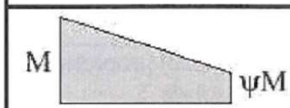
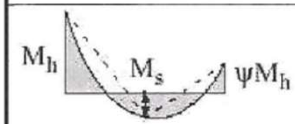
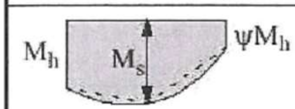
Interaction factors	Design assumptions: elastic cross-sectional properties class 3, class 4	Design assumptions: plastic cross-sectional properties class 1, class 2
k_{yy}	k_{yy} from Table B.1	k_{yy} from Table B.1
k_{yz}	k_{yz} from Table B.1	k_{yz} from Table B.1
k_{zy}	$\left[1-\frac{0{,}05\bar{\lambda}_z}{\left(C_{mLT}-0{,}25\right)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ $\geq\left[1-\frac{0{,}05}{\left(C_{mLT}-0{,}25\right)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$	$\left[1-\frac{0{,}1\bar{\lambda}_z}{\left(C_{mLT}-0{,}25\right)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ $\geq\left[1-\frac{0{,}1}{\left(C_{mLT}-0{,}25\right)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right]$ for $\bar{\lambda}_z<0{,}4$: $k_{zy}=0{,}6+\bar{\lambda}_z\leq 1-\frac{0{,}1\bar{\lambda}_z}{\left(C_{mLT}-0{,}25\right)}\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}$

Table B.1: Interaction factors k_{ij} for members not susceptible to torsional deformations

<table>
<tr><th rowspan="2">Interaction factors</th><th rowspan="2">Type of sections</th><th colspan="2">Design assumptions</th></tr>
<tr><th>elastic cross-sectional properties class 3, class 4</th><th>plastic cross-sectional properties class 1, class 2</th></tr>
<tr><td>k_{yy}</td><td>I-sections
RHS-sections</td><td>$C_{my}\left(1+0{,}6\bar{\lambda}_y \frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$
$\le C_{my}\left(1+0{,}6\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$</td><td>$C_{my}\left(1+\left(\bar{\lambda}_y-0{,}2\right)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$
$\le C_{my}\left(1+0{,}8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)$</td></tr>
<tr><td>k_{yz}</td><td>I-sections
RHS-sections</td><td>k_{zz}</td><td>$0{,}6\ k_{zz}$</td></tr>
<tr><td>k_{zy}</td><td>I-sections
RHS-sections</td><td>$0{,}8\ k_{yy}$</td><td>$0{,}6\ k_{yy}$</td></tr>
<tr><td rowspan="2">k_{zz}</td><td>I-sections</td><td rowspan="2">$C_{mz}\left(1+0{,}6\bar{\lambda}_z \frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
$\le C_{mz}\left(1+0{,}6\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$</td><td>$C_{mz}\left(1+\left(2\bar{\lambda}_z-0{,}6\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
$\le C_{mz}\left(1+1{,}4\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$</td></tr>
<tr><td>RHS-sections</td><td>$C_{mz}\left(1+\left(\bar{\lambda}_z-0{,}2\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$
$\le C_{mz}\left(1+0{,}8\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)$</td></tr>
<tr><td colspan="4">For I- and H-sections and rectangular hollow sections under axial compression and uniaxial bending $M_{y,Ed}$ the coefficient k_{zy} may be $k_{zy} = 0$.</td></tr>
</table>

k_{zz}	k_{zz} from Table B.1	k_{zz} from Table B.1

Table B.3: Equivalent uniform moment factors C_m in Tables B.1 and B.2

Moment diagram	range		C_{my} and C_{mz} and C_{mLT} uniform loading	concentrated load
	$-1 \le \psi \le 1$		$0{,}6 + 0{,}4\psi \ge 0{,}4$	
	$0 \le \alpha_s \le 1$	$-1 \le \psi \le 1$	$0{,}2 + 0{,}8\alpha_s \ge 0{,}4$	$0{,}2 + 0{,}8\alpha_s \ge 0{,}4$
	$-1 \le \alpha_s < 0$	$0 \le \psi \le 1$	$0{,}1 - 0{,}8\alpha_s \ge 0{,}4$	$-0{,}8\alpha_s \ge 0{,}4$
$\alpha_s = M_s/M_h$		$-1 \le \psi < 0$	$0{,}1(1-\psi) - 0{,}8\alpha_s \ge 0{,}4$	$0{,}2(-\psi) - 0{,}8\alpha_s \ge 0{,}4$
	$0 \le \alpha_h \le 1$	$-1 \le \psi \le 1$	$0{,}95 + 0{,}05\alpha_h$	$0{,}90 + 0{,}10\alpha_h$
	$-1 \le \alpha_h < 0$	$0 \le \psi \le 1$	$0{,}95 + 0{,}05\alpha_h$	$0{,}90 + 0{,}10\alpha_h$
$\alpha_h = M_h/M_s$		$-1 \le \psi < 0$	$0{,}95 + 0{,}05\alpha_h(1+2\psi)$	$0{,}90 - 0{,}10\alpha_h(1+2\psi)$

For members with sway buckling mode the equivalent uniform moment factor should be taken $C_{my} = 0{,}9$ or $C_{Mz} = 0{,}9$ respectively.

C_{my}, C_{mz} and C_{mLT} should be obtained according to the bending moment diagram between the relevant braced points as follows:

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

Annex AB [informative] – Additional design provisions

AB.1 Structural analysis taking account of material non-linearities

(1)B In case of material non-linearities the action effects in a structure may be determined by incremental approach to the design loads to be considered for the relevant design situation.

(2)B In this incremental approach each permanent or variable action should be increased proportionally.

AB.2 Simplified provisions for the design of continuous floor beams

(1)B For continuous beams with slabs in buildings without cantilevers on which uniformly distributed loads are dominant, it is sufficient to consider only the following load arrangements:

a) alternative spans carrying the design permanent and variable load ($\gamma_G G_k + \gamma_Q Q_k$), other spans carrying only the design permanent load $\gamma_G G_k$

b) any two adjacent spans carrying the design permanent and variable loads ($\gamma_G G_k + \gamma_Q Q_k$), all other spans carrying only the design permanent load $\gamma_G G_k$

NOTE 1 a) applies to sagging moments, b) to hogging moments.

NOTE 2 This annex is intended to be transferred to EN 1990 in a later stage.

TBYG3018 Design of Ocean Space Structures

Module 3 – Introduction to design of steel structures

Jomar Tørset, Assistant professor

Classification of beam sections

NS-EN 1993-1-1 Clause 5.5

Class 1: Can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance.

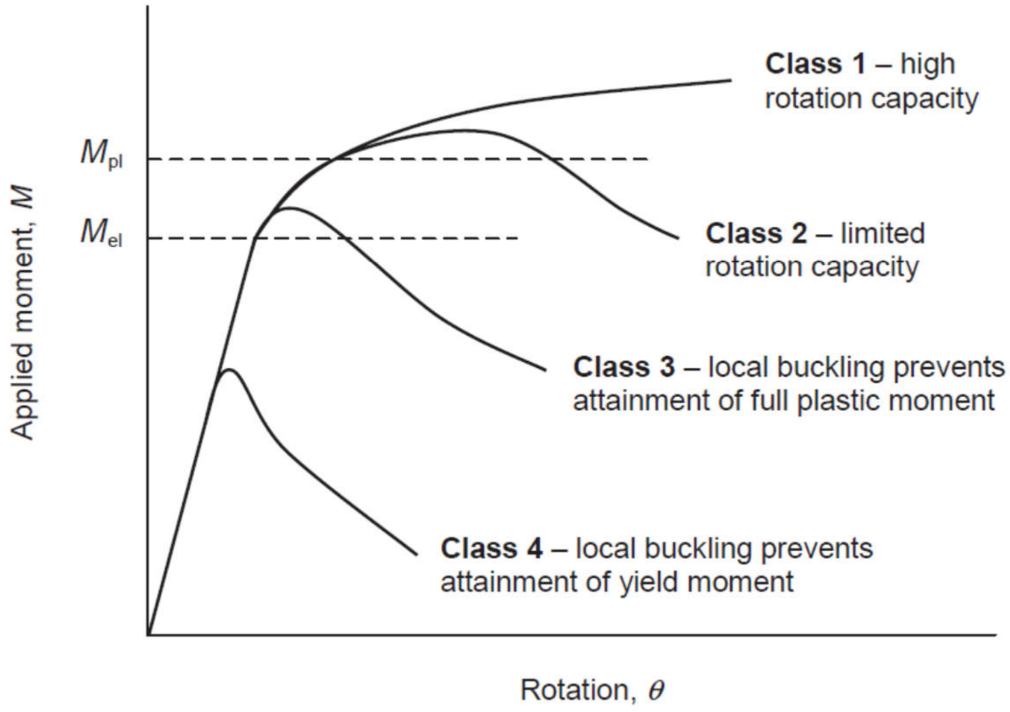
Class 2: Can develop their plastic moment resistance, but have limited rotation capacity because of local buckling..

Class 3: Can reach the yield strength in the extreme compression fibre, but local buckling is liable to prevent the development of the plastic moment resistance.

Class 4: Local buckling will occur before the attainment of the yield stress in one or more parts of the cross-section.

Classification of beam sections

NS-EN 1993-1-1 Clause 5.5



6.1.4.2 Classification

(1) Four classes of cross-sections are defined, as follows:

- Class 1 cross-sections are those that can form a plastic hinge with the rotation capacity required for plastic analysis without reduction of the resistance.

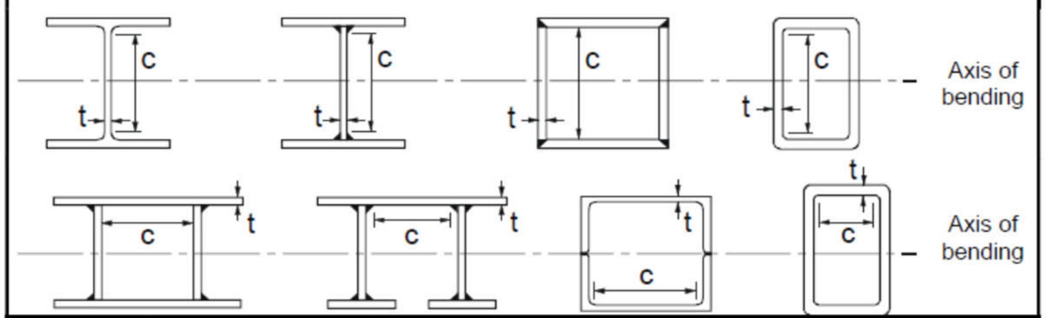
NOTE Further information on class 1 cross-sections is given in Annex G.

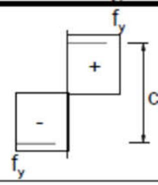
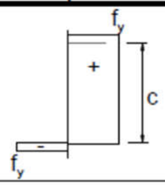
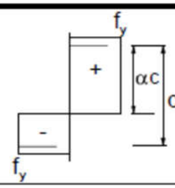
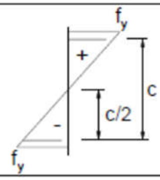
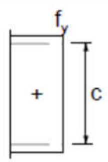
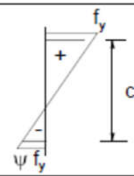
- Class 2 cross-sections are those that can develop their plastic moment resistance, but have limited rotation capacity because of local buckling.

- Class 3 cross-sections are those in which the calculated stress in the extreme compression fibre of the aluminium member can reach its proof strength, but local buckling is liable to prevent development of the full plastic moment resistance.
- Class 4 cross-sections are those in which local buckling will occur before the attainment of proof stress in one or more parts of the cross-section.

Table 5.2 (sheet 1 of 3): Maximum width-to-thickness ratios for compression parts

Internal compression parts



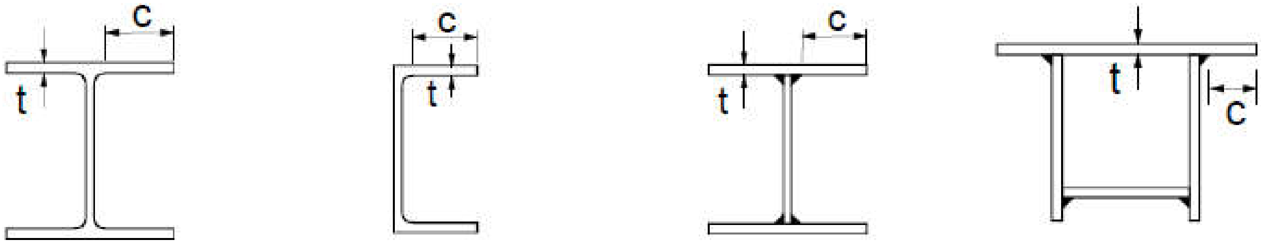
Class	Part subject to bending	Part subject to compression	Part subject to bending and compression
Stress distribution in parts (compression positive)			
1	$c/t \le 72\varepsilon$	$c/t \le 33\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{396\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{36\varepsilon}{\alpha}$
2	$c/t \le 83\varepsilon$	$c/t \le 38\varepsilon$	when $\alpha > 0{,}5$: $c/t \le \dfrac{456\varepsilon}{13\alpha - 1}$ when $\alpha \le 0{,}5$: $c/t \le \dfrac{41{,}5\varepsilon}{\alpha}$
Stress distribution in parts (compression positive)			
3	$c/t \le 124\varepsilon$	$c/t \le 42\varepsilon$	when $\psi > -1$: $c/t \le \dfrac{42\varepsilon}{0{,}67 + 0{,}33\psi}$ when $\psi \le -1$*): $c/t \le 62\varepsilon(1-\psi)\sqrt{(-\psi)}$

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

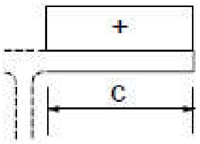
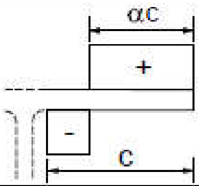
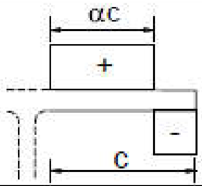
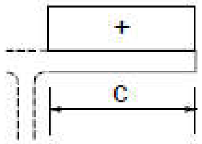
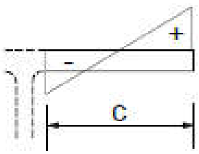
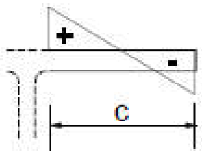
*) $\psi \le -1$ applies where either the compression stress $\sigma \le f_y$ or the tensile strain $\varepsilon_y > f_y/E$

Table 5.2 (sheet 2 of 3): Maximum width-to-thickness ratios for compression parts

Outstand flanges



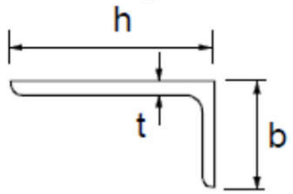
Rolled sections | Welded sections

Class	Part subject to compression	Part subject to bending and compression: Tip in compression	Part subject to bending and compression: Tip in tension
Stress distribution in parts (compression positive)			
1	$c/t \leq 9\varepsilon$	$c/t \leq \frac{9\varepsilon}{\alpha}$	$c/t \leq \frac{9\varepsilon}{\alpha\sqrt{\alpha}}$
2	$c/t \leq 10\varepsilon$	$c/t \leq \frac{10\varepsilon}{\alpha}$	$c/t \leq \frac{10\varepsilon}{\alpha\sqrt{\alpha}}$
Stress distribution in parts (compression positive)			
3	$c/t \leq 14\varepsilon$	$c/t \leq 21\varepsilon\sqrt{k_\sigma}$ For k_σ see EN 1993-1-5	

$\varepsilon = \sqrt{235/f_y}$	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71

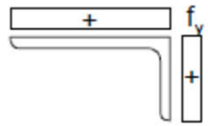
Table 5.2 (sheet 3 of 3): Maximum width-to-thickness ratios for compression parts

Angles

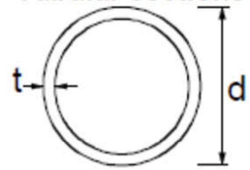


Refer also to "Outstand flanges" (see sheet 2 of 3)

Does not apply to angles in continuous contact with other components

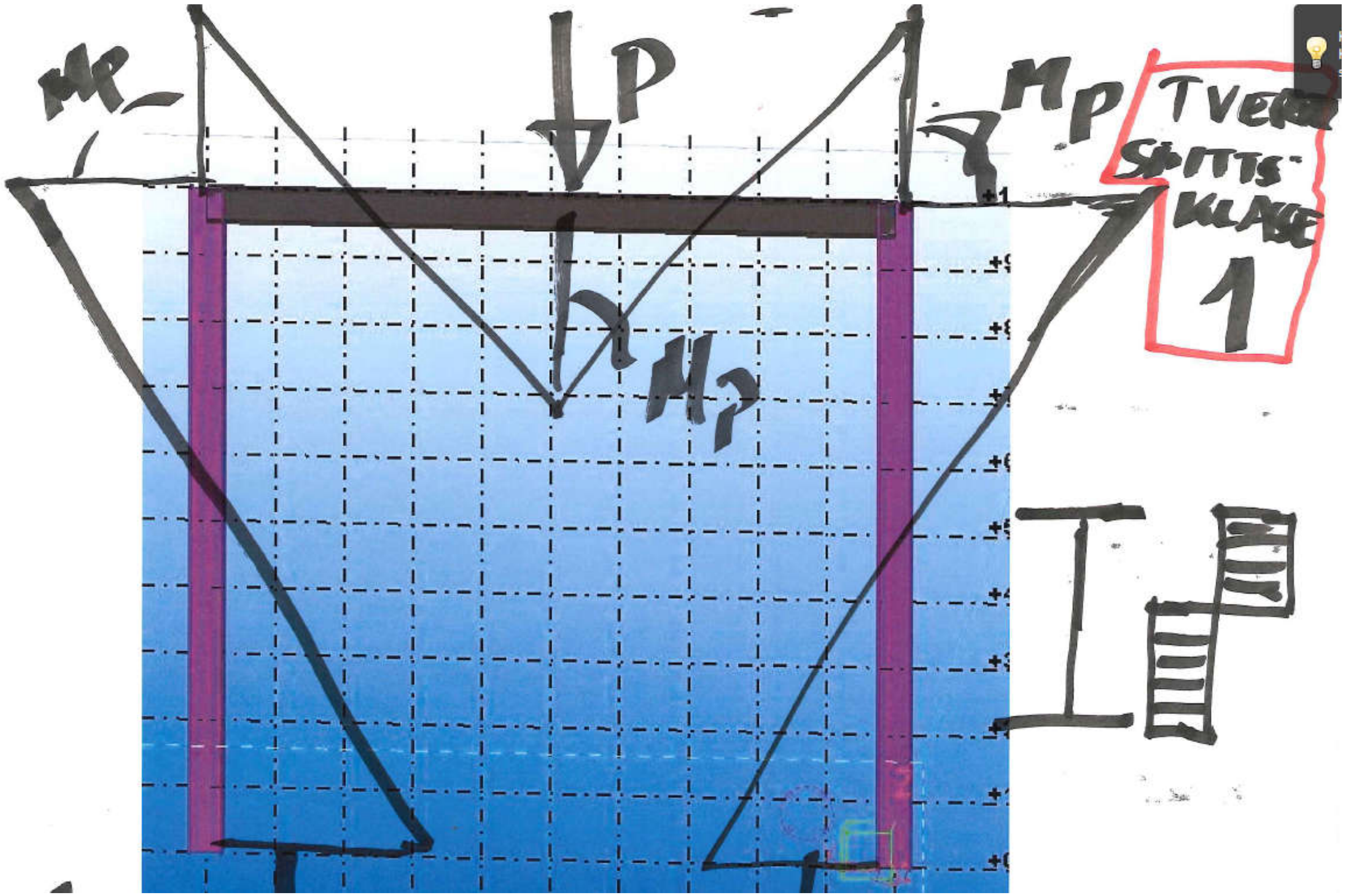
Class	Section in compression
Stress distribution across section (compression positive)	
3	$h/t \leq 15\varepsilon : \dfrac{b+h}{2t} \leq 11{,}5\varepsilon$

Tubular sections



Class	Section in bending and/or compression
1	$d/t \leq 50\varepsilon^2$
2	$d/t \leq 70\varepsilon^2$
3	$d/t \leq 90\varepsilon^2$ **NOTE** For $d/t > 90\varepsilon^2$ see EN 1993-1-6.

$\varepsilon = \sqrt{235/f_y}$						
	f_y	235	275	355	420	460
	ε	1,00	0,92	0,81	0,75	0,71
	ε^2	1,00	0,85	0,66	0,56	0,51



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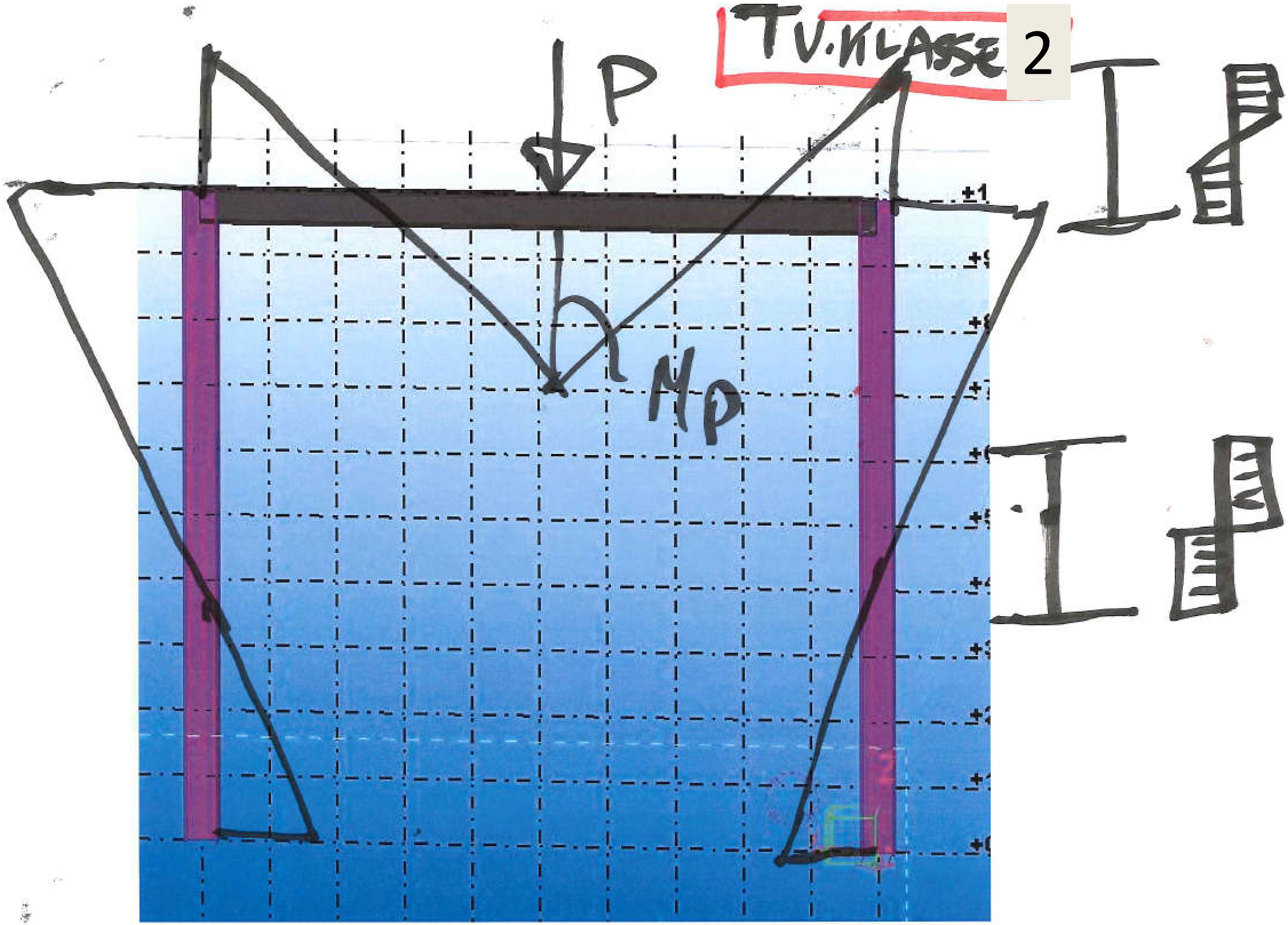
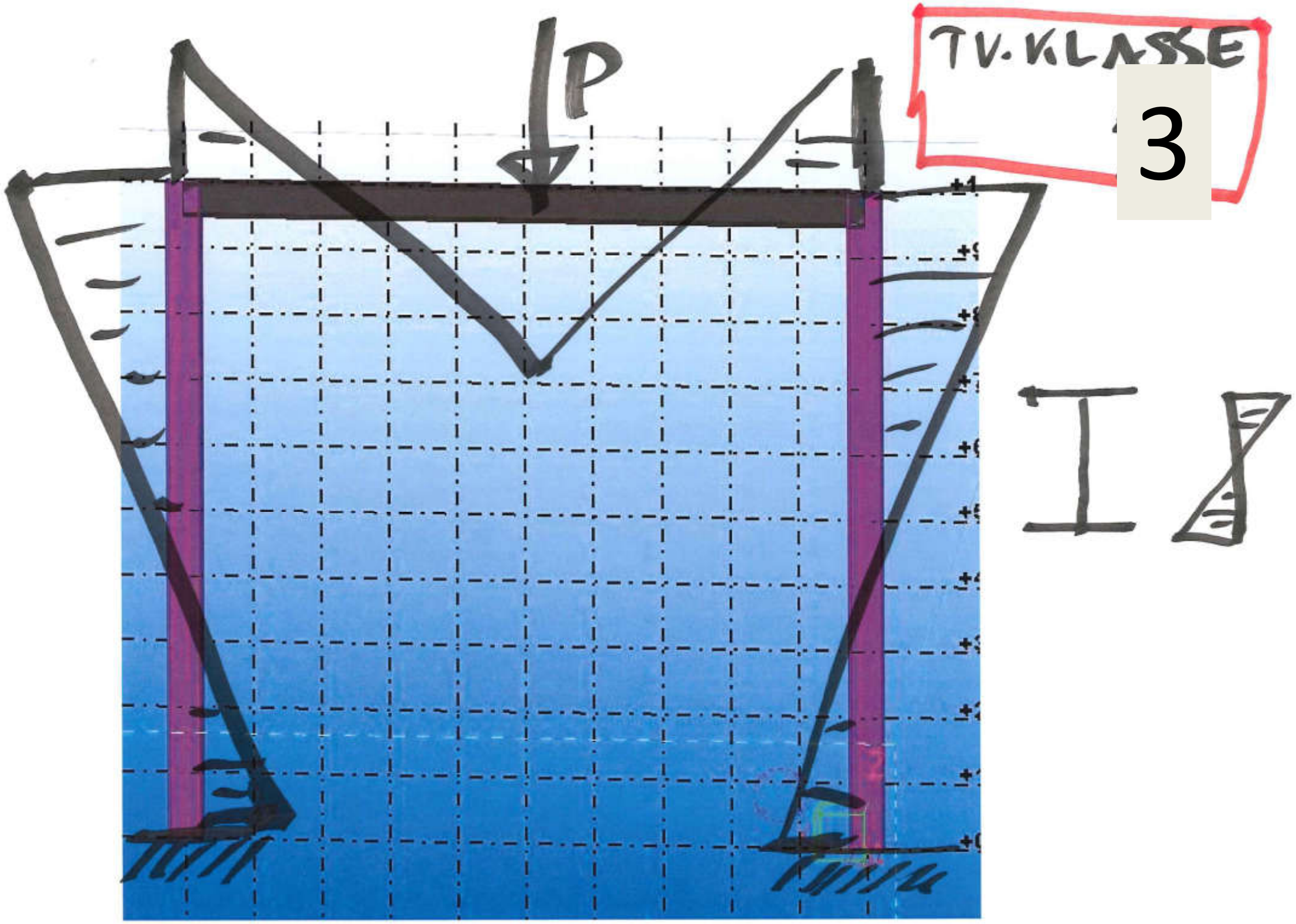
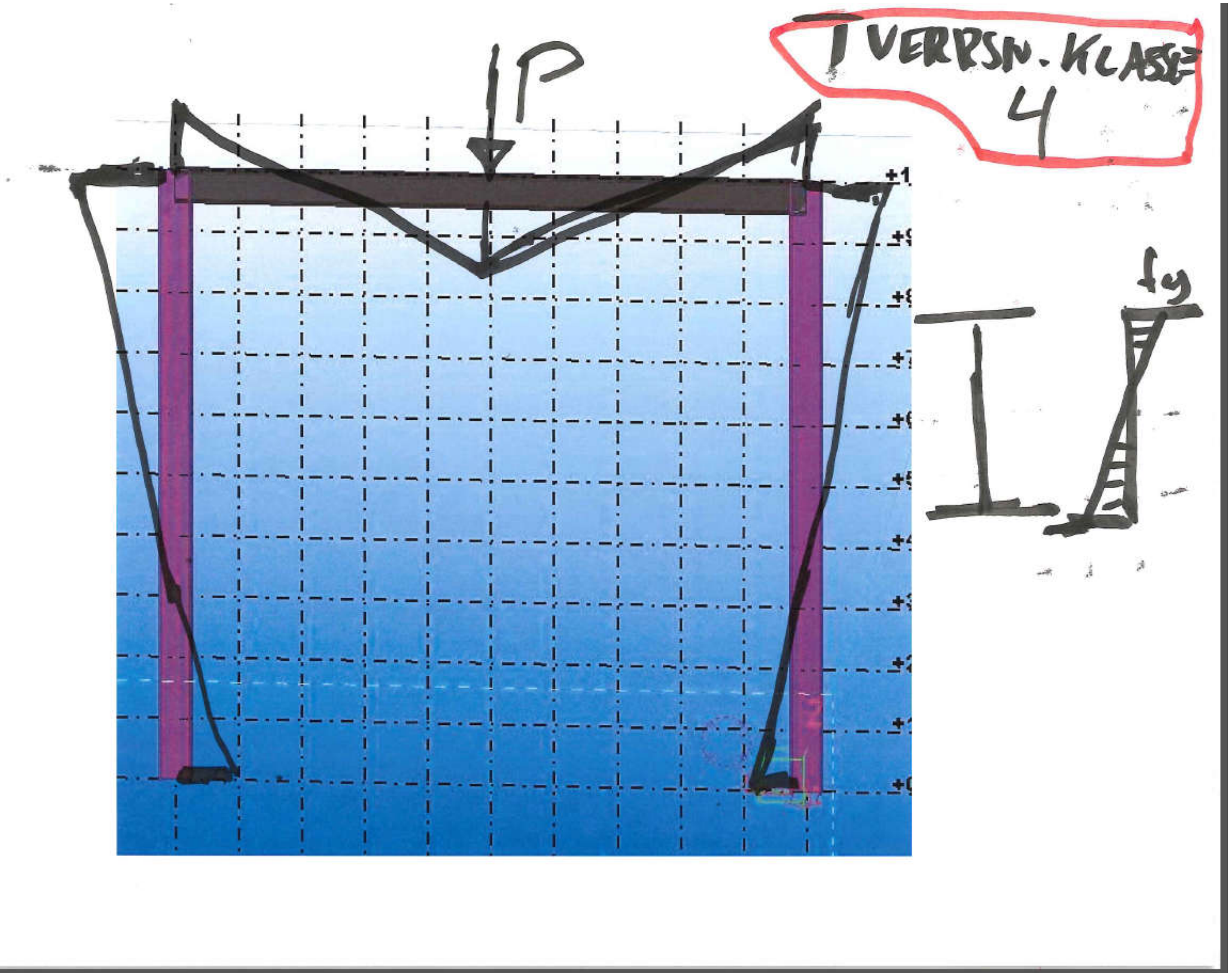
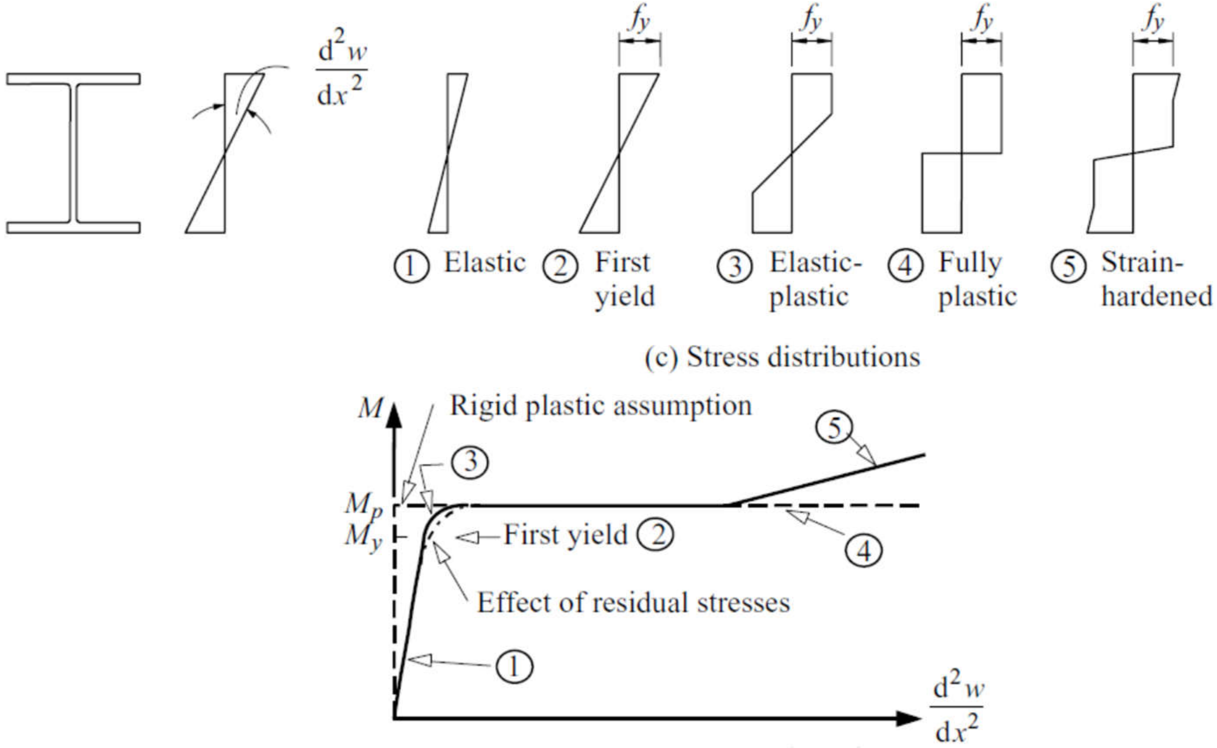

TV. KLASSE 2
P
Mp


P
TV. KLASSE
3


P
TVERRSN. KLASSE
4
f_y

Moment-curvature relationships for steel beams



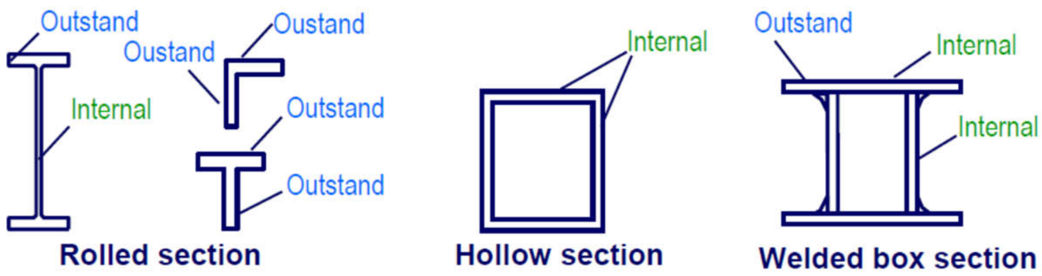
Lower-bound theorem

A chosen (assumed) distribution of stresses in a structure which satisfies equilibrium between internal and external forces and nowhere exceeds the plastic resistance, results in a resistance less or equal to the correct value.

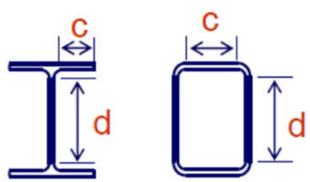
Basis of section classification

- Some parts are outstand:
  - Flanges of I beams
  - Legs of angles and tees
  - Flange part of welded sections

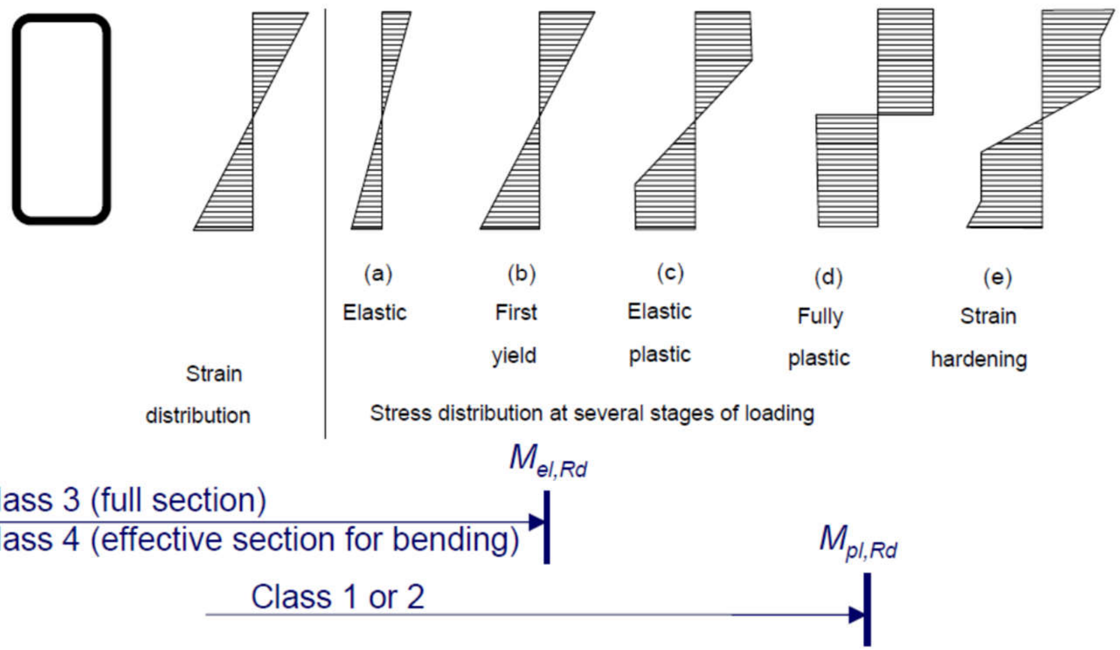
- Some parts are internal:
  - Webs of I beams
  - Walls of hollow sections
  - Flange part/web of welded box sections



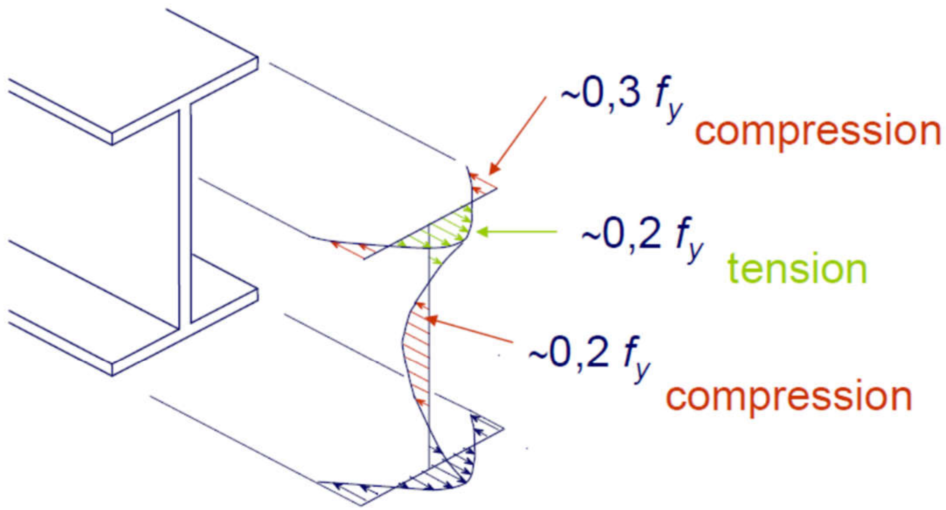
Section classification

Element 	Class 1	Class 2	Class 3
Flange	$c/t_f \leq 9\varepsilon$	$c/t_f \leq 10\varepsilon$	$c/t_f \leq 14\varepsilon$
Web subject to bending	$d/t_W \leq 72\varepsilon$	$d/t_W \leq 83\varepsilon$	$d/t_W \leq 124\varepsilon$
Web subject to compression	$d/t_W \leq 33\varepsilon$	$d/t_W \leq 38\varepsilon$	$d/t_W \leq 42\varepsilon$

Evolution of the direct stress distribution

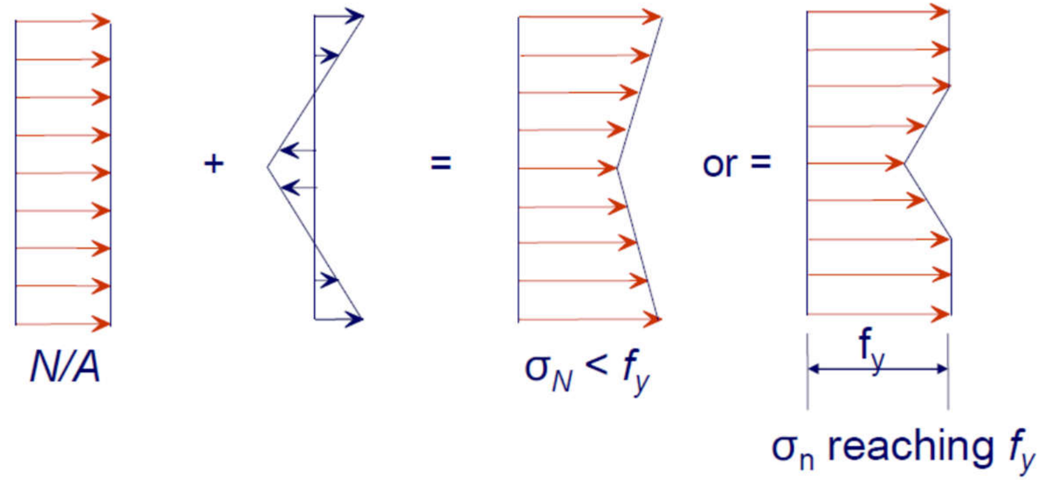


Residual stresses



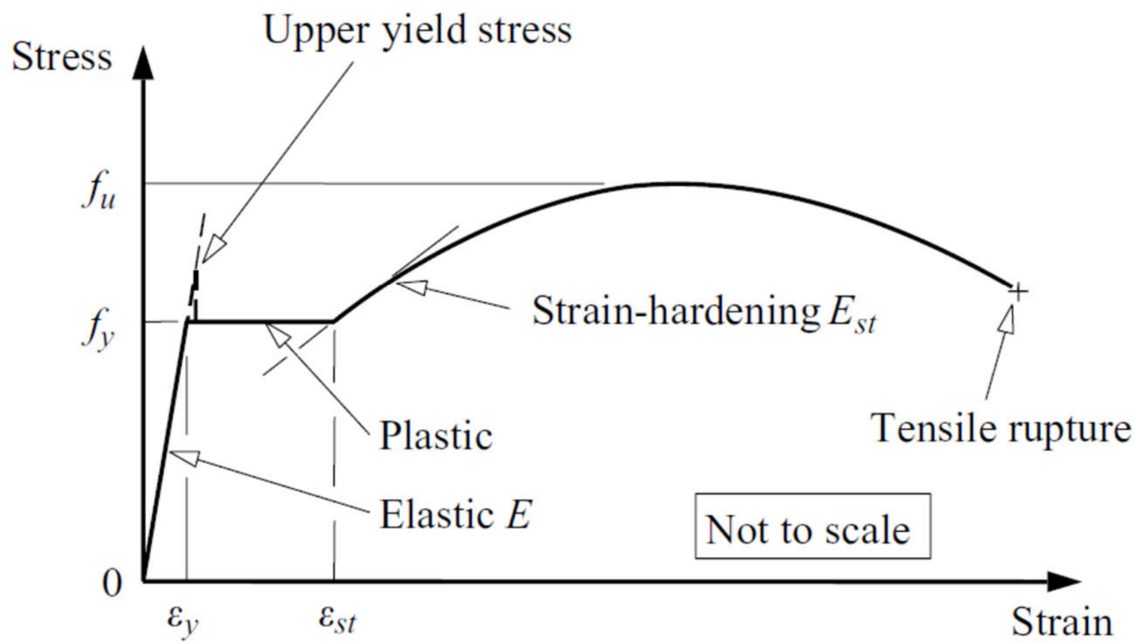
- Due to differential cooling during hot-rolling or welding.
- Above distribution is typical for a hot-rolled section.
- Peak residual stresses are larger (approaching f_y) for welded sections.

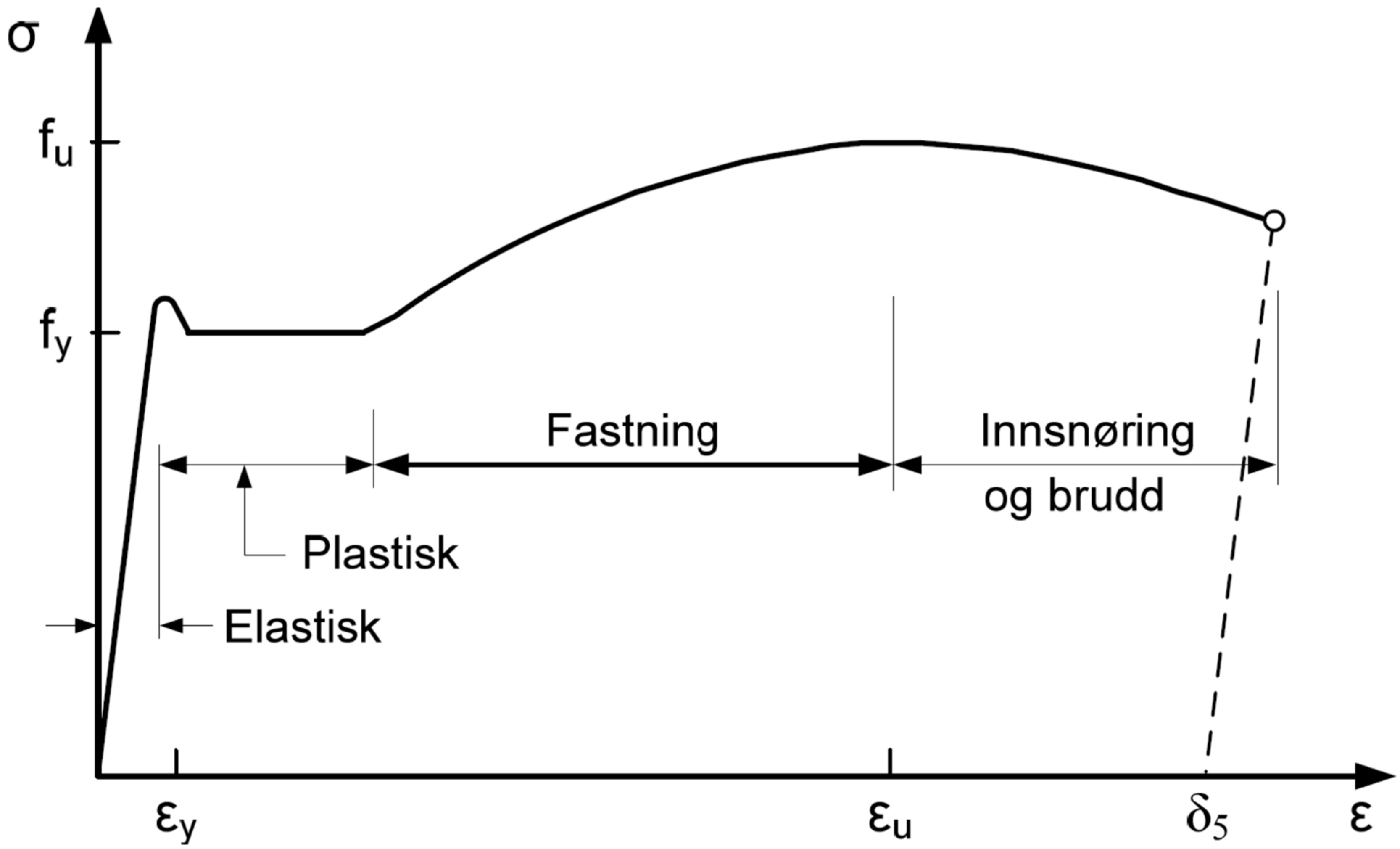
Effect of residual stresses



- Cause early yielding when combined with axial stresses.
- Reduces the flexural stiffness.

Idealised stress-strain behaviour

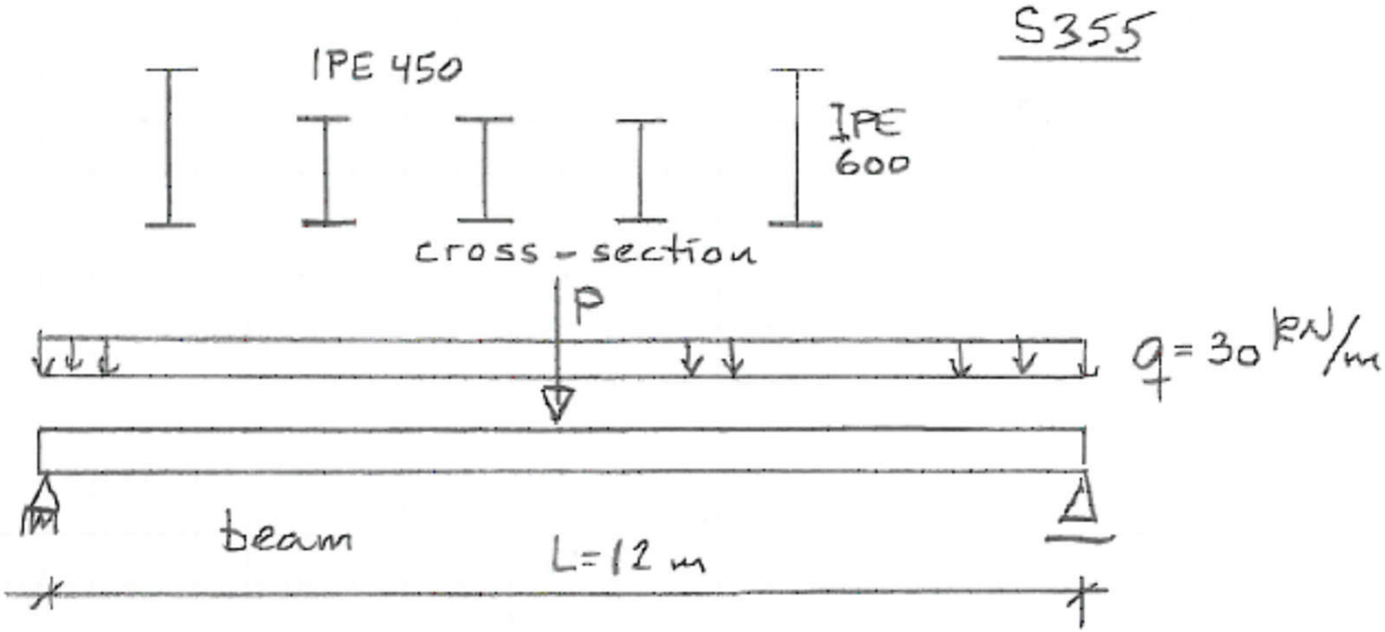



σ
f_u
f_y
Fastning
Innsnøring
og brudd
Plastisk
Elastisk
ε_y
ε_u
δ_5
ε

Moment resistance

The design value of the bending moment M_{Ed} at each cross-section shall satisfy

$$M_{Ed} \leq M_{c,Rd} = \begin{cases} M_{pl,Rd} = \dfrac{W_{pl} f_y}{\gamma_{M0}} & \text{Class 1 and 2} \\[2ex] M_{el,Rd} = \dfrac{W_{el,min} f_y}{\gamma_{M0}} & \text{Class 3} \\[2ex] M_{o,Rd} = \dfrac{W_{eff,min} f_y}{\gamma_{M0}} & \text{Class 4} \end{cases}$$


S355
IPE 450
IPE 600
cross - section
P
q = 30 kN/m
beam
L = 12 m

Gitt ei bru med lengde l = 12 m. Brua holdes oppe med 3 IPE 450 bjelker og 2 IPE 600 bjelker. Materialkvalitet S355 og materialkoefisient γ m= 1.05.

Det antas at sveisene har tilstrekkelig kapasitet og at tverrskottene er tilstrekkelig stive til ikke å deformeres i sitt plan når tverrsnittet belastes. Det er 3 meter mellom hvert tverrskott. Lasten N er en ren sentrisk last.

i), Beregn bruas elastiske kapasitet

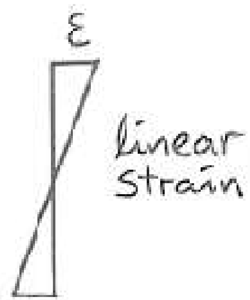
ii). Beregn bruas plasiske kapasitet.

Jevnt fordelt last: $q := 30 \cdot \frac{kN}{m}$

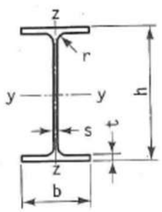
Material factor: $\gamma_m := 1.05$

Bridge length: $L := 12000 \cdot mm$

Det antas at dekket på brua ikke bidrar til kapasiteten. Bjelkene er koblet til dekket slik at de får samme defleksjon og identisk kurvatur. Spenningene blir størst i bjelkene med størst høyde siden tøyningen økes lineært med høyden.



TABELL 1.1 VARMVALSEDE IPE - BJELKER



Dimensjoner etter NS-EN 10 034
Materiale etter NS-EN 10 025

Betegnelse f.eks.: Bjelke NS-EN 10 034 IPE 200
Stål NS-EN 10 025 .....

IPE	Dimensjoner (mål i mm)					Masse	A $\cdot 10^{-3}$	y - y			z - z			I_T $\cdot 10^{-3}$	S_y $\cdot 10^{-3}$	C_w $\cdot 10^{-9}$	Hullavstand / hulldiam. (i mm)		
								$I \cdot 10^{-6}$	$W \cdot 10^{-3}$	i	$I \cdot 10^{-6}$	$W \cdot 10^{-3}$	i						
	h	b	s	t	r	kg/m	mm^2	mm^4	mm^3	mm	mm^4	mm^3	mm	mm^4	mm^3	mm^6	w	w_1	d
80	80	46	3,8	5,2	5	6,00	0,764	0,801	20,0	32,4	0,085	3,69	10,5	7,00	11,6	0,118	26	-	-
100	100	55	4,1	5,7	7	8,10	1,03	1,71	34,2	40,7	0,159	5,79	12,4	12,1	19,7	0,351	30	-	-
120	120	64	4,4	6,3	7	10,4	1,32	3,18	53,0	49,0	0,277	8,65	14,5	17,4	30,4	0,890	36	-	-
140	140	73	4,7	6,9	7	12,9	1,64	5,41	77,3	57,4	0,449	12,3	16,5	24,5	44,2	1,981	40	-	-
160	160	82	5,0	7,4	9	15,8	2,01	8,69	109	65,8	0,683	16,7	18,4	36,2	61,9	3,959	44	-	13
180	180	91	5,3	8,0	9	18,8	2,39	13,20	146	74,2	1,01	22,2	20,5	48,0	83,2	7,431	50	-	13
200	200	100	5,6	8,5	12	22,4	2,85	19,4	194	82,6	1,42	28,5	22,4	70,2	110	12,99	56	-	13
220	220	110	5,9	9,2	12	26,2	3,34	27,7	252	91,1	2,05	37,3	24,8	91,0	143	22,67	60	-	17
240	240	120	6,2	9,8	15	30,7	3,91	38,9	324	99,7	2,84	47,3	26,9	129	183	37,39	68	-	17
270	270	135	6,6	10,2	15	36,1	4,59	57,9	429	112	4,20	62,2	30,2	160	242	70,58	72	-	17
300	300	150	7,1	10,7	15	42,2	5,38	83,6	557	125	6,04	80,5	33,5	202	314	125,9	80	-	23
330	330	160	7,5	11,5	18	49,1	6,26	117,7	713	137	7,88	98,5	35,5	283	402	199,1	86	-	25
360	360	170	8,0	12,7	18	57,1	7,27	162,7	904	150	10,4	123	37,9	375	510	313,6	90	-	25
400	400	180	8,6	13,5	21	66,3	8,45	231,3	1160	165	13,2	146	39,5	514	654	490,0	96	-	28
450	450	190	9,4	14,6	21	77,6	9,88	337,4	1500	185	16,8	176	41,2	671	851	791,0	106	-	28
500	500	200	10,2	16,0	21	90,7	11,6	482,0	1930	204	21,4	214	43,1	907	1100	1249	110	-	28
550	550	210	11,1	17,2	24	106	13,4	671,2	2440	223	26,7	254	44,5	1240	1390	1884	116	-	28
600	600	220	12,0	19,0	24	122	15,6	920,8	3070	243	33,9	308	46,6	1660	1760	2846	120	-	28

Cross sectional area:

$H := 600 \cdot mm \qquad t_H := 12 \cdot mm$

$B := 220 \cdot mm \qquad t_B := 19 \cdot mm$

$A := \left[\left(H - 2 \cdot t_B \right) t_H + B \cdot t_B \cdot 2 \right]$

$A = 15104 \cdot mm^2$

Cross sectional Inertia strong axis :

$I_x := \frac{1}{12} \cdot t_H \cdot \left(H - 2 \cdot t_B \right)^3 + 2B \cdot t_B \cdot \left(\frac{H - t_B}{2} \right)^2 \qquad W_x := \frac{I_x}{\frac{H}{2}}$

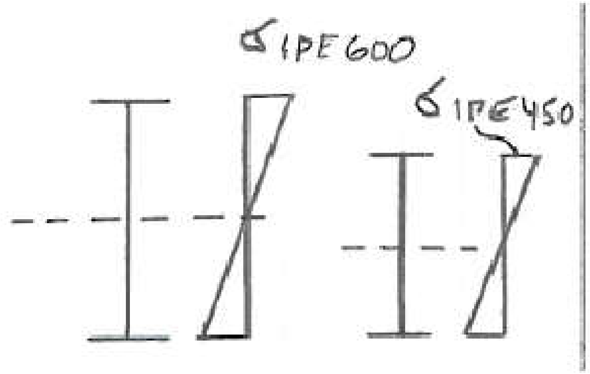
$I_x = 8.83 \times 10^8 \cdot mm^4$

$\boxed{W_x = 2.943 \times 10^6 mm^3}$

Cross sectional Inertia weak axis:

$I_y := \left[\frac{1}{12} \cdot t_B \cdot B^3 \cdot 2 + \frac{1}{12} \cdot \left(H - 2t_B \right) \cdot t_H^3 \right]$

$I_y = 3.38 \times 10^7 \cdot mm^4$



$$H_{IPE450} := 450 \cdot mm$$

$$H_{IPE600} := 600 \cdot mm$$

$$\alpha_{IPE450} := \frac{H_{IPE450}}{H_{IPE600}} \qquad \alpha_{IPE450} = 0.75$$

Momentbelastning midt i spennet:

$$M_{ed} := \frac{q \cdot L^2}{8} + \frac{P \cdot L}{4}$$

$$\boxed{W_{dIPE600} := 3070 \cdot 10^3 \cdot mm^3}$$

$$W_{dIPE450} := 1500 \cdot 10^3 \cdot mm^3$$

$$M_{dIPE600} := W_{dIPE600} \cdot f_d$$

$$M_{dIPE600} = 1.038 \times 10^3 \, kN \cdot m$$

$$M_{dIPE450} := W_{dIPE450} \cdot f_d$$

$$M_{dIPE450} = 507.143 \, kN \cdot m$$

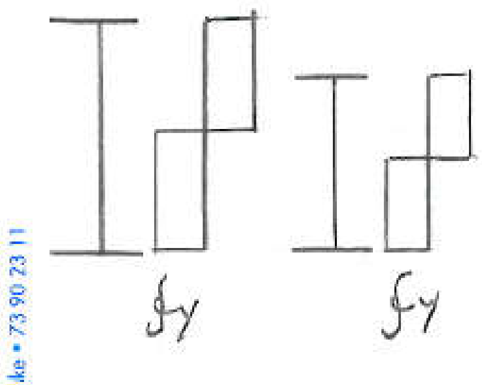
$$M_{kap} := 2 \cdot M_{dIPE600} + 3 \cdot \alpha_{IPE450} \cdot M_{dIPE450}$$

$$M_{kap} = 3217 \, kN \cdot m$$

Dvs. at kapasiteten til brua er:

$$P := \frac{\left(M_{kap} - \frac{q \cdot L^2}{8}\right) \cdot 4}{L} \qquad P = 892.325\,kN$$

Momentkapasitet:



Plastisk momentkapasitet:

$$A_{IPE600} := 15.6 \cdot 10^3 \cdot mm^2$$

$$A_{flens} := B \cdot t_B \qquad A_{flens} = 4180 \cdot mm^2$$

$$A_{steg} := (H - 2t_B) \cdot t_H \qquad A_{steg} = 6744 \cdot mm^2$$

$$W_{dIPE600} := \left[\frac{H - 2t_B}{2} \cdot t_H \cdot \left[\frac{(H - t_B)}{4}\right] + A_{flens} \cdot \left[\frac{(H - t_B)}{2}\right]\right] \cdot 2$$

$$W_{dIPE600} = 3.408 \times 10^6 \cdot mm^3$$

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$$I := W_{dIPE600} \cdot 300 \cdot mm$$

$$I = 1.022 \times 10^{9} \cdot mm^{4}$$

$$H := 450 \cdot mm$$

$$B := 190 \cdot mm$$

$$t_B := 14.6 \cdot mm$$

$$t_H := 9.4 \cdot mm$$

$$A_{IPE450} := 9.88 \cdot 10^{3} \cdot mm^{2}$$

$$A_{flens} := B \cdot t_B \qquad A_{flens} = 2774 \cdot mm^{2}$$

$$A_{steg} := (H - 2t_B) \cdot t_H \qquad A_{steg} = 3956 \cdot mm^{2}$$

$$W_{dIPE450} := \left[\frac{H - 2t_B}{2} \cdot t_H \cdot \left[\frac{(H - t_B)}{4}\right] + A_{flens} \cdot \left[\frac{(H - t_B)}{2}\right]\right] \cdot 2$$

$$W_{dIPE450} = 1.638 \times 10^{6} \cdot mm^{3}$$

$$M_{dIPE600} := W_{dIPE600} \cdot f_d$$

$$M_{dIPE600} = 1152.28 \cdot kN \cdot m$$

$$M_{dIPE450} := W_{dIPE450} \cdot f_d$$

$$M_{dIPE450} = 553.921 \cdot kN \cdot m$$

$$M_{kap} := 2 \cdot M_{dIPE600} + 3 \cdot M_{dIPE450} \qquad f_d = 338.095 \cdot MPa$$

$$M_{kap} = 3966 \cdot kN \cdot m$$

Dvs. at kapasiteten til brua er:

$$P := \frac{\left(M_{kap} - \frac{q \cdot L^2}{8}\right) \cdot 4}{L} \qquad P = 1142 \cdot kN$$

$$S_{XIPE600} := 1760 \cdot 10^3 \cdot mm^3$$

$$M_{dIPE450} := 2 \cdot S_{XIPE600} \cdot f_d \qquad M_{dIPE600} = 1152.28\,kN \cdot m$$

$$S_{XIPE450} := 851 \cdot 10^3 \cdot mm^3$$

$$M_{dIPE450} := 2 \cdot S_{XIPE450} \cdot f_d \qquad M_{dIPE450} = 575.438\,kN \cdot m$$

$$M_{kap} := 2 \cdot S_{XIPE600} \cdot 2 \cdot f_d + 3 \cdot S_{XIPE450} \cdot 2 \cdot f_d$$

$$M_{kap} = 4107 \cdot kN \cdot m$$

$$P := \frac{\left(M_{kap} - \frac{q \cdot L^2}{8}\right) \cdot 4}{L} \qquad P = 1189 \cdot kN$$