LVL Rib Panels

STRUCTURAL DESIGN MANUAL Aug-21

LVL Rib Panels by Stora Enso

Structural design manual

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STRUCTURAL DESIGN MANUAL

1. Product description

1.1 Basic product information

LVL Rib Panels by Stora Enso are intended to be used as structural or non-structural elements in buildings.
LVL Rib Panels by Stora Enso are composite slab elements made of X- and S types structural LVL (Laminated Veneer Lumber). The adhesive is of type EN 15425 polyurethane (PUR) adhesive. LVL Rib Panels by Stora Enso may contain screws and nails, but they do not have an influence on the composite effect but they are only used to fix secondary construction elements.
With regard to moisture behaviour of the product, the product shall be used in service classes 1,2 according to EN 1995 1.1. If LVL Rib Panels by Stora Enso are intended to be a part of the external envelope of the building, they shall be protected adequately, e.g. by a roof or by cladding.
LVL timber members comply with EN 14374. Figure 1 and Figure 10 show the rib panel and LVL material's principal direction of grain.

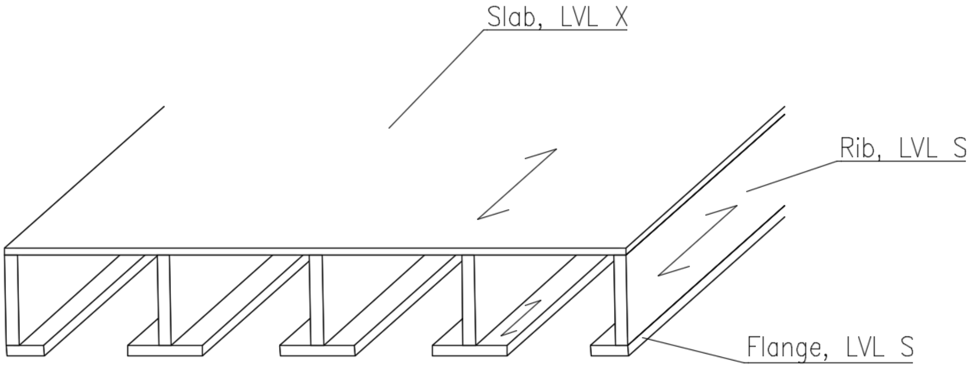


Figure 1: Rib panel. Arrows shows the main direction of grains.

The panel and the rib are glued to another. The pressure is being applied by screw-glue application. Figure 2, Figure 3 and Figure 4 show the different options for rib or box panels.

There are four types of Rib Panel available depending on the construction requirements:

- Open,
- Semi-Open
- Closed
- Inverted type.

LVL Rib Panel Open type

"T" shaped section with a top chord panel using LVL-X and a rib using LVL-S.

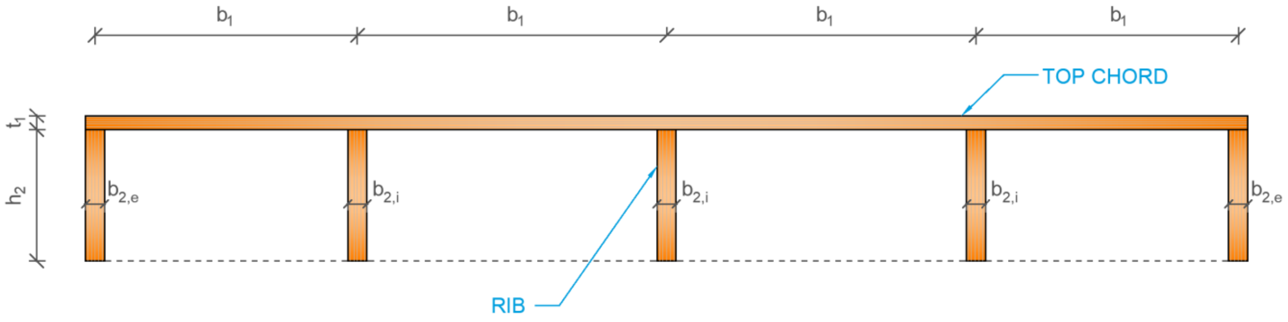


Figure 2: LVL Rib panel- Open type.

LVL Rib Panel Semi-Open type

"I" shaped section with a top chord panel using LVL-X, a rib using LVL-S and a bottom tension flange using LVL-S.

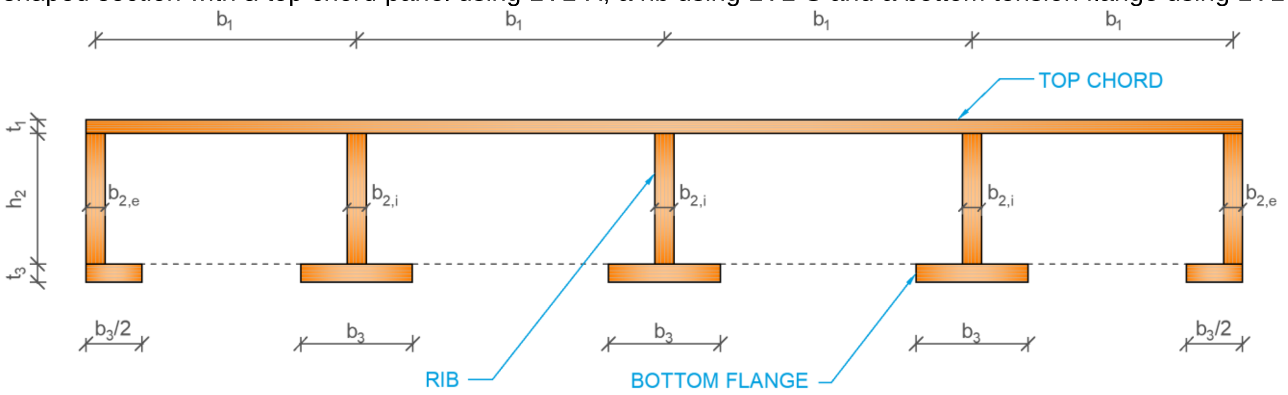


Figure 3: LVL Rib panel- Semi open type.

LVL Rib Panel Closed type

"I" shaped section with a top chord panel using LVL-X, a rib using LVL-S and a bottom chord using LVL-X. LVL-S can be an option if it is required.

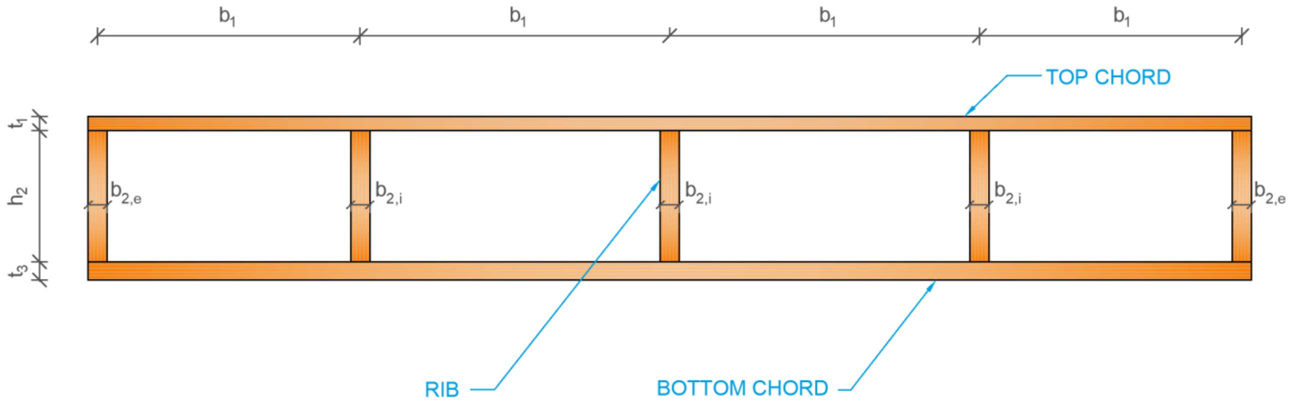


Figure 4: LVL Rib panel- Closed type.

LVL Rib Panel Inverted type

"I" shaped section with a bottom chord panel using LVL-X, and a rib using LVL-S .

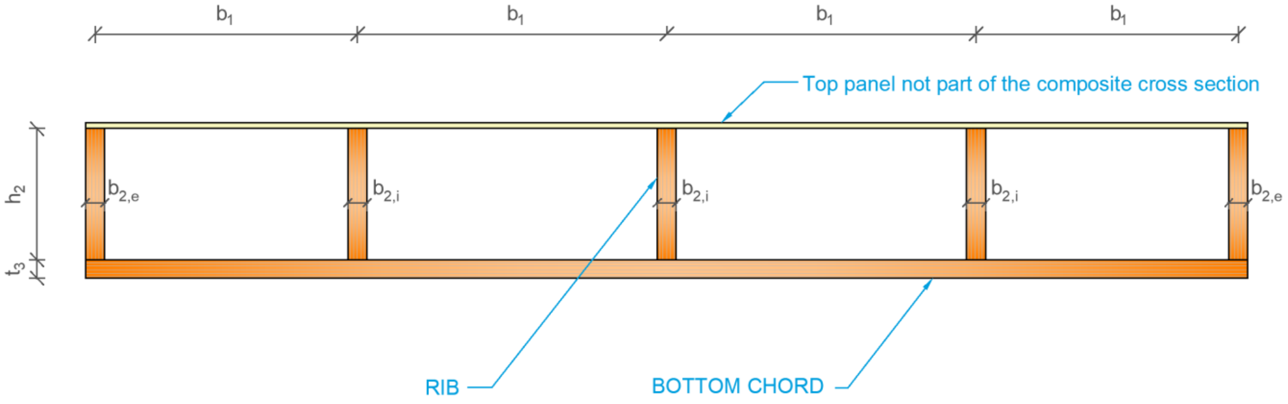


Figure 5: LVL Rib panel- Closed type.

The products are shaped rectangular or trapezoid with parallel longitudinal edges and at maximum 30 degrees angle of skew end/end. The maximum length of the elements is 24m and the typical height up to 1200mm. Typical lengths are from 6 m to 18 m and width up to 2,5m.

Note that the chord and flange nominal thicknesses t1 and t3 should be sanded.

1.2 Tolerances

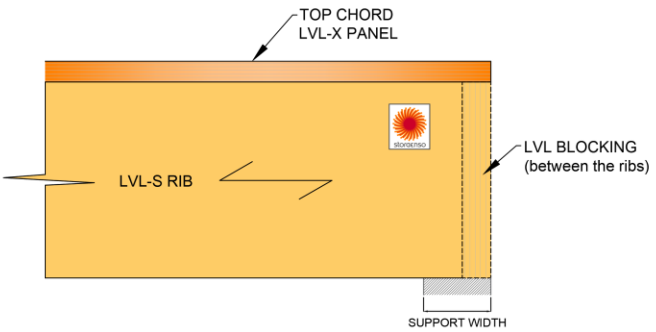
Tolerances of the LVL Rib Panels by Stora Enso

Dimension		Tolerance [mm] or [%]
Depth of the LVL Rib Panel	$h_{rib} \leq 300mm$	+/- 2 mm
	$300mm < h_{rib} \leq 600mm$	+/- 3 mm
	$h_{rib} > 600mm$	-0,5 %·h / +1 %·h
Width of the LVL Rib Panel		± 2 mm or ± 0,5 %
Length of the LVL Rib Panel		± 5 mm

All tolerances are based on standard LVL production with a moisture content of 10 ± 2%.

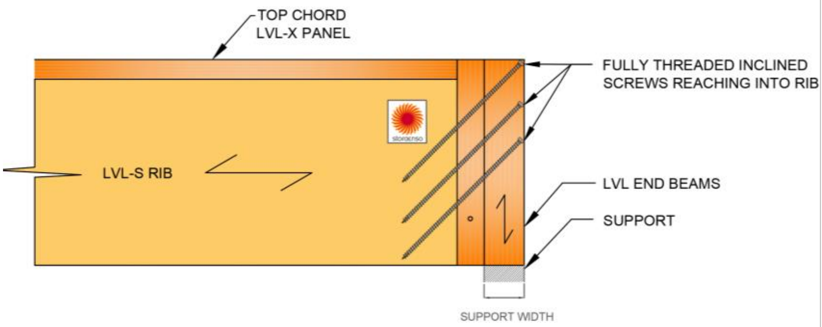
1.3 Support types

Simple support
LVL RP Open type



The ribs of LVL Rib Panels must be maintained at their supports. The simple configuration for maintaining the ribs is to place LVL-X blockings between the ribs. This configuration is recommended when an additional element put pressure on top like a wall at the supports. In this case, the blocking can carry vertical loads from the pressure above.
When the blocking cannot be placed at the end due to the type of connection used e.g. with a hanger supporting the rib, it is possible to shift the blocking row to allow more space. But if at the support, no bearing pressure is applied on the top cord, blockings are not mandatory.

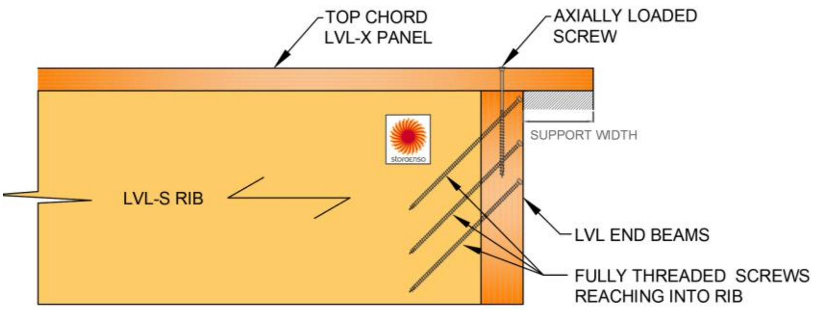
End beam support
LVL RP Open type



The ribs of LVL Rib Panels must be maintained at their supports. The basic configuration for maintaining the ribs is to close the element with a LVL-X end beam.

In case of end beam support the LVL Rib Panel is supported by end beam.
In the outer part of the end beam grains are vertical and this part bears the support reaction force. The inner part has horizontal grain direction and this part stiffens the outer part. The LVL RP is supported off the LVL-S rib and shall be hung off the end beam using fully threaded screws inserted from the side, in an angle of 45°, so the screws work in tension. Shear forces are transferred by inclined screwed connection between the rib and the end beam.
This special type of support is suited when the ends of the rib panel elements are between the lower and upper wall elements. The compression deformation is minimized by the end beams at the supports.

Suspended support
LVL RP Open type

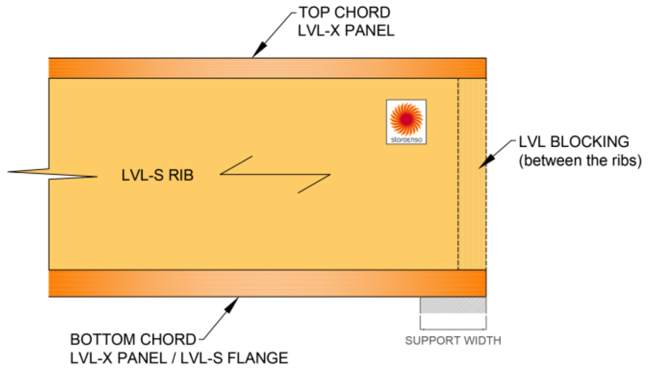


In a suspended support, the LVL RP is suspended by the slab. The slab is connected to the end beam (horizontal grains) with the structural screws axially loaded. The end beam is connected to the ribs with inclined structural screws as in the end beam support.
Support reaction is transferred from the slab to end beam by large head tensile screws and from end beam to the ribs by fully threaded inclined screws.

The same type of support is possible with LVL Rib Panel Semi-Open and Closed types.

Simple support

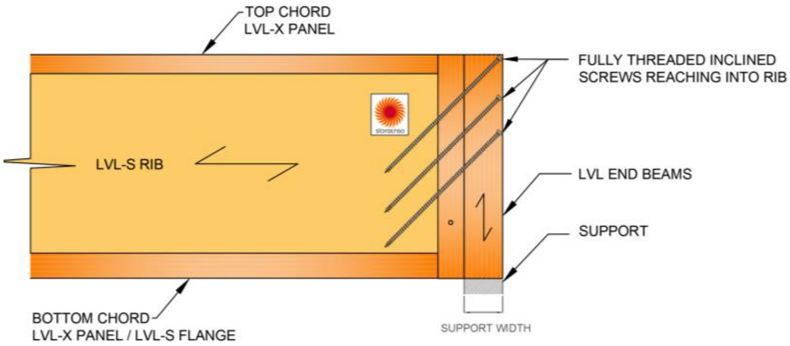
LVL RP Semi-Open/Closed type



The **support width** of LVL Rib Panel (L$_{support}$) is calculated so that forces can pass through the different elements of the Rib Panel by contact without exceeding the bearing pressure resistance capacity of the ribs and of the possible lower chord/flange.
It is necessary to provide a gap for the installation of LVL Rib Panels during the construction phase. The value of this gap depends on the tolerances of the element and the environment in which it is placed. This gap also allows to manage and prevent the dimensional variations (shrinkage & swelling) of the LVL elements which constitutes the Rib Panel.
Thus, in order to calculate a minimum support width, it is necessary to add to the calculated support width the functional gap as well as the width of the end beam.

End beam support

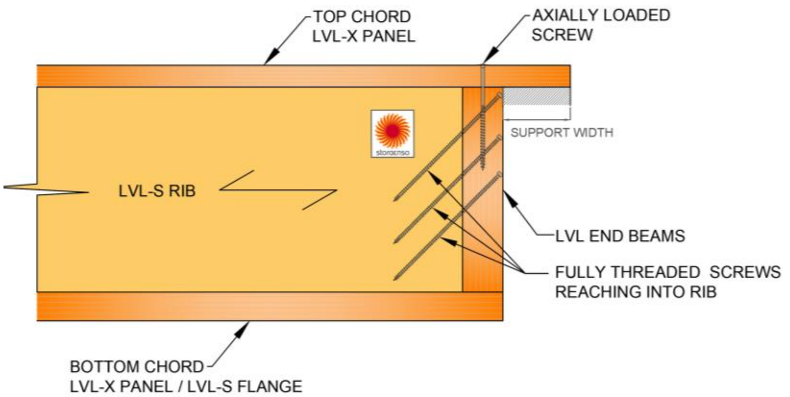
LVL RP Semi-Open/Closed type



At the end beam support, additional screws are usually needed to transfer the shear force between the parts. If the capacity of main screws on the outer beam is enough to transfer the shear load between the parts, additional screws are not needed. The additional screw type should be the same as the main screws and the screwing angle of additional screws should be same as for the main screws also.
In some cases, the first end beam can be used to close the rib panel at the end and placed on a simple support type.

Suspended support

LVL RP Semi-Open/Closed type



Depending on the intensity of the forces to carry, the configuration of the building, the habits of construction or installation work, it is possible to choose one or more types of support. The list of support details described in these pages is not exhaustive.

Figure 6:Different support types

The blocking elements shall be in LVL-X.

1.4 Blockings

At the support lines:

Adding blockings between the ribs at the supports is necessary in two cases:

1) When a bearing element applies a vertical linear force (blue) on the rib panel at the support, for example a wall from the upper level inducing compression at the support. In this case, blocking lines can carry the vertical loads originating from the compression of the upper element at the support and transfer them into the lower bearing elements.
2) When a rib panel is used as a diaphragm, horizontal forces originating from the upper vertical element (shear wall) transferred into the support line at the ends go through the blockings placed between the ribs and reach the lower resisting element. Thus, the role of the intermediate blockings at the supports is to transfer horizontal forces (green) in a continuous way from the upper vertical element to the lower vertical resisting element and preventing the rotation (torsion) of the ribs in transversal direction. (See Figure 7 to illustrate the role of the blockings at the supports)

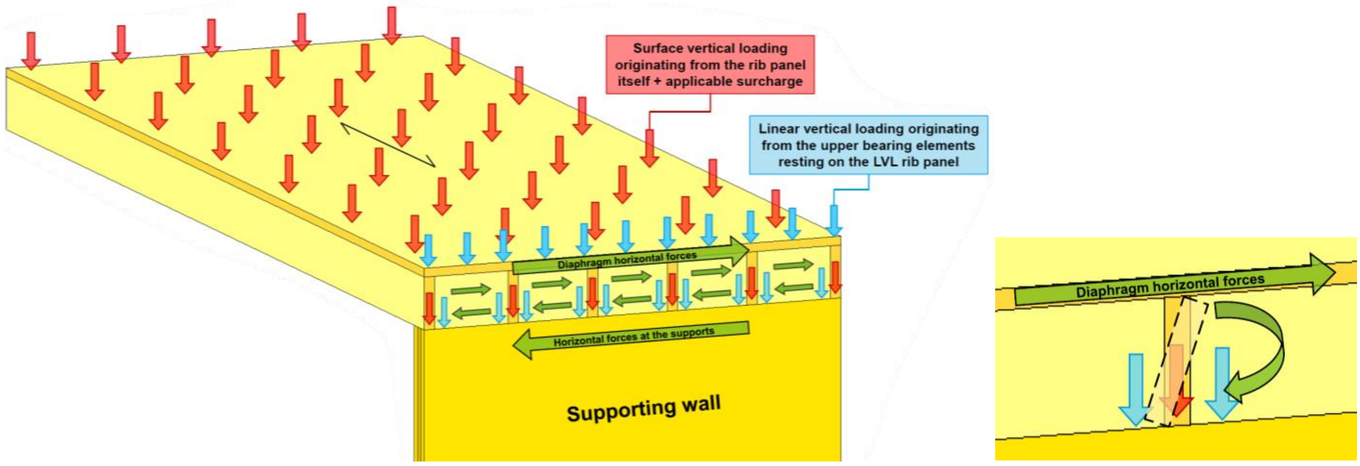


Figure 7: Load distribution into simple supports through blockings (blue) and through ribs (red)

Note : The closing end beam used as a continuous blocking assuring the same function as the intermediate blockings between the ribs presented in the Figure 7 can be taken as an alternative under the condition that the vertical loads originating from the rib panel and applicable surcharge (red) are correctly transferred from the ribs to the supports.

The blokings can prevent the lateral torsional buckling (LTB) of the ribs in some cases :

In the bay:

When vibration design in serviceability limit state doesn't meet the requirements, it is recommended to add blocking lines. The blocking line will act as a beam in the crosswise direction and significantly helps to achieve the requirement of deflection under 1kN point load. Blockings between the ribs can reduce deflection under point loads, but they need to be fixed well to the ribs to give the desired transverse stiffness and to avoid creaking in the long term.

For that purpose, it has been analyzed that two blocking lines close to the center of the span is a more efficient solution for long span floors than the blocking in e.g. 1/3 points.

Two blocking lines at 1m spacing at the center of the span is recommended for L>4m span length. (see Figure 8 and Figure 9).

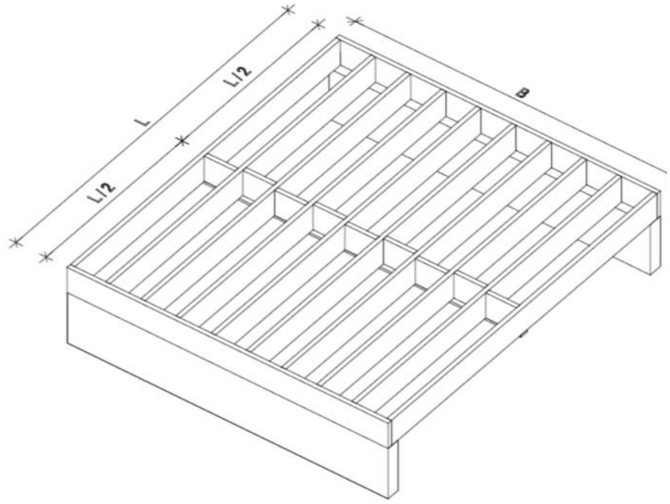


Figure 8: One line of blockings

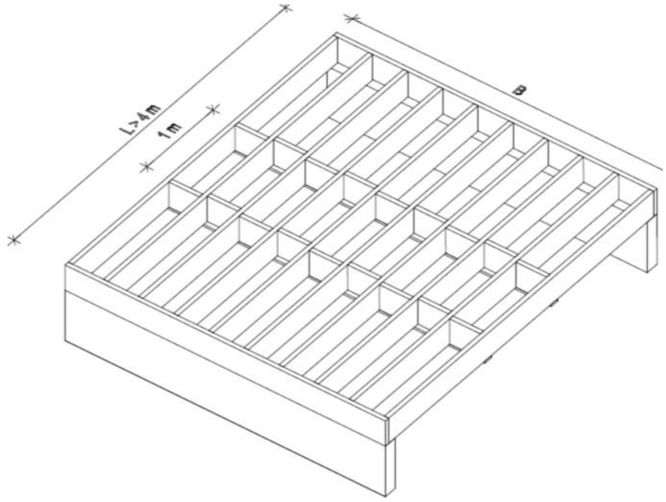


Figure 9: Two lines of blockings for span >4m

The stiffness of the bracing lines can be estimated according to EN1995-1-1 and is divided by the width of the floor.
In rib panels lateral torsional buckling is not an issue, but the lifting points need to be considered and therefore other locations of the transfer bracing lines may be needed.

2. Design principles

This design document is made for typical one span LVL rib panel without cantilever. The slab is exposed to dead loads, imposed loads and snow loads.

The slab is designed according to EN 1995.1-1 and actual National Annex.

3. Material

The material values are given in the following tables.

These values are based on declared values given in reports:

- “Varkaus 2016-12-21 DoP LVL by Stora Enso, S grade, thickness 24-75mm”
- “Varkaus 16.2.2017 DoP LVL by Stora Enso, X grade, thicknesses 24-69mm”
- “VTT analysis reports VTT-S-05550-17 [1] and VTT-S-05710-17” [2]

In Figure 10 strength and stiffness orientations for LVL are presented.

Table 1: Characteristic strength values.

Characteristic values [N/mm²]	**Symbol**	**Figure 10**	**LVL-S 24 - 75 mm**	**LVL-X 24 - 69 mm**
Bending strength: Edgewise (depth 300 mm)	$f_{m,0,edge,k}$	A	44	32
Bending strength: Size effect parameter	s	A	0.12	0.12
Bending strength: Flatwise, parallel to grain	$f_{m,0,flat,k}$	B	50	36
Bending strength: Flatwise, perpendicular to grain	$f_{m,90,flat,k}$	C	-	8
Tensile strength: Parallel to grain (length 3,000 mm)	$f_{t,0,k}$	D	35	26
Tensile strength: Perpendicular to grain, edgewise	$f_{t,90,edge,k}$	E	0.8	6
Tensile strength: Perpendicular to grain, flatwise	$f_{t,90,flat,k}$	F	0,35	-
Compressive strength: Parallel to grain	$f_{c,0,k}$	G	35	26
Compressive strength: Perpendicular to grain, edgewise	$f_{c,90,edge,k}$	H	6	9
Compressive strength: Perpendicular to grain, flatwise	$f_{c,90,flat,k}$	I	2,2	2.2
Shear strength: Edgewise	$f_{v,0,edge,k}$	J	4.2	4.5
Shear strength: Flatwise, parallel to grain	$f_{v,0,flat,k}$	K	2.3	1.3
Shear strength: Flatwise, perpendicular to grain	$f_{v,90,flat,k}$	L	-	0.6

Table 2: Characteristic stiffness values and density.

Characteristic values [N/mm²]	**Symbol**	**Figure 10**	**LVL-S 24 - 75 mm**	**LVL-X 24 - 69 mm**
Modulus of elasticity: Parallel to grain	$E_{0,k}$	ABDG	11600	8800
Modulus of elasticity: Compression perpendicular to grain, edgewise	$E_{c,90,edge,k}$	H	-	2000
Modulus of elasticity: Compression perpendicular to grain, flatwise	$E_{c,90,flat,k}$	I	-	-
Modulus of elasticity: Bending perpendicular to face veneer grain	$E_{m,90,k}$	C	-	1700
Shear modulus: Edgewise	$G_{0,edge,k}$	J	400	400
Shear modulus: Flatwise, parallel to grain	$G_{0,flat,k}$	K	250	100
Shear modulus: Flatwise, perpendicular to grain	$G_{90,flat,k}$	L	-	16
Characteristic density [kg/m³]	ρ_k	-	480	480

Table 3: Mean stiffness values and density.

Mean values [N/mm²]	**Symbol**	**Figure 10**	**LVL-S 24 - 75mm**	**LVL-X 24 - 69mm**
Modulus of elasticity: Parallel to grain	$E_{0,mean}$	ABDG	13800	10500
Modulus of elasticity: Compression perpendicular to grain, edgewise	$E_{c,90,edge,mean}$	H	-	2400
Modulus of elasticity: Compression perpendicular to grain, flatwise	$E_{c,90,flat,mean}$	I	-	-
Modulus of elasticity: Bending perpendicular to face veneer grain	$E_{m,90,mean}$	C	-	2000
Shear modulus: Edgewise	$G_{0/90,edge,mean}$	J	600	600
Shear modulus: Flatwise, parallel to grain	$G_{0,flat,mean}$	K	460	120
Shear modulus: Flatwise, perpendicular to grain	$G_{90,flat,mean}$	L	-	22
Mean density [kg/m³]	ρ_{mean}	-	510	510

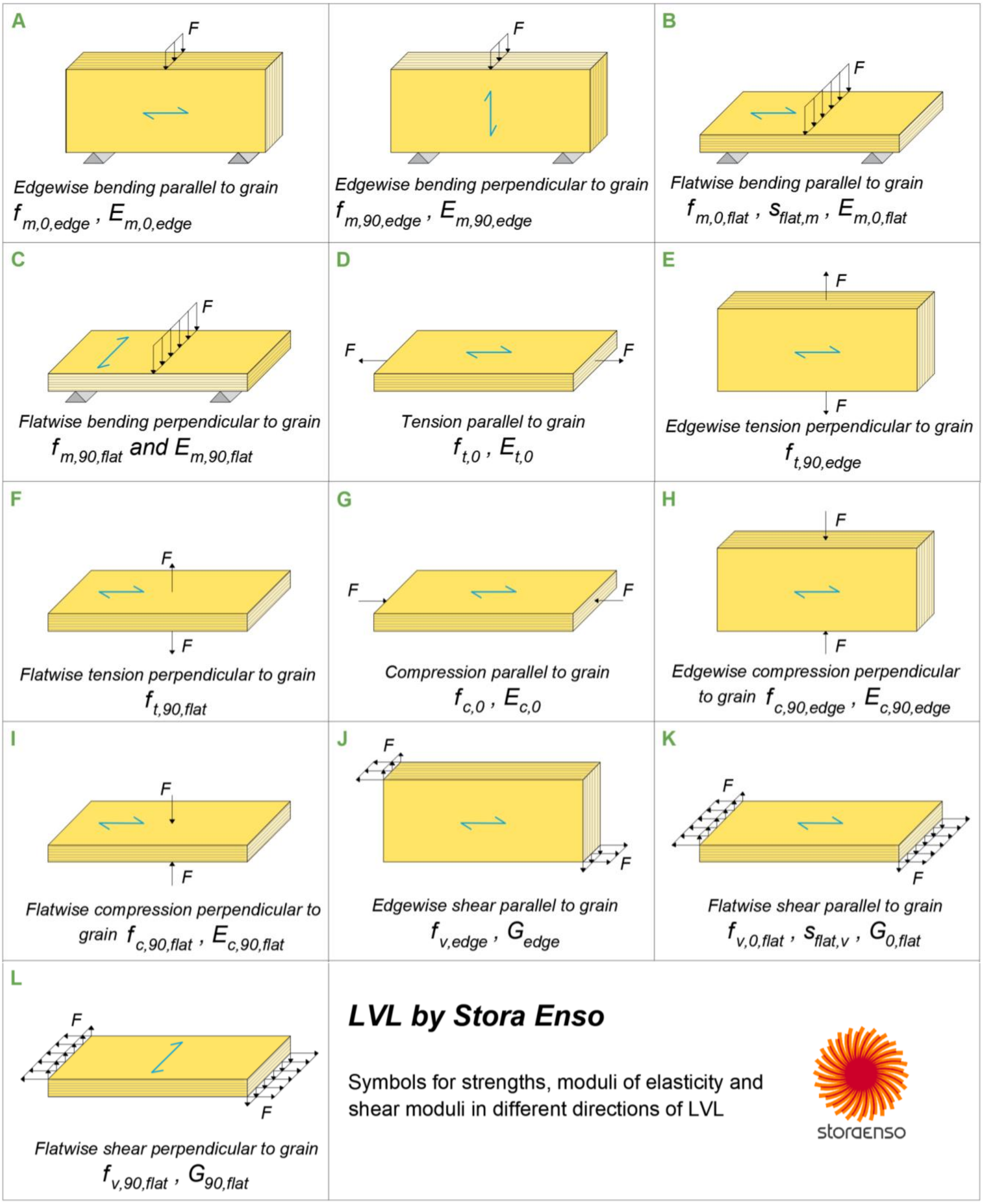


Figure 10: Strength and stiffness orientations.

3.1 Partial factor for the material

For LVL partial safety factor is according to EN 1995-1-1:

$\gamma_M = 1.2$

Exception: if national annex of EN 1995-1-1 is overruling this value.

3.2 Partial safety factors for loads and load combinations

The design value E_d for the effects of actions shall be calculated using EN 1990 and its national annexes. Partial factors for loads, load combinations and for different consequence classes are taken from the appropriate national annex.

3.3 Factors accounting for the load duration and moisture content

The moisture content and the load duration shall be taken according to the appropriate national annex. The modification factors for strength k_{mod} and deformation k_{def} shall be according to EN1995-1-1 and the applicable NA. A recommendation is given in the tables below.

Table 4 states load-duration class and load types.

Table 4: Load-duration class and load types.

Load-duration class	**Action time of the load**	**Load**
Permanent	More than 6 months	Self-weight, imposed load Category E
Medium-term	10 minutes - 6 months	Imposed floor load, snow
Instantaneus	Below 10 minutes	Wind, accidental load

In Table 5, values for k_{mod} and in Table 6, values for k_{def} can be found.

Table 5: Strength modification factor k_{mod}.

	Load-duration class		
Service class	Permanent action	Medium term action	Instantaneous action
1	0.60	0.80	1.10
2	0.60	0.80	1.10
3	0.50	0.65	0.90

Table 6: The values of the deformation factor k_{def}.

	Service class	
	1	2
LVL-X edgewise LVL-S edgewise LVL-S flatwise	0.60	0.80
LVL-X flatwise	0.80	1.00

The values in the second row are used only for flatwise bending and flatwise shear deformation of LVL X. Otherwise the K_{def} of LVL-X has the same values as for LVL S.

The service classes are defined in the standard EN 1995-1-1. Service class 1 means that the structure is in warmed in-doors conditions where the moisture content corresponds mainly to a temperature of 20ºC and the relative humidity of air exceeds 65% only for a few weeks per year.

Deformation factor k_{def} is a factor to evaluate creep deformation.
Most of the flexural rigidity originates from the ribs. Therefore, the simplification of applying a uniform k_{def} to the entire system shall be allowed. In that case, the k_{def} of the rib shall be used:

$$k_{def,uniform} = k_{def,LVL-S} \quad \text{Eq 1}$$

For the thin-flanged beam $k_{def,LVL-S}$ may be used as $k_{def,uniform}$, when the shear deformation is taken into account by supposing all the shear forces to the ribs.
However, a more precise result will be obtained, by applying the k_{def} factor to each individual layer (section component), as demonstrated in chapter 4.

3.4 Size effect parameter

The effect of the size of member shall be taken into consideration in the design (EN 1995-1-1).
For member in bending the characteristic value for $f_{m,0,edge,k}$ shall be multiplied by the factor

$$k_h = min\begin{cases}\left(\frac{300}{h}\right)^s \\ 1.2\end{cases} \quad \text{Eq 2}$$

s is the size effect parameter (defined in EN 14374)
h is the height of the member

For member in tension the characteristic value for $f_{t,0,k}$ shall be multiplied by the factor

$$k_l = min\begin{cases}\left(\frac{3000}{l}\right)^{s/2} \\ 1.1\end{cases} \quad \text{Eq 3}$$

l is the length of the member

3.5 Design values

The design values are calculated as

$$X_d = k_{mod}\frac{X_k}{\gamma_M} \quad \text{Eq 4}$$

X_d is a design strength
X_k is a characteristic strength

3.6 Ultimate limit state

Requirement:

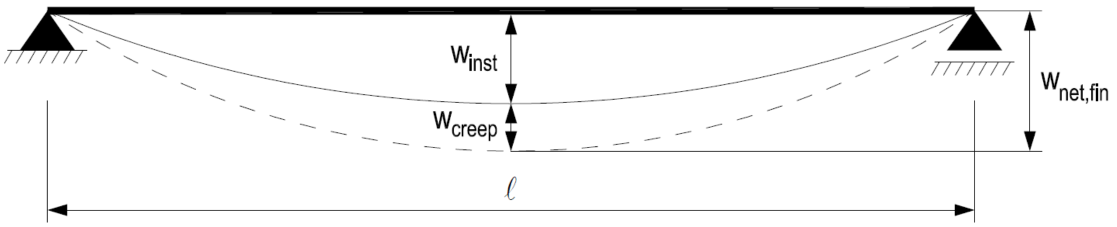
$$E_d \leq R_d \quad \text{Eq 5}$$

where
E_d is the design value of the effect of actions
R_d is the design value of the corresponding resistance

4. Serviceability limit state

4.1 Deformation

The deformation of a structure resulting from the effects of actions (such as axial and shear forces, bending moments and joint slip) and from moisture shall remain within appropriate limits, having regard to the possibility of damage to surfacing materials, ceilings, floors, partitions and finishes, and to the functional needs as well as any appearance requirements.
Limiting values for deformation are taken from EN1995-1-1 [3] and the applicable national annex.
Deformation need to be also limited so that action of the adjacent structures and the intended operation of the building is not disturbed.



Normally limiting deflections for the beams are according to the limits below:

For the instantaneous deflection limit is used

$$w_{inst} \leq \frac{L}{300} \; to \; \frac{L}{500}$$ Eq 6

and the final net deflection limit

$$w_{net,fin} \leq \frac{L}{250} \; to \; \frac{L}{350}$$ Eq 7

The final net deflection limit for slab between the ribs

$$w_{net,fin} \leq \frac{L}{200}$$ Eq 8

These limits shall be used in design according the suitable National Annex.
When calculating instantaneous deformation of the rib panel shear deformation shall be considered and mean values of stiffness properties and the effective cross-section shall be used in the analysis.

The instantaneous deflection may be calculated as:

$$w_{inst} = \underbrace{\sum_{j\geq 1} w_{inst,G,j} + w_{inst,Q1} + \sum_{i>1} \psi_{0,i} \cdot w_{inst,Q,i}}_{characteristic\ load\ combination}$$ Eq 9

The final deflection may be calculated as:

$$w_{fin} = w_{fin,G} + w_{fin,Q,1} + \sum w_{fin,Q,i}$$

$$w_{fin} = \underbrace{w_{inst}}_{characteristic\ load\ combination} + \underbrace{w_{creep}}_{quasi\ permanent\ load\ combination}$$

$$w_{fin} = \underbrace{\sum_{j\geq 1} w_{inst,G,j} + w_{inst,Q1} + \sum_{i>1} \psi_{0,i} \cdot w_{inst,Q,i}}_{characteristic\ load\ combination} + \underbrace{\left[\sum_{j\geq 1} w_{inst,G,j} + \sum_{i\geq 1} \psi_{2,i} \cdot w_{inst,Q,i}\right]}_{quasi\ permanent\ load\ combination} \cdot k_{def,i}$$ Eq 10

With:

Final deflection caused by permanent action

$$w_{fin,G} = w_{inst,G}\left(1 + k_{def}\right)$$ Eq 11

Final deflection caused by the leading variable action

$$w_{fin,Q,1} = w_{inst,Q,1}\left(1 + \psi_{2,1} \cdot k_{def}\right)$$ Eq 12

Final deflection caused by the accompanying variable action

$$w_{fin,Q,i} = w_{inst,Q,i}\left(\psi_{0,i} + \psi_{2,i} \cdot k_{def}\right)$$ Eq 13

The net final deflection may be calculated as:

$$w_{net,fin} = \underbrace{\sum_{j\geq 1} w_{inst,G,j} + w_{inst,Q1} + \sum_{i>1} \psi_{0,i} \cdot w_{inst,Q,i}}_{characteristic\ load\ combination} + \underbrace{\left[\sum_{j\geq 1} w_{inst,G,j} + \sum_{i\geq 1} \psi_{2,i} \cdot w_{inst,Q,i}\right] \cdot k_{def,i}}_{quasi\ permanent\ load\ combination} - \underbrace{w_c}_{camber}$$ Eq 14

$w_{inst,G}$ is instantaneous deformation for action G
$w_{inst,Q,1}$ is instantaneous deformation for action Q_1
$w_{inst,Q,i}$ is instantaneous deformation for action Q_i

4.2 Introducing the creep coefficient k_{def} in a hybrid system

The most precise method is to integrate the k_{def} factor in the flexural rigidity (and shear stiffness) for each individual partial section/material.
The equations below shall illustrate the method, given the following boundary conditions:
single span beam, pin-pin support, continuous loading, K_{def} integrated into the rigidities (instantaneous and final), shear rigidity of the composite section considered, shear deformation considered.

4.2.1 Precise method

Hybrid/mixed system:

$$w_{1,inst} = \underbrace{\frac{5 \cdot "1" \cdot L^4}{384 \cdot (EI)_{y,eff}}}_{\text{Deformation due to bending moment}} + \underbrace{\frac{"1" \cdot L^2}{8 \cdot (GA)_{y,eff}}}_{\text{Deformation due to shear}}$$

$$w_{1,creep} = \underbrace{\frac{5 \cdot "1" \cdot L^4}{384 \cdot (EI)_{y,creep,eff}}}_{\text{Deformation due to bending moment}} + \underbrace{\frac{"1" \cdot L^2}{8 \cdot (GA)_{y,creep,eff}}}_{\text{Deformation due to shear}}$$

Deformation due to bending moment $[mm/\frac{kN}{m}]$:

$$w_{net,fin} = \frac{5 \cdot q_{inst} \cdot L^4}{384 \cdot (EI)_{y,eff}} + \frac{5 \cdot q_{creep} \cdot L^4}{384 \cdot (EI)_{y,creep,eff}}$$

Deformation due to shear $[mm/\frac{kN}{m}]$:

$$w_{net,fin} = \frac{q_{inst} \cdot L^2}{8 \cdot (GA)_{y,eff}} + \frac{q_{creep} \cdot L^2}{8 \cdot (GA)_{y,creep,eff}}$$

with

- q_{inst}, characteristic combination load;
- q_{creep}, quasi permanent combination load.

Considering:

$$(EI)_{y,eff} = \sum_i E_i . I_{y,i} + \sum_i E_i . A_i . {e_i}^2 \quad \text{Eq 15}$$

and

$$(EI)_{y,creep,eff} = \sum_i E_{i,creep} . I_{y,i} + \sum_i E_{i,creep} . A_i . {e_i}^2 \quad \text{Eq 16}$$

with

$$E_{LVL-S,creep} = \frac{E_{0,mean,LVL-S}}{k_{def,LVL-S}} \quad \text{Eq 17}$$

$$E_{LVL-X,creep} = \frac{E_{0,mean,LVL-X}}{k_{def,LVL-X}} \quad \text{Eq 18}$$

Considering :

$$(GA)_{y,eff} = \sum_i (G_{i,inst} \cdot A_i) \cdot \kappa \quad \text{Eq 19}$$

and

$$(GA)_{creep,eff} = \sum_i (G_{i,creep} \cdot A_i) \cdot \kappa \quad \text{Eq 20}$$

with :

$$G_{LVL-S,creep} = \frac{G_{0,mean,LVL-S}}{k_{def,LVL-S}} \quad \text{Eq 21}$$

$$G_{LVL-X,creep} = \frac{G_{0,mean,LVL-X}}{k_{def,LVL-X}} \quad \text{Eq 22}$$

κ : Corrective shear coefficient

Summary of the deformations:

$$w_{inst} = w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{characteristic\ load} \cdot b \quad \text{Eq 23}$$

$$w_{fin} = \left\{ w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{ccharacteristic\ load} + w_{1,creep} \cdot \underbrace{[g_{1,k} + g_{2,k} + \psi_2 \cdot q_{1,k}]}_{Quasi\ permanent\ load} \right\} \cdot b$$

$$w_{net,fin} = \left\{ w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{ccharacteristic\ load} + w_{1,creep} \cdot \underbrace{[g_{1,k} + g_{2,k} + \psi_2 \cdot q_{1,k}]}_{Quasi\ permanent\ load} \right\} \cdot b$$

4.2.2

4.2.2 Simplified method

Most of the flexural rigidity is provided by the rib in the rib panel system. Therefore, it shall be sufficient to apply a shear rigidity (GA) considering the rib only.
The equations below shall illustrate the method, given the following boundary conditions:
single span beam, pin-pin support, continuous loading, K$_{def}$ integrated into the rigidities (instantaneous and final), shear rigidity of the rib section only considered, shear deformation considered.

$$w_{1,inst} = \underbrace{\frac{5\cdot "1"\cdot L^4}{384\cdot (EI)_{eff}}}_{\text{Deformation due to bending moment}} + \underbrace{\frac{"1"\cdot L^2}{8\cdot (GA)_{eff,rib}}}_{\text{Deformation due to shear}}$$

$$w_{1,creep} = \underbrace{\frac{5\cdot "1"\cdot L^4}{384\cdot (EI)_{creep,eff}}}_{\text{Deformation due to bending moment}} + \underbrace{\frac{"1"\cdot L^2}{8\cdot (GA)_{creep,eff,rib}}}_{\text{Deformation due to shear}}$$

considering:

$$(EI)_{eff} = \sum_i E_i . I_{y,i} + \sum_i E_i . A_i . {e_i}^2$$ Eq 24

and

$$(EI)_{creep,eff} = \sum_i E_{i,creep} . I_{y,i} + \sum_i E_{i,creep} . A_i . {e_i}^2$$ Eq 25

considering

$$(GA)_{eff,rib} = \sum_i (G_{inst,w}\cdot A_w)\cdot \kappa = \frac{G_{inst,w}\cdot A_w\cdot 5}{6}$$ Eq 26

and

$$(GA)_{creep,eff,rib} = \sum_i (G_{creep,w}\cdot A_w)\cdot \kappa = \frac{G_{creep,w}\cdot A_w\cdot 5}{6}$$ Eq 27

Summary of the deformations:

$$w_{inst} = w_{1,inst}\cdot \underbrace{[g_{1,k}+g_{2,k}+q_{1,k}]}_{charges\ caractéristique}\cdot b$$ Eq 28

$$w_{fin} = \left\{w_{1,inst}\cdot \underbrace{[g_{1,k}+g_{2,k}+q_{1,k}]}_{charges\ caractéristique} + w_{1,creep}\cdot \underbrace{[g_{1,k}+g_{2,k}+\psi_2\cdot q_{1,k}]}_{charges\ quasi-permanente}\right\}\cdot b$$

$$w_{net,fin} = \left\{w_{1,inst}\cdot \underbrace{[g_{1,k}+g_{2,k}+q_{1,k}]}_{charges\ caractéristique} + w_{1,creep}\cdot \underbrace{[g_{1,k}+g_{2,k}+\psi_2\cdot q_{1,k}]}_{charges\ quasi-permanente}\right\}\cdot b$$

$$w_{net,fin} = \frac{5\cdot q_{charact}\cdot L^4\cdot 1}{384\cdot E\cdot I} + \frac{5\cdot q_{qp}\cdot L^4\cdot k_{def}}{384\cdot E\cdot I}$$

<u>Homogenous system / one material only (or when using a mean value of k$_{def}$ - see chapter 3.3)</u>:
As explained previously, it is also possible to apply a uniform creep factor $k_{def,uniform}$ to the entire section.
The equations below shall illustrate the method, given the following boundary conditions:
single span beam, pin-pin support, continuous loading, $k_{def,uniform}$ not integrated into the rigidities and only applied to the creep deformation, shear rigidity of the rib section only considered, shear deformation considered.

$$w_{1,inst} = \underbrace{\frac{5\cdot "1"\cdot L^4}{384\cdot (EI)_{eff}}}_{\text{Deformation due to bending moment}} + \underbrace{\frac{"1"\cdot L^2}{8\cdot (GA)_{eff}}}_{\text{Deformation due to shear}}$$

$$w_{1,creep} = \left(\underbrace{\frac{5 \cdot "1" \cdot L^4}{384 \cdot (EI)_{eff}}}_{\substack{Deformation\ due\\ to\ bending\ moment}} + \underbrace{\frac{"1" \cdot L^2}{8 \cdot (GA)_{eff}}}_{\substack{\text{Deformation due}\\ \text{to shear}}} \right) \cdot k_{def,uniform}$$

considering

$$(EI)_{eff} = \sum_i E_i . I_{y,i} + \sum_i E_i . A_i . e_i^{\,2}$$ Eq 29

$$(GA)_{eff,rib} = \sum_i (G_{inst,w} \cdot A_w) \cdot \kappa = \frac{G_{inst,w} \cdot A_w \cdot 5}{6}$$ Eq 30

Summary of the deformations:

$$w_{inst} = w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{charges\ caractéristique} \cdot b$$ Eq 31

$$w_{fin} = \left\{ w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{charges\ caractéristique} + w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + \psi_2 \cdot q_{1,k}]}_{charges\ quasi-permanente} \cdot k_{def,uniforme} \right\} \cdot b$$

$$w_{net,fin} = \left\{ w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{charges\ caractéristique} + w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + \psi_2 \cdot q_{1,k}]}_{charges\ quasi-permanente} \cdot k_{def,uniforme} \right\} \cdot b$$

where

b Rib spacing [mm];

G_w Shear modulus of the rib (w: web) [N/mm²];

A_w Rib area [mm²].

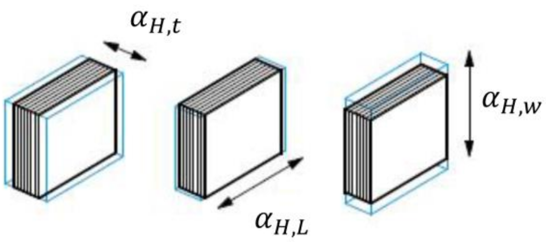
In that case, a mean k_{def} shall be calculated as given in the chapter 3.3

4.3 Dimensional stability of rib/box panels

The swelling and shrinkage behavior due to change in moisture content of LVL rib/box panels relates to the swelling and shrinkage behavior of the base material.

Table 7: Swelling and shrinkage coefficients in different direction of Stora Enso GLVL in % per % change of moisture content.

Type of timber and species	**Swelling and shrinkage in percent per percent change of moisture content** [3]		
	In the direction of the thickness $\alpha_{H,t}$	In the direction of the length $\alpha_{H,L}$	In the direction of the width $\alpha_{H,w}$
LVL-S (spruce)	0,30	0,006	0,31
LVL-X (spruce)	0,44	0,009	0,033



In normal conditions, harmful deformations due to moisture changes of the LVL Rib Panels by Stora Enso are not expected. When necessary, the dimensional change ΔL of the product due to change of moisture content can be calculated as follows:

$$\Delta L = \Delta\omega \cdot \alpha_H \cdot L \qquad \text{Eq 32}$$

with

ΔL deformation due to moisture content variation [mm]
$\Delta\omega$ change of moisture content from the equilibrium moisture content [%]
α_H dimensional variation coefficient
L initial dimension [mm].

5. Design of LVL edgewise loaded beams with unreinforced holes

Holes are openings with a clear dimension d ≥ h/10 with h, being the height of the beam or d ≥ 80mm.
According to ÖNORM B 1995-1-1:2015 [1] the application of unreinforced circular and rectangular holes is restricted to service class 1 and 2.

In general, it is not allowed to carry out on the building site openings not planned in design. For holes, a verification is necessary.

5.1 Geometric boundary conditions

In the current European standard for the design of timber structures EN 1995-1-1 [2] no explicit rules for the verification of holes in beams (loaded in bending) are given. Nevertheless, rules for the handling of this important topic are defined in a few National Annexes of the mentioned standard.

Until more specific rules are determined for Stora Enso LVL by experimental tests, relevant rules for reinforced holes (and notches) can be found in National Annexes to EN 1995-1-1 (e. g. ÖNORM B 1995-1-1).

In Figure 11 and Figure 12, the geometric boundary conditions regarding holes in beams are shown.

The rules given in the further sections of this chapter are only valid, for unreinforced holes and if the following geometric boundary conditions are respected.

General boundary condition:

- $l_V \geq h$
- $l_A \geq 0.5 \cdot h$

Additionally for circular holes:

- $h_d = d \leq 0.7 \cdot h$
- When the hole centre is situated at the neutral axis:
  $h_{ro} \geq 0.15 \cdot h$ and $h_{ru} \geq 0.15 \cdot h$
- When the hole centre is situated eccentrically to the neutral axis (eccentricity) :
  $h_{ro} \geq 0.25 \cdot h$ and $h_{ru} \geq 0.25 \cdot h$
- $l_z \geq max\begin{cases}0.50 \cdot h \\ 2.0 \cdot d\end{cases}$

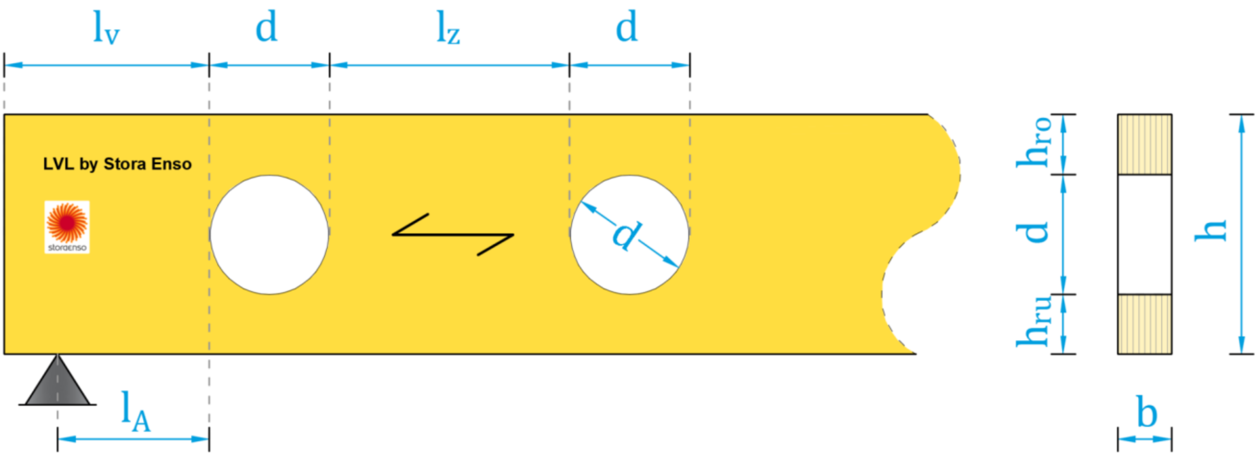


Figure 11: Definitions of geometric dimensions related to circular holes

<u>Additionally for rectangular holes</u>: The radius of curvature at each corner shall be at least 15 mm.

- $h_d \leq 0.3 \cdot h$ for LVL S ; $h_d \leq 0.4 \cdot h$ for LVL X
- $a \leq 1.5 \cdot h$
- $h_{ro} \geq 0.35 \cdot h$ and $h_{ru} \geq 0.35 \cdot h$ for LVL S ; $h_{ro} \geq 0.30 \cdot h$ and $h_{ru} \geq 0.30 \cdot h$ for LVL X
- $l_z \geq 1.5 \cdot h$

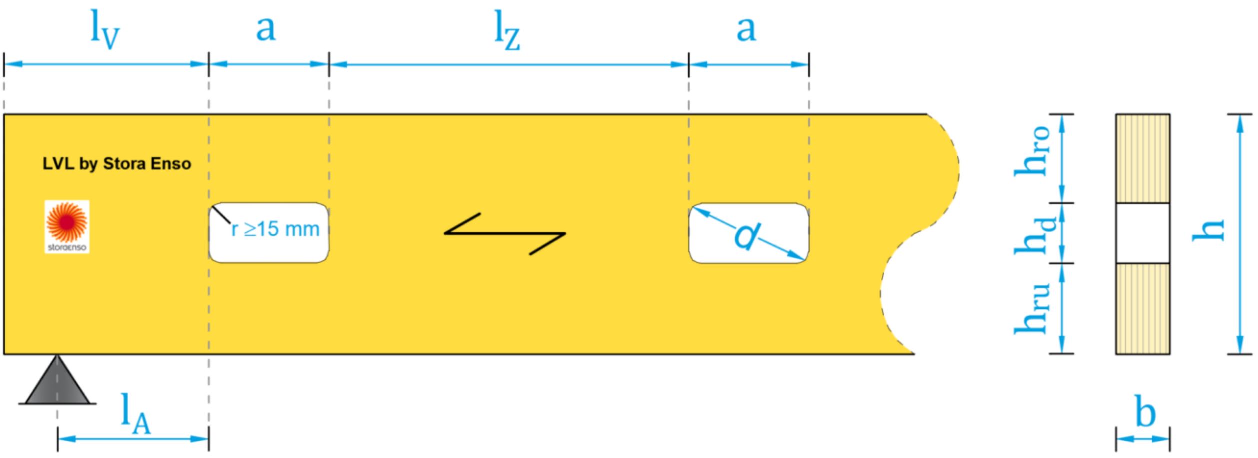


Figure 12: Definitions of geometric dimensions related to rectangular holes.

with

h Depth of the LVL beam [mm] ;
l_z Distance between two holes [mm];
h_d Diameter or height of the circular opening [mm];
a Length of the rectangular opening [mm].

5.2 Size and location of the holes

Beams with circular holes (LVL-S and LVL-X) loaded in bending

Geometrical limitations for circular holes in LVL-S and LVL-X by Stora Enso beams.

				Center of the hole on neutral axis		Center of the hole not on neutral axis	
BEAM HEIGHT	MIN DISTANCE FROM THE BEAM END	MIN DISTANCE FROM THE SUPPORT	MIN DISTANCE BETWEEN HOLES	MAXIMUM DIAMETER OF THE HOLE	DISTANCE FROM THE EDGES OF THE BEAM	MAXIMUM DIAMETER OF THE HOLE	DISTANCE FROM THE EDGES OF THE BEAM
h [mm]	L_V min [mm]	L_A min [mm]	L_Z min [mm]	d [mm]	h_{ro} and h_{ru} min [mm]	d [mm]	h_{ro} and h_{ru} min [mm]
200	200	100	280	140	30	100	50
240	240	120	336	168	36	120	60
300	300	150	420	210	45	150	75
350	350	175	490	245	52,5	175	87,5
400	400	200	560	280	60	200	100
450	450	225	630	315	67,5	225	112,5
500	500	250	700	350	75	250	125
600	600	300	840	420	90	300	150

Beams with rectangular holes (LVL-S and LVL-X) loaded in bending

Geometrical limitations for rectangular holes in LVL-S and LVL-X by Stora Enso beams.

BEAM HEIGHT	MIN DISTANCE FROM THE BEAM END	MIN DISTANCE FROM THE SUPPORT	MIN DISTANCE BETWEEN HOLES	MAXIMUM LENGTH OF TWO HOLE	MAXIMUM HEIGHT OF THE HOLE	DISTANCE FROM THE EDGES OF THE BEAM
h [mm]	L_V min [mm]	L_A min [mm]	L_Z min [mm]	a max [mm]	h_d max [mm]	h_{ro} and h_{ru} min [mm]
200	200	100	300	300	60 (S) / 80 (X)	70 (S) / 60 (X)
240	240	120	360	360	72 (S) / 96 (X)	84 (S) / 72 (X)
300	300	150	450	450	90 (S) / 120 (X)	105 (S) / 90 (X)
350	350	175	525	525	105 (S) / 140 (X)	122,5 (S) / 105 (X)
400	400	200	600	600	120 (S) / 160 (X)	140 (S) / 120 (X)
450	450	225	675	675	135 (S) / 180 (X)	157,5 (S) / 135 (X)
500	500	250	750	750	150 (S) / 200 (X)	175 (S) / 150 (X)
600	600	300	900	900	180 (S) / 240 (X)	210 (S) / 180 (X)

As a conservative approach in the design of unreinforced holes the initiation of the first crack is recommended to be considered as the “ultimate” load and stress level accordingly. The difference between the load level at the occurrence of the 1st crack and the maximum load has to be seen as a “buffer” for the missing interaction of stresses

(e. g. simultaneously acting stresses due to tensile stresses perpendicular to the grain and shear stresses) as well as loading effects from temperature and moisture content (shrinkage / swelling) in the vicinity of the hole not considered explicitly in the verification.

The following equations and design steps for unreinforced holes are based on the report from holz.bau forschungs gmbh [4]. The equations for the determination of the shear factor k_τ considering the stress concentrations at the contour of the circular and rectangular openings shall be used. This method is also based on a multitude of tests on LVL beams by Stora Enso with openings [5]. This includes alternatives to the method in the standard [6] for the verification of the tensile stresses perpendicular to grain which makes it possible to justify bigger ratios $\frac{h_d}{h}$ as given there.

Approach of Ardalany

The underlaying mechanical model is based on a beam on elastic foundation assuming that the lower part of the cracked beam is infinitely stiff in bending according to the approach of Ardalany [4].

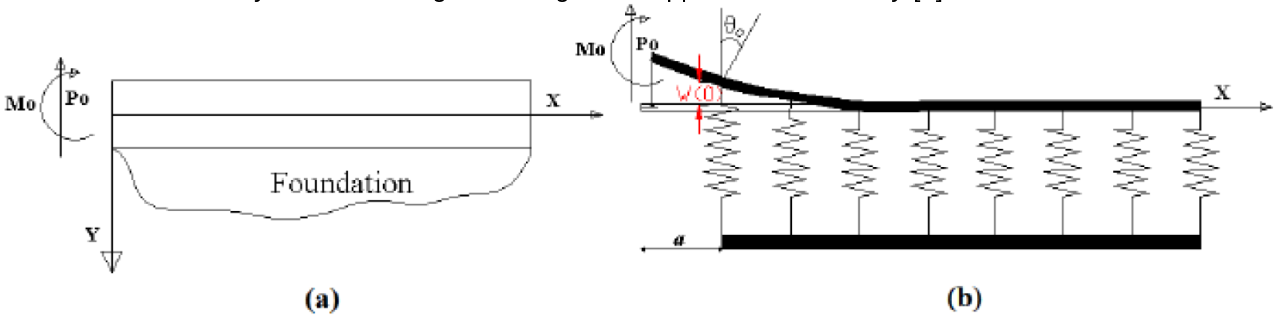


Figure 13 : Model of beam on elastic foundation: (a) beam on elastic foundation, (b) schematization of a beam on springs as elastic foundation, [7].

5.3 Verification of tension force perpendicular to grain (approach of Ardalany)

5.3.1 due to pure shear

The tension force perpendicular to grain for loadings in pure shear can be computed as follows:

$$F_{cr,t,90,V} = \frac{b \cdot f_{t,90}}{\sqrt{4 \cdot \lambda^2 + \frac{6}{5} \cdot \frac{K \cdot b}{G \cdot A}}} \quad \text{Eq 33}$$

with

$$\lambda = \sqrt[4]{\frac{K \cdot b}{4 \cdot E \cdot I}} \quad \text{Eq 34}$$

$$K = \frac{1}{2} \cdot \frac{f_{t,90}{}^2}{G_{I,f}}$$

The total shear resistance of the hole is twice this force as follows:

$$F_{cr,V} = 2 \cdot F_{cr,t,90,V} \quad \text{Eq 35}$$

$F_{cr,t,90,V}$	Shear resistance of the beam section for pure shear (at crack initiation), splitting force [N];
$F_{cr,V}$	Total shear resistance of the beam section for pure shear [N] ;
K	Spring stiffness in a beam on elastic foundation [N/mm³] ;
$G_{I,f}$	Fracture energy in mode I (opening) [N/mm] (see chapter 5.3.4);
$f_{t,90}$	Tensile strength perpendicular to grain [N/mm²];

b	Width of the beam [mm] ;
E	Modulus of elasticity [N/mm²] ;
I	Moment of inertia $I = \frac{b \cdot {h_{cr}}^3}{12}$ [mm⁴] ;
h_{cr}	Height of a portion of a beam above potential crack surface [mm];
	- for circular opening : $h_{cr} = \frac{h}{2} - \frac{h_d}{2} \cdot cos\ \alpha$, with $\alpha = 45°$
	- for rectangular opening : $h_{cr} = \frac{h}{2} - \frac{h_d}{2}$
λ	Parameter for beam on elastic foundation [1/mm] ;
G	Shear modulus of the beam section [N/mm²] ;
A	Shear area ($A = b \cdot h_{\mathrm{cr}}$) [mm²] .

5.3.2 due to pure bending

With the denotations used in standards, the local moment in the upper part of the beam on elastic foundation at crack initiation is as follows:

$$M_{cr,t,90} = \frac{b}{2 \cdot \lambda^2} \cdot f_{t,90}$$ Eq 36

The total moment is composed two parts:

a. Constant (rectangular) stress distribution

$$\sigma_N = \sigma_M \cdot \frac{h_{cr} + h_d \cdot cos\ \alpha}{h_{cr}}$$ Eq 37

with the resultant compression force

$$N = b \cdot \sigma_M \cdot (h_{cr} + h_d \cdot cos\ \alpha)$$ Eq 38

b. Triangular stress distribution

The triangular stresses are given by:

$$\sigma_{cr,t,90,M} = \frac{M_{cr,t,90}}{W}$$ Eq 39

Considering the moment of both portions of the beam (upper and lower portion) as well as from the resultant axial forces, the total moment $M_{cr,M}$ is given. Due to the different lever arm of circular and rectangular holes it is defined by:

- **for circular holes:**

$$M_{cr,M} = 2 \cdot M_{cr,t,90} + b \cdot \sigma_{cr,t,90,M} \cdot (h_{cr} + h_d \cdot cos\ \alpha) \cdot (h - h_{cr})$$ Eq 40

- **for rectangular holes:**

$$M_{cr,M} = 2 \cdot M_{cr,t,90} + b \cdot \sigma_{cr,t,90,M} \cdot (h_{cr} + h_d) \cdot (h - h_{cr})$$ Eq 41

where

b	Width of the beam [mm];
W	Cross section modulus $W = \frac{b \cdot {h_{cr}}^2}{6}$ [mm³];
h_{cr}	Height of a portion of a beam above potential crack surface [mm];
$\sigma_{cr,t,90,M}$	Stress due to bending moment [N/mm²];
$M_{cr,t,90}$	Moment resistance of the section [N·mm];
$M_{cr,M}$	Total resistance moment of the beam section for pure bending [N·mm].

5.3.3 Beam subjected to combined shear and bending moment (Splitting)

In general, combined actions form shear and bending moment occur in the beam. Thus an interaction equation has to be taken into account. The following empirical relationship shall be fulfilled:

$$\left(\frac{V_{cr}}{F_{cr,V}}\right) + \left(\frac{M_{cr}}{M_{cr,M}}\right)^2 = 1$$

Eq 42

where

M_{cr} Design bending moment [N·mm];
V_{cr} Design shear force at the location of the hole [N].

5.3.4 Fracture energy rate $G_{I,f}$ in mode I (opening) for LVL by Stora Enso

The determination of the fracture energy rate $G_{I,f}$ described in [4] is based on a test configuration of CIB-W18 standard draft proposed by Larsen and Gustafsson [8].

The experimentally determined values for LVL by Stora Enso are:

Mean value: $G_{I,f,mean} = 0.875\ N/mm$

Characteristic value: $G_{I,f,k} = 0.625\ N/mm$

Since the fracture energy value $G_{I,f}$ is a material parameter used in ULS verification, the modification factor K_{mod} (considering the moisture content and duration of load of the members) as well as a partial safety factor γ_M have to be considered for the computation of the design value $G_{I,f,d}$.

5.4 Verification of shear stresses for circular and rectangular holes in beams

In addition to the tensile stresses at the circumference (contour) of the hole, also the shear stresses have to be verified. In a first approximation these stresses can be determined applying the equation from the Beam Theory for the net cross section, but it is obvious that due to the redistribution of stresses in the vicinity of the hole stress peaks will occur.
By considering these concentration peaks in the design, the maximum shear stress at the contour of the hole can be multiplied by the factor for the shear stresses k_τ applicable for circular and rectangular holes.

The shear stress at the hole location should satisfy the following expression:

$$\tau_d = k_\tau \cdot 1{,}5 \cdot \frac{V_d}{b \cdot (h - h_d)} \le f_{v,d}$$

Eq 43

Since the redirection of stresses is expected to be smoother for circular holes and thus lead to minor pronounced stress peak at the contour of the hole, different k_τ factors shall be applied for each shape.
The following equations were determined by means of linear elastic FEM-simulations and shall be applied:

Rectangular holes

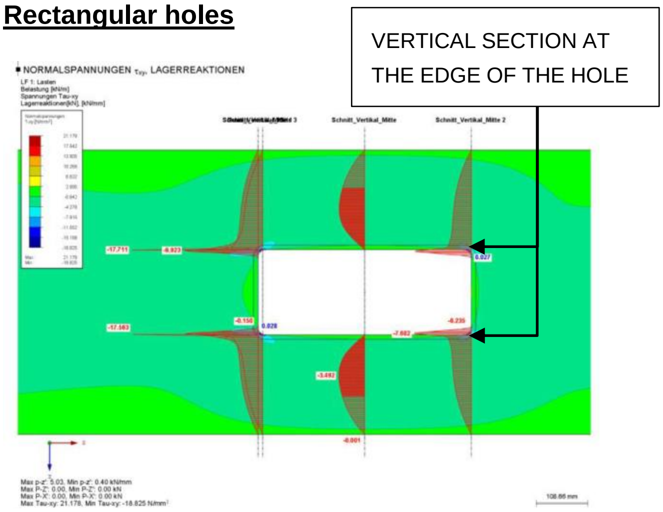


Figure 14: Shear stress distribution at the contour of a rectangular hole

For the computation of the maximum shear stresses, the shear factor k_τ at the corners of rectangular holes shall be determined according to [4] and [6].
as follows:

$$k_\tau = 1{,}85 \cdot \left(1 + \frac{a}{h}\right) \cdot \left(\frac{h_d}{h}\right)^{0{,}2}$$ *Eq 44*

with: $0.1 \leq \frac{a}{h} \leq 1.0$ and $0.1 \leq \frac{h_d}{h} \leq 0.4$

The maximum shear stress value should be determined in the vertical section at the edge of the hole closer to the supports.

Circular holes

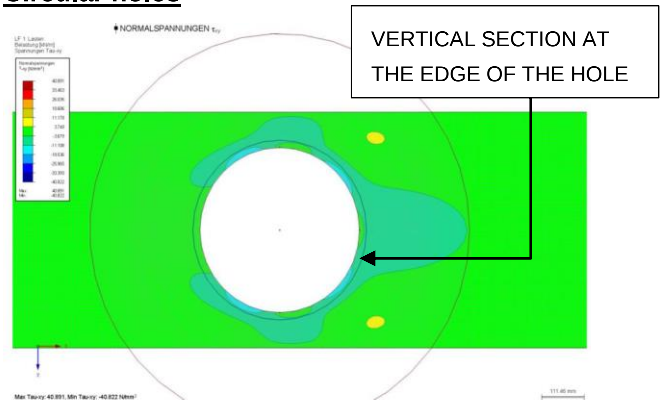


Figure 15: Shear stress distribution at the contour of a circular hole

For the computation of the maximum shear stresses, the shear factor k_τ of circular holes shall be determined as follows:

$$k_\tau = 0{,}62 \cdot \left[4{,}00 - \left(\frac{h_d}{h}\right)\right]$$ *Eq 45*

Remark:

(Compared to the currently given equation in ÖNORM B 1995-1-1:2015 [6] this equation given by Stora Enso leads to value for k_τ that are about 1/3 lower for circular holes of $\frac{h_d}{h} = 0{,}7$.)

5.5 Verification of longitudinal stresses – Bending

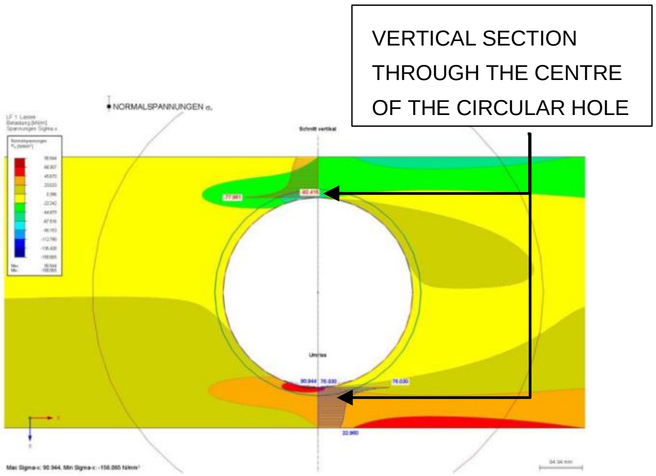


Figure 16: Longitudinal stresses at the contour of the hole and the edge of the beams

For circular holes:

While the longitudinal stresses for hd/h ratios up to ≈ 0.30 are greater at the edge of the beam (see Figure 15), for higher h_d/h values strongly increasing stresses at the circumference in the vertical section through the centre of the circular hole occur (see in Figure 16).

Thus, if $\frac{h_d}{h}$ ratios ≤ 0.7 shall be applied, in addition to the verification of the longitudinal stresses at the edge of the beam, given e. g. in the standard [6] , also the longitudinal stresses at the contour in the vertical section through the centre of the circular hole have to be verified (see Figure 16) .

For rectangular holes:

Similar to circular holes also at the contour, in particular at the corners of rectangular holes, pronounced stress peaks of the longitudinal stresses occur.

The bending stress at the hole location presented in Figure 14 for rectangular, and in Figure 15 and Figure 16 for circular holes (depending on the $\frac{h_d}{h}$ ratio) should satisfy the following expression:

$$\frac{\frac{M_d}{W_n} + \frac{M_{o,d}}{W_o}}{f_{m,d}} \leq 1 \qquad \text{Eq 46}$$

and

$$\frac{\frac{M_d}{W_n} + \frac{M_{u,d}}{W_u}}{f_{m,d}} \leq 1 \qquad \text{Eq 47}$$

where

$$M_{o,d} = \frac{A_o}{A_u + A_o} \cdot V_d \cdot \frac{a}{2} \qquad \text{Eq 48}$$

$$M_{u,d} = \frac{A_u}{A_u + A_o} \cdot V_d \cdot \frac{a}{2} \qquad \text{Eq 49}$$

$$A_o = b \cdot h_{ro} \qquad \text{Eq 50}$$

$$A_u = b \cdot h_{ru} \qquad \text{Eq 51}$$

$$W_o = \frac{b \cdot {h_{ro}}^2}{6} \qquad \text{Eq 52}$$

$$W_u = \frac{b \cdot {h_{ru}}^2}{6} \qquad \text{Eq 53}$$

with

M_d Design value of the bending moment at the edge of the opening [N.mm];

W_n	Effective cross section modulus of the beam at the position of the opening [mm³];
V_d	Design value of the transversal force at the edge of the opening [N];
$f_{m,d}$	Design value for the edgewise bending strength of the beam [N/mm²];
$h_{ro}; h_{ru}$	Remaining heights of the net cross section [mm]; According to the figures Figure 11 and Figure 12.

For circular holes it is sufficient to verify the bending stresses from the beam effect at the edges under consideration of the net cross section.

The bending stress at the location of a circular hole has to be verified by the equations:

$$\frac{\frac{M_d}{W_n}}{f_{m,d}} \leq 1$$

Eq 54

The verification of the resistance in tension perpendicular to the grain can be the most critical condition to fulfil the design of holes in LVL-S beams. LVL-X beams, on the other hand, offer a significant advantage for beams with holes, as the cross veneers act as reinforcement around the holes preventing cracking due to tension stresses perpendicular to the grain.

Remark:

It is critically mentioned that in the known design standards no interaction of stresses, i.e. tensile stresses perpendicular to grain and shear stresses as well as longitudinal stresses within the verification process is required, although they are simultaneously acting at the contour of the hole.

6. Vibration

The vibration control is made by setting limits to the natural frequency and on the flexibility. Each I and U section can be considered separately.

Eurocode

The vibration analysis shall be executed according to EN1995-1-1 and the applicable national annex.

In case an applicable national annex to a Eurocode standard is deviating from given recommendations in this document, automatically the national annex is governing.

In the basic edition of EN 1995-1-1, the vibration design is very poorly regulated. Currently the **Austrian national annex** of EN1995-1-1 contains the most extensive vibration design guide lines.

The following criterions on vibration design are available – most of them being mandatory to meet – others only optional:

- Layering criterion:

Floor class I	Floor class II
One floor element spanning across different occupancy units (apartments with different owners) on the same level	One floor element spanning within the same occupancy unit (apartments with same owner) on the same level
Wet floating screed installed on top of light or heavy granular fill.	Wet screed installed with or without granular fill below
Dry screed installed on top of heavy granular fill (> 60 kg/m²)	

- Frequency Criterion:

Floor class I	Floor class II
One floor element spanning across different occupancy units (apartments with different owners) on the same level	One floor element spanning within the same occupancy unit (apartments with same owner) on the same level
$f_1 \geq 8$ Hz	$f_1 \geq 6$ Hz

$$f_1 = \frac{\pi}{2 \cdot l^2} \sqrt{\frac{(EI)_{l,eff}}{m}} \cdot \underbrace{\sqrt{1 + \left(\frac{l}{b_R}\right)^4 \cdot \frac{(EI)_{b,eff,1m}}{(EI)_{l,eff,1m}}}}_{accounting\ for\ rigidity\ in\ cross\ direction}$$

Eq 55

Fundamental frequency f1 of the section I may be calculated as:

$$f_1 = \frac{\pi}{2L^2} \sqrt{\frac{EI}{b_1 m}} \cdot \underbrace{\sqrt{1 + \left(\frac{l}{b_R}\right)^4 \cdot \frac{(EI)_{b,eff,1m}}{(EI)_{l,eff,1m}}}}_{accounting\ for\ rigidity\ in\ cross\ direction}$$

Fundamental frequency f1 of the section U may be calculated as:

$$f_1 = \frac{\pi}{2L^2}\sqrt{\frac{2EI}{b_1 m}} \cdot \underbrace{\sqrt{1+\left(\frac{l}{b_R}\right)^4 \cdot \frac{(EI)_{b,eff,1m}}{(EI)_{l,eff,1m}}}}_{accounting\ for\ rigidity\ in\ cross\ direction}$$

f_1	First fundamental frequency [Hz]
$(EI)_{l,eff,1m}$	Flexural rigidity of the rib panel in longitudinal direction for a 1m wide rib panel (if rib spacing is not exactly 1m → extrapolation to a 1m wide element) in Nm²/m. The flexural rigidity is based on the mean value of the Young´s modulus and the effective moment of inertia. If a floating screed is present in the floor layup, the rigidity of the screed EI_{screed} can be added too.
$(EI)_{b,eff,1m}$	Analogous to $(EI)_{l,eff,1m}$, only in cross direction (perpendicular to the span direction [Nm²/m]. In case of a LVL rib panel, this is the flexural rigidity in cross direction of the LVL top flange + the rigidity of a screed, if any.
m	Mass of the structure in kg/m² = $\sum_{i\geq 1} G_{k,i}$ [kg/m²]
b_R	Width of the entire floor (not necessarily limited to the panel width – usually width of a room)
l	Span [m]

- Acceleration criterion (optional, if frequency criterion is not met):

Floor class I	Floor class II
$f_1 \geq 4{,}5$ Hz	$f_1 \geq 4{,}5$ Hz
$a_{rms} \leq 0{,}05$ m/s²	$a_{rms} \leq 0{,}10$ m/s²

$$a_{rms} = \frac{0{,}04 \cdot e^{-0{,}40 \cdot f_1} \cdot F_0}{2 \cdot \zeta \cdot \underbrace{\left[\frac{m \cdot l \cdot b_R}{2}\right]}_{modal\ mass\ M^*}}$$

Eq 56

f_1	First fundamental frequency [Hz]
F_0	Weight of 700N (defined in [4], Austrian NA, …related to the average weight of a human)
ζ	Damping ratio. For LVL RIB PANEL floors, a damping ratio of 4% shall be assumed (0,04)
m	Mass of the structure in kg/m² = $\sum_{i\geq 1} G_{k,i}$ [kg/m²]
b_R	Width of the entire floor (not necessarily limited to the panel width – usually width of a room)
l	Span [m]

- Stiffness criterion:

Floor class I	Floor class II
One floor element spanning across different occupancy units (apartments with different owners) on the same level	One floor element spanning within the same occupancy unit (apartments with same owner) on the same level
$w_{1kN} \leq 0{,}25$ mm	$w_{1kN} \leq 0{,}50$ mm

$$w_{1kN} = \frac{F \cdot l^3}{48 \cdot (EI)_{l,eff,1m} \cdot \left[\frac{l}{1{,}1} \cdot \sqrt[4]{\frac{(EI)_{b,eff,1m}}{(EI)_{l,eff,1m}}}\right]} + \frac{F \cdot l}{4 \cdot (GA)_{l,eff,1m} \cdot \left[\frac{l}{1{,}1} \cdot \sqrt[4]{\frac{(EI)_{b,eff,1m}}{(EI)_{l,eff,1m}}}\right]} \quad \text{Eq 57}$$

f_1	First fundamental frequency [Hz]
$(EI)_{l,eff,1m}$	Flexural rigidity of the rib panel in longitudinal direction for a 1m wide rib panel (if rib spacing is not exactly 1m → extrapolation to a 1m wide element) in Nm²/m. The flexural rigidity is based on the mean value of the Young´s modulus and the effective moment of inertia. If a floating screed is present in the floor layup, the rigidity of the screed EIscreed can be added too.
$(EI)_{b,eff,1m}$	Analogous to (EI)l,eff,1m, only in cross direction (perpendicular to the span direction [Nm²/m]. In case of a LVL rib panel, this is the flexural rigidity in cross direction of the LVL + the rigidity of a screed, if any.
$F_{for\ I\ section}$	Point load of 1 kN
$F_{for\ U\ section}$	Point load of 0.5 kN on edge ribs
$(GA)_{l,eff,1m}$	Shear stiffness of the rib panel in longitudinal direction for a 1m wide rib panel (if rib spacing is not exactly 1m → extrapolation to a 1m wide element)
l	Span [m]

The vibration design according to Austrian NA shall be summarized in the following flow chart:

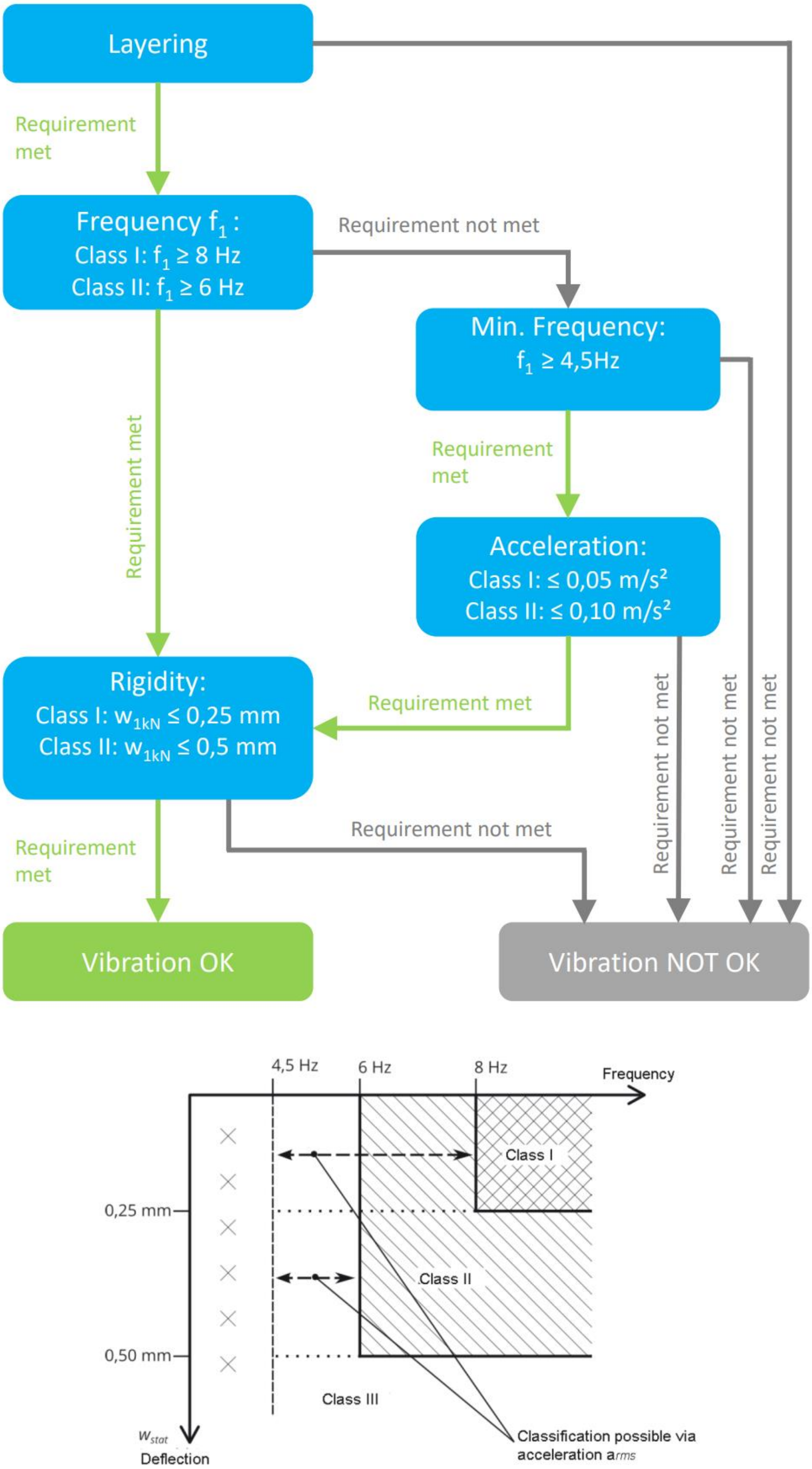


Figure 17: Vibration design flow chart

7. Cross-section

Standard dimensions for LVL-S are to be found in Table 8 and Table 9. These dimensions are used in ribs and bottom flanges of rib panels.

Table 8: Standard cross-section dimensions for LVL-S ribs.

Thickness [mm]	Height/Width [mm]							
45	200	240	300	350	400	450	500	600
51	200	240	300	350	400	450	500	600
57	-	240	300	350	400	-	-	-
63	-	240	300	350	400	450	500	600
69	-	240	300	350	400	-	-	-
75	200	240	300	350	400	450	500	600

Table 9: Standard cross-section dimensions for LVL-S flanges.

Thickness [mm]	Height/Width [mm]							
27	200	240	300	350	400	450	500	600
30	200	240	300	350	400	450	500	600
33	200	240	300	350	400	450	500	600
39	200	240	300	350	400	450	500	600
45	200	240	300	350	400	450	500	600
51	200	240	300	350	400	450	500	600
57	-	240	300	350	400	-	-	-
63	-	240	300	350	400	450	500	600
69	-	240	300	350	400	-	-	-
75	200	240	300	350	400	450	500	600

Standard dimensions for LVL-X (top chord LVL panel and bottom flange in box panels) are to be found in Table 10.

Table 10: Thicknesses and lay-ups for LVL-X top/bottom chord.

Thickness [mm]	Number of plies	Grain lay-up
27	9	II-III-II
30	10	II-IIII-II
33	11	II-IIIII-II
39	13	II-III-III-II
45	15	II-IIII-IIII-II
51	17	II-IIIII-IIIII-II
57	19	II-III-IIIII-III-II
63	21	II-III-III-III-III-II
69	23	II-IIII-III-III-IIII-II

The dimensions written in gray in the table above are available standard thickness for LVL but are not covered within this ETA.

Because of structural gluing, flat surface of LVL should be sanded. Sanding reduces cross-section 1 mm for each side and this reduction must be considered in a structural analysis.
The cross-section of rib panel is divided into sections I and U, see Figure 18.

7.1 Effective cross section in bending

The cross-section of LVL rib panels is divided to sections I and U - see Figures below. Each section should be calculated individually.

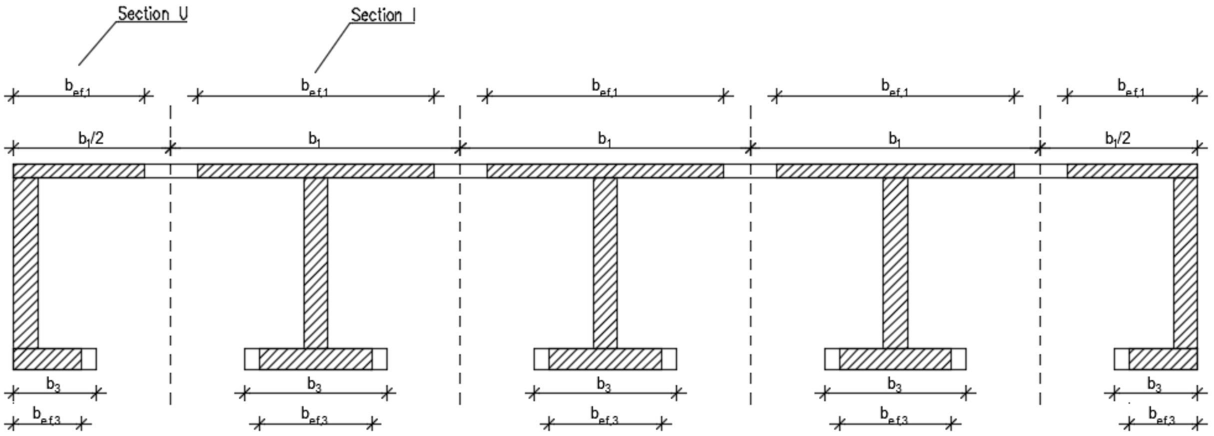


Figure 18: Sections I and U of rib panel (effective areas in bending hatched).

Figure 19 displays cross-sections and their designations.

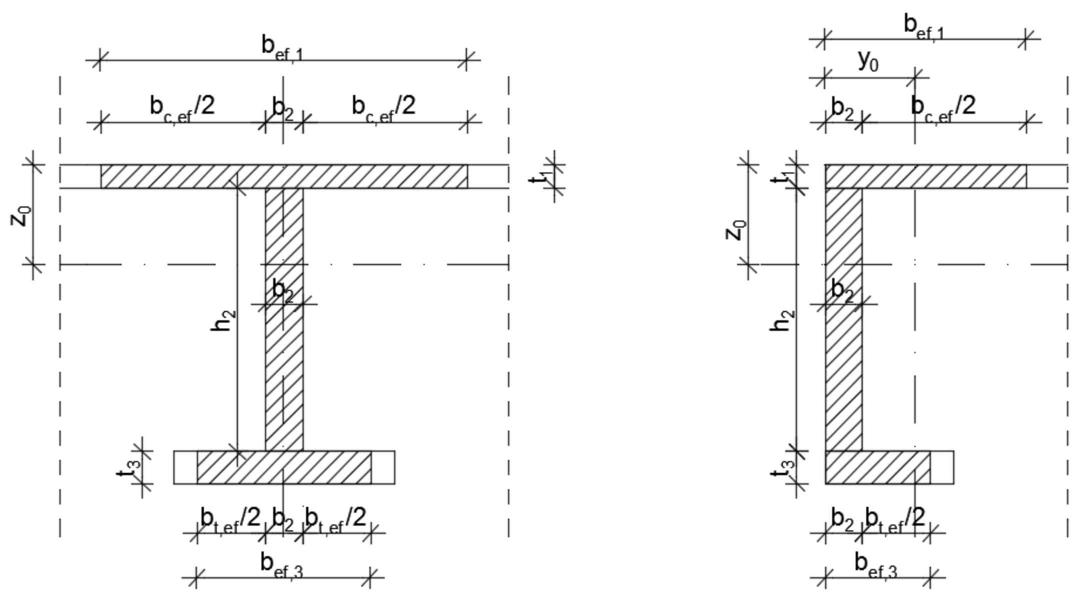


Figure 19: Parts of the effective cross-section and designations used in middle and edge ribs (Slab in compression).

7.2 Effective flange widths b_{ef}

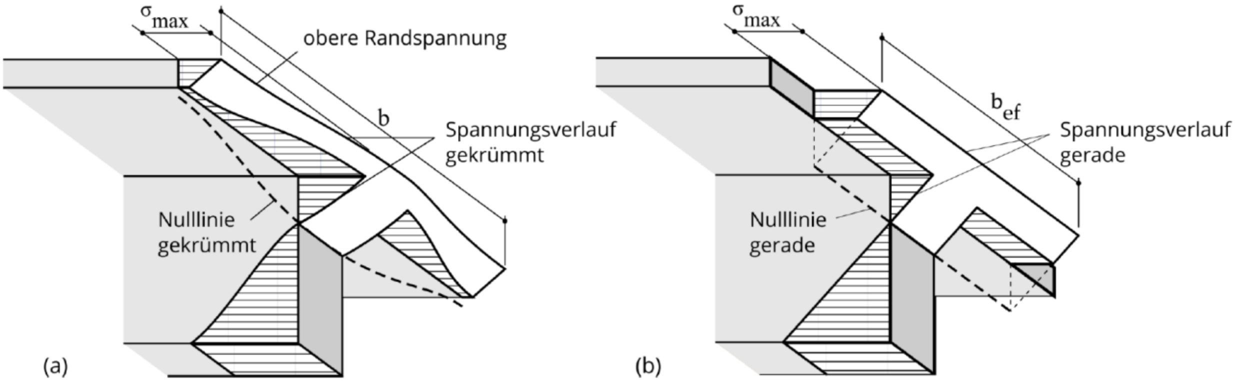


Figure 20: Effective width with (a) the actual stress distribution and (b) the linear stress distribution and effective width

Figure 20 shows the non-linear distribution of the bending stresses in a plate with a rib on the basis of Leonhardt, 1973. In order to attribute the problem to the beam theory with the assumption of a linear distribution of stresses, the effective width bef of the panel is determined such that the maximum edge stress in the panel σ_{max} is equal to the non-linear case.

Table 11: Effective flange width for I and U section (According to EN 1995-1-1, item 9.1.2)

Section I (middle ribs)	Section U (outer ribs)
$b_{ef} = \begin{cases} b_{c,ef} + b_2 \leq b_1 \\ b_{c,ef} + b_2 \leq b_3 \end{cases}$ or $b_{ef} = \begin{cases} b_{t,ef} + b_2 \leq b_1 \\ b_{t,ef} + b_2 \leq b_3 \end{cases}$	$b_{ef} = \begin{cases} 0.5b_{c,ef} + b_2 \leq \frac{b_1}{2} \\ 0.5b_{c,ef} + b_2 \leq b_3 \end{cases}$ or $b_{ef} = \begin{cases} 0.5b_{t,ef} + b_2 \leq \frac{b_1}{2} \\ 0.5b_{t,ef} + b_2 \leq b_3 \end{cases}$

$b_{c,ef}$ is the effective width on the compressed side
$b_{t,ef}$ is the effective width on the tension side
b_2 is the width of the rib

The value of $b_{t,ef}$ for the slab should not be greater than the values, given in Table 12, but less than 0.1l (l being the span of the beam).

Table 12: Maximum effective flange widths $b_{t,ef}$ in tension for open box slab (Research report No. VTT-S-04714-16)

LVL with grain direction in the outer plies:	Service limit state	Ultimate limit state
Parallel to the webs	296 mm	167.2 mm

The value of $b_{c,ef}$ and $b_{t,ef}$ for basic rib panels should not be greater than the minimum value from Table 13. Shear lag and plate buckling defines maximum values for flange width.

Table 13: Maximum effective flange widths $b_{c,ef}$ and $b_{t,ef}$.

LVL with grain direction in the outer plies:	Shear effect	Plate buckling
Parallel to the webs	0.1l	20h_f

h_f is the thickness of the slab

If a detailed buckling analysis is not made, the unrestrained flange width shall not be greater than twice the effective flange width due to plate buckling.

7.3 Flexural rigidity

For each I- and U-section the area and the flexural rigidity is analyzed as follows.
Cross-section area of the slab part may be calculated as

$$A_1 = b_{ef,1} \cdot t_1 \qquad \text{Eq 58}$$

$b_{ef,1}$ is the effective width of the upper slab
t_1 is the thickness of the upper slab

Cross-section area of the rib part may be calculated as

$$A_2 = b_2 \cdot h_2 \qquad \text{Eq 59}$$

h_2 is the height of the rib
Cross-section area of the bottom flange or slab part may be calculated as

$$A_3 = b_{ef,3} \cdot t_3 \qquad \text{Eq 60}$$

$b_{ef,3}$ is the effective width of the bottom flange or slab
t_3 is the thickness of the bottom flange or slab

Cross-section area of the entire section is

$$A = A_1 + A_2 + A_3 \qquad \text{Eq 61}$$

Location of the center of gravity (C.O.G.), related to the bottom of the rib panel section:

$$z_0 = \frac{\sum_i E_i \cdot A_i \cdot a_i}{\sum_i E_i \cdot A_i} \qquad \text{Eq 62}$$

A_i is the sectional area of the respective layer/rib

a_i is the distance from the reference edge (either top or bottom) of the rib panel section to the partial center of gravity of the respective layer/rib

E_i is the Young´s modulus of the respective layer/rib. For each situation with different Young´s modulus, the C.O.G. needs to be calculated individually (ULS t=0, ULS t=∞ (not needed to calculate with uniform k_{def}) , SLS t=0, SLS creep (not needed to calculate with uniform k_{def}))

Location of the neutral axis from the top edge of slab may be calculated as

$$z_0 = \frac{E_{0,mean,1} A_1 \frac{t_1}{2} + E_{0,mean,2} A_2 \left(t_1 + \frac{h_2}{2}\right) + E_{0,mean,3} A_3 \left(t_1 + h_2 + \frac{t_3}{2}\right)}{E_{0,mean,1} A_1 + E_{0,mean,2} A_2 + E_{0,mean,3} A_3} \qquad \text{Eq 63}$$

where
$E_{0,mean,1}$ is the modulus of elasticity of the slab
$E_{0,mean,2}$ is the modulus of elasticity of the rib
$E_{0,mean,3}$ is the modulus of elasticity of the bottom flange or slab

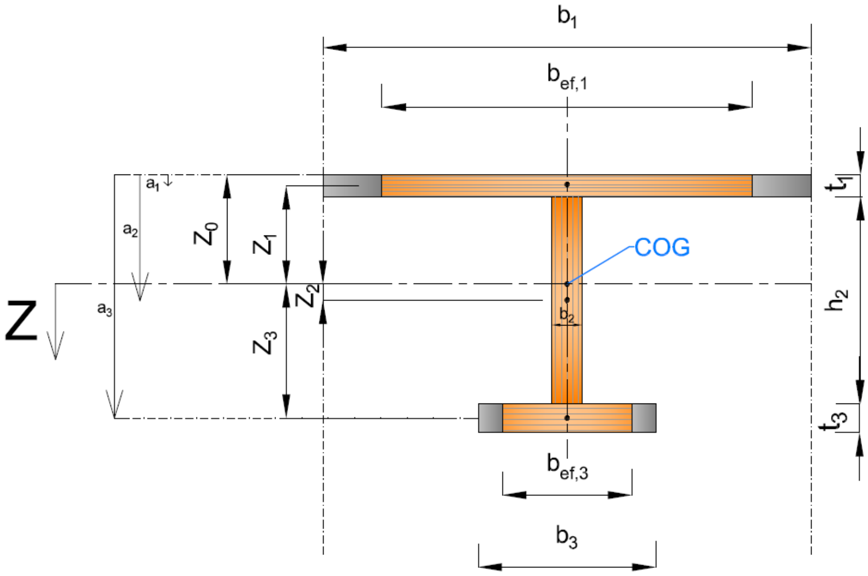


Figure 21: Cross section area- I section

Location of the neutral axis from edge of slab for U-section may be calculated as

$$y_0 = \frac{E_{0,mean,1}A_1\frac{b_{ef,1}}{2} + E_{0,mean,2}A_2\frac{b_2}{2} + E_{0,mean,3}A_3\frac{b_{ef,3}}{2}}{E_{0,mean,1}A_1 + E_{0,mean,2}A_2 + E_{0,mean,3}A_3}$$ Eq 64

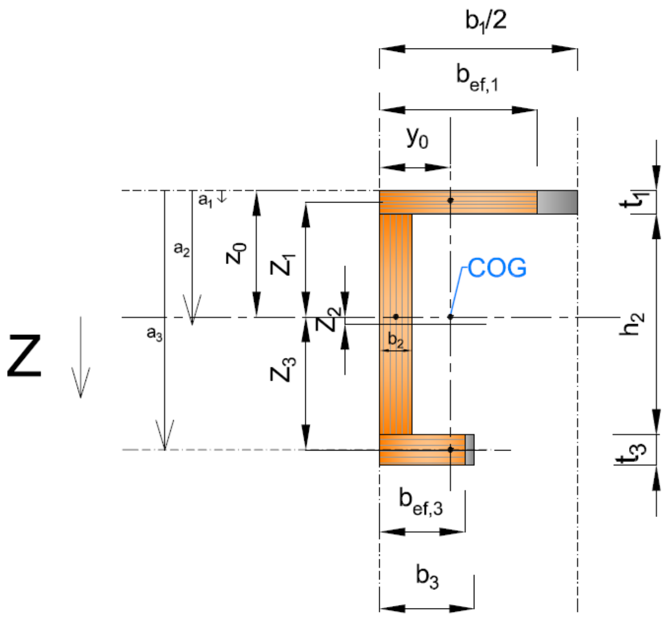


Figure 22;Cross section area- U section

Flexural rigidity about the Y-axis (Z-direction) of the partial sections shall be

$$EI_i = \frac{E_i \cdot b_{i,eff} \cdot {d_i}^3}{12} + E_i \cdot A_i \cdot {z_i}^2$$ Eq 65

Flexural rigidity of the section may be calculated in parts. If the effective widths are different in tension and compression the flexural rigidity needs to be calculated separately for positive and negative bending moment.

Flexural rigidity of the slab may be calculated as:

$$EI_1 = \frac{E_{0,mean,1} b_{ef,1} t_1^3}{12} + E_{0,mean,1} A_1 \left(z_0 - \frac{t_1}{2}\right)^2$$ Eq 66

Flexural rigidity of the rib may be calculated as

$$EI_2 = \frac{E_{0,mean,2} b_2 h_2^3}{12} + E_{0,mean,2} A_2 \left(z_0 - \left(t_1 + \frac{h_2}{2}\right)\right)^2$$ Eq 67

Flexural rigidity of the bottom flange or slab may be calculated as

$$EI_3 = \frac{E_{0,mean,3} b_{ef,3} t_3^3}{12} + E_{0,mean,3} A_3 \left(z_0 - \left(t_1 + h_2 + \frac{t_3}{2}\right)\right)^2$$ Eq 68

Flexural rigidity about the Y-axis (Z-direction) of the considered LVL Rib panel section may be calculated as

$$EI_{eff} = \sum_i EI_i$$

$$EI_{eff} = EI_1 + EI_2 + EI_3$$ Eq 69

Second moment of inertia of the upper slab perpendicular to the ribs per unit length may be calculated as

$$I_b = \frac{t_1^3}{12}$$ Eq 70

Flexural rigidity of the upper slab perpendicular to the ribs per unit width may be calculated as

$$EI_b = E_{m,90,mean,1} \frac{t_1^3}{12}$$ Eq 71

Since the stress distribution in the section is dependent on the Young´s modulus and the Young´s modulus is time dependent, the following cases need to be considered separately.

In a very precise analysis, the ULS at t=∞ could be included. Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

Case	Flexural rigidity	
ULS, t=0	$EI_{i,inst,d} = \frac{E_{i,inst,d} \cdot b_{i,eff} \cdot d_i^3}{12} + E_{i,inst,d} \cdot A_i \cdot z_{i,inst,d}^2$	Eq 72
ULS t=∞ [1]	$EI_{i,fin,d} = \frac{E_{i,fin,d} \cdot b_{i,eff} \cdot d_i^3}{12} + E_{i,fin,d} \cdot A_i \cdot z_{i,fin,d}^2$	Eq 73
SLS, t=0	$EI_{i,inst} = \frac{E_{i,inst} \cdot b_{i,eff} \cdot d_i^3}{12} + E_{i,inst} \cdot A_i \cdot z_{i,inst}^2$	Eq 74
SLS creep	$EI_{i,creep} = \frac{E_{i,creep} \cdot b_{i,eff} \cdot d_i^3}{12} + E_{i,creep} \cdot A_i \cdot z_{i,creep}^2$	Eq 75

When a uniform K_{def} is taken for the whole section, there is no need to analyze t=∞.

[1] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

7.4 Shear stiffness

7.4.1 Corrective shear coefficient

The followings equations can be found in the report of T. Bogensperger "Report hbf 05_2018 Shear Correction Factors for special cross sections of LVL Rib Panels - Graz, April 2018" [5].
The necessity for a shear stiffness correction arises, as indeed an exact formulation is used for pure bending, but only an approximation for shear deformations is introduced in the beam theory.
This approximation for the shear deformations results in a shear stiffness equal to GA. This product serves as a reference value for the shear stiffness, which needs an additional correction factor.

- **T-shaped section**
  Corrective shear coefficient

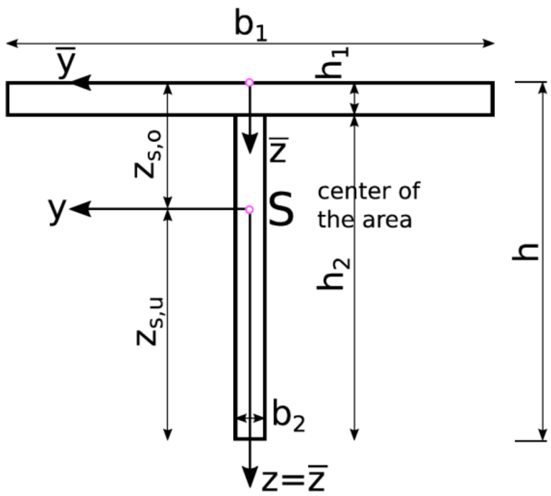


Figure 23:Geometry and notation of the T-shaped cross section

For the analysis, homogenized material stiffness parameter was used for each part of the section.

$$J=\int_A \frac{1}{G(z)}\cdot\left(\frac{ES(z)}{b(z)}\right)^2\cdot dA$$ Eq 76

This homogenization is different to the determination for CLT.

The term $ES(z)=\int_{\bar{z}=0}^{\bar{z}=z}\bar{z}\cdot b(\bar{z})\cdot d\,\bar{z}$ denotes the static moment of area, weighted by the particular modulus of elasticity.
The integral, which has to be solved now is given in the following Eq 77.The symbol s denotes the local natural coordinate along the thin parts of the sections. ES_0 in Eq 77represents the integration constant at the beginning of each part.

$$J=\int_A \frac{1}{G(s)}\cdot\left(\frac{ES(s)}{b(s)}\right)^2\cdot b(s)\cdot ds$$ Eq 77

$$\text{with}\quad ES(s)=\int_{\bar{s}=0}^{\bar{s}=s} z(\bar{s})\cdot b(\bar{s})\cdot d\,\bar{s}+ES_0$$

Solution "thick-wall cross section"

The above integral is solved for both parts of the cross section separately and the results of the integration can be found in the following equation.

J_1 denotes the results for the upper flange part and J_2 represents the bottom web part.

$$J_1 = \frac{E_1^{\ 2}}{G_1} \cdot b_1 \cdot h_1^{\ 3} \cdot \left(\frac{Z_{s,o}^{\ 2}}{3} - \frac{Z_{s,o} \cdot h1}{4} + \frac{h_1^{\ 2}}{20} \right)$$ Eq 78

$$J_2 = \frac{E_2^{\ 2}}{G_2} \cdot b_2 \cdot h_2^{\ 3} \cdot \left(\frac{Z_{s,u}^{\ 2}}{3} - \frac{Z_{s,u} \cdot h2}{4} + \frac{h_2^{\ 2}}{20} \right)$$ Eq 79

The shear correction factor κ for the T-shaped section is given by the following equation:

$$\kappa = \frac{GA}{EI_{eff}^{\ 2}} \cdot (J1 + J2)$$ Eq 80

κ	Corrective shear coefficient
EI_{eff}	Effective flexural rigidity of the LVL Rib panel
GA	Reference value for the shear stiffness

- **I-shaped section**

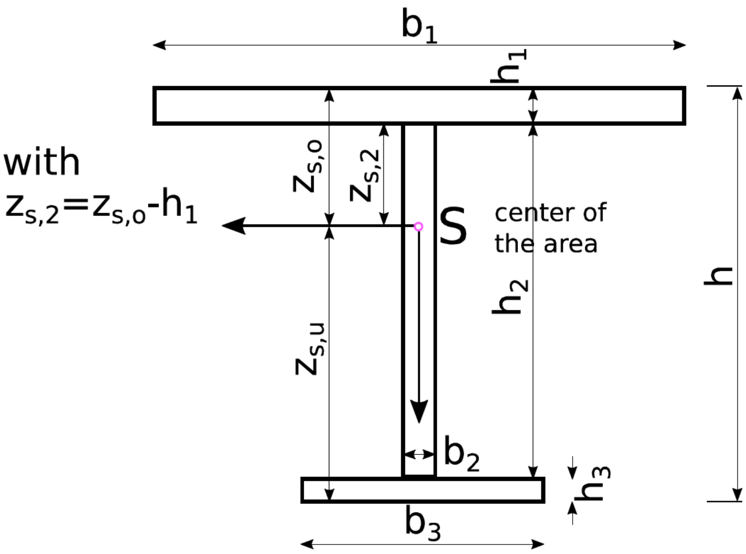


Figure 24:Geometry and notation of the I-shaped cross section

Solution "thick-wall cross section"

The integral Eq 77 has to be solved for all three parts of the I-section. The integration results can be found in as J1 for the upper flange 1, as J2 for the rib 2 and as J3 for the bottom flange 3.
Hint: don't mix $z_{s,2}$ and $z_{s,o}$

Part 1

$$J_1 = \frac{E_1^{\ 2}}{G_1} \cdot b_1 \cdot h_1^{\ 3} \cdot \left(\frac{Z_{s,o}^{\ 2}}{3} - \frac{Z_{s,o} \cdot h1}{4} + \frac{h_1^{\ 2}}{20} \right)$$ Eq 81

Part 2:

$$J_{2,1} = 15 \cdot b_1^{\ 2} \cdot E_1^{\ 2} \cdot h_1^{\ 2} \cdot (2 \cdot Z_{s,2} + h_1)^2$$ Eq 82

$$J_{2,2} = 10 \cdot b_1 \cdot b_2 \cdot h_1 \cdot h_2 \cdot E_1 \cdot E_2 \cdot (\ 2 \cdot Z_{s,2} + h_1) \cdot (\ 3 \cdot Z_{s,2} - h_2)$$ Eq 83

$$J_{2,3} = b_2^{\ 2} \cdot h_2^{\ 2} \cdot E_2^{\ 2} \cdot (20 \cdot Z_{s,2}^{\ 2} - 15 \cdot Z_{s,2} \cdot h_2 + 3 \cdot h_2^{\ 2})$$ Eq 84

Part 3:

$$J_3 = \frac{E_3{}^2}{G_3} \cdot b_3 \cdot h_3{}^3 \cdot \left(\frac{Z_{s,u}{}^2}{3} - \frac{Z_{s,u} \cdot h_3}{4} + \frac{h_3{}^2}{20} \right)$$ Eq 85

The shear correction factor κ for the I-shaped section is given by the following equation:

$$\kappa = \frac{GA}{EI_{eff}{}^2} \cdot \left(J_1 + (J_{2,1} + J_{2,2} + J_{2,3}) \cdot \frac{h_2}{60 \cdot b_2 \cdot G_2} + J_3 \right)$$ Eq 86

7.4.2 Effective shear stiffness:

$$(GA)_{eff} = \frac{\sum_i G_i \cdot A_i}{\kappa}$$ Eq 87

The shear stiffness is dependent on the shear modulus and needs to be analyzed for each of the following cases individually:

Case	Shear stiffness	
ULS, t=0	$(GA)_{eff} = \frac{\sum_i G_{i,inst,d} \cdot A_i}{\kappa}$	Eq 88
ULS t=∞ [2]	$(GA)_{eff} = \frac{\sum_i G_{i,fin,d} \cdot A_i}{\kappa}$	Eq 89
SLS, t=0	$(GA)_{eff} = \frac{\sum_i G_{i,inst} \cdot A_i}{\kappa}$	Eq 90
SLS creep	$(GA)_{eff} = \frac{\sum_i G_{i,creep} \cdot A_i}{\kappa}$	Eq 91

When it is supposed that all the shear force is carried by the ribs, **GA_{eff} of rib is used** and uniform $k_{def} = k_{def,LVL\text{-}S}$ may be used.

7.4.3 Simplified method for shear stiffness:

Instead of a precise derivation of the corrective shear coefficient, a more simplified approach is possible, by only considering the rib for shear deflection and neglecting the upper and (if present) the lower flange. In this case, the applicable corrective shear coefficient is the one for rectangular sections (5/6).

$$(GA)_{eff} = \frac{G_w \cdot A_w \cdot 5}{6}$$ Eq 92

7.5 Stiffness values

- Material parameters:

<u>LVL-S</u> (according to analysis report VTT-S-05710-17)

- $E_{0,mean,LVL\text{-}S}$ = 13,800 N/mm²
- $G_{0,mean,LVL\text{-}S}$ = 600 N/mm²
- $G_{0,flat,mean,LVL\text{-}S}$ = 460 N/mm²
- γ_M=1.20 | K_{def}=0.6 (SC1) | K_{def}=0.8 (SC2)

[2] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

LVL-X (according to analysis report VTT-S-05550-17)

- $E_{0,mean,LVL-X}$ = 10,500 N/mm²
- $E_{90,flat,mean,LVL-X}$ = 2000 N/mm²
- $G_{0,mean,LVL-X}$ = 600 N/mm²
- $G_{0,flat,mean,LVL-X}$ = 120 N/mm²
- $G_{90,flat,mean,LVL-X}$ = 22 N/mm²
- γ_M=1.20 | K_{def}=0.8 (SC1) | K_{def}=1.0 (SC2) (Values used for flatwise bending and flatwise shear only)

- **Loads**

Service class: 1&2 acc. to EN 1995-1-1

Example with K_{mod}=0.8 | ψ_2=0.3

Other K_{mod} values can be taken, depending on the load duration and service class.

Table 14:Young´s modulus for different design cases- Parallel to the grain (Example for precise method K_{def} in SC1)

Design	Time	Definition acc.to EN 1995-1-1	E_0 [N/mm²]		G_0 [N/mm²]		
			LVL-S	LVL-X	LVL-S edgewise	LVL-S flatwise	LVL-X flatwise
ULS	t = 0	$X_{inst} = X_{mean}$	13,800	10,500	600	460	120
		$n_{inst,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	1.31		-	3.83	
	t = ∞[3]	$X_{fin,d} = \frac{X_{mean}}{\gamma_M \cdot (1+\psi_2 \cdot k_{def})}$	9745.76	7056.45	423.73	324.86	80.65
		$n_{fin,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	1.38		-	4.03	
SLS	t = 0	$X_{inst} = X_{mean}$	13,800	10,500	600	460	120
		$n_{inst,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	1.31		-	3.83	
	Creep t = ∞	$X_{creep} = \frac{X_{mean}}{k_{def}}$	23,000	13,125	1,000	766.67	150
		$n_{creep,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	1.75		-	5.11	
	t = ∞	$X_{fin} = \frac{X_{mean}}{1+k_{def}}$	8625	5833.33	375	287.50	66.67
		$n_{fin,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	1.48		-	4.31	

[3] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

Table 15:Young´s modulus for different design cases- Perpendicular to the grain (Example precise method for K_{def} in SC1)

Design	Time	Definition acc.to EN 1995-1-1	E_{90} [N/mm²] LVL-X	G_{90} [N/mm²] LVL-X flatwise
ULS	t = 0	$X_{inst} = X_{mean}$	2000	22
	t = ∞[4]	$X_{fin,d} = \frac{X_{mean}}{\gamma_M \cdot (1+\psi_2 \cdot k_{def})}$	1344.10	14.78
SLS	t = 0	$X_{inst} = X_{mean}$	2000	22
	Creep t = ∞	$X_{creep} = \frac{X_{mean}}{k_{def}}$	2500	27.5
	t = ∞	$X_{fin} = \frac{X_{mean}}{1 + k_{def}}$	1111.11	12.22

n_{inst} and n_{fin} are ratios of the Young´s Modulus of 2 different materials. When calculating the center of gravity, one can either use the Young´s modulus in the equation or apply the n values to increase/decrease the related parts. If nothing else, the n factors give an idea about the difference in elasticity between 2 materials.

[4] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

8. Bending resistance

If the section is purely subject to out of plane loading that is causing a bending reaction on the rib panel, the normal stress in the section shall be calculated as follows: design at t=0 and t=∞ needs to be done with the precise method.

If one uniform K_{def} is considered as in the simplified method, design at t=0 is sufficient.

8.1 Bending stress

Bending stresses should be calculated in the following points, see Figure 25.

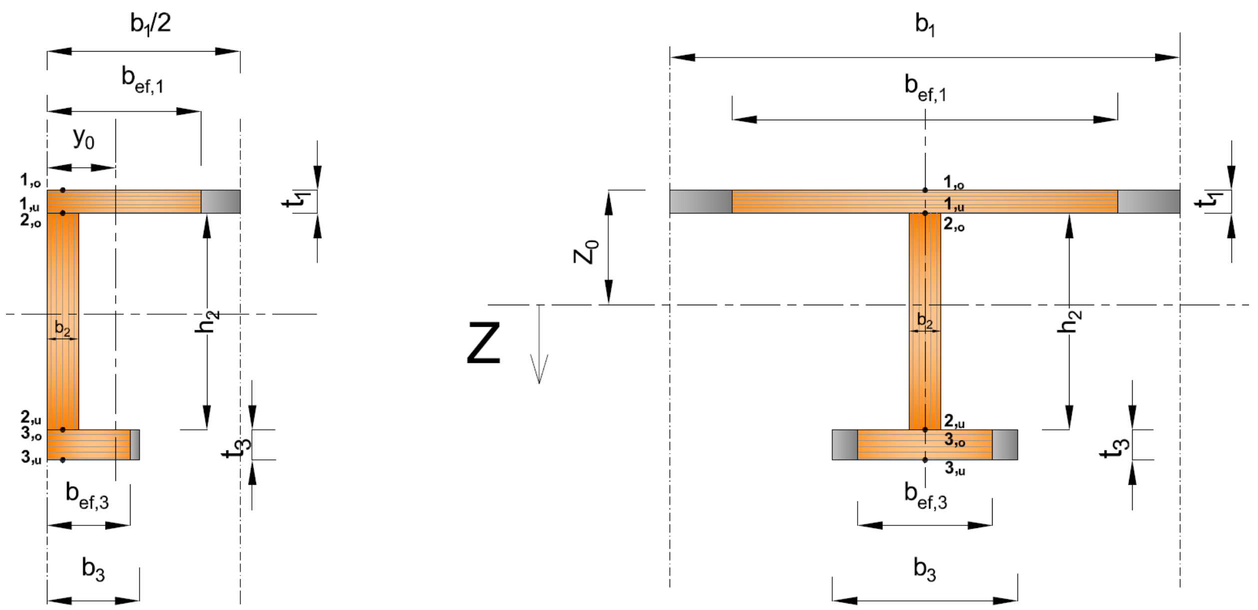


Figure 25: Calculation points of bending stress for rib panel.

For I-section and U-section in open box rib panel:
The mean stress or extreme fiber stress may be calculated as

$$\sigma_i(x,z) = \frac{E_i \cdot z_i \cdot M_y(x)}{EI_{eff}} \qquad \text{Eq 93}$$

$\sigma_i(x,z)$	Normal stress at location x in a rib panel beam at coordinate z in the section
E_i	Young´s Modulus at coordinate z
$M_y(x)$	Bending moment M_y at location x (positive when upper part is compressed)
z_i	coordinate z of the point “i” where the stress is being analysed (distance to the neutral axis (positive downwards))
EI_{eff}	Effective flexural rigidity

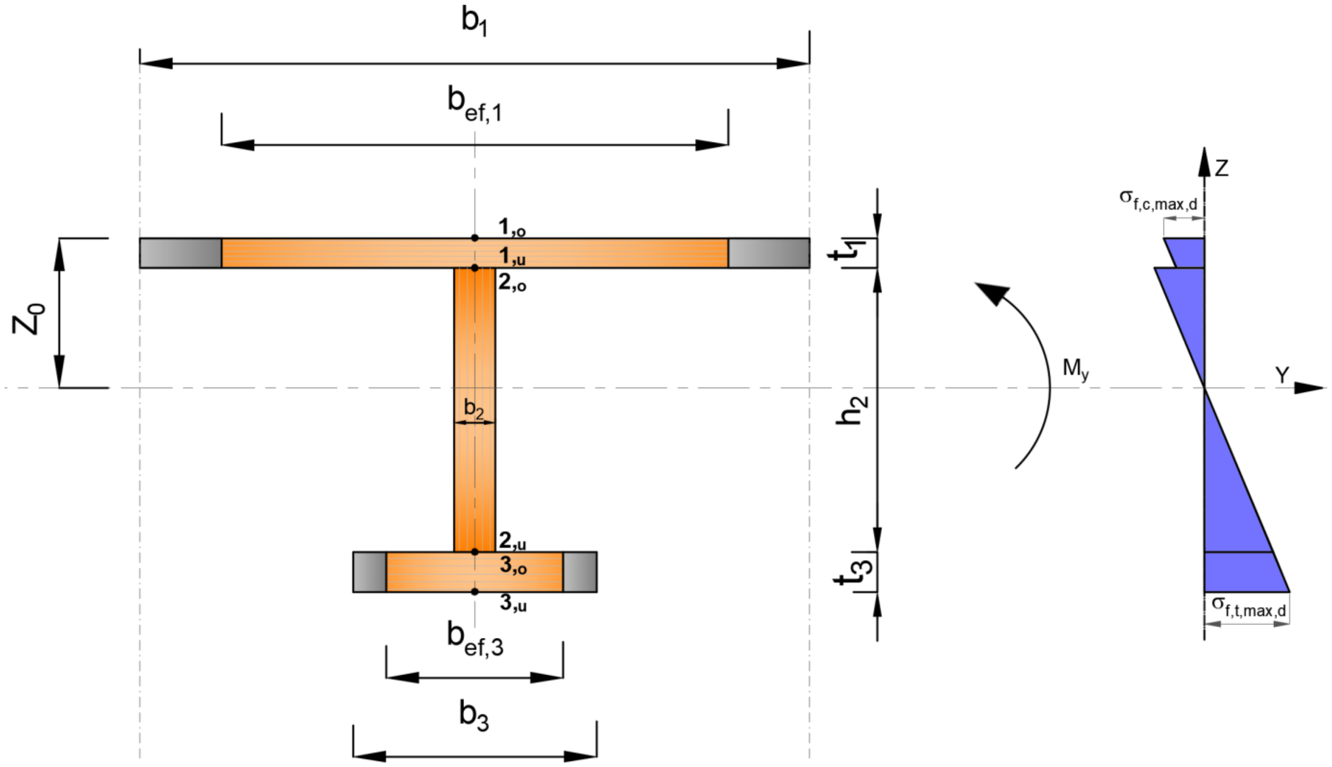


Figure 26: Bending stress distribution

The stress analysis shall be performed at the top and bottom flanges and at the LVL web(rib).
The compression/tension stresses for the flanges have to be verified in the center of the partial section and the flexural stress in the web has to be verified at the edges.
The stress analysis shall be performed according to EN 1995-1-1, item 9.1.2 since it is a **glued thin-flange beam**.

8.2 Normal stress analysis:

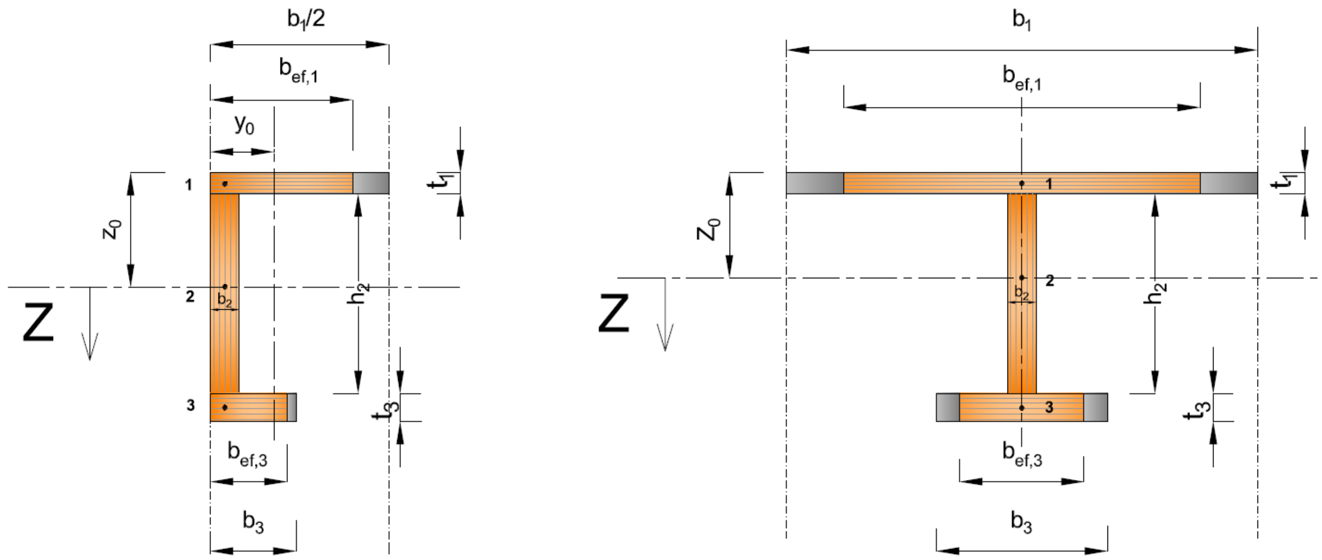


Figure 27: Calculation points of bending stress for rib panel

8.3 Verification of the stresses

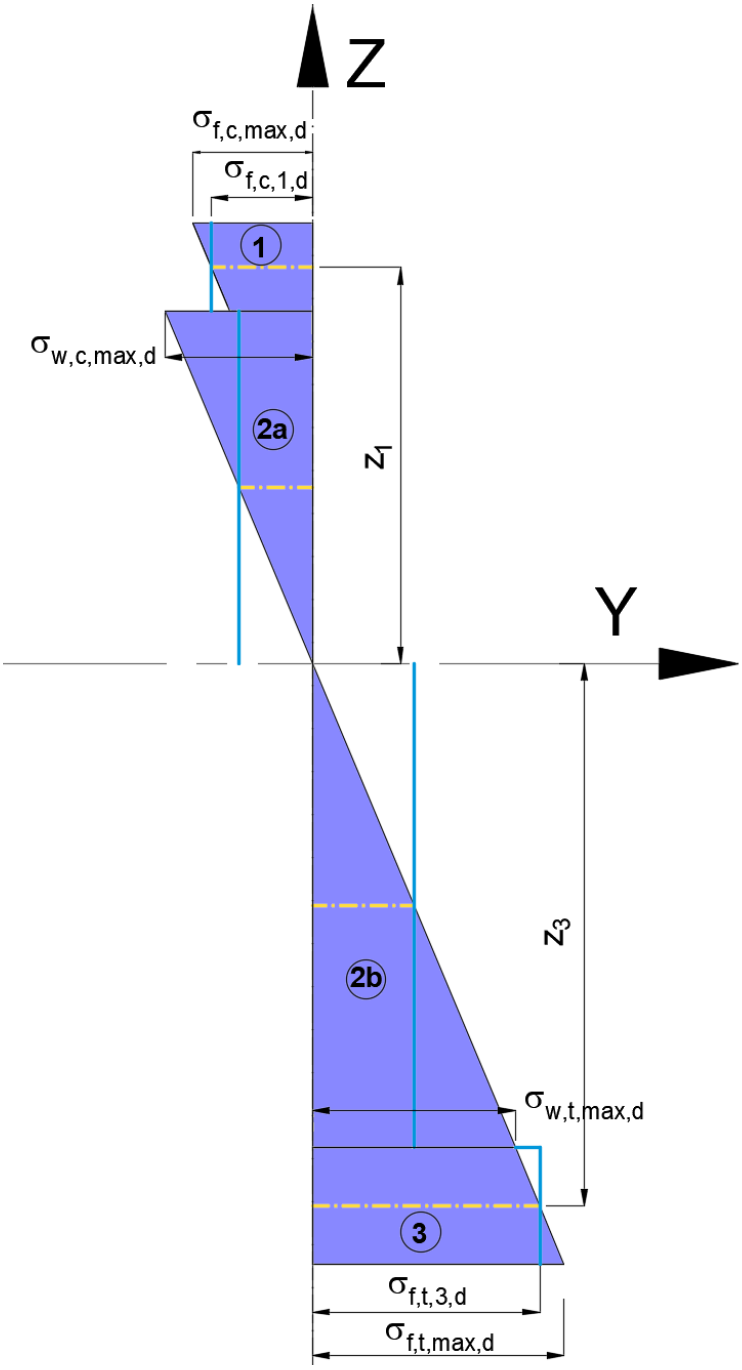


Figure 28: Stress distribution

Bending in transversal direction:
The bending stresses in the flanges should satisfy the following expressions
Maximum transversal bending stresses in a slab between ribs should satisfy the following expression

$$\sigma_d \leq f_{m,90,flat,d} \quad \text{Eq 94}$$

Compression-Tension:

At the same time the following requirements according to EN 1995-1-1, item 9.1.1(4) (for the rib) and 9.1.2(7) (for the flanges) shall be fulfilled.
The axial stresses in the flanges, based on the relevant effective flange width, should satisfy the following expressions:

For the flanges

$$|\sigma_{f,c,d}| \leq k_c \cdot f_{f,c,0,d}$$ *Eq 95*

$$\sigma_{f,t,d} \leq f_{f,t,0,d}$$ *Eq 96*

For the rib (web)

$$\sigma_{w,m,0,d} \leq f_{m,w,0,d}$$

with

$$\sigma_{w,m,0,d} = max(\sigma_{w,c,max,d}; \sigma_{w,t,max,d}) \leq f_{m,w,0,d}$$ *Eq 97*

where
$\sigma_{f,c,d}$ is the upper flange mean design compressive stress;
$\sigma_{f,t,d}$ is the bottom flange mean design tensile stress;
$\sigma_{w,c,d}$ is the rib design compressive stress;
$\sigma_{w,t,d}$ is the rib design tensile stress;
$\sigma_{w,m,0,d}$ is the rib design bending stress;
$f_{f,c,0,d}$ is the flange design compressive strength;
$f_{f,t,0,d}$ is the flange design tensile strength;
$f_{m,w,0,d}$ is the rib design bending strength.

Note that the bottom flange of the LVL Rib Panel needs to be in tension in any cases. The inclination angle (if there is one) should respect this condition. (specified in the ETA)

8.4 Normal stresses from axial force

$$\sigma_{c,d} = \frac{E_i \cdot N_{ed}}{EA_{eff}}$$ *Eq 98*

$$EA_{eff} = \sum E_i \cdot A_i$$ *Eq 99*

The normal stresses in the flanges and in the rib shall be executed according to EN 1995-1-1 and should satisfy the following expressions:

$$|\sigma_{c,d}| \leq k_c \cdot f_{c,0,d}$$ *Eq 100*

$$\sigma_{t,d} \leq f_{t,0,d}$$ *Eq 101*

8.5 Combined stress:

In case of an inclination of the rib panels, an axial load component will appear. These equations apply to the entire section, from top to bottom. For flanges, the axial stress from normal forces are directly added to the mean axial stresses from the bending moment. Therefore, the verification shall be only with the design compressive/tensile strength.

The stress design then shall be executed according to EN 1995-1-1, items 6.2.3 and 6.2.4:

Bending and tensile stress:

For the flanges

$$\frac{\sigma_{t,i,d}}{f_{t,i,d}} + \frac{\sigma_{f,t,max,d}}{f_{f,m,0,d}} \leq 1$$ Eq 102

For the rib (web)

$$\frac{\sigma_{t,i,d}}{f_{t,i,d}} + \frac{\sigma_{w,t,max,d}}{f_{m,w,0,d}} \leq 1$$ Eq 103

$\sigma_{t,i,d}$	Design tensile stress in the partial section i coming from normal force;
$\sigma_{f,t,max,d}$	Maximum design tensile stress in the bottom flange coming from the bending moment;
$f_{t,i,d}$	Design tensile strength of the partial section i;
$f_{f,m,0,d}$	Design bending strength of the flange;
$\sigma_{w,t,max,d}$	Maximum design tensile stress in the rib;
$f_{m,w,0,d}$	Design bending strength of the rib.

Bending and compressive stress:

For the flanges

$$\left(\frac{\sigma_{c,i,d}}{f_{c,i,d}}\right)^2 + \frac{\sigma_{f,c,max,d}}{f_{f,m,0,d}} \leq 1$$ Eq 104

For the rib (web)

$$\left(\frac{\sigma_{c,i,d}}{f_{c,i,d}}\right)^2 + \frac{\sigma_{w,c,max,d}}{f_{m,w,0,d}} \leq 1$$ Eq 105

$\sigma_{c,i,d}$	Design compressive stress in the partial section i
$\sigma_{f,c,max,d}$	Maximum design compressive stress in the upper flange coming from the bending moment
$f_{c,i,d}$	Design compressive strength of the partial section i
$f_{f,m,0,d}$	Design bending strength of the flange
$\sigma_{w,c,max,d}$	Maximum design compressive stress of the rib
$f_{m,w,0,d}$	Design bending strength of the rib

8.6 Bending strength

LVL-S Rib

$$f_{(LVL-S),m,0,edge,d} = \frac{k_{mod} \cdot f_{(LVL-S),m,0,edge,k} \cdot k_h}{\gamma_{m,LVL-S}}$$ Eq 106

$f_{(LVL-S),m,0,edge,d}$	Design bending strength for LVL-S (edgewise)
k_{mod}	Factor, according to EN1995-1-1, Table 3.1, taking a load duration (permanent – very short) and the service class (1-3) into consideration
$f_{(LVL-S),m,0,edge,k}$	Characteristic bending strength of the LVL-S lamination material, according to VTT-S-05710-17)
$\gamma_{m,LVL-S}$	Partial safety coefficient, applicable for LVL, according to either EN1995-1-1, Table 2.3, or some local regulations.
k_h	Depth factor in bending according to EN 1995-1-1, item 3.4 (3)

LVL-X panel

$$f_{(LVL-X),m,0,flat,d} = \frac{k_{mod} \cdot f_{(LVL-X),m,0,flat,k}}{\gamma_{m,LVL-X}}$$ Eq 107

$f_{(LVL-X),m,0,flat,d}$	Bending strength flatwise parallel to grain for LVL-X
k_{mod}	Factor, according to EN1995-1-1, Table 3.1, taking a load duration (permanent – very short) and the service class (1-3) into consideration
$f_{(LVL-X),m,0,flat,k}$	Characteristic bending strength flatwise parallel to grain of LVL-X, according to VTT-S-05550-17
$\gamma_{m,LVL-X}$	Partial safety coefficient, applicable for LVL, according to either EN1995-1-1, Table 2.3, or some local regulations.
k_{mod}	Partial safety coefficient, applicable for LVL, according to EN1995-1-1, Table 2.3

$$f_{(LVL-X),m,90,flat,d} = \frac{k_{mod} \cdot f_{(LVL-X),m,90,flat,k}}{\gamma_{m,LVL-X}}$$ Eq 108

$f_{(LVL-X),m,90,flat,d}$	Bending strength flatwise perpendicular to grain for LVL-X
k_{mod}	Factor, according to EN1995-1-1, Table 3.1, taking a load duration (permanent – very short) and the service class (1-3) into consideration
$f_{(LVL-X),m,90,flat,k}$	Characteristic bending strength flatwise perpendicular to grain of LVL-X, according to VTT-S-05550-17)
$\gamma_{m,LVL-X}$	Partial safety coefficient, applicable for LVL, according to either EN1995-1-1, Table 2.3, or some local regulations.
k_{mod}	Partial safety coefficient, applicable for LVL, according to EN1995-1-1, Table 2.3

8.7 Tensile strength

LVL-S rib and bottom flange

$$f_{(LVL-S),t,0,d} = \frac{k_{mod} \cdot f_{(LVL-S),t,0,k} \cdot k_l}{\gamma_{m,\mathrm{LVL-S}}}$$ Eq 109

$f_{(LVL-S),t,0,d}$	Design tensile strength for LVL-S
k_{mod}	Factor, according to EN1995-1-1, Table 3.1, taking a load duration (permanent – very short) and the service class (1-3) into consideration
$f_{(LVL-S),t,0,k}$	Characteristic tensile strength of the LVL-S lamination material, according VTT-S-05710-17)
$\gamma_{m\mathrm{LVL-S},}$	Partial safety coefficient, applicable for LVL, according to either EN1995-1-1, Table 2.3, or some local regulations.
k_l	Depth factor in tension according to EN 1995-1-1, item 3.4 (3)

8.8 Compressive strength

LVL-S rib

$$f_{(LVL-S),c,0,d} = \frac{k_{mod} \cdot f_{(LVL-S),c,0,k}}{\gamma_{m,LVL-S}}$$ Eq 110

$f_{(LVL-S),c,0,d}$	Design compressive strength for LVL-S
k_{mod}	Factor, according to EN1995-1-1, Table 3.1, taking a load duration (permanent – very short) and the service class (1-3) into consideration
$f_{(LVL-S),c,0,k}$	Characteristic compressive strength of the LVL material, according to VTT-S-05710-17)
$\gamma_{m,LVL-S}$	Partial safety coefficient, applicable for LVL, according to either EN1995-1-1, Table 2.3, or some local regulations.

LVL-X panel

$$f_{(LVL-X),c,0,d} = \frac{k_{mod} \cdot f_{(LVL-X),c,0,k}}{\gamma_{m,LVL-X}}$$ Eq 111

$f_{(LVL-X),c,0,d}$	Design compressive strength for LVL-X
k_{mod}	Factor, according to EN1995-1-1, Table 3.1, taking a load duration (permanent – very short) and the service class (1-3) into consideration
$f_{(LVL-X),c,0,k}$	Characteristic compressive strength of LVL-X, according to VTT-S-05550-17
$\gamma_{m,LVL-X}$	Partial safety coefficient, applicable for LVL, according to EN1995-1-1 [4], Table 2.3

8.8.1 Stability – buckling of compression members

Condition according to EN1995-1-1 [4], item 6.3.2 (3) needs to be fulfilled.

This applies to the LVL-X as well as to the LVL-S rib.

Buckling length

l_{ef} is the buckling length of the beam, according to the Table 16.

Table 16: Effective length as a ratio of the span

Beam type	Loading type	l_{ef}/l [a]
Simply supported	Constant moment Uniformly distributed load Concentrated force at the middle of the span	1,0 0,9 0,8
Cantilever	Uniformly distributed load Concentrated force at the free end	0,5 0,8
[a] The ratio between effective length l_{ef} and the span l is valid for a beam with torsionally restrained supports and loaded at the centre of gravity. If the load is applied at the compression edge of the beam, l_{ef} should be increased by 2h and may be decreased by 0.5 h for the tension edge of the beam.		

If the compressed side of the LVL Rip Panel is laterally supported with spacing “a” that may be used as effective length taking account the note in the Table 16 (usually l_{ef} = a + 2h).

8.8.2 Stability – lateral torsional buckling (LTB) of compression members

8.8.2.1 For Open, Semi-Open and Closed LVL rib panel with LVL-X above the ribs

No lateral torsional buckling is possible. At the compression side of the section, the LVL-X panel is providing stability out of plane. Nevertheless

8.8.2.2 For inverted LVL rib panels with LVL-X below the ribs

In this case the compression zone is in the rib and therefore the rib might be vulnerable to lateral torsional buckling. To account for lateral torsional buckling, it shall be sufficient, to run a buckling analysis (according to EN1995-1-1 [1], item 6.3.2 (3)) for the extreme half of the compression zone.
The length shall be the distance between the blockings that are placed between the ribs. If lateral torsional buckling is still critical, the distance between the blockings can be reduced.

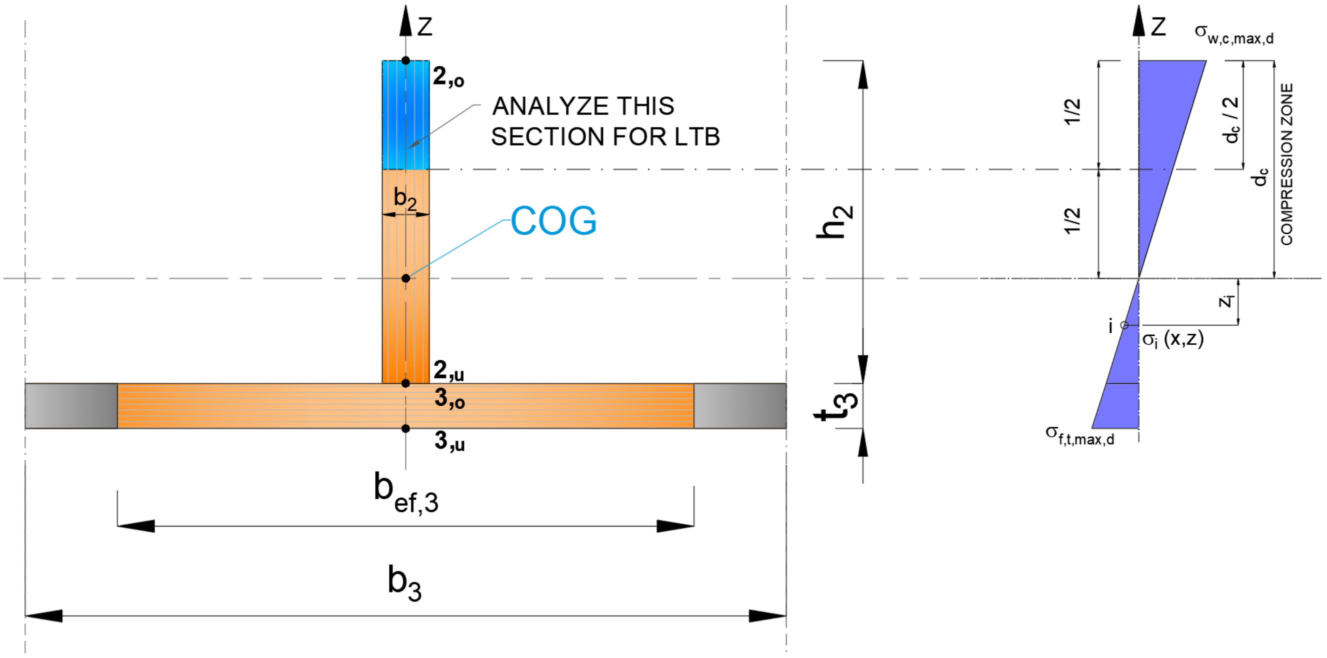


Figure 29: Inverted section for lateral torsional buckling analysis

Lateral torsional stability shall be verified both in the case where only a moment M_d exists about the strong axis and where a combination of moment $M_{d,b}$ and compressive force N_c exists.

The K_{crit} factor which takes into account the reduced bending strength due to lateral torsional buckling shall be calculated according to EN1995-1-1 [4] (6.3.3) and the Kc factor which is the instability factor shall be calculated according to EN1995-1-1 [4] (6.3.2)

The relative slenderness ratio may be calculated as

$$\lambda_{rel,m}=\sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} \qquad \text{Eq 112}$$

The critical bending stress may be calculated as

$$\sigma_{m,crit}=\frac{M_{y,crit}}{W_y}=\frac{\pi\sqrt{E_{0,05}\,I_z\,G_{0,05}I_{tor}}}{l_{ef}\,W_y} \qquad \text{Eq 113}$$

l_{ef} is the buckling length of the beam, distance between lateral supports

$$\left(\frac{\sigma_{m,d}}{K_{crit}\cdot f_{m,d}}\right)^2+\frac{\sigma_{c,d}}{K_c\cdot f_{c,0,d}}\leq 1 \qquad \text{Eq 114}$$

9. Shear resistance

9.1 Shear stress

Shear stress of the section needs to be checked in different locations. The most essential points within a section are:

- C.O.G. (center of gravity)
- Glue line between rib and LVL-S/X panels

Shear stresses should be calculated points given in Figure 30:

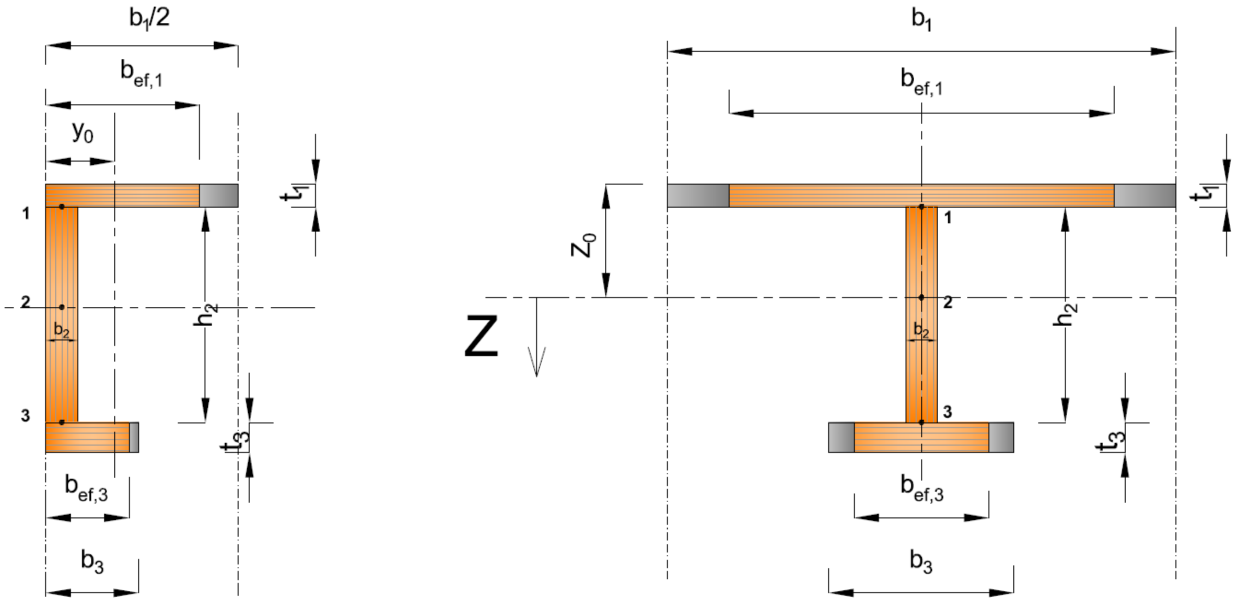


Figure 30: Calculation points of shear stress.

Shear stresses should satisfy the following expression in ribs

$$\tau_{i,d} \leq f_{v,LVL-S,0,edge,d} \qquad \text{Eq 115}$$

$f_{v,0,edge,d}$ is the design shear strength

Shear stresses in slab and flange at glued joints should satisfy the following expression

$$\tau_{mean,d} \leq \begin{cases} f_{v,LVL,0,flat,d} & for\ b_2 \leq 8h_f \\ f_{v,LVL,0,flat,d}\left(\dfrac{8h_f}{b_2}\right)^{0,8} & for\ b_2 > 8h_f \end{cases} \qquad \text{Eq 116}$$

where

$\tau_{mean,d}$ is design shear stress, assuming a uniform stress distribution
$f_{v,LVL,0,flat,d}$ is the design shear strength (LVL-S or LVL-X) flatwise
h_f is either t_1 or t_3

For edge parts of the rib panel limit $8h_f$ should be replaced by $4h_f$.

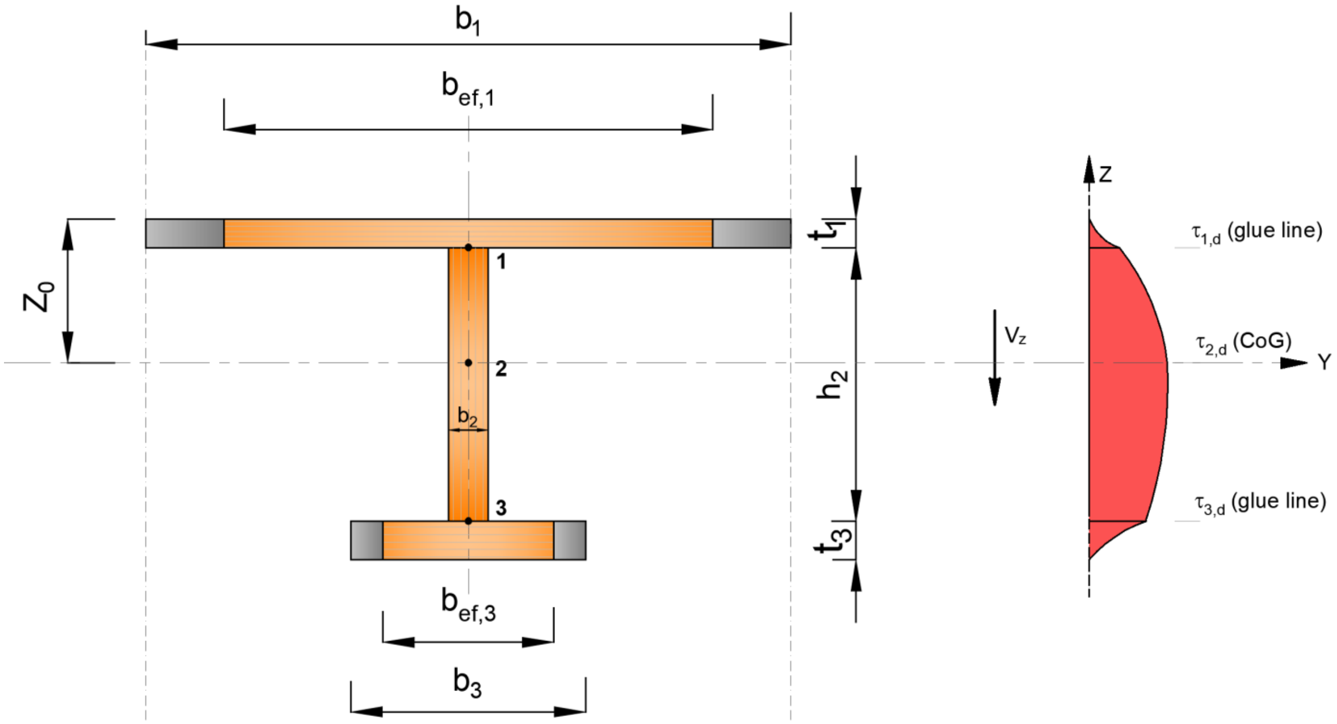


Figure 31: Shear stress distribution

9.2 Shear of the glue lines

Shear tests have shown that the shear strength of the glue line is higher than the shear strength in the timber parts that are glued to another and the shear failure will not occur directly in the glue joint, but the timber adjacent to the glue line will fail in shear parallel to the grain.

Therefore, it is assumed that sufficient shear strength in the joint is given, if the design equation in item 9.1 are met.

9.3 Analysis of shear stresses

Shear stresses should be calculated in points, given in Figure 30.

$$\tau(z)_d = E_i \cdot \frac{S_y(z) \cdot V_{z,d}}{EI_{y,ef} \cdot b(z)}$$ Eq 117

$\tau(z)_d$	Design shear stress at a given coordinate "z" [N/mm²]
E_i	Young´s modulus of the respective layer/rib
$S_y(z)$	Static moment at the coordinate "z", about the Y-axis [mm³]
$V_{z,d}$	Design shear force [N]
$I_{y,ef}$	Effective moment of inertia about the Y-axis [mm⁴]
$b(z)$	Width of the section at given "z" coordinate
$EI_{y,ef}$	Effective bending rigidity [N·mm²]

$$S_y(z) = \sum_i A_i \cdot e_{z,i}$$ Eq 118

$S_y(z)$	Static moment at the coordinate "z", about the Y-axis [mm³]
A_i	Area of the partial surface "i" [mm²]
$e_{z,i}$	Eccentricity of the partial surface "i" = distance between partial C.O.G. of partial surface "i" and total C.O.G. of the entire section [mm]

Without a detailed buckling analysis, the following expressions should be satisfied

$$h_2 \leq 70b_2 \quad \text{Eq 119}$$

and

$$V_{w,Ed} \leq \begin{cases} b_2 h_2 \left(1 + \frac{0.5(t_1 + t_3)}{h_2}\right) f_{v,0,edge,d} & for\ h_2 \leq 35b_2 \\ 35b_2^2 \left(1 + \frac{0.5(t_1 + t_3)}{h_2}\right) f_{v,0,edge,d} & for\ 35b_2 \leq h_2 \leq 70b_2 \end{cases} \quad \text{Eq 120}$$

where $V_{w,Ed}$ is the shear force acting on the rib. A conservative estimate is that $V_{w,Ed} = V_{Ed}$.
V_{Ed} is the design shear force

For I-section, T section and U-section in open/semi-open/box rib panel:

Shear stress at point 1 may be calculated as

$$\tau_{1,d} = \frac{V_{Ed} E_{0,mean,1} b_{ef,1} t_1 \left(z_0 - \frac{t_1}{2}\right)}{b_2 EI} \quad \text{Eq 121}$$

Shear stress at point 2 may be calculated as

$$\tau_{2,d} = \frac{V_{Ed} \left(E_{0,mean,2} \frac{1}{2} b_2 (z_0 - t_1)^2 + E_{0,mean,1} b_{ef,1} t_1 \left(z_0 - \frac{t_1}{2}\right)\right)}{b_2 EI} \quad \text{Eq 122}$$

Shear stress at point 3 may be calculated as

$$\tau_{3,d} = \frac{V_{Ed} E_{0,mean,3} b_{ef,3} t_3 \left(-z_0 + t_1 + h_2 + \frac{t_3}{2}\right)}{b_2 EI} \quad \text{Eq 123}$$

Shear stress in slab perpendicular to the ribs may be calculated as

$$\tau_d = \frac{3}{2} \frac{V_{Ed,slab}}{t_1} \quad \text{Eq 124}$$

$V_{Ed,slab}$ is the design shear force in slab per unit length

9.4 Shear strength

LVL-S Rib

$$f_{v(LVL-S),0,edge,d} = \frac{k_{mod} \cdot k_{cr} \cdot f_{v(LVL-S),0,edge,k}}{\gamma_{M,LVL-S}} \quad \text{Eq 125}$$

$f_{v(LVL-S),0,edge,d}$	Design shear strength for LVL-S
k_{mod}	Factor, according to EN1995-1-1 [4], Table 3.1, taking a load duration (permanent – very short) and the service class (1-3) into consideration
k_{cr}	Crack coefficient according to EN 1995-1-1, item 6.1.7 (Recommendation for LVL $\boldsymbol{k_{cr} = 1.0}$)
$f_{v(LVL-S),0,edge,k}$	Characteristic shear strength of the LVL-S, according to VTT-S-05710-17
$\gamma_{m,LVL-S}$	Partial safety coefficient, applicable for LVL, according to EN1995-1-1, Table 2.3, or local regulations.

LVL-X panel

$$f_{v(LVL-X),0,flat,d} = \frac{k_{mod} \cdot f_{v(LVL-X),0,flat,k}}{\gamma_{M,LVL-X}}$$ Eq 126

$f_{v(LVL-X),0,flat,d}$	Design shear strength for LVL-X flatwise parallel to the grain
k_{mod}	Factor, according to EN1995-1-1 [4], Table 3.1, taking a load duration (permanent – very short) and the service class (1-3) into consideration
$f_{v(LVL-X),0,flat,k}$	Characteristic shear strength of the LVL-X, according to VTT-S-05550-17
$\gamma_{m,LVL-X}$	Partial safety coefficient, applicable for LVL, according to EN1995-1-1, Table 2.3, or some local regulations

$$f_{v(LVL-X),90,flat,d} = k_{mod} \frac{f_{v(LVL-X),90,flat,k}}{\gamma_{M,LVL-X}}$$ Eq 127

$f_{v(LVL-X),90,flat,d}$	Design shear strength for LVL-X flatwise perpendicular to the grain
k_{mod}	Factor, according to EN1995-1-1 [4], Table 3.1, taking a load duration (permanent – very short) and the service class (1-3) into consideration
$f_{v(LVL-X),90,flat,k}$	Characteristic shear strength of the LVL-X, according to VTT-S-05550-17
$\gamma_{m,LVL-X}$	Partial safety coefficient, applicable for LVL, according to EN1995-1-1, Table 2.3, or some local regulations.

LVL-S bottom flange

$$f_{v(LVL-S),0,flat,d} = \frac{k_{mod} \cdot f_{v(LVL-S),0,flat,k}}{\gamma_{M,LVL-S}}$$ Eq 128

$f_{v(LVL-S),0,flat,d}$	Design shear strength for LVL-S flatwise parallel to the grain
k_{mod}	Factor, according to EN1995-1-1 [4], Table 3.1, taking a load duration (permanent – very short) and the service class (1-3) into consideration
$f_{v(LVL-S),0,flat,k}$	Characteristic shear strength of the LVL-S, according to VTT-S-05710-17
$\gamma_{m,LVL-S}$	Partial safety coefficient, applicable for LVL, according to EN1995-1-1, Table 2.3, or some local regulations.

10. Connection between rib panels and adjacent structures- Structural behaviour of LVL Rib Panel diaphragms

10.1 Longitudinal joints of LVL rib panels

The figures below show typical joint connections between two LVL rib panels. The connection shall be made between the LVL panels and designed in a way that all applicable shear forces, in the plane and/or out of the plane of LVL plate can be transferred.

The design of the screws shall be done according to the applicable ETA.

LVL rib panels can be joined in the longitudinal direction by means of:

(a) Screwed connection at the edge ribs of the chords or;
(b) Supporting pole connection screwed at the edge ribs of the chords or;
(c) Cover strip connection screwed between the chords without edge ribs.

The proposed solution of longitudinal joints connections are not exhaustive, other details are possible under the condition that they are justified in seismic zones and that the edge distances given by the screw's manufacturers are respected.

The connection between two Rib panels must ensure:

• the transmission of vertical shear forces (differential distributed loads, point loads, etc.)

• the transmission of longitudinal shear forces (overall bracing of the building by panels, horizontal differential loads, etc.)

• leveling (service)

• sealing for fire, acoustic, airtightness, etc.

10.2 Screwed connection at the edge ribs

The connection shall be done with fully threaded cylinder head screws. The screwed connection shall be executed crossed at 45°, with screws coming from either side of the joint. This screwing mode allows to carry the horizontal and vertical shear forces. The design of such connection between two LVL plates can be carried out using the software "CALCULATIS by Stora Enso".

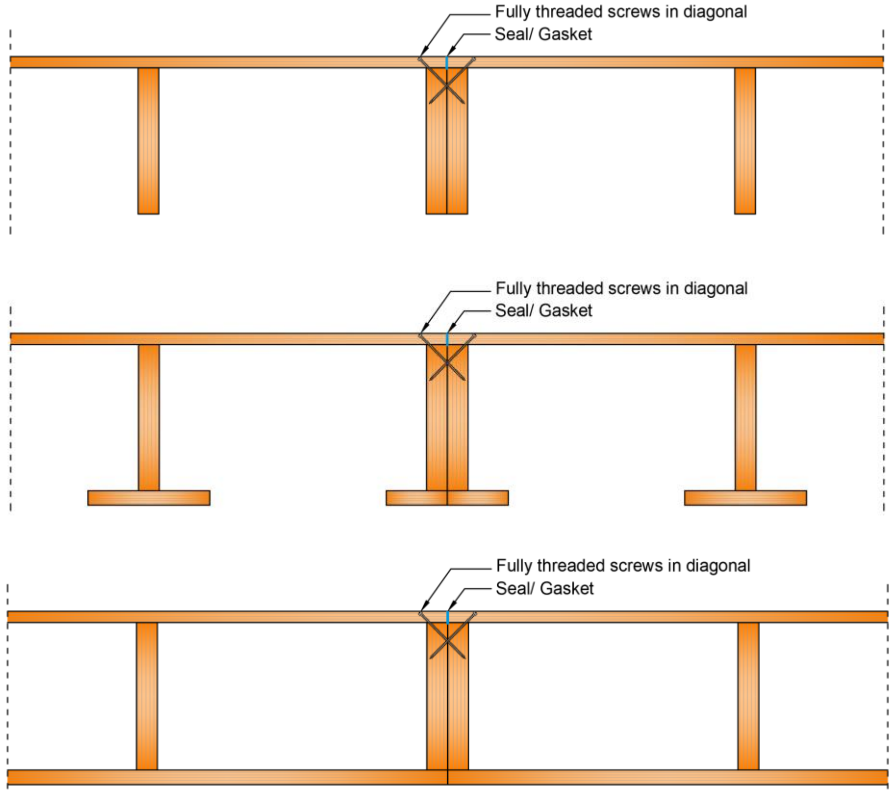


Figure 32: Assembly of adjacent LVL Rib Panels by means of digonal screwed connection at the edge ribs.

<u>Recommendation for the assembling on site:</u>

The fully threaded screws will not pull the two pieces tightly together because the threaded part works in both pieces, therefore partially threaded screws are first needed to lock the element in place and to avoid gaps at the longitudinal joint. This applies not only to the adjacent rib panel element, but also to e.g. supporting beams. Semi threaded screws will be used in the same way as the structural connection screws, but the quantity shall be according to the "locking" needs. Just a few screws on each connection should be enough, but this must be confirmed at site and can't be well quantified in instructions. After the connection is locked, the fully threaded screws can be places for the final connection.

Also, it is necessary to ensure a continuity of the diaphragm plane and avoid if possible, the transfer of the shear forces from a LVL plates to LVL ribs via the glue lines.

The connection type must be compatible with the assembly (top or bottom) and production constraints (open or closed type).

When shear forces are too high, it is possible to use perforated tapes nailed on top of the junction for taking the horizontal shear forces, and in addition the screws would take the vertical shear forces.

10.3 Supporting pole connection screwed at the edge ribs

Figure 33 shows another type of connection using a timber supporting pole between the two elements.

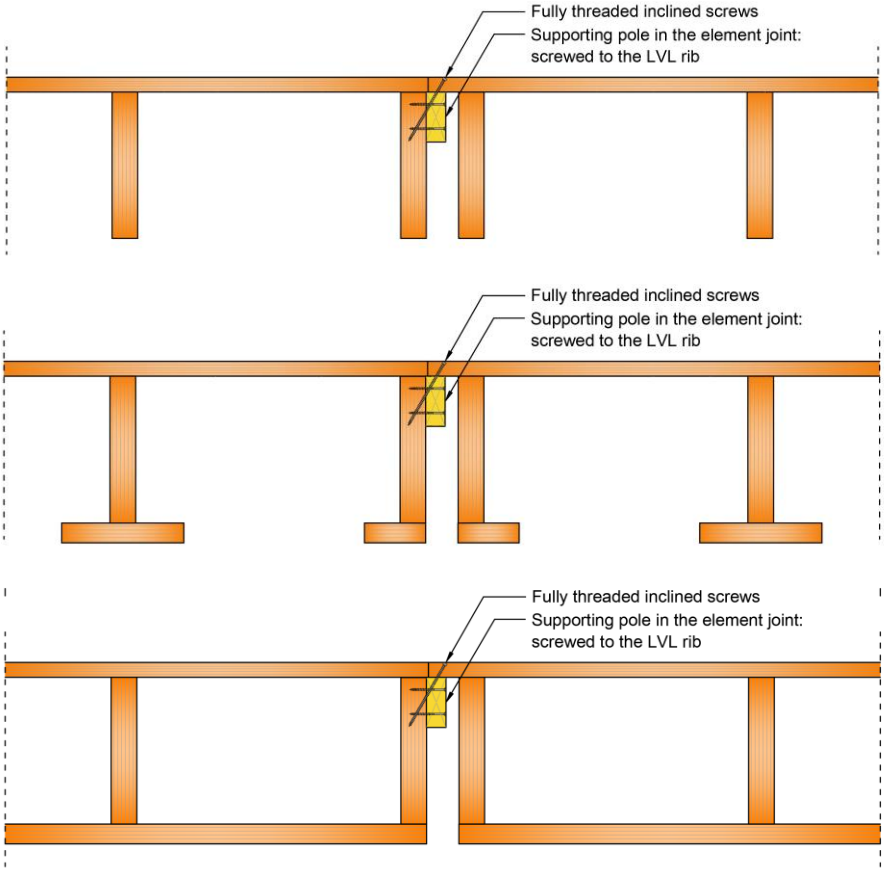


Figure 33: Assembly of adjacent LVL Rib Panels by means of supporting pole screwed at the edge ribs.

10.4 Cover strip connection screwed between the LVL chords without edge ribs

Figure 34 shows another connection type using cover strip between the LVL plates without edge ribs for different rib panel configurations.

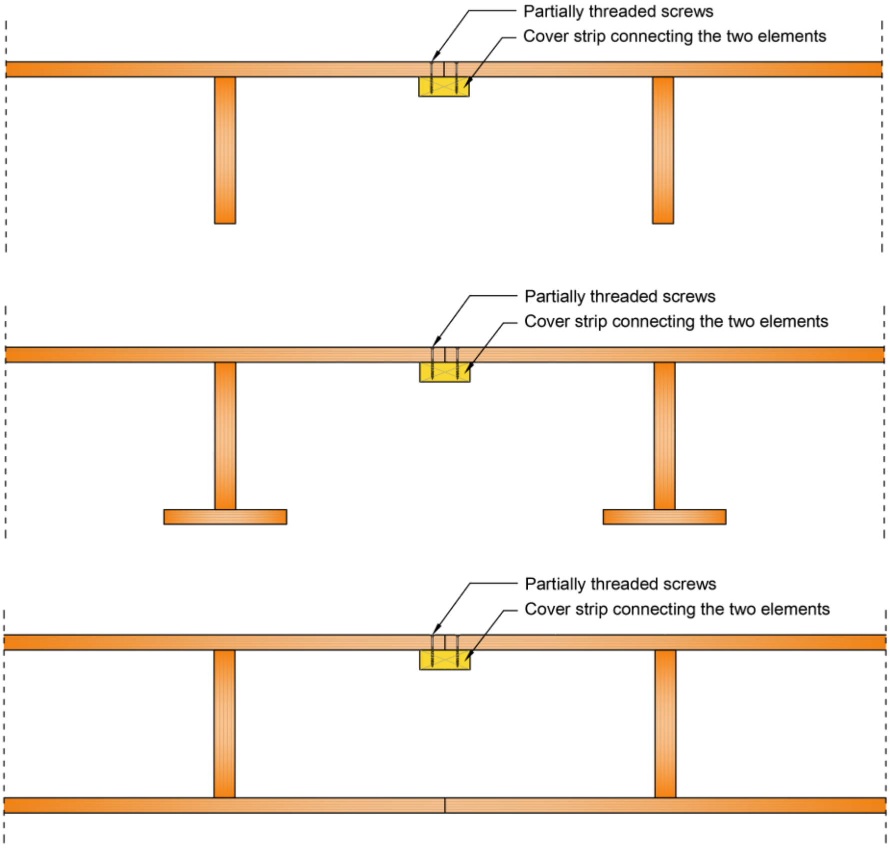


Figure 34: Assembly of adjacent LVL Rib Panels by means of cover strip connection screwed at the LVL plates with edge ribs.

10.4.1 Verification of the longitudinal joint without edge ribs

In EN 1995-1-1 no specification or rule for the vertical loads to be transferred by the longitudinal joint of slabs is given. Thus, an engineering judgement has to be applied for its determination. Since the stiffness of LVL rib panels in the cross direction is much smaller than in the longitudinal (rib) direction $(E \cdot I)_x >> (E \cdot I)_y$ and the twisting stiffness is also small $(<< (G \cdot I)_{xy})$ only a small portion of the load is carried in the cross direction. Nevertheless, forces at the coupling edges of adjacent elements should be considered based on experience.

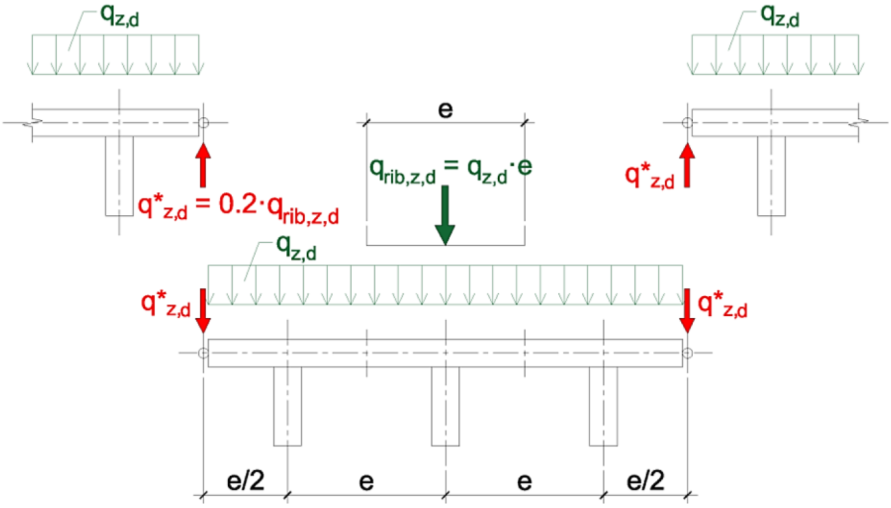


Figure 35: Design load q*z,d for longitudinal joints.

1) If the LVL rib panel is loaded by dominant continuously distributed loads, the longitudinal joints and its fasteners respectively shall be designed (in addition to the forces from the structural analysis) for a line load of 20% of the aliquot (part of portion) line load per rib based on a 1lm length.
2) Individual line (e. g. eccentrically (regarding the rib) situated walls) and single loads (e. g. due to machines) shall be considered using appropriate structural models. If up-lifting forces may occur the corresponding forces shall be transferred by reinforcements of the contact line between plate and rib.
3) The edges of all elements must be anchored to the walls and supporting elements respectively. (e.g. lintel beams) below.
4) If the LVL rib panel is used as diaphragm, the relevant combined vertical and horizontal loads and combinations must be considered in the design of the joint by appropriate models.

10.4.2 Vertical forces at the longitudinal edges

The following arguments can be quoted for the determination of the force to be taken into consideration:

Different loadings of adjacent elements

This effect will be demonstrated by means of a calculation example given below.

Example :

$e\ = 800\ mm$

Actions :

$\gamma_G = 1.35\ |\ \gamma_Q = 1.50\ |\ G_k = 2.00\ kN/m^2\ |\ P_k = 3.00\ kN/m^2$

Element 1:

$$E_{d,1} = \gamma_G \cdot G_k + \gamma_Q \cdot P_k = 1.35 \cdot 2.00 + 1.50 \cdot 3.00 = 7.20\ kN/m^2$$

Element 2:

$$E_{d,2} = \gamma_G \cdot G_k + \gamma_Q \cdot P_k = 1.35 \cdot 2.00 + 1.50 \cdot 0 = 2.70\ kN/m^2$$

Analysis is worked out for a 1.00 m wide stripe on a 1-D structural system. The ribs are simulated as springs with stiffness in the middle of the bay. The longitudinal joints are modelled as ideal hinge.

Stiffness of springs (ribs) in cross direction:

$$k = \frac{1}{w} = \frac{48 \cdot EI}{"1" \cdot l^3}\ [kN/m/m]$$

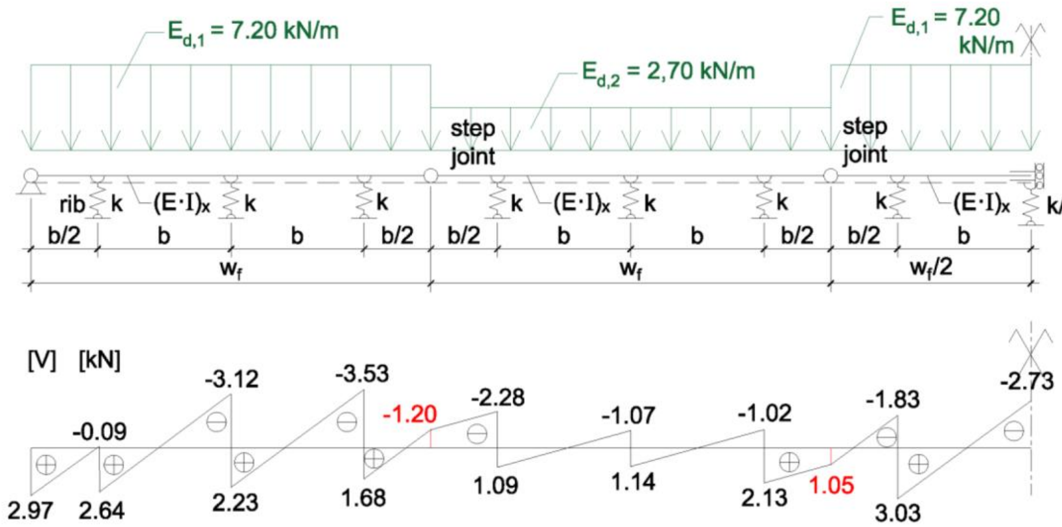


Figure 36: Analysed model and distribution of vertical forces

Result

In the example above, the (maximum) shear force at the hinge (in the middle of the field) is $v_{x,d}$ = 1.20 kN/m.

According to the proposed rule, the (vertical) linear load to consider in the design of the joint would be:

$$q_d^* = 20\% \cdot (E_{d,1} \cdot b) = 0.20 \cdot (7.20 \cdot 0.80) = 1.15\ kN/m$$

Comparable specifications

In addition to the already mentioned arguments, a look on specifications for other building materials is expedient.

For example:

Rules for solid slabs in EN 1992-1-1

Excerpt from EN 1992-1-1, Clause 9.3.1.1 (2)

"Secondary transverse reinforcement of not less than 20% of the principal reinforcement should be provided in one-way slabs."

This is in a comparable range as it is specified for concrete slabs in the cross direction. The proposed rule allows a pragmatic, simple and quick estimation of the vertical force in the joint.

Summary

As shown, it is arguable that longitudinal joints of LVL rib panels shall be designed for a certain load transfer in the vertical direction, although there are no rules given in standards. The two examples in this chapter demonstrate the effects and are non-exhaustive.

The vertical shear forces in the longitudinal hinge are approximately distributed in a sinusoidal form and the given values are the maximum in the middle of the longitudinal joint. It is recommended to define the vertical load to be transferred by the fasteners of the longitudinal joint with 20% of the total load per rib.

10.5 LVL Rib Panel as horizontal diaphragm

Bonded joints are not to be considered as dissipative zones within the norm EN 1998-1-1 (§8.2). The transfer of seismic shear forces at the sides of the panels to the side walls will therefore be carried out by means of connectors and mechanical fasteners.

Since the glue joint between the chord and the rib has not been tested under cyclic load, the shear transfer from the diaphragm to the walls will be handled in such a way that the forces are transferred from the LVL plates to the lateral walls directly.

- In the case where the LVL Rib Panel by Stora Enso rests directly on the walls, the transfer is done first from the top chord to the wall (CLT or GLVL for example) via metal brackets or to an equivalent system, then to the underside rib/blocking via a perforated metal plate for example. Finally, the transfer from the rib/blocking to the lower wall on which it rests can be carried out either via perforated metal plates or by inclined screws. In this case, glue lines are not stressed by seismic cyclic forces.

  Note: When the suspended support configuration (see chapter 1.3) is used, the shear force transfer is made directly from the LVL top chord to the support (e.g. lower wall) with shear brackets or screws as for a standard construction.

- In the case where the LVL Rib Panel by Stora Enso stops at the face of the wall (CLT or GLVL for example) with a metal connector (steel angle, hanger, etc.)

Only connectors that have shown their ability to take up seismic forces (behavior adapted to oligo-cyclic fatigue) and covered by an ETA can be used.

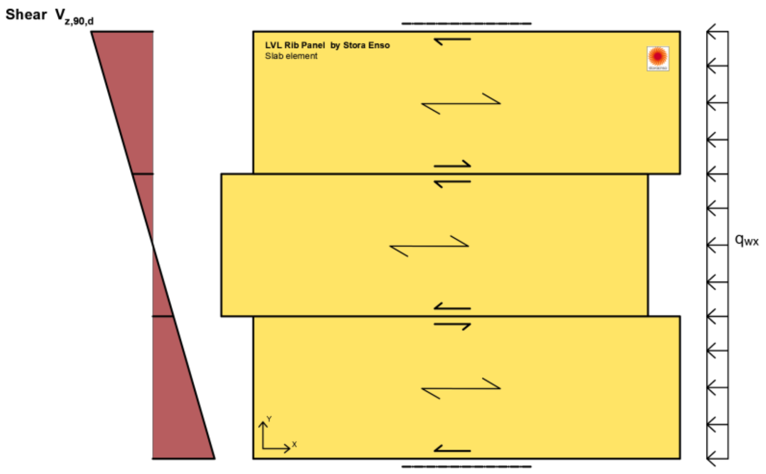
10.5.1 Horizontal forces at the longitudinal joints between the LVL rib panels

In addition to the load transfer in the vertical direction LVL rib panels are also able to carry loads in the horizontal (in-plane) directions. These actions result from e. g. wind and/or earthquake loadings on the global load-carrying structure and occur in two directions: Perpendicular and in the direction of the span (see Figure 37).The load transfer perpendicular to the span is often unproblematic. The first and the last LVL rib panel element transfer the load as a horizontal beam into the stabilizing (e.g. walls) structure below.

The load-carrying function parallel to the span requires some additional efforts in the design of the connection at the longitudinal edges of the elements. Due to the horizontal loading of the LVL rib panels, shear forces as well as compression and tensile forces in the longitudinal edges of the elements appear. A model for the determination of the relevant forces is derived in the subsequent sections of this chapter.

Finally, it should be mentioned that for the design of the fasteners and connection system the load-carrying capacity at the longitudinal edge of the elements, the vertical and horizontal forces from the different loadings must be combined.

a) Shear along the joints



Impacts in the direction of the longitudinal joints between the panels result in:

a) shear forces along the joints;

b) flange forces composed of tensile forces Z_d, and compression forces D_d caused by the bending perpendicular to grain of the diaphragm will occur at the same time at the intermediate joint edges.

The shear forces in the intermediate joints between the panels shall be taken by the respective fasteners designed accordingly.

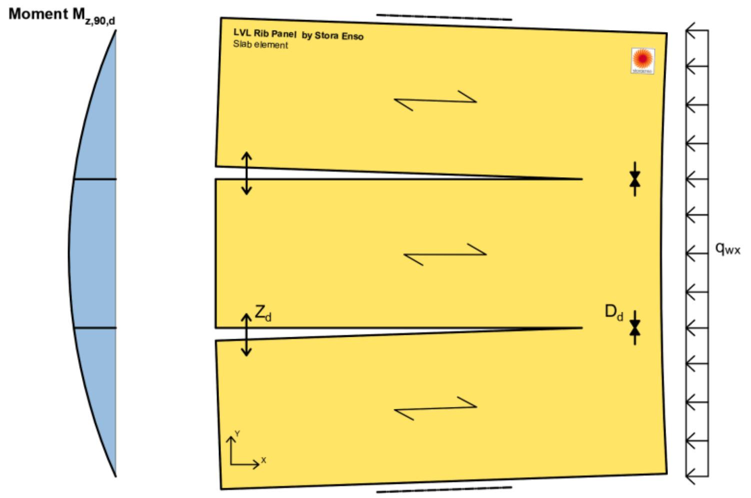
The tensile forces are taken jointly by:

• The connectors between floor panels

• Fixing systems also ensuring the floor / wall connection

• Other assemblies dedicated to carry these specific forces.

b) Flange forces (tension and compression) due to bending perpendicular to grain at the panel edges



c) Bending and shear due to bending parallel to grain as a horizontal girder

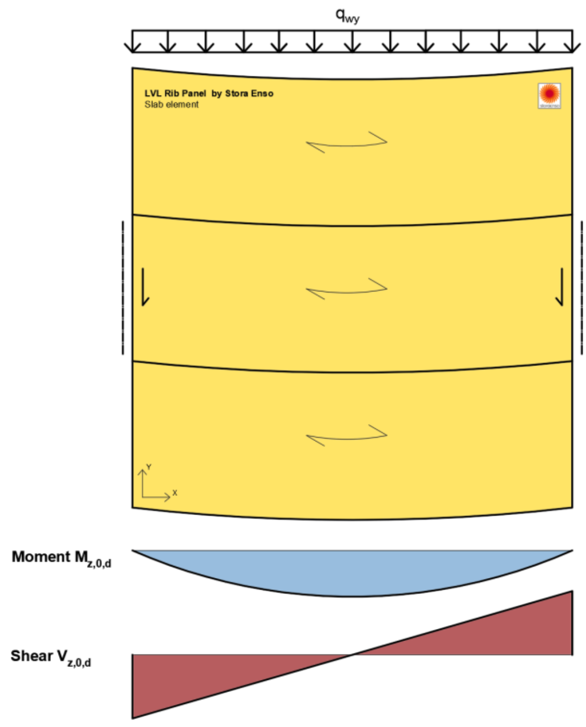


Figure 37: Failure mechanisms of diaphragms

Considering a beam with continuously distributed load:

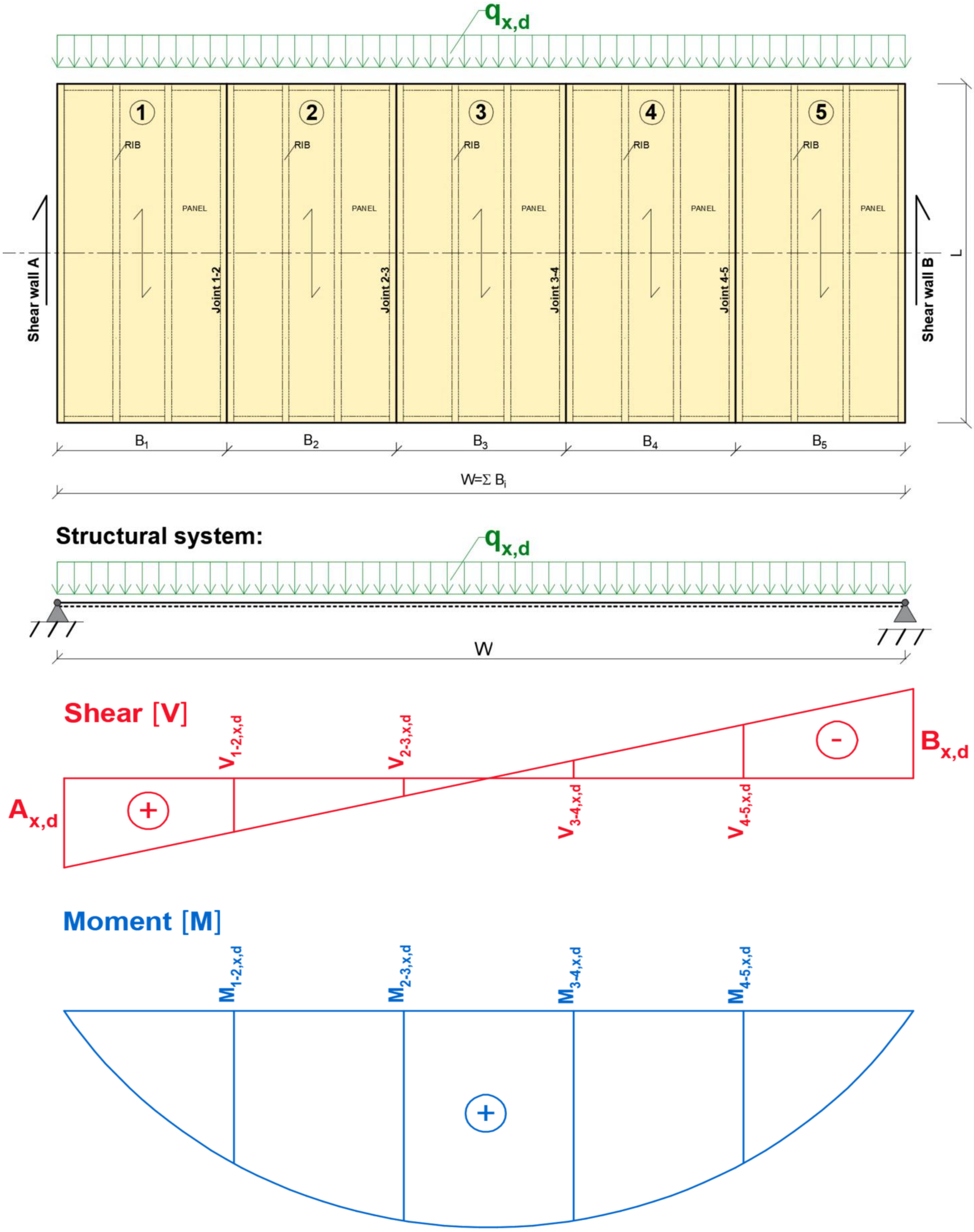


Figure 38: Modelling of LVL rib panels diaphragm in the direction of the span

Horizontal diaphragms are composed of several panels connected to each other along their length as presented in Figure 38. Shear forces are transferred by connectors through the longitudinal joints (1-2, 2-3, 3-4 and 4-5 for the example above) to the lateral shear walls (A and B for the example above). The shear value to considered at each joint is calculated by modelling the diaphragm as a horizontal girder oriented perpendicular to the horizontal forces.

The shear forces diagram of the girder can be determined and the reaction values at the supports correspond to the forces to be transferred to the lower shear walls.

10.5.2 Verification of the horizontal in-plane shear along the longitudinal joints

a) In-plane shear in the LVL-X plates:

The linear force in [N/mm] to be considered is:

$$n_{xy,d} = \frac{V_{x,d}}{L} \qquad \text{Eq 129}$$

$V_{x,d}$ Is the design shear force in the respective longitudinal joint [N]

L Is the length of the longitudinal joint (span) [mm]

The in-plane shear stress in the top/bottom chord shall be verified by considering the total LVL plate thickness

In top chord $$\tau_{(joint),d} = \frac{n_{xy,d}}{t_1}$$

In bottom chord $$\tau_{(joint),d} = \frac{n_{xy,d}}{t_3}$$

Eq 130

The following condition shall be fulfilled:

$$\tau_{joint,d} \leq f_{v,LVL-X,0,edge,d} \qquad \text{Eq 131}$$

b) Number of fasteners required to carry the shear force

$$n_{req,d} = \frac{F_{V,Ed}}{F_{V,Rd}} \qquad \text{Eq 132}$$

$$F_{V,Rd} = \frac{k_{mod} \cdot F_{V,Rk}}{\gamma_M} \qquad \text{Eq 133}$$

where :

$F_{V,Ed} = V_{x,d}/n_{plane}$ is the design shear force per shear plane to be transferred in the respective longitudinal joint [N];

$F_{V,Rk}$ is the characteristic load-carrying capacity per shear plane per fastener [N], according to Johansen equations in EN1995-1-1, equations (8.6) [34];

$F_{V,Rd}$ is the design load-carrying capacity per shear plane per fastener [N];

k_{mod} is the factor, according to EN1995-1-1, Table 3.1, taking a load duration (permanent – very short) and the service class (1-3) into consideration;

γ_M is the partial safety coefficient, applicable for dowel type connection design according to EN1995-1-1 [1].

The connection forces to the shear walls must be transferred with respective fasteners designed accordingly.

10.5.3 Determination of the compression zone and the limiting moment in a longitudinal joint of a diaphragm

The maximum moment of the longitudinal joint occurs when the maximum compression stress at the edge of the joint and the load carrying capacity of the outermost lying fastener is reached (Figure 39 and Figure 40).

Equilibrium

$$\sum F_x = 0 : \frac{1}{2} \cdot f_{c,90,edge,d} \cdot \sum t_{90} \cdot X - \sum_{i=1}^{n} F_i = 0$$ Eq 134

with

$\frac{1}{2} \cdot f_{c,90,edge,d} \cdot \sum t_{90} \cdot X \rightarrow$ Being the capacity of the compression force area in the LVL plate [N]

This compression will be carried by the total thickness of the LVL plate on its edge perpendicular to grain.

with

$$F_i = K_{ser} \cdot \varphi \cdot (X_i - X)$$ Eq 135

For the outermost lying fastener respectively:

$$F_n = K_{ser} \cdot \varphi \cdot (X_n - X) = F_d$$

$$K_{ser} \cdot \varphi = \frac{F_d}{(X_n - X)}$$ Eq 136

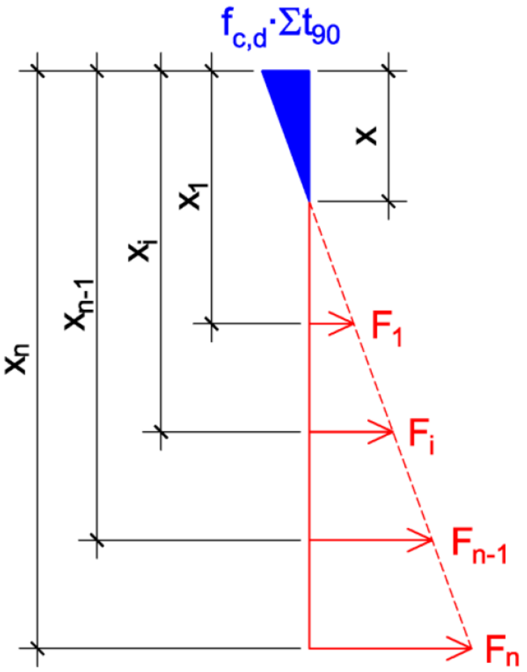


Figure 39: Compression zone perpendicular to grain (blue) at the edge of the joint and tension zone (red) with the load carrying capacity of the outermost lying fastener.

$F_n = F_d = R_d = n_{end} \cdot R_{d,i}$ is the total design carrying capacity of the outermost lying fasteners in [N] with n_{end} being the number of fasteners connecting the joint at the panel end.

φ is the angle of the strain plane

$$\varphi = \frac{F_d}{K_{ser} \cdot (X_n - X)}$$ Eq 137

$$\sum F_x = 0 :$$

$$\frac{1}{2} \cdot f_{c,90,edge,d} \cdot \sum t_{90} \cdot X - \sum_{i=1}^{n} K_{ser} \cdot \varphi \cdot (X_i - X) = 0$$

$$\frac{1}{2} \cdot f_{c,90,edge,d} \cdot \sum t_{90} \cdot X - \frac{F_d}{(X_n - X)} \cdot \sum_{i=1}^{n} (X_i - X) = 0$$ Eq 138

Quadratic equation for the distance X of the neutral axis from the edge loaded in compression

$$X^2 - \left(X_n + 2 \cdot \frac{n \cdot F_d}{f_{c,90,edge,d} \cdot \sum t_{90}}\right) \cdot X + 2 \cdot \frac{F_d}{f_{c,90,edge,d} \cdot \sum t_{90}} \cdot \sum_{i=1}^{n} X_i = 0$$

Distance X of the neutral axis from the edge loaded in compression

$$X_{1,2} = \left(\frac{X_n}{2} + \frac{n \cdot F_d}{f_{c,90,edge,d} \cdot \sum t_{90}}\right) \pm \sqrt{\left(\frac{X_n}{2} + \frac{n \cdot F_d}{f_{c,90,edge,d} \cdot \sum t_{90}}\right)^2 - 2 \cdot \frac{F_d}{f_{c,90,edge,d} \cdot \sum t_{90}} \cdot \sum_{i=1}^{n} X_i}$$ Eq 139

n is the number of fasteners along the longitudinal joint;
X_n is the distance of the outermost lying fastener from the panel extremity where the actions are applied [mm];
X_i is the distance of the i-th fastener [mm];
$f_{c,90,edge,d}$ is the design edgewise compressive strength perpendicular to grain of the LVL plate [N/mm²].
t_{90} Is the thickness of the LVL plate in compression perpendicular to grain [mm] ;
K_{ser} Is the slip modulus per shear plane per fastener [N/mm].

Since the “fastener springs” at the ends are weak, the neutral axis is close to the upper edge where the forces are applied. The distance X can also degenerate to X = 0 which is a conservative approach.

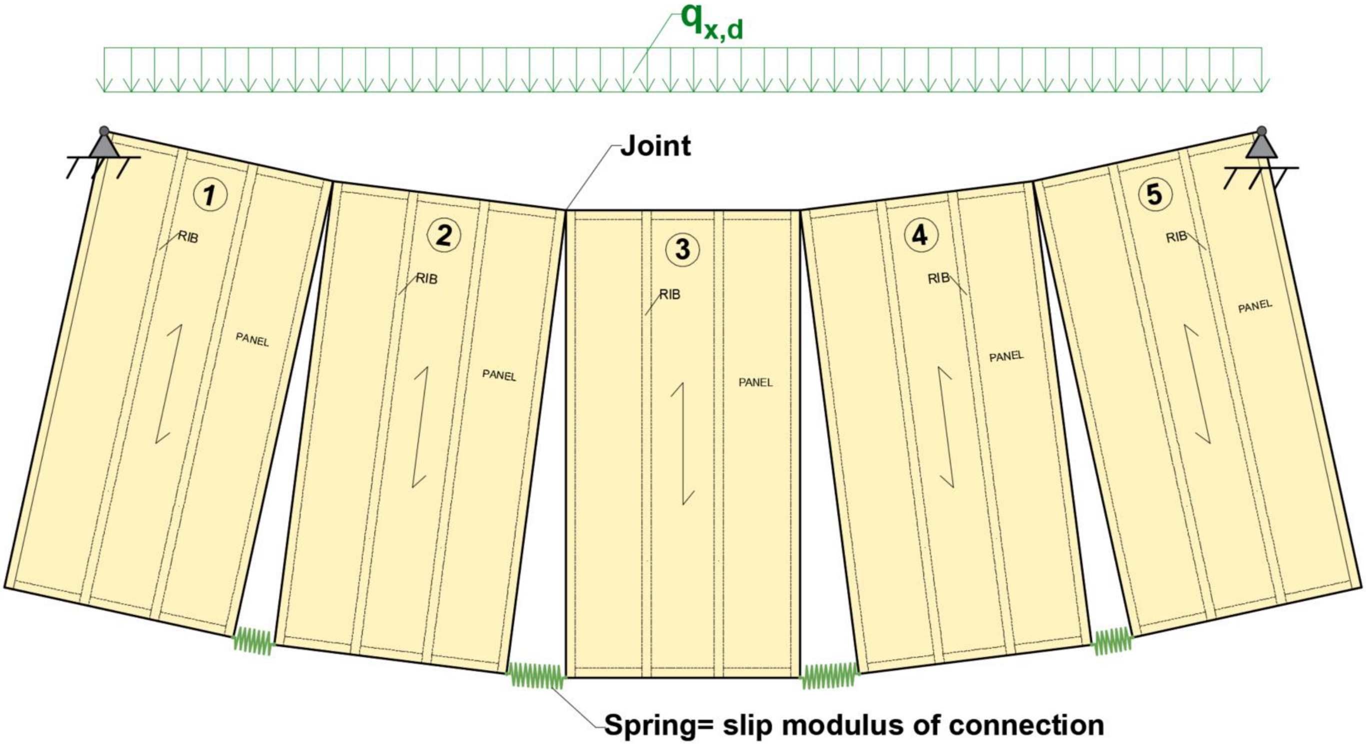


Figure 40: Diaphragm behaviour in longitudinal direction with springs in longitudinal joints at the end of the panels.

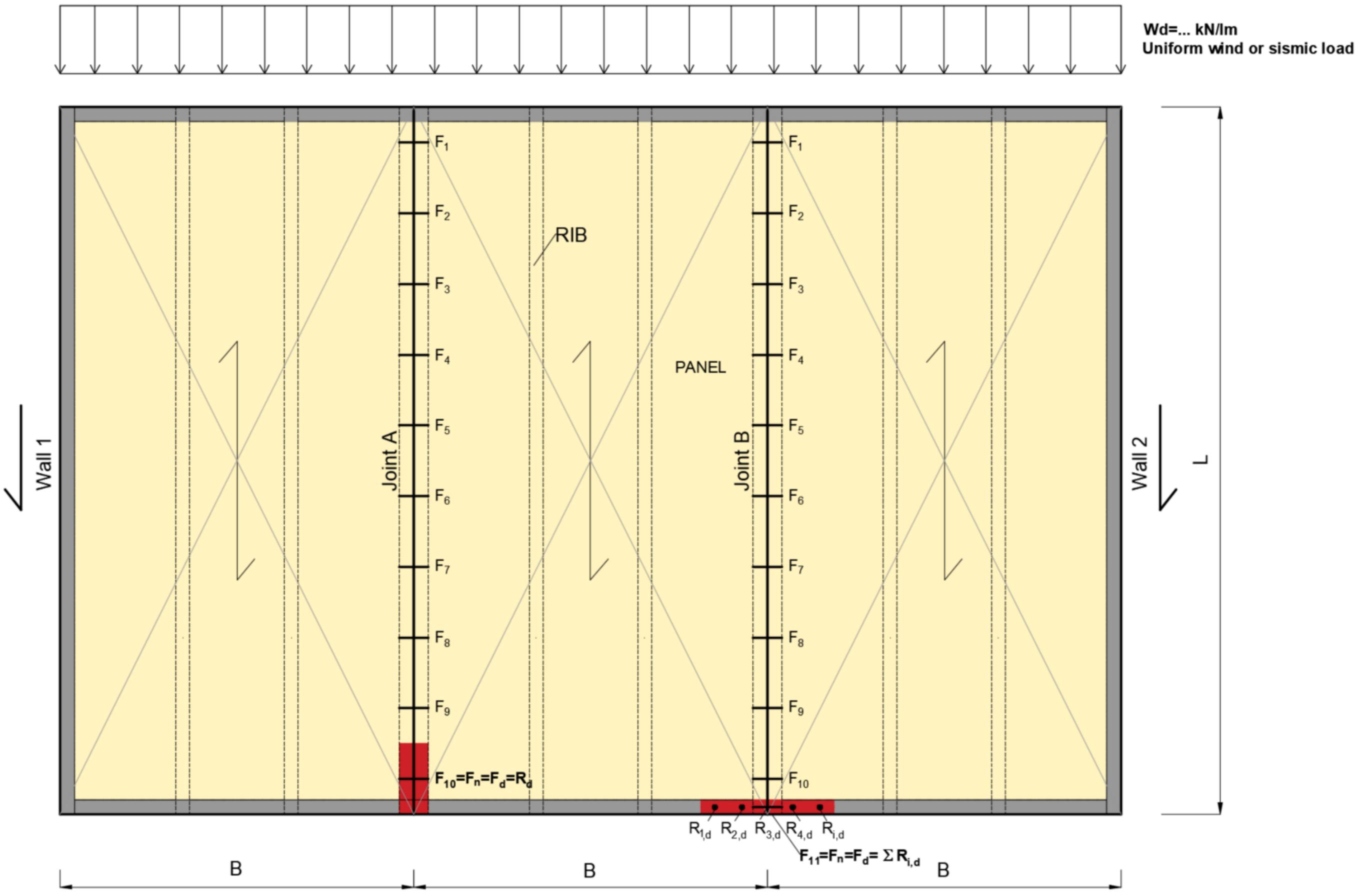


Figure 41: Fasteners placed in longitudinal joints and determination of the design carrying capacity of the outermost lying fastener Fd – Left (Joint A): Single fastener at the end; Right (Joint B):Multiple fasteners at the end .

Remark: We assume in this model that all joints have the same slip modulus. Meaning that the same fasteners should be used in the longitudinal joints, which is most often the case.

Limiting moment

$$\sum M = 0 : -\frac{1}{2} \cdot f_{c,90,edge,d} \cdot \sum t_{90} \cdot X \cdot \frac{X}{3} + K_{ser} \cdot \varphi \cdot \sum_{i=1}^{n} (X_i - X) \cdot X_i - M = 0$$

$$M_{limit} = \frac{F_d}{(X_n - X)} \cdot \left(\sum_{i=1}^{n} (X_i{}^2) - X \cdot \sum_{i=1}^{n} X_i \right) - \frac{X^2}{6} \cdot f_{c,90,edge,d} \cdot \sum t_{90}$$ *Eq 140*

Verification :

For each longitudinal joint present in the floor diaphragm shown in Figure 38, the respective design bending moment shall be compared with the limiting design moment M_{limit} calculated for the given joint:

$$M_{1,2,x} \leq M_{limit,1,2}$$
$$M_{2,3,x} \leq M_{limit,2,3}$$
$$M_{3,4,x} \leq M_{limit,3,4}$$
$$M_{4,5,x} \leq M_{limit,4,5}$$
$$M_{\dots,x} \leq M_{limit,\dots}$$

Eq 141

Resulting compression force

$$\rightarrow C = F_{c,d} = \frac{1}{2} \cdot f_{c,90,edge,d} \cdot \sum t_{90} \cdot X$$ Eq 142

Force in the i-th fastener

$$\rightarrow F_i = K_{ser} \cdot \varphi \cdot (X_i - X) = F_d \cdot \frac{(X_i - X)}{(X_n - X)}$$ Eq 143

It is a simple elastic model, not able to consider non-linear behavior which occurs. But a valid approach which allows the verification.

Summary:

- Start by defining the design capacity of the outmost lying fastener at the end of the panel, where most of the tension will occur.
- If there is a single fastener at the end connected in the joint (see Figure 41 -Joint A): we consider the total design capacity at the outermost lying fastener: $\boldsymbol{R_d = F_n = F_d}$
- If there are multiple fasteners at the end connected in the joint (see Figure 41 -Joint B) then we consider the total design capacity at the outermost lying fasteners : $\boldsymbol{R_d = n_{end} \cdot \sum R_{i,d} = F_n = F_d}$

➢ F_d is the maximum force that can be applied on the outermost connection in [N].

➢ $R_{i,d}$ is the design load-carrying capacity per shear plane per fastener [N], according to Johansen equations in EN1995-1-1, equations (8.6) . The tension is taken in shear by the connectors.

When knowing the value F_d and the K_{ser} of the connectors placed in the longitudinal joint, the forces F_i that will occur in each fastener depending on their distance from the extremity of the panel where the actions (like wind) are applied can be calculated.

The goal of the method presented is to calculate the maximum moment M_{limit} that can be applied to the floor diaphragm in longitudinal direction, based on the total design capacity at the outermost lying connection at the ends.

The edgewise compression perpendicular to grain of the LVL plate shall be verified as well if a compression zone is determined on the distance X previously introduced.

11. Verification at the supports

LVL Rib Panels can be supported in different ways depending on the structural detailing which influence the type of verification at the supports.
This chapter describes the design of LVL Rib Panel supports subjected to compression perpendicular to grain. Due to the stiffness of the ribs, all load applied to the entire rib panel (self-weight + additional surcharge) is directed to the supports through the load-bearing surface of the ribs (red arrows in Figure 7).
Any given linear load from wall resting on top of the rib panel will be directed through the blockings between the ribs in the support (blue arrows in Figure 7). In the case that a wall rests on top of the rib panel at the support line, blockings between the ribs are required to create a continuous transfer and distribute the compression uniformly.
A description of the role played by the end blockings is provided in chapter 1.4.

11.1 Design of simple supports with or without end blockings

11.1.1 Compression stresses perpendicular to grain at the end supports

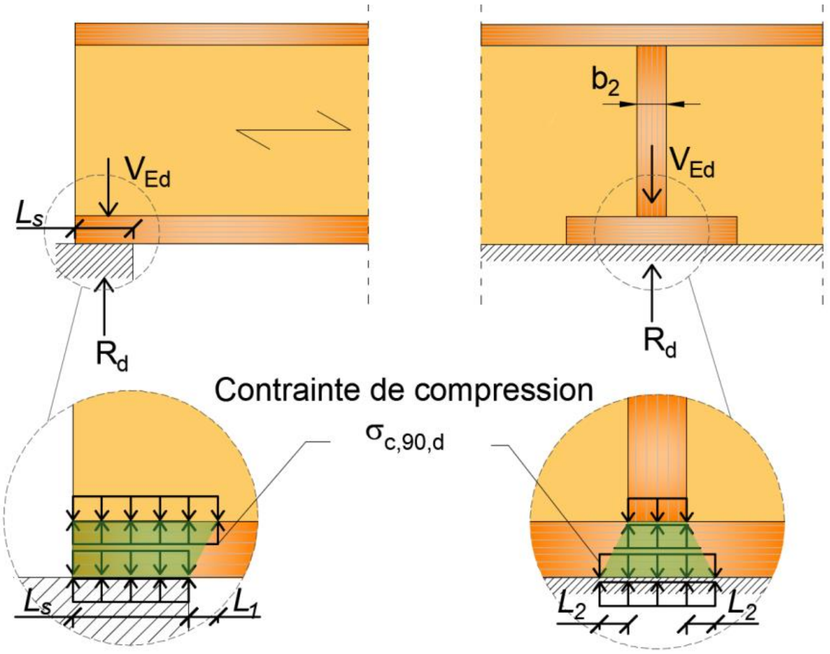


Figure 42: Effective contact area and stress distribution at a support.

The effective contact area perpendicular to the grain A_{ef} should be determined taking into account an effective contact length parallel to the grain, where the actual contact length L_s is increased at each side, and an effective contact width perpendicular to grain here the actual contact width b_2 is also increase at each side.

11.1.2 $k_{c,90}$ factor and contact length for LVL

Eurocode 5 does not include the parameters $k_{c,90}$ for LVL in different orientations of the material. The factors $k_{c,90}$ is as follows

Direction ***Compression perpendicular to the grain***		$k_{c,90}$	**Increase of the contact length [mm]**
Edgewise $f_{c,90,edge,k}$		1,0	$L_{1,edge} = 15mm$
Flatwise $f_{c,90,flat,k}$	Parallel to the grain of the surface veneers	1,4	$L_{1,flat} = 30mm$
	Perpendicular to the grain of the surface veneers		$L_2 = 15mm$

Increase of the contact length from one or two sides, but not more than “a” (distance between the element end and the support), Ls or *l/2* (distance between supports divided by two) according to EN 1995-1-1, part 6.1.5.
The increased contact length and factor $k_{c,90}$ are less favorable for LVL in edgewise direction, than in the flatwise direction or compared to other wood products have, due to the failure behavior of the products. LVL in the flatwise direction, solid wood and glulam have ductile behavior under compression perpendicular to the grain.

11.1.3 Effective compression area for LVL

The effective contact area may be calculated as:

Load bearing area on the rib:

$$A_{ef,rib} = b_2 \cdot (L_s + L_{1,edge})$$

Eq 144

Load bearing area on the bottom flange/chord:

$$A_{ef,flange/chord} = b_2 \cdot \left(L_s + L_{1,flat}\right) + 2 \cdot L_2 \cdot L_s$$

Eq 145

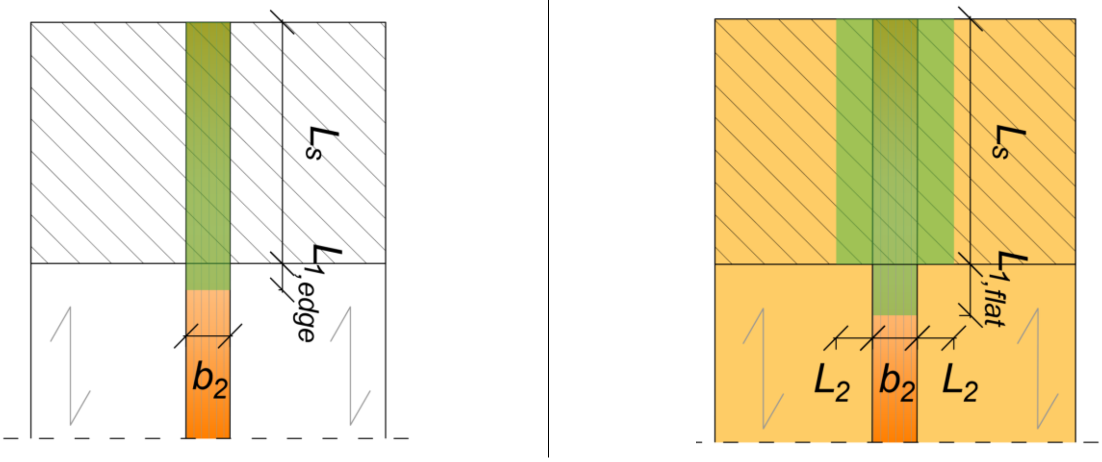


Figure 43: Load bearing area of the different part of the LVL Rib Panel section.

where

b_2 is the rib with [mm];
L_s is the actual contact length in compression according to EN 1995-1-1, part 6.1.5 [mm];
$L_{1,edge}$ is the increased contact length parallel to the grain on LVL edgewise [mm];
$L_{1,flat}$ is the increased contact length parallel to the grain on LVL flatwise [mm];
L_2 is the increased contact length perpendicular to the grain on LVL flatwise [mm].

The effective contact area at the support and the stress distribution is presented in Figure 42 and Figure 43.

11.1.4 Bearing pressure transferred through the ribs

11.1.4.1 Simply supported LVL rib panel

Open type

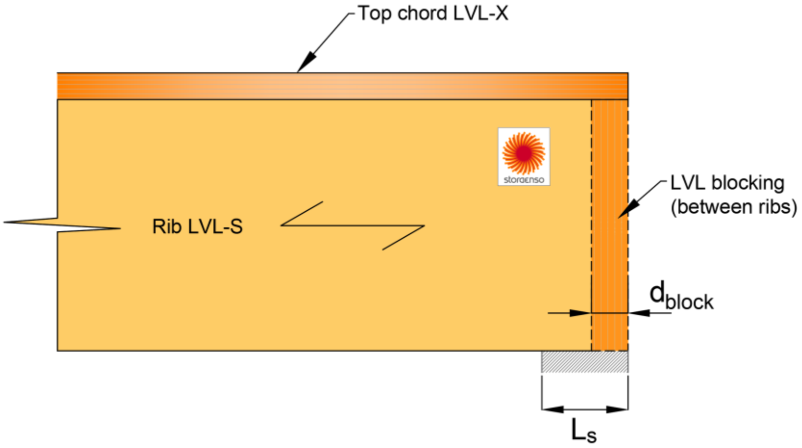


Figure 44: LVL Rib Panel simple support - Open

For the case of a LVL RP Open type, the following requirement shall be fulfilled for the rib:

Ribs:

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{c,90,LVL-S,edge,d}$$ *Eq 146*

with

$$\sigma_{c,90,d} = \frac{F_d}{A_{ef,rib}}$$ *Eq 147*

where

$\sigma_{c,90,d}$	design compressive stress perpendicular to the grain in the effective contact area [N/mm²];
$k_{c,90}$	factor considering the load configuration, the possibility of splitting and the degree of compressive deformation [-];
$f_{c,90,LVL-S,edge,d}$	design compressive strength perpendicular to the grain edgewise [N/mm²];
F_d	design support reaction per rib, originating from the rib panel itself and the surcharge applied to it [N]
$A_{ef,rib}$	effective contact area of the rib (see Figure 43) [mm²].

In these cases, there shall be a sufficient gap between the upper flange and blockings to prevent load bearing contact between the top chord and blocking due to the compression and/or shrinkage of the rib.

Semi-Open / Inverted / Closed type

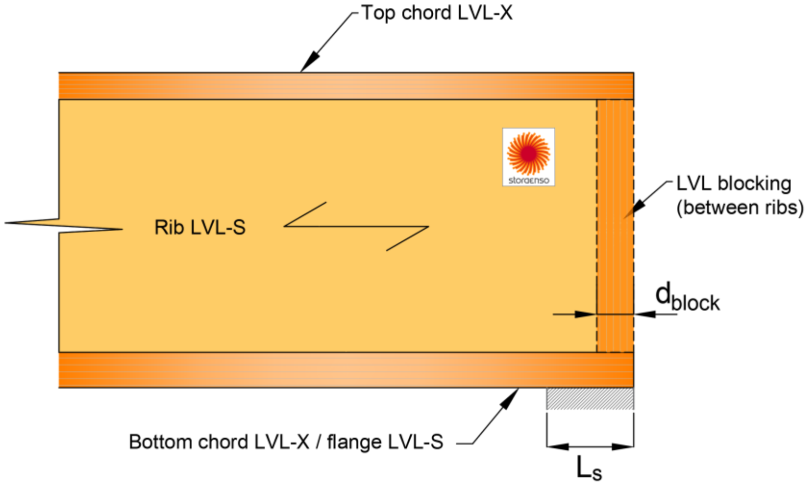


Figure 45: LVL Rib Panel simple support – Semi-Open/Closed/Inverted.

For the case of a LVL RP Semi-Open / Inverted / Closed, the following requirement shall be fulfilled <u>for the bottom chord/flange panel</u>:

Bottom flange (Semi-Open type):

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{c,90,LVL-S,flat,d} \quad \text{Eq 148}$$

Bottom chord (Closed type or Inverted type):

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{c,90,LVL-X,flat,d} \quad \text{Eq 149}$$

with

$$\sigma_{c,90,d} = \frac{F_d}{A_{ef,flange/chord}} \quad \text{Eq 150}$$

where

$\sigma_{c,90,d}$	design compressive stress perpendicular to the grain in the effective contact area [N/mm²];
$k_{c,90}$	factor considering the load configuration, the possibility of splitting and the degree of compressive deformation [-];
$f_{c,90,flat,d}$	design compressive strength perpendicular to the grain flatwise [N/mm²];
F_d	design support reaction per rib, originating from the rib panel itself and the surcharge applied to it [N];
$A_{ef,flange/chord}$	effective contact area of the flange/chord (see Figure 43) [mm²].

11.1.5 Bearing pressure transferred through the blockings

11.1.5.1 Simply supported LVL rib panel loaded from the top

Open type

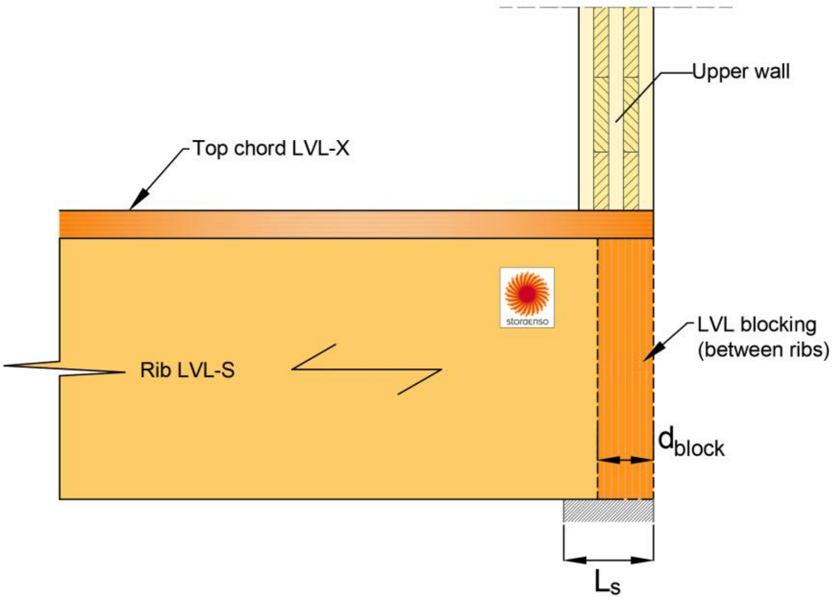


Figure 46: LVL Rib Panel simple support with wall above – Open.

For the case of a LVL RP with CLT Open under compression originating from the wall above (linear load), resting on top of the rib panel, aligned with the support, the following requirement shall be fulfilled for the blocking:

Blocking:

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{c,90,block,d} \quad \text{Eq 151}$$

with

$$\sigma_{c,90,d} = \frac{F_{wall,d}}{l_{block} \cdot min\,(L_s\,;d_{block})} \quad \text{Eq 152}$$

where

$\sigma_{c,90,d}$	design compressive stress perpendicular to the grain in the effective contact area of the blocking [N/mm²];
$k_{c,90}$	factor considering the load configuration, the possibility of splitting and the degree of compressive deformation [-];
$f_{c,90,block,d} = f_{c,90,LVL,edge,d}$	design compressive strength perpendicular to the grain edgewise of the blocking [N/mm²];
$F_{wall,d}$	Design support reaction per blocking, originating from the wall (linear load), resting on top of the rib panel, aligned with the support [N];
l_{block}	Length of the blocking [mm];
d_{block}	Thickness of the blocking [mm];
L_s	Depth of the support [mm].

In the presented case, the compressive stress perpendicular to grain applied on the LVL top chord should be verified as well:

Top chord:

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{c,90,LVL-X,flat,d}$$ *Eq 153*

with

$$\sigma_{c,90,d} = \frac{F_{wall,d}\ [N/mm]}{(t_{mur} + L_{1,flat})}$$ *Eq 154*

where

$F_{wall,d}$	Design support reaction per blocking, originating from the wall (linear load), resting on top of the rib panel, aligned with the support [N/mm];
t_{wall}	Upper wall thickness [mm];
$L_{1,flat}$	Increased contact length parallel to grain flatwise (see chapter 11.1.2) [mm].

Semi-Open / Inverted / Closed type

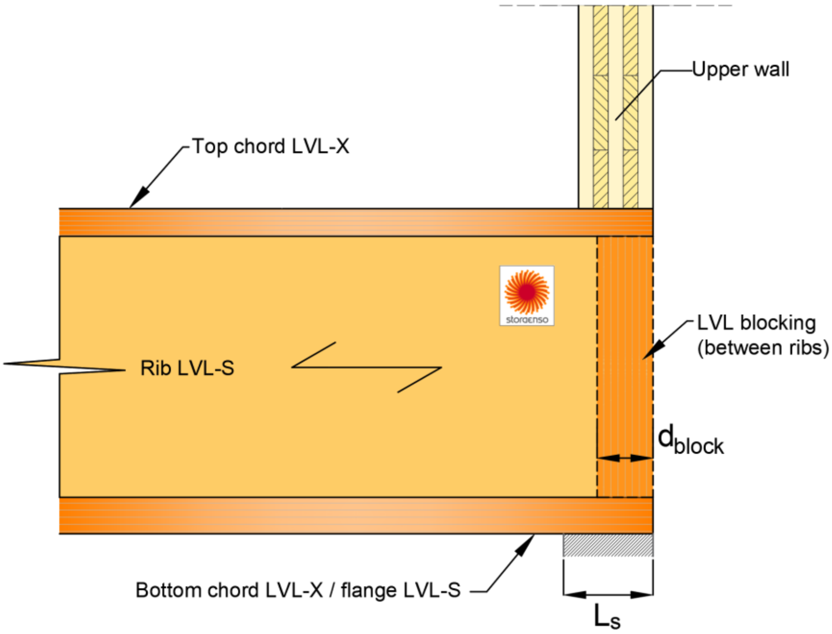


Figure 47: LVL Rib Panel simple support with wall above – Semi-Open/Closed/Inverted.

For the case of a LVL RP Semi-Open / Inverted / Closed under <u>compression originating from the wall above (linear load)</u>, resting on top of the rib panel, aligned with the support, the following requirement shall be fulfilled <u>for the LVL bottom flange/chord</u>:

Bottom flange (Semi-Open type):

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{c,90,LVL-S,flat,d}$$ *Eq 155*

with

$$\sigma_{c,90,d} = \frac{F_{wall,d}}{l_{block} \cdot [min\,(L_s\,; d_{block}) + L_{1,flat}]}$$ *Eq 156*

Bottom chord (Closed type or Inverted type):

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{c,90,LVL-X,flat,d}$$ *Eq 157*

with

$$\sigma_{c,90,d} = \frac{F_{wall,d}}{l_{block} \cdot [min\,(L_s\,; d_{block}) + L_{1,flat}]}$$ *Eq 158*

where

$\sigma_{c,90,d}$	design compressive stress perpendicular to the grain in the effective contact area of the bottom chord/flange [N/mm²];
$k_{c,90}$	factor considering the load configuration, the possibility of splitting and the degree of compressive deformation [-];
$f_{c,90,flat,d}$	design compressive strength perpendicular to the grain flatwise [N/mm²];
$F_{wall,d}$	Design support reaction per blocking, originating from the wall (linear load), resting on top of the rib panel, aligned with the support [N];
l_{block}	Length of the blocking [mm];
d_{block}	Thickness of the blocking [mm];
L_s	Depth of the support [mm];
$L_{1,flat}$	Increased contact length parallel to grain flatwise (see chapter 11.1.2) [mm].

In the presented case, the compressive stress perpendicular to grain applied on the blocking allowing the force transfer from the upper wall to the LVL flange/chord should be verified as well:

Blocking:

$$\sigma_{c,90,d} \leq k_{c,90} \cdot f_{c,90,block,d}$$ *Eq 159*

with

$$\sigma_{c,90,d} = \frac{F_{wall,d}}{l_{block} \cdot d_{block}}$$ *Eq 160*

where

$F_{wall,d}$	Design support reaction per blocking, originating from the wall (linear load), resting on top of the rib panel, aligned with the support [N];
l_{block}	Length of the blocking [mm];
d_{block}	Thickness of the blocking [mm].

In the presented case, the compressive stress perpendicular to grain applied on the LVL top chord should be verified as well:

- The verification presented above for the Open type LVL rib panel applies.

11.2 Supports with screwed connections

11.2.1 Capacity of axially loaded screws in tension

In general, the characteristic capacity $R_{T,Rk}$ of one screw connecting two members is

$$R_{T,Rk} = min\begin{cases} f_{ax,1,k} \cdot \left(\frac{8d}{l_{g,1}}\right)^{0.2} \cdot d \cdot l_{g,1} + f_{head,k} \cdot d_h^2 \\ f_{ax,2,k} \cdot \left(\frac{8d}{l_{g,2}}\right)^{0.2} d \cdot l_{g,2} \\ f_{tens,k} \end{cases} \quad \text{Eq 161}$$

In the first equation the angle between the screw axis and the grain direction shall be 60° ≥ α ≥ 30°. Otherwise the two parts cannot be added.

where

d	is the outer diameter of thread;
$f_{ax,1,k}$	is the characteristic withdrawal strength parameter in head-side member 1;
$l_{g,1}$	is the penetration length of the threaded part in head-side member 1
d_h	is the diameter of screw head (mm)
$f_{head,k}$	is the characteristic pull-through parameter of the screw in head-side member 1
$f_{ax,2,k}$	is the characteristic withdrawal strength parameter in point side member 2
$l_{g,2}$	is the penetration length of the threaded part in point side part 2
$f_{tens,k}$	is the characteristic tensile capacity of the screw.

The characteristic withdrawal strength parameters at angles $\alpha = 45°$ and $\alpha = 90°$ and pull-through parameter are given for tested Spax screws in Table 17. The values are based on tests reports (report VTT-S-04662-16 (diameter 8mm screw) [6] , VTT-S-05379-17 (diameter 5 and 6mm screws) [7].)

Tested screws:

- SPAX WIROX T-Star plus 8x140, flat countersunk head, partial thread, 4CUT point
- SPAX WIROX T-Star plus 8x200, cylindrical head, full thread, 4CUT point
- SPAX T-Star plus 6x120, flat countersunk head, partial thread, 4CUT point
- SPAX WIROX T-Star plus 6x160 cylindrical head, full thread, 4CUT point
- SPAX T-Star plus 5x100, flat countersunk head, partial thread, 4CUT

The withdrawal strength parameters are valid only if penetration length l_g ≥ 70 mm and if the minimum spacing, the end and edge distances are fulfilled (Table 18).

Table 17: Strength characteristic values and withdrawal strength parameter $f_{ax,\alpha,k}$ for threaded part of Spax T-Star screws. (ANALYSIS REPORT No VTT-S-04662-16 [6] , VTT-S-05379-17 [7])

Screw	LVL grade	$f_{ax,\alpha,\beta,k}$ [N/mm²]				$f_{head,k}$ [N/mm²]
		$\alpha = 45°$ $\beta = 0°$	$\alpha = 45°$ $\beta = 45°$	$\alpha = 90°$ $\beta = 0°$	$\alpha = 90°$ $\beta = 45°$	$\alpha = 90°$ $\beta = 0°$
Spax T-STAR plus d = 8 mm	X	-	17.1	11.5	17.9	18.1
	S	12.6	-	-	-	-
Spax T-STAR plus d = 6 mm	X	-	17.0	11.3	18.7	24
	S	14.8	-	-	-	-
Spax T-STAR plus d = 5 mm	X	-	17.4	13.7	20.5	26.7
	S	19.1	-	-	-	-

where

$f_{ax,\alpha,\beta,k}$ is the characteristic withdrawal strength parameter, which is determined at an angle of α and β.;

α denotes the angle between the screw axis and the grain direction (see Figure 48);

β is independent from the grain direction, it denotes the angle between the screw axis and the plane parallel to the flat face of LVL (veneer plane) (see Figure 48).

Partial factor for screw is
$\gamma_{M2} = 1.25$ according to EN1993-1-8

Partial factor for timber is
$\gamma_M = 1.30$ according to EN1995-1-1

The design withdrawal capacity may be calculated as

$$R_{T,Rd} = min \begin{cases} \frac{k_{mod}}{\gamma_M} \cdot \left(f_{ax,1,k} \cdot \left(\frac{8d}{l_{g,1}}\right)^{0.2} \cdot d \cdot l_{g,1} + f_{head,k} \cdot d_h^2 \right) \\ \frac{k_{mod}}{\gamma_M} \cdot \left(f_{ax,2,k} \cdot \left(\frac{8d}{l_{g,2}}\right)^{0.2} \cdot d \cdot l_{g,2} \right) \\ \frac{f_{tens,k}}{\gamma_{M2}} \end{cases} \quad \text{Eq 162}$$

Where $f_{tens,k} = 17kN$ for all SPAX T-Star plus $d = 8mm$ screws
$f_{tens,k} = 11kN$ for all SPAX T-Star plus $d = 6mm$ screws
$f_{tens,k} = 7.9kN$ for all SPAX T-Star plus $d = 5mm$ screws

According to ETA-12/0114 from Spax manufacturer. [8]

The design capacity of tension screwed connection may be calculated as

$$R_d = n^{0.9} \cdot R_{T,d} \quad \text{Eq 163}$$

n is the number of screws

11.2.2 Inclined screw connection

The design capacity of tension screw connection may be calculated as

$$R_d = n^{0.9} \cdot R_{T,d} \cdot (\cos\alpha + \mu \sin\alpha)$$ *Eq 164*

α is the angle between screw axis and the grain direction of the outer veneers of LVL, see Figure 48.

In connection of rib and end beam or parts of the end beam **friction factor *μ* may be 0.26**

The design equation for each connection between the parts should be respected:

$$V_{E,d} \le R_d$$ *Eq 165*

In this equation head side member 1 and point side member 2 are chosen accordingly for each connection plane. V_{Ed} is the shear force of the corresponding I or U section of the rib panel.

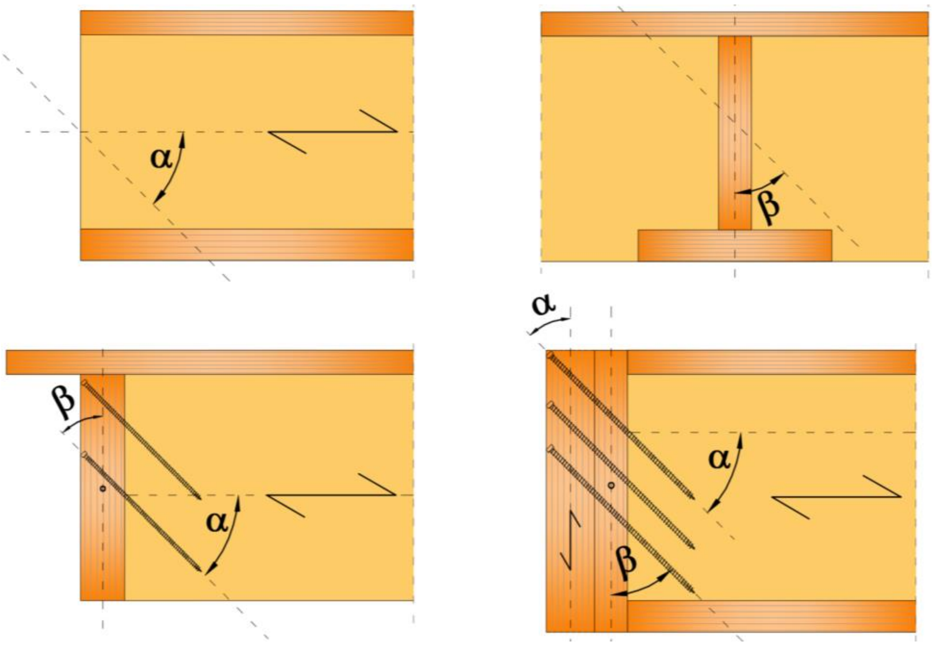


Figure 48: Screw angle α angle between screw axis and the grain direction and β between the screw axis and the plane parallel to the flat face of LVL.

11.3 End beam support

In end beam support the rib panel or the box panel is supported by end beam.
The rib panel is supported by an end beam, see Figure 50. In the outer part of the end beam grains are vertical and this part bears the support force. The inner part has horizontal grain direction and this part stiffens the outer part. The longer main screws also connect the end beam to the ribs. There may still be additional parts connected to the end beam but these are not part of the rib panel.

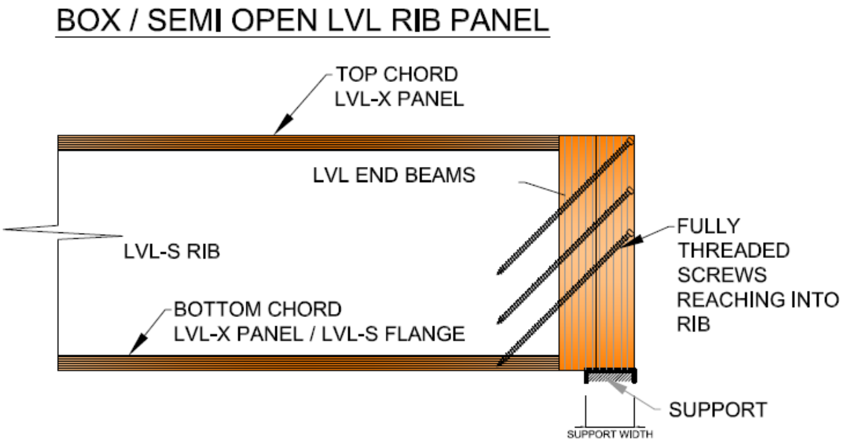


Figure 49 : Example with end beam support

Figure 49 is giving a possible solution for a rib panel. The rib panel is supported off the LVL-S rib. The rib or box panel shall be hung off the end beam by the use of fully threaded screws, in an angle of 45°, so the screws are in tension. Embedment in the rib is mandatory, additional embedment in the LVL is optional. The fully threaded screw can be inserted from the side of the rib panel or from the side of the end beam. The embedment at both ends needs to be sufficient for the given purpose and needs to be designed accordingly.

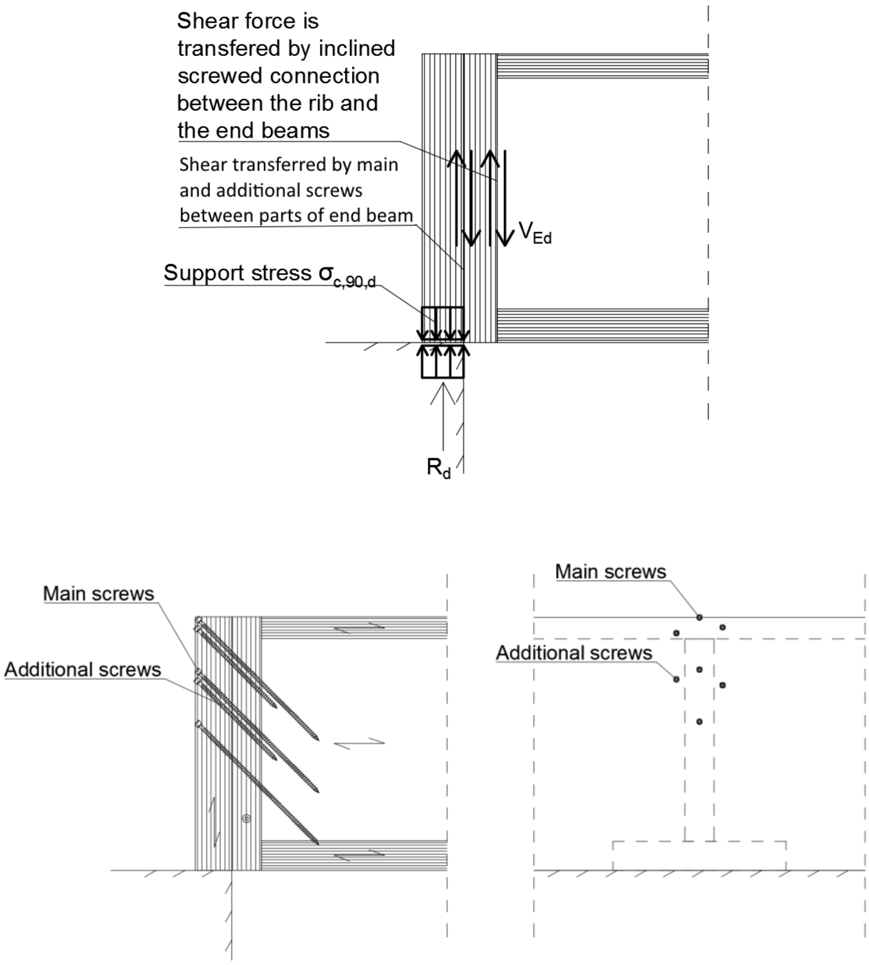


Figure 50: End beam support. Support reaction is transferred to shear force in ribs through end beam. ($V_{ed} = R_d$)

11.3.1 Screw connection capacity

When the end beam consists of multiple parts and only outer part beam is on the support, inclined screwed connection should be calculated as follows:
The additional screws are usually needed to transfer the shear force between the parts (). If the capacity of main screw on the outer beam is enough to transfer the shear load between the parts, additional screws are not needed. In Figure 51 is shown numbering of the sections of the connection. The additional screw type should be the same as the main screws and the screwing angle of additional screws should be same as for the main screws.

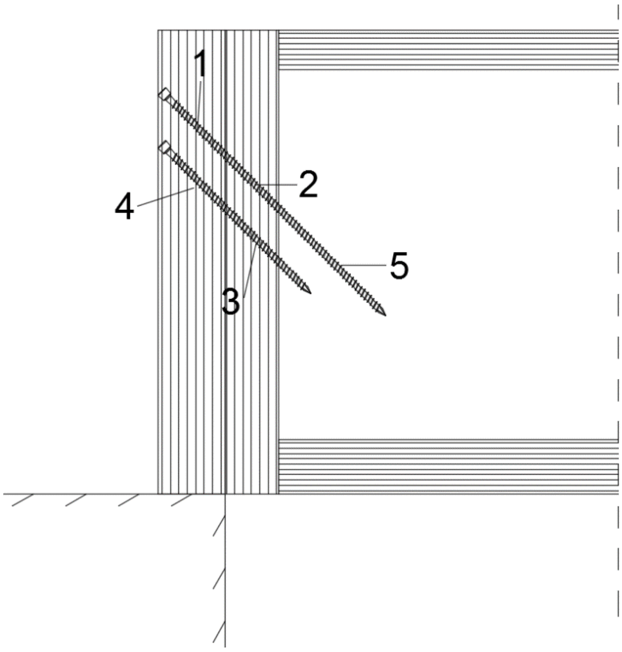


Figure 51: Numbering of the sections for connection.

The design withdrawal capacity of main screw in outer part of the end beam may be calculated as

$$R_{T,Rd,1} = min \begin{cases} \frac{k_{mod}}{\gamma_M} \cdot \left(f_{ax,1,k} \cdot \left(\frac{8d}{l_{g,1}} \right)^{0.2} \cdot d \cdot l_{g,1} + f_{head,k} \cdot d_h^2 \right) \\ \frac{f_{tens,k}}{\gamma_{M2}} \end{cases}$$ Eq 166

The design withdrawal capacity of main screw in inner part of the end beam may be calculated as

$$R_{T,Rd,2} = min \begin{cases} \frac{k_{mod}}{\gamma_M} \cdot \left(f_{ax,2,k} \cdot \left(\frac{8d}{l_{g,2}} \right)^{0.2} \cdot d \cdot l_{g,2} \right) \\ \frac{f_{tens,k}}{\gamma_{M2}} \end{cases}$$ Eq 167

The design withdrawal capacity of main screw in the rib may be calculated as

$$R_{T,Rd,5} = min \begin{cases} \frac{k_{mod}}{\gamma_M} \cdot \left(f_{ax,5,k} \cdot \left(\frac{8d}{l_{g,5}} \right)^{0.2} \cdot d \cdot l_{g,5} \right) \\ \frac{f_{tens,k}}{\gamma_{M2}} \end{cases}$$ Eq 168

The design withdrawal capacity of additional screw in outer part of the end beam is

$$R_{T,Rd,add,4} = R_{T,Rd,1} \quad \text{Eq 169}$$

The design withdrawal capacity of additional screw in inner part of the end beam is

$$R_{T,Rd,add,3} = R_{T,Rd,2} \quad \text{Eq 170}$$

The design capacity of main screws in the rib may be calculated as

$$R_{d,5} = n_{main}^{0.9} \cdot R_{T,Rd,5} \cdot (\cos\alpha + \mu \sin\alpha) \quad \text{Eq 171}$$

where n_{main} is the number of main screws

The design capacity of main screws in end beam may be calculated as

$$R_{d,12} = n_{main}^{0.9} \cdot (R_{T,Rd,1} + R_{T,Rd,2}) \cdot (\cos\alpha + \mu \sin\alpha) \quad \text{Eq 172}$$

The design capacity of all screws in outer part of the end beam may be calculated as

$$R_{d,14} = (n_{main} + n_{add})^{0.9} \cdot R_{T,Rd,1} \cdot (\cos\alpha + \mu \sin\alpha) \quad \text{Eq 173}$$

where n_{add} is the number of additional screws

The design capacity of all screws in inner part of the end beam may be calculated as

$$R_{d,23} = (n_{main} + n_{add})^{0.9} \cdot R_{T,Rd,2} \cdot (\cos\alpha + \mu \sin\alpha) \quad \text{Eq 174}$$

In the connection between the end beam and the rib shall be

$$V_{E,d} \leq R_d = min\begin{cases} R_{d,5} \\ R_{d,12} \end{cases} \quad \text{Eq 175}$$

where $V_{Ed}$is the shear force in the rib.

In the connection between the end beams shall be

$$V_{E,d} \leq R_d = min\begin{cases} R_{d,14} \\ R_{d,23} \end{cases} \quad \text{Eq 176}$$

11.3.2 Spacing, edge and end distance for screws

In Table 18 and is shown minimum spacing, end and edge distances to respect for the connections.
Only flatwise connections (perpendicular to the LVL surface) can be verified with the distances given in EN1995-1-1.
Minimum spacing and distances applicable to LVL material are presented in the LVL Handbook Europe [38] and in the ETA of the screw manufacturer.

The maximum distance from an additional screw to the nearest main screw should not be greater than 10d.

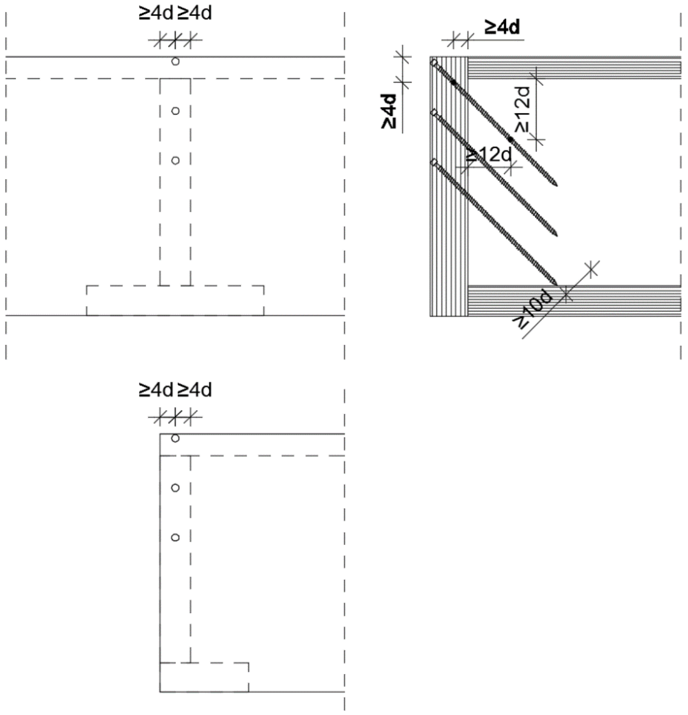


Figure 52: Minimum spacing, end and edge distances. The edge distance to the end of support beam should not be less than 5d.

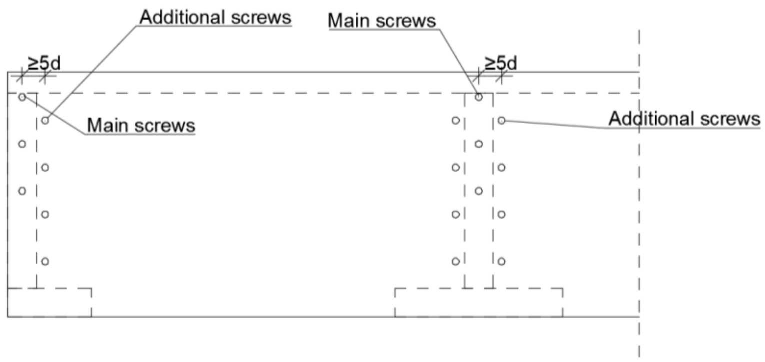


Figure 53: Screw instruction for additional screws.

Edgewise spacings and distances have been verified by tests for Spax T-Star plus screw.

Table 18: Minimum spacing and end and edge distances(d_{screws}=8mm/6mm/5mm)

Diameter	Minimum screw spacing in a plane parallel to the grain	Minimum screw spacing perpendicular to a plane parallel to the grain	Minimum end distance of the center of gravity of the threaded part of the screw in the member	Minimum edge distance of the center of gravity of the threaded part of the screw in the member	Minimum standard LVL-S rib width
Vis	**a_1**	**a_2**	**a_3**	**a_4**	**b**
Spax T-STAR plus	10d	5d	12d	4d	
8mm	80mm	40mm	96mm	32mm	69mm
6mm	60mm	30mm	72mm	24mm	51mm
5mm	50mm	25mm	60mm	20mm	45mm

11.3.3 Shear design in end of rib

If screws do not reach the bottom of the rib connection, it shall be calculated as a beam with a notch at the support.

Shear stress shall satisfy the following expression

$$\tau_d = \frac{3}{2} \cdot \frac{V_{Ed}}{b_2 h_{ef}} \leq k_v \cdot f_{v,d} \quad \text{Eq 177}$$

where

$$\alpha = \frac{h_{ef}}{h_2}$$

$$k_n = 4.5 \quad \text{Eq 178}$$

$$k_v = min \begin{cases} 1 \\ \dfrac{k_n}{\sqrt{h_2}\left(\sqrt{\alpha(1-\alpha)} + 0.8\dfrac{x}{h_2}\sqrt{\dfrac{1}{\alpha} - \alpha^2}\right)} \end{cases}$$

where
b_2 is the width of the rib [mm];
h_2 is the height of the rib [mm];
h_{ef} is distance from the lowest point of screw to the upper edge of rib[mm];
$f_{v,d}$ is the design shear strength edgewise [N/mm²];
x is the distance from point of the screw to the end of rib [mm].

In Figure 54 is defined distances x and h_{ef}.

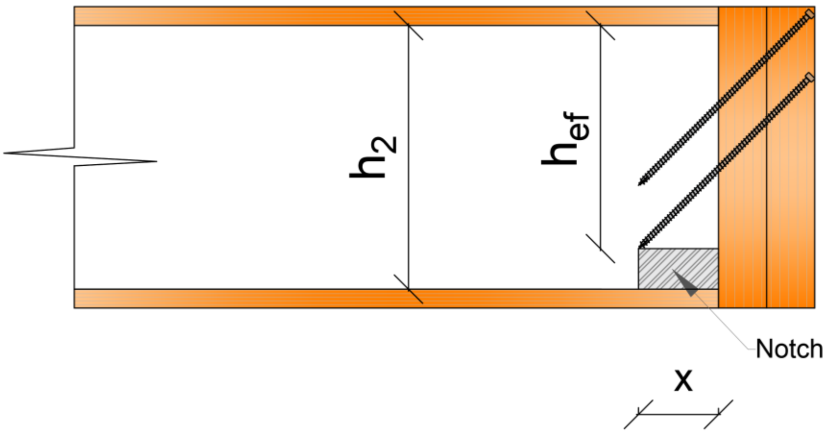


Figure 54: Determination of distances x and h_{ef}.

10.4 Suspended support

In a suspension support the rib panel is suspended by the slab. The slab is connected to the end beam (horizontal grains) with the structural screws. The end beam is connected to the ribs with diagonal structural screws as in the end beam support.

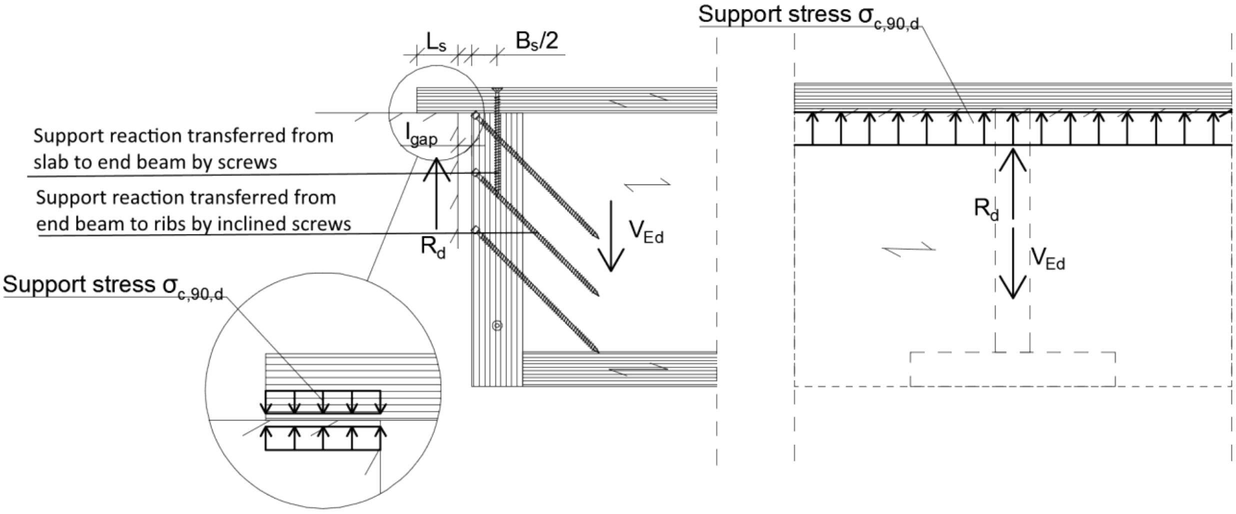


Figure 55: Suspension support. Support reaction is transferred to shear force in ribs through end beam. ($V_{ed} = R_d$)

In Figure 56 is shown suspended support condition.

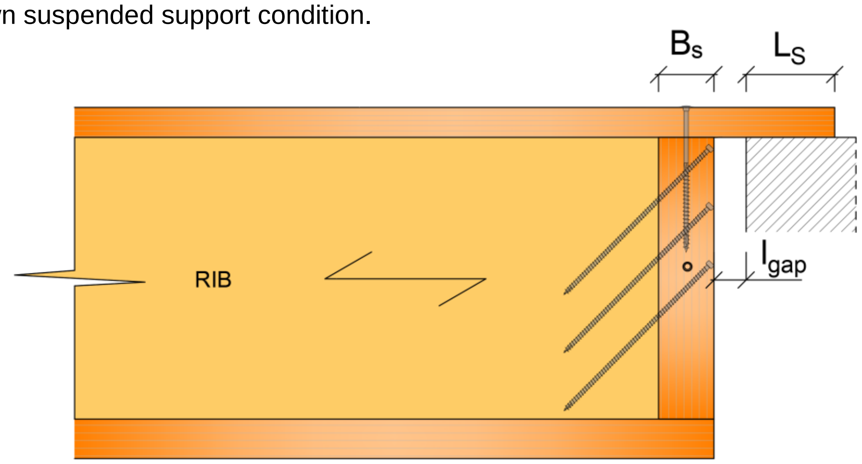


Figure 56: Suspension support geometry parameters.

The design of inclined screws is carried out in the same way as presented for the end beam support in chapter 11.3. The verification of the compression perpendicular to grain at the support shall be done as presented in chapter 11.1.

11.3.4 Bending resistance of the top chord slab

The bending stress should satisfy the following expression

$$\sigma_d \leq f_{(LVL-X),m,0,flat,d} \quad \text{Eq 179}$$

$f_{(LVL-X),m,0,flat,d}$ is the design bending strength of the LVL-X slab

Bending moment may be calculated as

$$M_{Ed,slab} = V_{Ed}\left(\frac{L_s}{2} + l_{gap} + \frac{B_s}{2}\right) \quad \text{Eq 180}$$

V_{Ed} is the shear force of the whole open box slab;

L_s is length of support;
l_{gap} is length of gap between end beam and support (the value should be the nominal value added with allowed tolerance);
B_s is width of the end beam.

Bending resistance may be calculated as

$$W = \frac{b_1 t_1^2}{6}$$ *Eq 181*

where b_1 and t_1 are the width and thickness of the slab.

Bending stress may be calculated as

$$\sigma_{m,d} = \frac{M_{Ed,slab}}{W}$$ *Eq 182*

11.3.5 Shear resistance of slab

The shear stress should satisfy the following expression

$$\tau_d \leq f_{(LVL-X)v,0,flat,d}$$ *Eq 183*

An improved shear strength was determined by testing when the top chord is structurally glued to the end beam. The following shear strength shall be taken:

$f_{(LVL-X)v,0,flat,k} = 2{,}3 \frac{N}{mm^2}$ for LVL-X 24-33mm

$f_{v,0,flat,d}$ is the design shear strength (according to VTT-S-00252-18 report [9])

Shear stress may be calculated as

$$\tau_d = \frac{3}{2} \frac{V_{Ed}}{b_1 t_1}$$ *Eq 184*

11.3.6 Axially loaded vertical screws

Design of screw connection is done according to previous chapter.
The screws are assumed to be connected to the rib closest to them and the design capacity of tension screwed connection may be calculated as

$$R_d = n^{0.9} R_{T,d}$$ *Eq 185*

n is the number of screws for rib.

The following design equation shall be fulfilled

$$V_{E,d} \leq R_d$$ *Eq 186*

Here V_{Ed} is the shear force of the rib considered.

11.3.7 Spacing, edge and edge distances for screws

In Table 18 and Figure 57 is shown minimum spacing, end and edge distances.

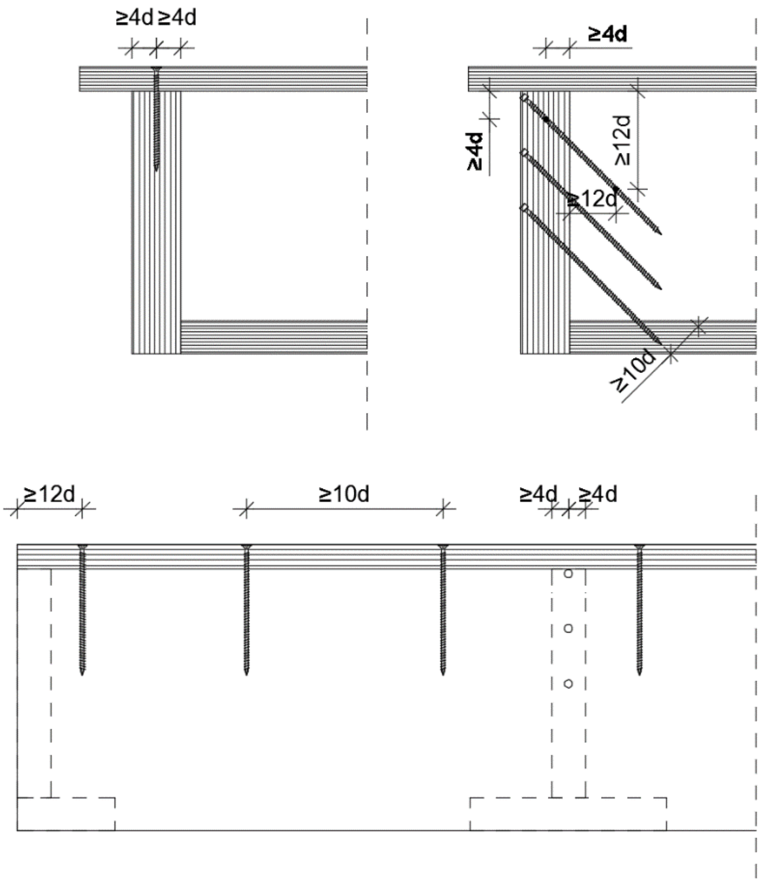
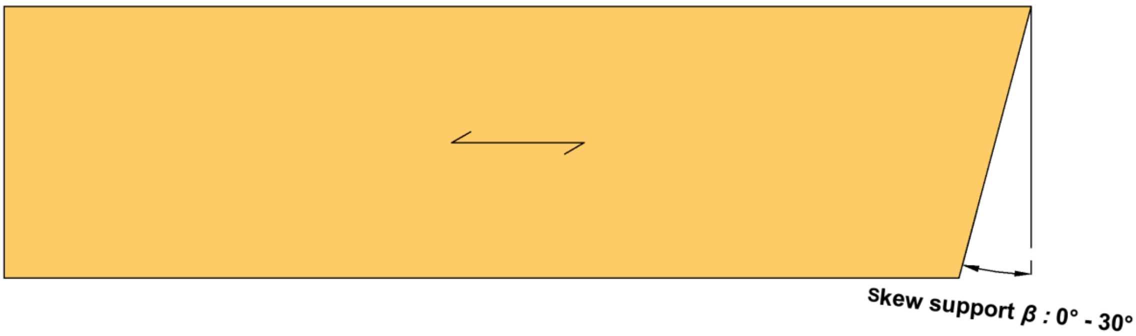


Figure 57: Minimum spacing, end and edge distances.

12. LVL Rib Panel with skew support



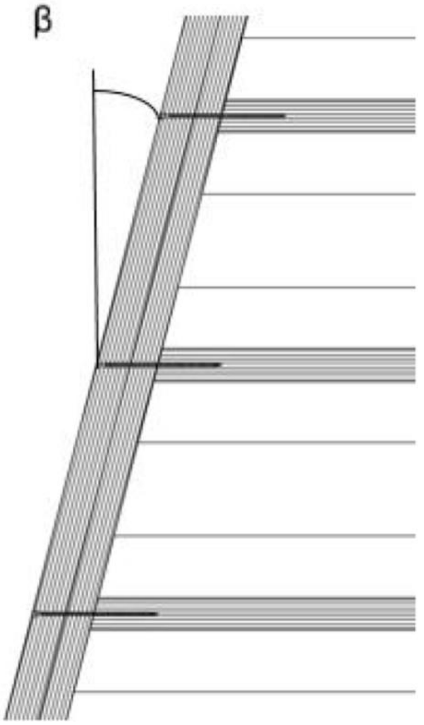


Figure 58: Skew support of open type LVL Rib Panel

When having a skew support, screws have to be screwed parallel to the ribs direction and designed according to chapter 11.2 .
In end beam parts, the values for $f_{ax,k}$ should be multiplied by the factor:

$$1 - \frac{\beta}{90}\left(1 - \frac{10\ N/mm^2}{f_{ax,k}}\right) \qquad \text{Eq 187}$$

Where β is the angle of skewness (0°-30°)
Angle between load and grain direction of the slab should considered material properties of the slab.
These properties are limited for a 63 mm slab thickness and utilization rates are limited to 80 % because of uncertainties of the great angle of the skew support.

Bending strength of the LVL-X slab with angle from 15° to 30˚:
$f_{m,\alpha,k} = 17.8\text{MPa}$
Shear strength of the LVL-X slab with angle from 15° to 30˚:
$f_{v,\alpha,k} = 0.76\text{MPa}$

In and are shown different slab layouts. **Suspended support type is not possible in skew supports**.
Rib slabs with skew supports can be designed as slabs with non-skew supports by considering each I- and U section separately (With different spans) due to the angle of skew support β which is limited to 30°.

LVL Rib Panels by Stora Enso

Analysis sample

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1. Introduction

The LVL rib panels, as they are laid out in the related ETA are being created, by gluing LVL-S ribs to a LVL-X panel. This connection between ribs and the is assumed to be rigid. Therefore, the proposed design method is based on the rigid composite theory.

2. Analysis example

2.1 Geometric data and loadings

Dimensions of rib panel are shown in Figure 59. Slab is intermediate floor in residential building.
The LVL rib panel in the following calculation example is built up by means of a **LVL-S-beam with the dimensions 51 x 350 mm, a LVL-X top panel of 37mm thick** and a **LVL-S bottom flange of 49 mm x 300mm for internal ribs** and **49 mm x 150mm for external ribs**. (After sanding)

The structural system is a single-span beam with a span ***l* = 7.12 m** and **width is 2.5 m**. The spacing distance between the ribs amounts **s = 625 mm**.

For the LVL-S and LVL-X elements, the material values according to VTT-S-05710-17 [2]and VTT-S-05550-17 [1] reports were used.

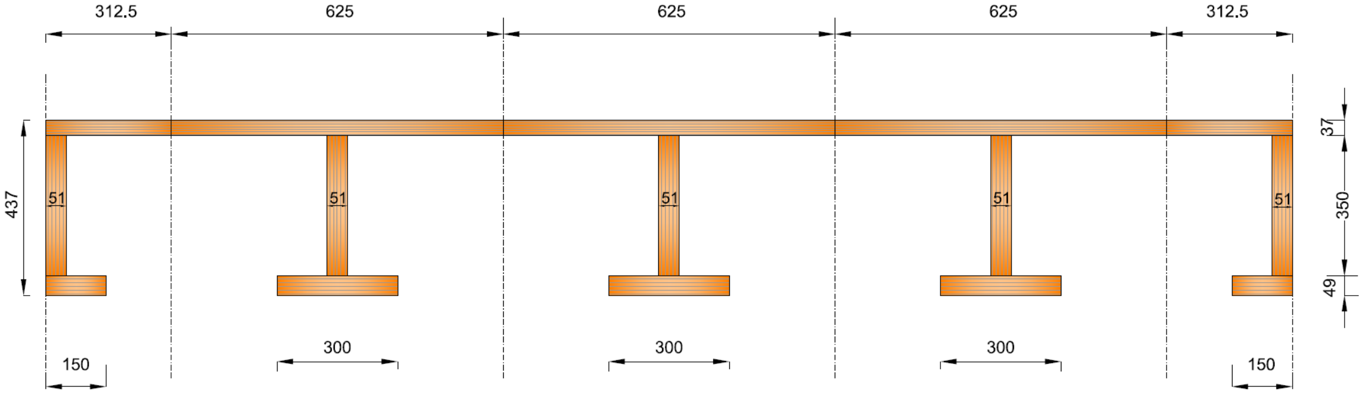

Figure 59: Cross-section of design example panel.

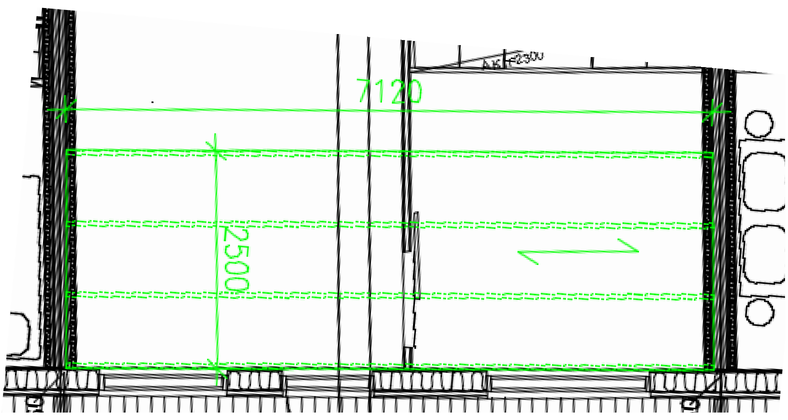

Figure 60: Design example slab.

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3. Verifications in the ultimate limit state (ULS)

3.1 Loadings

Dead load is
$g_k = 1.6\ kN/m^2$

Live load is
$q_k = 2.0kN/m^2$ (live-load of the category A acc. to EN 1991-1-1)
(The self-weight of the panel should be added as well)
Design loads coefficients are
$\gamma_G = 1.35$
$\gamma_Q = 1.5$

The load combination for medium term actions is calculated. Furthermore, also other load durations shall be checked.
$q_d = \gamma_G \cdot g_k + \gamma_Q \cdot q_k = 1.35 \cdot 1.6\ kN/m^2\ + 1.5 \cdot 2.0(\ kN)/m^2 = 5.16\ kN/m^2$

With the spacing between rib axis s=625mm for I section and s= 312.5mm for U section
$q_d = 5.16 \cdot 0.625 = 3.23\ kN/m$ for I section

$q_d = 5.16 \cdot 0.3125 = 1.61\ kN/m$ for U section

Structural calculations of rib panel are divided into three parts: middle rib, edge rib and slab perpendicular to ribs.

3.2 Internal forces

Maximum bending moment at midspan

for I section

$$M_{y,d} = \frac{q_d . L^2}{8} = \frac{3.23 \cdot 7.12^2}{8} = 20.47\ kNm$$

for U section

$$M_{y,d} = \frac{q_d . L^2}{8} = \frac{1.61 \cdot 7.12^2}{8} = 10.20\ kNm$$

- Maximum shear force at the supports

for I section

$$V_{z,d} = \frac{q_d . L}{2} = \frac{3.23 \cdot\ 7.12}{2} = 11.50\ kN$$

for U section

$$V_{z,d} = \frac{q_d . L}{2} = \frac{1.61\ \cdot 7.12}{2} = 5.73\ kN$$

Strength and stiffness values

Strength modification factor is

$$k_{mod} = 0.8$$

Partial factor for the material is

$$\gamma_M = 1.2$$

LVL-X panel
Compressive strength parallel to grain

$$f_{c,0,d} = k_{mod} \cdot \frac{f_{c,0,k}}{\gamma_M} = 0.8 \cdot \frac{26}{1.2} = 17.33\ N/mm^2$$

Shear strength flatwise parallel to grain

$$f_{v,0,flat,d} = k_{mod} \cdot \frac{f_{v,0,flat,k}}{\gamma_M} = 0.8 \cdot \frac{1.3}{1.2} = 0.87\ N/mm^2$$

Shear strength flatwise perpendicular to grain

$$f_{v,90,flat,d} = k_{mod} \cdot \frac{f_{v,90,flat,k}}{\gamma_M} = 0.8 \cdot \frac{0.6}{1.2} = 0.40\ N/mm^2$$

Bending strength flatwise perpendicular to grain

$$f_{m,90,flat,d} = k_{mod} \cdot \frac{f_{m,90,flat,k}}{\gamma_M} = 0.8 \cdot \frac{8}{1.2} = 5.33\ N/mm^2$$

Bending strength flatwise

$$f_{m,0,flat,d} = k_{mod} \cdot \frac{f_{m,0,flat,k}}{\gamma_M} = 0.8 \cdot \frac{36}{1.2} = 24\ N/mm^2$$

Mean modulus of elasticity parallel to grain

$$E_{0,mean} = 10{,}500\ N/mm^2$$

Mean modulus of elasticity perpendicular to grain

$$E_{m,90,mean} = 2{,}000\ N/mm^2$$

Mean shear modulus flatwise perpendicular to grain

$$G_{90,flat,mean} = 22\ N/mm^2$$

LVL-S Rib

Shear strength edgewise

$$f_{v,0,edge,d} = k_{mod} \cdot \frac{f_{v,0,edge,k}}{\gamma_M} = 0.8 \cdot \frac{4.1}{1.2} = 2.73\ N/mm^2$$

Size modification factor

$$k_h = min\begin{cases}\left(\frac{300}{h}\right)^s \\ 1.2\end{cases} = 0.98$$

Bending strength edgewise

$$f_{m,0,edge,d} = k_{mod} \cdot \frac{k_h \cdot f_{m,0,edge,k}}{\gamma_M} = 0.8 \cdot \frac{0.98 \cdot 44}{1.2} = 28.74\ N/mm^2$$

Mean modulus of elasticity parallel to grain

$E_{0,mean} = 13\,800\ N/mm^2$

Mean shear modulus edgewise

$G_{0,edge,mean} = 600\ N/mm^2$

LVL-S bottom flange

Size modification factor

$$k_l = min\begin{cases}\left(\frac{3000}{l}\right)^{s/2} \\ 1.1\end{cases} = 0.94$$

Tensile strength parallel to grain

$$f_{t,0,d} = k_{mod} \cdot \frac{k_l \cdot f_{t,0,k}}{\gamma_M} = 0.8 \cdot \frac{0.94 \cdot 35}{1.2} = 21.93\ N/mm^2$$

Shear strength flatwise parallel to grain

$$f_{v,0,flat,d} = k_{mod} \cdot \frac{f_{v,0,flat,k}}{\gamma_M} = 0.8 \cdot \frac{2.3}{1.2} = 1.53\ N/mm^2$$

Bending strength flatwise

$$f_{m,0,flat,d} = k_{mod} \cdot \frac{f_{m,0,flat,k}}{\gamma_M} = 0.8 \cdot \frac{50}{1.2} = 33.33\ N/mm^2$$

3.3 Stiffness values taken into account for the example

Table 19:Young´s modulus for different design cases- Parallel to the grain (Example for K_{def} in SC1)

Design	**Time**	**Definition acc.to EN 1995-1-1**	**E_0 [N/mm²]**		**G_0 [N/mm²]**		
			LVL-S	**LVL-X**	**LVL-S edgewise**	**LVL-S flatwise**	**LVL-X flatwise**
ULS	t = 0	$X_{inst} = X_{mean}$	13,800	10,500	600	460	120
		$n_{inst,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	1.31		-	3.83	
	t = ∞[5]	$X_{fin,d} = \frac{X_{mean}}{\gamma_M \cdot (1 + \psi_2 \cdot k_{def})}$	9745.76	7056.45	423.73	324.86	80.65
		$n_{fin,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	1.38		-	4.03	
SLS	t = 0	$X_{inst} = X_{mean}$	13,800	10,500	600	460	120
		$n_{inst,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	1.31		-	3.83	
	Creep t = ∞	$X_{creep} = \frac{X_{mean}}{k_{def}}$	23,000	13,125	1,000	766.67	150
		$n_{creep,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	1.75		-	5.11	
	t = ∞	$X_{fin} = \frac{X_{mean}}{1 + k_{def}}$	8625	5833.33	375	287.50	66.67
		$n_{fin,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	1.48		-	4.31	

[5] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

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Table 20:Young´s modulus for different design cases- Perpendicular to the grain (Example for K_{def} in SC1)

Design	Time	Definition acc.to EN 1995-1-1	E_{90} [N/mm²] LVL-X	G_{90} [N/mm²] LVL-X flatwise
ULS	t = 0	$X_{inst} = X_{mean}$	2000	22
		$n_{inst,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	-	
	t = ∞[6]	$X_{fin,d} = \frac{X_{mean}}{\gamma_M \cdot (1+\psi_2 \cdot k_{def})}$	1344.10	14.78
		$n_{fin,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	-	
SLS	t = 0	$X_{inst} = X_{mean}$	2000	22
		$n_{inst,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	-	
	Creep t = ∞	$X_{creep} = \frac{X_{mean}}{k_{def}}$	2500	27.5
		$n_{creep,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	-	
	t = ∞	$X_{fin} = \frac{X_{mean}}{1+k_{def}}$	1111.11	12.22
		$n_{fin,d} = \frac{X_{LVL-S}}{X_{LVL-X}}$	-	

[6] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

3.4 Section properties - Middle rib (I section)

Dimensions of rib panel

$b_1 = 625mm$
$t_1 = 37mm$
$b_2 = 51mm$
$h_2 = 350mm$
$b_3 = 300mm$
$t_3 = 49mm$

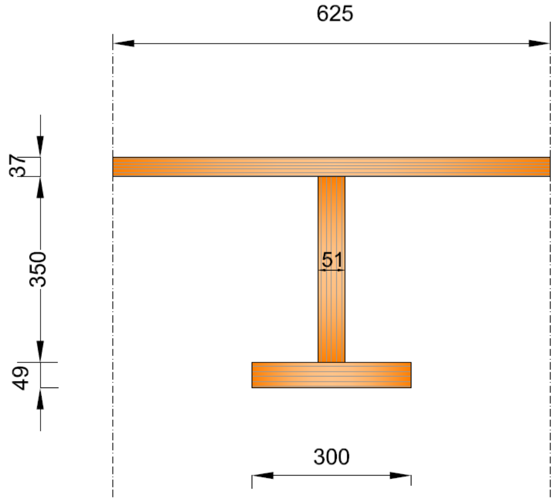

Figure 61: I section dimension

3.4.1 Effective width

Effective upper flange width limits are:

$$0.1L = 712mm$$
$$20h_f = 740mm$$

The value of $b_{c,ef}$ is smaller than the above values thus

$$b_{c,ef} = 712mm$$

In the ultimate limit state, the flexural rigidity is calculated according to effective flange dimensions

$$b_{ef,1} = 625mm$$

Effective lower flange width limit with strength criteria (Table 12) is

$$b_{t,ef} = 167.2mm$$

Because $b_{t,ef} + b_2 = 167.2mm + 51mm = 218.2mm < b_3 = 300mm$

$$b_{ef,3} = 218.2mm$$

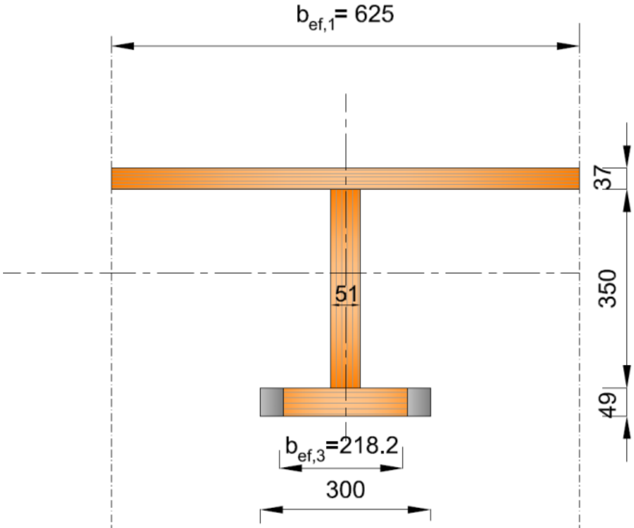

Figure 62:Effective width

3.4.2 Flexural rigidity

- **for the instantaneous situation (t = 0)**

Cross-section area of the slab is

$$A_1 = b_{ef,1} \cdot t_1 = 625 \cdot 37 = 23\,125 mm^2$$

Cross-section area of the rib is

$$A_2 = b_2 \cdot h_2 = 51 \cdot 350 = 17\,850 mm^2$$

Cross-section area of the bottom flange is

$$A_3 = b_{ef,3} \cdot t_3 = 49 \cdot 218.2 = 10\,692 mm^2$$

Place of the neutral axis from upper edge of slab is

$$z_0 = \frac{E_{1,inst,d} \cdot A_1 \cdot \frac{t_1}{2} + E_{2,inst,d} \cdot A_2 \cdot \left(t_1 + \frac{h_2}{2}\right) + E_{3,inst,d} \cdot A_3 \left(t_1 + h_2 + \frac{t_3}{2}\right)}{E_{1,inst,d} \cdot A_1 + E_{2,inst,d} \cdot A_2 + E_{3,inst,d} \cdot A_3}$$

$$= \frac{10500 \cdot 23125 \cdot 18.5 + 13800 \cdot 17850 \cdot 212 + 13800 \cdot 10692 \cdot 411.5}{10500 \cdot 23125 + 13800 \cdot 17850 + 13800 \cdot 10692} = 184.44\,mm$$

Flexural rigidity of the slab is

$$EI_1 = \frac{E_{1,inst,d} \cdot b_{ef,1} \cdot t_1^3}{12} + E_{1,inst,d} \cdot A_1 \cdot \left(z_0 - \frac{t_1}{2}\right)^2 = \frac{10500 \cdot 625 \cdot 37^3}{12} + 10500 \cdot 23125 \cdot \left(184.44 - \frac{37}{2}\right)^2$$

$$= 6.70 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the rib is

$$EI_2 = \frac{E_{2,inst,d} \cdot b_2 \cdot h_2^3}{12} + E_{2,inst,d} \cdot A_2 \left(z_0 - \left(t_1 + \frac{h_2}{2}\right)\right)^2$$

$$= \frac{13800 \cdot 51 \cdot 350^3}{12} + 13800 \cdot 17850 \left(184.44 - \left(37 + \frac{350}{2}\right)\right)^2 = 2.70 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the bottom flange is

$$EI_3 = \frac{E_{3,inst,d} \cdot b_{ef,3} \cdot t_3^3}{12} + E_{3,inst,d} \cdot A_3 \left(z_0 - \left(t_1 + h_2 + \frac{t_3}{2}\right)\right)^2$$

$$= \frac{13800 \cdot 218.2 \cdot 49^3}{12} + 13800 \cdot 10692 \left(184.44 - \left(37 + 350 + \frac{49}{2}\right)\right)^2 = 7.63 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the whole I section is

$$(EI)_{ef} = EI_1 + EI_2 + EI_3 = 6.70 \cdot 10^{12} + 2.70 \cdot 10^{12} + 7.63 \cdot 10^{12} = 1.70 \cdot 10^{13} Nmm^2$$

Flexural rigidity of the slab perpendicular to the ribs is

$$EI_b = E_{m,90,inst,1} \cdot \frac{t_1^3}{12} = 2000 \cdot \frac{37^3}{12} = 8.44 \cdot 10^6 \frac{Nmm^2}{mm}$$

- **for the final situation (t =∞)**[7]

In the ultimate limit state, the flexural rigidity is calculated according to effective flange dimensions

$$b_{ef,1} = 625mm$$

Effective lower flange width limit with strength criteria (Table 12) is

$$b_{t,ef} = 167.2mm$$

Because $b_{t,ef} + b_2 = 167.2mm + 51mm = 218.2mm < b_3 = 300mm$

$$b_{ef,3} = 218.2mm$$

Cross-section areas: of the bottom flange is

$A_1 = b_{ef,1} \cdot t_1 = 23\,125\ mm^2$

$A_2 = b_2 \cdot h_2 = 17\,850\ mm^2$

only bottom flange area is different compared to stiffness criteria

$A_3 = b_{ef,3} \cdot t_3 = 10\,692\ mm^2$

Place of the neutral axis from upper edge of slab is

$$z_0 = \frac{E_{1fin,d} \cdot A_1 \cdot \frac{t_1}{2} + E_{2fin,d} \cdot A_2 \cdot \left(t_1 + \frac{h_2}{2}\right) + E_{3fin,d} \cdot A_3 \cdot \left(t_1 + h_2 + \frac{t_3}{2}\right)}{E_{1fin,d} \cdot A_1 + E_{2fin,d} \cdot A_2 + E_{3fin,d} \cdot A_3}$$

$$= \frac{7056.45 \cdot 23125 \cdot 18.5 + 9745.76 \cdot 17850 \cdot 212 + 9745.76 \cdot 10692 \cdot 411.5}{7056.45 \cdot 23125 + 9745.76 \cdot 17850 + 9745.76 \cdot 10692} = \frac{8.28 \cdot 10^{10}}{441\,343\,888}$$

$$= 187.56\ mm$$

Flexural rigidity of the slab is

$$EI_1 = \frac{E_{1fin,d} \cdot b_{ef,1} \cdot t_1^3}{12} + E_{1fin,d} \cdot A_1 \cdot \left(z_0 - \frac{t_1}{2}\right)^2 = \frac{7056.45 \cdot 625 \cdot 37^3}{12} + 7056.45 \cdot 23\,125 \cdot \left(187.56 - \frac{37}{2}\right)^2$$

$$= 4.68 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the rib is

$$EI_2 = \frac{E_{2fin,d} \cdot b_2 \cdot h_2^3}{12} + E_{2fin,d} \cdot A_2 \cdot \left(z_0 - \left(t_1 + \frac{h_2}{2}\right)\right)^2$$

$$= \frac{9745.76 \cdot 51 \cdot 350^3}{12} + 9745.76 \cdot 17\,850 \left(187.56 - \left(37 + \frac{350}{2}\right)\right)^2 = 1.88 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the bottom flange is

$$EI_3 = \frac{E_{3fin,d} \cdot b_{ef,3} \cdot t_3^3}{12} + E_{3fin,d} \cdot A_3 \cdot \left(z_0 - \left(t_1 + h_2 + \frac{t_3}{2}\right)\right)^2 =$$

$$= \frac{9745.76 \cdot 218.2 \cdot 49^3}{12} + 9745.76 \cdot 10\,692 \cdot \left(187.56 - \left(37 + 350 + \frac{49}{2}\right)\right)^2 = 5.25 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the whole I is

$$(EI)_{ef} = EI_1 + EI_2 + EI_3 = 1.18 \cdot 10^{13} Nmm^2$$

Flexural rigidity of the slab perpendicular to the ribs is

$$EI_b = E_{m,90,mean,1} \cdot \frac{t_1^3}{12} = 1344.10 \cdot \frac{37^3}{12} = 5.67 \cdot 10^6 \frac{Nmm^2}{mm}$$

[7] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

3.4.3 Bending stresses

I section

The bending stresses should be calculated in four points, Figure 63.

- **for the instantaneous situation (t = 0)**

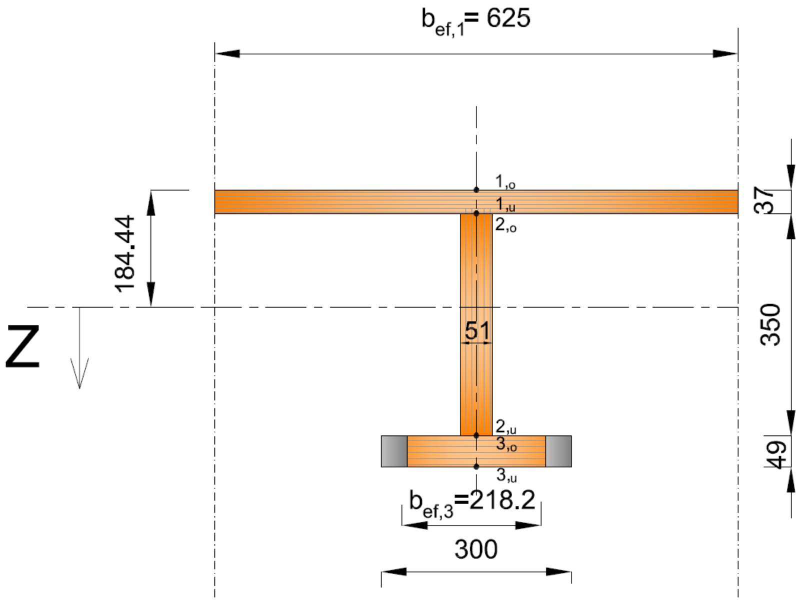

Figure 63: The calculation points of bending stress (t = 0)

- - Bending stresses at the edges of the layer

$$\sigma_{m,d,1,o} = \frac{E_{1,d}(-z_0)}{(EI)_{ef}} \cdot M_{Ed} = \frac{10\,500 \cdot (-184.44)}{1.70 \cdot 10^{13}} \cdot \ 20.47 \cdot 10^6 = -2.33MPa$$

$$\sigma_{m,d,1,u} = \frac{E_{1,d}(-z_0 + t_1)}{(EI)_{ef}} \cdot M_{Ed} = \frac{10\,500 \cdot (-184.44 + 37)}{1.70 \cdot 10^{13}} \cdot \ 20.47 \cdot 10^6 = -1.86MPa$$

$$\sigma_{m,d,2,o} = \frac{E_{2,d}(-z_0 + t_1)}{(EI)_{ef}} \cdot M_{Ed} = \frac{13\,800 \cdot (-184.44 + 37)}{1.70 \cdot 10^{13}} \cdot \ 20.47 \cdot 10^6 = -2.45MPa$$

$$\sigma_{m,d,2,u} = \frac{E_{2,d}(-z_0 + t_1 + h_2)}{(EI)_{ef}} \cdot M_{Ed} = \frac{13\,800 \cdot (-184.44 + 37 + 350)}{1.70 \cdot 10^{13}} \cdot \ 20.47 \cdot 10^6 = +3.36MPa$$

$$\sigma_{m,d,3,o} = \frac{E_{3,d}(-z_0 + t_1 + h_2)}{(EI)_{ef}} \cdot M_{Ed} = \frac{13\,800 \cdot (-184.44 + 37 + 350)}{1.70 \cdot 10^{13}} \cdot \ 20.47 \cdot 10^6 = +3.36MPa$$

$$\sigma_{m,d,3,u} = \frac{E_{3,d}(-z_0 + t_1 + h_2 + t_3)}{(EI)_{ef}} \cdot M_{Ed} = \frac{13\,800 \cdot (-184.44 + 37 + 350 + 49)}{1.70 \cdot 10^{13}} \cdot \ 20.47 \cdot 10^6 = +4.17MPa$$

- Normal stresses at the center of gravity of each layer

$$\sigma_1 = \frac{1}{2} \cdot \left(\sigma_{m,d;1,u} + \sigma_{m,d;1,o}\right) = \frac{1}{2} \cdot \left((-1.86) + (-2.33)\right) = -2.10N/mm^2$$

$$\sigma_2 = \frac{1}{2} \cdot \left(\sigma_{m,d;2,u} + \sigma_{m,d;2,o}\right) = \frac{1}{2} \cdot \left((+3.36) + (-2.45)\right) = 0.46N/mm^2$$

$$\sigma_3 = \frac{1}{2} \cdot \left(\sigma_{m,d;3,u} + \sigma_{m,d;3,o}\right) = \frac{1}{2} \cdot (+4.17 + (+3.36)) = 3.77\ N/mm^2$$

- **for the final situation (t =∞)**[8]

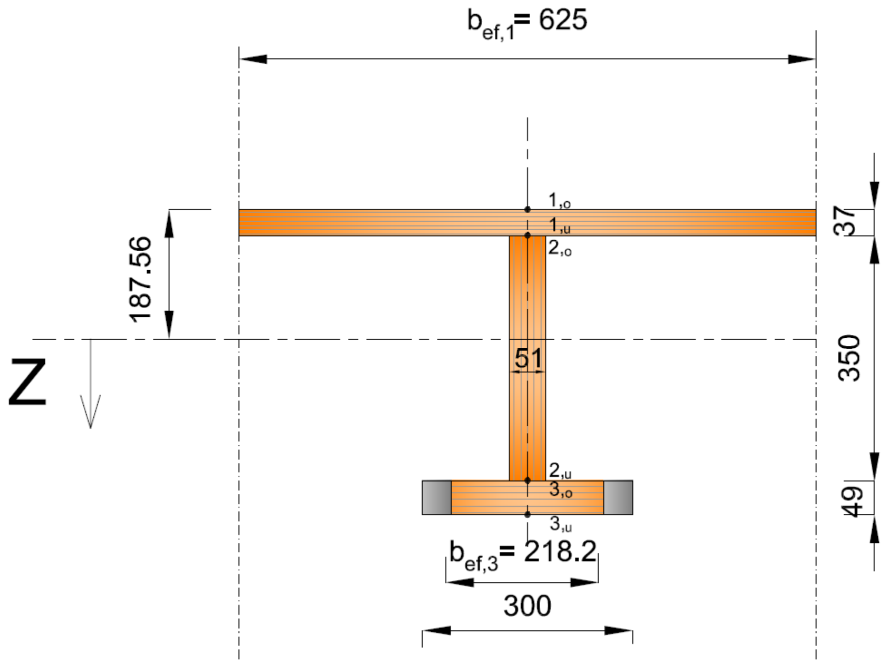

Figure 64:The calculation points of bending stress (t = ∞)

- Bending stresses at the edges of the layer

$$\sigma_{m,d,1,o} = \frac{E_{1,d}(-z_0)}{(EI)_{ef}} \cdot M_{Ed} = \frac{7056 \cdot 45 \cdot (-187.56)}{1.18 \cdot 10^{13}} \cdot\ 20.47 \cdot 10^6 = -2.30MPa$$
$$\sigma_{m,d,1,u} = \frac{E_{1,d}(-z_0 + t_1)}{(EI)_{ef}} \cdot M_{Ed} = \frac{7056.45 \cdot (-187.56 + 37)}{1.18 \cdot 10^{13}} \cdot\ 20.47 \cdot 10^6 = -1.84MPa$$

$$\sigma_{m,d,2,o} = \frac{E_{2,d}(-z_0 + t_1)}{(EI)_{ef}} \cdot M_{Ed} = \frac{9745.76 \cdot (-187.56 + 37)}{1.18 \cdot 10^{13}} \cdot\ 20.47 \cdot 10^6 = -2.56MPa$$
$$\sigma_{m,d,2,u} = \frac{E_{2,d}(-z_0 + t_1 + h_2)}{(EI)_{ef}} \cdot M_{Ed} = \frac{9745.76 \cdot (-187.56 + 37 + 350)}{1.18 \cdot 10^{13}} \cdot 20.47 \cdot 10^6 = +3.37MPa$$

$$\sigma_{m,d,3,o} = \frac{E_{3,d}(-z_0 + t_1 + h_2)}{(EI)_{ef}} \cdot M_{Ed} = \frac{9745.76 \cdot (-187.56 + 37 + 350)}{1.18 \cdot 10^{13}} \cdot\ 20.47 \cdot 10^6 = +3.37MPa$$
$$\sigma_{m,d,3,u} = \frac{E_{3,d}(-z_0 + t_1 + h_2 + t_3)}{(EI)_{ef}} \cdot M_{Ed} = \frac{9745.76 \cdot (-187.56 + 37 + 350 + 49)}{1.18 \cdot 10^{13}} \cdot\ 20.47 \cdot 10^6 = +4.20MPa$$

- Normal stresses at the center of gravity of each layer

$$\sigma_1 = \frac{1}{2} \cdot (\sigma_{m,d;1,u} + \sigma_{m,d;1,o}) = \frac{1}{2} \cdot ((-1.84) + (-2.30)) = -2.07N/mm^2$$
$$\sigma_2 = \frac{1}{2} \cdot (\sigma_{m,d;2,u} + \sigma_{m,d;2,o}) = \frac{1}{2} \cdot ((+3.37) + (-2.56)) = 0.41N/mm^2$$
$$\sigma_3 = \frac{1}{2} \cdot (\sigma_{m,d;3,u} + \sigma_{m,d;3,o}) = \frac{1}{2} \cdot (+4.20 + (+3.37)) = 3.79\ N/mm^2$$

[8] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

3.4.4 Normal stress design

- Analysis acc. to EN 1995-1-1, section 9.1.1 for glued thin-webbed beams

Remark:
The analysis procedure in EN 1995-1-1, section 9.1.2 for glued thin-flanged beams built up with flexible and rigid interfaces between is valid. Apart from the analysis of the bending stresses at the member's edges, also the normal stresses in compression and tension have to be checked. The later mentioned verifications in compression and tension will not become design governing when the range of parameters is considered.

LVL-X panel
Compressive strength parallel to grain

$$f_{c,0,d} = k_{mod} \cdot \frac{k_c \cdot f_{c,0,k}}{\gamma_M} = 0.8 \cdot \frac{1 \cdot 26}{1.2} = 17.33MPa$$

$$f_{m,0,flat,d} = k_{mod} \cdot \frac{f_{m,0,flat,k}}{\gamma_M} = 0.8 \cdot \frac{36}{1.2} = 24MPa$$

LVL-S Rib
Bending strength edgewise

$$f_{c,0,d} = k_{mod} \cdot \frac{f_{c,0,k}}{\gamma_M} = 0.8 \cdot \frac{35}{1.2} = 33.6MPa$$

$$f_{t,0,d} = k_{mod} \cdot \frac{k_l \cdot f_{t,0,k}}{\gamma_M} = 0.8 \cdot \frac{0.94 \cdot 35}{1.2} = 21.93MPa$$

$$f_{m,0,edge,d} = k_{mod} \cdot \frac{k_h \cdot f_{m,0,edge,k}}{\gamma_M} = 0.8 \cdot \frac{0.98 \cdot 44}{1.2} = 28.66MPa$$

LVL-S bottom flange
Tensile strength parallel to grain

$$f_{t,0,d} = k_{mod} \cdot \frac{k_l \cdot f_{t,0,k}}{\gamma_M} = 0.8 \cdot \frac{0.94 \cdot 35}{1.2} = 21.87MPa$$

$$f_{m,0,flat,d} = k_{mod} \cdot \frac{f_{m,0,flat,k}}{\gamma_M} = 0.8 \cdot \frac{50}{1.2} = 33.33MPa$$

3.4.5 Verification of the normal stresses

- **Verification to be fulfilled**

In the upper flange

$$|\sigma_{f,c,d}| \leq k_c \cdot f_{f,c,0,d}$$

$$\sigma_{f,c,\mathrm{d;LVL-X}} \leq \frac{k_c \cdot k_{mod} \cdot f_{c,0,k;\mathrm{LVL-X}}}{\gamma_M}$$

In the bottom flange

$$\sigma_{f,t,d} \leq f_{f,t,0,d}$$

$$\sigma_{f,t,\mathrm{d;LVL-S}} \leq \frac{k_{mod} . k_l . f_{t,0,k;LVL-S}}{\gamma_M}$$

In the Rib

$\sigma_{w,m,d} \leq f_{w,m,0,d}$

$|\sigma_{w,m,0,d;LVL-S}| \leq \frac{k_{mod} \cdot k_h \cdot f_{m,0,k;LVL-S}}{\gamma_M}$

- **for the instantaneous situation (t = 0)**

In the upper flange

Verification of the mean compression stress in the LVL-X panel

$| -2.10 N/mm^2 | \leq 0.8 \cdot \frac{1 \cdot 26}{1.2} = 17.33\ N/mm^2\ (\eta = 12.12\ \%)$

In the bottom flange

Verification of the mean tensile stress in the LVL-S bottom flange

$3.77 N/mm^2 \leq 0.8 \cdot \frac{0.94 \cdot 35}{1.2} = 21.87\ N/mm^2 (\eta = 17.20\ \%)$

In the Rib

Verification of the maximum bending stress in the LVL-S rib at edges

$+3.36 \mathrm{N}/mm^2 \leq 0.8 \cdot \frac{0.98 \cdot 44}{1.2} = 28.66\ N/mm^2\ (\eta = 12\ \%)$

- **for the final situation (t =∞)**[9]

In the upper flange

Verification of the mean compression stress in the LVL-X panel

$| -2.07 N/mm^2 | \leq 0.8 \cdot \frac{26}{1.2} = 17.33 N/mm^2\ (\eta = 12\ \%)$

In the bottom flange

Verification of the mean tensile stress in the LVL-S bottom flange

$3.79 N/mm^2 \leq 0.8 \cdot \frac{0.94 \cdot 35}{1.2} = 21.87 N/mm^2 (\eta = 17.30\ \%)$

In the Rib at the edges

Verification of the maximum bending stress in the LVL-S rib

$+3.37 N/mm^2 \leq 0.8 \cdot \frac{0.98 \cdot 44}{1.2} = 28.66\ N/mm^2\ (\eta = 12\ \%)$

[9] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

3.4.6 Shear stresses

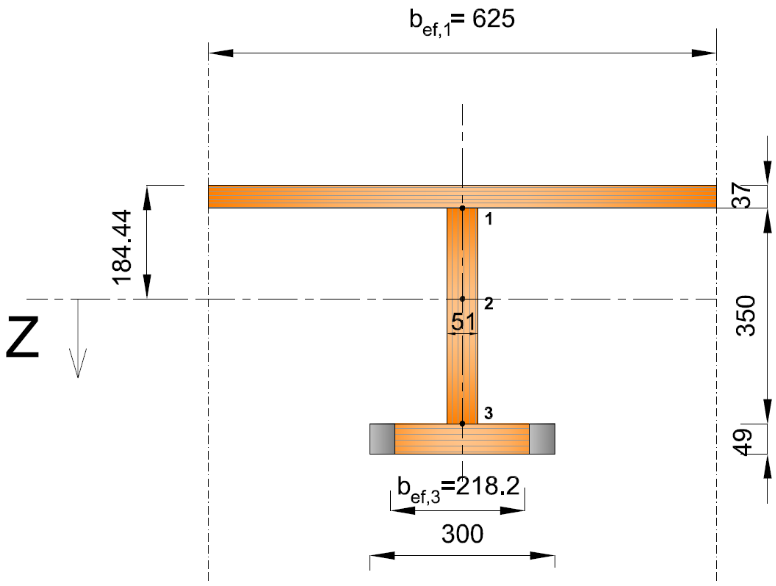

Figure 65. The shear stresses should be calculated in three points

$$V_{z,d} = \frac{q_d \cdot L}{2} = \frac{3.23 \cdot 7.12}{2} = 11.50\ kN$$

for I section

$$\tau(z)_d = E_i \cdot \frac{S_y(z) \cdot V_{z,d}}{EI_{y,ef} \cdot b(z)}$$

$$S_y(z) = \sum_i A_i \cdot e_{z,i}$$

- **for the instantaneous situation (t = 0)**

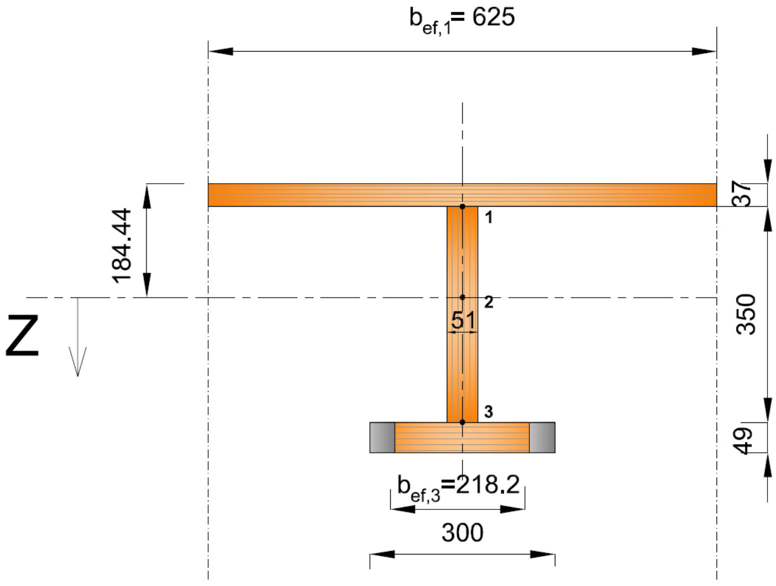

Figure 65: The calculation points of shear stress (t = 0)

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$(EI)_{ef} = EI_1 + EI_2 + EI_3 = 1.70 \cdot 10^{13} Nmm^2$

- at interface LVL-S / LVL-X

$S_{y,1}(z) = b_{ef,1} t_1 \left(z_0 - \frac{t_1}{2}\right) = 625 \cdot 37 \cdot \left(184.44 - \frac{37}{2}\right) = 3\,839\,212.5 = 3.84 \cdot 10^6\ mm^3$

$$\tau_{1,d} = E_{1,d} \cdot \frac{V_{Ed} \cdot S_{y,1}(z)}{b_2 \cdot EI} = 10\,500 \cdot \frac{11.50 \cdot 10^3 \cdot 3.84 \cdot 10^6}{51 \cdot 1.70 \cdot 10^{13}} = 0.54 MPa$$

- in the center of gravity

$$S_{y,2}(z) = b_2 \cdot \frac{(z_0 - t_1)^2}{2} = 51 \cdot \frac{(184.44 - 37)^2}{2} = 554315.80 = 5.54 \cdot 10^5\ mm3$$

$$\tau_{2,d} = \frac{V_{Ed}\left(E_{0,mean,2} \frac{1}{2} b_2 (z_0 - t_1)^2 + E_{0,mean,1} b_{ef,1} t_1 \left(z_0 - \frac{t_1}{2}\right)\right)}{b_2 EI}$$

$$\tau_{2,d} = \frac{V_{Ed}\left(E_{0,mean,2} \cdot S_{y,2}(z) + E_{0,mean,1} \cdot S_{y,1}(z)\right)}{b_2 EI} = \frac{11.50 \cdot 10^3 (13\,800 \cdot 5.54 \cdot 10^5 + 10\,500 \cdot 3.84 \cdot 10^6)}{51 \cdot 1.70 \cdot 10^{13}} = 0.63\ N/mm^2$$

- at interface LVL-S / LVL-S

$S_{y,3}(z) = b_{ef,3} \cdot t_3 \cdot \left(t_1 + h_2 + \frac{t_3}{2} - z_0\right) = 218.2 \cdot 49 \cdot \left(37 + 350 + \frac{49}{2} - 184.44\right) = 2.43 \cdot 10^6\ mm^3$

$$\tau_{3,d} = E_{3,d} \cdot \frac{V_{Ed} \cdot S_{y,3}(z)}{b_2 \cdot EI} = 13\,800 \cdot \frac{11.50 \cdot 10^3 \cdot 2.43 \cdot 10^6}{51 \cdot 1.70 \cdot 10^{13}} = 0.44\ N/mm^2$$

Shear force should be less than

$$h_2 \leq 35 \cdot b_2$$

$$V_{Ed} \leq b_2 h_2 \left(1 + \frac{0.5(t_1 + t_3)}{h_2}\right) f_{v,0,edge,d} = 54.8\ kN$$

=> O.K. ✓

- **for the final situation (t =∞)**[10]

[10] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

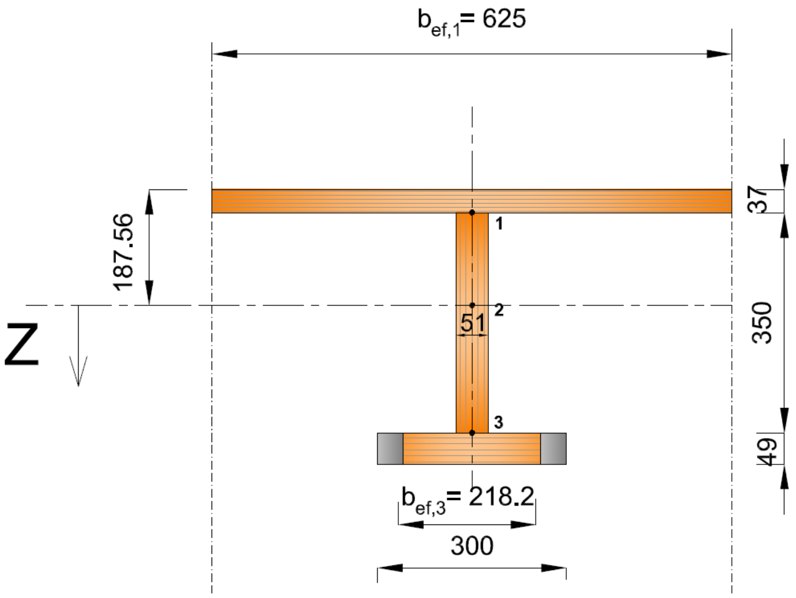

Figure 66:The calculation points of shear stress (t = ∞)

$$(EI)_{ef} = EI_1 + EI_2 + EI_3 = 1.18 \cdot 10^{13} Nmm^2$$

- at interface LVL-S / LVL-X

$$S_{y,1}(z) = b_{ef,1} \cdot t_1 \cdot \left(z_0 - \frac{t_1}{2}\right) = 625 \cdot 37 \cdot \left(187.56 - \frac{37}{2}\right) = 3.91 \cdot 10^6\ mm^3$$

$$\tau_{1,d} = E_{1,d} \cdot \frac{V_{Ed} \cdot S_{y,1}(z)}{b_2 \cdot EI} = 7056.45 \cdot \frac{11.50 \cdot 10^3 \cdot 3.91 \cdot 10^6}{51 \cdot 1.18 \cdot 10^{13}} = 0.53 MPa$$

- in the center of gravity

$$S_{y,2}(z) = b_2 \cdot \frac{(z_0 - t_1)^2}{2} = 51 \cdot \frac{(187.56 - 37)^2}{2} = 578021.70 = 5.78 \cdot 10^5\ mm^3$$

$$\tau_{2,d} = \frac{V_{Ed}\left(E_{0,mean,2} \cdot S_{y,2}(z) + E_{0,mean,1} \cdot S_{y,1}(z)\right)}{b_2 EI} = \frac{11.50 \cdot 10^3 (9745.76 \cdot 5.78 \cdot 10^5 + 7056.45 \cdot 3.91 \cdot 10^6)}{51 \cdot 1.18 \cdot 10^{13}} = 0.63 MPa$$

- at interface LVL-S / LVL-S

$$S_{y,3}(z) = b_{ef,3} \cdot t_3 \cdot \left(t_1 + h_2 + \frac{t_3}{2} - z_0\right) = 218.2 \cdot 49 \cdot \left(37 + 350 + \frac{49}{2} - 187.56\right) = 2.39 \cdot 10^6\ mm^3$$

$$\tau_{3,d} = E_{3,d} \cdot \frac{V_{Ed} \cdot S_{y,3}(z)}{b_2 \cdot EI} = 9745.76 \cdot \frac{11.50 \cdot 10^3 \cdot 2.39 \cdot 10^6}{51 \cdot 1.18 \cdot 10^{13}} = 0.45 MPa$$

3.4.7 Verification of the shear stresses

$$\tau_{max,d} \leq \frac{k_{mod} \cdot k_{cr} \cdot f_{v,0,,k}}{\gamma_M}$$

- **Verification to be fulfilled**

Crack coefficient according to EN 1995-1-1, item 6.1.7 (Recommendation for LVL $\boldsymbol{k_{cr} = 1.0}$)

LVL-S Rib

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$$\tau_{max,d} \leq \frac{k_{mod} \cdot k_{cr} \cdot f_{v(LVL-S),0,edge,k}}{\gamma_{M,LVL-S}} = \frac{0.8 \cdot 1.0 \cdot\ 4.1}{1.2} = 2.73MPa$$

LVL-X panel

$$f_{v(LVL-X),0,flat,d} = \frac{k_{mod} \cdot f_{v(LVL-X),0,flat,k}}{\gamma_{M,LVL-X}} = \frac{0.8 \cdot\ 1.3}{1.2} = 0.87MPa$$

LVL-S bottom flange

$$f_{v(LVL-S),0,flat,d} = \frac{k_{mod} \cdot f_{v(LVL-S),0,flat,k}}{\gamma_{M,LVL-S}} = \frac{0.8 \cdot 2.3}{1.2} = 1.53MPa$$

- **for the instantaneous situation (t = 0)**

➢ Verification at the interface LVL-S / LVL-X

For LVL-X

$$0.54MPa \leq \frac{0.8 \cdot\ 1.3}{1.2} = 0.87MPa\ (\eta = 62\ \%)$$

For LVL-S

$$0.54MPa \leq \frac{0.8 \cdot 1.0 \cdot 4.1}{1.2} = 2.73MPa\ (\eta = 20\ \%)$$

➢ Verification of the maximum shear stress in the LVL-S rib

$$0.63MPa \leq \frac{0.8 \cdot 1.0 \cdot 4.1}{1.2} = 2.73MPa\ \ (\eta = 23\ \%)$$

➢ Verification at the interface LVL-S / LVL-S

For LVL-S rib

$$0.44MPa \leq \frac{0.8 \cdot 1.0 \cdot\ 4.1}{1.2} = 2.73MPa\ (\eta = 16.11\ \%)$$

For LVL-S bottom flange

$$0.44MPa \leq \frac{0.8 \cdot 2.3}{1.2} = 1.53MPa\ (\eta = 29\ \%)$$

- **for the final situation (t =∞)**[11]

➢ Verification at the interface LVL-S / LVL-X

For LVL-X

$$0.53MPa \leq \frac{0.8 \cdot\ 1.3}{1.2} = 0.87MPa\ (\eta = 61\ \%)$$

For LVL-S

$$0.53MPa \leq \frac{0.8 \cdot 1.0 \cdot 4.1}{1.2} = 2.73MPa\ (\eta = 19.42\ \%)$$

➢ Verification of the maximum shear stress in the LVL-S rib

$$0.63MPa \leq \frac{0.8 \cdot 1.0 \cdot 4.1}{1.2} = 2.73MPa(\eta = 23.2\ \%)$$

➢ Verification at the interface LVL-S / LVL-S

For LVL-S rib

$$0.45MPa \leq \frac{0.8 \cdot 1.0 \cdot 4.1}{1.2} = 2.73MPa\ (\eta = 16.5\ \%)$$

For LVL-S bottom flange

$$0.45MPa \leq \frac{0.8 \cdot 2.3}{1.2} = 1.53MPa\ (\eta = 29.5\ \%)$$

[11] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

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3.4.8 Bearing pressure at supports

Assume that the support width is L_s = 100 mm.
The design stress is

$$\sigma_{c,90,d} = \frac{V_{Ed}}{b_2 \cdot L_s} = \frac{11.50 \cdot 10^3}{51 \ \cdot 100} = 2.25 MPa$$

- **Verification to be fulfilled**

  For LVL-S rib

$$f_{c(LVL-S),90,edge,d} = \frac{k_{mod} \cdot k_{c,90} \cdot f_{c(LVL-S),90,edge,k}}{\gamma_{M,LVL-S}} = \frac{0.8 \cdot 1{,}0 \ \cdot \ 6}{1.2} = 4 \ MPa$$

$2.25 MPa < 4 \ MPa$ => O.K. ✓

For LVL-S bottom flange

$$f_{c(LVL-S),90,flat,d} = \frac{k_{mod} \cdot k_{c,90} \cdot f_{c(LVL-S),90,flat,k}}{\gamma_{M,LVL-S}} = \frac{0.8 \cdot 1.0 \cdot 1.8}{1.2} = 1.20 MPa$$

$2.25 MPa > 1.20 \ MPa$ It has to be redesigned for the bearing pressure (the support width Ls has to be increased or the rib width can be increased)

3.5 Section properties - Edge rib (U section)

Dimensions of rib panel

$b_1 = 312.5mm$
$t_1 = 37mm$
$b_2 = 51mm$
$h_2 = 350mm$
$b_3 = 150mm$
$t_3 = 49mm$

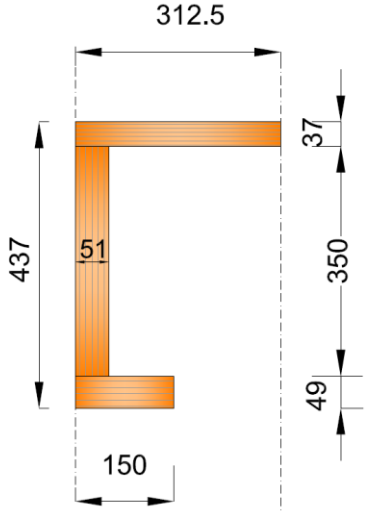

Figure 67:U section

3.5.1 Effective width

Effective upper flange width limits are

$$0.1L = 712mm$$
$$20h_f = 740mm$$

The value of $b_{c,ef}$ is smaller of the above values thus

$$b_{c,ef} = 712mm$$

In the ultimate limit state, the flexural rigidity is calculated according to effective flange dimensions

$$b_{ef,1} = 312.5mm$$

Effective lower flange width limit with strength criteria (Table 12) is

$$b_{t,ef} = 167.2mm$$

Because $0.5 \ b_{t,ef} + b_2 = 83.6mm + 51mm = 134.6mm < \frac{b_1}{2}$

$$b_{ef,3} = 134.6mm$$

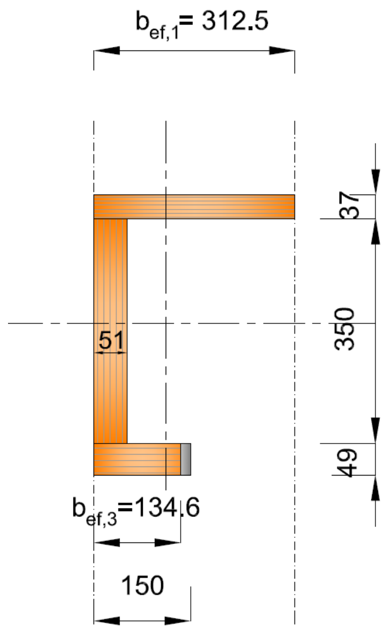

Figure 68: Effective width

3.5.2 Flexural rigidity

- **for the instantaneous situation (t = 0)**

Cross-section areas: of the bottom flange is

$$A_1 = b_{ef,1} \cdot t_1 = 312.5 \cdot 37 = 11563mm^2$$

Cross-section area of the rib is

$$A_2 = b_2 \cdot h_2 = 51 \cdot 350 = 17850mm^2$$

Cross-section area of the bottom flange is

$$A_3 = b_{ef,3} \cdot t_3 = 134.6 \cdot 49 = 6595mm^2$$

Place of the neutral axis from upper edge of slab is

$$z_0 = \frac{E_{1,inst,d} \cdot A_1 \cdot \frac{t_1}{2} + E_{2,inst,d} \cdot A_2 \cdot \left(t_1 + \frac{h_2}{2}\right) + E_{3,inst,d} \cdot A_3 \cdot \left(t_1 + h_2 + \frac{t_3}{2}\right)}{E_{1,inst,d} \cdot A_1 + E_{2,inst,d} \cdot A_2 + E_{3,inst,d} \cdot A_3}$$

$$= \frac{10500 \cdot 11563 \cdot 18.5 + 13800 \cdot 17850 \cdot 212 + 13800 \cdot 6595 \cdot 411.5}{10500 \cdot 11563 + 13800 \cdot 17850 + 13800 \cdot 6595} = 200.37\ mm$$

Flexural rigidity of the slab is

$$EI_1 = \frac{E_{1,inst,d} \cdot b_{ef,1} \cdot t_1^3}{12} + E_{1,inst,d} \cdot A_1 \left(z_0 - \frac{t_1}{2}\right)^2 = \frac{10500 \cdot 312.5 \cdot 37^3}{12} + 10500 \cdot 11563 \cdot \left(200.37 - \frac{37}{2}\right)^2$$

$$= 4.02 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the rib is

$$EI_2 = \frac{E_{2,inst,d} \cdot b_2 h_2^3}{12} + E_{2,inst,d} \cdot A_2 \left(z_0 - \left(t_1 + \frac{h_2}{2}\right)\right)^2$$

$$= \frac{13800 \cdot 51 \cdot 350^3}{12} + 13800 \cdot 17850 \cdot \left(200.37 - \left(37 + \frac{350}{2}\right)\right)^2 = 2.55 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the bottom flange is

$$EI_3 = \frac{E_{3,inst,d} \cdot b_{ef,3} \cdot t_3^3}{12} + E_{3,inst,d} \cdot A_3 \left(z_0 - \left(t_1 + h_2 + \frac{t_3}{2}\right)\right)^2$$
$$= \frac{13800 \cdot 134.6 \cdot 49^3}{12} + 13800 \cdot 6595 \cdot \left(200.37 - \left(37 + 350 + \frac{49}{2}\right)\right)^2 = 4.07 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the whole I section is

$$(EI)_{ef} = EI_1 + EI_2 + EI_3 = 3.36 \ \cdot 10^{12} + 2.12 \cdot 10^{12} + 3.40 \cdot 10^{12} = 1.07 \cdot 10^{13} Nmm^2$$

Flexural rigidity of the slab perpendicular to the ribs is

$$EI_b = E_{m,90,inst,1} \cdot \frac{t_1^3}{12} = 2000 \cdot \frac{37^3}{12} = 8.44 \cdot 10^6 \frac{Nmm^2}{mm}$$

- **for the final situation (t =∞)[12]**

Place of the neutral axis from upper edge of slab is

$$z_0 = \frac{E_{1fin,d} \cdot A_1 \cdot \frac{t_1}{2} + E_{2fin,d} \cdot A_2 \cdot \left(t_1 + \frac{h_2}{2}\right) + E_{3fin,d} \cdot A_3 \cdot \left(t_1 + h_2 + \frac{t_3}{2}\right)}{E_{1fin,d} \cdot A_1 + E_{2fin,d} \cdot A_2 + E_{3fin,d} \cdot A_3}$$
$$= \frac{7056.45 \cdot 11563 \cdot 18.5 + 9745.76 \cdot 17850 \cdot 212 + 9745.76 \cdot 6595 \cdot 411.5}{7056.45 \cdot 11563 + 9745.76 \cdot 17850 + 9745.76 \cdot 6595} = \frac{6.48 \cdot 10^{10}}{319\,828\,834.6}$$
$$= 202.73mm$$

Flexural rigidity of the slab is

$$EI_1 = \frac{E_{1fin,d} \cdot b_{ef,1} \cdot t_1^3}{12} + E_{1fin,d} \cdot A_1 \cdot \left(z_0 - \frac{t_1}{2}\right)^2 = \frac{7056.45 \cdot 312.5 \cdot 37^3}{12} + 7056.45 \cdot 11563 \cdot \left(202.73 - \frac{37}{2}\right)^2$$
$$= 2.77 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the rib is

$$EI_2 = \frac{E_{2fin,d} \cdot b_2 \cdot h_2^3}{12} + E_{2fin,d} \cdot A_2 \cdot \left(z_0 - \left(t_1 + \frac{h_2}{2}\right)\right)^2$$
$$= \frac{9745.76 \cdot 51 \cdot 350^3}{12} + 9745.76 \cdot 17\,850 \cdot \left(202.73 - \left(37 + \frac{350}{2}\right)\right)^2 = 1.79 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the bottom flange is

$$EI_3 = \frac{E_{3fin,d} \cdot b_{ef,3} \cdot t_3^3}{12} + E_{3fin,d} \cdot A_3 \cdot \left(z_0 - \left(t_1 + h_2 + \frac{t_3}{2}\right)\right)^2 =$$
$$= \frac{9745.76 \cdot 134.6 \cdot 49^3}{12} + 9745.76 \cdot 6595 \cdot \left(202.73 - \left(37 + 350 + \frac{49}{2}\right)\right)^2 = 2.81 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the whole I is

$$(EI)_{ef} = EI_1 + EI_2 + EI_3 = (2.78 + 1.79 + 2.82) \cdot 10^{12} = 7.39 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the slab perpendicular to the ribs is

$$EI_b = E_{m,90,mean,1} \cdot \frac{t_1^3}{12} = 1344.10 \cdot \frac{37^3}{12} = 5.67 \cdot 10^6 \frac{Nmm^2}{mm}$$

[12] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

3.5.3 Bending stresses

U section

The bending stresses should be calculated in four points, Figure 69.

- **for the instantaneous situation (t = 0)**

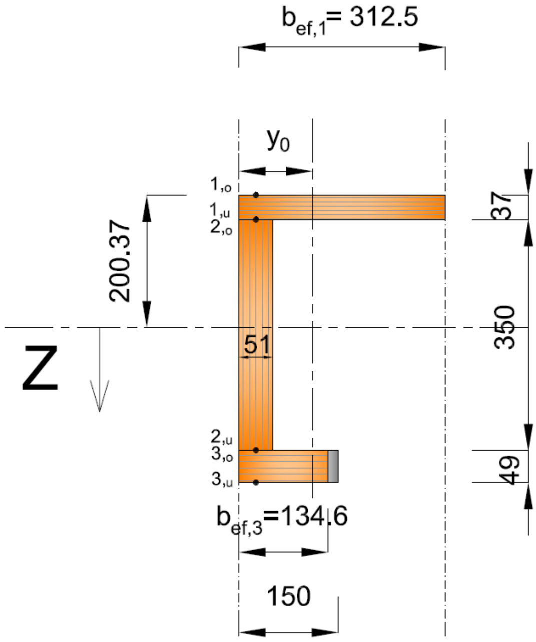

Figure 69: The calculation points of bending stress (t=0)

- - Bending stresses at the edges of the layer

$$\sigma_{m,d,1,o} = \frac{E_{1,d}(-z_0)}{(EI)_{ef}} \cdot M_{Ed} = \frac{10\,500 \cdot (-200.37)}{1.07 \cdot 10^{13}} \cdot 10.20 \cdot 10^6 = -2.01\ MPa$$

$$\sigma_{m,d,1,u} = \frac{E_{1,d}(-z_0 + t_1)}{(EI)_{ef}} \cdot M_{Ed} = \frac{10\,500 \cdot (-200.37 + 37)}{1.07 \cdot 10^{13}} \cdot 10.20 \cdot 10^6 = -1.64 MPa$$

$$\sigma_{m,d,2,o} = \frac{E_{2,d}(-z_0 + t_1)}{(EI)_{ef}} \cdot M_{Ed} = \frac{13\,800 \cdot (-200.37 + 37)}{1.07 \cdot 10^{13}} \cdot 10.20 \cdot 10^6 = -2.16 MPa$$

$$\sigma_{m,d,2,u} = \frac{E_{2,d}(-z_0 + t_1 + h_2)}{(EI)_{ef}} \cdot M_{Ed} = \frac{13\,800 \cdot (-200.37 + 37 + 350)}{1.07 \cdot 10^{13}} \cdot 10.20 \cdot 10^6 = +2.47 MPa$$

$$\sigma_{m,d,3,o} = \frac{E_{3,d}(-z_0 + t_1 + h_2)}{(EI)_{ef}} \cdot M_{Ed} = \frac{13\,800 \cdot (-200.37 + 37 + 350)}{1.07 \cdot 10^{13}} \cdot 10.20 \cdot 10^6 = +2.47 MPa$$

$$\sigma_{m,d,3,u} = \frac{E_{3,d}(-z_0 + t_1 + h_2 + t_3)}{(EI)_{ef}} \cdot M_{Ed} = \frac{13\,800 \cdot (-200.37 + 37 + 350 + 49)}{1.07 \cdot 10^{13}} \cdot 10.20 \cdot 10^6 = +3.12 MPa$$

- Normal stresses at the center of gravity of each layer

$$\sigma_1 = \frac{1}{2} \cdot (\sigma_{m,d;1,u} + \sigma_{m,d;1,o}) = \frac{1}{2} \cdot ((-1.64) + (-2.01)) = -1.83 N/mm^2$$

$$\sigma_2 = \frac{1}{2} \cdot (\sigma_{m,d;2,u} + \sigma_{m,d;2,o}) = \frac{1}{2} \cdot ((+2.47) + (-2.16)) = 0.16 N/mm^2$$

$$\sigma_3 = \frac{1}{2} \cdot (\sigma_{m,d;3,u} + \sigma_{m,d;3,o}) = \frac{1}{2} \cdot (+3.12 + (+2.47)) = 2.80\ N/mm^2$$

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- **for the final situation (t =∞)**[13]

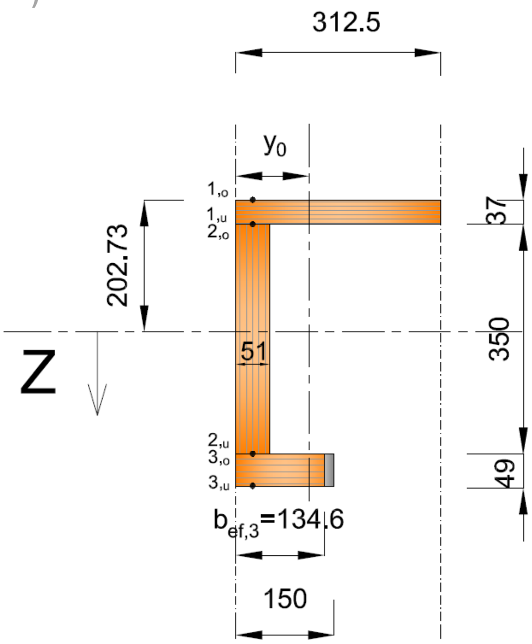

Figure 70:The calculation points of bending stress (t=∞)

- Bending stresses at the edges of the layer

$$\sigma_{m,d,1,o} = \frac{E_{1,d}(-z_0)}{(EI)_{ef}} \cdot M_{Ed} = \frac{7056.45 \cdot (-202.73)}{7.39 \cdot 10^{12}} \cdot 10.20 \cdot 10^6 = -1.97MPa$$

$$\sigma_{m,d,1,u} = \frac{E_{1,d}(-z_0 + t_1)}{(EI)_{ef}} \cdot M_{Ed} = \frac{7056.45 \cdot (-202.73 + 37)}{7.39 \cdot 10^{12}} \cdot 10.20 \cdot 10^6 = -1.62MPa$$

$$\sigma_{m,d,2,o} = \frac{E_{2,d}(-z_0 + t_1)}{(EI)_{ef}} \cdot M_{Ed} = \frac{9745.76 \cdot (-202.73 + 37)}{7.39 \cdot 10^{12}} \cdot 10.20 \cdot 10^6 = -2.23MPa$$

$$\sigma_{m,d,2,u} = \frac{E_{2,d}(-z_0 + t_1 + h_2)}{(EI)_{ef}} \cdot M_{Ed} = \frac{9745.76 \cdot (-202.73 + 37 + 350)}{7.39 \cdot 10^{12}} \cdot 10.20 \cdot 10^6 = +2.48MPa$$

$$\sigma_{m,d,3,o} = \frac{E_{3,d}(-z_0 + t_1 + h_2)}{(EI)_{ef}} \cdot M_{Ed} = \frac{9745.76 \cdot (-202.73 + 37 + 350)}{7.39 \cdot 10^{12}} \cdot 10.20 \cdot 10^6 = +2.48MPa$$

$$\sigma_{m,d,3,u} = \frac{E_{3,d}(-z_0 + t_1 + h_2 + t_3)}{(EI)_{ef}} \cdot M_{Ed} = \frac{9745.76 \cdot (-202.73 + 37 + 350 + 49)}{7.39 \cdot 10^{12}} \cdot 10.20 \cdot 10^6 = +3.14MPa$$

- Normal stresses at the center of gravity of each layer

$$\sigma_1 = \frac{1}{2} \cdot \left(\sigma_{m,d;1,u} + \sigma_{m,d;1,o}\right) = \frac{1}{2} \cdot \left((-1.62) + (-1.97)\right) = -1.80N/mm^2$$

$$\sigma_2 = \frac{1}{2} \cdot \left(\sigma_{m,d;2,u} + \sigma_{m,d;2,o}\right) = \frac{1}{2} \cdot \left((+2.48) + (-2.23)\right) = 0.13N/mm^2$$

$$\sigma_3 = \frac{1}{2} \cdot \left(\sigma_{m,d;3,u} + \sigma_{m,d;3,o}\right) = \frac{1}{2} \cdot (+3.14 + (+2.48)) = 2.81\ N/mm^2$$

[13] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

ANALYSIS SAMPLE Aug-21

3.5.4 Normal stress design

- Verification acc. to EN 1995-1-1, section 9.1.2 for glued thin-flanged beams

Remark:
The verification procedure in EN 1995-1-1, section 9.1.2 for glued thin-flanged beams built up with flexible and rigid interfaces between is valid. Apart from the verification of the bending stresses at the member's edges, also the normal stresses in compression and tension have to be verified. As can be shown, the later mentioned verifications in compression and tension will not become crucial when the range of parameters is considered.

LVL-X panel
Compressive strength parallel to grain

$$f_{c,0,d} = k_{mod} \cdot \frac{f_{c,0,k}}{\gamma_M} = 0.8 \cdot \frac{26}{1.2} = 17.33MPa$$

$$f_{m,0,flat,d} = k_{mod} \cdot \frac{f_{m,0,flat,k}}{\gamma_M} = 0.8 \cdot \frac{36}{1.2} = 24MPa$$

LVL-S Rib
Bending strength edgewise

$$f_{c,0,d} = k_{mod} \cdot \frac{k_c \cdot f_{c,0,k}}{\gamma_M} = 0.8 \cdot \frac{35}{1.2} = 33.6MPa$$

$$f_{t,0,d} = k_{mod} \cdot \frac{k_l \cdot f_{t,0,k}}{\gamma_M} = 0.8 \cdot \frac{0.94 \cdot 35}{1.2} = 21.93MPa$$

$$f_{m,0,edge,d} = k_{mod} \cdot \frac{k_h \cdot f_{m,0,edge,k}}{\gamma_M} = 0.8 \cdot \frac{0.98 \cdot 44}{1.2} = 28.66MPa$$

LVL-S bottom flange
Tensile strength parallel to grain

$$f_{t,0,d} = k_{mod} \cdot \frac{k_l \cdot f_{t,0,k}}{\gamma_M} = 0.8 \cdot \frac{0.94 \cdot 35}{1.2} = 21.87MPa$$

$$f_{m,0,flat,d} = k_{mod} \cdot \frac{f_{m,0,flat,k}}{\gamma_M} = 0.8 \cdot \frac{50}{1.2} = 33.33MPa$$

3.5.5 Verification of the normal stresses

- **Verification to be fulfilled**

In the upper flange

$$|\sigma_{f,c,d}| \leq k_c \cdot f_{f,c,0,d}$$

$$\sigma_{f,c,\mathrm{d;LVL-X}} \leq \frac{k_c \cdot k_{mod} \cdot f_{c,0,k;\mathrm{LVL-X}}}{\gamma_M}$$

In the bottom flange

$$\sigma_{f,t,d} \leq f_{f,t,0,d}$$

$$\sigma_{f,t,\mathrm{d;LVL-S}} \leq \frac{k_{mod}.k_l.f_{t,0,k;LVL-S}}{\gamma_M}$$

In the Rib

$\sigma_{w,m,d} \leq f_{w,m,0,d}$

$|\sigma_{w,m,0,d;LVL-S}| \leq \frac{k_{mod} \cdot k_h \cdot f_{m,0,k;LVL-S}}{\gamma_M}$

- **for the instantaneous situation (t = 0)**

In the upper flange

Verification of the mean compression stress in the LVL-X panel

$\sigma_{f,c,d;LVL-X} \leq \frac{k_c \cdot k_{mod} \cdot f_{c,0,k;LVL-X}}{\gamma_M}$

$| -1.83N/mm^2 | \leq 1.0 \cdot 0.8 \cdot \frac{26}{1.2} = 17.33N/mm^2 \ (\eta = 10.56\ \%)$

In the bottom flange

Verification of the mean tensile stress in the LVL-S bottom flange

$\sigma_{f,t,d;LVL-S} \leq \frac{k_{mod} \cdot k_l \cdot f_{t,0,k;LVL-S}}{\gamma_M}$

$2.80\ N/mm^2 \leq 0.8 \cdot \frac{0.94 \cdot 35}{1.2} = 21.87N/mm^2 \ (\eta = 12.70\ \%)$

In the Rib

Verification of the maximum bending stress in the LVL-S rib at edges

$+2.47N/mm^2 \leq 0.8 \cdot \frac{0.98 \cdot 44}{1.2} = 28.66\ N/mm^2 \ (\eta = 8.6\ \%)$

- **for the final situation (t =∞)**[14]

In the upper flange

Verification of the mean compression stress in the LVL-X panel

$| -1.80N/mm^2 | \leq 0.8 \cdot \frac{26}{1.2} = 17.33N/mm^2 \ (\eta = 10.40\ \%)$

In the bottom flange

Verification of the mean tensile stress in the LVL-S bottom flange

$2.81N/mm^2 \leq 0.8 \cdot \frac{0.94 \cdot 35}{1.2} = 21.87N/mm^2 (\eta = 12.81\ \%)$

In the Rib

Verification of the maximum bending stress in the LVL-S rib

$+2.48N/mm^2 \leq 0.8 \cdot \frac{0.98 \cdot 44}{1.2} = 28.66\ N/mm^2 \ (\eta = 8.6\ \%)$

[14] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

3.5.6 Shear stresses

The shear stresses should be calculated in three points, Figure 71.

$$V_{z,d} = \frac{q_d \cdot L}{2} = \frac{1.61 \cdot 7.12}{2} = 5.73\ kN \quad \text{for U section}$$

$$\tau(z)_d = E_i \cdot \frac{S_y(z) \cdot V_{z,d}}{EI_{y,ef} \cdot b(z)}$$

$$S_y(z) = \sum_i A_i \cdot e_{z,i}$$

- **for the instantaneous situation (t = 0)**

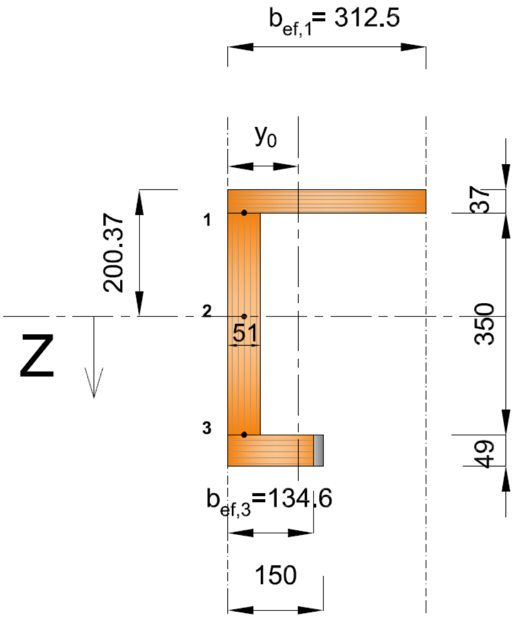

Figure 71: The calculation points of shear stress. (t = 0)

$$(EI)_{ef} = EI_1 + EI_2 + EI_3 = 3.36 \cdot 10^{12} + 2.12 \cdot 10^{12} + 3.40 \cdot 10^{12} = 1.07 \cdot 10^{13} Nmm^2$$

- at interface LVL-S / LVL-X

$$S_{y,1}(z) = b_{ef,1} \cdot t_1 \cdot \left(z_0 - \frac{t_1}{2}\right) = 312.5 \cdot 37 \cdot \left(200.37 - \frac{37}{2}\right) = 2.10 \cdot 10^6\ mm^3$$

$$\tau_{1,d} = E_{1,d} \cdot \frac{V_{Ed} \cdot S_{y,1}(z)}{b_2 \cdot EI} = 10\ 500 \cdot \frac{5.73 \cdot 10^3 \cdot 2.10 \cdot 10^6}{51 \cdot 1.07 \cdot 10^{13}} = 0.24 MPa$$

- in the center of gravity

$$S_{y,2}(z) = b_2 \cdot \frac{(z_0 - t_1)^2}{2} = 51 \cdot \frac{(200.37 - 37)^2}{2} = 680606.62\ mm3 = 6.81 \cdot 10^5\ mm3$$

$$\tau_{2,d} = \frac{V_{Ed}\left(E_{0,mean,2} \cdot S_{y,2}(z) + E_{0,mean,1} \cdot S_{y,1}(z)\right)}{b_2 EI} = \frac{11.50 \cdot 10^3 (13\ 800 \cdot 6.81 \cdot 10^5 + 10\ 500 \cdot 2.10 \cdot 10^6)}{51 \cdot 1.70 \cdot 10^{13}} = 0.33\ N/mm^2$$

- at interface LVL-S / LVL-S

$$S_{y,3}(z) = b_{ef,3} \cdot t_3 \cdot \left(t_1 + h_2 + \frac{t_3}{2} - z_0\right) = 134.6 \cdot 49 \cdot \left(37 + 350 + \frac{49}{2} - 200.37\right) = 1.39 \cdot 10^6\ mm^3$$

$$\tau_{3,d} = E_{3,d} \cdot \frac{V_{Ed} \cdot S_{y,3}(z)}{b_2 \cdot EI} = 13\ 800 \cdot \frac{5.73 \cdot 10^3 \cdot 1.39 \cdot 10^6}{51 \cdot 1.07 \cdot 10^{13}} = 0.21 MPa$$

- for the final situation (t =∞)[15]

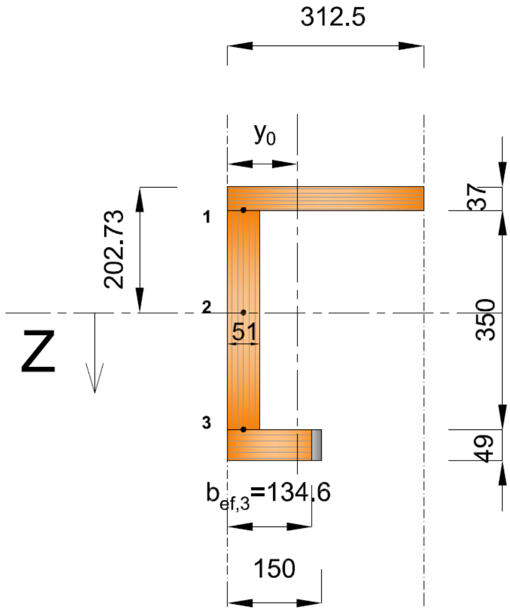

Figure 72:The calculation points of shear stress. (t = 0)

$$(EI)_{ef} = EI_1 + EI_2 + EI_3 = (2.78 + 1.79 + 2.82) \cdot 10^{12} = 7.39 \cdot 10^{12} Nmm^2$$

- at interface LVL-S / LVL-X

$$S_{y,1}(z) = b_{ef,1} \cdot t_1 \cdot \left(z_0 - \frac{t_1}{2}\right) = 312.5 \cdot 37 \cdot \left(202.73 - \frac{37}{2}\right) = 2.13 \cdot 10^6\ mm^3$$

$$\tau_{1,d} = E_{1,d} \cdot \frac{V_{Ed} \cdot S_{y,1}(z)}{b_2 \cdot EI} = 7056.45 \cdot \frac{5.73 \cdot 10^3 \cdot 2.13 \cdot 10^6}{51 \cdot 7.39 \cdot 10^{12}} = 0.23 MPa$$

- in the center of gravity

$$S_{y,2}(z) = b_2 \cdot \frac{(z_0 - t_1)^2}{2} = 51 \cdot \frac{(202.73 - 37)^2}{2} = 700404.92 = 7.00 \cdot 10^5\ mm^3$$

$$\tau_{2,d} = \frac{V_{Ed}\left(E_{0,mean,2} \cdot S_{y,2}(z) + E_{0,mean,1} \cdot S_{y,1}(z)\right)}{b_2 EI} = \frac{5.73 \cdot 10^3 \cdot (9745.76 \cdot 7.00 \cdot 10^5 + 7056.45 \cdot 2.13 \cdot 10^6)}{51 \cdot 7.39 \cdot 10^{12}} = 0.33 MPa$$

- at interface LVL-S / LVL-S

$$S_{y,3}(z) = b_{ef,3} \cdot t_3 \cdot \left(t_1 + h_2 + \frac{t_3}{2} - z_0\right) = 134.6 \cdot 49 \cdot \left(37 + 350 + \frac{49}{2} - 202.73\right) = 1.38 \cdot 10^6\ mm^3$$

$$\tau_{3,d} = E_{3,d} \cdot \frac{V_{Ed} \cdot S_{y,3}(z)}{b_2 \cdot EI} = 9745.76 \cdot \frac{5.73 \cdot 10^3 \cdot 1.38 \cdot 10^6}{51 \cdot 7.39 \cdot 10^{12}} = 0.21 MPa$$

3.5.7 Verification of the shear stresses

$$\tau_{max,d} \leq \frac{k_{mod} \cdot k_{cr} \cdot f_{v,0,k}}{\gamma_M}$$

[15] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

- **Verification to be fulfilled**

Crack coefficient according to EN 1995-1-1, item 6.1.7 (Recommendation for LVL $k_{cr} = 0.67$)

LVL-S Rib

$$\tau_{max,d} \leq \frac{k_{mod} \cdot k_{cr} \cdot f_{v(LVL-S),0,edge,k}}{\gamma_{M,LVL-S}} = \frac{0.8 \cdot 1.0 \cdot 4.1}{1.2} = 2.73MPa$$

LVL-X panel

$$f_{v(LVL-X),0,flat,d} = \frac{k_{mod} \cdot f_{v(LVL-X),0,flat,k}}{\gamma_{M,LVL-X}} = \frac{0.8 \cdot 1.3}{1.2} = 0.87MPa$$

LVL-S bottom flange

$$f_{v(LVL-S),0,flat,d} = \frac{k_{mod} \cdot f_{v(LVL-S),0,flat,k}}{\gamma_{M,LVL-S}} = \frac{0.8 \cdot 2.3}{1.2} = 1.53MPa$$

- **for the instantaneous situation (t = 0)**

- Verification at the interface LVL-S / LVL-X

For LVL-X

$$0.24MPa \leq \frac{0.8 \cdot 1.3}{1.2} = 0.87MPa\ (\eta = 27.60\ \%)$$

For LVL-S

$$0.24MPa \leq \frac{0.8 \cdot 1.0 \cdot 4.1}{1.2} = 2.73MPa(\eta = 8.80\ \%)$$

- Verification of the maximum shear stress in the LVL-S rib

$$0.33MPa \leq \frac{0.8 \cdot 1.0 \cdot 4.1}{1.2} = 2.73MPa\ (\eta = 12.2\ \%)$$

- Verification at the interface LVL-S / LVL-S

For LVL-S rib

$$0.21MPa \leq \frac{0.8 \cdot 1.0 \cdot 4.1}{1.2} = 2.73MPa\ (\eta = 7.70\ \%)$$

For LVL-S bottom flange

$$0.21MPa \leq \frac{0.8 \cdot 2.3}{1.2} = 1.53MPa(\eta = 13.80\ \%)$$

- **for the final situation (t =∞)[16]**

➢ Verification at the interface LVL-S / LVL-X

For LVL-X

$$0.23MPa \leq \frac{0.8 \cdot 1.3}{1.2} = 0.87MPa \ \ (\eta = 26.44\ \%)$$

For LVL-S

$$0.23MPa \leq \frac{0.8 \cdot 1.0 \cdot 4.1}{1.2} = 2.73MPa \ (\eta = 8.42\ \%)$$

➢ Verification of the maximum shear stress in the LVL-S rib

$$0.33MPa \leq \frac{0.8 \cdot 1.0 \cdot 4.1}{1.2} = 2.73MPa \ \ (\eta = 12.20\ \%)$$

➢ Verification at the interface LVL-S / LVL-S

For LVL-S rib

$$0.21MPa \leq \frac{0.8 \cdot 1.0 \cdot 4.1}{1.2} = 2.73MPa \ \ (\eta = 7.70\ \%)$$

For LVL-S bottom flange

$$0.21MPa \leq \frac{0.8 \cdot 2.3}{1.2} = 1.53MPa \ (\eta = 13.7\ \%)$$

3.5.8 Bearing pressure at supports

Assume that the support width is L_s = 100 mm.
The design stress is

$$\sigma_{c,90,d} = \frac{V_{Ed}}{b_2 \cdot L_s} = \frac{5.73 \cdot 10^3}{51 \cdot 100} = 1.12MPa$$

- **Verification to be fulfilled**

For LVL-S rib

$$f_{c(LVL-S),90,edge,d} = \frac{k_{mod} \cdot k_{c,90} \cdot f_{c(LVL-S),90,edge,k}}{\gamma_{M,LVL-S}} = \frac{0.8 \cdot 1{,}0 \cdot 6}{1.2} = 4\ MPa$$

$1.12MPa < 4\ MPa$ O.K. ✓

For LVL-S bottom flange

$$f_{c(LVL-S),90,flat,d} = \frac{k_{mod} \cdot k_{c,90} \cdot f_{c(LVL-S),90,flat,k}}{\gamma_{M,LVL-S}} = \frac{0.8 \cdot 1{,}0 \cdot 1.8}{1.2} = 1.20MPa$$

$1.12MPa < 1.20MPa$ O.K. ✓

[16] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

3.6 Slab perpendicular to ribs

Bending moment in cross direction is

$$M_{Ed,slab} = \frac{q_d \cdot b_1^2}{8} = \frac{5.16 \cdot 0.625^2}{8} = 0.25\ \frac{kNm}{m}$$

Shear force in cross direction is

$$V_{Ed,slab} = \frac{q_d \cdot b_1}{2} = \frac{5.16 \cdot 0.625}{2} = 1.61\ \frac{kN}{m}$$

3.6.1 Bending stress

$$q_d = \gamma_G g_k + \gamma_Q q_k = 1.35 \cdot 1.6\ \frac{kN}{m^2} + 1.5 \cdot 2.0 \frac{kN}{m^2} = 5.16\ kN/m^2$$

- **for the instantaneous situation (t = 0)**

Flexural rigidity of the slab perpendicular to the ribs is

$$EI_b = E_{m,90,inst,1} \cdot \frac{t_1^3}{12} = 2000 \cdot \frac{37^3}{12} = 8.44 \cdot 10^6 \frac{Nmm^2}{mm}$$

Bending stress is

$$\sigma_d = \frac{E_1 \cdot \frac{t_1}{2} \cdot M_{Ed,slab}}{EI_b} = \frac{2000 \cdot \frac{37}{2} \cdot 0.25 \cdot 10^3}{8.44 \cdot 10^6} = 1.10 MPa$$

- **Verification to be fulfilled**

  Bending strength flatwise perpendicular to grain

$$f_{m,90,flat,d} = k_{mod} \cdot \frac{f_{m,90,flat,k}}{\gamma_M} = 0.8 \cdot \frac{8}{1.2} = 5.33 MPa$$

For LVL-X

$$1.09 MPa \leq 5.33 MPa\ (\eta = 20.45\ \%)$$

- **for the final situation (t =∞)**[17]

Flexural rigidity of the slab perpendicular to the ribs is

$$EI_b = E_{m,90,mean,1} \cdot \frac{t_1^3}{12} = 1344.10 \cdot \frac{37^3}{12} = 5.67 \cdot 10^6 \frac{Nmm^2}{mm}$$

Bending stress is

$$\sigma_d = \frac{E_1 \cdot \frac{t_1}{2} \cdot M_{Ed,slab}}{EI_b} = \frac{1344.10 \cdot \frac{37}{2} \cdot 0.25 \cdot 10^3}{5.67 \cdot 10^6} = 1.10 MPa$$

- **Verification to be fulfilled**

  Bending strength flatwise perpendicular to grain

$$f_{m,90,flat,d} = k_{mod} \cdot \frac{f_{m,90,flat,k}}{\gamma_M} = 0.8 \cdot \frac{8}{1.2} = 5.33 MPa$$

For LVL-X

$$1.10 MPa \leq 5.33 MPa\ (\eta = 20.64\ \%)$$

[17] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

3.6.2 Shear stress

- **for the instantaneous situation (t = 0)**

Shear stress is

- in the center of gravity

$$S_y(z) = b_1 \cdot \left(\frac{t_1}{2}\right) \cdot \left(\frac{t_1}{4}\right) = 1000 \cdot \left(\frac{37}{2}\right) \cdot \left(\frac{37}{4}\right) = 171125.00 = 1.71 \cdot 10^5 \ mm3$$

$$\tau_{1,d} = E_{1,d} \cdot \frac{V_{Ed.} \cdot S_y(z)}{b_1 \cdot EI} = 2000 \cdot \frac{1.61 \cdot 1.71 \cdot 10^5}{1000 \cdot \ 8.44 \cdot 10^6} = 0.07MPa$$

- **Verification to be fulfilled**

Shear strength flatwise perpendicular to grain

$$f_{v,90,flat,d} = k_{mod} \cdot \frac{f_{v,90,flat,k}}{\gamma_M} = 0.8 \cdot \frac{0.6}{1.2} = 0.40MPa$$

For LVL-X

$0.07MPa \leq 0.40MPa$ (η = 16.3 %)

- **for the final situation (t =∞)**[18]

Shear stress is

- in the center of gravity

$$S_y(z) = b_1 \cdot \left(\frac{t_1}{2}\right) \cdot \left(\frac{t_1}{4}\right) = 1000 \cdot \left(\frac{37}{2}\right) \cdot \left(\frac{37}{4}\right) = 1.71 \cdot 10^5 mm3$$

$$\tau_{1,d} = E_{1,d} \cdot \frac{V_{Ed.} \cdot S_y(z)}{b_1 \cdot EI} = 1344.10 \cdot \frac{1.61 \cdot 1.71 \cdot 10^5}{1000 \cdot \ 5.67 \cdot 10^6} = 0.07MPa$$

- **Verification to be fulfilled**

Shear strength flatwise perpendicular to grain

$$f_{v,90,flat,d} = k_{mod} \cdot \frac{f_{v,90,flat,k}}{\gamma_M} = 0.8 \cdot \frac{0.6}{1.2} = 0.40MPa$$

For LVL-X

$0.07MPa \leq 0.40MPa$ (η = 16.3 %)

[18] Since in the given systems, the time effect on ULS design is in any case rather insignificant, this step can be skipped.

4. Verification of the Serviceability Limit State (SLS)

For the verification of the serviceability limit state the effective width b_{ef} can be determined in a good approximation with the rules for the uniformly distributed load Due to the fact, that no constricting effect due to single (point) loads (bearing loads) occurs, the so determined width is more or less constant over the total length of the beam.

4.1 Statics and deflection - Middle rib (I section)

4.1.1 Effective width

Maximum effective flange width of slab is

$$b_1 - b_2 = 625 - 51 = 574mm$$

Flange width is less than 712 mm, it can be used whole width of the slab

$$b_{ef,1} = 574 + 51 = 625mm$$

Flange width limit in tension with stiffness criteria (Table 12) is

$$b_{t,ef} = 296mm$$

Because $b_{t,ef} + b_2 = 296mm + 51mm = 347mm > b_3 = 300mm$ there is no reduction and

$$b_{ef,3} = 300mm$$

4.1.2 Flexural rigidity

- **for the instantaneous situation (t = 0)**

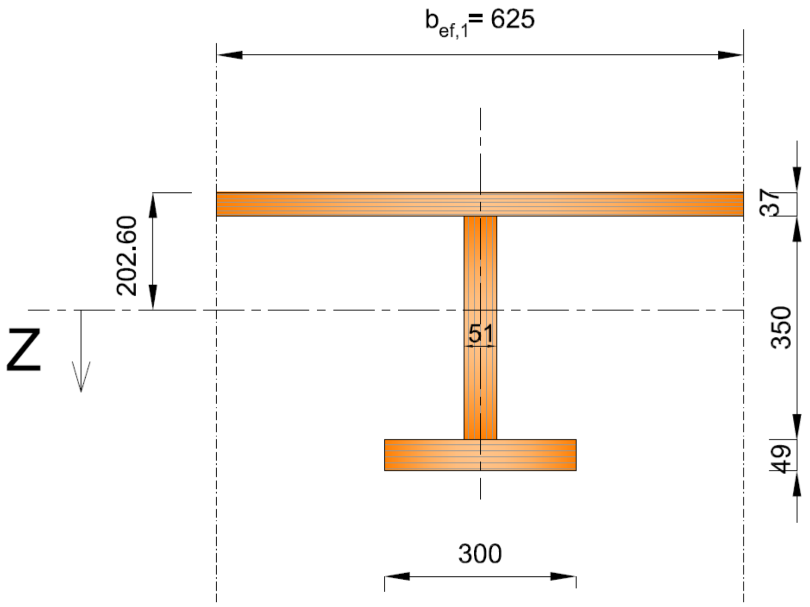

Figure 73: I section in SLS (t=0)

Cross-section area of the slab is

$$A_1 = b_{ef,1} \cdot t_1 = 37 \cdot 625 = 23\,125mm^2$$

Cross-section area of the rib is

$$A_2 = b_2 \cdot h_2 = 51 \cdot 350 = 17\,850mm^2$$

Cross-section area of the bottom flange is

$$A_3 = b_{ef,3} \cdot t_3 = 49 \cdot 300 = 14\,700mm^2$$

Place of the neutral axis from upper edge of slab is

$$z_0 = \frac{E_{1,mean,d} \cdot A_1 \cdot \frac{t_1}{2} + E_{2,mean,d} \cdot A_2 \left(t_1 + \frac{h_2}{2}\right) + E_{3,mean,d} \cdot A_3 \left(t_1 + h_2 + \frac{t_3}{2}\right)}{E_{1,mean,d} \cdot A_1 + E_{2,mean,d} \cdot A_2 + E_{3,mean,d} \cdot A_3}$$

$$= \frac{10\,500 \cdot 23\,125 \cdot 18.5 + 13\,800 \cdot 17\,850 \cdot 212 + 13\,800 \cdot 14\,700 \cdot 411.5}{10\,500 \cdot 23\,125 + 13\,800 \cdot 17\,850 + 13\,800 \cdot 14\,700} = \frac{1.402 \cdot 10^{11}}{692\,002\,500}$$

$$= 202.60mm$$

Flexural rigidity of the slab is

$$EI_1 = \frac{E_{1,mean,d} \cdot b_{ef,1} \cdot t_1^3}{12} + E_{1,mean,d} \cdot A_1 \left(z_0 - \frac{t_1}{2}\right)^2 = \frac{10\,500 \cdot 625 \cdot 37^3}{12} + 10\,500 \cdot 23125 \cdot \left(202.60 - \frac{37}{2}\right)^2$$

$$= 8.26 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the rib is

$$EI_2 = \frac{E_{2,mean,d} \cdot b_2 \cdot h_2^3}{12} + E_{2,mean,d} \cdot A_2 \left(z_0 - \left(t_1 + \frac{h_2}{2}\right)\right)^2$$

$$= \frac{13\,800 \cdot 51 \cdot 350^3}{12} + 13\,800 \cdot 17850 \cdot \left(202.60 - \left(37 + \frac{350}{2}\right)\right)^2 = 2.54 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the bottom flange is

$$EI_3 = \frac{E_{3,mean,d} \cdot b_{ef,3} \cdot t_3^3}{12} + E_{3,mean,d} \cdot A_3 \left(z_0 - \left(t_1 + h_2 + \frac{t_3}{2}\right)\right)^2$$

$$= \frac{13\,800 \cdot 300 \cdot 49^3}{12} + 13\,800 \cdot 14700 \cdot \left(202.60 - \left(37 + 350 + \frac{49}{2}\right)\right)^2 = 8.90 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the whole I section in serviceability state is

$$(EI)_{ef} = EI_1 + EI_2 + EI_3 = (8.26 + 2.54 + 8.90) \cdot 10^{12} = 1.97 \cdot 10^{13} Nmm^2$$

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- **for the final situation (t =∞) creep**

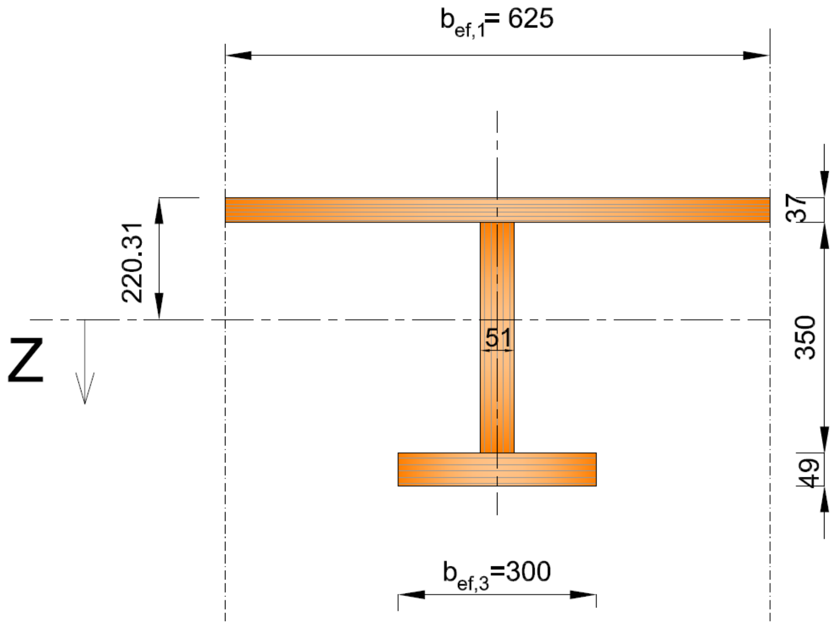

Figure 74:I section in SLS (t =∞) creep

Place of the neutral axis from upper edge of slab is

$$z_0 = \frac{E_{1,creep,d} \cdot A_1 \cdot \frac{t_1}{2} + E_{2,creep,d} \cdot A_2 \left(t_1 + \frac{h_2}{2}\right) + E_{3,creep,d} \cdot A_3 \cdot \left(t_1 + h_2 + \frac{t_3}{2}\right)}{E_{1,creep,d} \cdot A_1 + E_{2,creep,d} \cdot A_2 + E_{3,creep,d} \cdot A_3}$$

$$= \frac{13\,125 \cdot 23\,125 \cdot 18.5 + 23\,000 \cdot 17\,850 \cdot 212 + 23\,000 \cdot 14\,700 \cdot 411.5}{13\,125 \cdot 23\,125 + 23\,000 \cdot 17\,850 + 23\,000 \cdot 14\,700} = \frac{2.318 \cdot 10^{11}}{1\,052\,165\,625}$$

$$= 220.31 mm$$

Flexural rigidity of the slab is

$$EI_1 = \frac{E_{1,creep,d} \cdot b_{ef,1} \cdot t_1^3}{12} + E_{1,creep,d} \cdot A_1 \left(z_0 - \frac{t_1}{2}\right)^2 = \frac{13\,125 \cdot 625 \cdot 37^3}{12} + 13\,125 \cdot 23125 \cdot \left(220.31 - \frac{37}{2}\right)^2$$

$$= 1.24 \cdot 10^{13} Nmm^2$$

Flexural rigidity of the rib is

$$EI_2 = \frac{E_{2,creep,d} \cdot b_2 \cdot h_2^3}{12} + E_{2,creep,d} \cdot A_2 \left(z_0 - \left(t_1 + \frac{h_2}{2}\right)\right)^2$$

$$= \frac{23\,000 \cdot 51 \cdot 350^3}{12} + 23\,000 \cdot 17850 \cdot \left(220.31 - \left(37 + \frac{350}{2}\right)\right)^2 = 4.22 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the bottom flange is

$$EI_3 = \frac{E_{3,creep,d} \cdot b_{ef,3} \cdot t_3^3}{12} + E_{3,creep,d} \cdot A_3 \left(z_0 - \left(t_1 + h_2 + \frac{t_3}{2}\right)\right)^2$$

$$= \frac{23\,000 \cdot 300 \cdot 49^3}{12} + 23\,000 \cdot 14700 \cdot \left(220.31 - \left(37 + 350 + \frac{49}{2}\right)\right)^2 = 1.25 \cdot 10^{13} Nmm^2$$

Flexural rigidity of the whole I section in serviceability state is

$(EI)_{ef} = EI_1 + EI_2 + EI_3 = (1.24 + 1.25) \cdot 10^{13} + (4.22) \cdot 10^{12} = 2.91 \cdot 10^{13} Nmm^2$

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4.1.3 Effective shear stiffness:

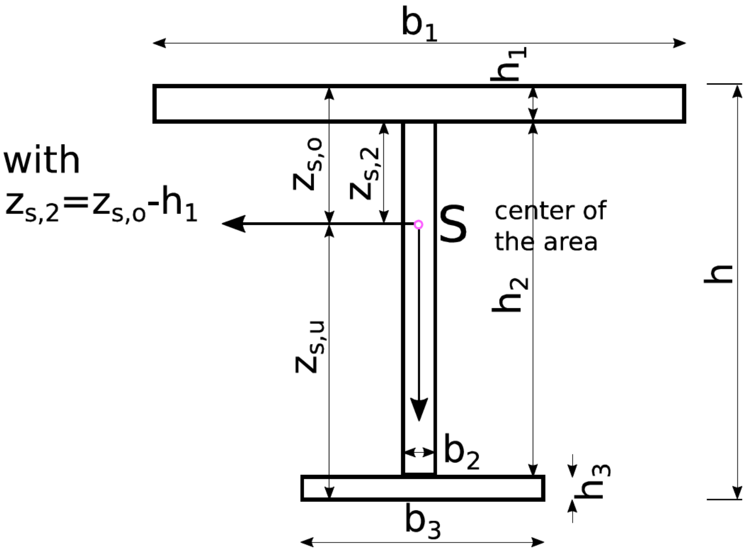

Figure 75:I shaped cross section

- **for the instantaneous situation (t = 0)**

$Z_{s,o} = 202.60mm$
$Z_{s,2} = 202.60 - 37 = 165.6mm$
$Z_{s,u} = h - Z_{s,o} = (37 + 350 + 49) - 202.60 = 436 - 202.60 = 233.4mm$
$(EI)_{ef} = EI_1 + EI_2 + EI_3 = (8.26 + 2.54 + 8.90) \cdot 10^{12} = 1.97 \cdot 10^{13} Nmm^2$

$$(GA) = \sum_i G_{i,inst} \cdot A_i = (120 \cdot 23\,125 + 600 \cdot 17\,850 + 460 \cdot 14\,700) = 2.025 \cdot 10^7 N$$

Part 1

$$J_1 = \frac{E_1{}^2}{G_1} \cdot b_1 \cdot h_1{}^3 \cdot \left(\frac{Z_{s,o}{}^2}{3} - \frac{Z_{s,o} \cdot h1}{4} + \frac{h_1{}^2}{20}\right)$$ Eq 188

$$= \frac{10500^2}{120} \cdot 625 \cdot 37^3 \cdot \left(\frac{202.60^2}{3} - \frac{202.60 \cdot 37}{4} + \frac{37^2}{20}\right) = 3.45397 \cdot 10^{17}$$

Part 2:

$$J_{2,1} = 15 \cdot b_1{}^2 \cdot E_1^2 \cdot h_1{}^2 \cdot \left(2 \cdot Z_{s,2} + h_1\right)^2 = 15 \cdot 625^2 \cdot 10500^2 \cdot 350^2 \cdot (2 \cdot 233.4 + 37)^2$$ Eq 189
$$= 1.19878 \cdot 10^{23}$$

$$J_{2,2} = 10 \cdot b_1 \cdot b_2 \cdot h_1 \cdot h_2 \cdot E_1 \cdot E_2 \cdot (2 \cdot Z_{s,2} + h_1) \cdot (3 \cdot Z_{s,2} - h_2)$$ Eq 190
$$= 10 \cdot 625 \cdot 51 \cdot 37 \cdot 350 \cdot 10500 \cdot 13800 \cdot (2 \cdot 165.6 + 37) \cdot (3 \cdot 165.6 - 37)$$
$$= 3.23188 \cdot 10^{22}$$

$$J_{2,3} = b_2{}^2 \cdot h_2{}^2 \cdot E_2{}^2 \cdot (20 \cdot Z_{s,2}{}^2 - 15 \cdot Z_{s,2} \cdot h_2 + 3 \cdot h_2{}^2)$$ Eq 191
$$= 51^2 \cdot 350^2 \cdot 13800^2 \cdot (20 \cdot 233.4^2 - 15 \cdot 233.4 \cdot 350 + 3 \cdot 350^2)$$
$$= 2.82456 \cdot 10^{21}$$

Part 3:

$$J_3 = \frac{E_3{}^2}{G_3} \cdot b_3 \cdot h_3{}^3 \cdot \left(\frac{Z_{s,u}{}^2}{3} - \frac{Z_{s,u} \cdot h_3}{4} + \frac{h_3{}^2}{20}\right)$$ Eq 192
$$= \frac{13800^2}{460} \cdot 300 \cdot 49^3 \cdot \left(\frac{233.4^2}{3} - \frac{233.4 \cdot 49}{4} + \frac{49^2}{20}\right) = 2.25335 \cdot 10^{17}$$

The shear correction factor κ for the I-shaped section is given by the following equation:

$$\kappa = \frac{GA}{EI_{eff}^{2}} \cdot \left(J_1 + (J_{2,1} + J_{2,2} + J_{2,3}) \cdot \frac{h_2}{60 \cdot b_2 \cdot G_2} + J_3\right)$$ Eq 193

$$= \frac{2.025 \cdot 10^7}{1.97 \cdot 10^{13^2}} \cdot \left(3.45397 \cdot 10^{17} + (1.19878 \cdot 10^{23} + 3.23188 \cdot 10^{22} + 2.82456 \cdot 10^{21}) \cdot \frac{350}{60 \cdot 51 \cdot 600} + 2.25335 \cdot 10^{17}\right) = 1.57$$

$$(GA)_{eff} = \frac{\sum_i G_{i,inst} \cdot A_i}{\kappa} = \frac{2.025 \cdot 10^7}{1.57} = 1.287 \cdot 10^7\ N$$

Simplified method with GA$_{eff}$ rib (for comparison):

$$G_w \cdot A_w = 600\ \cdot\ 17\,850 = 1.07 \cdot 10^7\ N$$

$$(GA)_{eff} = \frac{G_w \cdot A_w \cdot 5}{6} = \frac{1.07 \cdot 10^7 \cdot 5}{6} = 8.916 \cdot 10^6 N$$

Compared to the shear stiffness, applying the precise method, this is a difference of 44%.

- **for the final situation (t =∞) creep**

$$Z_{s,o} = 220.31mm$$
$$Z_{s,2} = 220.31mm - 37 = 183.31mm$$
$$Z_{s,u} = h - Z_{s,o} = (37 + 350 + 49) - 220.31 = 436 - 220.31 = 215.69mm$$
$$(EI)_{ef} = EI_1 + EI_2 + EI_3 = (1.24 + 1.25) \cdot\ 10^{13} + (4.22) \cdot\ 10^{12} = 2.91 \cdot 10^{13} Nmm^2$$

$$(GA)\ = \sum_i G_{i,inst} \cdot A_i = (\,150 \cdot 23\,125 + 1000 \cdot\ 17\,850 + 766.67 \cdot 14\,700) = 3.26 \cdot 10^7 N$$

Part 1

$$J_1 = \frac{E_1^{\,2}}{G_1} \cdot b_1 \cdot h_1^{\,3} \cdot \left(\frac{Z_{s,o}^{\,2}}{3} - \frac{Z_{s,o} \cdot h1}{4} + \frac{h_1^{\,2}}{20}\right)$$ Eq 194

$$= \frac{13125^2}{150} \cdot 625\ \cdot 37^3 \cdot \left(\frac{220.31^2}{3} - \frac{220.31 \cdot 37}{4} + \frac{37^2}{20}\right) = 5.16509 \cdot 10^{17}$$

Part 2:

$$J_{2,1} = 15 \cdot b_1^{\,2} \cdot E_1^2 \cdot h_1^{\,2} \cdot (2 \cdot Z_{s,2} + h_1)^2 = 15 \cdot 625^2 \cdot\ 13125^2 \cdot 350^2 \cdot (2 \cdot 183.31 + 37)^2 = 2.25064 \cdot 10^{23}$$ Eq 195

$$J_{2,2} = 10 \cdot b_1 \cdot b_2 \cdot h_1 \cdot h_2 \cdot E_1 \cdot E_2 \cdot (\,2 \cdot Z_{s,2} + h_1) \cdot (\,3 \cdot Z_{s,2} - h_2)$$ Eq 196

$$= 10 \cdot 625 \cdot 51 \cdot 37 \cdot 350 \cdot 13125 \cdot 23000 \cdot (\,2 \cdot 183.31 + 37) \cdot (\,3 \cdot 183.31 - 37) = 1.0051\ \cdot 10^{23}$$

$$J_{2,3} = b_2^{\,2} \cdot h_2^{\,2} \cdot E_2^{\,2} \cdot (20 \cdot Z_{s,2}^{\,2} - 15 \cdot Z_{s,2} \cdot h_2 + 3 \cdot h_2^{\,2})$$ Eq 197

$$= 51^2 \cdot 350^2 \cdot 23000^2 \cdot (20 \cdot 183.31 - 15 \cdot 183.31 \cdot 350 + 3 \cdot 350^{\,2}) = 1.30001 \cdot 10^{22}$$

Part 3:

$$J_3 = \frac{E_3^{\,2}}{G_3} \cdot b_3 \cdot h_3^{\,3} \cdot \left(\frac{Z_{s,u}^{\,2}}{3} - \frac{Z_{s,u} \cdot h_3}{4} + \frac{h_3^{\,2}}{20}\right)$$ Eq 198

$$= \frac{23000^2}{766.67} \cdot 300 \cdot 49^3 \cdot \left(\frac{215.69^2}{3} - \frac{215.69 \cdot 49}{4} + \frac{49^2}{20}\right) = 3.16303 \cdot 10^{17}$$

The shear correction factor κ for the I-shaped section is given by the following equation:

$$\kappa = \frac{GA}{EI_{eff}^{\,2}} \cdot \left(J_1 + (J_{2,1} + J_{2,2} + J_{2,3}) \cdot \frac{h_2}{60 \cdot b_2 \cdot G_2} + J_3\right)$$ Eq 199

$$= \frac{2.025 \cdot 10^7}{2.91 \cdot 10^{13^2}} \cdot \Big(5.16509 \cdot 10^{17} + (2.25064 \cdot 10^{23} + 1.0051 \cdot 10^{23} + 1.30001 \cdot 10^{22})$$

$$\cdot \frac{350}{60 \cdot 51 \cdot 1000} + 3.16303 \cdot 10^{17}\Big) = 1.53$$

$$(GA)_{creep,eff} = \frac{\sum_i G_{i,creep} \cdot A_i}{\kappa} = \frac{3.26 \cdot 10^7}{1.53} = 2.132 \cdot 10^7\ N$$

Simplified method with GA$_{eff}$ rib (for comparison):

$$G_w \cdot A_w = 1\,000 \cdot 17\,850 = 1.79 \cdot 10^7\ N$$

$$(GA)_{creep,eff} = \frac{G_w \cdot A_w \cdot 5}{6} = \frac{1.79 \cdot 10^7 \cdot 5}{6} = 1.49 \cdot 10^7 N$$

Compared to the shear stiffness, applying the precise method, this is a difference of 43%.

4.1.4 Deflection

- **for the instantaneous situation (t = 0)**

Instantaneous deflection $w_{1,inst}$ due to a "1.0" uniformly distributed load

$$w_{1,inst} = \frac{5 \cdot "1" \cdot l^4}{384 \cdot (E_0 \cdot I)_{y,ef}} + \frac{"1" \cdot l^2}{8 \cdot (GA)_{ef}} = \frac{5 \cdot "1" \cdot 7.12^4}{384 \cdot (1.97 \cdot 10^{13}) \cdot 10^{-9}} + \frac{"1" \cdot 7.12^2}{8 \cdot 1.287 \cdot 10^7 \cdot 10^{-3}} = 1.70\ mm + 0.49\ mm$$

$$= 2.19\ mm / \frac{kN}{m}$$

$$= (78\% + 22\%)$$

Simplified method with GA$_{eff}$ rib (for comparison):

$$w_{1,inst} = \frac{5 \cdot "1" \cdot l^4}{384 \cdot (E_0 \cdot I)_{y,ef}} + \frac{"1" \cdot l^2}{8 \cdot (GA)_{ef}} = \frac{5 \cdot "1" \cdot 7.12^4}{384 \cdot (1.97 \cdot 10^{13}) \cdot 10^{-9}} + \frac{"1" \cdot 7.12^2}{8 \cdot 8.916 \cdot 10^6 \cdot 10^{-3}} = 1.70\ mm + 0.71\ mm$$

$$= 2.41\ mm / \frac{kN}{m}$$

$$= (71\% + 29\%)$$

Compared to the shear stiffness, applying the precise method, this is a difference of 9% on the total deflection.

- Instantaneous deflection w_{inst} due to the characteristic load combination

$$w_{inst} = w_{1,inst} \cdot (g_k + q_k) \cdot b = 2.19 \cdot (0.45 + 1.6 + 2.0) \cdot 0.625 = 5.55mm \leq \frac{l}{300} = \frac{7120}{300} = 23.73mm\ (\eta = 23\%)$$

Simplified method with GA_{eff} rib (for comparison):

$$w_{inst} = w_{1,inst} \cdot (g_k + q_k) \cdot b = 2.41 \cdot (0.45 + 1.6 + 2.0) \cdot 0.625 = 6.10mm \leq \frac{l}{300} = \frac{7120}{300} = 23.73mm\ (\eta = 25.7\%)$$

Compared to the shear stiffness, applying the precise method, this is a difference of 9% on the total deflection.

- **for the final situation (t =∞) creep**

- Deflection $w_{1,creep}$ due to a "1.0" uniformly distributed load

$$w_{1,creep} = \frac{5 \cdot 1 \cdot l^4}{384 \cdot (EI)_{y,creep,ef}} + \frac{1 \cdot l^2}{8 \cdot (GA)_{creep,ef}} = \frac{5 \cdot "1" \cdot 7.12^4}{384 \cdot (2.91 \cdot 10^{13}) \cdot 10^{-9}} + \frac{"1" \cdot 7.12^2}{8 \cdot 2.132 \cdot 10^7 \cdot 10^{-3}} = 1.45\ mm/(\frac{kN}{lm})$$

Simplified method with GA_{eff} rib (for comparison):

$$w_{1,creep} = \frac{5 \cdot 1 \cdot l^4}{384 \cdot (EI)_{y,creep,ef}} + \frac{1 \cdot l^2}{8 \cdot (GA)_{creep,ef}} = \frac{5 \cdot "1" \cdot 7.12^4}{384 \cdot (2.91 \cdot 10^{13}) \cdot 10^{-9}} + \frac{"1" \cdot 7.12^2}{8 \cdot 1.49 \cdot 10^7 \cdot 10^{-3}} = 1.15 + 0.43 = 1.58\ mm/(\frac{kN}{lm})$$

Compared to the shear stiffness, applying the precise method, this is a difference of 8% on the total deflection.

- Net final deflection $w_{,net,fin}$ due to the characteristic and quasi-permanent load combinations

$$w_{net,fin} = \left\{ w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{characteristic\ loading} + w_{1,creep} \cdot \underbrace{[g_{1,k} + g_{2,k} + \psi_2 \cdot q_{1,k}]}_{quasi\ permanent\ loading} \right\} \cdot b$$
$$= \left\{ 2.19 \cdot \underbrace{[0.45 + 1.60 + 2.00]}_{characteristic\ loading} + 1.45 \cdot \underbrace{[0.45 + 1.60 + 0.3 \cdot 2.00]}_{quasi\ permanent\ loading} \right\} \cdot 0.625 = 7.95mm < \frac{l}{250} = \frac{7120}{250} = 28.48mm\ (\eta = 28\%)$$

Simplified method with GA_{eff} rib (for comparison):

$$w_{net,fin} = \left\{ w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{characteristic\ loading} + w_{1,creep} \cdot \underbrace{[g_{1,k} + g_{2,k} + \psi_2 \cdot q_{1,k}]}_{quasi\ permanent\ loading} \right\} \cdot b$$
$$= \left\{ 2.41 \cdot \underbrace{[0.45 + 1.60 + 2.00]}_{characteristic\ loading} + 1.58 \cdot \underbrace{[0.45 + 1.60 + 0.3 \cdot 2.00]}_{quasi\ permanent\ loading} \right\} \cdot 0.625 = 8.72mm < \frac{l}{250} = \frac{7120}{250} = 28.48mm\ (\eta = 30.61\%)$$

Compared to the shear stiffness, applying the precise method, this is a difference of 9% on the total deflection.

Simplified alternative using a uniform k_{def}:
As described in the LVL rib panel design manual, it shall be possible to apply to the entire structure a **uniform k_{def}**.

$$k_{def} = k_{def,LVL-s} = 0.60$$

$$w_{net,fin} = \left\{ w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{characteristic\ loading} + w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + \psi_2 \cdot q_{1,k}]}_{quasi\ permanent\ loading} \cdot k_{def} \right\} \cdot b$$
$$= \{2.41 \cdot [0.45 + 1.60 + 2.00] + 2.41 \cdot [0.45 + 1.60 + 0.3 \cdot 2.00] \cdot 0.60\} \cdot 0{,}625$$
$$= 8.5mm < \frac{l}{250} = \frac{7120}{250} = 28.48mm\ (\eta = 29.83\%)$$

Compared to the shear stiffness, applying the simplified method, this is a difference of 2.6%

- Final deflection $w_{,fin}$ due to the characteristic + quasi-permanent load combinations

$$w_{fin} = \left\{ w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{characteristic\ loading} + w_{1,creep} \cdot \underbrace{[g_{1,k} + g_{2,k} + \psi_2 \cdot q_{1,k}]}_{quasi\ permanent\ loading} \right\} \cdot b$$
$$= \left\{ 2.19 \cdot \underbrace{[0.45 + 1.60 + 2.00]}_{characteristic\ loading} + 1.45 \cdot \underbrace{[0.45 + 1.60 + 0.3 \cdot 2.00]}_{quasi\ permanent\ loading} \right\} \cdot 0.625 = 7.95mm < \frac{l}{150} = \frac{7120}{150}$$
$$= 47.47mm\ (\eta = 23\%)$$

Simplified method with GA_{eff} rib (for comparison):

$$w_{fin} = \left\{ w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{characteristic\ loading} + w_{1,creep} \cdot \underbrace{[g_{1,k} + g_{2,k} + \psi_2 \cdot q_{1,k}]}_{quasi\ permanent\ loading} \right\} \cdot b$$
$$= \{2.41 \cdot [0.45 + 1.60 + 2.00] + 1.58 \cdot [0.45 + 1.60 + 0.3 \cdot 2.00]\} \cdot 0.625 = 8.72mm < \frac{l}{150} = \frac{7120}{150}$$
$$= 47.47mm\ (\eta = 17\%)$$

Compared to the shear stiffness, applying the precise method, this is a difference of 9% on the total deflection.

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4.1.5 Vibration

Combined mass of the structure and the long-term part of the load is

Mass	on 1 section					
Mean density	510,00 Kg/m3					
Self weight	0,51t/m3					
Timber self weight	5,00 kN/m3					
t1	37,00 mm	0,12kN/m	0,19 kN/m²			density*(t1*b1)
t2	51,00 mm	0,09kN/m	0,26 kN/m²	**0,28kN/m**	**0,69 kN/m²**	density*(h2*b2)
t3	49,00 mm	0,07kN/m	0,25 kN/m²			density*(t3*b3)
m1		11,79 Kg/m	18,87 Kg/m²			about the rib
m2		9,10 Kg/m	26,01 Kg/m²	**28,39 Kg/m**	**45,43 Kg/m²**	spacing
m3		7,50 Kg/m	24,99 Kg/m²			(Kg/m) / b1
Screed self weight	65,00 mm					
	2200,00 Kg/m3	density*tscreed*b1				
	2,20t/m3	Timber+Screed				
	21,58 kN/m3	gk*b1				
	1,40 kN/m²	0,88kN/m	143,00 Kg/m²			
Total self weight		1,16kN/m				
Gk surcharge	1,40 kN/m²	0,88kN/m	**143,00 Kg/m²**			
L	7,12 m	71,20m²				
bR	**10,00 m**					
1m wide in cross direction	**1,00 m**					
total mass m	**15,2 t**	**188,43 Kg/m²**	m	Mass of the structure in kg/m² = $\sum_{i\geq1} G_{k,i}$ [kg/m²]		
Modal mass M*	**6708,14 Kg**					

$$m = 188{,}5\ \frac{kg}{m^2}$$

- Additional assumption:
  - Floor class II according to ÖNORM B 1995-1-1:2015
  - Width of the floor system: b_D = 10,0 m
  - Concrete screed (MOE = 25.000 N/mm²); Thickness: 65.0 mm

- <u>Fundamental frequency f1</u>

- Effective flexural rigidity in the longitudinal direction related to a "regular" rib of the ribbed plate:

$$(EI)_{RP,ef} = 1.97 \cdot 10^{13} Nmm^2 = 19.700\ kNm^2$$

$$(EI)_{screed} = 2.50 \cdot 10^7 \cdot \frac{0.625 \cdot 0.065^3}{12} = 3.58 \cdot 10^{11} Nmm^2$$

$$(EI)_{l,ef} = 1.97 \cdot 10^{13} + 3.58 \cdot 10^{11} = 2.00 \cdot 10^{13} Nmm^2 = 20{,}045\ kNm^2$$

Flexural rigidity in the longitudinal direction related to 1 m:

$$(EI)_{l,ef,1m} = \frac{(EI)_{l,ef}}{s} = \frac{20{,}045}{0.625} = 32{,}07\ kNm^2/m$$

Flexural rigidity of the slab perpendicular to the ribs related to 1m:

$$EI_{b,RP} = E_{m,90,inst,1} \cdot \frac{t_1^3}{12} = 2000 \cdot \frac{1000 \cdot 37^3}{12} = 8.44 \cdot 10^6 \frac{Nmm^2}{m} = 8.44 \cdot 10^9 \frac{kNm^2}{m}$$

$$EI_{b,screed} = E_{m,90,inst,1} \cdot \frac{t_1^3}{12} = 25\ 000 \cdot \frac{1000 \cdot 65^3}{12} = 5.72 \cdot 10^8 \frac{Nmm^2}{m} = 5.72 \cdot 10^{11} \frac{kNm^2}{m}$$

$$EI_b = EI_{b,RP} + EI_{b,screed} = 5.81 \cdot 10^8 \frac{Nmm^2}{m} = 5.81 \cdot 10^{11} \frac{kNm^2}{m} = 580.578\ kNm^2$$

$$f_1 = \frac{\pi}{2 \cdot l^2} \cdot \sqrt{\frac{(EI)_{l,ef}}{m}} \cdot \sqrt{1 + \left(\frac{l}{b_D}\right)^4 \cdot \frac{(EI)_{b,ef,1m}}{(EI)_{l,ef,1m}}} = \frac{\pi}{2 \cdot 7.12^2} \cdot \sqrt{\frac{32{,}070\ \cdot 10^3}{188{,}5}} \cdot \sqrt{1 + \left(\frac{7.12}{10}\right)^4 \cdot \frac{5.81 \cdot 10^8}{32{,}070\ \cdot 10^3}}$$
$$= 12.77 Hz$$

Frequency is greater than 9 Hz. => O.K. ✓

- Stiffness criterion

$$b_F = \frac{l}{1.1} \cdot \sqrt[4]{\frac{(EI)_{b,ef,1m}}{(EI)_{l,ef,1m}}} = \frac{7.12}{1.1} \cdot \sqrt[4]{\frac{5.81 \cdot 10^8}{32{,}070 \cdot 10^3}} = 2.37m$$

$$(GA)_{l,eff,1m} = \frac{(GA)_{l,ef}}{s} = \frac{1.287 \cdot 10^7\ N}{0.625\ m} = 2.06 \cdot 10^7 N/m$$

$$w_{1kN} = \frac{F \cdot l^3}{48 \cdot (EI)_{l,eff,1m} \cdot \left[\frac{l}{1,1} \cdot \sqrt[4]{\frac{(EI)_{b,eff,1m}}{(EI)_{l,eff,1m}}}\right]} + \frac{F \cdot l}{4 \cdot (GA)_{l,eff,1m} \cdot \left[\frac{l}{1,1} \cdot \sqrt[4]{\frac{(EI)_{b,eff,1m}}{(EI)_{l,eff,1m}}}\right]}$$

$$= \frac{1000 \cdot 7.12^3}{48 \cdot 32{,}070 \cdot 10^3 \cdot 2.37} + \frac{1000 \cdot 7.12}{4 \cdot 2.06 \cdot 10^7 \cdot 2.37} = 0.14mm < w_{limit,II} = 0.50mm$$

Deflection is below 0.5 mm limit. => O.K. ✓

- Acceleration criterion

$$a_{rms} = \frac{0{,}4 \cdot e^{-0{,}40 \cdot f_1} \cdot F_0}{2 \cdot \zeta \cdot \underbrace{\left[\frac{m \cdot l \cdot b_R}{2}\right]}_{modal\ mass\ M^*}} = \frac{0{,}4 \cdot e^{-0{,}40 \cdot 12.77} \cdot 700}{2 \cdot 0.04 \cdot \underbrace{\left[\frac{188{,}5 \cdot 7.12 \cdot 10}{2}\right]}_{modal\ mass\ M^*}} = 0.0031\ m/s^2$$

$a_{rms} \leq 0{,}10\ m/s^2$ => O.K. ✓

4.2 Edge rib (U section)

4.2.1 Effective width

Dimensions of rib panel

$$b_1 = 312.5mm$$
$$t_1 = 37mm$$
$$b_2 = 51mm$$
$$h_2 = 350mm$$
$$b_3 = 150mm$$
$$t_3 = 49mm$$

Effective upper flange width limits are

$$0.1L = 712mm$$
$$20h_f = 740mm$$

The value of $b_{c,ef}$ is smaller than the above values thus

$$b_{c,ef} = 712mm$$

Because $0.5\ b_{c,ef} + b_2 = 356mm + 51mm = 407mm > \frac{b_1}{2}$
effective upper flange width of slab is

$$b_{ef,1} = \frac{b_1}{2} = 312.5mm$$

Effective lower flange width limit with stiffness criteria (Table 12) is

$$b_{t,ef} = 296mm$$

Because $0.5\ b_{t,ef} + b_2 = 124mm + 51mm = 199mm$ there is no reduction and

$$b_{ef,3} = 150mm$$

4.2.2 Flexural rigidity

- **for the instantaneous situation (t = 0)**

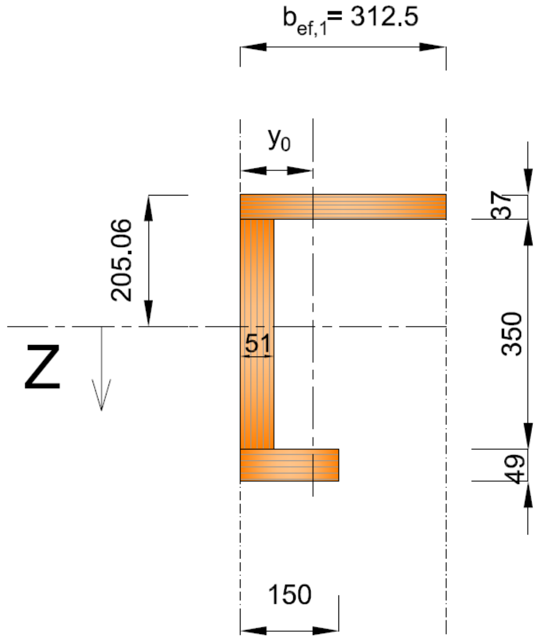

Figure 76:U section in SLS (t =0)

Cross-section areas: of the bottom flange is

$$A_1 = b_{ef,1} \cdot t_1 = 312.5 \cdot 37 = 11563mm^2$$

Cross-section area of the rib is

$$A_2 = b_2 \cdot h_2 = 51 \cdot 350 = 17850mm^2$$

Cross-section area of the bottom flange is

$$A_3 = b_{ef,3} \cdot t_3 = 150 \cdot 49 = 7350mm^2$$

Place of the neutral axis from upper edge of slab is

$$z_0 = \frac{E_{1,inst,d} \cdot A_1 \cdot \frac{t_1}{2} + E_{2,inst,d} \cdot A_2 \cdot \left(t_1 + \frac{h_2}{2}\right) + E_{3,inst,d} \cdot A_3 \cdot \left(t_1 + h_2 + \frac{t_3}{2}\right)}{E_{1,inst,d} \cdot A_1 + E_{2,inst,d} \cdot A_2 + E_{3,inst,d} \cdot A_3}$$

$$= \frac{10500 \cdot 11563 \cdot 18.5 + 13800 \cdot 17850 \cdot 212 + 13800 \cdot 7350 \cdot 411.5}{10500 \cdot 11563 + 13800 \cdot 17850 + 13800 \cdot 7350} = 205.06\,mm$$

Flexural rigidity of the slab is

$$EI_1 = \frac{E_{1,inst,d} \cdot b_{ef,1} \cdot t_1^3}{12} + E_{1,inst,d} \cdot A_1 \left(z_0 - \frac{t_1}{2}\right)^2 = \frac{10500 \cdot 312.5 \cdot 37^3}{12} + 10500 \cdot 11563 \cdot \left(205.06 - \frac{37}{2}\right)^2$$

$$= 4.24 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the rib is

$$EI_2 = \frac{E_{2,inst,d} \cdot b_2 h_2^3}{12} + E_{2,inst,d} \cdot A_2 \left(z_0 - \left(t_1 + \frac{h_2}{2}\right)\right)^2$$
$$= \frac{13800 \cdot 51 \cdot 350^3}{12} + 13800 \cdot 17850 \cdot \left(205.06 - \left(37 + \frac{350}{2}\right)\right)^2 = 2.53 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the bottom flange is

$$EI_3 = \frac{E_{3,inst,d} \cdot b_{ef,3} \cdot t_3^3}{12} + E_{3,inst,d} \cdot A_3 \left(z_0 - \left(t_1 + h_2 + \frac{t_3}{2}\right)\right)^2$$
$$= \frac{13800 \cdot 150 \cdot 49^3}{12} + 13800 \cdot 7350 \cdot \left(205.06 - \left(37 + 350 + \frac{49}{2}\right)\right)^2 = 4.34 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the whole I section is

$$(EI)_{ef} = EI_1 + EI_2 + EI_3 = 1.11 \cdot 10^{13} Nmm^2$$

- **for the final situation (t =∞) creep**

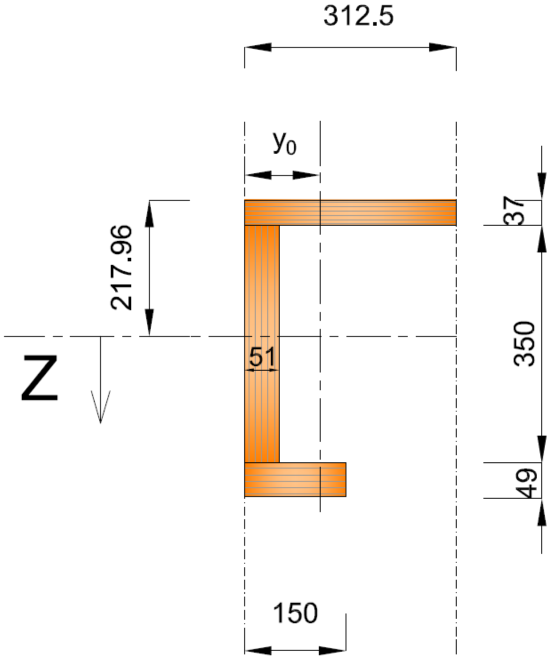

Figure 77:U section in SLS (t =∞) creep

Place of the neutral axis from upper edge of slab is

$$z_0 = \frac{E_{1fin,d} \cdot A_1 \cdot \frac{t_1}{2} + E_{2fin,d} \cdot A_2 \cdot \left(t_1 + \frac{h_2}{2}\right) + E_{3fin,d} \cdot A_3 \cdot \left(t_1 + h_2 + \frac{t_3}{2}\right)}{E_{1fin,d} \cdot A_1 + E_{2fin,d} \cdot A_2 + E_{3fin,d} \cdot A_3}$$

$$= \frac{13125 \cdot 11563 \cdot 18.5 + 23000 \cdot 17850 \cdot 212 + 23000 \cdot 7350 \cdot 411.5}{13125 \cdot 11563 + 23000 \cdot 17850 + 23000 \cdot 7350} = \frac{6.48 \cdot 10^{10}}{319\,828\,834.6} = 217.96mm$$

Flexural rigidity of the slab is

$$EI_1 = \frac{E_{1fin,d} \cdot b_{ef,1} \cdot t_1^3}{12} + E_{1fin,d} \cdot A_1 \cdot \left(z_0 - \frac{t_1}{2}\right)^2 = \frac{13125 \cdot 312.5 \cdot 37^3}{12} + 13125 \cdot 11563 \cdot \left(217.96 - \frac{37}{2}\right)^2$$
$$= 6.05 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the rib is

$$EI_2 = \frac{E_{2fin,d} \cdot b_2 \cdot h_2^3}{12} + E_{2fin,d} \cdot A_2 \cdot \left(z_0 - \left(t_1 + \frac{h_2}{2}\right)\right)^2$$

$$= \frac{23000 \cdot 51 \cdot 350^3}{12} + 23000 \cdot 17\,850 \cdot \left(217.96 - \left(37 + \frac{350}{2}\right)\right)^2 = 4.21 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the bottom flange is

$$EI_3 = \frac{E_{3fin,d} \cdot b_{ef,3} \cdot t_3^3}{12} + E_{3fin,d} \cdot A_3 \cdot \left(z_0 - \left(t_1 + h_2 + \frac{t_3}{2}\right)\right)^2 =$$

$$= \frac{23000 \cdot 1150 \cdot 49^3}{12} + 23000 \cdot 7350 \cdot \left(217.96 - \left(37 + 350 + \frac{49}{2}\right)\right)^2 = 6.37 \cdot 10^{12} Nmm^2$$

Flexural rigidity of the whole I is

$$(EI)_{ef} = EI_1 + EI_2 + EI_3 = 1.66 \cdot 10^{13} Nmm^2$$

4.2.3 Effective shear stiffness:

- **for the instantaneous situation (t = 0)**

$Z_{s,o} = 205.06mm$
$Z_{s,2} = 205.06 - 37 = 168.06mm$
$Z_{s,u} = h - Z_{s,o} = (37 + 350 + 49) - 202.60 = 436 - 205.06 = 230.94mm$
$(EI)_{ef} = EI_1 + EI_2 + EI_3 = 1.11 \cdot 10^{13} Nmm^2$

$$(GA) = \sum_i G_{i,inst} \cdot A_i = (120 \cdot 11563 + 600 \cdot 17\,850 + 460 \cdot 73500) = 1.55 \cdot 10^7 N$$

Part 1

$$J_1 = \frac{{E_1}^2}{G_1} \cdot b_1 \cdot {h_1}^3 \cdot \left(\frac{{Z_{s,o}}^2}{3} - \frac{Z_{s,o} \cdot h1}{4} + \frac{{h_1}^2}{20}\right) \quad \text{Eq 200}$$

$$= \frac{10500^2}{120} \cdot 312.5 \cdot 37^3 \cdot \left(\frac{205.06^2}{3} - \frac{205.06 \cdot 37}{4} + \frac{37^2}{20}\right) = 1.77249 \cdot 10^{17}$$

Part 2:

$$J_{2,1} = 15 \cdot {b_1}^2 \cdot E_1^2 \cdot {h_1}^2 \cdot \left(2 \cdot Z_{s,2} + h_1\right)^2 = 15 \cdot 312.5^2 \cdot 10500^2 \cdot 350^2 \cdot (2 \cdot 168.06 + 37)^2 = 3.07796 \cdot 10^{22} \quad \text{Eq 201}$$

$$J_{2,2} = 10 \cdot b_1 \cdot b_2 \cdot h_1 \cdot h_2 \cdot E_1 \cdot E_2 \cdot (2 \cdot Z_{s,2} + h_1) \cdot (3 \cdot Z_{s,2} - h_2) \quad \text{Eq 202}$$
$$= 10 \cdot 312.5 \cdot 51 \cdot 37 \cdot 350 \cdot 10500 \cdot 13800 \cdot (2 \cdot 168.06 + 37) \cdot (3 \cdot 168.06 - 37) = 1.72035 \cdot 10^{22}$$

$$J_{2,3} = {b_2}^2 \cdot {h_2}^2 \cdot {E_2}^2 \cdot \left(20 \cdot {Z_{s,2}}^2 - 15 \cdot Z_{s,2} \cdot h_2 + 3 \cdot {h_2}^2\right) \quad \text{Eq 203}$$
$$= 51^2 \cdot 350^2 \cdot 13800^2 \cdot \left(20 \cdot 168.06^2 - 15 \cdot 168.06 \cdot 350 + 3 \cdot 350^2\right) = 3.03791 \cdot 10^{21}$$

Part 3:

$$J_3 = \frac{{E_3}^2}{G_3} \cdot b_3 \cdot {h_3}^3 \cdot \left(\frac{{Z_{s,u}}^2}{3} - \frac{Z_{s,u} \cdot h_3}{4} + \frac{{h_3}^2}{20}\right) \quad \text{Eq 204}$$
$$= \frac{13800^2}{460} \cdot 300 \cdot 49^3 \cdot \left(\frac{233.4^2}{3} - \frac{233.4 \cdot 49}{4} + \frac{49^2}{20}\right) = 1.10094 \cdot 10^{17}$$

The shear correction factor κ for the I-shaped section is given by the following equation:

$$\kappa = \frac{GA}{{EI_{eff}}^2} \cdot \left(J_1 + (J_{2,1} + J_{2,2} + J_{2,3}) \cdot \frac{h_2}{60 \cdot b_2 \cdot G_2} + J_3\right) \quad \text{Eq 205}$$
$$= \frac{1.55 \cdot 10^7}{{1.11 \cdot 10^{13}}^2} \cdot \left(1.77249 \cdot 10^{17} + (3.07796 \cdot 10^{22} + 1.72035 \cdot 10^{22} + 3.03791 \cdot 10^{21}) \cdot \frac{350}{60 \cdot 51 \cdot 600} + 1.10094 \cdot 10^{17}\right) = 1.26$$

$$(GA)_{eff} = \frac{\sum_i G_{i,inst} \cdot A_i}{\kappa} = \frac{1.55 \cdot 10^7}{1.26} = 1.232 \cdot 10^7 \ N$$

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- **for the final situation (t =∞) creep**

$Z_{s,o} = 217.96mm$
$Z_{s,2} = 217.96mm - 37 = 180.96mm$
$Z_{s,u} = h - Z_{s,o} = (37 + 350 + 49) - 217.96 = 436 - 217.96 = 218.04mm$
$(EI)_{ef} = EI_1 + EI_2 + EI_3 = (1.24 + 1.25) \cdot 10^{13} + (4.22) \cdot 10^{12} = 1.66 \cdot 10^{13} Nmm^2$

$$(GA) = \sum_i G_{i,inst} \cdot A_i = (150 \cdot 11563 + 1000 \cdot 17\,850 + 766.67 \cdot 7350) = 2.52 \cdot 10^7 N$$

Part 1

$$J_1 = \frac{E_1{}^2}{G_1} \cdot b_1 \cdot h_1{}^3 \cdot \left(\frac{Z_{s,o}{}^2}{3} - \frac{Z_{s,o} \cdot h1}{4} + \frac{h_1{}^2}{20}\right) = \frac{13125^2}{150} \cdot 312.5 \cdot 37^3 \cdot \left(\frac{217.96^2}{3} - \frac{217.96 \cdot 37}{4} + \frac{37^2}{20}\right) = 2.52468 \cdot 10^{17}$$ Eq 206

Part 2:

$$J_{2,1} = 15 \cdot b_1{}^2 \cdot E_1^2 \cdot h_1{}^2 \cdot (2 \cdot Z_{s,2} + h_1)^2 = 15 \cdot 312.5^2 \cdot 13125^2 \cdot 350^2 \cdot (2 \cdot 180.96 + 37)^2 = 5.4976 \cdot 10^{22}$$ Eq 207

$$J_{2,2} = 10 \cdot b_1 \cdot b_2 \cdot h_1 \cdot h_2 \cdot E_1 \cdot E_2 \cdot (2 \cdot Z_{s,2} + h_1) \cdot (3 \cdot Z_{s,2} - h_2) = 10 \cdot 312.5 \cdot 51 \cdot 37 \cdot 350 \cdot 13125 \cdot 23000 \cdot (2 \cdot 180.96 + 37) \cdot (3 \cdot 180.96 - 37) = 4.79411 \cdot 10^{22}$$ Eq 208

$$J_{2,3} = b_2{}^2 \cdot h_2{}^2 \cdot E_2{}^2 \cdot (20 \cdot Z_{s,2}{}^2 - 15 \cdot Z_{s,2} \cdot h_2 + 3 \cdot h_2{}^2) = 51^2 \cdot 350^2 \cdot 23000^2 \cdot (20 \cdot 180.96 - 15 \cdot 180.96 \cdot 350 + 3 \cdot 350^2) = 1.22022 \cdot 10^{22}$$ Eq 209

Part 3:

$$J_3 = \frac{E_3{}^2}{G_3} \cdot b_3 \cdot h_3{}^3 \cdot \left(\frac{Z_{s,u}{}^2}{3} - \frac{Z_{s,u} \cdot h_3}{4} + \frac{h_3{}^2}{20}\right) = \frac{23000^2}{766.67} \cdot 150 \cdot 49^3 \cdot \left(\frac{218.04^2}{3} - \frac{218.04 \cdot 49}{4} + \frac{49^2}{20}\right) = 1.619E \cdot 10^{17}$$ Eq 210

The shear correction factor κ for the I-shaped section is given by the following equation:

$$\kappa = \frac{GA}{EI_{eff}{}^2} \cdot \left(J_1 + (J_{2,1} + J_{2,2} + J_{2,3}) \cdot \frac{h_2}{60 \cdot b_2 \cdot G_2} + J_3\right) = \frac{2.52 \cdot 10^7}{1.66 \cdot 10^{13^2}} \cdot \left(2.52468 \cdot 10^{17} + (5.4976 \cdot 10^{22} + 4.79411 \cdot 10^{22} + 1.22022 \cdot 10^{22}) \cdot \frac{350}{60 \cdot 51 \cdot 1000} + 1.619 \cdot 10^{17}\right) = 1.24$$ Eq 211

$$(GA)_{creep,eff} = \frac{\sum_i G_{i,creep} \cdot A_i}{\kappa} = \frac{2.52 \cdot 10^7}{1.24} = 2.035 \cdot 10^7 N$$

4.2.4 Deflection

- **for the instantaneous situation (t = 0)**

- Instantaneous deflection $w_{1,inst}$ due to a "1.0" uniformly distributed load

$$w_{1,inst} = \frac{5 \cdot "1" \cdot l^4}{384 \cdot (E_0 \cdot I)_{y,ef}} + \frac{"1" \cdot l^2}{8 \cdot (GA)_{ef}} = \frac{5 \cdot "1" \cdot 7.12^4}{384 \cdot (1.11 \cdot 10^{13}) \cdot 10^{-9}} + \frac{"1" \cdot 7.12^2}{8 \cdot 1.232 \cdot 10^7 \cdot 10^{-3}} = 3.01\,mm + 0.51\,mm$$
$$= 3.53 mm/(kN/m)$$
$$= (85\% + 15\%)$$

\- Instantaneous deflection w_{inst} due to the characteristic load combination

$$w_{inst} = w_{1,inst} \cdot (g_k + q_k) \cdot b = 3.53 \cdot (0.45 + 1.6 + 2.0) \cdot 0.3125 = 4.46mm \le \frac{l}{300} = \frac{7120}{300} = 23.73mm\ (\eta = 19\%)$$

- **for the final situation (t =∞) creep**

- Net final deflection $w_{1,creep}$ due to a "1.0" uniformly distributed load

$$w_{1,creep} = \frac{5 \cdot 1 \cdot l^4}{384 \cdot (EI)_{y,creep,ef}} + \frac{1 \cdot l^2}{8 \cdot (GA)_{creep,ef}}$$
$$= \frac{5 \cdot 1 \cdot 7.12^4}{384 \cdot (1.66 \cdot 10^{13}) \cdot 10^{-9}} + \frac{1 \cdot 7.12^2}{8 \cdot 2.035 \cdot 10^7 \cdot 10^{-3}}$$
$$= 2.01\,mmmm + 0.31\,mm = 2.32mm$$

\- Net final deflection $w_{,net,fin}$ due to the characteristic and quasi-permanent load combinations

$$w_{net,fin} = \left\{ w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{characteristic\ loading} + w_{1,creep} \cdot \underbrace{[g_{1,k} + g_{2,k} + \psi_2 \cdot q_{1,k}]}_{quasi\ permanent\ loading} \right\} \cdot b$$
$$= \left\{ 3.53 \cdot \underbrace{[0.45 + 1.60 + 2.00]}_{characteristic\ loading} + 2.32 \cdot \underbrace{[0.45 + 1.60 + 0.3 \cdot 2.00]}_{quasi\ permanent\ loading} \right\} \cdot 0.3125 = 6.39mm < \frac{l}{250} = \frac{7120}{250}$$
$$= 28.48mm\ (\eta = 22\%)$$

\- Final deflection $w_{,fin}$ due to the characteristic + quasi-permanent load combinations

$$w_{fin} = \left\{ w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{characteristic\ loading} + w_{1,creep} \cdot \underbrace{[g_{1,k} + g_{2,k} + \psi_2 \cdot q_{1,k}]}_{quasi\ permanent\ loading} \right\} \cdot b$$
$$= \left\{ 3.53 \cdot \underbrace{[0.45 + 1.60 + 2.00]}_{characteristic\ loading} + 2.32 \cdot \underbrace{[0.45 + 1.60 + 0.3 \cdot 2.00]}_{quasi\ permanent\ loading} \right\} \cdot 0.3125 = 6.39mm < \frac{l}{150} = \frac{7120}{150}$$
$$= 47.47mm\ (\eta = 13\%)$$

The comparison of the simplified deflection method with the uniform K_{def} and the shear stiffness GA_{eff} rib was described only for the I shaped section but the calculation shall be carried out in the same way.

4.2.5 Vibration

Combined mass of the structure and the long-term part of the load is

$$m = 220\ \frac{kg}{m^2}$$

- Additional assumption:
  - Floor class II according to ÖNORM B 1995-1-1:2015
  - Width of the floor system: b_D = 10,0 m

- Fundamental frequency f_1

- Effective flexural rigidity (stiffness of the screed not included in the example) in the longitudinal direction related to a "regular" rib of the ribbed plate:

$$(EI)_{ef} = 1.11 \cdot 10^{13} Nmm^2 = 11{,}100\ kNm^2$$

- Flexural rigidity in the longitudinal direction related to 1 m:

$$(EI)_{l,ef,1m} = \frac{(EI)_{l,ef}}{s} = \frac{11{,}100}{0.3125} = 35{,}520\ kNm^2/m = 35{,}520\ \cdot 10^3 Nm^2/m$$

Flexural rigidity of the slab perpendicular to the ribs related to 1m:

$$EI_b = E_{m,90,inst,1} \cdot \frac{t_1^3}{12} = 2000 \cdot \frac{1000 \cdot 37^3}{12} = 8.44 \cdot 10^6 \frac{Nmm^2}{mm} = 8.44 \cdot 10^9 \frac{kNm^2}{m} = 8.44 \cdot 10^6 \frac{Nm^2}{m}$$

$$f_1 = \frac{\pi}{2 \cdot l^2} \cdot \sqrt{\frac{(EI)_{l,ef}}{m}} \cdot \sqrt{1 + \left(\frac{l}{b_D}\right)^4 \cdot \frac{(EI)_{b,ef,1m}}{(EI)_{l,ef,1m}}} = \frac{\pi}{2 \cdot 7.12^2} \cdot \sqrt{\frac{35{,}520\ \cdot 10^3}{220}} \cdot \sqrt{1 + \left(\frac{7.12}{10}\right)^4 \cdot \frac{8.44 \cdot 10^6}{35{,}520\ \cdot 10^3}} =$$

$$= 12.82 Hz$$

Frequency is greater than 9 Hz. => O.K. ✓

- Stiffness criterion

$$b_F = \frac{l}{1.1} \cdot \sqrt[4]{\frac{(EI)_{b,ef,1m}}{(EI)_{l,ef,1m}}} = \frac{7.12}{1.1} \cdot \sqrt[4]{\frac{8.44 \cdot 10^6}{35{,}520\ \cdot 10^3}} = 0.804 m$$

$$(GA)_{l,eff,1m} = \frac{(GA)_{l,ef}}{s} = \frac{1.232 \cdot 10^7}{0.3125} = 3.94 \cdot 10^7 N/m$$

$$w_{0.5kN} = \frac{F \cdot l^3}{48 \cdot (EI)_{l,eff,1m} \cdot \left[\frac{l}{1,1} \cdot \sqrt[4]{\frac{(EI)_{b,eff,1m}}{(EI)_{l,eff,1m}}}\right]} + \frac{F \cdot l}{4 \cdot (GA)_{l,eff,1m} \cdot \left[\frac{l}{1,1} \cdot \sqrt[4]{\frac{(EI)_{b,eff,1m}}{(EI)_{l,eff,1m}}}\right]}$$

$$= \frac{500 \cdot 7.12^3}{48 \cdot 35{,}520\ \cdot 10^3 \cdot 0.804} + \frac{500 \cdot 7.12}{4 \cdot 3.94 \cdot 10^7\ \cdot 0.804} = 0.16 mm < w_{limit,II} = 0.50 mm$$

Deflection is below 0.5 mm limit. => O.K. ✓

- Acceleration criterion

$$a_{rms} = \frac{0{,}04 \cdot e^{-0{,}47 \cdot f_1} \cdot F_0}{2 \cdot \zeta \cdot \underbrace{\left[\frac{m \cdot l \cdot b_R}{2}\right]}_{modal\ mass\ M^*}} = \frac{0{,}04 \cdot e^{-0{,}47 \cdot 12.82} \cdot 700}{2 \cdot 0.04 \cdot \underbrace{\left[\frac{220 \cdot 7.12 \cdot 10}{2}\right]}_{modal\ mass\ M^*}} = 0.000108\ m/s^2$$

a_{rms} ≤ 0,10 m/s² => O.K. ✓
Deflection is below 0.5 mm limit. => O.K. ✓

4.3 Slab perpendicular to ribs

4.3.1 Deflection

- **for the instantaneous situation (t = 0)**

Instantaneous deflection $w_{1,inst}$ due to a "1.0" uniformly distributed load

$$EI_b = E_{m,90,inst,1} \cdot \frac{1m \cdot t_1^3}{12} = 2000 \cdot \frac{1000 \cdot 37^3}{12} = 8.44 \cdot 10^9 \frac{Nmm^2}{m}$$

$$(GA)_{ef} = (G_{90,LVL-X} \cdot 1m \cdot t_1) \cdot \kappa = 1000 \cdot 22 \cdot 37 \cdot \frac{5}{6} = 6.78 \cdot 10^5\ N/m$$

$$w_{1,inst} = \frac{5 \cdot "1" \cdot l^4}{384 \cdot (E_0 \cdot I)_{y,ef}} + \frac{"1" \cdot l^2}{8 \cdot (GA)_{ef}} = \frac{5 \cdot "1" \cdot 0.625^4}{384 \cdot (8.44 \cdot 10^9) \cdot 10^{-6}} + \frac{"1" \cdot 0.625^2}{8 \cdot 6.78 \cdot 10^5} = 0{,}24\ mm + 0{,}07\ mm$$
$$= 0.31 mm/(kN/m)$$

- Instantaneous deflection w_{inst} due to the characteristic load combination

$$w_{inst} = w_{1,inst} \cdot (g_k + q_k) \cdot b = 0.31 \cdot (0.45 + 1.6 + 2.0) \cdot 1.0 = 1.26mm \leq \frac{l}{300} = \frac{625}{300} = 2.08mm\ (\eta = 60.4\%)$$

- **for the final situation (t =∞) creep**

- Net final deflection $w_{1,creep}$ due to a "1.0" uniformly distributed load

$$EI_b = E_{m,90,mean,1} \cdot \frac{1m \cdot t_1^3}{12} = 1344.10 \cdot \frac{1000 \cdot 37^3}{12} = 5.67 \cdot 10^9 \frac{Nmm^2}{mm}$$

$$(GA)_{ef} = (G_{90,LVL-X} \cdot 1m \cdot t_1) \cdot \kappa = 27.5 \cdot 1000 \cdot 37 \cdot \frac{5}{6} = 8.48 \cdot 10^5\ N/m$$

$$w_{1,creep} = \frac{5 \cdot 1 \cdot l^4}{384 \cdot (EI)_{y,creep,ef}} + \frac{1 \cdot l^2}{8 \cdot (GA)_{creep,ef}}$$
$$= \frac{5 \cdot 1 \cdot 0.625^4}{384 \cdot (5.67 \cdot 10^9) \cdot 10^{-9}} + \frac{1 \cdot 0.625^2}{8 \cdot 847.92}$$
$$= 0{,}35\ mm + 0{,}06\ mm = 0{,}41\ mm/m$$

- Net final deflection $w_{,net,fin}$ due to the characteristic and quasi-permanent load combinations

$$w_{net,fin} = \left\{ w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{characteristic\ loading} + w_{1,creep} \cdot \underbrace{[g_{1,k} + g_{2,k} + \psi_2 \cdot q_{1,k}]}_{quasi\ permanent\ loading} \right\} \cdot b$$
$$= \left\{ 0.31 \cdot \underbrace{[0.45 + 1.60 + 2.00]}_{characteristic\ loading} + 0.41 \cdot \underbrace{[0.45 + 1.60 + 0.3 \cdot 2.00]}_{quasi\ permanent\ loading} \right\} \cdot 1.0 = 2.06mm$$
$$2.06mm < \frac{l}{200} = \frac{625}{200} = 3.13mm\ (\eta = 66\%)$$

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- Final deflection $w_{,fin}$ due to the characteristic + quasi-permanent load combinations

$$w_{fin} = \left\{ w_{1,inst} \cdot \underbrace{[g_{1,k} + g_{2,k} + q_{1,k}]}_{characteristic\ loading} + w_{1,creep} \cdot \underbrace{[g_{1,k} + g_{2,k} + \psi_2 \cdot q_{1,k}]}_{quasi\ permanent\ loading} \right\} \cdot b$$

$$= \left\{ 0.31 \cdot \underbrace{[0.45 + 1.60 + 2.00]}_{characteristic\ loading} + 0.41 \cdot \underbrace{[0.45 + 1.60 + 0.3 \cdot 2.00]}_{quasi\ permanent\ loading} \right\} \cdot 1.0 = 2.06mm < \frac{l}{150} = \frac{625}{150}$$

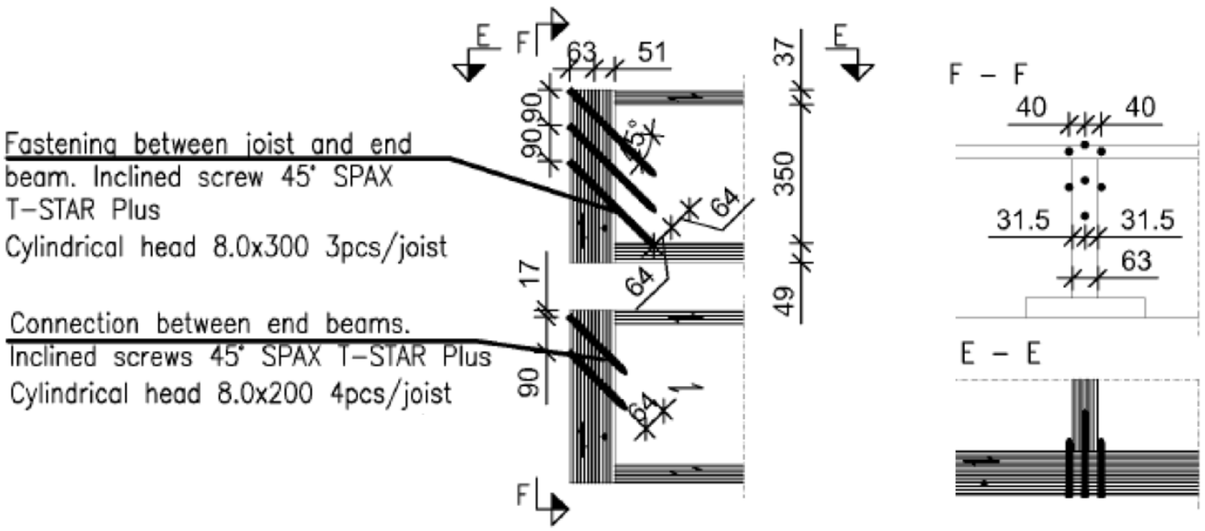
$$= 4.17mm\ (\eta = 49\%)$$

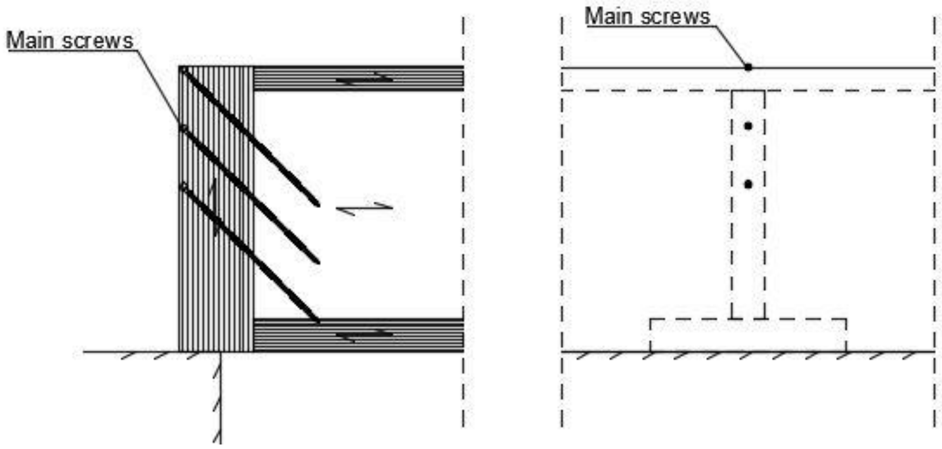
5. Screw connections

5.1 Design example- End beam support

These calculations show an example of a screw connection at I section of an end beam support.
See Figure 78 for I section beam support.

a) solid end beam





b) combined end beam

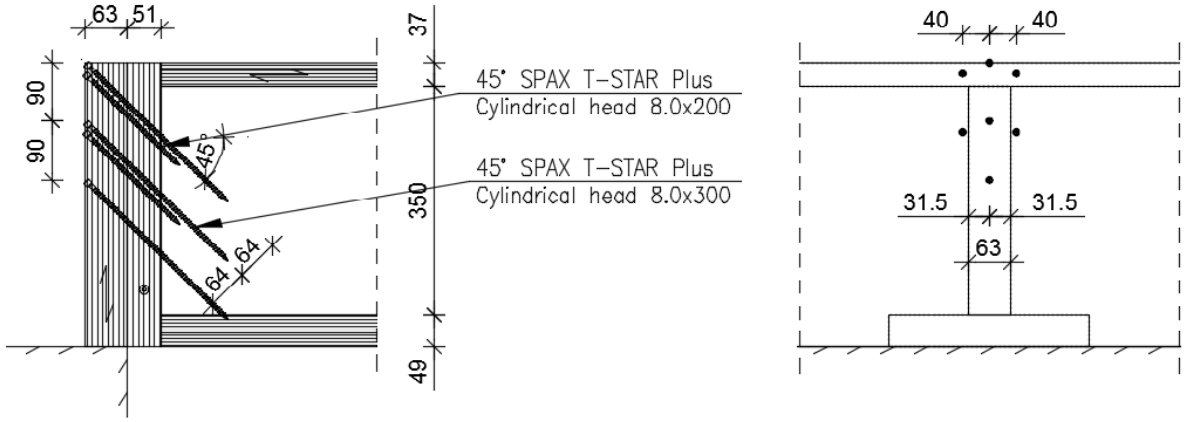


Figure 78: Illustration of an I section end beam support.

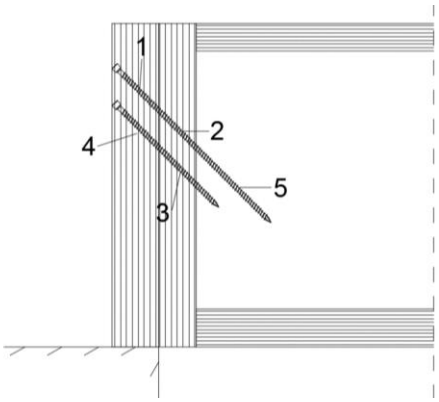
The design shear force for medium term action is

$$V_{z,d} = \frac{q_d.L}{2} = \frac{3.60.7.12}{2} = 12.83\ kN \text{ for I section}$$

In the example the chosen 63mm rib should be increased because of the a_4 edge distance which is 2*4*8mm=64mm.
In this case, there are two solutions:
- Switch the rib thickness from LVL-S 63mm to LVL-S 69mm
- Screw diameter for the screws which are in the rib can be decreased to 6mm.

LVL-S 63mm has been used in the design example for the calculation method, but to fulfill the edge distance requirement, one of the previous solutions has to be chosen.

The example is calculated with characteristic screw values in Table 17



$$f_{tens,k} = 17kN$$

$$f_{ax,1,k} = 17.1\ \frac{N}{mm^2} \qquad l_{g,1} = 63\sqrt{2}mm$$

$$f_{ax,2,k} = 17.9\ \frac{N}{mm^2} \qquad l_{g,2} = 51\sqrt{2}mm$$

$$f_{ax,5,k} = 12.6\ \frac{N}{mm^2} \qquad l_{g,5} = \left(300 - (63+51)\sqrt{2}\right)mm = 139mm$$

for a screw $d = 8mm$

- Design withdrawal capacity of main screw in outer part of the end beam

$$R_{T,Rd,1} = min \begin{cases} \frac{k_{mod}}{\gamma_M} \cdot \left(f_{ax,1,k} \cdot \left(\frac{8d}{l_{g,1}} \right)^{0.2} \cdot d \cdot l_{g,1} + f_{head,k} \cdot d_h^2 \right) \\ \frac{f_{tens,k}}{\gamma_{M2}} \end{cases} = min \begin{cases} 7.02\ kN \\ 13.6\ kN \end{cases}$$

where $k_{mod} = 0.8$, $\gamma_M = 1.3$, $\gamma_{M2} = 1.25$ and $f_{head,k} d_h^2$ is neglected.

- Design withdrawal capacity of main screw in inner part of the end beam

$$R_{T,Rd,2} = min \begin{cases} \frac{k_{mod}}{\gamma_M} \left(f_{ax,2,k} \left(\frac{8d}{l_{g,2}} \right)^{0.2} dl_{g,2} \right) \\ \frac{f_{tens,k}}{\gamma_{M2}} \end{cases} = min \begin{cases} 6.21\ kN \\ 13.6\ kN \end{cases}$$

- Design withdrawal capacity of main screw in the rib

$$R_{T,Rd,5} = min \begin{cases} \frac{k_{mod}}{\gamma_M} \left(f_{ax,5,k} \left(\frac{8d}{l_{g,5}} \right)^{0.2} dl_{g,5} \right) \\ \frac{f_{tens,k}}{\gamma_{M2}} \end{cases} = min \begin{cases} 7.40\ kN \\ 13.6\ kN \end{cases}$$

- Design withdrawal capacity of additional screw in outer part of the end beam

$$R_{T,Rd,add,4} = R_{T,Rd,1} = 7.02\ kN$$

- Design withdrawal capacity of additional screw in inner part of the end beam

$$R_{T,Rd,add,3} = R_{T,Rd,2} = 6.21\ kN$$

- **The design capacity of main screws in the rib may be calculated as**

$$R_{d,5} = n_{main}^{0.9} R_{T,Rd,5} (\cos\alpha + \mu \sin\alpha) = 3^{0.9} \cdot 7.40 \cdot (\cos 45 + 0.26 \cdot \sin 45) = 17.66\ kN$$

where $n_{main} = 3$ is the number of main screws, and $\mu = 0.26$ and $\alpha = 45°$.

- **The design capacity of main screws in end beam may be calculated as**

$$R_{d,12} = n_{main}^{0.9} (R_{T,Rd,1} + R_{T,Rd,2}) (\cos\alpha + \mu \sin\alpha) = 3^{0.9} \cdot (7.02 + 6.21\ ) \cdot (\cos 45 + 0.26 \cdot \sin 45) = 31.67\ kN$$

In the connection between the end beam and the rib

$$V_{E,d} = 11.50\ kN \le R_d = min \begin{cases} R_{d,5} \\ R_{d,12} \end{cases} = min \begin{cases} 17.66\ kN \\ 31.67\ kN \end{cases} = 17.66\ kN$$

where V_{Ed} is the shear force in the rib.

$V_{E,d} \le R_d$ **O.K. ✓**

- **The design capacity of all screws in outer part of the end beam**

$$R_{d,14} = (n_{main} + n_{add})^{0.9} \cdot R_{T,Rd,1} \cdot (\cos\alpha + \mu \sin\alpha) = (3 + 4)^{0.9} \cdot 7.02 \cdot (\cos 45 + 0.26 \cdot \sin 45) = 36.04\ kN$$

where $n_{add} = 4$ is the number of additional screws. See Figure 78 for screw instructions.

- **The design capacity of all screws in inner part of the end beam**

$$R_{d,23} = (n_{main} + n_{add})^{0.9} \cdot R_{T,Rd,2} \cdot (\cos\alpha + \mu \sin\alpha) = (3 + 4)^{0.9} \cdot 6.21 \cdot (\cos 45 + 0.26 \cdot \sin 45) = 31.86\ kN$$

In the connection between the parts of the end beam

$$V_{Ed} = 11.50\ kN \le R_d = min \begin{cases} R_{d,14} \\ R_{d,23} \end{cases} = min \begin{cases} 36.04\ kN \\ 31.86\ kN \end{cases} = 31.86\ kN$$

$V_{Ed} \le R_d$ **O.K. ✓**

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Because the lowest main screw practically extends to the bottom of the rib there is no need to check shear in the end of rib.

5.2 Design example- Suspended support

The rib width was modified for the example

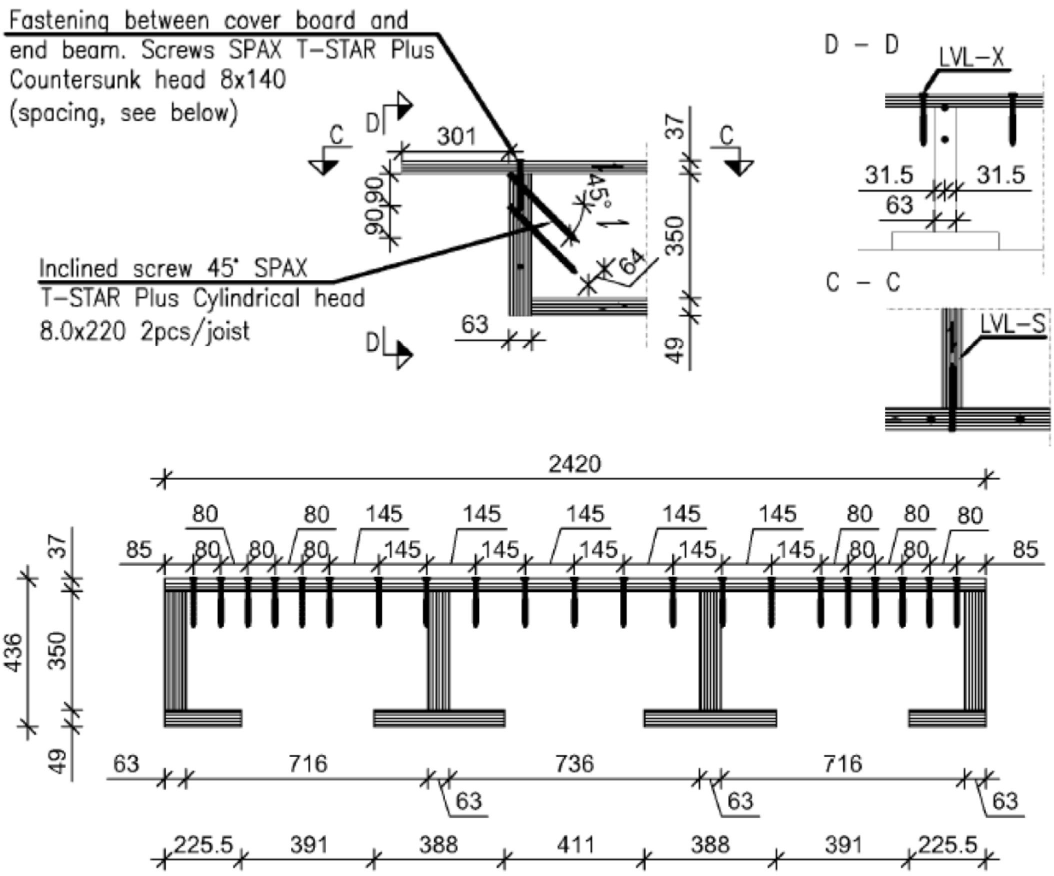

Figure 79: I section beam support and cross-section showing the position of suspension screws.

5.2.1 Slab bending

The bending stress in slab shall satisfy the following expression

$\sigma_d \leq f_{m,0,flat,d}$

$$q_d = \gamma_G g_k + \gamma_Q q_k = 1.35 \cdot (1.6 + 0.45)\,\frac{kN}{m^2} + 1.5 \cdot 2.0\,\frac{kN}{m^2} = 5.78\ kN/m^2$$

$$q_d = 5.78 \cdot 2.42 = 13.99\ kN/m$$

Maximum shear force at the suspended support

$$V_{z,d} = \frac{q_d . L}{2} = \frac{13.99 \cdot\ 7.12}{2} = 49.80\ kN$$

is the shear force for the whole open box slab

$L_s = 231mm$ is length of support

$l_{gap} = 70mm$ is length of gap between end beam and support

$B_s = 63mm$ is thickness of the end beam

Bending moment

$$M_{Ed,slab} = V_{Ed}\left(\frac{L_s}{2} + l_{gap} + \frac{B_s}{2}\right) = 49.80 \cdot \left(\frac{231}{2} + 70 + \frac{63}{2}\right) = 10.81\ kNm$$

Bending resistance

$$W = \frac{B \cdot t_1^2}{6} = \frac{2420 \cdot 37^2}{6} = 5.52 \cdot 10^5 mm^3$$

(The end beam needs to be longer than the width of the slab so that the minimum distance 12*d* to the end of the end beam is fulfilled.)

Bending stress may be calculated as

$$\sigma_d = \frac{M_{Ed,slab}}{W} = \frac{10.81 \cdot 10^6}{5.52 \cdot 10^5} = 19.60\ MPa$$

Bending strength

$$f_{m(LVL-X),0,flat,d} = k_{mod} \cdot \frac{f_{m,0,flat,k}}{\gamma_M} = 0.8 \cdot \frac{36}{1.2} = 24 MPa$$

$\sigma_d < f_{m,0,flat,d} \quad (\eta = 82\%)$

O.K. ✓. Bending capacity can further be increased by reducing the length of the support

5.2.2 Slab shear

The shear stress should satisfy the following expression

$\tau_d \leq f_{v(LVL-X),0,flat,d}$

Shear stress may be calculated as

$$\tau_d = \frac{3}{2}\frac{V_{Ed}}{B t_1} = \frac{3}{2} \cdot \frac{49.80 \cdot 10^3}{2420 \cdot 37} = 0.835\ MPa$$

Shear strength (increased strength see chapter 10.4)

$$f_{v(LVL-X),0,flat,d} = \frac{k_{mod} \cdot f_{v(LVL-X),0,flat,k}}{\gamma_M} = \frac{0.8 \cdot 2.3}{1.2} = 1.53 MPa$$

$\tau_d < f_{v(LVL-X),0,flat,d} \qquad (\eta = 53.9\%)$ **O.K. ✓**

when the shear force in one I section is

$$V_{I.Ed} = \frac{63 + \frac{716 + 736}{2}}{2420} \cdot 49.80\ kN = 16.24\ kN$$

5.2.3 Inclined screw connection

The calculations for screws are shown only for I section. The same principles are used for U section.

Design withdrawal capacity of one screw

$$R_{T,Rd} = min \begin{cases} \frac{k_{mod}}{\gamma_M} \cdot \left(f_{ax,1,k} \cdot \left(\frac{8d}{l_{g,1}}\right)^{0.2} \cdot d \cdot l_{g,1} + f_{head,k} \cdot d_h^2\right) \\ \frac{k_{mod}}{\gamma_M} \cdot \left(f_{ax,2,k} \cdot \left(\frac{8d}{l_{g,2}}\right)^{0.2} \cdot d \cdot l_{g,2}\right) \\ \frac{f_{tens,k}}{\gamma_{M2}} \end{cases} = min \begin{cases} 7.02\ kN \\ 704\ kN \\ 13.6\ kN \end{cases}$$

where $k_{mod} = 0.8$, $\gamma_M = 1.3$, $\gamma_{M2} = 1.25$ and $f_{head,k} d_h^2$ is neglected.

$f_{tens,k} = 17kN$
$f_{ax,1,k} = 17.9 \frac{N}{mm^2}$ $\quad l_{g,1} = 63\sqrt{2}mm$
$f_{ax,2,k} = 12.6 \frac{N}{mm^2}$ $\quad l_{g,2} = (220 - 63\sqrt{2})mm = 131mm$
for a screw $d = 8mm$

Design capacity of tension screw connection
$R_{T,d} = n^{0.9} R_{T,Rd}(\cos\alpha + \mu\sin\alpha) = 2^{0.9} \cdot 7.02 \cdot (\cos 45 + 0.26 \cdot \sin 45) = 11.67\ kN$
$n = 2$ is the number of screws

The design equation is
$V_{I.Ed} = 16.24\ kN \leq R_d = 11.67\ kN$

Is not fulfilled, the screw connection is too weak.

Proposal: increase the number of inclined screws per rib from 2 to 3, then the design will be **ok✓**

5.2.4 Suspension screws

One can estimate that $n = 5.5$ suspension screws are acting for one I section.

Design withdrawal or one screw

$$R_{S,Rd} = min \begin{cases} \frac{k_{mod}}{\gamma_M} \cdot f_{head,k} \cdot d_h^2 \\ \frac{k_{mod}}{\gamma_M} \cdot \left(f_{ax,k} \cdot \left(\frac{8d}{l_g}\right)^{0.2} \cdot d \cdot l_g \right) \\ \frac{f_{tens,k}}{\gamma_{M2}} \end{cases} = min \begin{cases} 2.54\ kN \\ 4.33\ kN \\ 13.60\ kN \end{cases}$$

where $k_{mod} = 0.8$, $\gamma_M = 1.3$, $\gamma_{M2} = 1.25$

$f_{tens,k} = 17kN$
$f_{ax,k} = 11.5 \frac{N}{mm^2}$ $\quad l_g = 80mm$ (partially thread screw)
$f_{head,k} = 18.1 \frac{N}{mm^2}$
for a screw $d = 8mm$ and $d_h = 15.1mm$

Design capacity of suspended screw connection
$R_{S.d} = n^{0.9} \cdot R_{S,Rd} = 5.5^{0.9} \cdot 2.54 = 11.78\ kN$
$n = 5.5$ is the number of screws
The design equation is
$V_{I.Ed} = 16.24\ kN \ \leq R_d = 11.78\ kN$
In the current setup, the resistance is too low.
With 8 screws (meaning a screw spacing of 80mm) instead of 5.5, $R_{S.d} = n^{0.9} \cdot R_{S,Rd} = 8^{0.9} \cdot 2.54 = 16.51\ kN$: it would be fullfilled.

Proposal: reduce the screw spacing to accommodate more screws, then the design will be **ok✓**

5.2.5 Shear in the rib

If screws do not reach the bottom of the rib connection, it should be calculated as a beam with a notch at the support.
Shear stress shall satisfy the following expression

$$\tau_d = \frac{3}{2}\frac{V_{I.Ed}}{b_2 h_{ef}} \leq k_v \cdot f_{v,d}$$

where the shear for one I section $V_{I.Ed} = 16.24\ kN$, and $h_{ef} = \left(\frac{220}{\sqrt{2}} + 64\sqrt{2}\right) mm = 246mm$ is calculated from the connection geometry.

$$\tau_d = \frac{3}{2}\frac{V_{I.Ed}}{b_2 h_{ef}} = \frac{3}{2}\frac{\cdot 16.24 \cdot 10^3}{63 \cdot 246} = 1.57\ MPa$$

when $b_{2\,rib} = 63mm$.

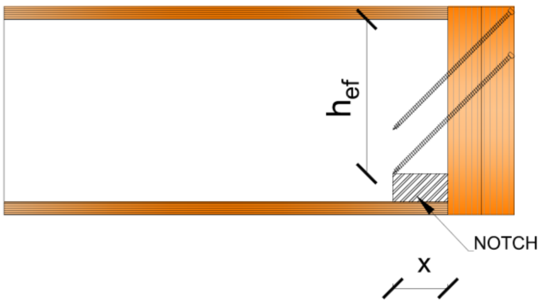

Figure 80: Determination of distances x and h_{ef} (should be adapted to the suspended support example)

$$k_v = min\begin{cases} 1 \\ \dfrac{k_n}{\sqrt{h_2}\left(\sqrt{\alpha(1-\alpha)} + 0.8\dfrac{x}{h_2}\sqrt{\dfrac{1}{\alpha} - \alpha^2}\right)} \end{cases} = min\begin{cases} 1 \\ 0.364 \end{cases}$$

where $h_2 = 350mm$ is the height of the rib (in mm)

$$k_n = 4.5$$

$$\alpha = \frac{h_{ef}}{h_2} = \frac{246.05}{350} = 0.703$$

x is the distance from point of the screw to the end of rib (in mm), $x = \frac{220-63\sqrt{2}}{\sqrt{2}} = 92.6mm$

Shear strength

$$f_{v,d} = k_{mod}\frac{f_{(LVL-S),v.k}}{\gamma_M} = 0.8 \cdot \frac{4.1}{1.2} = 2.73\frac{N}{mm^2}$$

Design strength of shear

$$k_v \cdot f_{v,d} = 0.364 \cdot 2.73\frac{N}{mm^2} = 0.995MPa$$

The design equation is

$$\tau_d > k_v f_{v,d}$$

$1.57MPa > 0.995MPa$ Connection is too weak.

Proposal: Shear capacity can be further increased by lowering the screws, or by increasing the rib width, then design would be **OK✓**
The rib spacing could also be reduces in order to reduce the shear force carried by one section.

13. References

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