FAKULTA
STAVEBNÍ
ČVUT V PRAZE

DIPLOMA PROJECT

Design of multistorey steel car park
Návrh budovy patrových garáží

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Branch of Study : Building Structure
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Prague, 2022

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ZADÁNÍ DIPLOMOVÉ PRÁCE

I. OSOBNÍ A STUDIJNÍ ÚDAJE

Příjmení: Koca Jméno: Ramazan Osobní číslo: 503274

Zadávající katedra: K134

Studijní program: N

Studijní obor: 3607T030 - Building Structures

II. ÚDAJE K DIPLOMOVÉ PRÁCI

Název diplomové práce: Návrh budovy patrových garáží

Název diplomové práce anglicky: Design of multistorey car park

Pokyny pro vypracování:
Design multistorey car park building with steel structural framework and concrete floors. Prepare plan for parking and circulation of cars. Design floor and bracing system. Design typical details and connections and main member sizes. Prepare documentation of structural analysis. Prepare drawings of the typical floor plan, anchoring plan, elevations and typical cross-sections including ramps.

Seznam doporučené literatury:
EN 1993-1-1, Eurocode 3: Design of steel structures - Part1-1: General rules for buildings
EN 1993-1-8, Eurocode 3: Design of steel structures - Part 1-8: Design of joints
Economical Carparks, A Design Guide, OneSteel
https://www.steelconstruction.info/Car_parks

Jméno vedoucího diplomové práce: Ing. Jiří Mareš, Phd

Datum zadání diplomové práce: 30.9.21 Termín odevzdání DP v IS KOS:
Údaj uveďte v souladu s datem v časovém plánu příslušného ak. roku

Podpis vedoucího práce Podpis vedoucího katedry

III. PŘEVZETÍ ZADÁNÍ

Beru na vědomí, že jsem povinen vypracovat diplomovou práci samostatně, bez cizí pomoci, s výjimkou poskytnutých konzultací. Seznam použité literatury, jiných pramenů a jmen konzultantů je nutné uvést v diplomové práci a při citování postupovat v souladu s metodickou příručkou ČVUT „Jak psát vysokoškolské závěrečné práce" a metodickým pokynem ČVUT „O dodržování etických principů při přípravě vysokoškolských závěrečných prací".

04.10.2021
Datum převzetí zadání Podpis studenta(ky)

Declaration of Authorship

I declare that I have prepared the submitted work independently and that I have listed all the information sources used in accordance with the Methodical Guidelines on the Ethical Preparation of University Thesis.

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Ramazan Koca

Prague, 02.01.2022

MULTI-STOREY STEEL CAR PARK DESIGN

Abstract

This thesis deals with design of multi-storey steel carpark which is planned for Prague. The structure has four storey , 12.6 meters height, 34 meters width and 111 meters long.

The steel framework is analysed in SCIA software. All hand calculations have been programmed in Python language by author and displayed in readable form in Jupyter notebook. A snapshot of the Python code is shown in Appendix.

Keywords

Car Park design, SCIA, Steel structure design, multi-storey carpark, steel framed structure, double tee floor system.

Acknowledgement

I would like to thank my supervisor , Jiří Mareš , for his valuable advice, comments and professional guidance.

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1.Introduction

Multi-storey car parks are essential buildings for cities with high population and for places that people gathered commonly such that train stations, airports, hospitals and shopping centres.

Car Park structures are often built above ground level as permanent structures however underground and basement car parking is used where it is necessary.

Steel construction is one of the best solutions to satisfy all the requirements of fine car park design. Since steel is:

* Ideal for long spans – providing column-free parking space
* Lightweight – reducing foundation requirements
* Robust and fire resistant (fire solutions do exist but it is not a subject of this study)
* Fast in construction – particularly relevant where the venue that is to be served is to remain operational during construction, e.g., retail, stations and hospitals
* Easily maintained
* Vandal resistant
* Economic.

A satisfactory steel car park design should fulfil some major objectives. These are:

* Utility or function (strength and serviceability)
* Safety (permanence)
* Economy (Low maintenance)
* Elegance
* Easy entry and exit to the car park and the parking stalls
* Uncomplicated and logical traffic flow around the car park
* Unimpeded movement
* Light and airy

The aim of this study is to present what are the steps for designing multistorey steel carpark and what is the design process for a given specific geometry of the structure. During designing process, the calculation is done using an advanced tool called Jupyter notebook. This tool is based on object-oriented programming language called Python. Procedures and algorithms were prepared and coded by the author of this thesis. This means that the final product of this thesis is not only the technical documentation itself but also a set of software tools that can be utilized in the future. Advanced analyse is obtained by SCIA Engineering Software, drawings are prepared by using AutoCAD and for 3D illustration TEKLA Structure.

2. Design Plan

The structural design plan shows us the phases from initial conception to the final plans and specifications. In this study the phases up to acceptance of structural project is planned, so let us consider the following list of phases and in detail of first three phases in this study:

A- Initial planning

B- Preliminary design

C- Final design

a. Analysis

b. Selection and proportioning of elements

c. Drawings, specifications and other contract document

D- Acceptance of project

E- Construction

F- Operation and maintenance

2.1. Initial planning

Initial planning starts with the requirements of the structure. At the stage of initial design, the geometry of the building, entrance, the capacity of parking space, parking bays and traffic flow are decided as shown below. In our study, building geometry is 34 meters wide and 111 meters long, traffic flow goes two ways with 90 ° angle parking. The parking floors are sloped for traffic to circulate from one level to another, which is shown in drawings at part 4, and satisfy the limit for interior ramp located at the middle of the structure. The full capacity of parking building is 501 vehicles.

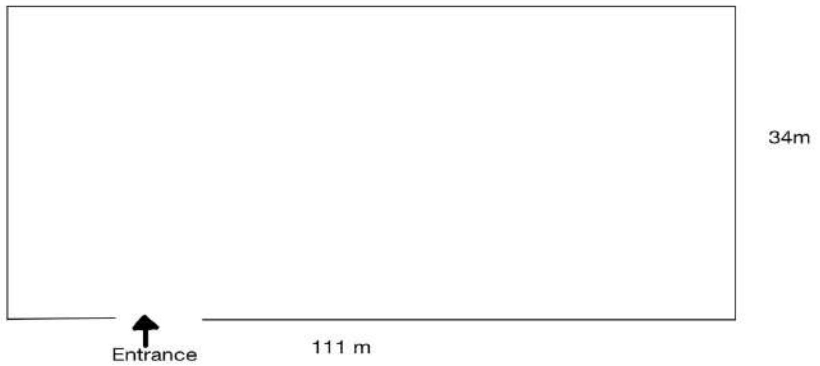


Figure 2. 1: Building geometry and entrance

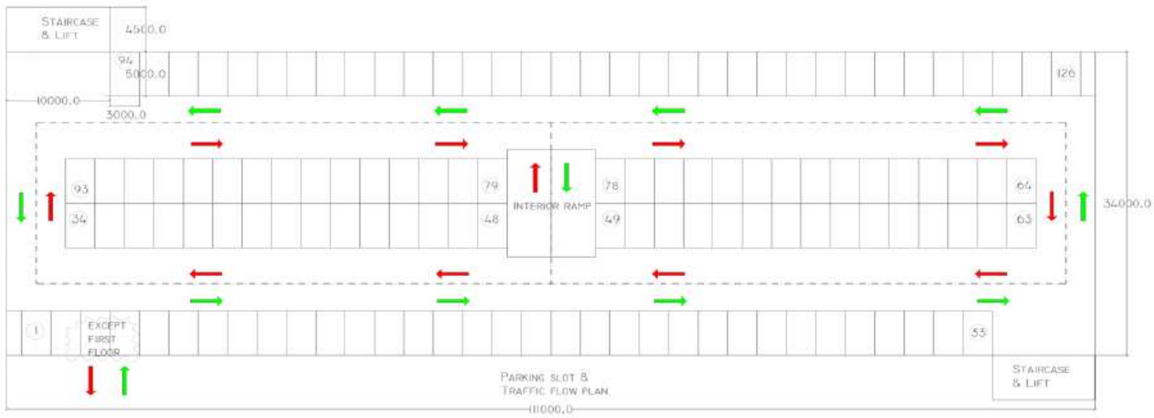


Figure 2. 2: Parking slots and traffic flow

The dimensions of large, standard and small size cars are well established and are given by the Institution of Structural Engineers [1]. According to the table the chosen aisles, parking angle and stall required at least 16.55 m bin width. The minimum dimensions based on a standard car, the bin width, the parking angle and stall width are shown in the following Table 2.1.1.

Effect of varying parking angle on parking bin requirements

Parking angle	Stall width (m)	Stall width (m) parallel to aisle	Aisle width (m)	Bin width (m) for stall length 4.80m
45°	2.3	3.25	3.6	13.65
	2.4	3.39		13.80
	2.5	3.54		13.95
60°	2.3	2.66	4.2	14.85
	2.4	2.77		14.95
	2.5	2.89		15.05
75°	2.3	2.38	4.98	15.45
	2.4	2.49		15.50
	2.5	2.59		15.55
90°	All widths	All widths	One way aisle 6.0	15.60
90°	All widths	All widths	Two way aisle 6.95	16.55

Table 2.1. 1: Effect of varying parking angle on parking bin requirement

2.2. Preliminary design stage

During the study, linear elastic analysis is assumed. All results are obtained and valid at elastic range.

The Standards for the material give values for the minimum yield strength (f_y) and ultimate strength(f_u); it should be noted that the yield strength is normally taken as the design strength. Steel grade was considered as S355 for primary members and S275 for secondary elements.

The following Table 2.2.1 shows value for steel strength at specified thickness range.

Structural Steel Strength Properties for elements with nominal thickness $t \leq 40$ mm

		EN10025-2 Hot rolled products - Non-alloy structural steels				EN10025-3 Hot rolled products - Normalized/normalized rolled weldable fine grain structural steels				EN10025-4 Hot rolled products - Thermomechanical rolled weldable fine grain structural steels				EN10025-5 Hot rolled products - Structural steels with improved atmospheric corrosion resistance		EN10025-6 Hot rolled products - High yield strength structural steels in the quenched and tempered condition
Symbol	Description	S235	S275	S355	S450	S275 N/NL	S355 N/NL	S420 N/NL	S460 N/NL	S275 M/ML	S355 M/ML	S420 M/ML	S460 M/ML	S235 W	S355 W	S460 Q/QL/QL1
f_y (MPa)	Yield strength	235	275	355	440	275	355	420	460	275	355	420	460	235	355	460
f_u (MPa)	Ultimate strength	360	430	490	550	390	490	520	540	370	470	520	540	360	490	570

Structural Steel Strength Properties for elements with nominal thickness 40 mm $< t \leq 80$ mm

		EN10025-2 Hot rolled products - Non-alloy structural steels				EN10025-3 Hot rolled products - Normalized/normalized rolled weldable fine grain structural steels				EN10025-4 Hot rolled products - Thermomechanical rolled weldable fine grain structural steels				EN10025-5 Hot rolled products - Structural steels with improved atmospheric corrosion resistance		EN10025-6 Hot rolled products - High yield strength structural steels in the quenched and tempered condition
Symbol	Description	S235	S275	S355	S450	S275 N/NL	S355 N/NL	S420 N/NL	S460 N/NL	S275 M/ML	S355 M/ML	S420 M/ML	S460 M/ML	S235 W	S355 W	S460 Q/QL/QL1
f_y (MPa)	Yield strength	215	255	335	410	255	335	390	430	255	335	390	430	215	335	440
f_u (MPa)	Ultimate strength	360	410	470	550	370	470	520	540	360	450	500	530	340	490	550

Table 2.2. 1: Yield strength (fy) and ultimate strength(fu)

2.2.1. Loads

The imposed loading for the parking areas and ramps in car parks is given in EN 1991-1-1 [2]. For a normal mix of vehicles, taking the maximum weight of any vehicle as 2500kg, the imposed uniformly distributed load given in EN 1991 [2] is 2.5kN/m^2.Other than that the building is located in Prague, the snow load given in ČSN EN 1991-1-3 [3] is 0.7kN/m^2 and the wind load is 0.32kN/m^2.

The weight of the structure itself and the weight of all loads permanently on it constitutes its dead load. Besides primary elements, the slab is the other dead load source. The slab system in the structure is chosen precast concrete double tee slab in order to reduce the construction time. The properties are shown in Table 2.2.1.1.

Double Tee flooring

Stahlton Double Tee is a large component, prestressed precast concrete unit which, with the addition of an in situ topping, forms a structural suspended floor suitable for use in most types of building. Double Tees are ideally suited to car-parks and supermarkets where large column-free spaces are required. The large unit size and large span capabilities of the Double Tee system result in very fast construction times.

- The large units require no temporary propping and once in place provide an immediate working platform.
- The large span capability can also considerably simplify the floor support system, and when used with the Stahlton Shell Beam gives further economies and time savings.

The Stahlton Double Tee, up to 550mm deep, has parallel sided 200 mm wide webs for superior fire rating and durability performance if required.

Double Tee

Unfactored maximum superimposed live load (Q) in kilopascals (kPa), (assuming no superimposed dead load ie SDL = 0kPa)

75mm of 25MPa topping concrete

Double Tee depth (mm)	Self wt (kPa)	Simply supported span (m) 5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
200	3.5	14.0	8.0	4.7	3.7											
250	3.7		12.5	7.5	6.4	4.8	3.4									
300	3.9			12.9	9.1	6.9	5.3	4.0								
350	4.1				11.7	9.5	7.1	5.3	4.1	3.2						
400	4.4					12.0	9.1	6.9	5.8	4.6	3.7					
450	4.6					13.0	10.5	9.2	7.4	5.8	4.5	4.1	2.7			
500	4.8						12.8	9.9	8.1	7.1	6.1	5.0	4.2	3.3	2.6	
550	5.0							14.0	11.7	9.0	7.5	5.9	4.8	4.0	3.3	2.6
600	4.8								12.5	10.1	8.2	6.7	5.6	4.3	3.5	2.7

Notes regarding load/span tables:

1. Consideration needs to be given to long term creep effects due to higher superimposed dead loads. Contact Stahlton's technical people for guidance.
2. The Load Span tables assume loads are uniformly distributed. Consideration is required for shear actions induced from point loads. Again, contact Stahlton's technical people for advice.
3. Theoretical cambers have been limited to span/400. Consider higher cambers for situations close to the tabulated load limits.
4. Refer to Stahlton's website www.stahlton.co.nz for more information.
5. The actual prestress used and the capacity of the floor varies according to the design load specified.

Double Tee section properties

Section properties are based on standard widths of 2.4m plus a 75mm concrete topping. Composite section modular ratio = 0.67

Double tee depth (mm)	Unit wt (kg/m)	Overall depth (mm)	Bare Unit: A x10^{3}mm^2	Y_b mm	I x10^{9}mm^4	Z_b x10^{6}mm^3	Composite unit: A x10^{3}mm^2	Y_b' mm	I' x10^{9}mm^4	Z_b' x10^{6}mm^3	Z_t' x10^{6}mm^3
200	495	275	194	138	0.59	4.31	374	179	1.42	7.95	34.82
250	546	325	214	173	1.09	6.32	394	218	2.21	10.16	20.70
300	594	375	233	205	1.86	9.08	413	252	3.32	13.19	27.02
350	655	425	252	235	2.92	12.41	437	289	4.95	17.12	36.37
400	717	475	281	266	4.24	15.96	461	121	6.77	21.08	43.95
450	770	525	302	298	5.71	19.16	482	357	8.89	24.89	52.89
500	821	575	322	334	7.94	24.51	502	387	11.97	30.98	63.67
550	857	625	336	352	10.35	29.41	516	419	15.11	36.54	74.32
*600	819	675	321	398	11.30	28.39	501	467	16.11	34.93	78.42

*tapered leg in Christchurch

Important information on Stahlton Double Tees

End seating

Stahlton Double tee flooring requires a recommended minimum, and the greater of, 75mm or L/180 seating onto unarmoured concrete beam or wall. A construction tolerance needs to be compensated for as per cl18.7.4 NZS3101:Part 1:2006. Stahlton and the code requires the use of low-friction bearing strips.

Temporary propping

Stahlton Double tee flooring does not usually require propping.

Camber

Stahlton Double tees will arrive on site with a camber. This is unavoidable due to pre-stressing. Cambers will vary and be influenced by the amount of prestress required to resist the induced loads, length and age of the units exposed to the elements.

Handling & storage

Stahlton Double tees are designed to be lifted using hooks and chains to Zeus hanger, 'Swift-lift' lifting anchors or strand lifting eyes located at ends.

Double tees need to be dunnaged as close as possible to the lifting location and with blocks in line with the block below, on solid and even ground. Ensure lifting equipment is regularly checked.

Penetrations

Stahlton Double tee flooring does allow flexibility for accommodating penetrations in specific locations avoiding the strands. If a strand is cut on-site, place a prop either side and contact Stahlton's Technical Department for a design review. Also refer to our standard drawings for acceptable details for attaching fixings to the soffit of the double tees.

Table 2.2.1 1:Double tee slab properties

The loading on foundations is greatly influenced by the material chosen for the superstructure. Steel is the lightest practical construction material for car parks and will often allow the use of simple foundations however heavier materials will not. On the other hand, foundation calculation depends on soil conditions, but this study is related to steel framework only. Foundation analysis is not part of this project but are shown in drawings just for illustration.

2.2.2. Preliminary beam design

The case study, which is shown below both as plan view and 3D view, has 4 stories and 111 m long and 34m wide. The first storey is 3.5m, second and third stories are 3.0 m, and fourth story is 3.1 m height.

The designs are based on the following assumptions:

- Design is given for internal and edge beams where appropriate for one bay of a car park
- Imposed loading is taken as 2.5kN/m^2
- Grade S355 is used for all main and secondary beams
- Grade S275 is used for all ties
- Approximate weights of steel given are based on a car park 111m long x 34m wide with car parking spaces at 2.4m wide.
- Column sizes are based on the lower length of a 4-storey car park
- Imposed load deflection limit: Span/360
- Total load deflection limit: Span/150
- Preliminary design is applied for linear analysis only

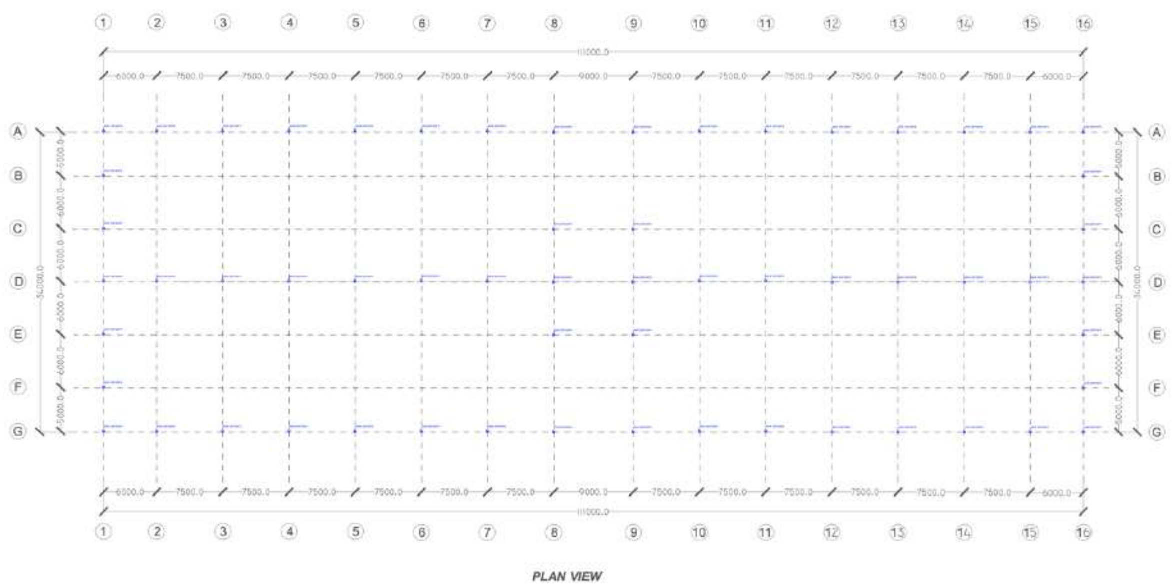


Figure 2.2.2. 1:Plan view

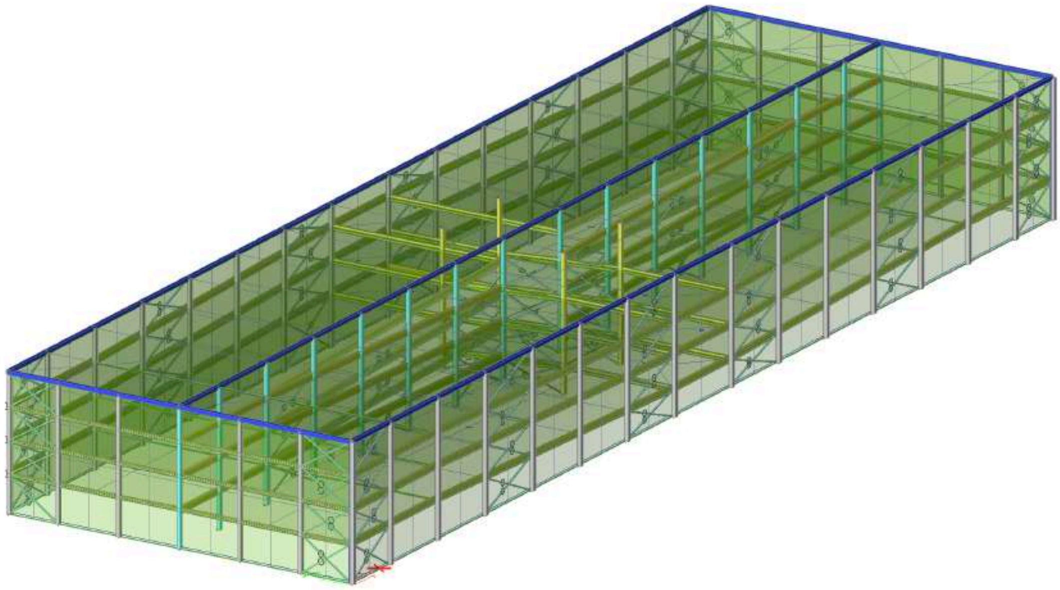


Figure 2.2.2. 2: 3D SCIA Model

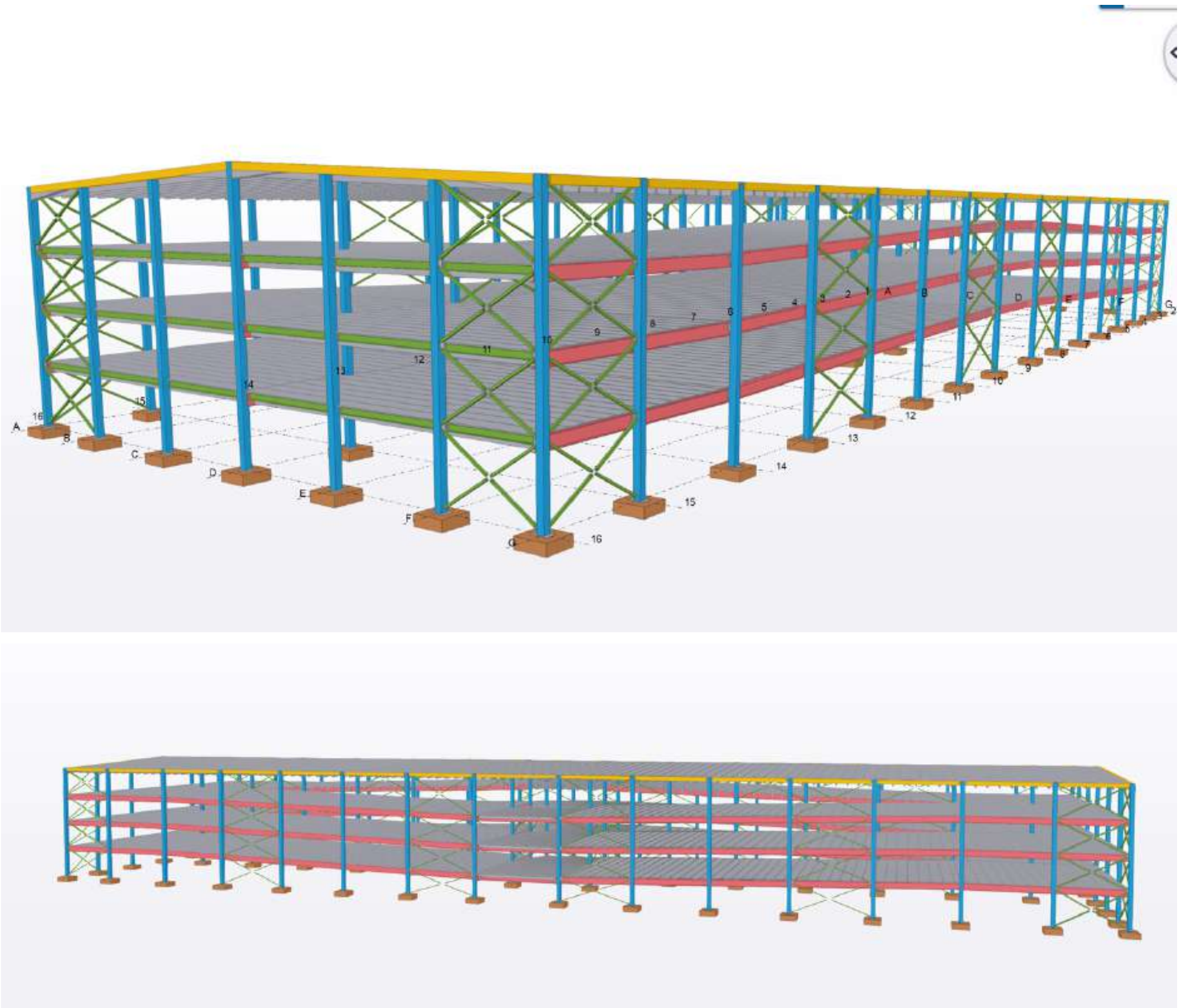


Figure 2.2.2. 3:Tekla model

The continuous beam as shown below has its top flange fully restrained laterally by a precast slab supported on secondary beams. Double tee flooring systems require a minimum of 75mm or span/180, whichever is greater, seating on beams. Concrete supports, double tee slab legs, are on steel beams the end seating can be reduced by 15mm. A construction tolerance of 10mm needs to be added to these figures. Double Tees can be supported at the end on the 55mm thick flange using hanger brackets. This avoids the contractor having to form the beams between the legs and allows the depth of the tee to be almost entirely within the required depth of the supporting beam. Other options are partial leg support and full leg support. The way of double tee flooring system is attached to beam flange is shown below.

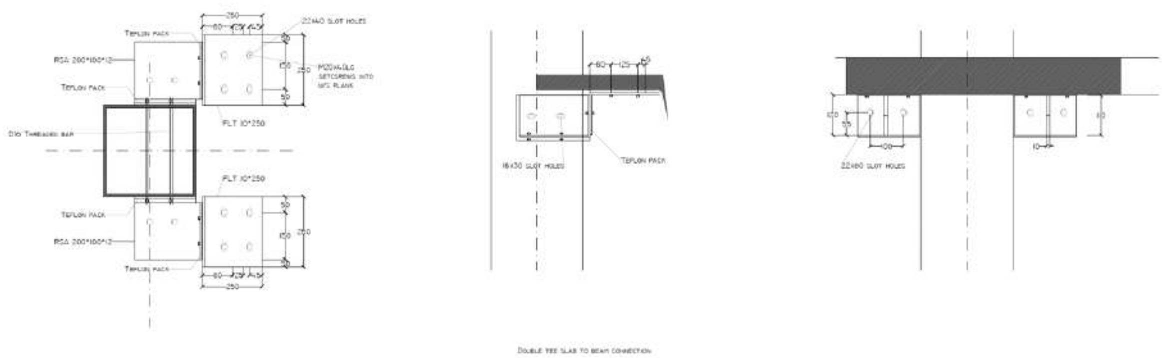


Figure 2.2.2. 4:Double tee floor to beam flange connection

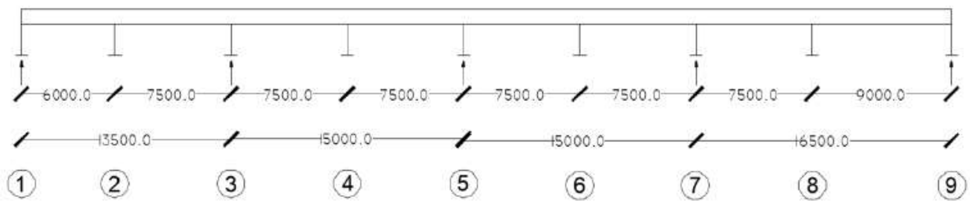


Figure 2.2.2. 5:Simplified continuous beam model

All actions are considered as concentrated loads acting at the nine numbered locations. Only the forces at 2,4,6 and 8 give rise to bending moments and shear forces.

2.2.2.1. Actions-Loading

Location: Czech Republic-Prague

Snow load= 0.7 kN/m^2

Wind Load = 0.32 kN/m^2

For category F, qk may be selected within the range 1.5 to 2.5 kN/m2 and Qk may be selected within the range 10 to 20 kN [2]

Slab selection (350 Double Tee with 75mm topping) (Table 2.2.1.1)
L=8.500 m (for half span)

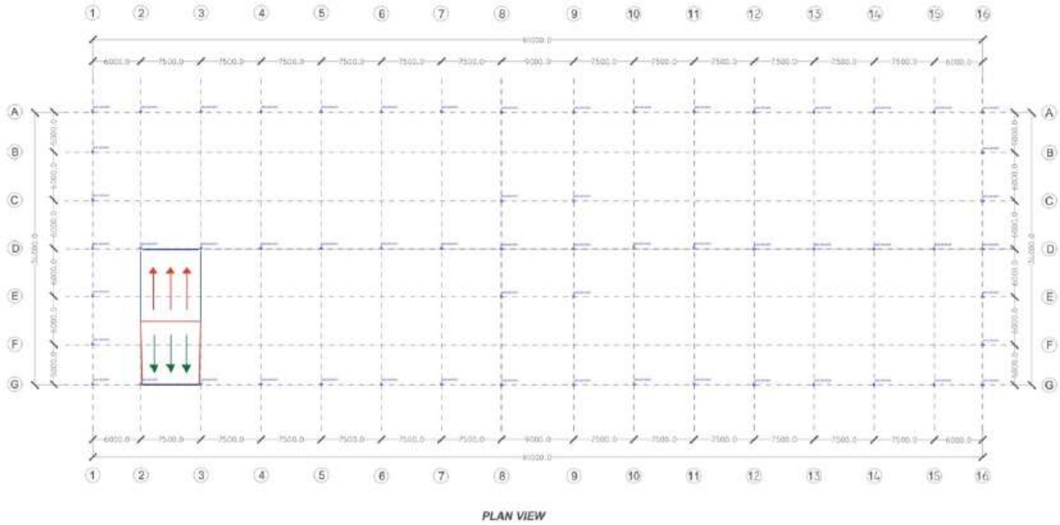


Figure 2.2.2.1. 1:Load distribution

$$g_f = 2.43 \cdot \frac{kN}{(m)^2} \cdot L = 2.43 \cdot \frac{kN}{(m)^2} \cdot 8.500\ m$$

$$g_f = 20.655 \frac{kN}{m} \text{ (Floor dead load)}$$

$$m_b = 660.000\ N/m \text{ (SelfWeight, HEA 600(S355), Assumption)}$$

$$g = 21.315 \frac{kN}{m} \text{(Total Dead Load)}$$

$$q_f = 21.250 \frac{kN}{m} \text{ (Floor Live load)}$$

$$q = 21.250 \frac{kN}{m} \text{ (Total Live Load)}$$

$$w_{uls} = g \cdot 1.35 + q \cdot 1.5 = 21.315\ \frac{kN}{m} \cdot 1.35 + 21.250\ \frac{kN}{m} \cdot 1.5$$

$$w_{uls} = 60.650 \frac{kN}{m} \text{ (Ultimate Limit State)}$$

$$w_{sls} = g + q = 21.315\ \frac{kN}{m} + 21.250\ \frac{kN}{m}$$

$$w_{sls} = 42.565\ \frac{kN}{m} \text{ (Serviceability Limit State)}$$

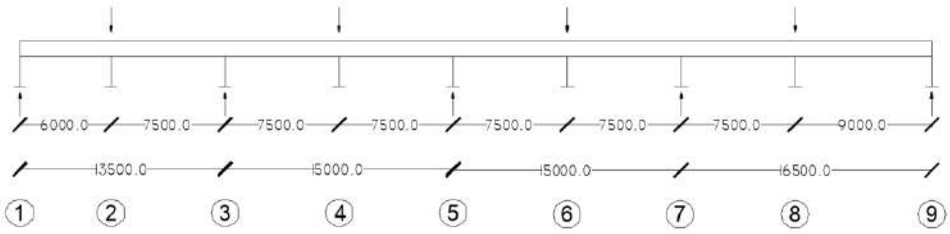
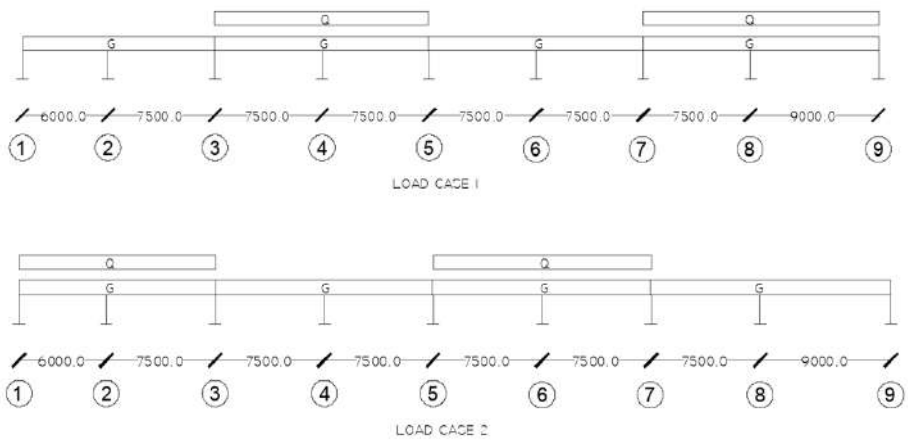


Figure 2.2.2.1. 2:Actions

2.2.2.2. Design bending moments and shear forces

For continuous beams with slabs in building without cantilevers where uniformly distributed loads are dominant, it is sufficient to consider only arrangement of actions shown in Figure 2.2.2.2. 1.



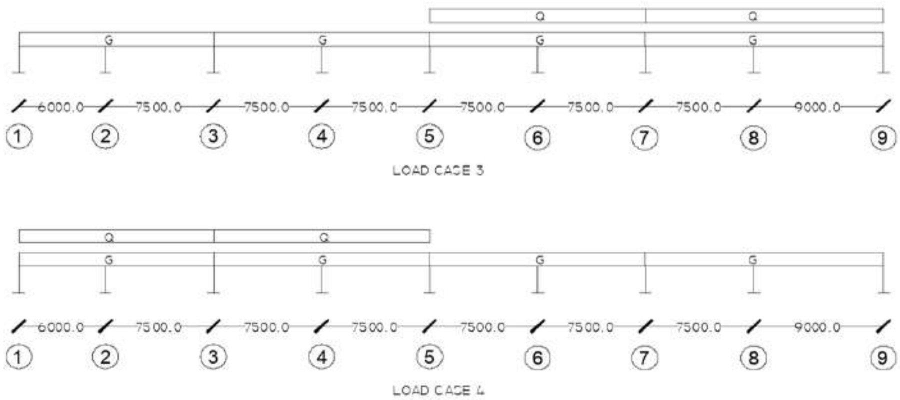


Figure 2.2.4. 1: Load Cases

The design bending moment and shear force diagrams are shown below. From inspection, the most onerous design values are obtained using the load case 1.

Maximum design bending moment occurs at point 8 for load case 1

M_{Ed} = 496.94 kNm (Figure 3.7)

Maximum design shear force occurs at point 8 for load case 1

V_{Ed} = 334.18 kN

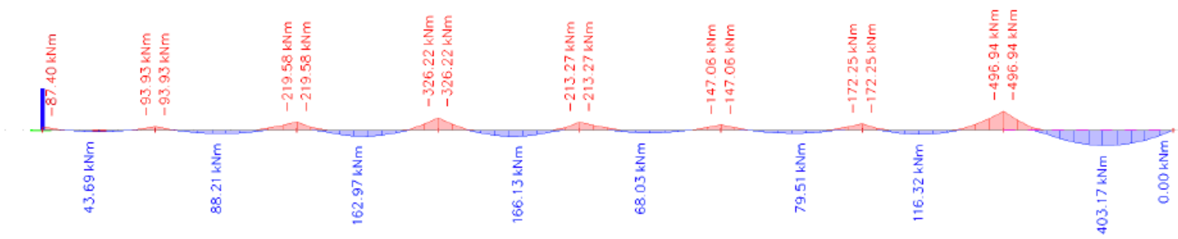


Figure 2.2.4. 2:Bending moment

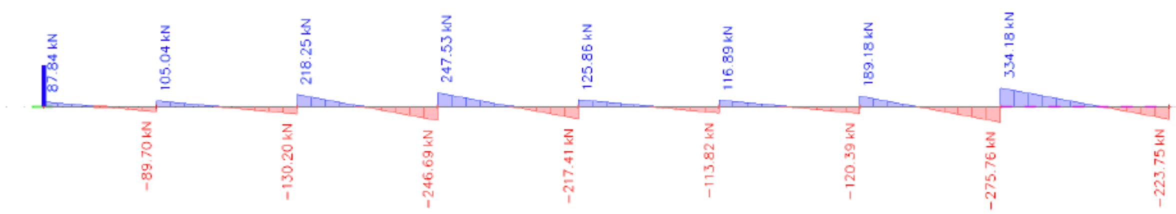


Figure 2.2.4. 3: Shear force

2.2.2.3. Beam design

Section properties of chosen beam, HEA600

$h = 590.000\,mm\,(depth)$ $b = 300.000\,mm\,(width)$ $t_w = 13.000\,mm\,(thickness)$

$t_f = 25.000\,mm$(flange thickness) r =27.000 mm(root radius)

m_w =177.800 kg $\cdot\, m^{-1}$(weight)

d = 486.000 mm (depth between fillets)

I_y =1412000000.000 mm^4 (second moment of area)

i_y =249.700 mm(radius of gyration)

I_z = 112700000.000 mm^4 (second moment of area)

i_z =70.500 mm(radius of gyration)

I_T=4075000.000 mm^4(torsion constant)

I_w=8.8790. 10^{12} mm^6(warping constant)

$A_v = 9321.000\,mm^2$

$Wpl_y =$ 5350000.000 mm^3 (plastic section modulus)

Wpl_z = 1156000.000 mm^3 (plastic section modulus)

$Wel_y =$ 4787000.000 mm^3 (elastic section modulus)

Wel_z = 751400.000 mm^3 (elastic section modulus)

$A = 22646.000\,mm^2$ (area)

E =210.000 MPa(elasticity modulus)

f_y =355.000 MPa(yield strength)

f_u =490.000 MPa(ultimate strength)

$\gamma_{M0} = 1.0$ (partial safety factor)

$\gamma_{m1} = 1.0$ (partial safety factor)

Cross-section classification

$$\varepsilon = \left(\frac{235 \cdot MPa}{f_y}\right)^{\left(\frac{1}{2}\right)}$$

$$\varepsilon = \left(\frac{235 \cdot MPa}{355.000\,MPa}\right)^{\left(\frac{1}{2}\right)}$$

$\varepsilon = 8.136 \times\ 10^{-1}$

- **Outstand of compression flange**

$$c = \frac{b - t_w - 2 \cdot r}{2} = \frac{300.000\,mm - 13.000\,mm - 2 \cdot 27.000\,mm}{2} = 116.500\,mm$$

$$\frac{c}{t_f} = \frac{116.500\,mm}{25.000\,mm} = 4.66$$

The limiting value for Class 1 is $\frac{c}{t_f} \leq 9\epsilon = 7.8624$

The flange in compression is Class 1

- **Web subject to bending**

$$c_b = 486.000\ mm$$

$$\frac{c}{t_w} = \frac{116.500\ mm}{13.000\ mm} = 8.962$$

The limiting value for Class 1 is $\frac{c}{t_w} \leq 72\epsilon = 62.8992$

The web under bending is Class 1

Cross-sectional resistance

- **Shear buckling**

The shear buckling resistance for webs should be verified according to section 5 EN 1993-1-5 [4] if:

$$\frac{h_w}{t_w} > 72 \cdot \frac{\epsilon}{\eta}$$

$$\eta = 1$$

$$h_w = h - 2 \cdot t_f = 590.000\ \text{mm} - 2 \cdot 25.000\ \text{mm} = 540.000\ \text{mm}$$

$$limit = \frac{h_w}{t_w} = \frac{540.000\ mm}{13.000\ mm} = 41.538$$

$$limit_2 = 72 \cdot \frac{\epsilon}{\eta} = 72 \cdot \frac{8.136\ \text{X}\ 10^{-1}}{1} = 58.58$$

The shear buckling resistance of the web does not need to be verified.

- **Shear resistance**

$$V_{pl_{Rd}} = \frac{A_v \cdot \left(\frac{f_y}{(3)^{\left(\frac{1}{2}\right)}}\right)}{\gamma_{M0}}$$

$$V_{pl_{Rd}} = \frac{9321.000 mm^2 \cdot \left(\frac{355.000\ \text{MPa}}{(3)^{\left(\frac{1}{2}\right)}}\right)}{1.0}$$

$V_{pl_{Rd}}$ =1.910 MN

Maximum design shear $V_{Ed} = 334.18\ kN$

If $\frac{V_{Ed}}{V_{c_{Rd}}} < 1.0$ the shear resistance of the section is adequate.

$V_{Ed} = 334.18\ kN$

$V_{c_{Rd}}$ =1.910 MN

$$limit = \frac{V_{Ed}}{V_{c_{Rd}}} = \frac{334.180\ kN}{1.910\ MN} = 1.749 \text{ X } 10^{-1}$$

The shear resistance of the section is adequate.

- **Resistance to bending**

At the point of maximum bending moment, check if the shear force will reduce the bending moment resistance of the section.

$$limit = \frac{V_{c_{Rd}}}{2} = \frac{1.910\ MN}{2} = 955.213 \text{ kN}$$

Shear force at maximum bending moment $V_{Ed} = 334.180\ kN < V_{c_{Rd}} = 955.213$ kN

Therefore, no reduction in resistance to bending due to shear is required.

$$M_{c_{Rd}} = M_{pl_{Rd}} = \frac{W_{pl_y \cdot f_y}}{\gamma_{M0}} = \frac{5350000.000\ mm^3 \cdot 355.000\ MPa}{1.0} = 1.899 \text{ MN} \cdot \text{m}$$

The design bending moment is $M_{Ed} = 496.94$ kNm

$$M_{Ed} = 496.94 \cdot \text{ kN} \cdot \text{m} = 496.94 \cdot \text{kN} \cdot 1.000 \text{ m} = 496.94 \text{ kN} \cdot \text{m}$$

$$\frac{M_{Ed}}{M_{c_{Rd}}} = \frac{496.94 \text{ kN} \cdot \text{m}}{1.899 \text{ MN} \cdot \text{m}} = 2.617 \text{ X } 10^{-1}$$

$$\frac{M_{Ed}}{M_{c_{Rd}}} < 1.0$$

Therefore, the bending moment capacity is adequate.

- **Resistance to lateral torsional buckling**

$UV = 0.9$

$\beta_w = 1$

$C_1 = 3.75$

$\bar{\lambda}_{LT0} = 0.4$

$\bar{\lambda}_{LT} = 4.01X\ 10^{-1}$

$$\phi_{LT} = 0.5 \cdot \left(1 + \alpha \cdot \left((\bar{\lambda}_{LT} - \bar{\lambda}_{LT0}) + \left(\beta \cdot (\bar{\lambda}_{LT})\right)^2\right)\right)$$

$$\phi_{LT} = 0.5 \cdot \left(1 + 0.49 \cdot \left((4.01X\ 10^{-1} - 0.4) + (0.75 \cdot (4.01X\ 10^{-1})^2)\right)\right)$$

$$\phi_{LT} = 5.298\ X\ 10^{-1}$$

$$\chi_{LT} = \frac{1}{\phi_{LT} + ((\phi_{LT})^2 - (\beta \cdot (\bar{\lambda}_T)^2))^{\left(\frac{1}{2}\right)}}$$

$$\chi_{LT} = \frac{1}{5.298\ X\ 10^{-1} + ((5.298\ X\ 10^{-1})^2 - (0.75 \cdot (4.01X\ 10^{-1})^2))^{\left(\frac{1}{2}\right)}}$$

$$\chi_{LT} = 1.0$$

$$M_{b_{Rd}} = \chi_{LT} \cdot W_{el} \cdot \frac{f_y}{\gamma_{M1}} = 1 \cdot 4787000.000\ mm^3 \cdot \frac{355.000\ MPa}{1.0} = 1.699\ MN \cdot m$$

$$\frac{M_{Ed}}{M_{b_{Rd}}} = \frac{496.94\ \mathrm{kN \cdot m}}{1.699\ \mathrm{MN \cdot m}} = 2.924\ \mathrm{X}\ 10^{-1}$$

$$\frac{M_{Ed}}{M_{b_{Rd}}} < 1.0$$

Therefore, the lateral torsional buckling moment capacity is adequate.

The chosen beam HEA600 satisfy the section requirements.

2.2.3. Secondary beam design

The importance of structural integrity during the construction stage is significant. In order to provide that secondary beam is used which its top flange fully restrained laterally by a precast slab.

L=9.000 m (for half span)

$$g_f = 2.43 \cdot \frac{kN}{(m)^2} \cdot L = 2.43 \cdot \frac{kN}{(m)^2} \cdot 9.000\ m$$

$$g_f = 21.870 \frac{kN}{m} \text{ (Floor dead load)}$$

$$m_b = 960.000\ N/m \text{ (SelfWeight, HEA 320(S355), Assumption)}$$

$$g = 22.830 \frac{kN}{m} \text{(Total Dead Load)}$$

$$q_f = 22.500 \frac{kN}{m} \text{ (Floor Live load)}$$

$$q = 22.500 \frac{kN}{m} \text{ (Total Live Load)}$$

$$w_{uls} = g \cdot 1.35 + q \cdot 1.5 = 22.830\ \frac{kN}{m} \cdot 1.35 + 22.500\ \frac{kN}{m} \cdot 1.5$$

$$w_{uls} = 64.570 \frac{kN}{m} \text{ (Ultimate Limit State)}$$

$w_{sls} = g + q = 22.830\ \frac{kN}{m} + 22.500\ \frac{kN}{m}$

w_{sls} =45.330 $\frac{kN}{m}$ (Serviceability Limit State)

Section properties of chosen beam, HEA320

$h = 310.000\ mm\ (depth)$ $b = 300.000\ mm\ (width)$

$t_w = 9.000\ mm\ (thickness)$

$t_f = 15.500\ mm$(flange thickness) r =27.000 mm(root radius)

m_w =97.600 kg $\cdot\ m^{-1}$(weight)

d = 225.000 mm (depth between fillets)

I_y =229300000.000 mm^4 (second moment of area)

i_y =135.800 mm(radius of gyration)

I_z = 69850000.000 mm^4 (second moment of area)

i_z =74.900 mm(radius of gyration)

$Wpl_y =$ 1628000.000 mm^3 (plastic section modulus)

Wpl_z = 709700.000 mm^3 (plastic section modulus)

$Wel_y =$ 1479000.000 mm^3 (elastic section modulus)

Wel_z = 465700.000 mm^3 (elastic section modulus)

A = 12437.000 mm^2 (area)

E =210.000 MPa(elasticity modulus)

f_y =355.000 MPa(yield strength)

f_u =490.000 MPa(ultimate strength)

γ_{M0} = 1.0 (partial safety factor)

γ_{m1} = 1.0 (partial safety factor)

Cross-section classification

$$\varepsilon = \left(\frac{235 \cdot MPa}{f_y}\right)^{\left(\frac{1}{2}\right)}$$

$$\varepsilon = \left(\frac{235 \cdot MPa}{355.000\ MPa}\right)^{\left(\frac{1}{2}\right)}$$

$\varepsilon = 8.136 \times\ 10^{-1}$

- **Outstand of compression flange**

$$c = \frac{b - t_w - 2 \cdot r}{2} = \frac{300.000\ mm - 9.000\ mm - 2 \cdot 27.000\ mm}{2} = 116.500\ mm$$

$$\frac{c}{t_f} = \frac{118.500\ mm}{15.500\ mm} = 7.645$$

The limiting value for Class 1 is $\frac{c}{t_f} \leq \ 9\epsilon \ = \ 7.645$

The flange in compression is Class 1

- **Web subject to bending**

$$c_b \ = 225.000\, mm$$

$$\frac{c}{t_w} = \frac{118.500\, mm}{9.000\, mm} = 13.167$$

The limiting value for Class 1 is $\frac{c}{t_w} \leq \ 72\epsilon \ = \ 62.8992$

The web under bending is Class 1

Cross-sectional resistance

- **Shear buckling**

The shear buckling resistance for webs should be verified according to section 5 EN 1993-1-5 [4] if:

$$\frac{h_w}{t_w} > 72 \cdot \frac{\epsilon}{\eta}$$

$$\eta = 1$$

$$h_w = h - 2 \cdot t_f = 310.000 \text{ mm} - 2 \cdot 15.500 \text{ mm} \ = 279.000 \text{ mm}$$

$$\frac{h_w}{t_w} = \frac{279.000\, mm}{9.000\, mm} = 31.0$$

$$72 \cdot \frac{\epsilon}{\eta} = 72 \cdot \frac{8.136 \text{ X } 10^{-1}}{1} = 58.58$$

The shear buckling resistance of the web does not need to be verified.

- **Shear resistance**

$$A_v = A - \left(2.b.t_f\right) + t_f.(2.r + t_w)$$
$$=12437.000\, mm^2 - (2.300.00\, mm.15.500mm) + 15.500\, mm\ (9.00mm$$
$$+2.27.000\, mm) \ = 4113.500\, mm^2$$

$$V_{pl_{Rd}} = \frac{A_v \cdot \left(\frac{f_y}{(3)^{\left(\frac{1}{2}\right)}}\right)}{\gamma_{M0}}$$

$$V_{pl_{Rd}} = \frac{4113.500mm^2 \cdot \left(\frac{355.000 \text{ MPa}}{(3)^{\left(\frac{1}{2}\right)}}\right)}{1.0}$$

$V_{pl_{Rd}}$ =843.100 kN

Maximum design shear $V_{Ed} = 195\ kN$

If $\frac{V_{Ed}}{V_{c_{Rd}}} < 1.0$ the shear resistance of the section is adequate.

$V_{Ed} = 195.000\ \ kN$

$V_{c_{Rd}}$ =843.100 kN

$$\frac{V_{Ed}}{V_{c_{Rd}}} = \frac{195.000\ kN}{843.100\ kN} = 2.313 \text{ X } 10^{-1}$$

The shear resistance of the section is adequate.

- **Resistance to bending**

At the point of maximum bending moment, check if the shear force will reduce the bending moment resistance of the section.

$$\frac{V_{c_{Rd}}}{2} = \frac{843.100\ kN}{2} = 421.550 \text{ kN}$$

Shear force at maximum bending moment $V_{Ed} = 195.00\ kN < V_{c_{Rd}} = 843.100$ kN

Therefore, no reduction in resistance to bending due to shear is required.

$$M_{c_{Rd}} = M_{pl_{Rd}} = \frac{W_{pl_y \cdot f_y}}{\gamma_{M0}} = \frac{1628000.000\ mm^3 \cdot 355.000\ MPa}{1.0} = 577.940 \text{ kN} \cdot \text{m}$$

The design bending moment is $M_{Ed} = 496.94$ kNm

$M_{Ed} = 496.94 \cdot \text{ kN} \cdot \text{m}$

$$limit = \frac{M_{Ed}}{M_{c_{Rd}}} = \frac{496.94 \text{ kN} \cdot \text{m}}{577.940 \text{ kN} \cdot \text{m}} = 8.598 \text{ X } 10^{-1}$$

$$\frac{M_{Ed}}{M_{c_{Rd}}} < 1.0$$

Therefore, the bending moment capacity is adequate.

The chosen beam HEA320 satisfy the section requirements.

2.2.4. Preliminary Column Design

2.2.4.1. Actions-Loading

The design load area shown in Figure 3.9 presents loading area of expected critical section.

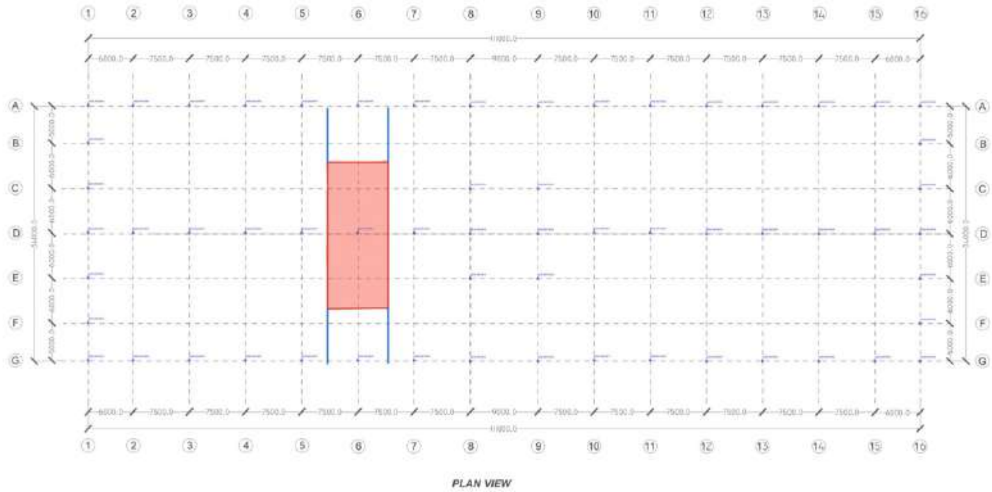


Figure 2.2.4.1. 1:Loading area

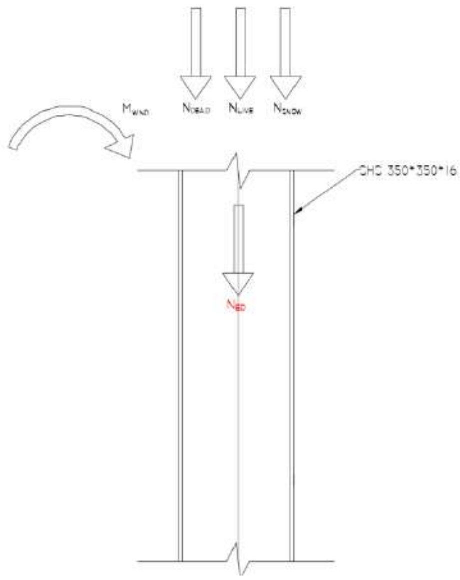


Figure 2.2.4.1. 2:Actions

Design value of force for ULS

$g_k = 2.430\ kPa$ $\quad q_k = 2.500\ kPa$ $\quad g_{sk} = 700.000\ Pa$

$q_w = 320.000\ Pa$ $\quad B = 7.500\ m$ $\quad H = 8.500\ m$

$n_{floor} = 3$ $\quad L_{Total} = 34.000\ m$ $\quad H_{Total} = 9.500\ m$

$g_{steel} = 785.000\ N/m^3$ $\quad A_{col} = 7524.000\ mm^2$

$\gamma_G = 1.35$ $\quad \gamma_Q = 1.5$ $\quad \gamma_{M0} = 1.0$ $\quad \psi_{0_{Snow}} = 0.5$

$\psi_{0_{wind}} = 0.6$ $\quad b_c = 3500.000\ mm\ (SHS\ 350 * 350 * 16)$

$A_{load} = B \cdot H \cdot 2 = 7.500\ m \cdot 8.500m \cdot 2 \qquad = 127.500\ m^2\ (Load\ Area)$

$N_{self} = A_{col} \cdot g_{steel} \cdot H_{Total} = 7524.000\, mm^2 \cdot 785.000 \dfrac{N}{m^3} \cdot 12.600\, m$

$N_{self} = 74.420\, N\, (Column\ self\ weight)$

$N_{dead} = A_{load} \cdot g_k \cdot n_{floor} = 127.500\, m^2 \cdot 2.430\, kPa \cdot 3$

$N_{dead} = 929.475\, kN\ \ (Dead\ Load)$

$N_{live} = A_{load} \cdot q_k \cdot n_{floor} = 127.500\, m^2 \cdot 2.500\, kPa \cdot 3$

$N_{live} = 956.250\, kN\ \ (Live\ Load)$

$N_{snow} = A_{load} \cdot g_{sk} = 127.500\, m^2 \cdot 700.000\, Pa$

$N_{snow} = 89.250\, kN\, (Snow\ Load)$

$M_{wind} = q_w \cdot \left(\dfrac{L_{total}}{2}\right) \cdot \left(\dfrac{(H_{Total})^2}{2}\right)$

$M_{wind} = 320.000\, Pa \cdot \left(\dfrac{34.000\, m}{2}\right) \cdot \left(\dfrac{(12.600\, m)^2}{2}\right)$

$M_{wind} = 431.827\, kN \cdot m$ ($Bending\ moment$ caused by wind load)

$N_{wind} = \dfrac{M_{wind}}{B} = \dfrac{431.827\, kN \cdot m}{7.500\, m}$

$N_{wind} = 57.577\, kN\, (Tension\ Force)$

$N_{ed} = \gamma_G \cdot (N_{self} + N_{dead}) + \gamma_Q \cdot (N_{live} + N_{snow} \cdot \psi_{0_{snow}} + N_{wind} \cdot \psi_{0_{wind}})$

$N_{ed} = 1.35 \cdot (74.420\, N + 929.475\, kN) + 1.5 \cdot (956.250\, kN + 89.250\, kN \cdot 0.5 + \cdots$

$\ldots + 57.577\, kN \cdot 0.6)$

$N_{ed} = 2.808\, MN\, (Design\ Load)$

2.2.4.2. Column Design

Section properties of chosen section which is SHS 350*350*16

h= 350.000 mm (width)

b= 350.000 mm (side dimension)

t = 16.000 mm (thickness)

L= 3.500 m(height)

r_0= 24.000 mm(outer rounding radius)

r_i= 16.000 mm(inner rounding radius)

A = 21101.000 mm^2 (area)

A_v= 10551.000 mm^2(shear area)

I= 389400000.000 mm^4(second moment of inertia)

W_{el}= 2225000.000 mm^3(elastic section modulus)

W_{pl}=2630000.000 mm^3(plastic section modulus)

$N_{pl_{Rd}}$= 7.491 MN(design plastic axial force resistance)

$V_{pl_{Rd}}$= 2.162 MN(design plastic shear axial force resistance)

$M_{el_{Rd}}$= 789.970 $kN.m$(design elastic bending moment resistance)

$M_{pl_{Rd}}$=933.530 $kN.m$(design plastic bending moment resistance)

$T_{pl_{Rd}}$= 668.910 kN(design plastic torsional moment resistance)

i= 117.000 mm(radius of gyration)

y_0= 0

W_T= 3264000.000 mm^3(torsion modulus)

I_T=609900000.000 mm^4(torsion constant)

f_y= 355.000 MPa(yield strength)

f_u= 490.000 MPa(ultimate strength)

γ_{M0} = 1.0 (resistance of cross − section)

γ_{M1} = 1.0 (resistance of members to instability)

G= 81.000 GPa(shear modulus)

α= 0.21(buckling curve)

π= 3.142(π pi constant)

E= 210.000 GPa(modulus of elasticity)

Cross-sectional resistance

Compression resistance should verify Ned/Nc, Rd ≤ 1.0

The design resistance of the cross-section for uniform compression is;

$$N_{C_{Rd}}=\mathrm{A}\cdot\frac{f_y}{\gamma_{M0}} = 21101.000mm^2 \cdot\frac{355.000\ MPa}{1.0} = 7.491\ MN\ (for\ class\ 1)$$

$$\frac{N_{Ed}}{N_{C_{Rd}}} = \frac{2.786\ MN}{7.491\ MN} = 3.719\ X\ 10^{-1}$$

$$\frac{N_{Ed}}{N_{C_{Rd}}} < 1.00;$$

Therefore, the compression resistance of the cross-section is adequate.

Member buckling resistance

The design buckling resistance is determined from:

L_{cr}=3.500 m

ε= 8.136 X 10^{-1}

λ_1=76.399

$$\bar{\lambda} = \left(\frac{L_{cr}}{i}\right) \cdot \left(\frac{1}{\lambda_1}\right) = \left(\frac{3.500\ m}{117.000\ mm}\right) \cdot \left(\frac{1}{76.399}\right) = 3.916x10^{-1}$$

$$\phi = 0.5 \cdot (1 + \alpha \cdot (\bar{\lambda} - 0.2\ ) + (\bar{\lambda})^2)$$

$$\phi = 0.5 \cdot (1 + 0.21 \cdot (3.916\ X\ 10^{-1} - 0.2\ ) + (3.916\ X\ 10^{-1})^2)$$

$$\phi = 5.968\ X\ 10^{-1}$$

$$\chi = \frac{1}{\phi + ((\phi)^2 - (\bar{\lambda})^2)^{\left(\frac{1}{2}\right)}}$$

$$\chi = \frac{1}{5.968\ X10^{-1} + ((5.968\ X10^{-1})^2 - (3.916\ X\ 10^{-1})^2)^{\left(\frac{1}{2}\right)}}$$

$$\chi = 0.955$$

$$N_{b_{Rd}} = \chi \cdot \mathrm{A}. \frac{f_y}{\gamma_{M1}} = 0.955 \cdot 21101.000\ mm^2\ \cdot \frac{355.000\ MPa}{1.0} = 7.154\ MN$$

$$\frac{N_{Ed}}{N_{b_{Rd}}} = \frac{2.808\ MN}{7.154\ MN} = 3.925\ X\ 10^{-1}$$

$$\frac{N_{Ed}}{N_{b_{Rd}}} < 1.00;$$

Therefore, the flexural buckling resistance of the section is adequate.

Torsional and torsional-flexural buckling resistance

$i_0 = 165.463\ mm$

$N_{c_{rT}} = 1.811\ GN$

$$\bar{\lambda}_T = \left(\frac{A \cdot f_y}{N_{c_{rT}}}\right)^{\left(\frac{1}{2}\right)}$$

$$\bar{\lambda}_T = \left(\frac{21101.000\ mm^2 \cdot 355.000\ MPa}{1.811\ GN}\right)^{\left(\frac{1}{2}\right)}$$

$\bar{\lambda}_T$=6.431 $X\ 10^{-2}$

$$\phi_T = 0.5 \cdot (1 + \alpha \cdot \left((\bar{\lambda}_T - 0.2\ ) + (\bar{\lambda})^2\right))$$

$$\phi_T = 0.5 \cdot (1 + 0.21 \cdot ((6.431\ X\ 10^{-2} - 0.2\ ) + (6.431\ X\ 10^{-2})^2))$$

$$\phi_T = 4.862\ X\ 10^{-1}$$

$$\chi_T = \frac{1}{\phi_T + ((\phi_T)^2 - (\bar{\lambda}_T)^2)^{\left(\frac{1}{2}\right)}}$$

$$\chi_T = \frac{1}{4.862\ X\ 10^{-1} + ((4.862\ X\ 10^{-1})^2 - (6.431\ X\ 10^{-2})^2)^{\left(\frac{1}{2}\right)}}$$

$$\chi_T = 1.033$$

$$\chi_T = 1.0$$

$$N_{bT_{Rd}} = \cdot \frac{\chi_T \cdot A \cdot f_y}{\gamma_{M1}} = \frac{1.0 \cdot 21101.000\ mm^2\ \cdot 355.000\ MPa}{1.0} = 7.491\ MN$$

$$\frac{N_{Ed}}{N_{bT_{Rd}}} < 1.00;$$

Therefore, the torsional buckling resistance of the section is adequate.

Lateral torsional buckling resistance

Connection eccentricity causes a design major axis bending moment is shown below.

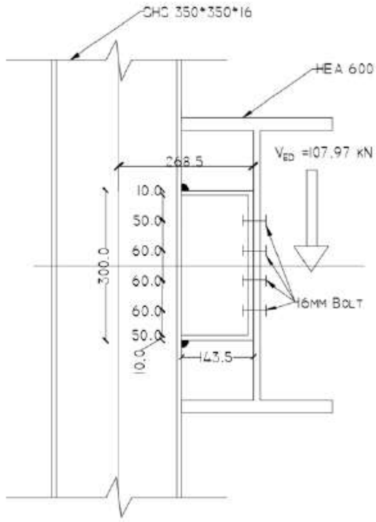


Figure 2.2.4.1. 3: Bending moment due to connection

$$e\ = 268.500\ mm$$

V_{Ed_6}=107.97 kN

$$M_{Ed} = M_{Ed_6} = V_{Ed_6} \cdot e = 107.97\ kN \cdot 268.500\ mm = 28.990\ kN \cdot m$$

$$UV = 0.9$$

$$\beta_w = 1$$

$$C_1 = 1$$

$\beta = 0.75$

$\bar{\lambda}_{LT0} = 0.4$

$\bar{\lambda}_{LT} = 3.524X\ 10^{-1}$

The slenderness for lateral torsional buckling is less than $\bar{\lambda}_{LT0}$ the effect of lateral torsional buckling is neglected. [3]

$$\phi_{LT} = 0.5 \cdot \left(1 + \alpha \cdot \left((\bar{\lambda}_{LT} - \bar{\lambda}_{LT0}) + \left(\beta \cdot (\bar{\lambda}_{LT})\right)^2\right)\right)$$

$$\phi_{LT} = 0.5 \cdot \left(1 + 0.21 \cdot \left((3.524X\ 10^{-1} - 0.4) + (0.75 \cdot (3.524X\ 10^{-1})^2)\right)\right)$$

$$\phi_{LT} = 5.048\ X\ 10^{-1}$$

$$\chi_{LT} = \frac{1}{\phi_{LT} + ((\phi_{LT})^2 - (\beta \cdot (\bar{\lambda}_T)^2))^{\left(\frac{1}{2}\right)}}$$

$$\chi_{LT} = \frac{1}{5.048\ X\ 10^{-1} + ((5.048\ X\ 10^{-1})^2 - (0.75 \cdot (3.524X\ 10^{-1})^2))^{\left(\frac{1}{2}\right)}}$$

$$\chi_{LT} = 1.103$$

$$\chi_{LT} = 1.0$$

$$M_{b_{Rd}} = \chi_{LT} \cdot W_{el} \cdot \frac{f_y}{\gamma_{M1}} = 1 \cdot 2225000.000\ mm^3 \cdot \frac{355.000\ MPa}{1.0} = 789.875\ kN \cdot m$$

Combined axial load plus bending

$$condition_3 = \left(\frac{N_{Ed}}{N_{b_{Rd}}}\right) + \left(\frac{M_{Ed}}{M_{b_{Rd}}}\right) = \left(\frac{2.808\text{MN}}{7.154\text{MN}}\right) + \left(\frac{28.990\ kN \cdot m}{789.875\ kN \cdot m}\right) = 4.292\ X\ 10^{-1}$$

$$\left(\frac{N_{Ed}}{N_{b_{Rd}}}\right) + \left(\frac{M_{Ed}}{M_{b_{Rd}}}\right) < 1.00$$

Therefore, combined axial load plus bending resistance is adequate. The chosen column section SHS 350x350x16 satisfies the section requirements.

2.2.5. Connection Design

Joining beam and column together is done by reverse channel connection where its web is connected to beam by bolting. The connection has eccentricity and it causes bending moment which is shown at column calculation.

It is assumed that project specifications require class 8.8 bolts.

f_{yb} =640.000 MPa

f_{ub} =800.000 MPa

f_y =355.000 MPa

b =160.000 mm

d_p =300.000 mm

t =10.000 mm

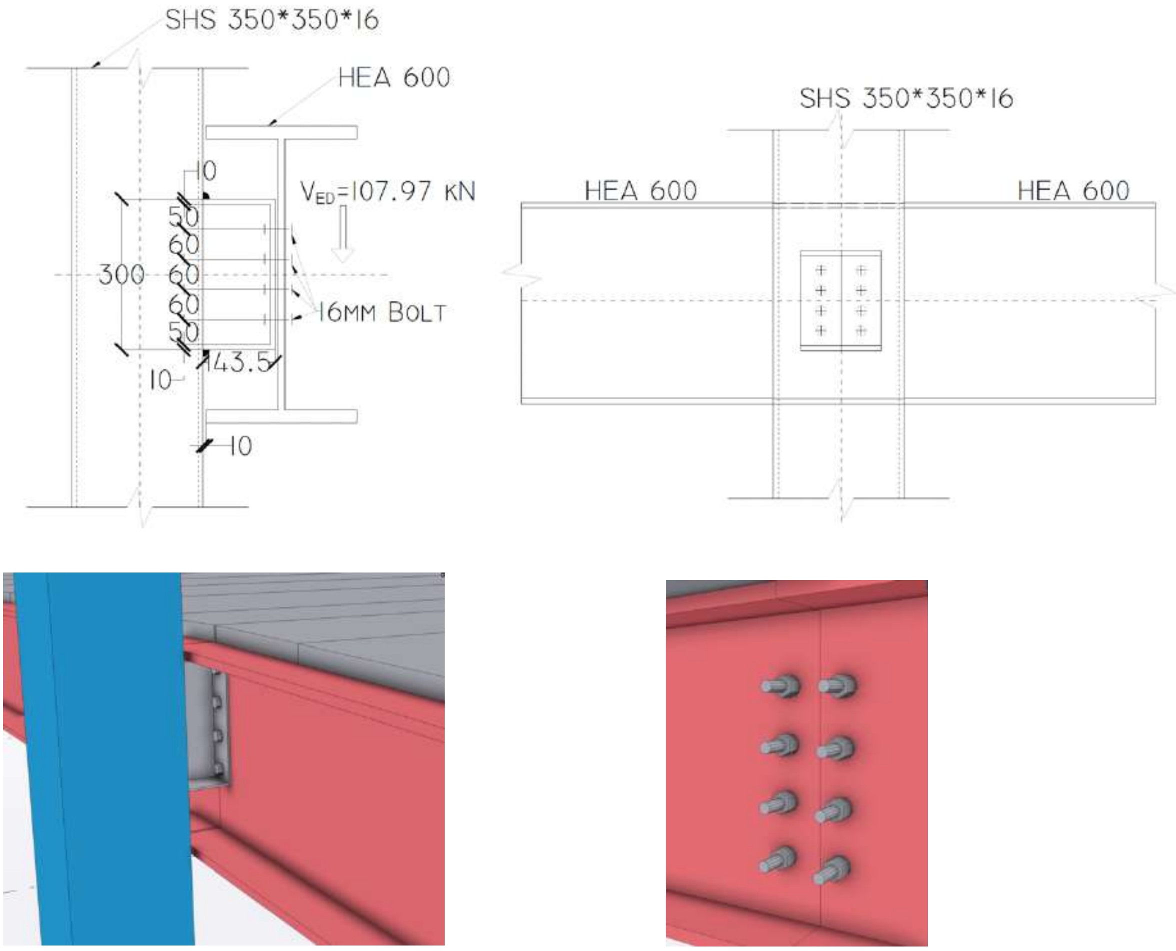


Figure 2.2.5. 1:Connection between column and beam

Design shear resistance of one fully treated bolt is

$\alpha_v = 0.6$

$\gamma_{M2} = 1.25$

$A_s = 157.000\ mm^2$

$$F_{vRd} = \frac{\alpha_v \cdot\ A_s \cdot f_{ub}}{\gamma_{M2}} = \frac{0.6 \cdot 157.000\ mm^2 \cdot\ 800.000\ \text{MPa}}{1.25} = 60.288\ \text{kN}$$

The design bearing resistance of bolts is evaluated by factors k1, a, αb as governed by the plate

$e_2 = 80.000\ mm$

$d_0 = 18.000$ mm

$$k_1 = \min\left(\left(\left(2.8 \cdot \frac{e_2}{d_0}\right) - 1.7\right), 2.5\right) = \min\left(\left(\left(2.8 \cdot \frac{80.000\ \text{mm}}{18.000\ \text{mm}}\right) - 1.7\right), 2.5\right) = 2.5$$

$e_1 = 50.000\ mm$

$p_1 = 60.000$ mm

f_u =360.000 MPa

$$\alpha_b = \min\left(\frac{e_1}{3 \cdot d_0}, \left(\frac{p_1}{3 \cdot d_0} - 0.25\right), \left(\frac{f_{ub}}{f_u}\right), 1\right)$$

$$\alpha_b = \min\left(\frac{50.000\ \text{mm}}{3 \cdot 18.000\ \text{mm}}, \left(\frac{60.000\ \text{mm}}{3 \cdot 18.000\ \text{mm}} - 0.25\right), \left(\frac{800.000\ \text{MPa}}{360.000\ \text{MPa}}\right), 1\right)$$

$\alpha_b = 8.611\text{X}\ 10^{-1}$

Analogous to the bearing resistance of the bolts, the bearing resistance of the beam web may be calculated by factors k_1 and α_b governed by the plate as follows:

$e_{1_1} = 170.000$ mm

$p_1 = 60.000$ mm

f_u =360.000 MPa

$$\alpha_b = \min\left(\frac{e_{1_1}}{3 \cdot d_0}, \left(\frac{p_1}{3 \cdot d_0} - 0.25\right), \left(\frac{f_{ub}}{f_u}\right), 1\right)$$

$$\alpha_b = \min\left(\frac{170.000\ \text{mm}}{3 \cdot 18.000\ \text{mm}}, \left(\frac{60.000\ \text{mm}}{3 \cdot 18.000\ \text{mm}} - 0.25\right), \left(\frac{800.000\ \text{MPa}}{360.000\ \text{MPa}}\right), 1\right)$$

$\alpha_b = 8.611\text{X}\ 10^{-1}$

The design bearing resistance of one bolt is

d =16.000 mm

t =10.000 mm

$$F_{bRd} = k_1 \cdot \alpha_b \cdot d \cdot t \cdot \frac{f_u}{\gamma_{M2}} = 2.5 \cdot 8.611\text{X}\ 10^{-1} \cdot 16.000\ \text{mm} \cdot 10.000\ \text{mm} \cdot \frac{360.000\ \text{MPa}}{1.25}$$

$F_{bRd} = 99.200$ kN

The design bearing resistance of four bolts are;

$V_{rd} = 4 \cdot F_{bRd} = 4 \cdot 99.200\ \text{kN} = 396.800\ \text{kN}$

$V_{rd} = 396.8\text{kN} > V_{majorEd} = 107.97\text{kN}$

The connection resistance is satisfactory.

Eccentricity of acting shear force in bolts creates a bending moment in welds:

The bolts group must resist the following bending moment coming from eccentricity.

$V_{Ed} = 107.970$ kN

$e_{\text{boltgroup}} = 268.500$ mm

$M_{ed} = e_{\text{boltgroup}} \cdot V_{Ed} = 268.500$ mm $\cdot$ 107.970 kN $= 28.990$ kN $\cdot$ m

The moments introduce stress σw is plane of the plate:

a =6.000 mm

l =300.000 mm

$$W_{el_w} = \left(\frac{2\,.\,a\,.\,(l)^2}{6}\right) = \left(\frac{2\;x\;6.000\;mm\,.\,(300.000\;mm)^2}{6}\right) = 180000.000\;mm^3$$

$$\sigma_w = \frac{M_{ed}}{W_{el_w}} = \frac{28.990\text{ kN}\cdot\text{m}}{180000.000\;mm^3} = 161.055\text{ MPa}$$

This stress is resolved into stress perpendicular and parallel to axes of weld throat calculated as follows:

$$\tau_{\perp} = \sigma_{\perp} = \frac{\sigma_w}{(2)^{1/2}} = \frac{161.055\text{ MPa}}{(2)^{1/2}} = 113.883\;MPa$$

$$\tau_{//} = \frac{V_{Ed}}{2.\,a.\,l} = \frac{107.970\text{ kN}}{2\text{x }6.0\text{mmx}300\text{mm}} = 29.992\;MPa$$

The design resistance of the fillet weld is sufficient if the following are satisfied.

$\beta_w = 0.9$

$$C_1 = ((\sigma_L)^2 + 3.((\tau_L)^2 - (\tau_{//})^2))^{\left(\frac{1}{2}\right)}$$

$$C_1 = ((113.833\;MPa)^2 + 3.((113.833MPa)^2 - (29.992\;MPa)^2))^{\left(\frac{1}{2}\right)}$$

$C_1 = 233.615\;MPa$

$$C_2 = \frac{f_u}{\beta_w.\gamma_{M2}} = \frac{360\;MPa}{0.9\;x\;1.25} = 320\;MPa$$

$\sigma_L = 113.883\;MPa$

$$C_3 = \frac{f_u}{\gamma_{M2}} = \frac{360\;MPa}{1.25} = 288\;MPa$$

$C_1 = 233.615\;MPa < C_2 = 320.00\;MPa\;\;and\;\;\sigma_L = 113.883\;MPa < C_3 = 288.00\;MPa$

The weld is satisfactory.

The design block tearing resistance of a plate is given by summing the resistance on critical area in tension and that in shear.

$$A_{nt} = t.\left(e_2 - \frac{d_0}{2}\right) = 10.00mm\,.(80.00\;mm - \frac{18.00\;mm}{2} = 710.00\;mm^2$$

$$A_{nv} = t.\left(e_2 + \; p_1 - d_0 \; - \frac{d_0}{2}\right) = 10mm\,.(80.00\;mm + 60.00\;mm - 18\;mm - \frac{18\;mm}{2}$$
$$= 1130.00\;mm^2$$

$\gamma_{M0} = 1.0$

$$V_{eff2Rd} = \left(\frac{0.5\,.A_{nt}.f_u}{\gamma_{M2}}\right) + \left(\frac{0.5\,.A_{nv}.f_y}{(3)^{\left(\frac{1}{2}\right)}.\gamma_{M0}}\right)$$

$$= \left(\frac{0.5\,.710.00mm^2\,.\;360.00\;MPa}{1.25}\right) + \left(\frac{1130.00\;mm^2.355\;MPa}{(3)^{\left(\frac{1}{2}\right)}.1.0}\right)$$
$$= 333.844\;kN$$

$V_{eff2Rd} = \; 333.844\;kN > V_{Ed} = 107.97\;kN$, The design block tearing resistance is satisfactory.

The shear resistance of gross area:

$A_v = 3000.0\;mm^2$

$$V_{plRd} = \left(\frac{A_v.f_y}{(3)^{\left(\frac{1}{2}\right)}.\gamma_{M0}}\right) = \frac{3000.00\;mm^2.355\;MPa}{(3)^{\left(\frac{1}{2}\right)}.1.0} = 614.878\;kN$$

$V_{plRd} = \; 614.878\;kN > V_{Ed} = 107.97\;kN$, The shear resistance of gross area is satisfactory.

For the beam web the design block shear resistance is evaluated in similar way as for the plate.

$t_w = 13.00mm$

$A_{ntw} = t_w.\left(e_2 - \frac{d_0}{2}\right) = 13.00mm\,.(80.00\,mm - \frac{18.00\,mm}{2} = 923.00\,mm^2$

$A_{nvw} = t_w.\left(120\,mm\; + e_1 + \; p_1 - d_0\; - \frac{d_0}{2}\right)$

$= 13.00mm\,.(120.00\,mm + 50.00\,mm + 60.00\,mm - 18\,mm - \frac{18\,mm}{2}$

$= 2639.00\,mm^2$

The beam shear resistance:

$$V_{eff2Rd} = \left(\frac{0.5\,.A_{ntw}.f_u}{\gamma_{M2}}\right) + \left(\frac{0.5\,.A_{nvw}.f_y}{(3)^{\left(\frac{1}{2}\right)}.\gamma_{M0}}\right)$$

$$= \left(\frac{0.5\,.923.00mm^2\,.\;360.00\,MPa}{1.25}\right) + \left(\frac{2639.00\,mm^2.355\,MPa}{(3)^{\left(\frac{1}{2}\right)}.1.0}\right)$$

$= 673.800\,kN$

$V_{eff2Rd} = \; 673.800\,kN > V_{Ed} = 107.97\,kN$, The beam shear resistance is satisfactory.

The bending resistance is checked for class 3 cross-section which resistance is;

$$M_{elRd} = \left(\frac{W_{elw}.f_y}{\gamma_{M0}}\right) = \left(\frac{180000mm^3\,.\;355.00\,MPa}{1.0}\right) = 63.900\,kNm$$

$M_{elRd} = \; 63.900\,kNm > M_{Ed} = 28.99\,kNm$;

Therefore, the connection is satisfactory.

2.2.6. Bracing Design

The robustness and stability are a critical structural design consideration of a structure. Multi-storey car park has that problem because it does not have internal walls which help stability. In order to provide stability, bracing member is placed at suitable bays.

Section properties of chosen section which is SHS 110*110*14.2 (S275 JR)

b= 120.000 mm (side dimension) L= 8.276 m(height)

A = 5790.000 mm^2 (area)

f_y =275.000 MPa t =14.200 mm

I_y=10500000.000 mm^4(second moment of inertia, y − y axis

I_z=10500000.000 mm^4(second moment of inertia, z − z axis)

$\gamma_{M0} = \; 1.0$ (resistance of cross − section)

$\gamma_{M1} = \; 1.0$ (resistance of members to instability)

α= 0.21(buckling curve)

π= 3.142(π pi constant)

E= 210.000 GPa(modulus of elasticity)

Cross-sectional resistance

N_{Ed} =259.220 kN

$$N_{cRd} = \left(\frac{A_s . f_y}{\gamma_{M0}}\right) = \left(\frac{5790.00\, mm^2\ .\ 275.00\, MPa}{1.0}\right) = 1.592 \text{ MN}$$

$$\left(\frac{N_{Ed}}{N_{cRd}}\right) = \left(\frac{259.220\, kN}{1.592 \text{ MN}}\right) = 1.628\, X\, 10^{-1}$$

$\left(\frac{N_{Ed}}{N_{cRd}}\right)$< 1.0, Resistance in tension is adequate.

Buckling resistance

L_{cr} =8.276 m

$$N_{cr} = \left(\frac{(\pi)^2 . E . I_z}{(L_{cr})^2}\right) = \frac{(3.142)^2 . 210.000\, GPa . 10500000.000\, mm^4}{(8.276 \text{ m})^2} = 317.737\, kN$$

$$\bar{\lambda}_T = \left(\frac{A \cdot f_y}{N_{cr}}\right)^{\left(\frac{1}{2}\right)}$$

$$\bar{\lambda}_T = \left(\frac{5790.000\, mm^2 \cdot 275.000\, MPa}{317.737\ \ kN}\right)^{\left(\frac{1}{2}\right)} = 2.239$$

$$\phi = 0.5 \cdot \left(1 + \alpha \cdot \left((\bar{\lambda} - 0.2\ ) + (\bar{\lambda})^2\right)\right)$$

$$\phi = 0.5 \cdot \left(1 + 0.21 \cdot \left((2.239 - 0.2\ ) + (2.239)^2\right)\right)$$

$$\phi = 3.22$$

$$\chi = \frac{1}{\phi + ((\phi)^2 - ((\bar{\lambda})^2))^{\left(\frac{1}{2}\right)}}$$

$$\chi = \frac{1}{3.22 + ((3.22)^2 - (2.239)^2))^{\left(\frac{1}{2}\right)}}$$

$$\chi = 1.807 X\, 10^{-1}$$

$$N_{b_{Rd}} = \chi \cdot A \cdot \frac{f_y}{\gamma_{M1}} = 1.807 X\, 10^{-1} \cdot 5790.000\, mm^2 \cdot \frac{275.000\, MPa}{1.0} = 287.734\, kN$$

$\left(\frac{N_{Ed}}{N_{bRd}}\right)$=$\left(\frac{259.220\ kN}{287.734\ kN}\right)$=9.009 X 10^{-1}

$\left(\frac{N_{Ed}}{N_{cRd}}\right)$< 1.0, Buckling resistance is adequate.

Therefore, chosen bracing member 110x110x14.2 is satisfied requirements.

2.2.7. Column Base Design

Column is subjected to the self-weight, dead load, snow load and live load with loading area defined in figure. Calculated column is part of bracing system-there are additional forces due to acting in bracing system.

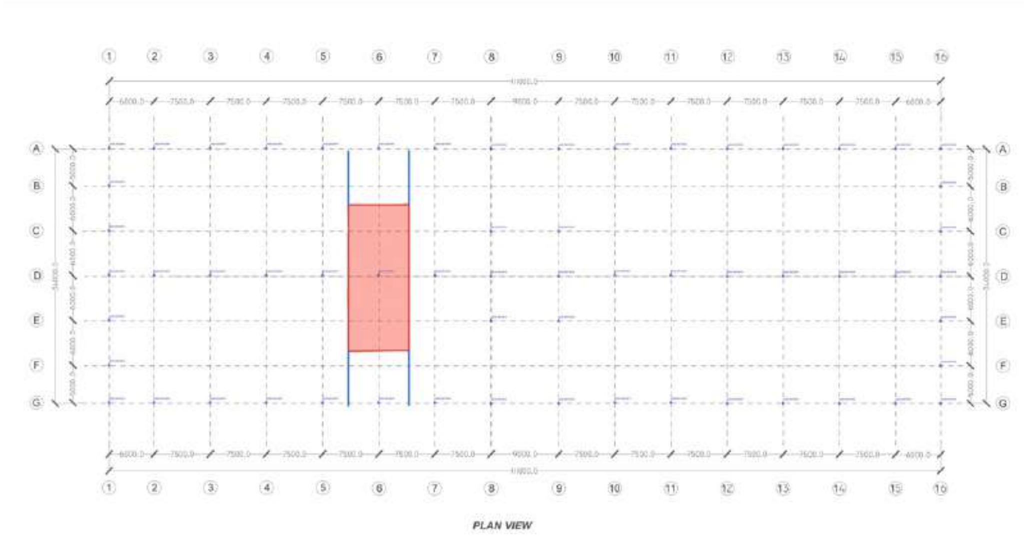


Figure 2.2.7. 1:Calculated column load area

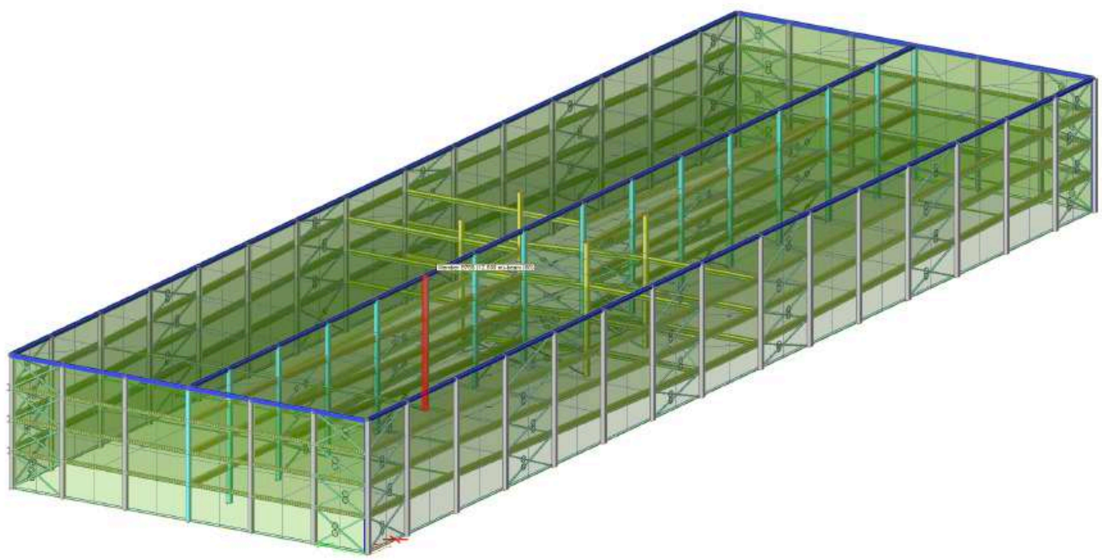


Figure 2.2.7. 2: Calculated column

$g_k = 2.430\ kPa$ $q_k = 2.500\ kPa$ $g_{sk} = 700.000\ Pa$

$q_w = 320.000\ Pa$ $B = 7.500\ m$ $H = 8.500\ m$

$n_{floor} = 3$ $L_{Total} = 34.000\ m$ $H_{Total} = 12.600\ m$

$g_{steel} = 785.000\ N/m^3$ $A_{col} = 7524.000\ mm^2$ $\gamma_G = 1.35$

$\gamma_Q = 1.5$ $\gamma_{M0} = 1.0$ $\psi_{0_{Snow}} = 0.5$

$\psi_{0_{wind}} = 0.6$ $b_c = 350.000\ mm\ (SHS\ 350 * 350 * 16)$

Maximum normal force in column base

Load Area

$A_{load} = B \cdot H \cdot 2 = 7.500\ m \cdot 8.500m \cdot 2 \qquad = 127.500\ m^2\ (Load\ Area)$

Column self-weight

$$N_{self} = A_{col} \cdot g_{steel} \cdot H_{Total} = 7524.000\ mm^2 \cdot 785.000 \frac{N}{m^3} \cdot 12.600m$$

N_{self} =74.420 N$(Column\ self\ weight)$

Dead Load

$N_{dead} = A_{load} \cdot g_k \cdot n_{floor} = 127.500\ m^2 \cdot 2.430\ kPa \cdot 3$

$N_{dead} = 929.475\ kN\ \ (Dead\ Load)$

Live Load

$N_{live} = A_{load} \cdot q_k \cdot n_{floor} = 127.500\ m^2 \cdot 2.500\ kPa \cdot 3$

$N_{live} = 956.250\ kN\ \ (Live\ Load)$

Snow Load

$N_{snow} = A_{load} \cdot g_{sk} = 127.500\ m^2 \cdot 700.000\ Pa$

$N_{snow} = 89.250\ kN\ (Snow\ Load)$

Bending moment on cantilever substituting truss bracing caused by wind load;

$$M_{wind} = q_w \cdot \left(\frac{L_{total}}{2}\right) \cdot \left(\frac{(H_{Total})^2}{2}\right)$$

$$M_{wind} = 320.000\ Pa \cdot \left(\frac{34.000\ m}{2}\right) \cdot \left(\frac{(12.600\ m)^2}{2}\right)$$

$M_{wind} = 431.827\ kN \cdot m$ $\quad$ ($Bending\ moment$ caused by wind load)

$$N_{wind} = \frac{M_{wind}}{B} = \frac{431.827\ kN \cdot m}{7.500\ m}$$

N_{wind} =57.577 kN ($Tencion\ Force$)

Total load combination

$$N_{ed} = \gamma_Q \cdot (N_{self} + N_{dead}) + \gamma_Q \cdot (N_{live} + N_{snow} \cdot \psi_{0_{snow}} + N_{wind} \cdot \psi_{0_{wind}})$$

$$N_{ed} = 1.5 \cdot (74.420\ \mathrm{N} + 929.475\ \mathrm{kN}) + 1.5 \cdot (956.250\ kN + 89.250\ kN \cdot 0.5 + \cdots$$

$$\ldots + 57.577\ \mathrm{kN} \cdot 0.6)$$

N_{ed} =2.947 MN ($Design\ Load$)

Column Base Geometry

t= 30.000 mm $\quad$ b_1= 450.000 mm $\quad$ d_1 = 450.000 mm

b = 500.000 mm $\quad$ d = 500.000 mm $\quad$ h = 500.000 mm

b_r= 425.000 mm $\quad$ f_y= 355.000 MPa $\quad$ E = 210.000 GPa

γ_0= 1.0 $\quad$ f_{ck}= 25.000 MPa $\quad$ γ_c= 1.5

Column Base Check

b_2= min (b_1+ 2 $\cdot b_r$, 3 $\cdot b_1$, b_1+ h)

b_2= min (450.000 mm + 2 ⋅ 425.000 mm, 3 ⋅ 450.000 mm ...,

, ... 450.000 mm + 500.000 mm)

b_2= 950.000 mm

d_2= min(d1 + 2 ⋅ br, 3 ⋅ d1, d1 + h)

d_2= min (450.000 mm + 2 ⋅ 425.000 mm, 3 ⋅ 450.000 mm ...,

... , 450.000 mm + 500.000 mm)

d_2= 950.000 mm

Stress concentration factor

$$k_j = \left(\frac{b_2 \cdot d_2}{b_1 \cdot d_1}\right)^{\left(\frac{1}{2}\right)} = \left(\frac{950.000\ \mathrm{mm} \cdot 950.000\ \mathrm{mm}}{450.000\ \mathrm{mm} \cdot 450.000\ \mathrm{mm}}\right)^{\left(\frac{1}{2}\right)} = 2.111$$

$\beta_j = \frac{2}{3} = 6.667 \mathrm{X}\ 10^{-1}$ (joint coeff.)

Design value of concrete concentrated pressure strength

$$f_{jd} = \frac{\beta_j \cdot k_j \cdot f_{ck}}{\gamma_c} = \frac{6.667 \mathrm{X}\ 10^{-1} \cdot 2.111 \cdot 25.000\ \mathrm{MPa}}{1.5} = 23.457\ \mathrm{MPa}$$

Effective width of flexible plate

$$c = t \cdot \left(\frac{f_y}{3 \cdot f_{jd} \cdot \gamma_{M0}}\right)^{\left(\frac{1}{2}\right)} = 30.000 \text{ mm} \cdot \left(\frac{355.000 \text{ MPa}}{3 \cdot 23.457 \text{ MPa} \cdot 1.0}\right)^{\left(\frac{1}{2}\right)} = 67.381 \text{ mm}$$

$A_{eff} = 234995.117\, mm^2$

Resistance of base plate

$N_{b_{Rd}} = A_{eff} \cdot f_{jd} = 234995.117\, mm^2 \cdot 23.457 \text{ MPa} = 5.512 \text{ MN}$

Reliability condition

$$\frac{N_{Ed}}{N_{b_{Rd}}} = \frac{2.947 \text{ MN}}{5.512 \text{ MN}} = 5.347\text{X } 10^{-1}$$

$$\frac{N_{Ed}}{N_{b_{Rd}}} = 0.5347 < 1.0\ Sadisfied$$

Minimum normal force in combination with horizontal force in column base

$$V_{Ed} = \gamma_Q \cdot q_w \cdot \left(\frac{L_{Total}}{2}\right) \cdot \left(\frac{H_{Total}}{2}\right)$$

$$V_{Ed} = 1.5 \cdot 320.000 \text{ Pa} \cdot \left(\frac{34.000 \text{ m}}{2}\right) \cdot \left(\frac{12.600\, m}{2}\right)$$

$V_{Ed} = 51.408 \text{ kN}$

Shear resistance of one bolt made of rod with diameter 20 mm, steel S355

f_{ub}= 490.000 MPa f_{yb}= 355.000 MPa $A_s = 245.000\, mm^2$

n_b= 2 α_v= 0.5 $\gamma_{M2} = 1.25$

α_b=3.335 X 10^{-1}

$F_{1vb_{Rd}} = 0.85 \cdot \frac{\alpha_v \cdot f_{ub} \cdot A_s}{\gamma_{M2}}$

$$F_{1vb_{Rd}} = 0.85 \cdot \frac{0.5 \cdot 490.000 \text{ MPa} \cdot 245.000\, mm^2}{1.25} = 40.817 \text{ kN}$$

$$F_{2vb_{Rd}} = \frac{\alpha_b \cdot f_{ub} \cdot A_s}{\gamma_{M2}}$$

$$F_{2vb_{Rd}} = \frac{3.335 \text{ X } 10^{-1} \cdot 490.000 \text{ MPa} \cdot 245.000\, mm^2}{1.25} = 32.029 \text{ kN}$$

$F_{vb_{Rd}} = \min\left(F_{1vb_{Rd}}, F_{2vb_{Rd}}\right) = \min(40.817 \text{ kN}, 32.029 \text{ kN}) = 32.029 \text{ kN}$

Resistance of two bolts in a column base

$V_{Rd} = n_b \cdot F_{vb_{Rd}} = 2 \cdot 32.029 \text{ kN} = 64.059 \text{ kN}$

Reliability condition

$$\frac{V_{Ed}}{V_{Rd}} = \frac{51.408 \text{ kN}}{64.059 \text{ kN}} = 8.025 \text{ X } 10^{-1}$$

$$\frac{V_{Ed}}{V_{Rd}} = 0.8025 < 1.0\ Satisfied$$

The chosen base plate dimensions are satisfied requirements.

2.2.3. Final design stage

Before a refined and detailed structural analysis can be carried out, it is necessary to make a final determination of the loads for which the structure is to be designed, including both loads determined from codes of practice and the self-weight of the structure. The analyse of whole structure is done by SCIA Engineer software. Drawings and diagrams are made in AutoCAD and Tekla Structure.

2.2.3.1. Geometry of structure

The structure is 111 meters long and 34 meters wide and column positions and grid plan is shown below.

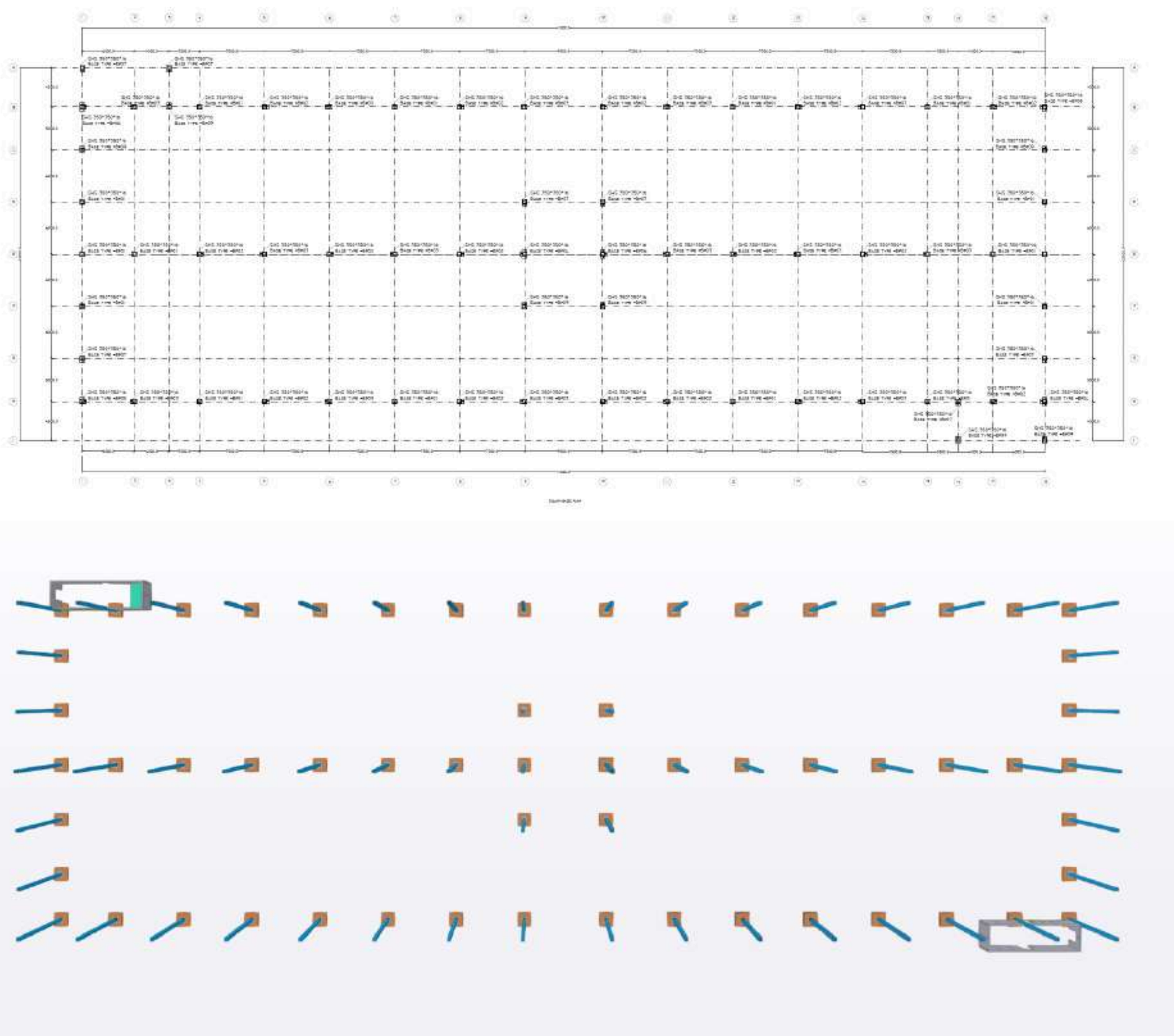


Figure 2.2.3.1. 1: Column positions and grids

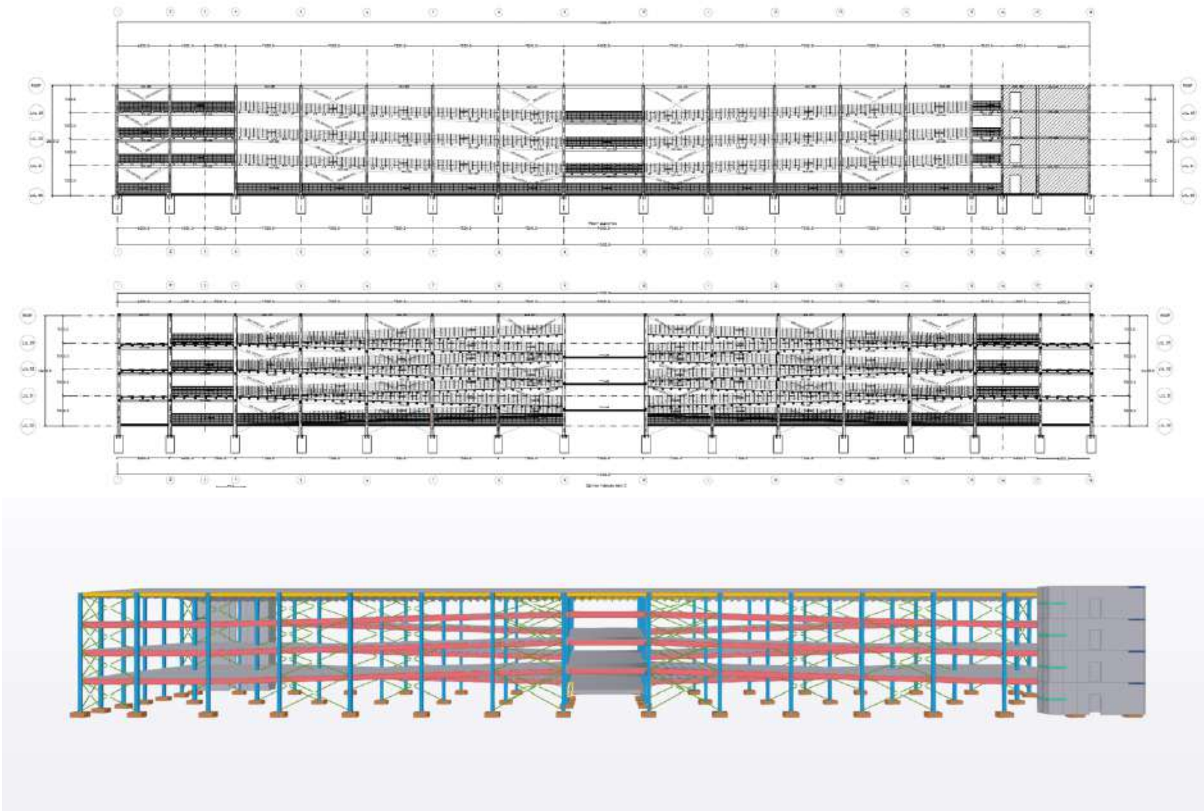


Figure 2.2.3.1. 2:Elevation of structure

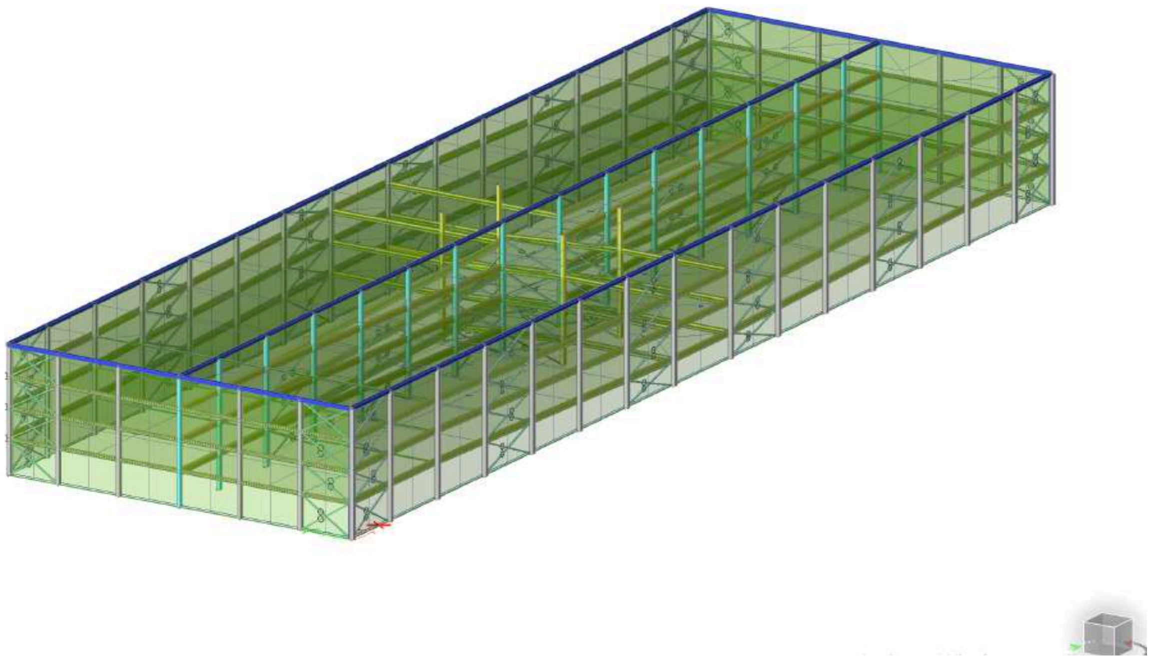


Figure 2.2.3.1. 3:Geometry of structure

2.2.3.2. Loads

After geometry of structure is defined, load cases should be applied the structure. There are eight load cases which are self-weight, dead load, live load, snow load and wind generator which is applied four different angles (0°,90°,180°,270°).

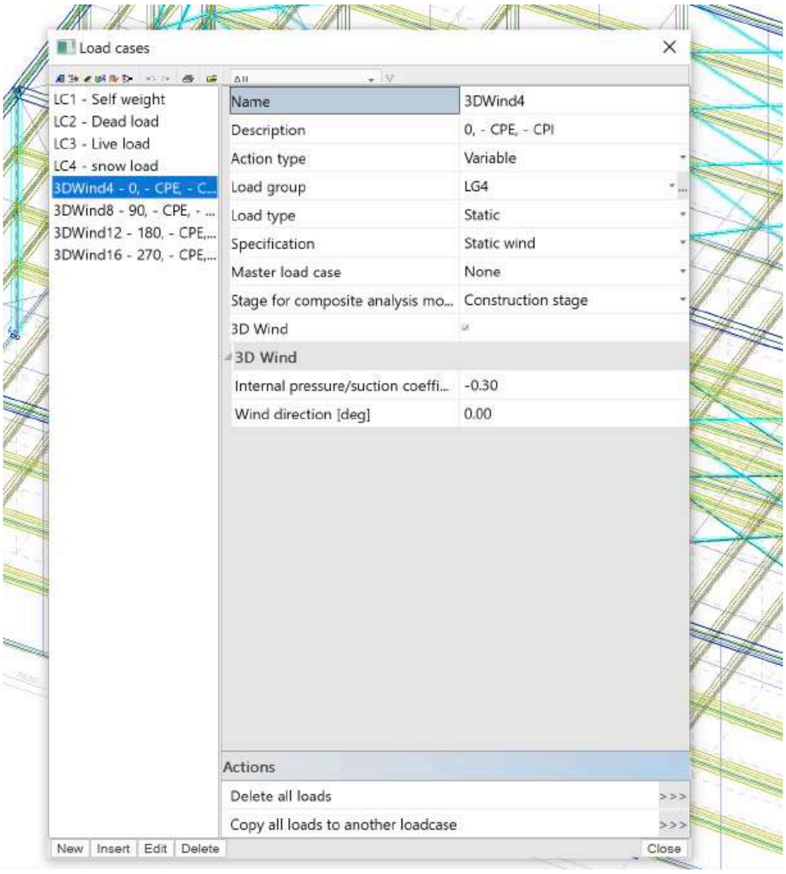


Figure 2.2.3.1. 4:Load cases

Each load case is shown below.

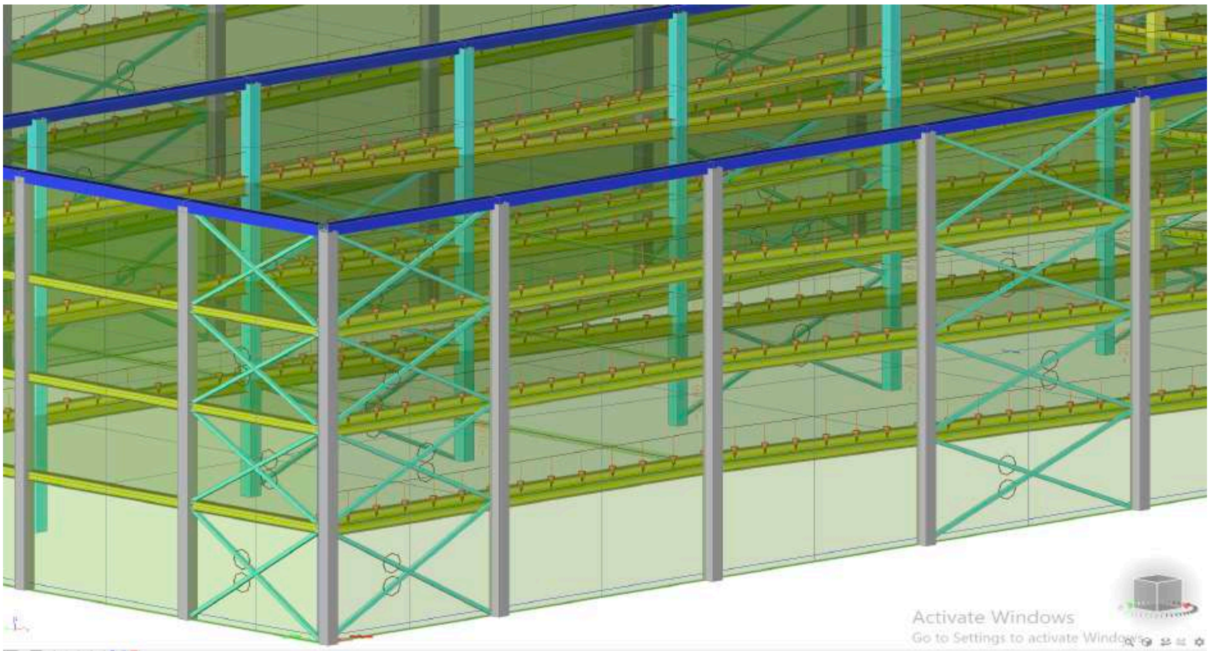


Figure 2.2.3.1. 5: Dead load

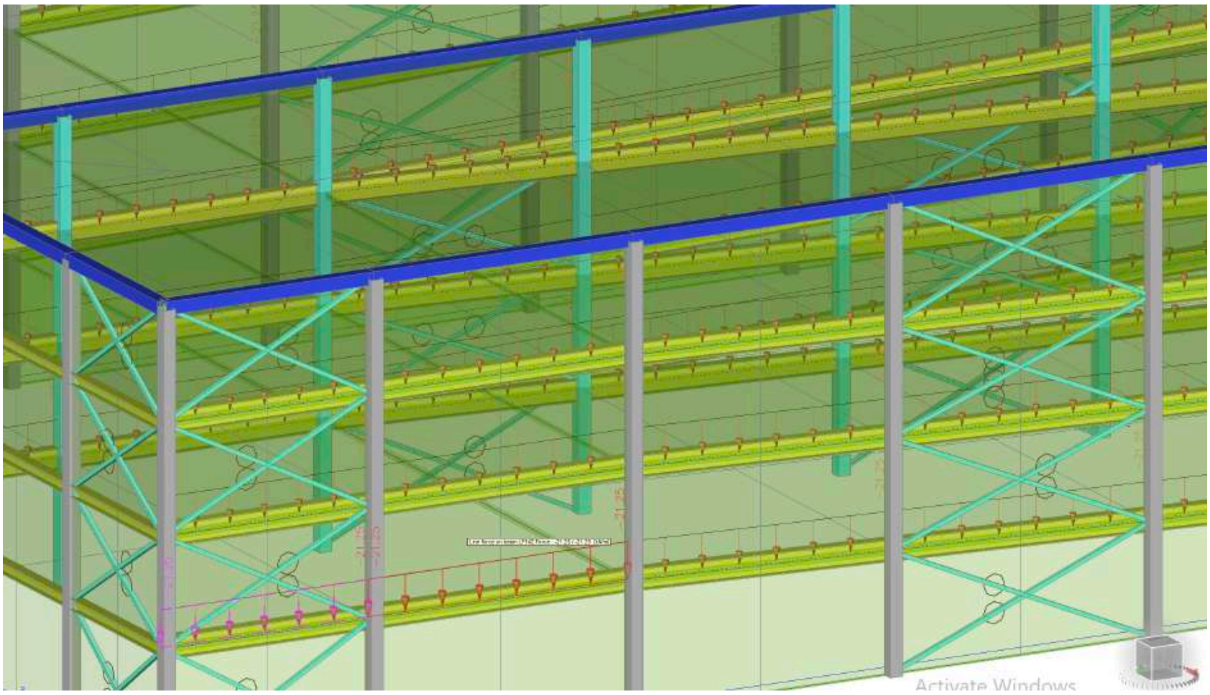


Figure 2.2.3.1. 6:Live load

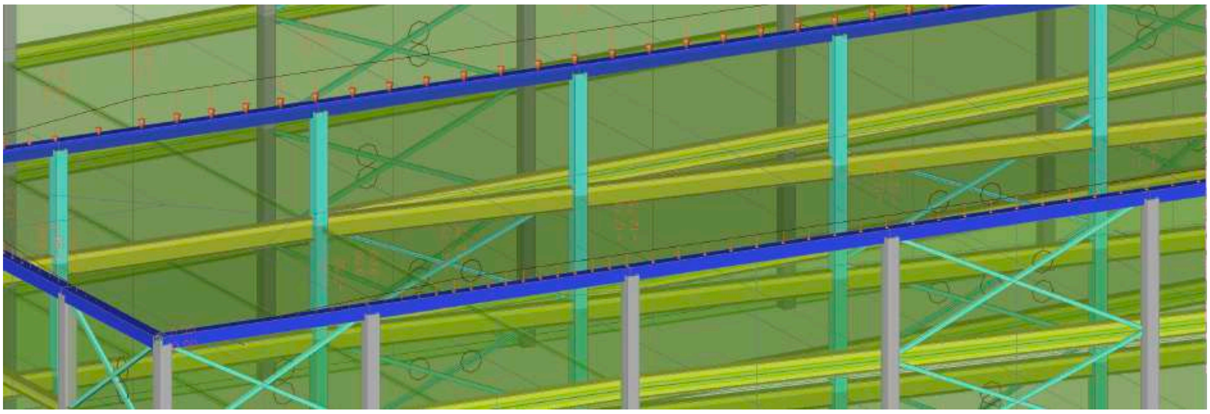


Figure 2.2.3.1. 7:Snow load

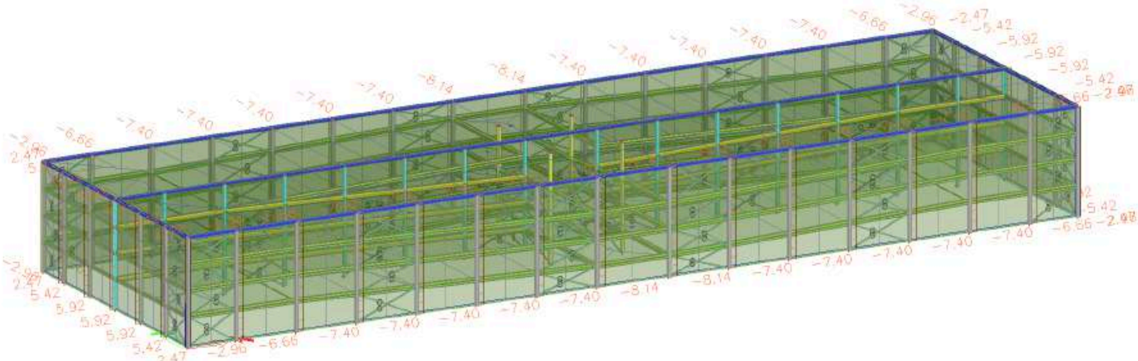


Figure 2.2.3.1. 8: 3D wind 0°

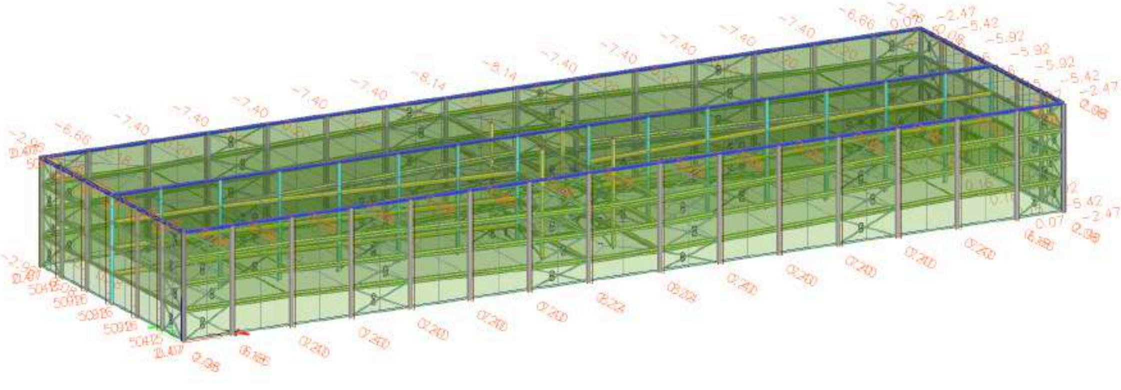


Figure 2.2.3.1. 9: 3D wind 90°

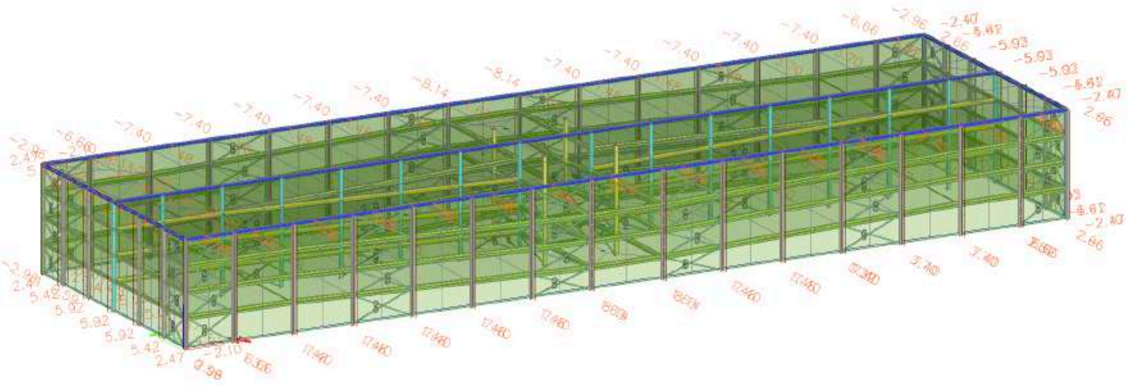


Figure 2.2.3.1. 10: 3D wind 180°

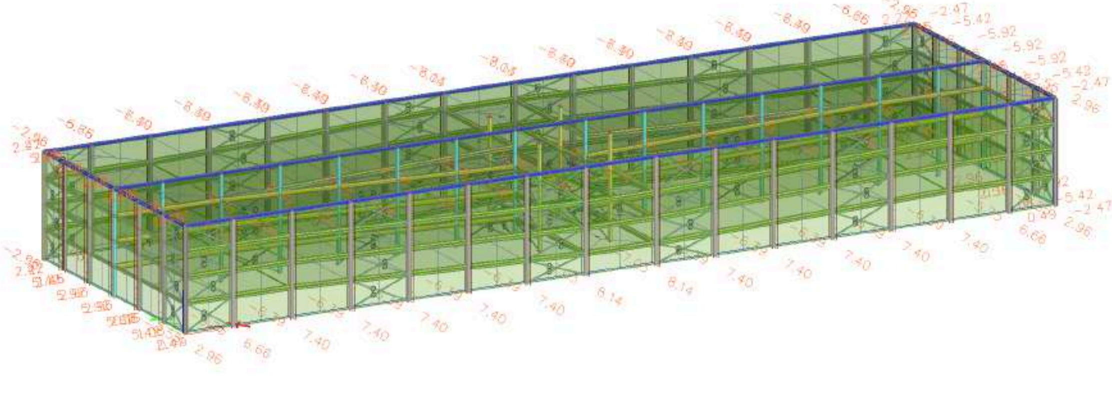


Figure 2.2.3.1. 11: 3D wind 270°

2.2.3.3. ULS Member Check

The ultimate limit state is the design for the safety of a structure and its users by limiting the stress that materials experience. In order to comply with engineering demands for strength and stability under design loads, ULS must be fulfilled as an established condition.

The necessity of checking sway mode of structure is important also it should be known that 'sensitive to buckling in a sway mode' does not mean the same as needing to consider second order effects due to the deformation of the structure. It means only that the geometrical deformation of the structure gives rise to additional effects in the members that must be taken into account in design. These additional effects may be only first order effects. If the geometrical deformation significantly affects the structural behaviour then second order effects also need to be considered.

First sway imperfection needs to be evaluated.

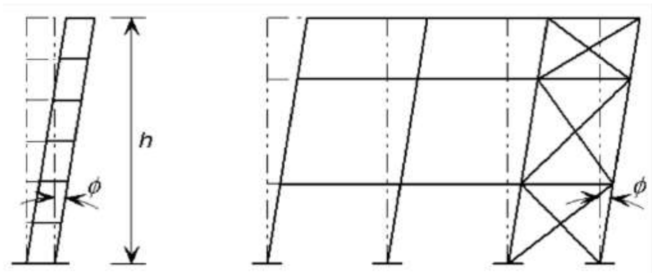


Figure 2.2.3.3. 1:Sway imperfection

$\Phi = \Phi_0 \; x \; \alpha_h \; x \alpha_m$

$\Phi_0 = \frac{1}{200} = 0.005$

$\alpha_h = \frac{2}{(h)^{\left(\frac{1}{2}\right)}} = \frac{2}{(12.6)^{\left(\frac{1}{2}\right)}} = 0.563$ but $0.666 \leq \;\; \alpha_h \leq 1.0$; $\alpha_h = 0.666$

$$\alpha_m = \sqrt{0.5\left(1 + \frac{1}{m}\right)} = \sqrt{0.5\left(1 + \frac{1}{3}\right)} = 0.816$$

$\Phi = \Phi_0 \; x \; \alpha_h \; x \alpha_m = 2.72 x \; 10^{-3}$

Equivalent horizontal load

$H_{Ed} = \Phi x \; g \; x \; l \; = 2.72 x \; 10^{-3} x 64.57 x 34 = 5.04 \; kN$

$\Sigma H_{Ed} = 5.04 + 57.577 = 62.617 \; kN$

$$\alpha_{cr} = \frac{\Sigma H_{Ed}}{\Sigma V_{Ed}} x \frac{h}{\delta_{H,Ed}} = \frac{62.617}{2195} x \frac{12600}{24} = 14.97$$

$\alpha_{cr} > 10$, therefore 1st order linear analysis will be taken into account. [3]

Name	dx [m]	Case	uy [mm]	uy,rel [1/xx]	uz [mm]	uz,rel [1/xx]
By4_5	6	ULS-Set B (auto)/1	-16.9	-1/356	0	0
By41_3	0	ULS-Set B (auto)/2	24	1/136	0	0
B1443	8.500+	ULS-Set B (auto)/3	0.5	1/10000	-30	-1/283
B601	4.500-	ULS-Set B (auto)/4	0	0	-22.1	-1/173
F48_2	5.883	ULS-Set B (auto)/5	0	0	31	1/190

Table 2.2.3.3. 1:Relative deformation(extreme)

2.2.3.3.2. Beam Check

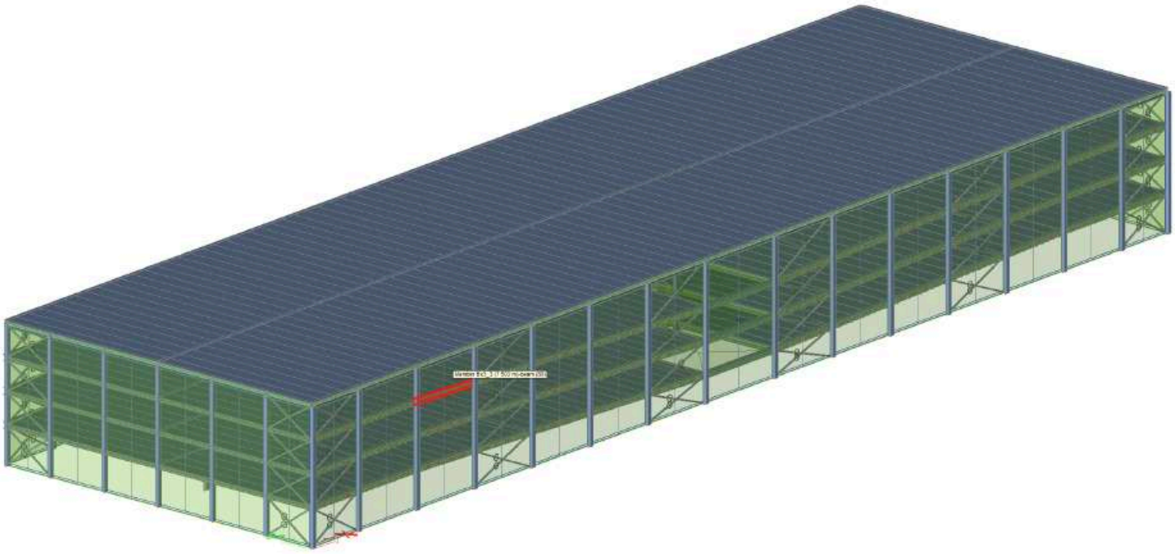


Figure 2.2.3.3. 2:Critical beam

EC-EN 1993 Steel check ULS

Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Global
Selection: All
Filter: Cross-section = CS12-Beam2 - HEA600

Student version

EN 1993-1-1 Code Check
National annex: Czech CSN-EN NA

Member Bx3_3	7.503 / 7.503 m	HEA600	S 355	ULS-Set B (auto)	0.83 -

Combination key
ULS-Set B (auto) / 1.15*LC1 + 1.15*LC2 + 1.50*LC3 + 1.50*LC4 + 0.90*3DWind16

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{M2} for resistance of net sections	1.25

Material			
Yield strength	f_y	355.0	MPa
Ultimate strength	f_u	490.0	MPa
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 7.503 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-61.17	kN
Shear force	$V_{y,Ed}$	-1.33	kN
Shear force	$V_{z,Ed}$	-263.12	kN
Torsion	T_{Ed}	0.11	kNm
Bending moment	$M_{y,Ed}$	-354.01	kNm
Bending moment	$M_{z,Ed}$	-9.95	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Student version

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	116	25	7.647e+04	8.676e+04	0.9	0.4	1.0	4.7	7.3	8.1	11.3	1
3	SO	116	25	7.055e+04	6.027e+04	0.9	0.5	1.0	4.7	7.3	8.1	11.9	1
4	I	486	13	6.361e+04	-5.821e+04	-0.9		0.5	37.4	56.2	65.0	92.2	1
5	SO	116	25	-7.107e+04	-8.136e+04								
7	SO	116	25	-6.515e+04	-5.487e+04								

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	2.2700e-02	m²
Compression resistance	$N_{c,Rd}$	8058.50	kN
Unity check		0.01	-

$$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M0}} = \frac{2.2700 \cdot 10^{-2}[m^2] \times 355.0[MPa]}{1.00} = 8058.50[kN] \quad \text{(EC3-1-1: 6.10)}$$

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{c,Rd}} = \frac{|-61.17[kN]|}{8058.50[kN]} = \mathbf{0.01 \leq 1.00} \quad \text{(EC3-1-1: 6.9)}$$

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	5.3333e-03	m³
Plastic bending moment	$M_{pl,y,Rd}$	1893.33	kNm
Unity check		0.19	-

$$M_{pl,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{5.3333 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 1893.33[kNm] \quad \text{(EC3-1-1: 6.13)}$$

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{pl,y,Rd}} = \frac{|-354.01[kNm]|}{1893.33[kNm]} = \mathbf{0.19 \leq 1.00} \quad \text{(EC3-1-1: 6.12)}$$

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Student version

Plastic section modulus	$W_{pl,z}$	1.1542e-03	m³
Plastic bending moment	$M_{pl,z,Rd}$	409.73	kNm
Unity check		0.02	-

$$M_{pl,z,Rd} = \frac{W_{pl,z} \times f_y}{\gamma_{M0}} = \frac{1.1542 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 409.73[kNm] \quad \text{(EC3-1-1: 6.13)}$$

$$\text{Unity check} = \frac{|M_{z,Ed}|}{M_{pl,z,Rd}} = \frac{|-9.95[kNm]|}{409.73[kNm]} = \mathbf{0.02 \leq 1.00} \quad \text{(EC3-1-1: 6.12)}$$

Student version

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1.20	
Shear area	A_v	1.5520e-02	m^2
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	3180.97	kN
Unity check		0.00	-

$$V_{pl,y,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{1.5520 \cdot 10^{-2}[m^2] \times \frac{355.0[MPa]}{\sqrt{3}}}{1.00} = 3180.97[kN] \quad \text{(EC3-1-1: 6.18)}$$

$$\text{Unity check} = \frac{|V_{y,Ed}|}{V_{c,y,Rd}} = \frac{|-1.33[kN]|}{3180.97[kN]} = \mathbf{0.00 \leq 1.00} \quad \text{(EC3-1-1: 6.17)}$$

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1.20	
Shear area	A_v	9.3750e-03	m^2
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	1921.49	kN
Unity check		0.14	-

$$V_{pl,z,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{9.3750 \cdot 10^{-3}[m^2] \times \frac{355.0[MPa]}{\sqrt{3}}}{1.00} = 1921.49[kN] \quad \text{(EC3-1-1: 6.18)}$$

$$\text{Unity check} = \frac{|V_{z,Ed}|}{V_{c,z,Rd}} = \frac{|-263.12[kN]|}{1921.49[kN]} = \mathbf{0.14 \leq 1.00} \quad \text{(EC3-1-1: 6.17)}$$

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	2	
Total torsional moment	τ_{Ed}	0.7	MPa
Elastic shear resistance	τ_{Rd}	205.0	MPa
Unity check		0.00	-

Student version

$$\tau_{Ed} = |T_{Ed}| \times \tau_{Ed,unit} = |105.45| \times 6.284[kN/m^2] = 0.7[MPa]$$

$$\tau_{Rd} = \frac{f_y}{\sqrt{3} \times \gamma_{M0}} = \frac{355.0[MPa]}{\sqrt{3} \times 1.00} = 205.0[MPa]$$

$$\text{Unity check} = \frac{\tau_{Ed}}{\tau_{Rd}} = \frac{0.7[MPa]}{205.0[MPa]} = \mathbf{0.00 \leq 1.00} \quad \text{(EC3-1-1: 6.23)}$$

Note: The unity check for torsion is lower than the limit value of 0.05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Plastic bending moment	$M_{pl,y,Rd}$	1893.33	kNm
Exponent of bending ratio y	α	2.00	
Plastic bending moment	$M_{pl,z,Rd}$	409.73	kNm
Exponent of bending ratio z	β	1.00	

Unity check (6.41) = 0.03 + 0.02 = 0.06 -

$$M_{pl,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{5.3333 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 1893.33[kNm] \quad \text{(EC3-1-1: 6.13)}$$

$$\alpha = 2.00$$

$$M_{pl,z,Rd} = \frac{W_{pl,z} \times f_y}{\gamma_{M0}} = \frac{1.1542 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 409.73[kNm] \quad \text{(EC3-1-1: 6.13)}$$

$$\beta = 1.00$$

$$\text{Unity check} = \left(\frac{|M_{y,Ed}|}{M_{pl,y,Rd}}\right)^{\alpha} + \left(\frac{|M_{z,Ed}|}{M_{pl,z,Rd}}\right)^{\beta} = \left(\frac{|-354.01[kNm]|}{1893.33[kNm]}\right)^{2.00} + \left(\frac{|-9.95[kNm]|}{409.73[kNm]}\right)^{1.00} = \mathbf{0.06 \leq 1.00} \quad \text{(EC3-1-1: 6.41)}$$

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.
Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.
Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the z-z axis is neglected.

The member satisfies the section check.

Student version

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 0.000 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	116	25	2.142e+03	2.142e+03	1.0	0.4	1.0	4.7	7.3	8.1	11.4	1
3	SO	116	25	2.142e+03	2.142e+03	1.0	0.4	1.0	4.7	7.3	8.1	11.4	1
4	I	486	13	2.142e+03	2.142e+03	1.0		1.0	37.4	22.8	27.7	30.9	4
5	SO	116	25	2.142e+03	2.142e+03	1.0	0.4	1.0	4.7	7.3	8.1	11.4	1
7	SO	116	25	2.142e+03	2.142e+03	1.0	0.4	1.0	4.7	7.3	8.1	11.4	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 4

Effective section N-
Effective width calculation
According to EN 1993-1-5 article 4.4

Id	Type	b_p [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	λ_p [-]	ρ [-]	b_e [mm]	b_{e1} [mm]	b_{e2} [mm]
1	SO	116	3.550e+05	3.550e+05	1.0	0.4	0.3	1.0	116		
3	SO	116	3.550e+05	3.550e+05	1.0	0.4	0.3	1.0	116		
4	I	486	3.550e+05	3.550e+05	1.0	4.0	0.8	0.9	437	219	219
5	SO	116	3.550e+05	3.550e+05	1.0	0.4	0.3	1.0	116		
7	SO	116	3.550e+05	3.550e+05	1.0	0.4	0.3	1.0	116		

Effective section My-
Effective width calculation
According to EN 1993-1-5 article 4.4

Id	Type	b_p [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	λ_p [-]	ρ [-]	b_e [mm]	b_{e1} [mm]	b_{e2} [mm]
1	SO	116	3.550e+05	3.550e+05	1.0	0.4	0.3	1.0	116		
3	SO	116	3.550e+05	3.550e+05	1.0	0.4	0.3	1.0	116		
4	I	486	3.054e+05	-3.054e+05	-1.0	23.9	0.3	1.0	243	97	146
5	SO	116	-3.550e+05	-3.550e+05							
7	SO	116	-3.550e+05	-3.550e+05							

Effective section Mz-
Effective width calculation
According to EN 1993-1-5 article 4.4

Student version

Student version

Effective section Mz-
Effective width calculation
According to EN 1993-1-5 article 4.4

Id	Type	b_p [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	λ_p [-]	ρ [-]	b_e [mm]	b_{e1} [mm]	b_{e2} [mm]
1	SO	116	3.550e+05	7.928e+04	0.2	0.5	0.3	1.0	116		
3	SO	116	-7.928e+04	-3.550e+05							
4	I	486	0.000e+00	0.000e+00							
5	SO	116	-7.928e+04	-3.550e+05							
7	SO	116	3.550e+05	7.928e+04	0.2	0.5	0.3	1.0	116		

Effective properties						
Effective area	A_{eff}	2.2017e-02	m²			
Effective second moment of area	$I_{eff,y}$	1.4123e-03	m⁴	$I_{eff,z}$	1.1271e-04	m⁴
Effective section modulus	$W_{eff,y}$	4.7874e-03	m³	$W_{eff,z}$	7.5143e-04	m³
Shift of the centroid	$e_{N,y}$	0	mm	$e_{N,z}$	0	mm

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	7.503	22.510	m
Buckling factor	k	3.24	1.00	
Buckling length	l_{cr}	24.310	22.509	m
Critical Euler load	N_{cr}	4945.03	462.25	kN
Slenderness	λ	97.54	319.03	
Relative slenderness	λ_{rel}	1.26	4.11	
Limit slenderness	$\lambda_{rel,0}$	0.20	0.20	
Buckling curve		a	b	
Imperfection	α	0.21	0.34	
Reduction factor	χ	0.50	0.05	
Buckling resistance	$N_{b,Rd}$	3868.90	426.75	kN

Flexural Buckling verification			
Cross-section effective area	A_{eff}	2.2017e-02	m²
Buckling resistance	$N_{b,Rd}$	426.75	kN
Unity check		0.14	-

$$N_{cr,y} = \frac{\pi^2 \times E \times I_y}{l_{cr,y}^2} = \frac{\pi^2 \times 210000.0[\text{MPa}] \times 1.4100 \cdot 10^{-3}[\text{m}^4]}{24.310[\text{m}]^2} = 4945.03[\text{kN}]$$

$$N_{cr,z} = \frac{\pi^2 \times E \times I_z}{l_{cr,z}^2} = \frac{\pi^2 \times 210000.0[\text{MPa}] \times 1.1300 \cdot 10^{-4}[\text{m}^4]}{22.509[\text{m}]^2} = 462.25[\text{kN}]$$

$$\lambda_y = \frac{l_{cr,y}}{i_y} = \frac{24.310[\text{m}]}{249[\text{mm}]} = 97.54$$

$$\lambda_z = \frac{l_{cr,z}}{i_z} = \frac{22.509[\text{m}]}{71[\text{mm}]} = 319.03$$

$$\lambda_{rel,y} = \frac{\lambda_y \times \sqrt{\frac{A_{eff}}{A}}}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{97.54 \times \sqrt{\frac{2.2017 \cdot 10^{-2}[\text{m}^2]}{2.2700 \cdot 10^{-2}[\text{m}^2]}}}{\pi \times \sqrt{\frac{210000.0[\text{MPa}]}{355.0[\text{MPa}]}}} = 1.26 \qquad \text{(EC3-1-1: 6.51)}$$

$$\lambda_{rel,z} = \frac{\lambda_z \times \sqrt{\frac{A_{eff}}{A}}}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{319.03 \times \sqrt{\frac{2.2017 \cdot 10^{-2}[\text{m}^2]}{2.2700 \cdot 10^{-2}[\text{m}^2]}}}{\pi \times \sqrt{\frac{210000.0[\text{MPa}]}{355.0[\text{MPa}]}}} = 4.11 \qquad \text{(EC3-1-1: 6.51)}$$

$$\varphi_y = 0.5 \times \left[1 + \alpha_y \times (\lambda_{rel,y} - \lambda_{rel,y,0}) + \lambda_{rel,y}^2\right] = 0.5 \times \left[1 + 0.21 \times (1.26 - 0.20) + 1.26^2\right] = 1.40$$

$$\varphi_z = 0.5 \times \left[1 + \alpha_z \times (\lambda_{rel,z} - \lambda_{rel,z,0}) + \lambda_{rel,z}^2\right] = 0.5 \times \left[1 + 0.34 \times (4.11 - 0.20) + 4.11^2\right] = 9.62$$

$$\chi_y = \min\left(\frac{1}{\varphi_y + \sqrt{\varphi_y^2 - \lambda_{rel,y}^2}}, \frac{1}{\lambda_{rel,y}^2}, 1\right) = \min\left(\frac{1}{1.40 + \sqrt{1.40^2 - 1.26^2}}, \frac{1}{1.26^2}, 1\right) = \min(0.50, 0.63, 1) = 0.50 \qquad \text{(EC3-1-1: 6.49)}$$

$$\chi_z = \min\left(\frac{1}{\varphi_z + \sqrt{\varphi_z^2 - \lambda_{rel,z}^2}}, \frac{1}{\lambda_{rel,z}^2}, 1\right) = \min\left(\frac{1}{9.62 + \sqrt{9.62^2 - 4.11^2}}, \frac{1}{4.11^2}, 1\right) = \min(0.05, 0.06, 1) = 0.05 \qquad \text{(EC3-1-1: 6.49)}$$

$$N_{b,y,Rd} = \frac{\chi_y \times A_{eff} \times f_y}{\gamma_{M1}} = \frac{0.50 \times 2.2017 \cdot 10^{-2}[\text{m}^2] \times 355.0[\text{MPa}]}{1.00} = 3868.90[\text{kN}] \qquad \text{(EC3-1-1: 6.48)}$$

$$N_{b,z,Rd} = \frac{\chi_z \times A_{eff} \times f_y}{\gamma_{M1}} = \frac{0.05 \times 2.2017 \cdot 10^{-2}[\text{m}^2] \times 355.0[\text{MPa}]}{1.00} = 426.75[\text{kN}] \qquad \text{(EC3-1-1: 6.48)}$$

$$N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}) = \min(3868.90[kN], 426.75[kN]) = 426.75[kN]$$

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{b,Rd}} = \frac{|-61.17[kN]|}{426.75[kN]} = \mathbf{0.14 \leq 1.00} \quad \text{(EC3-1-1: 6.46)}$$

Torsional(-Flexural) Buckling check

According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Note: For this I-section the Torsional(-Flexural) buckling resistance is higher than the resistance for Flexural buckling. Therefore Torsional(-Flexural) buckling is not printed on the output.

Lateral Torsional Buckling check

According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.3 and formula (6.54)

LTB parameters			
Method for LTB curve		Alternative case	
Effective section modulus	$W_{eff,y}$	4.7874e-03	m^3
Elastic critical moment	M_{cr}	548.50	kNm
Relative slenderness	$\lambda_{rel,LT}$	1.76	
Limit slenderness	$\lambda_{rel,LT,0}$	0.40	
LTB curve		b	
Imperfection	α_{LT}	0.34	
LTB factor	β	0.75	
Reduction factor	χ_{LT}	0.32	
Correction factor	k_c	0.86	
Correction factor	f	1.00	
Modified reduction factor	$\chi_{LT,mod}$	0.32	
Design buckling resistance	$M_{b,Rd}$	548.50	kNm
Unity check		0.65	-

Mcr parameters			
LTB length	l_{LT}	22.510	m
Influence of load position		no influence	
Correction factor	k	1.00	
Correction factor	k_w	1.00	
LTB moment factor	C_1	1.35	
LTB moment factor	C_2	0.63	
LTB moment factor	C_3	0.41	

Student version

Mcr parameters			
Shear centre distance	d_z	0	mm
Distance of load application	z_g	0	mm
Mono-symmetry constant	β_y	0	mm
Mono-symmetry constant	z_j	0	mm

Student version

$$M_{cr} = C_1 \times \frac{\pi^2 \times E \times I_z}{l_{LT}^2} \times \left[\sqrt{\left(\frac{k}{k_w}\right)^2 \times \frac{I_w}{I_z} + \frac{l_{LT}^2 \times G \times I_t}{\pi^2 \times E \times I_z} + (C_2 \times z_g - C_3 \times z_j)^2} - (C_2 \times z_g - C_3 \times z_j)\right] = 1.35$$

$$\times \frac{\pi^2 \times 210000.0[MPa] \times 1.1300 \cdot 10^{-4}[m^4]}{22.510[m]^2}$$

$$\times \left[\sqrt{\left(\frac{1.00}{1.00}\right)^2 \times \frac{8.9782 \cdot 10^{-6}[m^6]}{1.1300 \cdot 10^{-4}[m^4]} + \frac{22.510[m]^2 \times 80769.2[MPa] \times 3.9800 \cdot 10^{-6}[m^4]}{\pi^2 \times 210000.0[MPa] \times 1.1300 \cdot 10^{-4}[m^4]} + (0.63 \times 0[mm] - 0.41 \times 0[mm])^2} - (0.63 \times 0[mm] - 0.41 \times 0[mm])\right]$$

$$= 548.50[kNm]$$

$$\lambda_{rel,LT} = \sqrt{\frac{W_{eff,y} \times f_y}{M_{cr}}} = \sqrt{\frac{4.7874 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{548.50[kNm]}} = 1.76$$

$$\beta = 0.75$$

$$\chi_{LT} = \min\left(\frac{1}{\varphi_{LT} + \sqrt{\varphi_{LT}^2 - \beta \times \lambda_{rel,LT}^2}}, \frac{1}{\lambda_{rel,LT}^2}, 1\right) = \min\left(\frac{1}{1.89 + \sqrt{1.89^2 - 0.75 \times 1.76^2}}, \frac{1}{1.76^2}, 1\right) = \min(0.33, 0.32, 1) = 0.32 \quad \text{(EC3-1-1: 6.57)}$$

$$f = \min\left\{1 - 0.5 \times (1 - k_c) \times \left[1 - 2 \times (\lambda_{rel,LT} - 0.8)^2\right], 1\right\} = \min\left\{1 - 0.5 \times (1 - 0.86) \times \left[1 - 2 \times (1.76 - 0.8)^2\right], 1\right\} = \min\{1.06, 1\} = 1.00$$

$$\chi_{LT,mod} = \min\left(\frac{\chi_{LT}}{f}, 1\right) = \min\left(\frac{0.32}{1.00}, 1\right) = \min(0.32, 1) = 0.32$$

$$M_{b,Rd} = \chi_{LT,mod} \times W_{eff,y} \times \frac{f_y}{\gamma_{M1}} = 0.32 \times 4.7874 \cdot 10^{-3}[m^3] \times \frac{355.0[MPa]}{1.00} = 548.50[kNm] \quad \text{(EC3-1-1: 6.55)}$$

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{b,Rd}} = \frac{|-354.01[kNm]|}{548.50[kNm]} = \mathbf{0.65 \leq 1.00} \quad \text{(EC3-1-1: 6.54)}$$

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Note: The correction factor k_c is determined from C_1.

Bending and axial compression check

According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section effective area	A_{eff}	2.2017e-02	m^2
Effective section modulus	$W_{eff,y}$	4.7874e-03	m^3
Effective section modulus	$W_{eff,z}$	7.5143e-04	m^3
Design compression force	N_{Ed}	61.17	kN
Design bending moment (maximum)	$M_{y,Ed}$	-354.01	kNm
Design bending moment (maximum)	$M_{z,Ed}$	-12.65	kNm
Additional moment	$\Delta M_{y,Ed}$	0.00	kNm
Additional moment	$\Delta M_{z,Ed}$	0.00	kNm
Characteristic compression resistance	N_{Rk}	7815.95	kN
Characteristic moment resistance	$M_{y,Rk}$	1699.52	kNm
Characteristic moment resistance	$M_{z,Rk}$	266.76	kNm
Reduction factor	χ_y	0.50	
Reduction factor	χ_z	0.05	
Modified reduction factor	$\chi_{LT,mod}$	0.32	
Interaction factor	k_{yy}	0.91	
Interaction factor	k_{yz}	0.98	
Interaction factor	k_{zy}	0.99	
Interaction factor	k_{zz}	0.98	

Maximum moment $M_{y,Ed}$ is derived from beam Bx3_3 position 7.503 m.
Maximum moment $M_{z,Ed}$ is derived from beam Bx5_3 position 0.000 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.2	
Sway type y		sway	
Equivalent moment factor	C_{my}	0.90	
Resulting load type z		point load F	
End moment	$M_{h,z}$	0.00	kNm
Field moment	$M_{s,z}$	-12.65	kNm
Factor	$\alpha_{h,z}$	0.00	
Ratio of end moments	ψ_z	1.00	
Equivalent moment factor	C_{mz}	0.90	
Resulting load type LT		point load F	
End moment	$M_{h,LT}$	0.00	kNm
Field moment	$M_{s,LT}$	-354.01	kNm
Factor	$\alpha_{h,LT}$	0.00	
Ratio of end moments	ψ_{LT}	1.00	
Equivalent moment factor	C_{mLT}	0.90	

Unity check (6.61) = 0.02 + 0.59 + 0.05 = 0.65 -
Unity check (6.62) = 0.14 + 0.64 + 0.05 = 0.83 -

$$C_{my} = 0.90$$

$$\alpha_{h,z} = \frac{M_{h,z}}{M_{s,z}} = \frac{0.00[\mathrm{kNm}]}{-12.65[\mathrm{kNm}]} = 0.00$$

$$C_{mz} = 0.9 + 0.1 \times \alpha_{h,z} = 0.9 + 0.1 \times 0.00 = 0.90$$

$$\alpha_{h,LT} = \frac{M_{h,LT}}{M_{s,LT}} = \frac{0.00[\mathrm{kNm}]}{-354.01[\mathrm{kNm}]} = 0.00$$

$$C_{mLT} = 0.9 + 0.1 \times \alpha_{h,LT} = 0.9 + 0.1 \times 0.00 = 0.90$$

$$N_{Rk} = A_{eff} \times f_y = 2.2017 \cdot 10^{-2}[\mathrm{m^2}] \times 355.0[\mathrm{MPa}] = 7815.95[\mathrm{kN}]$$

$$M_{y,Rk} = W_{eff,y} \times f_y = 4.7874 \cdot 10^{-3}[\mathrm{m^3}] \times 355.0[\mathrm{MPa}] = 1699.52[\mathrm{kNm}]$$

$$M_{z,Rk} = W_{eff,z} \times f_y = 7.5143 \cdot 10^{-4}[\mathrm{m^3}] \times 355.0[\mathrm{MPa}] = 266.76[\mathrm{kNm}]$$

$$k_{yy} = \min\left[C_{my} \times \left(1 + 0.6 \times \lambda_{rel,y} \times \frac{N_{Ed}}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}}\right), C_{my} \times \left(1 + 0.6 \times \frac{N_{Ed}}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}}\right)\right]$$

$$= \min\left[0.90 \times \left(1 + 0.6 \times 1.26 \times \frac{61.17[\mathrm{kN}]}{0.50 \times \frac{7815.95[\mathrm{kN}]}{1.00}}\right), 0.90 \times \left(1 + 0.6 \times \frac{61.17[\mathrm{kN}]}{0.50 \times \frac{7815.95[\mathrm{kN}]}{1.00}}\right)\right] = \min[0.91, 0.91] = 0.91$$

$$k_{yz} = k_{zz} = 0.98$$

$$k_{zy} = \max\left(1 - \frac{0.05 \times \lambda_{rel,z}}{C_{mLT} - 0.25} \times \frac{N_{Ed}}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}}, 1 - \frac{0.05}{C_{mLT} - 0.25} \times \frac{N_{Ed}}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}}\right)$$

$$= \max\left(1 - \frac{0.05 \times 4.11}{0.90 - 0.25} \times \frac{61.17[\mathrm{kN}]}{0.05 \times \frac{7815.95[\mathrm{kN}]}{1.00}}, 1 - \frac{0.05}{0.90 - 0.25} \times \frac{61.17[\mathrm{kN}]}{0.05 \times \frac{7815.95[\mathrm{kN}]}{1.00}}\right) = \max(0.95, 0.99) = 0.99$$

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$$k_{zz} = \min\left[C_{mz} \times \left(1 + 0.6 \times \lambda_{rel,z} \times \frac{N_{Ed}}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}}\right), C_{mz} \times \left(1 + 0.6 \times \frac{N_{Ed}}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}}\right)\right]$$

$$= \min\left[0.90 \times \left(1 + 0.6 \times 4.11 \times \frac{61.17[kN]}{0.05 \times \frac{7815.95[kN]}{1.00}}\right), 0.90 \times \left(1 + 0.6 \times \frac{61.17[kN]}{0.05 \times \frac{7815.95[kN]}{1.00}}\right)\right] = \min[1.22, 0.98] = 0.98$$

$$\text{Unity check (6.61)} = \frac{|N_{Ed}|}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \times \frac{|M_{y,Ed}| + |\Delta M_{y,Ed}|}{\chi_{LT,mod} \times \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \times \frac{|M_{z,Ed}| + |\Delta M_{z,Ed}|}{\frac{M_{z,Rk}}{\gamma_{M1}}}$$

$$= \frac{|61.17[kN]|}{0.50 \times \frac{7815.95[kN]}{1.00}} + 0.91 \times \frac{|-354.01[kNm]| + |0.00[kNm]|}{0.32 \times \frac{1699.52[kNm]}{1.00}} + 0.98 \times \frac{|-12.65[kNm]| + |0.00[kNm]|}{\frac{266.76[kNm]}{1.00}} = \mathbf{0.65} \leq \mathbf{1.00} \qquad \text{(EC3-1-1: 6.61)}$$

$$\text{Unity check (6.62)} = \frac{|N_{Ed}|}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} \times \frac{|M_{y,Ed}| + |\Delta M_{y,Ed}|}{\chi_{LT,mod} \times \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \times \frac{|M_{z,Ed}| + |\Delta M_{z,Ed}|}{\frac{M_{z,Rk}}{\gamma_{M1}}}$$

$$= \frac{|61.17[kN]|}{0.05 \times \frac{7815.95[kN]}{1.00}} + 0.99 \times \frac{|-354.01[kNm]| + |0.00[kNm]|}{0.32 \times \frac{1699.52[kNm]}{1.00}} + 0.98 \times \frac{|-12.65[kNm]| + |0.00[kNm]|}{\frac{266.76[kNm]}{1.00}} = \mathbf{0.83} \leq \mathbf{1.00} \qquad \text{(EC3-1-1: 6.62)}$$

$$\text{Unity check} = \max(\text{Unity check (6.61)}, \text{Unity check (6.62)}) = \max(0.65, 0.83) = \mathbf{0.83} \leq \mathbf{1.00}$$

Shear Buckling check
According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

Shear Buckling parameters			
Buckling field length	a	7.503	m
Web		unstiffened	
Web height	h_w	540	mm
Web thickness	t	13	mm
Material coefficient	ε	0.81	
Shear correction factor	η	1.20	

Shear Buckling verification		
Web slenderness	h_w/t	41.54
Web slenderness limit		48.82

$$h_w/t = \frac{h_w}{t} = \frac{540[mm]}{13[mm]} = 41.54$$

$$\text{limit } h_w/t = \frac{72 \times \varepsilon}{\eta} = \frac{72 \times 0.81}{1.20} = 48.82$$

Note: The web slenderness is such that Shear Buckling effects may be ignored according to EN 1993-1-5 article 5.1(2).

The member satisfies the stability check.

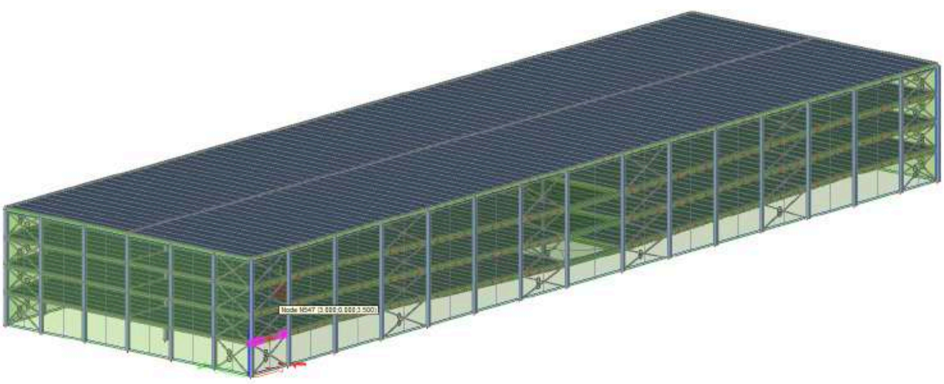

Figure 2.2.3.3. 3:Edge beam

EC-EN 1993 Steel check ULS

Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Global
Selection: Bx1_1
Filter: Cross-section = CS12-Beam2 - HEA600

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EN 1993-1-1 Code Check
National annex: Czech CSN-EN NA

Member Bx1_1	6.000 / 6.000 m	HEA600	S 355	ULS-Set B (auto)	0.29 -

Combination key
ULS-Set B (auto) / 1.15*LC1 + 1.15*LC2 + 1.50*LC3 + 1.50*LC4 + 0.90*3DWind16

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{M2} for resistance of net sections	1.25

Material			
Yield strength	f_y	355.0	MPa
Ultimate strength	f_u	490.0	MPa
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 6.000 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	52.34	kN
Shear force	$V_{y,Ed}$	0.51	kN
Shear force	$V_{z,Ed}$	-198.13	kN
Torsion	T_{Ed}	0.05	kNm
Bending moment	$M_{y,Ed}$	-251.70	kNm
Bending moment	$M_{z,Ed}$	6.12	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	116	25	4.622e+04	3.990e+04	0.9	0.5	1.0	4.7	7.3	8.1	11.8	1
3	SO	116	25	4.986e+04	5.618e+04	0.9	0.4	1.0	4.7	7.3	8.1	11.3	1
4	I	486	13	4.100e+04	-4.562e+04	-1.1		0.5	37.4	60.0	69.1	112.4	1
5	SO	116	25	-5.084e+04	-4.452e+04								
7	SO	116	25	-5.448e+04	-6.080e+04								

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Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Tension check
According to EN 1993-1-1 article 6.2.3 and formula (6.5)

Cross-section area	A	2.2700e-02	m²
Plastic tension resistance	$N_{pl,Rd}$	8058.50	kN
Ultimate tension resistance	$N_{u,Rd}$	8008.56	kN
Tension resistance	$N_{t,Rd}$	8008.56	kN
Unity check		0.01	-

$$N_{pl,Rd} = \frac{A \times f_y}{\gamma_{M0}} = \frac{2.2700 \cdot 10^{-2}[m^2] \times 355.0[MPa]}{1.00} = 8058.50[kN] \qquad \text{(EC3-1-1: 6.6)}$$

$$N_{u,Rd} = \frac{0.9 \times A \times f_u}{\gamma_{M2}} = \frac{0.9 \times 2.2700 \cdot 10^{-2}[m^2] \times 490.0[MPa]}{1.25} = 8008.56[kN] \qquad \text{(EC3-1-1: 6.7)}$$

$$N_{t,Rd} = \min(N_{pl,Rd}, N_{u,Rd}) = \min(8058.50[kN], 8008.56[kN]) = 8008.56[kN]$$

$$\text{Unity check} = \frac{N_{Ed}}{N_{t,Rd}} = \frac{52.34[kN]}{8008.56[kN]} = \mathbf{0.01 \leq 1.00} \qquad \text{(EC3-1-1: 6.5)}$$

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	5.3333e-03	m³
Plastic bending moment	$M_{pl,y,Rd}$	1893.33	kNm
Unity check		0.13	-

$$M_{pl,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{5.3333 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 1893.33[kNm] \qquad \text{(EC3-1-1: 6.13)}$$

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{pl,y,Rd}} = \frac{|-251.70[kNm]|}{1893.33[kNm]} = \mathbf{0.13 \leq 1.00} \qquad \text{(EC3-1-1: 6.12)}$$

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Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	1.1542e-03	m^3
Plastic bending moment	$M_{pl,z,Rd}$	409.73	kNm
Unity check		0.01	-

$$M_{pl,z,Rd} = \frac{W_{pl,z} \times f_y}{\gamma_{M0}} = \frac{1.1542 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 409.73[kNm]$$ (EC3-1-1: 6.13)

$$\text{Unity check} = \frac{|M_{z,Ed}|}{M_{pl,z,Rd}} = \frac{|6.12[kNm]|}{409.73[kNm]} = \mathbf{0.01 \leq 1.00}$$ (EC3-1-1: 6.12)

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1.20	
Shear area	A_v	1.5520e-02	m^2
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	3180.97	kN
Unity check		0.00	-

$$V_{pl,y,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{1.5520 \cdot 10^{-2}[m^2] \times \frac{355.0[MPa]}{\sqrt{3}}}{1.00} = 3180.97[kN]$$ (EC3-1-1: 6.18)

$$\text{Unity check} = \frac{|V_{y,Ed}|}{V_{c,y,Rd}} = \frac{|0.51[kN]|}{3180.97[kN]} = \mathbf{0.00 \leq 1.00}$$ (EC3-1-1: 6.17)

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1.20	
Shear area	A_v	9.3750e-03	m^2
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	1921.49	kN
Unity check		0.10	-

$$V_{pl,z,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{9.3750 \cdot 10^{-3}[m^2] \times \frac{355.0[MPa]}{\sqrt{3}}}{1.00} = 1921.49[kN]$$ (EC3-1-1: 6.18)

$$\text{Unity check} = \frac{|V_{z,Ed}|}{V_{c,z,Rd}} = \frac{|-198.13[kN]|}{1921.49[kN]} = \mathbf{0.10 \leq 1.00}$$ (EC3-1-1: 6.17)

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	2	
Total torsional moment	τ_{Ed}	0.3	MPa
Elastic shear resistance	τ_{Rd}	205.0	MPa
Unity check		0.00	-

$$\tau_{Ed} = |T_{Ed}| \times \tau_{Ed,unit} = |53.94| \times 6.284[kN/m^2] = 0.3[MPa]$$

$$\tau_{Rd} = \frac{f_y}{\sqrt{3} \times \gamma_{M0}} = \frac{355.0[MPa]}{\sqrt{3} \times 1.00} = 205.0[MPa]$$

$$\text{Unity check} = \frac{\tau_{Ed}}{\tau_{Rd}} = \frac{0.3[MPa]}{205.0[MPa]} = \mathbf{0.00 \leq 1.00}$$ (EC3-1-1: 6.23)

Note: The unity check for torsion is lower than the limit value of 0.05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Plastic bending moment	$M_{pl,y,Rd}$	1893.33	kNm
Exponent of bending ratio y	α	2.00	
Plastic bending moment	$M_{pl,z,Rd}$	409.73	kNm
Exponent of bending ratio z	β	1.00	

Unity check (6.41) = 0.02 + 0.01 = 0.03 -

$$M_{pl,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{5.3333 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 1893.33[kNm]$$ (EC3-1-1: 6.13)

$\alpha = 2.00$

$$M_{pl,z,Rd} = \frac{W_{pl,z} \times f_y}{\gamma_{M0}} = \frac{1.1542 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 409.73[kNm]$$ (EC3-1-1: 6.13)

$\beta = 1.00$

$$\text{Unity check} = \left(\frac{|M_{y,Ed}|}{M_{pl,y,Rd}}\right)^{\alpha} + \left(\frac{|M_{z,Ed}|}{M_{pl,z,Rd}}\right)^{\beta} = \left(\frac{|-251.70[kNm]|}{1893.33[kNm]}\right)^{2.00} + \left(\frac{|6.12[kNm]|}{409.73[kNm]}\right)^{1.00} = \mathbf{0.03 \leq 1.00}$$ (EC3-1-1: 6.41)

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.
Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.

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Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the z-z axis is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 6.000 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	116	25	4.622e+04	3.990e+04	0.9	0.5	1.0	4.7	7.3	8.1	11.8	1
3	SO	116	25	4.986e+04	5.618e+04	0.9	0.4	1.0	4.7	7.3	8.1	11.3	1
4	I	486	13	4.100e+04	-4.562e+04	-1.1		0.5	37.4	60.0	69.1	112.4	1
5	SO	116	25	-5.084e+04	-4.452e+04								
7	SO	116	25	-5.448e+04	-6.080e+04								

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.3 and formula (6.54)

LTB parameters			
Method for LTB curve		Alternative case	
Plastic section modulus	$W_{pl,y}$	5.3333e-03	m³
Elastic critical moment	M_{cr}	949.28	kNm
Relative slenderness	$\lambda_{rel,LT}$	1.41	
Limit slenderness	$\lambda_{rel,LT,0}$	0.40	
LTB curve		b	
Imperfection	α_{LT}	0.34	
LTB factor	β	0.75	
Reduction factor	χ_{LT}	0.47	
Correction factor	k_c	0.88	
Correction factor	f	0.99	
Modified reduction factor	$\chi_{LT,mod}$	0.47	
Design buckling resistance	$M_{b,Rd}$	897.39	kNm
Unity check		0.28	-

Mcr parameters			
LTB length	l_{LT}	13.500	m
Influence of load position		no influence	
Correction factor	k	1.00	
Correction factor	k_w	1.00	
LTB moment factor	C_1	1.29	
LTB moment factor	C_2	0.45	
LTB moment factor	C_3	0.41	
Shear centre distance	d_z	0	mm
Distance of load application	z_g	0	mm
Mono-symmetry constant	β_y	0	mm
Mono-symmetry constant	z_j	0	mm

$$M_{cr} = C_1 \times \frac{\pi^2 \times E \times I_z}{l_{LT}^2} \times \left[\sqrt{\left(\frac{k}{k_w}\right)^2 \times \frac{I_w}{I_z} + \frac{l_{LT}^2 \times G \times I_t}{\pi^2 \times E \times I_z} + (C_2 \times z_g - C_3 \times z_j)^2} - (C_2 \times z_g - C_3 \times z_j)\right] = 1.29$$

$$\times \frac{\pi^2 \times 210000.0[MPa] \times 1.1300 \cdot 10^{-4}[m^4]}{13.500[m]^2}$$

$$\times \left[\sqrt{\left(\frac{1.00}{1.00}\right)^2 \times \frac{8.9782 \cdot 10^{-6}[m^6]}{1.1300 \cdot 10^{-4}[m^4]} + \frac{13.500[m]^2 \times 80769.2[MPa] \times 3.9800 \cdot 10^{-6}[m^4]}{\pi^2 \times 210000.0[MPa] \times 1.1300 \cdot 10^{-4}[m^4]} + (0.45 \times 0[mm] - 0.41 \times 0[mm])^2} - (0.45 \times 0[mm] - 0.41 \times 0[mm])\right]$$

$$= 949.28[kNm]$$

$$\lambda_{rel,LT} = \sqrt{\frac{W_{pl,y} \times f_y}{M_{cr}}} = \sqrt{\frac{5.3333 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{949.28[kNm]}} = 1.41$$

$$\beta = 0.75$$

$$\chi_{LT} = \min\left(\frac{1}{\varphi_{LT} + \sqrt{\varphi_{LT}^2 - \beta \times \lambda_{rel,LT}^2}}, \frac{1}{\lambda_{rel,LT}^2}, 1\right) = \min\left(\frac{1}{1.42 + \sqrt{1.42^2 - 0.75 \times 1.41^2}}, \frac{1}{1.41^2}, 1\right) = \min(0.47, 0.50, 1) = 0.47 \quad \text{(EC3-1-1: 6.57)}$$

$$f = \min\left\{1 - 0.5 \times (1 - k_c) \times \left[1 - 2 \times (\lambda_{rel,LT} - 0.8)^2\right], 1\right\} = \min\left\{1 - 0.5 \times (1 - 0.88) \times \left[1 - 2 \times (1.41 - 0.8)^2\right], 1\right\} = \min\{0.99, 1\} = 0.99$$

$$\chi_{LT,mod} = \min\left(\frac{\chi_{LT}}{f}, 1\right) = \min\left(\frac{0.47}{0.99}, 1\right) = \min(0.47, 1) = 0.47$$

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times \frac{f_y}{\gamma_{M1}} = 0.47 \times 5.3333 \cdot 10^{-3}[m^3] \times \frac{355.0[MPa]}{1.00} = 897.39[kNm] \quad \text{(EC3-1-1: 6.55)}$$

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Mcr parameters			
LTB length	l_{LT}	13.500	m
Influence of load position		no influence	
Correction factor	k	1.00	
Correction factor	k_w	1.00	
LTB moment factor	C_1	1.29	
LTB moment factor	C_2	0.45	
LTB moment factor	C_3	0.41	
Shear centre distance	d_z	0	mm
Distance of load application	z_g	0	mm
Mono-symmetry constant	β_y	0	mm
Mono-symmetry constant	z_j	0	mm

$$M_{cr} = C_1 \times \frac{\pi^2 \times E \times I_z}{l_{LT}^2} \times \left[\sqrt{\left(\frac{k}{k_w}\right)^2 \times \frac{I_w}{I_z} + \frac{l_{LT}^2 \times G \times I_t}{\pi^2 \times E \times I_z} + (C_2 \times z_g - C_3 \times z_j)^2} - (C_2 \times z_g - C_3 \times z_j)\right] = 1.29$$

$$\times \frac{\pi^2 \times 210000.0[\mathrm{MPa}] \times 1.1300 \cdot 10^{-4}[\mathrm{m}^4]}{13.500[\mathrm{m}]^2}$$

$$\times \left[\sqrt{\left(\frac{1.00}{1.00}\right)^2 \times \frac{8.9782 \cdot 10^{-6}[\mathrm{m}^6]}{1.1300 \cdot 10^{-4}[\mathrm{m}^4]} + \frac{13.500[\mathrm{m}]^2 \times 80769.2[\mathrm{MPa}] \times 3.9800 \cdot 10^{-6}[\mathrm{m}^4]}{\pi^2 \times 210000.0[\mathrm{MPa}] \times 1.1300 \cdot 10^{-4}[\mathrm{m}^4]} + (0.45 \times 0[\mathrm{mm}] - 0.41 \times 0[\mathrm{mm}])^2} - (0.45 \times 0[\mathrm{mm}] - 0.41 \times 0[\mathrm{mm}])\right]$$

$$= 949.28[\mathrm{kNm}]$$

$$\lambda_{rel,LT} = \sqrt{\frac{W_{pl,y} \times f_y}{M_{cr}}} = \sqrt{\frac{5.3333 \cdot 10^{-3}[\mathrm{m}^3] \times 355.0[\mathrm{MPa}]}{949.28[\mathrm{kNm}]}} = 1.41$$

$$\beta = 0.75$$

$$\chi_{LT} = \min\left(\frac{1}{\varphi_{LT} + \sqrt{\varphi_{LT}^2 - \beta \times \lambda_{rel,LT}^2}}, \frac{1}{\lambda_{rel,LT}^2}, 1\right) = \min\left(\frac{1}{1.42 + \sqrt{1.42^2 - 0.75 \times 1.41^2}}, \frac{1}{1.41^2}, 1\right) = \min(0.47, 0.50, 1) = 0.47 \quad \text{(EC3-1-1: 6.57)}$$

$$f = \min\left\{1 - 0.5 \times (1 - k_c) \times \left[1 - 2 \times (\lambda_{rel,LT} - 0.8)^2\right], 1\right\} = \min\left\{1 - 0.5 \times (1 - 0.88) \times \left[1 - 2 \times (1.41 - 0.8)^2\right], 1\right\} = \min\{0.99, 1\} = 0.99$$

$$\chi_{LT,mod} = \min\left(\frac{\chi_{LT}}{f}, 1\right) = \min\left(\frac{0.47}{0.99}, 1\right) = \min(0.47, 1) = 0.47$$

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times \frac{f_y}{\gamma_{M1}} = 0.47 \times 5.3333 \cdot 10^{-3}[\mathrm{m}^3] \times \frac{355.0[\mathrm{MPa}]}{1.00} = 897.39[\mathrm{kNm}] \quad \text{(EC3-1-1: 6.55)}$$

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{b,Rd}} = \frac{|-251.70[\mathrm{kNm}]|}{897.39[\mathrm{kNm}]} = \mathbf{0.28 \leq 1.00} \quad \text{(EC3-1-1: 6.54)}$$

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.
Note: The correction factor k_c is determined from C_1.

Bending and axial tension check
According to EN 1993-1-3 article 6.3

Normal force	N_{Ed}	52.34	kN
Bending moment	$M_{y,Ed}$	-251.70	kNm
Bending moment	$M_{z,Ed}$	6.12	kNm
Tension resistance	$N_{t,Rd}$	8008.56	kN
Bending resistance	$M_{b,y,Rd}$	897.39	kNm
Bending resistance	$M_{c,z,Rd,com}$	409.73	kNm

Unity check = 0.28 + 0.01 - 0.01 = 0.29 -

$$N_{t,Rd} = \min(N_{pl,Rd}, N_{u,Rd}) = \min(8058.50[\mathrm{kN}], 8008.56[\mathrm{kN}]) = 8008.56[\mathrm{kN}]$$

$$M_{b,y,Rd} = \chi_{LT,mod} \times W_{pl,y} \times \frac{f_y}{\gamma_{M1}} = 0.47 \times 5.3333 \cdot 10^{-3}[\mathrm{m}^3] \times \frac{355.0[\mathrm{MPa}]}{1.00} = 897.39[\mathrm{kNm}]$$

$$M_{c,z,Rd,com} = \frac{W_{el,z,com} \times f_y}{\gamma_{M0}} = \frac{1.1542 \cdot 10^{-3}[\mathrm{m}^3] \times 355.0[\mathrm{MPa}]}{1.00} = 409.73[\mathrm{kNm}]$$

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{b,y,Rd}} + \frac{|M_{z,Ed}|}{M_{c,z,Rd,com}} - \frac{|N_{Ed}|}{N_{t,Rd}} = \frac{|-251.70[\mathrm{kNm}]|}{897.39[\mathrm{kNm}]} + \frac{|6.12[\mathrm{kNm}]|}{409.73[\mathrm{kNm}]} - \frac{|52.34[\mathrm{kN}]|}{8008.56[\mathrm{kN}]} = \mathbf{0.29 \leq 1.00}$$

Shear Buckling check
According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

Shear Buckling parameters			
Buckling field length	a	6.000	m
Web		unstiffened	
Web height	h_w	540	mm
Web thickness	t	13	mm
Material coefficient	ε	0.81	
Shear correction factor	η	1.20	

Shear Buckling verification		
Web slenderness	h_w/t	41.54
Web slenderness limit		48.82

Note: The web slenderness is such that Shear Buckling effects may be ignored according to EN 1993-1-5 article 5.1(2).

The member satisfies the stability check.

Student version

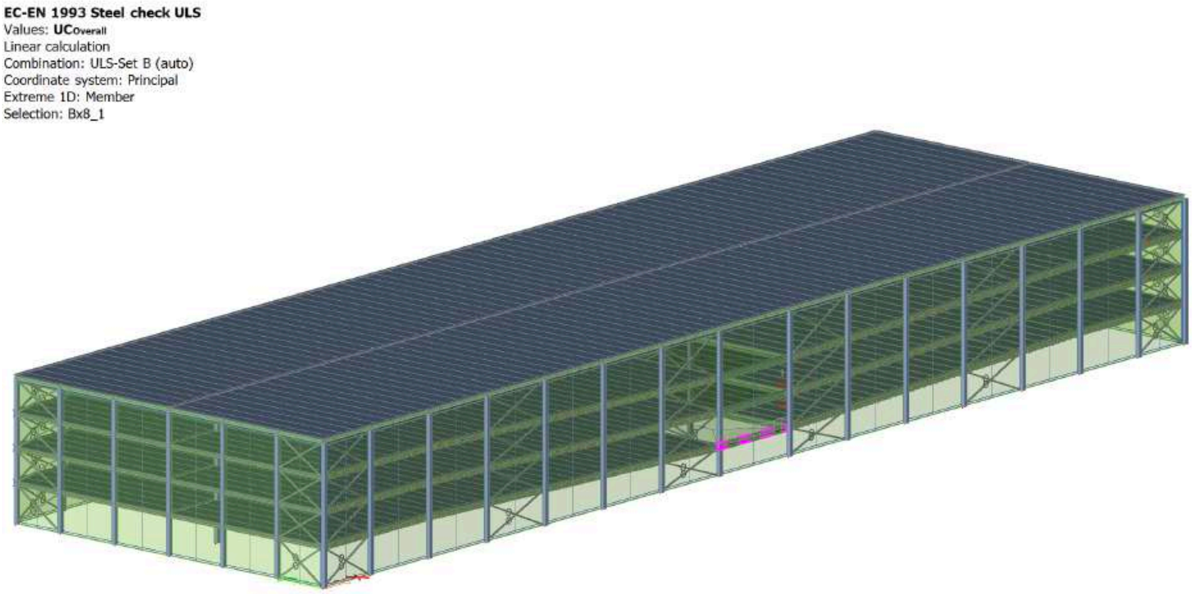


Figure 2.2.3.3. 4: Middle beam

EC-EN 1993 Steel check ULS

Student version

Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Global
Selection: Bx8_1
Filter: Cross-section = CS12-Beam2 - HEA600

EN 1993-1-1 Code Check
National annex: Czech CSN-EN NA

Member Bx8_1	**0.000 / 9.000 m**	**HEA600**	**S 355**	**ULS-Set B (auto)**	**0.10 -**

Combination key
ULS-Set B (auto) / 1.15*LC1 + 1.15*LC2 + 1.05*LC3 + 1.05*LC4 + 1.50*3DWind4

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{M2} for resistance of net sections	1.25

Material			
Yield strength	f_y	355.0	MPa
Ultimate strength	f_u	490.0	MPa
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 0.000 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-42.50	kN
Shear force	$V_{y,Ed}$	-0.13	kN
Shear force	$V_{z,Ed}$	72.27	kN
Torsion	T_{Ed}	0.00	kNm
Bending moment	$M_{y,Ed}$	-167.83	kNm
Bending moment	$M_{z,Ed}$	-0.48	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	116	25	3.559e+04	3.609e+04	1.0	0.4	1.0	4.7	7.3	8.1	11.2	1
3	SO	116	25	3.531e+04	3.481e+04	1.0	0.4	1.0	4.7	7.3	8.1	11.3	1
4	I	486	13	3.075e+04	-2.700e+04	-0.9		0.5	37.4	56.9	65.7	88.8	1
5	SO	116	25	-3.184e+04	-3.233e+04								
7	SO	116	25	-3.155e+04	-3.106e+04								

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	2.2700e-02	m²
Compression resistance	$N_{c,Rd}$	8058.50	kN
Unity check		0.01	-

$$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M0}} = \frac{2.2700 \cdot 10^{-2}[\mathrm{m}^2] \times 355.0[\mathrm{MPa}]}{1.00} = 8058.50[\mathrm{kN}] \qquad \text{(EC3-1-1: 6.10)}$$

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{c,Rd}} = \frac{|-42.50[\mathrm{kN}]|}{8058.50[\mathrm{kN}]} = \mathbf{0.01} \leq \mathbf{1.00} \qquad \text{(EC3-1-1: 6.9)}$$

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	5.3333e-03	m³
Plastic bending moment	$M_{pl,y,Rd}$	1893.33	kNm
Unity check		0.09	-

$$M_{pl,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{5.3333 \cdot 10^{-3}[\mathrm{m}^3] \times 355.0[\mathrm{MPa}]}{1.00} = 1893.33[\mathrm{kNm}] \qquad \text{(EC3-1-1: 6.13)}$$

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{pl,y,Rd}} = \frac{|-167.83[\mathrm{kNm}]|}{1893.33[\mathrm{kNm}]} = \mathbf{0.09} \leq \mathbf{1.00} \qquad \text{(EC3-1-1: 6.12)}$$

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	1.1542e-03	m³
Plastic bending moment	$M_{pl,z,Rd}$	409.73	kNm
Unity check		0.00	-

$$M_{pl,z,Rd} = \frac{W_{pl,z} \times f_y}{\gamma_{M0}} = \frac{1.1542 \cdot 10^{-3}[\mathrm{m}^3] \times 355.0[\mathrm{MPa}]}{1.00} = 409.73[\mathrm{kNm}] \qquad \text{(EC3-1-1: 6.13)}$$

$$\text{Unity check} = \frac{|M_{z,Ed}|}{M_{pl,z,Rd}} = \frac{|-0.48[\mathrm{kNm}]|}{409.73[\mathrm{kNm}]} = \mathbf{0.00} \leq \mathbf{1.00} \qquad \text{(EC3-1-1: 6.12)}$$

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1.20	
Shear area	A_v	1.5520e-02	m²
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	3180.97	kN
Unity check		0.00	-

$$V_{pl,y,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{1.5520 \cdot 10^{-2}[\mathrm{m}^2] \times \frac{355.0[\mathrm{MPa}]}{\sqrt{3}}}{1.00} = 3180.97[\mathrm{kN}] \qquad \text{(EC3-1-1: 6.18)}$$

$$\text{Unity check} = \frac{|V_{y,Ed}|}{V_{c,y,Rd}} = \frac{|-0.13[\mathrm{kN}]|}{3180.97[\mathrm{kN}]} = \mathbf{0.00} \leq \mathbf{1.00} \qquad \text{(EC3-1-1: 6.17)}$$

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1.20	
Shear area	A_v	9.3750e-03	m²
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	1921.49	kN
Unity check		0.04	-

$$V_{pl,z,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{9.3750 \cdot 10^{-3}[\mathrm{m}^2] \times \frac{355.0[\mathrm{MPa}]}{\sqrt{3}}}{1.00} = 1921.49[\mathrm{kN}] \qquad \text{(EC3-1-1: 6.18)}$$

$$\text{Unity check} = \frac{|V_{z,Ed}|}{V_{c,z,Rd}} = \frac{|72.27[\mathrm{kN}]|}{1921.49[\mathrm{kN}]} = \mathbf{0.04} \leq \mathbf{1.00} \qquad \text{(EC3-1-1: 6.17)}$$

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	2	
Total torsional moment	τ_{Ed}	0.0	MPa
Elastic shear resistance	τ_{Rd}	205.0	MPa
Unity check		0.00	-

Student version

$\tau_{Ed} = |T_{Ed}| \times \tau_{Ed,unit} = |3.73| \times 6.284[kN/m^2] = 0.0[MPa]$

$\tau_{Rd} = \dfrac{f_y}{\sqrt{3} \times \gamma_{M0}} = \dfrac{355.0[MPa]}{\sqrt{3} \times 1.00} = 205.0[MPa]$

Student version

Unity check $= \dfrac{\tau_{Ed}}{\tau_{Rd}} = \dfrac{0.0[MPa]}{205.0[MPa]} = \mathbf{0.00 \leq 1.00}$ (EC3-1-1: 6.23)

Note: The unity check for torsion is lower than the limit value of 0.05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Plastic bending moment	$M_{pl,y,Rd}$	1893.33	kNm
Exponent of bending ratio y	α	2.00	
Plastic bending moment	$M_{pl,z,Rd}$	409.73	kNm
Exponent of bending ratio z	β	1.00	

Unity check (6.41) = 0.01 + 0.00 = 0.01 -

$M_{pl,y,Rd} = \dfrac{W_{pl,y} \times f_y}{\gamma_{M0}} = \dfrac{5.3333 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 1893.33[kNm]$ (EC3-1-1: 6.13)

$\alpha = 2.00$

$M_{pl,z,Rd} = \dfrac{W_{pl,z} \times f_y}{\gamma_{M0}} = \dfrac{1.1542 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 409.73[kNm]$ (EC3-1-1: 6.13)

$\beta = 1.00$

Unity check $= \left(\dfrac{|M_{y,Ed}|}{M_{pl,y,Rd}}\right)^{\alpha} + \left(\dfrac{|M_{z,Ed}|}{M_{pl,z,Rd}}\right)^{\beta} = \left(\dfrac{|-167.83[kNm]|}{1893.33[kNm]}\right)^{2.00} + \left(\dfrac{|-0.48[kNm]|}{409.73[kNm]}\right)^{1.00} = \mathbf{0.01 \leq 1.00}$ (EC3-1-1: 6.41)

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.
Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.
Note: Since the axial force satisfies criteria (6.35) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the z-z axis is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 7.200 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	116	25	2.449e+03	3.925e+03	0.6	0.5	1.0	4.7	7.3	8.1	11.7	1
3	SO	116	25	1.600e+03	1.231e+02	0.1	1.4	1.0	4.7	7.3	8.1	20.1	1
4	I	486	13	2.004e+03	1.750e+03	0.9		1.0	37.4	22.8	27.7	32.3	4
5	SO	116	25	1.304e+03	-1.722e+02	-0.1	2.7	0.9	4.7	8.8	9.8	27.9	1
7	SO	116	25	2.154e+03	3.630e+03	0.6	0.5	1.0	4.7	7.3	8.1	11.7	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 4

Effective section N-
Effective width calculation
According to EN 1993-1-5 article 4.4

Id	Type	b_p [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	λ_p [-]	ρ [-]	b_e [mm]	b_{e1} [mm]	b_{e2} [mm]
1	SO	116	3.550e+05	3.550e+05	1.0	0.4	0.3	1.0	116		
3	SO	116	3.550e+05	3.550e+05	1.0	0.4	0.3	1.0	116		
4	I	486	3.550e+05	3.550e+05	1.0	4.0	0.8	0.9	437	219	219
5	SO	116	3.550e+05	3.550e+05	1.0	0.4	0.3	1.0	116		
7	SO	116	3.550e+05	3.550e+05	1.0	0.4	0.3	1.0	116		

Effective section My-
Effective width calculation
According to EN 1993-1-5 article 4.4

Id	Type	b_p [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	λ_p [-]	ρ [-]	b_e [mm]	b_{e1} [mm]	b_{e2} [mm]
1	SO	116	3.550e+05	3.550e+05	1.0	0.4	0.3	1.0	116		
3	SO	116	3.550e+05	3.550e+05	1.0	0.4	0.3	1.0	116		
4	I	486	3.054e+05	-3.054e+05	-1.0	23.9	0.3	1.0	243	97	146
5	SO	116	-3.550e+05	-3.550e+05							
7	SO	116	-3.550e+05	-3.550e+05							

Effective section Mz-
Effective width calculation
According to EN 1993-1-5 article 4.4

Student version

Id	Type	b_p [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	λ_p [-]	ρ [-]	b_e [mm]	b_{e1} [mm]	b_{e2} [mm]
1	SO	116	3.550e+05	7.928e+04	0.2	0.5	0.3	1.0	116		
3	SO	116	-7.928e+04	-3.550e+05							
4	I	486	0.000e+00	0.000e+00							
5	SO	116	-7.928e+04	-3.550e+05							
7	SO	116	3.550e+05	7.928e+04	0.2	0.5	0.3	1.0	116		

Effective properties						
Effective area	A_{eff}	2.2017e-02	m²			
Effective second moment of area	$I_{eff,y}$	1.4123e-03	m⁴	$I_{eff,z}$	1.1271e-04	m⁴
Effective section modulus	$W_{eff,y}$	4.7874e-03	m³	$W_{eff,z}$	7.5143e-04	m³
Shift of the centroid	$e_{N,y}$	0	mm	$e_{N,z}$	0	mm

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	9.000	9.000	m
Buckling factor	k	1.26	0.77	
Buckling length	l_{cr}	11.335	6.934	m
Critical Euler load	N_{cr}	22745.32	4871.29	kN
Slenderness	λ	45.48	98.28	
Relative slenderness	λ_{rel}	0.59	1.27	
Limit slenderness	$\lambda_{rel,0}$	0.20	0.20	

Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

$$N_{cr,y} = \frac{\pi^2 \times E \times I_y}{l_{cr,y}^2} = \frac{\pi^2 \times 210000.0[MPa] \times 1.4100 \cdot 10^{-3}[m^4]}{11.335[m]^2} = 22745.32[kN]$$

$$N_{cr,z} = \frac{\pi^2 \times E \times I_z}{l_{cr,z}^2} = \frac{\pi^2 \times 210000.0[MPa] \times 1.1300 \cdot 10^{-4}[m^4]}{6.934[m]^2} = 4871.29[kN]$$

$$\lambda_y = \frac{l_{cr,y}}{i_y} = \frac{11.335[m]}{249[mm]} = 45.48$$

$$\lambda_z = \frac{l_{cr,z}}{i_z} = \frac{6.934[m]}{71[mm]} = 98.28$$

$$\lambda_{rel,y} = \frac{\lambda_y \times \sqrt{\frac{A_{eff}}{A}}}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{45.48 \times \sqrt{\frac{2.2017 \cdot 10^{-2}[m^2]}{2.2700 \cdot 10^{-2}[m^2]}}}{\pi \times \sqrt{\frac{210000.0[MPa]}{355.0[MPa]}}} = 0.59 \quad \text{(EC3-1-1: 6.51)}$$

$$\lambda_{rel,z} = \frac{\lambda_z \times \sqrt{\frac{A_{eff}}{A}}}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{98.28 \times \sqrt{\frac{2.2017 \cdot 10^{-2}[m^2]}{2.2700 \cdot 10^{-2}[m^2]}}}{\pi \times \sqrt{\frac{210000.0[MPa]}{355.0[MPa]}}} = 1.27 \quad \text{(EC3-1-1: 6.51)}$$

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: For this I-section the Torsional(-Flexural) buckling resistance is higher than the resistance for Flexural buckling. Therefore Torsional(-Flexural) buckling is not printed on the output.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.3 and formula (6.54)

LTB parameters			
Method for LTB curve		Alternative case	
Effective section modulus	$W_{eff,y}$	4.7874e-03	m³
Elastic critical moment	M_{cr}	4734.28	kNm
Relative slenderness	$\lambda_{rel,LT}$	0.60	
Limit slenderness	$\lambda_{rel,LT,0}$	0.40	

Note: The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4).

Mcr parameters			
LTB length	l_{LT}	9.000	m
Influence of load position		no influence	
Correction factor	k	1.00	
Correction factor	k_w	1.00	
LTB moment factor	C_1	3.75	
LTB moment factor	C_2	1.66	
LTB moment factor	C_3	0.41	
Shear centre distance	d_z	0	mm
Distance of load application	z_g	0	mm
Mono-symmetry constant	β_y	0	mm
Mono-symmetry constant	z_j	0	mm

$$M_{cr} = C_1 \times \frac{\pi^2 \times E \times I_z}{l_{LT}^2} \times \left[\sqrt{\left(\frac{k}{k_w}\right)^2 \times \frac{I_w}{I_z} + \frac{l_{LT}^2 \times G \times I_t}{\pi^2 \times E \times I_z} + (C_2 \times z_g - C_3 \times z_j)^2} - (C_2 \times z_g - C_3 \times z_j)\right] = 3.75$$

$$\times \frac{\pi^2 \times 210000.0[MPa] \times 1.1300 \cdot 10^{-4}[m^4]}{9.000[m]^2}$$

$$\times \left[\sqrt{\left(\frac{1.00}{1.00}\right)^2 \times \frac{8.9782 \cdot 10^{-6}[m^6]}{1.1300 \cdot 10^{-4}[m^4]} + \frac{9.000[m]^2 \times 80769.2[MPa] \times 3.9800 \cdot 10^{-6}[m^4]}{\pi^2 \times 210000.0[MPa] \times 1.1300 \cdot 10^{-4}[m^4]} + (1.66 \times 0[mm] - 0.41 \times 0[mm])^2} - (1.66 \times 0[mm] - 0.41 \times 0[mm])\right]$$

$$= 4734.28[kNm]$$

$$\lambda_{rel,LT} = \sqrt{\frac{W_{eff,y} \times f_y}{M_{cr}}} = \sqrt{\frac{4.7874 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{4734.28[kNm]}} = 0.60$$

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section effective area	A_{eff}	2.2017e-02	m^2
Effective section modulus	$W_{eff,y}$	4.7874e-03	m^3
Effective section modulus	$W_{eff,z}$	7.5143e-04	m^3
Design compression force	N_{Ed}	42.50	kN
Design bending moment (maximum)	$M_{y,Ed}$	-167.83	kNm
Design bending moment (maximum)	$M_{z,Ed}$	-1.67	kNm
Additional moment	$\Delta M_{y,Ed}$	0.00	kNm
Additional moment	$\Delta M_{z,Ed}$	0.00	kNm
Characteristic compression resistance	N_{Rk}	7815.95	kN
Characteristic moment resistance	$M_{y,Rk}$	1699.52	kNm
Characteristic moment resistance	$M_{z,Rk}$	266.76	kNm

Bending and axial compression check parameters			
Reduction factor	χ_y	1.00	
Reduction factor	χ_z	1.00	
Modified reduction factor	$\chi_{LT,mod}$	1.00	
Interaction factor	k_{yy}	0.90	
Interaction factor	k_{yz}	0.72	
Interaction factor	k_{zy}	0.72	
Interaction factor	k_{zz}	0.72	

Maximum moment $M_{y,Ed}$ is derived from beam Bx8_1 position 0.000 m.
Maximum moment $M_{z,Ed}$ is derived from beam Bx8_1 position 9.000 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.1	
Sway type y		sway	
Equivalent moment factor	C_{my}	0.90	
Resulting load type z		linear moment M	
Ratio of end moments	ψ_z	0.29	
Equivalent moment factor	C_{mz}	0.72	
Resulting load type LT		line load q	
End moment	$M_{h,LT}$	-167.83	kNm
Field moment	$M_{s,LT}$	54.85	kNm
Factor	$\alpha_{s,LT}$	-0.33	
Ratio of end moments	ψ_{LT}	0.55	
Equivalent moment factor	C_{mLT}	0.40	

Unity check (6.61) = 0.01 + 0.09 + 0.00 = 0.10 -
Unity check (6.62) = 0.01 + 0.07 + 0.00 = 0.08 -

$$C_{my} = 0.90$$

$$C_{mz} = \max(0.6 + 0.4 \times \psi_z, 0.4) = \max(0.6 + 0.4 \times 0.29, 0.4) = \max(0.72, 0.4) = 0.72$$

$$\alpha_{s,LT} = \frac{M_{s,LT}}{M_{h,LT}} = \frac{54.85[kNm]}{-167.83[kNm]} = -0.33$$

$$C_{mLT} = \max(0.1 - 0.8 \times \alpha_{s,LT}, 0.4) = \max(0.1 - 0.8 \times -0.33, 0.4) = \max(0.36, 0.4) = 0.40$$

$$N_{Rk} = A_{eff} \times f_y = 2.2017 \cdot 10^{-2}[m^2] \times 355.0[MPa] = 7815.95[kN]$$

$$M_{y,Rk} = W_{eff,y} \times f_y = 4.7874 \cdot 10^{-3}[m^3] \times 355.0[MPa] = 1699.52[kNm]$$

$$M_{z,Rk} = W_{eff,z} \times f_y = 7.5143 \cdot 10^{-4}[m^3] \times 355.0[MPa] = 266.76[kNm]$$

$$k_{yy} = \min\left[C_{my} \times \left(1 + 0.6 \times \lambda_{rel,y} \times \frac{N_{Ed}}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}}\right), C_{my} \times \left(1 + 0.6 \times \frac{N_{Ed}}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}}\right)\right]$$

$$= \min\left[0.90 \times \left(1 + 0.6 \times 0.59 \times \frac{42.50[kN]}{1.00 \times \frac{7815.95[kN]}{1.00}}\right), 0.90 \times \left(1 + 0.6 \times \frac{42.50[kN]}{1.00 \times \frac{7815.95[kN]}{1.00}}\right)\right] = \min[0.90, 0.90] = 0.90$$

$k_{yz} = k_{zz} = 0.72$

$k_{zy} = 0.8 \times k_{yy} = 0.8 \times 0.90 = 0.72$

$$k_{zz} = \min\left[C_{mz} \times \left(1 + 0.6 \times \lambda_{rel,z} \times \frac{N_{Ed}}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}}\right), C_{mz} \times \left(1 + 0.6 \times \frac{N_{Ed}}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}}\right)\right]$$

$$= \min\left[0.72 \times \left(1 + 0.6 \times 1.27 \times \frac{42.50[kN]}{1.00 \times \frac{7815.95[kN]}{1.00}}\right), 0.72 \times \left(1 + 0.6 \times \frac{42.50[kN]}{1.00 \times \frac{7815.95[kN]}{1.00}}\right)\right] = \min[0.72, 0.72] = 0.72$$

$$\text{Unity check (6.61)} = \frac{|N_{Ed}|}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \times \frac{|M_{y,Ed}| + |\Delta M_{y,Ed}|}{\chi_{LT,mod} \times \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \times \frac{|M_{z,Ed}| + |\Delta M_{z,Ed}|}{\frac{M_{z,Rk}}{\gamma_{M1}}}$$

(EC3-1-1: 6.61)

$$= \frac{|42.50[kN]|}{1.00 \times \frac{7815.95[kN]}{1.00}} + 0.90 \times \frac{|-167.83[kNm]| + |0.00[kNm]|}{1.00 \times \frac{1699.52[kNm]}{1.00}} + 0.72 \times \frac{|-1.67[kNm]| + |0.00[kNm]|}{\frac{266.76[kNm]}{1.00}} = \mathbf{0.10 \leq 1.00}$$

$$\text{Unity check (6.62)} = \frac{|N_{Ed}|}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} \times \frac{|M_{y,Ed}| + |\Delta M_{y,Ed}|}{\chi_{LT,mod} \times \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \times \frac{|M_{z,Ed}| + |\Delta M_{z,Ed}|}{\frac{M_{z,Rk}}{\gamma_{M1}}}$$

(EC3-1-1: 6.62)

$$= \frac{|42.50[kN]|}{1.00 \times \frac{7815.95[kN]}{1.00}} + 0.72 \times \frac{|-167.83[kNm]| + |0.00[kNm]|}{1.00 \times \frac{1699.52[kNm]}{1.00}} + 0.72 \times \frac{|-1.67[kNm]| + |0.00[kNm]|}{\frac{266.76[kNm]}{1.00}} = \mathbf{0.08 \leq 1.00}$$

$\text{Unity check} = \max(\text{Unity check (6.61)}, \text{Unity check (6.62)}) = \max(0.10, 0.08) = \mathbf{0.10 \leq 1.00}$

Shear Buckling check

According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

Shear Buckling parameters			
Buckling field length	a	9.000	m
Web		unstiffened	
Web height	h_w	540	mm
Web thickness	t	13	mm
Material coefficient	ε	0.81	
Shear correction factor	η	1.20	

Shear Buckling verification		
Web slenderness	h_w/t	41.54
Web slenderness limit		48.82

$$h_w/t = \frac{h_w}{t} = \frac{540[mm]}{13[mm]} = 41.54$$

$$\text{limit } h_w/t = \frac{72 \times \varepsilon}{\eta} = \frac{72 \times 0.81}{1.20} = 48.82$$

Note: The web slenderness is such that Shear Buckling effects may be ignored according to EN 1993-1-5 article 5.1(2).

The member satisfies the stability check.

Student version

EC-EN 1993 Steel check ULS
Values: **UCoverall**
Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Member
Selection: Bx8_1

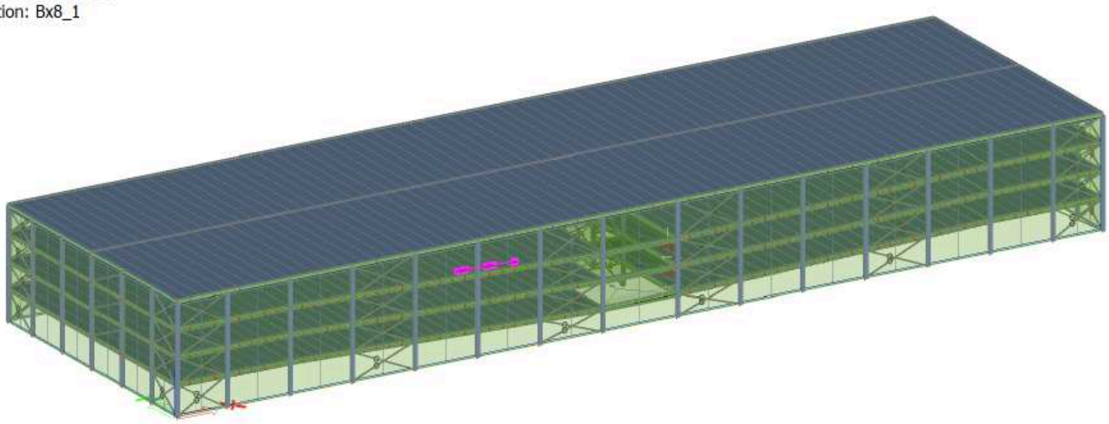


Figure 2.2.3.3. 5:Interior beam

EC-EN 1993 Steel check ULS

Student version

Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Global
Selection: Bx22_1
Filter: Cross-section = CS12-Beam2 - HEA600

EN 1993-1-1 Code Check
National annex: Czech CSN-EN NA

Member Bx22_1	**3.752 / 7.503 m**	**HEA600**	**S 355**	**ULS-Set B (auto)**	**0.30 -**

Combination key
ULS-Set B (auto) / 1.15*LC1 + 1.15*LC2 + 1.50*LC3 + 1.50*LC4 + 0.90*3DWind12

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{M2} for resistance of net sections	1.25

Material			
Yield strength	f_y	355.0	MPa
Ultimate strength	f_u	490.0	MPa
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 3.752 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	51.59	kN
Shear force	$V_{y,Ed}$	0.00	kN
Shear force	$V_{z,Ed}$	0.00	kN
Torsion	T_{Ed}	-0.18	kNm
Bending moment	$M_{y,Ed}$	405.05	kNm
Bending moment	$M_{z,Ed}$	0.00	kNm

The critical check is on position 3.752 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	51.59	kN
Shear force	$V_{y,Ed}$	0.00	kN
Shear force	$V_{z,Ed}$	0.00	kN
Torsion	T_{Ed}	-0.18	kNm
Bending moment	$M_{y,Ed}$	405.05	kNm
Bending moment	$M_{z,Ed}$	0.00	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	116	25	-8.330e+04	-8.330e+04								
3	SO	116	25	-8.330e+04	-8.330e+04								
4	I	486	13	-7.197e+04	6.742e+04	-1.1		0.5	37.4	60.0	69.1	107.8	1
5	SO	116	25	7.875e+04	7.875e+04	1.0	0.4	1.0	4.7	7.3	8.1	11.4	1
7	SO	116	25	7.875e+04	7.875e+04	1.0	0.4	1.0	4.7	7.3	8.1	11.4	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Tension check
According to EN 1993-1-1 article 6.2.3 and formula (6.5)

Cross-section area	A	2.2700e-02	m²
Plastic tension resistance	$N_{pl,Rd}$	8058.50	kN
Ultimate tension resistance	$N_{u,Rd}$	8008.56	kN
Tension resistance	$N_{t,Rd}$	8008.56	kN
Unity check		0.01	-

$$N_{pl,Rd} = \frac{A \times f_y}{\gamma_{M0}} = \frac{2.2700 \cdot 10^{-2}[m^2] \times 355.0[MPa]}{1.00} = 8058.50[kN] \qquad \text{(EC3-1-1: 6.6)}$$

$$N_{u,Rd} = \frac{0.9 \times A \times f_u}{\gamma_{M2}} = \frac{0.9 \times 2.2700 \cdot 10^{-2}[m^2] \times 490.0[MPa]}{1.25} = 8008.56[kN] \qquad \text{(EC3-1-1: 6.7)}$$

$$N_{t,Rd} = \min(N_{pl,Rd}, N_{u,Rd}) = \min(8058.50[kN], 8008.56[kN]) = 8008.56[kN]$$

$$\text{Unity check} = \frac{N_{Ed}}{N_{t,Rd}} = \frac{51.59[kN]}{8008.56[kN]} = \mathbf{0.01 \leq 1.00} \qquad \text{(EC3-1-1: 6.5)}$$

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	5.3333e-03	m³
Plastic bending moment	$M_{pl,y,Rd}$	1893.33	kNm
Unity check		0.21	-

$$M_{pl,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{5.3333 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 1893.33[kNm] \qquad \text{(EC3-1-1: 6.13)}$$

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{pl,y,Rd}} = \frac{|405.05[kNm]|}{1893.33[kNm]} = \mathbf{0.21 \leq 1.00} \qquad \text{(EC3-1-1: 6.12)}$$

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	2	
Total torsional moment	τ_{Ed}	1.1	MPa
Elastic shear resistance	τ_{Rd}	205.0	MPa
Unity check		0.01	-

$$\tau_{Ed} = |T_{Ed}| \times \tau_{Ed,unit} = |-181.66| \times 6.284[kN/m^2] = 1.1[MPa]$$

$$\tau_{Rd} = \frac{f_y}{\sqrt{3} \times \gamma_{M0}} = \frac{355.0[MPa]}{\sqrt{3} \times 1.00} = 205.0[MPa]$$

$$\text{Unity check} = \frac{\tau_{Ed}}{\tau_{Rd}} = \frac{1.1[MPa]}{205.0[MPa]} = \mathbf{0.01 \leq 1.00} \qquad \text{(EC3-1-1: 6.23)}$$

Note: The unity check for torsion is lower than the limit value of 0.05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.31)

Plastic bending moment	$M_{pl,y,Rd}$	1893.33	kNm
Unity check		0.21	-

$$M_{pl,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{5.3333 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 1893.33[kNm] \qquad \text{(EC3-1-1: 6.13)}$$

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{pl,y,Rd}} = \frac{|405.05[kNm]|}{1893.33[kNm]} = \mathbf{0.21 \leq 1.00} \qquad \text{(EC3-1-1: 6.31)}$$

Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.

The member satisfies the section check.

Student version

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 3.752 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	116	25	-8.330e+04	-8.330e+04								
3	SO	116	25	-8.330e+04	-8.330e+04								
4	I	486	13	-7.197e+04	6.742e+04	-1.1		0.5	37.4	60.0	69.1	107.8	1
5	SO	116	25	7.875e+04	7.875e+04	1.0	0.4	1.0	4.7	7.3	8.1	11.4	1
7	SO	116	25	7.875e+04	7.875e+04	1.0	0.4	1.0	4.7	7.3	8.1	11.4	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.3 and formula (6.54)

LTB parameters			
Method for LTB curve		Alternative case	
Plastic section modulus	$W_{pl,y}$	5.3333e-03	m³
Elastic critical moment	M_{cr}	1856.08	kNm
Relative slenderness	$\lambda_{rel,LT}$	1.01	
Limit slenderness	$\lambda_{rel,LT,0}$	0.40	
LTB curve		b	
Imperfection	α_{LT}	0.34	
LTB factor	β	0.75	
Reduction factor	χ_{LT}	0.69	
Correction factor	k_c	0.94	
Correction factor	f	0.97	
Modified reduction factor	$\chi_{LT,mod}$	0.71	
Design buckling resistance	$M_{b,Rd}$	1348.82	kNm
Unity check		0.30	-

Mcr parameters			
LTB length	l_{LT}	7.503	m
Influence of load position		no influence	
Correction factor	k	1.00	
Correction factor	k_w	1.00	
LTB moment factor	C_1	1.13	
LTB moment factor	C_2	0.45	
LTB moment factor	C_3	0.53	
Shear centre distance	d_z	0	mm
Distance of load application	z_g	0	mm
Mono-symmetry constant	β_y	0	mm
Mono-symmetry constant	z_j	0	mm

$$M_{cr} = C_1 \times \frac{\pi^2 \times E \times I_z}{l_{LT}^2} \times \left[\sqrt{\left(\frac{k}{k_w}\right)^2 \times \frac{I_w}{I_z} + \frac{l_{LT}^2 \times G \times I_t}{\pi^2 \times E \times I_z} + (C_2 \times z_g - C_3 \times z_j)^2} - (C_2 \times z_g - C_3 \times z_j)\right] = 1.13$$
$$\times \frac{\pi^2 \times 210000.0[MPa] \times 1.1300 \cdot 10^{-4}[m^4]}{7.503[m]^2}$$
$$\times \left[\sqrt{\left(\frac{1.00}{1.00}\right)^2 \times \frac{8.9782 \cdot 10^{-6}[m^6]}{1.1300 \cdot 10^{-4}[m^4]} + \frac{7.503[m]^2 \times 80769.2[MPa] \times 3.9800 \cdot 10^{-6}[m^4]}{\pi^2 \times 210000.0[MPa] \times 1.1300 \cdot 10^{-4}[m^4]} + (0.45 \times 0[mm] - 0.53 \times 0[mm])^2} - (0.45 \times 0[mm] - 0.53 \times 0[mm])\right]$$
$$= 1856.08[kNm]$$

$$\lambda_{rel,LT} = \sqrt{\frac{W_{pl,y} \times f_y}{M_{cr}}} = \sqrt{\frac{5.3333 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1856.08[kNm]}} = 1.01$$

$$\beta = 0.75$$

$$\chi_{LT} = \min\left(\frac{1}{\varphi_{LT} + \sqrt{\varphi_{LT}^2 - \beta \times \lambda_{rel,LT}^2}}, \frac{1}{\lambda_{rel,LT}^2}, 1\right) = \min\left(\frac{1}{0.99 + \sqrt{0.99^2 - 0.75 \times 1.01^2}}, \frac{1}{1.01^2}, 1\right) = \min(0.69, 0.98, 1) = 0.69 \qquad \text{(EC3-1-1: 6.57)}$$

$$f = \min\left\{1 - 0.5 \times (1 - k_c) \times \left[1 - 2 \times (\lambda_{rel,LT} - 0.8)^2\right], 1\right\} = \min\left\{1 - 0.5 \times (1 - 0.94) \times \left[1 - 2 \times (1.01 - 0.8)^2\right], 1\right\} = \min\{0.97, 1\} = 0.97$$

$$\chi_{LT,mod} = \min\left(\frac{\chi_{LT}}{f}, 1\right) = \min\left(\frac{0.69}{0.97}, 1\right) = \min(0.71, 1) = 0.71$$

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times \frac{f_y}{\gamma_{M1}} = 0.71 \times 5.3333 \cdot 10^{-3}[m^3] \times \frac{355.0[MPa]}{1.00} = 1348.82[kNm] \qquad \text{(EC3-1-1: 6.55)}$$

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{b,Rd}} = \frac{|405.05[kNm]|}{1348.82[kNm]} = \mathbf{0.30 \leq 1.00} \qquad \text{(EC3-1-1: 6.54)}$$

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.
Note: The correction factor k_c is determined from C_1.

The member satisfies the stability check.

Student version

2.2.3.3.3. Secondary Beam Check

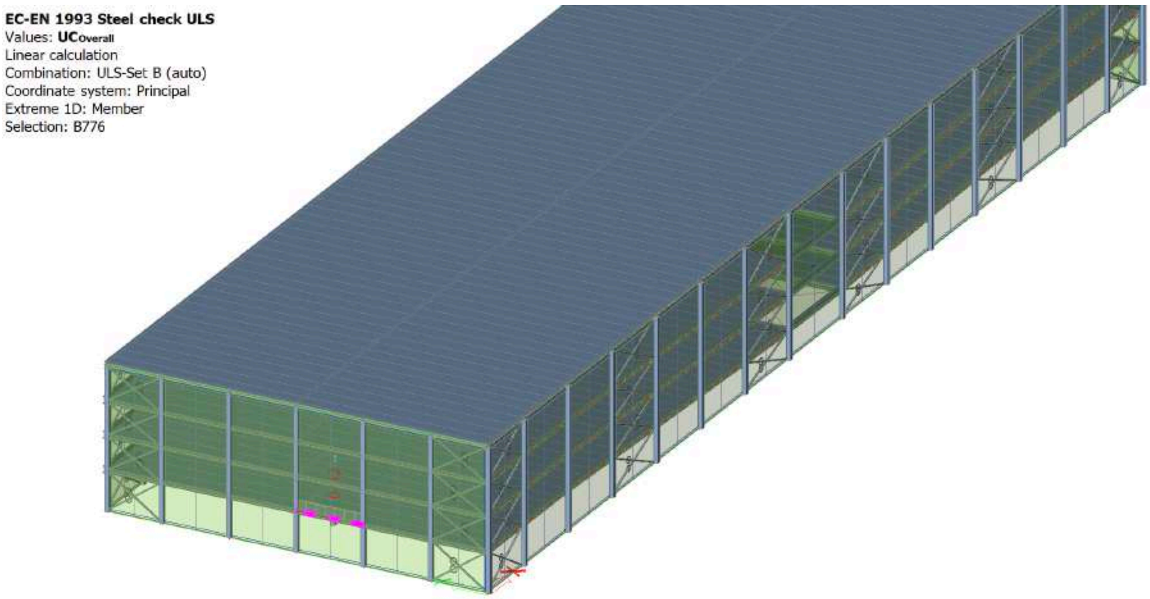


Figure 2.2.3.3. 6:Edge secondary beam

EC-EN 1993 Steel check ULS

Student version

Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Global
Selection: B776

EN 1993-1-1 Code Check
National annex: Czech CSN-EN NA

Member B776	**2.571 / 6.000 m**	**HEA320**	**S 355**	**ULS-Set B (auto)**	**0.01 -**

Combination key
ULS-Set B (auto) / 1.35*LC1 + 1.35*LC2 + 1.05*LC3 + 0.90*3DWind8

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{M2} for resistance of net sections	1.25

Material			
Yield strength	f_y	355.0	MPa
Ultimate strength	f_u	490.0	MPa
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 2.571 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-16.91	kN
Shear force	$V_{y,Ed}$	0.00	kN
Shear force	$V_{z,Ed}$	0.55	kN
Torsion	T_{Ed}	0.04	kNm
Bending moment	$M_{y,Ed}$	5.68	kNm
Bending moment	$M_{z,Ed}$	0.00	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	118	16	-2.290e+03	-2.290e+03								
3	SO	118	16	-2.290e+03	-2.290e+03								
4	I	225	9	-1.429e+03	4.148e+03	-0.3		0.5	25.0	56.5	65.3	58.0	1
5	SO	118	16	5.009e+03	5.009e+03	1.0	0.4	1.0	7.6	7.3	8.1	11.4	2
7	SO	118	16	5.009e+03	5.009e+03	1.0	0.4	1.0	7.6	7.3	8.1	11.4	2

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 2

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	1.2400e-02	m²
Compression resistance	$N_{c,Rd}$	4402.00	kN
Unity check		0.00	-

$$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M0}} = \frac{1.2400 \cdot 10^{-2}[m^2] \times 355.0[MPa]}{1.00} = 4402.00[kN] \quad \text{(EC3-1-1: 6.10)}$$

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{c,Rd}} = \frac{|-16.91[kN]|}{4402.00[kN]} = \mathbf{0.00 \leq 1.00} \quad \text{(EC3-1-1: 6.9)}$$

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	1.6292e-03	m³
Plastic bending moment	$M_{pl,y,Rd}$	578.36	kNm
Unity check		0.01	-

$$M_{pl,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{1.6292 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 578.36[kNm] \quad \text{(EC3-1-1: 6.13)}$$

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{pl,y,Rd}} = \frac{|5.68[kNm]|}{578.36[kNm]} = \mathbf{0.01 \leq 1.00} \quad \text{(EC3-1-1: 6.12)}$$

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1.20	
Shear area	A_v	4.0765e-03	m²
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	835.52	kN
Unity check		0.00	-

$$V_{pl,z,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{4.0765 \cdot 10^{-3}[m^2] \times \frac{355.0[MPa]}{\sqrt{3}}}{1.00} = 835.52[kN] \quad \text{(EC3-1-1: 6.18)}$$

$$\text{Unity check} = \frac{|V_{z,Ed}|}{V_{c,z,Rd}} = \frac{|0.55[kN]|}{835.52[kN]} = \mathbf{0.00 \leq 1.00} \quad \text{(EC3-1-1: 6.17)}$$

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	2	
Total torsional moment	τ_{Ed}	0.5	MPa
Elastic shear resistance	τ_{Rd}	205.0	MPa
Unity check		0.00	-

$$\tau_{Ed} = |T_{Ed}| \times \tau_{Ed,unit} = |38.11| \times 1.436 \cdot 10^{1}[kN/m^2] = 0.5[MPa]$$

$$\tau_{Rd} = \frac{f_y}{\sqrt{3} \times \gamma_{M0}} = \frac{355.0[MPa]}{\sqrt{3} \times 1.00} = 205.0[MPa]$$

$$\text{Unity check} = \frac{\tau_{Ed}}{\tau_{Rd}} = \frac{0.5[MPa]}{205.0[MPa]} = \mathbf{0.00 \leq 1.00} \quad \text{(EC3-1-1: 6.23)}$$

Note: The unity check for torsion is lower than the limit value of 0.05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.31)

Plastic bending moment	$M_{pl,y,Rd}$	578.36	kNm
Unity check		0.01	-

$$M_{pl,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{1.6292 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 578.36[kNm] \quad \text{(EC3-1-1: 6.13)}$$

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{pl,y,Rd}} = \frac{|5.68[kNm]|}{578.36[kNm]} = \mathbf{0.01 \leq 1.00} \quad \text{(EC3-1-1: 6.31)}$$

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.
Note: Since the axial force satisfies both criteria (6.33) and (6.34) of EN 1993-1-1 article 6.2.9.1(4) its effect on the moment resistance about the y-y axis is neglected.

The member satisfies the section check.

Student version

Student version

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 2.571 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	SO	118	16	-2.290e+03	-2.290e+03								
3	SO	118	16	-2.290e+03	-2.290e+03								
4	I	225	9	-1.429e+03	4.148e+03	-0.3		0.5	25.0	56.5	65.3	58.0	1
5	SO	118	16	5.009e+03	5.009e+03	1.0	0.4	1.0	7.6	7.3	8.1	11.4	2
7	SO	118	16	5.009e+03	5.009e+03	1.0	0.4	1.0	7.6	7.3	8.1	11.4	2

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 2

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	6.000	6.000	m
Buckling factor	k	1.00	1.00	
Buckling length	l_{cr}	6.000	6.000	m
Critical Euler load	N_{cr}	13184.15	4024.53	kN
Slenderness	λ	44.15	79.91	
Relative slenderness	λ_{rel}	0.58	1.05	
Limit slenderness	$\lambda_{rel,0}$	0.20	0.20	

Note: The slenderness or compression force is such that Flexural Buckling effects may be ignored according to EN 1993-1-1 article 6.3.1.2(4).

$$N_{cr,y} = \frac{\pi^2 \times E \times I_y}{l_{cr,y}^2} = \frac{\pi^2 \times 210000.0[\mathrm{MPa}] \times 2.2900 \cdot 10^{-4}[\mathrm{m}^4]}{6.000[\mathrm{m}]^2} = 13184.15[\mathrm{kN}]$$

$$N_{cr,z} = \frac{\pi^2 \times E \times I_z}{l_{cr,z}^2} = \frac{\pi^2 \times 210000.0[\mathrm{MPa}] \times 6.9900 \cdot 10^{-5}[\mathrm{m}^4]}{6.000[\mathrm{m}]^2} = 4024.53[\mathrm{kN}]$$

$$\lambda_y = \frac{l_{cr,y}}{i_y} = \frac{6.000[\mathrm{m}]}{136[\mathrm{mm}]} = 44.15$$

$$\lambda_z = \frac{l_{cr,z}}{i_z} = \frac{6.000[\mathrm{m}]}{75[\mathrm{mm}]} = 79.91$$

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{44.15}{\pi \times \sqrt{\frac{210000.0[\mathrm{MPa}]}{355.0[\mathrm{MPa}]}}} = 0.58 \quad \text{(EC3-1-1: 6.50)}$$

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{79.91}{\pi \times \sqrt{\frac{210000.0[\mathrm{MPa}]}{355.0[\mathrm{MPa}]}}} = 1.05 \quad \text{(EC3-1-1: 6.50)}$$

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: For this I-section the Torsional(-Flexural) buckling resistance is higher than the resistance for Flexural buckling. Therefore Torsional(-Flexural) buckling is not printed on the output.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1 & 6.3.2.3 and formula (6.54)

LTB parameters			
Method for LTB curve		Alternative case	
Plastic section modulus	$W_{pl,y}$	1.6292e-03	m³
Elastic critical moment	M_{cr}	943.89	kNm
Relative slenderness	$\lambda_{rel,LT}$	0.78	
Limit slenderness	$\lambda_{rel,LT,0}$	0.40	

Note: The slenderness or bending moment is such that Lateral Torsional Buckling effects may be ignored according to EN 1993-1-1 article 6.3.2.2(4).

Mcr parameters			
LTB length	l_{LT}	6.000	m
Influence of load position		no influence	
Correction factor	k	1.00	
Correction factor	k_w	1.00	
LTB moment factor	C_1	1.13	
LTB moment factor	C_2	0.45	
LTB moment factor	C_3	0.53	
Shear centre distance	d_z	0	mm
Distance of load application	z_g	0	mm
Mono-symmetry constant	β_y	0	mm
Mono-symmetry constant	z_j	0	mm

$$M_{cr} = C_1 \times \frac{\pi^2 \times E \times I_z}{l_{LT}^2} \times \left[\sqrt{\left(\frac{k}{k_w}\right)^2 \times \frac{I_w}{I_z} + \frac{l_{LT}^2 \times G \times I_t}{\pi^2 \times E \times I_z} + (C_2 \times z_g - C_3 \times z_j)^2} - (C_2 \times z_g - C_3 \times z_j) \right] = 1.13$$

$$\times \frac{\pi^2 \times 210000.0[MPa] \times 6.9900 \cdot 10^{-5}[m^4]}{6.000[m]^2}$$

$$\times \left[\sqrt{\left(\frac{1.00}{1.00}\right)^2 \times \frac{1.5124 \cdot 10^{-6}[m^6]}{6.9900 \cdot 10^{-5}[m^4]} + \frac{6.000[m]^2 \times 80769.2[MPa] \times 1.0800 \cdot 10^{-6}[m^4]}{\pi^2 \times 210000.0[MPa] \times 6.9900 \cdot 10^{-5}[m^4]} + (0.45 \times 0[mm] - 0.53 \times 0[mm])^2} - (0.45 \times 0[mm] - 0.53 \times 0[mm]) \right]$$

$$= 943.89[kNm]$$

$$\lambda_{rel,LT} = \sqrt{\frac{W_{pl,y} \times f_y}{M_{cr}}} = \sqrt{\frac{1.6292 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{943.89[kNm]}} = 0.78$$

Note: C parameters are determined according to ECCS 119 2006 / Galea 2002.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section area	A	1.2400e-02	m^2
Plastic section modulus	$W_{pl,y}$	1.6292e-03	m^3
Design compression force	N_{Ed}	16.91	kN
Design bending moment (maximum)	$M_{y,Ed}$	5.68	kNm
Design bending moment (maximum)	$M_{z,Ed}$	0.00	kNm
Characteristic compression resistance	N_{Rk}	4402.00	kN
Characteristic moment resistance	$M_{y,Rk}$	578.36	kNm
Reduction factor	χ_y	1.00	
Reduction factor	χ_z	1.00	
Modified reduction factor	$\chi_{LT,mod}$	1.00	
Interaction factor	k_{yy}	0.90	
Interaction factor	k_{zy}	0.54	

Maximum moment $M_{y,Ed}$ is derived from beam B776 position 2.571 m.
Maximum moment $M_{z,Ed}$ is derived from beam B776 position 0.000 m.

Student version

Interaction method 2 parameters			
Method for interaction factors		Table B.1	
Sway type y		sway	
Equivalent moment factor	C_{my}	0.90	
Resulting load type LT		line load q	
End moment	$M_{h,LT}$	0.00	kNm
Field moment	$M_{s,LT}$	5.68	kNm
Factor	$\alpha_{h,LT}$	0.00	
Ratio of end moments	ψ_{LT}	1.00	
Equivalent moment factor	C_{mLT}	0.95	

Unity check (6.61) = 0.00 + 0.01 + 0.00 = 0.01 -
Unity check (6.62) = 0.00 + 0.01 + 0.00 = 0.01 -

$$C_{my} = 0.90$$

$$\alpha_{h,LT} = \frac{M_{h,LT}}{M_{s,LT}} = \frac{0.00[kNm]}{5.68[kNm]} = 0.00$$

$$C_{mLT} = 0.95 + 0.05 \times \alpha_{h,LT} = 0.95 + 0.05 \times 0.00 = 0.95$$

$$N_{Rk} = A \times f_y = 1.2400 \cdot 10^{-2}[m^2] \times 355.0[MPa] = 4402.00[kN]$$

$$M_{y,Rk} = W_{pl,y} \times f_y = 1.6292 \cdot 10^{-3}[m^3] \times 355.0[MPa] = 578.36[kNm]$$

$$k_{yy} = \min\left\{ C_{my} \times \left[1 + (\lambda_{rel,y} - 0.2) \times \frac{N_{Ed}}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}}\right], C_{my} \times \left(1 + 0.8 \times \frac{N_{Ed}}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}}\right)\right\}$$

$$= \min\left\{ 0.90 \times \left[1 + (0.58 - 0.2) \times \frac{16.91[kN]}{1.00 \times \frac{4402.00[kN]}{1.00}}\right], 0.90 \times \left(1 + 0.8 \times \frac{16.91[kN]}{1.00 \times \frac{4402.00[kN]}{1.00}}\right)\right\} = \min\{0.90, 0.90\} = 0.90$$

$$k_{zy} = 0.6 \times k_{yy} = 0.6 \times 0.90 = 0.54$$

$$\text{Unity check (6.61)} = \frac{|N_{Ed}|}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \times \frac{|M_{y,Ed}| + |\Delta M_{y,Ed}|}{\chi_{LT,mod} \times \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \times \frac{|M_{z,Ed}| + |\Delta M_{z,Ed}|}{\frac{M_{z,Rk}}{\gamma_{M1}}}$$

$$= \frac{|16.91[kN]|}{1.00 \times \frac{4402.00[kN]}{1.00}} + 0.90 \times \frac{|5.68[kNm]| + |0.00[kNm]|}{1.00 \times \frac{578.36[kNm]}{1.00}} + 0.60 \times \frac{|0.00[kNm]| + |0.00[kNm]|}{\frac{251.46[kNm]}{1.00}} = \mathbf{0.01} \leq \mathbf{1.00} \quad \text{(EC3-1-1: 6.61)}$$

Student version

$$\text{Unity check (6.62)} = \frac{|N_{Ed}|}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} \times \frac{|M_{y,Ed}| + |\Delta M_{y,Ed}|}{\chi_{LT,mod} \times \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \times \frac{|M_{z,Ed}| + |\Delta M_{z,Ed}|}{\frac{M_{z,Rk}}{\gamma_{M1}}}$$

$$= \frac{|16.91[kN]|}{1.00 \times \frac{4402.00[kN]}{1.00}} + 0.54 \times \frac{|5.68[kNm]| + |0.00[kNm]|}{1.00 \times \frac{578.36[kNm]}{1.00}} + 1.01 \times \frac{|0.00[kNm]| + |0.00[kNm]|}{\frac{251.46[kNm]}{1.00}} = \mathbf{0.01} \leq \mathbf{1.00} \quad \text{(EC3-1-1: 6.62)}$$

$$\text{Unity check} = \max(\text{Unity check (6.61)}, \text{Unity check (6.62)}) = \max(0.01, 0.01) = \mathbf{0.01} \leq \mathbf{1.00}$$

Shear Buckling check
According to EN 1993-1-5 article 5 & 7.1 and formula (5.10) & (7.1)

Shear Buckling parameters			
Buckling field length	a	6.000	m
Web		unstiffened	
Web height	h_w	279	mm
Web thickness	t	9	mm
Material coefficient	ε	0.81	
Shear correction factor	η	1.20	

Shear Buckling verification		
Web slenderness	h_w/t	31.00
Web slenderness limit		48.82

$$h_w/t = \frac{h_w}{t} = \frac{279[mm]}{9[mm]} = 31.00$$

$$\text{limit } h_w/t = \frac{72 \times \varepsilon}{\eta} = \frac{72 \times 0.81}{1.20} = 48.82$$

Note: The web slenderness is such that Shear Buckling effects may be ignored according to EN 1993-1-5 article 5.1(2).

The member satisfies the stability check.

2.2.3.3.4. Column Check

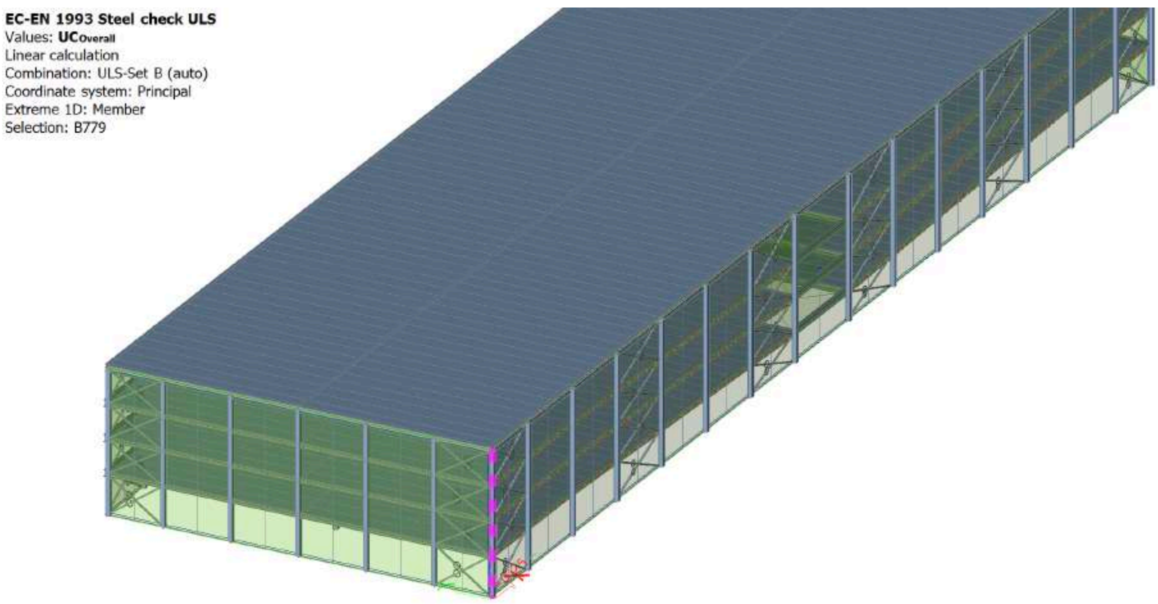


Figure 2.2.3.3. 7:Corner column

EN 1993-1-1 Code Check
National annex: Czech CSN-EN NA

Member B779	**12.600 / 12.600 m**	**SHS350/350/14.2**	**S 355**	**ULS-Set B (auto)**	**0.18 -**

Combination key
ULS-Set B (auto) / 1.15*LC1 + 1.15*LC2 + 1.05*LC3 + 1.05*LC4 + 1.50*3DWind4

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{M2} for resistance of net sections	1.25

Material			
Yield strength	f_y	355.0	MPa
Ultimate strength	f_u	490.0	MPa
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 12.600 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-636.53	kN
Shear force	$V_{y,Ed}$	-8.80	kN
Shear force	$V_{z,Ed}$	-33.66	kN
Torsion	T_{Ed}	10.61	kNm
Bending moment	$M_{y,Ed}$	-54.62	kNm
Bending moment	$M_{z,Ed}$	-10.41	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	307	14	6.435e+04	5.526e+04	0.9		1.0	21.6	22.8	27.7	32.5	1
3	I	307	14	5.264e+04	4.950e+03	0.1		1.0	21.6	22.8	27.7	45.1	1
5	I	307	14	3.167e+03	1.225e+04	0.3		1.0	21.6	22.8	27.7	41.6	1
7	I	307	14	1.488e+04	6.256e+04	0.2		1.0	21.6	22.8	27.7	42.0	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	1.8900e-02	m²
Compression resistance	$N_{c,Rd}$	6709.50	kN
Unity check		0.09	-

$$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M0}} = \frac{1.8900 \cdot 10^{-2}[m^2] \times 355.0[MPa]}{1.00} = 6709.50[kN] \quad \text{(EC3-1-1: 6.10)}$$

Student version

Unity check $= \frac{|N_{Ed}|}{N_{c,Rd}} = \frac{|-636.53[kN]|}{6709.50[kN]} = \mathbf{0.09 \leq 1.00}$ (EC3-1-1: 6.9)

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	2.3640e-03	m^3
Plastic bending moment	$M_{pl,y,Rd}$	839.22	kNm
Unity check		0.07	-

$$M_{pl,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{2.3640 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 839.22[kNm]$$ (EC3-1-1: 6.13)

Unity check $= \frac{|M_{y,Ed}|}{M_{pl,y,Rd}} = \frac{|-54.62[kNm]|}{839.22[kNm]} = \mathbf{0.07 \leq 1.00}$ (EC3-1-1: 6.12)

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	2.3640e-03	m^3
Plastic bending moment	$M_{pl,z,Rd}$	839.22	kNm
Unity check		0.01	-

$$M_{pl,z,Rd} = \frac{W_{pl,z} \times f_y}{\gamma_{M0}} = \frac{2.3640 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 839.22[kNm]$$ (EC3-1-1: 6.13)

Unity check $= \frac{|M_{z,Ed}|}{M_{pl,z,Rd}} = \frac{|-10.41[kNm]|}{839.22[kNm]} = \mathbf{0.01 \leq 1.00}$ (EC3-1-1: 6.12)

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1.20	
Shear area	A_v	9.4500e-03	m^2
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	1936.87	kN
Unity check		0.00	-

$$V_{pl,y,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{9.4500 \cdot 10^{-3}[m^2] \times \frac{355.0[MPa]}{\sqrt{3}}}{1.00} = 1936.87[kN]$$ (EC3-1-1: 6.18)

Unity check $= \frac{|V_{y,Ed}|}{V_{c,y,Rd}} = \frac{|-8.80[kN]|}{1936.87[kN]} = \mathbf{0.00 \leq 1.00}$ (EC3-1-1: 6.17)

Shear check for V_z
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1.20	
Shear area	A_v	9.4500e-03	m^2
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	1936.87	kN
Unity check		0.02	-

$$V_{pl,z,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{9.4500 \cdot 10^{-3}[m^2] \times \frac{355.0[MPa]}{\sqrt{3}}}{1.00} = 1936.87[kN]$$ (EC3-1-1: 6.18)

Unity check $= \frac{|V_{z,Ed}|}{V_{c,z,Rd}} = \frac{|-33.66[kN]|}{1936.87[kN]} = \mathbf{0.02 \leq 1.00}$ (EC3-1-1: 6.17)

Torsion check
According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	1	
Total torsional moment	τ_{Ed}	3.3	MPa
Elastic shear resistance	τ_{Rd}	205.0	MPa
Unity check		0.02	-

$$\tau_{Ed} = |T_{Ed}| \times \tau_{Ed,unit} = |10612.76| \times 3.123 \cdot 10^{-1}[kN/m^2] = 3.3[MPa]$$

$$\tau_{Rd} = \frac{f_y}{\sqrt{3} \times \gamma_{M0}} = \frac{355.0[MPa]}{\sqrt{3} \times 1.00} = 205.0[MPa]$$

Unity check $= \frac{\tau_{Ed}}{\tau_{Rd}} = \frac{3.3[MPa]}{205.0[MPa]} = \mathbf{0.02 \leq 1.00}$ (EC3-1-1: 6.23)

Note: The unity check for torsion is lower than the limit value of 0.05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check
According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Design plastic moment resistance reduced due to N_{Ed}	$M_{N,y,Rd}$	839.22	kNm
Exponent of bending ratio y	α	1.68	
Design plastic moment resistance reduced due to N_{Ed}	$M_{N,z,Rd}$	839.22	kNm
Exponent of bending ratio z	β	1.68	

Student version

Unity check (6.41) = 0.01 + 0.00 = 0.01 -

$$M_{N,y,Rd} = \min\left[\frac{M_{pl,y,Rd} \times (1-n)}{1-0.5\times ratio_{A,w}}, M_{pl,y,Rd}\right] = \min\left[\frac{839.22[kNm]\times(1-0.09)}{1-0.5\times 0.47}, 839.22[kNm]\right] = \min[995.60[kNm], 839.22[kNm]] = 839.22[kNm] \quad \text{(EC3-1-1: 6.39)}$$

$$\alpha = \max\left[\min\left(\frac{1.66}{1-1.13\times n^2}, 6\right), 1\right] = \max\left[\min\left(\frac{1.66}{1-1.13\times 0.09^2}, 6\right), 1\right] = \max[\min(1.68, 6), 1] = 1.68$$

$$M_{N,z,Rd} = \min\left[\frac{M_{pl,z,Rd} \times (1-n)}{1-0.5\times ratio_{A,f}}, M_{pl,z,Rd}\right] = \min\left[\frac{839.22[kNm]\times(1-0.09)}{1-0.5\times 0.47}, 839.22[kNm]\right] = \min[995.60[kNm], 839.22[kNm]] = 839.22[kNm] \quad \text{(EC3-1-1: 6.40)}$$

$$\beta = \max\left[\min\left(\frac{1.66}{1-1.13\times n^2}, 6\right), 1\right] = \max\left[\min\left(\frac{1.66}{1-1.13\times 0.09^2}, 6\right), 1\right] = \max[\min(1.68, 6), 1] = 1.68$$

$$\text{Unity check} = \left(\frac{|M_{y,Ed}|}{M_{N,y,Rd}}\right)^{\alpha} + \left(\frac{|M_{z,Ed}|}{M_{N,z,Rd}}\right)^{\beta} = \left(\frac{|-54.62[kNm]|}{839.22[kNm]}\right)^{1.68} + \left(\frac{|-10.41[kNm]|}{839.22[kNm]}\right)^{1.68} = \mathbf{0.01 \leq 1.00} \quad \text{(EC3-1-1: 6.41)}$$

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 9.100 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	307	14	-3.420e+03	1.438e+04	-0.2		0.8	21.6	29.8	35.8	54.2	1
3	I	307	14	1.757e+04	6.877e+04	0.3		1.0	21.6	22.8	27.7	41.7	1
5	I	307	14	7.031e+04	5.251e+04	0.7		1.0	21.6	22.8	27.7	33.9	1
7	I	307	14	4.932e+04	-1.877e+03	0.0		1.0	21.6	23.9	28.9	48.3	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	3.500	3.500	m
Buckling factor	k	1.95	0.58	
Buckling length	l_{cr}	6.840	2.020	m
Critical Euler load	N_{cr}	15597.86	178802.49	kN
Slenderness	λ	50.11	14.80	
Relative slenderness	λ_{rel}	0.66	0.19	
Limit slenderness	$\lambda_{rel,0}$	0.20	0.20	
Buckling curve		a	a	
Imperfection	α	0.21	0.21	
Reduction factor	χ	0.87	1.00	
Buckling resistance	$N_{b,Rd}$	5820.57	6709.50	kN

Flexural Buckling verification			
Cross-section area	A	1.8900e-02	m²
Buckling resistance	$N_{b,Rd}$	5820.57	kN
Unity check		0.11	-

$$N_{cr,y} = \frac{\pi^2 \times E \times I_y}{l_{cr,y}^2} = \frac{\pi^2 \times 210000.0[MPa] \times 3.5210\cdot 10^{-4}[m^4]}{6.840[m]^2} = 15597.86[kN]$$

$$N_{cr,z} = \frac{\pi^2 \times E \times I_z}{l_{cr,z}^2} = \frac{\pi^2 \times 210000.0[MPa] \times 3.5210\cdot 10^{-4}[m^4]}{2.020[m]^2} = 178802.49[kN]$$

$$\lambda_y = \frac{l_{cr,y}}{i_y} = \frac{6.840[m]}{136[mm]} = 50.11$$

$$\lambda_z = \frac{l_{cr,z}}{i_z} = \frac{2.020[m]}{136[mm]} = 14.80$$

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{50.11}{\pi \times \sqrt{\frac{210000.0[MPa]}{355.0[MPa]}}} = 0.66 \quad \text{(EC3-1-1: 6.50)}$$

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{14.80}{\pi \times \sqrt{\frac{210000.0[MPa]}{355.0[MPa]}}} = 0.19 \quad \text{(EC3-1-1: 6.50)}$$

$$\varphi_y = 0.5 \times \left[1 + \alpha_y \times (\lambda_{rel,y} - \lambda_{rel,y,0}) + \lambda_{rel,y}^2\right] = 0.5 \times \left[1 + 0.21 \times (0.66 - 0.20) + 0.66^2\right] = 0.76$$

$$\varphi_z = 0.5 \times \left[1 + \alpha_z \times (\lambda_{rel,z} - \lambda_{rel,z,0}) + \lambda_{rel,z}^2\right] = 0.5 \times \left[1 + 0.21 \times (0.19 - 0.20) + 0.19^2\right] = 0.52$$

Student version

$$\chi_y = \min\left(\frac{1}{\varphi_y + \sqrt{\varphi_y^2 - \lambda_{rel,y}^2}}, \frac{1}{\lambda_{rel,y}^2}, 1\right) = \min\left(\frac{1}{0.76 + \sqrt{0.76^2 - 0.66^2}}, \frac{1}{0.66^2}, 1\right) = \min(0.87, 2.32, 1) = 0.87 \quad \text{(EC3-1-1: 6.49)}$$

$$\chi_z = \min\left(\frac{1}{\varphi_z + \sqrt{\varphi_z^2 - \lambda_{rel,z}^2}}, \frac{1}{\lambda_{rel,z}^2}, 1\right) = \min\left(\frac{1}{0.52 + \sqrt{0.52^2 - 0.19^2}}, \frac{1}{0.19^2}, 1\right) = \min(1.00, 26.65, 1) = 1.00 \quad \text{(EC3-1-1: 6.49)}$$

$$N_{b,y,Rd} = \frac{\chi_y \times A \times f_y}{\gamma_{M1}} = \frac{0.87 \times 1.8900 \cdot 10^{-2}[m^2] \times 355.0[MPa]}{1.00} = 5820.57[kN] \quad \text{(EC3-1-1: 6.47)}$$

$$N_{b,z,Rd} = \frac{\chi_z \times A \times f_y}{\gamma_{M1}} = \frac{1.00 \times 1.8900 \cdot 10^{-2}[m^2] \times 355.0[MPa]}{1.00} = 6709.50[kN] \quad \text{(EC3-1-1: 6.47)}$$

$$N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}) = \min(5820.57[kN], 6709.50[kN]) = 5820.57[kN]$$

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{b,Rd}} = \frac{|-636.53[kN]|}{5820.57[kN]} = \mathbf{0.11 \leq 1.00} \quad \text{(EC3-1-1: 6.46)}$$

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: The cross-section concerns a RHS section which is not susceptible to Torsional(-Flexural) Buckling.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1
Note: The cross-section concerns an RHS section with 'h / b < 10 / $\lambda_{rel,z}$'.
This section is thus not susceptible to Lateral Torsional Buckling.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section area	A	1.8900e-02	m^2
Plastic section modulus	$W_{pl,y}$	2.3640e-03	m^3
Plastic section modulus	$W_{pl,z}$	2.3640e-03	m^3
Design compression force	N_{Ed}	636.53	kN
Design bending moment (maximum)	$M_{y,Ed}$	58.64	kNm
Design bending moment (maximum)	$M_{z,Ed}$	20.39	kNm
Characteristic compression resistance	N_{Rk}	6709.50	kN
Characteristic moment resistance	$M_{y,Rk}$	839.22	kNm
Characteristic moment resistance	$M_{z,Rk}$	839.22	kNm
Reduction factor	χ_y	0.87	
Reduction factor	χ_z	1.00	
Reduction factor	χ_{LT}	1.00	
Interaction factor	k_{yy}	0.94	
Interaction factor	k_{yz}	0.24	
Interaction factor	k_{zy}	0.57	
Interaction factor	k_{zz}	0.40	

Maximum moment $M_{y,Ed}$ is derived from beam B779 position 9.100 m.
Maximum moment $M_{z,Ed}$ is derived from beam B779 position 9.100 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.1	
Sway type y		sway	
Equivalent moment factor	C_{my}	0.90	
Resulting load type z		linear moment M	
Ratio of end moments	ψ_z	-0.51	
Equivalent moment factor	C_{mz}	0.40	
Resulting load type LT		line load q	
End moment	$M_{h,LT}$	58.64	kNm
Field moment	$M_{s,LT}$	3.14	kNm
Factor	$\alpha_{s,LT}$	0.05	
Ratio of end moments	ψ_{LT}	-0.93	
Equivalent moment factor	C_{mLT}	0.40	

Unity check (6.61) = 0.11 + 0.07 + 0.01 = 0.18 -
Unity check (6.62) = 0.09 + 0.04 + 0.01 = 0.14 -

$$C_{my} = 0.90$$

$$C_{mz} = \max(0.6 + 0.4 \times \psi_z, 0.4) = \max(0.6 + 0.4 \times -0.51, 0.4) = \max(0.40, 0.4) = 0.40$$

$$\alpha_{s,LT} = \frac{M_{s,LT}}{M_{h,LT}} = \frac{3.14[kNm]}{58.64[kNm]} = 0.05$$

$$C_{mLT} = \max(0.2 + 0.8 \times \alpha_{s,LT}, 0.4) = \max(0.2 + 0.8 \times 0.05, 0.4) = \max(0.24, 0.4) = 0.40$$

$$N_{Rk} = A \times f_y = 1.8900 \cdot 10^{-2}[m^2] \times 355.0[MPa] = 6709.50[kN]$$

$$M_{y,Rk} = W_{pl,y} \times f_y = 2.3640 \cdot 10^{-3}[m^3] \times 355.0[MPa] = 839.22[kNm]$$

student version
student version
student version

$M_{z,Rk} = W_{pl,z} \times f_y = 2.3640 \cdot 10^{-3}[m^3] \times 355.0[MPa] = 839.22[kNm]$

Student version

$$k_{yy} = \min\left\{C_{my} \times \left[1 + (\lambda_{rel,y} - 0.2) \times \frac{N_{Ed}}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}}\right], C_{my} \times \left(1 + 0.8 \times \frac{N_{Ed}}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}}\right)\right\}$$

$$= \min\left\{0.90 \times \left[1 + (0.66 - 0.2) \times \frac{636.53[kN]}{0.87 \times \frac{6709.50[kN]}{1.00}}\right], 0.90 \times \left(1 + 0.8 \times \frac{636.53[kN]}{0.87 \times \frac{6709.50[kN]}{1.00}}\right)\right\} = \min\{0.94, 0.98\} = 0.94$$

$k_{yz} = 0.6 \times k_{zz} = 0.6 \times 0.40 = 0.24$

$k_{zy} = 0.6 \times k_{yy} = 0.6 \times 0.94 = 0.57$

$$k_{zz} = \min\left[C_{mz}, C_{mz} \times \left(1 + 0.8 \times \frac{N_{Ed}}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}}\right)\right] = \min\left[0.40, 0.40 \times \left(1 + 0.8 \times \frac{636.53[kN]}{1.00 \times \frac{6709.50[kN]}{1.00}}\right)\right] = \min[0.40, 0.43] = 0.40$$

$$\text{Unity check (6.61)} = \frac{|N_{Ed}|}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \times \frac{|M_{y,Ed}| + |\Delta M_{y,Ed}|}{\chi_{LT} \times \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \times \frac{|M_{z,Ed}| + |\Delta M_{z,Ed}|}{\frac{M_{z,Rk}}{\gamma_{M1}}}$$

$$= \frac{|636.53[kN]|}{0.87 \times \frac{6709.50[kN]}{1.00}} + 0.94 \times \frac{|58.64[kNm]| + |0.00[kNm]|}{1.00 \times \frac{839.22[kNm]}{1.00}} + 0.24 \times \frac{|20.39[kNm]| + |0.00[kNm]|}{\frac{839.22[kNm]}{1.00}} = \mathbf{0.18 \leq 1.00} \quad \text{(EC3-1-1: 6.61)}$$

$$\text{Unity check (6.62)} = \frac{|N_{Ed}|}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} \times \frac{|M_{y,Ed}| + |\Delta M_{y,Ed}|}{\chi_{LT} \times \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \times \frac{|M_{z,Ed}| + |\Delta M_{z,Ed}|}{\frac{M_{z,Rk}}{\gamma_{M1}}}$$

$$= \frac{|636.53[kN]|}{1.00 \times \frac{6709.50[kN]}{1.00}} + 0.57 \times \frac{|58.64[kNm]| + |0.00[kNm]|}{1.00 \times \frac{839.22[kNm]}{1.00}} + 0.40 \times \frac{|20.39[kNm]| + |0.00[kNm]|}{\frac{839.22[kNm]}{1.00}} = \mathbf{0.14 \leq 1.00} \quad \text{(EC3-1-1: 6.62)}$$

Unity check = max (Unity check (6.61), Unity check (6.62)) = max (0.18, 0.14) = $\mathbf{0.18 \leq 1.00}$

The member satisfies the stability check.

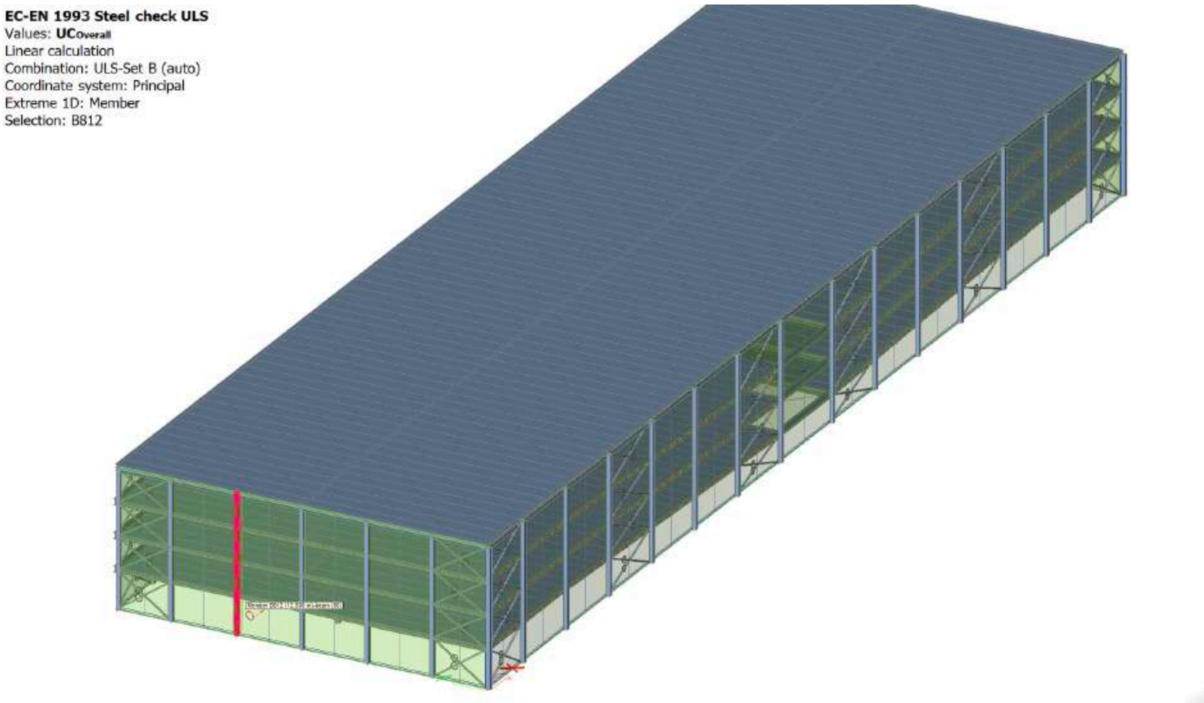


Figure 2.2.3.3. 8:Edge column

EN 1993-1-1 Code Check
National annex: Czech CSN-EN NA

Member B812	**12.600 / 12.600 m**	**SHS350/350/14.2**	**S 355**	**ULS-Set B (auto)**	**0.39 -**

Combination key
ULS-Set B (auto) / 1.15*LC1 + 1.15*LC2 + 1.05*LC3 + 1.05*LC4 + 1.50*3DWind16

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{M2} for resistance of net sections	1.25

Material			
Yield strength	f_y	355.0	MPa
Ultimate strength	f_u	490.0	MPa
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 12.600 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-111.36	kN
Shear force	$V_{y,Ed}$	-1.82	kN
Shear force	$V_{z,Ed}$	109.07	kN
Torsion	T_{Ed}	14.52	kNm
Bending moment	$M_{y,Ed}$	306.60	kNm
Bending moment	$M_{z,Ed}$	-5.14	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	307	14	-1.381e+05	-1.426e+05								
3	I	307	14	-1.304e+05	1.373e+05	-0.9		0.5	21.6	56.3	65.1	95.6	1
5	I	307	14	1.499e+05	1.544e+05	1.0		1.0	21.6	22.8	27.7	31.2	1
7	I	307	14	1.422e+05	-1.255e+05	-0.9		0.5	21.6	53.3	62.0	89.2	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	1.8900e-02	m²
Compression resistance	$N_{c,Rd}$	6709.50	kN
Unity check		0.02	-

$$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M0}} = \frac{1.8900 \cdot 10^{-2}[m^2] \times 355.0[MPa]}{1.00} = 6709.50[kN]$$ (EC3-1-1: 6.10)

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{c,Rd}} = \frac{|-111.36[kN]|}{6709.50[kN]} = \mathbf{0.02 \leq 1.00}$$ (EC3-1-1: 6.9)

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	2.3640e-03	m³
Plastic bending moment	$M_{pl,y,Rd}$	839.22	kNm
Unity check		0.37	-

$$M_{pl,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{2.3640 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 839.22[kNm]$$ (EC3-1-1: 6.13)

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{pl,y,Rd}} = \frac{|306.60[kNm]|}{839.22[kNm]} = \mathbf{0.37 \leq 1.00}$$ (EC3-1-1: 6.12)

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	2.3640e-03	m³
Plastic bending moment	$M_{pl,z,Rd}$	839.22	kNm
Unity check		0.01	-

$$M_{pl,z,Rd} = \frac{W_{pl,z} \times f_y}{\gamma_{M0}} = \frac{2.3640 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 839.22[kNm]$$ (EC3-1-1: 6.13)

$$\text{Unity check} = \frac{|M_{z,Ed}|}{M_{pl,z,Rd}} = \frac{|-5.14[kNm]|}{839.22[kNm]} = \mathbf{0.01 \leq 1.00}$$ (EC3-1-1: 6.12)

Shear check for V_y
According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1.20	
Shear area	A_v	9.4500e-03	m²
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	1936.87	kN
Unity check		0.00	-

Student version

Shear check for V_y

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1.20	
Shear area	A_v	9.4500e-03	m^2
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	1936.87	kN
Unity check		0.00	-

Student version

$$V_{pl,y,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{9.4500 \cdot 10^{-3}[m^2] \times \frac{355.0[MPa]}{\sqrt{3}}}{1.00} = 1936.87[kN] \quad \text{(EC3-1-1: 6.18)}$$

$$\text{Unity check} = \frac{|V_{y,Ed}|}{V_{c,y,Rd}} = \frac{|-1.82[kN]|}{1936.87[kN]} = \mathbf{0.00 \leq 1.00} \quad \text{(EC3-1-1: 6.17)}$$

Shear check for V_z

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1.20	
Shear area	A_v	9.4500e-03	m^2
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	1936.87	kN
Unity check		0.06	-

$$V_{pl,z,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{9.4500 \cdot 10^{-3}[m^2] \times \frac{355.0[MPa]}{\sqrt{3}}}{1.00} = 1936.87[kN] \quad \text{(EC3-1-1: 6.18)}$$

$$\text{Unity check} = \frac{|V_{z,Ed}|}{V_{c,z,Rd}} = \frac{|109.07[kN]|}{1936.87[kN]} = \mathbf{0.06 \leq 1.00} \quad \text{(EC3-1-1: 6.17)}$$

Torsion check

According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	1	
Total torsional moment	τ_{Ed}	4.5	MPa
Elastic shear resistance	τ_{Rd}	205.0	MPa
Unity check		0.02	-

$$\tau_{Ed} = |T_{Ed}| \times \tau_{Ed,unit} = |14521.49| \times 3.123 \cdot 10^{-1}[kN/m^2] = 4.5[MPa]$$

$$\tau_{Rd} = \frac{f_y}{\sqrt{3} \times \gamma_{M0}} = \frac{355.0[MPa]}{\sqrt{3} \times 1.00} = 205.0[MPa]$$

$$\text{Unity check} = \frac{\tau_{Ed}}{\tau_{Rd}} = \frac{4.5[MPa]}{205.0[MPa]} = \mathbf{0.02 \leq 1.00} \quad \text{(EC3-1-1: 6.23)}$$

Note: The unity check for torsion is lower than the limit value of 0.05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Student version

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Design plastic moment resistance reduced due to N_{Ed}	$M_{N,y,Rd}$	839.22	kNm
Exponent of bending ratio y	α	1.66	
Design plastic moment resistance reduced due to N_{Ed}	$M_{N,z,Rd}$	839.22	kNm
Exponent of bending ratio z	β	1.66	

Student version

Unity check (6.41) = 0.19 + 0.00 = 0.19 -

$$M_{N,y,Rd} = \min\left[\frac{M_{pl,y,Rd} \times (1-n)}{1 - 0.5 \times \text{ratio}_{A,w}}, M_{pl,y,Rd}\right] = \min\left[\frac{839.22[kNm] \times (1-0.02)}{1 - 0.5 \times 0.47}, 839.22[kNm]\right] = \min[1081.69[kNm], 839.22[kNm]] = 839.22[kNm] \quad \text{(EC3-1-1: 6.39)}$$

$$\alpha = \max\left[\min\left(\frac{1.66}{1 - 1.13 \times n^2}, 6\right), 1\right] = \max\left[\min\left(\frac{1.66}{1 - 1.13 \times 0.02^2}, 6\right), 1\right] = \max[\min(1.66, 6), 1] = 1.66$$

$$M_{N,z,Rd} = \min\left[\frac{M_{pl,z,Rd} \times (1-n)}{1 - 0.5 \times \text{ratio}_{A,f}}, M_{pl,z,Rd}\right] = \min\left[\frac{839.22[kNm] \times (1-0.02)}{1 - 0.5 \times 0.47}, 839.22[kNm]\right] = \min[1081.69[kNm], 839.22[kNm]] = 839.22[kNm] \quad \text{(EC3-1-1: 6.40)}$$

$$\beta = \max\left[\min\left(\frac{1.66}{1 - 1.13 \times n^2}, 6\right), 1\right] = \max\left[\min\left(\frac{1.66}{1 - 1.13 \times 0.02^2}, 6\right), 1\right] = \max[\min(1.66, 6), 1] = 1.66$$

$$\text{Unity check} = \left(\frac{|M_{y,Ed}|}{M_{N,y,Rd}}\right)^{\alpha} + \left(\frac{|M_{z,Ed}|}{M_{N,z,Rd}}\right)^{\beta} = \left(\frac{|306.60[kNm]|}{839.22[kNm]}\right)^{1.66} + \left(\frac{|-5.14[kNm]|}{839.22[kNm]}\right)^{1.66} = \mathbf{0.19 \leq 1.00} \quad \text{(EC3-1-1: 6.41)}$$

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design

Decisive position for stability classification: 12.600 m

Classification according to EN 1993-1-1 article 5.5.2

Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	307	14	-1.381e+05	-1.426e+05								
3	I	307	14	-1.304e+05	1.373e+05	-0.9		0.5	21.6	56.3	65.1	95.6	1
5	I	307	14	1.499e+05	1.544e+05	1.0		1.0	21.6	22.8	27.7	31.2	1
7	I	307	14	1.422e+05	-1.255e+05	-0.9		0.5	21.6	53.3	62.0	89.2	1

Student version

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	12.600	12.600	m
Buckling factor	k	1.33	0.50	
Buckling length	l_{cr}	16.729	6.313	m
Critical Euler load	N_{cr}	2607.66	18313.53	kN
Slenderness	λ	122.56	46.25	
Relative slenderness	λ_{rel}	1.60	0.61	
Limit slenderness	$\lambda_{rel,0}$	0.20	0.20	
Buckling curve		a	a	
Imperfection	α	0.21	0.21	
Reduction factor	χ	0.33	0.89	
Buckling resistance	$N_{b,Rd}$	2225.94	5957.91	kN

Flexural Buckling verification			
Cross-section area	A	1.8900e-02	m²
Buckling resistance	$N_{b,Rd}$	2225.94	kN
Unity check		0.05	-

$$N_{cr,y} = \frac{\pi^2 \times E \times I_y}{l_{cr,y}^2} = \frac{\pi^2 \times 210000.0[MPa] \times 3.5210 \cdot 10^{-4}[m^4]}{16.729[m]^2} = 2607.66[kN]$$

$$N_{cr,z} = \frac{\pi^2 \times E \times I_z}{l_{cr,z}^2} = \frac{\pi^2 \times 210000.0[MPa] \times 3.5210 \cdot 10^{-4}[m^4]}{6.313[m]^2} = 18313.53[kN]$$

$$\lambda_y = \frac{l_{cr,y}}{i_y} = \frac{16.729[m]}{136[mm]} = 122.56$$

$$\lambda_z = \frac{l_{cr,z}}{i_z} = \frac{6.313[m]}{136[mm]} = 46.25$$

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{122.56}{\pi \times \sqrt{\frac{210000.0[MPa]}{355.0[MPa]}}} = 1.60$$ (EC3-1-1: 6.50)

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{46.25}{\pi \times \sqrt{\frac{210000.0[MPa]}{355.0[MPa]}}} = 0.61$$ (EC3-1-1: 6.50)

$$\varphi_y = 0.5 \times \left[1 + \alpha_y \times (\lambda_{rel,y} - \lambda_{rel,y,0}) + \lambda_{rel,y}^2\right] = 0.5 \times \left[1 + 0.21 \times (1.60 - 0.20) + 1.60^2\right] = 1.93$$

$$\varphi_z = 0.5 \times \left[1 + \alpha_z \times (\lambda_{rel,z} - \lambda_{rel,z,0}) + \lambda_{rel,z}^2\right] = 0.5 \times \left[1 + 0.21 \times (0.61 - 0.20) + 0.61^2\right] = 0.73$$

$$\chi_y = \min\left(\frac{1}{\varphi_y + \sqrt{\varphi_y^2 - \lambda_{rel,y}^2}}, \frac{1}{\lambda_{rel,y}^2}, 1\right) = \min\left(\frac{1}{1.93 + \sqrt{1.93^2 - 1.60^2}}, \frac{1}{1.60^2}, 1\right) = \min(0.33, 0.39, 1) = 0.33$$ (EC3-1-1: 6.49)

$$\chi_z = \min\left(\frac{1}{\varphi_z + \sqrt{\varphi_z^2 - \lambda_{rel,z}^2}}, \frac{1}{\lambda_{rel,z}^2}, 1\right) = \min\left(\frac{1}{0.73 + \sqrt{0.73^2 - 0.61^2}}, \frac{1}{0.61^2}, 1\right) = \min(0.89, 2.73, 1) = 0.89$$ (EC3-1-1: 6.49)

$$N_{b,y,Rd} = \frac{\chi_y \times A \times f_y}{\gamma_{M1}} = \frac{0.33 \times 1.8900 \cdot 10^{-2}[m^2] \times 355.0[MPa]}{1.00} = 2225.94[kN]$$ (EC3-1-1: 6.47)

$$N_{b,z,Rd} = \frac{\chi_z \times A \times f_y}{\gamma_{M1}} = \frac{0.89 \times 1.8900 \cdot 10^{-2}[m^2] \times 355.0[MPa]}{1.00} = 5957.91[kN]$$ (EC3-1-1: 6.47)

$$N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}) = \min(2225.94[kN], 5957.91[kN]) = 2225.94[kN]$$

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{b,Rd}} = \frac{|-111.36[kN]|}{2225.94[kN]} = \mathbf{0.05 \leq 1.00}$$ (EC3-1-1: 6.46)

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: The cross-section concerns a RHS section which is not susceptible to Torsional(-Flexural) Buckling.

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1
Note: The cross-section concerns an RHS section with 'h / b < 10 / $\lambda_{rel,z}$'.
This section is thus not susceptible to Lateral Torsional Buckling.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Student version

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section area	A	1.8900e-02	m^2
Plastic section modulus	$W_{pl,y}$	2.3640e-03	m^3
Plastic section modulus	$W_{pl,z}$	2.3640e-03	m^3
Design compression force	N_{Ed}	111.36	kN
Design bending moment (maximum)	$M_{y,Ed}$	306.60	kNm
Design bending moment (maximum)	$M_{z,Ed}$	-5.14	kNm
Characteristic compression resistance	N_{Rk}	6709.50	kN
Characteristic moment resistance	$M_{y,Rk}$	839.22	kNm
Characteristic moment resistance	$M_{z,Rk}$	839.22	kNm
Reduction factor	χ_y	0.33	
Reduction factor	χ_z	0.89	
Reduction factor	χ_{LT}	1.00	
Interaction factor	k_{yy}	0.94	
Interaction factor	k_{yz}	0.24	
Interaction factor	k_{zy}	0.56	
Interaction factor	k_{zz}	0.40	

Maximum moment $M_{y,Ed}$ is derived from beam B812 position 12.600 m.
Maximum moment $M_{z,Ed}$ is derived from beam B812 position 12.600 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.1	
Sway type y		sway	
Equivalent moment factor	C_{my}	0.90	
Resulting load type z		point load F	
End moment	$M_{h,z}$	-5.14	kNm
Field moment	$M_{s,z}$	1.21	kNm
Factor	$\alpha_{s,z}$	-0.24	
Ratio of end moments	ψ_z	-0.81	
Equivalent moment factor	C_{mz}	0.40	
Resulting load type LT		line load q	
End moment	$M_{h,LT}$	306.60	kNm
Field moment	$M_{s,LT}$	-115.94	kNm
Factor	$\alpha_{s,LT}$	-0.38	
Ratio of end moments	ψ_{LT}	0.11	
Equivalent moment factor	C_{mLT}	0.40	

Unity check (6.61) = 0.05 + 0.34 + 0.00 = 0.39 -
Unity check (6.62) = 0.02 + 0.21 + 0.00 = 0.23 -

student version

$$C_{my} = 0.90$$

$$\alpha_{s,z} = \frac{M_{s,z}}{M_{h,z}} = \frac{1.21[kNm]}{-5.14[kNm]} = -0.24$$

$$C_{mz} = \max(-0.2 \times \psi_z - 0.8 \times \alpha_{s,z}, 0.4) = \max(-0.2 \times -0.81 - 0.8 \times -0.24, 0.4) = \max(0.35, 0.4) = 0.40$$

$$\alpha_{s,LT} = \frac{M_{s,LT}}{M_{h,LT}} = \frac{-115.94[kNm]}{306.60[kNm]} = -0.38$$

$$C_{mLT} = \max(0.1 - 0.8 \times \alpha_{s,LT}, 0.4) = \max(0.1 - 0.8 \times -0.38, 0.4) = \max(0.40, 0.4) = 0.40$$

$$N_{Rk} = A \times f_y = 1.8900 \cdot 10^{-2}[m^2] \times 355.0[MPa] = 6709.50[kN]$$

$$M_{y,Rk} = W_{pl,y} \times f_y = 2.3640 \cdot 10^{-3}[m^3] \times 355.0[MPa] = 839.22[kNm]$$

$$M_{z,Rk} = W_{pl,z} \times f_y = 2.3640 \cdot 10^{-3}[m^3] \times 355.0[MPa] = 839.22[kNm]$$

$$k_{yy} = \min\left\{C_{my} \times \left[1 + (\lambda_{rel,y} - 0.2) \times \frac{N_{Ed}}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}}\right], C_{my} \times \left(1 + 0.8 \times \frac{N_{Ed}}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}}\right)\right\}$$

$$= \min\left\{0.90 \times \left[1 + (1.60 - 0.2) \times \frac{111.36[kN]}{0.33 \times \frac{6709.50[kN]}{1.00}}\right], 0.90 \times \left(1 + 0.8 \times \frac{111.36[kN]}{0.33 \times \frac{6709.50[kN]}{1.00}}\right)\right\} = \min\{0.96, 0.94\} = 0.94$$

$$k_{yz} = 0.6 \times k_{zz} = 0.6 \times 0.40 = 0.24$$

$$k_{zy} = 0.6 \times k_{yy} = 0.6 \times 0.94 = 0.56$$

$$k_{zz} = \min\left\{C_{mz} \times \left[1 + (\lambda_{rel,z} - 0.2) \times \frac{N_{Ed}}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}}\right], C_{mz} \times \left(1 + 0.8 \times \frac{N_{Ed}}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}}\right)\right\}$$

$$= \min\left\{0.40 \times \left[1 + (0.61 - 0.2) \times \frac{111.36[kN]}{0.89 \times \frac{6709.50[kN]}{1.00}}\right], 0.40 \times \left(1 + 0.8 \times \frac{111.36[kN]}{0.89 \times \frac{6709.50[kN]}{1.00}}\right)\right\} = \min\{0.40, 0.41\} = 0.40$$

$$\text{Unity check (6.61)} = \frac{|N_{Ed}|}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \times \frac{|M_{y,Ed}| + |\Delta M_{y,Ed}|}{\chi_{LT} \times \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \times \frac{|M_{z,Ed}| + |\Delta M_{z,Ed}|}{\frac{M_{z,Rk}}{\gamma_{M1}}}$$

$$= \frac{|111.36[\mathrm{kN}]|}{0.33 \times \frac{6709.50[\mathrm{kN}]}{1.00}} + 0.94 \times \frac{|306.60[\mathrm{kNm}]| + |0.00[\mathrm{kNm}]|}{1.00 \times \frac{839.22[\mathrm{kNm}]}{1.00}} + 0.24 \times \frac{|-5.14[\mathrm{kNm}]| + |0.00[\mathrm{kNm}]|}{\frac{839.22[\mathrm{kNm}]}{1.00}} = \mathbf{0.39} \leq \mathbf{1.00} \qquad \text{(EC3-1-1: 6.61)}$$

Student version

$$\text{Unity check (6.62)} = \frac{|N_{Ed}|}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} \times \frac{|M_{y,Ed}| + |\Delta M_{y,Ed}|}{\chi_{LT} \times \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \times \frac{|M_{z,Ed}| + |\Delta M_{z,Ed}|}{\frac{M_{z,Rk}}{\gamma_{M1}}}$$

$$= \frac{|111.36[\mathrm{kN}]|}{0.89 \times \frac{6709.50[\mathrm{kN}]}{1.00}} + 0.56 \times \frac{|306.60[\mathrm{kNm}]| + |0.00[\mathrm{kNm}]|}{1.00 \times \frac{839.22[\mathrm{kNm}]}{1.00}} + 0.40 \times \frac{|-5.14[\mathrm{kNm}]| + |0.00[\mathrm{kNm}]|}{\frac{839.22[\mathrm{kNm}]}{1.00}} = \mathbf{0.23} \leq \mathbf{1.00} \qquad \text{(EC3-1-1: 6.62)}$$

Unity check = max (Unity check (6.61), Unity check (6.62)) = max (0.39, 0.23) = $\mathbf{0.39} \leq \mathbf{1.00}$

The member satisfies the stability check.

EC-EN 1993 Steel check ULS
Values: **$UC_{Overall}$**
Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Member
Selection: B761

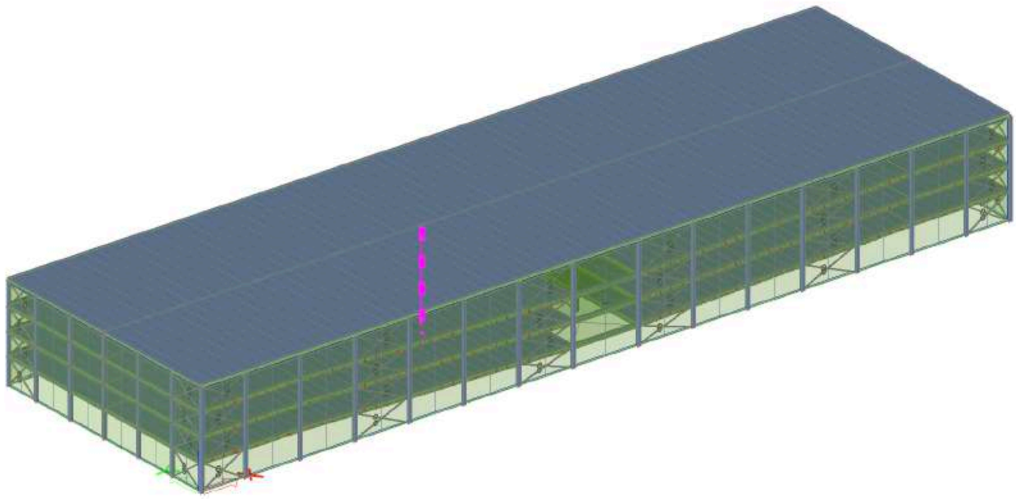


Figure 2.2.3.3. 9:Interior column

EN 1993-1-1 Code Check
National annex: Czech CSN-EN NA

Member B761	12.600 / 12.600 m	SHS350/350/16.0	S 355	ULS-Set B (auto)	0.74 -

Combination key
ULS-Set B (auto) / 1.15*LC1 + 1.15*LC2 + 1.50*LC3 + 1.50*LC4 + 0.90*3DWind8

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{M2} for resistance of net sections	1.25

Material			
Yield strength	f_y	355.0	MPa
Ultimate strength	f_u	490.0	MPa
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 12.600 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-2794.76	kN
Shear force	$V_{y,Ed}$	-4.12	kN
Shear force	$V_{z,Ed}$	12.98	kN
Torsion	T_{Ed}	13.56	kNm
Bending moment	$M_{y,Ed}$	10.69	kNm
Bending moment	$M_{z,Ed}$	112.82	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Student version

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	302	16	8.412e+04	1.716e+05	0.5		1.0	18.9	22.8	27.7	37.6	1
3	I	302	16	1.767e+05	1.850e+05	1.0		1.0	18.9	22.8	27.7	31.4	1
5	I	302	16	1.808e+05	9.329e+04	0.5		1.0	18.9	22.8	27.7	37.2	1
7	I	302	16	8.821e+04	7.992e+04	0.9		1.0	18.9	22.8	27.7	32.0	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	2.1100e-02	m²
Compression resistance	$N_{c,Rd}$	7490.50	kN
Unity check		0.37	-

$$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M0}} = \frac{2.1100 \cdot 10^{-2}[m^2] \times 355.0[MPa]}{1.00} = 7490.50[kN] \quad \text{(EC3-1-1: 6.10)}$$

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{c,Rd}} = \frac{|-2794.76[kN]|}{7490.50[kN]} = \mathbf{0.37 \leq 1.00} \quad \text{(EC3-1-1: 6.9)}$$

Bending moment check for M_y
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,y}$	2.6300e-03	m³
Plastic bending moment	$M_{pl,y,Rd}$	933.65	kNm
Unity check		0.01	-

$$M_{pl,y,Rd} = \frac{W_{pl,y} \times f_y}{\gamma_{M0}} = \frac{2.6300 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 933.65[kNm] \quad \text{(EC3-1-1: 6.13)}$$

$$\text{Unity check} = \frac{|M_{y,Ed}|}{M_{pl,y,Rd}} = \frac{|10.69[kNm]|}{933.65[kNm]} = \mathbf{0.01 \leq 1.00} \quad \text{(EC3-1-1: 6.12)}$$

Bending moment check for M_z
According to EN 1993-1-1 article 6.2.5 and formula (6.12),(6.13)

Plastic section modulus	$W_{pl,z}$	2.6300e-03	m³
Plastic bending moment	$M_{pl,z,Rd}$	933.65	kNm
Unity check		0.12	-

Student version

$$M_{pl,z,Rd} = \frac{W_{pl,z} \times f_y}{\gamma_{M0}} = \frac{2.6300 \cdot 10^{-3}[m^3] \times 355.0[MPa]}{1.00} = 933.65[kNm] \qquad \text{(EC3-1-1: 6.13)}$$

$$\text{Unity check} = \frac{|M_{z,Ed}|}{M_{pl,z,Rd}} = \frac{|112.82[kNm]|}{933.65[kNm]} = \mathbf{0.12 \leq 1.00} \qquad \text{(EC3-1-1: 6.12)}$$

Shear check for V_y

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1.20	
Shear area	A_v	1.0550e-02	m²
Plastic shear resistance for V_y	$V_{pl,y,Rd}$	2162.32	kN
Unity check		0.00	-

$$V_{pl,y,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{1.0550 \cdot 10^{-2}[m^2] \times \frac{355.0[MPa]}{\sqrt{3}}}{1.00} = 2162.32[kN] \qquad \text{(EC3-1-1: 6.18)}$$

$$\text{Unity check} = \frac{|V_{y,Ed}|}{V_{c,y,Rd}} = \frac{|-4.12[kN]|}{2162.32[kN]} = \mathbf{0.00 \leq 1.00} \qquad \text{(EC3-1-1: 6.17)}$$

Shear check for V_z

According to EN 1993-1-1 article 6.2.6 and formula (6.17)

Shear correction factor	η	1.20	
Shear area	A_v	1.0550e-02	m²
Plastic shear resistance for V_z	$V_{pl,z,Rd}$	2162.32	kN
Unity check		0.01	-

$$V_{pl,z,Rd} = \frac{A_v \times \frac{f_y}{\sqrt{3}}}{\gamma_{M0}} = \frac{1.0550 \cdot 10^{-2}[m^2] \times \frac{355.0[MPa]}{\sqrt{3}}}{1.00} = 2162.32[kN] \qquad \text{(EC3-1-1: 6.18)}$$

$$\text{Unity check} = \frac{|V_{z,Ed}|}{V_{c,z,Rd}} = \frac{|12.98[kN]|}{2162.32[kN]} = \mathbf{0.01 \leq 1.00} \qquad \text{(EC3-1-1: 6.17)}$$

Torsion check

According to EN 1993-1-1 article 6.2.7 and formula (6.23)

Index of fibre	Fibre	1	
Total torsional moment	τ_{Ed}	3.8	MPa
Elastic shear resistance	τ_{Rd}	205.0	MPa
Unity check		0.02	-

$$\tau_{Ed} = |T_{Ed}| \times \tau_{Ed,unit} = |13557.47| \times 2.801 \cdot 10^{-1}[kN/m^2] = 3.8[MPa]$$

$$\tau_{Rd} = \frac{f_y}{\sqrt{3} \times \gamma_{M0}} = \frac{355.0[MPa]}{\sqrt{3} \times 1.00} = 205.0[MPa]$$

$$\text{Unity check} = \frac{\tau_{Ed}}{\tau_{Rd}} = \frac{3.8[MPa]}{205.0[MPa]} = \mathbf{0.02 \leq 1.00} \qquad \text{(EC3-1-1: 6.23)}$$

Note: The unity check for torsion is lower than the limit value of 0.05. Therefore torsion is considered as insignificant and is ignored in the combined checks.

Combined bending, axial force and shear force check

According to EN 1993-1-1 article 6.2.9.1 and formula (6.41)

Design plastic moment resistance reduced due to N_{Ed}	$M_{N,y,Rd}$	764.69	kNm
Exponent of bending ratio y	α	1.97	
Design plastic moment resistance reduced due to N_{Ed}	$M_{N,z,Rd}$	764.69	kNm
Exponent of bending ratio z	β	1.97	

Unity check (6.41) = 0.00 + 0.02 = 0.02 -

$$M_{N,y,Rd} = \min\left[\frac{M_{pl,y,Rd} \times (1-n)}{1 - 0.5 \times \text{ratio}_{A,w}}, M_{pl,y,Rd}\right] = \min\left[\frac{933.65[kNm] \times (1-0.37)}{1 - 0.5 \times 0.47}, 933.65[kNm]\right] = \min[764.69[kNm], 933.65[kNm]] = 764.69[kNm] \qquad \text{(EC3-1-1: 6.39)}$$

$$\alpha = \max\left[\min\left(\frac{1.66}{1 - 1.13 \times n^2}, 6\right), 1\right] = \max\left[\min\left(\frac{1.66}{1 - 1.13 \times 0.37^2}, 6\right), 1\right] = \max[\min(1.97, 6), 1] = 1.97$$

$$M_{N,z,Rd} = \min\left[\frac{M_{pl,z,Rd} \times (1-n)}{1 - 0.5 \times \text{ratio}_{A,f}}, M_{pl,z,Rd}\right] = \min\left[\frac{933.65[kNm] \times (1-0.37)}{1 - 0.5 \times 0.47}, 933.65[kNm]\right] = \min[764.69[kNm], 933.65[kNm]] = 764.69[kNm] \qquad \text{(EC3-1-1: 6.40)}$$

$$\beta = \max\left[\min\left(\frac{1.66}{1 - 1.13 \times n^2}, 6\right), 1\right] = \max\left[\min\left(\frac{1.66}{1 - 1.13 \times 0.37^2}, 6\right), 1\right] = \max[\min(1.97, 6), 1] = 1.97$$

$$\text{Unity check} = \left(\frac{|M_{y,Ed}|}{M_{N,y,Rd}}\right)^{\alpha} + \left(\frac{|M_{z,Ed}|}{M_{N,z,Rd}}\right)^{\beta} = \left(\frac{|10.69[kNm]|}{764.69[kNm]}\right)^{1.97} + \left(\frac{|112.82[kNm]|}{764.69[kNm]}\right)^{1.97} = \mathbf{0.02 \leq 1.00} \qquad \text{(EC3-1-1: 6.41)}$$

Note: Since the shear forces are less than half the plastic shear resistances their effect on the moment resistances is neglected.

The member satisfies the section check.

Student version

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 8.220 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	302	16	4.134e+04	1.915e+05	0.2		1.0	18.9	22.8	27.7	42.5	1
3	I	302	16	1.994e+05	1.979e+05	1.0		1.0	18.9	22.8	27.7	31.0	1
5	I	302	16	1.899e+05	3.976e+04	0.2		1.0	18.9	22.8	27.7	42.6	1
7	I	302	16	3.188e+04	3.331e+04	1.0		1.0	18.9	22.8	27.7	31.4	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	2.620	12.600	m
Buckling factor	k	1.39	0.70	
Buckling length	l_{cr}	3.651	8.820	m
Critical Euler load	N_{cr}	60559.46	10374.75	kN
Slenderness	λ	26.87	64.92	
Relative slenderness	λ_{rel}	0.35	0.85	
Limit slenderness	$\lambda_{rel,0}$	0.20	0.20	
Buckling curve		a	a	
Imperfection	α	0.21	0.21	
Reduction factor	χ	0.97	0.77	
Buckling resistance	$N_{b,Rd}$	7229.00	5738.48	kN

Flexural Buckling verification			
Cross-section area	A	2.1100e-02	m²
Buckling resistance	$N_{b,Rd}$	5738.48	kN
Unity check		0.49	-

$$N_{cr,y} = \frac{\pi^2 \times E \times I_y}{l_{cr,y}^2} = \frac{\pi^2 \times 210000.0[MPa] \times 3.8940 \cdot 10^{-4}[m^4]}{3.651[m]^2} = 60559.46[kN]$$

$$N_{cr,z} = \frac{\pi^2 \times E \times I_z}{l_{cr,z}^2} = \frac{\pi^2 \times 210000.0[MPa] \times 3.8940 \cdot 10^{-4}[m^4]}{8.820[m]^2} = 10374.75[kN]$$

$$\lambda_y = \frac{l_{cr,y}}{i_y} = \frac{3.651[m]}{136[mm]} = 26.87$$

$$\lambda_z = \frac{l_{cr,z}}{i_z} = \frac{8.820[m]}{136[mm]} = 64.92$$

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{26.87}{\pi \times \sqrt{\frac{210000.0[MPa]}{355.0[MPa]}}} = 0.35 \qquad \text{(EC3-1-1: 6.50)}$$

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{64.92}{\pi \times \sqrt{\frac{210000.0[MPa]}{355.0[MPa]}}} = 0.85 \qquad \text{(EC3-1-1: 6.50)}$$

$$\varphi_y = 0.5 \times \left[1 + \alpha_y \times (\lambda_{rel,y} - \lambda_{rel,y,0}) + \lambda_{rel,y}^2\right] = 0.5 \times \left[1 + 0.21 \times (0.35 - 0.20) + 0.35^2\right] = 0.58$$

$$\varphi_z = 0.5 \times \left[1 + \alpha_z \times (\lambda_{rel,z} - \lambda_{rel,z,0}) + \lambda_{rel,z}^2\right] = 0.5 \times \left[1 + 0.21 \times (0.85 - 0.20) + 0.85^2\right] = 0.93$$

$$\chi_y = \min\left(\frac{1}{\varphi_y + \sqrt{\varphi_y^2 - \lambda_{rel,y}^2}}, \frac{1}{\lambda_{rel,y}^2}, 1\right) = \min\left(\frac{1}{0.58 + \sqrt{0.58^2 - 0.35^2}}, \frac{1}{0.35^2}, 1\right) = \min(0.97, 8.08, 1) = 0.97 \qquad \text{(EC3-1-1: 6.49)}$$

$$\chi_z = \min\left(\frac{1}{\varphi_z + \sqrt{\varphi_z^2 - \lambda_{rel,z}^2}}, \frac{1}{\lambda_{rel,z}^2}, 1\right) = \min\left(\frac{1}{0.93 + \sqrt{0.93^2 - 0.85^2}}, \frac{1}{0.85^2}, 1\right) = \min(0.77, 1.39, 1) = 0.77 \qquad \text{(EC3-1-1: 6.49)}$$

$$N_{b,y,Rd} = \frac{\chi_y \times A \times f_y}{\gamma_{M1}} = \frac{0.97 \times 2.1100 \cdot 10^{-2}[m^2] \times 355.0[MPa]}{1.00} = 7229.00[kN] \qquad \text{(EC3-1-1: 6.47)}$$

$$N_{b,z,Rd} = \frac{\chi_z \times A \times f_y}{\gamma_{M1}} = \frac{0.77 \times 2.1100 \cdot 10^{-2}[m^2] \times 355.0[MPa]}{1.00} = 5738.48[kN] \qquad \text{(EC3-1-1: 6.47)}$$

$$N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}) = \min(7229.00[kN], 5738.48[kN]) = 5738.48[kN]$$

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{b,Rd}} = \frac{|-2794.76[kN]|}{5738.48[kN]} = \mathbf{0.49} \leq \mathbf{1.00} \qquad \text{(EC3-1-1: 6.46)}$$

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: The cross-section concerns a RHS section which is not susceptible to Torsional(-Flexural) Buckling.

Student version

Lateral Torsional Buckling check
According to EN 1993-1-1 article 6.3.2.1
Note: The cross-section concerns an RHS section with 'h / b < 10 / $\lambda_{rel,z}$'.
This section is thus not susceptible to Lateral Torsional Buckling.

Bending and axial compression check
According to EN 1993-1-1 article 6.3.3 and formula (6.61),(6.62)

Bending and axial compression check parameters			
Interaction method		alternative method 2	
Cross-section area	A	2.1100e-02	m^2
Plastic section modulus	$W_{pl,y}$	2.6300e-03	m^3
Plastic section modulus	$W_{pl,z}$	2.6300e-03	m^3
Design compression force	N_{Ed}	2794.76	kN
Design bending moment (maximum)	$M_{y,Ed}$	-23.32	kNm
Design bending moment (maximum)	$M_{z,Ed}$	178.53	kNm
Characteristic compression resistance	N_{Rk}	7490.50	kN
Characteristic moment resistance	$M_{y,Rk}$	933.65	kNm
Characteristic moment resistance	$M_{z,Rk}$	933.65	kNm
Reduction factor	χ_y	0.97	
Reduction factor	χ_z	0.77	
Reduction factor	χ_{LT}	1.00	
Interaction factor	k_{yy}	0.95	
Interaction factor	k_{yz}	0.76	
Interaction factor	k_{zy}	0.57	
Interaction factor	k_{zz}	1.27	

Maximum moment $M_{y,Ed}$ is derived from beam B761 position 9.980 m.
Maximum moment $M_{z,Ed}$ is derived from beam B761 position 8.660 m.

Interaction method 2 parameters			
Method for interaction factors		Table B.1	
Sway type y		sway	
Equivalent moment factor	C_{my}	0.90	
Resulting load type z		point load F	
End moment	$M_{h,z}$	112.82	kNm
Field moment	$M_{s,z}$	178.53	kNm
Factor	$\alpha_{h,z}$	0.63	
Ratio of end moments	ψ_z	0.49	
Equivalent moment factor	C_{mz}	0.96	
Resulting load type LT		point load F	
End moment	$M_{h,LT}$	-39.67	kNm
Field moment	$M_{s,LT}$	50.87	kNm
Factor	$\alpha_{h,LT}$	-0.78	
Ratio of end moments	ψ_{LT}	-0.27	
Equivalent moment factor	C_{mLT}	0.86	

Student version

Unity check (6.61) = 0.39 + 0.02 + 0.15 = 0.56 -
Unity check (6.62) = 0.49 + 0.01 + 0.24 = 0.74 -

$$C_{my} = 0.90$$

$$\alpha_{h,z} = \frac{M_{h,z}}{M_{s,z}} = \frac{112.82[\mathrm{kNm}]}{178.53[\mathrm{kNm}]} = 0.63$$

$$C_{mz} = 0.9 + 0.1 \times \alpha_{h,z} = 0.9 + 0.1 \times 0.63 = 0.96$$

$$\alpha_{h,LT} = \frac{M_{h,LT}}{M_{s,LT}} = \frac{-39.67[\mathrm{kNm}]}{50.87[\mathrm{kNm}]} = -0.78$$

$$C_{mLT} = 0.9 + 0.1 \times \alpha_{h,LT} \times (1 + 2 \times \psi_{LT}) = 0.9 + 0.1 \times -0.78 \times (1 + 2 \times -0.27) = 0.86$$

$$N_{Rk} = A \times f_y = 2.1100 \cdot 10^{-2}[\mathrm{m^2}] \times 355.0[\mathrm{MPa}] = 7490.50[\mathrm{kN}]$$

$$M_{y,Rk} = W_{pl,y} \times f_y = 2.6300 \cdot 10^{-3}[\mathrm{m^3}] \times 355.0[\mathrm{MPa}] = 933.65[\mathrm{kNm}]$$

$$M_{z,Rk} = W_{pl,z} \times f_y = 2.6300 \cdot 10^{-3}[\mathrm{m^3}] \times 355.0[\mathrm{MPa}] = 933.65[\mathrm{kNm}]$$

$$k_{yy} = \min\left\{ C_{my} \times \left[1 + (\lambda_{rel,y} - 0.2) \times \frac{N_{Ed}}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}}\right], C_{my} \times \left(1 + 0.8 \times \frac{N_{Ed}}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}}\right)\right\}$$

$$= \min\left\{ 0.90 \times \left[1 + (0.35 - 0.2) \times \frac{2794.76[\mathrm{kN}]}{0.97 \times \frac{7490.50[\mathrm{kN}]}{1.00}}\right], 0.90 \times \left(1 + 0.8 \times \frac{2794.76[\mathrm{kN}]}{0.97 \times \frac{7490.50[\mathrm{kN}]}{1.00}}\right)\right\} = \min\{0.95, 1.18\} = 0.95$$

Student version

$$k_{yz} = 0.6 \times k_{zz} = 0.6 \times 1.27 = 0.76$$

$$k_{zy} = 0.6 \times k_{yy} = 0.6 \times 0.95 = 0.57$$

Student version

$$k_{zz} = \min\left\{ C_{mz} \times \left[1 + (\lambda_{rel,z} - 0.2) \times \frac{N_{Ed}}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}}\right], C_{mz} \times \left(1 + 0.8 \times \frac{N_{Ed}}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}}\right)\right\}$$

$$= \min\left\{ 0.96 \times \left[1 + (0.85 - 0.2) \times \frac{2794.76[kN]}{0.77 \times \frac{7490.50[kN]}{1.00}}\right], 0.96 \times \left(1 + 0.8 \times \frac{2794.76[kN]}{0.77 \times \frac{7490.50[kN]}{1.00}}\right)\right\} = \min\{1.27, 1.34\} = 1.27$$

$$\text{Unity check (6.61)} = \frac{|N_{Ed}|}{\chi_y \times \frac{N_{Rk}}{\gamma_{M1}}} + k_{yy} \times \frac{|M_{y,Ed}| + |\Delta M_{y,Ed}|}{\chi_{LT} \times \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz} \times \frac{|M_{z,Ed}| + |\Delta M_{z,Ed}|}{\frac{M_{z,Rk}}{\gamma_{M1}}}$$

$$= \frac{|2794.76[kN]|}{0.97 \times \frac{7490.50[kN]}{1.00}} + 0.95 \times \frac{|-23.32[kNm]| + |0.00[kNm]|}{1.00 \times \frac{933.65[kNm]}{1.00}} + 0.76 \times \frac{|178.53[kNm]| + |0.00[kNm]|}{\frac{933.65[kNm]}{1.00}} = \mathbf{0.56 \leq 1.00} \qquad \text{(EC3-1-1: 6.61)}$$

$$\text{Unity check (6.62)} = \frac{|N_{Ed}|}{\chi_z \times \frac{N_{Rk}}{\gamma_{M1}}} + k_{zy} \times \frac{|M_{y,Ed}| + |\Delta M_{y,Ed}|}{\chi_{LT} \times \frac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz} \times \frac{|M_{z,Ed}| + |\Delta M_{z,Ed}|}{\frac{M_{z,Rk}}{\gamma_{M1}}}$$

$$= \frac{|2794.76[kN]|}{0.77 \times \frac{7490.50[kN]}{1.00}} + 0.57 \times \frac{|-23.32[kNm]| + |0.00[kNm]|}{1.00 \times \frac{933.65[kNm]}{1.00}} + 1.27 \times \frac{|178.53[kNm]| + |0.00[kNm]|}{\frac{933.65[kNm]}{1.00}} = \mathbf{0.74 \leq 1.00} \qquad \text{(EC3-1-1: 6.62)}$$

$$\text{Unity check} = \max(\text{Unity check (6.61)}, \text{Unity check (6.62)}) = \max(0.56, 0.74) = \mathbf{0.74 \leq 1.00}$$

The member satisfies the stability check.

2.2.3.3.5. Bracing Check

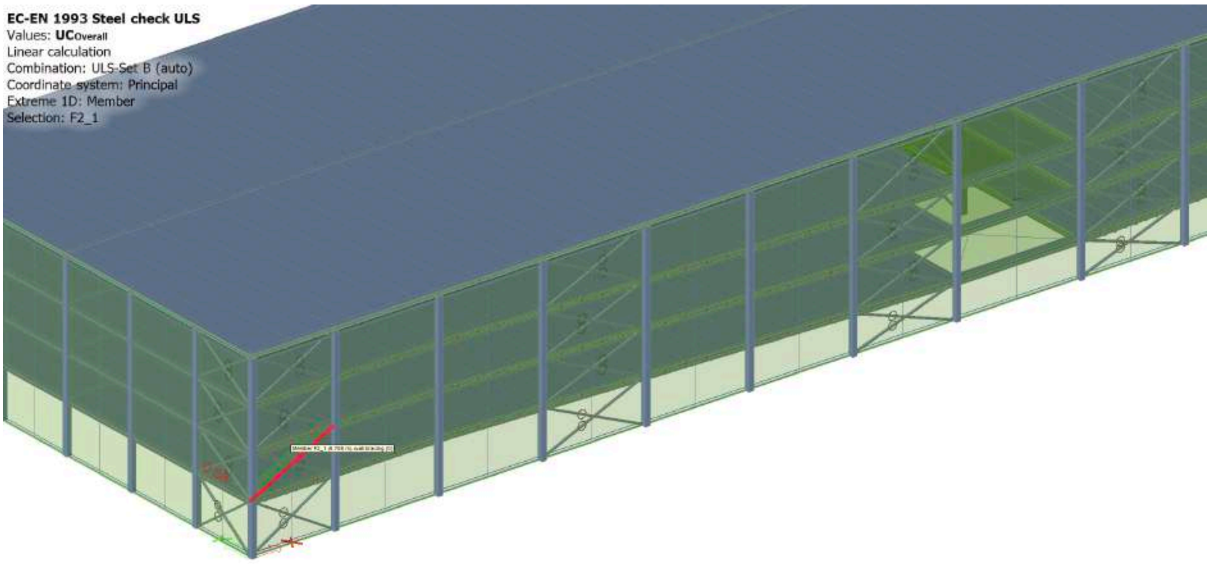


Figure 2.2.3.3. 10:Corner brace

EC-EN 1993 Steel check ULS

Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Member
Selection: F2_1

Student version

EN 1993-1-1 Code Check
National annex: Czech CSN-EN NA

Member F2_1	**6.708 / 6.708 m**	**SHS110/110/14.2**	**S 275 JR (EN 10025-2)**	**ULS-Set B (auto)**	**0.58 -**

Combination key
ULS-Set B (auto) / 1.15*LC1 + 1.15*LC2 + 1.05*LC3 + 1.05*LC4 + 1.50*3DWind4

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{M2} for resistance of net sections	1.25

Material			
Yield strength	f_y	275.0	MPa
Ultimate strength	f_u	410.0	MPa
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 6.708 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-184.29	kN
Shear force	$V_{y,Ed}$	0.00	kN
Shear force	$V_{z,Ed}$	0.00	kN
Torsion	T_{Ed}	0.00	kNm
Bending moment	$M_{y,Ed}$	0.00	kNm
Bending moment	$M_{z,Ed}$	0.00	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Student version

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	67	14	3.528e+04	3.528e+04	1.0		1.0	4.7	25.9	31.4	35.1	1
3	I	67	14	3.528e+04	3.528e+04	1.0		1.0	4.7	25.9	31.4	35.1	1
5	I	67	14	3.528e+04	3.528e+04	1.0		1.0	4.7	25.9	31.4	35.1	1
7	I	67	14	3.528e+04	3.528e+04	1.0		1.0	4.7	25.9	31.4	35.1	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	5.2300e-03	m²
Compression resistance	$N_{c,Rd}$	1438.25	kN
Unity check		0.13	-

$$N_{c,Rd} = \frac{A \times f_y}{\gamma_{M0}} = \frac{5.2300 \cdot 10^{-3}[\mathrm{m}^2] \times 275.0[\mathrm{MPa}]}{1.00} = 1438.25[\mathrm{kN}] \qquad \text{(EC3-1-1: 6.10)}$$

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{c,Rd}} = \frac{|-184.29[\mathrm{kN}]|}{1438.25[\mathrm{kN}]} = \mathbf{0.13 \leq 1.00} \qquad \text{(EC3-1-1: 6.9)}$$

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 6.708 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	67	14	3.528e+04	3.528e+04	1.0		1.0	4.7	25.9	31.4	35.1	1
3	I	67	14	3.528e+04	3.528e+04	1.0		1.0	4.7	25.9	31.4	35.1	1
5	I	67	14	3.528e+04	3.528e+04	1.0		1.0	4.7	25.9	31.4	35.1	1
7	I	67	14	3.528e+04	3.528e+04	1.0		1.0	4.7	25.9	31.4	35.1	1

Student version

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	6.708	6.708	m
Buckling factor	k	1.00	1.00	
Buckling length	l_{cr}	6.708	6.708	m
Critical Euler load	N_{cr}	357.41	357.43	kN
Slenderness	λ	174.15	174.15	
Relative slenderness	λ_{rel}	2.01	2.01	
Limit slenderness	$\lambda_{rel,0}$	0.20	0.20	
Buckling curve		a	a	
Imperfection	α	0.21	0.21	
Reduction factor	χ	0.22	0.22	
Buckling resistance	$N_{b,Rd}$	318.81	318.82	kN

Flexural Buckling verification			
Cross-section area	A	5.2300e-03	m^2
Buckling resistance	$N_{b,Rd}$	318.81	kN
Unity check		0.58	-

$$N_{cr,y} = \frac{\pi^2 \times E \times I_y}{l_{cr,y}^2} = \frac{\pi^2 \times 210000.0[MPa] \times 7.7600 \cdot 10^{-6}[m^4]}{6.708[m]^2} = 357.41[kN]$$

$$N_{cr,z} = \frac{\pi^2 \times E \times I_z}{l_{cr,z}^2} = \frac{\pi^2 \times 210000.0[MPa] \times 7.7600 \cdot 10^{-6}[m^4]}{6.708[m]^2} = 357.43[kN]$$

$$\lambda_y = \frac{l_{cr,y}}{i_y} = \frac{6.708[m]}{39[mm]} = 174.15$$

$$\lambda_z = \frac{l_{cr,z}}{i_z} = \frac{6.708[m]}{39[mm]} = 174.15$$

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{174.15}{\pi \times \sqrt{\frac{210000.0[MPa]}{275.0[MPa]}}} = 2.01$$ (EC3-1-1: 6.50)

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{174.15}{\pi \times \sqrt{\frac{210000.0[MPa]}{275.0[MPa]}}} = 2.01$$ (EC3-1-1: 6.50)

$$\varphi_y = 0.5 \times \left[1 + \alpha_y \times (\lambda_{rel,y} - \lambda_{rel,y,0}) + \lambda_{rel,y}^2\right] = 0.5 \times \left[1 + 0.21 \times (2.01 - 0.20) + 2.01^2\right] = 2.70$$

$$\varphi_z = 0.5 \times \left[1 + \alpha_z \times (\lambda_{rel,z} - \lambda_{rel,z,0}) + \lambda_{rel,z}^2\right] = 0.5 \times \left[1 + 0.21 \times (2.01 - 0.20) + 2.01^2\right] = 2.70$$

$$\chi_y = \min\left(\frac{1}{\varphi_y + \sqrt{\varphi_y^2 - \lambda_{rel,y}^2}}, \frac{1}{\lambda_{rel,y}^2}, 1\right) = \min\left(\frac{1}{2.70 + \sqrt{2.70^2 - 2.01^2}}, \frac{1}{2.01^2}, 1\right) = \min(0.22, 0.25, 1) = 0.22$$ (EC3-1-1: 6.49)

$$\chi_z = \min\left(\frac{1}{\varphi_z + \sqrt{\varphi_z^2 - \lambda_{rel,z}^2}}, \frac{1}{\lambda_{rel,z}^2}, 1\right) = \min\left(\frac{1}{2.70 + \sqrt{2.70^2 - 2.01^2}}, \frac{1}{2.01^2}, 1\right) = \min(0.22, 0.25, 1) = 0.22$$ (EC3-1-1: 6.49)

$$N_{b,y,Rd} = \frac{\chi_y \times A \times f_y}{\gamma_{M1}} = \frac{0.22 \times 5.2300 \cdot 10^{-3}[m^2] \times 275.0[MPa]}{1.00} = 318.81[kN]$$ (EC3-1-1: 6.47)

$$N_{b,z,Rd} = \frac{\chi_z \times A \times f_y}{\gamma_{M1}} = \frac{0.22 \times 5.2300 \cdot 10^{-3}[m^2] \times 275.0[MPa]}{1.00} = 318.82[kN]$$ (EC3-1-1: 6.47)

$$N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}) = \min(318.81[kN], 318.82[kN]) = 318.81[kN]$$

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{b,Rd}} = \frac{|-184.29[kN]|}{318.81[kN]} = \mathbf{0.58 \leq 1.00}$$ (EC3-1-1: 6.46)

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: The cross-section concerns a RHS section which is not susceptible to Torsional(-Flexural) Buckling.

The member satisfies the stability check.

Student version
Student version

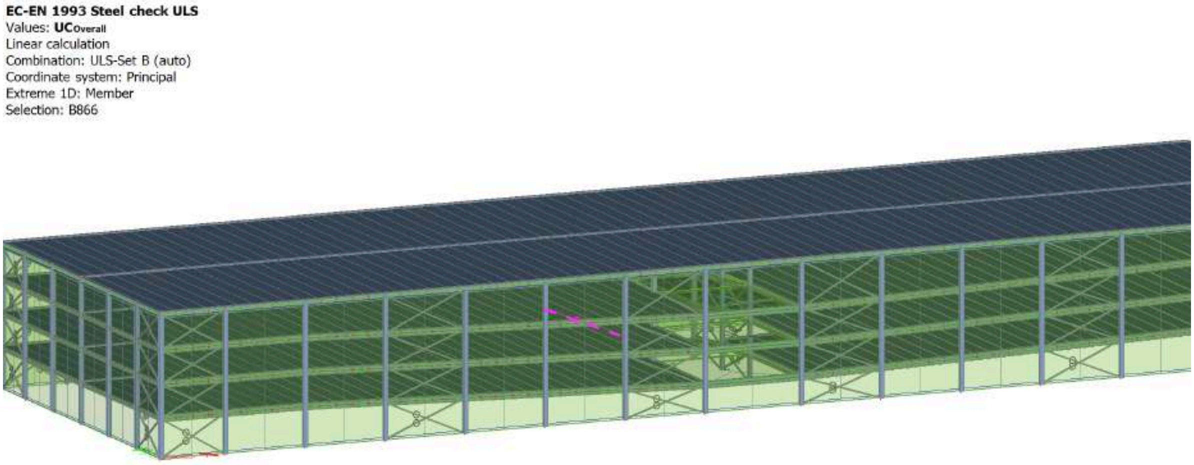


Figure 2.2.3.3. 11:Internal brace

EC-EN 1993 Steel check ULS

Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Member
Selection: B866

Student version

EN 1993-1-1 Code Check
National annex: Czech CSN-EN NA

Member B866	**8.078 / 8.078 m**	**SHS110/110/14.2**	**S 275 JR (EN 10025-2)**	**ULS-Set B (auto)**	**0.81 -**

Combination key
ULS-Set B (auto) / 1.35*LC1 + 1.35*LC2 + 1.05*LC3 + 1.05*LC4 + 0.90*3DWind16

Partial safety factors	
γ_{M0} for resistance of cross-sections	1.00
γ_{M1} for resistance to instability	1.00
γ_{M2} for resistance of net sections	1.25

Material			
Yield strength	f_y	275.0	MPa
Ultimate strength	f_u	410.0	MPa
Fabrication		Rolled	

...::SECTION CHECK::...

The critical check is on position 8.078 m

Internal forces		Calculated	Unit
Normal force	N_{Ed}	-183.41	kN
Shear force	$V_{y,Ed}$	0.00	kN
Shear force	$V_{z,Ed}$	0.00	kN
Torsion	T_{Ed}	0.00	kNm
Bending moment	$M_{y,Ed}$	0.00	kNm
Bending moment	$M_{z,Ed}$	0.00	kNm

Classification for cross-section design
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	67	14	3.511e+04	3.511e+04	1.0		1.0	4.7	25.9	31.4	35.1	1
3	I	67	14	3.511e+04	3.511e+04	1.0		1.0	4.7	25.9	31.4	35.1	1
5	I	67	14	3.511e+04	3.511e+04	1.0		1.0	4.7	25.9	31.4	35.1	1
7	I	67	14	3.511e+04	3.511e+04	1.0		1.0	4.7	25.9	31.4	35.1	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Compression check
According to EN 1993-1-1 article 6.2.4 and formula (6.9)

Cross-section area	A	5.2300e-03	m²
Compression resistance	$N_{c,Rd}$	1438.25	kN
Unity check		0.13	-

$$N_{c.Rd} = \frac{A \times f_y}{\gamma_{M0}} = \frac{5.2300 \cdot 10^{-3}[m^2] \times 275.0[MPa]}{1.00} = 1438.25[kN]$$ (EC3-1-1: 6.10)

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{c.Rd}} = \frac{|-183.41[kN]|}{1438.25[kN]} = \mathbf{0.13 \leq 1.00}$$ (EC3-1-1: 6.9)

The member satisfies the section check.

...::STABILITY CHECK::...

Classification for member buckling design
Decisive position for stability classification: 8.078 m
Classification according to EN 1993-1-1 article 5.5.2
Classification of Internal and Outstand parts according to EN 1993-1-1 Table 5.2 Sheet 1 & 2

Id	Type	c [mm]	t [mm]	σ_1 [kN/m²]	σ_2 [kN/m²]	Ψ [-]	k_σ [-]	α [-]	c/t [-]	Class 1 Limit [-]	Class 2 Limit [-]	Class 3 Limit [-]	Class
1	I	67	14	3.511e+04	3.511e+04	1.0		1.0	4.7	25.9	31.4	35.1	1
3	I	67	14	3.511e+04	3.511e+04	1.0		1.0	4.7	25.9	31.4	35.1	1
5	I	67	14	3.511e+04	3.511e+04	1.0		1.0	4.7	25.9	31.4	35.1	1
7	I	67	14	3.511e+04	3.511e+04	1.0		1.0	4.7	25.9	31.4	35.1	1

Note: The Classification limits have been set according to Semi-Comp+.
The cross-section is classified as Class 1

Flexural Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)

Buckling parameters		yy	zz	
Sway type		sway	non-sway	
System length	L	8.078	8.078	m
Buckling factor	k	1.00	1.00	
Buckling length	l_{cr}	8.078	8.078	m
Critical Euler load	N_{cr}	246.49	246.50	kN
Slenderness	λ	209.71	209.70	
Relative slenderness	λ_{rel}	2.42	2.42	
Limit slenderness	$\lambda_{rel,0}$	0.20	0.20	
Buckling curve		a	a	
Imperfection	α	0.21	0.21	
Reduction factor	χ	0.16	0.16	
Buckling resistance	$N_{b,Rd}$	225.20	225.21	kN

Flexural Buckling verification			
Cross-section area	A	5.2300e-03	m²
Buckling resistance	$N_{b,Rd}$	225.20	kN
Unity check		0.81	-

$$N_{cr,y} = \frac{\pi^2 \times E \times I_y}{l_{cr,y}^2} = \frac{\pi^2 \times 210000.0[MPa] \times 7.7600 \cdot 10^{-6}[m^4]}{8.078[m]^2} = 246.49[kN]$$

$$N_{cr,z} = \frac{\pi^2 \times E \times I_z}{l_{cr,z}^2} = \frac{\pi^2 \times 210000.0[MPa] \times 7.7600 \cdot 10^{-6}[m^4]}{8.078[m]^2} = 246.50[kN]$$

$$\lambda_y = \frac{l_{cr,y}}{i_y} = \frac{8.078[m]}{39[mm]} = 209.71$$

$$\lambda_z = \frac{l_{cr,z}}{i_z} = \frac{8.078[m]}{39[mm]} = 209.70$$

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{209.71}{\pi \times \sqrt{\frac{210000.0[MPa]}{275.0[MPa]}}} = 2.42$$ (EC3-1-1: 6.50)

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi \times \sqrt{\frac{E}{f_y}}} = \frac{209.70}{\pi \times \sqrt{\frac{210000.0[MPa]}{275.0[MPa]}}} = 2.42$$ (EC3-1-1: 6.50)

Student version

Student version

$$\varphi_y = 0.5 \times \left[1 + \alpha_y \times (\lambda_{rel,y} - \lambda_{rel,y,0}) + \lambda_{rel,y}^2\right] = 0.5 \times \left[1 + 0.21 \times (2.42 - 0.20) + 2.42^2\right] = 3.65$$

$$\varphi_z = 0.5 \times \left[1 + \alpha_z \times (\lambda_{rel,z} - \lambda_{rel,z,0}) + \lambda_{rel,z}^2\right] = 0.5 \times \left[1 + 0.21 \times (2.42 - 0.20) + 2.42^2\right] = 3.65$$

$$\chi_y = \min\left(\frac{1}{\varphi_y + \sqrt{\varphi_y^2 - \lambda_{rel,y}^2}}, \frac{1}{\lambda_{rel,y}^2}, 1\right) = \min\left(\frac{1}{3.65 + \sqrt{3.65^2 - 2.42^2}}, \frac{1}{2.42^2}, 1\right) = \min(0.16, 0.17, 1) = 0.16 \qquad \text{(EC3-1-1: 6.49)}$$

$$\chi_z = \min\left(\frac{1}{\varphi_z + \sqrt{\varphi_z^2 - \lambda_{rel,z}^2}}, \frac{1}{\lambda_{rel,z}^2}, 1\right) = \min\left(\frac{1}{3.65 + \sqrt{3.65^2 - 2.42^2}}, \frac{1}{2.42^2}, 1\right) = \min(0.16, 0.17, 1) = 0.16 \qquad \text{(EC3-1-1: 6.49)}$$

$$N_{b,y,Rd} = \frac{\chi_y \times A \times f_y}{\gamma_{M1}} = \frac{0.16 \times 5.2300 \cdot 10^{-3}[m^2] \times 275.0[MPa]}{1.00} = 225.20[kN] \qquad \text{(EC3-1-1: 6.47)}$$

$$N_{b,z,Rd} = \frac{\chi_z \times A \times f_y}{\gamma_{M1}} = \frac{0.16 \times 5.2300 \cdot 10^{-3}[m^2] \times 275.0[MPa]}{1.00} = 225.21[kN] \qquad \text{(EC3-1-1: 6.47)}$$

$$N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd}) = \min(225.20[kN], 225.21[kN]) = 225.20[kN]$$

$$\text{Unity check} = \frac{|N_{Ed}|}{N_{b,Rd}} = \frac{|-183.41[kN]|}{225.20[kN]} = \mathbf{0.81 \leq 1.00} \qquad \text{(EC3-1-1: 6.46)}$$

Torsional(-Flexural) Buckling check
According to EN 1993-1-1 article 6.3.1.1 and formula (6.46)
Note: The cross-section concerns a RHS section which is not susceptible to Torsional(-Flexural) Buckling.

The member satisfies the stability check.

2.2.3.4. Results

2.2.3.4.1. Stress Distribution

1D stresses
Values: σ_x
Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Global
Selection: All

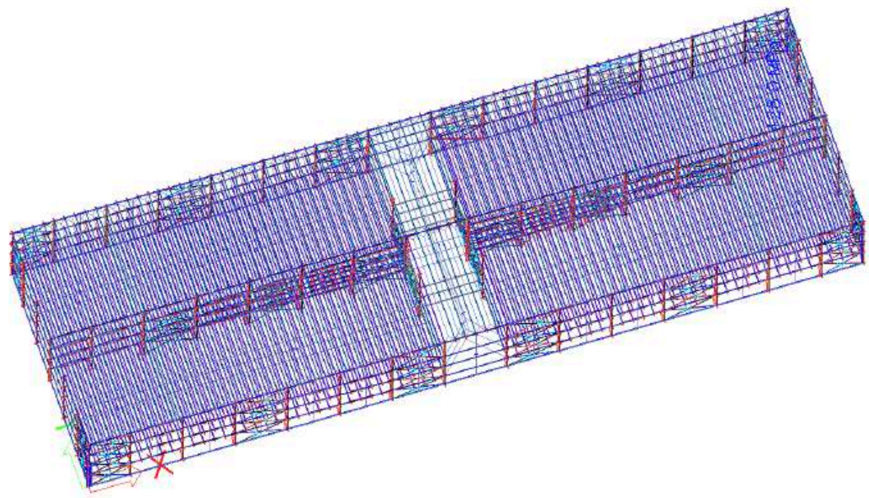


Figure 2.2.3.4. 1:Stress distribution

The maximum and minimum stress distribution on the structure is shown above. The red colour represents compression and blue represents tension. The stress values are in the acceptable range.

1D stresses

Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Global
Selection: All

Student version

Name	dx [m]	Fibre	Case	σ_x [MPa]	τ_{xy} [MPa]	τ_{xz} [MPa]	τ_{tor} [MPa]
B769	8.660+	8	ULS-Set B (auto)/1	**-510.5**	-1.4	-42.7	-15.3
Bx28_3	4.500+	3	ULS-Set B (auto)/2	**428.0**	0.0	0.0	0.0

Name	Combination key
ULS-Set B (auto)/1	1.35*LC1 + 1.35*LC2 + 1.05*LC3 + 1.05*LC4 + 0.90*3DWind16
ULS-Set B (auto)/2	1.35*LC1 + 1.35*LC2 + 1.05*LC3 + 1.05*LC4 + 0.90*3DWind12

2.2.3.4.2. Deformation

1D deformations
Values: u_y
Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Global
Selection: All

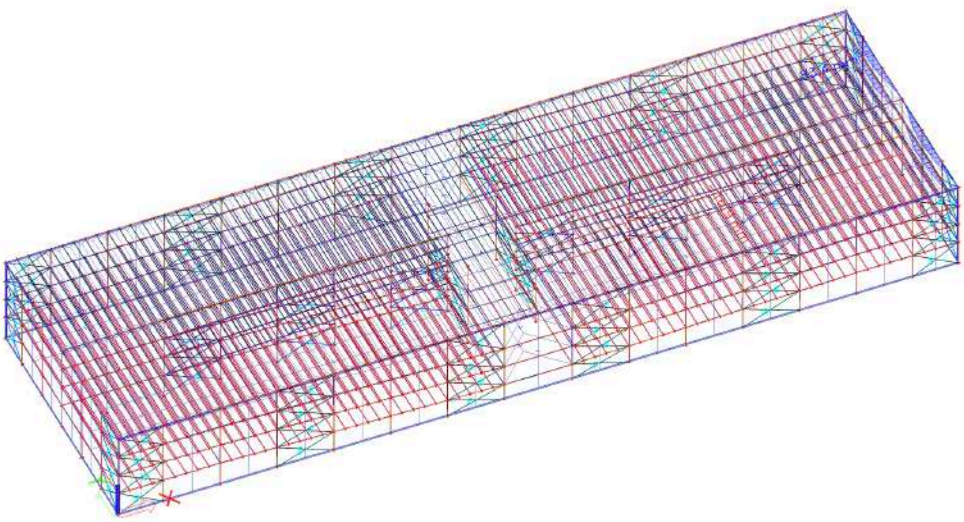


Figure 2.2.3.4. 2:Deformation u_y

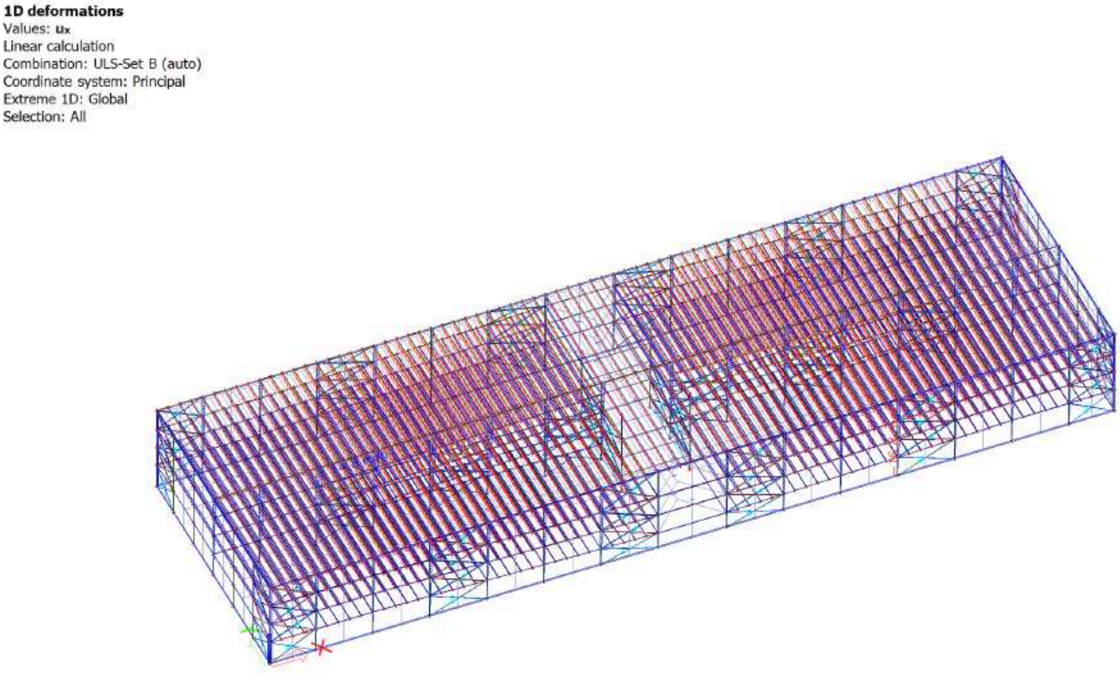


Figure 2.2.3.4. 3:Deformation u_x

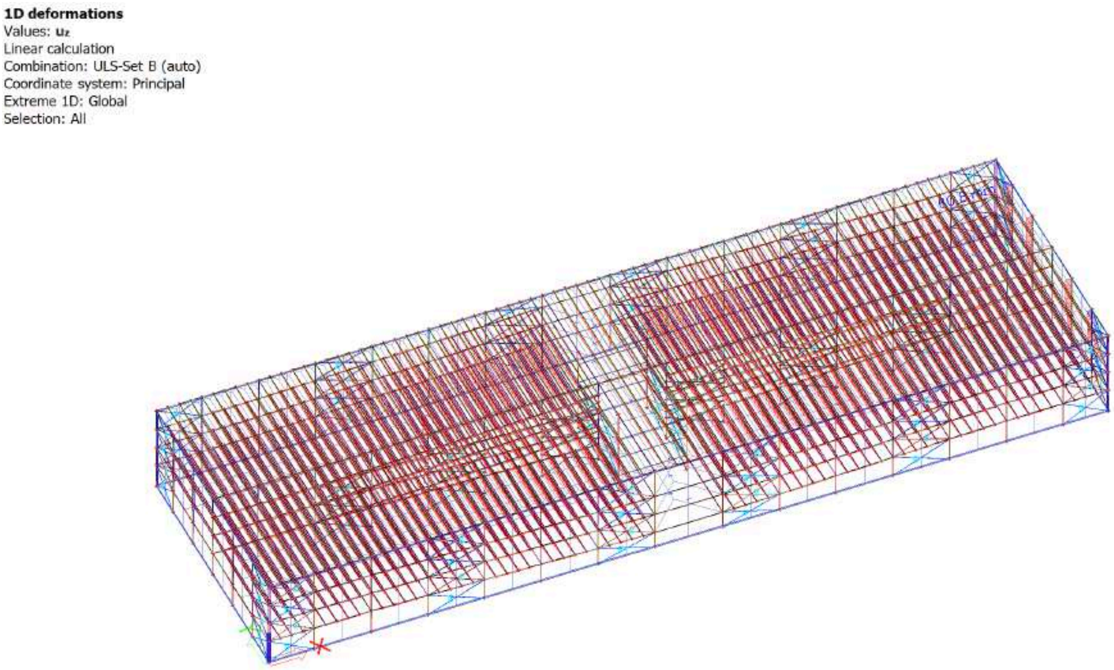


Figure 2.2.3.4. 4:Deformation u_z

1D deformations

Linear calculation
Combination: ULS-Set B (auto)
Coordinate system: Principal
Extreme 1D: Global
Selection: All
Deformations

Name	dx [m]	Case	u_x [mm]	u_y [mm]	u_z [mm]	φ_x [mrad]	φ_y [mrad]	φ_z [mrad]	U_{total} [mm]
B1056	17.000	ULS-Set B (auto)/1	**-16.8**	0.0	-6.6	-0.3	-3.6	-0.2	18.0
B763	0.000	ULS-Set B (auto)/2	**14.8**	9.0	2.4	0.0	0.3	-1.6	17.5
Bx12_4	7.500	ULS-Set B (auto)/1	0.0	**-16.9**	-6.4	3.6	-0.3	-0.2	18.1
By43_4	2.571	ULS-Set B (auto)/3	4.0	**33.4**	0.3	-1.1	-0.1	-1.3	33.5
B1413	8.947	ULS-Set B (auto)/4	-10.4	1.7	**-34.4**	-2.1	-0.3	0.0	35.1
F48_1	5.883	ULS-Set B (auto)/5	-3.7	2.3	**30.8**	0.0	0.0	0.0	31.0
Bx55_3	4.500-	ULS-Set B (auto)/2	-0.8	-7.8	-10.1	**-12.5**	-0.7	0.2	12.8
B1070	0.000	ULS-Set B (auto)/6	-2.9	1.9	-26.3	**12.1**	-0.7	0.0	26.5
Bx28_3	7.500	ULS-Set B (auto)/7	1.9	2.4	-10.7	-0.4	**-15.8**	-0.5	11.1
Bx28_2	0.000	ULS-Set B (auto)/8	0.8	2.2	-9.5	-1.1	**15.2**	-0.3	9.8
By41_4	2.571	ULS-Set B (auto)/9	3.9	39.3	-1.9	-1.2	0.7	**-16.4**	39.5
By42_4	3.429	ULS-Set B (auto)/10	4.3	35.7	-2.2	-0.9	-0.8	**16.7**	36.0
B1413	8.947	ULS-Set B (auto)/1	-11.3	1.5	-34.4	-2.1	-0.3	0.0	**35.2**

Name	Combination key
ULS-Set B (auto)/1	1.35*LC1 + 1.35*LC2 + 1.05*LC3 + 1.05*LC4 + 0.90*3DWind12
ULS-Set B (auto)/2	1.35*LC1 + 1.35*LC2 + 1.05*LC3 + 1.05*LC4 + 0.90*3DWind4
ULS-Set B (auto)/3	LC1 + LC2 + 1.05*LC3 + 1.50*3DWind12
ULS-Set B (auto)/4	1.35*LC1 + 1.35*LC2 + 1.05*LC3 + 1.05*LC4 + 0.90*3DWind8
ULS-Set B (auto)/5	1.15*LC1 + 1.15*LC2 + 1.05*LC3 + 1.05*LC4 + 1.50*3DWind12
ULS-Set B (auto)/6	1.35*LC1 + 1.35*LC2 + 1.05*LC3 + 1.05*LC4 + 0.90*3DWind16
ULS-Set B (auto)/7	1.35*LC1 + 1.35*LC2 + 1.05*LC3 + 0.90*3DWind16
ULS-Set B (auto)/8	1.35*LC1 + 1.35*LC2 + 1.05*LC3 + 0.90*3DWind4
ULS-Set B (auto)/9	1.15*LC1 + 1.15*LC2 + 1.05*LC4 + 1.50*3DWind12
ULS-Set B (auto)/10	1.15*LC1 + 1.15*LC2 + 1.05*LC3 + 1.50*3DWind12

Student version

Table 2.2.3.4. 1:Extreme values of deformations

2.2.3.4.3. Internal Forces and Reactions

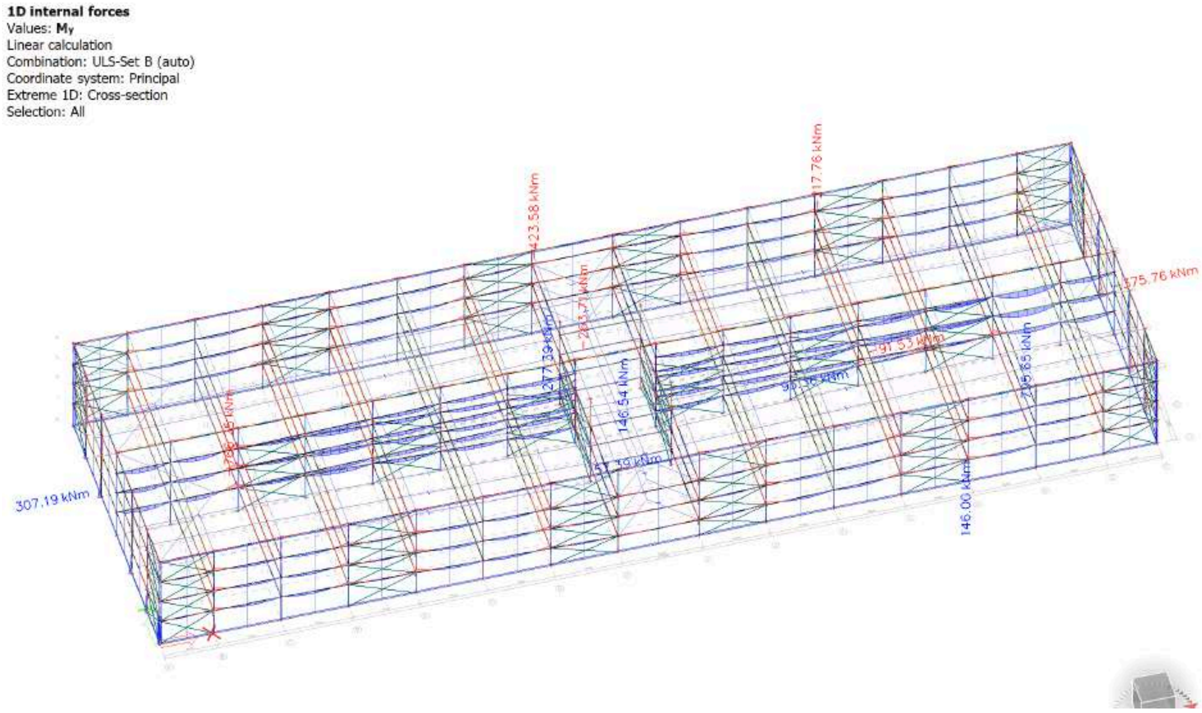


Figure 2.2.3.4. 5: My bending moments

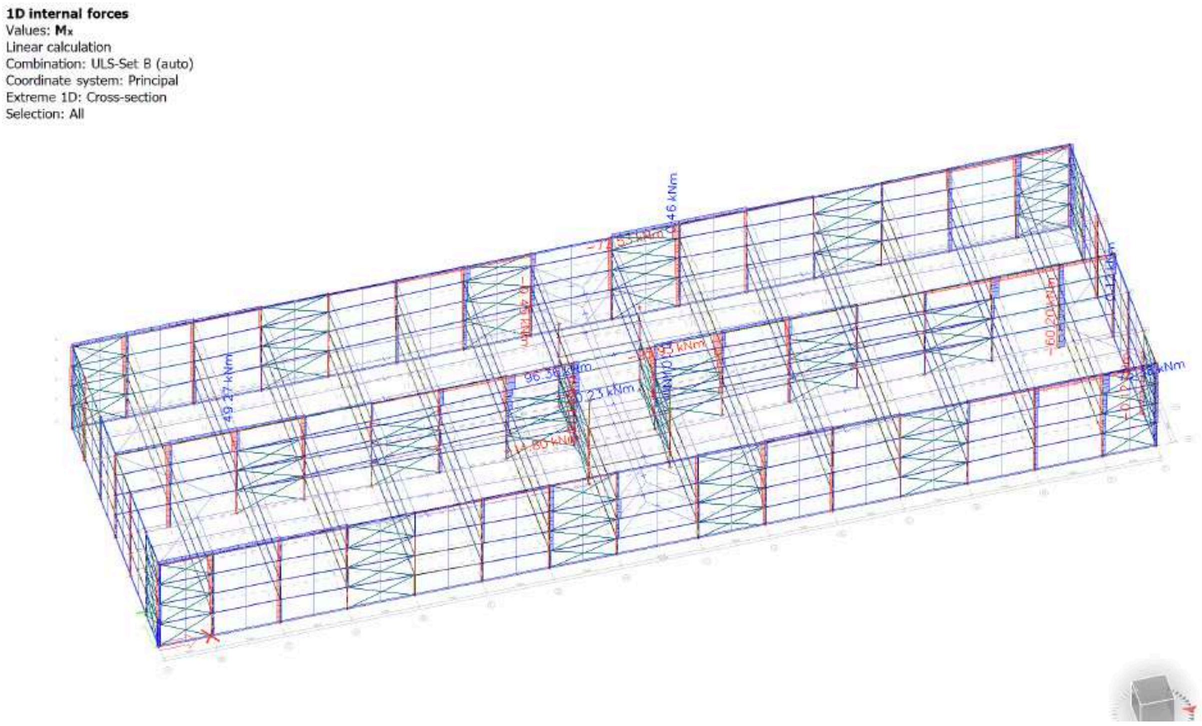


Figure 2.2.3.4. 6:Mx bending moments

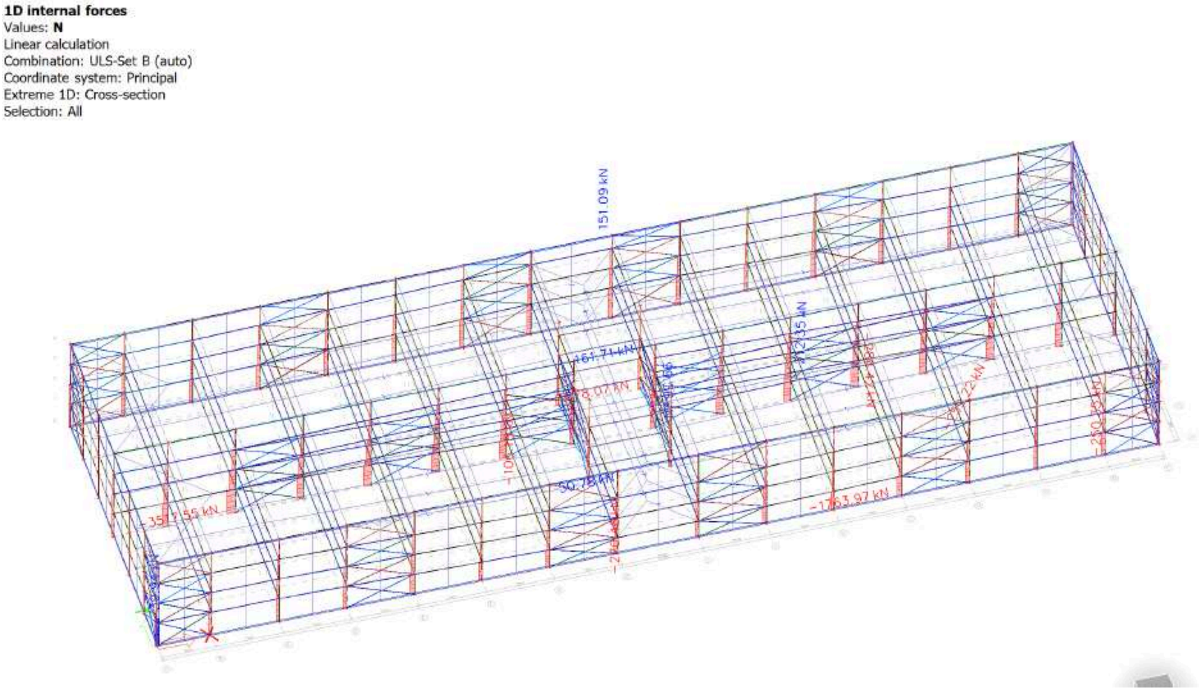


Figure 2.2.3.4. 7:N axial forces

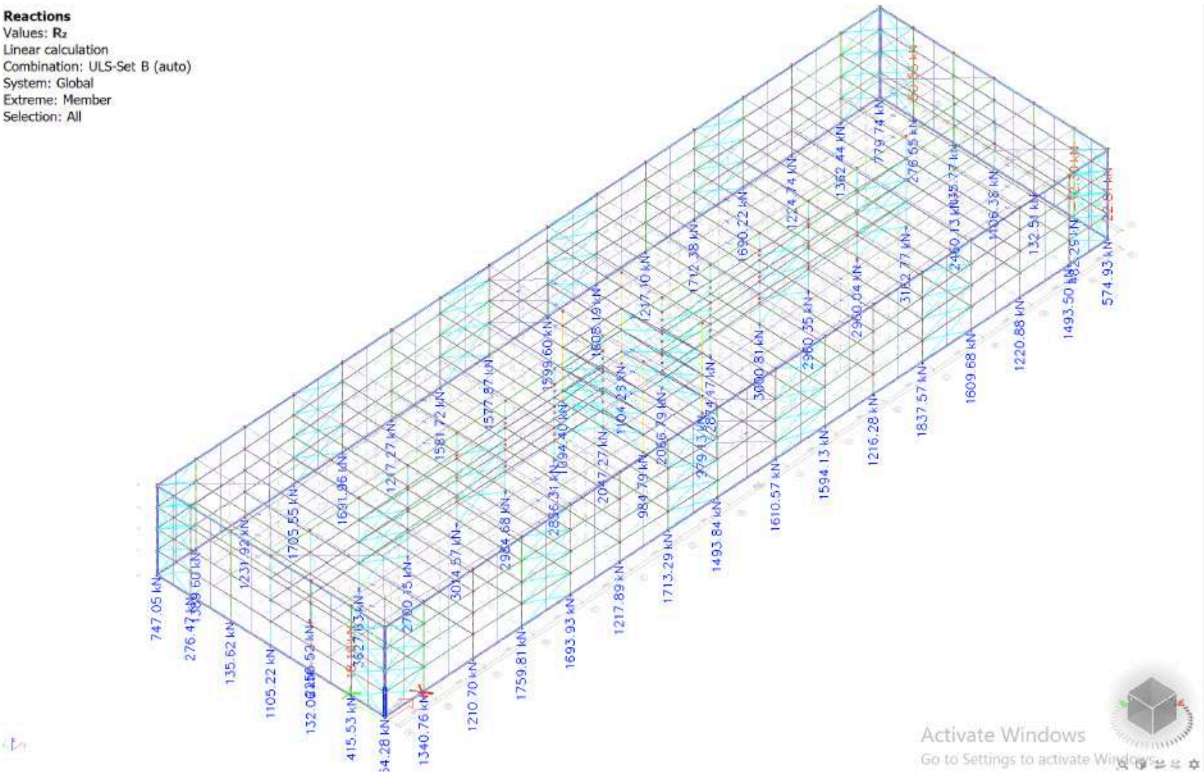


Figure 2.2.3.4. 8:Reactions

The cross-sectional extreme internal forces are shown below table.

Name	dx [m]	Case	Cross-section	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
Bx26_3	0	ULS-Set B (auto)/1	CS12-Beam2 - HEA600	-284.42	0	180.08	0.02	0	0
Bx39_1	7.503	ULS-Set B (auto)/2	CS12-Beam2 - HEA600	212.35	0	-215.94	0.03	0	0
Bx52_3	0	ULS-Set B (auto)/2	CS12-Beam2 - HEA600	-90.95	-11.18	219.49	0.46	-282.02	34.58
Bx15_3	0	ULS-Set B (auto)/3	CS12-Beam2 - HEA600	-130.5	16.41	164.7	-0.34	-207.39	-28.85
Bx28_1	0	ULS-Set B (auto)/4	CS12-Beam2 - HEA600	-31.34	0.5	520.76	-0.18	-723.11	-3.75
Bx17_1	7.5	ULS-Set B (auto)/5	CS12-Beam2 - HEA600	-25.27	0.28	-526.52	0.07	-766.25	2.09
Bx28_2	3.75	ULS-Set B (auto)/6	CS12-Beam2 - HEA600	1.59	0	0	-0.27	795.65	0
Bx50_3	0	ULS-Set B (auto)/2	CS12-Beam2 - HEA600	-91.1	11.19	213.55	-0.45	-262.41	-51.1
Bx15_3	6	ULS-Set B (auto)/1	CS12-Beam2 - HEA600	-129.57	16.39	-124.58	-0.36	-86.2	70.15

Name	dx [m]	Case	Cross-section	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
By21_3	0	ULS-Set B (auto)/13	CS13_Secondary beam2 - HEA320	-98.45	0	4.14	-0.11	0	0
By24_3	0	ULS-Set B (auto)/2	CS13_Secondary beam2 - HEA320	-119.28	-1.83	115.16	0.1	-198.95	7.04
By22_3	11	ULS-Set B (auto)/15	CS13_Secondary beam2 - HEA320	-71.75	-1.24	-150.74	0.06	-423.58	-13.59
By22_3	4.583	ULS-Set B (auto)/5	CS13_Secondary beam2 - HEA320	-107.82	-0.74	-10.07	0.06	277.39	-3.41
By20_3	6.013	ULS-Set B (auto)/2	CS13_Secondary beam2 - HEA320	-182.57	-5.61	-70.48	0.09	-66.84	-22.45
By23_3	11	ULS-Set B (auto)/2	CS13_Secondary beam2 - HEA320	-251.66	2.71	-194.44	-0.05	-379.42	13.76
B763	12.6	ULS-Set B (auto)/2	CS1-Column1 - SHS350/350/16.0	-3517.55	19.8	-5.99	-13.16	-10	205.42
B772	0	ULS-Set B (auto)/16	CS1-Column1 - SHS350/350/16.0	-3.75	-29	-18.89	40.13	27.92	18.86
B770	2.880+	ULS-Set B (auto)/17	CS1-Column1 - SHS350/350/16.0	-585.64	-101.83	12.65	0.36	-3.63	145.51
B770	9.320+	ULS-Set B (auto)/1	CS1-Column1 - SHS350/350/16.0	-2388.86	61.04	2.85	3.07	-9.44	145.2
B769	8.660+	ULS-Set B (auto)/2	CS1-Column1 - SHS350/350/16.0	-2460.93	-13.9	-308.02	-64.84	78.94	282.21
B767	0	ULS-Set B (auto)/2	CS1-Column1 - SHS350/350/16.0	-292.7	-15.97	-51.62	-95.93	53.57	57.59
B761	0	ULS-Set B (auto)/2	CS1-Column1 - SHS350/350/16.0	-292.34	-16.72	37.76	96.36	-32.32	56.76
B769	8.660-	ULS-Set B (auto)/5	CS1-Column1 - SHS350/350/16.0	-1790.43	-32.64	-55.35	9.67	-91.53	48.66
B769	9.540+	ULS-Set B (auto)/4	CS1-Column1 - SHS350/350/16.0	-2531.41	16.23	-45.26	-0.05	93.36	247.84
B770	12.6	ULS-Set B (auto)/11	CS1-Column1 - SHS350/350/16.0	-1105.34	-75.22	3.03	-0.94	3.64	-247.65
B770	9.100-	ULS-Set B (auto)/2	CS1-Column1 - SHS350/350/16.0	-2468.93	20.13	216.56	69.89	18.95	357.83
B788	12.6	ULS-Set B (auto)/8	CS3-Column3 - SHS350/350/16.0	-1763.97	-45.21	-16.87	9.58	-25.92	87.53
B783	0	ULS-Set B (auto)/4	CS3-Column3 - SHS350/350/16.0	50.78	-6.89	11.13	41.18	-12.46	2.84
B778	4.200+	ULS-Set B (auto)/2	CS3-Column3 - SHS350/350/16.0	-679.14	-185.65	-31.65	-22.17	49.01	349.38
B803	2.000+	ULS-Set B (auto)/2	CS3-Column3 - SHS350/350/16.0	-717.58	115.73	-25.67	-39.7	36.89	-156.72
B804	0	ULS-Set B (auto)/2	CS3-Column3 - SHS350/350/16.0	-117.76	-16.65	-38.29	-72.53	25.4	20.31
B791	0	ULS-Set B (auto)/1	CS3-Column3 - SHS350/350/16.0	-52.96	-44.94	25.28	73.38	-32.78	25.84
B820	12.6	ULS-Set B (auto)/3	CS3-Column3 - SHS350/350/16.0	-52.82	2.19	-134.12	-25.18	-375.76	7.94
B812	12.6	ULS-Set B (auto)/18	CS3-Column3 - SHS350/350/16.0	-55.59	-1.91	109.17	14.21	307.19	-5.42
B797	12.6	ULS-Set B (auto)/11	CS3-Column3 - SHS350/350/16.0	-666.53	-108.01	-2.76	2.47	-2.99	-266.39
B817	12.6	ULS-Set B (auto)/2	CS4-Column4 - SHS350/350/16.0	-1078.07	64.14	-6.43	5.29	8.97	42.88
B818	5.000+	ULS-Set B (auto)/15	CS4-Column4 - SHS350/350/16.0	-466.32	-66.45	-6.04	-2.57	39.02	104.38
B815	10.200+	ULS-Set B (auto)/8	CS4-Column4 - SHS350/350/16.0	-922.03	24.84	-8.42	-10.02	48.77	-45.26
B817	2.000+	ULS-Set B (auto)/2	CS4-Column4 - SHS350/350/16.0	-129.65	12.68	0.76	-1.21	13.26	22.1
B815	4.200+	ULS-Set B (auto)/4	CS4-Column4 - SHS350/350/16.0	-295.74	72.02	-5.73	-14.8	31.72	-125.36
B818	11.000+	ULS-Set B (auto)/17	CS4-Column4 - SHS350/350/16.0	-723.37	26.5	-5.45	10.23	35.98	-38.31
B815	0	ULS-Set B (auto)/19	CS4-Column4 - SHS350/350/16.0	0	0	0	0	0	0
B816	10.200+	ULS-Set B (auto)/20	CS4-Column4 - SHS350/350/16.0	-865.64	31.01	-6.5	-8.22	57.39	-52.04
B816	4.200+	ULS-Set B (auto)/2	CS4-Column4 - SHS350/350/16.0	-282.13	75.77	-4.54	-7.31	32.27	-132.32

Table 2.2.3.4. 2: Extreme internal forces

3. Conclusion

The study of multi-storey steel carpark design is focused on designing process , calculation and analysis of framed structure both by hand calculation and software.

The slab floor system used in study makes construction faster ,besides it provides structural stability during the construction. The interior sloped floor system design is provided to stay in Eurocode boundary while designing interior ramp.

The study provides calculations which can be utilized in the future design. Besides the study shows material quantity that are used in design in order to give an idea about financial cost.

PROFILE	MATERIAL	NUMBER	LENGTH [mm]	WEIGHT [kg/m]	TOTAL WEIGHT [kg]
SHS 350*350*16	S355	43	12600	165.6	89722.08
		2	12850	165.6	4255.92
		2	13150	165.6	4355.28
		16	13450	165.6	35637.12
SHS 110*110*14.2	S275JR	104	7500	41	31980
HEA 600	S355	60	7550	178	80634
		4	5785	178	4118.92
		8	5765	178	8209.36
		22	6000	178	23496
		6	5000	178	5340
		8	9000	178	12816
		4	4885	178	3478.12
		8	6015	178	8565.36
		6	4500	178	4806
		24	7500	178	32040
HEA 320	S275	12	5000	97.6	5856
		24	6000	97.6	14054.4
TOTAL					**369364.56**

Table 4. 1:List of profiles

Appendix A - Analysis Procedure

In this study, the calculations are prepared on Jupyter notebook so that it can be utilized in the future design.

Python Code of Beam Design

```
import handcalcs.render
from math import sqrt
import forallpeople
forallpeople.environment('structural', top_level = True)
%%render
L =(8.5*m) #for half span
g_f= 2.43*kN/(m**2)*L  #Floor dead load
m_b=(0.66*kN/m)    #SelfWeight,HEA 600(S355),Assumption
g=(g_f + m_b) #Total Dead Load
q_f=2.5*(kN/(m**2))*L #Floor Live load
q=q_f #Total Live Load
w_uls = g*1.35 + q*1.5 #Ultimate Limit State
w_sls = g+q       #Serviceability Limit State
%%render
#Parameters
h=590*mm #depth
b=300*mm #width
t_w=13*mm #thickness
t_f=25*mm #flange thickness
r=27*mm #root radius
m_w=177.8*kg/m #weight
%%render

d=h-2*(t_f+r) #depth between fillets
```

```
I_y=1412*10**6*mm**4 #second moment of area
pi=3.14159265359 #π pi constant
i_y=249.7*mm #radius of gyration
I_T=4075*10**3*mm**4 #torsion constant
I_w=8.8790*10**12*mm**6 #warping constant
G=81000*MPa #shear modulus
%%render
#Parameters
I_z=112.7*10**6*mm**4 #second moment of area
i_z=70.5*mm #radius of gyration
%%render
#Parameters
Wpl_y=5350*10**3*mm**3 #plastic section modulus
Wpl_z=1156*10**3*mm**3 #plastic section modulus
%%render
#Parameters
Wel_y=4787*10**3*mm**3 #elastic section modulus
Wel_z=751.4*10**3*mm**3 #elastic section modulus
%%render
alpha= 0.49
A=22646*mm**2 #area
E=210*MPa #elasticity modulus
f_y=355*MPa #yield strength
f_u=490*MPa #ultimate strength
%%render
#Parameters
gamma_M0= 1.0 #partial safety factor
gamma_M1= 1.0 #partial safety factor
%%render
epsilon= ((235*MPa)/f_y)**(1/2)
%%render
c= (b-t_w-2*r)/2
limit= c/t_f
print("The flange in compression is Class 1") if c/t_f < 9*epsilon else print("The flange in compression is Class 2 ")
%%render
c_b=d
limit= c/t_w
print("The web under bending is Class 1") if c_b/t_w < 72*epsilon else print("The web under bending is Class 2")
%%render
eta= 1
h_w= h-2*t_f
limit= h_w/t_w
limit_2= 72*epsilon/eta
print("The shear buckling resistance of the web does not need to be verified.") if h_w/t_w < 72*epsilon/eta else print("The shear buckling resistance of the web need to be verified.")
%%render
A_v= A-(2*b*t_f)+t_f*(t_w+2*r)
Vpl_Rd = (A_v*(f_y/3**(1/2)))/gamma_M0
%%render
V_Ed =334.18*kN
Vc_Rd=Vpl_Rd
limit= V_Ed/Vc_Rd
print("The shear resistance of the section is adequate.") if V_Ed/Vc_Rd <1.0 else print("The shear resistance of the section is not adequate.")
%%render
limit= Vc_Rd/2

#Shear force at maximum bending moment V_Ed= 334.180 kN
print("Therefore,no reduction in resistance to bending due to shear is required.") if Vc_Rd>V_Ed  else print("Need reduction in resistance to bending due to shear is required.")
%%render
Mc_Rd= Mpl_Rd= (Wpl_y*f_y)/gamma_M0
```

```
%%render
M_Ed= 496.94*kN*m
limit= M_Ed/Mc_Rd
print("Therefore,the bending moment capacity is adequate.") if M_Ed/Mc_Rd <1.0  else print("Therefore,the bending moment capacity is not adequate.")
%%render
L_cr=9000*mm
epsilon=((235*MPa)/f_y)**(1/2)
lambda_1=93.9*epsilon
i=i_z
lambdabar= (L_cr/i)*(1/lambda_1)
UV=0.9
beta_w=1
C_1=3.75
beta=0.75
lambdabar_LT0=0.4
lambdabar_LT= 1*UV*lambdabar*(beta_w**1/2)/(C_1**1/2)
phi_LT = 0.5*(1+alpha*((lambdabar_LT-lambdabar_LT0)+(beta*lambdabar_LT**2)))
chi_LT = 1/(phi_LT+((phi_LT**2)-(beta*lambdabar_LT**2))**(1/2))
chi_LT=1
M_b_Rd= chi_LT*Wel_y*f_y/gamma_M1
print("Therefore,the lateral torsional buckling moment capacity is adequate.") if M_Ed/M_b_Rd <1.0  else print("Therefore,the lateral torsional buckling moment capacity is not adequate.")
```

Python Code of Column Design

```
import handcalcs.render
from math import sqrt
import forallpeople
forallpeople.environment('structural', top_level = True)
%%render
# Parameters
g_k=0.00243*MPa
q_k=0.0025*MPa
g_sk=0.0007*MPa
q_w=0.00032*MPa
B=7500*mm
H=8500*mm
n_floor=3
L_total=34000*mm
H_total=12600*mm
g_steel=7.85e-7*N/mm**3
A_col=7524*mm**2
gamma_G=1.35
gamma_Q=1.5
gamma_M0=1.0
psi_0_snow=0.5
psi_0_wind=0.6
b_c=350*mm #SHS 350*350*16
%%render
# Parameters
#Section properties (SHS 350*350*16)
h=350*mm #width
b=350*mm #side dimension
t=16*mm #thickness
L=3500*mm #height
r_0=24*mm #outer rounding radius
r_i=16*mm #inner rounding radius
A=21101*mm**2 #area
A_v=10551*mm**2 #shear area
%%render

I=389.4*10**6*mm**4 #second moment of inertia
```

```
W_el=2225*10**3*mm**3 #elastic section modulus
W_pl=2630*10**3*mm**3 #plastic section modulus
N_pl_Rd=7490.96*kN #design plastic axial force resistance
V_pl_Rd=2162.46*kN #design plastic shear axial force resistance
M_el_Rd=789.97*kN*m #design elastic bending moment resistance
M_pl_Rd=933.53*kN*m #design plastic bending moment resistance
T_pl_Rd=668.91*kN #design plastic torsional moment resistance
i=i_y=i_z=117*mm #radius of gyration
y_0=0
W_T=3264*10**3*mm**3 #torsion modulus
I_T=609900*10**3*mm**4 #torsion constant
f_y=355*MPa #yield strength
f_u=490*MPa #ultimate strength
gamma_M0=1.00 #resistance of cross-section
gamma_M1=1.00 #resistance of members to instability
G=81000*MPa #shear modulus
alpha=0.21 # buckling curve
pi=3.14159265359 #π pi constant
I_w=1.058*10**12*mm**6 #warping constant
E=210*GPa #modulus of elasticity
%%render
N_c_Rd= A*f_y/gamma_M0#for class 1
%%render
condition_1= N_Ed/N_c_Rd
%%render
L_cr=3500*mm
epsilon=((235*MPa)/f_y)**(1/2)
lambda_1=93.9*epsilon
lambdabar= (L_cr/i)*(1/lambda_1)
phi = 0.5*(1+alpha*(lambdabar-(0.2))+(lambdabar**2))
chi= 1/(phi+((phi**2)-(lambdabar**2))**(1/2))
N_b_Rd= chi*A*f_y/gamma_M1
%%render
condition_2= N_Ed/N_b_Rd
%%render
i_0=(i_y**2+i_z**2+y_0**2)**(1/2)
N_cr_T=(1/i_0**2)*(G*I_T+((pi**2*E*I_w)/L**2))
lambdabar_T= ((A*f_y)/N_cr_T)**(1/2)
phi_T = 0.5*(1+alpha*((lambdabar_T-0.2)+(lambdabar_T**2)))
chi_T = 1/(phi_T+((phi_T**2)-(lambdabar_T**2))**(1/2))
%%render
chi_T = 1.00
N_bT_Rd= (chi_T*A*f_y)/gamma_M1
%%render
e=268.5*mm
V_Ed_6=107.97*kN
M_Ed=M_Ed_6y= V_Ed_6*e
%%render
UV=0.9
beta_w=1
C_1=1
beta=0.75
lambdabar_LT0=0.4
lambdabar_LT=1*UV*lambdabar*(beta_w**1/2)/(C_1**1/2)
%%render
phi_LT = 0.5*(1+alpha*((lambdabar_LT-
lambdabar_LT0)+(beta*lambdabar_LT**2)))
chi_LT = 1/(phi_LT+((phi_LT**2)-(beta*lambdabar_LT**2))**(1/2))
%%render
chi_LT=1
M_b_Rd= chi_LT*W_el*f_y/gamma_M1
%%render
condition_3= (N_Ed/N_b_Rd)+(M_Ed/M_b_Rd)
```

Python Code of Secondary Beam Design

```
import handcalcs.render
from math import sqrt
import forallpeople
forallpeople.environment('structural', top_level = True)
%%render
L =(9*m) #for longest span
g_f= 2.43*kN/(m**2)*L  #Floor dead load
m_b=(0.96*kN/m)   #SelfWeight,HEA 320(S355),Assumption
g=(g_f + m_b) #Total Dead Load
q_f=2.5*(kN/(m**2))*L #Floor Live load
q=q_f #Total Live Load
w_uls = g*1.35 + q*1.5 #Ultimate Limit State
w_sls = g+q       #Serviceability Limit State
%%render
#Parameters
h=310*mm #depth
b=300*mm #width
t_w=9*mm #thickness
t_f=15.5*mm #flange thickness
r=27*mm #root radius
m_w=97.6*kg/m #weight
%%render

d=h-2*(t_f+r) #depth between fillets
I_y=229.3*10**6*mm**4 #second moment of area

i_y=135.8*mm #radius of gyration
%%render
I_z=69.85*10**6*mm**4 #second moment of area
i_z=74.9*mm #radius of gyration
%%render
Wpl_y=1628*10**3*mm**3 #plastic section modulus
Wpl_z=709.7*10**3*mm**3 #plastic section modulus
%%render
Wel_y=1479*10**3*mm**3 #elastic section modulus
Wel_z=465.7*10**3*mm**3 #elastic section modulus
%%render
A=12437*mm**2 #area
E=210*MPa #elasticity modulus
f_y=355*MPa #yield strength
f_u=490*MPa #ultimate strength
%%render
#Parameters
gamma_M0= 1.0 #partial safety factor
gamma_M1= 1.0 #partial safety factor
%%render
epsilon= ((235*MPa)/f_y)**(1/2)
%%render
c= (b-t_w-2*r)/2
limit= c/t_f
print("The flange in compression is Class 1") if c/t_f < 9*epsilon else print
("The flange in compression is Class 2 ")
%%render
c_b=d
limit= c/t_w
print("The web under bending is Class 1") if c_b/t_w < 72*epsilon else print(
"The web under bending is Class 2")
%%render
eta= 1
h_w= h-2*t_f
limit= h_w/t_w
limit_2= 72*epsilon/eta
```

```
print("The shear buckling resistance of the web does not need to be verified.") if h_w/t_w < 72*epsilon/eta else print("The shear buckling resistance of the web need to be verified.")
%%render
A_v= A-(2*b*t_f)+t_f*(t_w+2*r)
Vpl_Rd = (A_v*(f_y/3**(1/2)))/gamma_M0
%%render
V_Ed =195*kN
Vc_Rd=Vpl_Rd
limit= V_Ed/Vc_Rd
print("The shear resistance of the section is adequate.") if V_Ed/Vc_Rd <1.0 else print("The shear resistance of the section is not adequate.")
%%render
limit= Vc_Rd/2

#Shear force at maximum bending moment V_Ed= 334.180 kN
print("Therefore,no reduction in resistance to bending due to shear is required.") if Vc_Rd>V_Ed  else print("Need reduction in resistance to bending due to shear is required.")
%%render
Mc_Rd= Mpl_Rd= (Wpl_y*f_y)/gamma_M0
%%render
M_Ed= 496.94*kN*m
limit= M_Ed/Mc_Rd
print("Therefore,the bending moment capacity is adequate.") if M_Ed/Mc_Rd <1.0  else print("Therefore,the bending moment capacity is not adequate.")
```

Python Code of Connection Design

```
import handcalcs.render
from math import sqrt
import forallpeople
forallpeople.environment('structural', top_level = True)
%%render
f_yb=640*(N/mm/mm)
f_ub=800*(N/mm/mm)
f_y=355*(MPa)
#Parameters
b=160*mm
d_p=300*mm
t=10*mm
%%render
alpha_v = 0.6
gamma_M2 = 1.25
A_s=157*(mm*mm)

F_vRd = (alpha_v * A_s * f_ub )/ gamma_M2
%%render
e_2 = 80*mm
d_0 = 18*mm
k_1 = min (((2.8*e_2/d_0)-1.7) , 2.5)
%%render
e_1= (50*mm)
p_1= (60*mm)
f_u= (360*MPa)

alpha_b = min((e_1/(3*d_0),(p_1/(3*d_0)-0.25),(f_ub/f_u),(1)))
%%render
e_1_1= (170*mm)
p_1= (60*mm)
f_u= (360*MPa)

alpha_b = min((e_1_1/(3*d_0),(p_1/(3*d_0)-0.25),(f_ub/f_u),(1)))
%%render
d=16*mm
```

```
t=10*mm
F_bRd = k_1*alpha_b*d*t*f_u/gamma_M2
%%render
V_rd= 4*F_bRd
%%render
V_Ed=107.97*kN
e_boltgroup=268.5*mm
M_ed = e_boltgroup * V_Ed
%%render
a=(6*mm)
l=(300*mm)
W_el_w= ((2*a*l**2)/6)
sigma_w = M_ed/ W_el_w
%%render
tau_L =sigma_L = sigma_w/2**(1/2)
tau_II= V_Ed/(2*a*l)
%%render
beta_w = 0.9
C_1= ((sigma_L**2+3*(tau_L**2+tau_II**2))**(1/2))
C_2= f_u/(beta_w*gamma_M2)
%%render
sigma_L=sigma_w/2**(1/2)
C_3= f_u/gamma_M2
%%render
A_nt = t*(e_2-(d_0/2))
A_nv = t*(e_2+p_1-d_0-d_0/2)
gamma_M0=1.00
V_eff2Rd= ((0.5* A_nt*f_u)/gamma_M2)+ (A_nv*f_y)/(3**(1/2)*gamma_M0)
%%render
A_v=t*d_p
V_plRd= (A_v*f_y)/(3**(1/2)*gamma_M0)
%%render
t_w=13*mm
A_ntw= t_w*(e_2-d_0/2)
A_nvw= t_w*(120*mm+e_1+p_1-d_0-(d_0/2))
%%render
V_eff2Rd= ((0.5*A_ntw*f_u)/gamma_M2)+(A_nvw*f_y)/(3**(1/2)*gamma_M0)
%%render
M_elRd = (W_el_w * f_y)/gamma_M0
```

Python Code of Column Base Plate Design

```
import handcalcs.render
from math import sqrt
import forallpeople
forallpeople.environment('structural', top_level = True)
%%render
# Parameters
g_k=2.43*kN/m**2
q_k=2.5*kN/m**2
g_sk=0.7*kN/m**2
q_w=0.32*kN/m**2
B=7.5*m
H=8.5*m
n_floor=3
L_total=34*m
H_total=12.6*m
g_steel=785*N/m**3
A_col=7524*mm**2
gamma_G=1.35
gamma_Q=1.5
gamma_M0=1.0
psi_0_snow=0.5
psi_0_wind=0.6
```

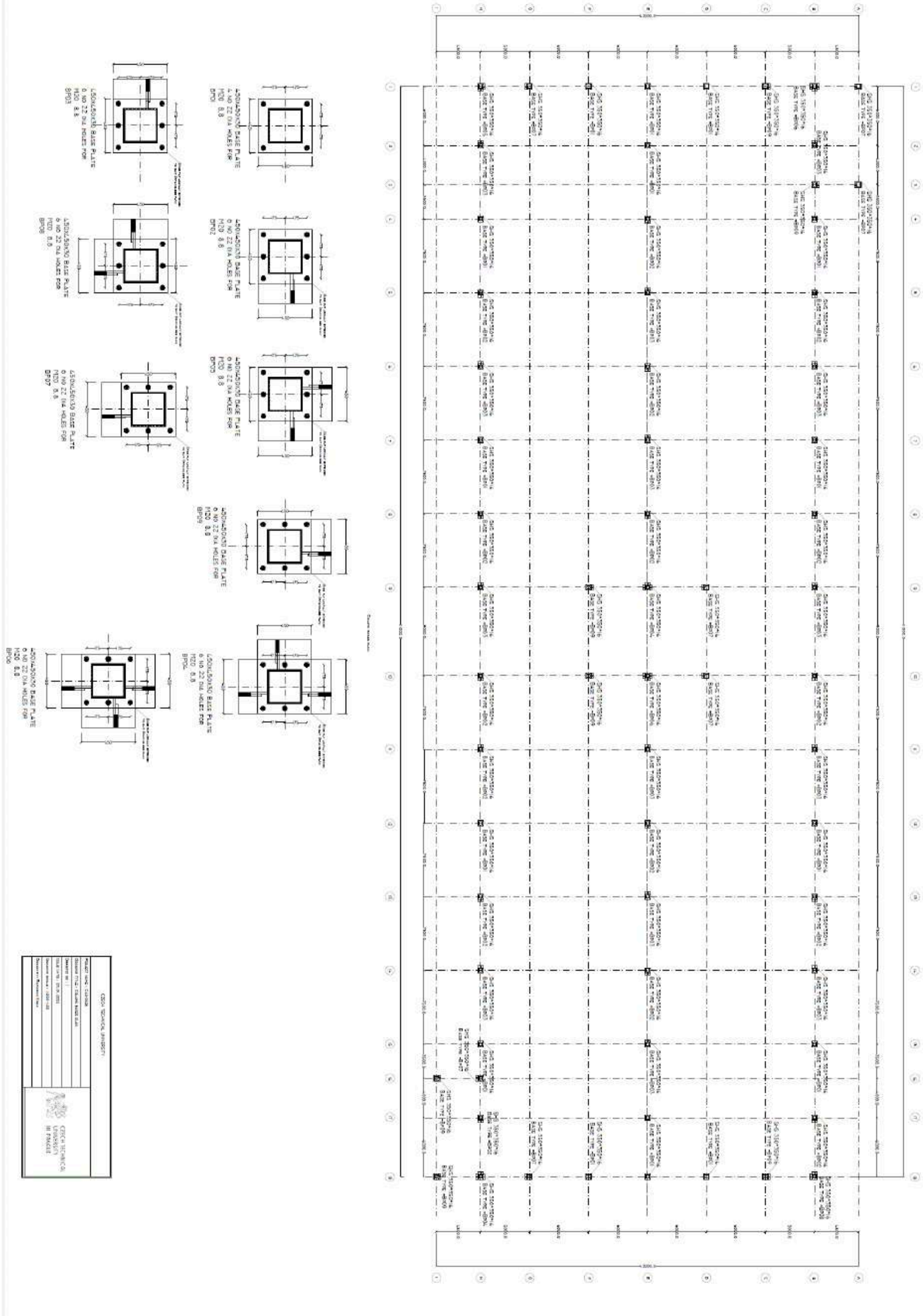
```
b_c=350*mm #SHS 350*350*16
%%render
A_load= B*H*2
%%render
N_self= A_col*g_steel*H_total
%%render
N_dead= A_load*g_k*n_floor
%%render
N_live= A_load*q_k*n_floor
%%render
N_snow= A_load*g_sk
%%render
M_wind= q_w*(L_total/2)*((H_total**2)/2)
N_wind= M_wind/B    #Tension force
%%render
N_Ed= gamma_Q*(N_self+N_dead)+gamma_Q*(N_live+N_snow*psi_0_snow+N_wind*psi_0_wind)
%%render
# Parameters
t=30*mm
b_1=450*mm
d_1=450*mm
b=500*mm
d=500*mm
h=500*mm
b_r=425*mm
#Column Base Material
f_y=355*MPa
E=210*GPa
gamma_0=1.0
#Pad footing material: concrete C25/30
f_ck=25*MPa
gamma_c=1.5
#Grout not specified
%%render
b_2= min(b_1+2*b_r,3*b_1,b_1+h)
d_2= min(d_1+2*b_r,3*d_1,d_1+h)
%%render
k_j= ((b_2*d_2)/(b_1*d_1))**(1/2)
beta_j =2/3 #joint coeff.
%%render
f_jd= (beta_j*k_j*f_ck)/gamma_c
%%render
c= t*(f_y/(3*f_jd*gamma_M0))**(1/2)
%%render
A_eff=(2*c+b_c)**2
%%render
N_b_Rd= A_eff*f_jd
%%render
R_condition = N_Ed/N_b_Rd
%%render
V_Ed= gamma_Q*q_w*(L_total/2)*(H_total/2)
%%render
# Parameters
f_ub=490*MPa
f_yb=355*MPa
A_s=245*mm**2
n_b=2
alpha_v=0.5
gamma_M2=1.25
%%render
alpha_b=0.44-0.0003*1/MPa*f_yb
F_1vb_Rd= 0.85*(alpha_v*f_ub*A_s)/gamma_M2
F_2vb_Rd= (alpha_b*f_ub*A_s)/gamma_M2
%%render
F_vb_Rd= min(F_1vb_Rd,F_2vb_Rd)
```

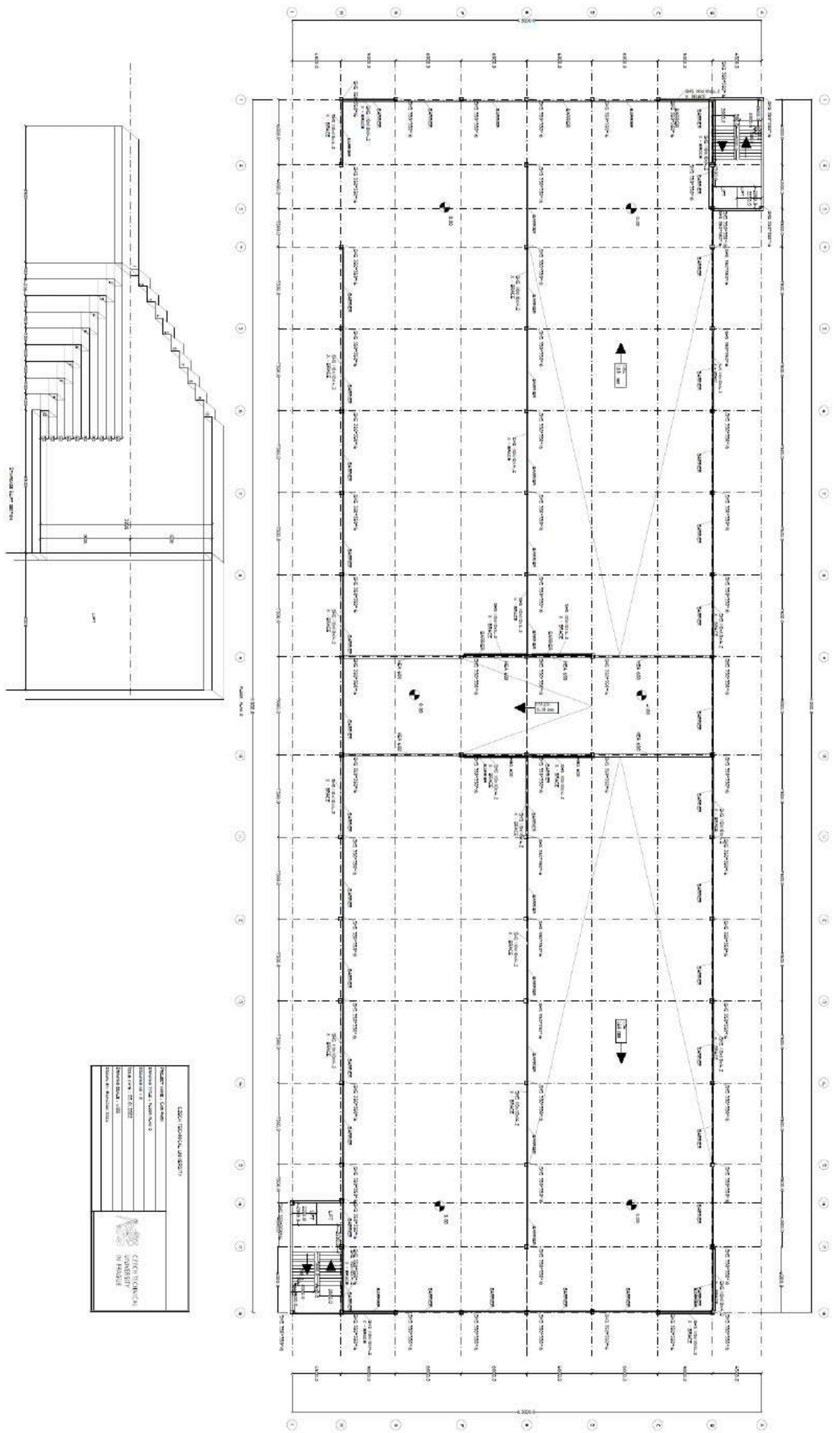
```
%%render
V_Rd= n_b*F_vb_Rd
%%render
R_condition2= V_Ed/V_Rd
```

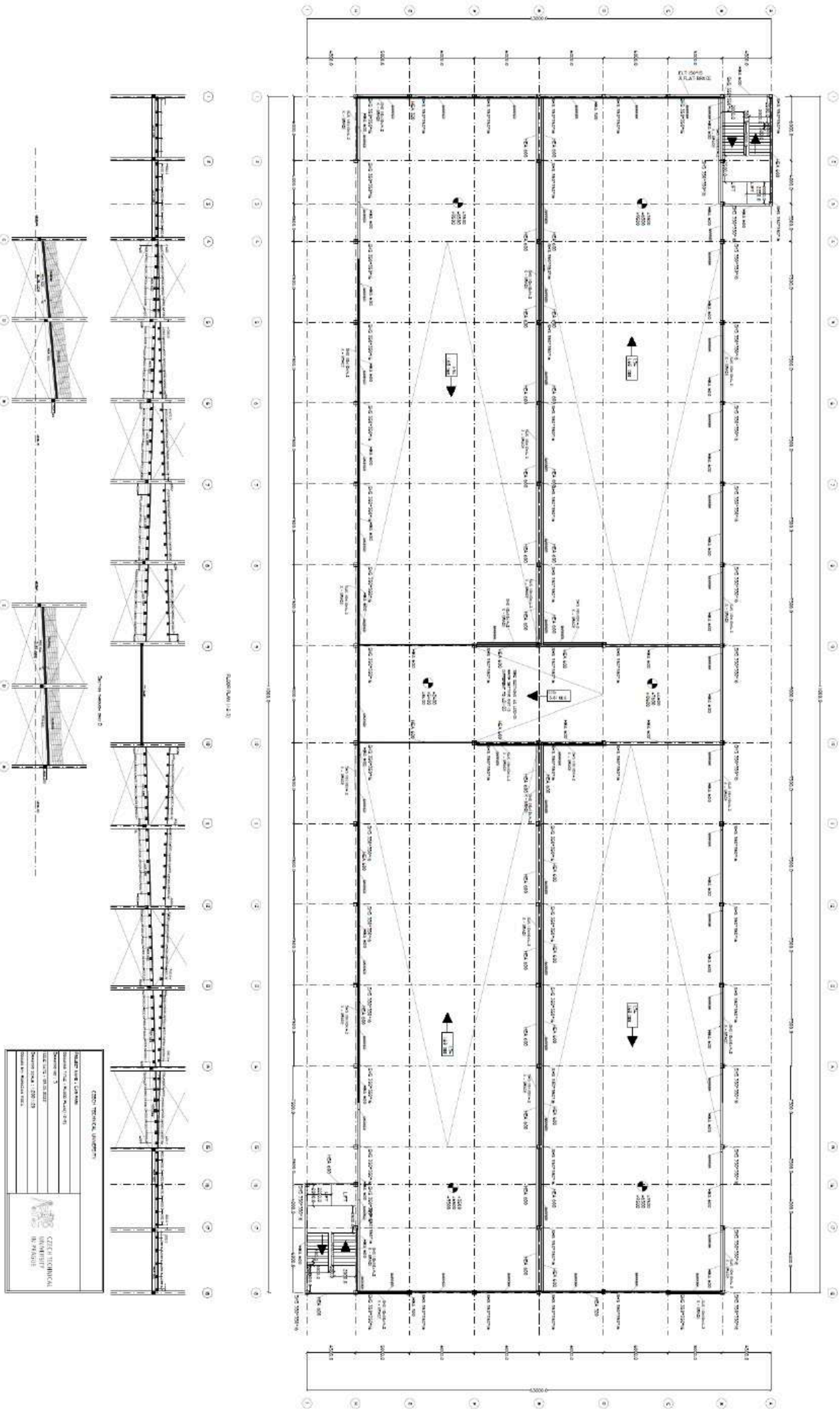
Python Code of Bracing Member Design

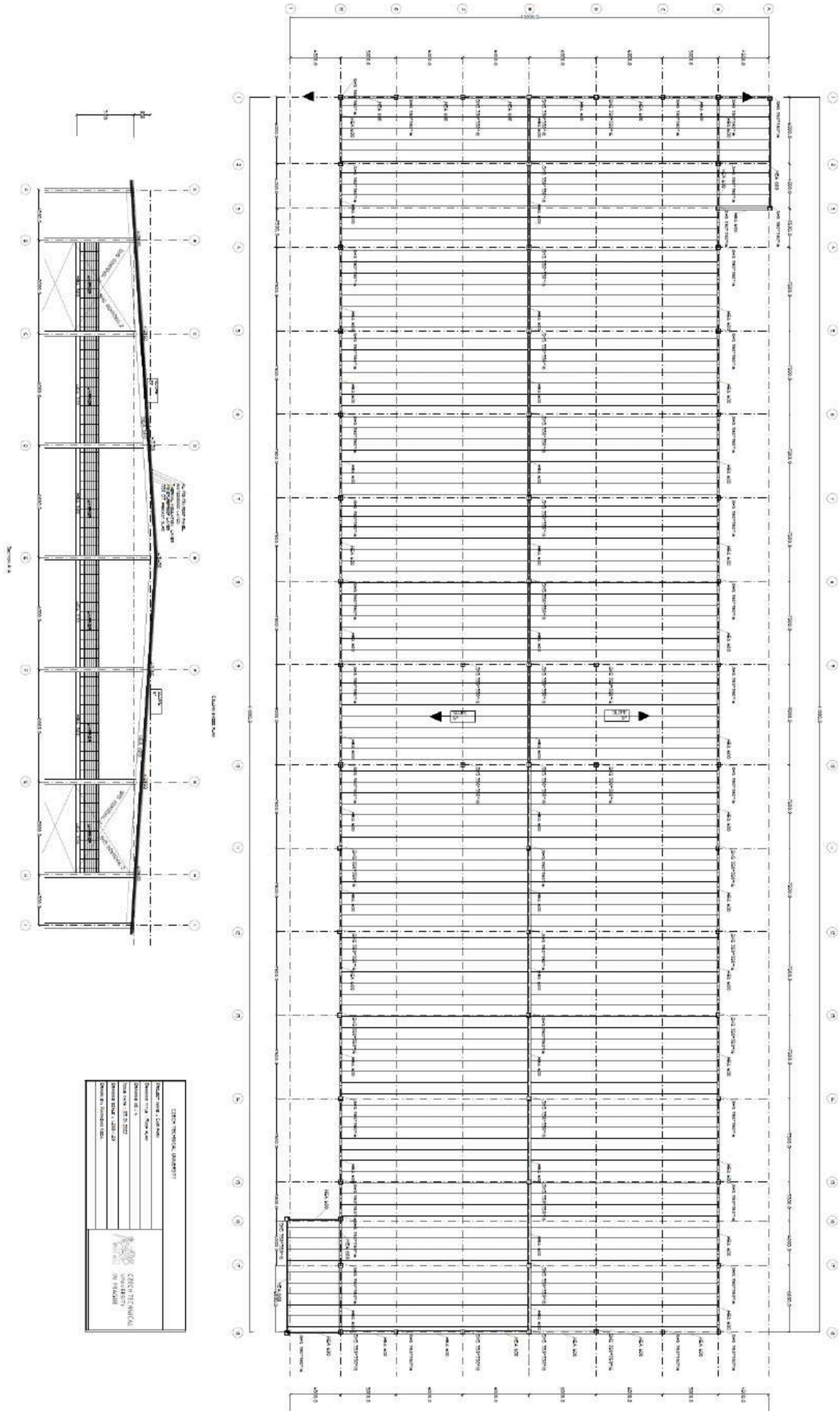
```
import handcalcs.render
from math import sqrt
import forallpeople
forallpeople.environment('structural', top_level = True)
%%render
b=120*mm
t=14.2*mm
A_s=5790*mm**2
f_y=275*MPa
L=8276*mm
E=210*GPa
I_y=10500000*mm**4
I_z=10500000*mm**4
gamma_M0=1.0
gamma_M1=1.0
alpha=0.21
pi=3.14159265359
%%render
N_Ed=259.22*kN #from SCIA
N_c_Rd= (A_s*f_y)/gamma_M0
%%render
C_1= N_Ed/N_c_Rd
%%render
L_cr=L
N_cr= (pi**2*E*I_z)/L_cr**2
lambdabar= ((A_s*f_y)/N_cr)**(1/2)
phi = 0.5*(1+alpha*(lambdabar-(0.2))+(lambdabar**2))
chi= 1/(phi+((phi**2)-(lambdabar**2))**(1/2))
N_b_Rd= chi*A_s*f_y/gamma_M1
%%render
C_2= N_Ed/N_b_Rd
```

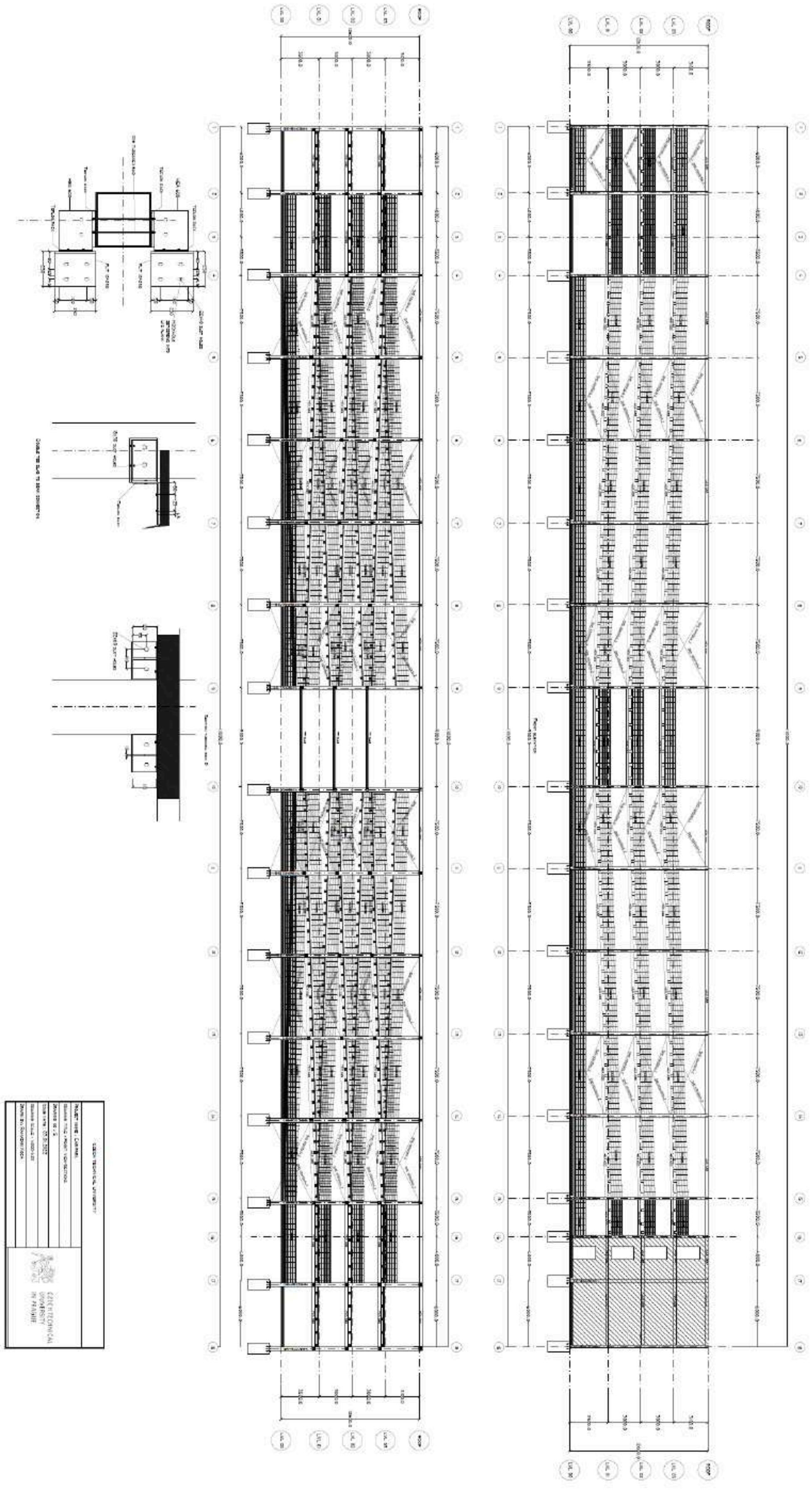
Appendix B - Drawings

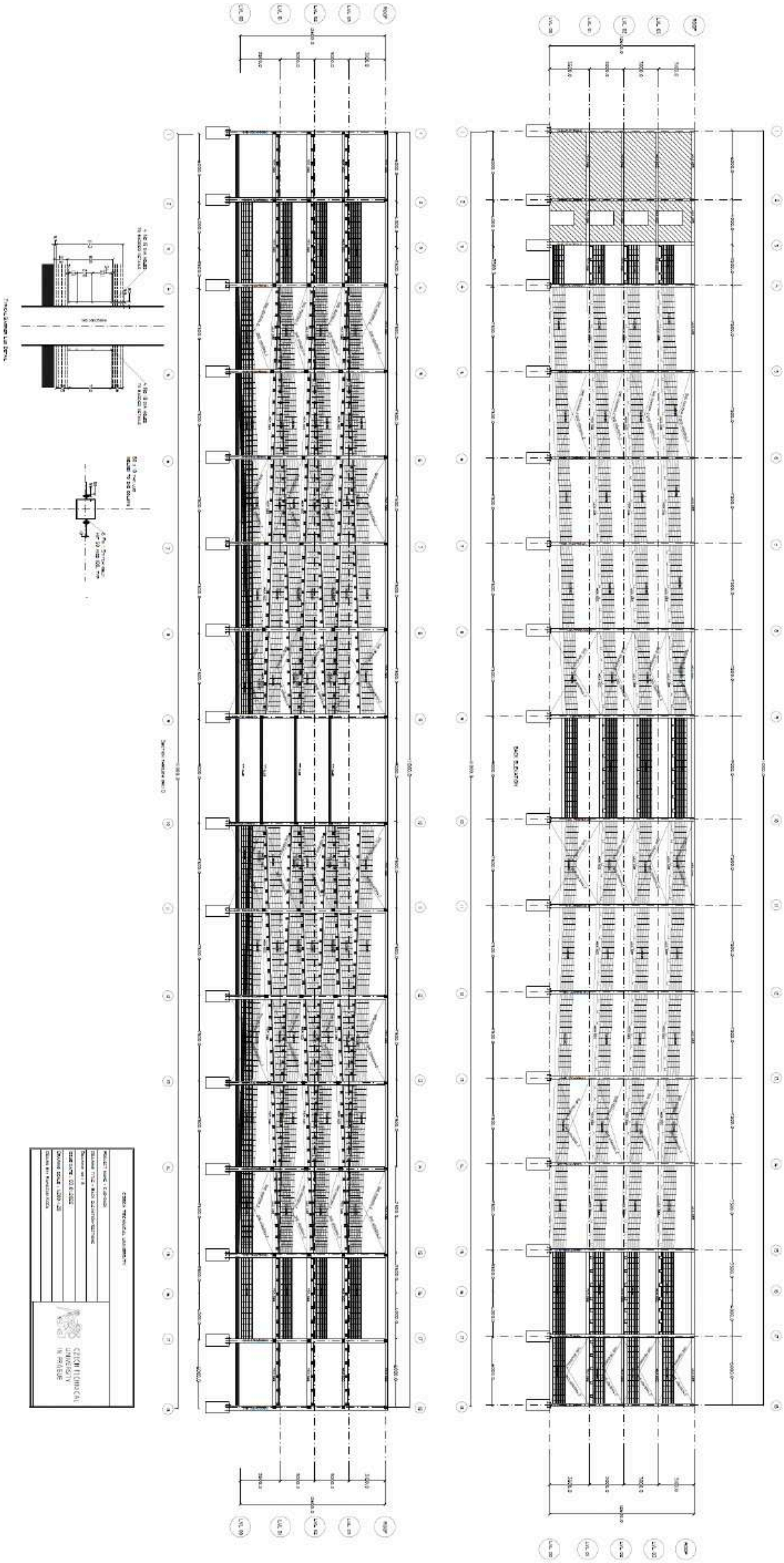


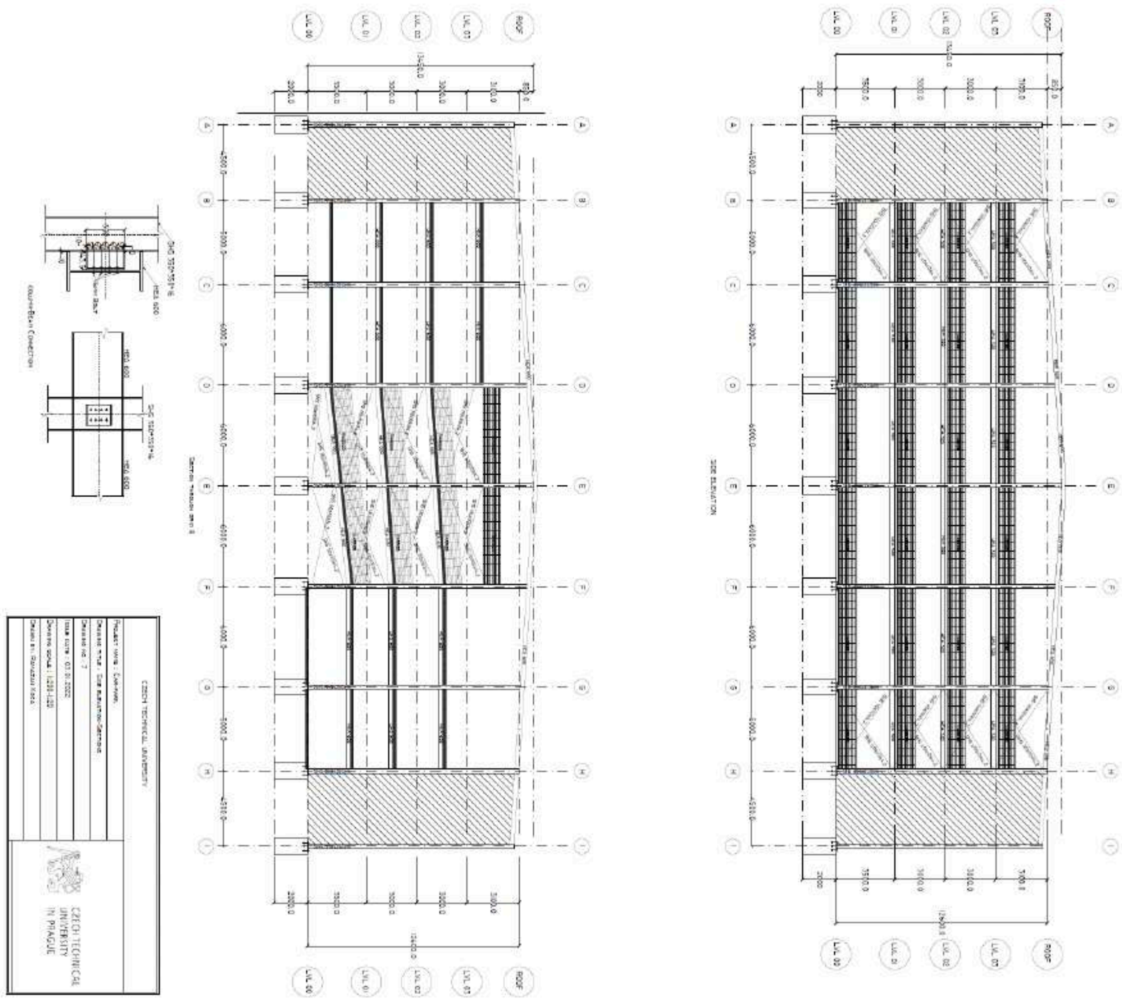












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Further Reading

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List of Figures

List of Tables