

DEGREE OF FREEDOM

PROJECT	Frederikke – Rehabilitering fasade og tekniske anlegg					
PHASE	For anbud					
TITLE	Limtre søyler i fasaden, gesims og baldakin - beregningsrapport					
CLIENT	Universitetet i Oslo					
DATE	29.05.2026					
REPORT ID	25086-R01					
SAMMENDRAG Rapporten omhandler beregning og verifisering av nye limtresøyler og gesims og innfesting av disse i fasaden på Frederikkebygget på Blindern. I tillegg er det gjort en beregning av innfesting for baldakin (demonteres/remonteres) på fasaden mot Lucy Smiths Hus. Alle beregninger er basert på gjeldende standarder og lovverk. FORBEHOLD - De beregnede sideveis deformasjonene pga vindlast er betydelige for glassfasaden og det må tas hensyn til ved prosjektering av glassfasaden. Bevegelseskapasiteten til innfesting, pakninger og fuger skal verifiiseres slik at deformasjonene kan opptas uten å påføre sekundærspenninger eller utilsiktet lastoverføring til glassfeltene. - Eksisterende armering i prefab bjelkene må detekteres før innfesting av nye stålplater slik at disse ikke kommer i konflikt med eksisterende armering. - Baldakin innfesting: Mål og vekt på baldakin må kontrolleres. Innfesting er beregnet med tanke på demontering og remontering av eksisterende baldakin.						
01	29/05/2026	Parapet design and connections details	42	JDO	FI	ER
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1 SCOPE

Degree of Freedom has been engaged to carry out the design and structural verification of the internal timber mullions and the connection for the new parapet forming part of the new façade structure associated with the façade and roof refurbishment of the BL03 Frederikke building at the University of Oslo.



Figure 1-1. Frederikke building location within the Oslo University campus.

The project intervention is located on the east and west façades, which are the two longest elevations of the building.

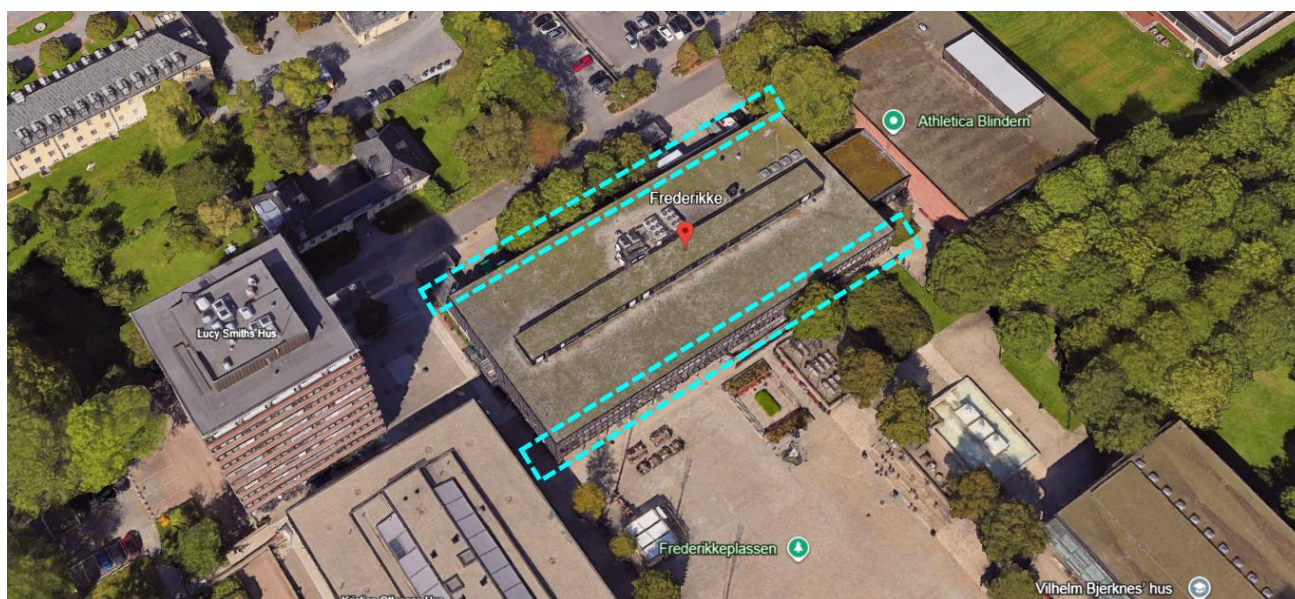


Figure 1-2. Frederikke building façades under evaluation.

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Figure 1-3. View from the exterior of the current Frederikke building façade.

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2 PROJECT BACKGROUND

The exterior of the Frederikke building, including the façades and the main roof, has largely remained unchanged since its completion in 1961, with only minor exceptions. The ground-floor façade has undergone a prolonged process of deterioration and is currently in a degraded condition as a result of long-term exposure to weathering, giving rise to an almost urgent need for rehabilitation.

The load-bearing structures affected by the refurbishment project comprise the roof supporting structure and the load-bearing façade structure at ground floor level (first floor). The ground-floor façade load-bearing system consists of glued laminated timber columns with a cross-section of 5" × 5", spaced at 1,25 m centres. According to the original drawings attached below, these columns are vertically supported on a timber sill beam located at the front edge of the concrete floor slab (Detail 2). Horizontal stability against actions perpendicular to the façade plane is provided by a steel IPE beam (Detail 1).

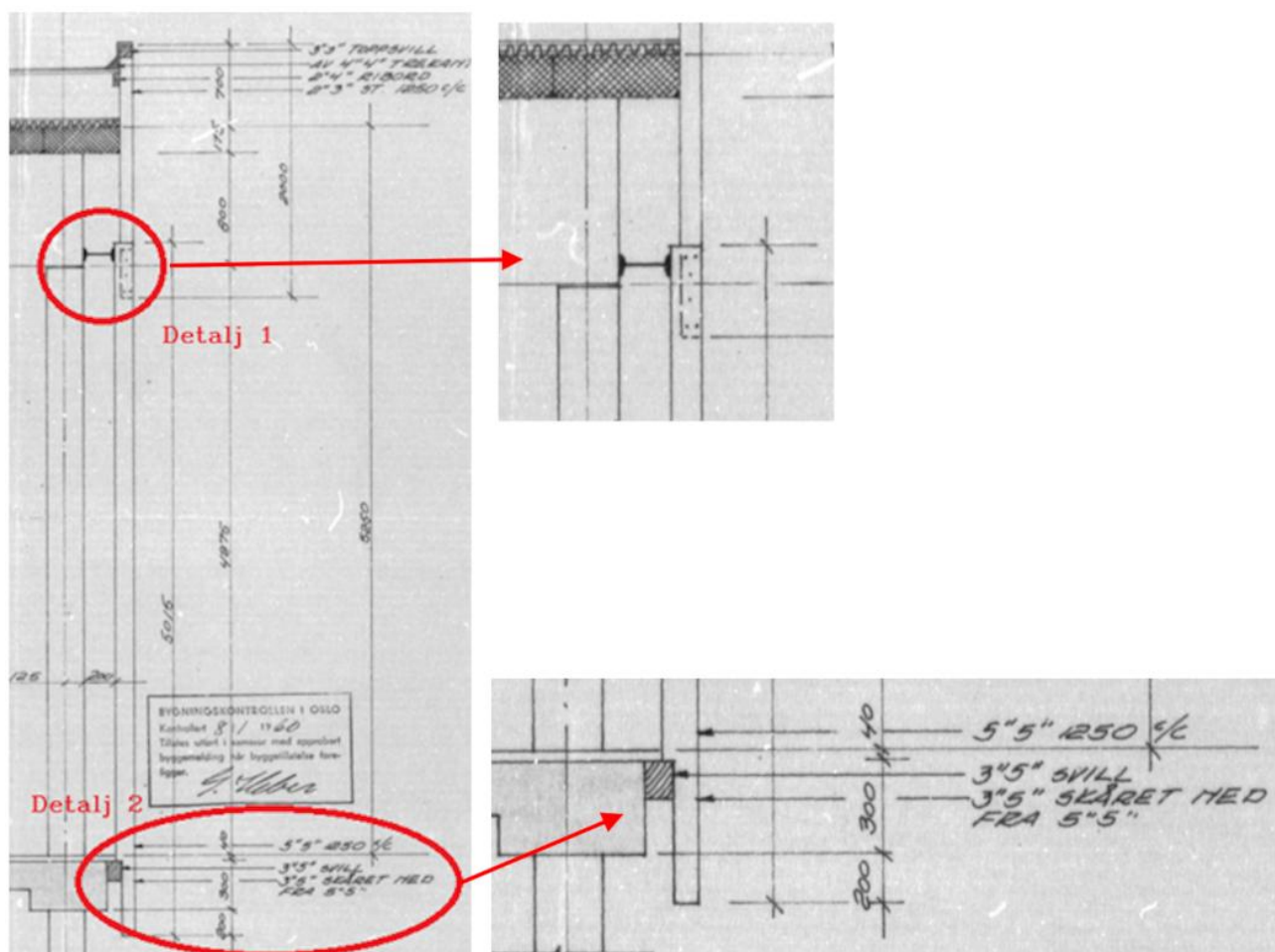


Figure 2-1. Existing drawings

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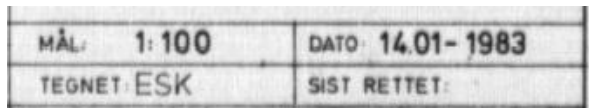
2.1 MATERIALS IN THE EXISTING STRUCTURE

Below is a chronology of codes of practice for reinforced concrete design in Norway. The oldest code available is 1926 – *Forskrift for betongarbeider*.

Norsk betongstandard historie

1893 Ingeniørkaptein Kolderups lærebok
1926 Forskrift for betongarbeider
1939 NS 427, Regler for utførelse av arbeider i armert betong
1963 NS 427 A, Del 1 til Del 5
1973 NS 3473, Første utgave
1974 NS 3474, Materialer, Utførelse og kontroll
1986 NS 3420 del L erstatter NS 3474
1989 NS 3473, Hovedrevisjon, 3. utgave
2001 NS-EN 206-1, Betong med NA erstatter NS 3420 del L
2003 NS 3465, Utførelse av betongkonstruksjoner, erstatter NS3420 Del L
2003 NS 3473, 6. utgave
2008 NS-EN 1992-1-1 med NA
2010 NS 3473 og NS 3465 trekkes tilbake
2010 NS-EN 13670 Utførelse av betongkonstruksjoner med NA

The project data is around the 80's



As the existing concrete strength has not been found, it is taken B300 as the most restrictive one.

Tabell 2.1.2—1 — Betongens karakteristiske trykkfasthet, f_{ck}

Byggeår	NS 427 (av 1939)	NS 427A (av 1962)	NS 3473 (1973-2003)	NS 3473 (2003-2010)		NS-EN 1992-1-1 (NA 3.1.2)	
	Betongkvalitet	Betongkvalitet	Fasthetsklasses	Fasthetsklasses	f_{cn} (N/mm ²)	Fasthetsklasses	f_{ck} (N/mm ²)
Før 1920	C-betong	B 200	C 15	B 10	11,2	B 12	12
1920-1945	B-betong	B 250	C 20	B 16	14,0	B 16	16
Etter 1945	A-betong	B 300	C 25	B 20	16,8	B 20	20
		B 350	C 30	B 25	20,3	B 25	25
		B 400	C 35	B 28	22,4	B 28	28
		B 450	C 40	B 32	25,2	B 32	32
			C 45	B 35	27,3	B 35	35
		B 600	C 55	B 45	34,3	B 45	45

Figure 2-2. Concrete strength

Hence it is assumed $f_{ck} = 20 \text{ N/mm}^2$

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Tabell 2.1.1—1 — Materialfaktorer for betong og armeringsstål

Materiale	Materialfaktorer γ_c og γ_s			
	Bruddgrensetilstand	Brukgrensetilstand	Ulykkesgrensetilstand	Utmattingsgrensetilstand
Armert betong, γ_c	1,50	1,0	1,20	1,50
Armering før 1920, γ_s	1,50 ⁽¹⁾	1,0	1,30	1,30
Armering 1920-1955, γ_s	1,30 ⁽²⁾	1,0	1,20	1,20
Armering 1955-2010, γ_s	1,25	1,0	1,10	1,15
Armering etter 2010, γ_s	1,15	1,0	1,00	1,15

(1) For brudekker uten tegn til armeringskorrosjon av betydning for bæreevne i kritiske snitt, kan $\gamma_s = 1,30$ benyttes.

(2) For bruer uten tegn til armeringskorrosjon av betydning for bæreevne i kritiske snitt, kan $\gamma_s = 1,25$ benyttes.

Table 2-1 Material factors for concrete and reinforcement

Material factor for concrete: $\gamma_c = 1.5$

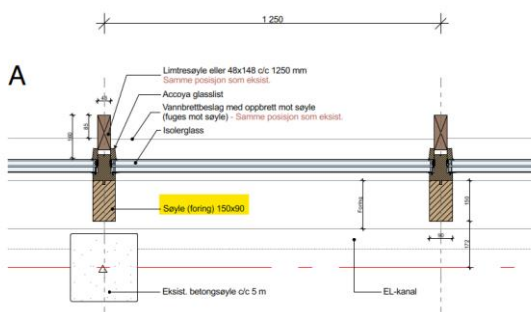
2.2 DOCUMENT REFERENCES

- [1] Architectural drawings and details - A50-5 Detaljer yttervegg - div. alternativer, received on 06/01/2026

3 NEW FAÇADE BUILD-UP

The architectural information provided includes two alternatives for the timber mullion, with a fixed height of 150 mm and width of 90 mm. In addition, a façade section has been provided showing the different façade components and their arrangement.

Forslag 04 horisontal detalj (foretrukket)



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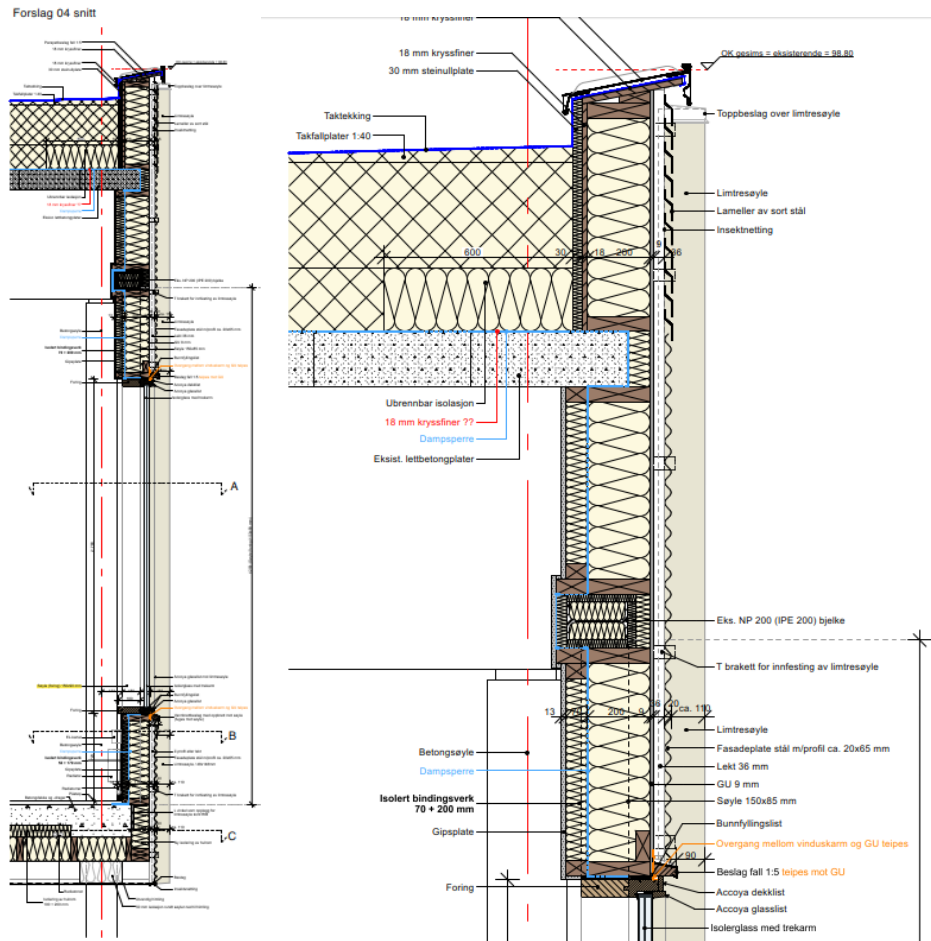


Figure 3-1. New façade alternatives and build-up provided by ARK.

In the new façade configuration, steel T-shaped angle profiles are installed along the front edge of the floor slab to provide vertical support for the curtain wall (Detail 2), while the existing IPE beam is maintained to ensure the horizontal stability of the system (Detail 1).

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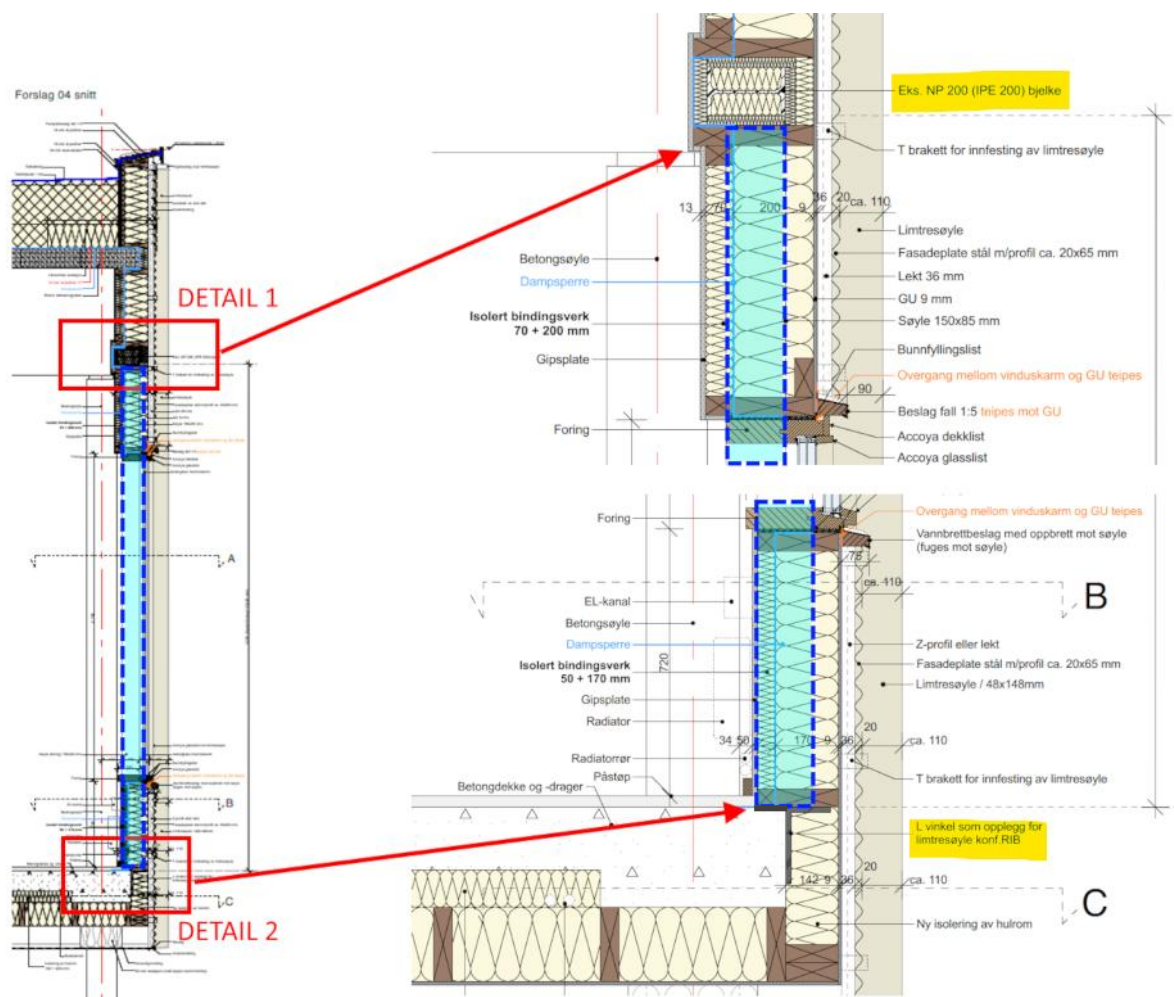


Figure 3-2. ARK proposal - New façade detailing.

4 DESIGN ASSUMPTIONS AND DISCLAIMERS

Degree of Freedom (DOF) is responsible for the structural design and structural verification of the internal timber mullion forming part of the new façade system of the building and the connection for the new parapet to the existing structure.

In order to carry out this task, a number of assumptions and observations have been adopted, which are considered relevant and are therefore highlighted for coordination and information purposes with the other disciplines.

- It is assumed that the timber mullion supports the self-weight of all façade elements located above it until the roof slab level, including the weight of the glazing.
This assumption is made on the basis that all façade elements located above the roof slab level are assumed to be supported directly by the slab itself. However, the load transfer mechanism for the elements within the intermediate zone in between the top of the mullion and the roof slab (with an approximate height of 1 meter) is not clearly defined; therefore, and in order to remain on the safe side, their weight is conservatively assumed to be carried by the timber mullion under design.

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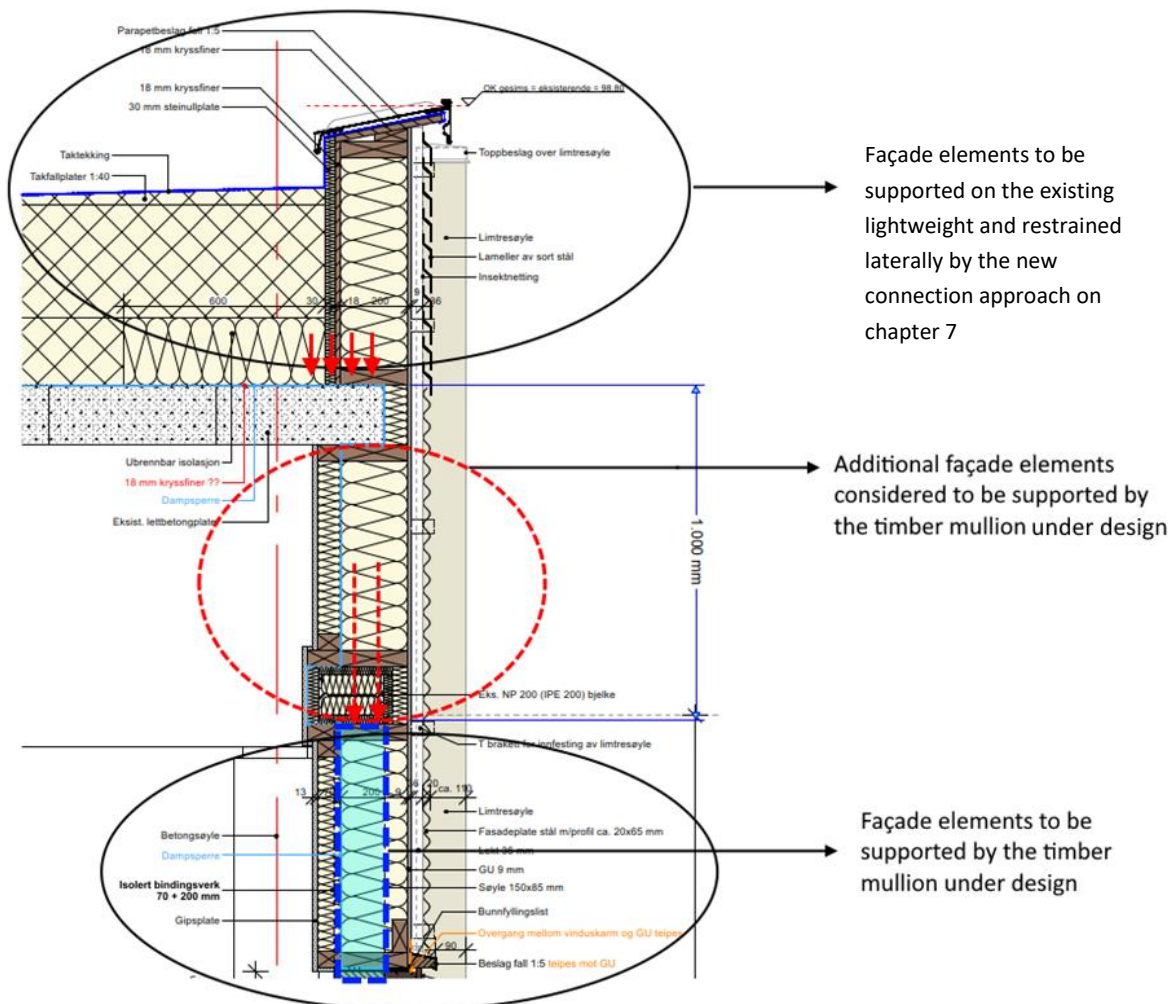


Figure 4-1. Vertical load considered into the timber mullion design.

- As for the structural behaviour of the timber mullion under design, it is assumed to be vertically and horizontally restraint at its base while only horizontally restraint at the top, where the existing IPE profile is located. This mechanism it is coherent with the original design and with the provided drawings.
A steel plate is proposed as the connection element between the timber mullion and the existing IPE profile. The plate will be welded to the flange of the IPE and bolted to the timber member through vertically slotted holes, allowing vertical movement and thereby preventing the transfer of vertical loads to the IPE.
- Based on the drawings and details provided to date, the intended load path and fixing arrangement are not clear. The key structural issue for this zone is the portion of the façade that projects above the roof slab level and therefore behaves as a cantilever. An indicative sketch is attached, showing one possible solution consistent with the situation described.

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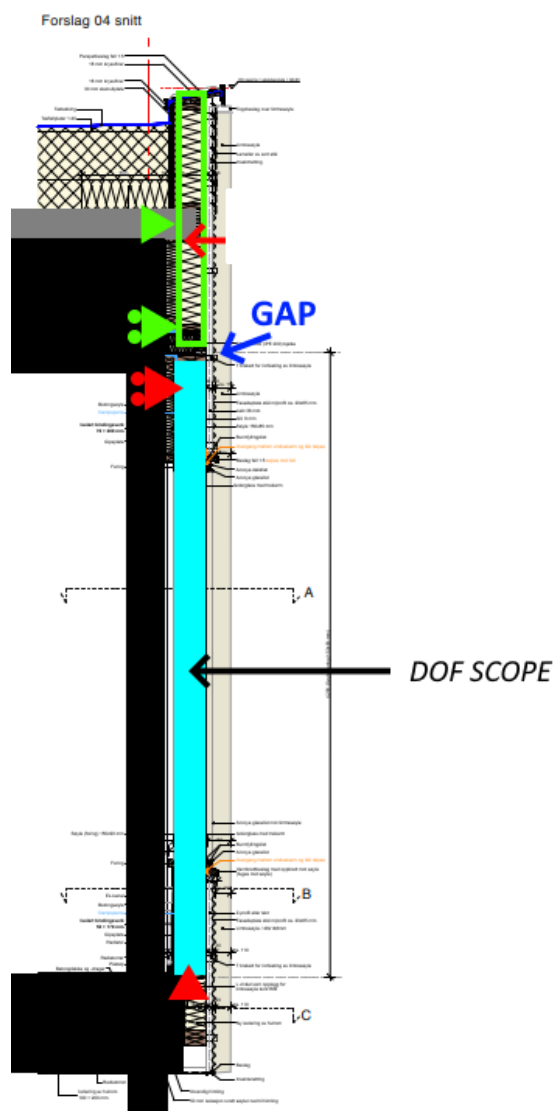


Figure 4-2. Illustrative sketch of a possible total façade solution.

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- Based on the information provided by RIBr drawings, no fire resistance requirements apply to the façade elements within the project scope; therefore, no fire design verifications have been performed.

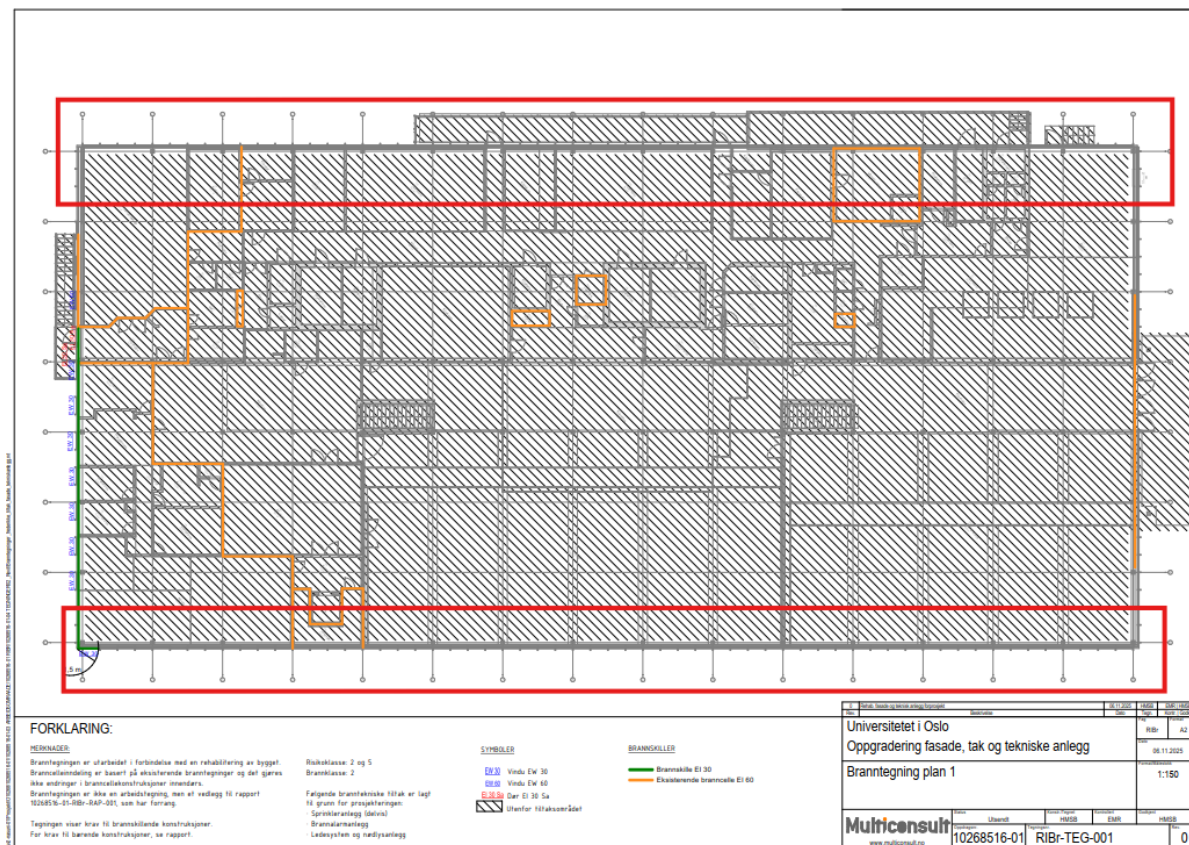


Figure 4-3. Façade areas under evaluation in RIBr drawing from Plan 1.

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5 LOADS CONSIDERED

5.1 PERMANENT LOADS

The vertical load carried by the timber mullion and steel members to undertake the connection of the parapet corresponds to the self-weight of all façade elements considered, as justified in the previous section. Accordingly, the following load components are considered bearing in mind that the timber mullions are spaced at 1250 mm centres and taking as reference the *Byggforskserien 471.031 Egenlaster for bygningsmaterialer, byggevarer og bygningsdeler (v2013)* and an additional factor of 1,10 has been applied to the resulting total vertical load to account for additional components that may not have been explicitly considered, such as fixings, joint elements, or other secondary items not defined in the sketch provided by ARK.

Profiled steel façade cladding, Sinus 18 (Fasadeplate stål m/profil / SIN 18 veggprofil)

Assumed self weight	1 kN/m ³
Thickness	50 mm
Height	3700 mm

Total weight load 0.231 kN

Gypsum wind barrier board 9 mm (GU / Vindsperre, "UV bestanding GU/Vindsperre", 9 mm)

Assumed self weight	9 kN/m ³
Thickness	9 mm
Height	3700 mm

Total weight load 0.375 kN

Internal gypsum board 13 mm (Gips – 1 eller 2 lag)

Assumed self weight	9 kN/m ³
Thickness	13 mm
Height	1800 mm

Total weight load 0.263 kN

Mineral wool insulation in timber stud zone (Isolert bindingsverk 70+200 mm)

Assumed self weight	0.3 kN/m ³
Thickness	270 mm
Height	1700 mm

Total weight load 0.172 kN

Vapour barrier (Dampsperre / "papp")

Assumed self weight	0.003 kN/m ³
Thickness	1000 mm
Height	1800 mm

Total weight load 0.007 kN

Internal timber lining / casing (Foring)

Assumed self weight	0.3 kN/m ³
Thickness	220 mm
Height	650 mm

Total weight load 0.054 kN

Timber battens 36 mm (Lekt 36 mm)

Assumed self weight	5 kN/m ³
Thickness	36 mm
Height	3700 mm

Total weight load 0.833 kN

Accoya detailing

Assumed self weight	0.04 kN/m
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Total weight load 0.050 kN

Timber battens 36 mm (Lekt 36 mm)

Assumed self weight	5 kN/m ³
Thickness	36 mm
Height	3700 mm

Total weight load 0.833 kN

Accoya detailing

Assumed self weight	0.04 kN/m
---------------------	-----------

Total weight load 0.050 kN

Sloped flashing taped to GU (Beslag fall 1:5 teipes mot GU)

Assumed self weight	78.5 kN/m ³
Thickness	1 mm
Height	150 mm

Total weight load 0.015 kN

External timber frame (48x148)

Assumed self weight	5 kN/m ³
b	148 mm
h	48 mm
Height	5000 mm

Total weight load 0.178 kN

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- Weight load obtained (Q_k) = $2,20 \cdot 1,10 = \mathbf{2,50 \text{ kN at each mullion}}$
- Vertical load for the steel plate to undertake the connection of parapet: it is considered that it rests on the lightweight concrete slab as the recess is not undertaken, as it was earlier.

Following the recommendations from *Byggforskserien 471.031 – Egenlaster for bygningsmaterialer, byggevarer og bygningsdeler*, structural glass weight is assumed to be equal to 25 kN/m³.

In accordance with the documentation provided by ARK, the glazing is assumed to be a double-layer configuration. To account for additional components such as glass support profiles, fixings, setting blocks, gaskets, sealants, cover caps and installation tolerances, an additional allowance factor of 1,15 is applied.

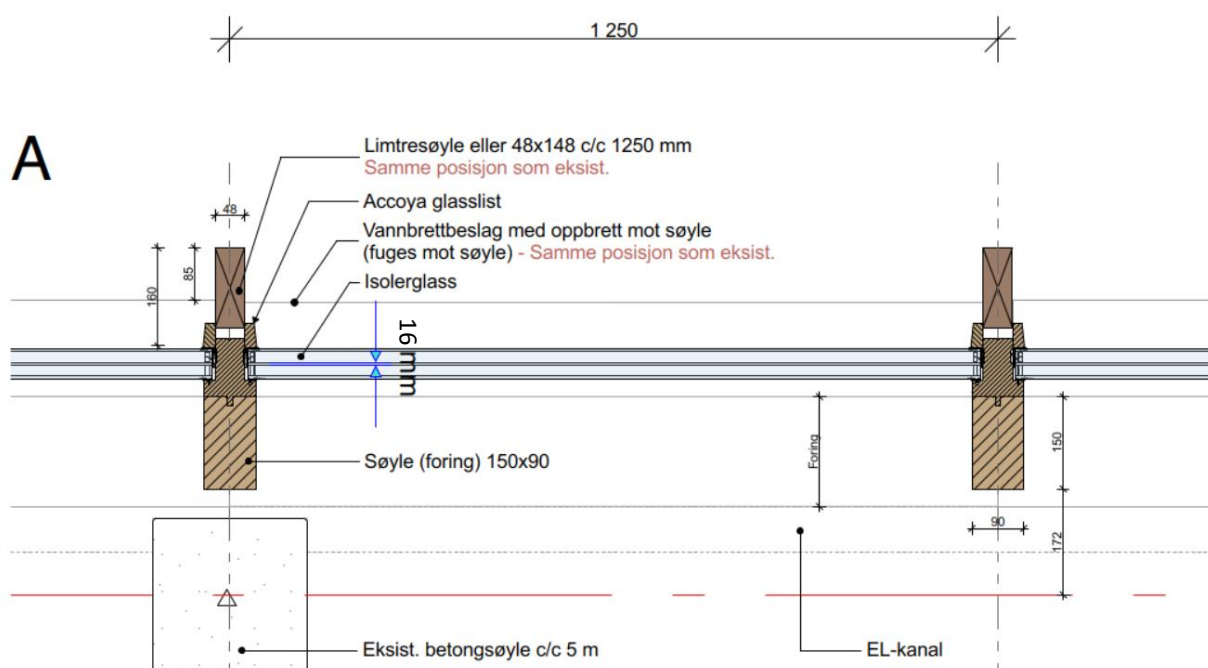


Figure 5-1. Glass configuration considered.

Then, bearing in mind a mullion spacing of 1,25 m and a total glass height of 2,70 m:

$$Q_{\text{glass}} = (2 \cdot 8 \text{ mm} \cdot 1,25 \text{ m} \cdot 2,70 \text{ m} \cdot 25 \text{ kN/m}^3) \cdot 1,15 = \mathbf{1,60 \text{ kN at each mullion}}$$

Then the total vertical load applied at the top of each mullion:

$$Q_k = 2,50 \text{ kN} + 1,60 \text{ kN} = \mathbf{4,10 \text{ kN}}$$

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5.2 VARIABLE LOADS

5.2.1 WIND

Wind actions have been calculated using TEDDS software, assuming building dimensions of 74 m × 36 m × 9.5 m and adopting a Terrain Category III (Table NA.4.1).

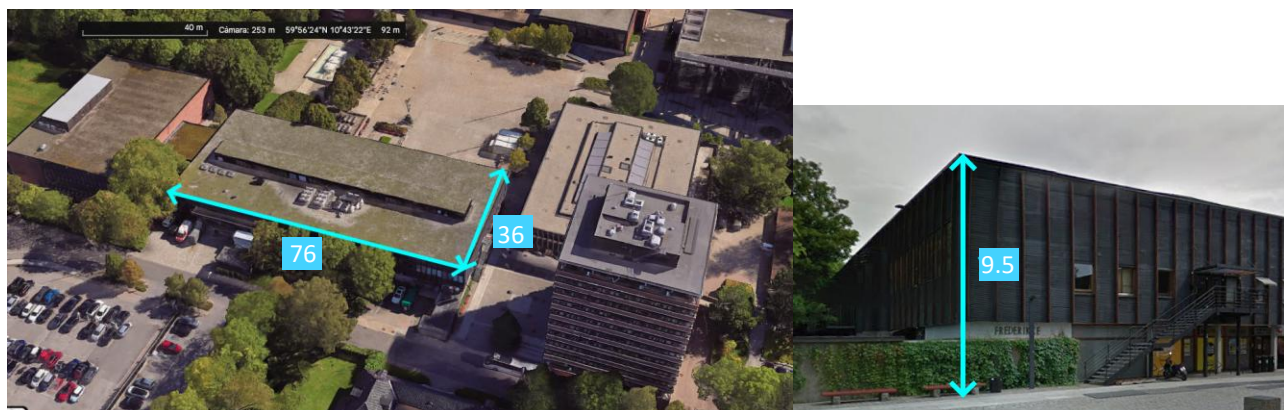


Figure 5-2. Frederikke building.

Table NA.4.1 – Terrain categories and terrain parameters

Category number	Terrain roughness category	k_r	z_0 (m)	z_{min} (m)
0	Open, rough sea.	0,16	0,003	2
I	Rough sea nearby the coast. Exposed, open country coastal areas without trees and bush vegetation.	0,17	0,01	2
II	Agricultural areas, areas with isolated small buildings or trees.	0,19	0,05	4
III	Areas with regular cover of small buildings, industrial areas or forested areas.	0,22	0,3	8
IV	Areas in which at least 15% of the surface is covered with buildings and their average height exceeds 15 m. Coniferous forest areas.	0,24	1,0	16

Basic wind velocity = 22 m/s in accordance with Table NA.4(901.1):

Table NA.4(901.1) – Fundamental value of the basic wind velocity, $v_{b,0}$, for the municipalities

Municipality	$v_{b,0}$ m/s	County
Oslo	22	Oslo
Kongsvinger	22	Hedmark
Hamar	22	Hedmark
Ringsaker	22	Hedmark
Løten	22	Hedmark

Municipality	$v_{b,0}$ m/s	County
Nord-Aurdal	22	Oppland
Vestre Slidre	22	Oppland
Øystre Slidre	22	Oppland
Vang	24	Oppland
Drammen	22	Buskerud

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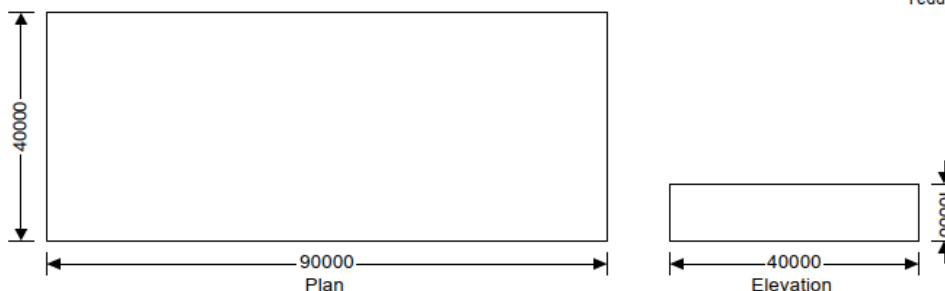
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WIND LOADING

In accordance with EN1991-1-4:2005+A1:2010 and the Norwegian national annex

Tedds calculation version 3.0.29



Building data

Type of roof	Flat
Length of building	L = 90000 mm
Width of building	W = 40000 mm
Height to eaves	H = 10000 mm
Eaves type	Sharp
Total height	h = 10000 mm

Basic values

Fundamental basic wind velocity	$V_{b,0} = 22.0$ m/s
Region	Area 1
Altitude above sea level	$H_{alt} = 0$
Altitude where correction begins (Table NA.4)	$H_0 = 900$ m
Alt. where max. correction is reached (Table NA.4)	$H_{topp} = 1500$ m
Altitude correction factor	$C_{alt} = \max(1 + ((V_0 - V_{b,0}) \times (\min(H_{alt}, H_{topp}) - H_0) / (V_{b,0} \times (H_{topp} - H_0))), 1)$ $= 1.00$
Direction factor	$C_{dir} = 1.00$
Season factor	$C_{season} = 1.00$
Shape parameter K	$K = 0.2$
Exponent n	$n = 0.5$
Air density	$\rho = 1.250$ kg/m ³

Probability factor	$C_{prob} = [(1 - K \times \ln(-\ln(1-p)))/(1 - K \times \ln(-\ln(0.98)))]^n = 1.00$
Basic wind velocity (Exp. 4.1)	$V_b = C_{dir} \times C_{season} \times C_{alt} \times C_{prob} \times V_{b,0} = 22.0$ m/s
Reference mean velocity pressure	$q_b = 0.5 \times \rho \times V_b^2 = 0.303$ kN/m ²

Orography

Orography factor not significant	$C_o = 1.0$
Terrain category	III
Displacement height (sheltering effect excluded)	$h_{dis} = 0$ mm

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height h is less than b (cl.7.2.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 1 part as the height h is less than b (cl.7.2.2)

Peak velocity pressure - windward wall - Wind 0 deg and roof

Reference height (at which q is sought) $z = 10000$ mm

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Displacement height (sheltering effects excluded)	$h_{dis} = 0 \text{ mm}$
Roughness length (Table 4.1)	$Z_0 = 300 \text{ mm}$
Roughness length (Category II)	$Z_{0,II} = 50 \text{ mm}$
Minimum height (Table 4.1)	$Z_{min} = 8000 \text{ mm}$
Maximum height	$Z_{max} = 200000 \text{ mm}$
Terrain factor	$k_r = 0.220$
Roughness factor	$C_r = k_r \times \ln(Z / Z_0) = 0.77$
Mean wind	$V_m = C_r \times C_o \times V_b = 17.0 \text{ m/s}$
Turbulence factor	$k_l = 1.0$
Turbulence intensity	$I_v = k_l / (C_o \times \ln(Z / Z_0)) = 0.285$
Peak velocity pressure	$q_p = (1 + 2 \times k_p \times I_v) \times 0.5 \times \rho \times V_m^2 = 0.54 \text{ kN/m}^2$
Structural factor	
Structural factor	$C_s C_d = 1.000$
Peak velocity pressure - windward wall - Wind 90 deg and roof	
Reference height (at which q is sought)	$z = 10000 \text{ mm}$
Displacement height (sheltering effects excluded)	$h_{dis} = 0 \text{ mm}$
Terrain factor	$k_r = 0.220$
Roughness factor	$C_r = k_r \times \ln(Z / Z_0) = 0.77$
Mean wind	$V_m = C_r \times C_o \times V_b = 17.0 \text{ m/s}$
Turbulence factor	$k_l = 1.0$
Turbulence intensity	$I_v = k_l / (C_o \times \ln(Z / Z_0)) = 0.285$
Peak velocity pressure	$q_p = (1 + 2 \times k_p \times I_v) \times 0.5 \times \rho \times V_m^2 = 0.54 \text{ kN/m}^2$
Peak velocity pressure for internal pressure	
Peak velocity pressure – internal (as roof press.)	$q_{p,i} = 0.54 \text{ kN/m}^2$
Pressures and forces	
Net pressure	$p = C_s C_d \times q_p \times C_{pe} - q_{p,i} \times C_{pi}$
Net force	$F_w = p_w \times A_{ref}$

The external and internal pressure coefficients are defined in accordance with NS-EN 1991-1-4 (cl.7.2.9 and Table 7.1 respectively).

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

Zone	A		B		C		D		E	
<i>h/d</i>	<i>C</i> _{pe,10}	<i>C</i> _{pe,1}	<i>C</i> _{pe,10}	<i>C</i> _{pe,1}	<i>C</i> _{pe,10}	<i>C</i> _{pe,1}	<i>C</i> _{pe,10}	<i>C</i> _{pe,1}	<i>C</i> _{pe,10}	<i>C</i> _{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	

NOTE 2 Where it is not possible, or not considered justified, to estimate μ for a particular case then C_{pi} should be taken as the more onerous of +0,2 and -0,3.

Figure 5-3. Eurocode NS-EN 1991-1-4 recommended values for pressure coefficients.

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To determine the design wind force acting on the local façade element (the timber mullion under design), the external pressure coefficient (c_{pe}) has been calculated for the tributary area of the mullion by interpolation, in accordance with the recommendations given in NS-EN 1991-1-4 (Figure 7.2).

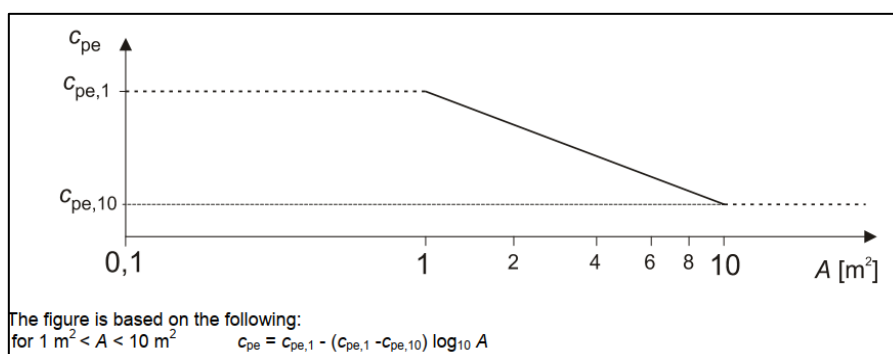


Figure 5-4. Eurocode NS-EN 1991-1-4 recommended procedure for determining the external pressure coefficient

The two wind directions considered and the corresponding façade zonification in accordance with NS-EN 1991-1-4 are shown in the figures attached below.

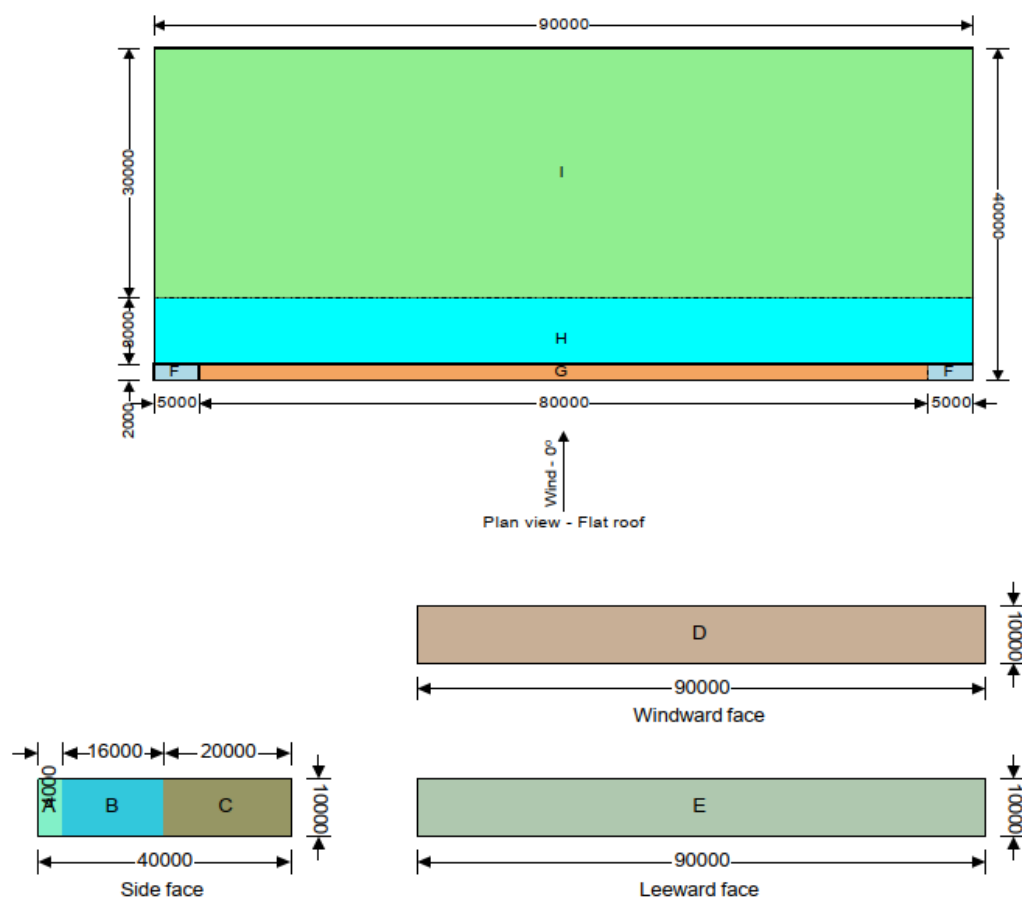


Figure 5-5. Schema of the affected areas by 0°-degree wind direction.

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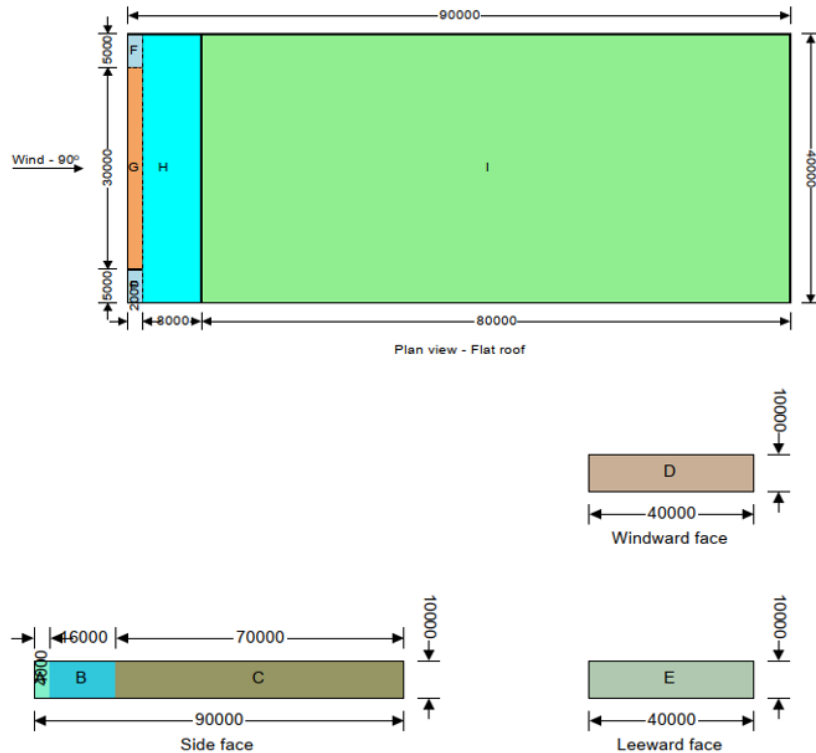


Figure 5-6. Schema of the affected areas by 90°-degree wind direction.

Results obtained assuming a tributary height for the mullion of 5,30 m (distance between plan 1 and roof slabs):

Local element data	
htrib	5.3 m
spacing	1.25 m
Tributary area	6.63 m ²
log10(A)	0.82
Building data	
h	10 m
d	90 m
h/d	0.11
Internal coef, cpi	
cpi+	0.20
cpi-	-0.30

Regarding the façade evaluation for the parapet, this area of influence is higher as shown:

- Tributary width: 5 m
- Height of the top face of the parapet and the existing IPE 200: 1.7 m
- $A_t = 8.5 \text{ m}^2$

Therefore, Cpe1 values are calculated for the most restrictive case (mullions evaluation)

External coef, cpe

ZONE A			ZONE B			ZONE C			ZONE D			ZONE E		
cpe1	cpe10	cpe(A)	cpe1	cpe10	cpe(A)	cpe1	cpe10	cpe(A)	cpe1	cpe10	cpe(A)	cpe1	cpe10	cpe(A)
-1.40	-1.20	-1.36	-1.10	-0.80	-0.85	-0.50	-0.50	-0.50	1.00	0.70	0.75	-0.30	-0.30	-0.30

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Façade load case 1 - Wind 0, cpi = +0.20 / -cpe					
Zone	cpe	cpi	cpnet	qpz	Net pressure (p)
A	-1.36	0.20	-1.56	0.54	-0.84 kN/m²
B	-0.85	0.20	-1.05	0.54	-0.57 kN/m²
C	-0.50	0.20	-0.70	0.54	-0.38 kN/m²
D	0.75	0.20	0.55	0.54	0.30 kN/m²
E	-0.30	0.20	-0.50	0.54	-0.27 kN/m²

Façade load case 3 - Wind 90, cpi = +0.20 / -cpe					
Zone	cpe	cpi	cpnet	qpz	Net pressure (p)
A	-1.36	0.20	-1.56	0.54	-0.84 kN/m²
B	-0.85	0.20	-1.05	0.54	-0.57 kN/m²
C	-0.50	0.20	-0.70	0.54	-0.38 kN/m²
D	0.75	0.20	0.55	0.54	0.30 kN/m²
E	-0.30	0.20	-0.50	0.54	-0.27 kN/m²

Façade load case 2 - Wind 0, cpi = -0.30 / +cpe					
Zone	cpe	cpi	cpnet	qpz	Net pressure (p)
A	-1.36	-0.30	-1.06	0.54	-0.57 kN/m²
B	-0.85	-0.30	-0.55	0.54	-0.30 kN/m²
C	-0.50	-0.30	-0.20	0.54	-0.11 kN/m²
D	0.75	-0.30	1.05	0.54	0.57 kN/m²
E	-0.30	-0.30	0.00	0.54	0.00 kN/m²

Façade load case 4 - Wind 90, cpi = -0.30 / +cpe					
Zone	cpe	cpi	cpnet	qpz	Net pressure (p)
A	-1.36	-0.30	-1.06	0.54	-0.57 kN/m²
B	-0.85	-0.30	-0.55	0.54	-0.30 kN/m²
C	-0.50	-0.30	-0.20	0.54	-0.11 kN/m²
D	0.75	-0.30	1.05	0.54	0.57 kN/m²
E	-0.30	-0.30	0.00	0.54	0.00 kN/m²

Figure 5-7. Net pressure values obtained under the calculation assumptions stated.

Since the timber mullions are equally spaced 1250 mm around all the building, the following wind design loads have been considered:

- Global verification (mullion at the windward face):

$$Q = \text{MAX}(\text{net pressure obtained at D/E areas}) * 1,25 \text{ m} = 0,57 \text{ kN/m}^2 * 1,25 \text{ m} = \mathbf{0,71 \text{ kN/m}}$$

- Local verification (mullion at the building corner):

$$Q = \text{MAX}(\text{net pressure obtained at A/B/C areas}) * 1,25 \text{ m} = 0,84 \text{ kN/m}^2 * 1,25 \text{ m} = \mathbf{1,05 \text{ kN/m}}$$

As a conservative approach, the maximum wind load value reached at the corner area will be used as design force.

Note: for baldakin is used in the safety side:

- 1 kN/m² on the whole area
- resultant pressure/4 → at 0.2 m from the edge
 - o 1.2 kN on the bars which are on the extremes
 - o 2.4 kN on the bars which are on the middle

5.2.2 SNOW

It is considered that amount of snow can be accumulated on the top of the parapet or the baldakin. The rest of the snow accumulated on the roof is considered to be supported by the lightweight concrete slab.

5.2.2.1 CHARACTERISTIC SNOW LOAD ON GROUND

The characteristic ground snow load (s_k) is determined based on a 50-year return period for all municipalities in the country, in accordance with Table NA.4.1(901) from NS-EN 1991-1-3:2003+A1:2015+NA:2018.

The project is located at an altitude of 8 m above sea level: 89.9 m

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EU89, UTM-sone 33

☐ Vis flere koordinatsystemer.

NORD 6652510.91

ØST 261029.12

Høyde (anslått ved interpolering): 89.9 moh

The values corresponding to the project location are shown below.

Oslo				
0–150 m.o.h.	3,5	–	–	–
151–250 m.o.h.	4,5	–	–	–
251–350 m.o.h.	5,5	–	–	–
> 350 m.o.h.	6,5	–	–	–

Figure 5-8. Snow load on Oslo 89.9 m

5.2.2.2 SNOW LOAD IN PARAPET AND BALDAKIN

For the persistent/transient design situations, the snow load on roofs (s) shall be determined as recommended in NS EN 1991-1-3, §5.2(3):

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

Where:

μ_i : snow load shape coefficient.

C_e : exposure coefficient.

C_t : thermal coefficient.

s_k : characteristic value of snow load on ground.

For accidental design situations where exceptional snow load/drift is the accidental action, see NS-EN 1991-3, §5.2(3).

For this project, a normal topography category is assumed, where there is no significant removal of snow by wind on construction work. Then, $C_e = 1$.

Additionally, no snow load reduction on roofs due melting caused by heat loss has been considered, then $C_t = 1$.

The snow load shape coefficients (μ_1 , μ_2) used to estimate the load on each roof are defined in Table 5.2 of the NS-EN 1991-1-3:2003, which is attached below along a graphically representation of the coefficient variation according to the angle of pitch of roof.

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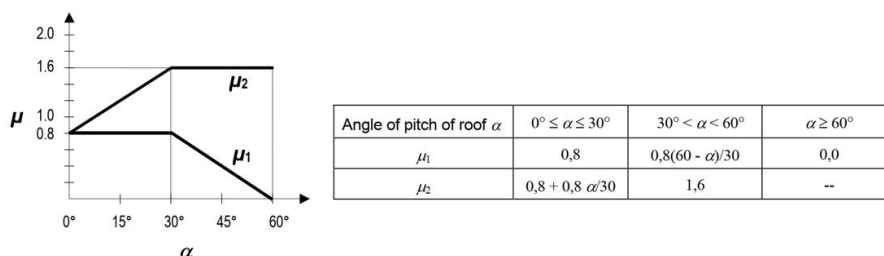


Figure 5-9. Snow load on Oslo 89.9 m

Therefore, $\mu_1 = 0.8$ with an angle of parapet of 11° (most unfavourable value as the angle of the baldakin is 0°)

As a result, the design snow load is $s = 0.8 \cdot 1 \cdot 1 \cdot 3.5 = 2.8 \text{ kN/m}^2$

Considering that the width of the parapet is 350 mm and the steel members for the connection of the parapet are each 5000 mm:

$$s = 2.8 \cdot 0.35 \cdot 5 = 4.9 \text{ kN per million}$$

5.2.2.3 ROOF ABUTTING AND CLOSE TO TALLER CONSTRUCTION WORKS (BALDAKIN)

The snow load shape coefficients that should be used for roofs abutting to taller construction works are given in the following expressions and shown in the figure attached below. In this case, the main façade is acting as a taller construction works.

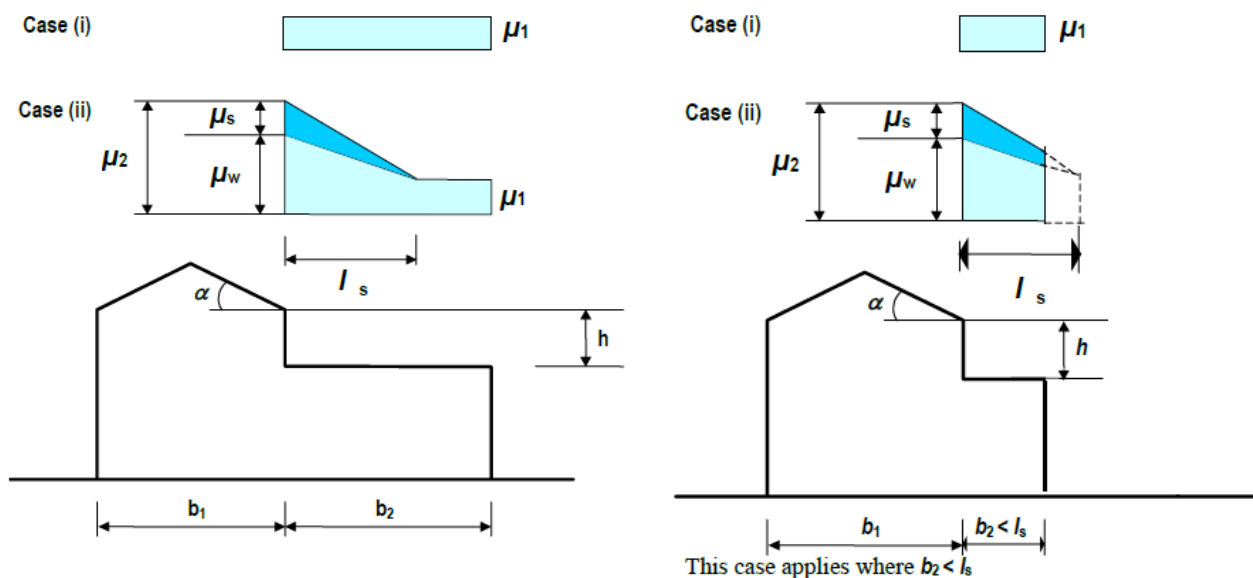
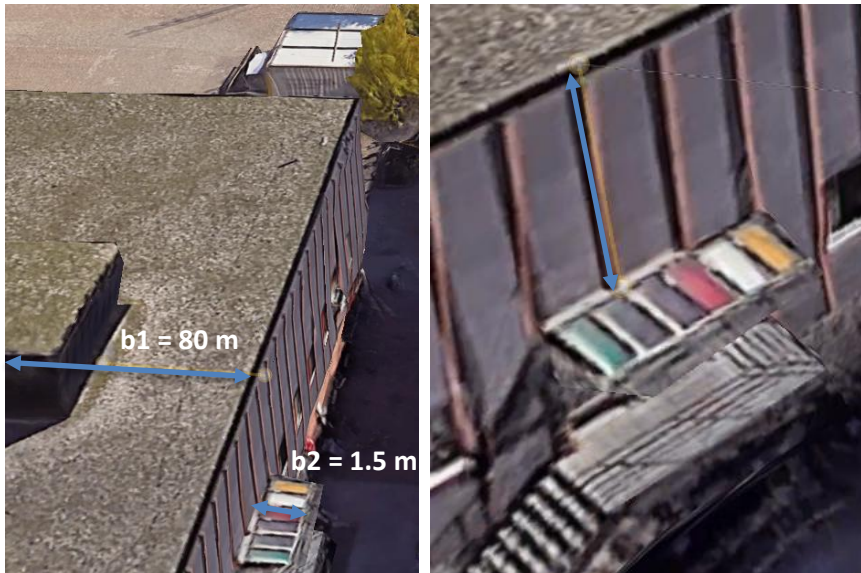


Figure 5-10 Snow load shape coefficients for roofs abutting to taller construction works

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In accordance with NS-EN 1991-1-3:20033+A1:2015+NA:2018 (§5.3.6):

- $\mu_1 = 0,80 \rightarrow$ assuming the lower roof is flat
- $\mu_2 = \mu_s + \mu_w = 0 + 4 = 4$. Following the interpolation, the final value is 3.79

where:

μ_s snow load shape coefficient due to sliding of snow from the upper roof (taking the façade with the duo pitched roof)

For $\alpha \leq 15^\circ \rightarrow \mu_s = 0$.

$$\mu_s = 0 \cdot 0.8 = 0$$

μ_w snow load shape coefficient due to wind

$\mu_w = (b_1 + b_2)/2h \leq \gamma \cdot h/s_k \rightarrow (80 + 2)/(2 \cdot 8) = 5.2 \leq 2 \cdot 8/3.5 = 4.6$ As it does not comply, the lower limit of NA.5.3.6 is taken

where:

γ : weight density of snow (recommended value = 2 kN/m³)

An upper and lower value of μ_w should be specified. The recommended range according to NA.5.3.6 is:

$$0,80 \leq \mu_w \leq 4,00. \text{ Therefore, } \mu_w \text{ is limited to } 4.$$

The drift length is determined as follows:

$$I_s = 2h = 2 \cdot 8 = 16m$$

A restriction for I_s is given by NA.5.3.6:

$$5 \text{ m} \leq I_s \leq 15 \text{ m}$$

Therefore, I_s is taken as 15 m

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As $b_2 < l_s$ the coefficient at the end of the lower roof is determined by interpolation between μ_1 and μ_2 truncated at the end of the lower roof.

$$s = \mu_i \cdot C_e \cdot C_t \cdot s_k = 3.79 \cdot 1 \cdot 1 \cdot 3.5 = 13.3 \text{ kN/m}^2$$

Finally,

$$s_{r,a} = 13.3 - 2.8 = 10.5 \frac{\text{kN}}{\text{m}^2}$$

6 90x150 MULLION CALCULATION

6.1 INPUT DATA

The calculation has been performed in Calculatis software, with the following input data.

Material → Timber GL30c

Cross section evaluated → 90x150

Mullion height → 4300 mm

Design loads: $Q_{\text{PERM}} = 4,10 \text{ kN}$; $WIND_{\text{CORNER}} = 1,05 \text{ kN/m}$

In accordance with EN 1995-1-1:2004+A2:2014, geometric imperfections of timber members shall be taken into account in stability verifications. Clause 2.3.2.2(1) specifies that members subject to compression shall be analysed considering an initial lack of straightness. For glued laminated timber members, the recommended value of the initial bow imperfection is $e_0 = L/500 = 4,30\text{m}/500 = 0,010 \text{ m}$.

Service class 2 considered.

No fire design was performed.

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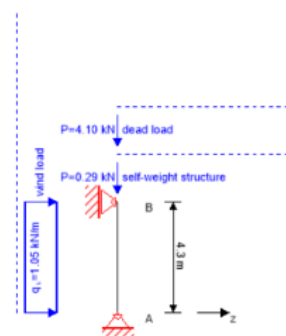
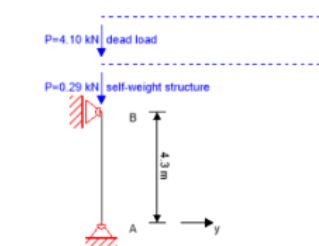
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6.2 RESULTS

6.2.1 ULS VERIFICATION

Geometry and loading



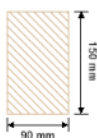
Utilization ratios

Global utilization ratio	ULS	ULS Fire
61 %	61 %	-

Utilization ratios

Global utilization ratio	ULS	ULS Fire
49 %	49 %	-

Section Wooden beam 9/15



Section width	Section height	Area	I _y	I _z
[cm]	[cm]	[mm ²]	[mm ⁴]	[mm ⁴]
9	15	13,500	25,312,500	9,112,501

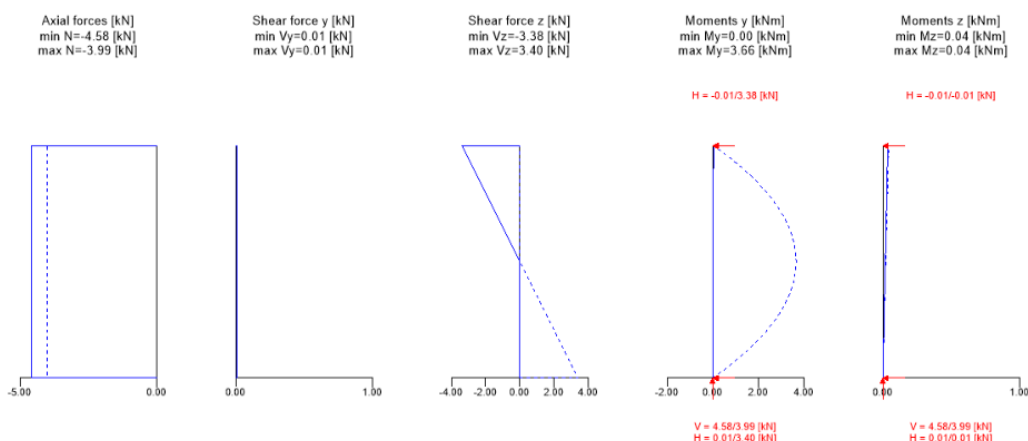
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Ultimate limit state (ULS) - results of governing load combination

Combination: 1.15/1.00 * LC1:self-weight structure + 1.15/1.00 * LC3:dead load + 1.50/0.00 * LC2:wind load



Flexural design

Dist.	$f_{m,k}$	$f_{c0,k}$	$f_{t0,k}$	γ_m	k_{mod}	$k_{s,y,y}$	$k_{h,m,y}$	$k_{h,m,z}$	k_l	$f_{m,y,d}$	$f_{m,z,d}$	$f_{c0,d}$	$f_{t0,d}$
[m]	[N/mm ²]	[N/mm ²]	[N/mm ²]	[-]	[-]	[-]	[-]	[-]	[-]	[N/mm ²]	[N/mm ²]	[N/mm ²]	[N/mm ²]
2.15	30.00	30.00	24.00	1.20	0.90	1.00	1.10	1.10	1.00	24.75	24.75	22.50	18.00

Dist.	$M_{y,d}$	$M_{z,d}$	$N_{c,d}$	$N_{t,d}$	$\sigma_{m,y,d}$	$\sigma_{m,z,d}$	$\sigma_{c,d}$	$\sigma_{t,d}$	Utilization
[m]	[kNm]	[kNm]	[kN]	[kN]	[N/mm ²]	[N/mm ²]	[N/mm ²]	[N/mm ²]	
2.15	3.66	0.02	-5.04	0.00	10.86	0.12	0.37	0.00	44%

Shear analysis Y

Dist.	$f_{v,k}$	γ_m	k_{mod}	$k_{h,v}$	$f_{v,d}$	V_d	$\tau_{v,d}$	Utilization
[m]	[N/mm ²]	[-]	[-]	[-]	[N/mm ²]	[kN]	[N/mm ²]	
0.0	3.50	1.20	0.60	1.00	1.25	0.01	0.00	0%

Shear analysis Z

Dist.	$f_{v,k}$	γ_m	k_{mod}	$k_{h,v}$	$f_{v,d}$	V_d	$\tau_{v,d}$	Utilization
[m]	[N/mm ²]	[-]	[-]	[-]	[N/mm ²]	[kN]	[N/mm ²]	
0.0	3.50	1.20	0.90	1.00	1.87	3.40	0.38	20%

Shear analysis Combined

Dist.	$f_{v,k}$	γ_m	k_{mod}	$f_{v,d}$	$V_{y,d}$	$V_{z,d}$	$\tau_{v,y,d}$	$\tau_{v,z,d}$	Ratio
[m]	[N/mm ²]	[-]	[-]	[N/mm ²]	[kN]	[kN]	[N/mm ²]	[N/mm ²]	
0.0	3.50	1.20	0.90	1.87	0.01	3.40	0.00	0.38	4%

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Buckling design

Dist.	$f_{m,k}$	$f_{c0,k}$	γ_m	k_{mod}	$k_{sys,y}$	$k_{sys,z}$	I_{ky}	I_{kz}	λ_y	λ_z	$\lambda_{rel,y}$	$\lambda_{rel,z}$	β_c	$k_{h,m,y}$	k_i
[m]	[N/mm ²]	[N/mm ²]	[-]	[-]	[-]	[-]	[m]	[m]	[-]	[-]	[-]	[-]	[-]	[-]	[-]
2.15	30.00	30.00	1.20	0.90	1.00	1.00	4.300	4.300	99	166	1.63	2.71	0.1	1.10	1.00

Dist.	k_y	k_z	k_{cy}	k_{cz}	$f_{m,y,d}$	$f_{m,z,d}$	$f_{c0,d}$	$M_{y,d}$	$M_{z,d}$	$N_{c,d}$	$\sigma_{c,d}$	$\sigma_{m,y,d}$	$\sigma_{m,z,d}$	Utilization
[m]	[-]	[-]	[-]	[-]	[N/mm ²]	[N/mm ²]	[N/mm ²]	[kNm]	[kNm]	[kN]	[N/mm ²]	[N/mm ²]	[N/mm ²]	
2.15	1.89	4.30	0.35	0.13	24.75	24.75	22.50	3.66	0.02	-5.04	0.37	10.86	0.12	49%

Lateral torsional buckling design

Dist.	$f_{m,k}$	$f_{c0,k}$	γ_m	k_{mod}	$k_{sys,y}$	I_{ef}	I_k	λ_y	$\lambda_{rel,y}$	$\lambda_{rel,m}$	β_c	k_y	k_{cy}	$k_{h,m,y}$	$k_{h,m,z}$	k_i
[m]	[N/mm ²]	[N/mm ²]	[-]	[-]	[-]	[m]	[m]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]
2.15	30.00	30.00	1.20	0.90	1.00	3.870	4.300	99	1.63	0.62	0.1	1.89	0.35	1.10	1.10	1.00

$\sigma_{m,crit,y}$	$\sigma_{m,crit,z}$	k_{crit}	$f_{m,y,d}$	$f_{m,z,d}$	$f_{c0,d}$	$M_{y,d}$	$M_{z,d}$	$N_{c,d}$	$\sigma_{c,d}$	$\sigma_{m,y,d}$	$\sigma_{m,z,d}$	Utilization
[N/mm ²]	[N/mm ²]	[-]	[N/mm ²]	[N/mm ²]	[N/mm ²]	[kNm]	[kNm]	[kN]	[N/mm ²]	[N/mm ²]	[N/mm ²]	
85.40	237.23	1.00	24.75	24.75	22.50	3.66	0.02	-5.04	0.37	10.86	0.12	49%

6.2.1 SLS VERIFICATION

The mullion was idealised as a prismatic, simply supported (pin-pin) member spanning 4,30 m and subjected to a uniformly distributed lateral wind line load acting about the strong axis (with a maximum value of 1,05 kN/m reached at the corner areas and 0,71 kN/m at the rest of the façade area).

6.2.1.1 CALCULATION DATA AND ASSUMPTIONS

- In accordance with EN 1995-1-1, (Cl. 2.2.2(1)), a linear elastic first-order analysis was performed using mean stiffness properties. The elastic modulus was taken as $E_{0mean} = 13000 \text{ N/mm}^2$ for GL30c as given in EN 14080 (table of laminated classes).
- The second moment of area about the strong axis was calculated for the rectangular section ($b \cdot h = 90 \times 150$ as ($I = b \cdot h^3 / 12 = 25,31 \cdot 10^{-6} \text{ mm}^4$).
- The maximum instantaneous deflection under UDL for a simply supported member was then obtained from the classical beam expression $w_{max} = 5 \cdot q \cdot L^4 / (384 \cdot E \cdot I)$ at mid-span.
- Wind action was classified as short-term in accordance with NS-EN 1995-1-1 Table 2.2; therefore, the serviceability verification was based on instantaneous deflection without amplification for creep. For façade applications, deformability limits are not governed directly by generic structural span/deflection ratios but by the performance requirements of the glazing system.

6.2.1.2 VERIFICATION

$$I = b \cdot h^3 / 12 = 90 \text{ mm} \cdot (150 \text{ mm})^3 / 12 = 25312500 \text{ mm}^4$$

At the intermediate area of the studied building façades:

$$W_{max,interm} = 5 \cdot q \cdot L^4 / 384 \cdot E I = 5 \cdot 0,71 \text{ kN/m} \cdot (4,3 \text{ m})^4 / 384 \cdot 13000 \text{ N/mm}^2 \cdot 25312500 \text{ mm}^4 = 9,60 \text{ mm}$$

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At the intermediate area of the studied building façades:

$$W_{\max, \text{corner}} = 5 \cdot q \cdot L^4 / 384 \cdot EI = 5 \cdot 1,05 \text{ kN/m} \cdot (4,3 \text{ m})^4 / 384 \cdot 13000 \text{ N/mm}^2 \cdot 25312500 \text{ mm}^4 = \mathbf{14.20 \text{ mm}}$$

6.2.1.3 INTERPRETATION OF RESULTS

The 90x150 mm GL30c mullion, modelled as a simply supported member with a span of 4,30 m, develops a maximum instantaneous mid-span deflection of approximately 14,20 mm in the façade corner zones, defined as the first 4,00 m from each corner towards the interior, where the design wind load is 1,05 kN/m acting about the strong axis. In the remaining central façade areas, where the wind load reduces to 0,71 kN/m, the corresponding maximum instantaneous deflection decreases proportionally to approximately 9,60 mm.

From a façade engineering perspective, both displacement values are structurally acceptable at ultimate limit state (around L/300 and L/450 respectively); however, they are highly relevant for the design of glazing interfaces. A lateral displacement of 14.2 mm at mid-span in the corner zones represents a significant relative movement between mullion and glass edge, particularly when combined with support rotations. Even the reduced 9,60 mm deflection in central areas is non-negligible in the context of insulating glass units with limited edge clearance and defined bite requirements. These movements must therefore be explicitly accommodated in the design of mullion brackets, pressure plates, setting blocks, gaskets and sealants. The fixity concept must allow for differential movement without inducing secondary bending in the glass, loss of edge cover, excessive gasket compression or hard contact between glass and framing. Consequently, the calculated deflection values are directly relevant design parameters for the glazing support and restraint system and must be incorporated into joint detailing, tolerance strategy and movement capacity verification.

6.3 CONNECTIONS

6.3.1 TIMBER MULLION TO EXISTING IPE200

Horizontal reactions:

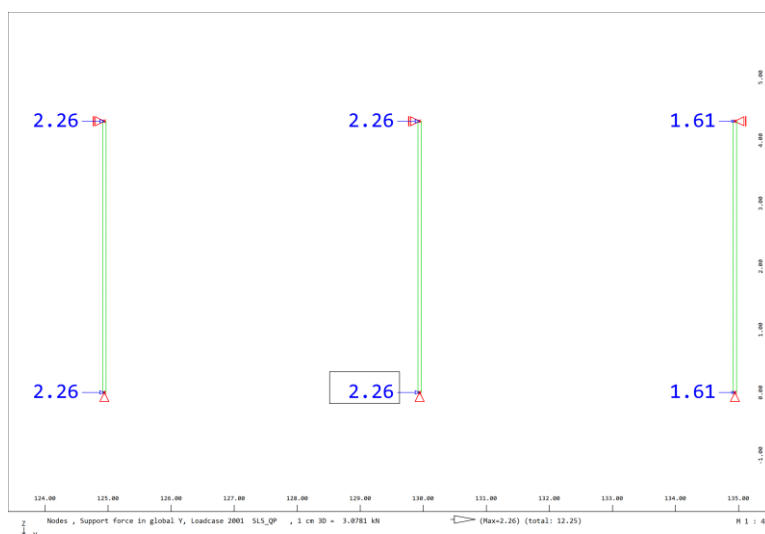


Figure 6-1. Horizontal reactions

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Vertical reaction:

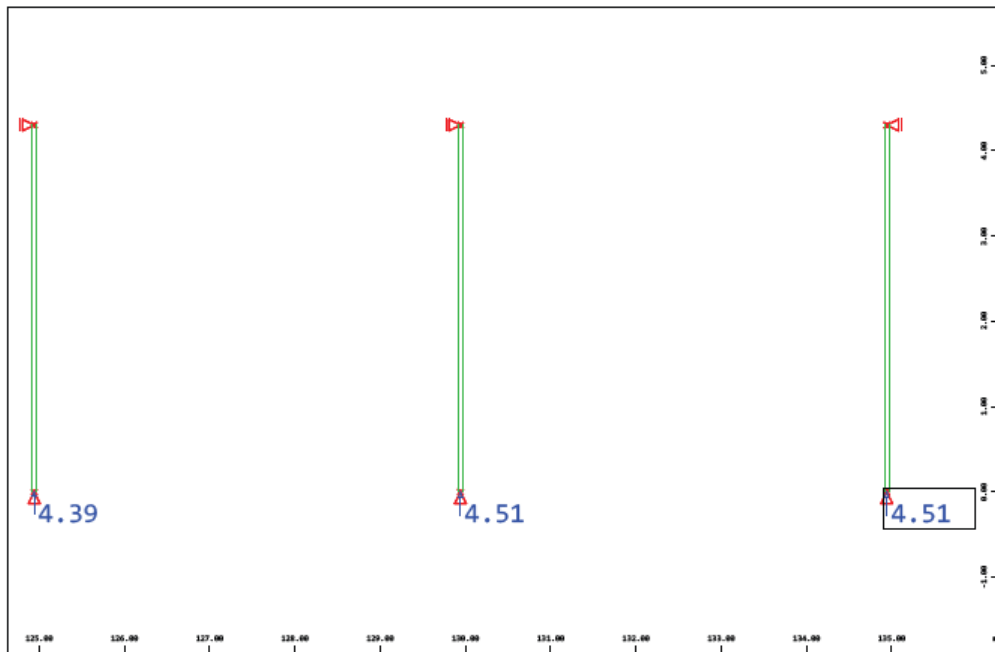


Figure 6-2. Vertical reactions

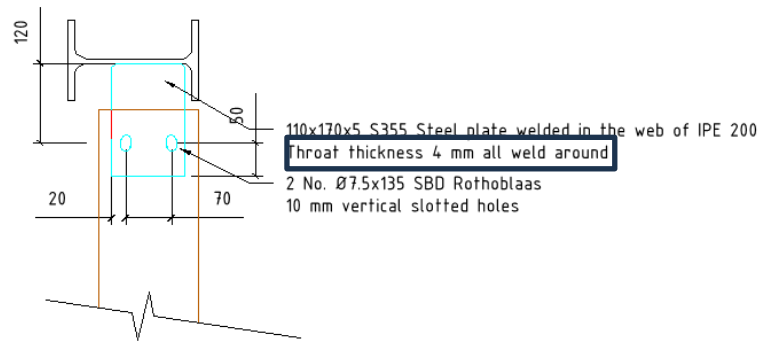
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6.3.1.1 WELDING BETWEEN STEEL PLATE AND IPE 200 WEB

Introducing the moment generated on the welding: $2.3 \text{ kN} \cdot 0.12 \text{ m} = 0.3 \text{ kNm}$



ULS VERIFICATION FOR FILLET WELDS SIMPLIFIED METHOD AS PER §4.5.3.2 - EN 1993-1-8			
CODE		NS-EN 1993-1	
	V_{a1}	1,05	§NA.6.1 NS-EN 1993-1-1
	V_{a2}	1,25	§NA.6.1 NS-EN 1993-1-1
PLATE	distance between welds (t)	5	mm
	steel	S 355	
	f_y	355	N/mm ²
	f_u	470	N/mm ²
	L_w	110	mm
	L_{eff}	102	mm
	L_{min}	24	mm
ACTIONS	N	0,0	kN
	V1	2,3	kN
	V2	0,0	kN
	T	0,0	
	M1	0,0	kNm
	M2	0,3	kNm
WELD	a1 (throat thickness)	4,0	mm
	a2 (throat thickness)	4,0	mm
	h	102	mm
	A1	408	mm ²
	A2	408	mm ²
	Atot	816	mm ²
	Wel	13872	mm ³
	β_w	0,9	
	ultimate weld stress $f_{vw,d}$	241,2	N/mm ²
	F1	2,8	N/mm ²
	F2	0,0	N/mm ²
	F3	21,6	N/mm ²
	weld stress	21,8	N/mm ²
	factor	9%	

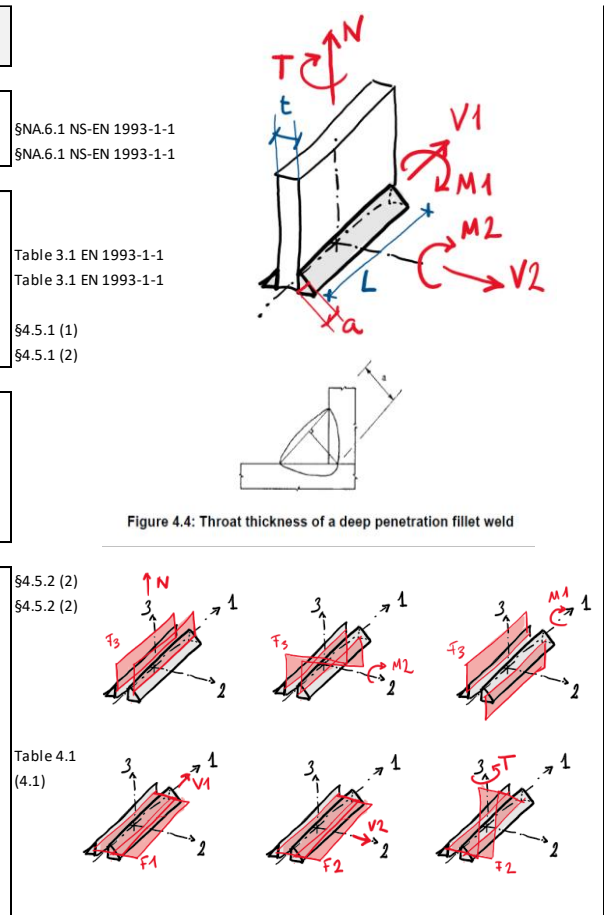


Figure 6-3. Welding calculations

Welding: 4 mm throat thickness all weld around

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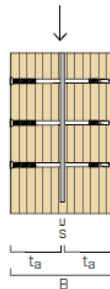
6.3.1.2 DOWELS BETWEEN IPE 200 AND TIMBER MULLION

For 2.3 kN horizontal shear reaction:

STRUCTURAL VALUES | TIMBER-TO-METAL-TO-TIMBER

CHARACTERISTIC VALUES
EN 1995:2014

1 INTERNAL PLATE - DOWEL HEAD INSTALLATION DEPTH 0 mm



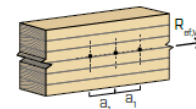
			7,5x55	7,5x75	7,5x95	7,5x115	7,5x135	7,5x155	7,5x175	7,5x195	7,5x215	7,5x235
beam width	B	[mm]	60	80	100	120	140	160	180	200	220	240
head insertion depth	p	[mm]	0	0	0	0	0	0	0	0	0	0
exterior wood	t _a	[mm]	27	37	47	57	67	77	87	97	107	117
R _{V,k} [kN]	load-to-grain angle	0°	7,48	9,20	12,10	12,88	13,97	15,27	16,69	17,65	18,41	18,64
		30°	6,89	8,59	11,21	11,96	12,88	13,99	15,23	16,42	17,09	17,65
		45°	6,41	8,09	10,34	11,20	11,99	12,96	14,05	15,22	16,00	16,62
		60°	6,00	7,67	9,62	10,58	11,25	12,10	13,07	14,12	15,08	15,63
		90°	5,66	7,31	9,01	10,04	10,62	11,37	12,24	13,18	14,19	14,79

EFFECTIVE NUMBER FOR SHEAR-STRESSED DOWELS

The load-bearing capacity of a connection made with several dowels, all of the same type and size, may be lower than the sum of the load-bearing capacities of the individual connection system.

For a row of n dowels arranged parallel to the direction of the grain (α = 0°) at a distance a₁, the characteristic effective load-bearing capacity is equal to:

$$R_{ef,V,k} = n_{ef} \cdot R_{V,k}$$



The nef value is given in the table below as a function of n and a₁.

		40	50	60	70	a ₁ ^(*) [mm]	80	90	100	120	140
n	2	1,49	1,58	1,65	1,72	1,78	1,83	1,88	1,97	2,00	
	3	2,15	2,27	2,38	2,47	2,56	2,63	2,70	2,83	2,94	
	4	2,79	2,95	3,08	3,21	3,31	3,41	3,50	3,67	3,81	
	5	3,41	3,60	3,77	3,92	4,05	4,17	4,28	4,48	4,66	
	6	4,01	4,24	4,44	4,62	4,77	4,92	5,05	5,28	5,49	

2 No. dowels for shear connection → n_{ef} = 1.72

$$R_{V,d} = 1.72 \cdot \frac{10.62 \cdot 0.8}{1.3} = 11.24 > 2.3 \text{ kN}$$

Dowels: 2 No. 7.5x135 SBD

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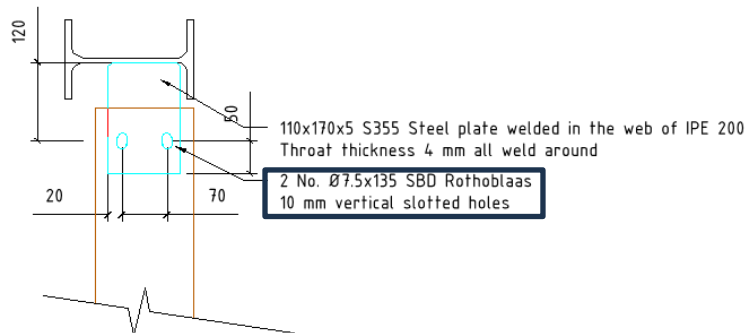


Figure 6-4. Sketch showing the position and properties of dowels

6.3.2 TIMBER MULLION TO EXISTING CONCRETE BEAM

6.3.2.1 DOWELS BETWEEN TIMBER MULLION AND T SECTION

- 4.5 kN vertical shear reaction
- 2.3 kN horizontal shear reaction
- Red, tot = $\sqrt{R_v^2 + R_h^2} = 5 \text{ kN}$

STRUCTURAL VALUES | TIMBER-TO-METAL-TO-TIMBER

CHARACTERISTIC VALUES
EN 1995:2014

1 INTERNAL PLATE - DOWEL HEAD INSTALLATION DEPTH 0 mm



			7,5x55	7,5x75	7,5x95	7,5x115	7,5x135	7,5x155	7,5x175	7,5x195	7,5x215	7,5x235
beam width	B	[mm]	60	80	100	120	140	160	180	200	220	240
head insertion depth	p	[mm]	0	0	0	0	0	0	0	0	0	0
exterior wood	t _a	[mm]	27	37	47	57	67	77	87	97	107	117
R _{v,k} [kN]	load-to-grain angle	0°	7,48	9,20	12,10	12,88	13,97	15,27	16,69	17,65	18,41	18,64
		30°	6,89	8,59	11,21	11,96	12,88	13,99	15,23	16,42	17,09	17,65
		45°	6,41	8,09	10,34	11,20	11,99	12,96	14,05	15,22	16,00	16,62
		60°	6,00	7,67	9,62	10,58	11,25	12,10	13,07	14,12	15,08	15,63
		90°	5,66	7,31	9,01	10,04	10,62	11,37	12,24	13,18	14,19	14,79

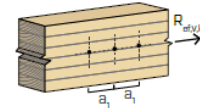
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EFFECTIVE NUMBER FOR SHEAR-STRESSED DOWELS

The load-bearing capacity of a connection made with several dowels, all of the same type and size, may be lower than the sum of the load-bearing capacities of the individual connection system.
For a row of n dowels arranged parallel to the direction of the grain ($\alpha = 0^\circ$) at a distance a_1 , the characteristic effective load-bearing capacity is equal to:

$$R_{efV,k} = n_{ef} \cdot R_{V,k}$$



The n_{ef} value is given in the table below as a function of n and a_1 .

	a ₁ (*) [mm]									
	40	50	60	70	80	90	100	120	140	
n	2	1,49	1,58	1,65	1,72	1,78	1,83	1,88	1,97	2,00
	3	2,15	2,27	2,38	2,47	2,56	2,63	2,70	2,83	2,94
	4	2,79	2,95	3,08	3,21	3,31	3,41	3,50	3,67	3,81
	5	3,41	3,60	3,77	3,92	4,05	4,17	4,28	4,48	4,66
	6	4,01	4,24	4,44	4,62	4,77	4,92	5,05	5,28	5,49

2 No. dowels for shear connection in one direction $\rightarrow n_{ef} = 1.72$

$$R_{V,d} = 1.72 \cdot \frac{10.62 \cdot 0.8}{1.3} = 11.24 > 5.0 \text{ kN}$$

Dowels: 4 No. 7.5x135 SBD

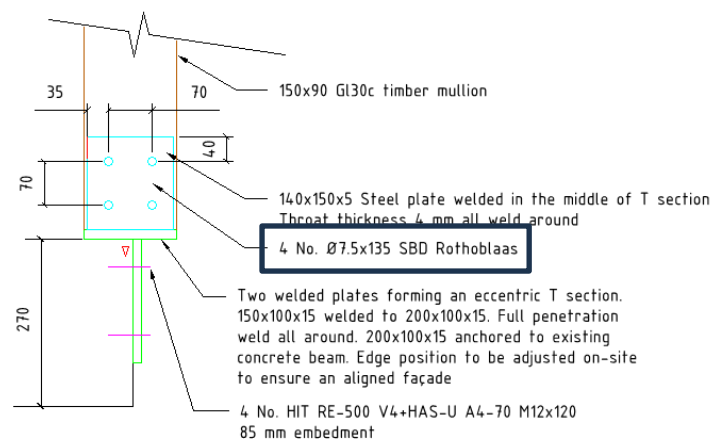


Figure 6-5. Sketch showing the position and properties of dowels

6.3.2.2 WELDING BETWEEN STEEL PLATE AND T SECTION

Same calculations than chapter 6.3.1.1 as this one is most unfavourable because it has more eccentricity $\rightarrow$ welding: 4 mm throat thickness all weld around.

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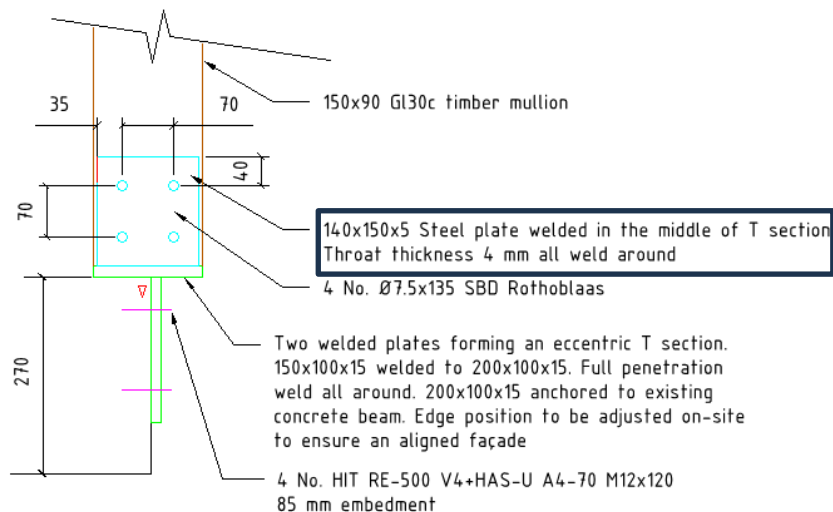


Figure 6-6. Sketch showing the position and properties of welding

6.3.2.3 ANCHORS BETWEEN T SECTION AND EXISTING CONCRETE BEAM

Existing concrete beam shown below, whose geometry is taken from the ark's drawings [1]. Contractor to confirm dimensions on-site.

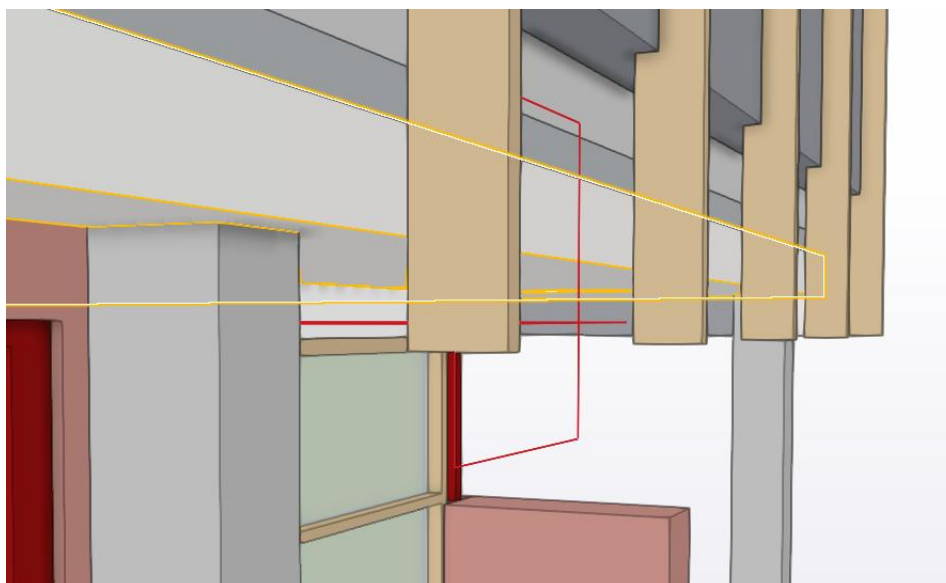


Figure 6-7. Existing concrete beam

Beam depth: 270 mm

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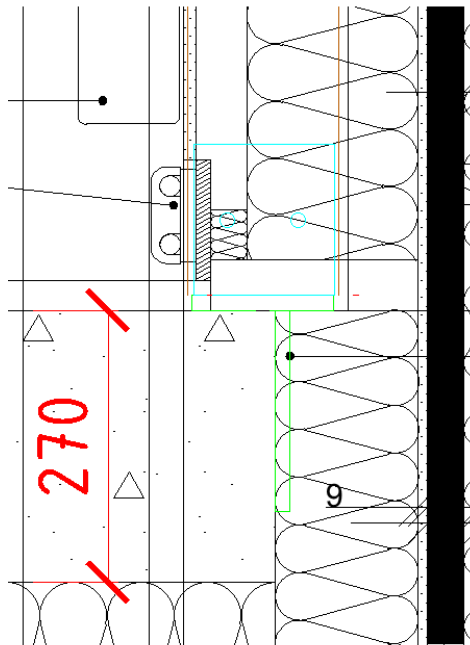


Figure 6-8. Existing concrete beam depth

Modelling only the vertical plate and the edge distances:

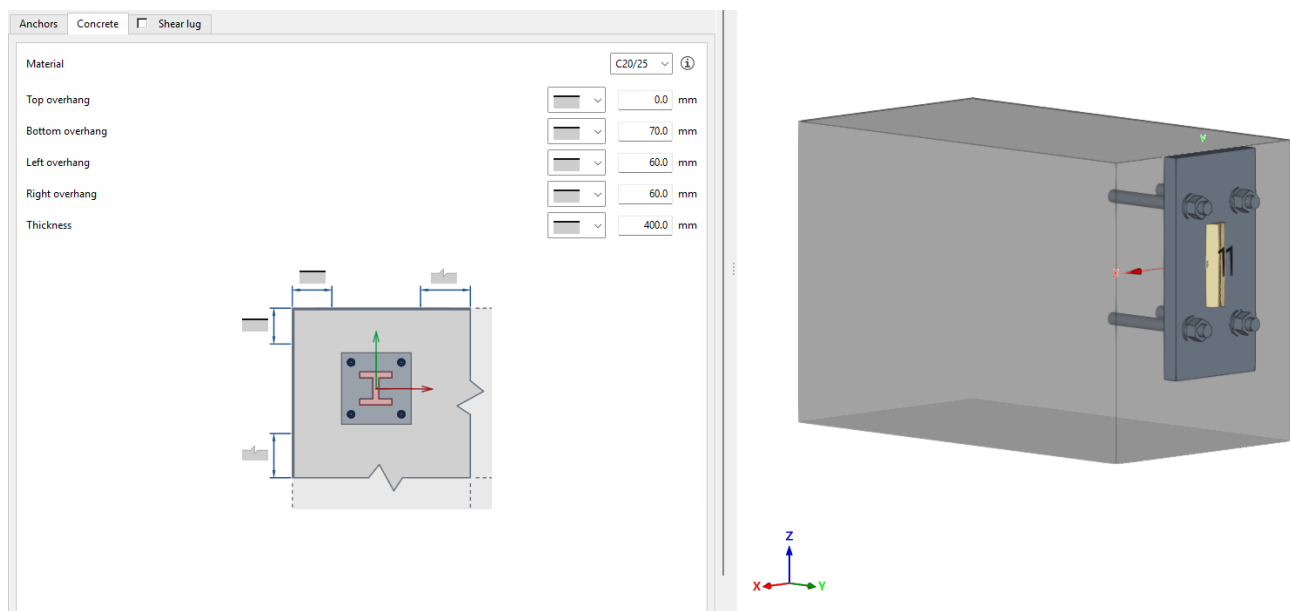


Figure 6-9. Edge distances regarding the web of T section

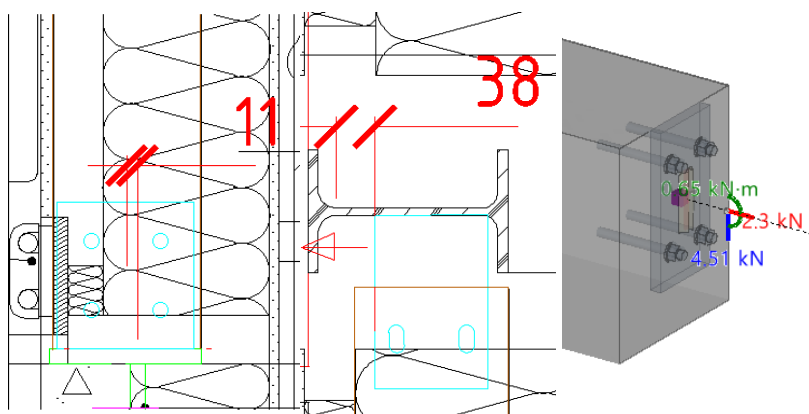
Considering the most unfavourable case, where the wind can be transformed like a suction and taking into account the potential most large eccentricity of the timber mullion regarding the vertical plate (18 mm actual position + 30 mm potential unfavourable position = 50 mm):

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- $M = 4.5 \text{ kN} \cdot 0.05 \text{ m} + 2.3 \text{ kN} \cdot 0.18 \text{ m} = 0.65 \text{ kNm}$
- $V = 4.5 \text{ kN}$
- $N = 2.3 \text{ kN}$



Use 4 No. HIT RE-500 V4+HAS-U A4-70 M12x120, with 85 mm embedment:

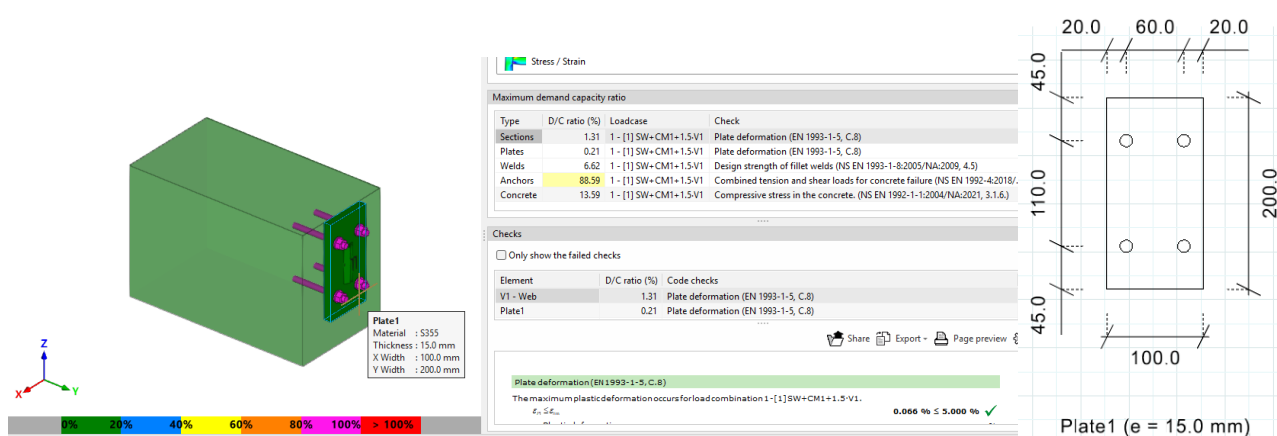


Figure 6-10. Checks of anchors to existing concrete beam and holes disposition

Note: see full report of checks in Annex II

7 PARAPET CONNECTION CALCULATION

The new parapet proposed by ARK needs to be fixed to the existing structure. The solution which is proposed consists on designing steel brackets to connect the parapet to the end of the prestressed concrete beams on top and on the existing IPE 200 without transmitting vertical loads as shown on the next figure, which is detailed on an independent sketch document.

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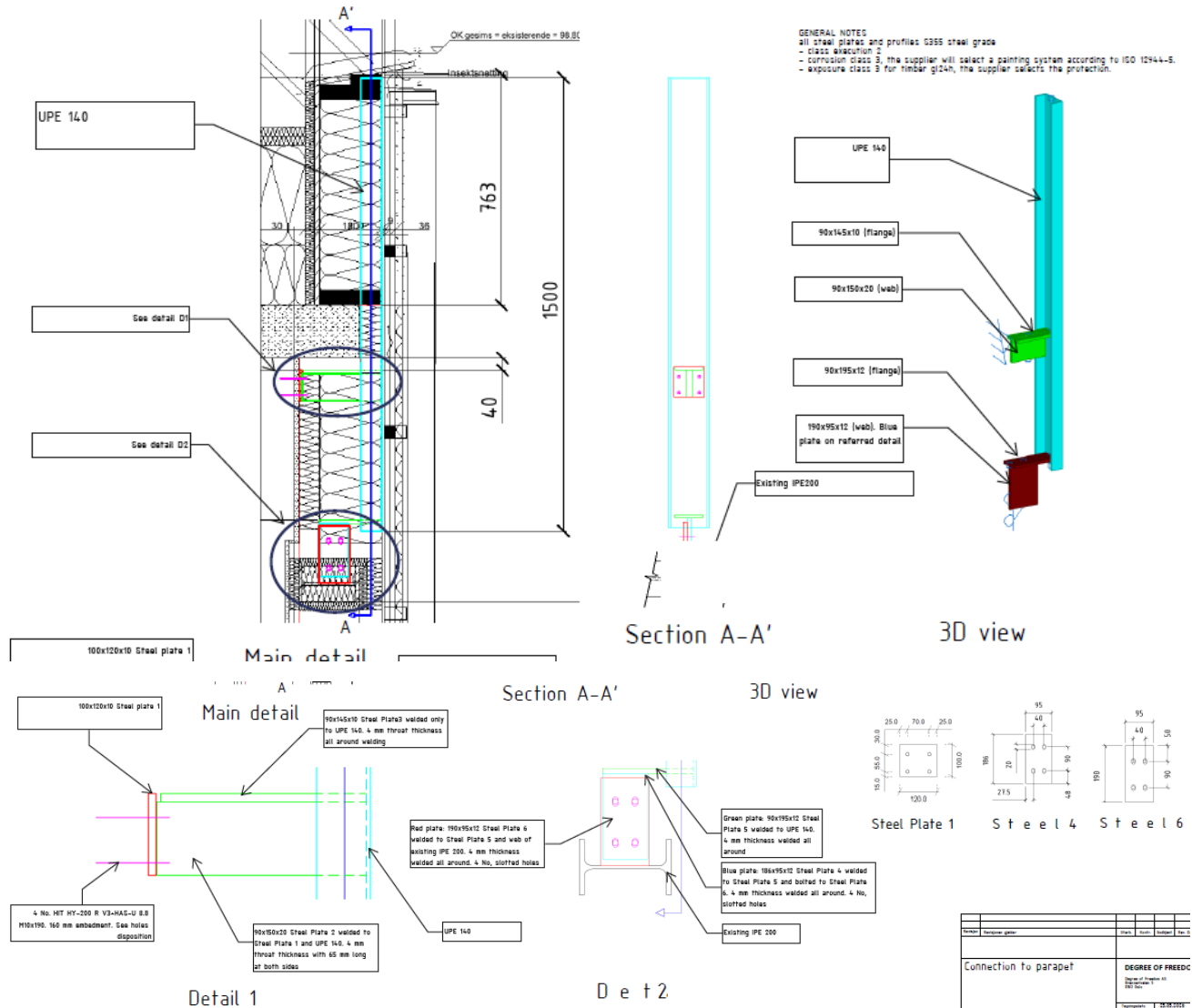


Figure 7-1. Sketch proposal for parapet connection

7.1 CALCULATION PROGRAM

The structure was modelled with Cype 3D v26, using bar elements to represent each steel plates and to obtain the reactions on the supports and calculate the connections.

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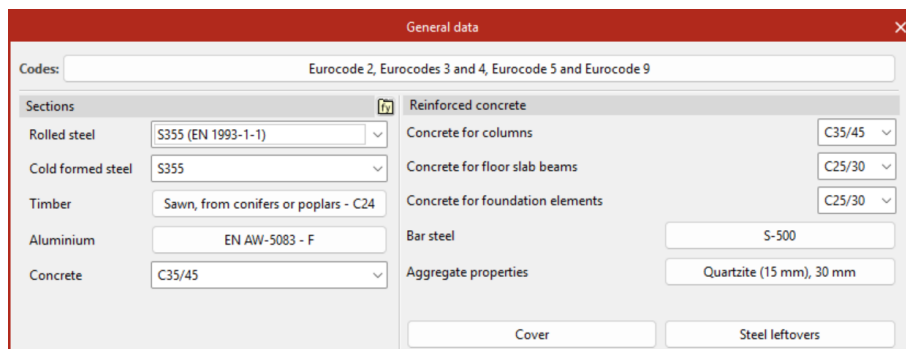


Figure 7-2. General data Cype 3D v26

7.2 INPUT DATA

7.2.1 PRELIMINARY CONSIDERATIONS

The scheme which has been considered following the Figure 7-1, is modelling the blue long profile UPE which covers the length of the parapet as a cantilever, being on the safety side considering that the upper extreme of the parapet is not restrained.

The middle green element which consists of a welded steel plates forming a T-section is provided to connect the UPE to the prestressed concrete beam, which are separated 250 mm aprox. This end support restrains the vertical and horizontal displacements, and it allows rotation. This plate would be located on the top flange of the existing prestressed concrete beam.

On the other hand, the bottom part of the UPE needs to be connected to the existing IPE 200, because there is not enough space on the 85 mm prestressed concrete beam web in this position, as the minimum width to anchor an existing concrete element must be 100 mm.

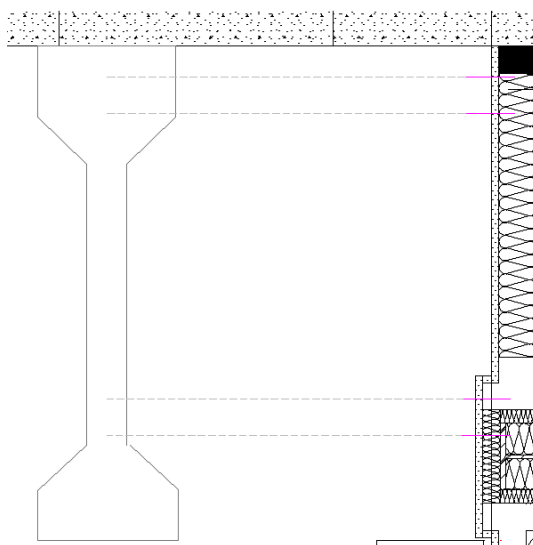


Figure 7-3. Sketch showing that the bottom plate would coincide with the web of the prestressed beam (85 mm)

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To avoid transferring vertical forces to the existing IPE 200 profile, a support is designed to allow vertical displacement by means of slotted holes. The way to connect the UPE profile to the web of the of the IPE 200, welded steel plates forming a T-section (brown) are located to be bolted to a vertical plate welded on the web of the IPE 200.

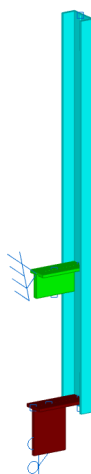


Figure 7-4. Steel plate scheme

7.2.2 GEOMETRY AND SECTIONS

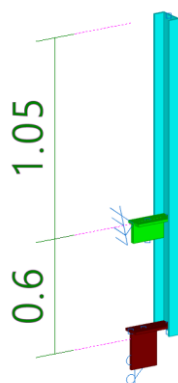


Figure 7-5. Geometry

Dimensions of profiles modelled:

- Blue steel profile: UPE 140 S355
- Green steel plate (welded steel plates forming a T-section): 90x150x20 (web) and 90x145x10 (flange) S355. *Note: The top flange shall be slightly shorter than the web to allow only web to be connected to the anchor plate.*
- Brown steel plate (welded steel plates forming a T-section): and 186x95x12 (web) and 90x195x12 (flange) S355.

7.3 BUCKLING

Automatically set by the software.

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7.4 LOADS APPLIED

This support for the parapet is connected to the prestressed concrete beams and to existing IPE 200, being installed each 5 m. Considering the loads which have been calculated previously on chapter 5:

- Wind (most unfavourable one) = $0.84 \text{ kN/m}^2 \rightarrow 0.84 \text{ kN/m}^2 \cdot 5 \text{ m} = 4.2 \text{ kN/m}$
- Snow = 4.9 kN

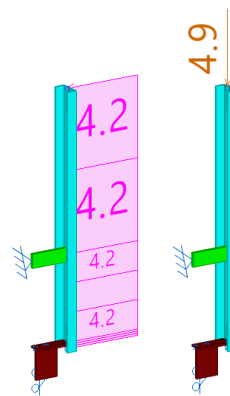


Figure 7-6. Loads applied

7.5 RESULTS

7.5.1 ULS VERIFICATION

Regarding the most unfavourable part of UPE, it is checked that it works with a ratio resistance of 31%.

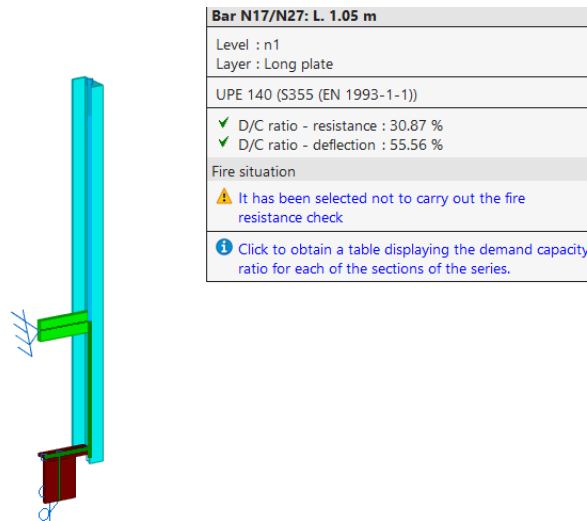


Figure 7-7. ULS ratio – 31%

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7.5.2 SLS VERIFICATION

Following with the same restrictions which have been discussed previously regarding the deflection ($L/300$), it is worth mentioning that for the cantilever part the limitation is $L/150$. $1100/150 = 7.3 \text{ mm} > 4 \text{ mm}$

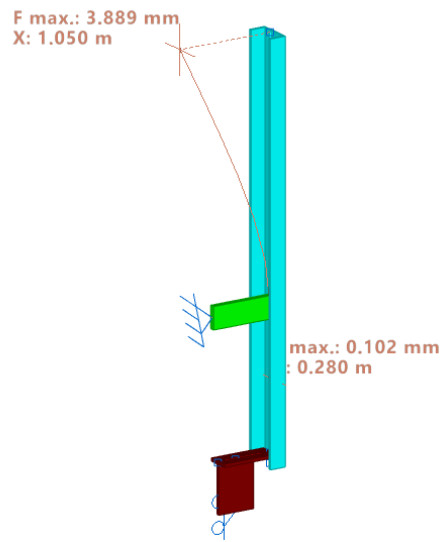


Figure 7-8. SLS ratio – 56%

If a recess needs to be undertaken due to the geometry of the UPE 140, a thick steel plate with dimensions 1550x140x35 could be used, taking into consideration that this system is less rigid than the UPE and the deflections would increase to 6.5 mm.

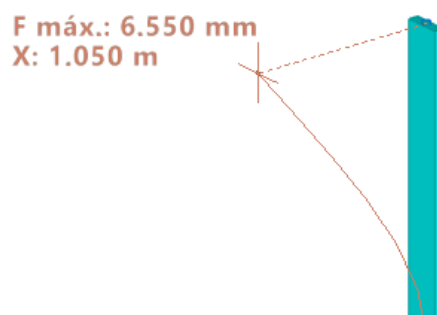


Figure 7-9. Deflection of thick plate – 93%

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7.5.3 REACTIONS

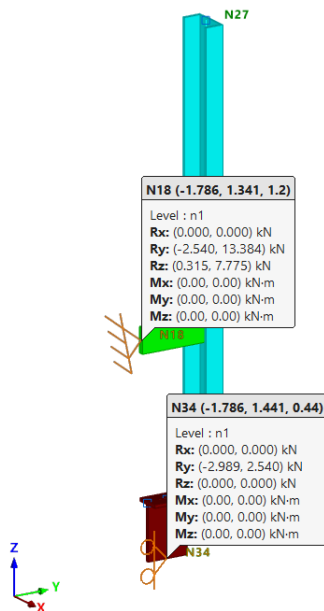


Figure 7-10. Reactions – Envelope

The 3 kN horizontal reaction on the bottom must be transmitted to the existing IPE 200 accordingly each 5 m. This horizontal reaction must be checked independently in the IPE 200 in order to verify if there is enough stress capacity on the section.

7.6 CONNECTIONS

It has been used Cypeconnect.

Steel Timber Concrete

Norway - NS EN 1993

Friction coefficient for prestressed bolts 0.30

☒ Length of fillet welds (NS EN 1993-1-8:2005/NA:2009, 4.5.1)

☒ Effective throat thickness of fillet welds (NS EN 1993-1-8:2005/NA:2009, 4.5.2)

☒ Maximum angle of the fillet welds (NS EN 1993-1-8:2005/NA:2009, 4.3.2.1)

☒ Minimum angle of the fillet welds (NS EN 1993-1-8:2005/NA:2009, 4.3.2.1)

Bolt hole layout (NS EN 1993-1-8:2005/NA:2009, 3.5) Unprotected steel (EN 10025-5)

Rotational stiffness

Limiting stiffness factor for moment connections, kb 25.00

Limiting stiffness factor for pinned connections, kb' 0.50

Plate deformation

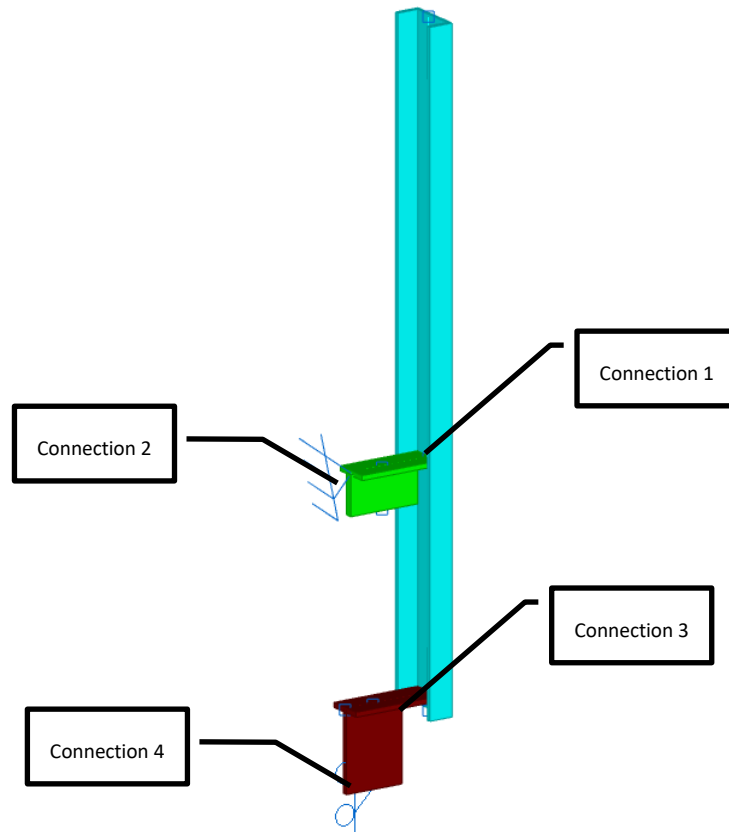
Plastic deformation limit 5.00 %

Figure 7-11. General data Cypeconnect

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7.6.1 CONNECTION 1. GREEN WELDED STEEL PLATES FORMING A T-SECTION TO UPE 140

Use throat thickness: 4 mm

Weld all around

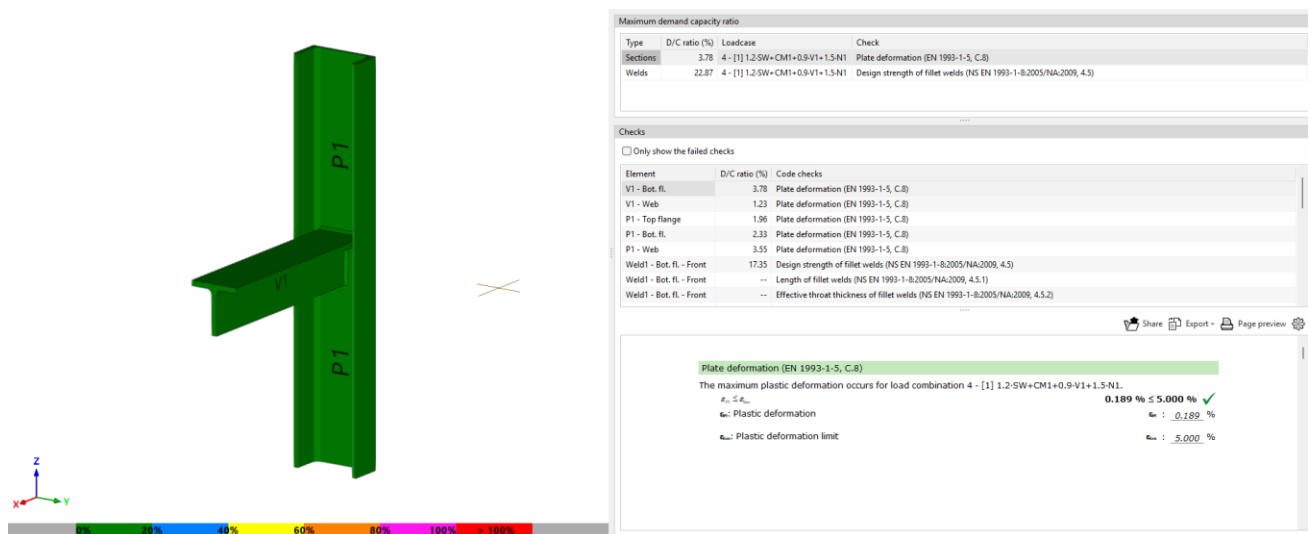


Figure 7-12. Welds checks

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Note: see full report of checks in Annex II

7.6.2 CONNECTION 2. 90x150x20 (WEB) TO PRESTRESSED BEAM

The prestressed beams are located 5 metres apart from each other:

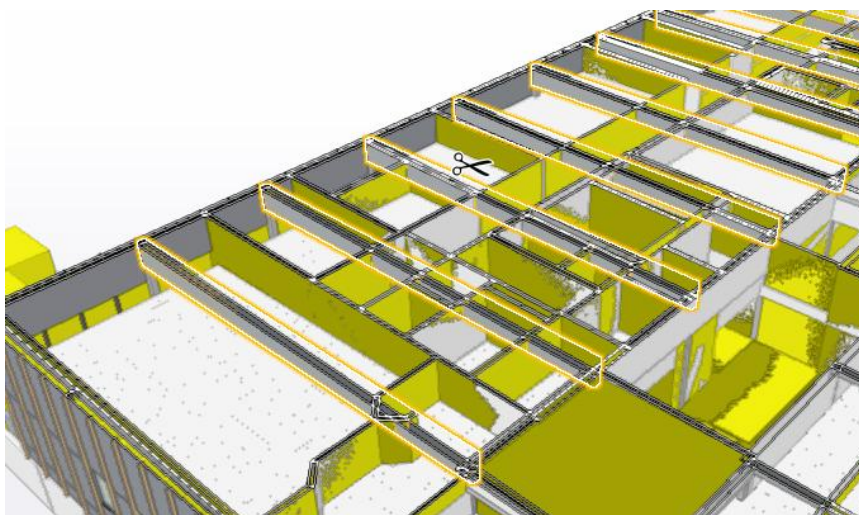


Figure 7-13. Prestressed beams configuration

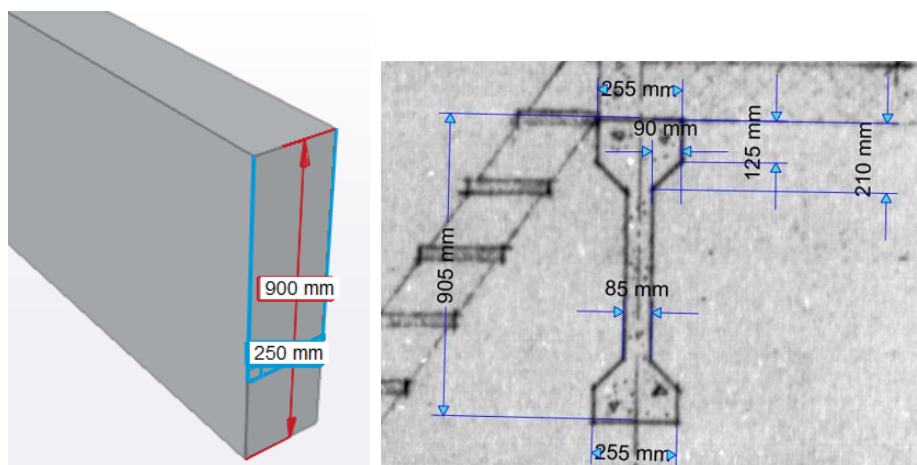


Figure 7-14. Prestressed beam dimensions

Considering the following:

- 120 x 100 x 10 S355 Steel plate
- Information about the existing resistance of the prestressed beam is not found. It is taken C20 on the safety side as explained on “Materials on existing structure” chapter.
- Distance clearances:

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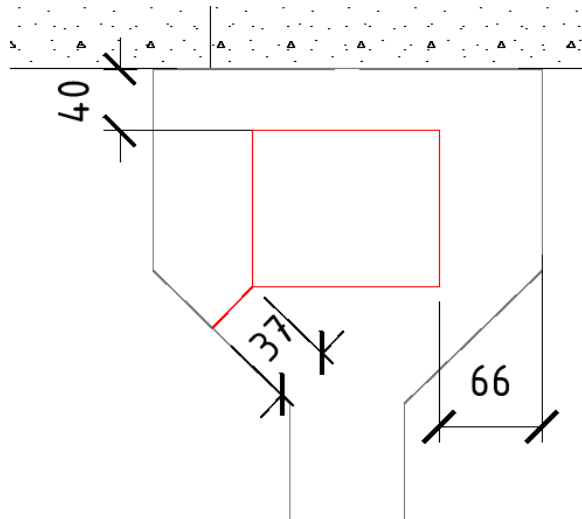


Figure 7-15. Position of the anchored base plate with regards to the top flange of the prestressed beam

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Therefore, the input on the program is:

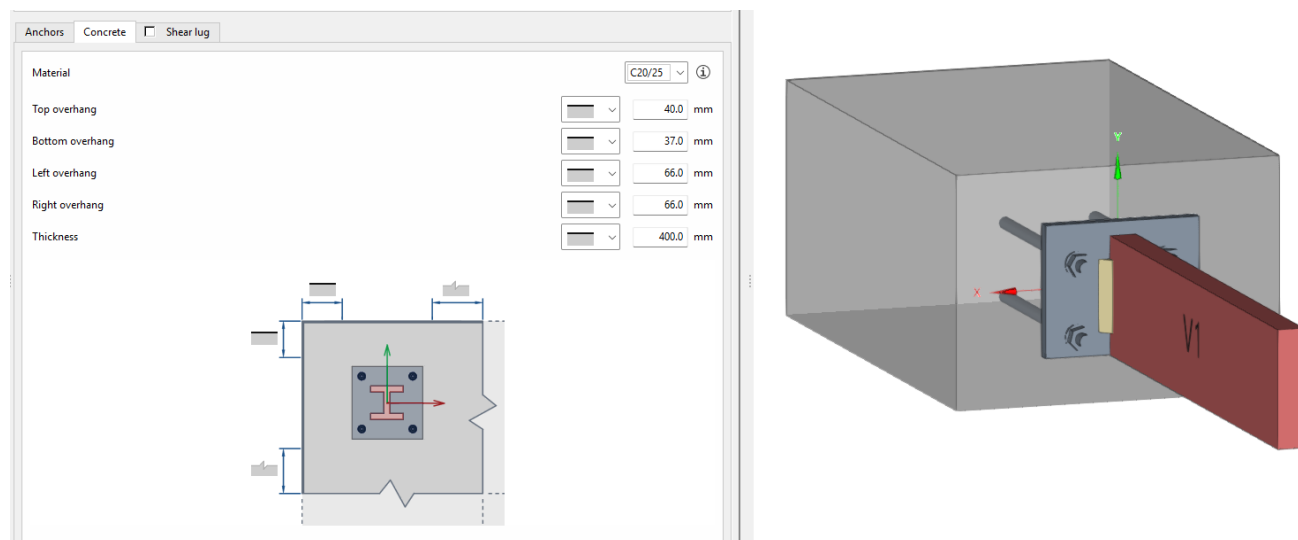


Figure 7-16. Input for dimensions of concrete element on Cypeconnect

Use 4 No. HIT HY-200 R V3+HAS-U 8.8 M10x190, with 160 mm embedment.

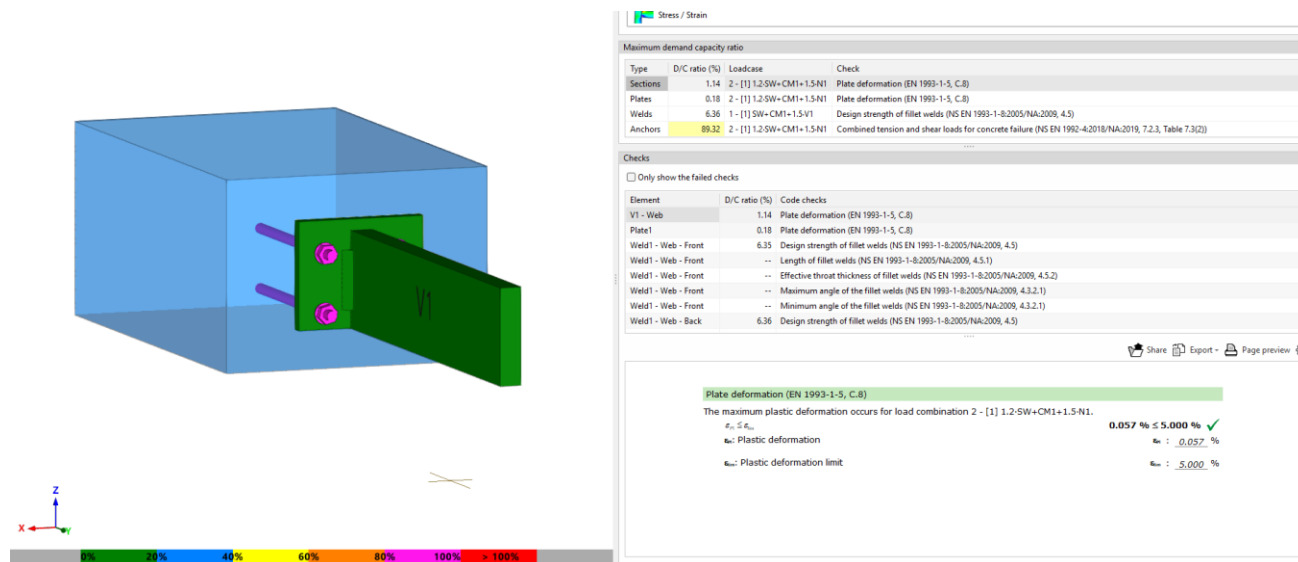


Figure 7-17. Main checks of anchors and welds

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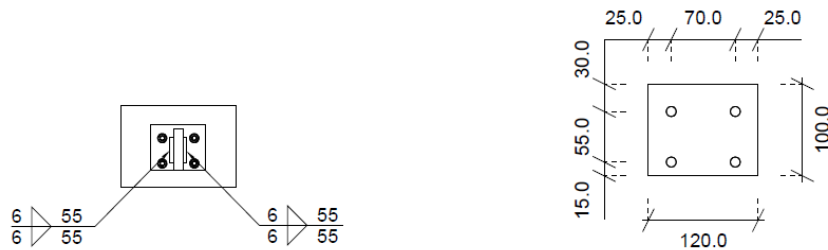


Figure 7-18. Holes disposition on base plate connected to prestressed beams

Note: see full report of checks in Annex II

7.6.3 CONNECTION 3. 90x195x10 (FLANGE) TO UPE 140

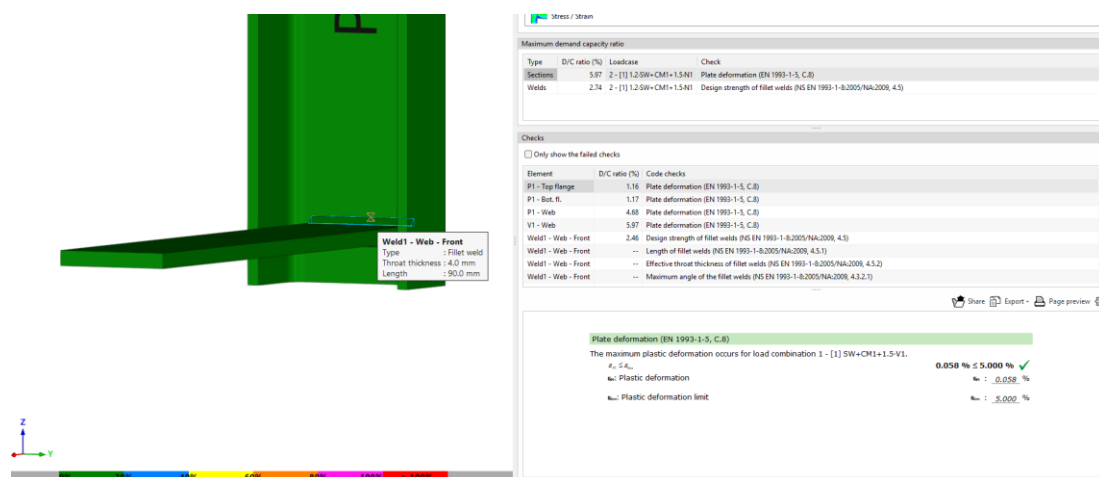


Figure 7-19. Welds checks

Note: see full report of checks in Annex II

Properties of the welding:

- Throat thickness: 4 mm
- Welded all around

7.6.4 CONNECTION 4. VERTICAL STEEL PLATE WELDED TO IPE 200'S WEB

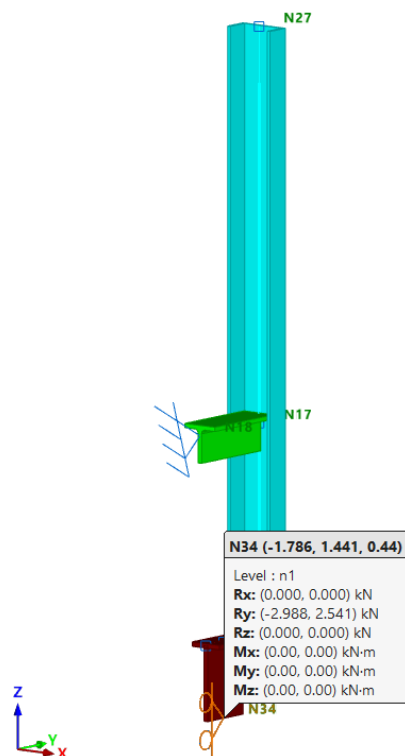
This plate is receiving around 3 kN as horizontal reaction.

The calculation for the throat of the welding is the following:

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ULS VERIFICATION FOR FILLET WELDS DIRECTIONAL METHOD AS PER §4.5.3.2 - EN 1993-1-8			
CODE		NS-EN 1993-1	
Y_{a1}		1,05	
Y_{a2}		1,25	
PLATE	Plate thickness (t)	12	mm
	steel	S 355	
	f_y	355	N/mm ²
	f_u	470	N/mm ²
WELD	Throat thickness a	4,0	mm
	Weld length L_w	95	mm
	L_{eff}	87	mm
	L_{min}	24	mm
	A_w	348	mm ²
ACTIONS ON 1 WELD	F1	3,0	kN
	F2	0,0	kN
	F3	0,0	kN
RESULTS	σ_T	0,0	N/mm ²
	τ_T	0,0	N/mm ²
	τ_{II}	8,6	N/mm ²
	β_w	0,9	
	Long Lap Joint	YES	
	β_{Lw1}	1,00	
	ultimate weld stress	417,8	N/mm ²
	max σ_T	338,4	N/mm ²
	weld stress	14,9	N/mm ²
	factor	4%	

As a result, the properties of the welding is:

- Throat thickness: 4 mm
- Welded all around

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7.6.5 BOLTS TO CONNECT PARAPET AND STEEL PLATE

Considering that the total vertical reaction is 15 kN, and taking into account that the parapet has a total height of 800 and bolts are located each 200 mm.

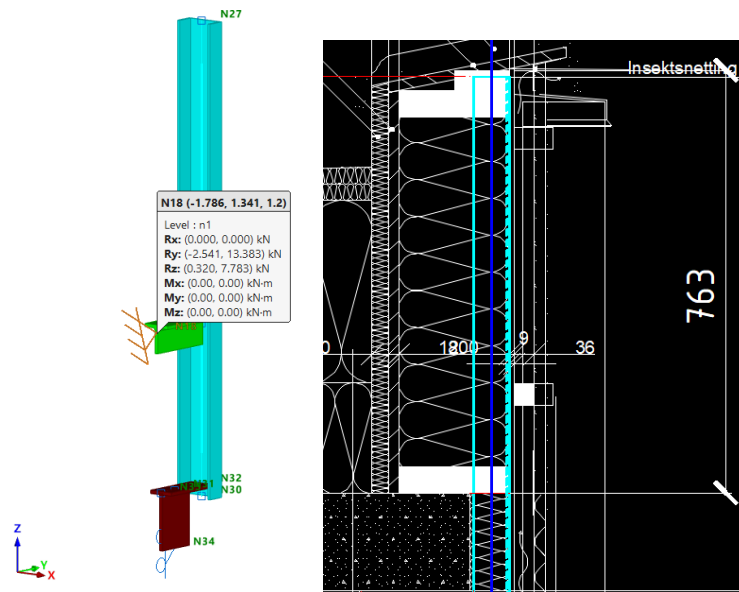


Figure 7-20. Vertical reaction and parapet dimensions

Each bolt is receiving:

- N° of bolts: $800 / 200 = 4$
- $15 / 4 = 3.75$ kN each

Contractor to ensure that bolts chosen to connect the parapet to the blue steel plate must resist 3.75 kN at least.

7.6.6 SLOTTED HOLES

The maximum vertical displacement is 3.5 mm. Therefore, the length of the slotted holes using 4 No. M12 must be, as minimum:

- Length: $3.5 + 12 + 3.5 = 19$ mm $\rightarrow$ 20 mm, to allow displacement both upwards and downwards. The bolt must be positioned on the centre of the hole

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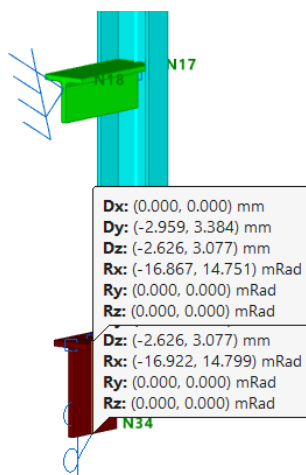


Figure 7-21. Maximum displacement

8 BALDAKIN CONNECTION TO THE TIMBER MULLIONS

8.1 CALCULATION PROGRAM

8.2 INPUT DATA

The structure was modelled with Cype 3D v26, using bar elements to represent each part of the existing structure to obtain the reactions on the supports and calculate the connections. The verifications of the existing frame are not undertaken.

8.2.1 PRELIMINARY CONSIDERATIONS

The aim of this chapter is to recalculate the connections of the existing baldakin to the new vertical structural elements of the façade. To model the platform of the baldakin, 150x50 mm sections are modelled for the frame to have into account a similar shape.

The frame is supported on four vertical structural members of the façade. Considering that the design snow loads in Norway have increased compared with the values typically used approximately 50 years ago, when the canopy was originally designed, the existing exterior elements supporting the cables are proposed to be replaced by new steel columns.

This replacement is particularly relevant because these exterior elements receive loads directly from the canopy frame and therefore form part of the primary load path. The new steel columns will provide a more robust and verifiable support system in accordance with the current design assumptions.

Regarding the internal columns, they are receiving the load from the frame at one point each.

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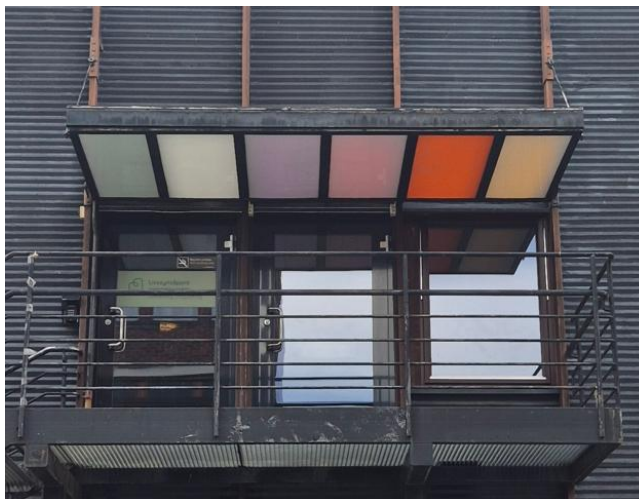


Figure 8-1. Existing situation of the baldakin.

8.2.2 GEOMETRY AND SECTIONS

The dimensions of the platform below are taken into account to approximate the dimensions of the baldakin:

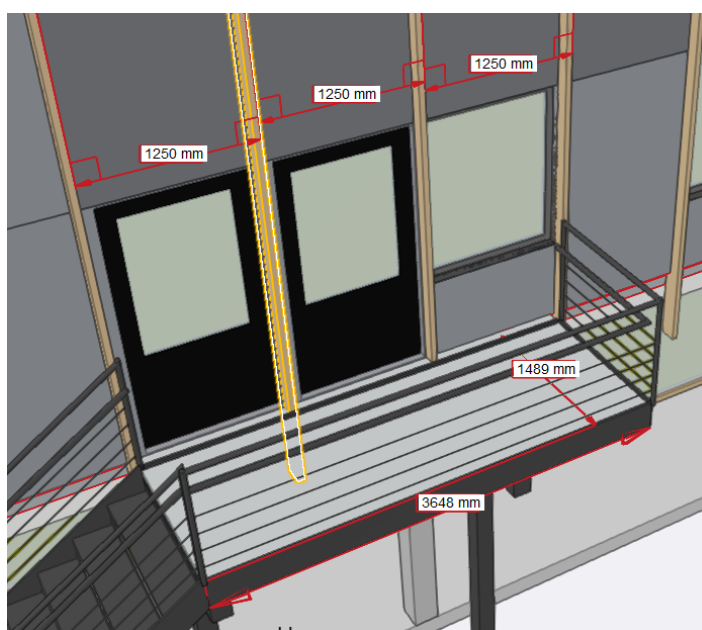


Figure 8-2. Dimensions of the existing platform to consider the dimensions of the baldakin

- Length: 3.8 m
- Width: 1.5 m

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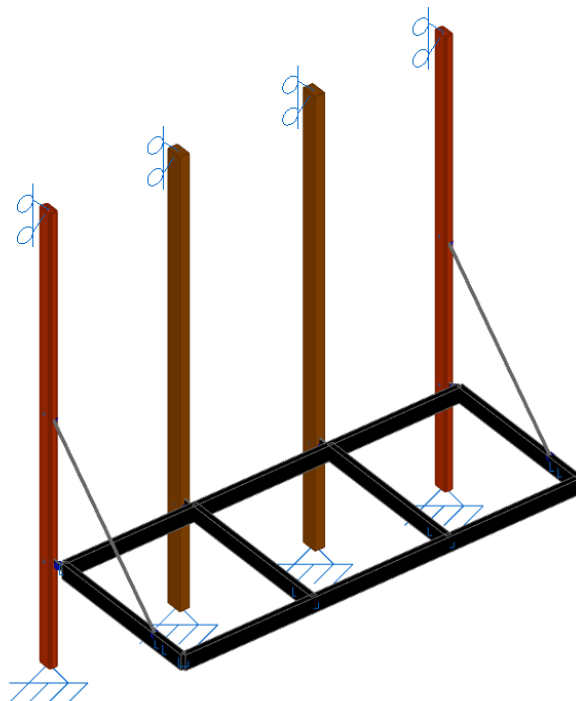


Figure 8-3. Baldakin model with the vertical supports of the façade.

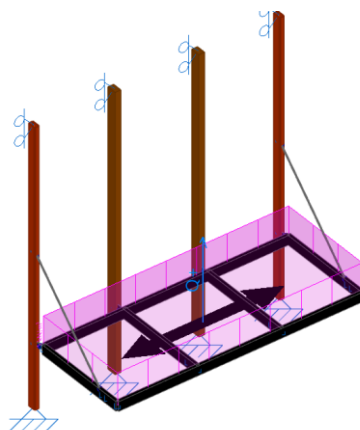
- Internal columns: GI30c, 4.3 m high
- External columns: RHS 140x80x10 S355, 4.3 m high

8.3 BUCKLING

Automatically set by the software.

8.4 LOADS APPLIED

- Area loads:



Loads on panel		
Loadcase	Positive direction	Value
CM 1	Downward vertical	0.200
V 1	Upward vertical	-1.000
N 1	Downward vertical	13.300

Figure 8-4. Loads considered on the panel

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- Point loads following Figure 7.16 EN 1991-1-4:2005 (E)

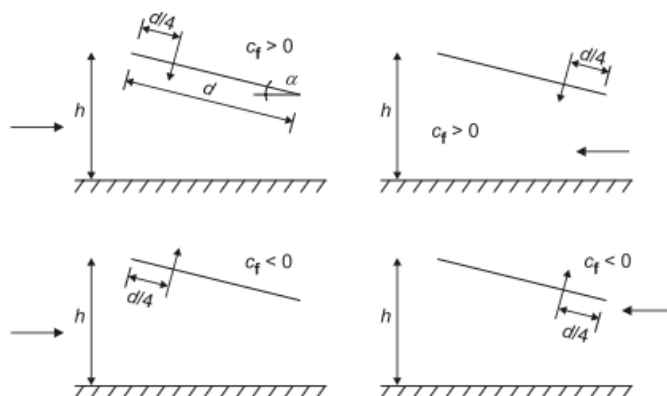


Figure 7.16 — Location of the centre of force for monopitch canopies

- Area: $4.5 \times 1.5 = 6.75 \text{ m}^2$
- Resultant to $d/4 = 1.5 / 4 = 0.2 \text{ m}$
 - External areas: $(1.5 \times 1.5 / 2) = 1.0 \text{ kN}$
 - Internal areas: $(1.5 \times 1.5) = 2.3 \text{ kN}$

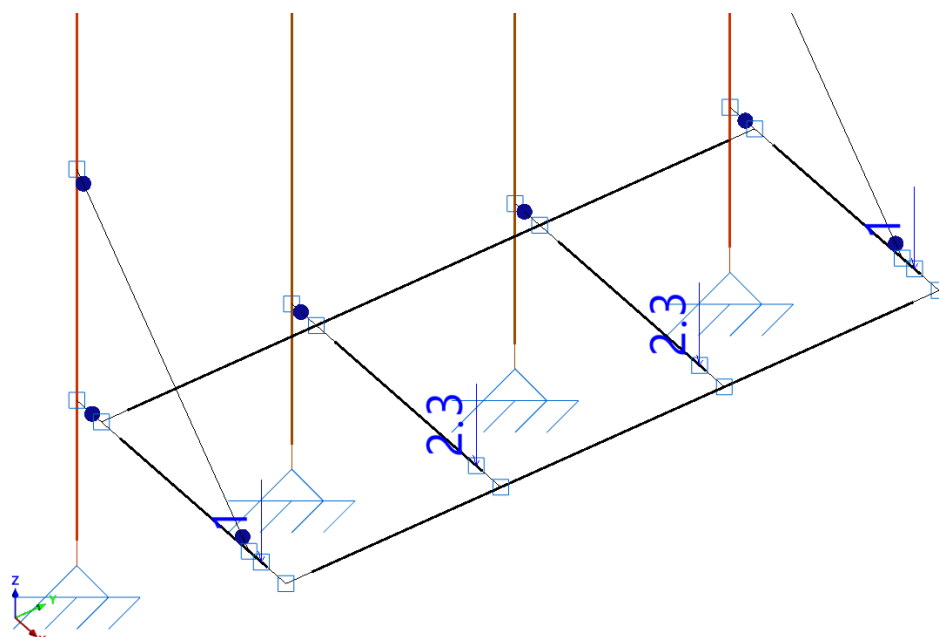


Figure 8-5. Point loads due to the wind

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8.5 RESULTS

The elements which are checked are only the vertical columns, as they are new elements. It is checked that they comply in green.

8.5.1 ULS AND SLS VERIFICATION

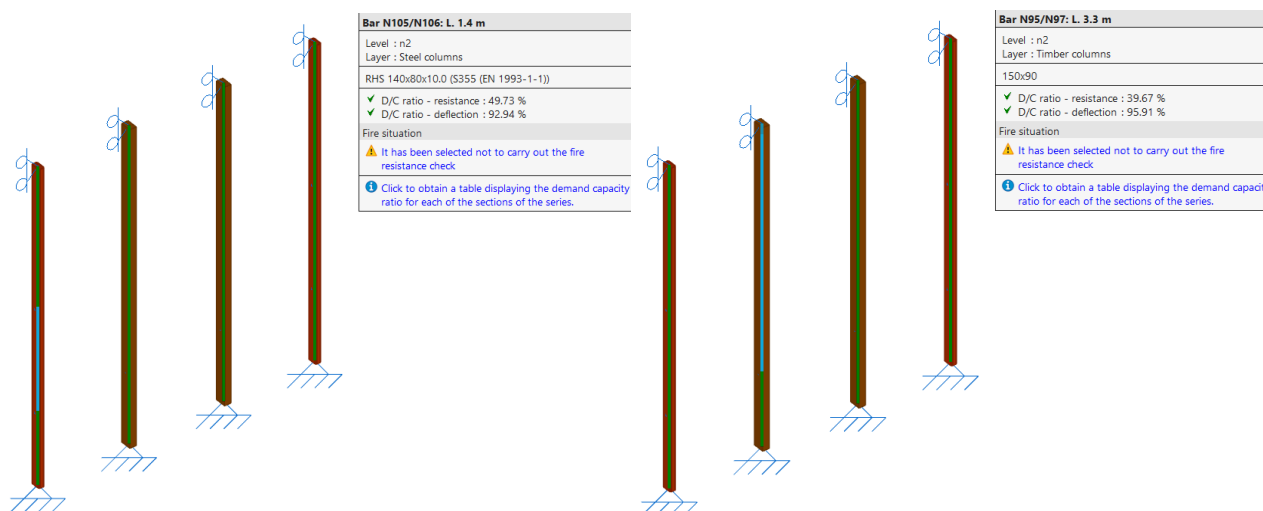


Figure 8-6. Checks of new steel columns and timber columns

8.5.2 REACTIONS

The internal forces of the elements of the baldakin to the columns are automatically considered with Cypeconnect to calculate the connections 1 and 2.

For connection 3, Ved = 16 kN:

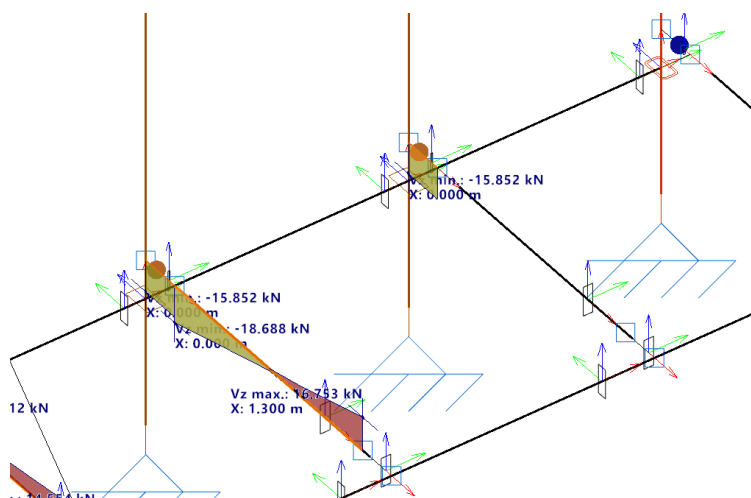


Figure 8-7. Checks of new steel columns and timber columns

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8.6 CONNECTIONS

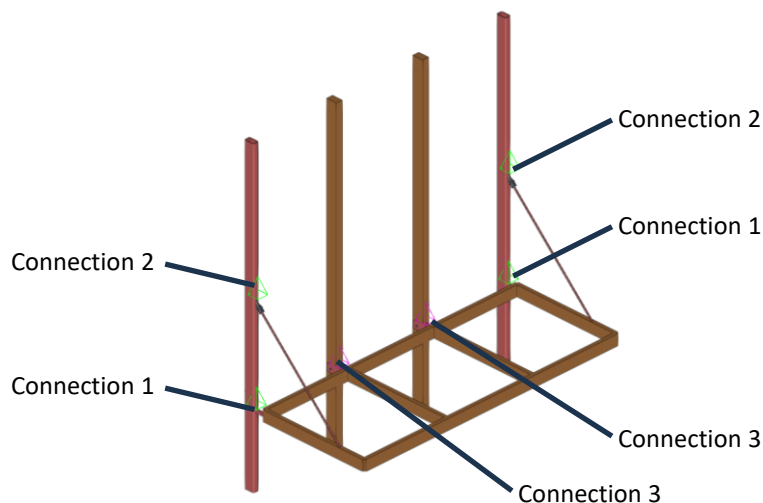


Figure 8-8. Connection types

8.6.1 CONNECTION 1

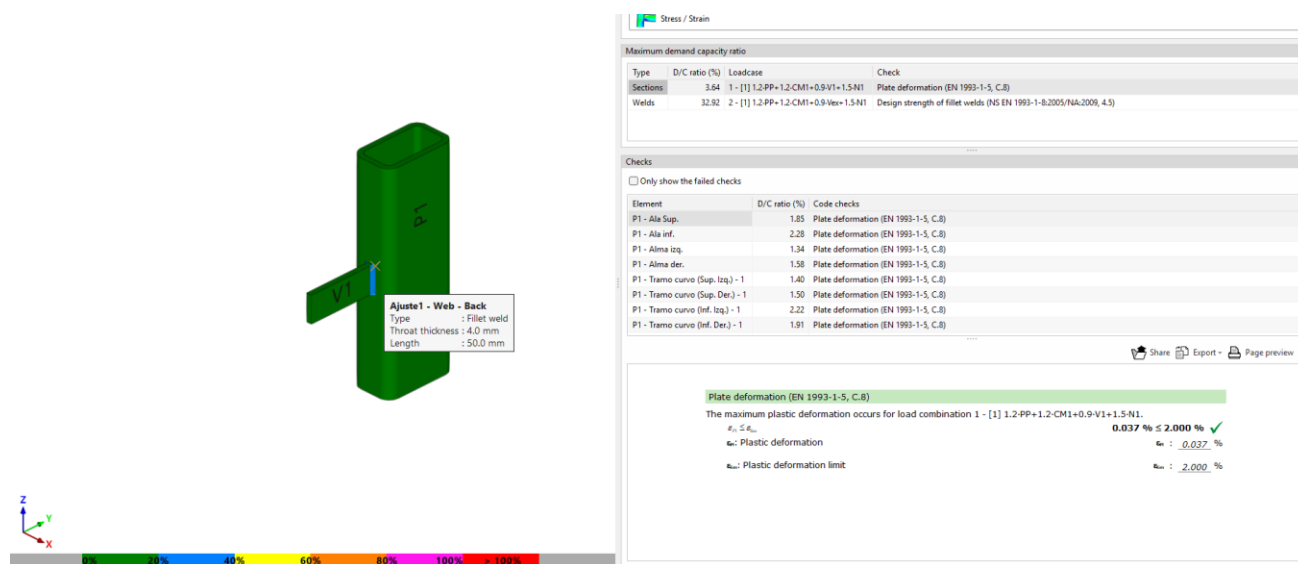


Figure 8-9. Connection 1 checks

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8.6.2 CONNECTION 2

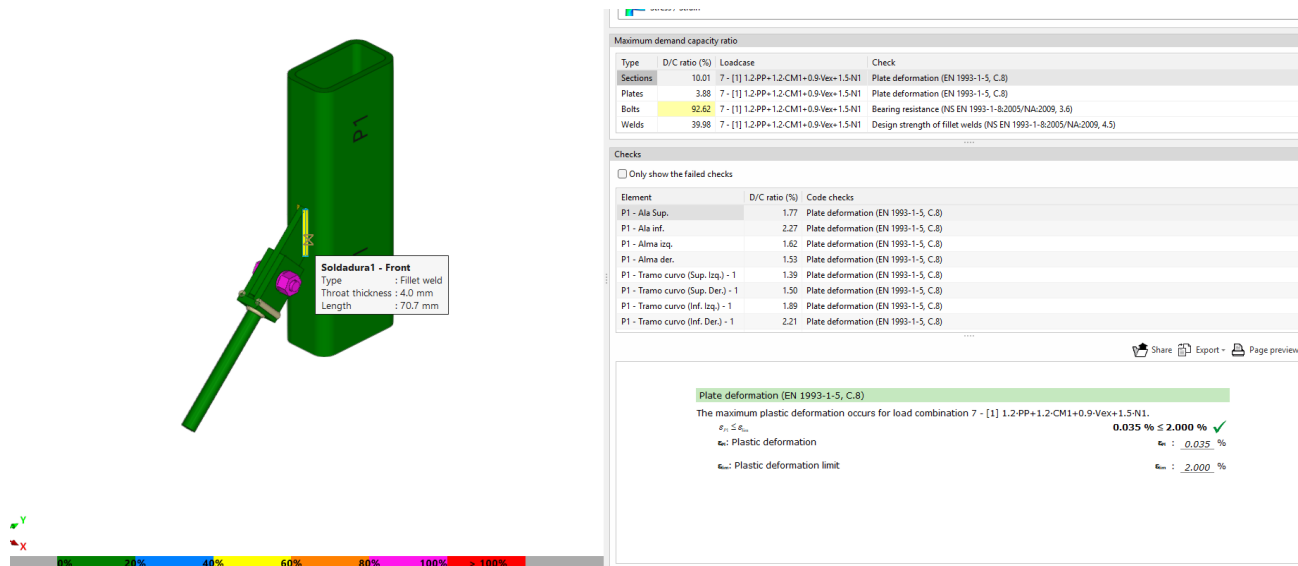


Figure 8-10. Connection 2 checks

Throat weld: 4 mm.

Existing bolt: to be minimum M16 8.8.

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8.6.3 CONNECTION 3

Considering the following, to be confirmed by contractor on site:

- T_{min} plate = 5 mm
- Ø_{min} bolt = 20 mm
- For a thin steel plate in single shear:

$$F_{v,Rk} = \min \begin{cases} 0,4 f_{h,k} t_1 d & (a) \\ 1,15 \sqrt{2 M_{y,Rk} f_{h,k} d} + \frac{F_{ax,Rk}}{4} & (b) \end{cases} \quad (8.9)$$

- For a thick steel plate in single shear:

eq. 8.9	F_{v,Rk} (N)	(a)	(b)	F_{v,Rk} min kN
thin plate single shear		18420	21913	18,4

18.4 > 16 kN. Therefore, it complies.

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9 CONSTRUCTION PROCESS

1. Preparation and setting-out

- Remove the existing finishes locally to expose the existing IPE profile and the reinforced concrete beam.
- Identify the axis of the existing concrete beam on site and use it as the reference for setting out the new support elements.
- Set out the supports at 5.00 m spacing, with a tolerance of ± 50 mm.
- Fix the base anchor plate to the existing concrete beam using Hilti anchors, or approved equivalent.

2. Installation of the UPE support

- Weld the vertical green plate to the base anchor plate.
- Position and weld the UPE profile to the vertical green plate.
- Weld the horizontal green plate to the UPE profile only. This plate shall not be welded to the base anchor plate.
- The green plates form a built-up T-section. Only the web plate is connected to the base anchor plate, while both the web and flange plates are welded to the UPE profile.

3. Connection to the existing IPE 200

- Weld the horizontal brown plate to the UPE profile.
- Install the vertical brown plate and bolt it to the plate connected to the existing IPE 200 using vertical slotted holes.
- The slotted holes shall allow vertical displacement both upwards and downwards, avoiding the transfer of vertical forces to the existing IPE 200.

4. Alternative installation sequence

- As an alternative, the full bracket may be prefabricated as a welded assembly.
- In this case, the final site operation shall be the welding of the connection plate to the existing IPE 200 and the installation of the bolted slotted-hole connection.

5. Final checks

- Check the final position, alignment and verticality of the installed UPE profile.
- Inspect all welds.
- Repair any damaged corrosion protection after installation

10 CONCLUSION

The 90x150 timber mullion satisfies the structural requirements established by the applicable Eurocode provisions for strength and serviceability under wind loading. Nevertheless, the calculated lateral deflections, particularly in the façade corner zones, are significant from a façade engineering perspective and must be explicitly considered in the glazing design. The movement capacity of fixings, pressure plates, gaskets and joints shall be verified to accommodate these displacements without inducing secondary stresses or unintended load transfer to the glass units.

The main calculation assumptions and governing design considerations adopted in this assessment are summarised in Chapter 4 of this report.

DEGREE OF FREEDOM

Limtre søyler i fasaden, gesims og baldakin - beregningsrapport
Frederikke – Rehabilitering fasade og tekniske anlegg
25086-R01

Revision	Date	Done By	Controlled By	Approved By
01	29/05/2026	JDO	FI	ER

On the other hand, a connection for the parapet to the existing structure has been designed, where the horizontal reaction to the existing IPE 200 must be checked independently in order to verify if there is enough stress capacity on the section. Finally, existing reinforcement of prestressed beam must be detected and notify to RIB if there is any conflict.

DEGREE OF FREEDOM

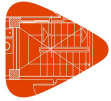
Limtre søyler i fasaden, gesims og baldakin - beregningsrapport
Frederikke – Rehabilitering fasade og tekniske anlegg
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ANNEX I

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1. JOB DATA

1.1. Codes considered

Rolled and welded steel: Eurocodes 3 and 4

1.2. Limit states

Fracture U.L.S. Rolled steel	Norway
Displacements	Characteristic loads

1.2.1. Project situations

The load combinations will be defined according to the following criteria for the different project situations:

- With combination coefficients

$$\sum_{j \geq 1} \gamma_{Gj} G_{kj} + \gamma_P P_k + \gamma_{Q1} \Psi_{p1} Q_{k1} + \sum_{i \geq 1} \gamma_{Qi} \Psi_{ai} Q_{ki}$$

- Without combination coefficients

$$\sum_{j \geq 1} \gamma_{Gj} G_{kj} + \gamma_P P_k + \sum_{i \geq 1} \gamma_{Qi} Q_{ki}$$

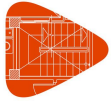
- Where:

- G_k Permanent load
- P_k Prestressing action
- Q_k Variable load
- γ_G Permanent load partial safety factor
- γ_P Partial safety coefficient for prestressing action
- $\gamma_{Q,1}$ Main variable load partial safety factor
- $\gamma_{Q,i}$ Accompanying variable load partial safety factor
- $\Psi_{p,1}$ Main variable load combination coefficient
- $\Psi_{a,i}$ Accompanying variable load combination coefficient

For each project situation and limit state, the loading coefficients will be determined by:

Fracture U.L.S. Rolled steel: Eurocodes 3 and 4

6.10a (Cat A)				
	Partial safety factors (γ)		Combination coefficients (ψ)	
	Favourable	Unfavourable	Main (ψ_p)	Accompanying (ψ_a)
Dead load (G)	1.000	1.350	-	-
Wind (Q)	0.000	1.500	0.600	0.600
Snow (Q)	0.000	1.500	0.700	0.700



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6.10b (Cat A)				
	Partial safety factors (γ)		Combination coefficients (ψ)	
	Favourable	Unfavourable	Main (ψ_p)	Accompanying (ψ_s)
Dead load (G)	1.000	1.200	-	-
Wind (Q)	0.000	1.500	1.000	0.600
Snow (Q)	0.000	1.500	1.000	0.700

Displacements

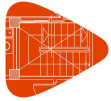
Variable loads without seismic loading		
	Partial safety factors (γ)	
	Favourable	Unfavourable
Dead load (G)	1.000	1.000
Wind (Q)	0.000	1.000
Snow (Q)	0.000	1.000

1.2.2. Combinations

▪ Loadcase names

SW Self weight
CM 1 CM 1
V 1 V 1
N 1 N 1

▪ Fracture U.L.S. Rolled steel



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Comb.	SW	CM 1	V 1	N 1
1	1.000	1.000		
2	1.350	1.000		
3	1.000	1.350		
4	1.350	1.350		
5	1.350	1.000	0.900	
6	1.000	1.350	0.900	
7	1.350	1.350	0.900	
8	1.350	1.000		1.050
9	1.000	1.350		1.050
10	1.350	1.350		1.050
11	1.350	1.000	0.900	1.050
12	1.000	1.350	0.900	1.050
13	1.350	1.350	0.900	1.050
14	1.200	1.200		
15	1.000	1.000	1.500	
16	1.200	1.000	1.500	
17	1.000	1.200	1.500	
18	1.200	1.200	1.500	
19	1.000	1.000		1.500
20	1.200	1.000		1.500
21	1.000	1.200		1.500
22	1.200	1.200		1.500
23	1.000	1.000	0.900	1.500
24	1.200	1.000	0.900	1.500
25	1.000	1.200	0.900	1.500
26	1.200	1.200	0.900	1.500
27	1.000	1.000	1.500	1.050
28	1.200	1.000	1.500	1.050
29	1.000	1.200	1.500	1.050
30	1.200	1.200	1.500	1.050

▪ Displacements

Comb.	SW	CM 1	V 1	N 1
1	1.000	1.000		
2	1.000	1.000	1.000	
3	1.000	1.000		1.000
4	1.000	1.000	1.000	1.000

2. STRUCTURE

2.1. Geometry

2.1.1. Nodes

References:

$\Delta_x, \Delta_y, \Delta_z$: Prescribed displacements in global axes.

$\theta_x, \theta_y, \theta_z$: Prescribed rotations in global axes.

U_x, U_y, U_z : Direction vector of the straight line or vector normal to the dependency plane

Each degree of freedom is marked with 'X' if it has external restrictions and, in the opposite case, with '-':

[illegible]

2.1.2. Bars

2.1.2.1. Materials used

Materials used							
Material		E (MPa)	ν	G (MPa)	f_y (MPa)	α_t (m/m°C)	γ (kN/m ³)
Type	Designation						
Rolled steel	S355 (EN 1993-1-1)	210000.00	0.300	81000.00	355.00	0.000012	77.01
Notes: <i>E: Modulus of Elasticity</i> <i>ν: Poisson's ratio</i> <i>G: Shear Modulus</i> <i>f_y: Yield strength</i> <i>α_t: Coefficient of thermal expansion</i> <i>γ: Unit weight</i>							

2.1.2.2. Description

Description									
Material		Bar (Ni/Nf)	Element (Ni/Nf)	Section(Series)	Length (m)	β_{xy}	β_{xz}	Lb _{Top} (m)	Lb _{Bot.} (m)
Type	Designation								
Rolled steel	S355 (EN 1993-1-1)	N18/N17	N18/N17	FL 90 x 20 (Pletinas)	0.250	1.00	1.00	0.250	0.250
		N30/N32	N30/N27	UPE 140 (UPE)	0.040	1.00	1.00	0.040	0.040
		N32/N17	N30/N27	UPE 140 (UPE)	0.560	1.00	1.00	0.560	0.560
		N17/N27	N30/N27	UPE 140 (UPE)	1.050	2.00	2.00	1.050	1.050
		N34/N31	N34/N31	FL 150 x 12 (Pletinas)	0.200	1.00	1.00	0.200	0.200
		N35/N31	N35/N32	FL 90 x 12 (Pletinas)	0.075	1.00	1.00	0.075	0.075
		N31/N32	N35/N32	FL 90 x 12 (Pletinas)	0.150	1.00	1.00	0.150	0.150
Notes: <i>Ni: Initial node</i> <i>Nf: Final node</i> <i>β_{xy}: Buckling coefficient in the 'XY' plane</i> <i>β_{xz}: Buckling coefficient in the 'XZ' plane</i> <i>Lb_{Top}: Separation between bracings of the top flange</i> <i>Lb_{Bot}: Separation between bracinas of the bottom flange</i>									



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2.1.2.3. Mechanical characteristics

Element types	
Ref.	Elements
1	N18/N17
2	N30/N27
3	N34/N31
4	N35/N32

Mechanical characteristics									
Material		Ref.	Description	A (cm ²)	Avy (cm ²)	Avz (cm ²)	Iyy (cm4)	Izz (cm4)	It (cm4)
Type	Designation								
Rolled steel	S355 (EN 1993-1-1)	1	FL 90 x 20, (Pletinas)	18.00	15.00	15.00	121.50	6.00	20.56
		2	UPE 140, (UPE)	18.40	8.78	5.49	600.00	78.80	4.05
		3	FL 150 x 12, (Pletinas)	18.00	15.00	15.00	337.50	2.16	8.63
		4	FL 90 x 12, (Pletinas)	10.80	9.00	9.00	72.90	1.30	4.73
Notes: Ref.: Reference A: Area of the transverse section Avy: Shear area of the section in the local 'Y' axis Avz: Shear area of the section in the local 'Z' axis Iyy: Inertia of the section about the local 'Y' axis Izz: Inertia of the section about the local 'Z' axis It: Torsional inertia The mechanical characteristics of the elements correspond to those of their cross section at their mid-point.									

2.1.2.4. Quantities table

Quantities table						
Material		Element (Ni/Nf)	Section(Series)	Length (m)	Volume (m³)	Weight (kg)
Type	Designation					
Rolled steel	S355 (EN 1993-1-1)	N18/N17	FL 90 x 20 (Pletinas)	0.250	0.000	3.53
		N30/N27	UPE 140 (UPE)	1.650	0.003	23.83
		N34/N31	FL 150 x 12 (Pletinas)	0.200	0.000	2.83
		N35/N32	FL 90 x 12 (Pletinas)	0.225	0.000	1.91
Notes: Ni: Initial node Nf: Final node						

2.1.2.5. Quantities summary

Quantities summary												
Material		Series	Section	Length			Volume			Weight		
Type	Designation			Section (m)	Series (m)	Material (m)	Section (m³)	Series (m³)	Material (m³)	Section (kg)	Series (kg)	Material (kg)
Rolled steel	S355 (EN 1993-1-1)	Pletinas	FL 90 x 20	0.250	0.675		0.000	0.001		3.53	8.27	
			FL 150 x 12	0.200			0.000			2.83		
			FL 90 x 12	0.225			0.000			1.91		
		UPE	UPE 140	1.650	1.650	2.325	0.003	0.003	0.004	23.83	23.83	32.10

2.1.2.6. Surface area quantities



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Rolled steel: Quantities of the surface areas to paint				
Series	Section	Unit surface area (m ² /m)	Length (m)	Surface (m ²)
Pletinas	FL 90 x 20	0.220	0.250	0.055
	FL 150 x 12	0.324	0.200	0.065
	FL 90 x 12	0.204	0.225	0.046
UPE	UPE 140	0.530	1.650	0.875
Total				1.040

2.2. Loads

2.2.1. Nodes

Loads on nodes					
Reference	Loadcase	Point loads (kN)	Direction		
			X	Y	Z
N27	N 1	4.90	0.000	0.000	-1.000

2.2.2. Bars

References:

'P1', 'P2':

- Point moments, point, uniform and strip loads: 'P1' is the load value. 'P2' is not used.
- Trapezoidal loads: 'P1' is the value of the load at the point where it begins (L1) and 'P2' is the value of the load where it ends (L2).
- Triangular loads: 'P1' is the maximum value of the load. 'P2' is not used.
- Temperature increments: 'P1' and 'P2' are the temperature values on the external faces or surfaces of the element. The position of the temperature increment over the transverse section will depend on the selected direction.

'L1', 'L2':

- Point loads and moments: 'L1' is the distance between the initial node of the bar and the position where the load is applied. 'L2' is not used.
- Trapezoidal, strip and triangular loads: 'L1' is the distance between the initial node of the bar and the position where the load begins, 'L2' is the distance between the initial node of the bar and the position where the load ends.

Units:

- Point loads: kN
- Point moments: kN·m.
- Uniform, strip, triangular and trapezoidal loads: kN/m.
- Temperature increments: °C.

Loads on bars										
Bar	Loadcase	Type	Values		Position		Direction			
			P1	P2	L1 (m)	L2 (m)	Centre axis	X	Y	Z
N18/N17	Self weight	Uniform	0.139	-	-	-	Global	0.000	0.000	-1.000
N30/N32	Self weight	Uniform	0.142	-	-	-	Global	0.000	0.000	-1.000
N30/N32	V 1	Uniform	4.200	-	-	-	Global	0.000	-1.000	0.000
N32/N17	Self weight	Uniform	0.142	-	-	-	Global	0.000	0.000	-1.000
N32/N17	V 1	Uniform	4.200	-	-	-	Global	0.000	-1.000	0.000
N17/N27	Self weight	Uniform	0.142	-	-	-	Global	0.000	0.000	-1.000
N17/N27	V 1	Uniform	4.200	-	-	-	Global	0.000	-1.000	0.000
N34/N31	Self weight	Uniform	0.139	-	-	-	Global	0.000	0.000	-1.000



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Loads on bars										
Bar	Loadcase	Type	Values		Position		Direction			
			P1	P2	L1 (m)	L2 (m)	Centre axis	X	Y	Z
N35/N31	Self weight	Uniform	0.083	-	-	-	Global	0.000	0.000	-1.000
N31/N32	Self weight	Uniform	0.083	-	-	-	Global	0.000	0.000	-1.000

2.3. Results

2.3.1. Nodes

2.3.1.1. Reactions

References:

Rx, Ry, Rz: Node reactions with displacements with external fixity (forces).

Mx, My, Mz: Node reactions with rotations with external fixity (moments).

2.3.1.1.1. Envelope

Node reaction envelopes								
Reference	Combination		Reactions in global axes					
	Type	Description	Rx (kN)	Ry (kN)	Rz (kN)	Mx (kN·m)	My (kN·m)	Mz (kN·m)
N18	Foundation concrete	Minimum envelope value	0.000	-2.540	0.315	0.00	0.00	0.00
		Maximum envelope value	0.000	13.384	7.775	0.00	0.00	0.00
	Ground bearing pressures	Minimum envelope value	0.000	-2.186	0.315	0.00	0.00	0.00
		Maximum envelope value	0.000	11.587	6.685	0.00	0.00	0.00
N34	Foundation concrete	Minimum envelope value	0.000	-2.989	0.000	0.00	0.00	0.00
		Maximum envelope value	0.000	2.540	0.000	0.00	0.00	0.00
	Ground bearing pressures	Minimum envelope value	0.000	-2.578	0.000	0.00	0.00	0.00
		Maximum envelope value	0.000	2.186	0.000	0.00	0.00	0.00

Note: The indicated concrete combinations are the same as those used to check the equilibrium limit state of the foundations.

2.3.2. Bars

2.3.2.1. Forces

References:

N: Axial force (kN)

Vy: Shear force in the local Y axis of the bar. (kN)

Vz: Shear force in the local Z axis of the bar. (kN)

Mt: Torsional moment (kN·m)

My: Bending moment in the 'XZ' plane (section rotation with respect to the local 'Y' axis of the bar). (kN·m)

Mz: Bending moment in the 'XY' plane (section rotation with respect to the local 'Z' axis of the bar). (kN·m)

2.3.2.1.1. Envelope

Bar force envelopes					
Bar	Combination type	Force	Positions on the bar		
			0.000 m	0.125 m	0.250 m
N18/N17	Rolled steel	N _{min}	-13.384	-13.384	-13.384
		N _{max}	2.526	2.526	2.526
		Vy _{min}	0.000	0.000	0.000
		Vy _{max}	0.000	0.000	0.000
		Vz _{min}	-7.728	-7.707	-7.686
		Vz _{max}	-0.315	-0.298	-0.280



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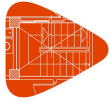
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Bar force envelopes					
Bar	Combination type	Force	Positions on the bar		
			0.000 m	0.125 m	0.250 m
		$M_{t_{min}}$	0.00	0.00	0.00
		$M_{t_{max}}$	0.00	0.00	0.00
		$M_{y_{min}}$	0.00	0.04	0.07
		$M_{y_{max}}$	0.00	0.96	1.93
		$M_{z_{min}}$	0.00	0.00	0.00
		$M_{z_{max}}$	0.00	0.00	0.00

Bar force envelopes					
Bar	Combination type	Force	Positions on the bar		
			0.000 m	0.020 m	0.040 m
N30/N32	Rolled steel	N_{min}	0.000	0.003	0.006
		N_{max}	0.000	0.004	0.008
		$V_{y_{min}}$	0.000	-0.127	-0.254
		$V_{y_{max}}$	0.000	0.000	0.000
		$V_{z_{min}}$	0.000	0.000	0.000
		$V_{z_{max}}$	0.000	0.000	0.000
		$M_{t_{min}}$	0.00	0.00	0.00
		$M_{t_{max}}$	0.00	0.00	0.00
		$M_{y_{min}}$	0.00	0.00	0.00
		$M_{y_{max}}$	0.00	0.00	0.00
		$M_{z_{min}}$	0.00	0.00	0.00
		$M_{z_{max}}$	0.00	0.00	0.01

Bar force envelopes					
Bar	Combination type	Force	Positions on the bar		
			0.000 m	0.280 m	0.560 m
N32/N17	Rolled steel	N_{min}	0.052	0.092	0.131
		N_{max}	0.070	0.124	0.177
		$V_{y_{min}}$	-3.242	-5.006	-6.769
		$V_{y_{max}}$	2.526	2.526	2.526
		$V_{z_{min}}$	0.000	0.000	0.000
		$V_{z_{max}}$	0.000	0.000	0.000
		$M_{t_{min}}$	0.00	0.00	0.00
		$M_{t_{max}}$	0.00	0.00	0.00
		$M_{y_{min}}$	0.00	0.00	0.00
		$M_{y_{max}}$	0.00	0.00	0.00
		$M_{z_{min}}$	-0.51	-1.22	-1.93
		$M_{z_{max}}$	0.60	1.75	3.40

Bar force envelopes									
Bar	Combination type	Force	Positions on the bar						
			0.000 m	0.175 m	0.350 m	0.525 m	0.700 m	0.875 m	1.050 m
N17/N27	Rolled steel	N_{min}	-7.529	-7.499	-7.469	-7.439	-7.410	-7.380	-7.350
		N_{max}	-0.149	-0.124	-0.099	-0.074	-0.050	-0.025	0.000



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Bar force envelopes									
Bar	Combination type	Force	Positions on the bar						
			0.000 m	0.175 m	0.350 m	0.525 m	0.700 m	0.875 m	1.050 m
		Vy _{min}	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Vy _{max}	6.615	5.513	4.410	3.308	2.205	1.103	0.000
		Vz _{min}	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Vz _{max}	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Mt _{min}	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		Mt _{max}	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		My _{min}	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		My _{max}	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		Mz _{min}	0.00	0.00	0.00	0.00	0.00	0.00	0.00
		Mz _{max}	3.47	2.41	1.54	0.87	0.39	0.10	0.00

Bar force envelopes					
Bar	Combination type	Force	Positions on the bar		
			0.000 m	0.100 m	0.200 m
N34/N31	Rolled steel	N _{min}	0.000	0.014	0.028
		N _{max}	0.000	0.019	0.037
		Vy _{min}	0.000	0.000	0.000
		Vy _{max}	0.000	0.000	0.000
		Vz _{min}	-2.526	-2.526	-2.526
		Vz _{max}	2.989	2.989	2.989
		Mt _{min}	0.00	0.00	0.00
		Mt _{max}	0.00	0.00	0.00
		My _{min}	0.00	-0.30	-0.60
		My _{max}	0.00	0.25	0.51
		Mz _{min}	0.00	0.00	0.00
		Mz _{max}	0.00	0.00	0.00

Bar force envelopes					
Bar	Combination type	Force	Positions on the bar		
			0.000 m	0.037 m	0.075 m
N35/N31	Rolled steel	N _{min}	0.000	0.000	0.000
		N _{max}	0.000	0.000	0.000
		Vy _{min}	0.000	0.003	0.006
		Vy _{max}	0.000	0.004	0.008
		Vz _{min}	0.000	0.000	0.000
		Vz _{max}	0.000	0.000	0.000
		Mt _{min}	0.00	0.00	0.00
		Mt _{max}	0.00	0.00	0.00
		My _{min}	0.00	0.00	0.00
		My _{max}	0.00	0.00	0.00
		Mz _{min}	0.00	0.00	0.00
		Mz _{max}	0.00	0.00	0.00



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Bar force envelopes					
Bar	Combination type	Force	Positions on the bar		
			0.000 m	0.075 m	0.150 m
N31/N32	Rolled steel	N _{min}	-2.526	-2.526	-2.526
		N _{max}	2.989	2.989	2.989
		Vy _{min}	0.034	0.040	0.046
		Vy _{max}	0.046	0.054	0.063
		Vz _{min}	0.000	0.000	0.000
		Vz _{max}	0.000	0.000	0.000
		Mt _{min}	0.00	0.00	0.00
		Mt _{max}	0.00	0.00	0.00
		My _{min}	0.00	0.00	0.00
		My _{max}	0.00	0.00	0.00
		Mz _{min}	-0.51	-0.51	-0.51
		Mz _{max}	0.60	0.59	0.59

2.3.2.2. Resistance

References:

N: Axial force (kN)

Vy: Shear force in the local Y axis of the bar. (kN)

Vz: Shear force in the local Z axis of the bar. (kN)

Mt: Torsional moment (kN·m)

My: Bending moment in the 'XZ' plane (section rotation with respect to the local 'Y' axis of the bar). (kN·m)

Mz: Bending moment in the 'XY' plane (section rotation with respect to the local 'Z' axis of the bar). (kN·m)

The indicated forces are those corresponding to the worst case combination, in other words, that requiring the maximum resistance of the section.

Origin of worst case loads:

- G: Only gravity loads
- GW: Gravity loads + wind
- GE: Gravity loads + earthquake
- GWE: Gravity loads + wind + earthquake

η : Demand capacity ratio - resistance. The bar complies with the code resistance conditions if $\eta \leq 100\%$.

Resistance check										
Bar	η (%)	Position (m)	Worst case forces						Origin	Status
			N (kN)	Vy (kN)	Vz (kN)	Mt (kN·m)	My (kN·m)	Mz (kN·m)		
N18/N17	20.97	0.250	-5.558	0.000	-7.686	0.00	1.93	0.00	GW	Verified
N30/N32	0.10	0.040	0.006	-0.254	0.000	0.00	0.00	0.01	GW	Verified
N32/N17	28.86	0.560	0.131	-6.769	0.000	0.00	0.00	3.40	GW	Verified
N17/N27	30.87	0.000	-5.324	6.615	0.000	0.00	0.00	3.47	GW	Verified
N34/N31	3.75	0.200	0.028	0.000	2.989	0.00	-0.60	0.00	GW	Verified
N35/N31	0.00	0.075	0.000	0.008	0.000	0.00	0.00	0.00	G	Verified
N31/N32	78.70	0.000	2.989	0.034	0.000	0.00	0.00	0.60	GW	Verified



Reports

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2.3.2.3. Deflections

References:

Pos.: Local 'X' coordinate value of the deflection group at the point at which the worst case deflection value occurs.

L.: Distance between two consecutive cut-off points of the deformed shape on the straight line that joins the end nodes of the deflection group.

Deflections								
Group	Absolute maximum deflection xy Relative maximum deflection xy		Absolute maximum deflection xz Relative maximum deflection xz		Absolute active deflection xy Relative active deflection xy		Absolute active deflection xz Relative active deflection xz	
	Pos. (m)	Deflection (mm)	Pos. (m)	Deflection (mm)	Pos. (m)	Deflection (mm)	Pos. (m)	Deflection (mm)
N18/N17	0.000	0.00	0.125	0.02	0.000	0.00	0.125	0.02
	-	L/(>1000)	0.125	L/(>1000)	-	L/(>1000)	0.125	L/(>1000)
N17/N27	1.050	3.89	0.000	0.00	1.050	3.89	0.000	0.00
	1.050	L/270.0	-	L/(>1000)	1.050	L/270.0	-	L/(>1000)
N30/N17	0.320	0.30	0.000	0.00	0.320	0.52	0.000	0.00
	0.320	L/(>1000)	-	L/(>1000)	0.320	L/(>1000)	-	L/(>1000)
N31/N32	0.075	0.40	0.000	0.00	0.075	0.76	0.000	0.00
	0.075	L/372.7	-	L/(>1000)	0.075	L/393.9	-	L/(>1000)
N34/N31	0.000	0.00	0.100	0.00	0.000	0.00	0.100	0.00
	-	L/(>1000)	0.100	L/(>1000)	-	L/(>1000)	0.100	L/(>1000)
N35/N31	0.037	0.00	0.000	0.00	0.000	0.00	0.000	0.00
	0.037	L/(>1000)	-	L/(>1000)	-	L/(>1000)	-	L/(>1000)

2.3.2.4. U.L.S. Checks (Summarised)

Bars	CHECKS (EUROCODE 3 EN 1993-1-1: 2005)													Status
	N _t	N _c	M _y	M _z	V _z	V _y	M _y V _z	M _z V _y	NM _y M _z	NM _y M _z V _y V _z	M _t	M _t V _z	M _t V _y	
N18/N17	η = 0.4	η = 2.1	x: 0.25 m η = 20.1	M _{Ed} = 0.00 D.N.P. ⁽¹⁾	x: 0 m η = 2.1	V _{Ed} = 0.00 D.N.P. ⁽²⁾	x: 0.125 m η < 0.1	D.N.P. ⁽³⁾	x: 0.25 m η = 21.0	x: 0.125 m η < 0.1	M _{Ed} = 0.00 D.N.P. ⁽⁴⁾	D.N.P. ⁽⁵⁾	D.N.P. ⁽⁵⁾	VERIFIED η = 21.0
N34/N31	x: 0.2 m η < 0.1	N _{Ed} = 0.00 D.N.P. ⁽⁶⁾	x: 0.2 m η = 3.7	M _{Ed} = 0.00 D.N.P. ⁽¹⁾	η = 0.8	V _{Ed} = 0.00 D.N.P. ⁽²⁾	x: 0.1 m η < 0.1	D.N.P. ⁽³⁾	x: 0.2 m η = 3.7	x: 0.1 m η < 0.1	M _{Ed} = 0.00 D.N.P. ⁽⁴⁾	D.N.P. ⁽⁵⁾	D.N.P. ⁽⁵⁾	VERIFIED η = 3.7
N35/N31	N _{Ed} = 0.00 D.N.P. ⁽⁷⁾	N _{Ed} = 0.00 D.N.P. ⁽⁶⁾	M _{Ed} = 0.00 D.N.P. ⁽¹⁾	M _{Ed} = 0.00 D.N.P. ⁽¹⁾	V _{Ed} = 0.00 D.N.P. ⁽²⁾	x: 0.075 m η < 0.1	D.N.P. ⁽³⁾	D.N.P. ⁽³⁾	D.N.P. ⁽⁸⁾	D.N.P. ⁽⁹⁾	M _{Ed} = 0.00 D.N.P. ⁽⁴⁾	D.N.P. ⁽⁵⁾	D.N.P. ⁽⁵⁾	VERIFIED η < 0.1
N31/N32	η = 0.8	η = 0.7	M _{Ed} = 0.00 D.N.P. ⁽¹⁾	x: 0 m η = 51.9	V _{Ed} = 0.00 D.N.P. ⁽²⁾	x: 0.15 m η < 0.1	D.N.P. ⁽³⁾	η < 0.1	x: 0 m η = 78.7	η < 0.1	M _{Ed} = 0.00 D.N.P. ⁽⁴⁾	D.N.P. ⁽⁵⁾	D.N.P. ⁽⁵⁾	VERIFIED η = 78.7

Bars	CHECKS (EUROCODE 3 EN 1993-1-1: 2005)													Status
	λ _{web}	N _t	N _c	M _y	M _z	V _z	V _y	M _y V _z	M _z V _y	NM _y M _z	NM _y M _z V _y V _z	M _t	M _t V _z	
N30/N32	D.N.P. ⁽¹⁰⁾	x: 0.04 m η < 0.1	N _{Ed} = 0.00 D.N.P. ⁽⁶⁾	M _{Ed} = 0.00 D.N.P. ⁽¹⁾	x: 0.04 m η < 0.1	V _{Ed} = 0.00 D.N.P. ⁽²⁾	x: 0.04 m η = 0.1	D.N.P. ⁽³⁾	x: 0.04 m η < 0.1	x: 0.04 m η < 0.1	x: 0.04 m η < 0.1	M _{Ed} = 0.00 D.N.P. ⁽⁴⁾	D.N.P. ⁽⁵⁾	VERIFIED η = 0.1
N32/N17	D.N.P. ⁽¹⁰⁾	x: 0.56 m η < 0.1	N _{Ed} = 0.00 D.N.P. ⁽⁶⁾	M _{Ed} = 0.00 D.N.P. ⁽¹⁾	x: 0.56 m η = 28.8	V _{Ed} = 0.00 D.N.P. ⁽²⁾	x: 0.56 m η = 2.7	D.N.P. ⁽³⁾	η < 0.1	x: 0.56 m η = 28.9	η < 0.1	M _{Ed} = 0.00 D.N.P. ⁽⁴⁾	D.N.P. ⁽⁵⁾	VERIFIED η = 28.9
N17/N27	D.N.P. ⁽¹⁰⁾	N _{Ed} = 0.00 D.N.P. ⁽⁷⁾	x: 0 m η = 1.2	M _{Ed} = 0.00 D.N.P. ⁽¹⁾	x: 0 m η = 29.5	V _{Ed} = 0.00 D.N.P. ⁽²⁾	x: 0 m η = 2.6	D.N.P. ⁽³⁾	x: 0 m η < 0.1	x: 0 m η = 30.9	x: 0 m η < 0.1	M _{Ed} = 0.00 D.N.P. ⁽⁴⁾	D.N.P. ⁽⁵⁾	VERIFIED η = 30.9

Notes:

- N_t: Resistance to axial tension
- N_c: Resistance to axial compression
- M_y: Y - Axis bending resistance
- M_z: Z - Axis bending resistance
- V_z: Resistance to shear in the Z direction
- V_y: Resistance to shear in the Y direction
- M_yV_z: Combined bending moment Y and shear force Z resistance
- M_zV_y: Combined bending moment Z and shear force Y resistance
- NM_yM_z: Combined bending and axial resistance
- NM_yM_zV_yV_z: Combined bending, axial and shear resistance
- M_t: Torsional resistance
- M_tV_z: Combined Z shear and torsional resistance
- M_tV_y: Combined Y shear and torsional resistance
- x: Distance to the origin of the bar
- η: Demand capacity ratio (%)
- D.N.P.: Not applicable
- λ_{web}: Crushing of the web induced by the compressed flange



Reports

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Checks that do not proceed (D.N.P.):

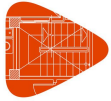
- ⁽¹⁾ The check does not proceed, as there is no bending moment.
- ⁽²⁾ The check does not proceed, as there is no shear force.
- ⁽³⁾ The check does not proceed, as there is no interaction between shear forces and bending moments for any loadcase combination.
- ⁽⁴⁾ The check does not proceed, as there is no torsional moment.
- ⁽⁵⁾ There is no interaction between torsional moment and shear force for any combination. Therefore the check does not proceed.
- ⁽⁶⁾ The check does not proceed, as there is no compressive axial force.
- ⁽⁷⁾ The check does not proceed, as there is no tensile axial force.
- ⁽⁸⁾ There is no interaction between the axial force and the bending moment or between bending moments in both directions for any combination. Therefore the check does not proceed.
- ⁽⁹⁾ There is no interaction between the bending moment, axial force and shear for any combination. Therefore, the check does not proceed.
- ⁽¹⁰⁾ The check does not proceed, as there is no bending moment compressing a flange, hence the phenomenon of web crushing induced by the compressed flange cannot be developed.

DEGREE OF FREEDOM

Limtre søyler i fasaden, gesims og baldakin - beregningsrapport
Frederikke – Rehabilitering fasade og tekniske anlegg
25086-R01

Revision	Date	Done By	Controlled By	Approved By
01	29/05/2026	JDO	FI	ER

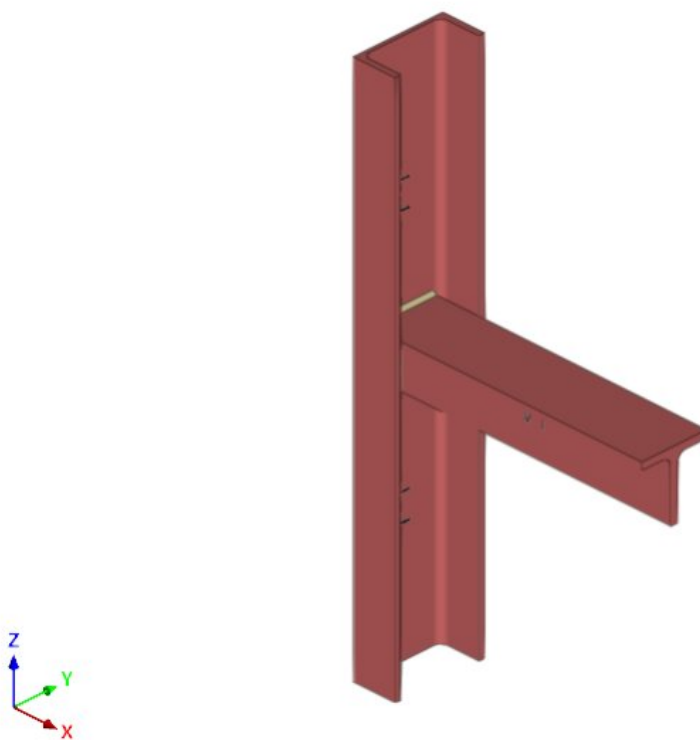
ANNEX II



1. JOINTS

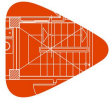
1.1. Connection 1

1.1.1. Connection components



3D View

Steel sections					
Element	Description	Geometry	Steel		
			Type	f_y (MPa)	f_u (MPa)
V1	T-100x11		S355 (EN 1993-1-1)	355.00	510.00
P1	UPE 140		S355 (EN 1993-1-1)	355.00	510.00



Reports

25086 Connection 1

Date: 06/12/26

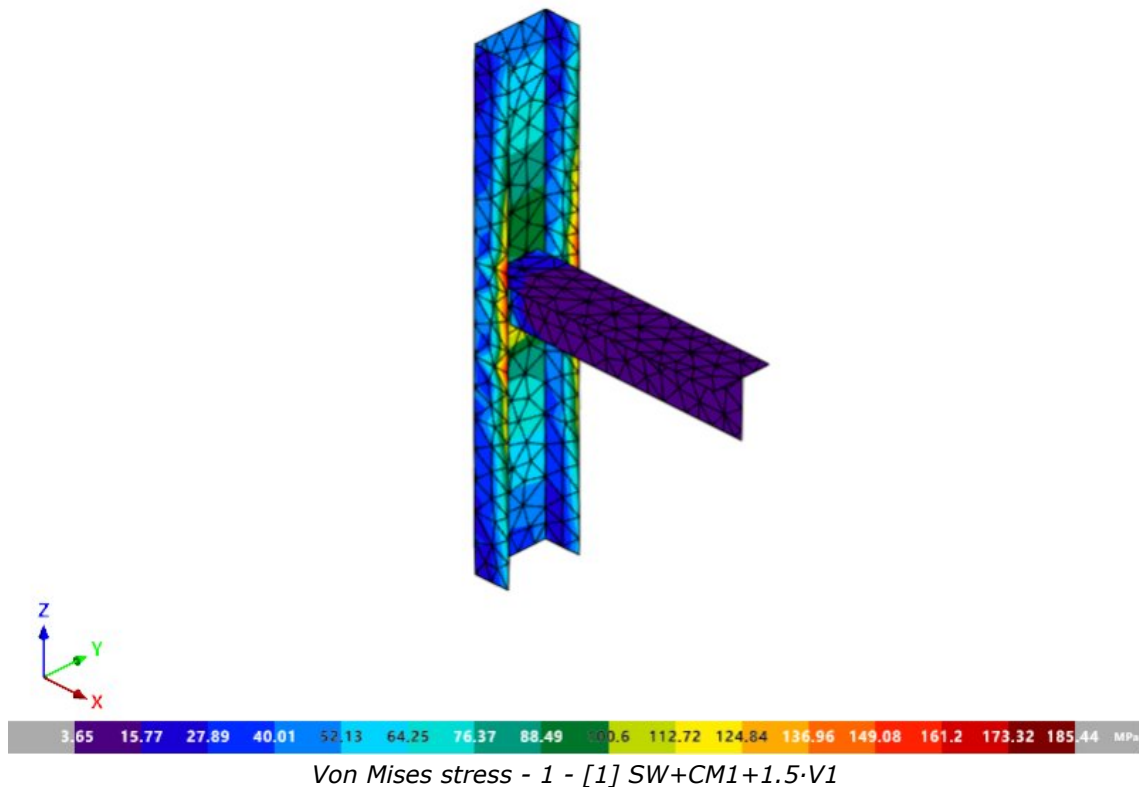
Welds							
Element	Type	Electrode	a (mm)	z (mm)	l (mm)	t (mm)	Angle (°)
Weld1 - Bot. fl. - Front	Fillet weld		4.0	5.7	100.0	5.0	90.0
Weld1 - Bot. fl. - Back 1 Weld1 - Bot. fl. - Back 2	Fillet weld		4.0	5.7	33.5	5.0	90.0
Weld2 - Web - Front Weld2 - Web - Back	Fillet weld		4.0	5.7	78.0	5.0	90.0

a: Throat thickness
z: Leg length
l: Weld length
t: Thickness of the thinnest member to join

1.1.2. Loads

V1 (T-100x11) - Forces at the end of the bar							
Load cases	N (kN)	Vy (kN)	Vz (kN)	Mx (kN·m)	My (kN·m)	Mz (kN·m)	
1 - [1] SW+CM1+1.5·V1	-13.38	0.00	-0.28	0.00	0.08	0.00	
2 - [1] 1.2·SW+CM1+1.5·V1	-13.36	0.00	-0.34	0.00	0.09	0.00	
3 - [1] 1.2·SW+CM1+1.5·N1	2.53	0.00	-7.69	0.00	1.93	0.00	
4 - [1] 1.2·SW+CM1+0.9·V1+1.5·N1	-5.56	0.00	-7.69	0.00	1.93	0.00	
5 - [1] SW+CM1+1.5·V1+1.05·N1	-11.69	0.00	-5.43	0.00	1.36	0.00	
6 - [1] 1.2·SW+CM1+1.5·V1+1.05·N1	-11.67	0.00	-5.48	0.00	1.38	0.00	

1.1.3. Stress / Strain



1.1.4. Summary of code checks

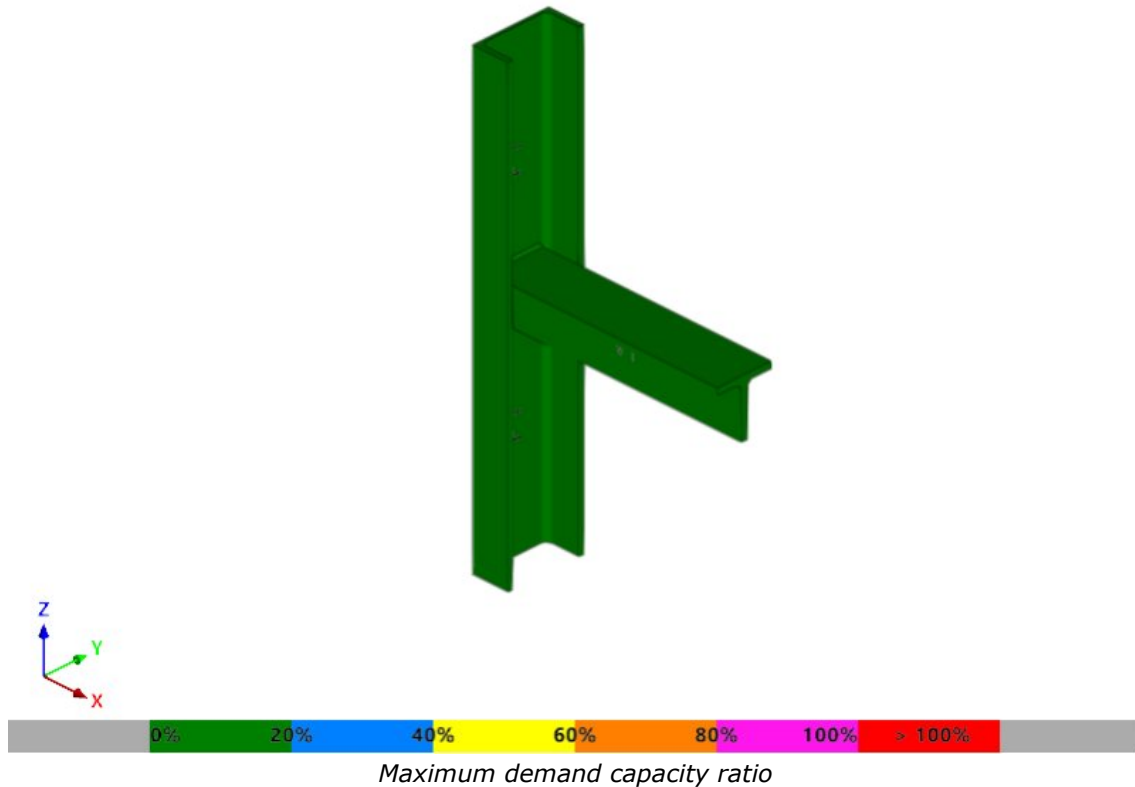


Reports

25086 Connection 1

Date: 06/12/26

1.1.4.1. Stress / Strain



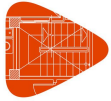
Elongation - Sections				
Element	Comb.	Plastic deformation (%)	Limit (%)	D/C ratio (%)
V1 - Bot. fl.	4 - [1] 1.2·SW+CM1+0.9·V1+1.5·N1	0.189	5.000	3.78
V1 - Web	3 - [1] 1.2·SW+CM1+1.5·N1	0.062	5.000	1.23
P1 - Top flange	6 - [1] 1.2·SW+CM1+1.5·V1+1.05·N1	0.098	5.000	1.96
P1 - Bot. fl.	6 - [1] 1.2·SW+CM1+1.5·V1+1.05·N1	0.116	5.000	2.33
P1 - Web	3 - [1] 1.2·SW+CM1+1.5·N1	0.177	5.000	3.55

Resistance - Welds									
Element	Von Mises stress					Normal stress		f_u (MPa)	β_w
	$\sigma_{\perp}$ (MPa)	$\tau_{\perp}$ (MPa)	$\tau_{//}$ (MPa)	Value (MPa)	D/C ratio (%)	$\sigma_{\perp}$ (MPa)	D/C ratio (%)		
Weld1 - Bot. fl. - Front	39.35	31.23	23.92	78.68	17.35	56.07	15.27	510.00	0.90
Weld1 - Bot. fl. - Back 1	13.09	16.60	36.40	70.51	15.55	13.09	3.56	510.00	0.90
Weld1 - Bot. fl. - Back 2	33.31	45.83	33.38	103.70	22.87	33.31	9.07	510.00	0.90
Weld2 - Web - Front	3.71	4.54	45.97	80.09	17.67	32.43	8.83	510.00	0.90
Weld2 - Web - Back	4.54	3.71	45.97	80.01	17.65	32.36	8.81	510.00	0.90

Notes

$\sigma_{\perp}$: Normal stress perpendicular to the throat of the weld.

$\tau_{\perp}$: Shear stress (in the plane of the throat) perpendicular to the axis of the weld.

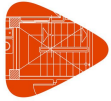


Reports

25086 Connection 1

Date: 06/12/26

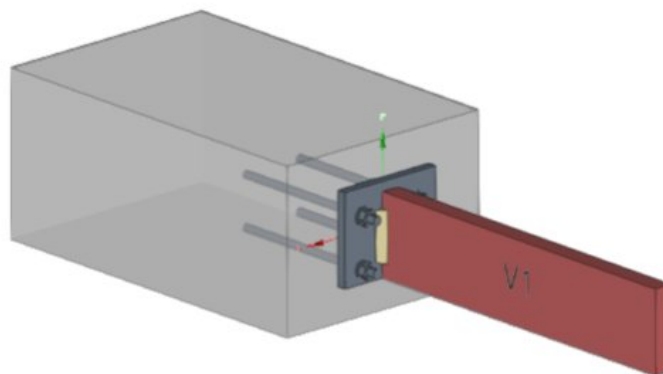
$\tau_{\parallel}$: Shear stress (in the plane of the throat) parallel to the axis of the weld.
 f_u : Ultimate nominal tensile resistance of the weakest part of the connection
 β_w : Correlation coefficient for fillet welds.



1. JOINTS

1.1. Connection 2

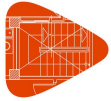
1.1.1. Connection components



3D View

Steel sections					
Element	Description	Geometry	Steel		
			Type	f_y (MPa)	f_u (MPa)
V1	FL 90 x 20		S355 (EN 1993-1-1)	355.00	510.00

Plates					
Element	Material	Thickness (mm)	Holes (mm)	Area (mm ²)	Weight (kg)
Plate1	S355	10.0	4 x 11.00	12000.00	0.94



Reports

25086 Parapet_connections 2 and 3

Date: 06/12/26

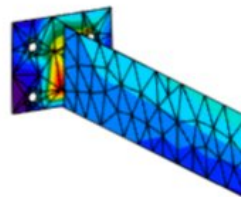
Welds							
Element	Type	Electrode	a (mm)	z (mm)	l (mm)	t (mm)	Angle (°)
Weld1 - Web - Front Weld1 - Web - Back	Fillet weld		6.0	8.5	55.0	10.0	90.0
a: Throat thickness z: Leg length l: Weld length t: Thickness of the thinnest member to join							

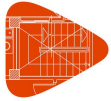
Post-installed anchor		
Operation	Type	Quantity
Anchorage1 - 1	Hilti - HIT HY-200 R V3+HAS-U 8.8 M10x190	4

1.1.2. Loads

V1 (FL 90 x 20) - Forces at the end of the bar							
Load cases	N (kN)	Vy (kN)	Vz (kN)	Mx (kN·m)	My (kN·m)	Mz (kN·m)	
1 - [1] SW+CM1+1.5·V1	-13.38	0.00	-0.31	0.00	0.00	0.00	
2 - [1] 1.2·SW+CM1+1.5·N1	2.53	0.00	-7.73	0.00	0.00	0.00	
3 - [1] 1.2·SW+CM1+0.9·V1+1.5·N1	-5.56	0.00	-7.73	0.00	0.00	0.00	

1.1.3. Stress / Strain





Reports

25086 Parapet_connections 2 and 3

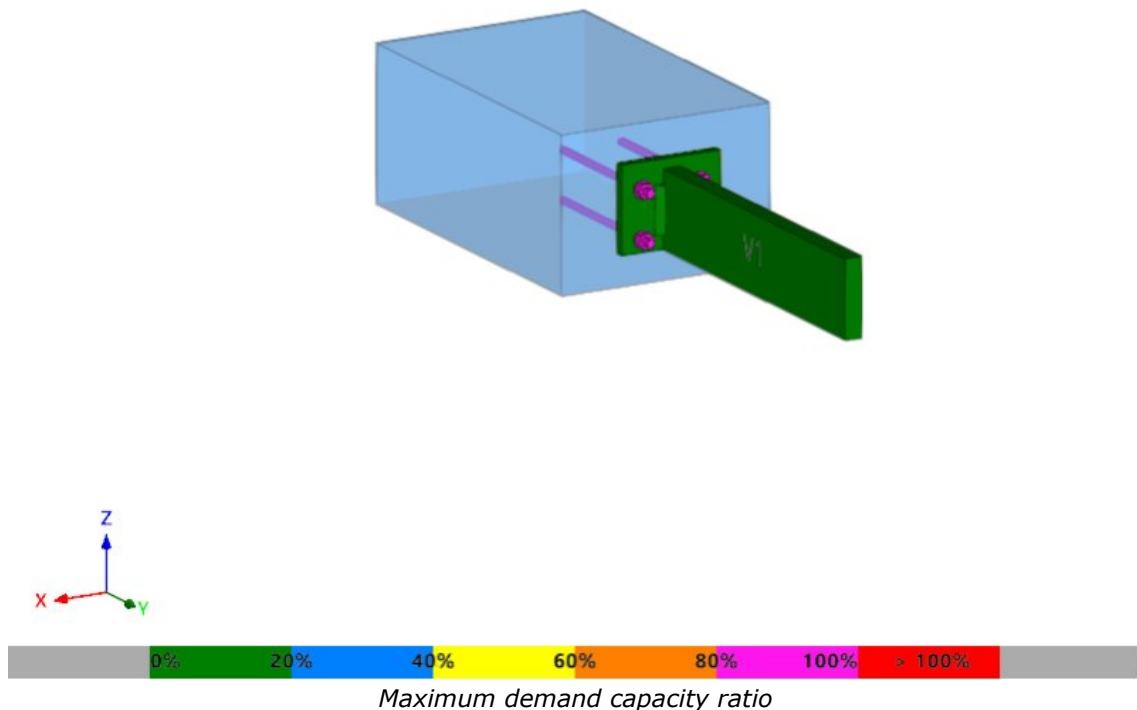
Date: 06/12/26

1.1.4. Forces in the anchors

Forces in the anchors				
Anchorage	Loadcase	N (kN)	Vx (kN)	Vy (kN)
Anchorage1 - 1	1 - [1] SW+CM1+1.5·V1	0.00	-0.02	-0.09
Anchorage1 - 1	2 - [1] 1.2·SW+CM1+1.5·N1	0.67	0.00	-1.93
Anchorage1 - 1	3 - [1] 1.2·SW+CM1+0.9·V1+1.5·N1	0.00	-0.02	-1.93
Anchorage1 - 2	1 - [1] SW+CM1+1.5·V1	0.00	0.02	-0.08
Anchorage1 - 2	2 - [1] 1.2·SW+CM1+1.5·N1	0.64	0.00	-1.93
Anchorage1 - 2	3 - [1] 1.2·SW+CM1+0.9·V1+1.5·N1	0.00	0.02	-1.93
Anchorage1 - 3	1 - [1] SW+CM1+1.5·V1	0.00	-0.02	-0.07
Anchorage1 - 3	2 - [1] 1.2·SW+CM1+1.5·N1	0.82	0.02	-1.93
Anchorage1 - 3	3 - [1] 1.2·SW+CM1+0.9·V1+1.5·N1	0.00	0.00	-1.93
Anchorage1 - 4	1 - [1] SW+CM1+1.5·V1	0.00	0.02	-0.07
Anchorage1 - 4	2 - [1] 1.2·SW+CM1+1.5·N1	0.90	-0.02	-1.93
Anchorage1 - 4	3 - [1] 1.2·SW+CM1+0.9·V1+1.5·N1	0.00	0.01	-1.93

1.1.5. Summary of code checks

1.1.5.1. Stress / Strain



Elongation - Sections				
Element	Comb.	Plastic deformation (%)	Limit (%)	D/C ratio (%)
V1 - Web	2 - [1] 1.2·SW+CM1+1.5·N1	0.057	5.000	1.14



Reports

25086 Parapet_connections 2 and 3

Date: 06/12/26

Compressive stress in the concrete.				
Element	Comb.	$\sigma_{c,Ed}$ (MPa)	$\sigma_{c,Rd}$ (MPa)	D/C ratio (%)
Anchorage1 - Concrete	1 - [1] SW+CM1+1.5·V1	4.51	11.33	39.75

Elongation - Plates				
Element	Comb.	Plastic deformation (%)	Limit (%)	D/C ratio (%)
Plate1	2 - [1] 1.2·SW+CM1+1.5·N1	0.009	5.000	0.18

Resistance - Welds									
Element	Von Mises stress					Normal stress		f_u (MPa)	β_w
	$\sigma_{\perp}$ (MPa)	$\tau_{\perp}$ (MPa)	$\tau_{//}$ (MPa)	Value (MPa)	D/C ratio (%)	$\sigma_{\perp}$ (MPa)	D/C ratio (%)		
Weld1 - Web - Front	13.82	13.79	1.04	27.66	6.35	13.82	3.92	490.00	0.90
Weld1 - Web - Back	13.79	13.82	1.04	27.69	6.36	13.79	3.91	490.00	0.90

Notes

$\sigma_{\perp}$: Normal stress perpendicular to the throat of the weld.
 $\tau_{\perp}$: Shear stress (in the plane of the throat) perpendicular to the axis of the weld.
 $\tau_{//}$: Shear stress (in the plane of the throat) parallel to the axis of the weld.
 f_u : Ultimate nominal tensile resistance of the weakest part of the connection
 β_w : Correlation coefficient for fillet welds.

Resistance - Group of anchors								
Element	Tension				Shear			Interaction
	$N_{Rd,c}$ (kN)	$N_{Rd,p}$ (kN)	$N_{Rd,sp}$ (kN)	D/C ratio (%)	$V_{Rd,cp}$ (kN)	$V_{Rd,c}$ (kN)	D/C ratio (%)	D/C ratio (%)
Anchorage1	13.95	38.54	30.47	21.70	29.22	9.03	85.61	89.32

Resistance - Anchors					
Element	Tension		Shear		Interaction
	$N_{Rd,s}$ (kN)	D/C ratio (%)	$V_{Rd,s}$ (kN)	D/C ratio (%)	D/C ratio (%)
Anchorage1 - 1	30.93	2.18	18.56	10.43	1.13
Anchorage1 - 2	30.93	2.07	18.56	10.42	1.13
Anchorage1 - 3	30.93	2.64	18.56	10.41	1.15
Anchorage1 - 4	30.93	2.91	18.56	10.41	1.17

Notes

$N_{Rd,s}$: design value of steel resistance of a fastener or a channel bolt under tension load
 $N_{Rd,p}$: design resistance in case of pull-out failure under tension load
 $N_{Rd,c}$: design resistance in case of concrete cone failure under tension load
 $N_{Rd,sp}$: design resistance in case of concrete splitting failure under tension load
 $V_{Rd,s}$: design value of steel resistance of a fastener or a channel bolt under shear load
 $V_{Rd,c}$: design resistance in case of concrete edge failure under shear load
 $V_{Rd,cp}$: design resistance in case of concrete pry-out failure under shear load

1.2. Connection 3

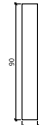


Reports

25086 Parapet_connections 2 and 3

Date: 06/12/26

1.2.1. Connection components

Steel sections					
Element	Description	Geometry	Steel		
			Type	f_y (MPa)	f_u (MPa)
P1	UPE 140		S355 (EN 1993-1-1)	355.00	510.00
V1	FL 90 x 12		S355 (EN 1993-1-1)	355.00	510.00

Welds							
Element	Type	Electrode	a (mm)	z (mm)	l (mm)	t (mm)	Angle (°)
Weld1 - Web - Front Weld1 - Web - Back	Fillet weld		4.0	5.7	90.0	5.0	90.0
<i>a: Throat thickness z: Leg length l: Weld length t: Thickness of the thinnest member to join</i>							

1.2.2. Loads

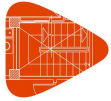
V1 (FL 90 x 12) - Forces at the end of the bar						
Load cases	N (kN)	Vy (kN)	Vz (kN)	Mx (kN·m)	My (kN·m)	Mz (kN·m)
1 - [1] SW+CM1+1.5·V1	2.99	0.05	0.00	0.00	0.00	-0.59
2 - [1] 1.2·SW+CM1+1.5·N1	-2.53	0.06	0.00	0.00	0.00	0.51

1.2.3. Summary of code checks

1.2.3.1. Stress / Strain

Elongation - Sections				
Element	Comb.	Plastic deformation (%)	Limit (%)	D/C ratio (%)
P1 - Top flange	1 - [1] SW+CM1+1.5·V1	0.058	5.000	1.16
P1 - Bot. fl.	1 - [1] SW+CM1+1.5·V1	0.058	5.000	1.17
P1 - Web	2 - [1] 1.2·SW+CM1+1.5·N1	0.234	5.000	4.68
V1 - Web	2 - [1] 1.2·SW+CM1+1.5·N1	0.298	5.000	5.97

Resistance - Welds									
Element	Von Mises stress					Normal stress		f_u (MPa)	β_w
	$\sigma_{\perp}$ (MPa)	$\tau_{\perp}$ (MPa)	$\tau_{//}$ (MPa)	Value (MPa)	D/C ratio (%)	$\sigma_{\perp}$ (MPa)	D/C ratio (%)		
Weld1 - Web - Front	3.62	2.69	5.46	11.14	2.46	6.69	1.82	510.00	0.90
Weld1 - Web - Back	2.01	6.69	2.30	12.42	2.74	3.93	1.07	510.00	0.90



Reports

25086 Parapet_connections 2 and 3

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Notes

$\sigma_{\perp}$: Normal stress perpendicular to the throat of the weld.
 $\tau_{\perp}$: Shear stress (in the plane of the throat) perpendicular to the axis of the weld.
 $\tau_{\parallel}$: Shear stress (in the plane of the throat) parallel to the axis of the weld.
 f_u : Ultimate nominal tensile resistance of the weakest part of the connection
 β_w : Correlation coefficient for fillet welds.



1. JOINTS

1.1. Timber mullion to existing concrete beam

1.1.1. Connection components

Steel sections					
Element	Description	Geometry	Steel		
			Type	f_y (MPa)	f_u (MPa)
V1	FL 75 x 8		S355 (EN 1993-1-1)	355.00	510.00

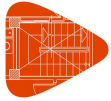
Plates					
Element	Material	Thickness (mm)	Holes (mm)	Area (mm ²)	Weight (kg)
Plate1	S355	15.0	4 x 13.00	20000.00	2.36

Welds							
Element	Type	Electrode	a (mm)	z (mm)	l (mm)	t (mm)	Angle (°)
Weld1 - Web - Front Weld1 - Web - Back	Fillet weld		6.0	8.5	75.0	8.0	90.0
<i>a: Throat thickness</i> <i>z: Leg length</i> <i>l: Weld length</i> <i>t: Thickness of the thinnest member to join</i>							

Post-installed anchor		
Operation	Type	Quantity
Anchorage1 - 1	Hilti - HIT RE-500 V4+HAS-U A4-70 M12x120	4

1.1.2. Loads

V1 (FL 75 x 8) - Forces at the end of the bar						
Load cases	N (kN)	V _y (kN)	V _z (kN)	M _x (kN·m)	M _y (kN·m)	M _z (kN·m)
1 - [1] SW+CM1+1.5·V1	2.30	0.00	-4.51	0.00	-0.65	0.00
2 - [1] 1.2·SW+CM1+1.5·N1	0.00	0.00	0.00	0.00	0.00	0.00
3 - [1] 1.2·SW+CM1+0.9·V1+1.5·N1	0.00	0.00	0.00	0.00	0.00	0.00



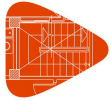
1.1.3. Stress / Strain



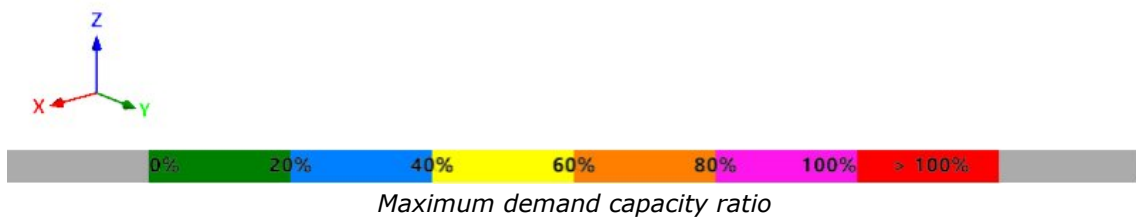
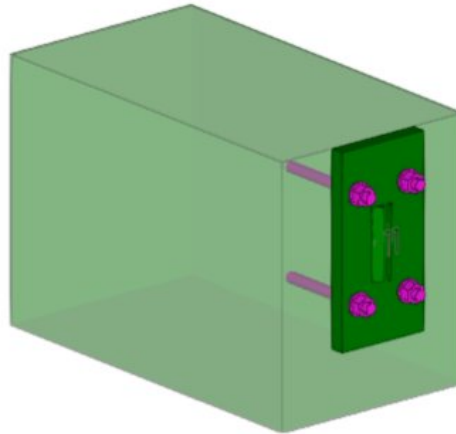
1.1.4. Forces in the anchors

Forces in the anchors				
Anchorage	Loadcase	N (kN)	Vx (kN)	Vy (kN)
Anchorage1 - 1	1 - [1] SW+CM1+1.5·V1	3.61	0.02	-1.13
Anchorage1 - 1	2 - [1] 1.2·SW+CM1+1.5·N1	0.00	0.00	0.00
Anchorage1 - 1	3 - [1] 1.2·SW+CM1+0.9·V1+1.5·N1	0.00	0.00	0.00
Anchorage1 - 2	1 - [1] SW+CM1+1.5·V1	3.20	-0.03	-1.13
Anchorage1 - 2	2 - [1] 1.2·SW+CM1+1.5·N1	0.00	0.00	0.00
Anchorage1 - 2	3 - [1] 1.2·SW+CM1+0.9·V1+1.5·N1	0.00	0.00	0.00
Anchorage1 - 3	1 - [1] SW+CM1+1.5·V1	0.00	-0.01	-1.13
Anchorage1 - 3	2 - [1] 1.2·SW+CM1+1.5·N1	0.00	0.00	0.00
Anchorage1 - 3	3 - [1] 1.2·SW+CM1+0.9·V1+1.5·N1	0.00	0.00	0.00
Anchorage1 - 4	1 - [1] SW+CM1+1.5·V1	0.00	0.01	-1.12
Anchorage1 - 4	2 - [1] 1.2·SW+CM1+1.5·N1	0.00	0.00	0.00
Anchorage1 - 4	3 - [1] 1.2·SW+CM1+0.9·V1+1.5·N1	0.00	0.00	0.00

1.1.5. Summary of code checks



1.1.5.1. Stress / Strain



Elongation - Sections				
Element	Comb.	Plastic deformation (%)	Limit (%)	D/C ratio (%)
V1 - Web	1 - [1] SW+CM1+1.5·V1	0.066	5.000	1.31

Compressive stress in the concrete.				
Element	Comb.	$\sigma_{c,Ed}$ (MPa)	$\sigma_{c,Rd}$ (MPa)	D/C ratio (%)
Anchorage1 - Concrete	1 - [1] SW+CM1+1.5·V1	1.54	11.33	13.59

Elongation - Plates				
Element	Comb.	Plastic deformation (%)	Limit (%)	D/C ratio (%)
Plate1	1 - [1] SW+CM1+1.5·V1	0.011	5.000	0.21

Resistance - Welds									
Element	Von Mises stress					Normal stress		f_u (MPa)	β_w
	$\sigma_{\perp}$ (MPa)	$\tau_{\perp}$ (MPa)	$\tau_{//}$ (MPa)	Value (MPa)	D/C ratio (%)	$\sigma_{\perp}$ (MPa)	D/C ratio (%)		
Weld1 - Web - Front	14.00	13.87	6.33	29.89	6.59	14.00	3.81	510.00	0.90
Weld1 - Web - Back	13.87	14.00	6.33	30.01	6.62	13.87	3.78	510.00	0.90



Reports

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Notes

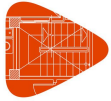
$\sigma_{\perp}$: Normal stress perpendicular to the throat of the weld.
 $\tau_{\perp}$: Shear stress (in the plane of the throat) perpendicular to the axis of the weld.
 $\tau_{\parallel}$: Shear stress (in the plane of the throat) parallel to the axis of the weld.
 f_u : Ultimate nominal tensile resistance of the weakest part of the connection
 β_w : Correlation coefficient for fillet welds.

Resistance - Group of anchors								
Element	Tension				Shear			Interaction
	$N_{Rd,c}$ (kN)	$N_{Rd,p}$ (kN)	$N_{Rd,sp}$ (kN)	D/C ratio (%)	$V_{Rd,cp}$ (kN)	$V_{Rd,c}$ (kN)	D/C ratio (%)	D/C ratio (%)
Anchorage1	16.64	26.22	43.51	40.91	35.70	6.17	73.04	88.59

Resistance - Anchors					
Element	Tension		Shear		Interaction
	$N_{Rd,s}$ (kN)	D/C ratio (%)	$V_{Rd,s}$ (kN)	D/C ratio (%)	D/C ratio (%)
Anchorage1 - 1	31.56	11.43	18.91	5.97	1.66
Anchorage1 - 2	31.56	10.14	18.91	5.98	1.39
Anchorage1 - 3	--	0.00	18.91	5.95	0.00
Anchorage1 - 4	--	0.00	18.91	5.95	0.00

Notes

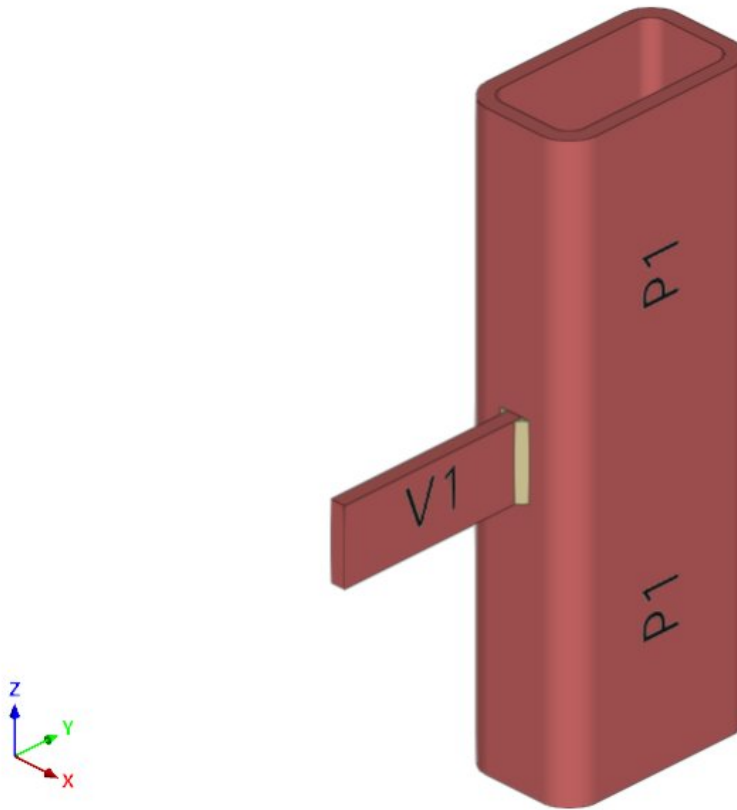
$N_{Rd,s}$: design value of steel resistance of a fastener or a channel bolt under tension load
 $N_{Rd,p}$: design resistance in case of pull-out failure under tension load
 $N_{Rd,c}$: design resistance in case of concrete cone failure under tension load
 $N_{Rd,sp}$: design resistance in case of concrete splitting failure under tension load
 $V_{Rd,s}$: design value of steel resistance of a fastener or a channel bolt under shear load
 $V_{Rd,c}$: design resistance in case of concrete edge failure under shear load
 $V_{Rd,cp}$: design resistance in case of concrete pry-out failure under shear load



1. JOINTS

1.1. Baldakin. C1

1.1.1. Connection components



3D View

Steel sections					
Element	Description	Geometry	Steel		
			Type	f_y (MPa)	f_u (MPa)
P1	RHS 140x80x10.0		S355 (EN 1993-1-1)	355.00	510.00
V1	FL 50 x 10		S355 (EN 1993-1-1)	355.00	510.00



Reports

25086_Frederikke_Canopy

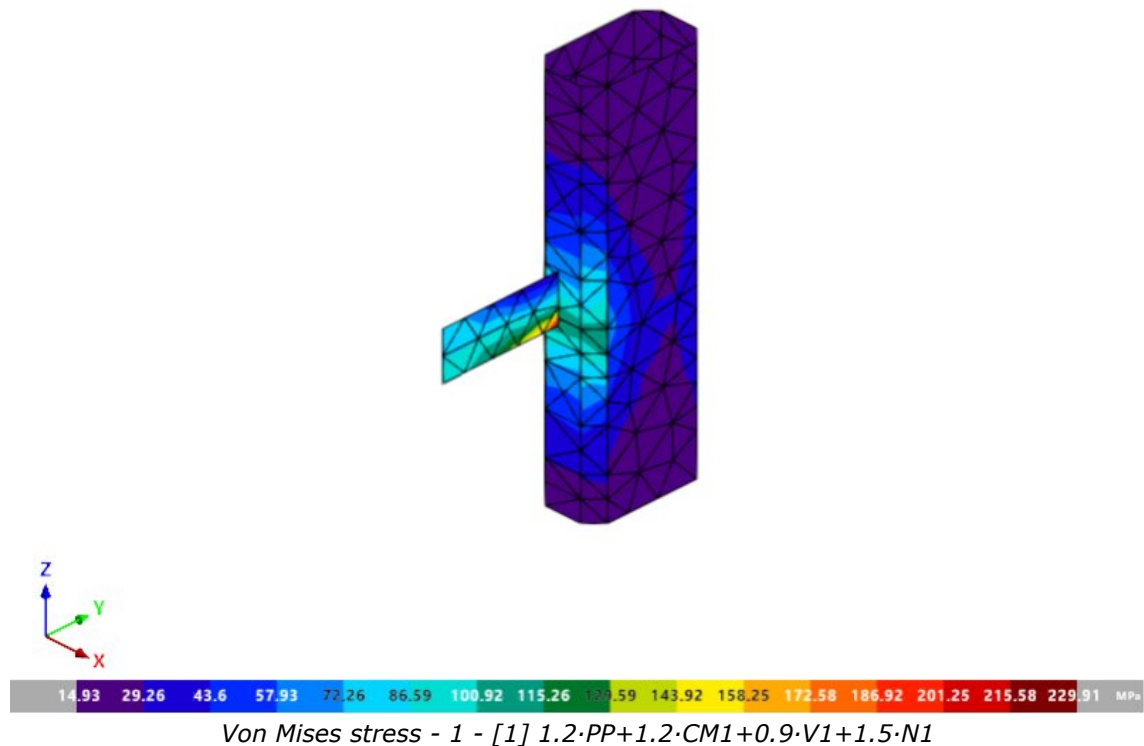
Date: 06/12/26

Welds							
Element	Type	Electrode	a (mm)	z (mm)	l (mm)	t (mm)	Angle (°)
Ajuste1 - Web - Front Ajuste1 - Web - Back	Fillet weld		4.0	5.7	50.0	10.0	90.0
<i>a: Throat thickness</i> <i>z: Leg length</i> <i>l: Weld length</i> <i>t: Thickness of the thinnest member to join</i>							

1.1.2. Loads

V1 (FL 50 x 10) - Forces at the end of the bar							
Load cases	N (kN)	Vy (kN)	Vz (kN)	Mx (kN·m)	My (kN·m)	Mz (kN·m)	
1 - [1] 1.2·PP+1.2·CM1+0.9·V1+1.5·N1	-41.70	0.00	-4.42	0.00	0.88	0.00	
2 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	-41.95	0.00	-3.85	0.00	0.77	0.00	

1.1.3. Stress / Strain



1.1.4. Summary of code checks

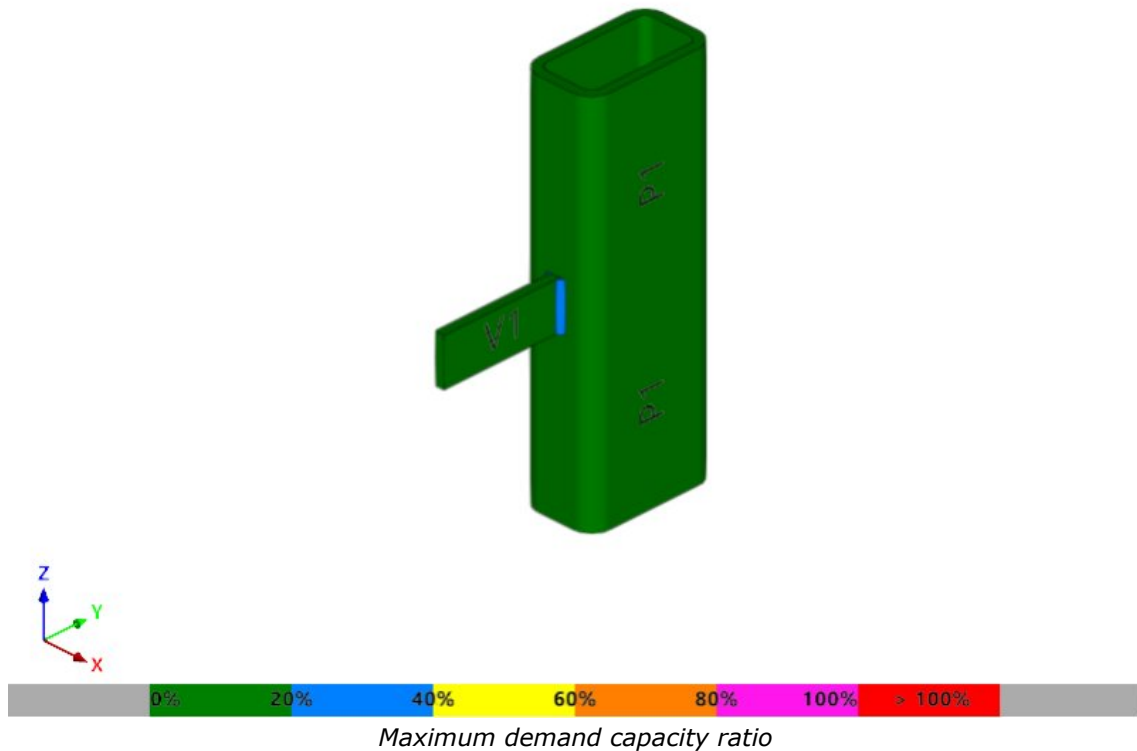


Reports

25086_Frederikke_Canopy

Date: 06/12/26

1.1.4.1. Stress / Strain



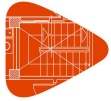
Elongation - Sections					
Element	Comb.	Plastic deformation (%)	Limit (%)	D/C ratio (%)	
P1 - Ala Sup.	1 - [1] 1.2·PP+1.2·CM1+0.9·V1+1.5·N1	0.037	2.000	1.85	
P1 - Ala inf.	1 - [1] 1.2·PP+1.2·CM1+0.9·V1+1.5·N1	0.046	2.000	2.28	
P1 - Alma izq.	1 - [1] 1.2·PP+1.2·CM1+0.9·V1+1.5·N1	0.027	2.000	1.34	
P1 - Alma der.	1 - [1] 1.2·PP+1.2·CM1+0.9·V1+1.5·N1	0.032	2.000	1.58	
P1 - Tramo curvo (Sup. Izq.) - 1	2 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.028	2.000	1.40	
P1 - Tramo curvo (Sup. Der.) - 1	2 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.030	2.000	1.50	
P1 - Tramo curvo (Inf. Izq.) - 1	1 - [1] 1.2·PP+1.2·CM1+0.9·V1+1.5·N1	0.044	2.000	2.22	
P1 - Tramo curvo (Inf. Der.) - 1	1 - [1] 1.2·PP+1.2·CM1+0.9·V1+1.5·N1	0.038	2.000	1.91	
V1 - Alma	1 - [1] 1.2·PP+1.2·CM1+0.9·V1+1.5·N1	0.073	2.000	3.64	

Resistance - Welds									
Element	Von Mises stress					Normal stress		f_u (MPa)	β_w
	$\sigma_{\perp}$ (MPa)	$\tau_{\perp}$ (MPa)	$\tau_{//}$ (MPa)	Value (MPa)	D/C ratio (%)	$\sigma_{\perp}$ (MPa)	D/C ratio (%)		
Ajuste1 - Alma - Frontal	74.16	74.16	9.63	149.25	32.92	74.16	20.19	510.00	0.90
Ajuste1 - Alma - Trasero	74.16	74.16	9.63	149.25	32.92	74.16	20.19	510.00	0.90

Notes

$\sigma_{\perp}$: Normal stress perpendicular to the throat of the weld.

$\tau_{\perp}$: Shear stress (in the plane of the throat) perpendicular to the axis of the weld.



Reports

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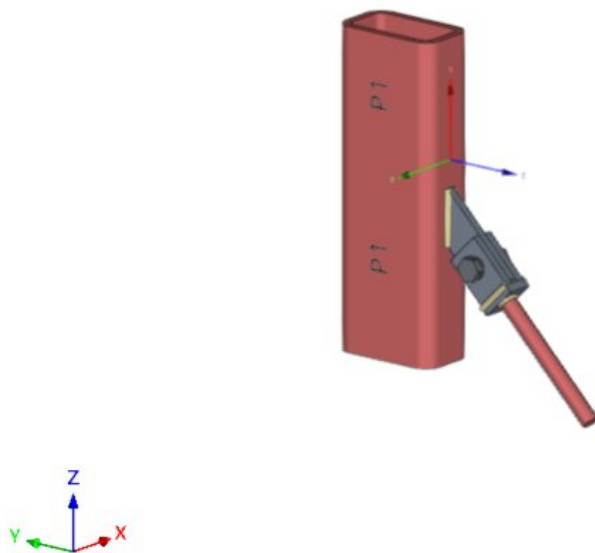
$\tau_{//}$: Shear stress (in the plane of the throat) parallel to the axis of the weld.
 f_u : Ultimate nominal tensile resistance of the weakest part of the connection
 β_w : Correlation coefficient for fillet welds.

1.1.5. Buckling

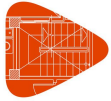
Buckling		
Loadcase	Buckling mode	Critical load factor
1 - [1] 1.2·PP+1.2·CM1+0.9·V1+1.5·N1	1	48.654
	2	94.395
	3	196.612
	4	283.888
	5	328.151
2 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	1	48.938
	2	96.189
	3	200.362
	4	279.696
	5	321.304

1.2. Baldakin. C2

1.2.1. Connection components



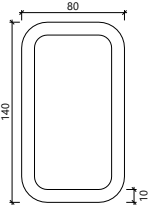
3D View



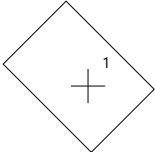
Reports

25086_Frederikke_Canopy

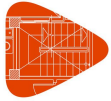
Date: 06/12/26

Steel sections					
Element	Description	Geometry	Steel		
			Type	f_y (MPa)	f_u (MPa)
P1	RHS 140x80x10.0		S355 (EN 1993-1-1)	355.00	510.00
V1	20		S355	355.00	490.00

Plates					
Element	Material	Thickness (mm)	Holes (mm)	Area (mm ²)	Weight (kg)
Placa1	S355	10.0	1 x 18.00	5406.92	0.42
Placa2	S355	10.0	1 x 18.00	3500.03	0.27
Placa3	S355	10.0	1 x 18.00	3500.03	0.27
Placa4	S355	10.0	--	1750.00	0.14

Bolt elements						
Element	Type	Series	Material	Dimensions	Length (mm)	Quantity
	Bolt	ISO 4017	8.8	16	55.0	1
	Nut	ISO 4032	8	16	--	1
	Washer	ISO 7089	200 HV	16	--	2

Welds							
Element	Type	Electrode	a (mm)	z (mm)	l (mm)	t (mm)	Angle (°)
Soldadura1 - Front Soldadura1 - Back	Fillet weld		4.0	5.7	70.7	10.0	90.0
Soldadura2 Soldadura3	Single bevel butt weld		--	--	49.9	10.0	45.0
Ajuste1	Single bevel butt weld		--	--	62.4	7.8	45.0
<i>a: Throat thickness z: Leg length l: Weld length t: Thickness of the thinnest member to join</i>							



Reports

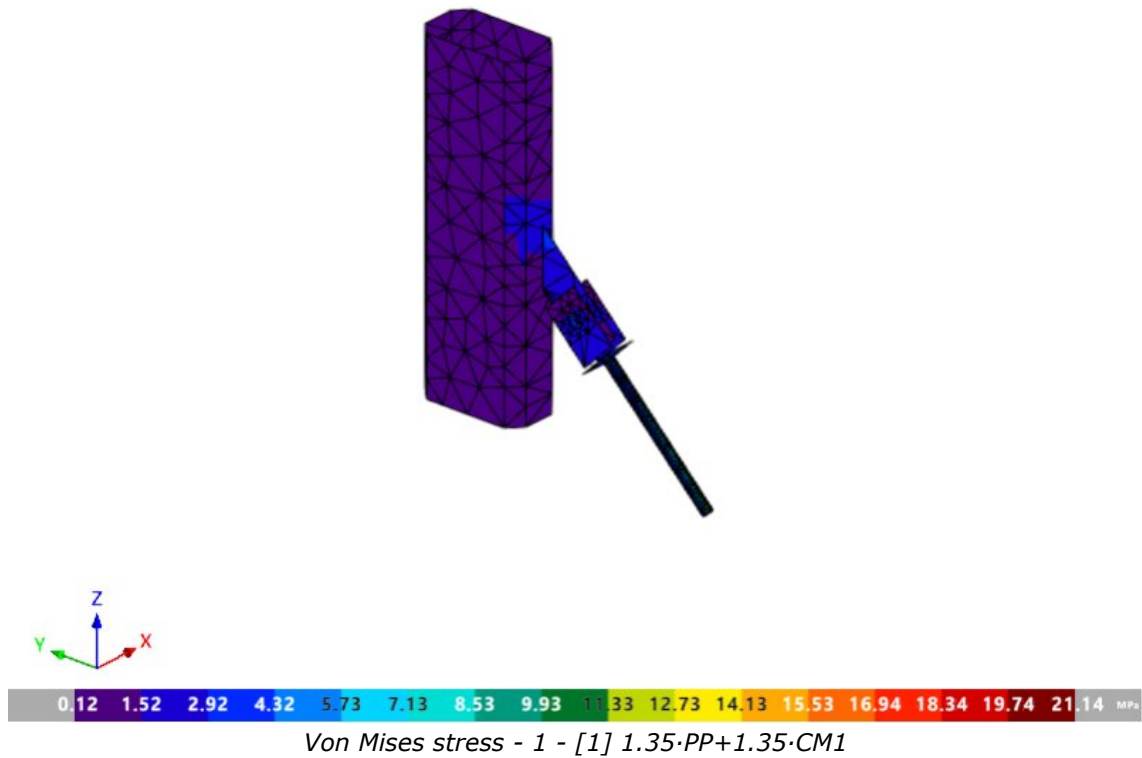
25086_Frederikke_Canopy

Date: 06/12/26

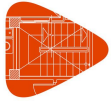
1.2.2. Loads

V1 (20) - Forces at the end of the bar						
Load cases	N (kN)	Vy (kN)	Vz (kN)	Mx (kN·m)	My (kN·m)	Mz (kN·m)
1 - [1] 1.35·PP+1.35·CM1	1.09	0.00	-0.02	0.00	0.00	0.00
2 - [1] 1.35·PP+1.35·CM1+0.9·V1	3.52	0.00	-0.02	0.00	0.00	0.00
3 - [1] 1.35·PP+1.35·CM1+0.9·Vex	5.24	0.00	-0.02	0.00	0.00	0.00
4 - [1] 1.35·PP+1.35·CM1+1.05·N1	38.70	0.00	-0.02	0.00	0.00	0.00
5 - [1] 1.35·PP+1.35·CM1+0.9·V1+1.05·N1	41.53	0.00	-0.02	0.00	0.00	0.00
6 - [1] 1.35·PP+1.35·CM1+0.9·Vex+1.05·N1	42.87	0.00	-0.02	0.00	0.00	0.00
7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	58.87	0.00	-0.02	0.00	0.00	0.00

1.2.3. Stress / Strain



1.2.4. Summary of code checks

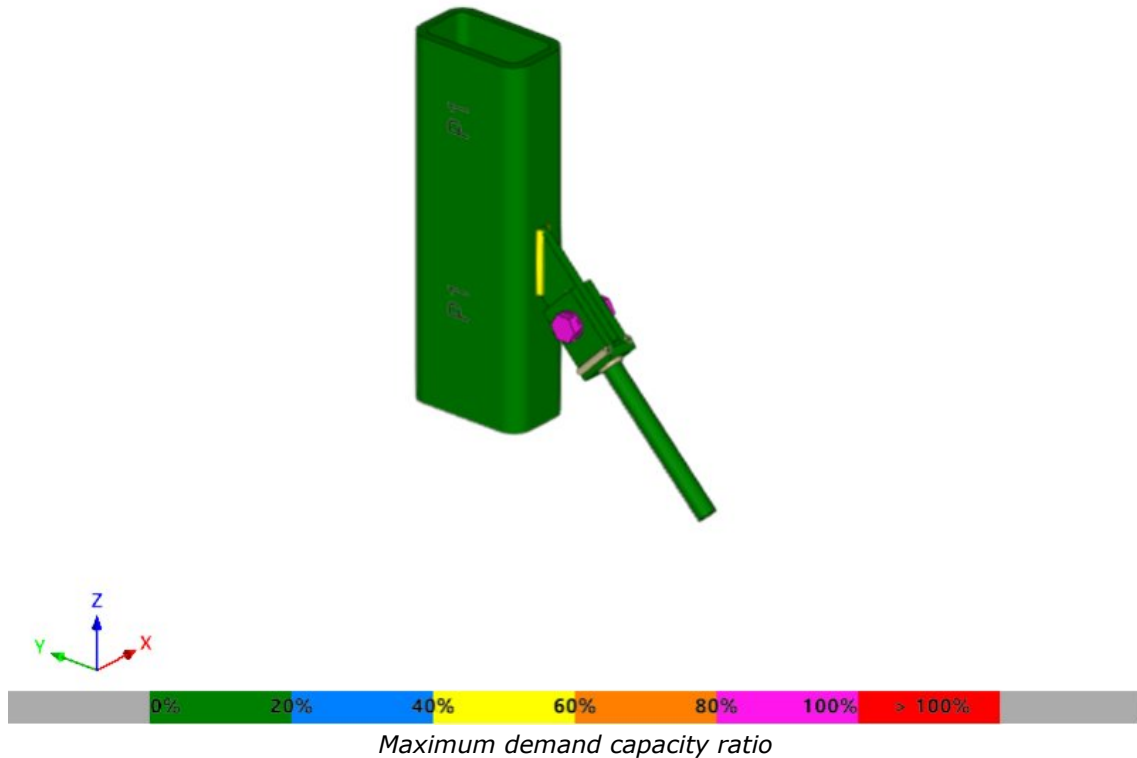


Reports

25086_Frederikke_Canopy

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1.2.4.1. Stress / Strain



Elongation - Sections					
Element	Comb.	Plastic deformation (%)	Limit (%)	D/C ratio (%)	
P1 - Ala Sup.	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.035	2.000	1.77	
P1 - Ala inf.	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.045	2.000	2.27	
P1 - Alma izq.	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.032	2.000	1.62	
P1 - Alma der.	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.031	2.000	1.53	
P1 - Tramo curvo (Sup. Izq.) - 1	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.028	2.000	1.39	
P1 - Tramo curvo (Sup. Der.) - 1	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.030	2.000	1.50	
P1 - Tramo curvo (Inf. Izq.) - 1	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.038	2.000	1.89	
P1 - Tramo curvo (Inf. Der.) - 1	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.044	2.000	2.21	
V1 - Lado - 1	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.200	2.000	10.01	
V1 - Lado - 2	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.167	2.000	8.36	
V1 - Lado - 3	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.097	2.000	4.83	
V1 - Lado - 4	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.094	2.000	4.71	
V1 - Lado - 5	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.094	2.000	4.71	
V1 - Lado - 6	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.096	2.000	4.82	
V1 - Lado - 7	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.167	2.000	8.35	
V1 - Lado - 8	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.200	2.000	9.98	
V1 - Lado - 9	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.198	2.000	9.92	
V1 - Lado - 10	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.167	2.000	8.37	
V1 - Lado - 11	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.097	2.000	4.85	
V1 - Lado - 12	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.084	2.000	4.18	
V1 - Lado - 13	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.084	2.000	4.18	
V1 - Lado - 14	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.097	2.000	4.86	



Reports

25086_Frederikke_Canopy

Date: 06/12/26

Elongation - Sections				
Element	Comb.	Plastic deformation (%)	Limit (%)	D/C ratio (%)
V1 - Lado - 15	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.168	2.000	8.38
V1 - Lado - 16	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.199	2.000	9.94

Elongation - Plates				
Element	Comb.	Plastic deformation (%)	Limit (%)	D/C ratio (%)
Placa1	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.072	2.000	3.61
Placa2	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.058	2.000	2.92
Placa3	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.058	2.000	2.92
Placa4	7 - [1] 1.2·PP+1.2·CM1+0.9·Vex+1.5·N1	0.078	2.000	3.88

Layout - Bolts					
Element	d ₀ (mm)	e ₁ (mm)	e ₂ (mm)	p ₁ (mm)	p ₂ (mm)
Tornillos1 - 1	18.0	25.0	25.0	--	--

Resistance - Bolts									
Element	Tension				Shear				Interaction
	Ft,Ed (kN)	Ft,Rd (kN)	Bp,Rd (kN)	D/C ratio (%)	Fv,Ed (kN)	Fv,Rd (kN)	Fb,Rd (kN)	D/C ratio (%)	D/C ratio (%)
Tornillos1 - 1	5.18	90.43	212.55	5.73	29.44	60.29	63.56	48.83	52.92

Resistance - Welds									
Element	Von Mises stress					Normal stress		f _u (MPa)	β _w
	σ _⊥ (MPa)	τ _⊥ (MPa)	τ _∥ (MPa)	Value (MPa)	D/C ratio (%)	σ _⊥ (MPa)	D/C ratio (%)		
Soldadura1 - Frontal	49.70	49.76	82.50	174.12	39.98	49.70	14.09	490.00	0.90
Soldadura1 - Trasero	49.76	49.70	82.50	174.08	39.97	49.76	14.10	490.00	0.90

Notes

d₀: Hole diameter

e₁: Distance from the center of the fastener hole to the adjacent end of any element, measured in the direction of load transfer

e₂: Distance from the center of the fastener hole to the adjacent end of any element, measured at right angles to the direction of load transfer

p₁: Distance between the centres of the fasteners in line in the direction of the transmission of the load

p₂: Distance between adjacent rows of fasteners, measured perpendicular to the direction of the transmission of the load

Ft,Ed: Design value of the acting tensile force per bolt for the ultimate limit state

Ft,Rd: Tensile strength of the bolt

Bp,Rd: Punching shear resistance

Fv,Ed: Design value of the acting shear force per bolt for the ultimate limit state

Fv,Rd: Shear resistance of the bolt

Fb,Rd: Bearing resistance

σ_⊥: Normal stress perpendicular to the throat of the weld.

τ_⊥: Shear stress (in the plane of the throat) perpendicular to the axis of the weld.

τ_∥: Shear stress (in the plane of the throat) parallel to the axis of the weld.

f_u: Ultimate nominal tensile resistance of the weakest part of the connection

β_w: Correlation coefficient for fillet welds.