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II – DRAWINGS

1. Foundation reinforcement drawing

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I - TEXTUAL PART

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1 Standards

European norms (EN):

- EN 1990:2002, Basis of structural design
- EN 1991-1-1:2002, Actions on structures, Part 1-1: General actions – Densities, self-weight, imposed loads for buildings
- EN 1991-1-3:2003, Actions on structures, Part 1-3: General actions – Snow loads
- EN 1991-1-4:2005, Actions on structures, Part 1-: General actions – Wind actions
- EN 1992-1-1-2004 General rules and rules for buildings
- EN 1993-1-1-2005 General rules and rules for buildings
- EN 1995-1-1-2004 General – Common rules and rules for building

2 Primary data

2.1 Product info

Structure is shown in the picture below:

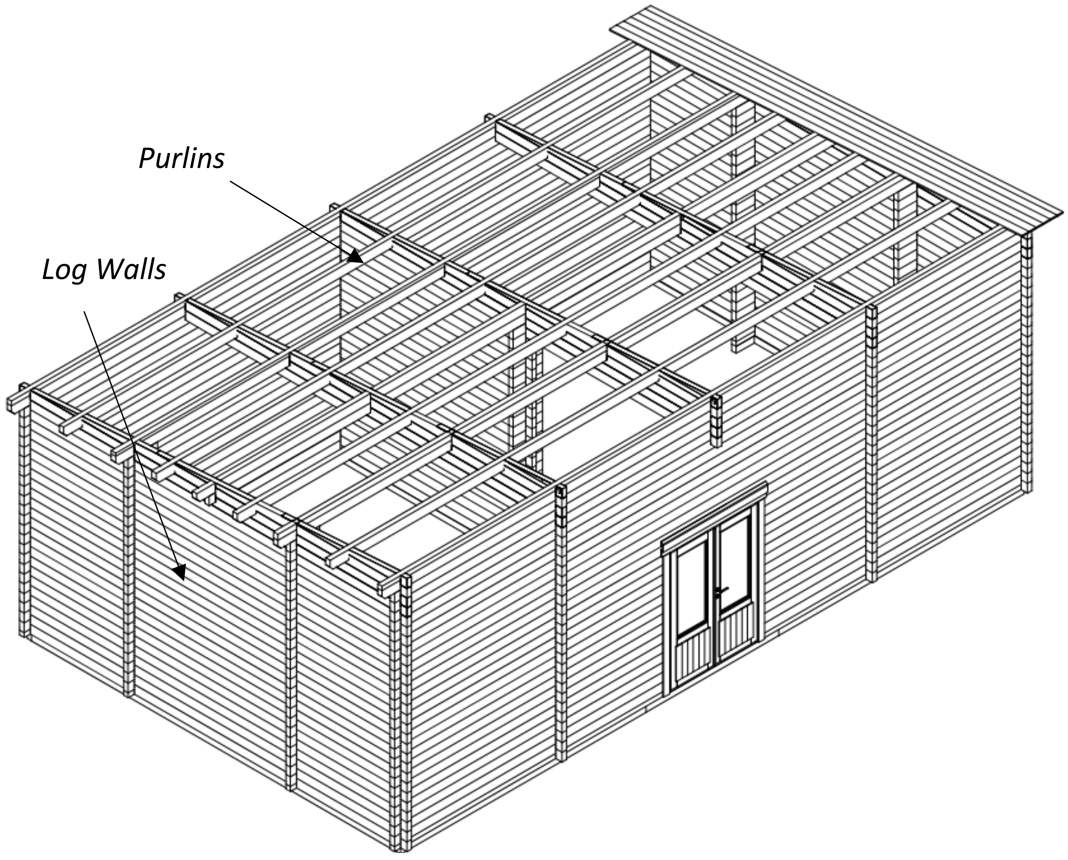

Fig. 1: Special todhunter 620x1020 70

Location info:

Penwortham – United Kingdom
Geographic latitude: 53.750° N
Geographic longitude: -2.720° W
Elevation: <50m

Wooden Purlin info:

Cross-section width (b)	70	mm
Cross-section height (h)	145	mm
Strength class of timber	C24	

Wooden Wall info:

Cross-section width (b)	70	mm
Strength class of timber	C24	

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3 Material properties

3.1 Normative properties of the material

Wood

Bending strength	$f_{m,k}$	24	N/mm²
Shear strength	$f_{v,k}$	2.5	N/mm²
Compressive strength	$f_{c,90,k}$	2.5	N/mm²
Average modulus of elasticity of longitudinal fibers	$E_{m,0,mean}$	11000	N/mm²
5% value of the modulus of elasticity in longitudinal section	$E_{m,0.5,k}$	7400	N/mm²

Concrete C25/30

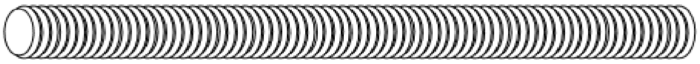
Compression strength	$f_{c,k}$	25	N/mm²
Average modulus of elasticity of longitudinal fibers	E_{cm}	31476	N/mm²

Reinforcing steel B500B

Tensile strength	$f_{y,k}$	500	N/mm²
Average modulus of elasticity of longitudinal fibers	E_s	200000	N/mm²

Steel reinforcing elements:

The element bearing characteristitcs are threaded bar 8.8.



KLi – Threaded Rod M12 – Tie element

The element bearing characteristitcs are threaded bar 8.8.

Normal strength	$f_{y,k}$	640	N/mm²
Yeald strength	$f_{u,k}$	800	N/mm²

UVi – Metal bar M12 – Shear element

The element bearing characteristitcs are steel grade S235.

Normal strength	$f_{y,k}$	235	N/mm²
Shear strength	$f_{v,k}$	135	N/mm²
Average modulus of elasticity of longitudinal fibers	$E_{m,0,mean}$	200000	N/mm²

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3.2 Partial Safety Factors

Wood

Load – duration class	Short term	
Service class	2	
Terrain factor	2	
Partial safety factor of the material	γ_m	1.3
Modification factor	k_{mod}	0.9
Cross – sectional factor	k_h	1.00
System strength factor	k_{sys}	1.1
Fraction factor	k_{cr}	0.67
Auxiliary factor	$k_{c,90}$	1

Concrete

Partial safety factor of the material	γ_m	1.50

Reinforcing steel B500B

Partial safety factor of the material	γ_s	1.15

Structural steel S235

General resistance of cross section	γ_{M0}	1.00
Global stability of element	γ_{M1}	1.00
Conection verifications	γ_{M2}	1.25

3.3 Calculated properties of the material

Wood

Bending strength: $f_{md}=\cos(\beta)\cdot((k_{mod}\cdot k_h\cdot k_{sys}\cdot f_{mk})/\gamma_m)$	$f_{m,d}$	18.28	N/mm²
Shearing strength: $f_{vd}=(k_{mod}\cdot k_{sys}\cdot f_{vk})/\gamma_m$	$f_{v,d}$	1.90	N/mm²
Compressive strength: $f_{c,90,d}=(k_{mod}\cdot k_{sys}\cdot f_{c,90,k})/\gamma_m$	$f_{c,90,d}$	1.90	N/mm²

Concrete C25/30

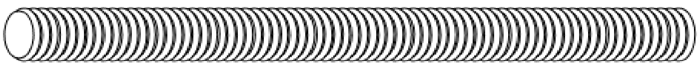
Design strength	$f_{c,d}$	16.5	N/mm²

Reinforcing steel B500B

Design Tensile strength	$f_{y,d}$	435	N/mm²

Steel reinforcing elements:

The element bearing characteristitcs are threaded bar 8.8.



KLi – Threaded Rod M12 – Tie element

The element bearing characteristitcs are threaded bar 8.8.

Normal strength	$f_{y,d}$	512	N/mm²
Yeald strength	$f_{u,d}$	800	N/mm²

UVi – Metal bar M12 – Shear element

The element bearing characteristitcs are steel grade S235.

Normal strength	$f_{y,d}$	188	N/mm²
Shear strength	$f_{v,d}$	135	N/mm²

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4 Modeling

4.1 Model informations

The timber structure was modeled using a spatial FEM (Finite Element Method) computational model. To ensure the model accurately represents the structural behavior and incorporates all relevant characteristics of the timber system, the engineer developed a set of specialized element configurations. The figure below illustrates the computational model of the structure.

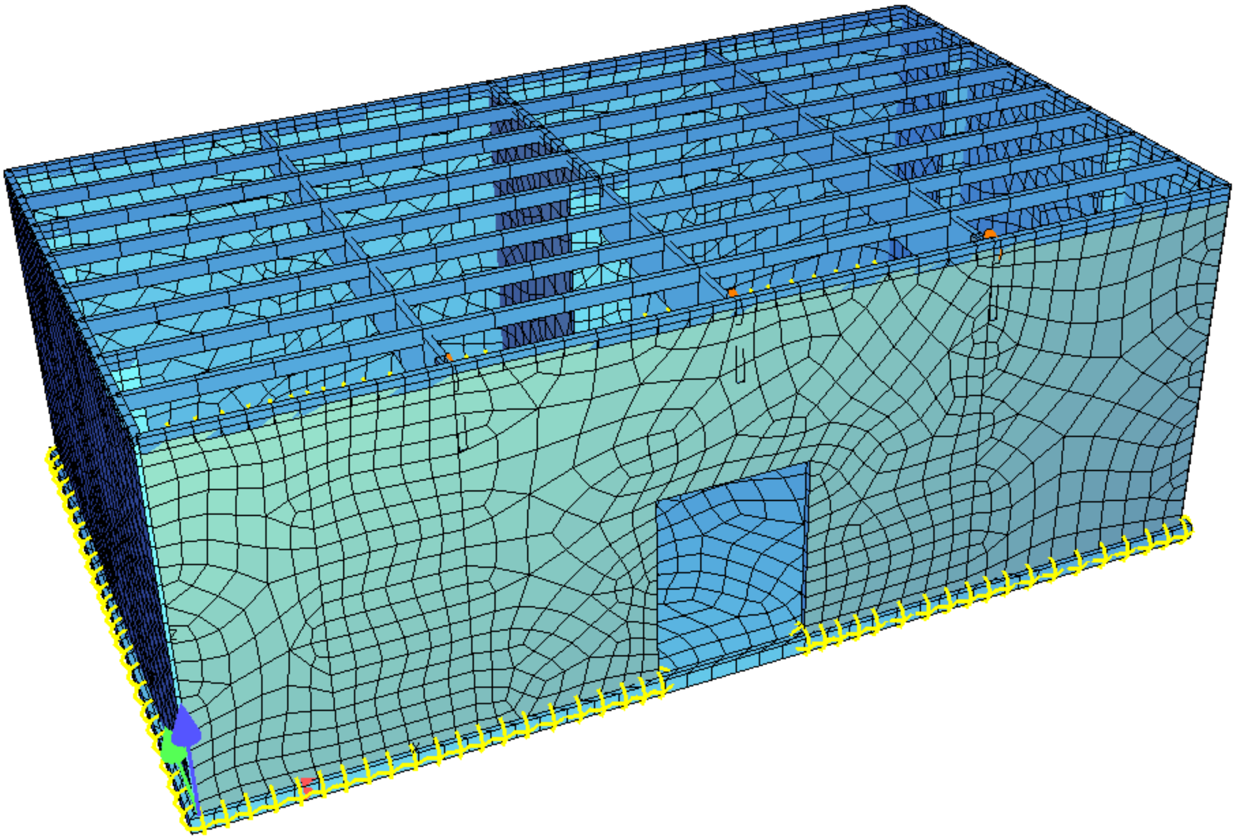

Fig. 2: Structural model of the building

The structure is divided into the following groups based on the load transfer mechanism:

- (1) Beam elements (purlins)
- (2) Wall diaphragms that ensure the global stability of the structure and resist lateral loads
- (3) Walls designed to resist only lateral loads
- (4) Threaded rod M12 – Grade 8.8 – Tie element
- (5) Metal bar M12 - S235 – Shear element
- (6) Foundation (concrete slab)

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The following figure shows the elements according to their load-bearing function.

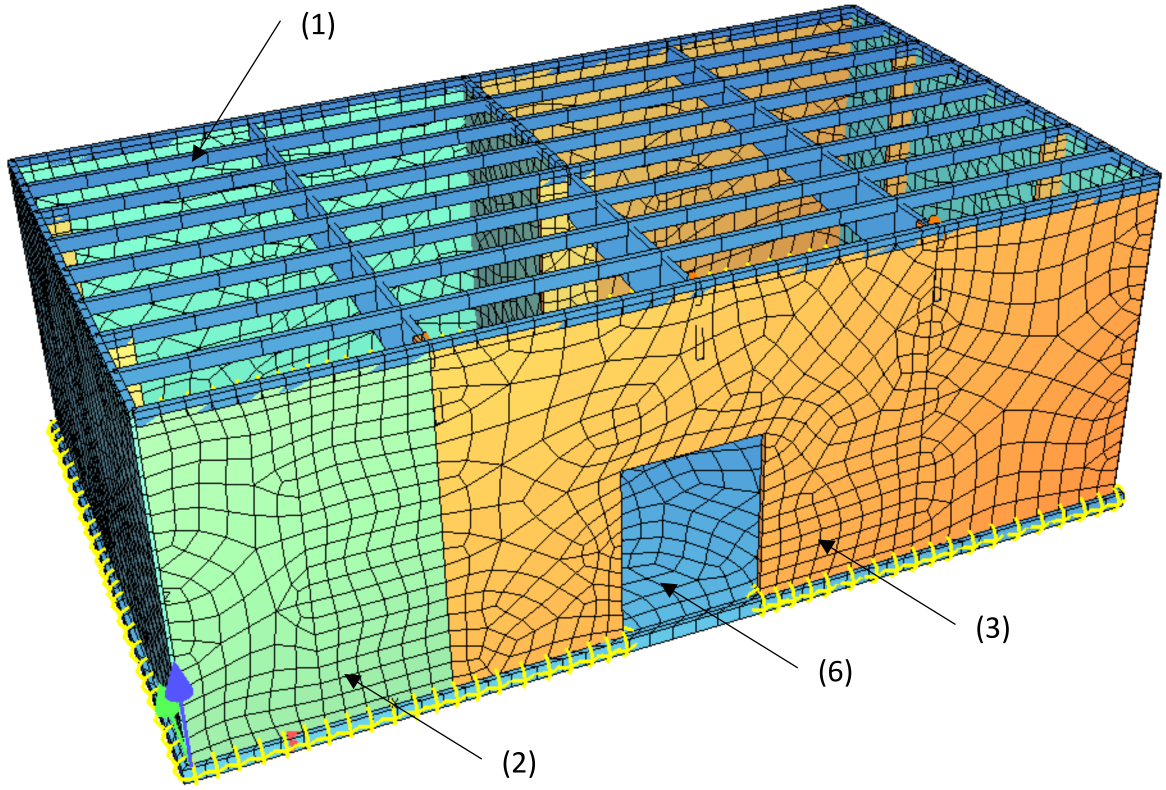

Fig. 3: Model groups

The walls (2) act as (bracings) shear walls, and in order to fully enable the load transfer mechanism, they include additional steel elements (4) and (5) that ensure continuity of force flow and maintain the structural integrity.

The shear walls are therefore converted into standard beam elements, and according to the well-known principles, the load-bearing capacity of the walls is then verified. Mathematically, the main wall is considered as an equivalent beam element for design, using the results from the shell elements.

$$M_y = \int m_{xx} dy$$

$$M_z = \int n_{xx} dy$$

.

.

etc.

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4.2 Purlins Layout

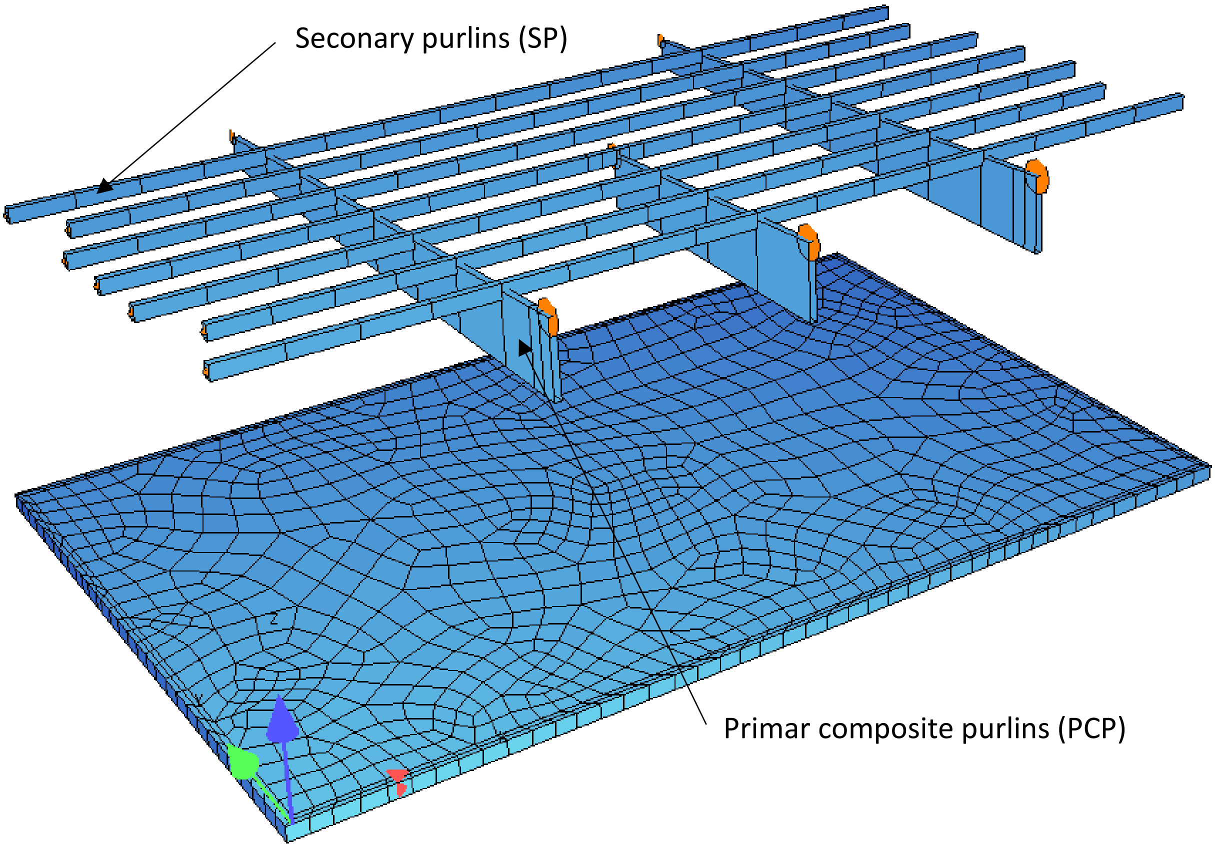

Fig. 4: Purlins layout

There are two sets of purlins for the design:

- Secondary purlins – 70/145mm wooden beam
- Primary composite purlins – composed of 4 to 6 beams (70/145), connected with a steel threaded rod to maintain their position and provide a girder-like structural response

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4.3 Wall design

Wall design verification

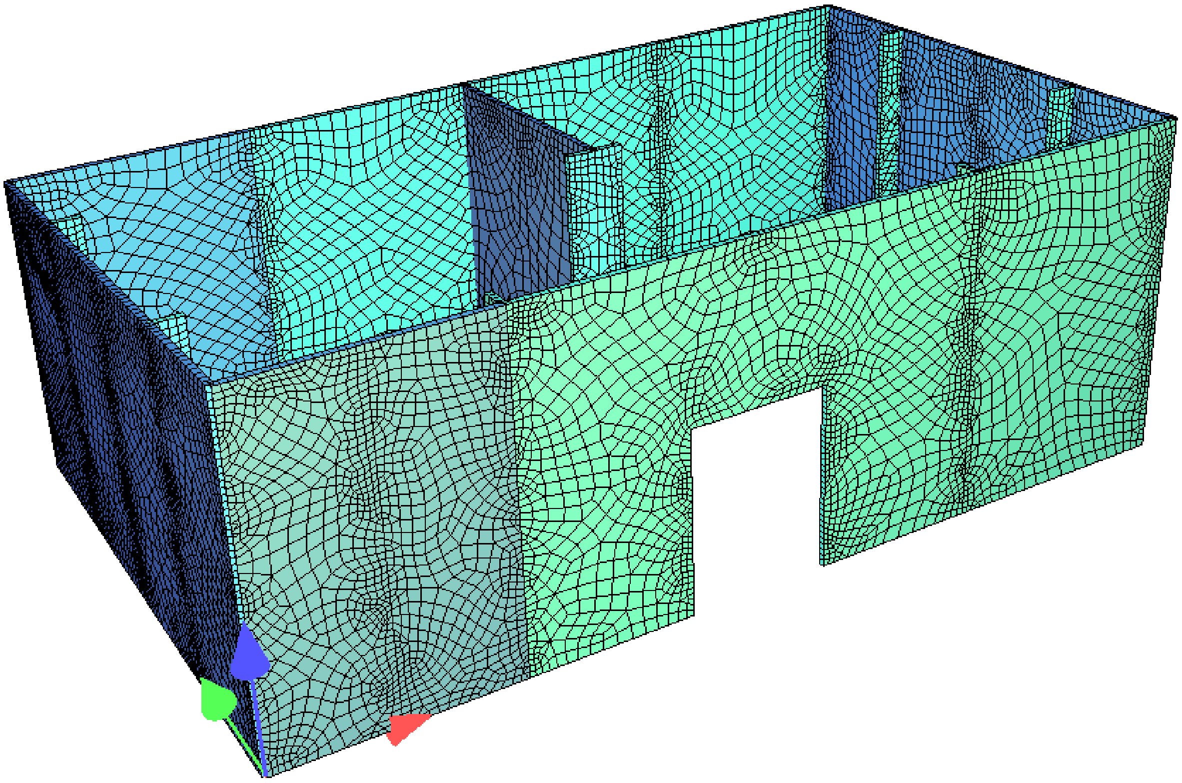

Fig. 5: Walls layout

Walls are designed solely to transfer vertical loads and wind loads acting perpendicular to their central plane. To ensure simplified structural behavior, the wall parameters are adjusted so that the axial in plane stiffness in the horizontal direction is approximately zero (i.e., $A_x \cdot E_x \approx 0$). This assumption reflects the lack of friction between the logs and represents a conservative (safe-side) design approach.

Verification of these walls is carried out by applying local vertical support reactions from the purlins and analyzing them for one-way bending and shear in the x-direction under direct wind loading.

To ensure the global stability of the structure, certain walls are equipped with additional steel elements that enable them to resist in-plane loads required for overall structural performance. These walls function as shear walls, and their verification is performed through a separate, dedicated check.

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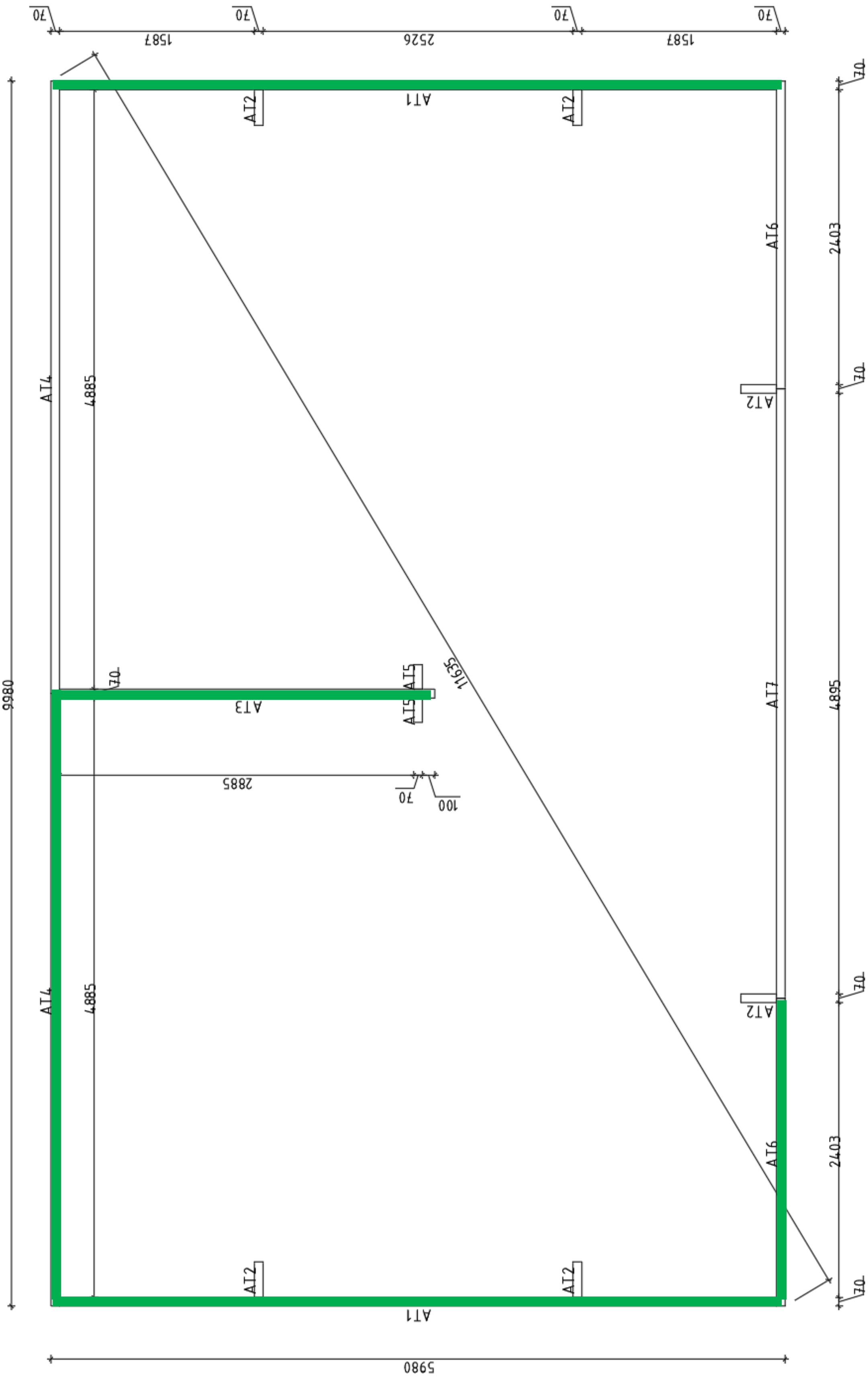
Shear Wall verification



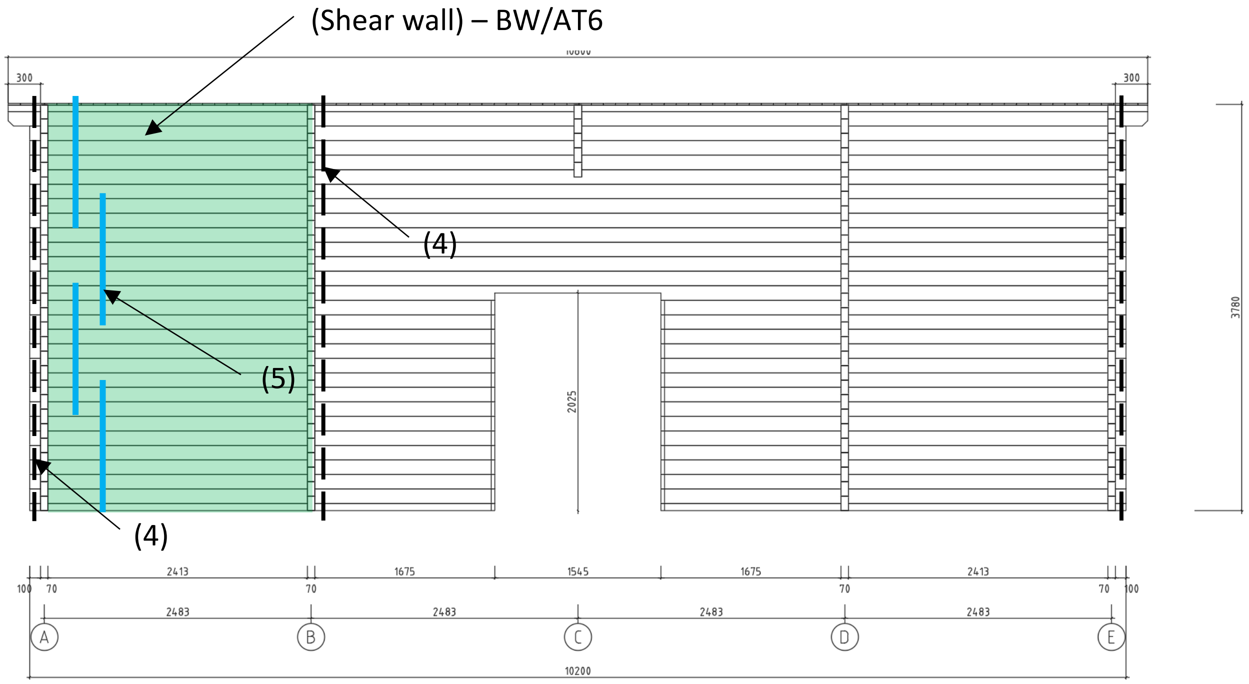


Fig. 6: Walls AT6 – AT7 – AT6

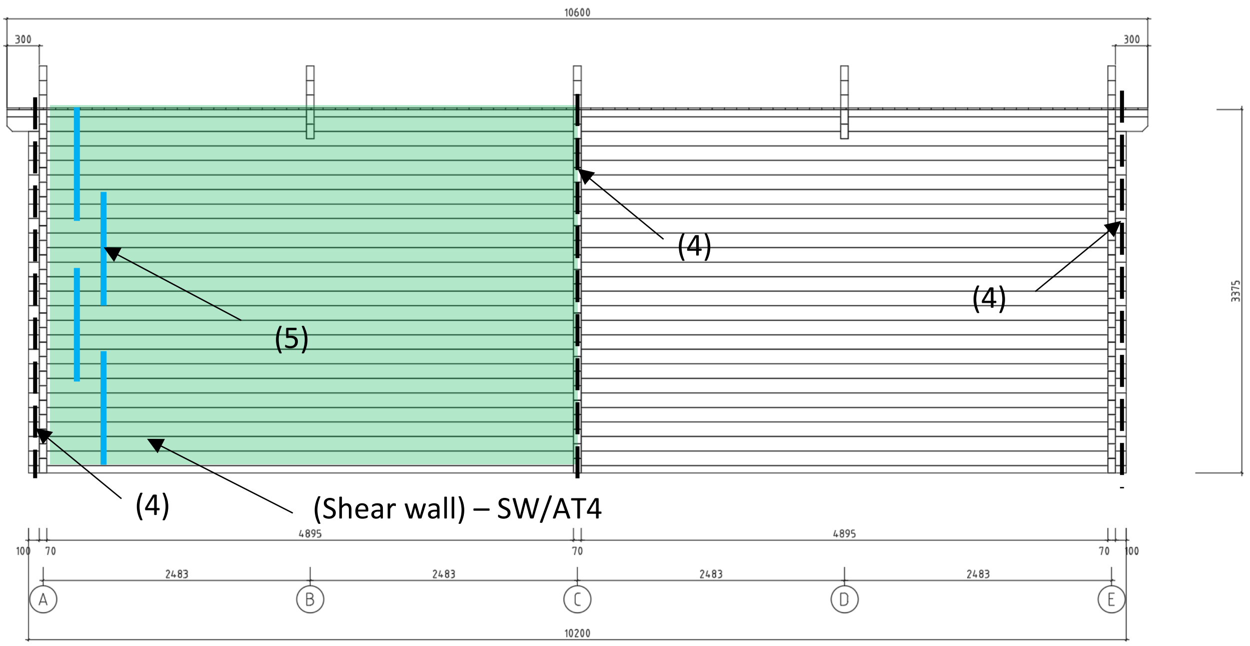


Fig. 7: Walls AT4 – AT4

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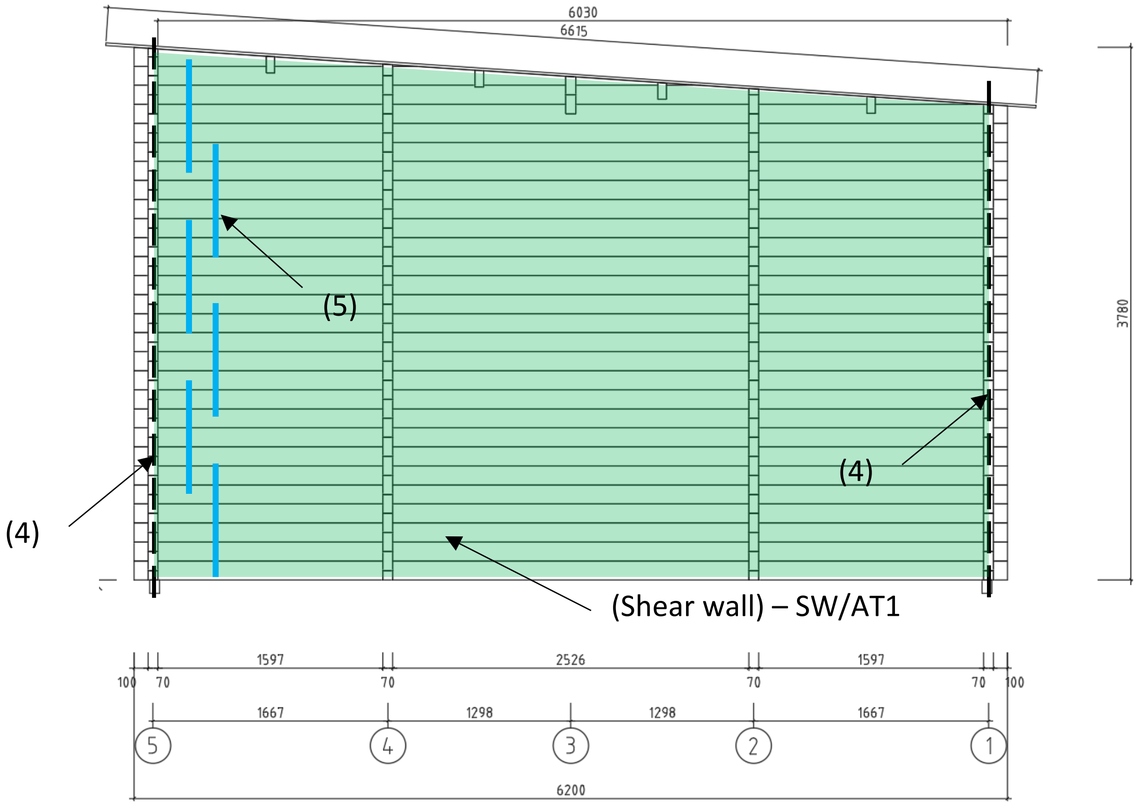

Fig. 8: Wall AT1 (on both sides)

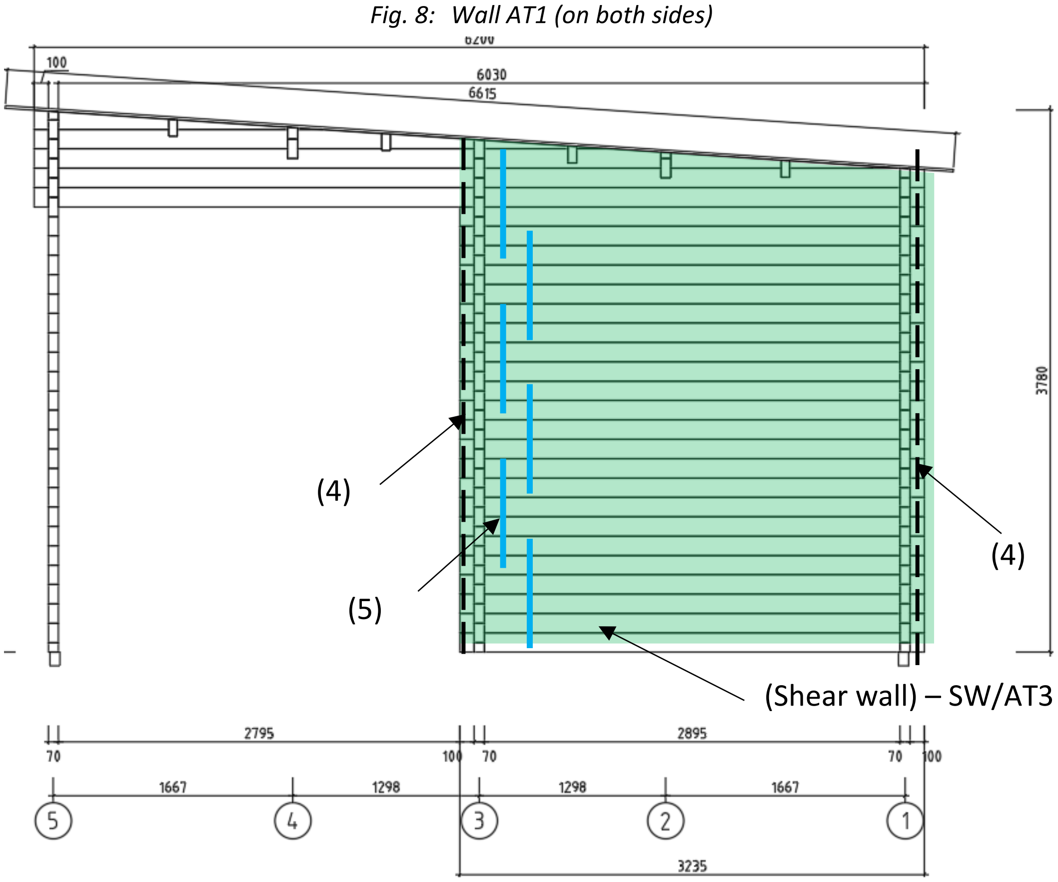

Fig. 9: Wall AT3 (on both sides)

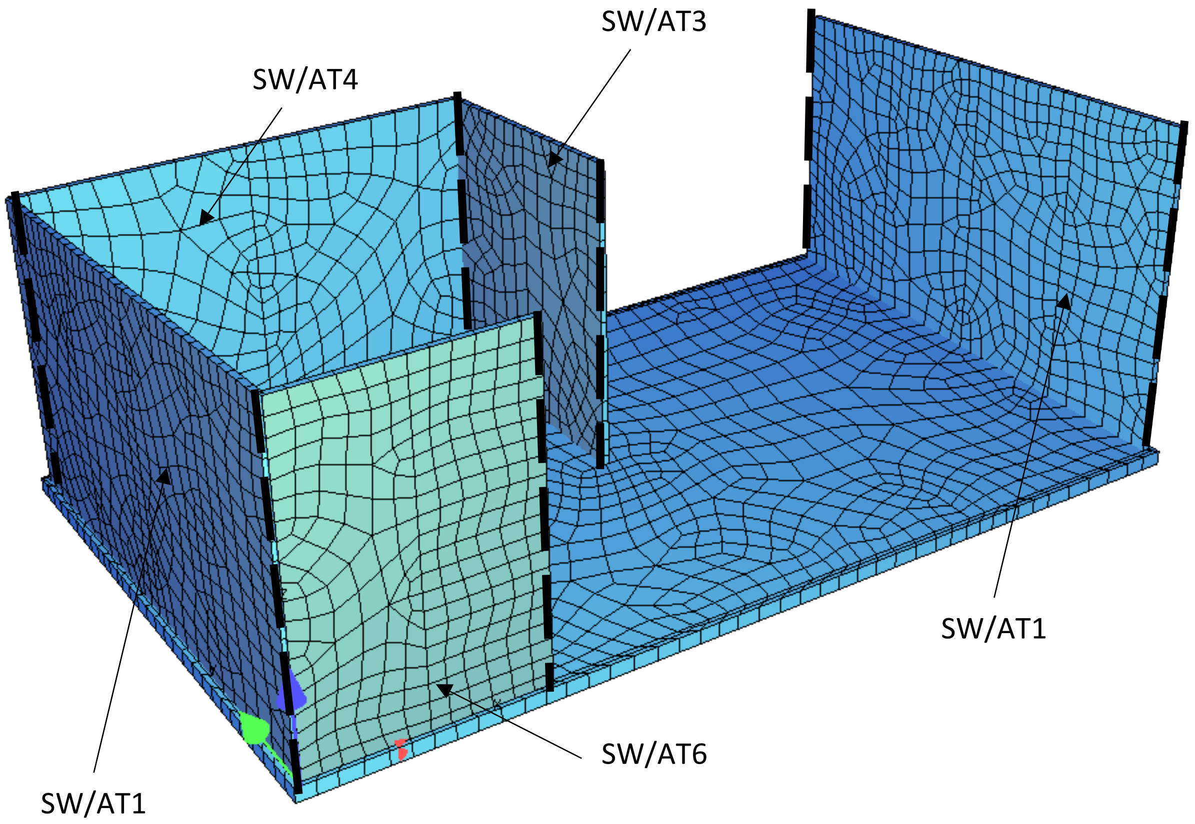

Fig. 10: Shear walls for special desing calculations

Threaded bars 7pcs.
SW/AT1 2pcs.
SW/AT3 1pcs.
SW/AT4 1pcs.
SW/AT6 1pcs.

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5 Loads

5.1 Load combination rules

For load combination rules the EN 1990 is applied. The standard verification combinations are listed below:

ULS DESIGN SITUATION

$$\sum_{j\geq 1} \gamma_{G,j} \cdot G_{k,j} \oplus \gamma_p \cdot P \oplus \gamma_{Q,1} \cdot Q_{k,1} \oplus \sum_{i>1} \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{k,i}$$

CHARACTERISTIC DESIGN SITUATION

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

FREQUENT DESIGN SITUATION

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

QUASI-PERMANENT DESIGN SITUATION

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

Where the factors are defined as:

$\oplus$ → "to be combined with"
G → Dead weight
P → Prestressing
Q_1 → Leading live action
Q_i → following live action

Design load combinations applied are as follows:

$1.35 \cdot g_k + 1.50 \cdot q_{snow,k}$ ⟶ Applied snow combinations
$1.00 \cdot g_k + 1.50 \cdot q_{wind,k,i}$ ⟶ Applied wind combinations

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Table A1.1 - Recommended values of ψ factors for buildings

Action	ψ_0	ψ_1	ψ_2
Imposed loads in buildings, category (see EN 1991-1-1)			
Category A : domestic, residential areas	0,7	0,5	0,3
Category B : office areas	0,7	0,5	0,3
Category C : congregation areas	0,7	0,7	0,6
Category D : shopping areas	0,7	0,7	0,6
Category E : storage areas	1,0	0,9	0,8
Category F : traffic area, vehicle weight ≤ 30kN	0,7	0,7	0,6
Category G : traffic area, 30kN < vehicle weight ≤ 160kN	0,7	0,5	0,3
Category H : roofs	0	0	0
Snow loads on buildings (see EN 1991-1-3)*			
Finland, Iceland, Norway, Sweden	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H > 1000 m a.s.l.	0,70	0,50	0,20
Remainder of CEN Member States, for sites located at altitude H ≤ 1000 m a.s.l.	0,50	0,20	0
Wind loads on buildings (see EN 1991-1-4)	0,6	0,2	0
Temperature (non-fire) in buildings (see EN 1991-1-5)	0,6	0,5	0
NOTE The ψ values may be set by the National annex. * For countries not mentioned below, see relevant local conditions.			

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5.2 Dead weight

Dead loads of self weight are automaticaly assumed in the software for the following parameteres:

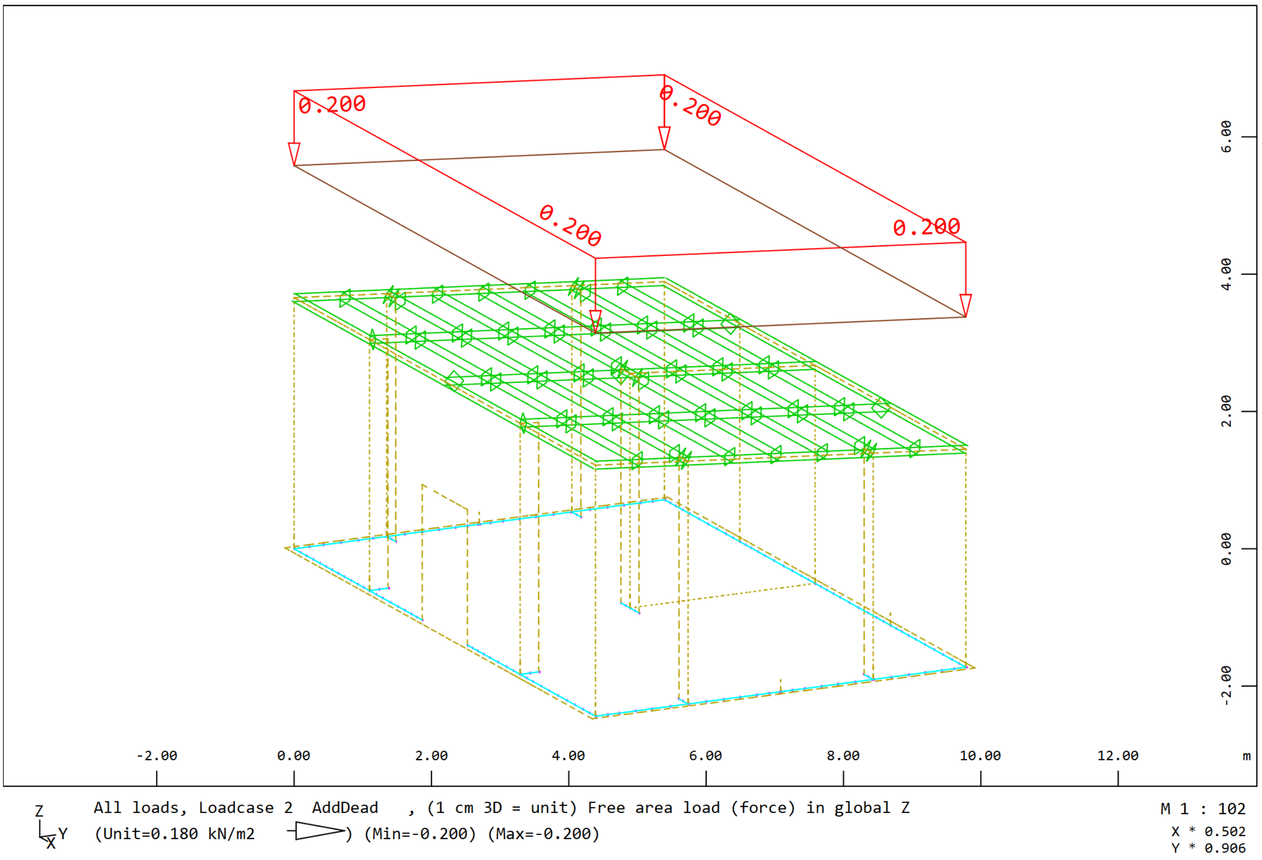
Self weight

Wood specific weight	γ_w	10	kN/m^3
Concrete specific weight	γ_c	25	kN/m^3

The additional dead load from the coverings is assumed to be:

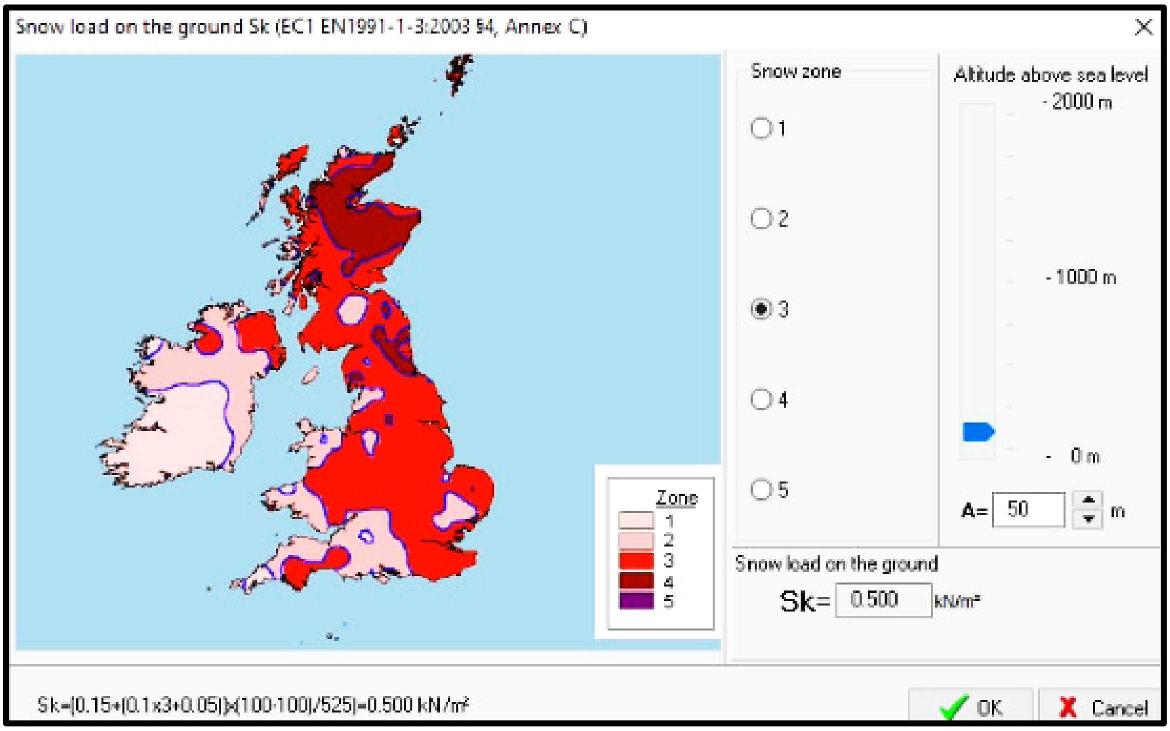
Additional dead weight g_{add}

Weight of the roof layers	g_{add}	0,20	kN/m^2



5.3 Snow loads

The snow load is taken from the national load map of UK:



s_k=0,50kN/m^2

The roof snow distribution is calculated from the following equation:

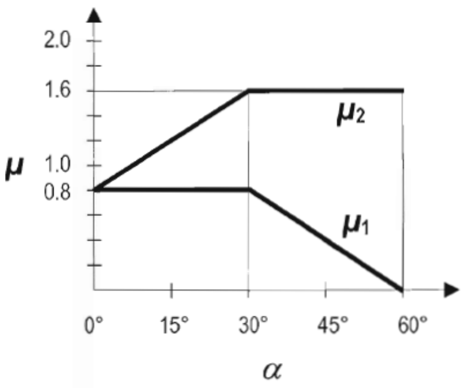
$$s=\mu_i \cdot C_e \cdot C_t \cdot s_k$$

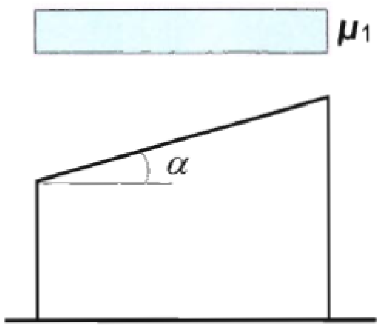
μ_i ⟶ Shape coeffitient

C_e⟶ Exposure coeffitient (1.00)

C_t⟶ Thermal coeffitient (1.00)

For the monopitch roof, the shape depends on the roof angle, and the coefficient is determined accordingly:



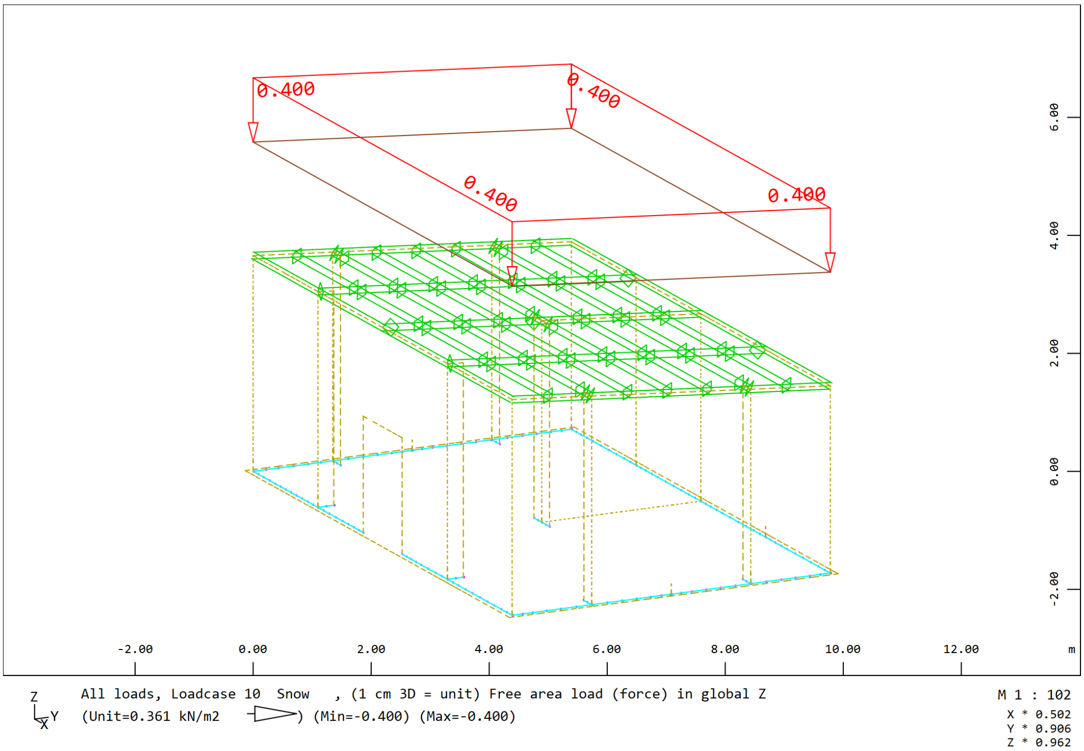


For α<30

μ_i =0.80

$$s=0.80 \cdot 1.00 \cdot 1.00 \cdot 0.50=0.40 kN/m^2$$

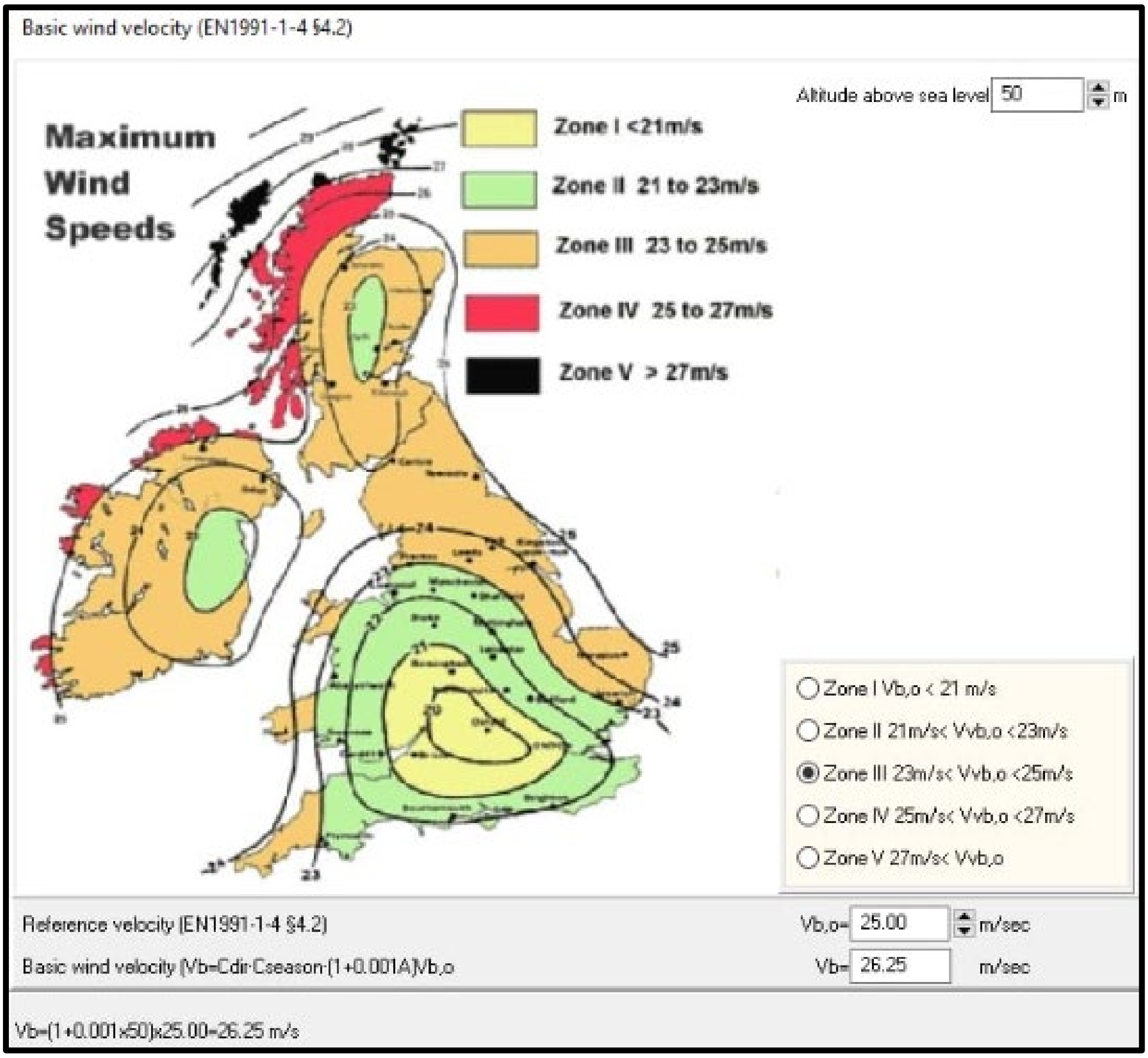
Applied snow load is shown below:



Snow load	s	0,40	kN/m^2

5.4 Wind loads

The wind velocity map is shown below:



$$v_b = 26.25 m/s$$

v_b ⟶ Basic wind velocity

Wind pressure varies with height and several other factors. In the following, the calculation formulas are presented along with the computed values.

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Basic wind pressure is obtaind by the following eq.

$$q_b=0.50\cdot\rho\cdot v_b^2$$

$q_b \longrightarrow$ Basic wind pressure
$\rho \longrightarrow$ Air density (1.25 kg/m^3)

Than expousure factor is obtained by:

$$c_e(z)= [1+7\cdot I_v(z)]$$

$$q_p(z)= c_e(z)\cdot q_b$$

$$I_v(z)=\frac{k_I}{c_0\cdot \ln(z/z_0)} \text{ for } z_{min}\leq z\leq z_{max}$$

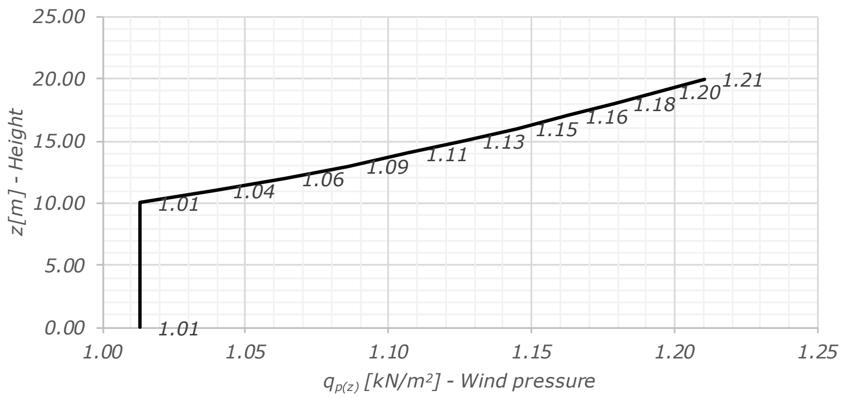
$$I_v(z)=I_v(z_{min}) \text{ for } z\leq z_{min}$$

$z_0 \longrightarrow$ Roughness length
$c_0 \longrightarrow$ orography factor
$k_I \longrightarrow$ Turbulence factor k_I=1.00

Wind pressure calculation EN 1991-1-4

$v_{b,0}$=	26.25	m/s	Basic wind velocity
c_{season}=	1.00	-	Season coefficient
c_{dir}=	1.00	-	Direction coefficient
T=	1.00	/50 god	Return period of 50 years (1/50 probability of occurrence per year)
c_{prob}=	1.00		
v_b=	26.25	m/s	$v_b=v_0\cdot c_{prob}\cdot c_{season}\cdot c_{dir}$
Terrain category			II
$z_{0,II}$=	0.05	m	
z_{min}=	2.00		
z_0=	0.050	m	
H_{min}=	10.00	m	
H_{max}=	20.00	m	
k_r=	0.19	-	
$c_{0(z)}$=	1.00	-	
ρ=	1.25	kg/m^3	Air density
$c_{f,0}$=	1.00		Shape coefficient for w ind load (assuming no influence from the structural geometry, take 1.00)

z	z_{ref}	$c_{r(z)}$	$I_{v\,(z)}$	$v_{m(z)}$	$c_{e(z)}$	$q_{p(z)}$
[m]	[m]	[-]	[-]	[m/s]	[-]	[kN/m^2]
0.00	10.00	1.01	0.189	26.43	2.35	1.01
10.00	10.00	1.01	0.189	26.43	2.352	1.01
11.00	11.00	1.02	0.185	26.90	2.413	1.04
12.00	12.00	1.04	0.182	27.33	2.469	1.06
13.00	13.00	1.06	0.180	27.73	2.521	1.09
14.00	14.00	1.07	0.177	28.10	2.570	1.11
15.00	15.00	1.08	0.175	28.45	2.616	1.13
16.00	16.00	1.10	0.173	28.77	2.659	1.15
17.00	17.00	1.11	0.172	29.07	2.700	1.16
18.00	18.00	1.12	0.170	29.36	2.738	1.18
19.00	19.00	1.13	0.168	29.63	2.775	1.20
20.00	20.00	1.14	0.167	29.88	2.810	**1.21**



Wind basic load	**w**	**1,00**	**kN/m²**

Roof angle for calculation of pressure coefficients is assumed to be:

$\alpha=5°$

Friction forces are obtained from the following equation:

$$w_{fr}=c_{fr}\cdot q_p$$

$c_{fr}=0.02$

Friction force is:

$$w_{fr}=0.02\cdot 1.00$$

Friction wind load	**w_{fr}**	**0,02**	**kN/m²**

Shape coefficients for vertical walls are as follows:

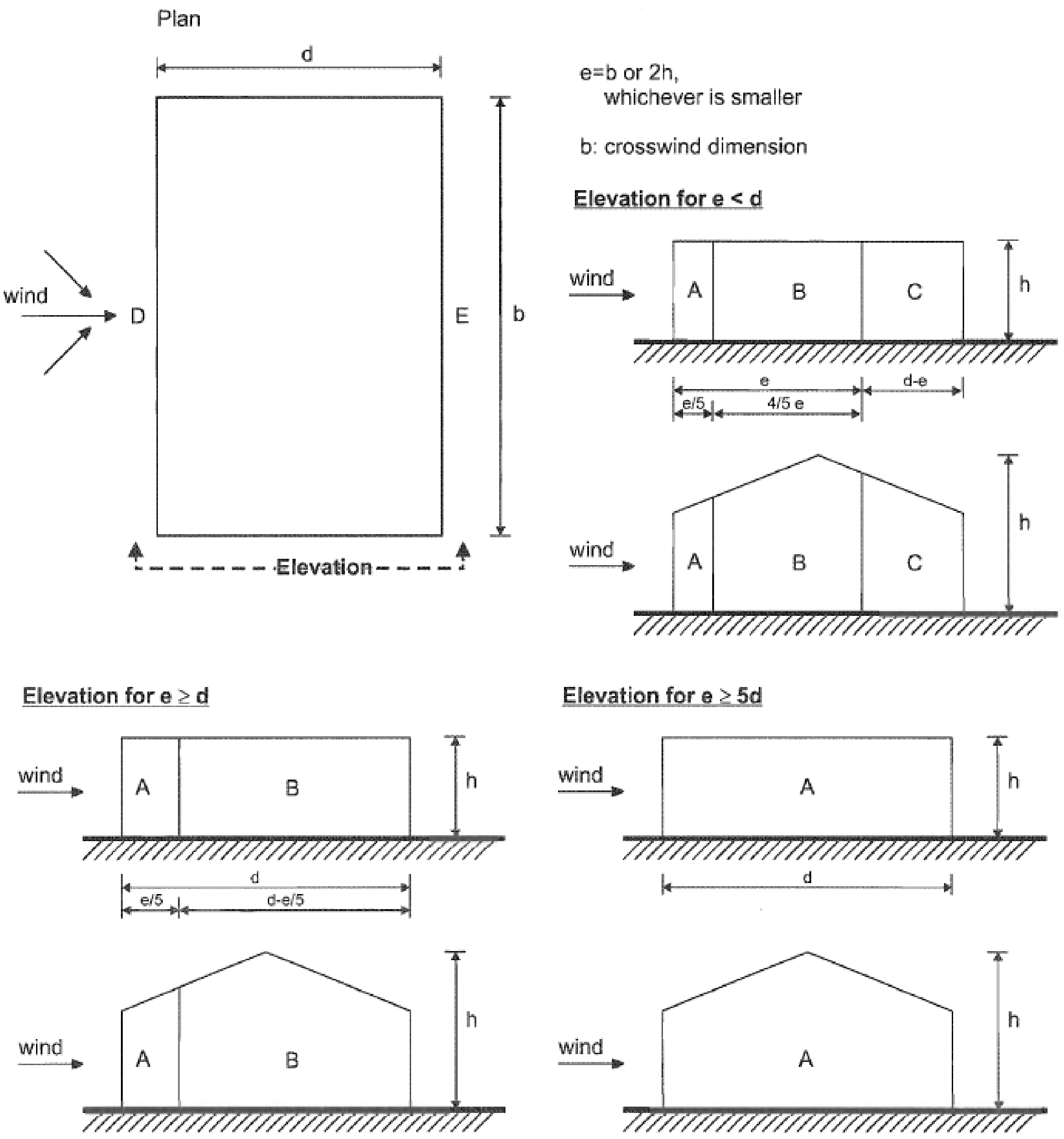


Table 7.1 — Recommended values of external pressure coefficients for vertical walls of rectangular plan buildings

Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
≤ 0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

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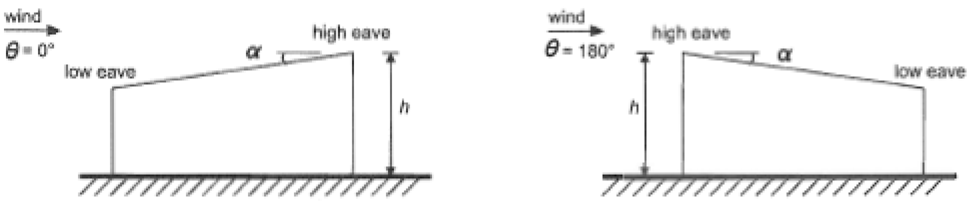
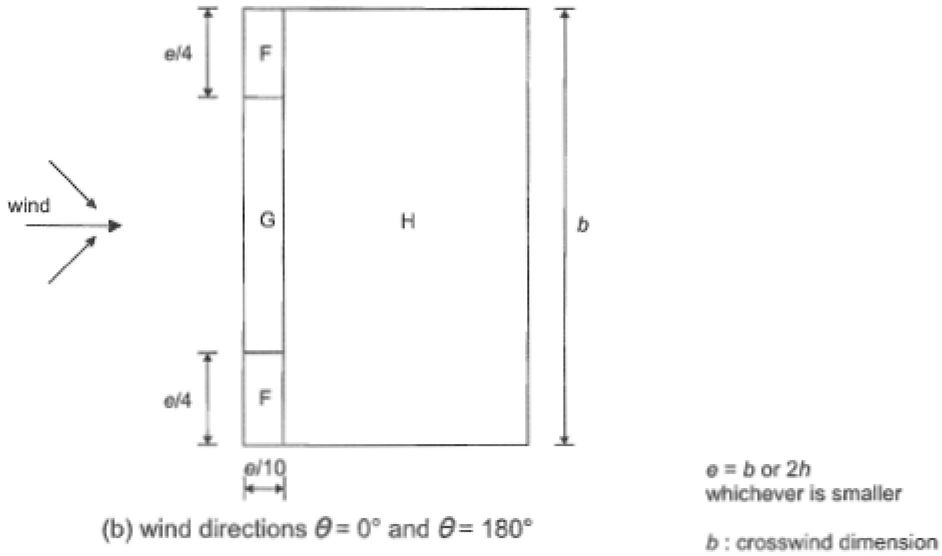
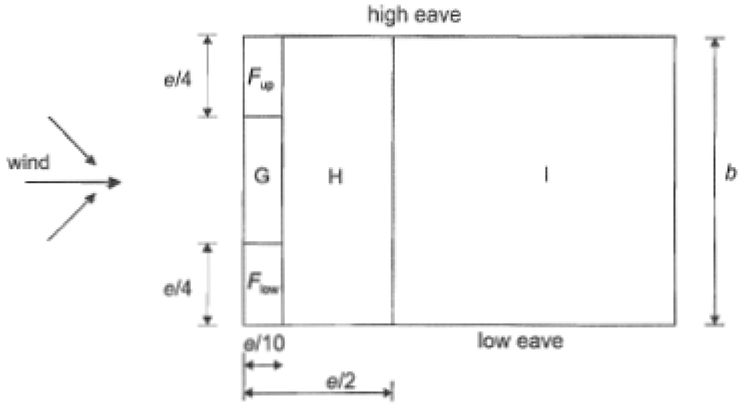
Shape coefficients for roof are as follows:



(a) general



(b) wind directions θ = 0° and θ = 180°



(c) wind direction θ = 90°

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Table 7.3a — Recommended values of external pressure coefficients for monopitch roofs

Pitch Angle α	Zone for wind direction $\theta = 0°$						Zone for wind direction $\theta = 180°$					
	F		G		H		F		G		H	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5°	-1,7	-2,5	-1,2	-2,0	-0,6	-1,2	-2,3	-2,5	-1,3	-2,0	-0,8	-1,2
	+0,0		+0,0		+0,0							
15°	-0,9	-2,0	-0,8	-1,5	-0,3		-2,5	-2,8	-1,3	-2,0	-0,9	-1,2
	+0,2		+0,2		+ 0,2							
30°	-0,5	-1,5	-0,5	-1,5	-0,2		-1,1	-2,3	-0,8	-1,5	-0,8	
	+0,7		+0,7		+0,4							
45°	-0,0		-0,0		-0,0		-0,6	-1,3	-0,5		-0,7	
	+0,7		+0,7		+0,6							
60°	+0,7		+0,7		+0,7		-0,5	-1,0	-0,5		-0,5	
75°	+0,8		+0,8		+0,8		-0,5	-1,0	-0,5		-0,5	

Table 7.3b — Recommended values of external pressure coefficients for monopitch roofs

Pitch Angle α	Zone for wind direction $\theta = 90°$									
	F_{up}		F_{low}		G		H		I	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5°	-2,1	-2,6	-2,1	-2,4	-1,8	-2,0	-0,6	-1,2	-0,5	
15°	-2,4	-2,9	-1,6	-2,4	-1,9	-2,5	-0,8	-1,2	-0,7	-1,2
30°	-2,1	-2,9	-1,3	-2,0	-1,5	-2,0	-1,0	-1,3	-0,8	-1,2
45°	-1,5	-2,4	-1,3	-2,0	-1,4	-2,0	-1,0	-1,3	-0,9	-1,2
60°	-1,2	-2,0	-1,2	-2,0	-1,2	-2,0	-1,0	-1,3	-0,7	-1,2
75°	-1,2	-2,0	-1,2	-2,0	-1,2	-2,0	-1,0	-1,3	-0,5	

NOTE 1 At $\theta = 0°$ (see table a)) the pressure changes rapidly between positive and negative values around a pitch angle of $\alpha = +5°$ to $+45°$, so both positive and negative values are given. For those roofs, two cases should be considered: one with all positive values, and one with all negative values. No mixing of positive and negative values is allowed on the same face.

NOTE 2 Linear interpolation for intermediate pitch angles may be used between values of the same sign. The values equal to 0.0 are given for interpolation purposes

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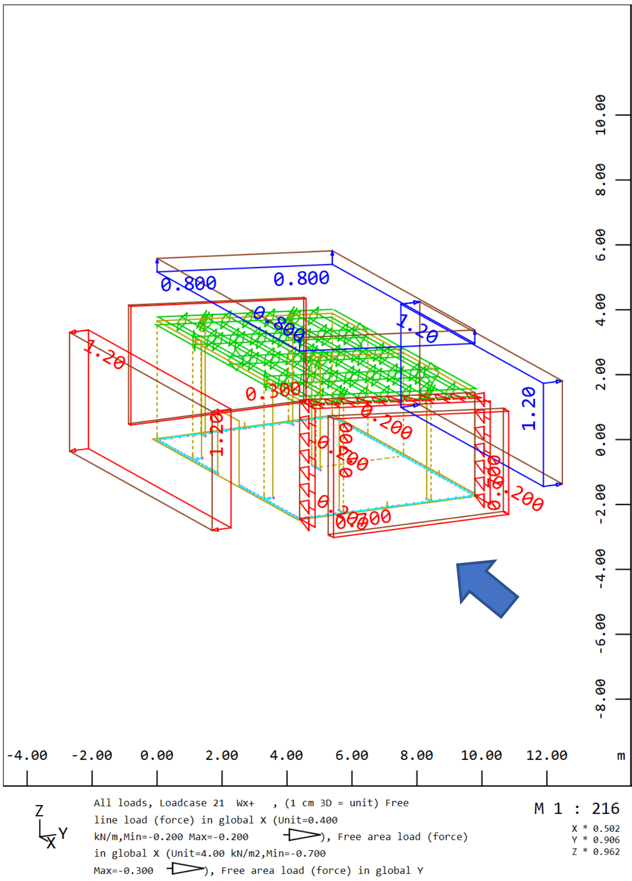
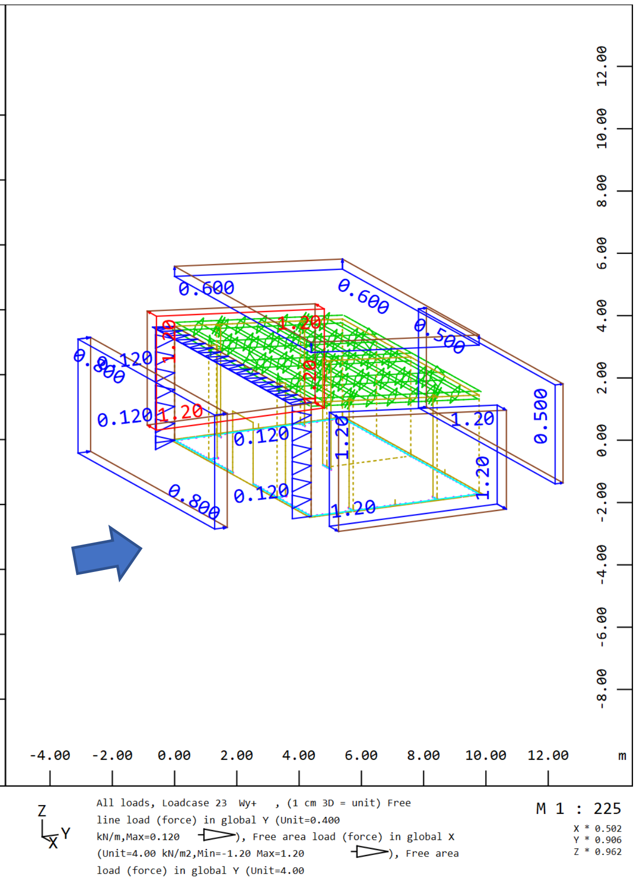
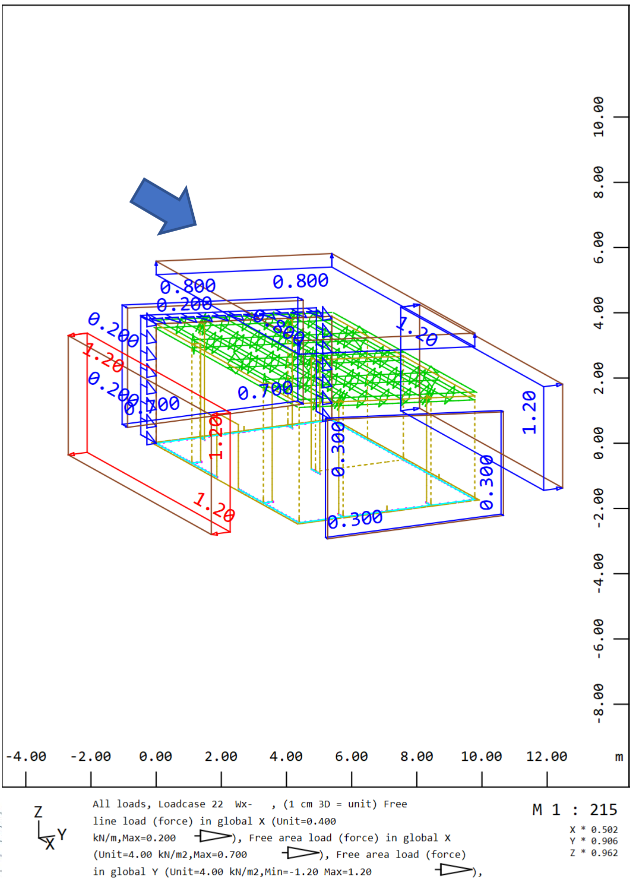
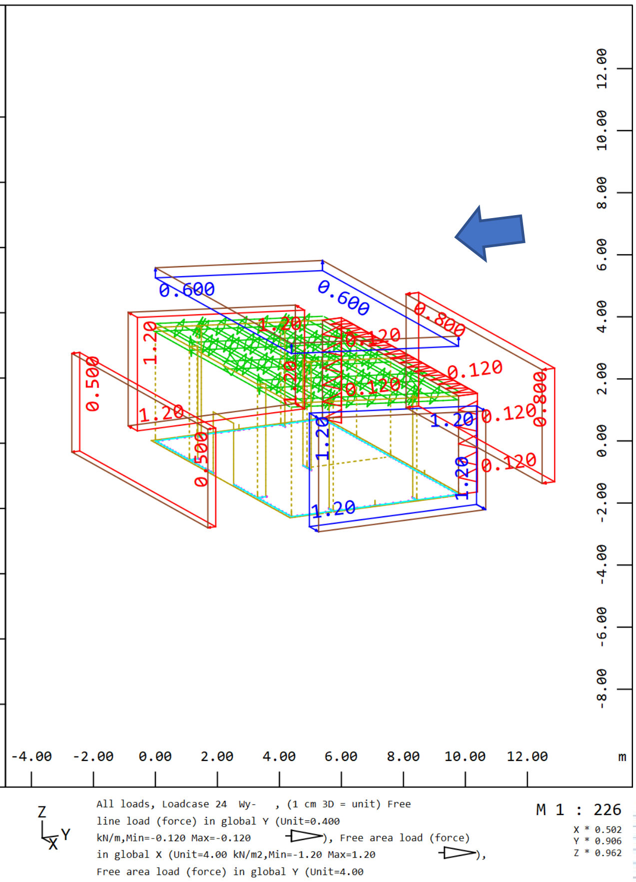
Wind loads applied to the structure are shown below:





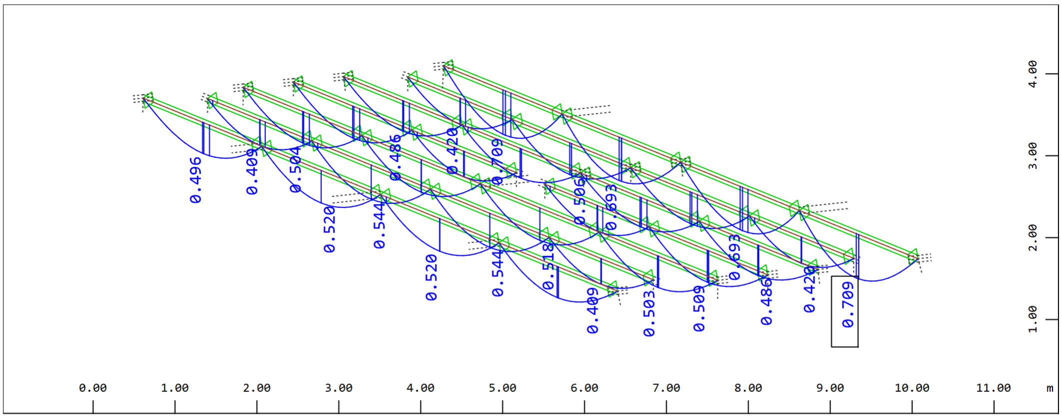




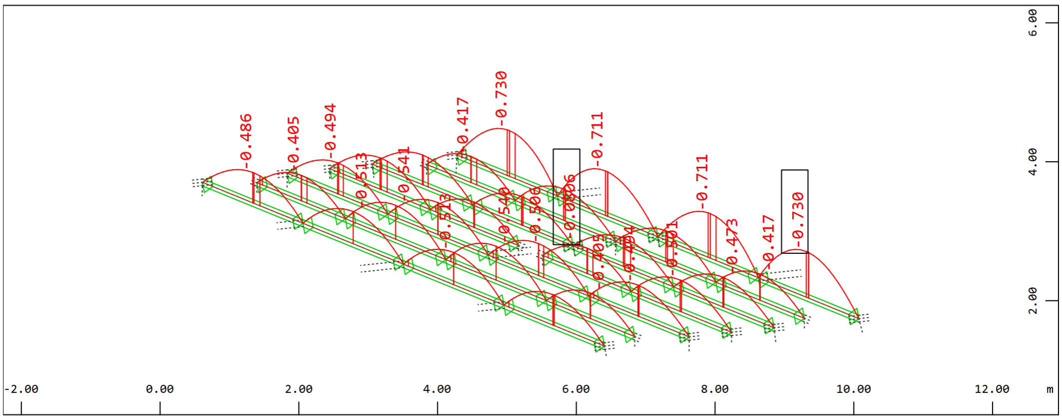
6 Design of purlins

6.1 Secondary purlins SP

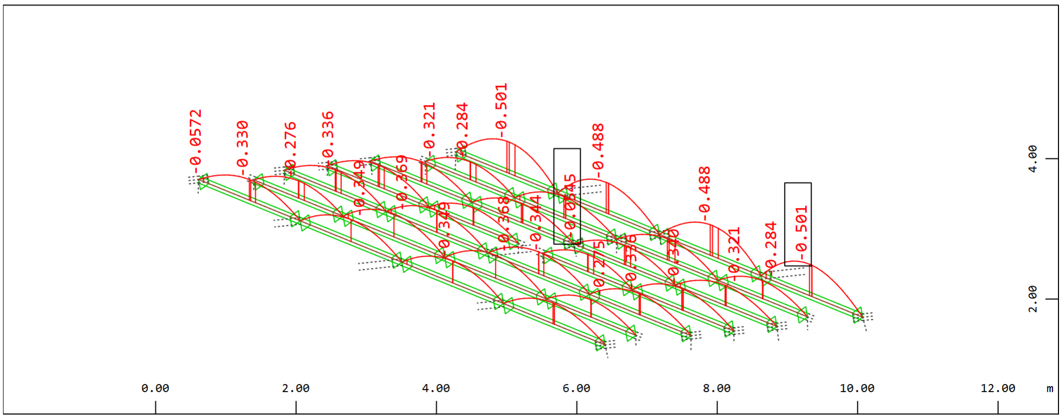
The values of bending moments and shear forces were obtained from software analysis.



Sector of system Group 10 M 1 : 72
Beam Elements , Bending moment My, nonlinear Loadcase 1001 1.35G_1+1.35G_1+1.5S , 1 cm X * 0.625
3D = 0.902 kNm (Min=-1.6453e-15) (Max=0.709) Y * 0.832 Z * 0.957



Sector of system Group 10 M 1 : 85
Beam Elements , Bending moment My, nonlinear Loadcase 1002 G_1+G_1+1.5W , 1 cm 3D = X * 0.625
0.902 kNm (Min=-0.730) (Max= 9.3604e-16) Y * 0.832 Z * 0.957



Sector of system Group 10 M 1 : 84
Beam Elements , Bending moment My, nonlinear Loadcase 1004 G_1+G_1+1.5W , 1 cm 3D = X * 0.625
0.902 kNm (Min=-0.501) (Max= 8.2421e-16) Y * 0.832 Z * 0.957

Fig. 11: Governing bending M_y moments for beam design

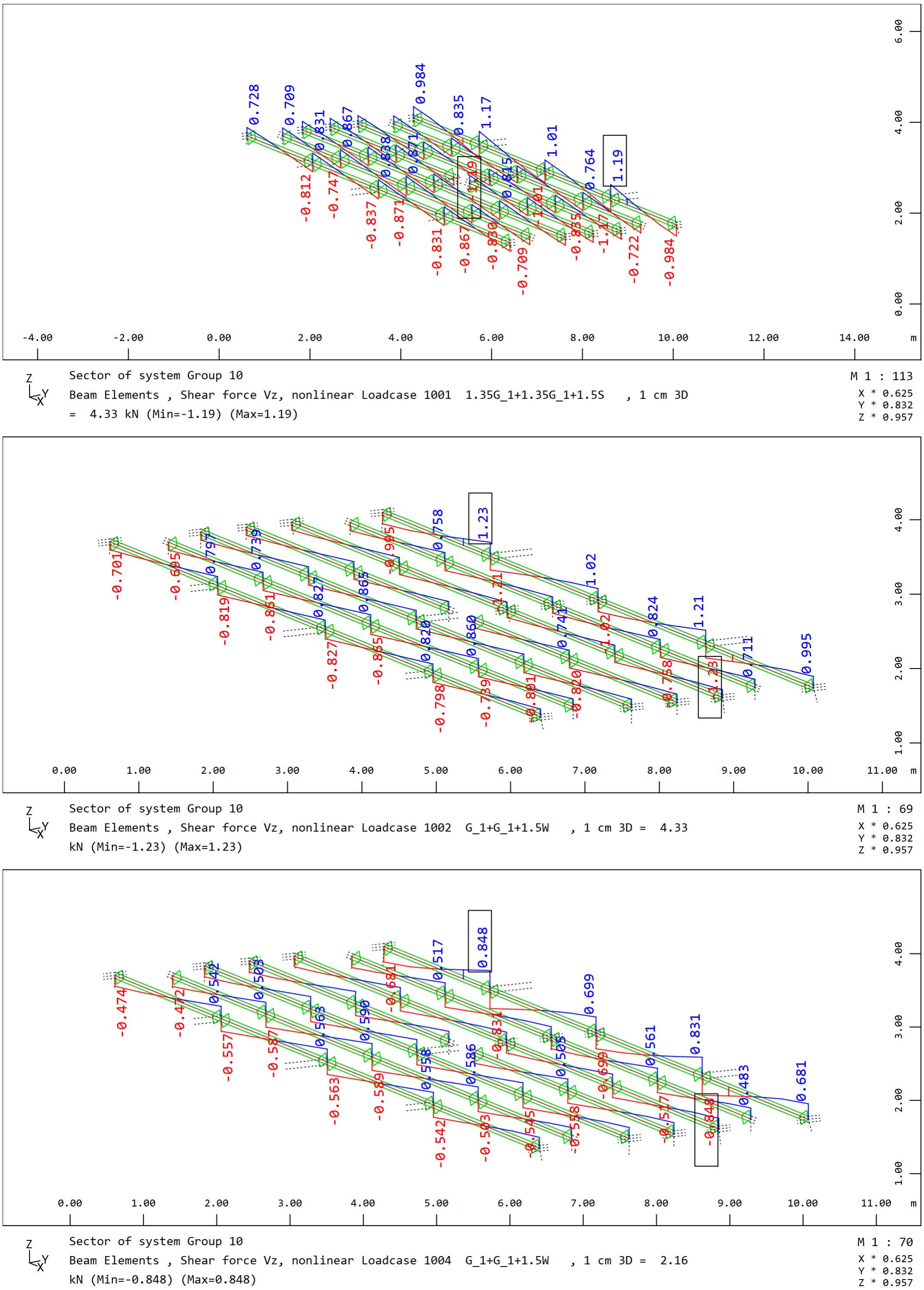
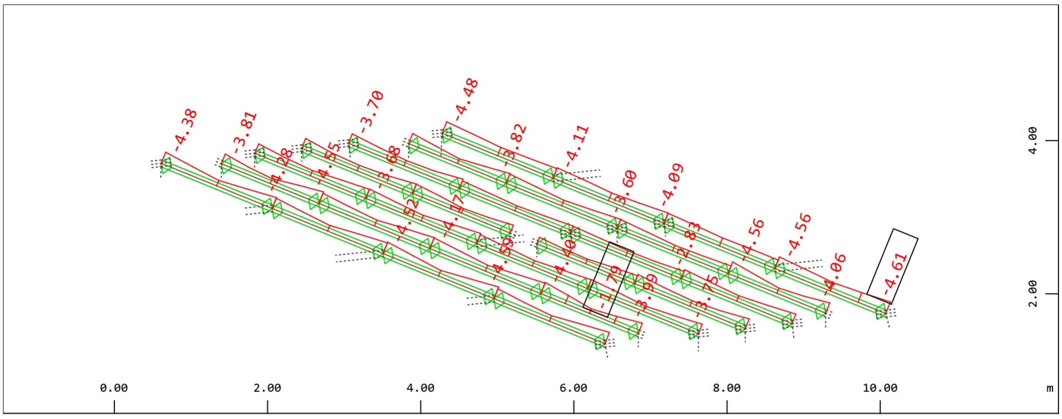


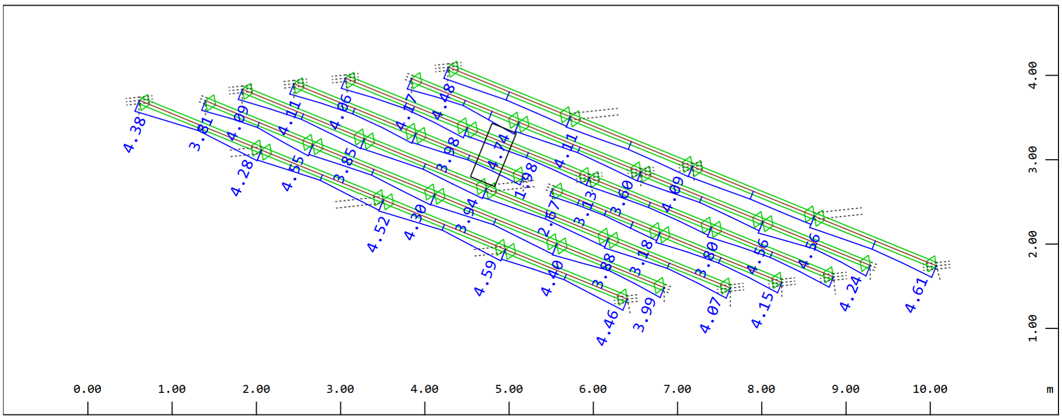
Fig. 12: Governing shear forces V_z for beam design

6.2 Capacity check

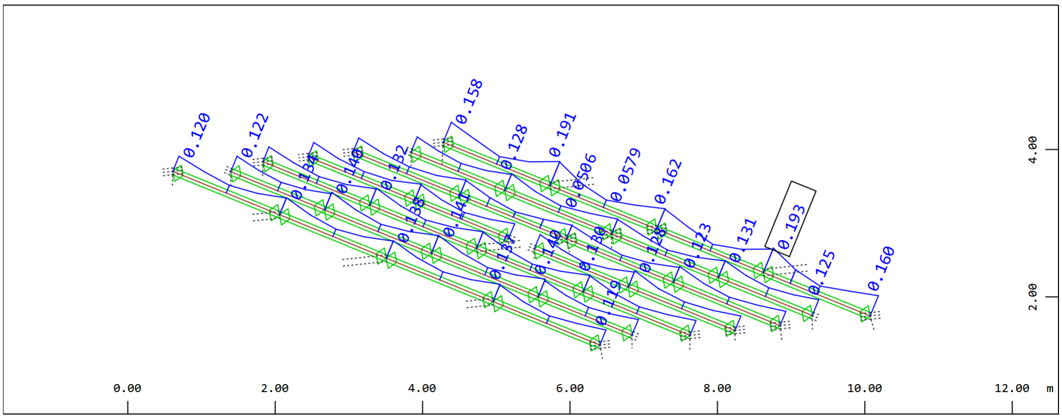
Stresses are shown in the figure below:



Sector of system Group 10 M 1 : 77
Beam Elements , Maximum compression stress, Design Case 901 GlobalMAX SIGMA, Material 1 C 24 Cl.1 (EN 1995) , 1 cm 3D = 21.6 MPa (Min=-4.61) (Max=-1.79)
X * 0.625 Y * 0.832 Z * 0.957



Sector of system Group 10 M 1 : 70
Beam Elements , Maximum tensile stress, Design Case 901 GlobalMAX SIGMA, Material 1 C 24 Cl.1 (EN 1995) , 1 cm 3D = 21.6 MPa (Max=4.74)
X * 0.625 Y * 0.832 Z * 0.957



Sector of system Group 10 M 1 : 80
Beam Elements , Shear stress, Design Case 901 GlobalMAX SIGMA, Material 1 C 24 Cl.1 (EN 1995) , 1 cm 3D = 0.433 MPa (Max=0.193)
X * 0.625 Y * 0.832 Z * 0.957

Fig. 13: Maximum stresses

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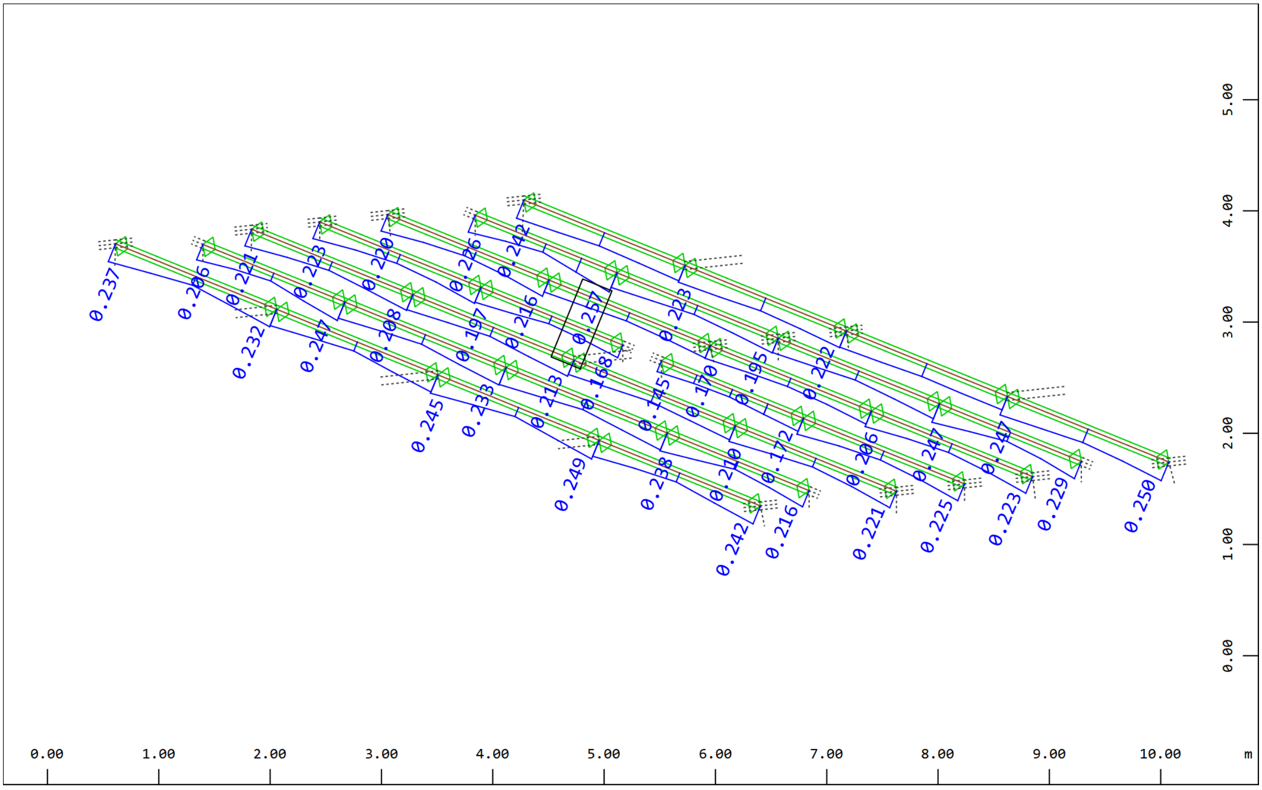
The utilisation level is automatically calculated in the software based on the following criteria:

Bending: $\sigma_{m,d}/f_{m,d}$
Shear: $\tau_{m,d}/f_{v,d}$



Sector of system Group 10
Beam Elements , Utilisation level Decisive - Total, Design Case 901 GlobalMAX SIGMA , 1
cm 3D = 0.866 - (Max=0.257)

M 1 : 63
X * 0.625
Y * 0.832
Z * 0.957

$$\eta = E_d / R_d$$

η =0.257	<	η_{lim}=1.000
OK!		

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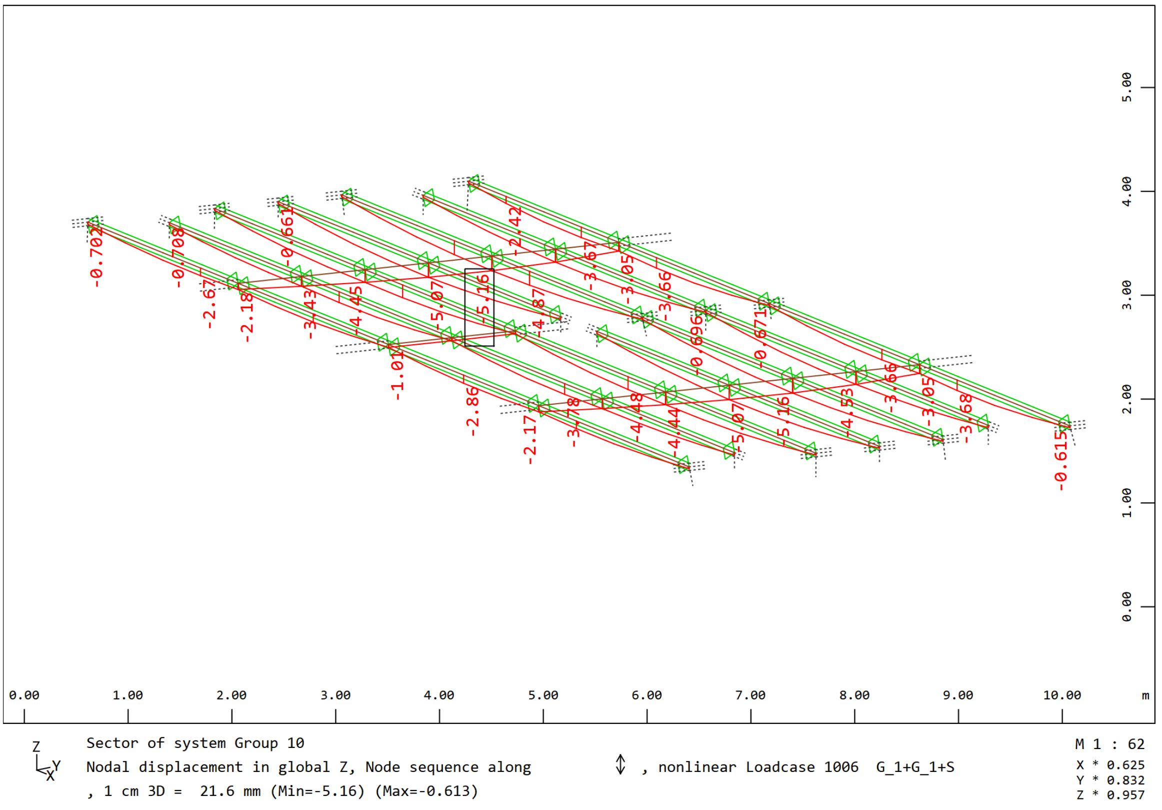
6.3 Deflection check



v=5.16mm

v_{lim}=L/150=2500/150=16.66mm

v =5.16 mm	<	v_{lim}=16.66 mm
OK!		

6.4 Local compression check

Strength condition: $\sigma_{c,90,d} < k_{c,90} \cdot f_{c,90,d}$
Effective cross-section area for compression:
A_{ef}= b·l
A_{ef} = 4900 mm^2
R_d=2.50kN

Calculated compressive stress:
$\sigma_{c,90,d}=R_d/A_{ef}$
$\sigma_{c,90,d}$ =0.51 N/mm^2

$\sigma_{c,90,d}$ =0.51 N/mm^2	<	$f_{c,90,d}$ =1.90 N/mm^2
OK!		

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7 Design of Primary composite purlins (PCP)

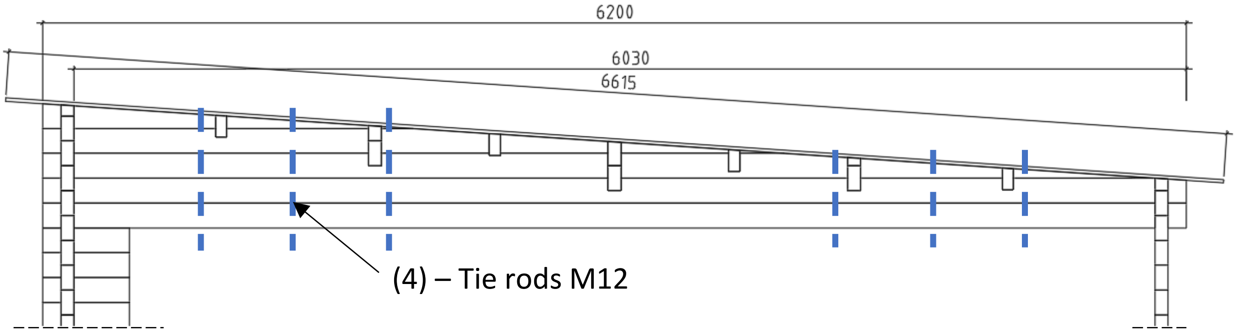

Fig. 14: Typical composite purlins

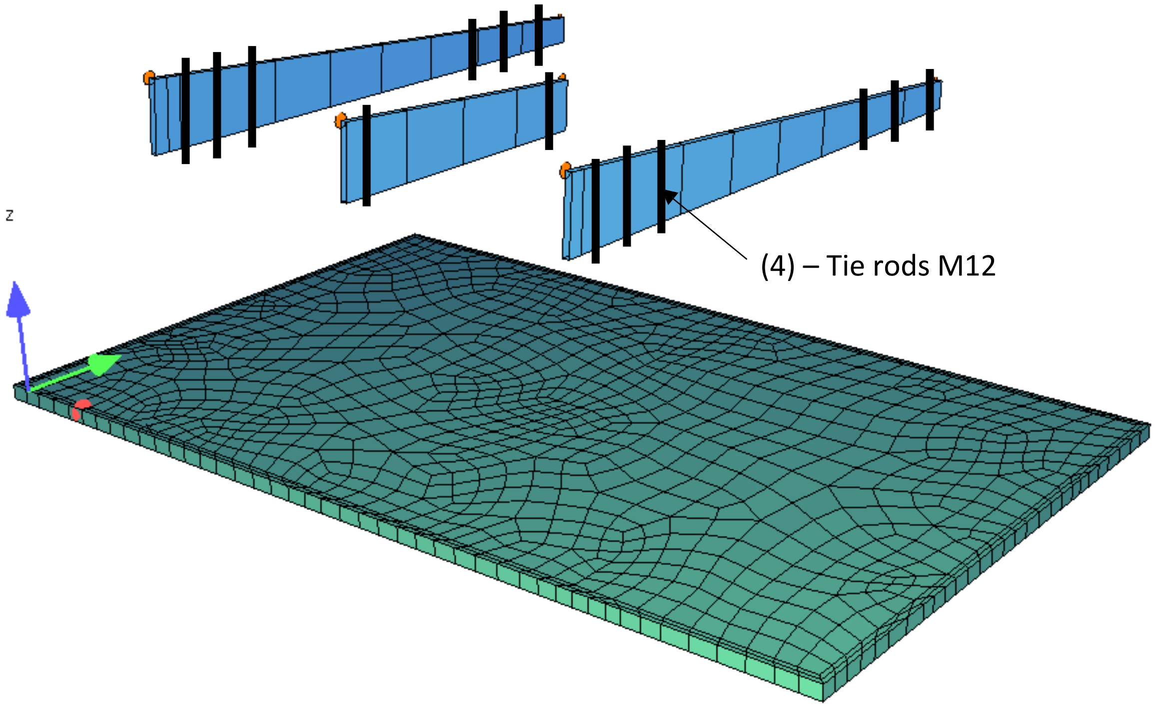

Fig. 15: Model elements of composite purlins

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7.1 Purlins PCP

The values of bending moments and shear forces were obtained from software analysis.

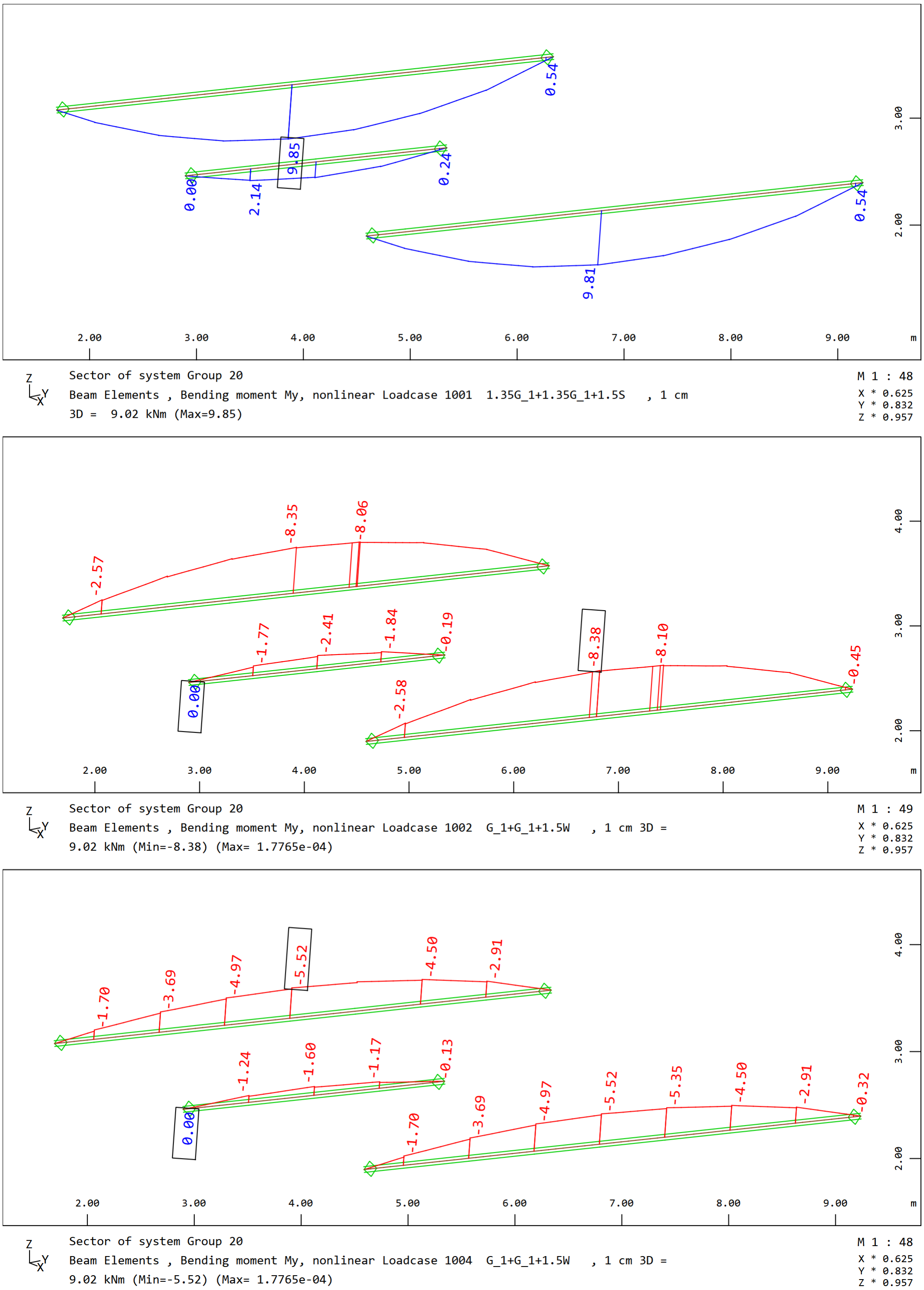

Fig. 16: Governing bending M_y moments for beam design

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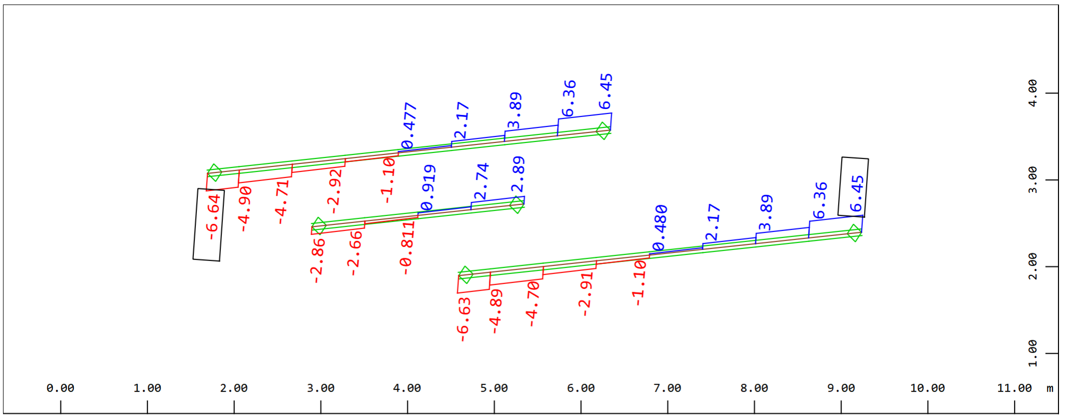
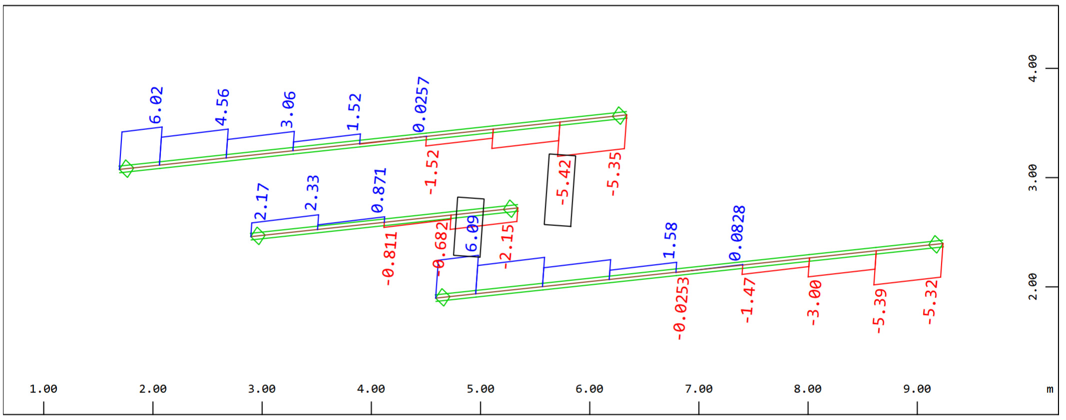
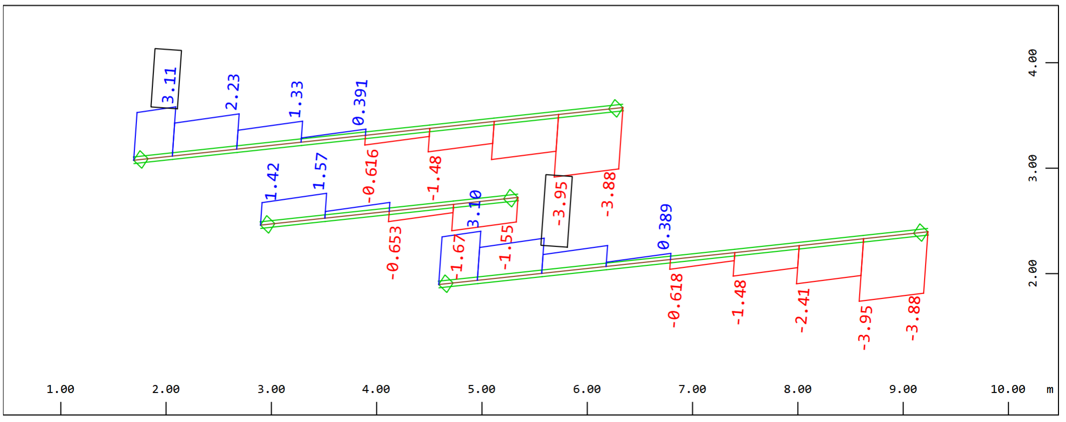


Z Y X Sector of system Group 20 M 1 : 68
Beam Elements , Shear force Vz, nonlinear Loadcase 1001 1.35G_1+1.35G_1+1.5S , 1 cm 3D = 21.6 kN (Min=-6.64) (Max=6.45)
X * 0.625
Y * 0.832
Z * 0.957



Z Y X Sector of system Group 20 M 1 : 54
Beam Elements , Shear force Vz, nonlinear Loadcase 1002 G_1+G_1+1.5W , 1 cm 3D = 9.02 kN (Min=-5.42) (Max=6.09)
X * 0.625
Y * 0.832
Z * 0.957



Z Y X Sector of system Group 20 M 1 : 56
Beam Elements , Shear force Vz, nonlinear Loadcase 1004 G_1+G_1+1.5W , 1 cm 3D = 3.61 kN (Min=-3.95) (Max=3.11)
X * 0.625
Y * 0.832
Z * 0.957

Fig. 17: Governing shear forces V_z for beam design

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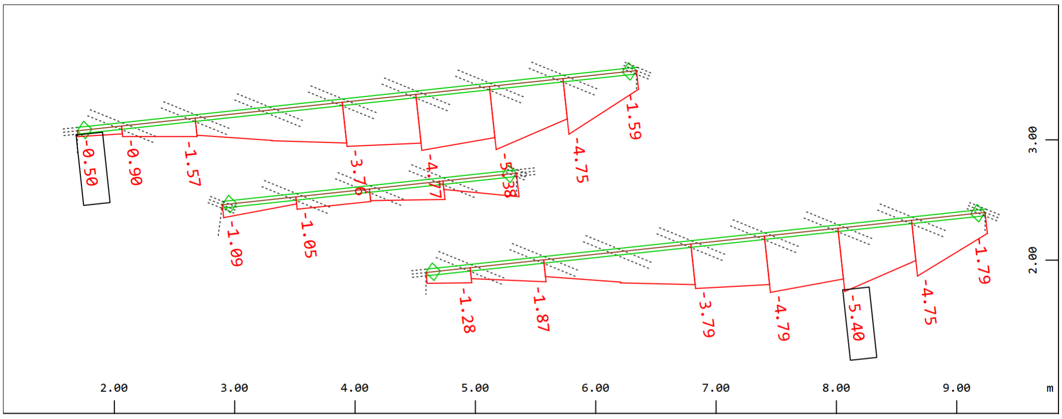
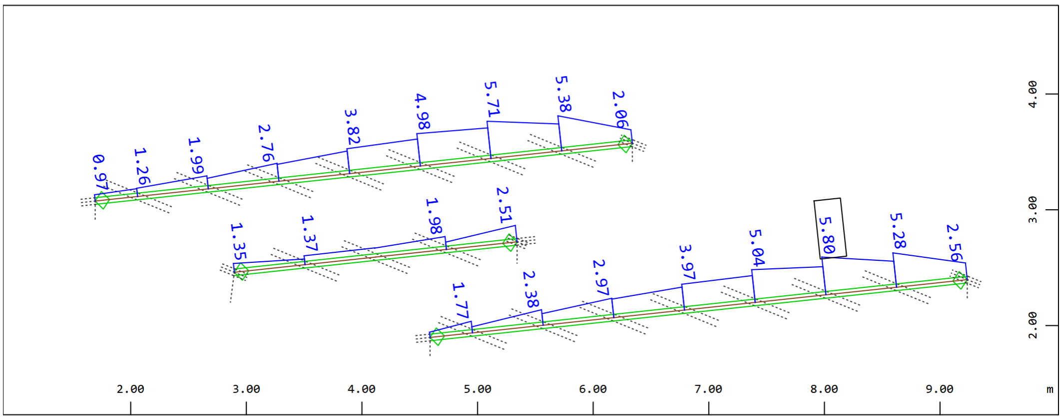
7.2 Capacity check

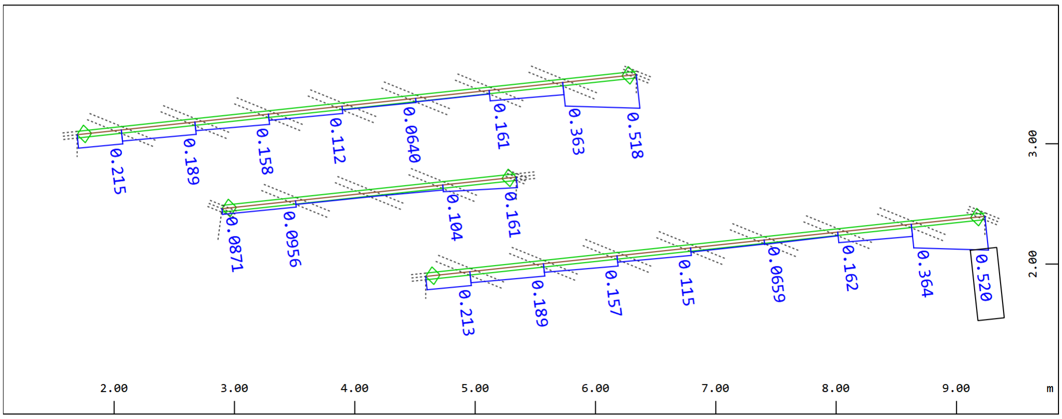
Stresses are shown on the picture below:



Sector of system Group 20
Beam Elements , Maximum compression stress, Design Case 901 GlobalMAX SIGMA, Material 1 C 24 Cl.1 (EN 1995) , 1 cm 3D = 5.00 MPa (Min=-5.40) (Max=-0.501)
M 1 : 49
X * 0.625
Y * 0.832
Z * 0.957



Sector of system Group 20
Beam Elements , Maximum tensile stress, Design Case 901 GlobalMAX SIGMA, Material 1 C 24 Cl.1 (EN 1995) , 1 cm 3D = 9.02 MPa (Max=5.80)
M 1 : 51
X * 0.625
Y * 0.832
Z * 0.957



Sector of system Group 20
Beam Elements , Shear stress, Design Case 901 GlobalMAX SIGMA, Material 1 C 24 Cl.1 (EN 1995) , 1 cm 3D = 0.902 MPa (Max=0.520)
M 1 : 49
X * 0.625
Y * 0.832
Z * 0.957

Fig. 18: Maximum stresses

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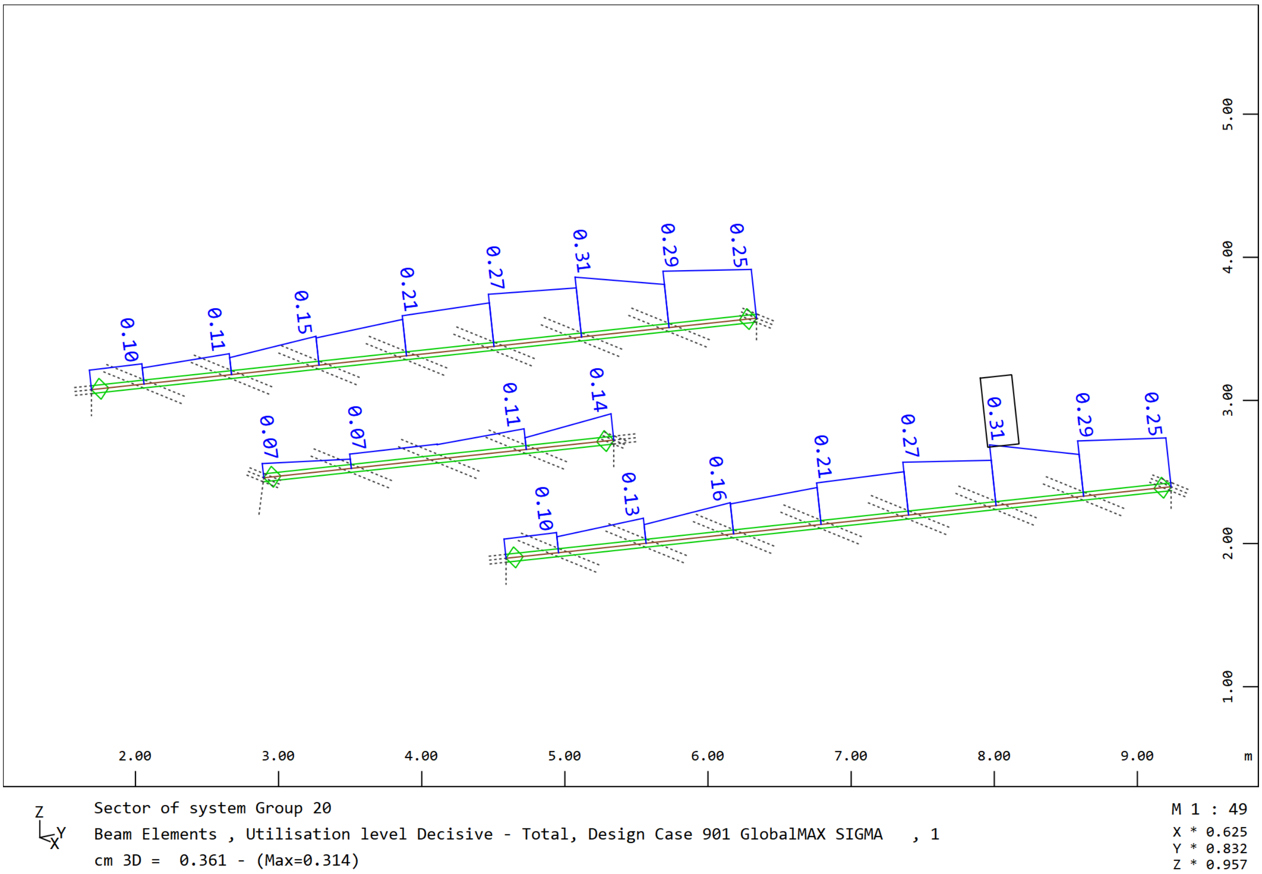
The utilisation level is automatically calculated in the software based on the following criteria:

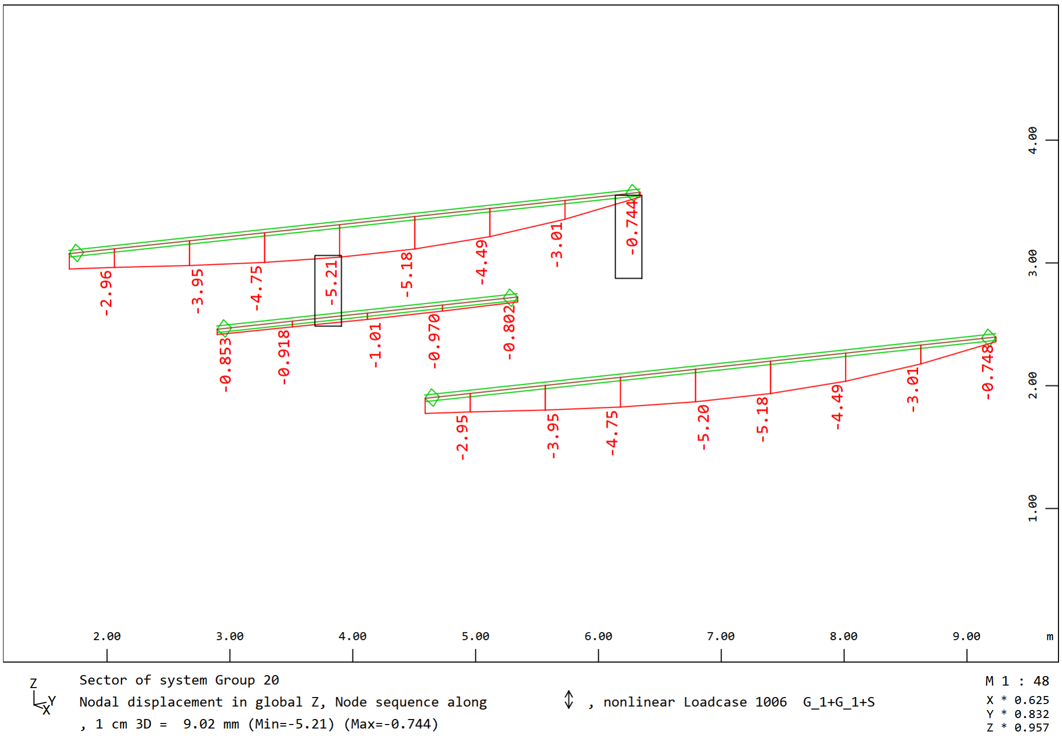
Bending: $\sigma_{m,d}/f_{m,d}$
Shear: $\tau_{m,d}/f_{v,d}$



$$\eta = E_d / R_d$$

η =0.310	<	η_{lim}=1.000
OK!		

7.3 Deflection check



v=5.21mm

v_{lim}=L/150=2500/150=16.66mm

v =5.21 mm	<	v_{lim}=16.66 mm
OK!		

7.4 Local compression check

Strength condition: $\sigma_{c,90,d} < k_{c,90} \cdot f_{c,90,d}$

Effective cross-section area for compression:

A_{ef}= b·l

A_{ef} = 4900 mm^2

R_d=6.64kN

Calculated compressive stress:

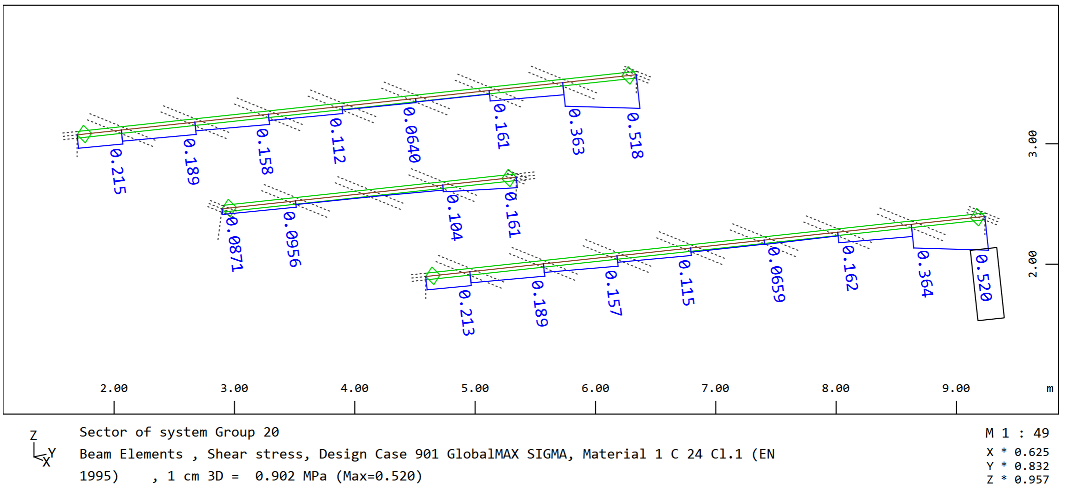
$\sigma_{c,90,d}=R_d/A_{ef}$

$\sigma_{c,90,d}$ =1.35 N/mm^2

$\sigma_{c,90,d}$ =1.35 N/mm^2	<	$f_{c,90,d}$ =1.90 N/mm^2
OK!		

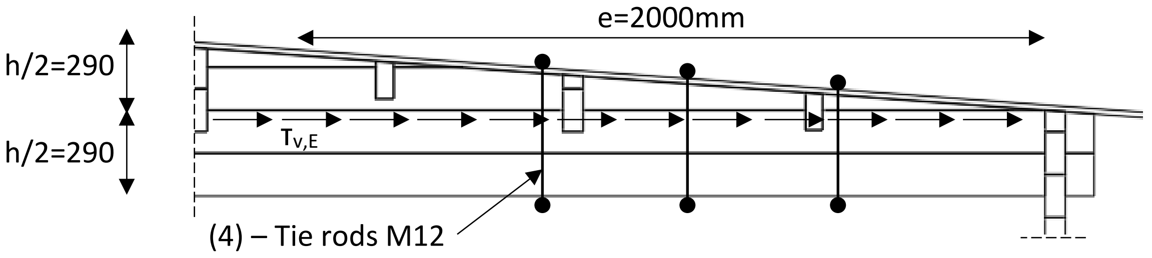
7.5 Shear connection check

Shear conection check is calculated from the shear stress results as follows:



$\tau_{v,E}$=0.150MPa ⟶ Average shear stress

Shear force on connection threaded bars is calculated from the following equation:



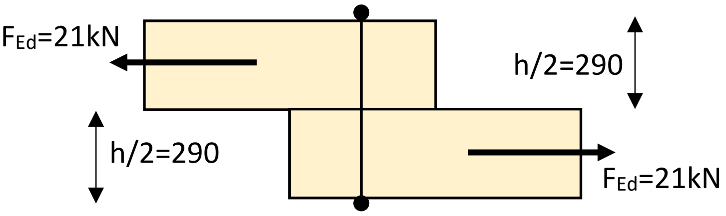
$$F_{Ed}= \tau_{v,E}\cdot b\cdot e$$

b ⟶ Width of the beam

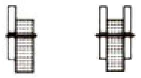
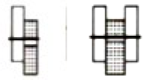
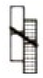
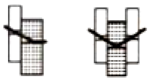
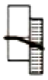
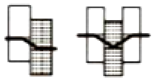
e ⟶ Shear length of the beam

$$F_{Ed}= 0.150\cdot 7\cdot 200\cdot 10^{-1}=21kN$$

Capacity of the shear connectors are calculated using the following failure mechanism:



The wooden conection capacity is calculated from the EN 1995-1-1 equations:

$F_{1,k}=$	76.235	kN			⇒	$f_{h,1,k}\cdot t_1\cdot d$
$F_{2,k}=$	76.235	kN			⇒	$f_{h,2,k}\cdot t_2\cdot d$
$F_{3,k}=$	109.159	kN		-	⇒	$\frac{f_{h,1,k}\cdot t_1\cdot d}{1+\beta}\cdot\left[\sqrt{\beta+2\cdot\beta^2\cdot\left[1+\frac{t_2}{t_1}+\left(\frac{t_2}{t_1}\right)^2\right]+\beta^3\cdot\left(\frac{t_2}{t_1}\right)^2}-\beta\cdot\left(1-\frac{t_2}{t_1}\right)\right]+F_{ax,Rk}\cdot\Psi$
$F_{4,k}=$	28.391	kN			⇒	$1.05\cdot\frac{f_{h,1,k}\cdot t_1\cdot d}{2+\beta}\cdot\left[\sqrt{2\cdot\beta\cdot[1+\beta]+\frac{4\cdot\beta\cdot(2+\beta)}{f_{h,1,k}\cdot d}\cdot\frac{M_{y,Rk}}{t_1^2}}-\beta\right]+F_{ax,Rk}\cdot\Psi$
$F_{5,k}=$	28.391	kN		-	⇒	$1.05\cdot\frac{f_{h,1,k}\cdot t_1\cdot d}{1+2\cdot\beta}\cdot\left[\sqrt{2\cdot\beta^2\cdot[1+\beta]+\frac{4\cdot\beta\cdot(1+2\cdot\beta)}{f_{h,1,k}\cdot d}\cdot\frac{M_{y,Rk}}{t_2^2}}-\beta\right]+F_{ax,Rk}\cdot\Psi$
$F_{6,k}=$	9.709	kN			⇒	$1.15\cdot\sqrt{\frac{2\cdot\beta}{1+\beta}}\cdot\sqrt{2\cdot M_{y,Rk}\cdot f_{h,1,k}\cdot d}+F_{ax,Rk}\cdot\Psi$
$F_{v,Rk}=$	9.709	kN				
$F_{v,Rd}=$	**7.282**	kN	⇒			$F_{V,Rd}=k_{mod}\frac{MIN(F_{i,k})}{\gamma_M}$

$F_{Rd}=$**7.28**kN

$n=F_{Ed}/F_{Rd}=21.00/7.28=2.88$ pcs. ⟶ **3pcs. adopted**

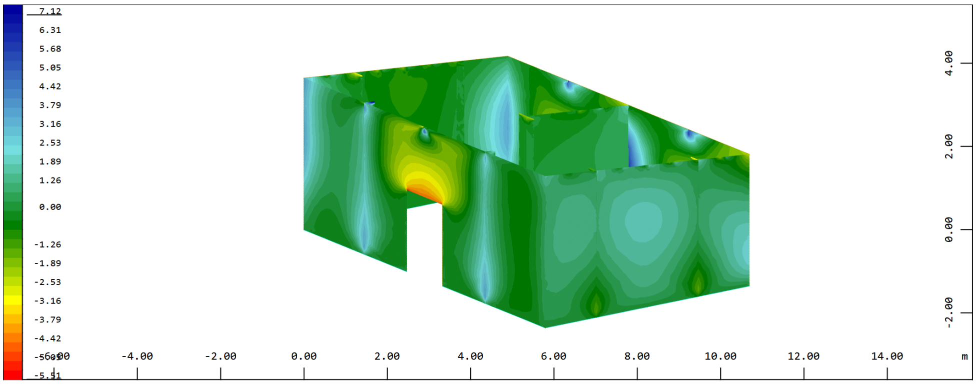
$\eta = E_d / R_d=2.88/3.00=0.960$

η =0.960	<	η_{lim}=1.000
OK!		

8 Design of wall

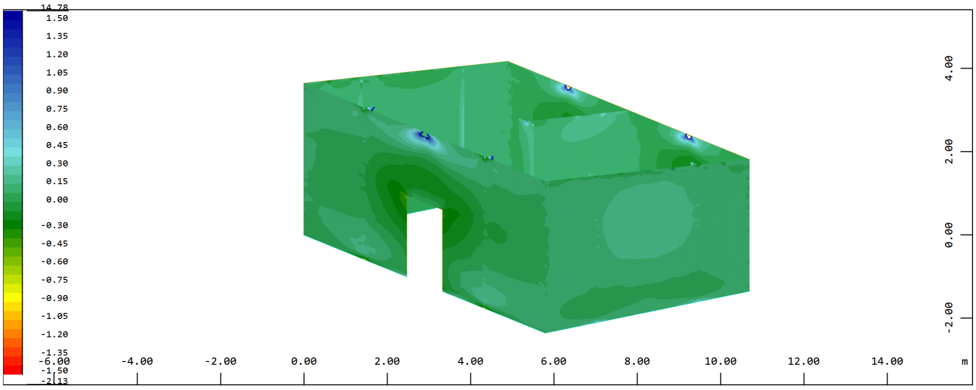
8.1 Wall design verification

Stresses in the walls are obtained from the relevant load cases and are shown in the figures below:



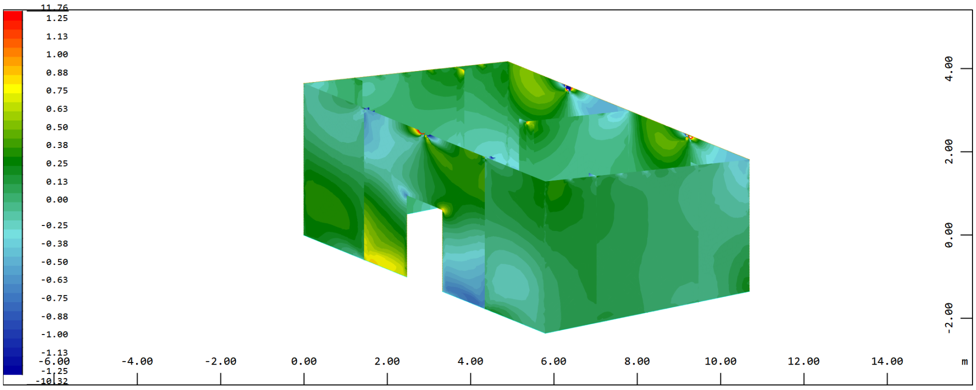
Sector of system Quadrilateral Elements Group 41 42 51 52
Top Stress in local x in Node, nonlinear Loadcase 1002 G_1+G_1+1.5W , from -5.51 to 7.12 step 0.316 MPa

M 1 : 130
X * 0.625
Y * 0.832
Z * 0.957



Sector of system Quadrilateral Elements Group 41 42 51 52
Top Stress in local y in Node , nonlinear Loadcase 1002 G_1+G_1+1.5W , from -1.50 to 1.50 step 0.0750 MPa

M 1 : 130
X * 0.625
Y * 0.832
Z * 0.957



Sector of system Quadrilateral Elements Group 41 42 51 52
Top Membrane shear stress tau-xy in Node, nonlinear Loadcase 1003 G_1+G_1+1.5W , from -1.25 to 1.25 step 0.0625 MPa

M 1 : 130
X * 0.625
Y * 0.832
Z * 0.957

$\sigma_{c,0,d}$ =7.12 N/mm^2	<	$f_{c,0,d}$ =18.28 N/mm^2
$\sigma_{c,90,d}$ =1.50 N/mm^2	<	$f_{c,90,d}$ =1.90 N/mm^2
$\tau_{v,d}$ =1.25 N/mm^2	<	$f_{v,d}$ =1.90 N/mm^2
OK!		

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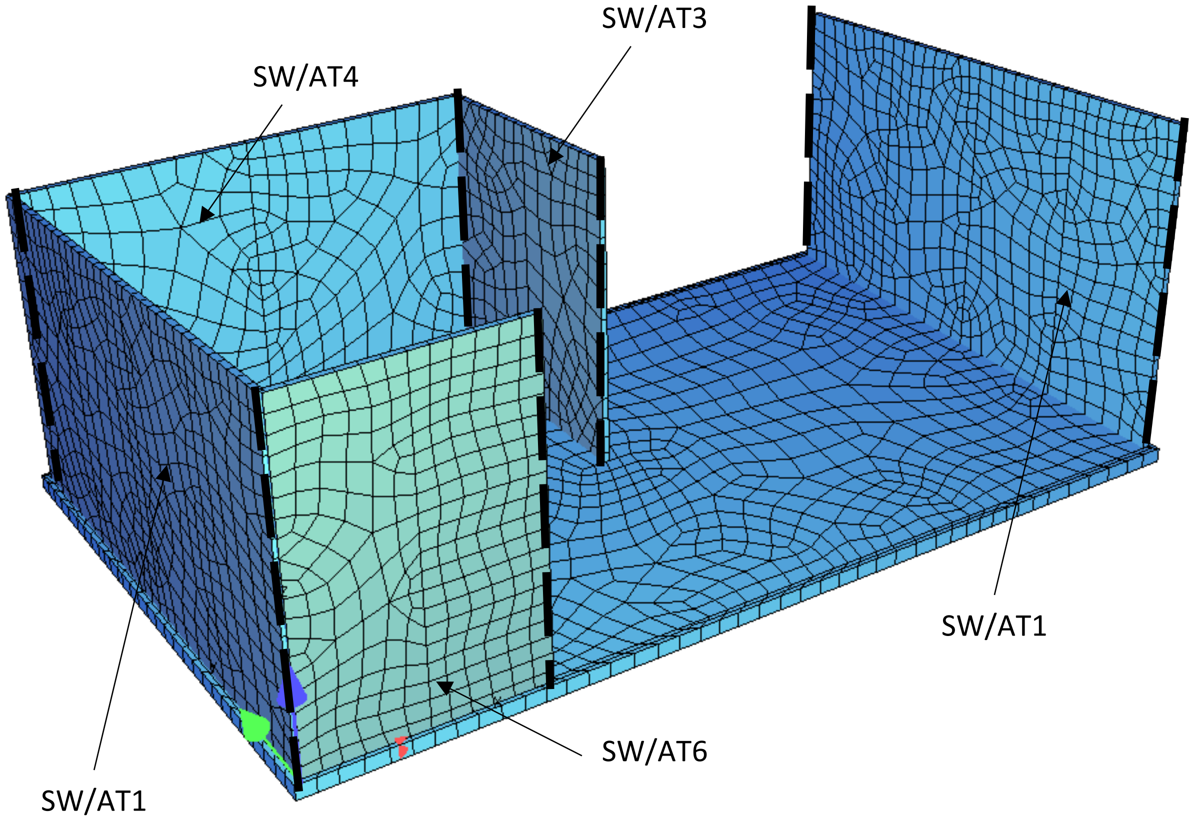
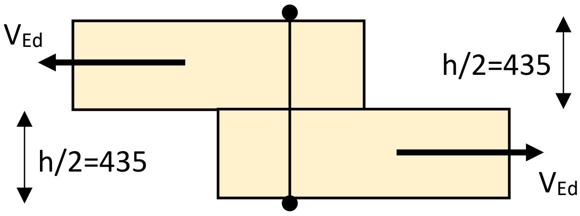
8.2 Shear wall verification

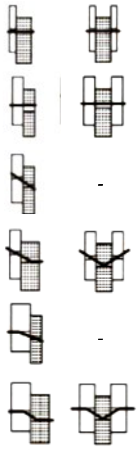
Shear walls formed in the model are:

SW/AT1 2pcs.
SW/AT3 1pcs.
SW/AT4 1pcs.
SW/AT6 1pcs.



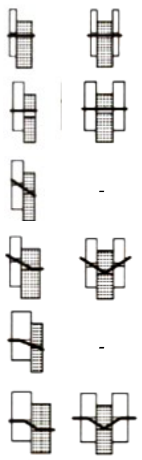
The shear resistance of the wall is verified using the following principles:





$F_{1,k}=$	114.353	kN	$\Rightarrow f_{h,1,k}\cdot t_1\cdot d$
$F_{2,k}=$	114.353	kN	$\Rightarrow f_{h,2,k}\cdot t_2\cdot d$
$F_{3,k}=$	170.436	kN	$\Rightarrow \frac{f_{h,1,k}\cdot t_1\cdot d}{1+\beta}\cdot\left[\sqrt{\beta+2\cdot\beta^2\cdot\left[1+\frac{t_2}{t_1}+\left(\frac{t_2}{t_1}\right)^2\right]+\beta^3\cdot\left(\frac{t_2}{t_1}\right)^2}-\beta\cdot\left(1-\frac{t_2}{t_1}\right)\right]+F_{ax,Rk}\cdot\Psi$
$F_{4,k}=$	48.934	kN	$\Rightarrow 1.05\cdot\frac{f_{h,1,k}\cdot t_1\cdot d}{2+\beta}\cdot\left[\sqrt{2\cdot\beta\cdot[1+\beta]+\frac{4\cdot\beta\cdot(2+\beta)}{f_{h,1,k}\cdot d}\cdot\frac{M_{y,Rk}}{t_1^2}}-\beta\right]+F_{ax,Rk}\cdot\Psi$
$F_{5,k}=$	48.934	kN	$\Rightarrow 1.05\cdot\frac{f_{h,1,k}\cdot t_1\cdot d}{1+2\cdot\beta}\cdot\left[\sqrt{2\cdot\beta^2\cdot[1+\beta]+\frac{4\cdot\beta\cdot(1+2\cdot\beta)}{f_{h,1,k}\cdot d}\cdot\frac{M_{y,Rk}}{t_2^2}}-\beta\right]+F_{ax,Rk}\cdot\Psi$
$F_{6,k}=$	14.961	kN	$\Rightarrow 1.15\cdot\sqrt{\frac{2\cdot\beta}{1+\beta}}\cdot\sqrt{2\cdot M_{y,Rk}\cdot f_{h,1,k}\cdot d}+F_{ax,Rk}\cdot\Psi$
$F_{V,Rk}=$	14.961	kN	
$F_{v,Rd}=$	**10.357**	kN	$\Rightarrow F_{V,Rd}=k_{mod}\frac{MIN(F_{i,k})}{\gamma_M}$

The resistance of threaded bar M12 (8.8) → $F_{vR,d,1}$=10.35kN



$F_{1,k}=$	114.353	kN	$\Rightarrow f_{h,1,k}\cdot t_1\cdot d$
$F_{2,k}=$	114.353	kN	$\Rightarrow f_{h,2,k}\cdot t_2\cdot d$
$F_{3,k}=$	164.920	kN	$\Rightarrow \frac{f_{h,1,k}\cdot t_1\cdot d}{1+\beta}\cdot\left[\sqrt{\beta+2\cdot\beta^2\cdot\left[1+\frac{t_2}{t_1}+\left(\frac{t_2}{t_1}\right)^2\right]+\beta^3\cdot\left(\frac{t_2}{t_1}\right)^2}-\beta\cdot\left(1-\frac{t_2}{t_1}\right)\right]+F_{ax,Rk}\cdot\Psi$
$F_{4,k}=$	43.295	kN	$\Rightarrow 1.05\cdot\frac{f_{h,1,k}\cdot t_1\cdot d}{2+\beta}\cdot\left[\sqrt{2\cdot\beta\cdot[1+\beta]+\frac{4\cdot\beta\cdot(2+\beta)}{f_{h,1,k}\cdot d}\cdot\frac{M_{y,Rk}}{t_1^2}}-\beta\right]+F_{ax,Rk}\cdot\Psi$
$F_{5,k}=$	43.295	kN	$\Rightarrow 1.05\cdot\frac{f_{h,1,k}\cdot t_1\cdot d}{1+2\cdot\beta}\cdot\left[\sqrt{2\cdot\beta^2\cdot[1+\beta]+\frac{4\cdot\beta\cdot(1+2\cdot\beta)}{f_{h,1,k}\cdot d}\cdot\frac{M_{y,Rk}}{t_2^2}}-\beta\right]+F_{ax,Rk}\cdot\Psi$
$F_{6,k}=$	7.733	kN	$\Rightarrow 1.15\cdot\sqrt{\frac{2\cdot\beta}{1+\beta}}\cdot\sqrt{2\cdot M_{y,Rk}\cdot f_{h,1,k}\cdot d}+F_{ax,Rk}\cdot\Psi$
$F_{V,Rk}=$	7.733	kN	
$F_{v,Rd}=$	**5.354**	kN	$\Rightarrow F_{V,Rd}=k_{mod}\frac{MIN(F_{i,k})}{\gamma_M}$

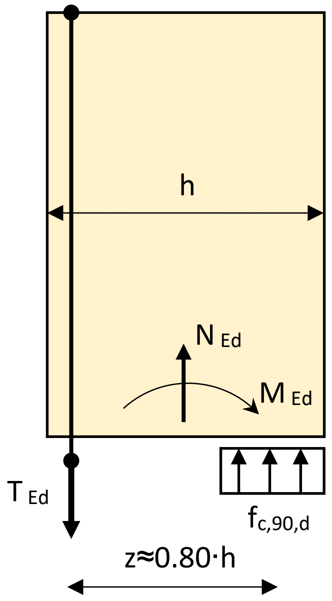
The resistance of metal bar M12 (S235) → $F_{vR,d,2}$=5.34kN

Total resistance of the shear wall is calculeted from the folowing equation:

$$V_{Rd,wall}= m\cdot F_{Rd,M12,1}+ n\cdot F_{Rd,M12,2}$$

m → number of threaded bars in shear wall
n → number of metal bars in shear wall

The bending resistance of the wall is verified using the following principles:



Although simplified, the presented model is recognized and well known in engineering practice.

The tie force is calculated as follows:

$$T_{Ed}=M_{Ed}/0.80\cdot h + 0.50\cdot N_{Ed}$$

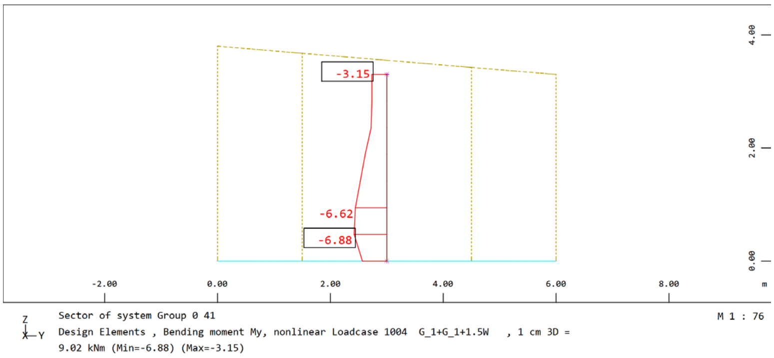
Tie tensile resistance of M8 threaded bar is:

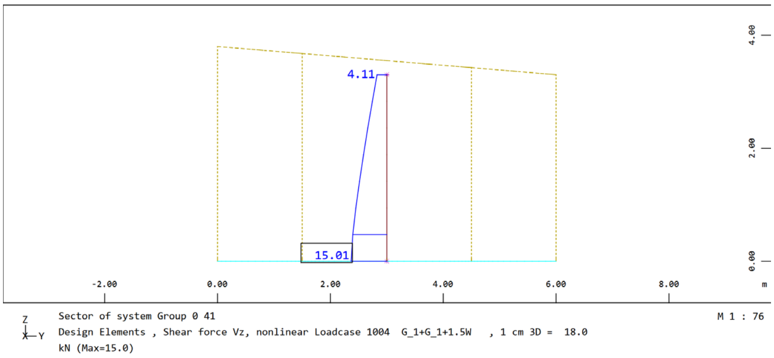
$F_{t,Rd}=A\cdot f_{yd}\cdot \gamma_{M2}^{-1}=1.13\cdot 64.0/1.25=$**56.32kN**

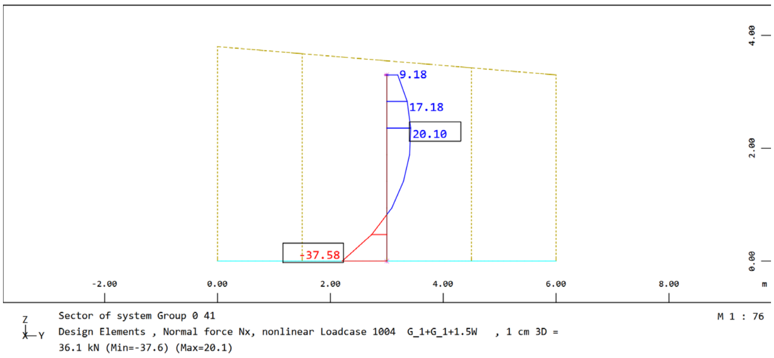
The verification for all walls types is listed in text.

SW/AT1 - verification

Shear walls forces are shown below:







Bending wall resistace is:

$F_{t,Ed}= M/0.80 \cdot h+0.50 \cdot N_{Ed} =6.88/[0.80 \cdot 6]=1.43kN$

$F_{t,Ed}/F_{t,Rd}=1.43/56.32=0.03<1.00$ ...OK

The shear wall resistace is:

$V_{Ed}=15.01kN$

$V_{Rd,wall}= m \cdot F_{Rd,M12,1}+ n \cdot F_{Rd,M12,2}= 1 \cdot 5.34+ 2 \cdot 10.35=26.04kN$

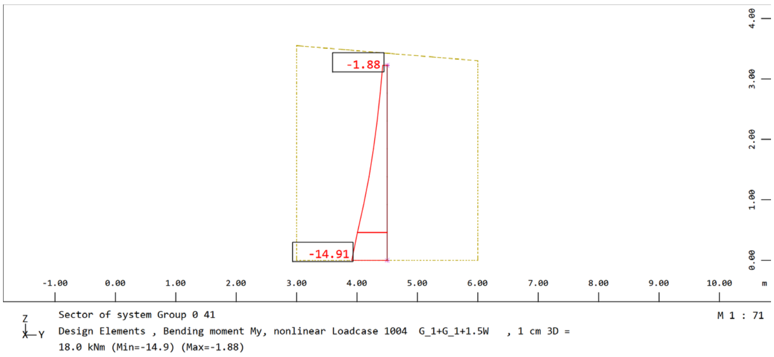
$V_{Ed}/V_{Rd}=15.01/26.04=0.58<1.00$ ...OK

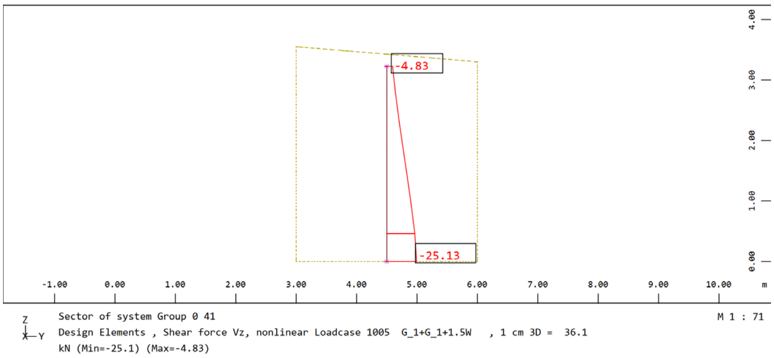
$$\eta = E_d / R_d$$

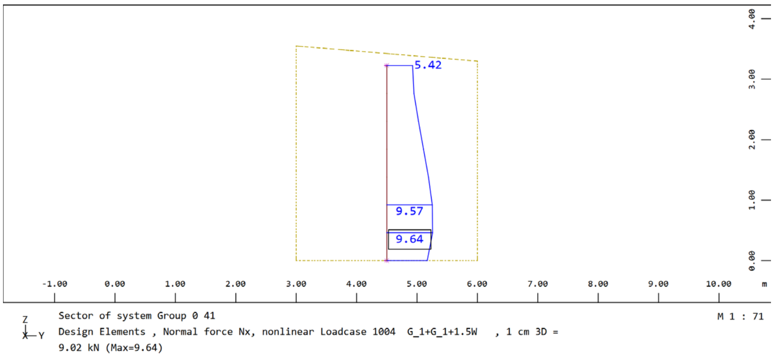
η =0.58	<	η_{lim}=1.00
OK!		

SW/AT3 - verification

Shear walls forces are shown below:







Bending wall resistace is:

$$F_{t,Ed}=M/0.80\cdot h+0.50\cdot N_{Ed}=14.91/[0.80\cdot 3]+0.50\cdot 9.64=11.03kN$$

$$F_{t,Ed}/F_{t,Rd}=11.03/56.32=0.20<1.00 \text{ ...OK}$$

The shear wall resistace is:

V_{Ed}=25.13kN

$$V_{Rd,wall}= m\cdot F_{Rd,M12,1}+ n\cdot F_{Rd,M12,2}= 1\cdot 5.34+ 2\cdot 10.35=26.04kN$$

$$V_{Ed}/V_{Rd}=25.13/26.04=0.96<1.00 \text{ ...OK}$$

$$\eta = E_d / R_d$$

η =0.96	<	η_{lim}=1.00
OK!		

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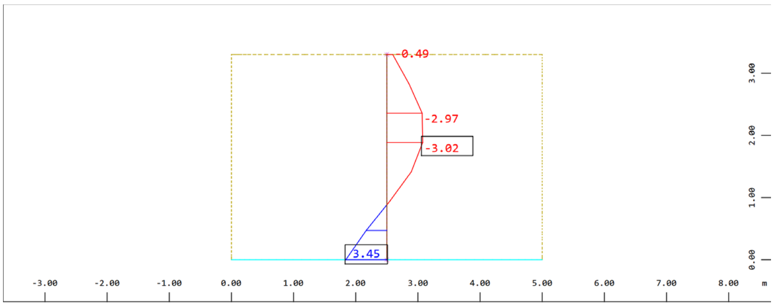
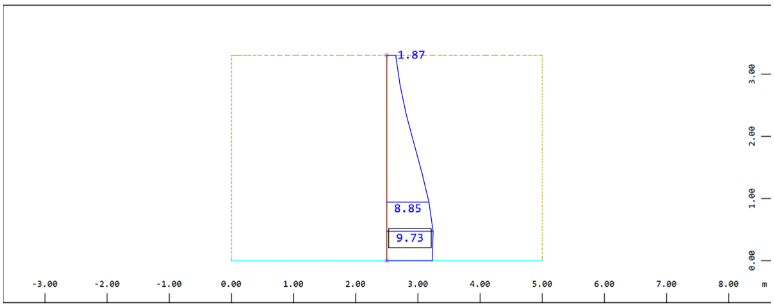
SW/AT4 - verification

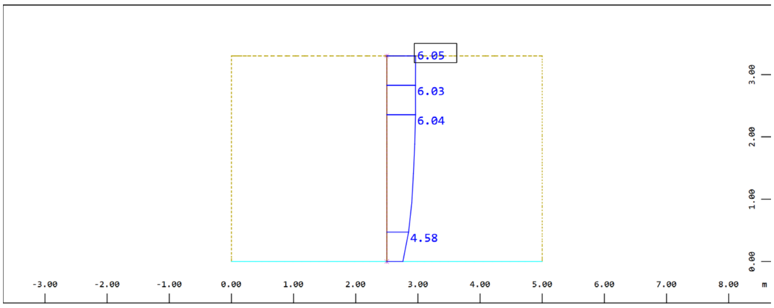
Shear walls forces are shown below:



Z X Sector of system Group 0 42 M 1 : 69
Design Elements , Bending moment My, nonlinear Loadcase 1003 G_1+G_1+1.5W , 1 cm 3D =
3.61 kNm (Min=-3.02) (Max=3.45)



Z X Sector of system Group 0 42 M 1 : 69
Design Elements , Shear force Vz, nonlinear Loadcase 1002 G_1+G_1+1.5W , 1 cm 3D = 9.02
kN (Max=9.73)



Z X Sector of system Group 0 42 M 1 : 69
Design Elements , Normal force Nx, nonlinear Loadcase 1003 G_1+G_1+1.5W , 1 cm 3D =
9.02 kN (Max=6.05)

Bending wall resistace is:

$$F_{t,Ed}=M/0.80\cdot h+0.50\cdot N_{Ed}=3.45/[0.80\cdot 5]+0.50\cdot 4.58=3.15kN$$

$$F_{t,Ed}/F_{t,Rd}=3.15/56.32=0.06<1.00 \ldots OK$$

The shear wall resistace is:

V_{Ed}=9.73kN

$$V_{Rd,wall}= m\cdot F_{Rd,M12,1}+ n\cdot F_{Rd,M12,2}= 1\cdot 5.34+ 2\cdot 10.35=26.04kN$$

$$V_{Ed}/V_{Rd}=9.73/26.04=0.37<1.00 \ldots OK$$

$$\eta = E_d / R_d$$

η =0.37	<	η_{lim}=1.00
OK!		

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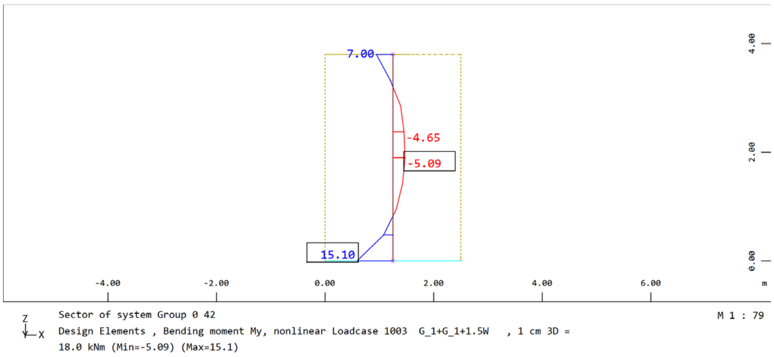
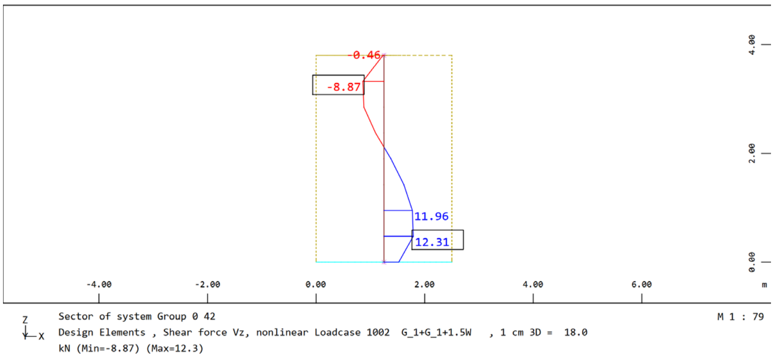
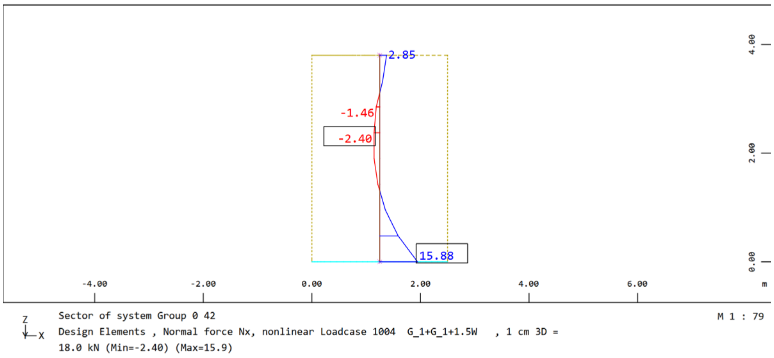
SW/AT6 - verification

Shear walls forces are shown below:







Bending wall resistace is:

$$F_{t,Ed}=M/0.80\cdot h+0.50\cdot N_{Ed}=15.10/[0.80\cdot 2.5]+0.50\cdot 15.88=15.49kN$$

$$F_{t,Ed}/F_{t,Rd}=15.49/56.32=0.26<1.00 \text{ ...OK}$$

The shear wall resistace is:

$V_{Ed}=12.31kN$

$$V_{Rd,wall}= m\cdot F_{Rd,M12,1}+ n\cdot F_{Rd,M12,2}= 1\cdot 5.34+ 2\cdot 10.35=26.04kN$$

$$V_{Ed}/V_{Rd}=12.31/26.04=0.47<1.00 \text{ ...OK}$$

$$\eta = E_d / R_d$$

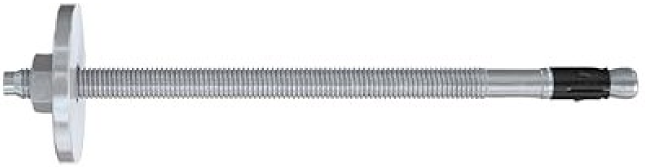
η =0.47	<	η_{lim}=1.00
OK!		

9 Uplift connections recomendations

To provide uplift resistance for the wind loads secundary and primary composide purlins needs to be connected to the wall structure, as well as wall structure needs to be connected to concerete foundations.

Therefore additional equipment needs to be used. For this equipment the following is recommended by designer:

(A) – Metal connectors (or simmilar steel element) $HxBxt_{min}$=2.5mm (Height can be variable)



(B) – Mechanical bolt – Example Fisher M8 Faz II L≥80mm or simmilar with same capacity

(C) - Wooden screw
(C) Type 1 – Ø5×70 mm

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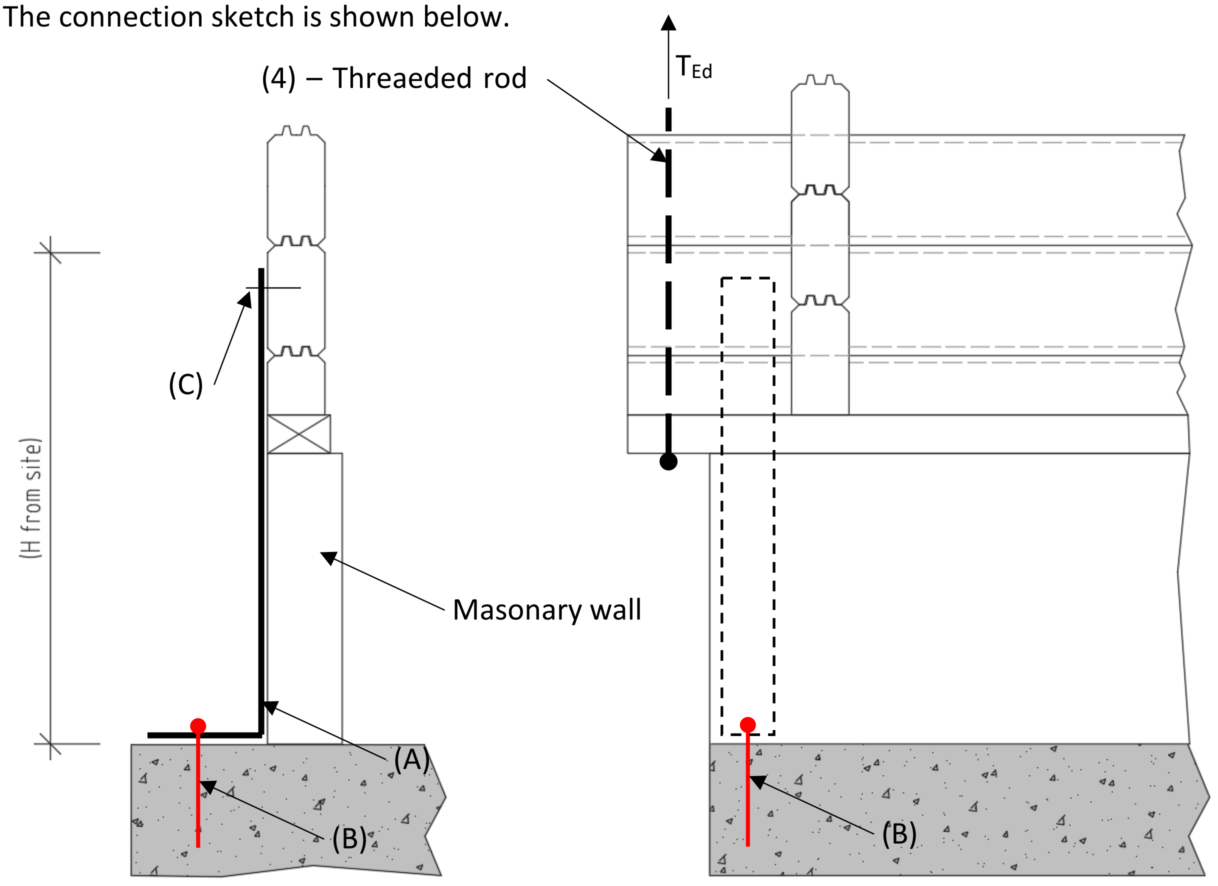
9.1 Uplift fixing 1 (UF1) - Wall purling connection with concrete slab

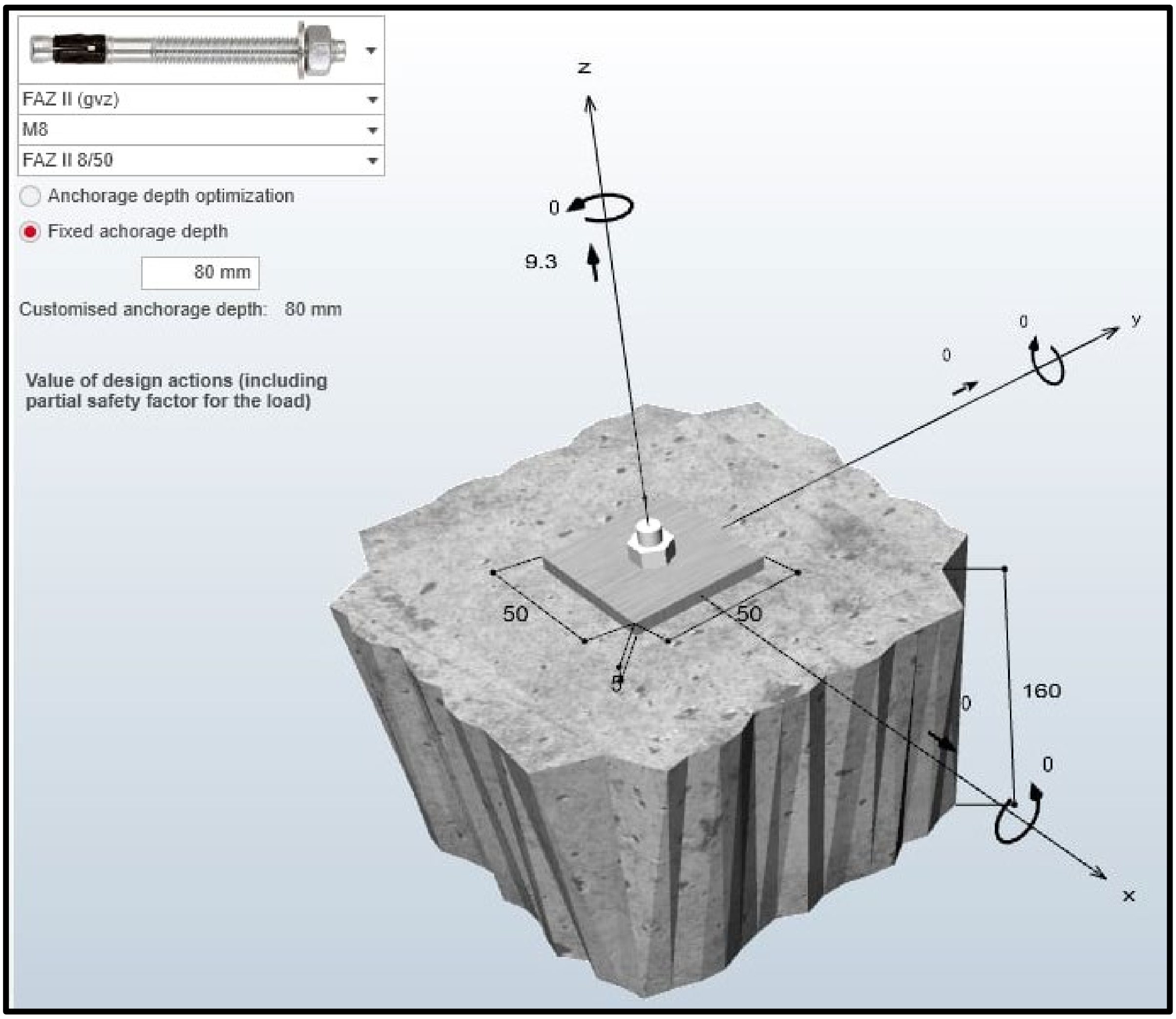
The connection sketch is shown below.



The connection resistance is calculated from the two criteria:

- (B) Anchor resistance
- (C) Screws resistance

Maximum anchor force is:



Steel failure		
Utilisation	βN,s	84.04 %
Considered anchors		1
NRk,s	kN	16.60
γMs	-	1.50
NRd,s	kN	11.07
N^{h}_{Ed}	kN	9.30

Pullout failure		
Utilisation	βN,p	99.64 %
Considered anchors		1
NRk,p	kN	14.00
ψc	-	1.000
γMp	-	1.50
NRd,p	kN	9.33
N^{h}_{Ed}	kN	9.30

Concrete cone failure		
Utilisation	βN,c	39.63 %
Considered anchors		1
$A^{0}_{c,N}$	mm²	57,600
$A_{c,N}$	mm²	57,600
$N^{0}_{Rk,c}$	kN	35.20
ψs,N	-	1.000
ψre,N	-	1.000
ψec,Nx	-	1.000
ψec,Ny	-	1.000
ψM,N	-	1.000
NRk,c	kN	35.20
γMc	-	1.50
NRd,c	kN	23.47
N^{g}_{Ed}	kN	9.30

$N_{c,Rd}$=9,30kN ⟶ Bolt/Concrete resistance

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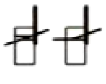
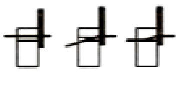
Maximum screws resistance is:

For thin plates		
$F_{1,ka}$ =	3.270	kN
$F_{1,kb}$ =	2.767	kN
$F_{min,1}$ =	**2.767**	kN
For tick plates		
$F_{1,kc}$ =	8.175	kN
$F_{1,kd}$ =	8.313	kN
$F_{1,ke}$ =	3.408	kN
$F_{min,2}$ =	**3.408**	kN





$$F_{v,Rk} = \min \begin{cases} 0,4 f_{h,k} t_1 d & \text{(a)} \\ 1,15\sqrt{2M_{y,Rk} f_{h,k} d} + F_{ax,Rk} \cdot \Psi & \text{(b)} \end{cases}$$

$$F_{v,Rk} = \min \begin{cases} f_{h,k} t_1 d & \text{(c)} \\ f_{h,k} t_1 d \left[\sqrt{2 + \frac{4M_{y,Rk}}{f_{h,k} d t_1^2}} - 1\right] + F_{ax,Rk} \cdot \Psi & \text{(d)} \\ 2,3\sqrt{M_{y,Rk} f_{h,k} d} + F_{ax,Rk} \cdot \Psi & \text{(e)} \end{cases}$$

$F_{V,Rk}$ = 2.880 kN $\qquad F_{V,Rk} = \min(F_{min,i})$

$F_{V,Rd}$ = **1.772** kN $\qquad F_{V,Rd} = k_{mod} \frac{F_{Rd,k}}{\gamma_M}$

$F_{V,Rd}$=1.77kN

4 screws adopted per connector on the purlins side so the total force is equal to:

$N_{w,Rd}$=4·1.77=7.08kN → Screw/Wood resistance

Connection capacity is ilustrated below:

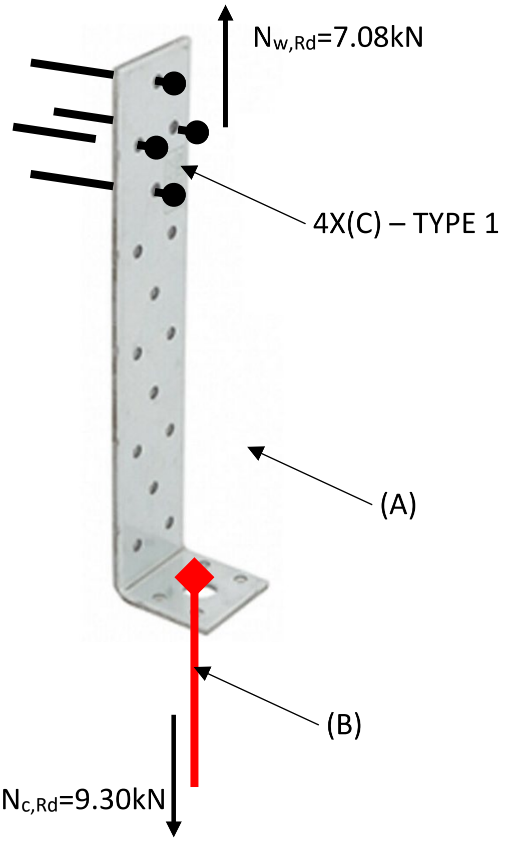


Fig. 19: UF1 – connection capacity

T_{Rd}=MIN(9.30;7,08)=7,08kN → Connection resistance

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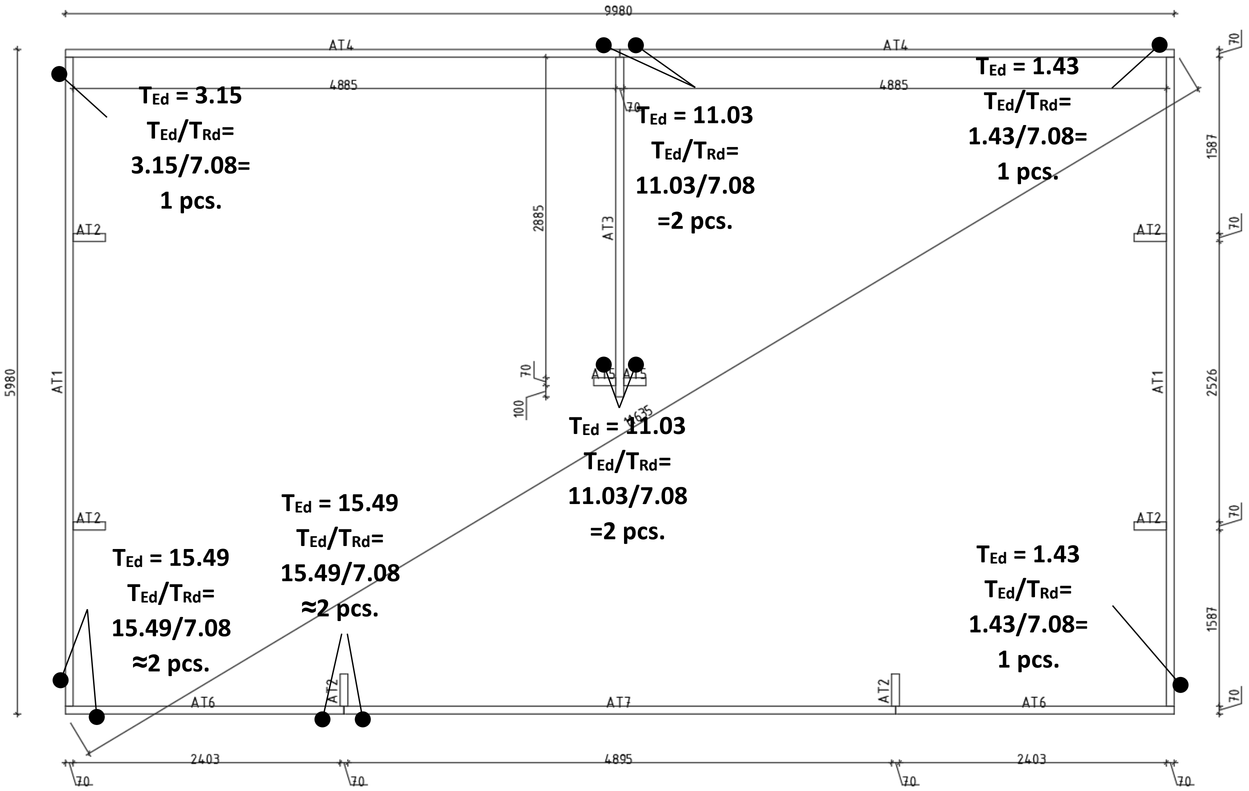
Number of metal connectors per threaded bar M12 is calculated according to equation:

$$n = T_{Ed}/T_{Rd}$$

The calculated number of connectors is shown bellow:



UF1 connectoN:

- (A) – Metal connectors 11 pcs.
- (B) – M8 bolts 11pcs.
- (C) – Type 1 screws 44pcs.

η =1.000	=	η lim=1.000
OK!		

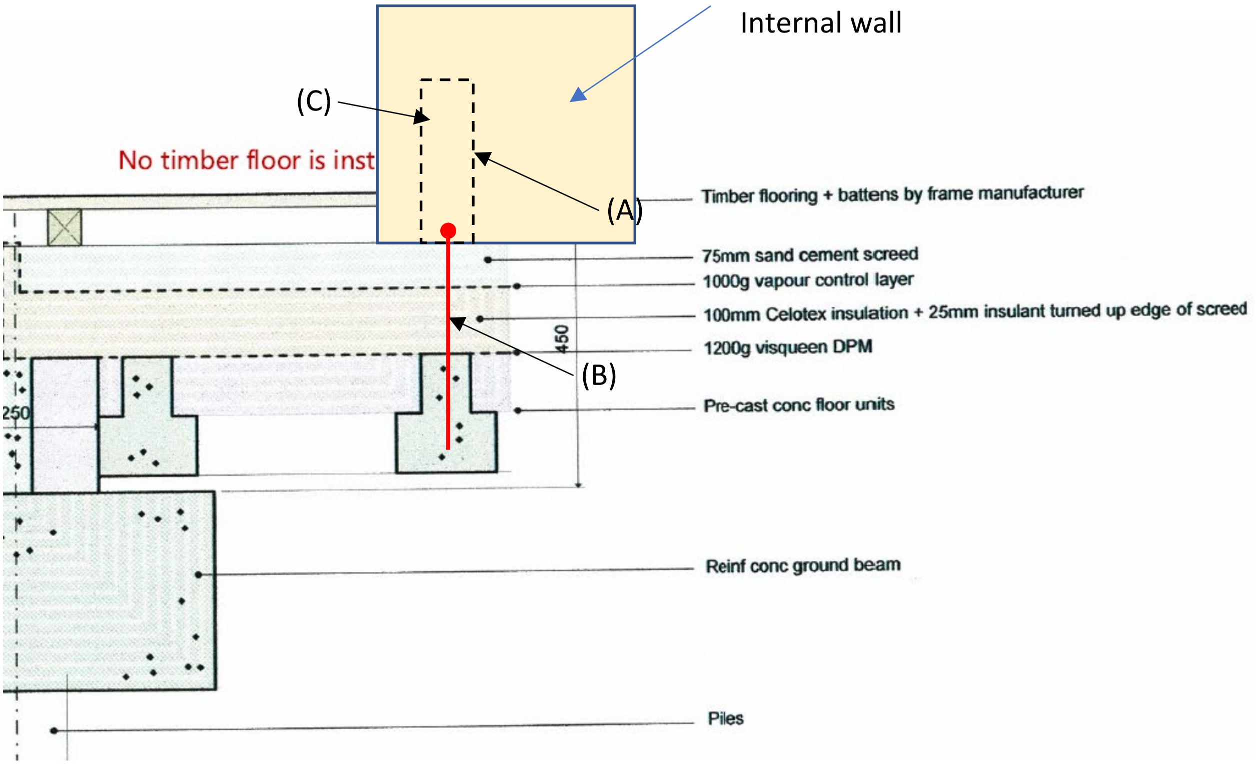


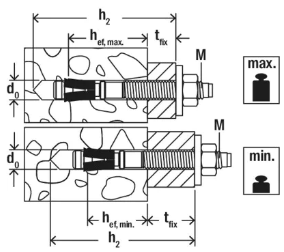
Fig. 20: Additional detail for internall wall

Internall wall can be fixed with longer bold directly connected to the pre-cast concerete floor units. Bolt details that can be used are:

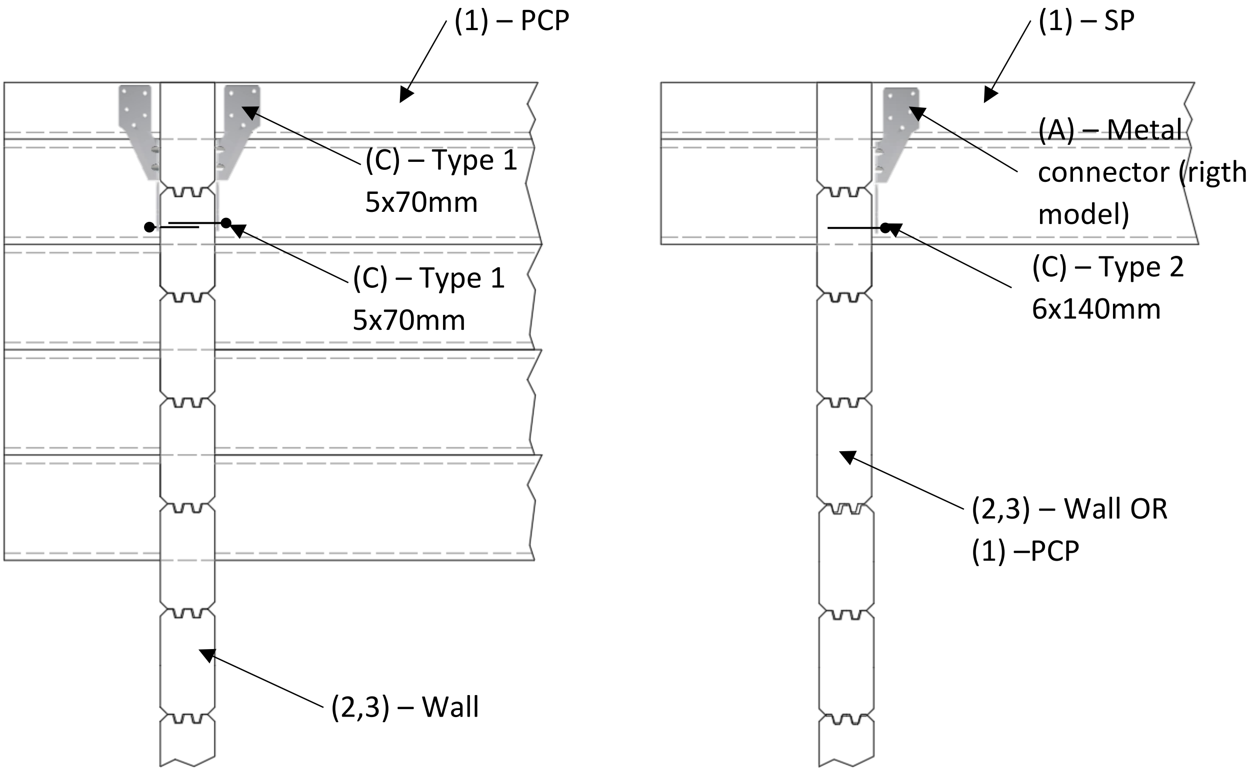
FAZ II Plus 12/180 GS ZP → Art.-No. 564659	12 [mm]	270 [mm]	180 / 200 [mm]	280 [mm]	M12 x 186 [mm]

Attributes

ETA-approval	✓
Seismic-Approval	C1 / C2
Drill diameter (d_0)	12 mm
Min. drill hole depth for through fixings (h_2)	270 mm
Max. usable length hef,stand./hef,min. (t_{fix})	180 / 200 mm
Anchor length (l)	280 mm
Thread (ø x length)	M12 x 186 mm
Washer (outer diameter x thickness)	44 x 4 mm
Width across nut	19 mm
Packaging	–
Amount	20 pcs
GTIN (EAN-Code)	4048962462845



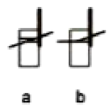
9.2 Uplift fixing 2 (UF2) – Rafters / wall connection

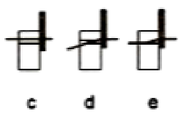


For the uplifting force of the secondary and primary purlins, verticaly bored wooden screwer type 2, d=6mm L=140mm is invisioned. The anchor uplift resistance is calculated as follows:

Maximum screws resistance is:

For thin plates		
$F_{1,ka}$ =	*3.270*	*kN*
$F_{1,kb}$ =	*2.767*	*kN*
$F_{min,1}$ =	***2.767***	*kN*
For tick plates		
$F_{1,kc}$ =	*8.175*	*kN*
$F_{1,kd}$ =	*8.313*	*kN*
$F_{1,ke}$ =	*3.408*	*kN*
$F_{min,2}$ =	***3.408***	*kN*





$$F_{v,Rk} = \min \begin{cases} 0,4 f_{h,k} t_1 d & \text{(a)} \\ 1,15\sqrt{2M_{y,Rk} f_{h,k} d} + F_{ax,Rk} \cdot \Psi & \text{(b)} \end{cases}$$

$$F_{v,Rk} = \min \begin{cases} f_{h,k} t_1 d & \text{(c)} \\ f_{h,k} t_1 d \left[\sqrt{2 + \frac{4M_{y,Rk}}{f_{h,k} d t_1^2}} - 1\right] + F_{ax,Rk} \cdot \Psi & \text{(d)} \\ 2,3\sqrt{M_{y,Rk} f_{h,k} d} + F_{ax,Rk} \cdot \Psi & \text{(e)} \end{cases}$$

$F_{V,Rk}$ = *2.880* *kN* $\quad F_{V,Rk} = min(F_{min,i})$

$F_{V,Rd}$ = ***1.772*** *kN* $\quad F_{V,Rd} = k_{mod} \frac{F_{Rd,k}}{\gamma_M}$

$F_{V,Rd}$=1.77kN

2 screws addopted per side of bracket:

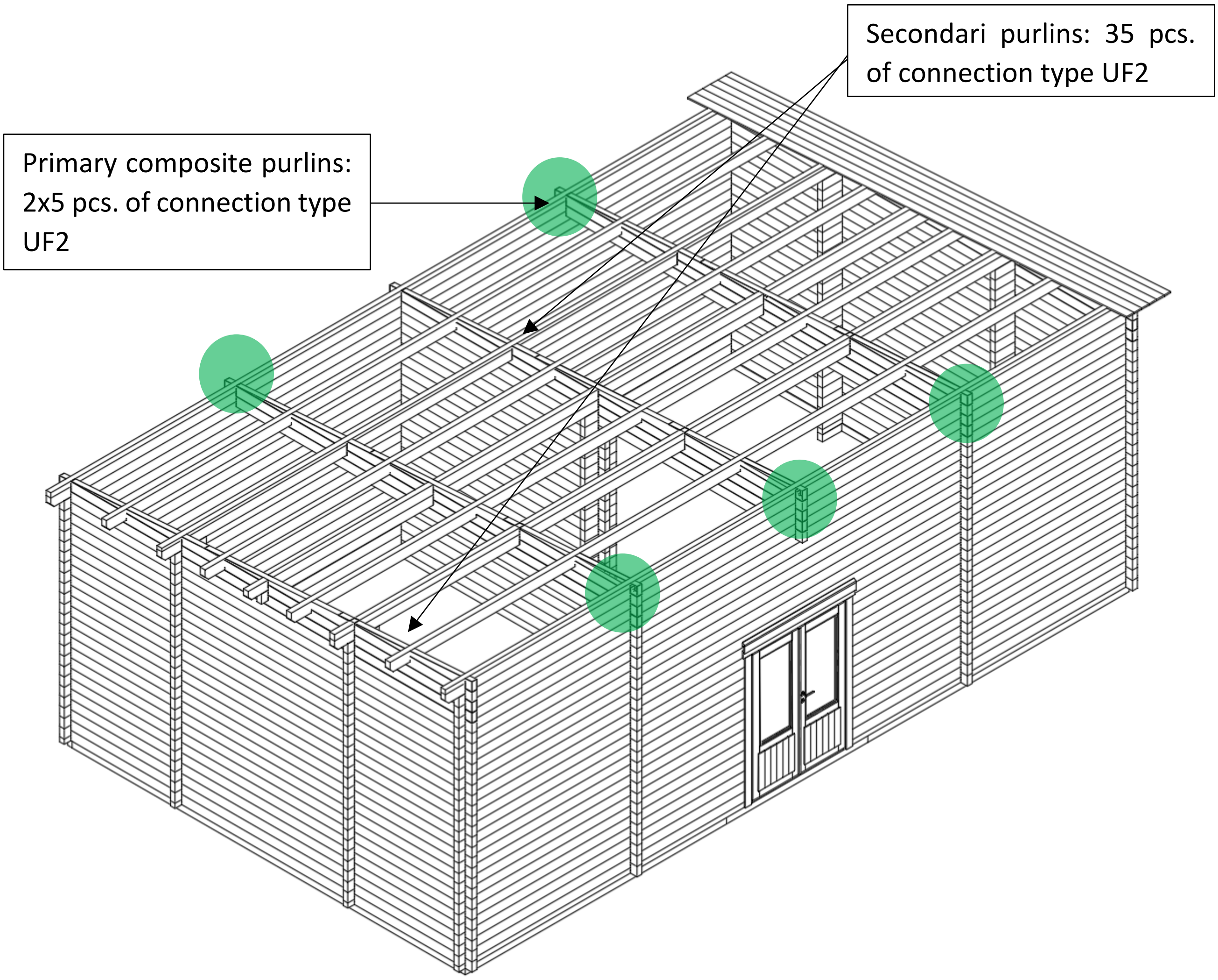
T_{Rd}=2·1.77=3.54kN

The maximum uplift forces are:

$T_{Ed,SP}$ = 3kN ⟶ Secondary purlins 1 bracket/purlin T_{Ed}/T_{Rd}=0.85<1.00 …Ok

$T_{Ed,PCP}$ = 6.5kN ⟶ Primary composite purlins 2 bracket/purlin T_{Ed}/T_{Rd}=0.91<1.00 …Ok

T_{Ed}/T_{Rd}=0.85<1.00

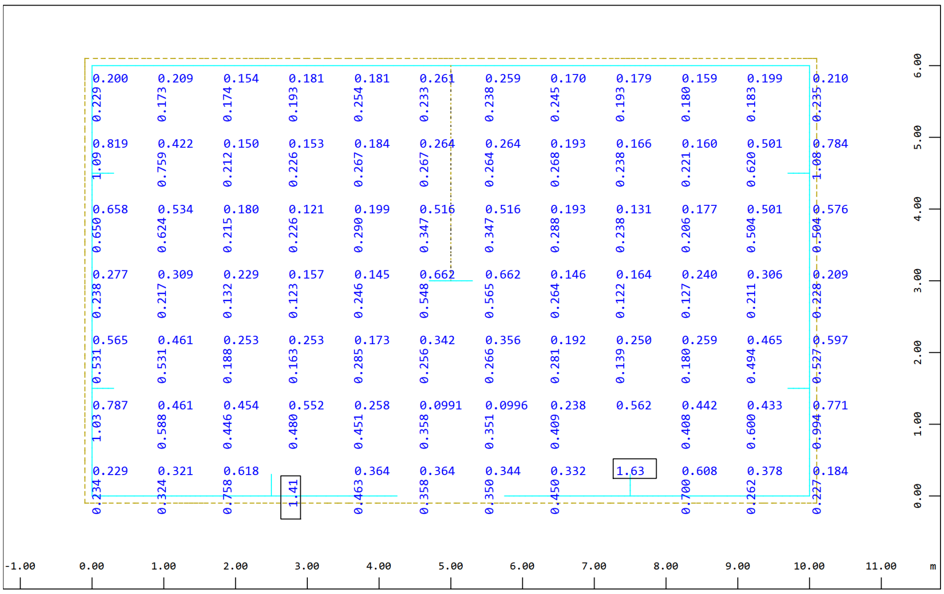


η =0.910	<	η lim=1.000
OK!		

10 Design of concrete foundation

The reinforcement calculation is performed automatically by the software, and the results are displayed below:

Sector of system Group 60
Quadrilateral Elements , upper Reinforcements in Node in cm2/m, Design Case 1 ULS design
(Max=2.10)
M 1 : 73



Sector of system Group 60
Quadrilateral Elements , lower Reinforcements in Node in cm2/m, Design Case 1 ULS design
(Max=1.63)
M 1 : 73

Reinforcement provided in the slab is:

Top layers ortogonal: ϕ8/15 → a_{prov}=3.35cm^2/m'>2.10 cm^2/m'

Bottom layers ortogonal: ϕ8/15 → a_{prov}=3.35cm^2/m'>1.63 cm^2/m'

$$\eta = E_d / R_d$$

η =0.62	<	η_{lim}=1.00
OK!		

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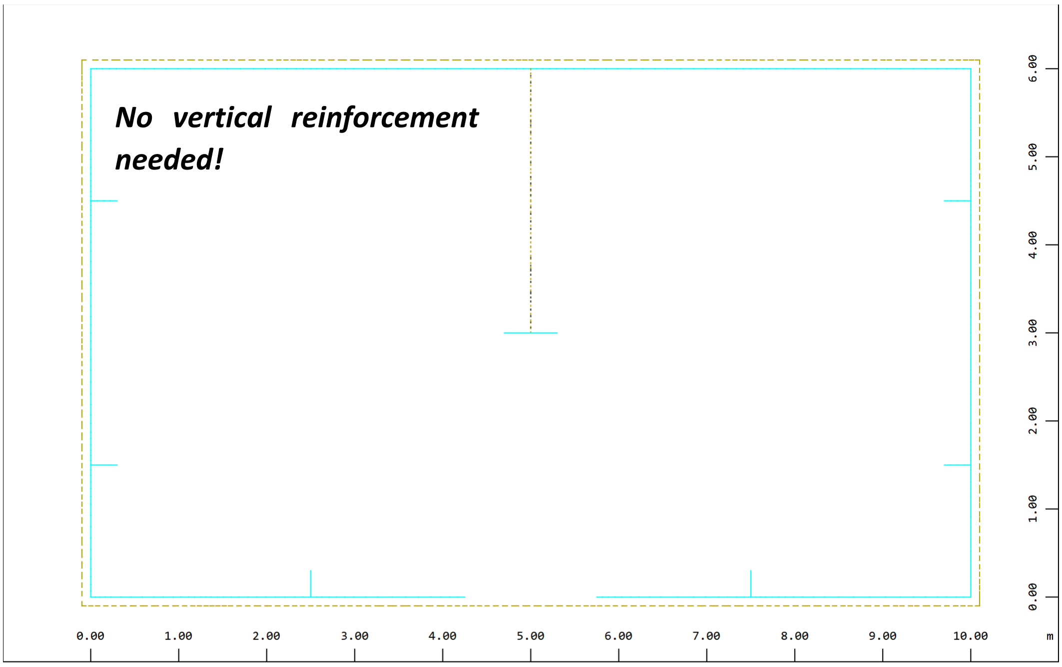
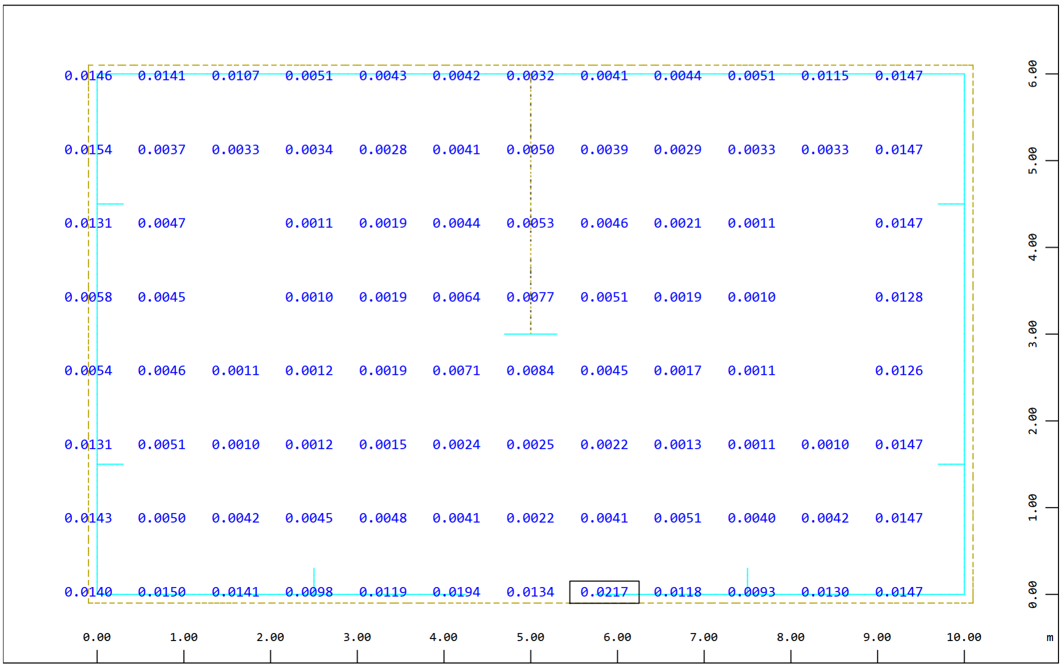


Sector of system Group 60 M 1 : 67
Quadrilateral Elements , Shear reinforcement in Node in cm2/m2, Design Case 1 ULS design
(Max=0)



Sector of system Group 60 M 1 : 68
max.VED / VRDmax in Node in -, Design Case 1 ULS design (Max=0.0217)

$$\eta = E_d / R_d \longrightarrow V_{Ed}/V_{Rd,max}=0.022$$

η =0.02	<	η_{lim}=1.00
OK!		

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11 Summary

The structural analysis of the modular wooden house was made in accordance with European standards (EC). The designer has peformed a detailed analysis of the structure and it can be concluded that it has all the necessary load capacity to overtake the loads imposed by modern regulations.

Element	**Dominant failure mode**	**Utilisation level (%) $\eta = E_d / R_d$**	**Status**
Secondary purling	Deflection check	31%	**Satisfactory**
Primary composite purling	Mechanical coupling	96%	**Satisfactory**
Walls	Bending	39%	**Satisfactory**
Shear walls	Mechanical coupling	96%	**Satisfactory**
Concrete slab	Bending	62%	**Satisfactory**
Uplift connections	-	100%	**Satisfactory**

E_d ⟶ Effects

R_d ⟶ Resistance

The capacity of the timber structure is **Satisfactory**.

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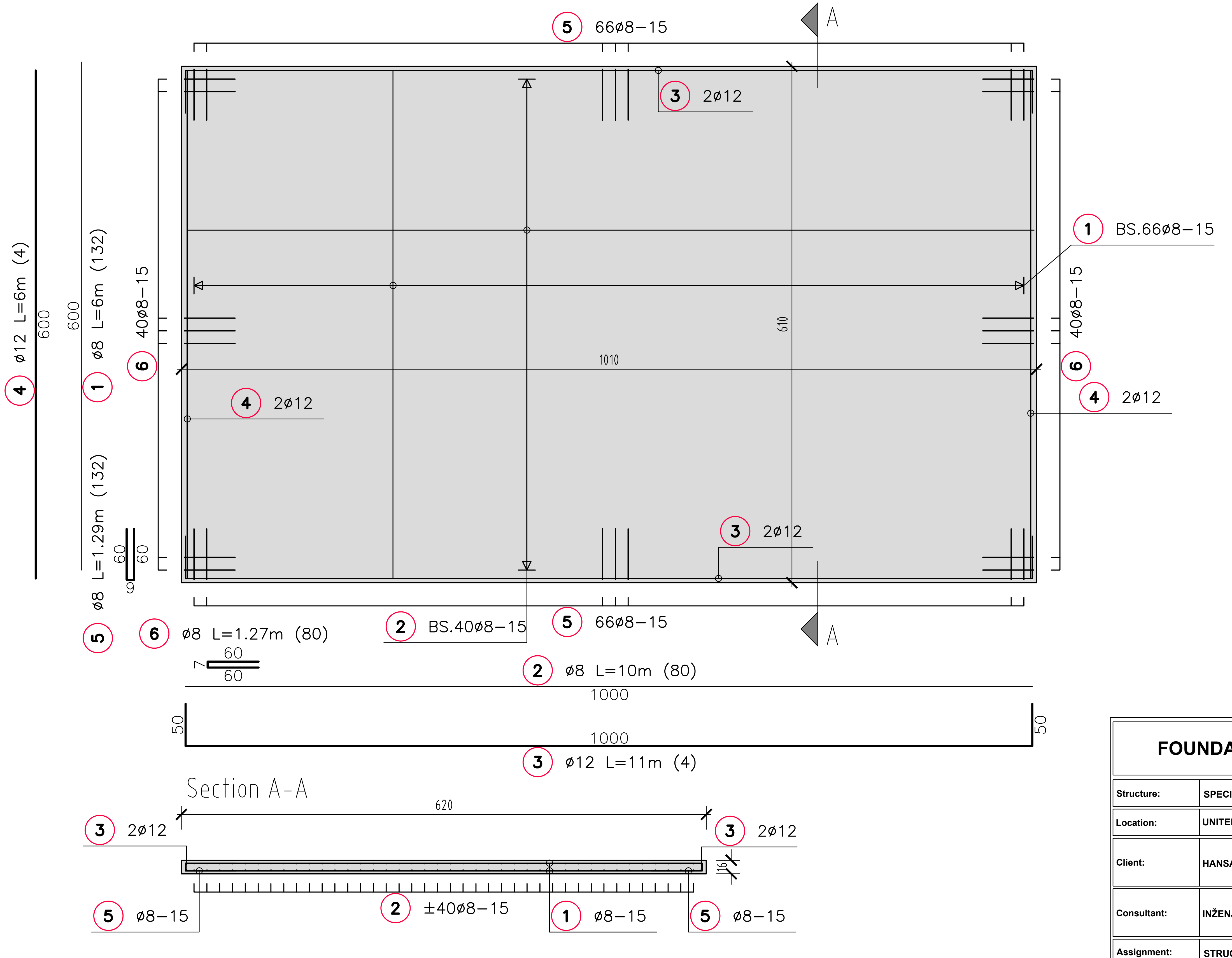
II- DRAWINGS

Foundation reinforcement
Sc. 1:50



Pos	Shape and dimensions [cm]	Ø [mm]	L1 [m]	n [pcs.]	L (total) [m]	g (weight) [kg]
SlabRebar (1 pcs.)						
1	610	8	6.10	132	805.20	318.05
2	1010	8	10.10	80	808.00	319.16
3	50 / 1000 / 50	12	11.00	4	44.00	39.07
4	600	12	6.00	4	24.00	21.31
5	60 / 9 / 60	8	1.29	132	170.28	67.26
6	60 / 7 / 60	8	1.27	80	101.60	40.13

Recapitulation			
Ø [mm]	L (total) [m]	Unit w. [kg/m']	Weight [kg]
S500, Ø <= 12 mm			
8	1885.08	0.40	744.61
12	68.00	0.89	60.38
Ukupno (S500, Ø <= 12 mm)			804.99
Ukupno			804.99

FOUNDATION REINFORCEMENT DRAWING

Structure:	SPECIAL TODHUNTER 620X1020 70
Location:	UNITED KINGDOM, PENWORTHAM
Client:	HANSA24 GROUP OÜ
Consultant:	INŽENJERSKA GRUPACIJA GLOBAL LLC MOSTAR
Assignment:	STRUCTURAL DESIGN OF SPECIAL TODHUNTER 620X1020 70
Date: October, 2025.	scale: 1:50

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drawing no.: 01.