MOMENT RESISTING STEEL-CONCRETE COMPOSITE FRAMES UNDER THE COLUMN LOSS SCENARIO: DESIGN OF AN EXPERIMENTAL STUDY

Riccardo Zandonini [1]
Nadia Baldassino [1]
Fabio Freddi [2]

[1] Department of Civil, Environmental and Mechanical Engineering, University of Trento, Trento, Italy
[2] Department of Civil, Environmental & Geomatic Engineering, University College of London, London, WC1E 6BT, U.K., formerly Department of Civil, Environmental and Mechanical Engineering, University of Trento, Trento, Italy

UNIVERSITÀ DEGLI STUDI DI TRENTO

ISBN (online) 978-88-8443-816-4

DOI 10.15168/11572_219133

Publisher: Università degli Studi di Trento, via Calepina, 14 – 38122 Trento (Italy)

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Date of publication: November 2018

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1 INTRODUCTION

Robustness is a very topical research issue, and several research work have been carried out in recent years. The University of Trento was involved in the project 'Robust impact design of steel and composite building structures' (acronym ROBUSTIMPACT). The project was financially supported in the framework of the European RFCS program (Research Fund for Coal and Steel). This document concerns the experimental activities carried out at the Laboratory of Material and Structural Testing (LMST) of the University of Trento.

The project focuses on the behaviour of composite steel and concrete framed buildings subject to accidental actions. Within the project, several experimental analyses were performed by the partners ranging from the local to the global response.

As to the global behaviour, at LMST two 3D full-scale tests on steel-concrete composite sub-frames simulating the total loss of an impacted column were performed. The purpose of the study is to improve the state of knowledge on the contribution offered by the joints and by the 3D slab system in terms of activation of an alternative mechanisms of resistance. The research assumes two reference case studies of steel and concrete five-story buildings differing for in-plane column layout. Two sub-frames were 'extracted' from these structures and tested in the laboratory.

The plan of the experimental tests required preliminary studies devoted to:

- selection and definition of a reference buildings assumed as case studies,
- design of the reference buildings according to the Eurocodes;
- identification of representative substructures (slab-beam system, columns and joints) from the reference structures to be experimentally investigated;
- design of the testing set-up

This report concern the design of the case studies, of the specimens and of the testing set-up.

In particular, Section 2 illustrates the design, based on the relevant Eurocodes, of the reference structures for both the geometric configurations. Section 3 reports the design of the sub-structures including also the numerical analysis and the design of the testing setup. Finally, Section 4 provides the details of the components of the two specimens as needed for their fabrication, and of the testing set-up. All the related drawings are reported in Annexes A-C.

2 DESIGN OF THE CASE STUDY STRUCTURES

2.1 Geometry

A five-story composite steel and concrete structure has been selected as case study structure. The total dimensions of the building are 34.2 *m* in *X* direction, 11.4 *m* in *Y* direction and the total height is of 18 *m* and it consists of six bays in the *X* direction and two bays in the *Y* direction. Two different geometric configurations of the frames are investigated. The first configuration is symmetric with respect to *X* and *Y* direction (Figure 2-1) while the second configuration is symmetric only with respect to the *Y* direction (Figure 2-6). The two case study structures will be called hereinafter as "*Symmetric*" and "*Asymmetric*" configurations respectively.

Both the building configurations are made by using the same steel sections type for beams (*IPE 240*) and columns (*HEB 220*) and the same thickness of the slab (150 *mm*). This choice is made in order to reduce the number of variables and hence, in order to simplify the comparison of the results between the two structures. Also the joint connections of the two geometric configurations are made in the same way and the only difference is in the rebars dimension and layout of the slab.

The steel braces designed to resist the horizontal forces in *X* direction are positioned in the frames *A* and *C* (Figure 2-1 and Figure 2-6), while, those needed to resist the horizontal forces in *Y* direction are positioned in the frames *4* and *7* (Figure 2-1 and Figure 2-6). Even if it is not the optimal solution to obtain a good seismic behavior, this choice is made in order to identify a portion of structure that is free from steel braces which can be hence simply reproduced in laboratory. Moreover, this make the sub-structure more representative of a general case.

In the *Symmetric* structure the two bays in *Y* direction have the same dimension of 5.7 *m*. Figure 2-1, Figure 2-2 and Figure 2-3 reports the typical floor framing plan and the frame elevation respectively in *X* and *Y* direction. The characteristics of the floor framing plan are the same at all the stories. In the *Asymmetric* structure the two bays in *Y* direction have different dimensions of 7.125 *m* and 4.275 *m*. Figure 2-6, Figure 2-7 and Figure 2-8 reports the typical floor framing plan and the frame elevation respectively in *X* and *Y* direction. The characteristics of the floor framing plan are the same at all the stories.

The cover of the rebars is equal to 20 *mm.* Wires of ϕ10/150×150 *mm* are uniformly distributed in the top and bottom side of the slab. Moreover, several additional reinforcements are required in several zones. Figure 2-4 and Figure 2-5 illustrate the distribution of the reinforcements in the slab respectively in the upper side and in the lower side of the *Symmetric* structure. While, Figure 2-9 and Figure 2-10 illustrate the distribution of the reinforcements in the slab respectively in the upper side and in the lower side of the *Asymmetric* structure.

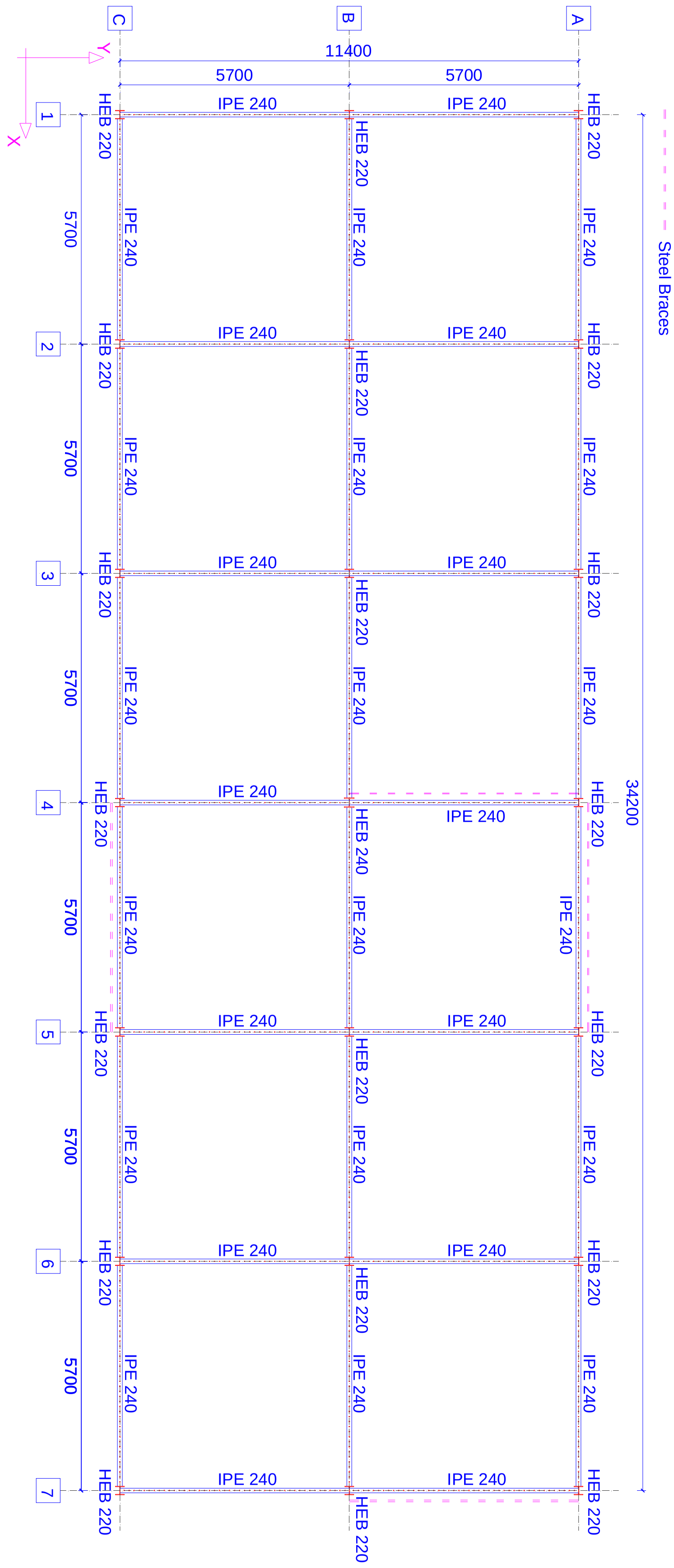


Figure 2-1. Floor Framing Plan – Symmetric Configuration (length unit mm)

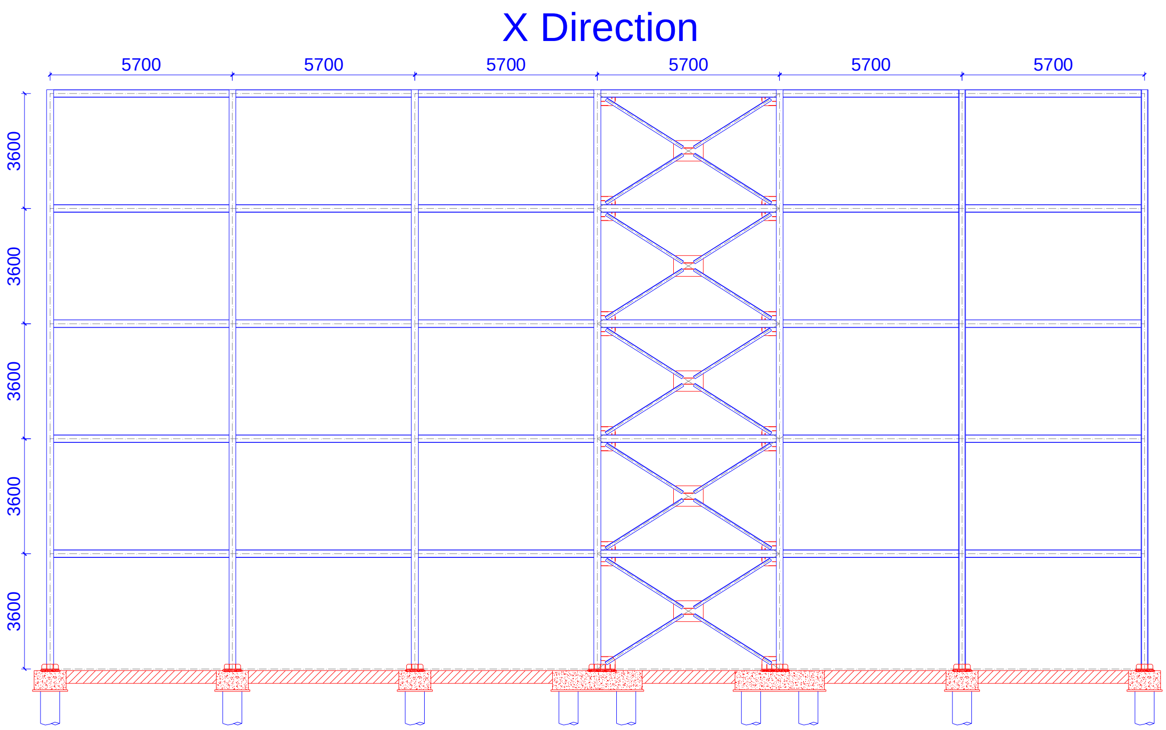


Figure 2-2. Frame Elevation – X direction – Symmetric Configuration (length unit mm)

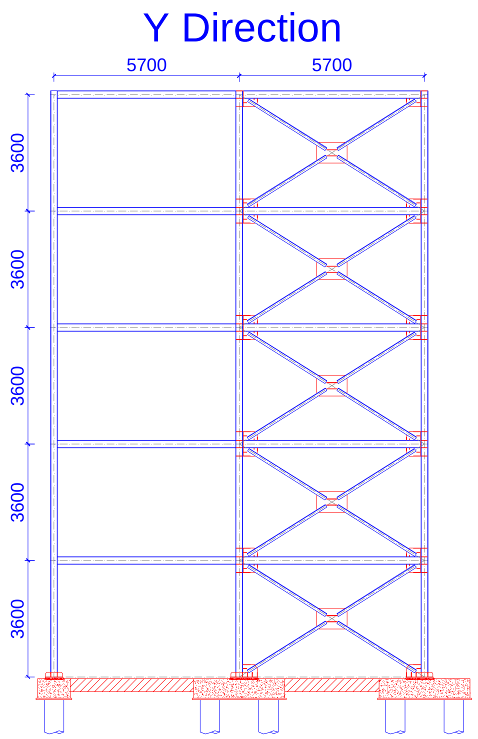


Figure 2-3. Frame Elevation – Y direction – Symmetric Configuration (length unit mm)

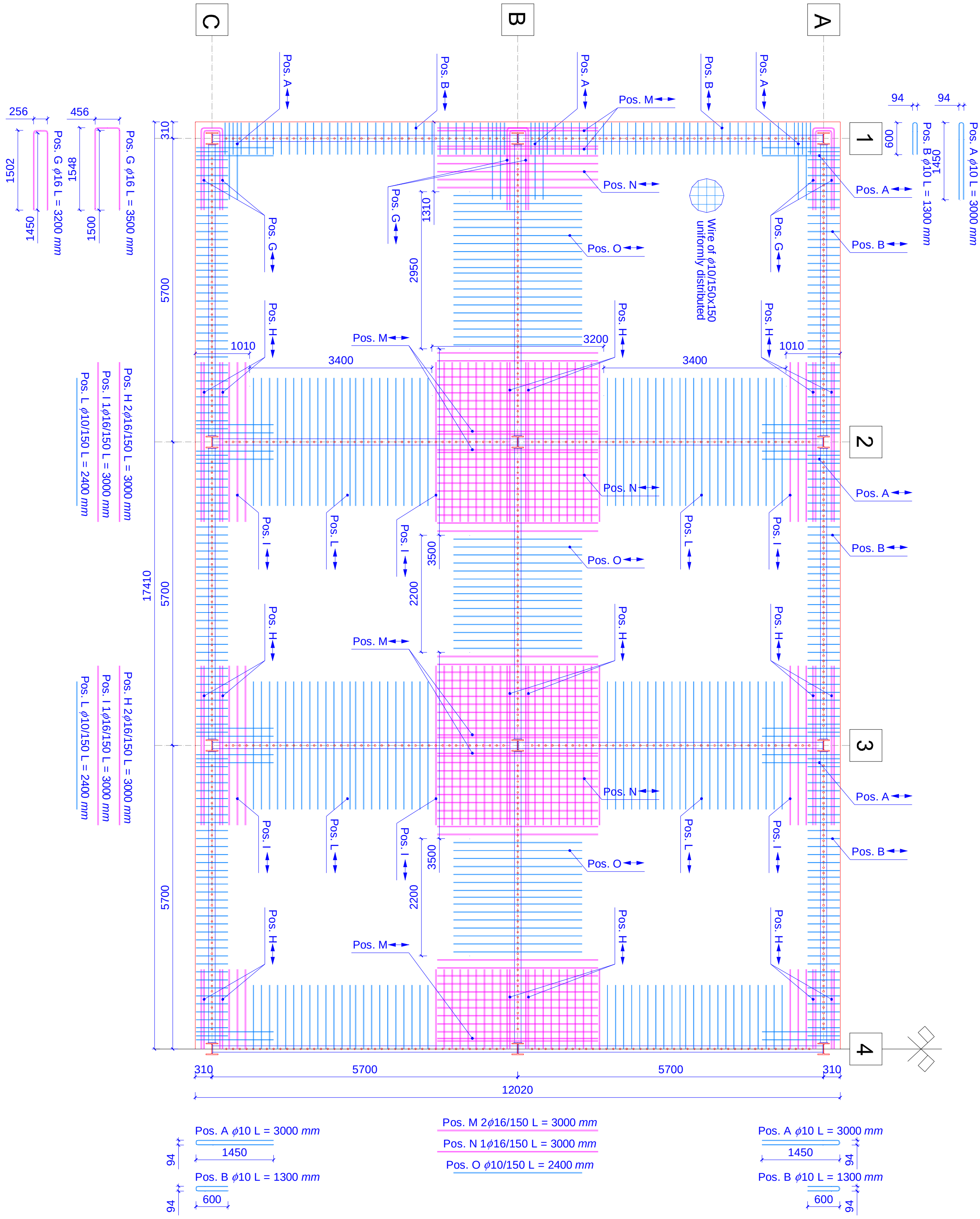


Figure 2-4. Slab Rebars - Upper Side – Symmetric Configuration (length unit mm)

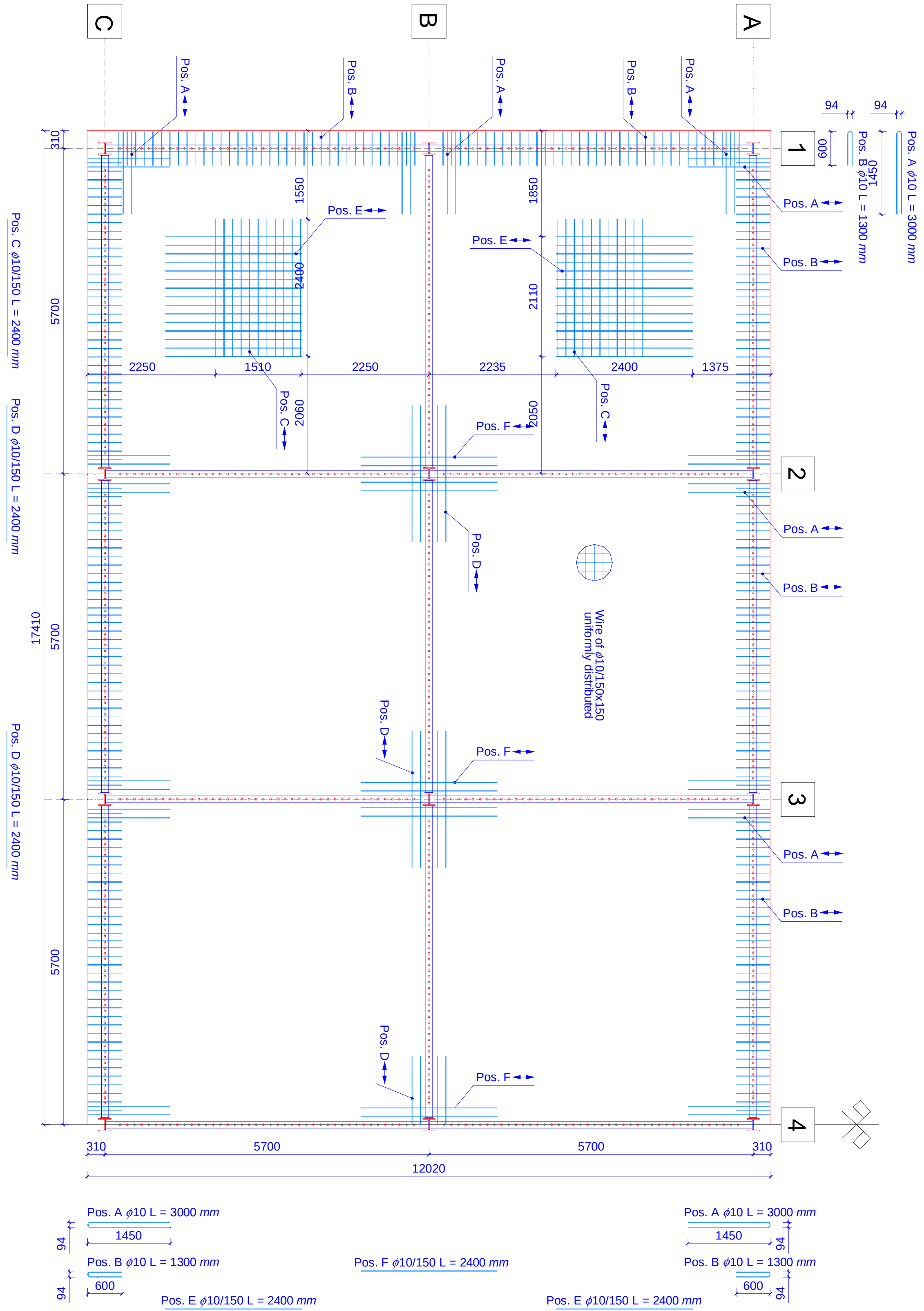


Figure 2-5. Slab Rebars - Lower Side – Symmetric Configuration (length unit mm)

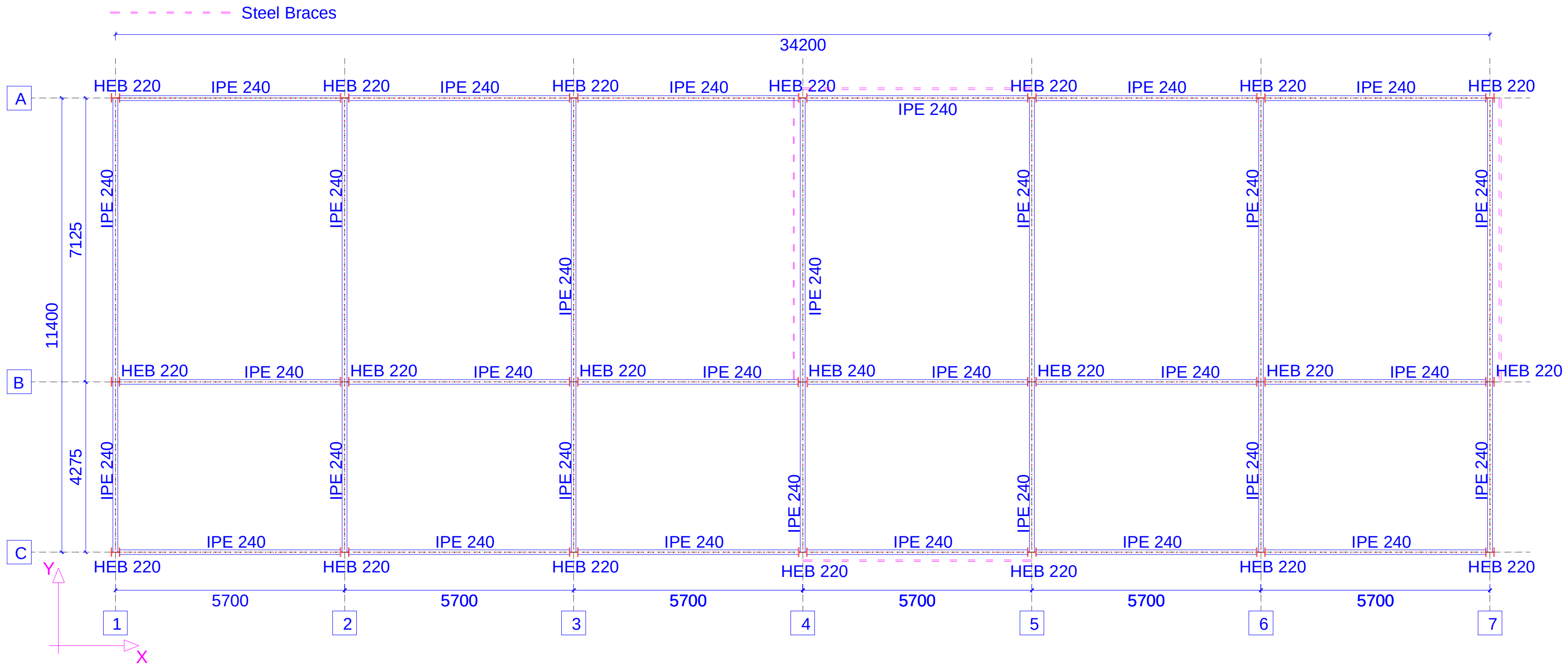


Figure 2-6. Floor Framing Plan – Asymmetric Configuration (length unit mm)

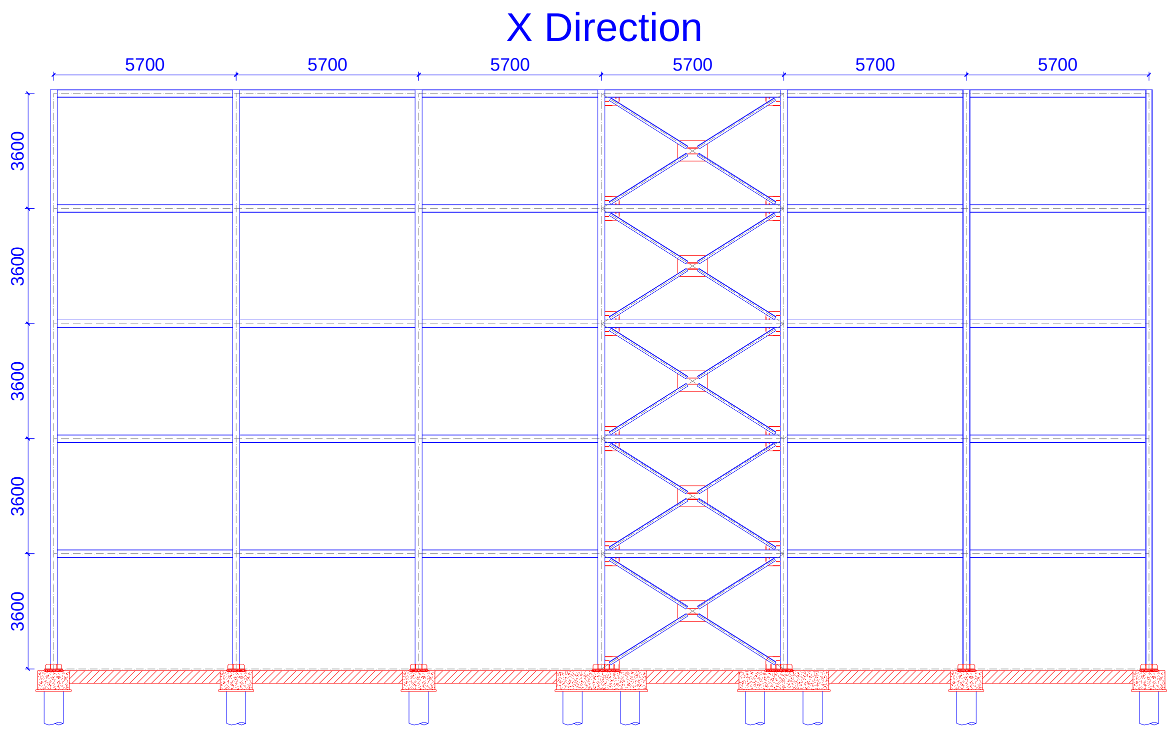


Figure 2-7. Frame Elevation – X direction – Asymmetric Configuration (length unit mm)

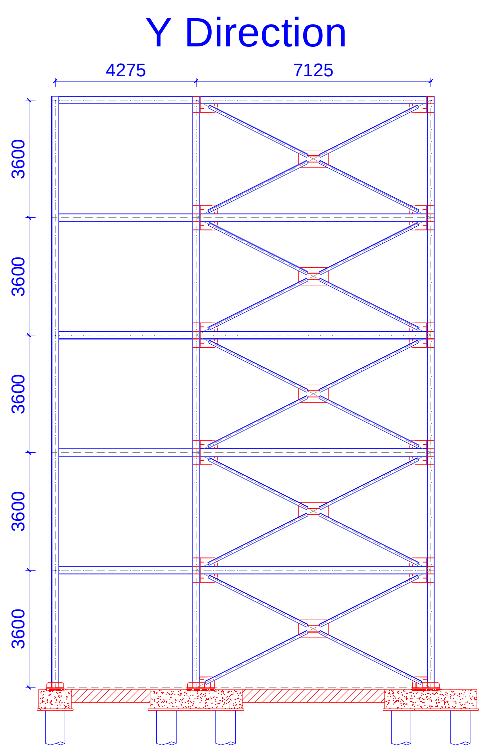


Figure 2-8. Frame Elevation – Y direction – Asymmetric Configuration (length unit mm)

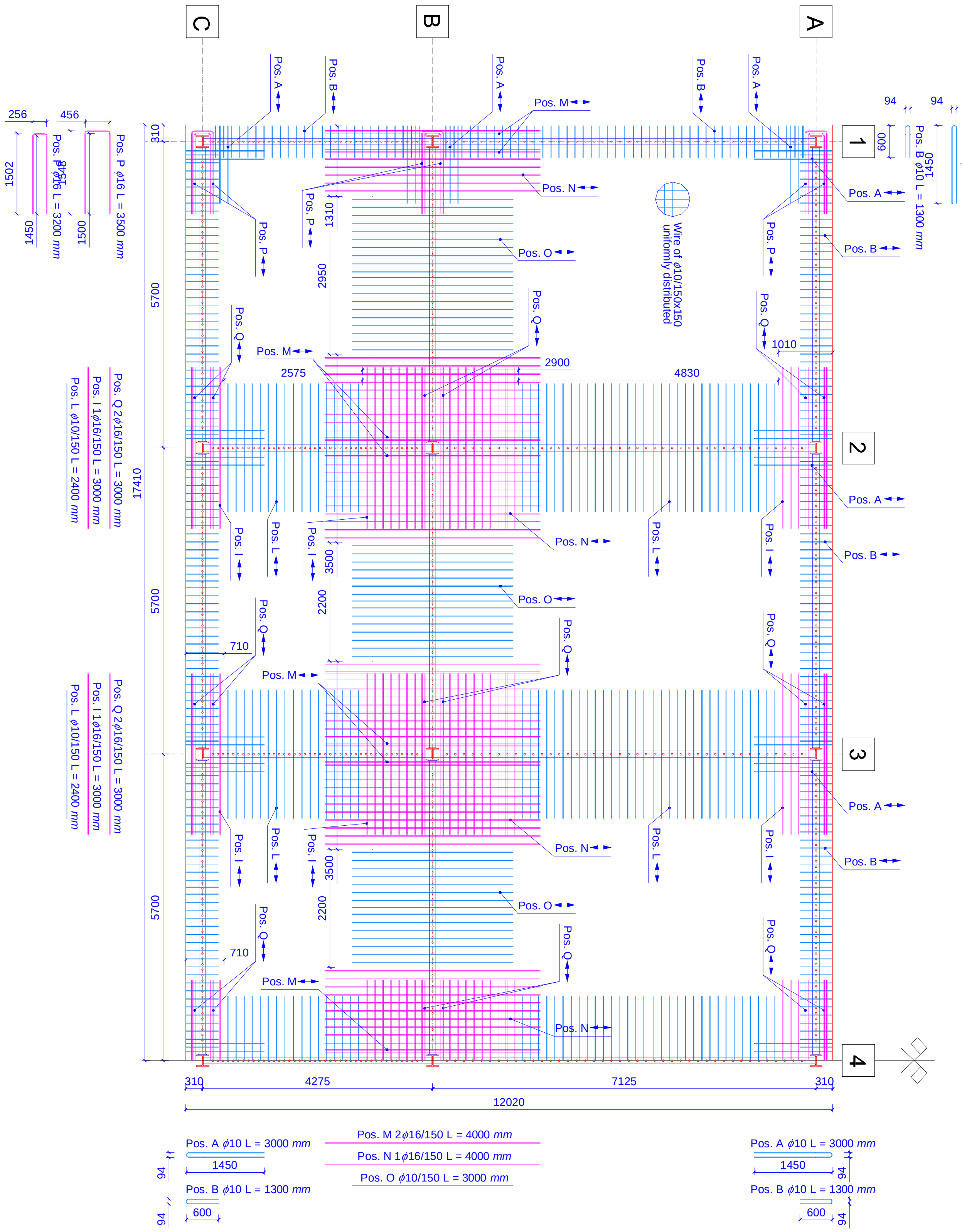


Figure 2-9. Slab Rebars - Upper Side – Asymmetric Configuration (length unit mm)

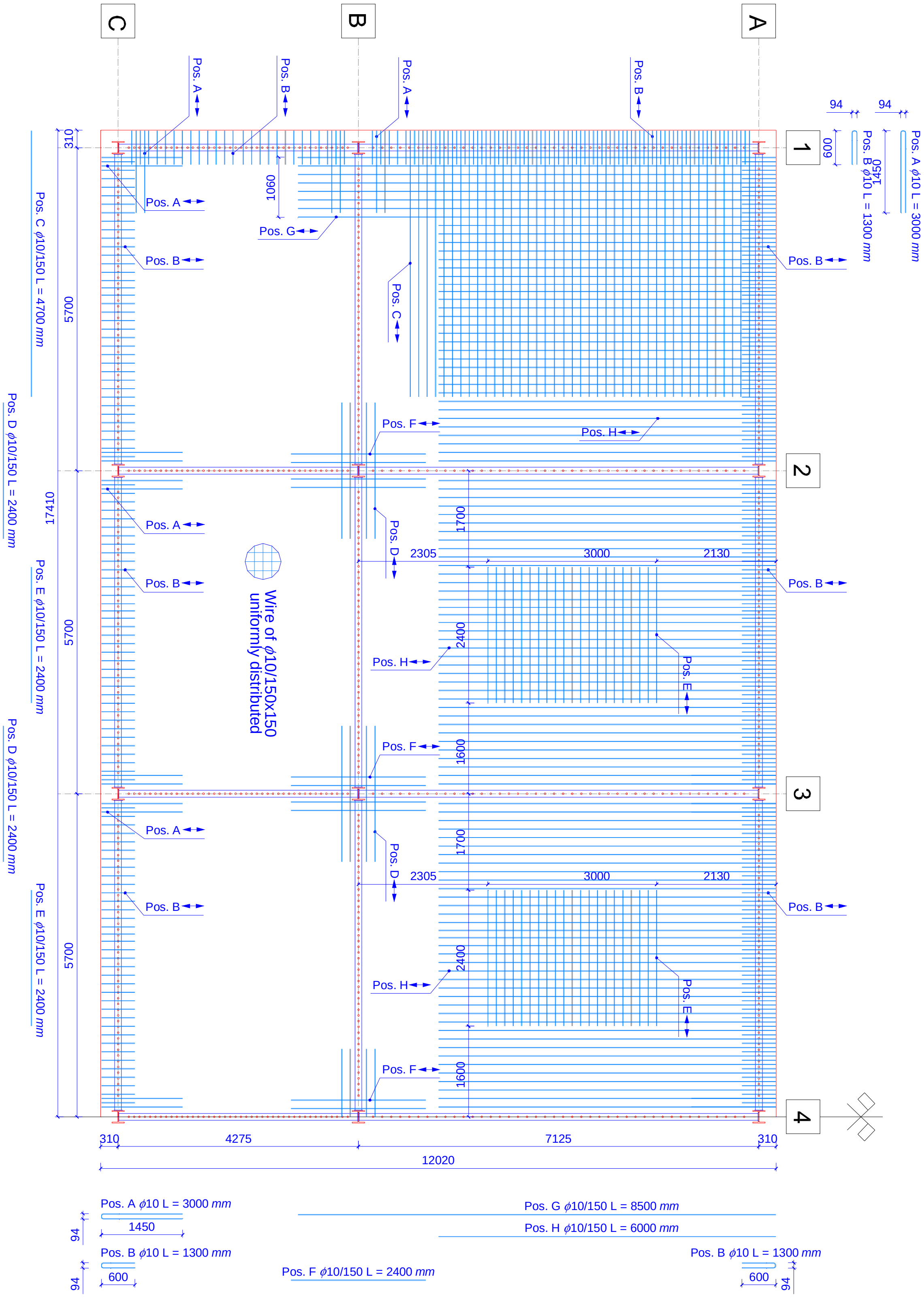


Figure 2-10. Slab Rebars - Lower Side – Asymmetric Configuration (length unit mm)

2.2 Finite Element Model

In order to design the case study structures, the Finite Element Model of the 3-D frame has been developed by using the SAP 2000 program [1] . The frame is fixed at the base in both the directions and employs steel braces modeled as elastic elements to resist to the horizontal forces. The model employs the elastic 2-D elements "*Frame*" to model the behavior of beams and columns and elastic "*Shell*" elements to model the behavior of the slab. The slab is rigidly connected to the beams in order to simulate the behavior of the complete interaction given by the shear connection. The connection between beams and columns is modeled by a pinned connection in the *Y* direction where the beams are connected to the web of the column, differently, in the *X* direction the beam-column connections are characterized by an adequate stiffness calculated by following the instructions of EN 1993-1-8 [2].

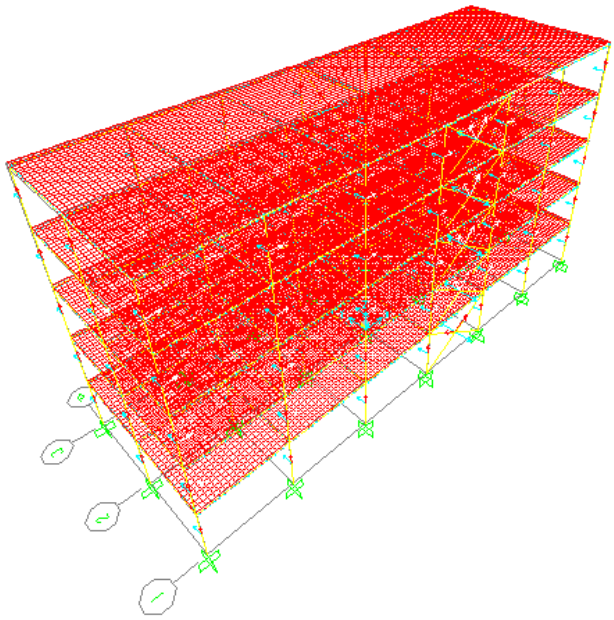


Figure 2-11. 3-D Finite Element Model – Symmetric Configuration

2.3 Materials

The materials used for the design of the structures are listed below. There is no difference between the materials used in the *Symmetric* and in the *Asymmetric* structure.

Concrete – C30/37 (EN 1992-1-1 §2.4.2.4 and EN 1992-1-1 Table 2.1N [3])

$f_{ck} = 30$ MPa;

$R_{ck} = 37$ MPa;

$f_{cm} = 38$ MPa;

$f_{ctm} = 2.896$ MPa;

$E_{cm} = 32.836$ GPa;

$\gamma_c = 1.5$ for persistent and transient design situations;

$\gamma_c = 1.2$ for accidental design situations;

Rebars – B450C (EN 1992-1-1 §2.4.2.4 and EN 1992-1-1 Table 2.1N [3])

$f_{yk} = 450\ \mathrm{MPa}$;

$f_{tk} = 540\ \mathrm{MPa}$;

$E_s = 210\ \mathrm{GPa}$;

$\gamma_s = 1.15$ for persistent and transient design situations;

$\gamma_s = 1.00$ for accidental design situations;

Steel – S355 (EN 1993-1-1 Table 3.1 and EN 1993-1-1 §6.1 [4])

$f_y = 355\ \mathrm{MPa}$;

$f_u = 510\ \mathrm{MPa}$;

$E_s = 210\ \mathrm{GPa}$;

$\gamma_{M0} = 1.00$ for resistance of cross-sections;

$\gamma_{M1} = 1.00$ for resistance of members to instability;

$\gamma_{M2} = 1.25$ for resistance of cross-section in tension to fracture;

Bolts – Class 10.9 (EN 1993-1-8 Table 3.1 [2] and EN 1992-1-1 Table 2.1N [3])

$f_{yb} = 900\ \mathrm{MPa}$;

$f_{ub} = 1000\ \mathrm{MPa}$;

$\gamma_{M2} = 1.25$ for resistance of bolts;

2.4 Actions

The actions considered for the design of the structures are reported in the following sections. There is no difference between the actions of the *Symmetric* and of the *Asymmetric* structure.

2.4.1 *Self-Weight*

- Slab $G_{k,Slab} = 3.75\mathrm{kN/m^2}$
- Beam IPE 240 $G_{k,Beam} = 0.301\mathrm{kN/m}$
- Column HEB 220 $G_{k,Col} = 0.701\mathrm{kN/m}$
- Finishes $G_{k,2} = 2.00\mathrm{kN/m^2}$

2.4.2 *Variable Action*

- Imposed load, Category B $Q_k = 3.00\mathrm{kN/m^2}$
- Movable partition $G_{k,mp} = 1.20\mathrm{kN/m^2}$

2.4.3 *Wind Load*

The wind load is evaluated by following the EN 1991-1-4 §4.3.3 [5].

- Wind on the long side (Y direction)

$$q_p(z_e) = q_p(z_i) = 0.823 \frac{kN}{m^2}$$

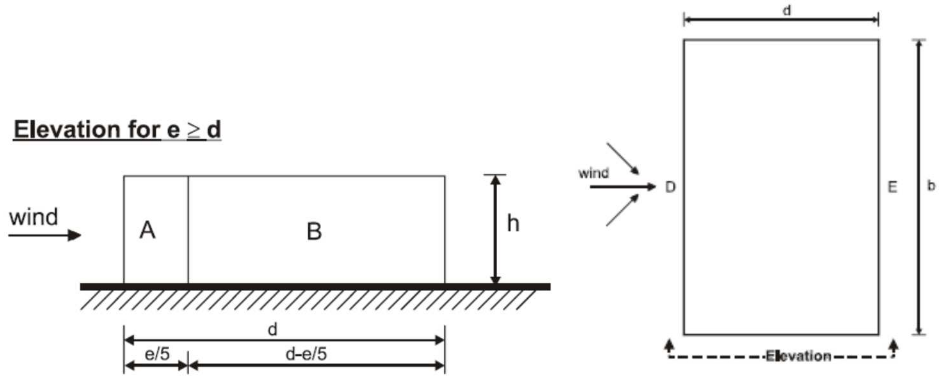


Figure 2-12. Key for vertical walls

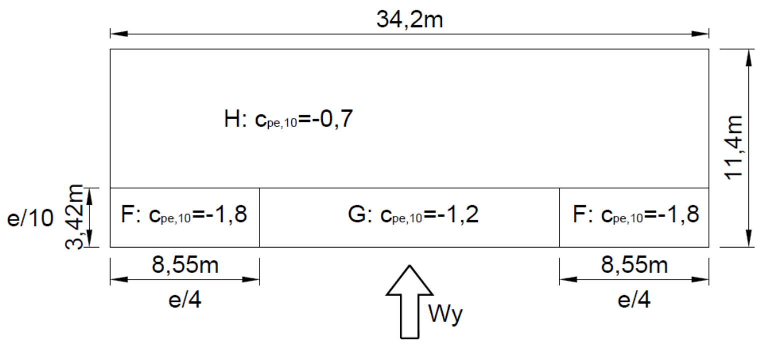


Figure 2-13. Key for flat roof with wind in Y direction

Table 2-1. Wind pressures for normal design situations on the long side (Y direction)

Wind pressure $[kN/m^2]$							
	Vertical walls				Roof		
zone	A	B	D	E	F	G	H
c_{pe}	-1,2	-0,8	0,8	-0,529	-1,8	-1,2	-0,7
$c_{pi} = +0,2$	-0,974	-0,704	0,375	-0,521	-1,379	-0,974	-0,637
$c_{pi} = -0,3$	-0,562	-0,293	0,787	-0,110	-0,967	-0,562	-0,225

Table 2-2. Wind pressures for accidental design situations on the long side (Y direction)

Wind pressure $[kN/m^2]$							
	Vertical walls				Roof		
zone	A	B	D	E	F	G	H
c_{pe}	-1,2	-0,8	0,8	-0,529	-1,8	-1,2	-0,7
$c_{pi} = +0,72$	-1,402	-1,132	-0,053	-0,949	-1,807	-1,402	-1,065
$c_{pi} = -1,08$	0,080	0,349	1,429	0,532	-0,325	0,080	0,417

- Wind on the short side (X direction)

$$q_p(z_e) = q_p(z_i) = 0.701 \frac{kN}{m^2} \quad for \quad 0\,m < z < 11.4\,m$$

$$q_p(z_e) = q_p(z_i) = 0.823 \frac{kN}{m^2} \quad for \quad 11.4\,m < z < 18\,m$$

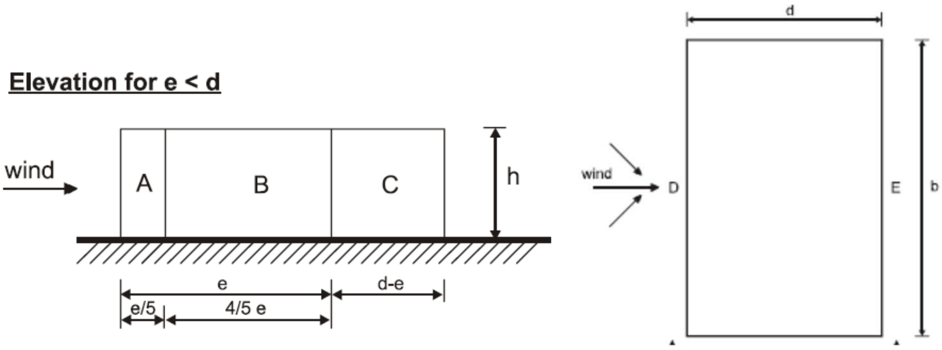


Figure 2-14. Key for vertical walls

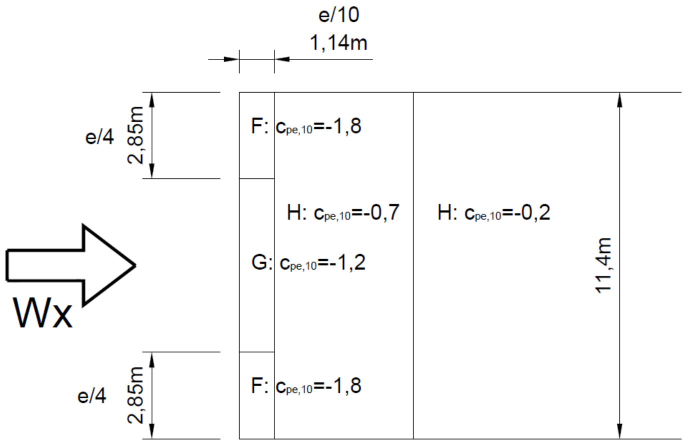


Figure 2-15. Key for flat roof with wind in X direction

Table 2-3. Wind pressures for normal design situations on the short side (X direction)

Wind pressure $[kN/m^2]$										
		Vertical walls					Roof			
zone		A	B	C	D	E	F	G	H	I
c_{pe}		-1,2	-0,8	-0,5	0,8	-0,529	-1,8	-1,2	-0,7	-0,2
$c_{pi} = +0,2$	$0 \le z \le 11,4\ m$	-0,872	-0,628	-0,445	0,309	-0,368	-1,238	-0,872	-0,567	-0,262
	$11,4 < z \le 18\ m$	-1,024	-0,738	-0,523	0,363	-0,432	-1,454	-1,024	-0,666	-0,308
$c_{pi} = -0,3$	$0 \le z \le 11,4\ m$	-0,522	-0,278	-0,095	0,660	-0,018	-0,888	-0,522	-0,217	0,088
	$11,4 < z \le 18\ m$	-0,612	-0,326	-0,111	0,775	-0,021	-1,042	-0,612	-0,254	0,104

Table 2-4. Wind pressures for accidental design situations on the short side (X direction)

Wind pressure $[kN/m^2]$										
		Vertical walls					Roof			
zone		A	B	C	D	E	F	G	H	I
c_{pe}		-1,2	-0,8	-0,5	0,8	-0,529	-1,8	-1,2	-0,7	-0,2
$c_{pi} = +0,72$	$0 \le z \le 11,4\ m$	-1,237	-0,993	-0,810	-0,055	-0,733	-1,603	-1,237	-0,932	-0,627
	$11,4 < z \le 18\ m$	-1,452	-1,166	-0,951	0,065	-0,860	-1,882	-1,452	-1,094	-0,736
$c_{pi} = -1,08$	$0 \le z \le 11,4\ m$	0,025	0,269	0,452	1,207	0,529	-0,341	0,025	0,330	0,635
	$11,4 < z \le 18\ m$	0,030	0,316	0,531	1,417	0,621	-0,400	0,030	0,388	0,746

2.4.4 *Snow Load*

The snow loads on the roof are obtained by following the instruction of EN 1991-1-3 §5 [6].

- Persistent/Transient design situations $s = 1.2kN/m^2$
- Accidental design situations $s = 2.4kN/m^2$

2.5 Load Combinations

The load combinations are defined in accordance with the EN 1990 [7]. In all the combinations, the *self-weight* is uniformly distributed overall the structure. Differently, the *variable* actions are distributed by following different load distributions in order to maximize the stresses in all the structural elements. Figure 2-16 shows all the load combination schemes considered for the typical floor framing plan for the variable loads for both the *Symmetric* and *Asymmetric* structures.

The symbols used in Figure 2-16 are:

- q_1 Variable loads distribution to maximize the forces on the slab;
- q_2 Variable loads distribution to maximize the forces on the slab;
- q_{1v} Variable loads distribution to maximize the forces on the beams in *X* direction;
- q_{2v} Variable loads distribution to maximize the forces on the beams in *X* direction;
- q_{1o} Variable loads distribution to maximize the forces on the beams in *Y* direction;
- q_{2o} Variable loads distribution to maximize the forces on the beams in *Y* direction;
- $q_1 + q_2$ Variable loads distribution to maximize the forces on the supports;

All the considered variable loads are listed in the follow:

- q Imposed load;
- q_s Snow load;
- q_{mp} Mobile partitions load;
- $w_{x\_cpi+0.2}$ Wind in *X* direction with coefficient c_{pi} = + 0.2;
- $w_{x\_cpi-0.3}$ Wind in *X* direction with coefficient c_{pi} = - 0.3;
- $w_{y\_cpi+0.2}$ Wind in *Y* direction with coefficient c_{pi} = + 0.2;
- $w_{y\_cpi-0.3}$ Wind in *Y* direction with coefficient c_{pi} = - 0.3.

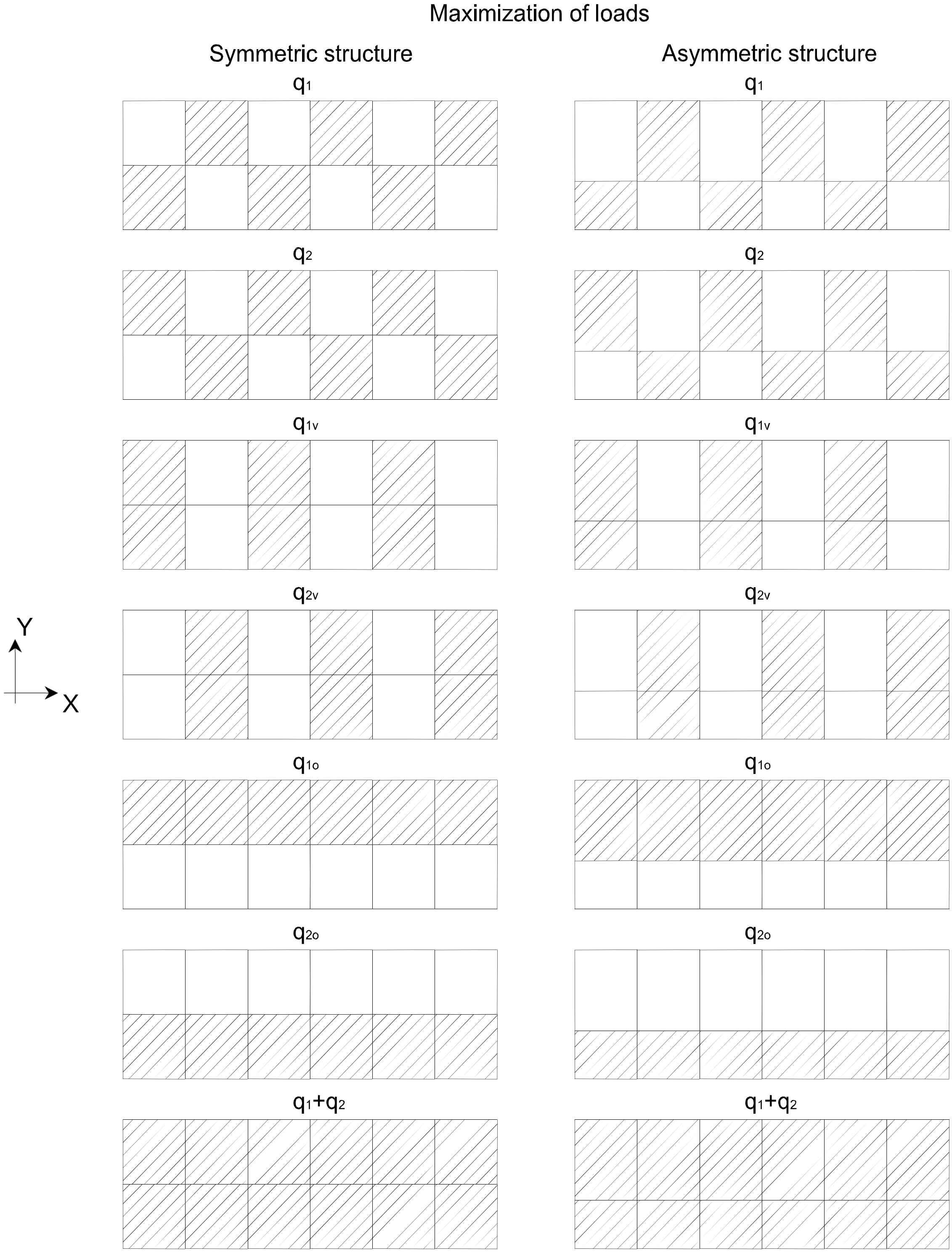


Figure 2-16. Load Combination Schemes on the Typical Floor Framing Plan

The load combination schemes on the *Symmetric* structure for the variable loads in the transversal (*Y* direction) and longitudinal (*X* direction) frames are reported in the follow (Figure 2-17 to Figure 2-20).

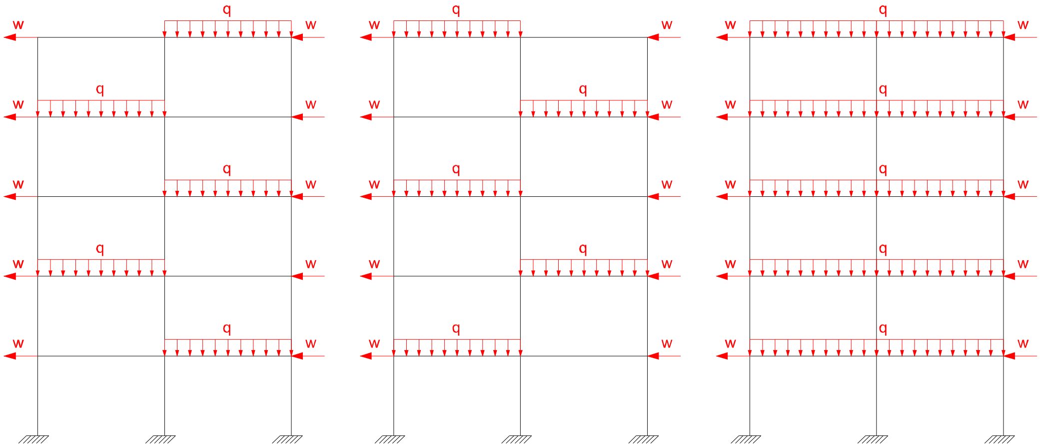


Figure 2-17. Load Combination Schemes on the Frame in Y direction

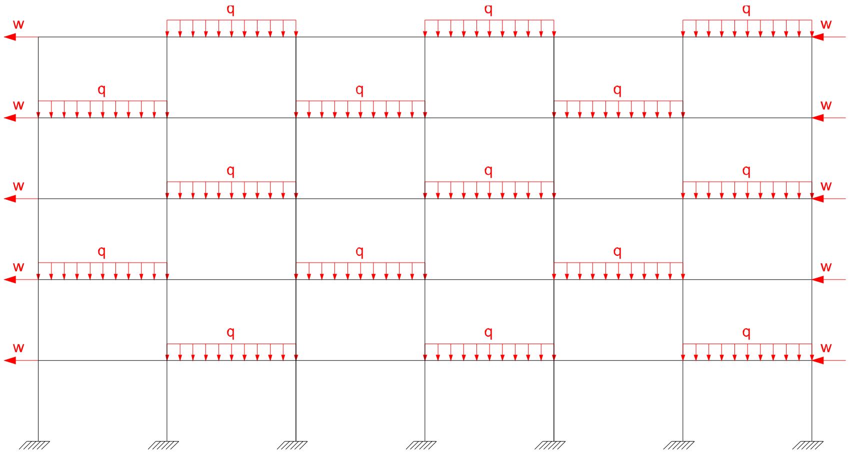


Figure 2-18. Load Combination Schemes on the Frame in X direction

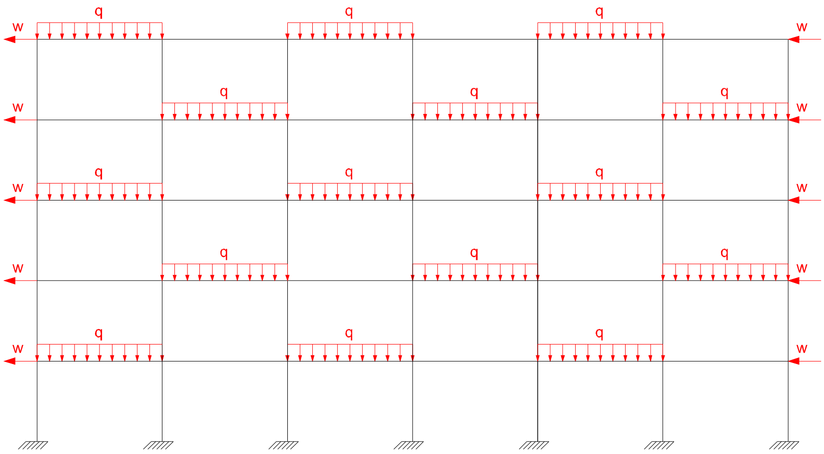


Figure 2-19. Load Combination Schemes on the Frame in X direction

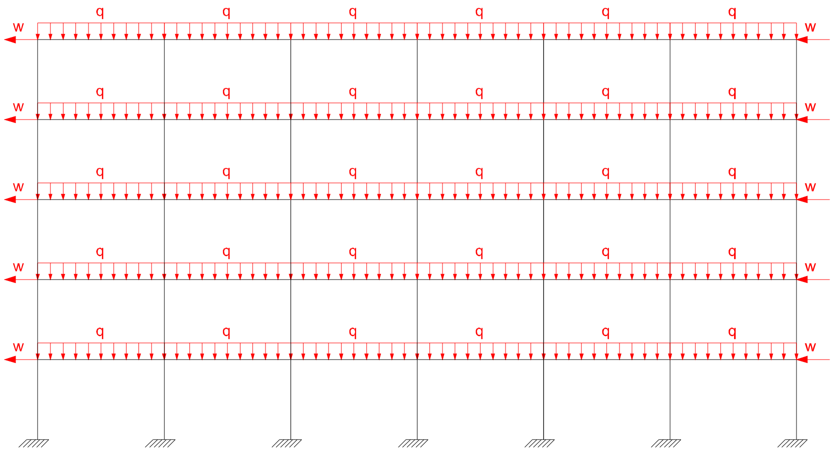


Figure 2-20. Load Combination Schemes on the Frame in X direction

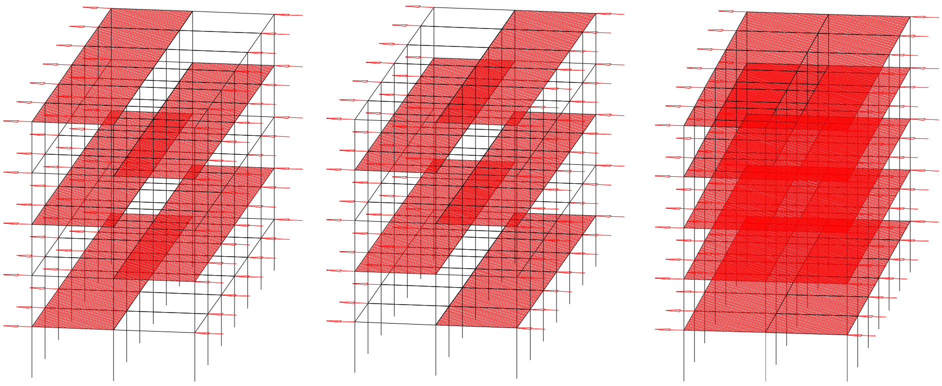


Figure 2-21. 3-D Load Combination Schemes on the Frame in Y direction

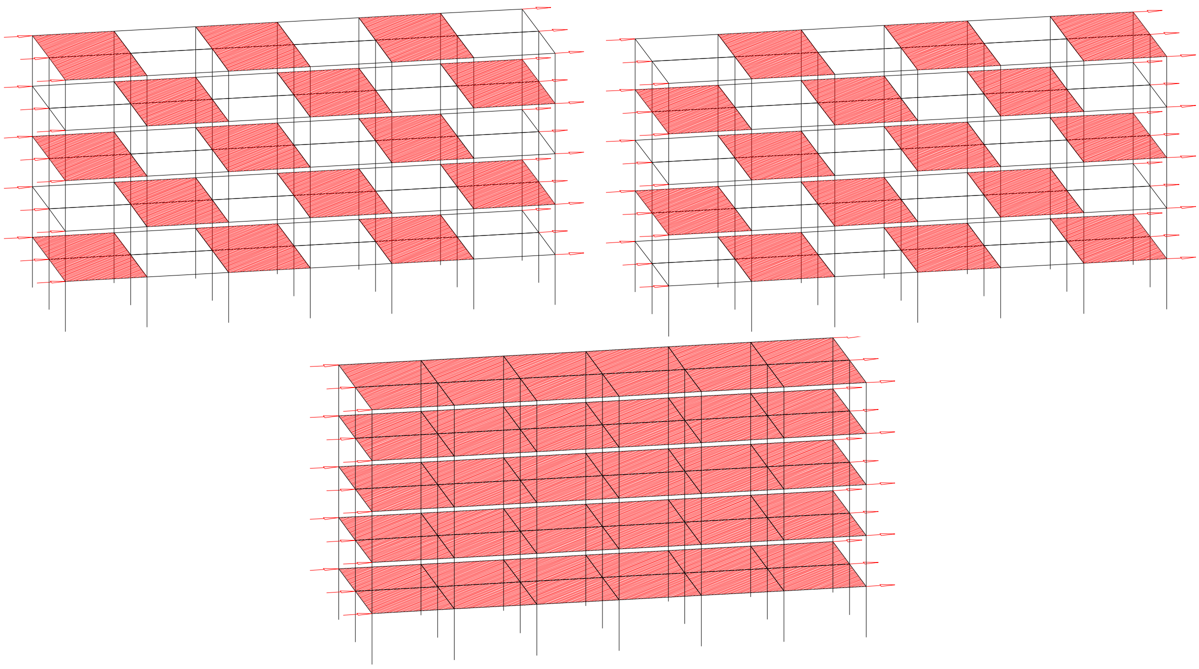


Figure 2-22. 3-D Load Combination Schemes on the Frame in X direction

The load combination schemes on the *Symmetric* frames for the variable loads in the transversal (*Y* direction) and longitudinal (*X* direction) frames are reported in the follow (Figure 2-23 to Figure 2-26).

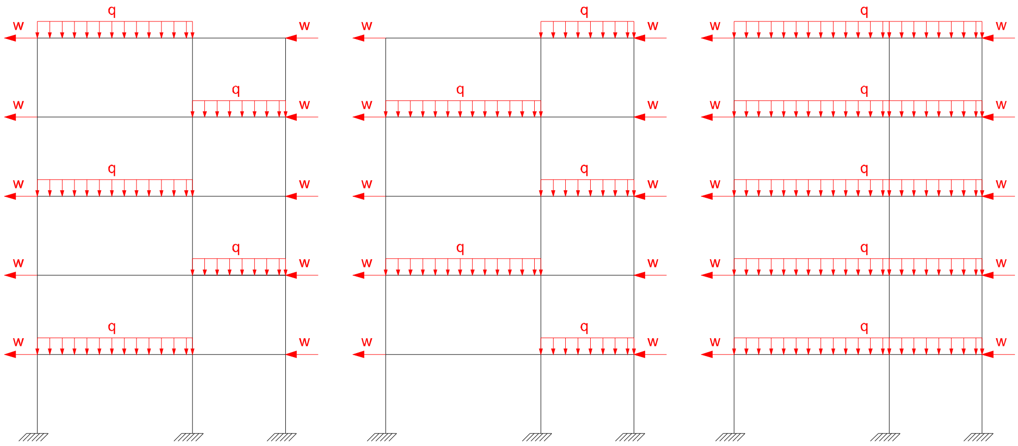


Figure 2-23. Load Combination Schemes on the Frame in Y direction

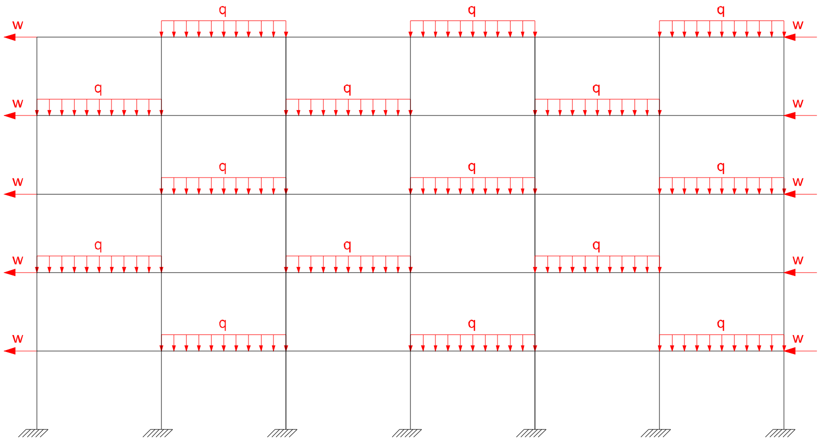


Figure 2-24. Load Combination Schemes on the Frame in X direction

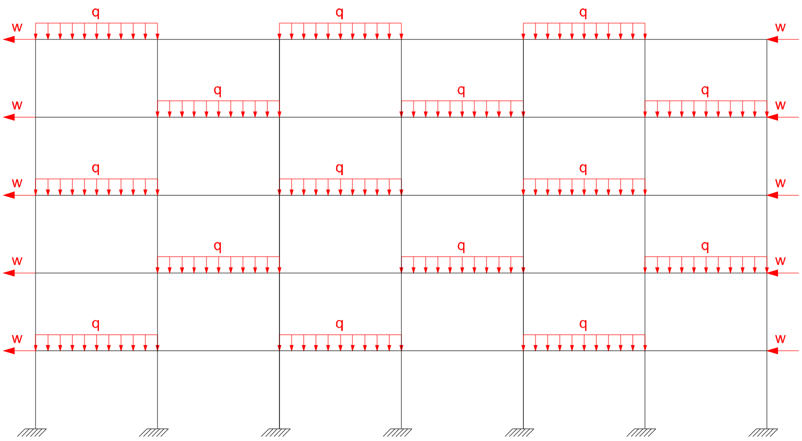


Figure 2-25. Load Combination Schemes on the Frame in X direction

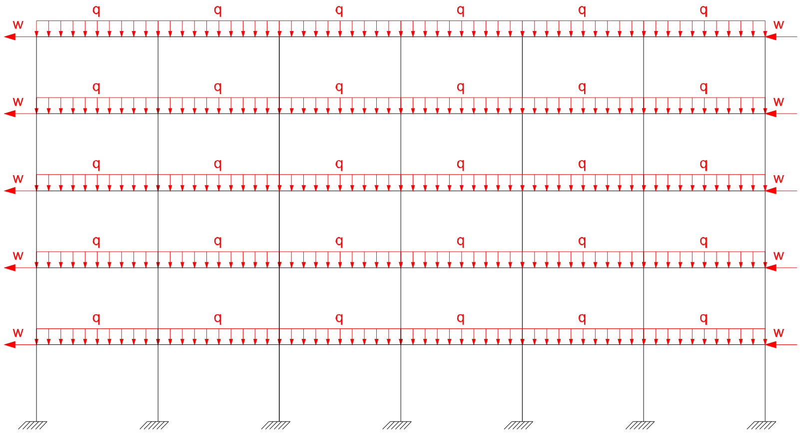


Figure 2-26. Load Combination Schemes on the Frame in X direction

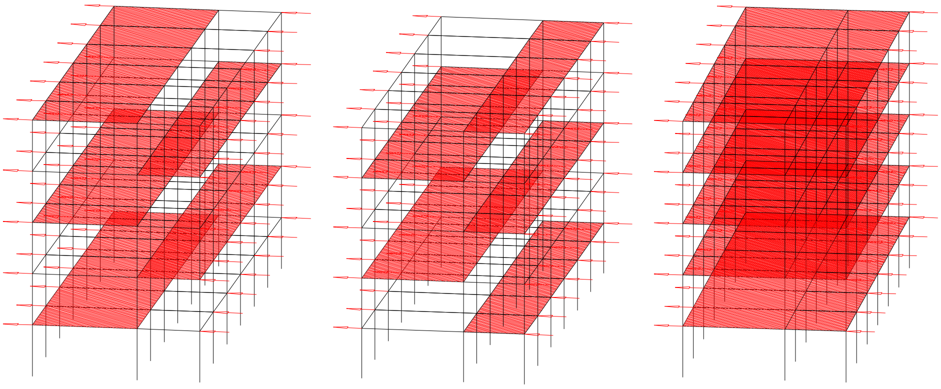


Figure 2-27. 3-D Load Combination Schemes on the Frame in Y direction

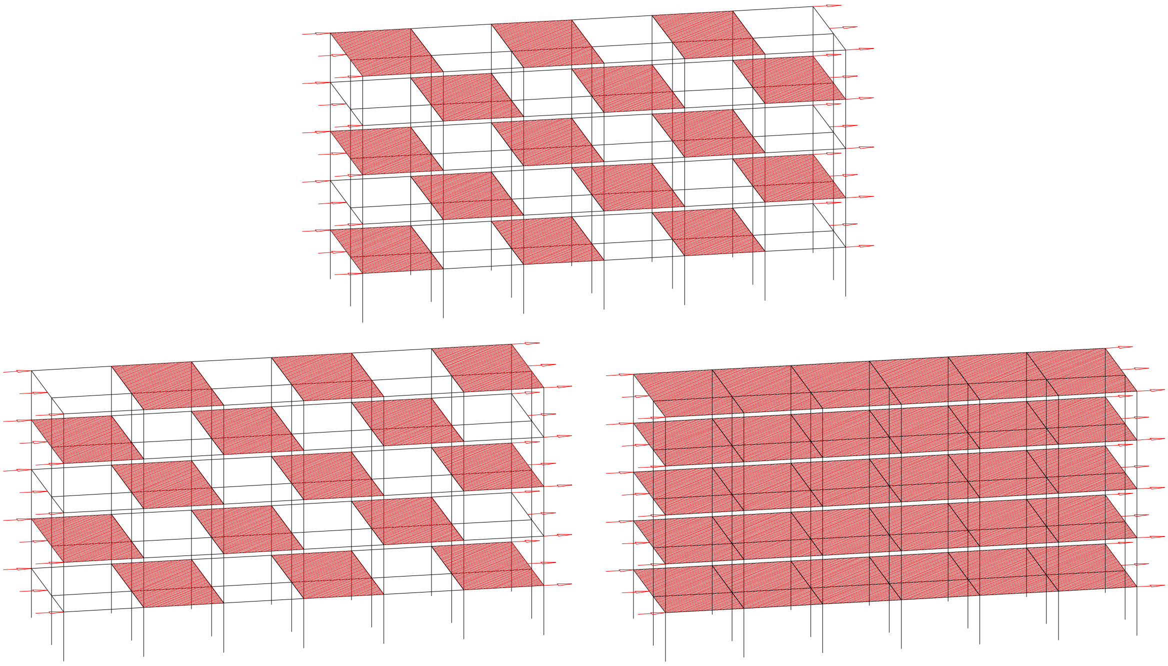


Figure 2-28. 3-D Load Combination Schemes on the Frame in X direction

2.5.1 *Ultimate Limit State - ULS*

The symbols defined in Figure 2-16 (q_1, q_2, q_{1o}, q_{2o}, q_{1v} and q_{2v}) are used in the following tables to report the Load Combinations considered in the study.

The load combinations used for the *Ultimate Limit State* (ULS) are:

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j}\text{"+"}\gamma_p P\text{"+"}\gamma_{Q,1}Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \quad (2.1)$$

Table 2-5. Load Combinations for Ultimate Limit State

$\gamma_{G,1}$	G_1	+ $\gamma_{G,2}$	G_2 + $\gamma_{Q,1}$	Q_1	+ $\gamma_{Q,2}$ $\psi_{0,2}$	Q_2	+ $\gamma_{Q,3}$ $\psi_{0,3}$	Q_3	+$\gamma_{Q,4}$ $\psi_{0,4}$	Q_4
1,35	DEAD	+ 1,35	g_2 + 1,5	q_1	+ 1,5 0,7	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{1s}	+1,5 0,7	q_{1mp}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_1	+ 1,5 0,7	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{1s}	+1,5 0,7	q_{1mp}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_2	+ 1,5 0,7	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{2s}	+1,5 0,7	q_{2mp}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_2	+ 1,5 0,7	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{2s}	+1,5 0,7	q_{2mp}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_1+q_2	+ 1,5 0,7	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{1s}+q_{2s}	+1,5 0,7	q_{1mp}+q_{2mp}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_1+q_2	+ 1,5 0,7	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{1s}+q_{2s}	+1,5 0,7	q_{1mp}+q_{2mp}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_1	+ 1,5 0,7	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{1s}	+1,5 0,7	q_{1mp}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_1	+ 1,5 0,7	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{1s}	+1,5 0,7	q_{1mp}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_2	+ 1,5 0,7	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{2s}	+1,5 0,7	q_{2mp}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_2	+ 1,5 0,7	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{2s}	+1,5 0,7	q_{2mp}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_1+q_2	+ 1,5 0,7	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{1s}+q_{2s}	+1,5 0,7	q_{1mp}+q_{2mp}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_1+q_2	+ 1,5 0,7	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{1s}+q_{2s}	+1,5 0,7	q_{1mp}+q_{2mp}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{1o}	+ 1,5 0,7	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{1so}	+1,5 0,7	q_{1mpo}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{1o}	+ 1,5 0,7	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{1so}	+1,5 0,7	q_{1mpo}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{2o}	+ 1,5 0,7	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{2so}	+1,5 0,7	q_{2mpo}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{2o}	+ 1,5 0,7	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{2so}	+1,5 0,7	q_{2mpo}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{1o}	+ 1,5 0,7	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{1so}	+1,5 0,7	q_{1mpo}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{1o}	+ 1,5 0,7	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{1so}	+1,5 0,7	q_{1mpo}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{2o}	+ 1,5 0,7	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{2so}	+1,5 0,7	q_{2mpo}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{2o}	+ 1,5 0,7	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{2so}	+1,5 0,7	q_{2mpo}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{1v}	+ 1,5 0,7	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{1sv}	+1,5 0,7	q_{1mpv}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{1v}	+ 1,5 0,7	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{1sv}	+1,5 0,7	q_{1mpv}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{2v}	+ 1,5 0,7	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{2sv}	+1,5 0,7	q_{2mpv}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{2v}	+ 1,5 0,7	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{2sv}	+1,5 0,7	q_{2mpv}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{1v}	+ 1,5 0,7	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{1sv}	+1,5 0,7	q_{1mpv}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{1v}	+ 1,5 0,7	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{1sv}	+1,5 0,7	q_{1mpv}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{2v}	+ 1,5 0,7	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{2sv}	+1,5 0,7	q_{2mpv}
1,35	DEAD	+ 1,35	g_2 + 1,5	q_{2v}	+ 1,5 0,7	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{2sv}	+1,5 0,7	q_{2mpv}
1,35	DEAD	+ 1,35	g_2 + 1,5	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_1	+ 1,5 0,7	q_{1s}	+1,5 0,7	q_{1mp}
1,35	DEAD	+ 1,35	g_2 + 1,5	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_1	+ 1,5 0,7	q_{1s}	+1,5 0,7	q_{1mp}
1,35	DEAD	+ 1,35	g_2 + 1,5	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_2	+ 1,5 0,7	q_{2s}	+1,5 0,7	q_{2mp}

1,35 DEAD + 1,35 g_2 + 1,5	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_2	+ 1,5 0,7	q_{2s}	+1,5 0,7	q_{2mp}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_1+q_2	+ 1,5 0,7	$q_{1s}+q_{2s}$	+1,5 0,7	$q_{1mp}+q_{2mp}$
1,35 DEAD + 1,35 g_2 + 1,5	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_1+q_2	+ 1,5 0,7	$q_{1s}+q_{2s}$	+1,5 0,7	$q_{1mp}+q_{2mp}$
1,35 DEAD + 1,35 g_2 + 1,5	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_1	+ 1,5 0,7	q_{1s}	+1,5 0,7	q_{1mp}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_1	+ 1,5 0,7	q_{1s}	+1,5 0,7	q_{1mp}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_2	+ 1,5 0,7	q_{2s}	+1,5 0,7	q_{2mp}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_2	+ 1,5 0,7	q_{2s}	+1,5 0,7	q_{2mp}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_1+q_2	+ 1,5 0,7	$q_{1s}+q_{2s}$	+1,5 0,7	$q_{1mp}+q_{2mp}$
1,35 DEAD + 1,35 g_2 + 1,5	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_1+q_2	+ 1,5 0,7	$q_{1s}+q_{2s}$	+1,5 0,7	$q_{1mp}+q_{2mp}$
1,35 DEAD + 1,35 g_2 + 1,5	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{1o}	+ 1,5 0,7	q_{1so}	+1,5 0,7	q_{1mpo}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{1o}	+ 1,5 0,7	q_{1so}	+1,5 0,7	q_{1mpo}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{2o}	+ 1,5 0,7	q_{2so}	+1,5 0,7	q_{2mpo}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{2o}	+ 1,5 0,7	q_{2so}	+1,5 0,7	q_{2mpo}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{1o}	+ 1,5 0,7	q_{1so}	+1,5 0,7	q_{1mpo}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{1o}	+ 1,5 0,7	q_{1so}	+1,5 0,7	q_{1mpo}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{2o}	+ 1,5 0,7	q_{2so}	+1,5 0,7	q_{2mpo}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{2o}	+ 1,5 0,7	q_{2so}	+1,5 0,7	q_{2mpo}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{1v}	+ 1,5 0,7	q_{1sv}	+1,5 0,7	q_{1mpv}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{1v}	+ 1,5 0,7	q_{1sv}	+1,5 0,7	q_{1mpv}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{2v}	+ 1,5 0,7	q_{2sv}	+1,5 0,7	q_{2mpv}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{2v}	+ 1,5 0,7	q_{2sv}	+1,5 0,7	q_{2mpv}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{1v}	+ 1,5 0,7	q_{1sv}	+1,5 0,7	q_{1mpv}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{1v}	+ 1,5 0,7	q_{1sv}	+1,5 0,7	q_{1mpv}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{2v}	+ 1,5 0,7	q_{2sv}	+1,5 0,7	q_{2mpv}
1,35 DEAD + 1,35 g_2 + 1,5	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{2v}	+ 1,5 0,7	q_{2sv}	+1,5 0,7	q_{2mpv}
1,35 DEAD + 1,35 g_2 + 1,5	q_{1s}	+ 1,5 0,7	q_1	+ 1,5 0,7	$w_{y\_cpi+0,2}$	+1,5 0,7	q_{1mp}
1,35 DEAD + 1,35 g_2 + 1,5	q_{1s}	+ 1,5 0,7	q_1	+ 1,5 0,7	$w_{y\_cpi-0,3}$	+1,5 0,7	q_{1mp}
1,35 DEAD + 1,35 g_2 + 1,5	q_{2s}	+ 1,5 0,7	q_2	+ 1,5 0,7	$w_{y\_cpi+0,2}$	+1,5 0,7	q_{2mp}
1,35 DEAD + 1,35 g_2 + 1,5	q_{2s}	+ 1,5 0,7	q_2	+ 1,5 0,7	$w_{y\_cpi-0,3}$	+1,5 0,7	q_{2mp}
1,35 DEAD + 1,35 g_2 + 1,5	$q_{1s}+q_{2s}$	+ 1,5 0,7	q_1+q_2	+ 1,5 0,7	$w_{y\_cpi+0,2}$	+1,5 0,7	$q_{1mp} + q_{2mp}$
1,35 DEAD + 1,35 g_2 + 1,5	$q_{1s}+q_{2s}$	+ 1,5 0,7	q_1+q_2	+ 1,5 0,7	$w_{y\_cpi-0,3}$	+1,5 0,7	$q_{1mp} + q_{2mp}$
1,35 DEAD + 1,35 g_2 + 1,5	q_{1s}	+ 1,5 0,7	q_1	+ 1,5 0,7	$w_{x\_cpi+0,2}$	+1,5 0,7	q_{1mp}
1,35 DEAD + 1,35 g_2 + 1,5	q_{1s}	+ 1,5 0,7	q_1	+ 1,5 0,7	$w_{x\_cpi-0,3}$	+1,5 0,7	q_{1mp}
1,35 DEAD + 1,35 g_2 + 1,5	q_{2s}	+ 1,5 0,7	q_2	+ 1,5 0,7	$w_{x\_cpi+0,2}$	+1,5 0,7	q_{2mp}
1,35 DEAD + 1,35 g_2 + 1,5	q_{2s}	+ 1,5 0,7	q_2	+ 1,5 0,7	$w_{x\_cpi-0,3}$	+1,5 0,7	q_{2mp}
1,35 DEAD + 1,35 g_2 + 1,5	$q_{1s}+q_{2s}$	+ 1,5 0,7	q_1+q_2	+ 1,5 0,7	$w_{x\_cpi+0,2}$	+1,5 0,7	$q_{1mp} + q_{2mp}$
1,35 DEAD + 1,35 g_2 + 1,5	$q_{1s}+q_{2s}$	+ 1,5 0,7	q_1+q_2	+ 1,5 0,7	$w_{x\_cpi-0,3}$	+1,5 0,7	$q_{1mp} + q_{2mp}$
1,35 DEAD + 1,35 g_2 + 1,5	q_{1so}	+ 1,5 0,7	q_{1o}	+ 1,5 0,7	$w_{y\_cpi+0,2}$	+1,5 0,7	q_{1mpo}
1,35 DEAD + 1,35 g_2 + 1,5	q_{1so}	+ 1,5 0,7	q_{1o}	+ 1,5 0,7	$w_{y\_cpi-0,3}$	+1,5 0,7	q_{1mpo}
1,35 DEAD + 1,35 g_2 + 1,5	q_{2so}	+ 1,5 0,7	q_{2o}	+ 1,5 0,7	$w_{y\_cpi+0,2}$	+1,5 0,7	q_{2mpo}
1,35 DEAD + 1,35 g_2 + 1,5	q_{2so}	+ 1,5 0,7	q_{2o}	+ 1,5 0,7	$w_{y\_cpi-0,3}$	+1,5 0,7	q_{2mpo}
1,35 DEAD + 1,35 g_2 + 1,5	q_{1so}	+ 1,5 0,7	q_{1o}	+ 1,5 0,7	$w_{x\_cpi+0,2}$	+1,5 0,7	q_{1mpo}
1,35 DEAD + 1,35 g_2 + 1,5	q_{1so}	+ 1,5 0,7	q_{1o}	+ 1,5 0,7	$w_{x\_cpi-0,3}$	+1,5 0,7	q_{1mpo}
1,35 DEAD + 1,35 g_2 + 1,5	q_{2so}	+ 1,5 0,7	q_{2o}	+ 1,5 0,7	$w_{x\_cpi+0,2}$	+1,5 0,7	q_{2mpo}

1,35	DEAD +	1,35	g_2 + 1,5	q_{2so}	+ 1,5 0,7	q_{2o}	+ 1,5 0,7	$w_{x\_cpi-0,3}$	+1,5 0,7	q_{2mpo}
1,35	DEAD +	1,35	g_2 + 1,5	q_{1sv}	+ 1,5 0,7	q_{1v}	+ 1,5 0,7	$w_{y\_cpi+0,2}$	+1,5 0,7	q_{1mpv}
1,35	DEAD +	1,35	g_2 + 1,5	q_{1sv}	+ 1,5 0,7	q_{1v}	+ 1,5 0,7	$w_{y\_cpi-0,3}$	+1,5 0,7	q_{1mpv}
1,35	DEAD +	1,35	g_2 + 1,5	q_{2sv}	+ 1,5 0,7	q_{2v}	+ 1,5 0,7	$w_{y\_cpi+0,2}$	+1,5 0,7	q_{2mpv}
1,35	DEAD +	1,35	g_2 + 1,5	q_{2sv}	+ 1,5 0,7	q_{2v}	+ 1,5 0,7	$w_{y\_cpi-0,3}$	+1,5 0,7	q_{2mpv}
1,35	DEAD +	1,35	g_2 + 1,5	q_{1sv}	+ 1,5 0,7	q_{1v}	+ 1,5 0,7	$w_{x\_cpi+0,2}$	+1,5 0,7	q_{1mpv}
1,35	DEAD +	1,35	g_2 + 1,5	q_{1sv}	+ 1,5 0,7	q_{1v}	+ 1,5 0,7	$w_{x\_cpi-0,3}$	+1,5 0,7	q_{1mpv}
1,35	DEAD +	1,35	g_2 + 1,5	q_{2sv}	+ 1,5 0,7	q_{2v}	+ 1,5 0,7	$w_{x\_cpi+0,2}$	+1,5 0,7	q_{2mpv}
1,35	DEAD +	1,35	g_2 + 1,5	q_{2sv}	+ 1,5 0,7	q_{2v}	+ 1,5 0,7	$w_{x\_cpi-0,3}$	+1,5 0,7	q_{2mpv}
1	DEAD +	1	g_2 + 1,5	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_1	+ 1,5 0,7	q_{1s}	+1,5 0,7	q_{1mp}
1	DEAD +	1	g_2 + 1,5	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_1	+ 1,5 0,7	q_{1s}	+1,5 0,7	q_{1mp}
1	DEAD +	1	g_2 + 1,5	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_2	+ 1,5 0,7	q_{2s}	+1,5 0,7	q_{2mp}
1	DEAD +	1	g_2 + 1,5	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_2	+ 1,5 0,7	q_{2s}	+1,5 0,7	q_{2mp}
1	DEAD +	1	g_2 + 1,5	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_1+q_2	+ 1,5 0,7	q_{1s}+q_{2s}	+1,5 0,7	q_{1mp}+q_{2mp}
1	DEAD +	1	g_2 + 1,5	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_1+q_2	+ 1,5 0,7	q_{1s}+q_{2s}	+1,5 0,7	q_{1mp}+q_{2mp}
1	DEAD +	1	g_2 + 1,5	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_1	+ 1,5 0,7	q_{1s}	+1,5 0,7	q_{1mp}
1	DEAD +	1	g_2 + 1,5	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_1	+ 1,5 0,7	q_{1s}	+1,5 0,7	q_{1mp}
1	DEAD +	1	g_2 + 1,5	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_2	+ 1,5 0,7	q_{2s}	+1,5 0,7	q_{2mp}
1	DEAD +	1	g_2 + 1,5	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_2	+ 1,5 0,7	q_{2s}	+1,5 0,7	q_{2mp}
1	DEAD +	1	g_2 + 1,5	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_1+q_2	+ 1,5 0,7	q_{1s}+q_{2s}	+1,5 0,7	q_{1mp}+q_{2mp}
1	DEAD +	1	g_2 + 1,5	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_1+q_2	+ 1,5 0,7	q_{1s}+q_{2s}	+1,5 0,7	q_{1mp}+q_{2mp}
1	DEAD +	1	g_2 + 1,5	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{1o}	+ 1,5 0,7	q_{1so}	+1,5 0,7	q_{1mpo}
1	DEAD +	1	g_2 + 1,5	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{1o}	+ 1,5 0,7	q_{1so}	+1,5 0,7	q_{1mpo}
1	DEAD +	1	g_2 + 1,5	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{2o}	+ 1,5 0,7	q_{2so}	+1,5 0,7	q_{2mpo}
1	DEAD +	1	g_2 + 1,5	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{2o}	+ 1,5 0,7	q_{2so}	+1,5 0,7	q_{2mpo}
1	DEAD +	1	g_2 + 1,5	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{1o}	+ 1,5 0,7	q_{1so}	+1,5 0,7	q_{1mpo}
1	DEAD +	1	g_2 + 1,5	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{1o}	+ 1,5 0,7	q_{1so}	+1,5 0,7	q_{1mpo}
1	DEAD +	1	g_2 + 1,5	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{2o}	+ 1,5 0,7	q_{2so}	+1,5 0,7	q_{2mpo}
1	DEAD +	1	g_2 + 1,5	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{2o}	+ 1,5 0,7	q_{2so}	+1,5 0,7	q_{2mpo}
1	DEAD +	1	g_2 + 1,5	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{1v}	+ 1,5 0,7	q_{1sv}	+1,5 0,7	q_{1mpv}
1	DEAD +	1	g_2 + 1,5	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{1v}	+ 1,5 0,7	q_{1sv}	+1,5 0,7	q_{1mpv}
1	DEAD +	1	g_2 + 1,5	$w_{y\_cpi+0,2}$	+ 1,5 0,7	q_{2v}	+ 1,5 0,7	q_{2sv}	+1,5 0,7	q_{2mpv}
1	DEAD +	1	g_2 + 1,5	$w_{y\_cpi-0,3}$	+ 1,5 0,7	q_{2v}	+ 1,5 0,7	q_{2sv}	+1,5 0,7	q_{2mpv}
1	DEAD +	1	g_2 + 1,5	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{1v}	+ 1,5 0,7	q_{1sv}	+1,5 0,7	q_{1mpv}
1	DEAD +	1	g_2 + 1,5	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{1v}	+ 1,5 0,7	q_{1sv}	+1,5 0,7	q_{1mpv}
1	DEAD +	1	g_2 + 1,5	$w_{x\_cpi+0,2}$	+ 1,5 0,7	q_{2v}	+ 1,5 0,7	q_{2sv}	+1,5 0,7	q_{2mpv}
1	DEAD +	1	g_2 + 1,5	$w_{x\_cpi-0,3}$	+ 1,5 0,7	q_{2v}	+ 1,5 0,7	q_{2sv}	+1,5 0,7	q_{2mpv}

2.5.2 *Serviceability Limit State - ULS*

The symbols defined in Figure 2-16 (q_1, q_2, q_{1o}, q_{2o}, q_{1v} and q_{2v}) are used in the following tables to report the load combinations considered in the study. The load combinations used for the *Serviceability Limit State* (SLS) are reported in the follow.

SLS: *Characteristic load combination*

$$\sum_{j\geq1} G_{k,j} \text{"+"} P \text{"+"} Q_{k,1} \text{"+"} \sum_{i>1} \psi_{0,i} Q_{k,i} \qquad (2.2)$$

Table 2-6. Load Combinations for Serviceability Limit State - Characteristic

G_1	+	G_2	+	Q_1	+	$\psi_{0,2}$	Q_2	+	$\psi_{0,3}$	Q_3	+	$\psi_{0,4}$	Q_4
DEAD	+	g_2	+	q_1+q_2	+	0,7	$W_{y\_cpi+0,2}$	+	0,7	$q_{1s}+q_{2s}$	+	0,7	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	q_1+q_2	+	0,7	$W_{y\_cpi-0,3}$	+	0,7	$q_{1s}+q_{2s}$	+	0,7	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	q_1+q_2	+	0,7	$W_{x\_cpi+0,2}$	+	0,7	$q_{1s}+q_{2s}$	+	0,7	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	q_1+q_2	+	0,7	$W_{x\_cpi-0,3}$	+	0,7	$q_{1s}+q_{2s}$	+	0,7	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	$W_{y\_cpi+0,2}$	+	0,7	q_1+q_2	+	0,7	$q_{1s}+q_{2s}$	+	0,7	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	$W_{y\_cpi-0,3}$	+	0,7	q_1+q_2	+	0,7	$q_{1s}+q_{2s}$	+	0,7	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	$W_{x\_cpi+0,2}$	+	0,7	q_1+q_2	+	0,7	$q_{1s}+q_{2s}$	+	0,7	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	$W_{x\_cpi-0,3}$	+	0,7	q_1+q_2	+	0,7	$q_{1s}+q_{2s}$	+	0,7	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	$q_{1s}+q_{2s}$	+	0,7	q_1+q_2	+	0,7	$W_{y\_cpi+0,2}$	+	0,7	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	$q_{1s}+q_{2s}$	+	0,7	q_1+q_2	+	0,7	$W_{y\_cpi-0,3}$	+	0,7	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	$q_{1s}+q_{2s}$	+	0,7	q_1+q_2	+	0,7	$W_{x\_cpi+0,2}$	+	0,7	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	$q_{1s}+q_{2s}$	+	0,7	q_1+q_2	+	0,7	$W_{x\_cpi-0,3}$	+	0,7	$q_{1mp}+q_{2mp}$

SLS: *Frequent Load Combination*

$$\sum_{j\geq1} G_{k,j} \text{"+"} P \text{"+"} \psi_{1,1} Q_{k,1} \text{"+"} \sum_{i>1} \psi_{2,i} Q_{k,i} \qquad (2.3)$$

Table 2-7. Load Combinations for Serviceability Limit State - Frequent

G_1	+	G_2	+	$\psi_{1,1}$	Q_1	+	$\psi_{2,2}$	Q_2	+	$\psi_{2,3}$	Q_3	+	$\psi_{2,4}$	Q_4
DEAD	+	g_2	+	0,5	q_1+q_2	+	0,3	$W_{y\_cpi+0,2}$	+	0,3	$q_{1s}+q_{2s}$	+	0,3	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	0,5	q_1+q_2	+	0,3	$W_{y\_cpi-0,3}$	+	0,3	$q_{1s}+q_{2s}$	+	0,3	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	0,5	q_1+q_2	+	0,3	$W_{x\_cpi+0,2}$	+	0,3	$q_{1s}+q_{2s}$	+	0,3	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	0,5	q_1+q_2	+	0,3	$W_{x\_cpi-0,3}$	+	0,3	$q_{1s}+q_{2s}$	+	0,3	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	0,5	$W_{y\_cpi+0,2}$	+	0,3	q_1+q_2	+	0,3	$q_{1s}+q_{2s}$	+	0,3	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	0,5	$W_{y\_cpi-0,3}$	+	0,3	q_1+q_2	+	0,3	$q_{1s}+q_{2s}$	+	0,3	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	0,5	$W_{x\_cpi+0,2}$	+	0,3	q_1+q_2	+	0,3	$q_{1s}+q_{2s}$	+	0,3	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	0,5	$W_{x\_cpi-0,3}$	+	0,3	q_1+q_2	+	0,3	$q_{1s}+q_{2s}$	+	0,3	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	0,5	$q_{1s}+q_{2s}$	+	0,3	q_1+q_2	+	0,3	$W_{y\_cpi+0,2}$	+	0,3	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	0,5	$q_{1s}+q_{2s}$	+	0,3	q_1+q_2	+	0,3	$W_{y\_cpi-0,3}$	+	0,3	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	0,5	$q_{1s}+q_{2s}$	+	0,3	q_1+q_2	+	0,3	$W_{x\_cpi+0,2}$	+	0,3	$q_{1mp}+q_{2mp}$
DEAD	+	g_2	+	0,5	$q_{1s}+q_{2s}$	+	0,3	q_1+q_2	+	0,3	$W_{x\_cpi-0,3}$	+	0,3	$q_{1mp}+q_{2mp}$

SLS: *Quasi Permanent Load Combination*

$$\sum_{j\geq 1} G_{k,j} \text{"+"} P \text{"+"} \sum_{i>1} \psi_{2,i} Q_{k,i} \tag{2.4}$$

Table 2-8. Load Combinations for Serviceability Limit State – Quasi-Permanent

G_1	+	G_2	+	$\psi_{1,1}$	Q_1	+	$\psi_{2,2}$	Q_2	+	$\psi_{2,3}$	Q_3
DEAD	+	g_2	+	0,3	q_1+q_2	+	0,3	$w_{y\_cpi+0,2}$	+	0,3	$q_{1s}+q_{2s}$
DEAD	+	g_2	+	0,3	q_1+q_2	+	0,3	$w_{y\_cpi-0,3}$	+	0,3	$q_{1s}+q_{2s}$
DEAD	+	g_2	+	0,3	q_1+q_2	+	0,3	$w_{x\_cpi+0,2}$	+	0,3	$q_{1s}+q_{2s}$
DEAD	+	g_2	+	0,3	q_1+q_2	+	0,3	$w_{x\_cpi-0,3}$	+	0,3	$q_{1s}+q_{2s}$

2.6 Imperfection for global analysis of frames

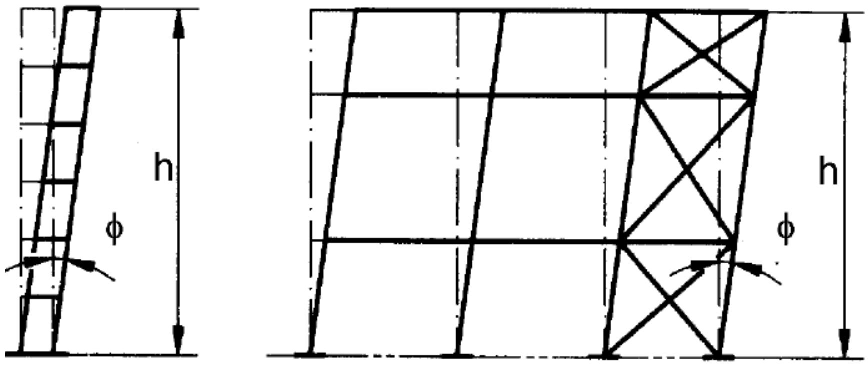


Figure 2-29. Equivalent sway imperfections

The effect of the global initial sway imperfections has been accounted by including these in the geometry of the finite element model. The effect of the bow imperfections has been considered in the study as suggested in the EN 1993-1-1 §6.3 [4]. The global initial sway imperfection may be evaluated by the following formula (EN 1993-1-1 §5.3.2 [4]) :

$$\phi = \phi_0 \cdot \alpha_h \cdot \alpha_m \tag{2.5}$$

The global initial sway imperfections for both the directions are reported in the follow:

- Global initial sway imperfection - X direction

$$\phi_0 = 1/200 \qquad \alpha_h = 0.667 \qquad \alpha_m = 0.756 \qquad \phi_x = 0.00252$$

- Global initial sway imperfection - Y direction

$$\phi_0 = 1/200 \qquad \alpha_h = 0.667 \qquad \alpha_m = 0.816 \qquad \phi_y = 0.00272$$

2.7 Creep and Shrinkage of the concrete

The calculation of the creep coefficient based on EN 1992-1-1 Annex B and EN 1992-1-1 §5.4.2.2 [3]. The relative humidity of the ambient is RH = 75 % and the age of the concrete considered is t = ∞. The creep coefficient is calculated as follow.

$$\varphi(t, t_0) = \varphi_0 \cdot \beta_c(t, t_0) \tag{2.6}$$

For the creep, the age of loading t_0 is assumed to be 28 days while for shrinkage, the age of loading t_0 is assumed to be 1 day.

$$\varphi(t, 28) = \varphi_0 \cdot \beta_c(t, 28) = 1.893 \qquad \varphi(t, 1) = \varphi_0 \cdot \beta_c(t, 1) = 3.523$$

The modular ratios are calculated for short and for long term loading by following the EN 1994-1-1 §5.4.2.2 [8]. For short-term loading:

$$n_0 = E_s / E_{cm} = 6.395$$

For long-term loading the creep multiplier (ψ_L) depends on the type of loading. It can be taken as 1.1 for permanent load and 0.55 for primary and secondary effect of shrinkage. The modular ratios and effective modulus of elasticity of the concrete for long-term loading are calculated as follow:

- Permanent Load

$$n_L = n_0\left(1+\psi_L\varphi(t,t_0)\right) = 19.706 \qquad E_{c,eff} = E_s/n_L = 10656.5\ MPa$$

- Primary and secondary effects of shrinkage

$$n_L = n_0\left(1+\psi_L\varphi(t,t_0)\right) = 18.782 \qquad E_{c,eff} = E_s/n_L = 11180.7\ MPa$$

2.8 Symmetric Structure

The following sections report the calculation of the components of the Symmetric structure.

2.8.1 *Slab - Maximum Bending Moments - ULS*

The bending moments for the design of the slab in both the directions have been obtained by using the finite element model previously described. Figure 2-30 up to Figure 2-33 show the maximum and minimum bending moments in *X* and *Y* direction for the Envelope of all the ULS combinations.

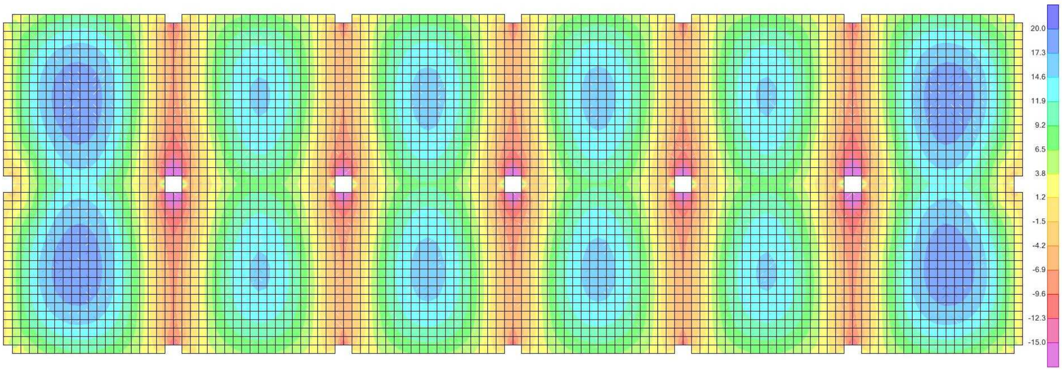


Figure 2-30. Maximum Bending Moments in X direction (ULS Envelope)

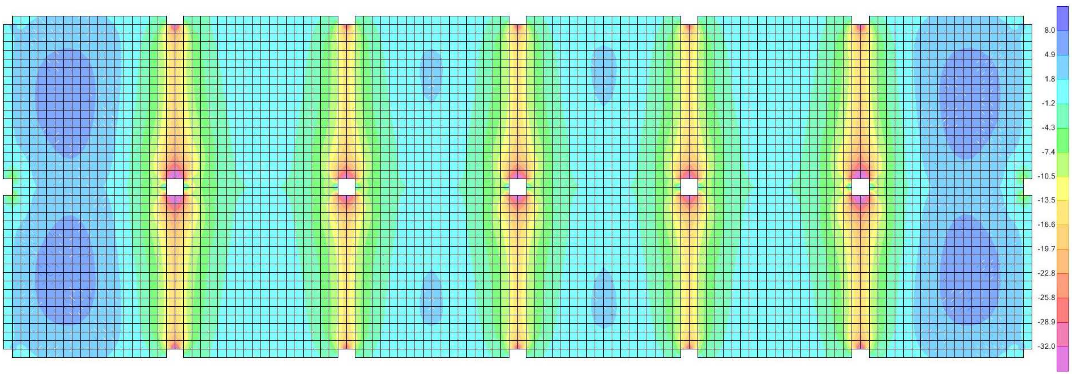


Figure 2-31. Minimum Bending Moments in X direction (ULS Envelope)

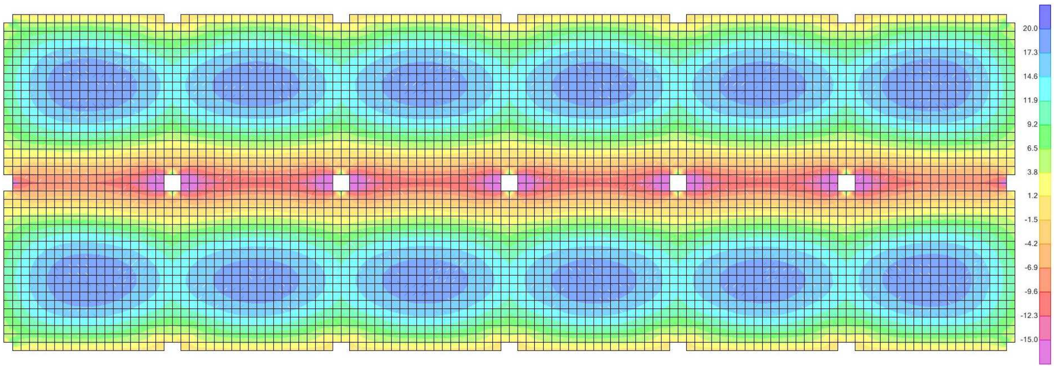


Figure 2-32. Maximum Bending Moments in Y direction (ULS Envelope)

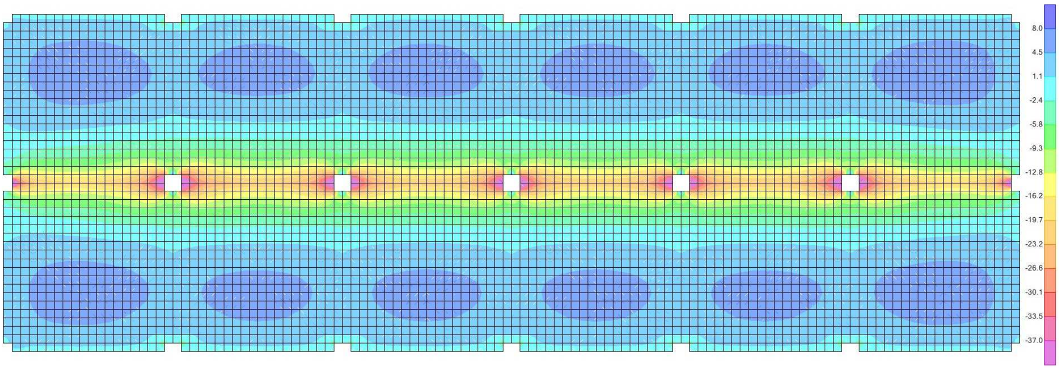


Figure 2-33. Minimum Bending Moments in Y direction (ULS Envelope)

2.8.2 *Slab - Maximum traction force in the layers of rebars - ULS*

The amount of rebars is based on the maximum traction force in each layer of rebars for the Envelope of all the ULS combinations as reported in Figure 2-34 up to Figure 2-37. The minimum and maximum steel percentages and the maximum spacing of rebars are defined in the EN 1992-1-1 §9.2.1 and EN 1992-1-1 §9.3 [3]. They are respectively:

$A_{s,min} = 150.8\ {mm^2}/{m}$ minimum steel percentage

$A_{s,max} = 6000\ {mm^2}/{m}$ maximum steel percentage

$s_{max,slab} = 400\ mm$ maximum spacing of rebars

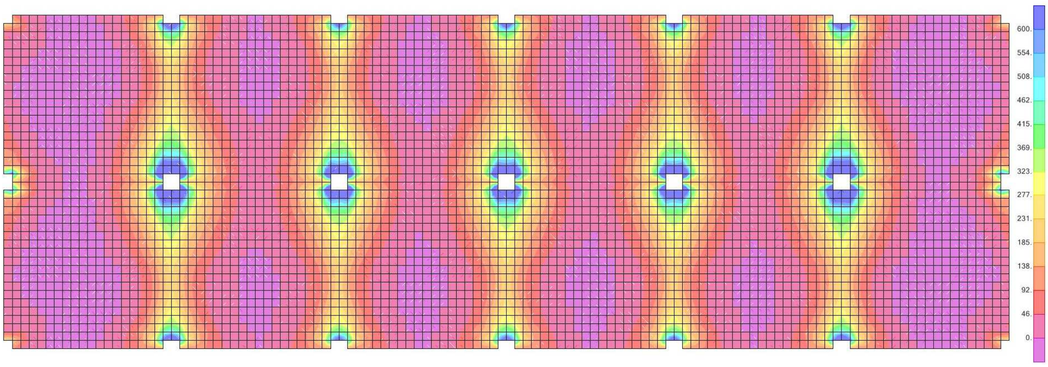


Figure 2-34. Maximum Traction Force in the upper rebars in X direction (ULS Envelope)

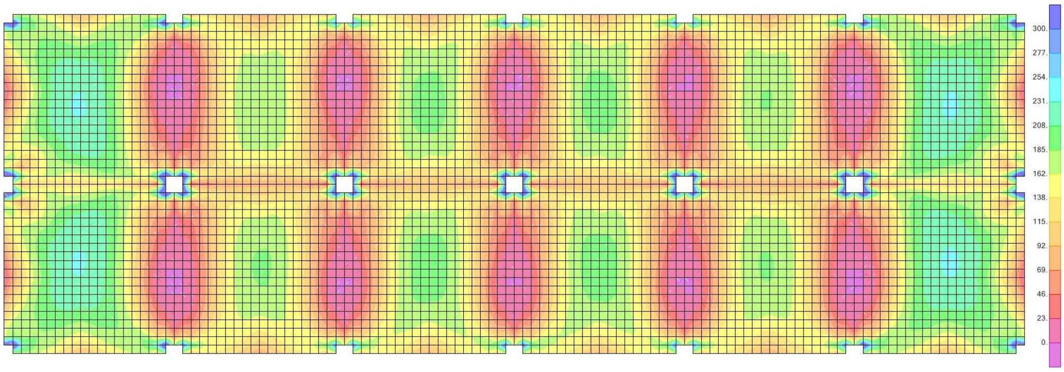


Figure 2-35. Maximum Traction Force in the lower rebars in X direction (ULS Envelope)

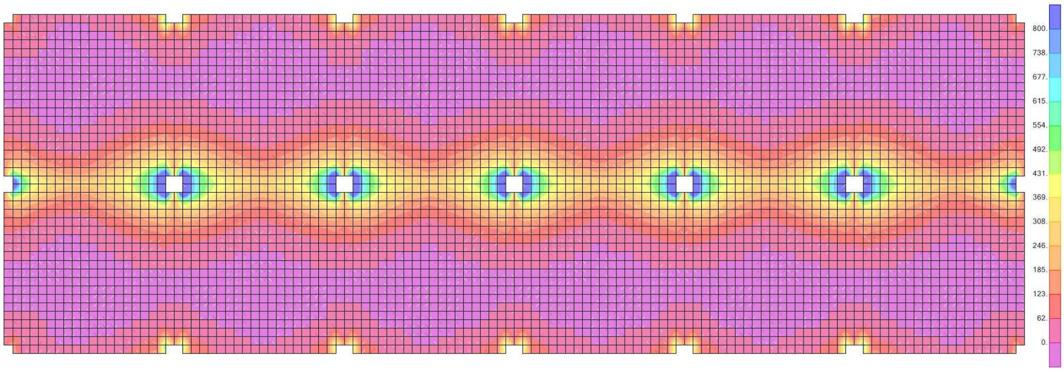


Figure 2-36. Maximum Traction Force in the upper rebars in Y direction (ULS Envelope)

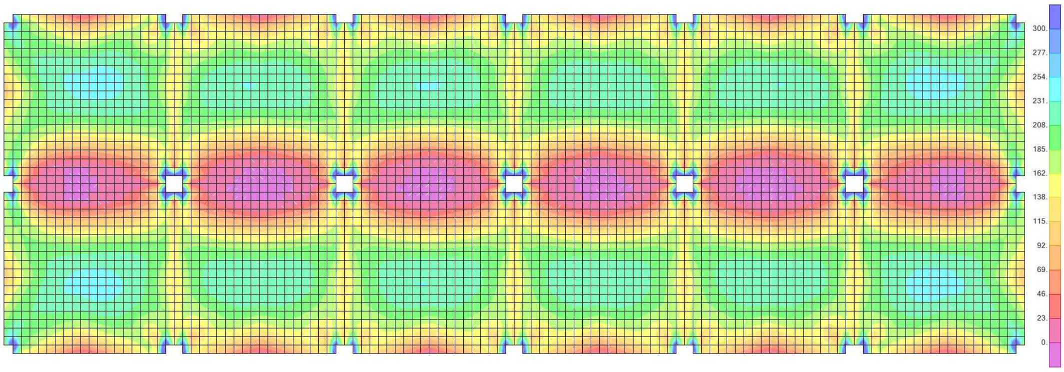


Figure 2-37. Maximum Traction Force in the lower rebars in Y direction (ULS Envelope)

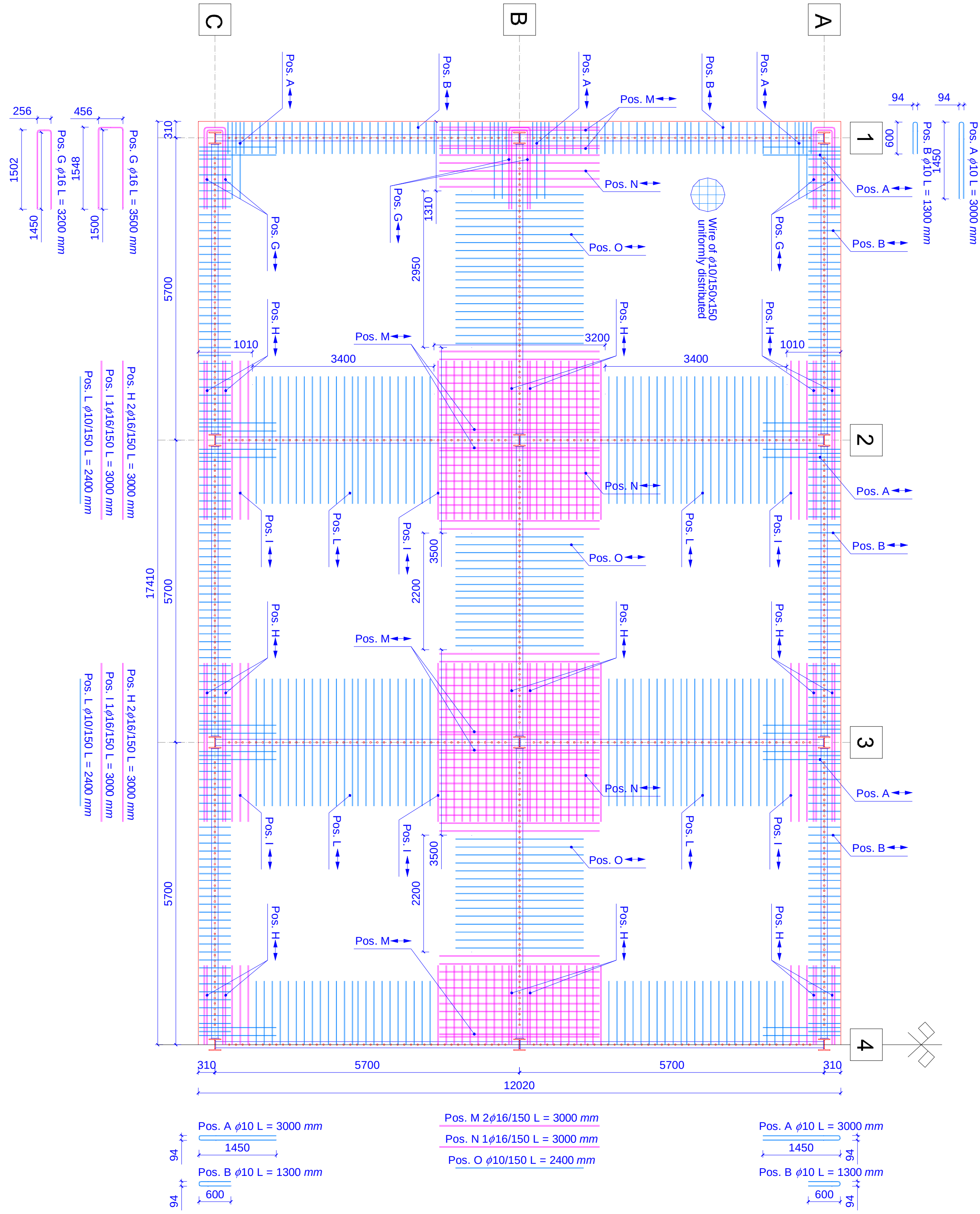


Figure 2-38. Slab Rebars - Upper Side – Symmetric Configuration (length unit mm)

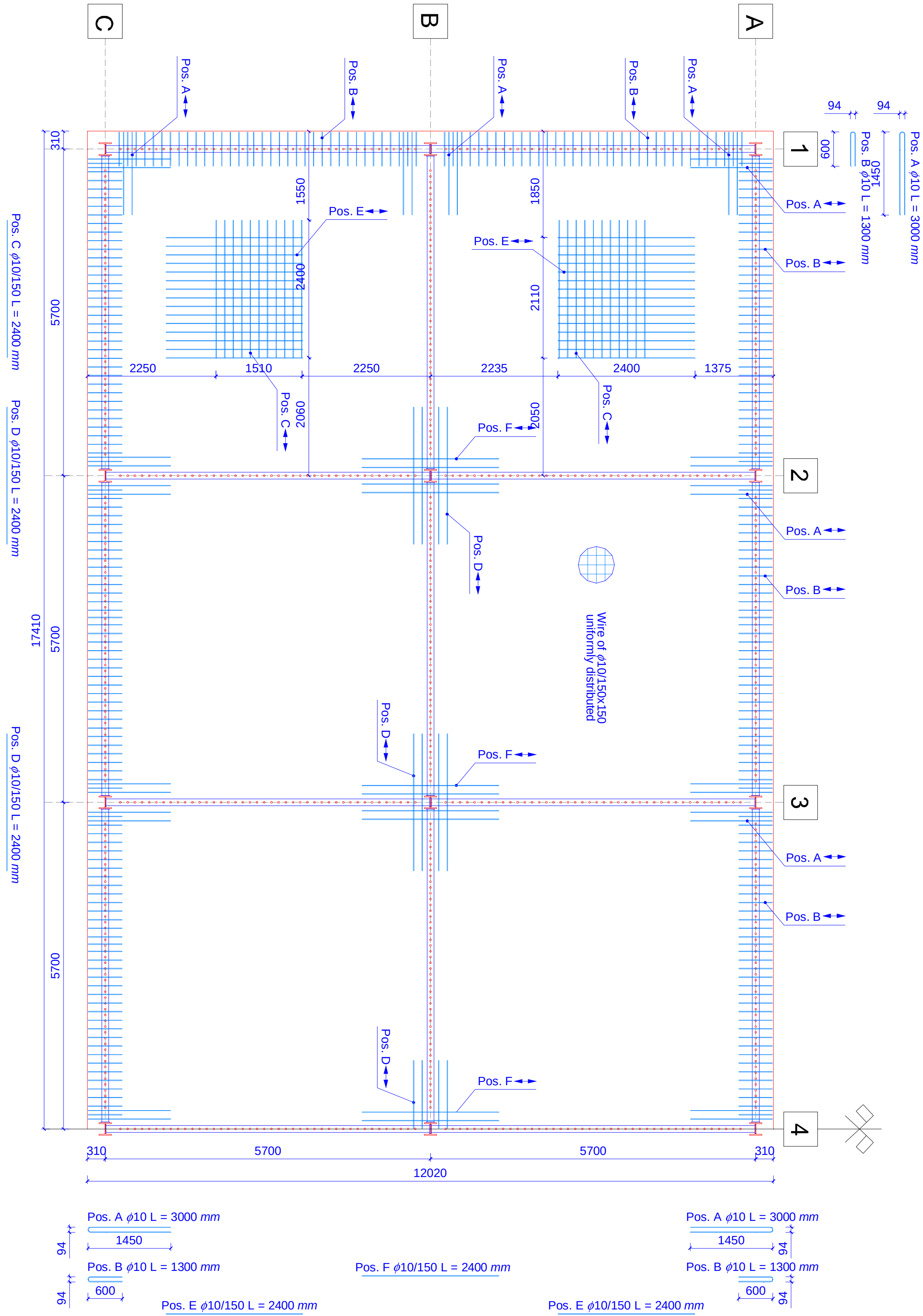


Figure 2-39. Slab Rebars - Lower Side – Symmetric Configuration (length unit mm)

2.8.3 *Slab Shear - ULS*

The slab is verified against shear forces by following the EN 1992-1-1 §6.2 [3]. The design forces at the ULS are:

$V_{Ed,\max} = 22\ kN/m$ $\qquad$ $N_{Ed,\max} = -40\ kN/m$ (traction)

The design value for the shear resistance $V_{Rd,c}$ in members not requiring design shear reinforcement is calculated as specified in the EN 1992-1-1 §6.2.2 [3]:

$$V_{Rd,c} = \max\left\{\left[C_{Rd,c}k\left(100\rho_l f_{ck}\right)^{1/3} + k_1\sigma_{cp}\right]b_w d; \left(v_{\min} + k_1\sigma_{cp}\right)b_w d\right\} = 78.97\ kN/m > V_{Ed}$$

2.8.4 *Slab Deflection - SLS*

The maximum slab deflection has been evaluated by considering the ***Quasi Permanent*** load combination as required in EN 1992-1-1 §7.4 [3]. The design value of the bending moment in a portion of the slab of unitary width is:

$M_{Ed,\max} = 8.58\ kNm$

Considering a portion of the slab of unitary width and neglecting the presence of rebars the bending moment at cracking is:

$f_{ctm,fl} = \max\left[\left(1.6 - h/1000\right) f_{ctm}; f_{ctm}\right] = 4.20\ MPa$ $\qquad$ $M_{cr} = f_{ctm,fl} \cdot \dfrac{b \cdot h^2}{6} = 15.75\ kNm > M_{Ed}$

The slab is not cracked since $M_{cr} > M_{Ed}$. The evaluation of the maximum deflection is conducted considering the moment of inertia of the uncracked section. The finite element model with the effective modulus of elasticity of the concrete in the case of long-term permanent load have been employed for the deflection evaluation as required in the EN 1992-1-1 §7.4.3(4) [3]. The maximum displacement is equal to δ = 11.09 *mm*. The maximum dimension of the slab is equal to L = 5.700 *m* and hence, the ratio L/δ is equal to 513.98.

2.8.5 *Slab Stresses - SLS*

The stresses on the concrete and on the rebars of the slab are checked with reference to the ***Characteristic*** and the ***Quasi-Permanent*** load combinations as required in EN 1992-1-1 §7.2 [3]. The stresses are directly obtained as results of the model of the analysis.

- Characteristic Load Combination

$$\sigma_{c,max} = 8.35\ MPa < 0.6\ f_{ck} = 18\ MPa$$

$$\sigma_{s,max} = 268\ MPa < 0.8\ f_{yk} = 360\ MPa$$

- Quasi-Permanent Load Combination

$$\sigma_{c,max} = 5.81\ MPa < 0.45\ f_{ck} = 13.5\ MPa$$

2.8.6 *Beams - Effective width of flange for shear lag*

The beams section type is IPE 240. It is a section class 1 in bending and section class 2 in compression. The dimensions of the effective width of flange for shear lag is calculated by following the indications of EN 1994-1-1 §5.4.1.2 [8].

L_e in the different parts of the beams is defined by following the indications reported in Figure 5.1 of EN 1994-1-1 [8]. Figure 2-40 and Figure 2-41 reports the effective width of flanges for shear lag of beams respectively in X e Y directions.

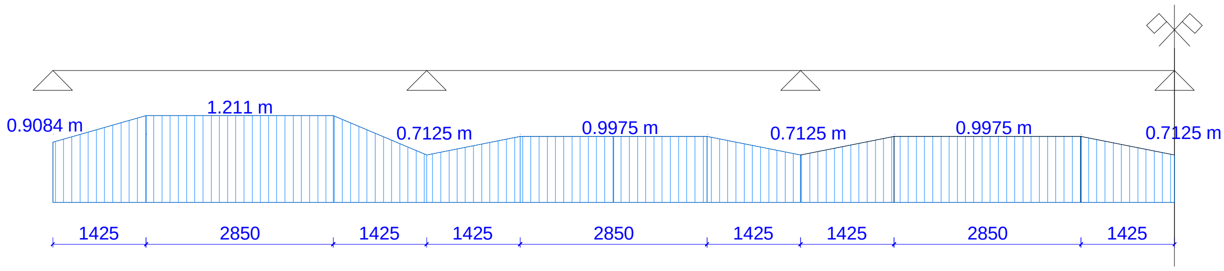


Figure 2-40. Effective width of flanges for shear lag of beams in X direction

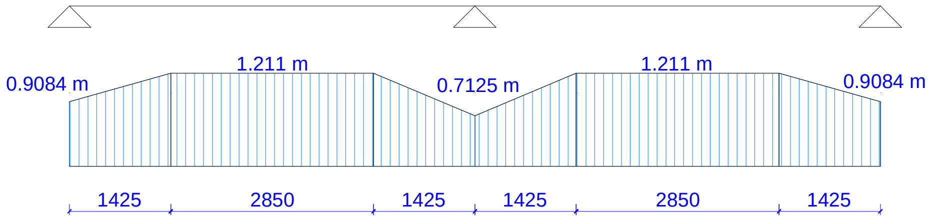
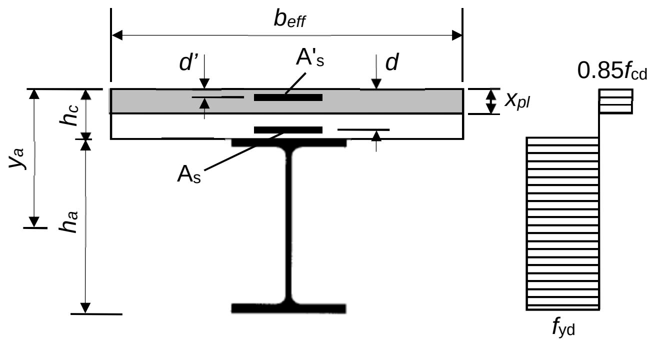


Figure 2-41. Effective width of flanges for shear lag of beams in Y direction

2.8.7 *Beams - Maximum positive bending moment at mid span - ULS*

The maximum positive bending moment of the beam at mid-span is equal to M_{Ed} = 130.49 *kNm*.



The beams section type is IPE 240 and the effective width of flange in this position is equal to b_{eff} = 1.2110 *m*. The calculation of the resisting moment for the ULS condition is reported in the follow.

$$F_{c,\max} = 0.85 \cdot h_c \cdot b_{eff} \frac{f_{ck}}{\gamma_c} = 3088.05\ kN$$ Plastic resistance of the concrete section

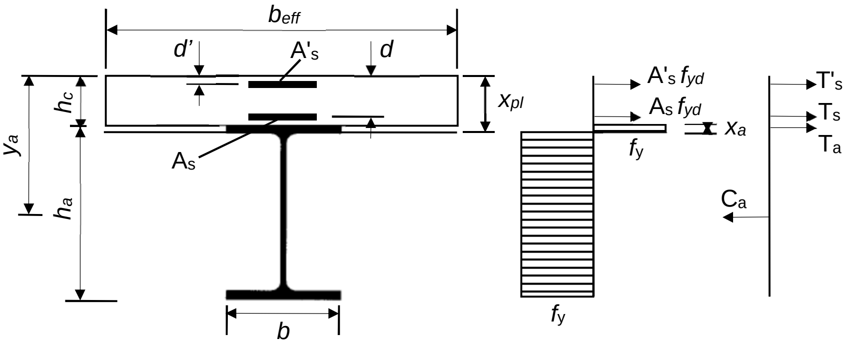
$$F_{a,\max} = A \frac{f_y}{\gamma_{M0}} = 1388.05\ kN$$ Plastic resistance of the steel section

$$x_{pl} = \frac{F_{a,\max}}{0.85 \frac{f_{ck}}{\gamma_c} b_{eff}} = 67.42\ mm$$ Neutral axis position

$$M_{pl}=F_{s,max}\left(\frac{h_a}{2}+h_c-\frac{x_{pl}}{2}\right)=327.98\ \mathrm{kNm}>\mathrm{M_{Ed}}$$

2.8.8 *Beams - Maximum negative bending moment at the support- ULS*

The maximum negative bending moment of the beam at the support is equal to M_{Ed} = 109.18 *kNm*.



The beams section type is IPE 240 and the effective width of flange in this position is equal to b_{eff} = 712.5 *mm*. The amount of rebars contained in the effective width of flange are A_s' = 1363.45 mm^2 (2ϕ10 + 6ϕ16, d' = 34 *mm*) and A_s = 471.24 mm^2 (6ϕ10, d = 116 *mm*) respectively for the upper and lower layers.

$$T_s'=A_s'\frac{f_{yd}}{\gamma_s}=533.52\ kN$$ Tensile force in the rebars;

$$T_s=A_s\frac{f_{yd}}{\gamma_s}=184.40\ kN$$ Tensile force in the rebars;

$$T_a=\frac{1}{2}\left(A\frac{f_y}{\gamma_{M0}}-T_s'-T_s\right)=335.42\ kN$$ Tensile force in steel section;

$$x_{pl}=h_c+\frac{T_a}{b\frac{f_y}{\gamma_{M0}}}=157.87\ mm$$ Neutral axis position

$$C_a=A\frac{f_y}{\gamma_{M0}}-T_a=1053.34\ kN$$ Compressive force in the steel section;

$$M_{pl}=C_a(x_{Ca}-x_{pl})+T_a(x_{pl}-x_{Ta})+T_s(x_{pl}-x_s)+T_s'(x_{pl}-x_s')=189.09\ kN>M_{Ed}$$

2.8.9 *Beams - Maximum shear force at the support- ULS*

As suggested in EN 1993-1-1 §6.2.6, only the beams section IPE 240 is considered for the shear resistance. The maximum shear force on the beam is equal to V_{Ed} = 145.68 *kN.*

$$V_{Rd}=\frac{A_v\frac{f_{yk}}{\gamma_{M0}}}{\sqrt{3}}=392.45\ kN>V_{Ed}$$

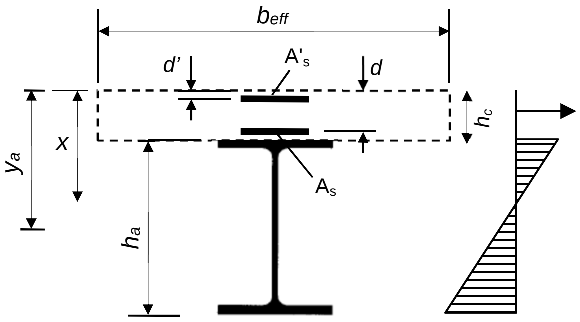
2.8.10 *Beams - Calculation of the crack widths - ULS*

The crack width is calculated by following the EN 1992-1-1 §7.3.4 [3] as follow.

$$w_k = s_{r,max}(\varepsilon_{sm} - \varepsilon_{cm}) \tag{2.7}$$

The structure is subjected to the exposure class XC3 where the maximum crack width is equal to $w_{max} = 0.3\ mm$ while considering the ***Quasi-Permanent*** load combination. (EN 1992-1-1 Table 7.1N [3]). The beams section type is IPE 240 and the effective width of flange in this position is equal to $b_{eff} = 712.5\ mm$

The amount of rebars contained in the effective width of flange are $A'_s = 1363.45\ mm^2$ $(2\emptyset 10 + 6\emptyset 16, d' = 34\ mm)$ and $A'_s = 471.24\ mm^2$ $(6\emptyset 10,\ d = 116\ mm)$ respectively for the upper and lower layers. The negative bending moment of the beam at the support is equal to $M_{Ed} = 54.87\ kNm$.



$A = A_a + A_s + A'_s = 5746.69\ mm^2$ Total area of resisting elements in tension;

$x = \frac{1}{A}(A_a y_a + A_s d + A'_s d') = 201.38\ mm$ Position of the neutral axis;

$J = J_a + A_a(y_a - x)^2 + A_s(x-d)^2 + A'_s(x-d')^2 = 6100.45\ cm^4$ Moment of inertia

$\sigma_{s,t} = \frac{M}{J}(x - d') = 150.55\ MPa$ Tensile stress in the upper layer of rebars;

$$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_{s,t}}{E_s} = 0.000717$$

$$s_{r,max} = 115.6$$

$$w_k = s_{r,max}(\varepsilon_{sm} - \varepsilon_{cm}) = 0.0829\ mm < w_{max}$$

2.8.11 *Columns - Maximum axial force- ULS*

The column section type is HEB 220. It is class 1 both in bending and in compression. The maximum axial force on the column is checked by the EN 1993-1-1 §6.2.4 [4]. The maximum axial force on the columns is equal to $N_{Ed} = 2596.91\ kN$

$$N_{Rd} = \frac{A f_y}{\gamma_{M0}} = 3231\ kN > N_{Ed}$$

2.8.12 *Columns - Maximum shear force- ULS*

The column section type is HEB 220. It is class 1 both in bending and in compression. The maximum shear force on the column is checked by the EN 1993-1-1 §6.2.6 [4]. The maximum shear force on the column is equal to $V_{Ed} = 22.72\ kN$.

$$A_v = A - 2bt_f + (t_w + 2r)t_f = 2792.00\ mm^2$$

$$V_{Rd} = \frac{A_v \dfrac{f_{yk}}{\gamma_{M0}}}{\sqrt{3}} = 572.25\ kN > V_{Ed}$$

2.8.13 *Columns - Member in bending and axial compression - ULS*

The column section type is HEB 220. It is class 1 both in bending and in compression. In order to verify the members which are subjected to combined bending and axial compression, the formulations reported in EN 1993-1-1 §6.3.3 [4] have to be satisfied.

The forces acting in the more stressed column are:

$$N_{Ed} = 2596.91\ kN$$

$$M_{y,Ed,A} = -14.72\ kNm \qquad M_{z,Ed,A} = -7.50\ kNm$$

$$M_{y,Ed,B} = 4.42\ kNm \qquad M_{z,Ed,B} = 4.43\ kNm$$

The reduction factors for buckling for lateral torsional buckling χ_y, χ_z and χ_{LT} have been calculated by following the instruction of EN 1993-1-1 §6.2.1.2, EN 1993-1-1 §6.3.2.2 and EN 1993-1-1 §6.3.2.3 [4].

$$\chi_y = \frac{1}{\phi_y + \sqrt{\phi_y^2 - \bar{\lambda}_y^2}} = 0.989 \qquad \chi_z = \frac{1}{\phi_z + \sqrt{\phi_z^2 - \bar{\lambda}_z^2}} = 0.917 \qquad \chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2}} = 0.963$$

The parameters N_{Rk}, $M_{y,Rk}$ and $M_{z,Rk}$ are calculated as define in EN 1993-1-1 Table 6.7 [4].

$$N_{Rk} = f_y A = 355\ MPa \cdot 9104\ mm^2 = 3231.92\ kN$$

$$M_{y,Rk} = f_y W_{pl,y} = 355\ MPa \cdot 827 \cdot 10^3\ mm^3 = 293.59\ kNm$$

$$M_{z,Rk} = f_y W_{pl,z} = 355\ MPa \cdot 393 \cdot 10^3\ mm^3 = 139.83\ kNm$$

The interaction coefficients have been calculated by using the approach reported in EN 1993-1-1 Annex B [4].

$$k_{yy} = \min\left(C_{my}\left(1 + \left(\bar{\lambda}_y - 0.2\right)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right); C_{my}\left(1 + 0.8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\right) = 0.499$$

$$k_{zz} = \min\left(C_{mz}\left(1 + \left(2\bar{\lambda}_z - 0.6\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right); C_{mz}\left(1 + 1.4\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)\right) = 0.485$$

$$k_{yz} = 0.6k_{zz} = 0.291$$

$$k_{zy} = 0.6k_{yy} = 0.300$$

$$\frac{N_{Ed}}{\dfrac{\chi_y N_{Rk}}{\gamma_{M1}}} + k_{yy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\dfrac{M_{y,Rk}}{\gamma_{M1}}} + k_{yz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\dfrac{M_{z,Rk}}{\gamma_{M1}}} = 0.854 \le 1$$

$$\frac{N_{Ed}}{\dfrac{\chi_z N_{Rk}}{\gamma_{M1}}} + k_{zy}\frac{M_{y,Ed} + \Delta M_{y,Ed}}{\chi_{LT}\dfrac{M_{y,Rk}}{\gamma_{M1}}} + k_{zz}\frac{M_{z,Ed} + \Delta M_{z,Ed}}{\dfrac{M_{z,Rk}}{\gamma_{M1}}} = 0.935 \le 1$$

2.8.14 *Steel Diagonal Bracing - ULS*

The forces on the steel braces have been obtained by employing the finite element model previously described without the compressed braces assuming that they do not have compressive resistance as consequence of the buckling.

The bracing section type is the equal length angle L120×120×10 in *X* direction and the double equal length angle L120×120×10 in *Y* direction. The resistance of the net section is evaluated by following the instructions of EN 1993-1-8 §3.10.3 [2]. The use of bolts M20 is considered.

The maximum axial force acting in the braces for the X direction is equal to $\mathrm{N_{Ed}} = 208.04\ \mathrm{kN}$.

$$\mathrm{A_{Net}} = \mathrm{A} - \mathrm{d_0 t} = 2098\ \mathrm{mm^2} \qquad \mathrm{N_{u,Rd}} = \frac{\beta_3 \mathrm{A_{Net} f_u}}{\gamma_{\mathrm{M2}}} = 599.2\ \mathrm{kN} > \mathrm{N_{Ed}}$$

The maximum axial force acting in the braces for the Y direction is equal to $\mathrm{N_{Ed}} = 635.38\ \mathrm{kN}$.

$$\mathrm{A_{Net}} = \mathrm{A} - \mathrm{d_0 t} = 2098\ \mathrm{mm^2} \qquad \mathrm{N_{u,Rd}} = 2\frac{\beta_3 \mathrm{A_{Net} f_u}}{\gamma_{\mathrm{M2}}} = 1198.4\ \mathrm{kN} > \mathrm{N_{Ed}}$$

2.9 Asymmetric Structure

The following sections report the calculation of the components of the Asymmetric structure.

2.9.1 *Slab - Maximum Bending Moments - ULS*

The bending moments for the design of the slab in both the directions have been obtained by using the finite element model previously described. Figure 2-42 up to Figure 2-45 show the maximum and minimum bending moments in *X* and *Y* direction for the Envelope of all the ULS combinations.

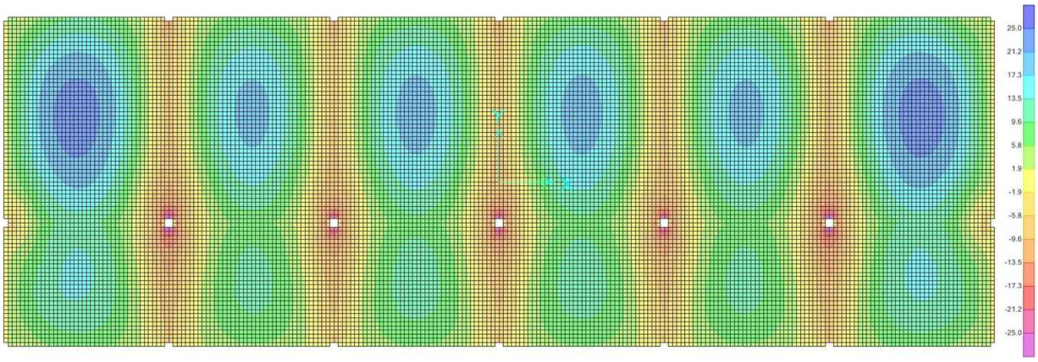


Figure 2-42. Maximum Bending Moments in X direction (ULS Envelope)

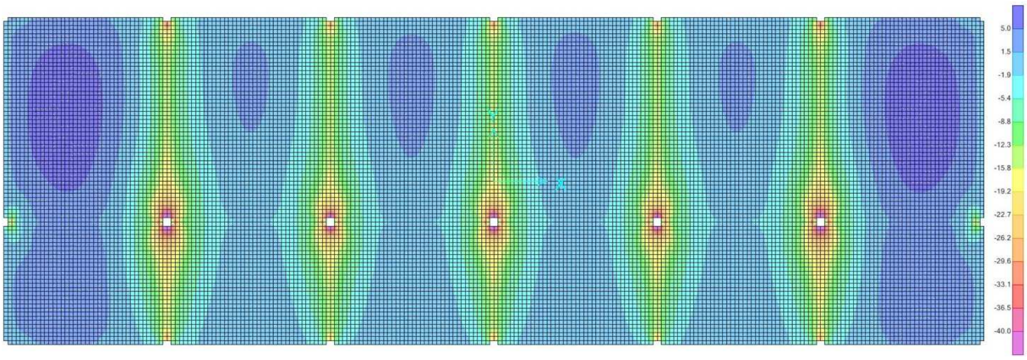


Figure 2-43. Minimum Bending Moments in X direction (ULS Envelope)

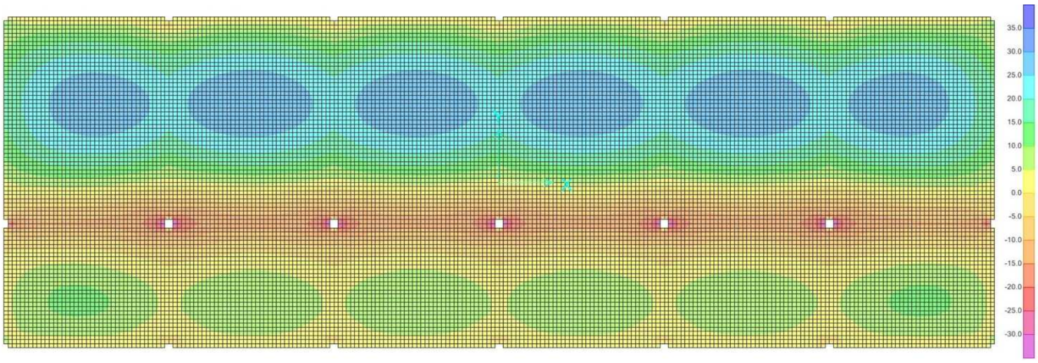


Figure 2-44. Maximum Bending Moments in Y direction (ULS Envelope)

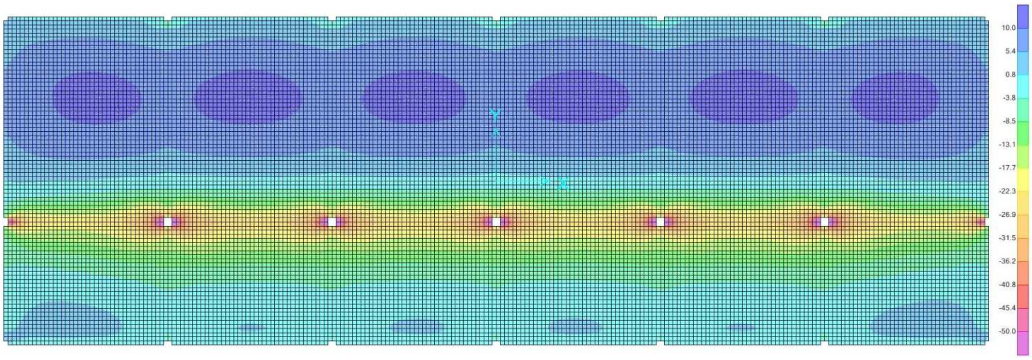


Figure 2-45. Minimum Bending Moments in Y direction (ULS Envelope)

2.9.2 *Slab - Maximum traction force in the layers of rebars - ULS*

The amount of rebars is based on the maximum traction force in each layer of rebars for the Envelope of all the ULS combinations as reported in Figure 2-46 up to Figure 2-49.

The minimum and maximum steel percentages and the maximum spacing of rebars are defined in the EN 1992-1-1 §9.2.1 and EN 1992-1-1 §9.3 [3]. They are respectively:

$A_{s,min} = 150.8\ mm^2/m$ minimum steel percentage

$A_{s,max} = 6000\ mm^2/m$ maximum steel percentage

$s_{max,slab} = 400\ mm$ maximum spacing of rebars

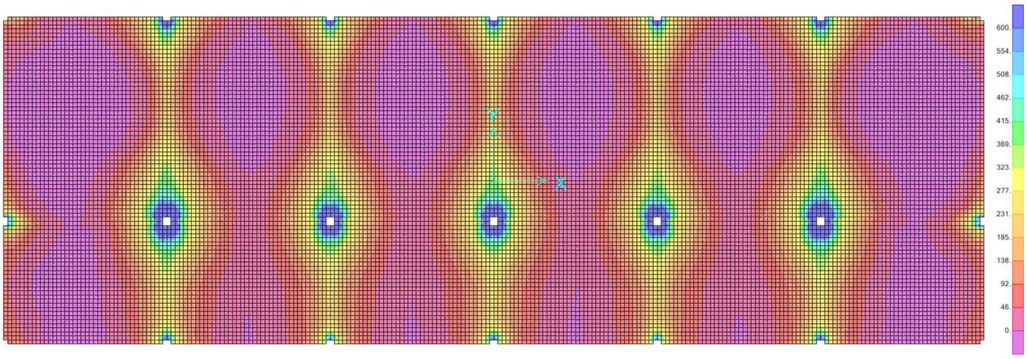


Figure 2-46. Maximum Traction Force in the upper rebars in X direction (ULS Envelope)

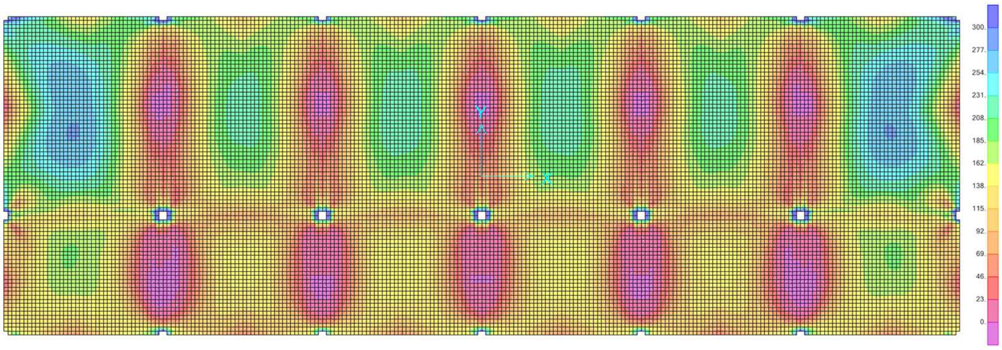


Figure 2-47. Maximum Traction Force in the lower rebars in X direction (ULS Envelope)

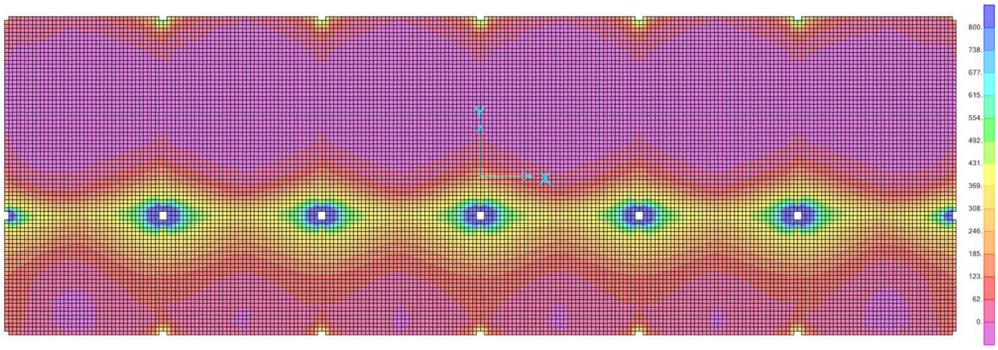


Figure 2-48. Maximum Traction Force in the upper rebars in Y direction (ULS Envelope)

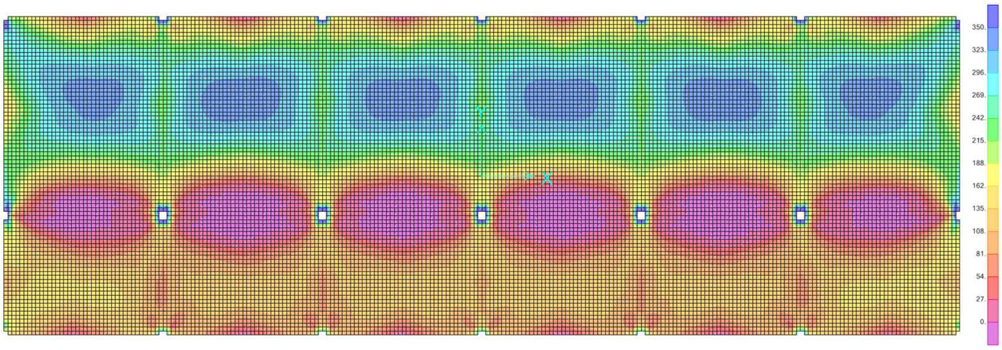


Figure 2-49. Maximum Traction Force in the lower rebars in Y direction (ULS Envelope)

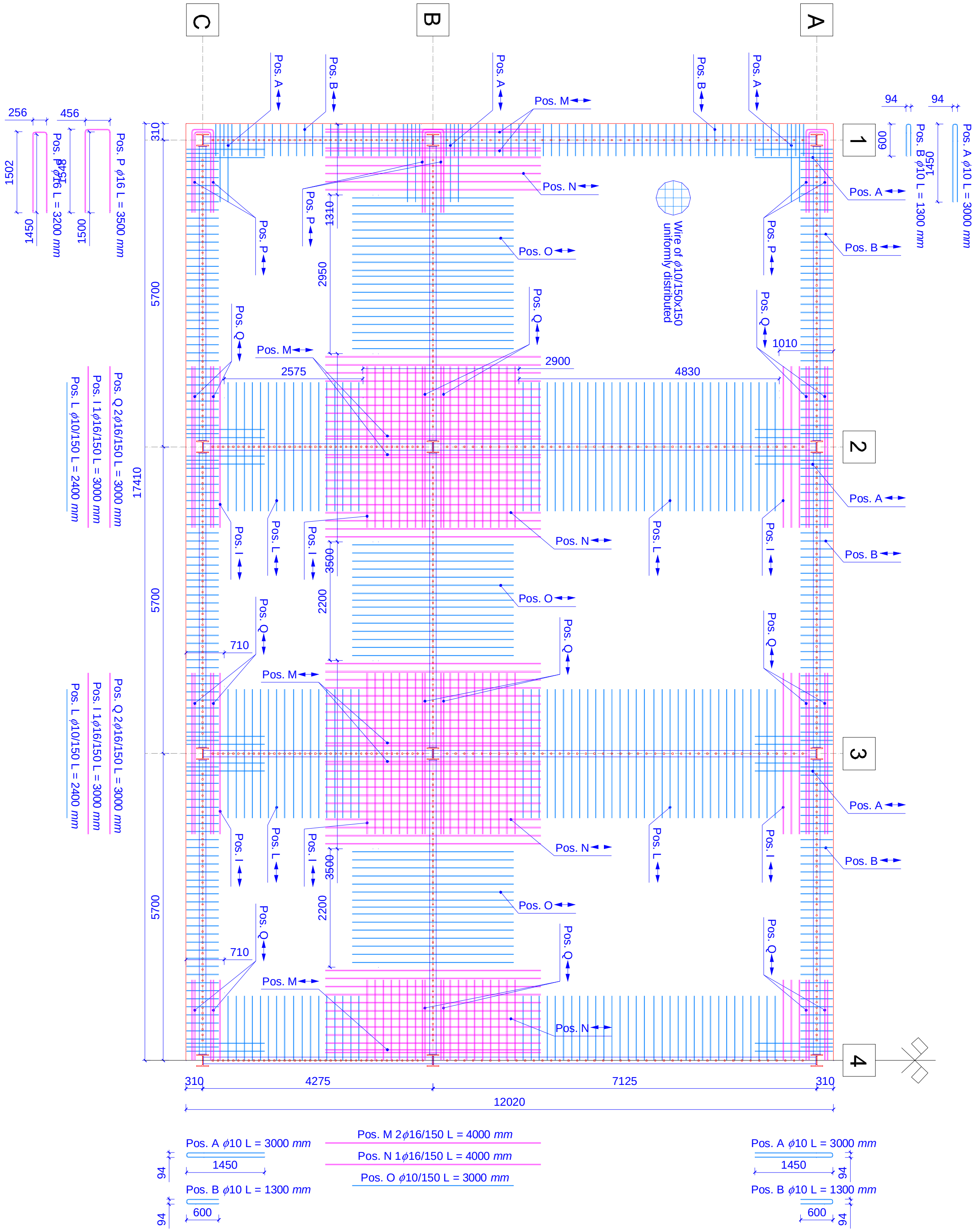


Figure 2-50. Slab Rebars - Upper Side – Asymmetric Configuration (length unit mm)

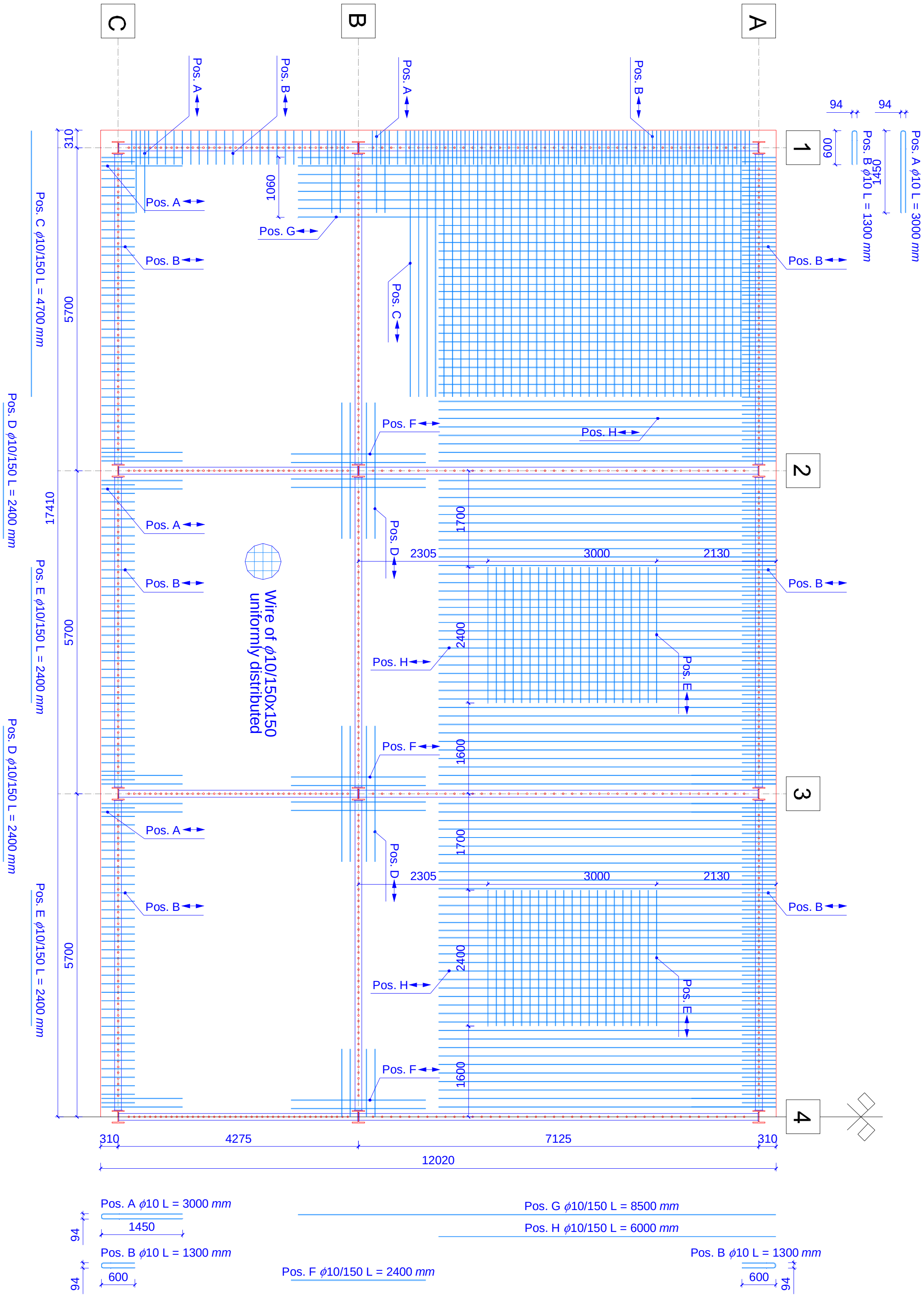


Figure 2-51. Slab Rebars - Lower Side – Asymmetric Configuration (length unit mm)

2.9.3 *Slab Shear - ULS*

The slab is verified against shear forces by following the EN 1992-1-1 §6.2 [3]. The design forces at the ULS are:

$V_{Ed,\max} = 30\, kN/m$ $\quad\quad N_{Ed,\max} = -40\, kN/m$ (traction)

The design value for the shear resistance $V_{Rd,c}$ in members not requiring design shear reinforcement is calculated as specified in the EN 1992-1-1 §6.2.2 [3]:

$$V_{Rd,c} = \max\left\{\left[C_{Rd,c} k \left(100 \rho_l f_{ck}\right)^{1/3} + k_1 \sigma_{cp}\right] b_w d; \left(v_{\min} + k_1 \sigma_{cp}\right) b_w d\right\} = 76.65\, kN/m > V_{Ed}$$

2.9.4 *Slab Deflection - SLS*

The maximum slab deflection has been evaluated by considering the ***Quasi Permanent*** load combination as required in EN 1992-1-1 §7.4 [3]. The design value of the bending moment in a portion of the slab of unitary width is:

$M_{Ed,\max} = 12.33\, kNm$

Considering a portion of the slab of unitary width and neglecting the presence of rebars the bending moment at cracking is:

$$f_{ctm,fl} = \max\left[\left(1.6 - h/1000\right) f_{ctm}; f_{ctm}\right] = 4.20\, MPa \quad\quad M_{cr} = f_{ctm,fl} \cdot \frac{b \cdot h^2}{6} = 15.75\, kNm > M_{Ed}$$

The slab is not cracked since $M_{cr} > M_{Ed}$. The evaluation of the maximum deflection is conducted considering the moment of inertia of the uncracked section. The finite element model with the effective modulus of elasticity of the concrete in the case of long-term permanent load have been employed for the deflection evaluation as required in the EN 1992-1-1 §7.4.3(4) [3]. The maximum displacement is equal to δ = 20.476 *mm*. The maximum dimension of the slab is equal to L = 7.125 *m* and hence, the ratio L/δ is equal to 347.97.

2.9.5 *Slab Stresses - SLS*

The stresses on the concrete and on the rebars of the slab are checked with reference to the ***Characteristic*** and the ***Quasi-Permanent*** load combinations as required in EN 1992-1-1 §7.2 [3]. The stresses are directly obtained as results of the model of the analysis.

- Characteristic Load Combination

$$\sigma_{c,max} = 12.28\, MPa < 0.6\, f_{ck} = 18\, MPa$$

$$\sigma_{s,max} = 348\, MPa < 0.8\, f_{yk} = 360\, MPa$$

- Quasi-Permanent Load Combination

$$\sigma_{c,max} = 8.55\, MPa < 0.45\, f_{ck} = 13.5\, MPa$$

2.9.6 *Beams - Effective width of flange for shear lag*

The beams section type is IPE 240 and it is a section class 1 in bending and section class 2 in compression. The dimensions of the effective width of flange for shear lag is calculated by following the indications of EN 1994-1-1 §5.4.1.2 [8]. L_e in the different parts of the beams is defined by following the indications reported in Figure 5.1 of EN 1994-1-1 [8]. Figure 2-52 and

Figure 2-53 reports the effective width of flanges for shear lag of beams respectively in X e Y directions.

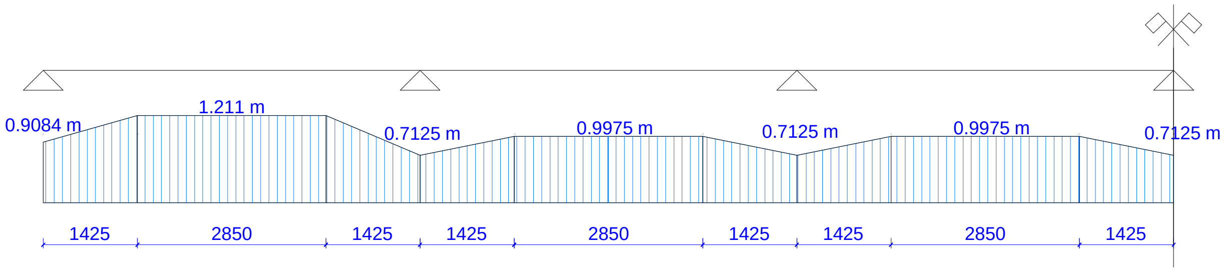


Figure 2-52. Effective width of flanges for shear lag of beams in X direction

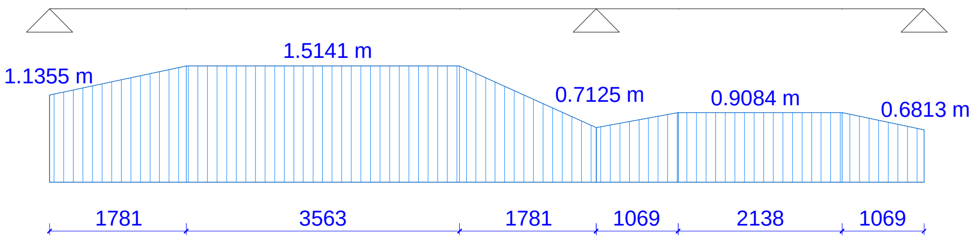
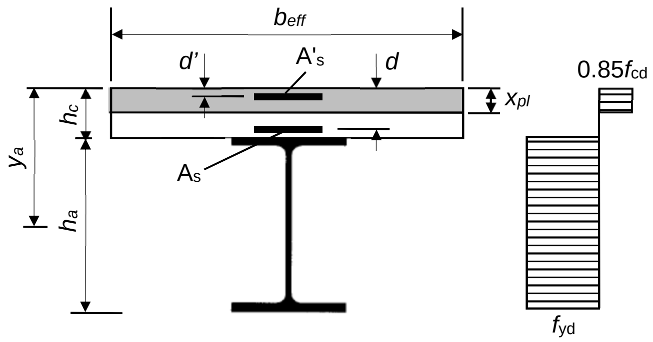


Figure 2-53. Effective width of flanges for shear lag of beams in Y direction

2.9.7 *Beams - Maximum positive moment at mid span - Long Span - ULS*

The maximum positive bending moment of the beam at mid-span is equal to M_{Ed} = 216.93 *kNm*. The beams section type is IPE 240 and the effective width of flange in this position is equal to b_{eff} = 1.514 *m*. The calculation of the resisting moment for the ULS condition is reported in the follow.



$$F_{c,\max} = 0.85 \cdot h_c \cdot b_{eff} \frac{f_{ck}}{\gamma_c} = 3860.70\ kN$$ Plastic resistance of the concrete section

$$F_{a,\max} = A \frac{f_y}{\gamma_{M0}} = 1388.05\ kN$$ Plastic resistance of the steel section

$$x_{pl} = \frac{F_{a,\max}}{0.85 \frac{f_{ck}}{\gamma_c} b_{eff}} = 53.93\ mm$$ Neutral axis position

$$M_{pl} = F_{s,\max}\left(\frac{h_a}{2} + h_c - \frac{x_{pl}}{2}\right) = 337.34\ \mathrm{kNm} > \mathrm{M_{Ed}}$$

2.9.8 *Beams - Maximum positive moment at mid span - Short Span - ULS*

The maximum positive bending moment of the beam at mid-span is equal to M_{Ed} = 49.30 *kNm*. The beams section type is IPE 240 and the effective width of flange in this position is equal to b_{eff} = 0.9084 *m*. The calculation of the resisting moment for the ULS condition is reported in the follow.

$$F_{c,\max}=0.85\cdot h_c\cdot b_{eff}\frac{f_{ck}}{\gamma_c}=2316.42\ kN$$ Plastic resistance of the concrete section

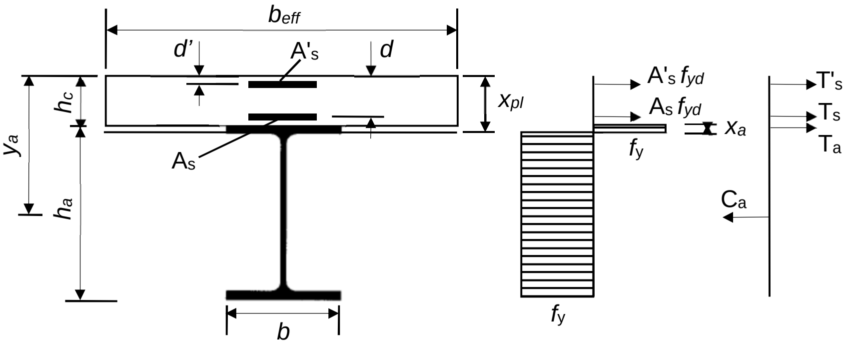
$$F_{a,\max}=A\frac{f_y}{\gamma_{M0}}=1388.05\ kN$$ Plastic resistance of the steel section

$$x_{pl}=\frac{F_{a,\max}}{0.85\frac{f_{ck}}{\gamma_c}b_{eff}}=89.88\ mm$$ Neutral axis position

$$M_{pl}=F_{s,\max}\left(\frac{h_a}{2}+h_c-\frac{x_{pl}}{2}\right)=312.39\ \mathrm{kNm}>\mathrm{M_{Ed}}$$

2.9.9 *Beams - Maximum negative bending moment at the support- ULS*

The maximum negative bending moment of the beam at the support is equal to M_{Ed} = 149.88 *kNm*.



The beams section type is IPE 240 and the effective width of flange in this position is equal to b_{eff} = 712.5 *mm*. The amount of rebars contained in the effective width of flange are A_s' = 1363.45 mm^2 (2ϕ10 + 6ϕ16, d' = 34 *mm*) and A_s = 471.24 mm^2 (6ϕ10, d = 116 *mm*) respectively for the upper and lower layers.

$$T_s'=A_s'\frac{f_{yd}}{\gamma_s}=533.52\ kN$$ Tensile force in the rebars;

$$T_s=A_s\frac{f_{yd}}{\gamma_s}=184.40\ kN$$ Tensile force in the rebars;

$$T_a=\frac{1}{2}\left(A\frac{f_y}{\gamma_{M0}}-T_s'-T_s\right)=335.42\ kN$$ Tensile force in steel section;

$$x_{pl}=h_c+\frac{T_a}{b\frac{f_y}{\gamma_{M0}}}=157.87\ mm$$ Neutral axis position

$C_a = A\frac{f_y}{\gamma_{M0}} - T_a = 1053.34\,kN$ Compressive force in the steel section;

$M_{pl} = C_a(x_{Ca} - x_{pl}) + T_a(x_{pl} - x_{Ta}) + T_s(x_{pl} - x_s) + T_s'(x_{pl} - x_s') = 189.09\,kN > M_{Ed}$

2.9.10 *Beams - Maximum shear force at the support- ULS*

As suggested in EN 1993-1-1 §6.2.6 [4], only the beams section IPE 240 is considered for the shear resistance. The maximum shear force on the beam is equal to V_{Ed} = 173.56 *kN.*

$$V_{Rd} = \frac{A_v \frac{f_{yk}}{\gamma_{M0}}}{\sqrt{3}} = 392.45\,kN > V_{Ed}$$

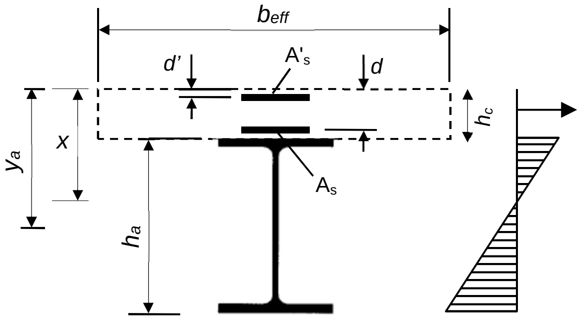
2.9.11 *Beams - Calculation of the crack widths - ULS*

The crack width is calculated by following the EN 1992-1-1 §7.3.4 [3] as follow.

$$w_k = s_{r,max}(\varepsilon_{sm} - \varepsilon_{cm}) \quad (2.8)$$

The structure is subjected to the exposure class XC3 where the maximum crack width is equal to $w_{max} = 0.3\ mm$ while considering the ***Quasi-Permanent*** load combination. (EN 1992-1-1 Table 7.1N [3]).

The beams section type is IPE 240 and the effective width of flange in this position is equal to $b_{eff} = 712.5\ mm$ The amount of rebars contained in the effective width of flange are $A'_s = 1363.45\ mm^2$ $(2\emptyset10 + 6\emptyset16, d' = 34\ mm)$ and $A'_s = 471.24\ mm^2$ $(6\emptyset10,\ d = 116\ mm)$ respectively for the upper and lower layers. The negative bending moment of the beam at the support is equal to $M_{Ed} = 77.56\ kNm$.



$A = A_a + A_s + A_s' = 5746.69\,mm^2$ Total area of resisting elements in tension;

$x = \frac{1}{A}\left(A_a y_a + A_s d + A_s' d'\right) = 201.38\,mm$ Position of the neutral axis;

$J = J_a + A_a\left(y_a - x\right)^2 + A_s\left(x - d\right)^2 + A_s'\left(x - d'\right)^2 = 6100.45\,cm^4$ Moment of inertia

$\sigma_{s,t} = \frac{M}{J}\left(x - d'\right) = 212.80\,MPa$ Tensile stress in the upper layer of rebars;

$\varepsilon_{sm} - \varepsilon_{cm} = \frac{\sigma_{s,t}}{E_s} = 0.001013$

$s_{r,\max} = 115.6$

$$w_k = s_{r,\max}\left(\varepsilon_{sm} - \varepsilon_{cm}\right) = 0.1171\, mm < w_{\max}$$

2.9.12 *Columns - Maximum axial force- ULS*

The column section type is HEB 220. It is class 1 both in bending and in compression. The maximum axial force on the column is checked by the EN 1993-1-1 §6.2.4 [4]. The maximum axial force on the columns is equal to $\mathrm{N_{Ed}} = 2689.47\ \mathrm{kN}$

$$N_{Rd} = \frac{A f_y}{\gamma_{M0}} = 3231\, kN > N_{Ed}$$

2.9.13 *Columns - Maximum shear force- ULS*

The column section type is HEB 220. It is class 1 both in bending and in compression. The maximum shear force on the column is checked by the EN 1993-1-1 §6.2.6 [4]. The maximum shear force on the column is equal to $\mathrm{V_{Ed}} = 22.75\ \mathrm{kN}$.

$$A_v = A - 2bt_f + \left(t_w + 2r\right)t_f = 2792.00\, mm^2$$

$$V_{Rd} = \frac{A_v \dfrac{f_{yk}}{\gamma_{M0}}}{\sqrt{3}} = 572.25\, kN > V_{Ed}$$

2.9.14 *Columns - Member in bending and axial compression - ULS*

The column section type is HEB 220. It is class 1 both in bending and in compression. In order to verify the members which are subjected to combined bending and axial compression, the formulations reported in EN 1993-1-1 §6.3.3 [4] have to be satisfied.

The forces acting in the more stressed column are:

$$N_{Ed} = 2689.47\, kN$$

$$M_{y,Ed,A} = -17.48\, kNm \qquad M_{z,Ed,A} = -7.67\, kNm$$

$$M_{y,Ed,B} = 2.15\, kNm \qquad M_{z,Ed,B} = 3.91\, kNm$$

The reduction factors for buckling for lateral torsional buckling χ_y, χ_z and χ_{LT} have been calculated by following the instruction of EN 1993-1-1 §6.2.1.2, EN 1993-1-1 §6.3.2.2 and EN 1993-1-1 §6.3.2.3 [4].

$$\chi_y = \frac{1}{\phi_y + \sqrt{\phi_y^2 - \bar{\lambda}_y^2}} = 0.989 \qquad \chi_z = \frac{1}{\phi_z + \sqrt{\phi_z^2 - \bar{\lambda}_z^2}} = 0.917 \qquad \chi_{LT} = \frac{1}{\phi_{LT} + \sqrt{\phi_{LT}^2 - \beta\bar{\lambda}_{LT}^2}} = 0.963$$

The parameters N_{Rk}, $M_{y,Rk}$ and $M_{z,Rk}$ are calculated as define in EN 1993-1-1 Table 6.7 [4].

$$N_{Rk} = f_y A = 355\, MPa \cdot 9104\, mm^2 = 3231.92\, kN$$

$$M_{y,Rk} = f_y W_{pl,y} = 355\, MPa \cdot 827 \cdot 10^3\, mm^3 = 293.59\, kNm$$

$$M_{z,Rk} = f_y W_{pl,z} = 355\, MPa \cdot 393 \cdot 10^3\, mm^3 = 139.83\, kNm$$

The interaction coefficients have been calculated by using the approach reported in EN 1993-1-1 Annex B [4].

$$k_{yy} = \min\left(C_{my}\left(1+\left(\bar{\lambda}_y - 0.2\right)\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right);C_{my}\left(1+0.8\frac{N_{Ed}}{\chi_y N_{Rk}/\gamma_{M1}}\right)\right)=0.574$$

$$k_{zz} = \min\left(C_{mz}\left(1+\left(2\bar{\lambda}_z - 0.6\right)\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right);C_{mz}\left(1+1.4\frac{N_{Ed}}{\chi_z N_{Rk}/\gamma_{M1}}\right)\right)=0.488$$

$$k_{yz} = 0.6k_{zz} = 0.293$$

$$k_{zy} = 0.6k_{yy} = 0.344$$

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}}+k_{yy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}}+k_{yz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}}=0.894\le 1$$

$$\frac{N_{Ed}}{\frac{\chi_z N_{Rk}}{\gamma_{M1}}}+k_{zy}\frac{M_{y,Ed}+\Delta M_{y,Ed}}{\chi_{LT}\frac{M_{y,Rk}}{\gamma_{M1}}}+k_{zz}\frac{M_{z,Ed}+\Delta M_{z,Ed}}{\frac{M_{z,Rk}}{\gamma_{M1}}}=0.977\le 1$$

2.9.15 *Steel Diagonal Bracing - ULS*

The forces on the steel braces have been obtained by employing the finite element model previously described without the compressed braces assuming that they do not have compressive resistance as consequence of the buckling.

The bracing section type is the equal length angle L120×120×10 in *X* direction and the double equal length angle L120×120×10 in *Y* direction. The resistance of the net section is evaluated by following the instructions of EN 1993-1-8 §3.10.3 [2]. The use of bolts M20 is considered.

The maximum axial force acting in the braces for the X direction is equal to $N_{Ed} = 238.78\ kN$.

$A_{Net} = A - d_0 t = 2098\ mm^2$ $\qquad N_{u,Rd} = \frac{\beta_3 A_{Net} f_u}{\gamma_{M2}} = 599.2\ kN > N_{Ed}$

The maximum axial force acting in the braces for the Y direction is equal to $N_{Ed} = 670.68\ kN$.

$A_{Net} = A - d_0 t = 2098\ mm^2$ $\qquad N_{u,Rd} = 2\frac{\beta_3 A_{Net} f_u}{\gamma_{M2}} = 1198.4\ kN > N_{Ed}$

2.10 Calculation of the shear connectors - ULS

The beams section type is IPE 240 and it is a section class 1 in bending and section class 2 in compression. The design resistance of a headed stud automatically welded is calculated as reported in EN 1994-1-1 §6.6.3.1 [8]. The number of studs is designed for the full shear connection. The properties of the studs employed are reported in the follow:

$d = 19\ mm$ — Diameter of the shank of the stud;

$h_{sc} = 100\ mm$ — Overall nominal height of the stud;

$f_u = 450\ MPa$ — Ultimate tensile strength of the material of the stud;

$$P_{s,Rd} = \frac{0.8\cdot f_u \cdot \frac{\pi\cdot d^2}{4}}{\gamma_V} = 81.66\ kN$$

$$P_{c,Rd} = \frac{0.29 \cdot \alpha \cdot d^2 \sqrt{f_{ck} E_{cm}}}{\gamma_V} = 83.13\ kN$$

$$P_{Rd} = \min\left(P_{c,Rd}, P_{s,Rd}\right) = 81.66\ kN$$ Design resistance of a headed stud

2.10.1 *Beams in X direction*

As conservative assumption the internal beam is considered for the design of the studs.

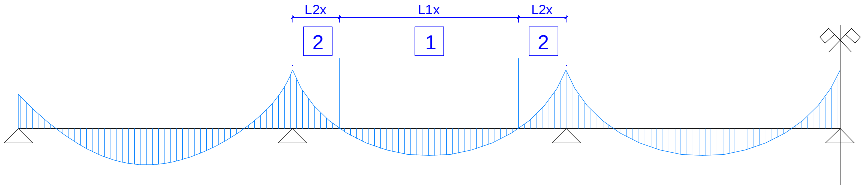


Figure 2-54. Zones for the calculation of the shear connection – X direction

Zone 1

$$V_L = \min\left(F_{c,\max}, F_{a,\max}\right) = 1388.76\ kN \qquad n_{stud} = 2\frac{V_L}{P_{Rd}} = 2\frac{1388.76\ kN}{81.66\ kN} = 34$$

Zone 2

$$V_L = \min\left(F_{s,\max}, F_{a,\max}\right) = 717.92\ kN \qquad n_{stud} = \frac{V_L}{P_{Rd}} = \frac{717.92\ kN}{81.66\ kN} = 8.8$$

The total number of studs on a beam is equal to $n_{Tot} = 34 + 2 \cdot 9 = 52$. The connectors are spaced uniformly over the length of the beam as reported in EN 1994-1-1 §6.6.1.3 (3) [8].

2.10.2 *Beams in Y direction*

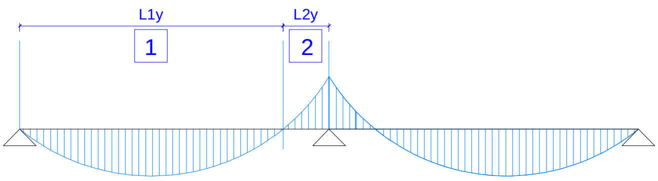


Figure 2-55. Zones for calculation of shear connection – Symmetric structure – Y direction

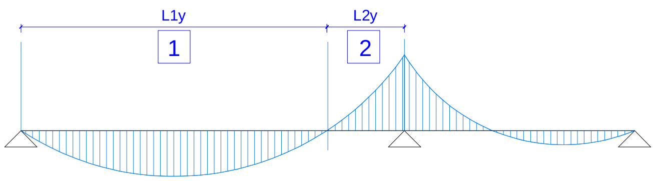


Figure 2-56. Zones for calculation of shear connection – Asymmetric structure – Y direction

Zone 1

$$V_L = \min\left(F_{c,\max}, F_{a,\max}\right) = 1388.76\ kN \qquad n_{stud} = 2\frac{V_L}{P_{Rd}} = 2\frac{1388.76\ kN}{81.66\ kN} = 34$$

Zone 2

$$V_L = \min\left(F_{s,\max}, F_{a,\max}\right) = 717.92\ kN \qquad n_{stud} = \frac{V_L}{P_{Rd}} = 8.8$$

The total number of studs on a beam is equal to $n_{Tot} = 34 + 9 = 43$. The connectors are spaced uniformly over the length of the beam as reported in EN 1994-1-1 §6.6.1.3 (3) [8].

2.11 Design of composite joints

These composite joints employed for this structure are beam-to-column flush end-plate connections. The evaluation of moment resistance and stiffness is based on EN 1993-1-1 [4], EN 1993-1-8 [2] and EN 1994-1-1 [8]. Six different joint configurations have been identified in the model as illustrated in Figure 2-59.

The elements employed in the joints are:

- Column HE 220B;
- Beam IPE 240;
- Solid slab with h_c = *150 mm*;
- Bolts M20 Class 10.9;

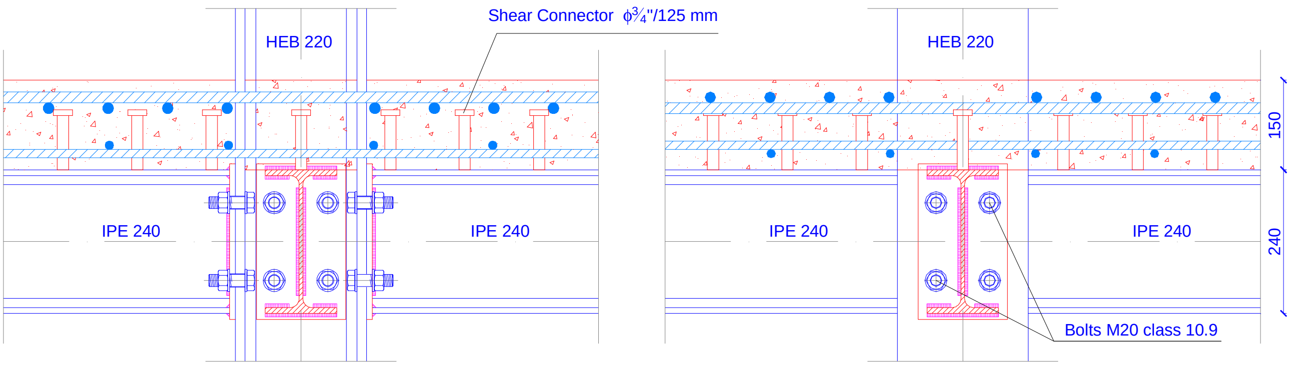


Figure 2-57. Details of the Interior Joint

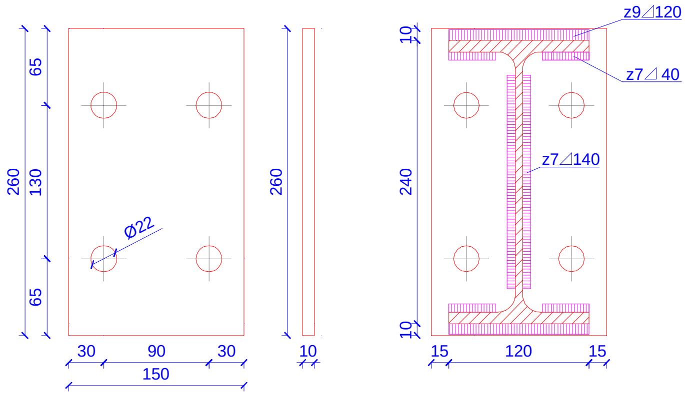


Figure 2-58. Details of the End Plate

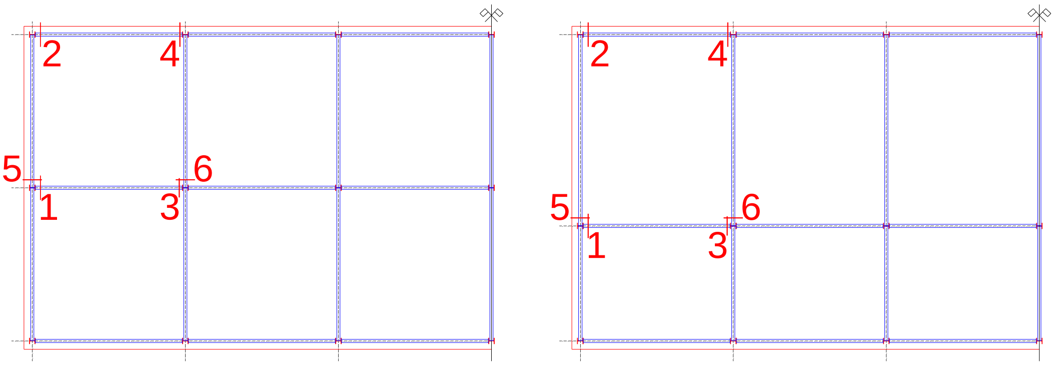


Figure 2-59. Different Joint types for the Symmetric and Asymmetric structure

Joints 1 and 2 are single side joints with the beam that is connected on the flange of the column, which differs for the amount of rebars within the effective width.

Joints 3 and 4 are double side joints with the beam that is connected on the flange of the column, which differs for the amount of rebars within the effective width.

Joints 5 and 6 are double side joints with the beam that is connected on the web of the column, which differs for the amount of rebars within the effective width.

2.11.1 *Joint type 1, X direction (single sided joint, β=1)*

The components involved in the calculation of the resistance and stiffness of the joint illustrated in

Figure *2-60* are reported in the follow:

- usrt - Upper slab reinforcement in tension
- lsrt - Lower slab reinforcement in tension
- cws - Column web panel in shear EN 1993-1-8 §6.2.6.1 and §6.3.2
- cwc - Column web in transverse compression EN 1993-1-8 §6.2.6.2 and §6.3.2
- cwt - Column web in transverse tension EN 1993-1-8 §6.2.6.3 and §6.3.2
- cfb - Column flange in bending EN 1993-1-8 §6.2.6.4 and §6.3.2
- epb - End-plate in bending EN 1993-1-8 §6.2.6.5 and §6.3.2
- bfwc - Beam flange and web in compression EN 1993-1-8 §6.2.6.7 and §6.3.2
- bwt - Beam web in tension EN 1993-1-8 §6.2.6.8 and §6.3.2
- bt - Bolt in tension

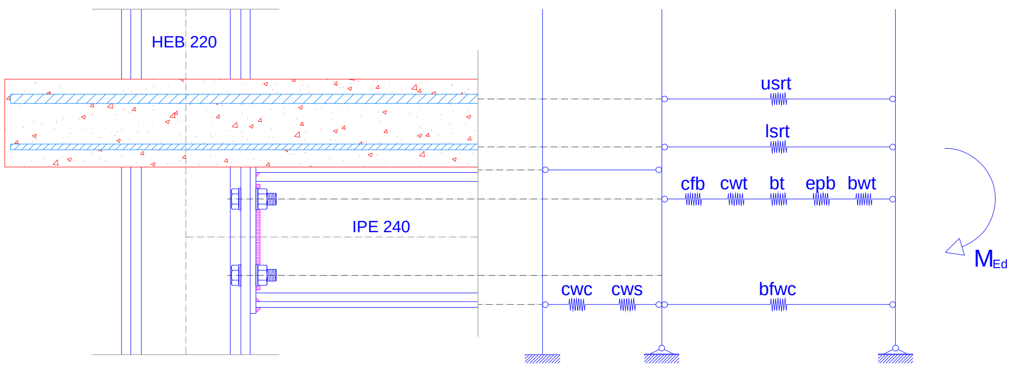


Figure 2-60. Single Side Beam-to-Column Joint connected on column flange (Type 1 and 2)

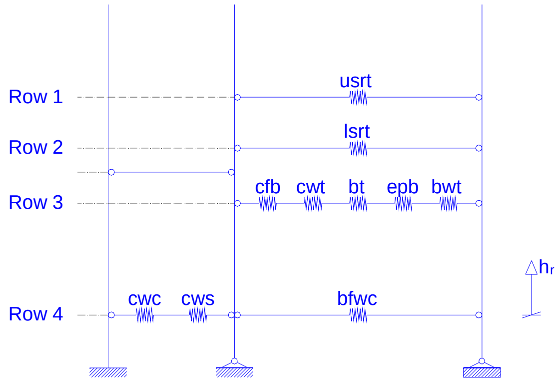


Figure 2-61. Arrangement of the components (Type 1 and 2)

Row 1 $(h_1 = 351.1\,mm)$

- usrt - Upper slab reinforcement in tension $F_{t,As1,Rd} = 499.10\,kN$ $k_{13,1,t} = 1.080\,mm$

Row 2 $(h_2 = 269.1\,mm)$

- lsrt - Lower slab reinforcement in tension $F_{t,As2,Rd} = 184.40\,kN$ $k_{13,2,t} = 0.435\,mm$

Row 3 $(h_3 = 180.1\,mm)$

- cwt - Column web in transverse tension $F_{t,wc,Rd} = 463.43\,kN$ $k_3 = 7.11\,mm$
- cfb - Column flange in bending $F_{t,cfb,Rd} = 339.74\,kN$ $k_4 = 34.66\,mm$
- epb - End-plate in bending $F_{t,epb,Rd} = 186.38\,kN$ $k_5 = 3.59\,mm$
- bwt - Beam web in tension $F_{t,wb,Rd} = 419.46\,kN$ $k_8 = \infty$
- bt - Bolt in tension $2 \cdot F_{t,Rd} = 352.80\,kN$ $k_{10} = 8.46\,mm$

Row 4 $(h_4 = 0\,mm)$

- cws - Column web panel in shear $V_{wp,Rd} = 515.02\,kN$ $k_1 = 3.77\,mm$
- cwc - Column web in transverse compression $F_{c,wc,Rd} = 544.80\,kN$ $k_2 = 2.59\,mm$
- bfwc - Beam flange and web in compression $F_{c,fb,Rd} = 565.35\,kN$ $k_7 = \infty$

The design resistance moment of the composite joint with full shear connection is determined by analogy to provisions for steel joints given in EN 1993-1-8 §6.2.7 [2] taking account of the contribution of reinforcement as specified in EN 1994-1-1 §8.3.2(2) [8]. The moment plastic resistance of the joint is reached as consequence of the failure of the column web in shear.

$$M_{Rd} = F_{t,As1,Rd} \cdot h_1 + \left(V_{wp,Rd} - F_{t,As1,Rd}\right) h_2 = 179.52\,kNm$$

The rotational stiffness of the composite joint with full shear connection is determined by analogy to provisions for steel joints given in EN 1993-1-8 §6.2.7 [2] taking account of the contribution of reinforcement as specified in EN 1994-1-1 §8.3.2(2) [8].

The general method described in EN 1993-1-8 §6.3.3.1 [2] is applied in order to account for the 3 row in tension. In the considered case the effective stiffness coefficients are calculated based on EN 1993-1-8 §6.3.3.1(2) (6.30) [2].

The rotational stiffness of the joint is calculated as specified in EN 1993-1-8 §6.3.1(4) (6.27) [2].

$$S_{j,ini} = \frac{E_s \cdot z_{eq}^2}{\mu\left(\frac{1}{k_{eff,4}} + \frac{1}{k_{eq}}\right)} = 15814.5 \frac{kNm}{rad}$$

2.11.2 *Joint type 2, X direction (single sided joint, β=1)*

The components involved in the calculation of the resistance and stiffness of the joint are the same of the Joint type 1 and are illustrated in

Figure *2-60*. Joints 1 and 2 are single side joints with the beam that is connected on the flange of the column, which differs for the amount of rebars within the effective width. The components which differ from Joint 1 are the upper slab reinforcement in tension (*usrt*) and the lower slab reinforcement in tension (*lsrt*). Arrangement of the components is reported in Figure 2-61.

Row 1 $(h_1 = 351.1\,mm)$

- usrt - Upper slab reinforcement in tension $F_{t,As1,Rd} = 406.90\,kN$ $k_{13,1,t} = 0.910\,mm$

Row 2 $(h_2 = 269.1\,mm)$

- lsrt - Lower slab reinforcement in tension $F_{t,As2,Rd} = 92.20\,kN$ $k_{13,2,t} = 0.230\,mm$

Row 3 $(h_3 = 180.1\,mm)$

- cwt - Column web in transverse tension $F_{t,wc,Rd} = 463.43\,kN$ $k_3 = 7.11\,mm$
- cfb - Column flange in bending $F_{t,cfb,Rd} = 339.74\,kN$ $k_4 = 34.66\,mm$
- epb - End-plate in bending $F_{t,epb,Rd} = 186.38\,kN$ $k_5 = 3.59\,mm$
- bwt - Beam web in tension $F_{t,wb,Rd} = 419.46\,kN$ $k_8 = \infty$
- bt - Bolt in tension $2 \cdot F_{t,Rd} = 352.80\,kN$ $k_{10} = 8.46\,mm$

Row 4 $(h_4 = 0\,mm)$

- cws - Column web panel in shear $V_{wp,Rd} = 515.02\,kN$ $k_1 = 3.77\,mm$
- cwc - Column web in transverse compression $F_{c,wc,Rd} = 544.80\,kN$ $k_2 = 2.59\,mm$
- bfwc - Beam flange and web in compression $F_{c,fb,Rd} = 565.35\,kN$ $k_7 = \infty$

The design resistance moment of the composite joint with full shear connection is determined by analogy to provisions for steel joints given in EN 1993-1-8 §6.2.7 [2] taking account of the contribution of reinforcement as specified in EN 1994-1-1 §8.3.2(2) [8]. The moment plastic resistance of the joint is reached as consequence of the failure of the column web in shear.

$$M_{Rd} = F_{t,As1,Rd} \cdot h_1 + F_{t,As2,Rd} \cdot h_2 + \left(V_{wp,Rd} - F_{t,As1,Rd} - F_{t,As2,Rd}\right) h_3 = 170.54\,kNm$$

The rotational stiffness of the composite joint with full shear connection is determined by analogy to provisions for steel joints given in EN 1993-1-8 §6.2.7 [2] taking account of the contribution of reinforcement as specified in EN 1994-1-1 §8.3.2(2) [8].

The general method described in EN 1993-1-8 §6.3.3.1 [2] is applied in order to account for the 3 row in tension. In the considered case the effective stiffness coefficients are calculated based on EN 1993-1-8 §6.3.3.1 (2) (6.30) [2].

The rotational stiffness of the joint is calculated as specified in EN 1993-1-8 §6.3.1(4) (6.27) [2].

$$S_{j,ini} = \frac{E_s \cdot z_{eq}^2}{\mu\left(\frac{1}{k_{eff,4}} + \frac{1}{k_{eq}}\right)} = 14392.1 \frac{kNm}{rad}$$

2.11.3 *Joint type 3, X direction (double sided joint, β=0)*

The components involved in the calculation of the resistance and stiffness of the joint are the same of the Joint type 1 and are illustrated in Figure 2-62. Joints 3 and 4 are double side joints with the beam that is connected on the flange of the column These joints differ from the Joints 1 and 2 for the absence of the component column web panel in shear (*cws*). The components which differ from Joint 1 are the upper slab reinforcement in tension (*usrt*), the lower slab reinforcement in tension (*lsrt*), the column web in transverse compression (*cwc*) and the column web in transverse tension (*cwt*). Only the calculation of these components is reported in the following.

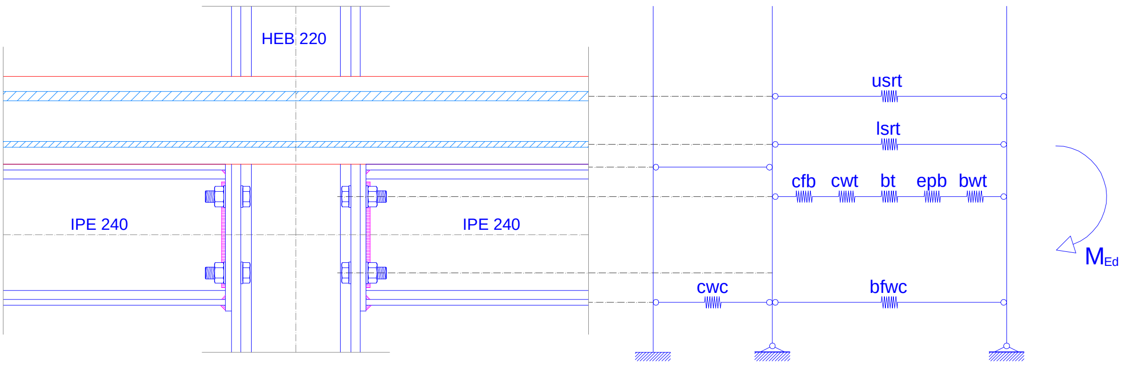


Figure 2-62. Double Side Beam-to-Column Joint connected on column flange (Type 3 and 4)

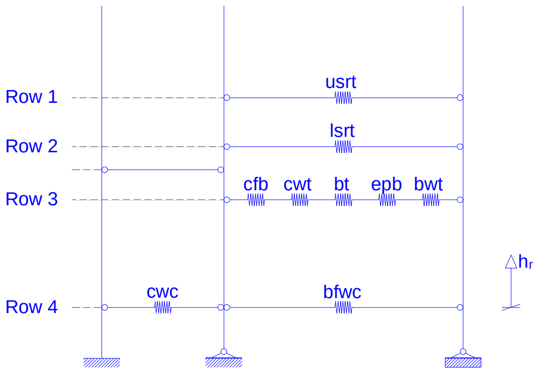


Figure 2-63. Arrangement of the components (Type 3 and 4)

Row 1 $(h_1 = 351.1\,mm)$

- usrt - Upper slab reinforcement in tension $F_{t,As1,Rd} = 533.52\,kN$ $k_{13,1,t} = 2.660\,mm$

Row 2 $(h_2 = 269.1\,mm)$

- lsrt - Lower slab reinforcement in tension $F_{t,As2,Rd} = 184.40\,kN$ $k_{13,2,t} = 1.170\,mm$

Row 3 $(h_3 = 180.1\,mm)$

- cwt - Column web in transverse tension $F_{t,wc,Rd} = 547.76\,kN$ $k_3 = 7.11\,mm$
- cfb - Column flange in bending $F_{t,cfb,Rd} = 339.74\,kN$ $k_4 = 34.66\,mm$
- epb - End-plate in bending $F_{t,epb,Rd} = 186.38\,kN$ $k_5 = 3.59\,mm$
- bwt - Beam web in tension $F_{t,wb,Rd} = 419.46\,kN$ $k_8 = \infty$
- bt - Bolt in tension $2 \cdot F_{t,Rd} = 352.80\,kN$ $k_{10} = 8.46\,mm$

Row 4 $(h_4 = 0\,mm)$

- cwc - Column web in transverse compression $F_{c,wc,Rd} = 699.12\,kN$ $k_2 = 2.59\,mm$
- bfwc - Beam flange and web in compression $F_{c,fb,Rd} = 565.35\,kN$ $k_7 = \infty$

The design resistance moment of the composite joint with full shear connection is determined by analogy to provisions for steel joints given in EN 1993-1-8 §6.2.7 [2] taking account of the contribution of reinforcement as specified in EN 1994-1-1 §8.3.2(2) [8]. The moment plastic resistance of the joint is reached as consequence of the failure of the beam flange and web in compression.

$$M_{Rd} = F_{t,As1,Rd} \cdot h_1 + \left(F_{c,fb,Rd} - F_{t,As1,Rd}\right) h_2 = 195.88\,kNm$$

The rotational stiffness of the composite joint with full shear connection is determined by analogy to provisions for steel joints given in EN 1993-1-8 §6.2.7 [2] taking account of the contribution of reinforcement as specified in EN 1994-1-1 §8.3.2(2) [8].

The general method described in EN 1993-1-8 §6.3.3.1 [2] is applied in order to account for the 3 row in tension. In the considered case the effective stiffness coefficients are calculated based on EN 1993-1-8 §6.3.3.1(2) (6.30) [2].

The rotational stiffness of the joint is calculated as specified in EN 1993-1-8 §6.3.1(4) (6.27) [2].

$$S_{j,ini} = \frac{E_s \cdot z_{eq}^2}{\mu\left(\frac{1}{k_{eff,4}} + \frac{1}{k_{eq}}\right)} = 32741.4\,\frac{kNm}{rad}$$

2.11.4 *Joint type 4, X direction (double sided joint, β=0)*

The components involved in the calculation of the resistance and stiffness of the joint are the same of the Joint type 1 and are illustrated in Figure 2-62. Joints 3 and 4 are double side joints with the beam that is connected on the flange of the column, which differs for the amount of rebars within the effective width. The components which differ from Joint 3 are the upper slab reinforcement in tension (*usrt*) and the lower slab reinforcement in tension (*lsrt*). Only the calculation of these two components is reported in the following. Arrangement of the components is reported in Figure 2-63.

Row 1 $(h_1 = 351.1\,mm)$

- usrt - Upper slab reinforcement in tension $F_{t,As1,Rd} = 454.85\,kN$ $k_{13,1,t} = 2.410\,mm$

Row 2 $(h_2 = 269.1\,mm)$

- lsrt - Lower slab reinforcement in tension $F_{t,As2,Rd} = 61.47\,kN$ $k_{13,2,t} = 0.530\,mm$

Row 3 $(h_3 = 180.1\,mm)$

- cwt - Column web in transverse tension $F_{t,wc,Rd} = 547.76\,kN$ $k_3 = 7.11\,mm$
- cfb - Column flange in bending $F_{t,cfb,Rd} = 339.74\,kN$ $k_4 = 34.66\,mm$
- epb - End-plate in bending $F_{t,epb,Rd} = 186.38\,kN$ $k_5 = 3.59\,mm$
- bwt - Beam web in tension $F_{t,wb,Rd} = 419.46\,kN$ $k_8 = \infty$
- bt - Bolt in tension $2 \cdot F_{t,Rd} = 352.80\,kN$ $k_{10} = 8.46\,mm$

Row 4 $(h_4 = 0\,mm)$

- cwc - Column web in transverse compression $F_{c,wc,Rd} = 699.12\,kN$ $k_2 = 2.59\,mm$
- bfwc - Beam flange and web in compression $F_{c,fb,Rd} = 565.35\,kN$ $k_7 = \infty$

The design resistance moment of the composite joint with full shear connection is determined by analogy to provisions for steel joints given in EN 1993-1-8 §6.2.7 [2] taking account of the contribution of reinforcement as specified in EN 1994-1-1 §8.3.2(2) [8]. The moment plastic resistance of the joint is reached as consequence of the failure of the beam flange and web in compression.

$$M_{Rd} = F_{t,As1,Rd} \cdot h_1 + F_{t,As2,Rd} \cdot h_2 + \left(F_{c,fb,Rd} - F_{t,As1,Rd} - F_{t,As2,Rd}\right) h_3 = 185.07\,kNm$$

The rotational stiffness of the composite joint with full shear connection is determined by analogy to provisions for steel joints given in EN 1993-1-8 §6.2.7 [2] taking account of the contribution of reinforcement as specified in EN 1994-1-1 §8.3.2(2) [8].

The general method described in EN 1993-1-8 §6.3.3.1 [2] is applied in order to account for the 3 row in tension. In the considered case the effective stiffness coefficients are calculated based on EN 1993-1-8 §6.3.3.1(2) (6.30) [2].

The rotational stiffness of the joint is calculated as specified in EN 1993-1-8 §6.3.1(4) (6.27) [2].

$$S_{j,ini} = \frac{E_s \cdot z_{eq}^2}{\mu\left(\frac{1}{k_{eff,4}} + \frac{1}{k_{eq}}\right)} = 30778.7\,\frac{kNm}{rad}$$

2.11.5 *Joint type 5, Y direction (double sided joint, β=0)*

The components involved in the calculation of the resistance and stiffness of the joint illustrated in Figure 2-64 are reported in the follow:

- usrt - Upper slab reinforcement in tension
- lsrt - Lower slab reinforcement in tension
- epb - End-plate in bending EN 1993-1-8 §6.2.6.5 and §6.3.2

- bfwc - Beam flange and web in compression EN 1993-1-8 §6.2.6.7 and §6.3.2
- bwt - Beam web in tension EN 1993-1-8 §6.2.6.8 and §6.3.2
- bt - Bolt in tension

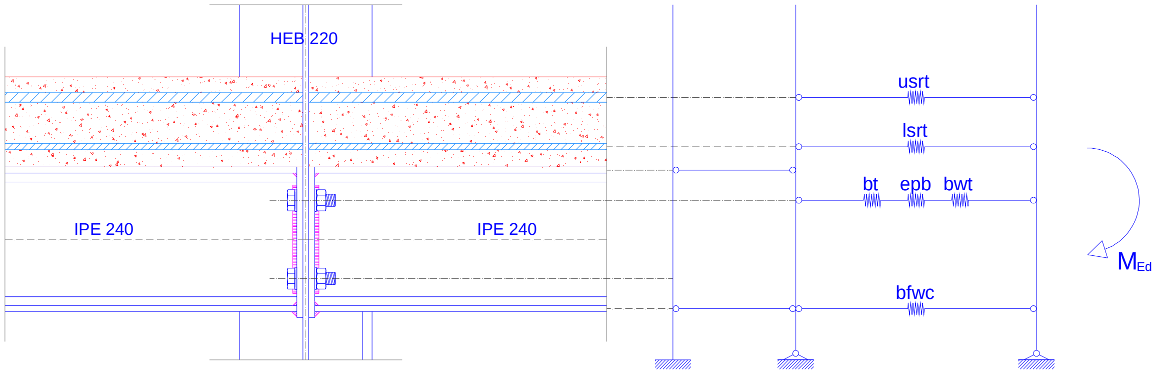


Figure 2-64. Double Side Beam-to-Column Joint connected on column web (Type 5 and 6)

Joints 5 and 6 are double side joints with the beam that is connected on the web of the column. These joints differ from the joints from 1 up to 4 for the absence of the following components:

- cws - Column web panel in shear
- cwc - Column web in transverse compression
- cwt - Column web in transverse tension
- cfb - Column flange in bending

The components which differ from those of Joint 1 are the upper slab reinforcement in tension (*usrt*), the lower slab reinforcement in tension (*lsrt*) and the bolt in tension (*bt*).

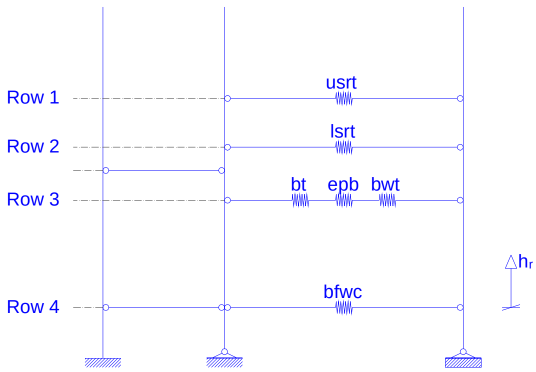


Figure 2-65. Arrangement of the components (Type 5 and 6)

Row 1 $(h_1 = 351.1\,mm)$

- usrt - Upper slab reinforcement in tension $F_{t,As1,Rd} = 454.85\,kN$ $k_{13,1,t} = 2.130\,mm$

Row 2 $(h_2 = 269.1\,mm)$

- lsrt - Lower slab reinforcement in tension $F_{t,As2,Rd} = 61.47\,kN$ $k_{13,2,t} = 0.500\,mm$

Row 3 $(h_3 = 180.1\,mm)$

- epb - End-plate in bending $F_{t,epb,Rd} = 186.38\,kN$ $k_5 = 3.59\,mm$
- bwt - Beam web in tension $F_{t,wb,Rd} = 419.46\,kN$ $k_8 = \infty$
- bt - Bolt in tension $2 \cdot F_{t,Rd} = 352.80\,kN$ $k_{10} = 15.73\,mm$

Row 4 $(h_4 = 0\,mm)$

- bfwc - Beam flange and web in compression $F_{c,fb,Rd} = 565.35\,kN$ $k_7 = \infty$

The design resistance moment of the composite joint with full shear connection is determined by analogy to provisions for steel joints given in EN 1993-1-8 §6.2.7 [2] taking account of the contribution of reinforcement as specified in EN 1994-1-1 §8.3.2(2) [8]. The moment plastic resistance of the joint is reached as consequence of the failure of the beam flange and web in compression.

$$M_{Rd} = F_{t,As1,Rd} \cdot h_1 + F_{t,As2,Rd} \cdot h_2 + \left(F_{c,fb,Rd} - F_{t,As1,Rd} - F_{t,As2,Rd}\right) h_3 = 185.07\,kNm$$

The rotational stiffness of the composite joint with full shear connection is determined by analogy to provisions for steel joints given in EN 1993-1-8 §6.2.7 [2] taking account of the contribution of reinforcement as specified in EN 1994-1-1 §8.3.2(2) [8].

The general method described in EN 1993-1-8 §6.3.3.1 [2] is applied in order to account for the 3 row in tension. In the considered case the effective stiffness coefficients are calculated based on EN 1993-1-8 §6.3.3.1(2) (6.30) [2].

The rotational stiffness of the joint is calculated as specified in EN 1993-1-8 §6.3.1(4) (6.27) [2].

$$S_{j,ini} = \frac{E_s \cdot z_{eq}^2}{\mu \left(\frac{1}{k_{eff,4}} + \frac{1}{k_{eq}} \right)} = 82601.1 \frac{kNm}{rad}$$

Moment resistance of the joint

2.11.6 *Joint type 6, Y direction (double sided joint, β=0)*

The components involved in the calculation of the resistance and stiffness of the joint are the same of the Joint type 5 and are illustrated in Figure 2-64. The components which differ from Joint 5 are the upper slab reinforcement in tension (*usrt*) and the lower slab reinforcement in tension (*lsrt*). Arrangement of the components is reported in Figure 2-65.

Row 1 $(h_1 = 351.1\,mm)$

- usrt - Upper slab reinforcement in tension $F_{t,As1,Rd} = 533.52\,kN$ $k_{13,1,t} = 2.330\,mm$

Row 2 $(h_2 = 269.1\,mm)$

- lsrt - Lower slab reinforcement in tension $F_{t,As2,Rd} = 184.40\,kN$ $k_{13,2,t} = 1.090\,mm$

Row 3 $(h_3 = 180.1\,mm)$

- epb - End-plate in bending $F_{t,epb,Rd} = 186.38\,kN$ $k_5 = 3.59\,mm$
- bwt - Beam web in tension $F_{t,wb,Rd} = 419.46\,kN$ $k_8 = \infty$
- bt - Bolt in tension $2 \cdot F_{t,Rd} = 352.80\,kN$ $k_{10} = 15.73\,mm$

Row 4 $(h_4 = 0\,mm)$

- bfwc - Beam flange and web in compression $F_{c,fb,Rd} = 565.35\,kN$ $k_7 = \infty$

The design resistance moment of the composite joint with full shear connection is determined by analogy to provisions for steel joints given in EN 1993-1-8 §6.2.7 [2] taking account of the contribution of reinforcement as specified in EN 1994-1-1 §8.3.2(2) [8]. The moment plastic resistance of the joint is reached as consequence of the failure of the beam flange and web in compression.

$M_{Rd} = F_{t,As1,Rd} \cdot h_1 + \left(F_{c,fb,Rd} - F_{t,As1,Rd}\right) h_2 = 195.88\,kNm$ Moment resistance EN 1993-1-8 §6.2.7.2 [2].

The rotational stiffness of the composite joint with full shear connection is determined by analogy to provisions for steel joints given in EN 1993-1-8 §6.2.7 [2] taking account of the contribution of reinforcement as specified in EN 1994-1-1 §8.3.2(2) [8].

The general method described in EN 1993-1-8 §6.3.3.1 [2] is applied in order to account for the 3 row in tension. In the considered case the effective stiffness coefficients are calculated based on EN 1993-1-8 §6.3.3.1(2) (6.30) [2].

The rotational stiffness of the joint is calculated as specified in EN 1993-1-8 §6.3.1(4) (6.27) [2].

$$S_{j,ini} = \frac{E_s \cdot z_{eq}^2}{\mu\left(\dfrac{1}{k_{eff,4}} + \dfrac{1}{k_{eq}}\right)} = 96749.0\,\frac{kNm}{rad}$$ Moment resistance of the joint

2.11.7 *Shear components for Joints*

- Bolt in shear

The joints employ 4 bolts where 2 are in traction and 2 works in shear. The shear resistance of the single bolt is calculated as reported in EN 1993-1-8 §3.6.1 Table 3.4 [2]. The shear resistance has been reduced by factor 0,4/1,4 due to tension in bolts.

$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} = 98.00\,kN$ Resistance of the single bolt

$V_{Rd,1} = 252.1\,kN$

- End plate in bearing

The bearing resistance of the end plate is calculated as reported in EN 1993-1-8 §3.6.1 Table 3.4 [2].

$k_1 = \min\left(2.8\frac{e_2}{d_0}/1.7;2.5\right) = 2.118$ $\alpha_b = \min\left(\frac{f_{ub}}{f_u};1\right) = 1$

$F_{b,Rd} = \frac{k_1 \alpha_b f_u dt}{\gamma_{M2}} = 172.83\,kN$

$V_{Rd,2} = 691.3\,kN$

- Column flange in bearing

The bearing resistance of the column flange is calculated as reported in EN 1993-1-8 §3.6.1 Table 3.4 [2].

$k_1 = \min\left(2.8\frac{e_2}{d_0}/1.7;2.5\right) = 2.5$ $\alpha_b = \min\left(\frac{f_{ub}}{f_u};1\right) = 1$

$F_{b,Rd} = \frac{k_1 \alpha_b f_u dt}{\gamma_{M2}} = 326.40\,kN$

$V_{Rd,3} = 1305.60\,kN$

- Shear resistance of joints

The shear resistance of the joint is equal to the shear resistance of the weakest component.

$V_{Rd} = 252.1\,kN$

2.11.8 *Joint design for the Symmetric Configuration - ULS*

- *Joint type 1 - X direction (single side joint, β=1)*

The maximum bending moment and the maximum shear force on the joint type 1 of the *Symmetric* structure are respectively $M_{Ed} = 4.68\,kNm$ and $V_{Ed} = 124.46\,kN$.

$M_{Rd} = 179.52\,kNm > M_{Ed}$

$V_{Rd} = 252.1\,kN > V_{Ed}$

- *Joint type 2 - X direction (single side joint, β=1)*

The maximum bending moment and the maximum shear force on the joint type 2 of the *Symmetric* structure are respectively $M_{Ed} = 5.87\,kNm$ and $V_{Ed} = 56.23\,kN$.

$M_{Rd} = 170.54\,kNm > M_{Ed}$

$V_{Rd} = 252.1\,kN > V_{Ed}$

- *Joint type 3 - X direction (double side joint, β=0)*

The maximum bending moment and the maximum shear force on the joint type 3 of the *Symmetric* structure are respectively $M_{Ed} = 98.05\,kNm$ and $V_{Ed} = 145.68\,kN$.

$M_{Rd} = 195.88\,kNm > M_{Ed}$

$V_{Rd} = 252.1\,kN > V_{Ed}$

- *Joint type 4 - X direction (double side joint, β=0)*

The maximum bending moment and the maximum shear force on the joint type 4 of the *Symmetric* structure are respectively $M_{Ed} = 33.89\,kNm$ and $V_{Ed} = 54.81\,kN$.

$M_{Rd} = 185.07\,kNm > M_{Ed}$

$V_{Rd} = 252.1\,kN > V_{Ed}$

- *Joint type 5 - Y direction (double side joint, β=0)*

The maximum bending moment and the maximum shear force on the joint type 5 of the *Symmetric* structure are respectively $M_{Ed} = 41.42\,kNm$ and $V_{Ed} = 56.11\,kN$.

$M_{Rd} = 185.07\,kNm > M_{Ed}$

$V_{Rd} = 252.1\,kN > V_{Ed}$

- *Joint type 6 - Y direction (double side joint, β=0)*

The maximum bending moment and the maximum shear force on the joint type 5 of the *Symmetric* structure are respectively $M_{Ed} = 109.18\,kNm$ and $V_{Ed} = 141.51\,kN$.

$M_{Rd} = 195.88\,kNm > M_{Ed}$

$V_{Rd} = 252.1\,kN > V_{Ed}$

2.11.9 *Joint design for the Asymmetric Configuration - ULS*

- *Joint type 1 - X direction (single side joint, β=1)*

The maximum bending moment and the maximum shear force on the joint type 1 of the *Asymmetric* structure are respectively $M_{Ed} = 16.63\,kNm$ and $V_{Ed} = 134.56\,kN$.

$M_{Rd} = 179.52\,kNm > M_{Ed}$

$V_{Rd} = 252.1\,kN > V_{Ed}$

- *Joint type 2 - X direction (single side joint, β=1)*

The maximum bending moment and the maximum shear force on the joint type 2 of the *Asymmetric* structure are respectively $M_{Ed} = 11.29\,kNm$ and $V_{Ed} = 62.10\,kN$.

$M_{Rd} = 170.54\,kNm > M_{Ed}$

$V_{Rd} = 252.1\,kN > V_{Ed}$

- *Joint type 3 - X direction (double side joint, β=0)*

The maximum bending moment and the maximum shear force on the joint type 3 of the *Asymmetric* structure are respectively $M_{Ed} = 104.31\,kNm$ and $V_{Ed} = 149.92\,kN$.

$M_{Rd} = 195.88\,kNm > M_{Ed}$

$V_{Rd} = 252.1\,kN > V_{Ed}$

- *Joint type 4 - X direction (double side joint, β=0)*

The maximum bending moment and the maximum shear force on the joint type 4 of the *Asymmetric* structure are respectively $M_{Ed} = 42.10\,kNm$ and $V_{Ed} = 63.86\,kN$.

$M_{Rd} = 185.07\,kNm > M_{Ed}$

$V_{Rd} = 252.1\,kN > V_{Ed}$

- *Joint type 5 - Y direction (double side joint, β=0)*

The maximum bending moment and the maximum shear force on the joint type 5 of the *Asymmetric* structure are respectively $M_{Ed} = 65.32\,kNm$ and $V_{Ed} = 74.25\,kN$.

$M_{Rd} = 185.07\,kNm > M_{Ed}$

$V_{Rd} = 252.1\,kN > V_{Ed}$

- *Joint type 6 - Y direction (double side joint, β=0)*

The maximum bending moment and the maximum shear force on the joint type 5 of the *Asymmetric* structure are respectively $M_{Ed} = 149.88\,kNm$ and $V_{Ed} = 173.56\,kN$.

$M_{Rd} = 195.88\,kNm > M_{Ed}$

$V_{Rd} = 252.1\,kN > V_{Ed}$