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List of Appendices

1. Introduction and General Considerations in Conceptual Design

In this report, a simple multi-storey building will be designed in accordance with the brief. Some general considerations in conceptual design will be summarized in this section.

Brief introduction of the two proposals will be given in Section 2. The two proposals will be compared at the end of the section. Sections 3 to 7 will provide the detailed calculations of composite slabs, composite beams, steel columns, wind bracing and cantilever truss system. Some assumptions and basic design considerations will also be given in the corresponding sections. Sections 8 to 12 will give the design of all the connections. At last, a parametric analysis summarizing some aborted proposals and some better proposals to be studied in the future will be shown in Section 13. A conclusion, reference list and all the appendices enclosing all the typical drawings and calculation spreadsheets will also be included at the end of the report.

In this section, some general design considerations and a general design procedure, illustrated using the example of this project, will be given.

1.1. General Design Considerations

First of all, the client's requirements and site constraints need to be taken into account during the design. This is because the requirements are the primary aims of the building design. Without meeting the design requirements, the building designed may not be meet the original planned functions. On the other hand, site constraints provide all the limitations to the design, which ensure the design proposals to be viable and able to construct on site. Furthermore, some construction issues need to be considered to minimize any construction problems from happening during the construction stage, so that the proposals can be put into construction.

Technically speaking, it is also needed to meet all the requirements from code of practice. This is because it is important to ensure the building to have met all the technical requirements, such that the safety and comfort of people inside the building can be safeguarded by fulfilling all the Ultimate Limit State and Serviceability Limit State requirements.

In addition, with the growing importance of achieve carbon net zero, the design also needs to be sustainable. To achieve a sustainable design, the proposed design should have economic, social and environmental benefits.

For economic benefits, it can be achieved by optimizing the design. This is because an optimal design can help reduce material costs. Some measures to help reduce the

construction difficulties can also reduce construction costs or time. If the construction programme can be benefited, it can also reduce the construction costs. Other than the material and construction costs, operation costs also need to be considered. Owing to this, design minimizing the operation costs can also be considered. For instance, the operation costs may be controlled by the using of energy-efficient facilities, adopting proper insulation measures and by many other means. In order to achieve it, some innovative technologies, such as the use of Building Information Modelling (BIM), can be used. For example, Project Information Model and Asset Information Model can be built to model the building from cradle to grave, so that the overall performance can be modelled.

In terms of social benefits, it depends on the nature of the project in order to propose a scheme which can benefit the most in the society. In environmental aspect, they can be considered by using more sustainable materials, insulation as mentioned and adopting some green measures, such as green roof, in the building.

By considering the above considerations, the best proposal can be designed.

1.2. Design Procedures

Typical design procedures will be provided in this section by using this project as an example.

1.2.1. Initial Design Considerations

Initially, all the constraints and design requirements must be gathered before formulating any viable scheme. As mentioned in the earlier subsection, the principle of building design is to achieve the most sustainable and optimum design while meeting all the client's and technical design requirements.

For example, there is an old Victoria tunnel located at the north-west part of the building. Owing to this, no columns can be installed to the ground at the region. In addition, there are also requirements on span of beams and fire requirements. Without understanding all the site constraints and requirements, viable schemes will not be able to be formulated.

1.2.2. Grid Selection and Element Sizing

The second step is grid selection and element sizing. This step is very important as it determines the spacing of different elements and the size of elements to be used. Should the span between beams and columns be designed to be far apart, the loading acting on each element may be larger. Therefore, a larger member size is required

to provide sufficient capacity in terms of strength, buckling and deflections.

The grid selection is also affected by the architectural layout. For example, an open office may require a larger common area in the middle. It is desirable if the column spacing can be maximized, so that the effective area which is available for use can be larger. Similarly, some clients may also want to maintain a minimum headroom. In this design, a minimum headroom of 2.90 metres needs to be maintained. Therefore, it also affects the thickness of slabs and beams to be adopted.

In order to achieve the most optimum performance, some general rules of thumbs can be considered. This is because the longer the span between elements, the capacity required for each element needs to be larger. Therefore, the overall material cost may be higher. However, reducing the span may also increase the number of elements to be used. Therefore, it is good to find an optimum solution, which can balance the benefits and drawbacks from both sides. The tables below show the general golden rules which may be adopted in building design.

Table 1.1 Span to depth ratios for different beam solutions. Retrieved from Table 2 of The Design of a New Administrative Building for the University of Guyana at the Turkeyen Campus by Lewis, p.25.

Non-composite primary beams	Floor = span/20 Roof = span/25
Non-composite secondary beams	Floor = span/25 Roof = span/30
Composite beams	Span/16 to span/18 (Note depth is steel beam plus slab) Long span solutions tend to be shallower, up to span/20

Table 1.2 Typical structural depths (floor to ceiling). Retrieved from Table 3 of The Design of a New Administrative Building for the University of Guyana at the Turkeyen Campus by Lewis, p.25.

Flooring system	Target floor depth (mm)
Composite beam construction	800 – 1,200
Cellular beams (with service integration)	800 – 1,100
Downstand beams with precast concrete floor slabs	1,200 – 1,450
Shallow floor or integrated beams	600 – 800

Table 1.3 Typical column sizes for small and medium span composite floors. Retrieved from Table 4.1 of Steel Buildings in Europe, p.2-30.

Number of floors supported	Universal Column (UC) Height h
1	152
2 - 4	203
3 - 8	254
5 - 12	305
10 - 40	356

1.2.3. Load Calculation

Before designing the elements, the actions acting on individual element should be calculated by studying the load path. According to the load path of the building in this brief, vertical dead loads and imposed from slabs are transferred to secondary beams. Due to the minimum requirements on the span of secondary beams, the composite slabs are all designed as one-way slabs as the ratio between the length and width of the slab exceeds 25. Owing to this, the loads from slabs are designed to be transferred to secondary beams. Further transfer of loads to columns from secondary beams via primary beams are considered during the design of primary beams and columns. At the cantilever region near the north-west corner of the building, vertical loads are transferred via the trusses to the adjacent columns.

For lateral loads, loads are transferred via the sets of wind bracing to the columns. The bracing also enhances the lateral stability of the frame. Equivalent horizontal force (EHF) and amplification factor can then be calculated to determine the amplified lateral wind load and the EHF as appropriate.

1.2.4. Design of Elements

The design of members is then carried out as per the loads calculated. In this project, the members are mainly designed in accordance with Eurocodes 3 and 4. For example, cross-section and buckling resistance should be checked for columns under ULS. For SLS, deflection under loading for beams may be critical in design. Welds and bolted connections should also be designed properly in accordance with BSEN1993-1-8.

1.2.5. Other Considerations

Sensitivity to Sway

For multi-storey steel buildings, the control of sway is important as the deflection

is large. In general, there are three ways to alleviate the situation:

(i) Reduce the deflection by increasing the member size as stiffness increases with the member size. However, it is not desirable as it increases the use of materials.

(ii) Amplify first-order moments and forces. It is only applicable for α_{cr} is between 3 and 10.

(iii) Conduct a second-order analysis to account for secondary effects explicitly. It requires more sophisticated modelling techniques and it is usually adopted when α_{cr} is smaller than 3, in which the situation is very critical.

Fire

Fire protection is very important as steel degrades when temperature increases. Therefore, fire protection can be provided as per the requirements set out in BSEN1993-1-2 to ensure the temperature of steel members, especially columns, does not reach a critical temperature before the fire protection period. It is also important to design effective means of escape as per the UK Government Buildings Regulations. More details will be provided in Section 2 during the design of services.

2. Design Proposals

According to the coursework requirements limited by Eurocode and UK National Annex, a physical, conceptual, and analytical model was created for the proposed building design to better visualize and interpret the design challenges at hand.

For structural design, SAP2000 was used to determine the general configuration and section properties of primary beams, secondary beams, wind bracing, truss for cantilever.

In the detailed check, it was also used to check the resistance of truss and bracing. (N.B. Isolated SAP2000 Truss Modelling Analysis will be presented in the truss chapter)

In this report, not only the details of the two confirmed proposals will be discussed, aborted proposals and some improvements to the two proposals which are worth investigating will also be mentioned.

For architectural and detailing modeling, Autodesk Revit 2014 or Twinmotion were used to create the various models. The following figures highlight some of the many model views created to gain insight into the problem.

2.1. Coursework Requirements

Some coursework requirements stated in the brief are shown below:

2.1.1. Design Requirements

There are a couple of design requirements that would need to be fulfilled. The requirements can be generally divided into three categories:

Restrictions to Column Arrangement

According to the coursework brief, there is an old Victorian tunnel running across the northwest corner of the site. Therefore, no columns can be placed over the specified region and the part of the structure needs to cantilever over the tunnel.

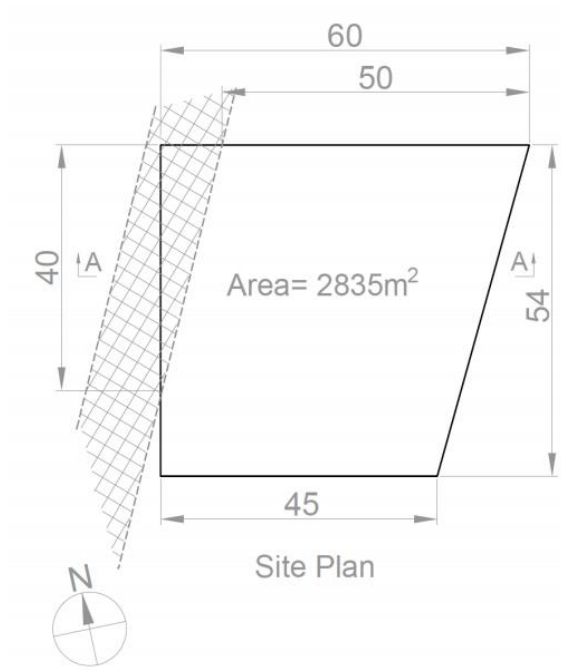


Figure 2.1 Site Plan requirement. Retrieved from SIMPLE MULTI-STOREY BUILDING DESIGN coursework by Louca, 2022.

Floor Area and Headroom Requirements

In order to maximize the effective area for use, the column spacing for most secondary beam spans is limited to at least 12m. Therefore, the span of the secondary beam is larger than that of the primary beam. In addition, around 15% to 20% of the floor area needs to be reserved for services, such as lifts and toilets. Minimum headroom of 2.9m between floor to ceiling also needs to be maintained in the design.

Services Design Requirements

The design of lifts and services were designed based on the brief and the HM Government Approved Document B on fire safety requirements of The Building Regulations 2010. The design aims at minimising the area used by lifts and services, while fulfilling all the requirements stated in statutory regulations and brief. Therefore, the effective area to be used can be maximised.

According to the brief, the net floor area is around 80% of the gross floor area, which is around is 2268m^2. Assuming an occupancy level of one person per 10m^2 of net floor area, the number of occupants is around 227 persons per floor. As the building is designed as an open area without individual compartments, so the fire safety requirements can be given as follows:

Minimum Number of Escape Routes from a storey: 2 nos. (Table 2.2 of Approved Document B, HM Government, 2010)

Minimum Width of Escape Routes: The requirement is 5mm per person, i.e. 1135mm (Table 2.3 of Approved Document B, HM Government, 2010). However, the width is taken as 1750mm in design for better evacuation efficiency.

Limitations on Travel Distance: 45m as more than one direction of escape routes are provided (Table 2.1 of Approved Document B, HM Government, 2010)

Width of Final Exit: 2m (Section 2.23 of Approved Document B, HM Government, 2010)

Firefighter Lift: 2.5m x 2.2m per floor

For lifts and services, they are provided as follows:

Number of Lifts: 8 16-person lifts (From Assignment Brief)

Size of Lift Shaft: 2.6m x 3.3m each

Size of Service Cores: 5m x 2.5m per floor

Toilet: 7.5m x 3.5m per floor

By incorporating all the requirements given above, the architectural layout plan has been provided as shown in Appendix A1.

Other Design Requirements

Other design requirements include fire protection of at least 1.5 hours and the use of composite floors.

2.1.2. Design Assumptions

Simple design assumption

While the effects of joint behavior on the distribution of internal forces and moments inside a structure and the overall deformations of the structure are often disregarded (depending on the connection way of the structure). However, they should be considered when they are considerable (for instance, in the case of semi-continuous joints).

In this project, simply supported connections (pinned connection) should be implemented. According to EC3, clause.5.1.2(2), the definition for simply

connection is that the moment does not transfer through the joints.

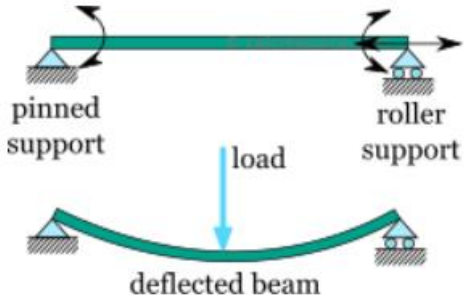


Figure 2.2 simply supported the diagram. Retrieved from calcresource. 2022. *Simply supported beam diagrams: article*. [online] Available at: <https://calcresource.com/statics-simple-beam-diagrams.html> [Accessed 27 March 2022].

Diaphragm Stiffness Assumptions

A diaphragm is a horizontal structural component that distributes gravity loads to vertical components by passing storey shears and torsional moments to force-resisting materials on the lateral side. The relative stiffness of the diaphragm in comparison to the stiffness of the lateral parts at the diaphragm level determines how it transmits shears and torsional moments. Additionally, it disperses gravity loads in a one- or two-directional way. To accomplish this goal, lateral analysis diaphragm motion may be characterized as Rigid, Semi-rigid, Pseudo-flexible, or Flexible/None. In gravity analysis, the kind of deck used inside the diaphragm determines how gravity loads are distributed on the deck.

This project is mainly a steel frame building project, according to Dr.Louca's suggestion, for simplicity, the stiffness of diaphragms should not be considered. As a result, the diaphragm prevents frames from interacting, and each frame in the building may translate independently of the others. This is maybe the simplest structural analysis since no element or constraint representing the diaphragm is included.

Therefore, flexible diaphragms assumption is used in this project.

(Reference: Analysis of Buildings with Rigid, Semirigid and Pseudo-Flexible Diaphragms by Bulent and Rakesh)

2.1.3. Required Loading Conditions

According to the brief, the loads to be considered are as follows:

Table 2.1 Loads need to be considered in this project

Roof		**Floor**	
Dead Load		*Dead Load*	
Topping	1 kPa	Tiles (screed)	0.7 kPa
Slab	Depends on material	Slab	Depends on material
Steel	0.2 kPa	Steel	0.3 kPa
Ceiling	0.5 kPa	Partitions	1 kPa
Services	1 kPa	Services	1.5 kPa
		Ceiling	0.5 kPa
Dead Load of beams and columns are also considered			
Imposed Load	1.5 kPa	*Imposed Load*	4 kPa
Other Loads			
Wind Load	1.3 kPa		
Cladding	1.3 kPa (for perimeter beams)		

The following load combinations have been considered in the design:

Load Case 1: $1.35G_k+1.50Q_k+EHF$

Load Case 2: $1.35G_k+1.50Q_k+0.75W_k+EHF$

Load Case 3: $1.35G_k+1.05Q_k+1.50W_k+EHF$

Load Case 4: $1.0G_k+1.50W_k+EHF$

2.2. Proposal 1

The details for Proposal 1 are summarized below.

2.2.1. Global View and Dimensions of Structure

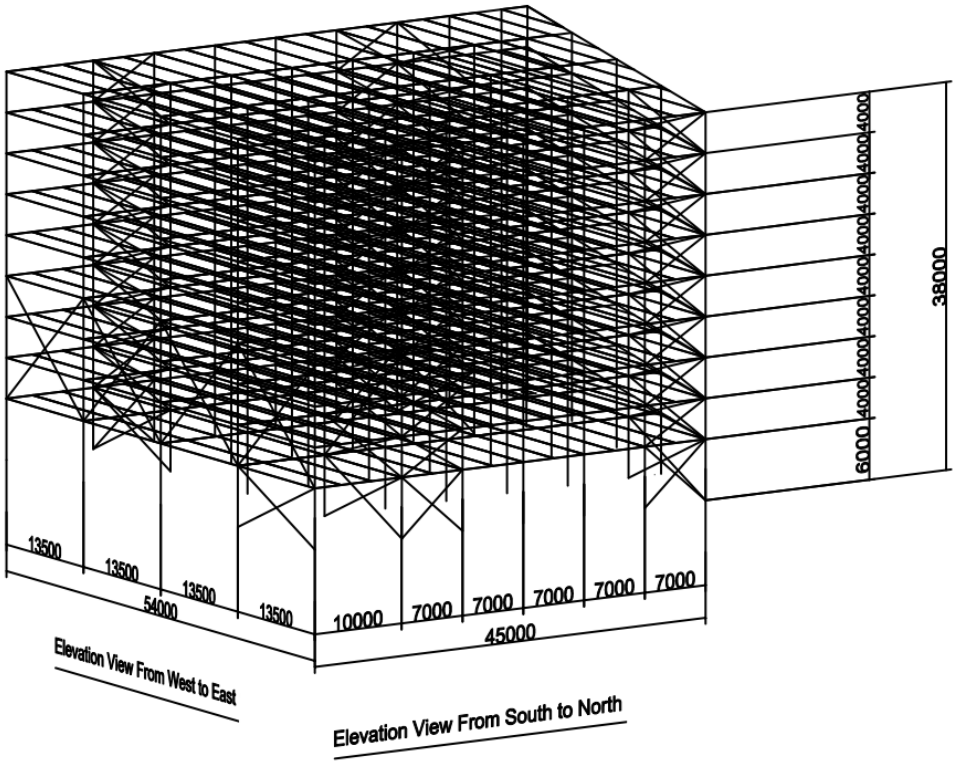


Figure 2.3 An overall View for Proposal one with dimensions

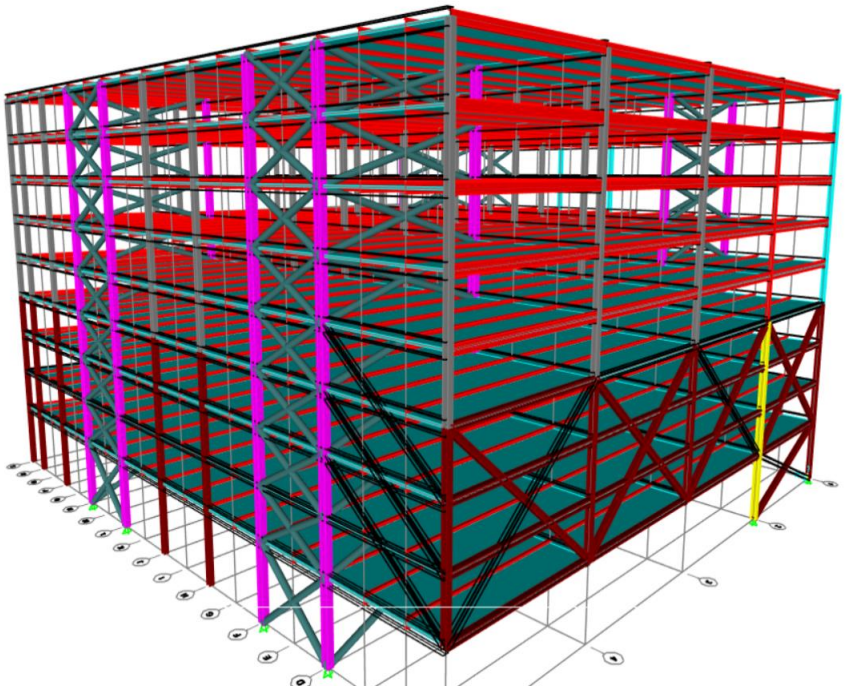


Figure 2.4 Global Structural Model for Proposal 1 (SAP2000)

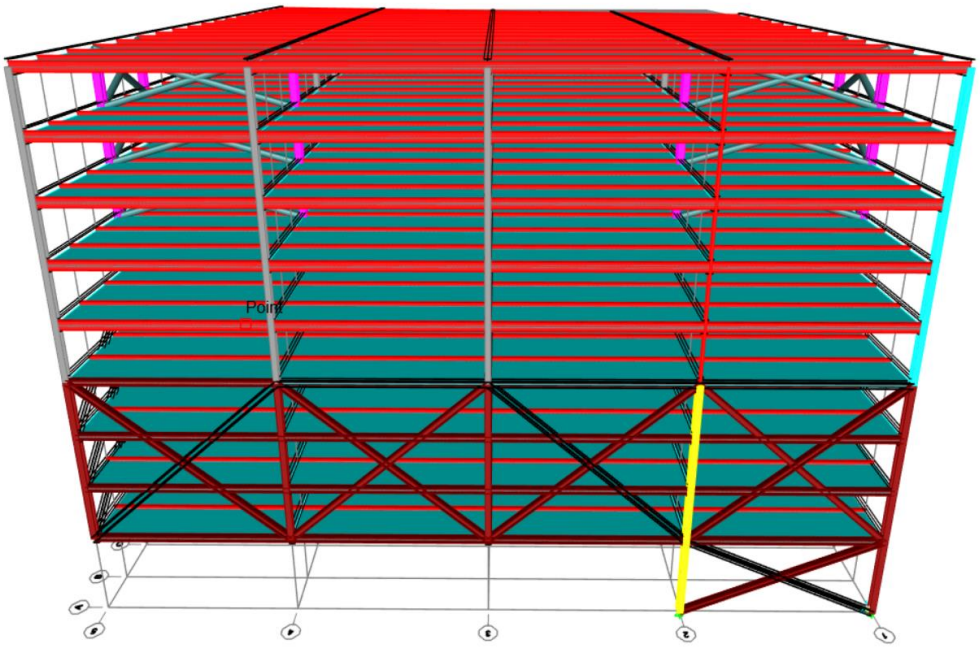


Figure 2.5 Global Structural Model for Proposal 1 (SAP2000)

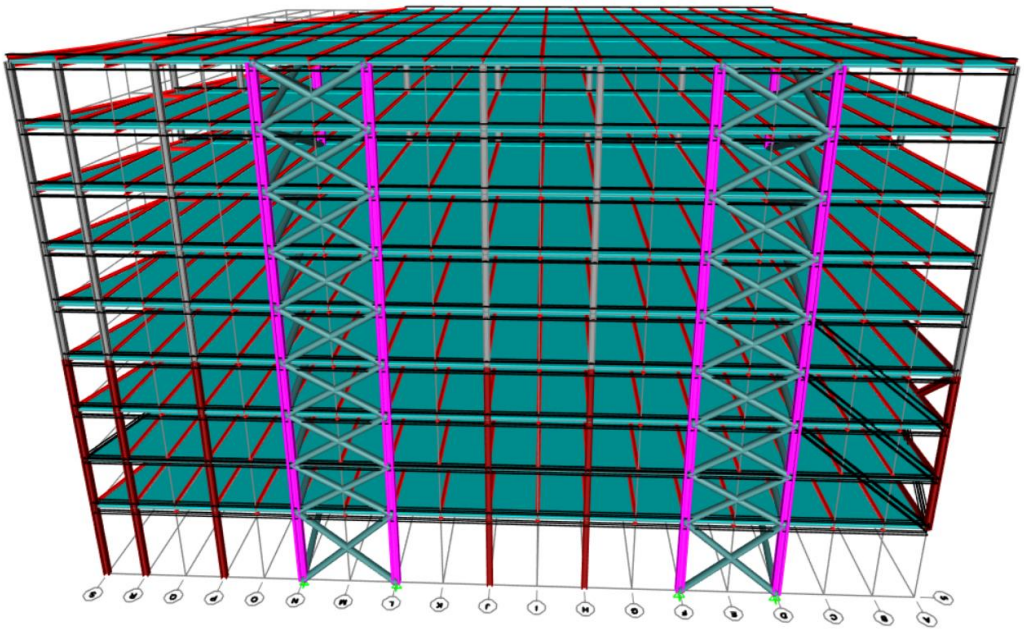


Figure 2.6 Elevation View from West to East for Proposal 1 (SAP2000)

2.2.2. Column Grid Arrangement and Cross-section in Proposal 1

For making sure that each junction between the primary beams and edge beams will have columns. For more efficient use of steel, most of the columns are using variable cross-section according to the level of floor (the lower level of the floor, the higher axial force will be, hence, larger area of cross-section required).

Based on our calculation both in SAP2000 and by hand calculation, it can be found that the columns located in the cantilever regions will tend to have higher axial forces.

The columns for wind bracing bays are isolated to be analyzed by hand calculation and show that the biggest area of a steel cross-section is required. In the region far from the cantilever region tends to be fewer axial forces, as a result, a smaller area of cross-section is required.

Table 2.2 Column Cross-section

Location of the Columns	Cross-section Selection (UKC)	Represented Colors in SAP2000
Typical Internal Columns from 4th Floor to Roof	356×368×202	
Typical internal Columns from 4th Floor to Ground	356×406x467	
Typical Peripheral Columns from 4th Floor to Roof	356×368× 202	
Typical Peripheral Columns from 4th Floor to Ground	356×406×235	
Columns for Wind Bracing Bays from Roof to Ground	356*406*1202	
Columns near the cantilever region from 4th Floor to Ground	356×406×634	

Column Grid Arrangement

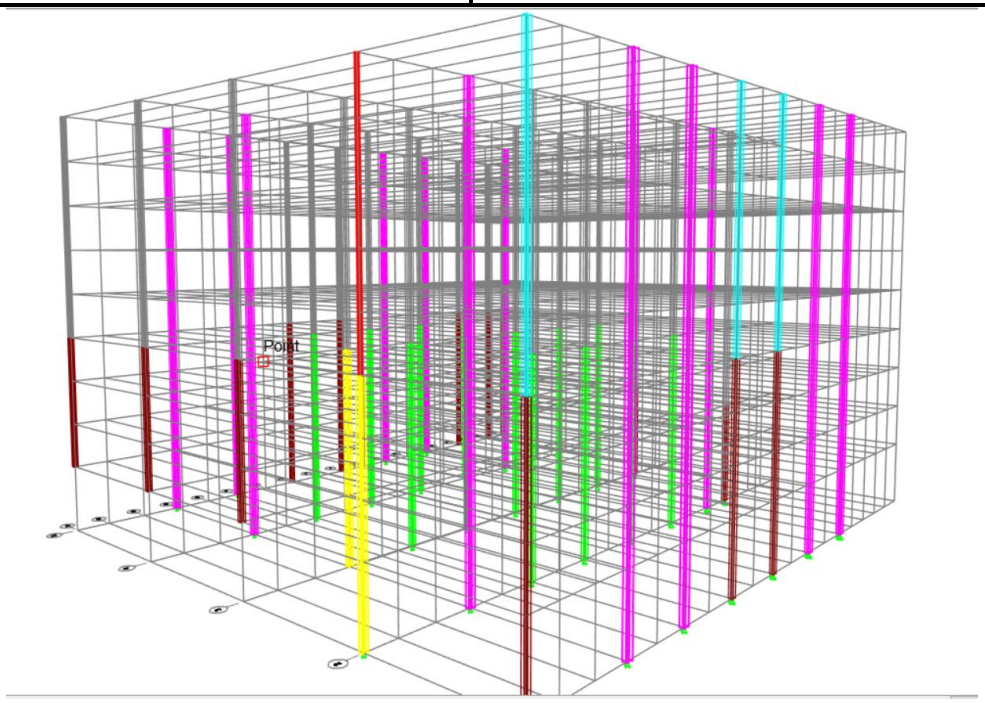


Figure 2.7 An overview for the layout of Columns (SAP2000)

Details of the column size at different locations of the building can be referred to Appendix A4.

2.2.3. Beam Selection

Primary and secondary beam arrangement

Based on the requirement of the coursework, for most of the secondary beams, the column spacing should be bigger than 12m, and the span of primary beams could be less than 12m. As general consideration mentioned before, the arrangement of beams should try to be as uniform as possible. By running and calculating many possible layouts of primary beams to analyze that efficiency (it will be discussed in the possible alternative solutions in Section 13). Also, for satisfying the requirement for cantilever area, uniform spans with 13.5m are chosen to be used.

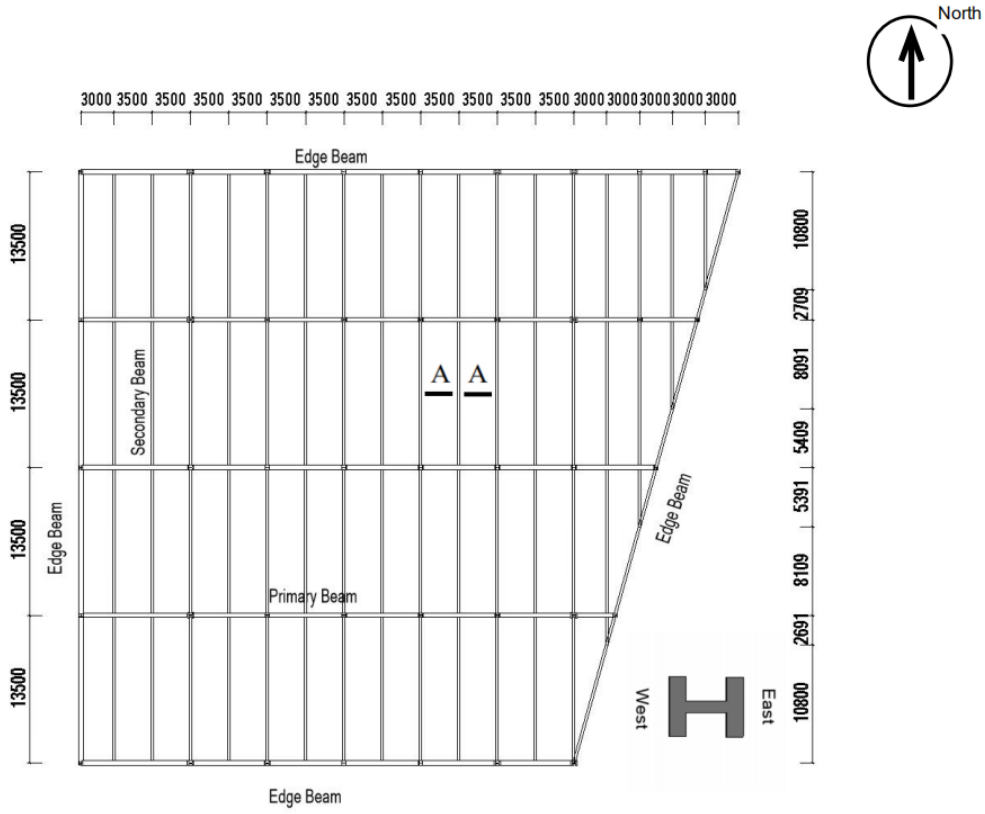


Figure 2.8 Beam Arrangement with critical section (Revit)

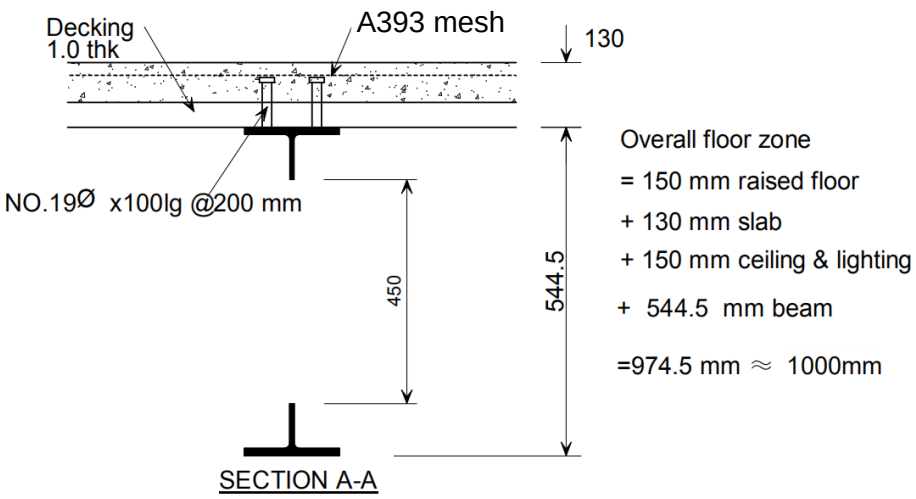


Figure 2.9 Details with critical section A-A

Beam Cross-section

Table 2.3 Cross-Section selection and represented colors in SAP2000

Location of the Columns	Cross-section Selection (UKB)	Represented Colors in SAP2000
Primary Beam from level 1 to Level Roof	533×210×122	
Secondary Beam from level 1 to Level Roof	533×210×122	
Edge Beam from level 5 to Level Roof	533×210×122	
Edge Beam from level 1 to Level 4	Depends on Truss	

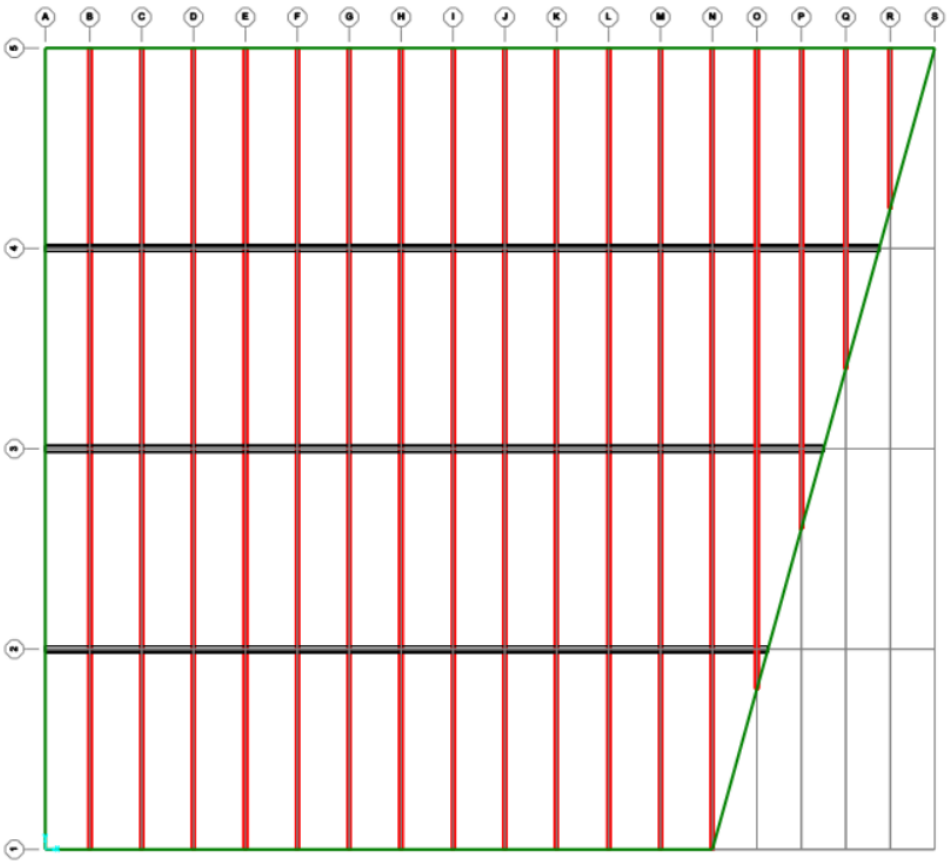


Figure 2.10 Primary Beam Arrangement

2.2.4. Bracing

The structure's braced frame system is generally composed of truss members that serve as bracing components. These bracings are often employed to reinforce constructions that are susceptible to lateral stresses. They resist lateral stresses primarily via the compression or tension of the brace elements. As a result, the bracing system is very effective in resisting lateral stresses. Additionally, the braced frame method is efficient since it stiffens the structure laterally. With the least amount of material added to the frame, it provides an economically sound structure for any height.

For steel bracing system, various kinds of sections, such as channel sections, angle sections, I section, and tubular sections, may be utilized in steel braces. In our case, proposal one's bracing configuration is shown below. For resisting both X and Y direction horizontal forces, the bracing system on both sides needs to be provided. In addition, a symmetrical layout is considered for easy calculations.

Besides, due to the 'simple design approach is used in this project, the connection design is a simple one with do not transfer moment. Meanwhile, considering the connection difficulty and lower bracing efficiency when bracing cross more than one floor, X bracing layer-by-layer is chosen for our design.

In terms of the design of cross-section bracing, three types of imperfections, axial forces, and lateral deflection are considered. Meanwhile, lateral deflection is the most critical factor in our design.

When analyzing the horizontal forces in different forces combination, even the forces from North to South in the load combination is the biggest one, but the provided bracing in the West to East direction is shorter than that of North to the South due to the less second moment of the area will be provided. For this reason, North to South direction is the most critical one.

Cross-section Properties for Bracing Systems

Table 2.4 Cross-section Properties for Bracing Systems

Location of the Columns	Cross-section Selection
Columns for Bracing	533×210×122
X-Bracing	533×210×122
Beams in Bracing	533×210×122(UKB)

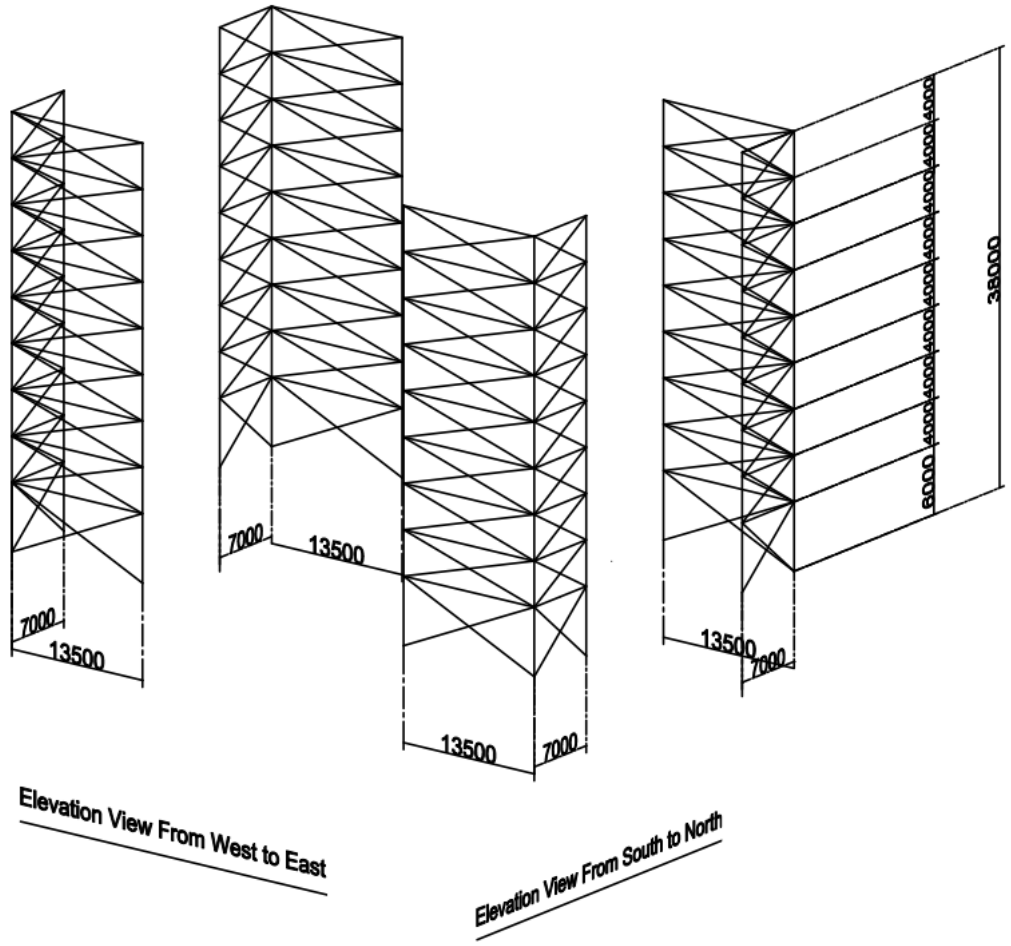


Figure 2.11 3D View for Global Bracing System in Proposal 1 (Autodesk CAD)

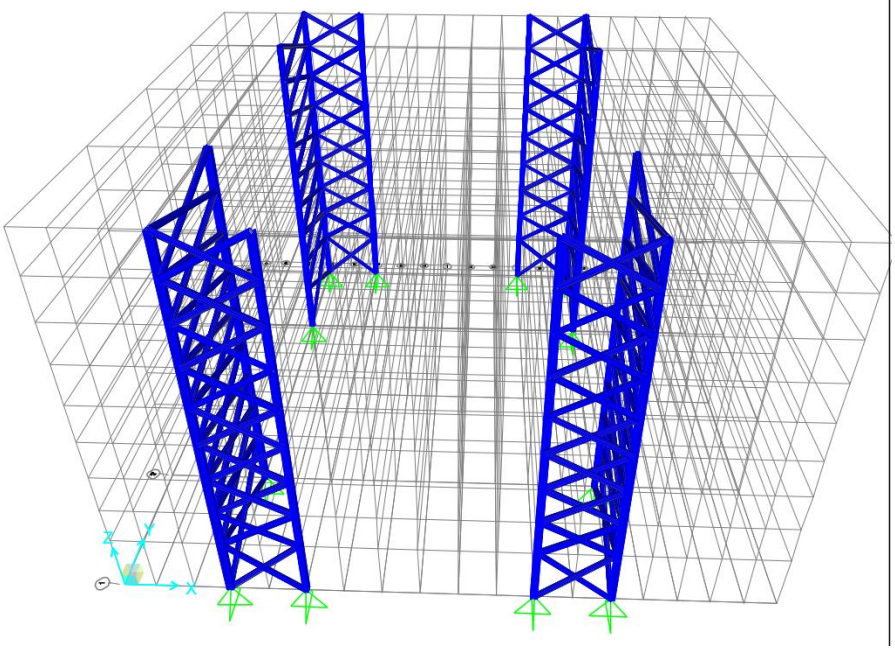


Figure 2.12 Global layout of the bracing system (in SAP2000)

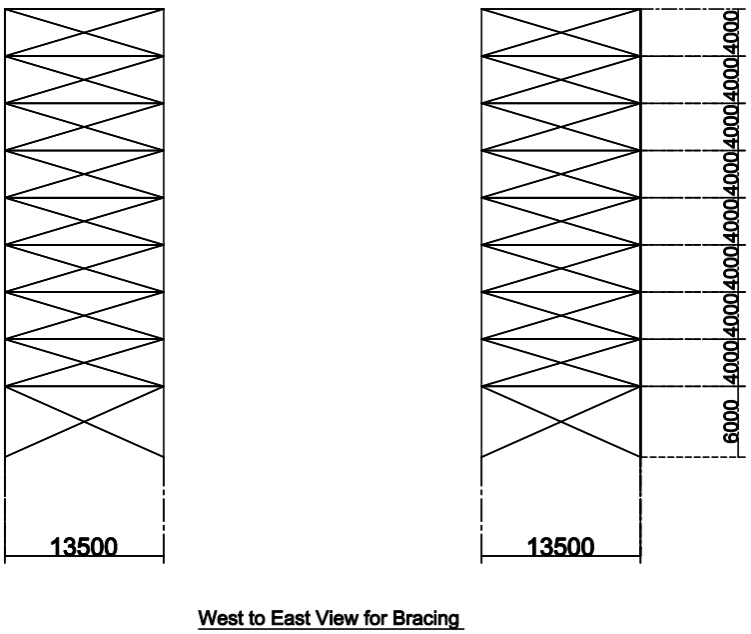


Figure 2.13 Elevation View for Bracing from West to East (Autodesk CAD)

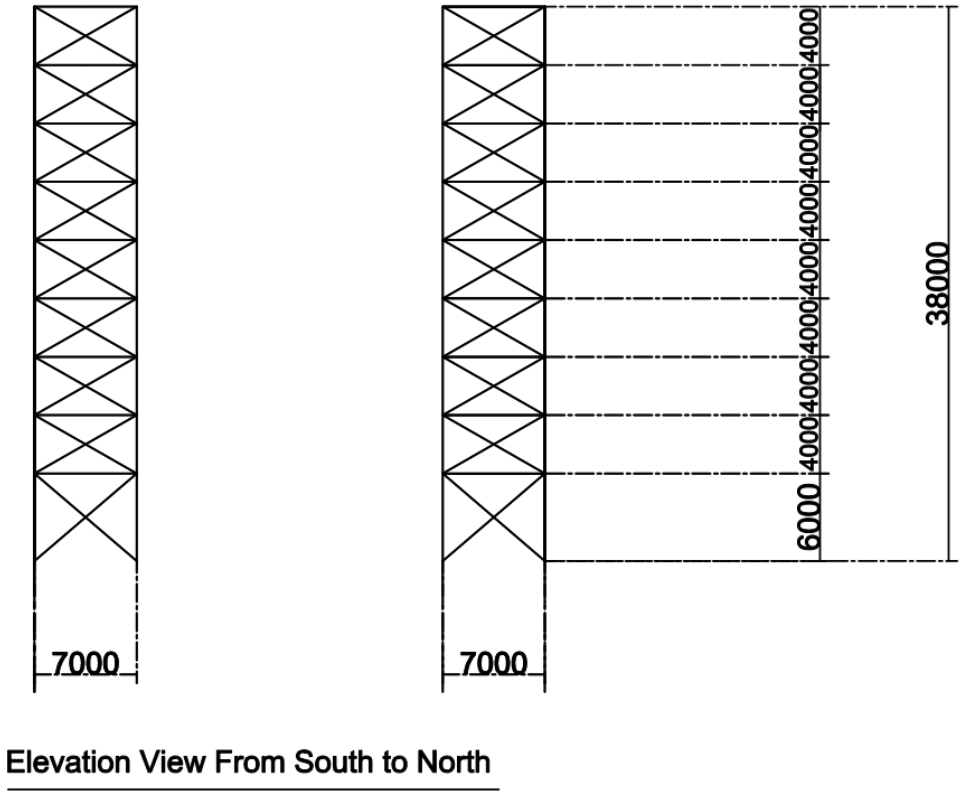


Figure 2.14 Elevation View for Bracing from South to North (Autodesk CAD)

2.2.5. Cantilever Region Design

For proposal 1, our detailed cantilever design is shown below, and the axial forces are shown by red and blue bars. The magnitude of axial forces varies with the sizes the bars and members in tension and compression are shown by colors of blue and red respectively.

X bracing across three floors in the west face and the three truss members in the north face are employed to resist vertical and horizontal forces.

On the North-West side of the cantilever region, vertical loads transferred from upper floors will transfer to our truss system through columns, as it is shown below, most forces will transfer to the first edge columns and hence very high compressive forces. More details will be discussed in the truss section.

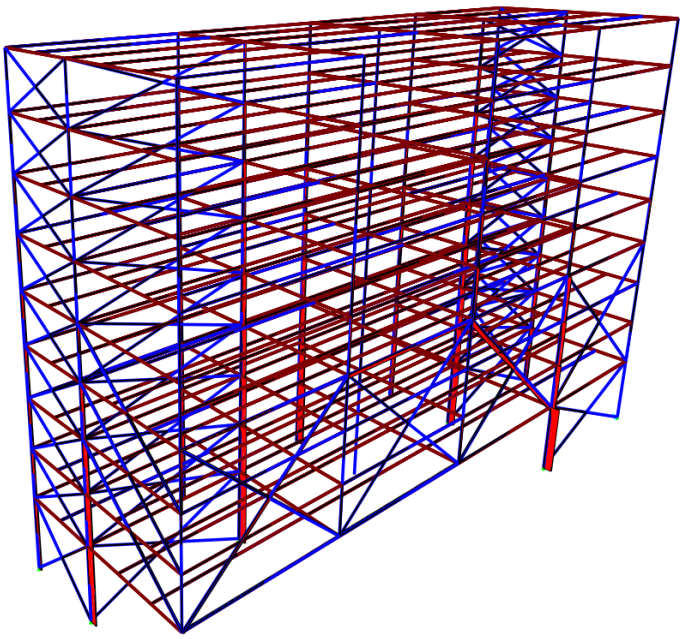


Figure 2.15 Isolated cantilever region

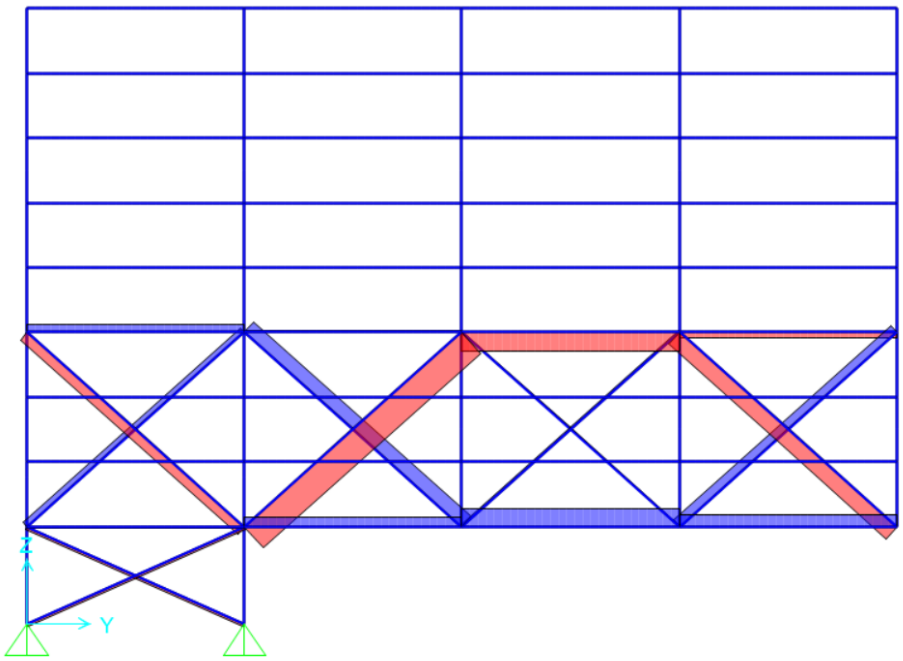


Figure 2.16 Cantilever region Loading path from West to East

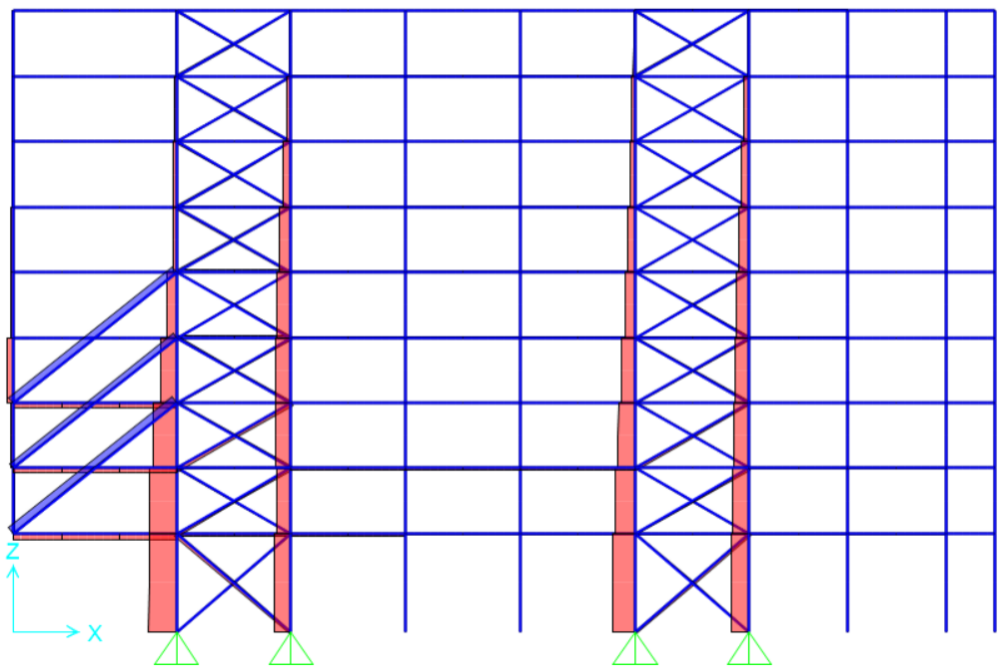


Figure 2.17 Cantilever Region View from North to South

2.2.6. Overall Tonnage of Proposal 1

Table 2.5 The overall self-weight of steel of S355

Section	Unit	Total length (m)	Total weight (kN)
UKB(533×210×122) Primary	43	2257500	8596
UKB(533×210×122) Secondary	202	8116597	11147
UKC356*406*235	41	408647	942
UKC356*368*202	50	536000	1187
UKC356*406*467	26	542861	2487
UKC356*406*634	2	36000	223
356*406*1202	44	456000	5373
CHS 508*14.2	144	1609055	2728
Total Self-Weight			**32686**

2.3. Proposal 2

In proposal 2, the layout of columns and beams remain the same, but the layout for bracing and truss in the cantilever region are relocated.

2.3.1. Global View and Dimensions for the Structure

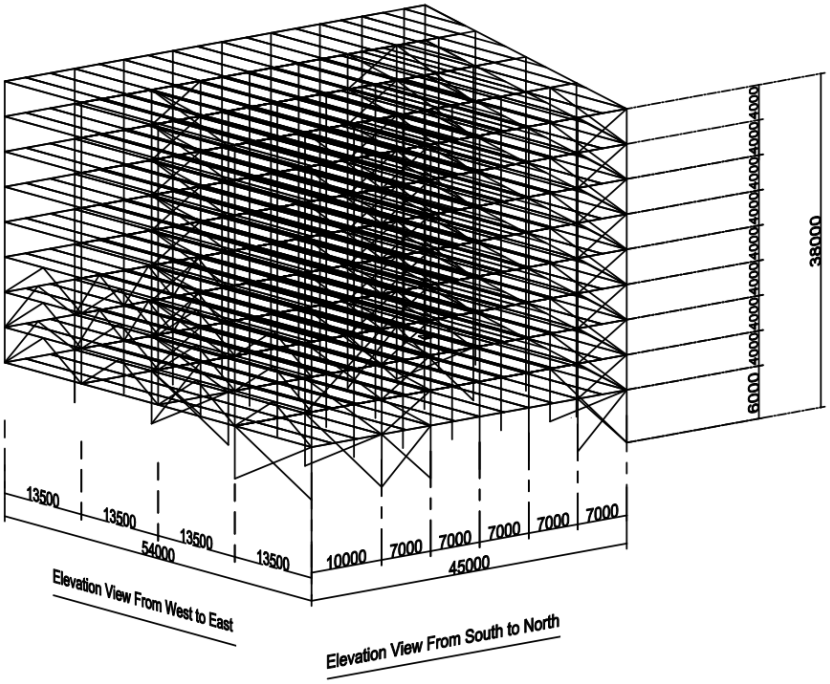


Figure 2.18 Global Structural Model for Proposal 2 (Autodesk)

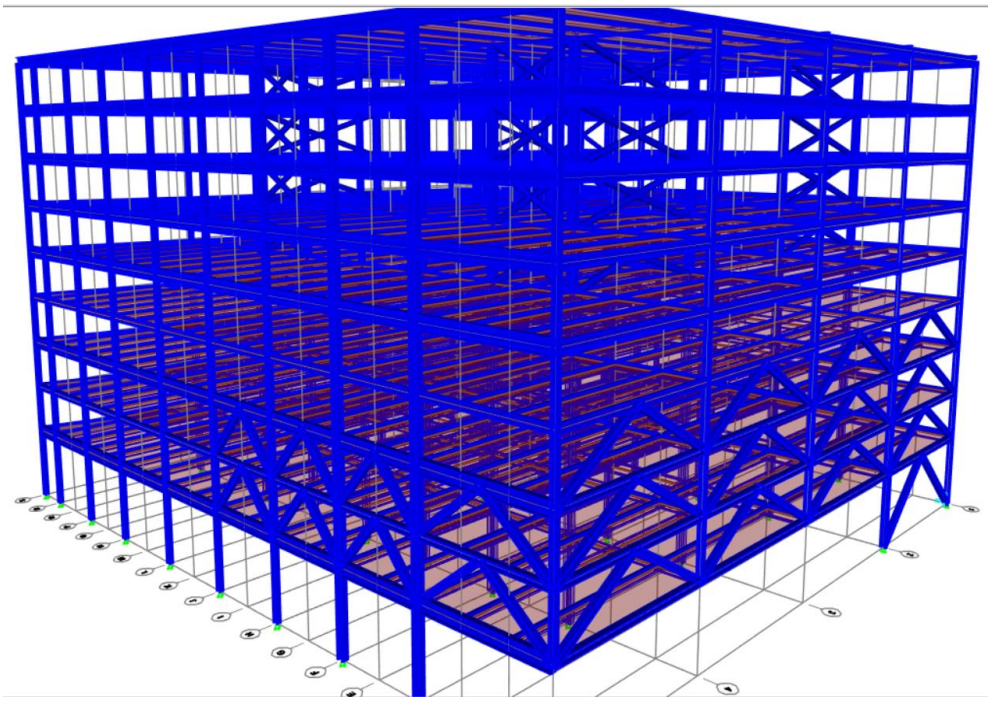


Figure 2.19 Global Structural Model for Proposal 2 (SAP2000)

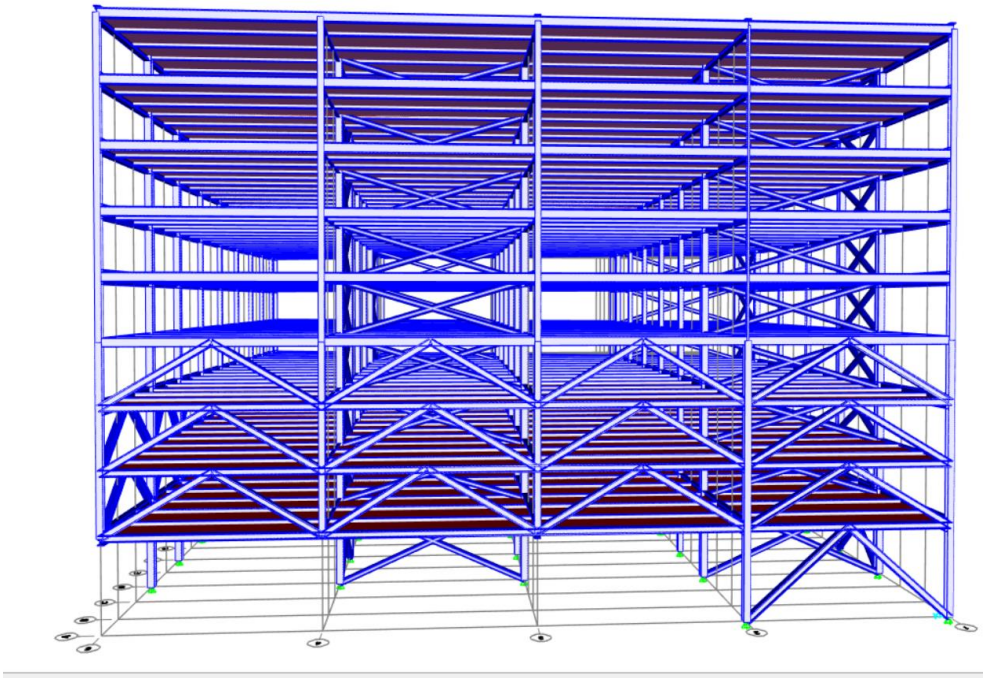


Figure 2.20 Global Structural Model for Proposal 2 (SAP2000)

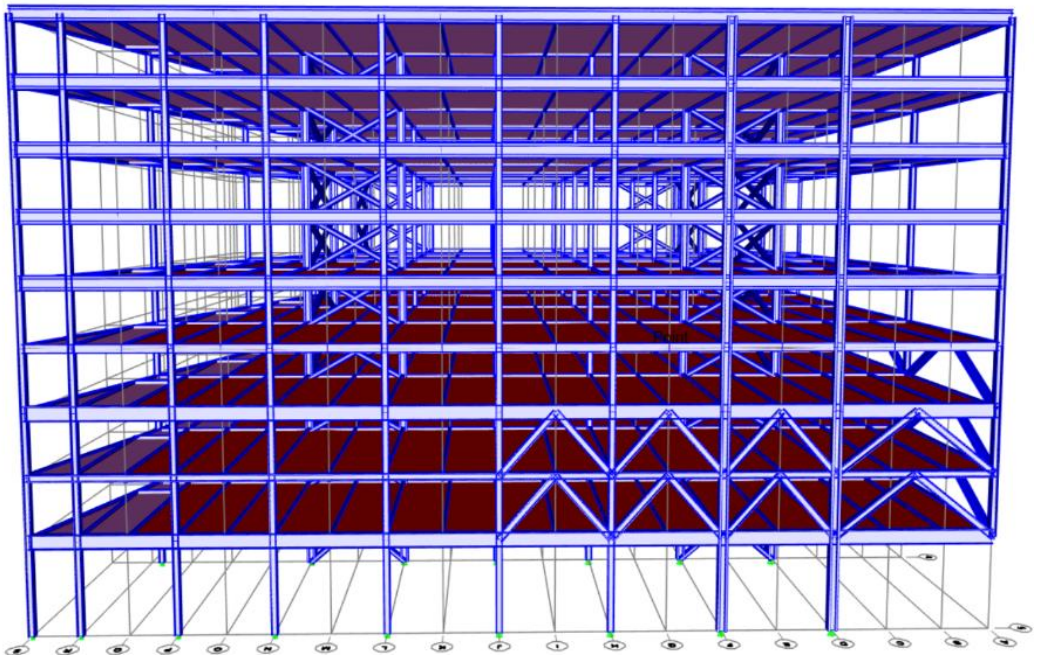


Figure 2.21 Elevation View from West to East (SAP2000)

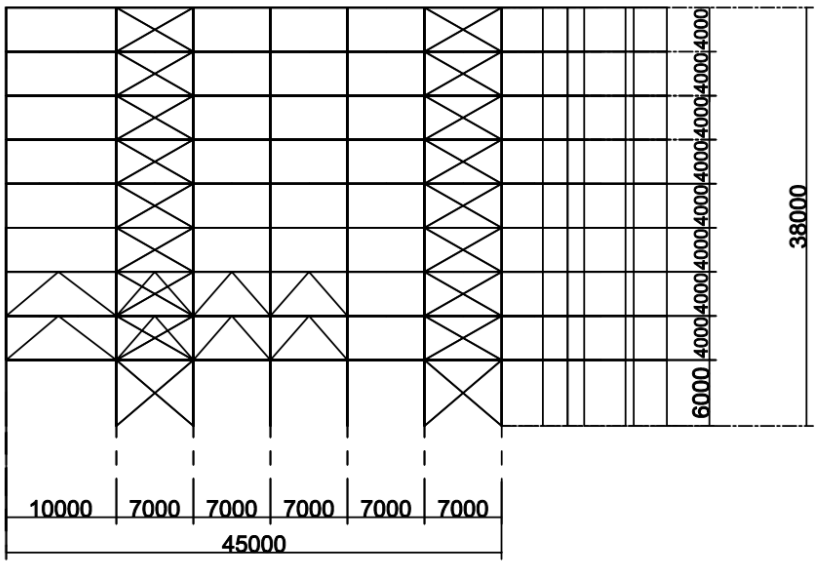


Figure 2.22 Elevation View from South to North

To be noticed, the elevation drawings are perspective and truss system and bracing system do not intersect with each other

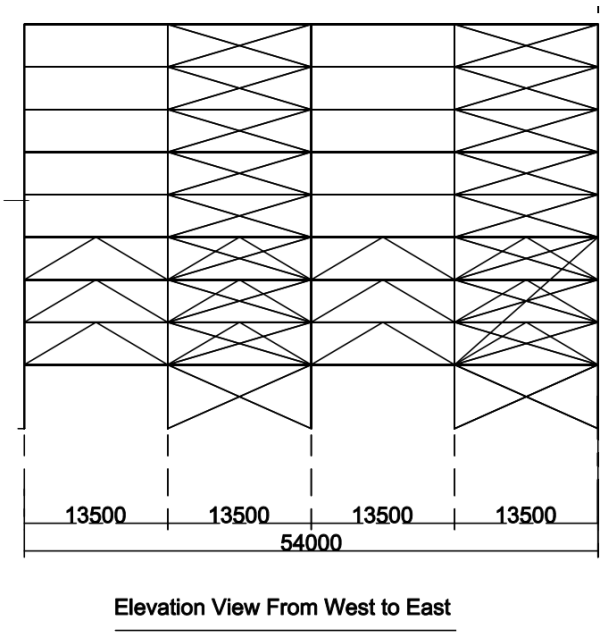


Figure 2.23 Elevation View from West to East

2.3.2. Column Arrangement and Cross-section Dimensions

The layout of columns in proposal 2 remains that of in proposal one, the changes are mainly due to the movement of wind bracing. The columns around the bracing need UKC 356*406*1202.

The detailed columns with corresponding cross-sections are shown below.

Table 2.6 Cross-section properties in SAP2000

Location of the Columns	Cross-section Selection (UKC)	Represented Colors in SAP2000
Typical Internal Columns from 4th Floor to Roof	356×368×202	
Typical internal Columns from 4th Floor to Ground	356×406x467	
Typical Peripheral Columns from 4th Floor to Roof	356×368× 202	
Typical Peripheral Columns from 4th Floor to Ground	356×406×235	
Columns for Wind Bracing Bays from Roof to Ground	356*406*1202	
Columns near the cantilever region from 4th Floor to Ground	356×406×634	

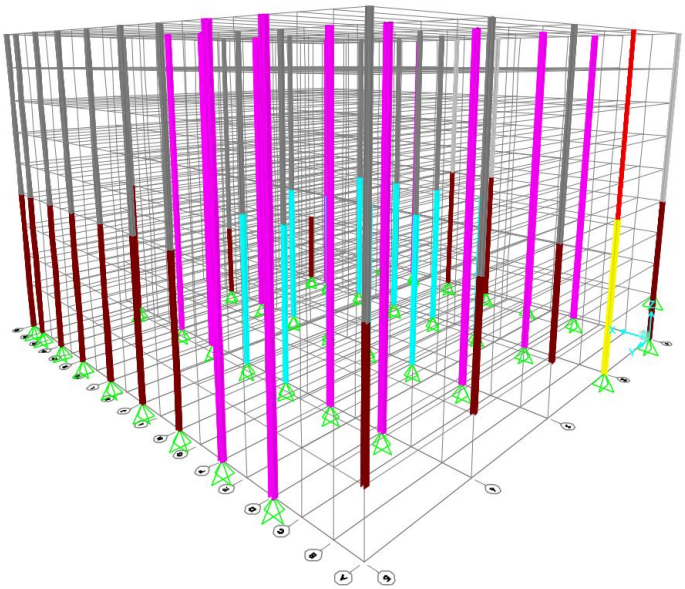


Figure 2.24 Global View for Columns (SAP2000)

Details of the column size at different parts of the buildings can be referred to Appendix A4.

2.3.3. Beam Arrangement and Cross-section Selection

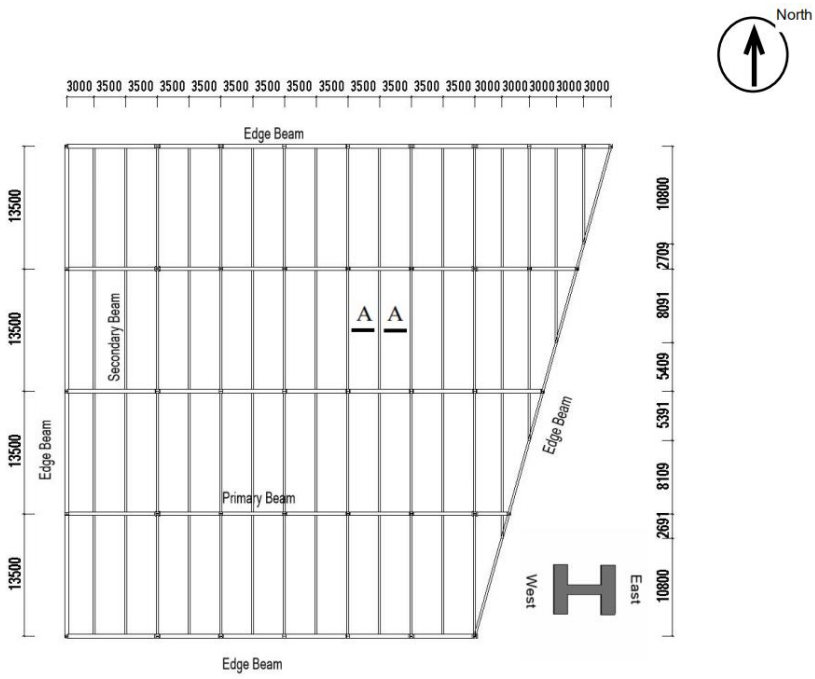


Figure 2.25 Beam Arrangement with critical section (Revit)

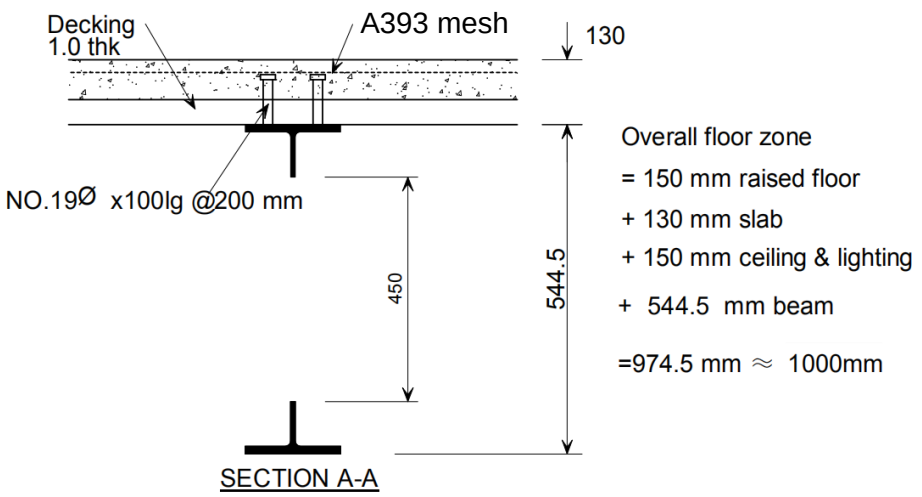


Figure 2.26 Details with critical section A-A

2.3.4. Bracing

The bracing system will intersect with the truss system if bracing remains at the location under proposal 1's arrangement. It will bring a lot of problems such as connection erection. The loads from the cantilever region are also more likely to be taken by the bracing system, which makes the hand calculations more complicated. To prevent this from happenning, the bracing system is moved inward from North to South View.

By providing a bracing system in both X and Y directions with asymmetric layout, it can not only resist the lateral force such as wind forces, equivalent horizontal forces due to the gravity and imperfection of construction, etc., but also can resist torsional forces to the structure.

The details with load transfer and resistance check will be explained in the bracing chapter.

The location of the bracing system is highlighted in the following figure (as highlighted in red).

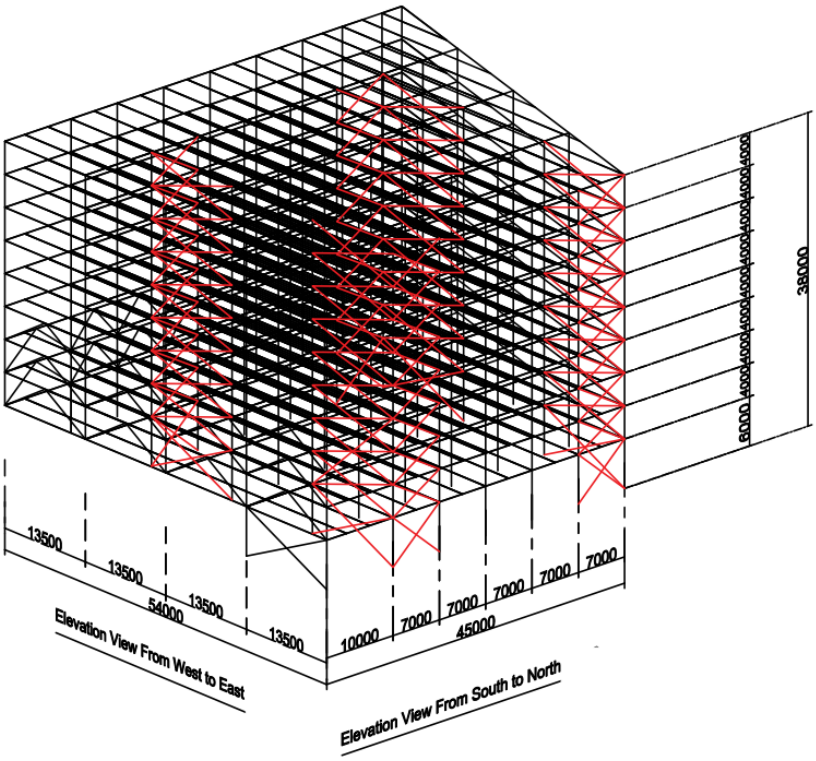


Figure 2.27 Global View for Bracing in the Structure

An isolated bracing system with dimensions is shown below

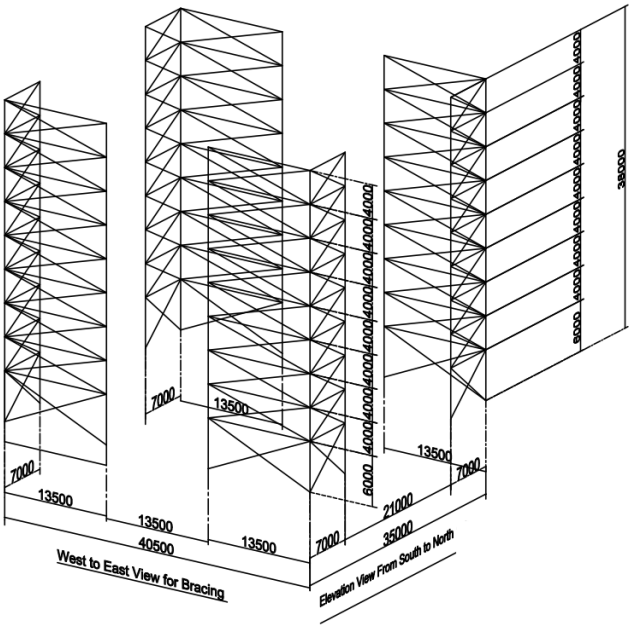


Figure 2.28 Overview for isolated bracing system

Elevation views from South to North and from West to East is shown below

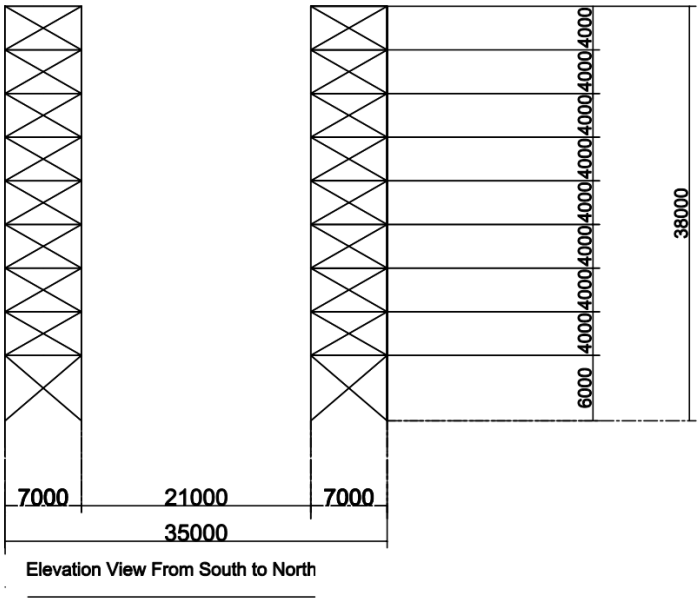


Figure 2.8 Elevation View from South to North for Bracing System

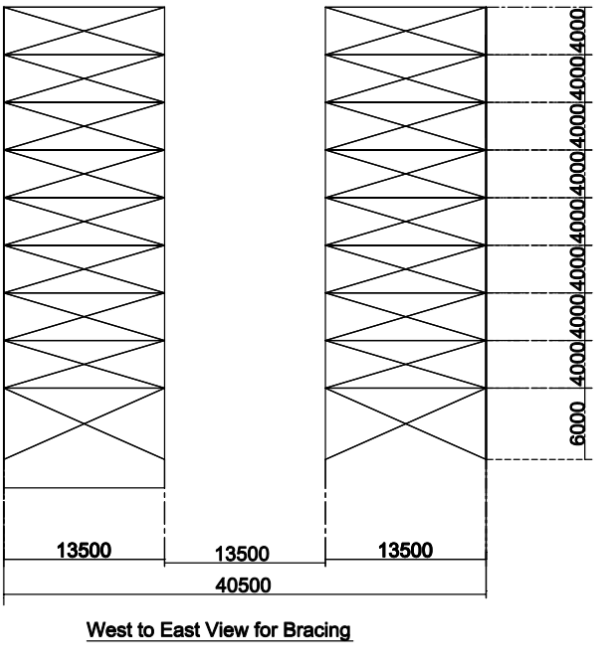


Figure 2.9 Elevation View from West to East for Bracing System

2.3.5. Cantilever Region Design

In proposal 2, the truss is designed for resisting horizontal and vertical forces (mainly to resist vertical forces in the cantilever region) as shown in the following figure instead of the X-bracing truss in proposal 1.

The tension and compression members in each truss unit are in each primary and secondary span unit. On the west face side, three floors of warrant truss are deployed, and the loads are transferred to the ground through the columns were in the RHS.

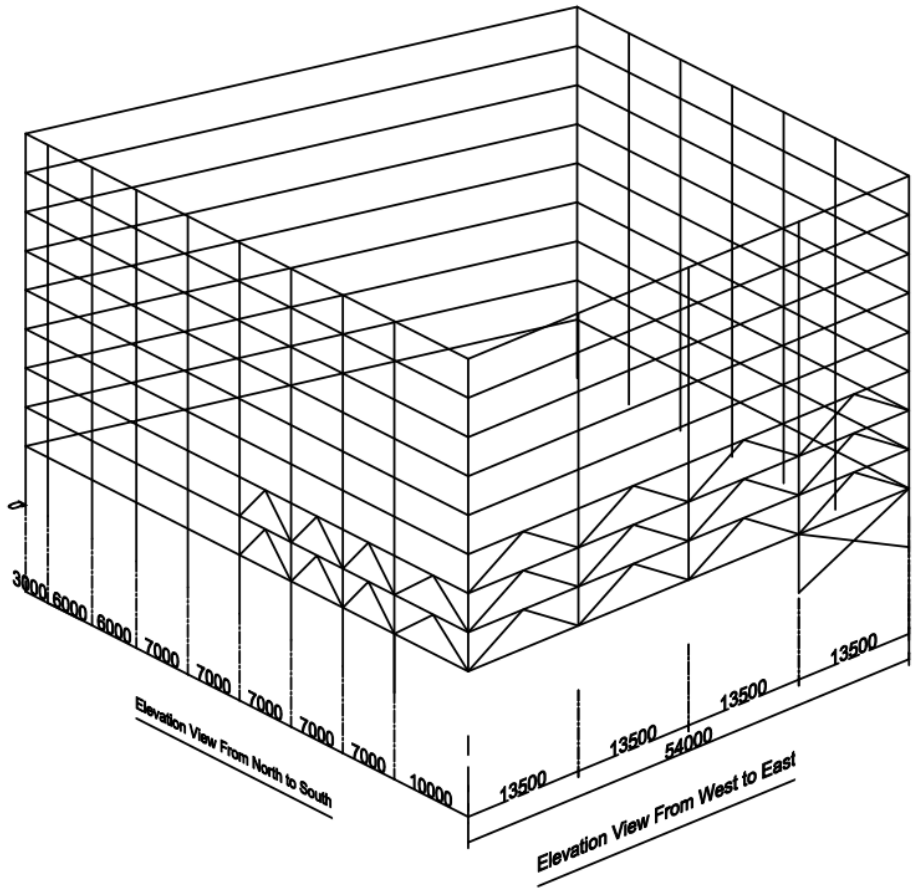


Figure 2.10 Global View for Cantilever Region (Autodesk CAD)

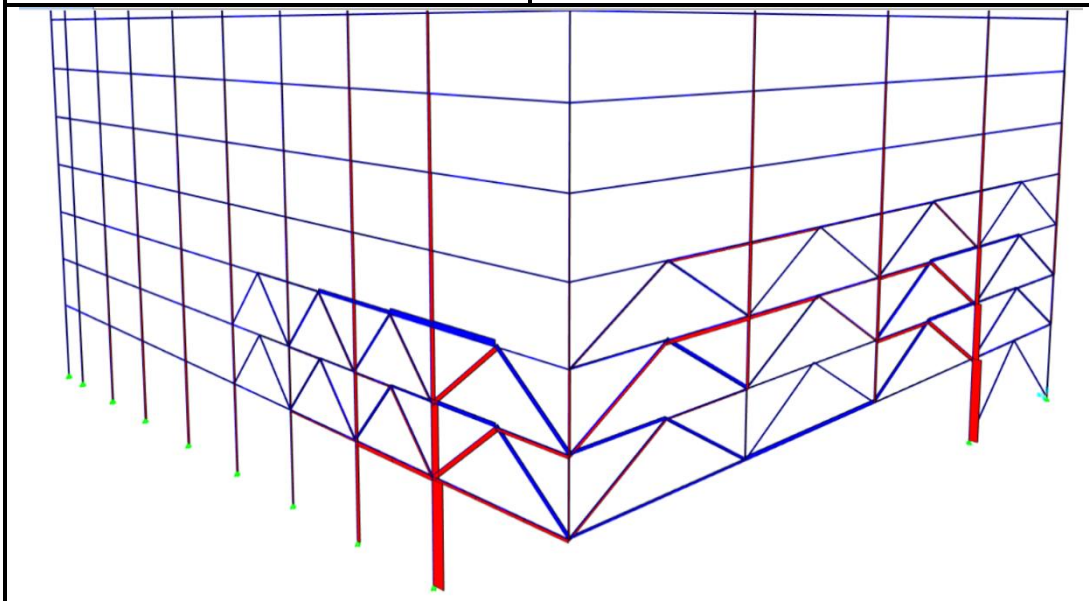


Figure 2.11 Global View for Cantilever Region (SAP2000)

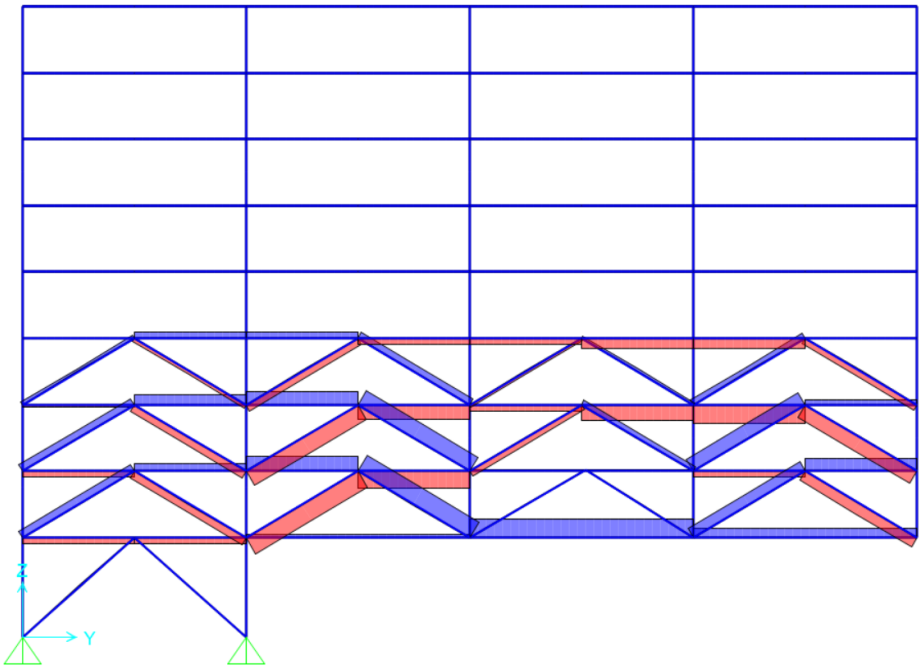


Figure 2.12 West to East view for loading path

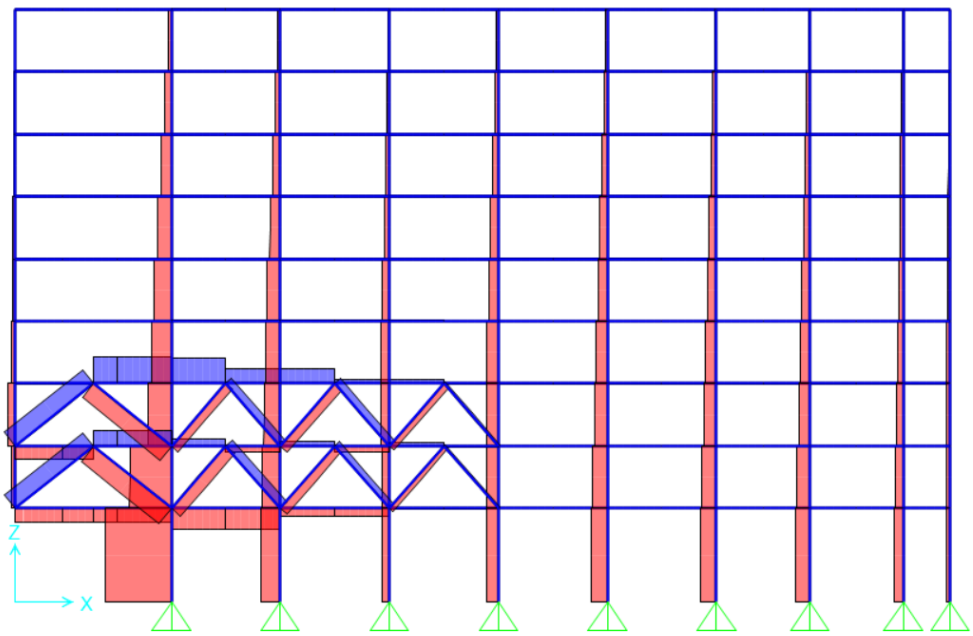


Figure 2.13 North to South view for loading path

2.3.6. The overall tonnage of the structural proposal 2

Table 2.7 The overall self-weight of steel of S355

Section	Unit	Total Length (m)	Total Weight (kN)
UKC (356x368x202)	3	60	117
UKB (610x229x140)	176	68183	9364
UKC356*368*202	1	20	6.565
UKC356*406*235	37	254	586
UKC356*368*202	52	496	983
UKC356*406*467	51	709	3250
UKC356*406*634	1	18	111
UKC356*406*1202	38	532	6269
CHS 508*14.2	144	16092	2728
UKC356*406*340	16	89	409
UKB 533×210×122	886	6880	8242
Total weight			**32070**

2.4. Comparison Between Proposals 1 and 2

The two proposals are compared in this section and it is found that Proposal 2 is relatively better in terms of amount of materials used.

As shown in Tables 2.5 and 2.7, the total tonnage of steel is around 32686kN and 32070.5kN. Considering the self-weight of steel used, Proposal 2 is more economical as it involves less materials, so that the cost is also lower. Assuming a unit price of £700 per ton for structural steel, there can be a saving of nearly £50,000 saving for using Proposal 2. For other materials, such as concrete used in composite slabs and beams, it is considered as not critical when compared with the quantity of steel used in the design. The quantity of other materials used in the two proposals can also be considered as similar, so that the exact amount is not compared.

Other factors, such as the ease of construction, have been considered in the formulating the two proposals. Both proposals are found to be similar in terms of construction difficulties. Some other proposals which are not adopted, and some better proposals which may be considered in the future are discussed in Section 13.

3. Composite Slabs Design

For the composite slabs design, the slabs are designed by considering the fire rating, manufacturer's data for the material ComFlor 60 composite slab, the loading on the slab and the corresponding bending and shear resistance.

A double-span slab depth of 130mm and a thickness of steel profile of 1.2mm is adopted for both proposals. According to the manufacturer's data, double-span slabs with a depth of 130mm offer the largest maximum span between secondary beams, while complying the subsequent deflection checks. Therefore, a span depth of 130mm is adopted for the design. The reason for choosing 1.2mm of steel profile is because a thicker deck gives a larger moment of inertia and ultimate moment capacity without significantly increase the self-weight of the slab. The large moment of inertia and ultimate moment capacity can then offer a better resistance of the slab to deflection and bending.

Checks to both construction stage and operation stage after the composite action has been formed have been conducted.

3.1. ULS and SLS Checks for Construction Stage

For the construction stage, an ULS check on the moment capacity has been performed. The design moment has been compared against the ultimate bending resistance of the slab with reference to the manufacturer's data to ensure the slab complies with the requirements.

For SLS check, deflection of the slab during construction stage has been found to be critical in determining the depth of slab adopted. This is because a thicker of slab causes a larger self-weight, which increases the deflection induced. With reference to Clause 9.3.2(2) of BSEN1994-1-1, it has been found that the ponding effect cannot be ignored as the deflection of the slab induced by the self-weight is larger than 10% of the slab depth. Therefore, the deflection resistance has been re-verified by considering the nominal thickness of the concrete increases by 0.7 times of the central deflection as stated in Clause 9.3.2(2) of BSEN1994-1-1. By re-checking against the deflection limit of L/130 stated in NA.2.15 of BSEN1994-1-1, where L denotes the span of the slab, it is found that the deflection calculated is smaller than maximum permissible limit. Therefore, the proposed composite slab complies with the requirements set out by the code of practice during the construction stage.

3.2. ULS and SLS Checks for Operation Stage

For operation stage, bending, longitudinal and vertical shear checks have been performed for ULS. Cracking check has been performed for SLS to determine the

reinforcement needed to resist concrete cracking.

For the bending resistance, the bending resistance has been calculated after obtaining the neutral axis as per the requirements set out in Clause 6.2.1.2 and 9.7.2(5) of BSEN1994-1-1. Full shear connection has also been assumed to determine the bending resistance.

For calculating the longitudinal shear resistance, m-k method stated in Clause 9.7.3 of BSEN1994-1-1 has been used. The longitudinal shear resistance can then be obtained using the following equation from 9.7.3(4) of BSEN1994-1-1:

$$V_{l,Rd} = \frac{bd_p}{\gamma_{vs}}\left(\frac{mA_p}{bL_s} + k\right)$$

, whereA_p, L_s and γ_{vs} are nominal cross-section of steel profile, shear span and ultimate partial safety factor under ULS. m and k are empirical factors obtained from slab tests, b and d_p are 1000mm and depth of slab minus the height of neutral axis above the soffit respectively.

The vertical shear resistance has been calculated based on BSEN19920101 Equation 6.2b and the cracking resistance of concrete under SLS has been calculated based on Clause 9.8.1(2) of BSEN1994-1-1. According to the code, a minimum cross-sectional area of reinforcement of 0.4% of cross-sectional area of concrete for unpropped condition should be provided. After calculation, it is decided to provide A393 mesh for the composite slab.

3.3. Summary of Design of Composite Slab

As the design tributary area for slabs remain unchanged in the two proposals, same design is adopted for both proposals. To summarise, a double-span composite slab with depth 130mm and 1.2mm of steel profile using ComFlor 60 is proposed. A393 mesh is proposed for the slab to meet the requirements related to concrete cracking.

3.4. Detailed Calculations

Propose ComFlor 60 Bar Fire Method/Unpropped double-span slab,

<u>From ComFlor 60 manufacturer's data:</u>

Total Depth of Slab t=130mm

Depth of Profile h_p=1.2mm

Span L=3.5m

Cross Section Area A_{pe}=1721mm^2/m

Second Moment of Area I_p=132.91cm^4/m

Yield Strength of Deck f_{yp}=350MPa

Design Bending Resistance (Sagging) M_{Rd}=15.21kNm/m

Height of NA above soffit=33.85mm

Assuming a concrete class C25/30,

Density (wet)=26kN/m^3

Density (dry)=25kN/m^3

Cylinder Strength f_{ck}=25MPa

Concrete Design Strength=0.85(25)/1.5=14.17MPa

Elastic Modulus E_{cm}=31GPa

<u>Actions</u>

Concrete Self Weight (Wet)=2.46kPa

Concrete Self Weight (Dry)=2.36kPa

<u>Permanent Actions</u>

Load (Construction Stage)=0.14kPa

Load (Operation Stage)=0.7+0.3+1+1.5+0.5+2.36=6.5kPa

<u>Variable Actions</u>

Load (Construction Stage)=Construction Load+Concrete SW (Wet)

=1.5+ 2.46=3.96kPa

Load (Operation Stage)=4kPa

ULS Partial Factors (Consider Load Case 1 to be Conservative)

γ_G=1.35

γ_Q=1.5

Load (ULS)

Construction Stage :0.14(1.35)+3.96(1.5)=6.13kPa

Operation Stage: 6.36(1.35)+4(1.5)=14.78kPa

Design Moment and Shear Force (ULS)

Construction Stage

$M_{Ed}=FL^2/8=6.1(3.5)^2/8=9.39$kNm/m width

$V_{Ed}=FL/2=6.1(3.5)/2=10.73$kN/m width

Operation Stage

$M_{Ed}=FL^2/8=14.6(3.5)^2/8=22.62$kNm/m width

$V_{Ed}=FL/2=14.6(3.5)/2=25.86$kN/m width

ULS Moment Design Check (Construction Stage)

According to BSEN1993-1-3 Clause 6.1.1,

$M_{Ed}/M_{Rd}=10.53/15.21=0.62<=1$ **OK**

SLS Deflection Check (Construction Stage)

Load under SLS=0.14+2.96=2.6kPa

According to BSEN 1994-1-1 Clause 9.3.2(2),

$\delta_s=5FL^4/(384EI)=18.20\text{mm}>13\text{mm}$ **FURTHER CHECK**

Therefore, ponding cannot be ignored and the thickness of concrete increases by $0.7\delta_s$.

Revised Load under SLS=2.6+0.7(18/1000)(26)=2.93kPa

Revised δ_s=5FL^4/(384EI)=20.52mm

$\delta_{s,max}$=L/130=3500/130=26.92mm<=30mm **OK**

According to BSEN 1994-1-1 NA2.15,

δ_s=20.5mm<=$\delta_{s,max}$=26.9mm, so it is OK.

<u>ULS Composite Slab Check (Operation Stage)</u>

<u>Bending Resistance</u>

According to BSEN1994-1-1 Clause 6.2.1.2,

Max. compressive design force in concrete (assuming pna is below solid part of slab):

$N_c=f_{cd}(A_c)$=14.2(130-60)=991.67kN/m

Max. tensile resistance of profiled steel sheet:

$N_p=f_{yp,d}(A_p)$=350(1721)/1000=602.35kN/m

As $N_p<N_c$, netural axis lies above profiled sheeting.

According to BSEN1994-1-1 Clause 9.7.2(5),

Depth of concrete in compression:

$x_{pl}=A_{pe}(f_{yp,d})/(bf_{cd})$=1721(350)/1000/14.2=42.52mm

Assuming full shear connection:

$M_{pl,Rd}=A_pf_{yd}(d_p-x_{pl}/2)$=45.11kNm/m

$M_{Ed}/M_{pl,Rd}$=22.33/45.11=0.50<=1 **OK**

Longitudinal Shear Resistance

From BSEN1994-1-1 Clause 9.7.3,

$V_{l,Rd}=bd_p(mA_p/bL_s+k)/\gamma_{vs}$

From manufacturer's data,

m=157.2MPa

k=0.12MPa

For uniform load across span length,

$L_s=L/4=3500/4=875mm$

$V_{l,Rd}=33.26kN/m$

$V_{Ed}/V_{l,Rd}=25.5/33.3=0.78<=1$ **OK**

Vertical Shear Resistance

$V_{v,Rd}=v_{min}b_sd_p$

According to BSEN1992-1-1 NA2 Table NA.1

$v_{min}=0.035k^{1.5}f_{ck}^{0.5}$

, where $k=1+(200/d_p)^{0.5}=2.44>2$ **FURTHER CHECK**

Therefore, take k=2

$v_{min}=0.035(2)^{1.5}(25)^{0.5}=0.49MPa$

$V_{v,Rd}=0.049\ (130-33.9)=47.59kN/m$

$V_{Ed}/V_{v,Rd}=25.5/47.6=0.54<=1$ **OK**

SLS Composite Slab Check (Operation Stage)

Cracking of Concrete

According to BSEN1994-1-1 Clause 9.8.1(2), minimum reinforcement of 0.4% of concrete area should be provided.

Therefore, provide 0.4%(130-60)(1000)=280mm^2

Therefore, provide A393 instead.

4. Composite Beams Design

In this section the design of the beams will be described in detail. All beams in this project are composite beams and it is assumed that the supports are all simply supported. There are three types of beams, secondary, primary and edge beams, and in view of the need to achieve as uniform a section as possible in practice, this section will design the most critical parts of each of the three types of beams and apply the resulting sections to each of the corresponding types of beams. As the calculation process is similar for the different types of beams, the most critical secondary beam from proposal 1 will be used as an example in this section. Due to the unchanged loads from slabs, same set of results from Proposal 1 will be adopted in Proposal 2.

4.1. Selection of beam types

Composite beams are used in this project due to the following advantages.

(1) Composite beams generally have a better bending stiffness and structural performance than pure steel or concrete beams. They are also more suitable to use in regions which are susceptible to earthquakes.

(2) Due to the better structural performance, a smaller size of beams can be used, which can increase the inter-storey clear height.

(3) As steel is more expensive, the use of composite beams can enhance the structural capacity of the beams with the same amount of steel used.

(4) It can save time as composite beams are normally used in conjunction with composite slabs, which can be manufactured in advanced.

4.2. Beam section design results

The beam sections are designed according to the load, span etc. and are verified by first selecting the sections and then carrying out the load carrying capacity and deflection tests etc. The detailed steps will be shown at the end of this section.

Table 4.1: Table showing the UKB sections for beams adopted for the design

Type	Section
primary beam (floor)	533x210x122UKB
secondary beam (floor)	533x210x122UKB
edge beam (floor)	533x210x122UKB
primary beam (roof)	533x210x122UKB
secondary beam (roof)	533x210x122UKB
edge beam (roof)	533x210x122UKB

4.3. Detailed Calculation

Design data

Beam span	L = 14.0 m
Beam spacing	s = 3.5 m
Total slab depth	h = 130 mm
Depth of concrete above profile	h_c = 70 mm
Deck profile height	h_p = 60 mm
Width of the bottom of the trough	b_{bot} = 120 mm
Width of the top of the trough	b_{top} = 170 mm approx

Shear connectors

Diameter	d = 19 mm
Overall height before welding	h_{sc} = 100 mm
Height after welding	95 mm

Materials

Structural Steel:
For grade S355 and maximum thickness (t) less than 40 mm

Yield strength	f_y = 355 N/mm^2
Ultimate strength	f_u = 510 N/mm^2

Steel reinforcement:

Yield strength	f_{yk} = 500 N/mm^2

Concrete:
Normal weight concrete strength class C25/30

Density	26 kN/m³ (wet)
	25 kN/m³ (dry)
Cylinder strength	f_{ck} = 25 N/mm^2
Secant modulus of elasticity	E_{cm} = 31 kN/mm^2

<u>Actions</u>
Use the data from slab part.
Ultimate Limit State
Combination of actions for Ultimate Limit State
The design value of combined actions are:
Construction stage:
Distributed load (0.925×1.35×0.44)+(1.5×3.96) = 6.49 kN/m^2
Total load F_d = 5.26×14.0×3.5 = 317.98 kN
Composite stage:
Distributed load (0.925×1.35×6.50)+(1.5×4.00) = 14.12 kN/m2
Total load Fd = 14.12×14.0×3.5 = 691.73 kN

Design values of moment and shear force at ULS

Construction stage

Maximum design moment (at mid span)

$$M_{y,Ed} = \frac{F_d L}{8} = \frac{317.98 \times 14.0}{8} = 556.5 \text{ kNm}$$

Composite stage

Maximum design moment (at mid span)

$$M_{y,Ed} = \frac{F_d L}{8} = \frac{691.73 \times 14.0}{8} = 1210.5 \text{ kNm}$$

Maximum design shear force (at supports)

$$V_{E_d} = \frac{F_d}{2} = \frac{691.7}{2} = 345.9 \text{ kN}$$

Partial factors for resistance

Structural steel	$\gamma_{M0} = 1.0$
Concrete	$\gamma_c = 1.5$
Reinforcement	$\gamma_s = 1.15$
Shear connectors	$\gamma_v = 1.25$
Longitudinal shear	$\gamma_{vs} = 1.25$

Trial section

The plastic modulus that is required to resist the construction stage maximum design bending moment is determined as:

$$W_{\text{pl,y}} = \frac{M_{\text{y,Ed}}\gamma_{\text{MO}}}{f_{\text{y}}} = \frac{556.5 \times 10^3 \times 1.0}{355} = 1568 \text{ cm}^3$$

From the tables of section properties try section 533×165×75 UKB, S355, which has $W_{pl,y} = 2100$ cm^3

Depth of cross-section	$h_a = 529.1$ mm
Width of cross-section	$b = 165.9$ mm
Depth between fillets	$d = 476.5$ mm
Web thickness	$t_w = 9.7$ mm
Flange thickness	$t_f = 13.6$ mm
Radius of root fillet	$r = 12.7$ mm
Cross-section area	$A_a = 95.2$ cm^2
Plastic section modulus (y-y)	$W_{pl,y} = 1810$ cm^3
$t_f < 40$ mm, therefore	$f_y = 355$ N/mm^2
Modulus of elasticity	$E = 210000$ N/mm^2
Second moment of area (y-y)	$I_a = 41100$ cm^4

Section classification

For section classification the coefficient e is:

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81$$

Outstand flange: flange under uniform compression

$$c = \frac{(b - t_w - 2r)}{2} = \frac{(165.9 - 9.7 - 2 \times 12.7)}{2} = 65.4 \text{ mm}$$

$$\frac{c}{t_f} = \frac{65.4}{13.6} = 4.81$$

The limiting value for Class 1 is $\frac{c}{t} \leq 9\varepsilon = 9 \times 0.81 = 7.32$

4.81 < 7.32

Therefore, the flange outstand in compression is Class 1.

Internal compression part: web under pure bending

c = d = 476.5 mm

$$\frac{c}{t_w} = \frac{476.5}{9.7} = 49.12$$

The limiting value for Class 1 is $\frac{c}{t} \leq 72\varepsilon = 72 \times 0.81 = 58.58$

49.12 < 58.58

Therefore, the web in pure bending is Class 1.

Therefore, the section is Class 1 under pure bending.

<u>Composite stage member resistance checks</u>

Concrete

Design value of concrete compressive strength $f_{cd} = \alpha_{cc} \times f_{ck}/\gamma_c$

$\alpha_{cc} = 0.85$

$f_{cd} = 0.85 \times \frac{25}{1.5} = 14.2\ N/mm^2$

Compression resistance of concrete slab

At mid-span the effective width of the compression flange of the composite beam is determined from:

$b_{eff} = b_0 + \sum b_{ei}$

$b_{ei} = \frac{L_e}{8} = \frac{L}{8} = \frac{14}{8} = 1.75$ m ($L_e = L$ for simply supported beams)

Assume a single line of shear studs, therefore, $b_0 = 0$ m

$b_{eff} = 0 + (2 \times 1.75) = 3.50m \leq 3.5m$ (beam spacing)

Compression resistance of concrete slab is determined from:

$$N_{c,\text{ slab}} = f_{cd} b_{eff} h_c$$

where h_c is the depth of the solid concrete above the decking

$N_{c,slab} = 14.2 \times 3500 \times 70 \times 10^{-3} = 3471$ kN

Tensile resistance of steel section

$$N_{pl,a} = f_d A_a = \frac{f_y A_a}{\gamma_{MO}}$$

$$N_{pl,a} = \frac{355 \times 95.2 \times 10^2}{1.0} \times 10^{-3} = 3380 \text{ kN}$$

Location of neutral axis

Since $N_{pl,a} < N_{c,slab}$ the plastic neutral axis lies in the concrete flange.

Design bending resistance with full shear connection

As the plastic neutral axis lies in the concrete flange, the plastic resistance moment of the composite beam with full shear connection is:

$$M_{pl,Rd} = N_{pl,a}\left[\frac{h_s}{2} + h - \frac{N_{pl,a}}{N_{c,slab}} \times \frac{h_c}{2}\right]$$

$$M_{pl,Rd} = 3380\left[\frac{529.1}{2} + 130 - \frac{3380}{3471} \times \frac{70}{2}\right] \times 10^{-3} = 1218 \text{ kNm}$$

Bending moment at mid span $M_{y,Ed}$ = 1210.5 kNm

$$\frac{M_{y,Ed}}{M_{pl,Rd}} = \frac{1210.5}{1218} = 0.99 < 1.0$$

Therefore, the design bending resistance of the composite beam is adequate, assuming full shear connection.

Shear connector resistance

The design shear resistance of a single shear connector in a solid slab is the smaller of:

$$P_{Rd} = \frac{0.29\alpha d^2\sqrt{f_{ck}E_{cm}}}{\gamma_v} \text{ and}$$

$$P_{Rd} = \frac{0.8 f_u(\pi d^2/4)}{\gamma_v}$$

$$\frac{h_{SC}}{d} = \frac{100}{19} = 5.26$$

As $\frac{h_{Sc}}{d} > 4.0 \qquad \alpha = 1.0$

$$P_{\text{Rd}} = \frac{0.29 \times 1.0 \times 19^2\sqrt{25 \times 31 \times 10^3}}{1.25} \times 10^{-3} = 73.7 \text{ kN}$$

or

$$P_{\text{Rd}} = \frac{0.8 \times 450 \times \left(\pi \times \frac{19^2}{4}\right)}{1.25} \times 10^{-3} = 81.7 \text{ kN}$$

As 73.7 kN < 81.7 kN $P_{\text{Rd}} = 73.7$ kN

Influence of deck shape
Deck crosses the beam (i.e., ribs transverse to the beam)
One stud per trough, $n_r = 1.0$

Reduction factor

$$k_t = \left(\frac{0.7}{\sqrt{n_p}}\right)\left(\frac{b_0}{h_p}\right)\left(\frac{h_{sc}}{h_p} - 1\right) \leq 1.0$$

For trapezoidal profiles, b_0 is the average width of a trough, taken here as (120 + 170) ÷ 2 = 145 mm

$$k_t = \left(\frac{0.7}{\sqrt{1}}\right) \times \left(\frac{145}{60}\right) \times \left(\frac{100}{60} - 1\right) = 1.13 \text{ but not more than } 1.0$$

Therefore, as $k_t = 1.0$ no reduction in shear connector resistance is required.

Therefore, $P_{Rd} = 73.7$ kN

Number of shear studs in half span
Use one shear connector per trough, therefore,
Stud spacing along beam = 175 mm
Centre line to centre line span of 7 m should be reduced to allow for the primary beam width or the column width (assume 254 mm).

$$n = \frac{7000-(254/2)}{300} = 39 \text{ stud shear connectors per half span}$$

Degree of shear connection
Total resistance of 39 shear connectors

$$R_q = 39P_{Rd} = 39 \times 73.7 = 2875.5\text{kN}$$

$$\frac{R_q}{N_{pl,a}} = \frac{2875.5}{3380} = 0.85 < 1.0$$

As this is less than 1.0, this beam has partial shear connection.
Therefore, the minimum shear connection requirements must be checked, and the moment resistance reassessed.

Minimum shear connection requirements
The minimum shear connection requirement is calculated from:
(for $L_e < 25$m):

$$\eta \geq 1 - \left(\frac{355}{f_y}\right)(0.75 - 0.03L_e), \qquad \eta \geq 0.4$$

For a simply supported beam, L_e is equal to the span.

$$\eta \geq 1 - \left(\frac{355}{355}\right)(0.75 - 0.03 \times 14) = 0.67, \qquad \eta \geq 0.4$$

Therefore, the degree of shear connection must be at least 0.67. As shown above, there are a sufficient number of shear connectors to achieve this.

Design bending moment resistance with partial shear connection

The design bending moment can conservatively be calculated using:

$M_{Rd} = M_{pl,a,Rd} + (M_{pl,Rd} - M_{pl,a,Rd})\eta$

$M_{pl,a,Rd}$ is the plastic moment resistance of the steel section:

$M_{pl,a,Rd} = f_{yd}W_{pl,y} = 355 \times 1810 \times 10^{-3} = 642.6$ kNm

Therefore, the design bending moment resistance is:

$M_{Rd} = 642.6 + (1218 - 642.6) \times 0.85 = 1132.4$ kNm

$$\frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{Rd}}} = 1 \leq 1.0$$

Therefore, the design bending resistance of the composite beam with partial shear connection is adequate.

<u>Shear buckling resistance of the uncased web</u>

For unstiffened webs if $\frac{h_w}{t} > \frac{72}{\eta}\varepsilon$ the shear buckling resistance of the web should be checked.

Where:

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81$$

$\eta = 1.0$ (conservative)

$h_w = h_a - 2t_f = 529.1 - (2 \times 13.6) = 501.9$ mm

$$\frac{72}{\eta}\varepsilon = \left(\frac{72}{1.0}\right) \times 0.81 = 58.58$$

$$\frac{h_w}{t} = \frac{h_w}{t_w} = \frac{501.9}{9.7} = 51.74$$

As 51.74 < 58.58 the shear buckling resistance of the web does not need to be checked.

<u>Resistance to vertical shear</u>

Vertical shear resistance of the composite beam is:

$$V_{pl,Rd} = V_{pl,a,Rd} = \frac{A(f_y/\sqrt{3})}{\gamma_{MO}}$$

For rolled I and H sections loaded parallel to the web:

$A_v = A - 2bt_f + t_f(t_w + 2r)$ but not less than $\eta h_w t_w$

$A_v = 9520 - (2 \times 165.9 \times 13.6) + 13.6 \times [9.7 + (2 \times 12.7)]$

$A_v = 5485 \text{ mm}^2$

$\eta = 1.0$ (Conservatively)

$\eta h_w t_w = 1.0 \times 501.9 \times 9.7 = 4868 \text{ mm}^2$

$5485 \text{ mm}^2 > 4868 \text{ mm}^2$

Therefore, $A = 5485 \text{ mm}^2$

$$V_{pl,Rd} = 5485 \times \frac{355}{\sqrt{3} \times 1.0} \times 10^{-3} = 1124 \text{ kN}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{76.5}{247} = 0.31 < 1.0$$

Therefore, the design resistance to vertical shear is adequate.

As there is no shear force at the point of maximum bending moment (mid span) no reduction (due to shear) in bending resistance is required.

<u>Design of the transverse reinforcement</u>

For simplicity, neglect the contribution of the decking and check the resistance of the reinforced concrete flange to splitting.

The area of reinforcement (A_{sf}) can be determined using the following equation:

$$\frac{A_{sf} f_{yd}}{s_f} > \frac{V_{Ed} h_f}{\cot \theta_f} \quad \text{therefore,} \quad \frac{A_{sf}}{s_f} > \frac{V_{Ed} h_f}{f_{yd} \cot \theta_f}$$

where:

h_f is the depth of concrete above the metal decking, therefore,

$h_f = h_c = 70$ mm

S_f is the spacing of the transverse reinforcement

$$f_{yd} = \frac{f_y}{\gamma_s} = \frac{500}{1.15} = 435 \text{N/mm}^2$$

For compression flanges $26.5° \leq \theta_f \leq 45°$

The longitudinal shear stress is the stress transferred from the steel beam to the concrete. This is determined from the minimum resistance of the steel, concrete and shear connectors. In this example, with partial shear connection, that maximum force that can be transferred is limited by the resistance of the shear connectors over half of the span, and is given by $R_q = 2875.5$ kN

This force must be transferred over each half-span. As there are two shear planes (one on either side of the beam, running parallel to it), the longitudinal shear stress is:

$$v_{Ed} = \frac{R_q}{2h_f \Delta x} = \frac{2875.5 \times 1000}{2 \times 70 \times 7000} = 2.93 \text{ N/mm}^2$$

For minimum area of transverse reinforcement assume $\theta = 26.5°$

$$\frac{A_{sf}}{s_f} \geq \frac{V_{Ed}h_f}{f_{yd}\cot\theta_f} = \frac{2.93 \times 70}{435 \times \cot 26.5} \times 10^3 = 235.5 \text{ mm}^2/\text{m}$$

Therefore, provide A393 mesh reinforcement (393 mm^2 /m) in the slab.

<u>Crushing of the concrete compression strut</u>

The model for resisting the longitudinal shear assumes compression struts form in the concrete slab

Verify that:

$v_{Ed} \leq vf_{cd}\sin\ \theta_f \cos\ \theta_f$

where:

$$v = 0.6\left[1 - \frac{f_{ck}}{250}\right]$$

$$v = 0.6\left[1 - \frac{25}{250}\right] = 0.54$$

$$vf_{cd}\sin\theta_f\cos\theta_f = 0.54 \times 14.2 \times \sin 26.5 \times \cos 26.5 = 3.06 \text{ N/mm}^2$$

$$v_{Ed} = 2.93 \text{ N/mm}^2 < 3.06 \text{ N/mm}^2$$

Therefore, the crushing resistance of the compression strut is adequate.

<u>Serviceability limit state</u>

Deflection

Check deflection in service under imposed load only assuming full composite action.

Check for position of neutral axis:

$$A_a(z_g - h_c) \leq \frac{b_{eff}h_c^2}{2n}$$

$$Z_g = 529.1/2 + 130 = 394.55\text{mm}$$

Note: n takes $2n_0$

$$\frac{b_{eff}h_c^2}{2n} = \frac{3500 \times 70^2}{2 \times 2 \times 6.77} = 632917$$

$$A_a(z_g - h_c) = 9520 \times (394.55 - 70) = 3089716 > 632917$$

Therefore, neutral axis depth exceeds h_c

From $A_a(z_g - x) = \frac{b_{eff}h_c(x - h_c/2)}{n}$, x = 159 mm

$$I = I_a + A_a(z_g - x)^2 + \frac{b_{eff}h_c}{n}\left[\frac{h_c^2}{12} + \left(x - \frac{h_c}{2}\right)^2\right] = 1.22 \times 10^9 \text{ mm}^4$$

$$\delta_c = \frac{5wL^4}{384EI} = \frac{5 \times 4 \times 3.5 \times 14000\text{^}4}{384 \times 210000 \times 1.22 \times 10\text{^}9} = 27.23 \text{ mm}$$

Allowable = span/360 = 39 mm

Check deflection with partial shear connection expression:

$\delta = \delta_c + 0.3(1-\eta)(\delta_s - \delta_c)$

$$\delta_s = \frac{5 \times 4 \times 3.5 \times 14000^4}{384 \times 21000 \times 41100 \times 10000} = 81.14 \text{ mm}$$

$\delta = \delta_c + 0.30 \times (1 - 0.85)(81.14 - 27.23) = 30 \text{ mm}$

Therefore, OK.

5. Steel Columns Design

In this section, the design of columns for this project will be described in detail. The choice of member size for different columns of the building will be explained with justifications. Overall procedures of the design calculations will also be given, followed by the full calculations for each type of columns.

5.1. Determination of Column Size

During the analysis of columns, the sections of columns were determined based on the locations of the columns and the loads acting on the columns. For example, the columns near the cantilever region and at the bottom of the building are expected to have a larger axial load. For the regions away from the cantilever region, the loads at the peripheral columns should be less than internal column as internal columns need to support larger loads from slabs and beams.

5.1.1. Change in Column Size across Floors

The change in column section across floors was considered after studying the general load path. As the axial load of columns at the top of the building is generally smaller than that at the bottom of the building, so that a smaller column size can be adopted for the columns at the top of the building. By considering this, the materials used can be minimised, which can also reduce the dead load of the entire structure. According to proposal one, the bottom truss system transferring forces from the cantilever region runs up to the fourth floor of the building. To maintain a constant cross-sectional property for the calculation of truss actions, it was decided that the section properties of columns running from the ground floor to the fourth floor to be unchanged, while smaller sections were considered for columns from the fourth floor to the roof.

5.1.2. Different Column Size for Internal and Peripheral Columns

Besides, smaller cross sections were also considered for columns at the peripheral region. This was because the tributary areas for slabs supporting by the peripheral columns are smaller than those for internal columns, so that a smaller cross section could be considered since the cross section and buckling resistance required are smaller.

5.1.3. Different Column Size for Cantilever Region

For the cantilever region, there are three columns fall within the cantilever region. For hand calculation, the loads of these columns were assumed to have transferred equally to the two columns at each end of the truss system as the axial stiffness (EA/L) of columns are larger than truss member. As such, each of the two columns

need to carry 2.5 times of the original load. However, part of the loads of the cantilever region were also spread to the next row of columns behind the cantilever region as per the SAP2000 analysis. The values were found to be similar with around 10% error. To be conservative, a larger value was adopted for the analysis to produce the size of the section. Details regarding the size of different columns can be referred to Figures 5.1 and 5.2 in the later sub-section.

5.1.4. Different Column Size for Wind Bracing Bays

However, when studying the wind bracing system, the cross-sectional area of columns affects the moment of inertia, which was used for the calculation of flexural component of the building drift. The equation is shown below:

$$I = 2A_c({}^{L}/_{2})^2 = {}^{A_c L^2}/_{2}$$

, where I, A_c, and L denote the moment of inertia of the wind bracing system, cross-sectional area of columns and the span of wind bracing bay respectively.

To maintain a sufficient moment of inertia of the bracing system such that the bracing is stiff enough to withstand the lateral wind loads and EHF, a larger cross-sectional area for the columns at the bracing bays was adopted.

5.2. Determination of Types of Columns used

Composite columns were once considered when the axial loads calculated were found to be exceptionally large as they offer a larger capacity. Otherwise, a very large section of I-columns might be needed, which may not be practical in construction and the effective area of the floor may be reduced. It may also not be cost-effective to use a very large I-section. However, composite columns were found to be unnecessary as the largest axial load obtained was only around 19000kN (Proposal 1) and 21721kN (Proposal 2), so UKC sections were adopted for all columns.

5.3. Overall Design Procedures

With reference to the above considerations for the design of columns, the resistance of columns was checked to ensure the capacity of proposed cross sections is sufficient to withstand the required axial loads.

In this design, the design calculations follow BSEN1993-1-1. ULS checks for both cross-section resistance and buckling resistance were performed. As pinned connections were assumed for columns, effective length of columns was taken as the actual length of columns. Bending resistance and combined axial and bending were not checked due to the assumption of pinned connections. Owing to the fact

that only axial resistance was checked, only load case 1 was considered, that is $1.35G_k + 1.5Q_k$ + EHF, as it gives the largest vertical axial force.

5.4. Summary of Results

After performing the calculations, the size of columns obtained is summarised in the table:

Table 5.1: Table showing the proposed column sizes

Region	Column Size (UKC)
Internal Columns	
Roof to 4/F	356 x 368 x 202 (Grey)
G/F to 4/F	356 x 406 x 467 (Green)
Peripheral Columns	
Roof to 4/F	356 x 368 x 202 (Grey)
G/F to 4/F	356 x 406 x 235 (Brown)
Cantilever Region	
Roof to 4/F	356 x 368 x 202 (Grey)
G/F to 4/F	356 x 406 x 634 (Yellow)
Wind Bracing Bay	
All Floors	356 x 406 x 1202 (Purple)

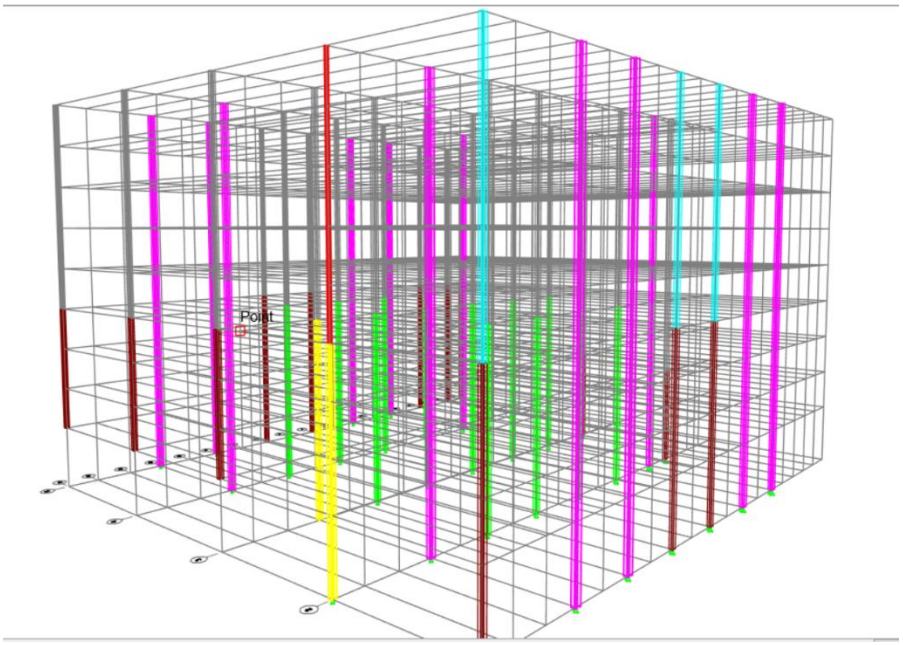


Figure 5.1: Figure showing the Proposed Column Sizes (Proposal 1)

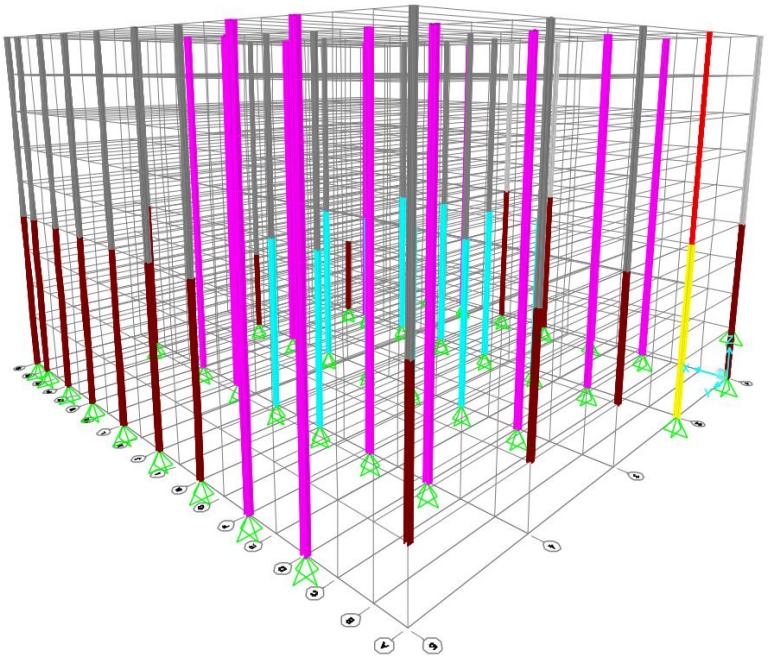


Figure 5.2: Figure showing the Proposed Column Sizes (Proposal 2)

For Proposal 2, the maximum design load of around 21721kN was found at one of the columns of the wind bracing bays near the cantilever region. For other columns, the design loads were found to be smaller than the capacity of 19744kN for UKC columns 356 x 406 x 634. Therefore, the same set of column sizes was adopted for Proposal 2, except that the location of wind bracing bays have been altered.

5.5. Calculation Results

The detailed calculation for each type of columns mentioned above are detailed from the next page. Note that the axial load of Proposal 2 was adopted for check for column UKC 356 x 406 x 1202 as it was more critical.

Design Load for Internal Columns (Roof - 4th Floor)

The slab is designed as one-way slab.

Tributary Area = 94.5 m^2

Assume ComFlor 60 Composite Slab of 150 mm

Weight of ComFlor Slab = 2.84 kPa

Assume 533x210x122 secondary beam,

Mass of Sec. Beam = 122 kg/m

Length of Sec. Beam supported by a column= 27.0 m/column

SW of Sec. Beam on each column = 32 kN/column

Assume 533x210x122 primary beam,

Mass of Primary Beam supported by a column= 122 kg/m

Length of Primary Beam = 7 m/column

SW of Primary Beam on each column = 8 kN/column

As the axial load is the largest at the bottom of columns

Max. Dead Load on a Column = 3150 kN

Max. Live Load on a Column = 1654 kN

Load Case

Load Case 1: 1.35 G_k + 1.5 Q_k

It is the most critical load case as the vertical forces are the greatest.

Factored Dead Load on a Column = 4252 kN

Factored Live Load on a Column = 2481 kN

Design of Compressive Columns

Section 356 x 368 x 202 UC

Section Properties

Grade 355

f_y 355 MPa

h 374.6 mm

b 374.7 mm

t_w 16.5 mm

t_f 27 mm

r 15.2 mm

A 257 cm^2

E 210 GPa

Axis	y-y	z-z
I (cm^4)	66300	23700

Wpl (cm3)	3970	1920	*Class 1-2*
Wel (cm3)	3540	1260	*Class 3*

Classification of Steel Sections (Section 5.5.2 of BSEN1993-1-1)

ε	0.81			
For Web,	c=h-2t_f-2r			
c	290.2	mm		
c/t	17.59	=	21.62	ε
Class	**1**	**web.**		
For Flange	c=b/2-t_w/2-r			
c	163.9			
c/t	6.07	=	7.46	ε
Class	**1**	**flange.**		
Overall Class	**1**	**section.**		

Design Load

N_{Ed}	6733	kN
$M_{y,Ed}$	0	kNm
$M_{z,Ed}$	0	kNm

Cross-Section Resistance (Section 6.2.4 of BSEN1993-1-1)

$N_{c,Rd} = Af_y/\gamma_{M0}$ =	9123.5	kN	
Because $N_{c,Rd}$	>=	$N_{c,Ed}$	**OK**

Buckling Resistance (Section 6.3.1 of BSEN1993-1-1)

Axis	y-y	z-z	
L_{cr} (m)	4	4	
$N_{cr} = \pi^2EI/L^2$(kN)	85884.06	30700.6	
$\lambda = (Af_y/N_{cr})^{0.5}$	0.33	0.55	
Buckling Curve	b	c	*Table 6.2*
α	0.34	0.49	
$\Phi = 0.5[1+\alpha(\lambda-0.2)+\lambda^2)$	0.57	0.73	
$\chi = 1/[\Phi+(\Phi^2-\lambda^2)^{0.5}]$	0.95	0.82	
$N_{b,Rd} = \chi Af_y/\gamma_{M1}$ (kN)	8708.56	7457.62	

Because $N_{b,Rd}$	>=	$N_{b,Ed}$	**OK**

Design Load for Internal Columns (Bottom - 4th Floor)

The slab is designed as one-way slab.

Tributary Area = 94.5 m^2

Assume ComFlor 60 Composite Slab of 150 mm

Weight of ComFlor Slab = 2.84 kPa

Assume 533x210x122 secondary beam,

Mass of Sec. Beam = 122 kg/m

Length of Sec. Beam supported by a column= 27.0 m/column

SW of Sec. Beam on each column = 32 kN/column

Assume 533x210x122 primary beam,

Mass of Primary Beam supported by a column= 122 kg/m

Length of Primary Beam = 7 m/column

SW of Primary Beam on each column = 8 kN/column

As the axial load is the largest at the bottom of columns

Max. Dead Load on a Column = 5735 kN

Max. Live Load on a Column = 3166 kN

Load Case

Load Case 1: 1.35 G_k + 1.5 Q_k

It is the most critical load case as the vertical forces are the greatest.

Factored Dead Load on a Column = 7743 kN

Factored Live Load on a Column = 4749 kN

Design of Compressive Columns

Section 356 x 406 x 467 UC

Section Properties

Grade	355	
f_y	335	MPa
h	436.6	mm
b	412.2	mm
t_w	35.8	mm
t_f	58	mm
r	15.2	mm
A	595	cm^2
E	210	GPa

Axis	y-y	z-z
I (cm^4)	183000	67800

Wpl (cm3)	10000	5030	*Class 1-2*
Wel (cm3)	8380	3290	*Class 3*

Classification of Steel Sections (Section 5.5.2 of BSEN1993-1-1)

ε	0.84			
For Web,	c=h-2t_f-2r			
c	290.2	mm		
c/t	8.11	=	9.68	ε
Class	**1**	**web.**		
For Flange	c=b/2-t_w/2-r			
c	173			
c/t	2.98	=	3.56	ε
Class	**1**	**flange.**		
Overall Class	**1**	**section.**		

Design Load

N_{Ed}	12491	kN
$M_{y,Ed}$	0	kNm
$M_{z,Ed}$	0	kNm

Cross-Section Resistance (Section 6.2.4 of BSEN1993-1-1)

$N_{c,Rd} = Af_y/\gamma_{M0}$ =	19932.5	kN	
Because $N_{c,Rd}$	>=	$N_{c,Ed}$	**OK**

Buckling Resistance (Section 6.3.1 of BSEN1993-1-1)

Axis	y-y	z-z	
L_{cr} (m)	6	6	
$N_{cr} = \pi^2EI/L^2$(kN)	105358.0	39034.2	
$\lambda = (Af_y/N_{cr})^{0.5}$	0.43	0.71	
Buckling Curve	b	c	*Table 6.2*
α	0.34	0.49	
$\Phi = 0.5[1+\alpha(\lambda-0.2)+\lambda^2)$	0.63	0.88	
$\chi = 1/[\Phi+(\Phi^2-\lambda^2)^{0.5}]$	0.91	0.72	
$N_{b,Rd} = \chi Af_y/\gamma_{M1}$ (kN)	18177.59	14264.43	
Because $N_{b,Rd}$	>=	$N_{b,Ed}$	**OK**

Design Load for Peripheral Columns (Roof - 4th Floor)

The slab is designed as one-way slab.

Tributary Area = 49 m^2

Assume ComFlor 60 Composite Slab of 150 mm

Weight of ComFlor Slab = 2.84 kPa

Assume 533x210x122 secondary beam,

Mass of Sec. Beam = 122 kg/m

Length of Sec. Beam supported by a column= 14.0 m/column

SW of Sec. Beam on each column = 17 kN/column

Assume 533x210x122 primary beam,

Mass of Primary Beam supported by a column= 122 kg/m

Length of Primary Beam = 7 m/column

SW of Primary Beam and Cladding on each column = 45 kN/column

As the axial load is the largest at the bottom of columns

Max. Dead Load on a Column = 1674 kN

Max. Live Load on a Column = 858 kN

Load Case

Load Case 1: 1.35 G_k + 1.5 Q_k

It is the most critical load case as the vertical forces are the greatest.

Factored Dead Load on a Column = 2259 kN

Factored Live Load on a Column = 1286 kN

Design of Compressive Columns

Section 356 x 368 x 202 UC

Section Properties

Grade	355	
f_y	355	MPa
h	374.6	mm
b	374.7	mm
t_w	16.5	mm
t_f	27	mm
r	15.2	mm
A	257	cm^2
E	210	GPa

Axis	y-y	z-z

I (cm^4)	66300	23700	
Wpl (cm3)	3970	1920	*Class 1-2*
Wel (cm3)	3540	1260	*Class 3*

Classification of Steel Sections (Section 5.5.2 of BSEN1993-1-1)

ε	0.81			
For Web,	$c=h-2t_f-2r$			
c	290.2	mm		
c/t	17.59	=	21.62	ε
Class	**1**	**web.**		
For Flange	$c=b/2-t_w/2-r$			
c	163.9			
c/t	6.07	=	7.46	ε
Class	**1**	**flange.**		
Overall Class	**1**	**section.**		

Design Load

N_{Ed}	3546	kN
$M_{y,Ed}$	0	kNm
$M_{z,Ed}$	0	kNm

Cross-Section Resistance (Section 6.2.4 of BSEN1993-1-1)

$N_{c,Rd} = Af_y/\gamma_{M0}$ =	9123.5	kN	
Because $N_{c,Rd}$	>=	$N_{c,Ed}$	**OK**

Buckling Resistance (Section 6.3.1 of BSEN1993-1-1)

Axis	y-y	z-z	
L_{cr} (m)	6	6	
$N_{cr} = \pi^2EI/L^2$(kN)	38170.70	13644.7	
$\lambda = (Af_y/N_{cr})^{0.5}$	0.49	0.82	
Buckling Curve	b	c	*Table 6.2*
α	0.34	0.49	
$\Phi = 0.5[1+\alpha(\lambda-0.2)+\lambda^2)$	0.67	0.99	
$\chi = 1/[\Phi+(\Phi^2-\lambda^2)^{0.5}]$	0.89	0.65	
$N_{b,Rd} = \chi Af_y/\gamma_{M1}$ (kN)	8111.72	5939.74	
Because $N_{b,Rd}$	>=	$N_{b,Ed}$	**OK**

Design Load for Peripheral Columns (Bottom - 4th Floor)

The slab is designed as one-way slab.

Tributary Area =	49	m^2
Assume ComFlor 60 Composite Slab of	150	mm
Weight of ComFlor Slab =	2.84	kPa

Assume 533 x 210 x 122 secondary beam,

Mass of Sec. Beam =	122	kg/m
Length of Sec. Beam supported by a column=	14.0	m/column
SW of Sec. Beam on each column =	17	kN/column

Assume 533x312x273 primary beam,

Mass of Primary Beam supported by a column=	273.3	kg/m
Length of Primary Beam =	7	m/column
SW of Primary Beam on each column =	55	kN/column

As the axial load is the largest at the bottom of columns

Max. Dead Load on a Column =	3025	kN
Max. Live Load on a Column =	1642	kN

Load Case

Load Case 1: 1.35 G_k + 1.5 Q_k

It is the most critical load case as the vertical forces are the greatest.

Factored Dead Load on a Column =	4083	kN
Factored Live Load on a Column =	2462	kN

Design of Compressive Columns

Section 356 x 406 x 235 UC

Section Properties

Grade	355	
f_y	355	MPa
h	381	mm
b	394.8	mm
t_w	18.4	mm
t_f	30.2	mm
r	15.2	mm
A	299	cm^2
E	210	GPa

Axis	y-y	z-z
I (cm^4)	79100	31000

Wpl (cm3)	4690	2380	*Class 1-2*
Wel (cm3)	4150	1570	*Class 3*

Classification of Steel Sections (Section 5.5.2 of BSEN1993-1-1)

ε	0.81			
For Web,	$c=h-2t_f-2r$			
c	290.2	mm		
c/t	15.77	=	19.38	ε
Class	**1**	**web.**		
For Flange	$c=b/2-t_w/2-r$			
c	173			
c/t	5.73	=	7.04	ε
Class	**1**	**flange.**		
Overall Class	**1**	**section.**		

Design Load

N_{Ed}	6546	kN
$M_{y,Ed}$	0	kNm
$M_{z,Ed}$	0	kNm

Cross-Section Resistance (Section 6.2.4 of BSEN1993-1-1)

$N_{c,Rd} = Af_y/\gamma_{M0}$ =	10614.5	kN	
Because $N_{c,Rd}$	>=	$N_{c,Ed}$	**OK**

Buckling Resistance (Section 6.3.1 of BSEN1993-1-1)

Axis	y-y	z-z	
L_{cr} (m)	6	6	
$N_{cr} = \pi^2EI/L^2$(kN)	45540.00	17847.5	
$\lambda = (Af_y/N_{cr})^{0.5}$	0.48	0.77	
Buckling Curve	b	c	*Table 6.2*
α	0.34	0.49	
$\Phi = 0.5[1+\alpha(\lambda-0.2)+\lambda^2)$	0.66	0.94	
$\chi = 1/[\Phi+(\Phi^2-\lambda^2)^{0.5}]$	0.89	0.68	
$N_{b,Rd} = \chi Af_y/\gamma_{M1}$ (kN)	9465.61	7220.51	
Because $N_{b,Rd}$	>=	$N_{b,Ed}$	**OK**

Design Load for Columns near Cantilever Region (Proposal One)

There are three columns in cantilever region.
Assuming the loads are transferred to the two adjacent columns, each column takes 2.5 times of the normal load of a peripheral column. Based on the assumption, the load is 16365kN.
However, the loads were found to be spread to the row of columns behind the cantilever region.
To be conservative, a larger load from SAP2000 (19022kN) was taken for the analysis.

Design of Compressive Columns

Section 356 x 406 x 634 UC

Section Properties

Grade	355	
f_y	335	MPa
h	474.6	mm
b	424	mm
t_w	47.6	mm
t_f	77	mm
r	15.2	mm
A	808	cm^2
E	210	GPa

Axis	y-y	z-z	
I (cm^4)	275000	98100	
Wpl (cm3)	14200	7110	*Class 1 and 2*
Wel (cm3)	11600	4630	*Class 3*

Classification of Steel Sections (Section 5.5.2 of BSEN1993-1-1)

ε	0.84		
For Web,	$c=h-2t_f-2r$		
c	290.2	mm	
c/t	6.10	= 7.28 ε	7.28
Class	**1**	**web.**	
For Flange	$c=b/2-t_w/2-r$		
c	173		
c/t	2.25	= 2.68 ε	2.68
Class	**1**	**flange.**	
Overall Class	**1**	**section.**	

Design Load

N_{Ed}	19022	kN

$M_{y,Ed}$	0	kNm
$M_{z,Ed}$	0	kNm

Cross-Section Resistance (Section 6.2.4 of BSEN1993-1-1)

$N_{c,Rd} = Af_y/\gamma_{M0}$ =	27068	kN	
Because $N_{c,Rd}$	>=	$N_{c,Ed}$	**OK**

Buckling Resistance (Section 6.3.1 of BSEN1993-1-1)

Axis	y-y	z-z	
L_{cr} (m)	6	6	
$N_{cr} = \pi^2EI/L^2$(kN)	158324.90	56478.81	
$\lambda = (Af_y/N_{cr})^{0.5}$	0.41	0.69	
Buckling Curve	b	c	*Table6.2*
α	0.34	0.49	
$\Phi = 0.5[1+\alpha(\lambda-0.2)+\lambda^2)$	0.62	0.86	
$\chi = 1/[\Phi+(\Phi^2-\lambda^2)^{0.5}]$	0.92	0.73	
$N_{b,Rd} = \chi Af_y/\gamma_{M1}$ (kN)	24921.23	19744.9	

Because $N_{b,Rd}$	>=	$N_{b,Ed}$	**OK**

Design Load for Columns at Cantilever and Wind Bracing (Proposal 2)

There are three columns in cantilever region.
Assuming the loads are transferred to the two asdjacent columns, each column takes 2.5 times of the normal load of a peripheral column.
Based on the assumption, the load is 16365kN and the value from SAP2000 is 21721kN.
To be conservative, take load as 21721kN for analysis.

Design of Compressive Columns

Section 356 x 406 x 1202 UC

Section Properties

Grade	355	
f_y	335	MPa
h	580	mm
b	471	mm
t_w	95	mm
t_f	130	mm
r	15.4	mm
A	1531	cm^2
E	210	GPa

Axis	y-y	z-z

I (cm^4)	664000	229000	
Wpl (cm3)	30000	15200	*Class 1-2*
Wel (cm3)	22900	9710	*Class 3*

Classification of Steel Sections (Section 5.5.2 of BSEN1993-1-1)

ε	0.84	
For Web,	$c=h-2t_f-2r$	
c	289.2	mm
c/t	3.04	= 3.63 ε
Class	**1**	**web.**
For Flange	$c=b/2-t_w/2-r$	
c	172.6	
c/t	1.33	= 1.59 ε
Class	**1**	**flange.**
Overall Class	**1**	**section.**

Design Load

N_{Ed}	21721	kN
$M_{y,Ed}$	0	kNm
$M_{z,Ed}$	0	kNm

Cross-Section Resistance (Section 6.2.4 of BSEN1993-1-1)

$N_{c,Rd} = Af_y/\gamma_{M0}$ = 51288.5 kN

Because $N_{c,Rd}$ >= $N_{c,Ed}$ **OK**

Buckling Resistance (Section 6.3.1 of BSEN1993-1-1)

Axis	y-y	z-z	
L_{cr} (m)	6	6	
$N_{cr} = \pi^2EI/L^2$(kN)	382282.68	131841.4	
$\lambda = (Af_y/N_{cr})^{0.5}$	0.37	0.62	
Buckling Curve	b	c	*Table 6.2 of BSEN1993-1-1*
α	0.34	0.49	
$\Phi = 0.5[1+\alpha(\lambda-0.2)+\lambda^2)$	0.60	0.80	
$\chi = 1/[\Phi+(\Phi^2-\lambda^2)^{0.5}]$	0.94	0.77	
$N_{b,Rd} = \chi Af_y/\gamma_{M1}$ (kN)	48172.33	39555.86	

Because $N_{b,Rd}$ >= $N_{b,Ed}$ **OK**

6. Wind Bracing Design

The design follows the principle of "simple construction", so beams and columns are assumed to be pinned together, which column will be designed to resist bending moments from beams' eccentric reactions. Bracing system in the building will be designed to resist horizontal force. There are two types of bracing systems resisting horizontal and vertical forces. In this design, vertical bracing system will be used to mainly provide resistance to wind loading. According to Brown, Iles &Yandzio (2009), there should be at least three vertical bracing planes in a multi-storey building. The structure (proposal 1) of 9-storey building in the design was supported by 8 vertical planes of bracing as shown in Figure 6.1. Considering negative effect of disproportionate collapse, each bracing plane is facing the other plane. Since the building is not symmetrical from left to right, the four east-west bracings are set inside the building to achieve symmetry.

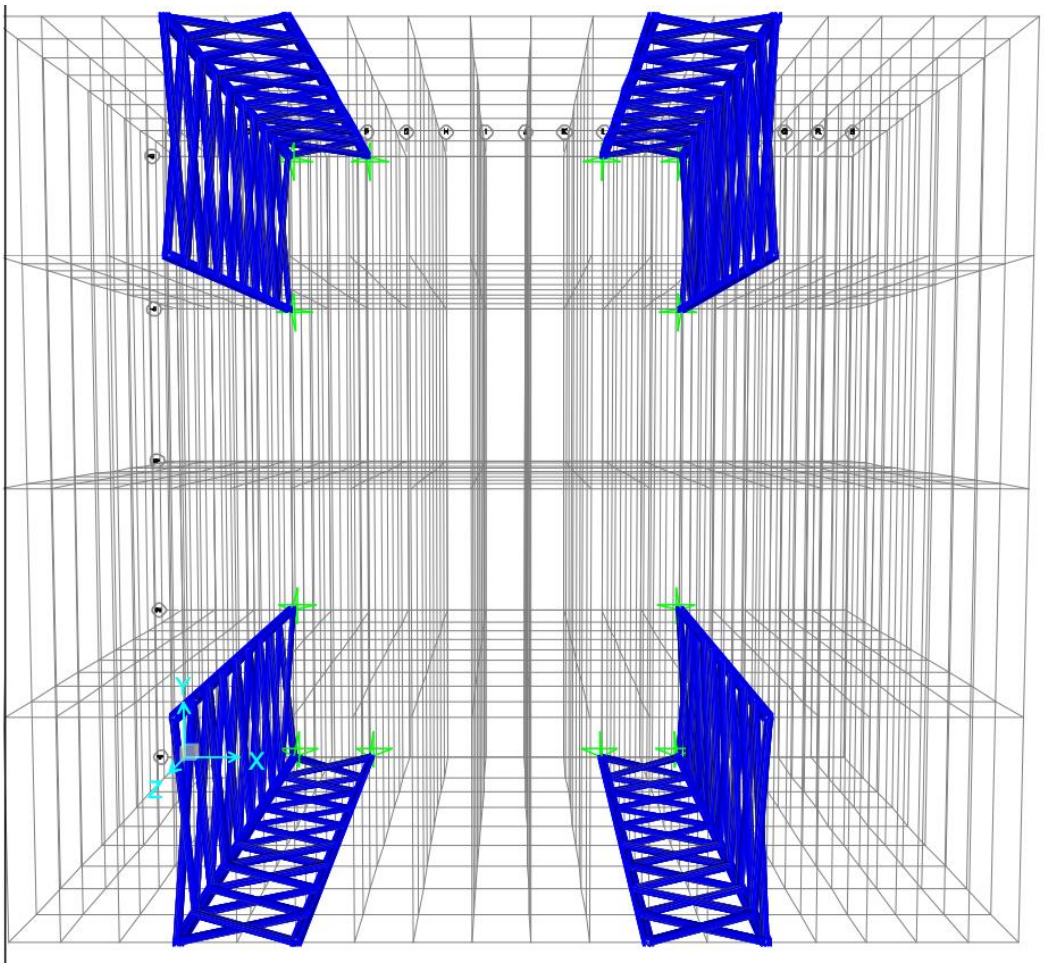


Figure 6.1 Designed location for 8 bracing systems (Plan View)

The bracing system is supported by crossed diagonals which is a typical bracing

system called X-bracing or Cross bracing. In the figure 6.2, when the lateral force caused by wind load on the structure, one of the braces is in tension and another is in compression. The resistance of the bracing must be larger than second order effects caused by sway, wind loads, and horizontal forces caused by frame imperfections (EHF). The stability on resistance to sway can be shown roughly from the shape of the structure. Wide bracing has high stability on resistance to sway and the horizontal deflection of the system is smaller. The angle from horizontal for the bracing in the design is from 15.9° to 40.6° and this is caused by different storey height and different bracing width.

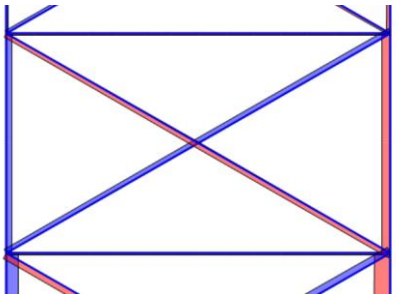


Figure 6.2 X- bracing used in the design (Blue is tension and red is compression)

The type of bracing for proposal 1 and 2 is the same, but north-south bracing frames is inside the building which reduce the cladding load from perimeter beams. The calculation in the report will check the critical load in proposal 1.

6.1. Actions of the whole building

To choose and check the frame sections for the bracing system, actions with different combinations should be calculated first. As what has been calculated before, the permanent action and variable action for both roof and floors are known. When calculating these unfactored actions, the permanent action of the vertical loads should include self-weight of columns and beams. All four north-south vertical braced frames are all located on the perimeter of the building, so they will additionally carry cladding load of 1.3 kN/m^2. East-west direction bracing is inside the building, so variable action is simply 4 kN/m^2.

For horizontal loads, wind pressure is the only external load on the system. The north-south bracing system will bear wind load from east and west, so the critical wind load is on the projected area of the building. Figure 6.3 shows the specific area used in the calculation for two directional bracings. The projected area of east wall is the same with the circled west wall. For getting the critical load of the east-west bracings, north wall's area which is the largest projected area should be taken in the calculation.

Then, characteristic wind load on the face of the building is the product of wind

pressure and projected area with a wind force coefficient 1.1. Since four bracings is designed to resist the horizontal loads, the load on each bracing should be divided by 4.

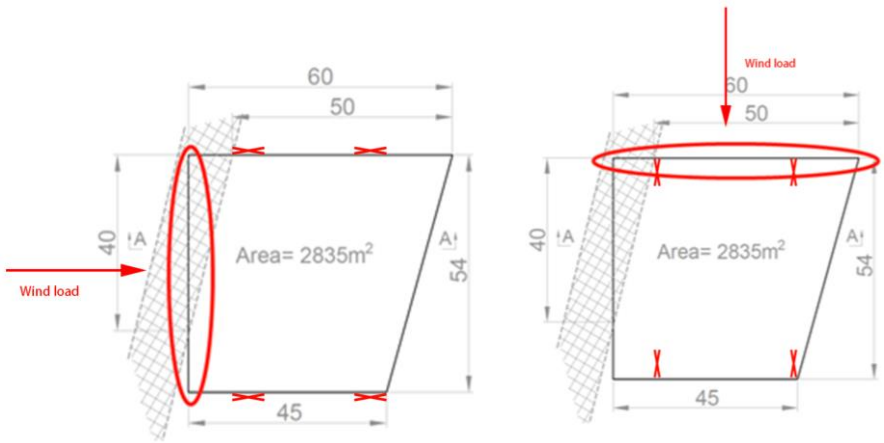


Figure 6.3 Plan view showing projected area of the four north-south (left) and east-west (right) bracings

6.2. ULS with combination of actions

There are four load combinations required in the project and only three of the load combinations are considered when checking the bracing systems because the function of vertical bracing is wind resistance. The first combination does not have wind load (Wk) so there is no need for check.

1.35Gk + 1.50Qk +EHF ×

1.35Gk + 1.50Qk + 0.75Wk + EHF √ (Combination 1 and 4)

1.35Gk + 1.05Qk + 1.50Wk + EHF √ (Combination 2 and 5)

1.0Gk + 1.50Wk + EHF√ (Combination 3 and 6)

Two equations show two ways of actions combination:

$\sum_{j\geq 1}\gamma_{\mathrm{G,j}}G_{k,j}+\gamma_{\mathrm{p}}P+\gamma_{\mathrm{Q},1}\psi_{0,1}Q_{k,1}+\sum_{i>1}\gamma_{\mathrm{Q,i}}\psi_{0,i}Q_{k,1}$
6.10a

$\sum_{j\geq 1}\xi_j\gamma_{\mathrm{G,j}}G_{k,j}+\gamma_{\mathrm{p}}P+\gamma_{\mathrm{Q},1}Q_{k,1}+\sum_{i>1}\gamma_{\mathrm{Q,i}}\psi_{0,i}Q_{k,1}$
6.10b

BS EN 1990, 6.4.3.2.

ψ_0 is applied to all variable actions in 6.10a because they are considered as

accompanying action. In 6.10b, there is leading variable action without factors and permanent action with reduction factor ξ. With factors in 3.2.6.2, the actions can be clearly shown in the table below:

Table 6.1 Combination of actions

Combination	Permanent actions		Leading variable action	Accompanying variable actions	
	Unfavourable	Favourable		Main	Others
6.10 a	$1.35G_{kj,sup}$	$1.00G_{kj,inf}$		$1.5\psi_{0,1}Q_{k,1}$	$1.5\psi_{0,i}Q_{k,j}$
6.10 b	$0.925 \times 1.35G_{kj,sup}$	$1.00G_{kj,inf}$	$1.5Q_{k,1}$		$1.5\psi_{0,i}Q_{k,j}$

Value of ψ for Category B: office areas is 0.7, so 6.10b is more onerous than 6.10a unless permanent load is 4.5 times of the imposed loads.

6.3. Design values of actions

Wind is assumed to be the leading action when checking resistance of bracing.

There are two directions bracing frames and three combinations for each of them. Combination 1, 2 and 3 is for north-south direction and combination 4, 5 and 6 is for east-west direction. The final vertical loads for all combinations are shown in the table below:

Table 6.2 Actions with all combinations

Combination		Vertical loads	Horizontal Loads
1 & 4	Roof	$1.25G + 0.75Q_{imp}$	$0.75Q_w$
	Floor	$1.25G + 1.05Q_{imp}$	
2 & 5	Roof	$1.25G + 0.53Q_{imp}$	$1.5Q_w$
	Floor	$1.25G + 0.74Q_{imp}$	
3 & 6	Roof	$0.925G$	$1.5Q_w$
	Floor	$0.925G$	

Design value of wind load:

In the calculation, roof and floor truss carry the load as shown in Figure 6.4. The roof truss is considered to bear the load of the upper half of the floor, and floor truss bears half the wind load of both upper and lower floors.

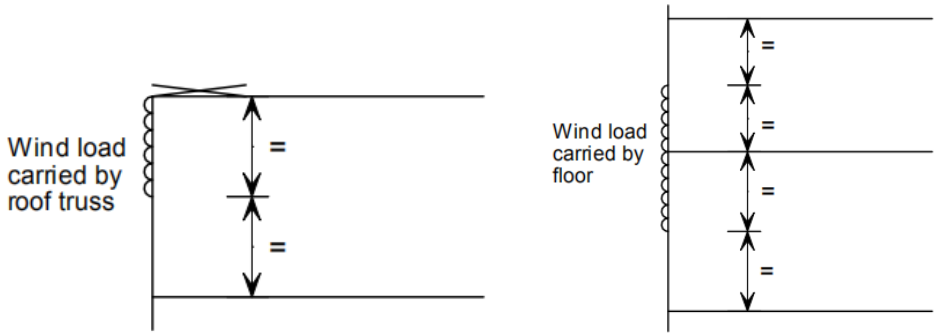


Figure 6.4 The way wind load applying on the roof and floors. Retrieved from "Steel Building Design: Medium rise braced frames" by The Steel Construction Institute (p.21), 2009

Wind leading combination calculated above can be used to check bracing system and a clear table is listed below:

Table 6.3 North-south final actions

	$G\ (kN/m^2)$	$Q_{imp}\ (kN/m^2)$	LC1 (kN/m^2)	LC2 (kN/m^2)	LC3 (kN/m^2)
Roof	7.09	1.5	9.98	9.64	6.56
Floor	7.21	5.3	14.57	12.90	6.67

Table 6.4 East-west final actions

	$G\ (kN/m^2)$	$Q_{imp}\ (kN/m^2)$	LC1 (kN/m^2)	LC2 (kN/m^2)	LC3 (kN/m^2)
Roof	7.09	1.5	9.98	9.64	6.56
Floor	7.21	4	13.20	11.94	6.67

6.4. Sway stiffness

To check the sway stiffness of bracing system, equivalent horizontal force (EHF) is the last force needed to calculate. EHF is used to replace sway structure's vertical geometrical deformation due to imperfection. There are two reasons: One is that it is simpler to adding force than change the geometry of the structure. Another reason is that the force on different floors has no effect when length changing. EHF will vary with different load combination because its design value is ϕN_{Ed}, where ϕ is global initial sway imperfections with equation:

BS EN 1993-1-1, 5.3.2(7)

$$\phi = \phi_0 \alpha_h \alpha_m$$

Where ϕ_0 is 1/200. For the bracing, α_h and α_m is 1 for being treated conservative and since ϕ is 1/200. N_{Ed} is the total vertical design forces.

BS EN 1993-1-1 5.3.2(3)

With all horizontal loads and forces calculated, the point force on each floor can be determined and it can be used to figure out deflection of the bracing system.

Vertical load on floor and on roof is different because they have different loads, but the area of each level is the same. For per bracing plane, the design value must be divided by 4. This is because all the four bracing planes are in the orthogonal direction, the horizontal force is assumed to be equally divided by these four bracing systems.

After calculating all the load needed for deflection, the size of bracing member can be selected:

Assumed column: UKC $356 \times 406 \times 1202$

Assumed beam: UKC $914 \times 419 \times 388$

Assumed bracing: CHS 508×14.2

6.4.1. Deflection with elastic analysis

Deflection can be used to show the stiffness of the bracing system and it can be calculated by adding horizontal force on the bracing plane. It can be ideally divided into shear deformation and flexural deformation where shear deformation is the main reason for the deflection. Shear is resisted by the diagnoals part of the frame and moment is resisted by the column part. As what mentioned in the beginning of 3.2.6, all joints in the frame is pinned which keeps the same with the whole building.

Figure 6.5 shows how shear deformation happens in the frame and equation below can show the calcualtion process:

$$\Delta_{sh} = \frac{F L_d}{A_d E} \left(\frac{L_d}{L}\right)^2$$

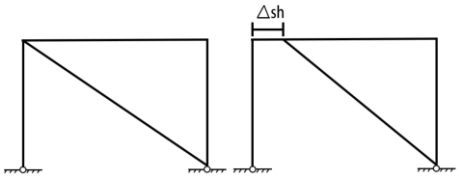


Figure 6.5 Shear deformation

Shear deformation doesn't consider flexural deformation, so flexural deformation is calculated by trearing the frame a cantilever. For getting more accurate result, a refined method is used by determining bending moment seperataly for each floor.

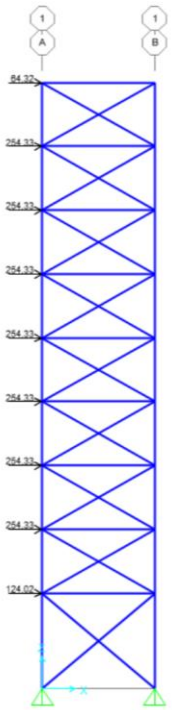


Figure 6.6 Deflection due to horizontal forces H_{Ed}

With actions shown in figure 6.6, sepcific calculation method and the table of deflections for all combinations are shown:

Table 6.5 Deflections calculated with different method

Combination	Hand calculated deflection (mm)	Software calculated deflection (mm)
1	32.26	30.65
2	56.35	53.57
3	53.33	50.70
4	23.14	21.60
5	41.34	38.70
6	39.59	37.10

Inertia I: $I = \frac{AL^2}{2}$ Where A is the section area of column in the frame and L is the width of the frame.

Moment: The moment for each floor is different and lever length for each force is different. For the 9th storey in figure 6.7, the force is assumed on a cantilever from the middle of storey to the top of the sotrey, so L1 is half of the 9th sotrey height. Moment for 9th storey is:

$$M_9 = F1 \times L1$$

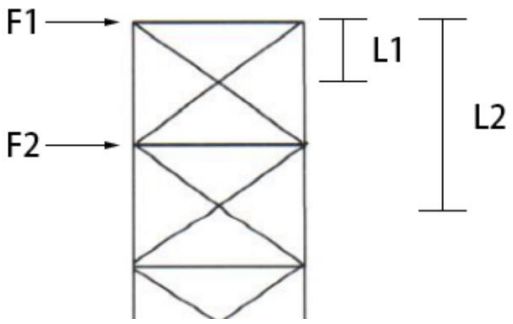


Figure 6.7 Method for calculating moment

There are two forces when calculating moment for 8th floor and two forces both have effect on the moment which shows moment is accumulating. So for 8th sotrey, moment is:

$$M_8 = F1 \times L1 + F2 \times L2$$

Here, L2 is from the top of the building to the middle of 8th storey. Next, the moment calculation method of each floor is analogous, and the moment should show a gradually increasing trend.

$\delta\theta_i$: It means change in inclination at each storey and it is figured out from equation for curvature:

$$\phi = \frac{\delta\theta}{h} = \frac{M}{EI}$$

$$\text{So, } \delta\theta_i = \frac{Mh}{EI}$$

Where M is calculated in the last step.

Storey inclination: Storey inclination is the sum of $\delta\theta_i$ up to required level, so it's increasing with higher floors. For instance:

2nd storey: $\sum \delta\theta_i = \delta\theta_1 + \delta\theta_2$

3rd storey: $\sum \delta\theta_i = \delta\theta_1 + \delta\theta_2 + \delta\theta_3$

Sotrey drift: $\theta_i h$ where h is height of each storey.

Shear drift: Shear drift uses the equation mentioned before, but it has to be divided by 2 for cross bracing because there are two diagonal bracing memebers which will provide stronger resistance:

$$\Delta_{sh} = \frac{1}{2}\frac{FL_d}{A_d E}\left(\frac{L_d}{L}\right)^2$$

Where F is the point force on each floor, L_d is the length of diagonal member, A_d is the cross section area of diagonal memebr, E is equal to 210000 MPa and L is width of the frame.

Total deflection δ: It adds storey drift and shear drift and it means deflection of each floor.

Deflection added together Δ: This is the sum of each deflection and it will attain the maximum on the 9th storey.

The value of deflection should be checked at seviceability limit state. For a sway frame resisting wind load, the maximum deflection for the whole building should

EN 1990, 3.4

be limited to H/600 which is 63mm for this 9-storey high building. From table 6.5, deflection of all combinations is smaller than the limitation.

With deflection and all horizontal actions, the sway stiffnesss can be checked with equation:

$$\alpha_{cr} = (\frac{H_{Ed}}{V_{Ed}})(\frac{h}{\delta_{H,Ed}})$$

BS EN 1993-1-1 5.2.1 (4)B Eqn (5.2)

Where H_{Ed} is sum of horizontal load and equivalent horizontal loads, V_{Ed} is vertical load on the bottom of each floor, h is the height of each floor, and $\delta_{H,Ed}$ is the horizontal deflection of the whole building which is on the top of the building.

α_{cr} got here is from first order analysis and there is possibility for second order analysis. Figure below shows difference of force from first and second order effects. There will be a amplifier for second order displacements and forces. Amplifier is $\frac{1}{1-1/\alpha_{cr}}$. To check whether use amplifier, BS EN 1993-1-1, 5.2.1(2) shows that if α_{cr} determined from elastic analysis with effects from first order is smaller than 10. This means deformations of the frame increase or modify structure significantly.

BS EN 1993-1-1, 5.2.1(2)

BS EN 1993-1-1 5.2.1 (3)

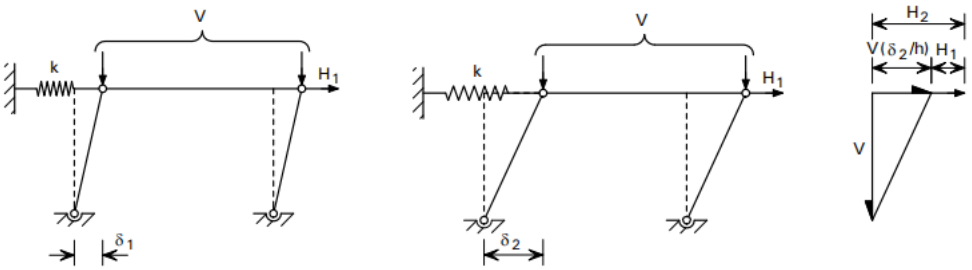


Figure 6.8 First and second order effects. Retrieved from "Steel Building Design: Medium rise braced frames" by The Steel Construction Institute (p.28), 2009

α_{cr} has to be calculated for all combinations.

Table 6.6 Value of α_{cr}

Combination	1	2	3	4	5	6
Worst α_{cr}	13	11.1	15.23	28.58	31.41	54.3

The worst case is larger than 10, so first order analysis alone is sufficient. Second order effects and load amplifier do not need to be considered.

6.5. Design of bracing

From the calculation for α_{cr}, H_{Ed} for each floor is figured out and it is found that H_{Ed} between 1st floor and the ground is the largest. This can be used to find critical design load in diagonal bracing member.

6.5.1. Axial force in diagonal member

With horizontal force, axial force can be calculated by times tilt angle of diagonal member. here are two diagonal members in one storey, so force in one diagonal member has to be divided by 2. Combination 1, 2 and 3 have the same structure of frame, so the buckling resistance is the same. The design force needed to be resist is largest one of this three combinations, which is $N_{Ed} = 2265.63\ kN$. The same for combination 4, 5 and 6, largest force to be resisted is $N_{Ed} = 2040.56\ kN$.

6.5.2. Member checks

Buckling length

The effective length factor for hollow sections of diagonal part is 1.0 because gusset plate connecting diagonal member and other part is flexible out of the plane.

Slenderness for flexural buckling

For axial compression in members, non-dimensional slenderness $\bar{\lambda}$ is:

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \left(\frac{L_{cr}}{i}\right)\left(\frac{1}{\lambda_1}\right) \quad \text{for Class 1, 2 and 3 cross-sections}$$

BSEN1993-1-1 6.3.1.3 (1) Eqn. (6.50)

Where L_{cr} is the buckling length calculated before and i is radius of gyration about the axis. λ_1 can be calculated with equation: $\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.9\varepsilon$ and $\varepsilon = \sqrt{\frac{235}{f_y}}$

For $\bar{\lambda} \leq 0.2$, the buckling effects can be ignored. However, $\bar{\lambda} > 0.2$, so reduction factor χ must be calculated for buckling resistance.

BSEN1993-1-1 6.3.1.2(4)

Design buckling resistance

Requirement for section:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0$$

BSEN1993-1-1 6.3.1.1(1) Eqn. (6.46)

Design buckling resistance is:

$$N_{b,Rd} = \frac{\chi\, A f_y}{\gamma_{M1}} \text{ (For Class 1, 2 and cross sections)}$$

BSEN1993-1-1 6.3.1.1(3) Eqn. (6.47)

Where $\gamma_{M1} = 1.0$, $\chi = \frac{1}{\phi^2 + \sqrt{\phi^2 - \bar{\lambda}^2}} \leq 1.0$ and $\phi = 0.5 \times (1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2)$

A is section area and $f_y = 355\ N/mm^2$.

BSEN1993-1-1 Table 6.2

From Table 6.2 and 6.1 in EN 1993-1-1, the buckling curve for any hot finished hollow section is "a" and imperfection factor α for curve a is 0.21.

BSEN1993-1-1 Table 6.1

6.5.3. Two additional imperfections checks

In addition to design buckling resistance check for the frames, two design situations caused by imperfections should also be taken into the consideration. One kind of imperfections happens at splices, and it generates the force local to the floor level. Another imperfection is subjected to the lateral force of the floor diaphragms. When checking these two imperfections, equivalent horizontal force is not included because imperfections can be considered separately. Check for two imperfections is because connection resistance may not be sufficient for beam shear and imperfections.

Verification of bracing adjacent to column splices

BS EN 1993-1-1, 5.3.3(4)

Both external load and lateral force at a splice are applied simultaneously on the bracing system when checking imperfections in the bracing frames. These forces are checked on the bracing members above and below the level of splices shown in figure 6.9. Since the direction of the force generated by splice cannot be determined, the force will be distributed to the diagonal members in figure 6.9to check.

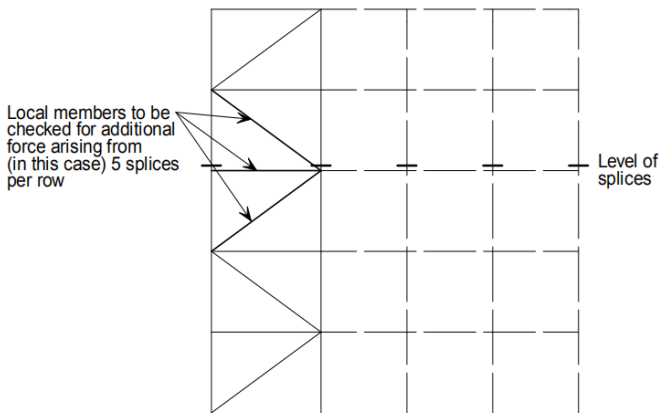


Figure 6.9 Check for bracing members. Retrieved from "Steel Building Design: Medium rise

braced frames" by The Steel Construction Institute (p.27), 2009

External loading

Without equivalent horizontal force, figure 6.10 shows the external loading (wind load) on the frame. Then, axial forces in the bracing elements can be calculated. The forces are accumulated to the lower level.

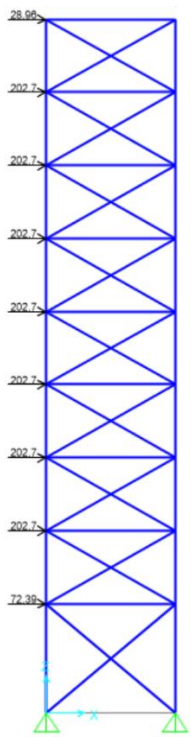


Figure 6.10 Bracing frame with only external load

The structure and sections of the bracing members is the same between 2nd to 1st floor and 1st floor to the ground, and the difference between their length is relatively small. It means that the critical load to be checked can use $833.80\ kN$ from 2nd floor to 1st floor and the whole structure can bear imperfection forces if resistance of bracing system between 2nd floor and 1st floor is sufficient.

Local lateral force is:

The total force to be resisted is the proportion of the total axial force in each column spliced at that level. There are 91 columns in one level of the building.

$$\alpha_m N_{Ed}/1000$$

Where $\alpha_m = \sqrt{0.5(1+\frac{1}{m})}$ and m is the number of columns to be restrained

The local force is assumed to be split equally between above and below bracing member in the figure below.

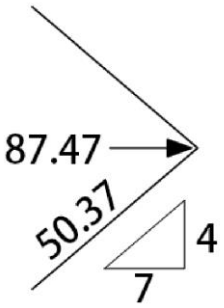


Figure 6.11 Splitting of local force

Verification of restraint to columns provided by bracing

BS EN 1993-1-1, 5.3.2(5)

Another imperfection of the structure is the horizontal forces of floor diaphragms which needs to be restrained by columns. As shown in figure 6.12, sway imperfection ϕ can be used to calculate horizontal force. Equation for the process is:

$$H = \phi N_{Ed}$$

Where has been mentioned in 3.2.6.4 sway stiffness: $\phi = \phi_0 \alpha_h \alpha_m$ and $\phi_0 = \frac{1}{200}\alpha_h = 1.0$ $\alpha_m = \sqrt{0.5(1+\frac{1}{m})} = \sqrt{0.5 \times (1+\frac{1}{91})} = 0.50$

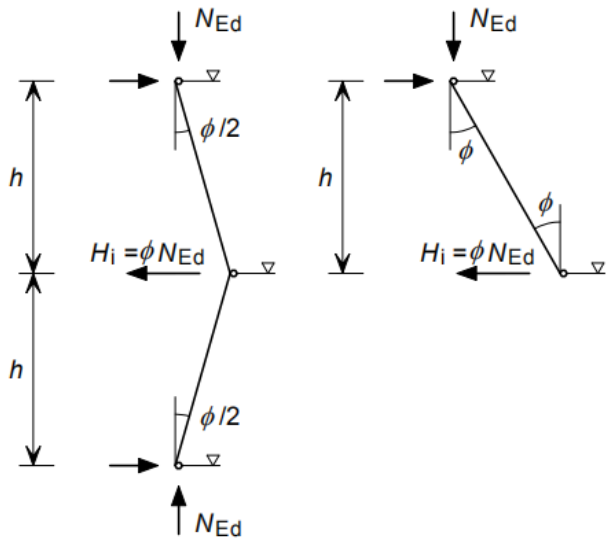


Figure 6.12 Determination of sway imperfection ϕ. Retrieved from "Steel Building Design: Medium rise braced frames" by The Steel Construction Institute (p.26), 2009

In conclusion, the section CHS 508×14.2 selected fulfil every limitation and the deflection limited is close to the requirement. Also, resistance to the various imperfections and external load is sufficient for each bracing member.

6.6. Calculation process

6.6.1. Actions of the whole building

North-south direction bracing:

Roof vertical loads:

Permanent action $g_{k,r} = 7.09\ kN/m^2$

Variable action $g_{k,r} = 1.5\ kN/m^2$

Floors vertical loads:

Permanent action $g_{k,r} = 7.21\ kN/m^2$

Variable action $g_{k,r} = 4 + 1.3 = 5.3\ \ kN/m^2$

Horizontal loads for both roof and floors:

Wind pressure $= 1.3\ kN/m^2$

Overall wind force coefficient $= 1.1$

The projected area of the vertical face of building (east-west direction) with consideration for bracing:$= 54 \times 38 = 2052\ m^2$

Characteristic wind load on the face of the building: $= 1.3 \times 1.1 \times 2052 = 2934.36\ kN$

Since four bracings is designed to resist the horizontal loads, the load on each bracing should be divided by $4 = 2934.36 \div 4 = 733.59\ kN$

East-west direction bracing:

Roof vertical loads:

Permanent action $g_{k,r} = 7.09\ kN/m^2$

Variable action $g_{k,r} = 1.5\ kN/m^2$

Floors vertical loads:

Permanent action $g_{k,r} = 7.21\ kN/m^2$

Variable action $g_{k,r} = 4\ kN/m^2$

Horizontal loads for both roof and floors:

Wind pressure $= 1.3\ kN/m^2$

Overall wind force coefficient $= 1.1$

The projected area of the vertical face of building (east-west direction) with consideration for bracing:$= 54 \times 60 = 2280\ m^2$

Characteristic wind load on the face of the building: $= 1.3 \times 1.1 \times 2280 = 3260.4\ kN$

Since four bracings is designed to resist the horizontal loads, the load on each bracing should be divided by $4 = 3260.4 \div 4 = 815.1\ kN$

6.6.2. Factors on actions

According to the National Annex to BS EN 1990. From NA.2.2.3.2 and Table

BS EN 1990 A1.3.1(4)

NA.A1.2:

Partial factors:

BS EN 1990 Table NA.A1.2(B)

Permanent actions (Unfavourable) $\gamma_{\mathrm{Gj,sup}} = 1.35$

Reduction factor for unfavourable permanent actions (6.10b) $\xi = 0.925$

Variable actions (Unfavourable) $\gamma_{\mathrm{Q,1}} = 1.5$

Imposed load on floors and wind load $\gamma_{\mathrm{Q,i}} = 1.5$

Factors on accompanying actions:

BS EN 1990 Table NA.A1.1

Imposed loads on buildings for Category B: Office areas $\psi_0 = 0.7$

Wind loads on buildings $\psi_0 = 0.5$

Snow loads (Altitude < 1000 m above sea level) $\psi_0 = 0.5$

Note that for favourable actions:

Permanent actions (Favourable) $\gamma_{\mathrm{Gj,sup}} = 1$

Variable actions (Favourable) $\gamma_{\mathrm{Q,1}} = 0$

Imposed load on floors and wind load $\gamma_{\mathrm{Q,i}} = 0$

6.6.3. ULS with combination of actions

Design values of actions

Wind is assumed to be the leading action when checking resistance of bracing.

There are two directions bracing frames and three combinations for each of them. Combination 1, 2, 3 is for north-south direction and combination 4, 5, 6 is for east-west direction.

Combination 1 of North-South direction (1.35Gk + 1.50Qk + 0.75Wk + EHF) with 6.10b

Roof vertical loads:

$$0.925 \times 1.35G + 1.5 \times 0.5Q_{imp} = 0.925 \times 1.35 \times 7.09 + 1.5 \times 0.5 \times 1.5$$
$$= 9.98\ kN/m^2$$

Floor vertical loads:

$$0.925 \times 1.35G + 1.5 \times 0.7Q_{imp} = 0.925 \times 1.35 \times 7.21 + 1.5 \times 0.5 \times 5.3$$
$$= 14.57\ kN/m^2$$

Horizontal loads:

Design value of total horizontal wind load per bracing system for combination 1 is:

$$0.75 \times 733.59 = 550.19\ kN$$

Roof level= $\frac{4\times0.5}{38} \times 550.19 = 28.96\ kN$

2nd to 8th floor level= $\frac{2\times4\times0.5}{38} \times 550.19 = 202.70\ kN$

1st floor level= $\frac{(4+6)\times0.5}{38} \times 550.19 = 72.39\ kN$

Combination 2 of North-South direction (1.35Gk + 1.05Qk + 1.50Wk + EHF) with 6.10b

Roof vertical loads:

$$0.925 \times 1.35G + 1.05 \times 0.5Q_{imp} = 0.925 \times 1.35 \times 7.09 + 1.05 \times 0.5 \times 1.5$$
$$= 9.64\ kN/m^2$$

Floor vertical loads:

$$0.925 \times 1.35G + 1.05 \times 0.7Q_{imp} = 0.925 \times 1.35 \times 7.21 + 1.05 \times 0.5 \times 5.3$$
$$= 12.90\ kN/m^2$$

Horizontal loads:

Design value of total horizontal wind load per bracing system for combination 2 is:

$$1.5 \times 733.59 = 1100.39kN$$

Design value of wind load:

Roof level= $\frac{4\times0.5}{38}\times 1100.39 = 57.915\ kN$

2nd to 8th floor level= $\frac{2\times4\times0.5}{38}\times 1100.39 = 405.41\ kN$

1st floor level= $\frac{(4+6)\times0.5}{38}\times 1100.39 = 144.79\ kN$

Combination 3 of North-South direction (1.0Gk + 1.50Wk + EHF) with 6.10b

Roof vertical loads:

$$0.925\times G + 0\times 0.5Q_{imp} = 0.925\times 1\times 7.09 + 0\times 0.5\times 1.5 = 6.56\ kN/m^2$$

Floor vertical loads:

$$0.925\times G + 0\times 0.7Q_{imp} = 0.925\times 1\times 7.21 + 0\times 0.5\times 5.3 = 6.67\ kN/m^2$$

Horizontal loads:

Design value of total horizontal wind load per bracing system for combination 3 is:

$$1.5\times 733.59 = 1100.39kN$$

Design value of wind load:

Roof level= $\frac{4\times0.5}{38}\times 1100.39 = 57.915\ kN$

2nd to 8th floor level= $\frac{2\times4\times0.5}{38}\times 1100.39 = 405.41\ kN$

1st floor level= $\frac{(4+6)\times0.5}{38}\times 1100.39 = 144.79\ kN$

Wind leading combination calculated above can be used to check bracing system and a clear table is listed below:

Table 6.7 Loads with different combinations of north-south direction

	$G\ (kN/m^2)$	$Q_{imp}\ (kN/m^2)$	LC1 (kN/m^2)	LC2 (kN/m^2)	LC3 (kN/m^2)
Roof	7.09	1.5	9.98	9.64	6.56
Floor	7.21	5.3	14.57	12.90	6.67

Vertical design forces on columns of braced frame will be calculated in the column

section.

Combination 4 of East-West direction (1.35Gk + 1.50Qk + 0.75Wk + EHF) with 6.10b

Roof vertical loads:

$$0.925 \times 1.35G + 1.5 \times 0.5Q_{imp} = 0.925 \times 1.35 \times 7.09 + 1.5 \times 0.5 \times 1.5 = 9.98\ kN/m^2$$

Floor vertical loads:

$$0.925 \times 1.35G + 1.5 \times 0.7Q_{imp} = 0.925 \times 1.35 \times 7.21 + 1.5 \times 0.5 \times 4 = 13.20\ kN/m^2$$

Horizontal loads:

Design value of total horizontal wind load per bracing system for combination 4 is:

$$0.75 \times 815.1 = 611.33\ kN$$

Roof level= $\frac{4\times0.5}{38} \times 611.33 = 32.18\ kN$

2nd to 8th floor level= $\frac{2\times4\times0.5}{38} \times 611.33 = 225.23\ kN$

1st floor level= $\frac{(4+6)\times0.5}{38} \times 611.33 = 80.44\ kN$

Combination 5 of East-West direction (1.35Gk + 1.05Qk + 1.50Wk + EHF) with 6.10b

Roof vertical loads:

$$0.925 \times 1.35G + 1.05 \times 0.5Q_{imp} = 0.925 \times 1.35 \times 7.09 + 1.05 \times 0.5 \times 1.5 = 9.64\ kN/m^2$$

Floor vertical loads:

$$0.925 \times 1.35G + 1.05 \times 0.7Q_{imp} = 0.925 \times 1.35 \times 7.21 + 1.05 \times 0.5 \times 4 = 11.94\ kN/m^2$$

Horizontal loads:

Design value of total horizontal wind load per bracing system for combination 5 is:

$$1.5 \times 815.1 = 1222.65kN$$

Design value of wind load:

Roof level= $\frac{4\times0.5}{38} \times 1222.65 = 64.35\ kN$

2nd to 8th floor level= $\frac{2\times4\times0.5}{38} \times 122 = 405.45\ kN$

1st floor level= $\frac{(4+6)\times0.5}{38} \times 1100.39 = 160.88\ kN$

Combination 6 of East-West direction (1.0Gk + 1.50Wk + EHF) with 6.10b

Roof vertical loads:

$$0.925 \times G + 0 \times 0.5Q_{imp} = 0.925 \times 1 \times 7.09 + 0 \times 0.5 \times 1.5 = 6.56\ kN/m^2$$

Floor vertical loads:

$$0.925 \times G + 0 \times 0.7Q_{imp} = 0.925 \times 1 \times 7.21 + 0 \times 0.5 \times 5.3 = 6.67\ kN/m^2$$

Horizontal loads:

Design value of total horizontal wind load per bracing system for combination 6 is:

$$1.5 \times 815.1 = 1222.65\ kN$$

Design value of wind load:

Roof level= $\frac{4\times0.5}{38} \times 1222.65 = 64.35\ kN$

2nd to 8th floor level= $\frac{2\times4\times0.5}{38} \times 1222.65 = 450.45\ kN$

1st floor level= $\frac{(4+6)\times0.5}{38} \times 1222.65 = 160.88\ kN$

Wind leading combination calculated above can be used to check bracing system and a clear table is listed below:

Table 6.7 Loads with different combinations of east-west direction

	$G\ (kN/m^2)$	$Q_{imp}\ (kN/m^2)$	**LC1** (kN/m^2)	**LC2** (kN/m^2)	**LC3** (kN/m^2)
Roof	7.09	1.5	9.98	9.64	6.56
Floor	7.21	4	13.20	11.94	6.67

6.6.4. Sway stiffness

Combination 1

Vertical load on floor and on roof is different because they have different loads, but the area of each level is the same. For per bracing plane, the design value must be divided by 4. This is because all the four bracing planes are in the orthogonal direction, the horizontal force is assumed to be equally divided by these four bracing systems.

EHF per floor, per bracing plane:

Roof: $0.005 \times (0.25 \times 2835) \times 9.98 = 35.36\ kN$

Floor: $0.005 \times (0.25 \times 2835) \times 14.57 = 51.63\ kN$

Total horizontal loads H_{Ed} which is equal to wind load add EHF:

Roof: $28.96 + 35.36 = 64.32\ kN$

2nd floor to 8th floor: $202.70 + 51.63 = 254.33\ kN$

1st floor: $72.39 + 51.63 = 124.02\ kN$

Combination 2

EHF per floor, per bracing plane:

Roof: $0.005 \times (0.25 \times 2835) \times 9.64 = 34.17\ kN$

Floor: $0.005 \times (0.25 \times 2835) \times 12.90 = 45.71\ kN$

Total horizontal loads H_{Ed} which is equal to wind load add EHF:

Roof: $57.92 + 34.17 = 92.08\ kN$

2nd floor to 8th floor: $405.41 + 45.71 = 451.12\ kN$

1st floor: $144.79 + 45.71 = 190.50\ kN$

Combination 3

EHF per floor, per bracing plane:

Roof: $0.005 \times (0.25 \times 2835) \times 6.56 = 23.24\ kN$

Floor: $0.005 \times (0.25 \times 2835) \times 6.67 = 23.63\ kN$

Total horizontal loads H_{Ed} which is equal to wind load add EHF:

Roof: $57.92 + 23.24 = 81.16\ kN$

2nd floor to 8th floor: $405.41 + 23.63 = 429.04\ kN$

1st floor: $144.79 + 23.63 = 168.42\ kN$

Combination 4

EHF per floor, per bracing plane:

Roof: $0.005 \times (0.25 \times 2835) \times 9.98 = 35.36\ kN$

Floor: $0.005 \times (0.25 \times 2835) \times 13.20 = 46.79\ kN$

Total horizontal loads H_{Ed} which is equal to wind load add EHF:

Roof: $32.18 + 35.36 = 67.54\ kN$

2nd floor to 8th floor: $225.23 + 46.79 = 272.02\ kN$

1st floor: $80.44 + 46.79 = 127.23kN$

Combination 5

EHF per floor, per bracing plane:

Roof: $0.005 \times (0.25 \times 2835) \times 9.64 = 34.17\ kN$

Floor: $0.005 \times (0.25 \times 2835) \times 11.94 = 42.32\ kN$

Total horizontal loads H_{Ed} which is equal to wind load add EHF:

Roof: $64.35 + 35.36 = 98.52\ kN$

2nd floor to 8th floor: $450.45 + 42.32 = 492.78\ kN$

1st floor: $160.88 + 42.32 = 203.2\ kN$

Combination 6

EHF per floor, per bracing plane:

Roof: $0.005 \times (0.25 \times 2835) \times 6.56 = 23.24\ kN$

Floor: $0.005 \times (0.25 \times 2835) \times 6.67 = 23.63\ kN$

Total horizontal loads H_{Ed} which is equal to wind load add EHF:

Roof: $64.35 + 23.24 = 87.59\ kN$

2nd floor to 8th floor: $450.45 + 23.63 = 474.08\ kN$

1st floor: $160.88 + 23.63 = 184.51\ kN$

After calculating all the load needed for deflection, the size of bracing member can be selected:

Assumed column: UKC $356 \times 406 \times 1202$

Assumed beam: UKC $914 \times 419 \times 388$

Assumed bracing: CHS 508×14.2

Deflection with elastic analysis

Table 6.8 Combination 1

Storey	Storey height (m)	Inertia I (m^2)	Moment (kNm)	$\delta\theta_i$ (rads)	Storey inclination	Storey drift (mm)	Shear drift (mm)	Total deflection (mm)	Deflection added together (mm)
9	4	3.75	128638	6.53E-07	0.00068	2.72	0.074	2.80	32.26
8	4	3.75	894574	4.54E-06	0.00068	2.72	0.294	3.09	29.46
7	4	3.75	2677829	1.36E-05	0.00068	2.70	0.294	3.37	26.37
6	4	3.75	5478402	2.78E-05	0.00066	2.65	0.294	3.61	23.00

5	4	3.75	9296293	4.72E-05	0.00063	2.54	0.294	3.79	19.39
4	4	3.75	14131503	7.18E-05	0.00059	2.35	0.294	3.90	15.60
3	4	3.75	19984031	0.00010	0.00052	2.06	0.294	3.90	11.71
2	4	3.75	26853877	0.00014	0.00041	1.66	0.294	3.79	7.81
1	6	3.75	36449071	0.00028	0.00028	1.67	0.215	4.02	4.02

Deflection of combination 1 is 32.26mm which should be smaller than the limitation 63mm. To verify the accuracy of hand calculation, SAP2000 is used for getting results. U1 in figure 6.13 euqals to 30.65mm which means deflection on the horizontal direction. The error between hand calculation and sofware is 1.61 and it has about 5.25% error rate.

$$\frac{32.26 - 30.65}{30.65} \times 100\% = 5.25\%$$

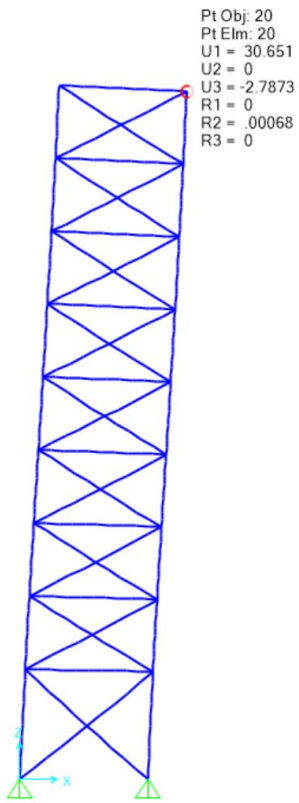


Figure 6.13 Software results for deflection of combination 1

Table 6.9 Combination 2

Storey	Storey height (m)	Inertia I (m^2)	Moment (kNm)	$\delta\theta_i$ (rads)	Storey inclination	Storey drift (mm)	Shear drift (mm)	Total deflection (mm)	Deflection added together (mm)

9	4	3.75	184161.562	9.35E-07	0.0012	4.75	0.11	4.86	56.35
8	4	3.75	1454716.26	7.39E-06	0.0012	4.75	0.52	5.37	51.50
7	4	3.75	4529734.106	2.30E-05	0.0012	4.72	0.52	5.87	46.12
6	4	3.75	9409215.099	4.78E-05	0.0012	4.62	0.52	6.30	40.26
5	4	3.75	16093159.24	8.17E-05	0.0011	4.43	0.52	6.63	33.96
4	4	3.75	24581566.53	0.00012	0.0010	4.11	0.52	6.82	27.33
3	4	3.75	34874436.97	0.00018	0.0009	3.61	0.52	6.85	20.51
2	4	3.75	46971770.55	0.00024	0.0007	2.90	0.52	6.66	13.67
1	6	3.75	63792721.86	0.00049	0.0005	2.92	0.33	7.01	7.01

Total horizontal deflection is 56.35mm and software result is 53.57mm. The error is $\frac{56.35-53.57}{53.57} \times 100\% = 5.19\%$ which is less than 10%. 56.35mm<63mm so that the total deflection is less than the limit.

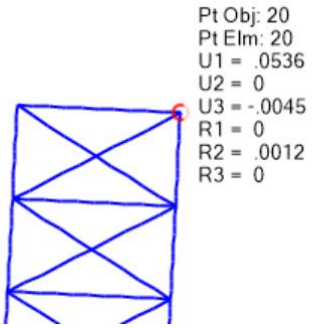


Figure 6.14 Software results for deflection of combination 2

Table 6.10 Combination 3

Storey	Storey height (m)	Inertia I (m^2)	Moment (kNm)	$\delta\theta_i$ (rads)	Storey inclination	Storey drift (mm)	Shear drift (mm)	Total deflection (mm)	Deflection added together (mm)
9	4	3.75	162311.60	8.24E-07	0.0011	4.49	0.093934	4.59	53.33
8	4	3.75	1345013.1	6.83E-06	0.0011	4.49	0.496591	5.079	48.75
7	4	3.75	4243871.22	2.16E-05	0.0011	4.46	0.496591	5.55	43.67
6	4	3.75	8858885.96	4.50E-05	0.0011	4.37	0.496591	5.96	38.12
5	4	3.75	15190057.32	7.71E-05	0.0010	4.19	0.496591	6.28	32.16
4	4	3.75	23237385.3	0.00012	0.00097	3.89	0.496591	6.46	25.89
3	4	3.75	33000869.9	0.00017	0.00085	3.41	0.496591	6.49	19.42
2	4	3.75	44480511.11	0.00023	0.00069	2.74	0.496591	6.31	12.94
1	6	3.75	60407925.48	0.00046	0.00046	2.76	0.291513	6.622	6.62

Total horizontal deflection is 53.33mm and software result is 50.70mm. The error is $\frac{53.33-50.70}{50.70} \times 100\% = 5.19\%$ which is less than 10%. 53.33mm<63mm so that

the total deflection is less than the limit.

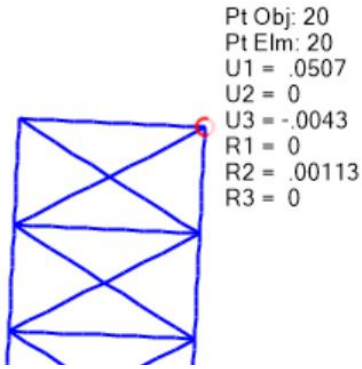


Figure 6.15 Software results for deflection of combination 3

Table 6.11 Combination 4

Storey	Storey height (m)	Inertia I (m^2)	Moment (kNm)	$\delta\theta_i$ (rads)	Storey inclination	Storey drift (mm)	Shear drift (mm)	Total deflection (mm)	Deflection added together (mm)
9	4	15.00	135073.59	1.84416E-07	0.000195503	0.782011063	0.111949135	0.893960199	23.08
8	4	15.00	949250.50	1.29601E-06	0.000195318	0.7812734	0.450892399	1.344114934	22.19
7	4	15.00	2851486.83	3.89313E-06	0.000194022	0.776089354	0.450892399	1.789823287	20.84
6	4	15.00	5841782.61	7.97578E-06	0.000190129	0.760516819	0.450892399	2.22514315	19.05
5	4	15.00	9920137.82	1.3544E-05	0.000182153	0.728613685	0.450892399	2.644132415	16.83
4	4	15.00	15086552.47	2.05977E-05	0.000168609	0.674437844	0.450892399	3.040848973	14.19
3	4	15.00	21341026.55	2.91369E-05	0.000148012	0.592047188	0.450892399	3.409350715	11.14
2	4	15.00	28683560.06	3.91617E-05	0.000118875	0.475499607	0.450892399	3.743695533	7.73
1	6	15.00	38923446.18	7.97132E-05	7.97132E-05	0.478279491	0.24359725	3.990072667	3.99

Total horizontal deflection is 23.14mm and software result is 21.60mm. The error is $\frac{23.14-21.60}{21.60} \times 100\% = 7.13\%$ which is less than 10%. 23.14mm<63mm so that the total deflection is less than the limit.

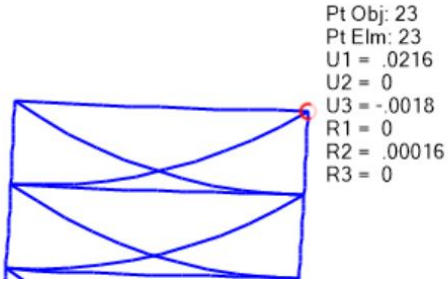


Figure 6.16 Software results for deflection of combination 4

Table 6.12 Combination 5

Storey	Storey height (m)	Inertia I (m^2)	Moment (kNm)	$\delta\theta_i$ (rads)	Storey inclination	Storey drift (mm)	Shear drift (mm)	Total deflection (mm)	Deflection added together (mm)
9	4	15.00	197031.56	2.69007E-07	0.000348127	1.392507198	0.163299965	1.555807163	41.24
8	4	15.00	1576644.15	2.15259E-06	0.000347858	1.39143117	0.816824429	2.371555563	39.68
7	4	15.00	4927355.68	6.72732E-06	0.000345705	1.382820803	0.816824429	3.179769625	37.31
6	4	15.00	10249166.14	1.39932E-05	0.000338978	1.355911536	0.816824429	3.969684787	34.13
5	4	15.00	17542075.54	2.39502E-05	0.000324985	1.299938806	0.816824429	4.730536486	30.16
4	4	15.00	26806083.87	3.65983E-05	0.000301035	1.204138051	0.816824429	5.45156016	25.43
3	4	15.00	38041191.14	5.19376E-05	0.000264436	1.057744708	0.816824429	6.121991246	19.98
2	4	15.00	51247397.35	6.99681E-05	0.000212499	0.849994215	0.816824429	6.731065182	13.86
1	6	15.00	69596691.14	0.000142531	0.000142531	0.855183014	0.389058587	7.125312568	7.13

Total horizontal deflection is 41.34mm and software result is 38.70mm. The error is $\frac{41.34-38.7}{21.60} \times 100\% = 6.82\%$ which is less than 10%. 41.34mm<63mm so that the total deflection is less than the limit.

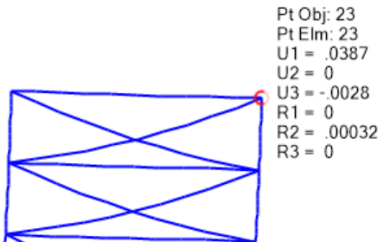


Figure 6.17 Software results for deflection of combination 5

Table 6.13 Combination 6

Storey	Storey height (m)	Inertia I (m^2)	Moment (kNm)	$\delta\theta_i$ (rads)	Storey inclination	Storey drift (mm)	Shear drift (mm)	Total deflection (mm)	Deflection added together (mm)
9	4	15.00	175181.60	2.39175 E-07	0.000333084	1.332334623	0.145190691	1.477525314	39.49
8	4	15.00	1473713.1	2.01206 E-06	0.000332844	1.331377922	0.785842886	2.262411498	38.02
7	4	15.00	4668581.22	6.37401 E-06	0.000330832	1.323329682	0.785842886	3.040206145	35.75
6	4	15.00	9759785.96	1.3325 E-05	0.000324458	1.297833634	0.785842886	3.800552982	32.71
5	4	15.00	16747327.32	2.28651 E-05	0.000311133	1.244533506	0.785842886	4.53309574	28.91
4	4	15.00	25631205.3	3.49943 E-05	0.000288268	1.153073029	0.785842886	5.227478149	24.38
3	4	15.00	36411419.9	4.97125 E-05	0.000253274	1.013095931	0.785842886	5.873343937	19.15
2	4	15.00	49087971.11	6.70198 E-05	0.000203561	0.814245942	0.785842886	6.460336834	13.28
1	6	15.00	66672397.98	0.000136542	0.000136542	0.819250188	0.353272466	6.818613545	6.82

Total horizontal deflection is 39.59mm and software result is 37.10mm. The error is $\frac{39.59-37.10}{37.10} \times 100\% = 6.71\%$ which is less than 10%. 39.59mm<63mm so that the total deflection is less than the limit.

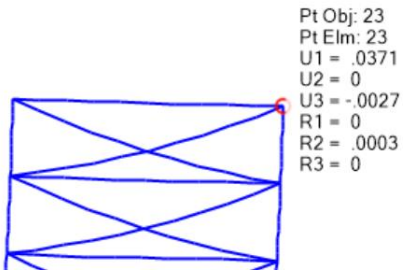


Figure 6.18 Software results for deflection of combination 6

α_{cr} has to be calculated for all combinations.

Combination 1

For the roof to 8th floor:

$H_{Ed} = 64.32\ kN$

$V_{Ed} = 0.25 \times 2835 \times 9.98 = 7072.36\ kN$

Also, V_{Ed} here is for each braced bay, so it must be divided by 4 (times 0.25).

$h = 4m$

$\delta = 2.80\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{64.32}{7072.36}\right)\left(\frac{4000}{2.8}\right) = 13.00$$

For 8th to 7th floor:

$H_{Ed} = 64.32 + 254.33 = 318.65\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57) = 17397.8\ kN$

$h = 4m$

$\delta = 3.09\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{318.65}{17397.8}\right)\left(\frac{4000}{3.09}\right) = 23.71$$

For 7th to 6th floor:

$H_{Ed} = 64.32 + 254.33 \times 2 = 572.98\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57 \times 2) = 27723.2\ kN$

$h = 4m$

$\delta = 3.37\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{572.98}{27723.2}\right)\left(\frac{4000}{3.37}\right) = 24.56$$

For 6th to 5th floor:

$H_{Ed} = 64.32 + 254.33 \times 3 = 827.31\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57 \times 3) = 38048.6\ kN$

$h = 4m$

$\delta = 3.61\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{827.31}{38048.6}\right)\left(\frac{4000}{3.61}\right) = 24.11$$

For 5th to 4th floor:

$$H_{Ed} = 64.32 + 254.33 \times 4 = 1081.64\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57 \times 4) = 48374\ kN$$

$$h = 4m$$

$$\delta = 3.79\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1081.64}{48374}\right)\left(\frac{4000}{3.79}\right) = 23.60$$

For 4th to 3rd floor:

$$H_{Ed} = 64.32 + 254.33 \times 5 = 1335.97\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57 \times 5) = 58699.4\ kN$$

$$h = 4m$$

$$\delta = 3.90\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1335.97}{58699.4}\right)\left(\frac{4000}{3.90}\right) = 23.37$$

For 3rd to 2nd floor:

$$H_{Ed} = 64.32 + 254.33 \times 6 = 1590.3\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57 \times 6) = 69024.9\ kN$$

$$h = 4m$$

$$\delta = 3.90\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1590.3}{69024.9}\right)\left(\frac{4000}{3.90}\right) = 23.61$$

For 2nd to 1st floor:

$H_{Ed} = 64.32 + 254.33 \times 7 = 1844.63\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57 \times 7) = 79350.3\ kN$

$h = 4m$

$\delta = 3.79\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1844.63}{79350.3}\right)\left(\frac{4000}{3.79}\right) = 24.53$$

For 1st floor to the ground:

$H_{Ed} = 64.32 + 254.33 \times 7 + 124.02 = 1968.65\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57 \times 8) = 89675.7\ kN$

$h = 6m$

$\delta = 4.02\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1968.65}{89675.7}\right)\left(\frac{6000}{4.02}\right) = 32.80$$

The worst case is $\alpha_{cr} = 13.00 > 10$, so first order analysis alone is sufficient. Second order effects and load amplifier do not need to be considered.

Combination 2

For the roof to 8th floor:

$H_{Ed} = 92.08\ kN$

$V_{Ed} = 0.25 \times 2835 \times 9.64 = 6833.16\ kN$

Also, V_{Ed} here is for each braced bay, so it must be divided by 4 (times 0.25).

$h = 4m$

$\delta = 4.86\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{92.08}{6833.16}\right)\left(\frac{4000}{4.86}\right) = 11.10$$

For 8th to 7th floor:

$$H_{Ed} = 92.08 + 451.116 = 543.20\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90) = 15975.3\ kN$$

$$h = 4m$$

$$\delta = 5.37\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{543.20}{15975.3}\right)\left(\frac{4000}{5.37}\right) = 25.31$$

For 7th to 6th floor:

$$H_{Ed} = 92.08 + 451.116 \times 2 = 994.31\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90 \times 2) = 25117.5\ kN$$

$$h = 4m$$

$$\delta = 5.87\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{994.31}{25117.5}\right)\left(\frac{4000}{5.87}\right) = 27.00$$

For 6th to 5th floor:

$$H_{Ed} = 92.08 + 451.116 \times 3 = 1445.43\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90 \times 3) = 34259.6\ kN$$

$$h = 4m$$

$$\delta = 6.30\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1445.43}{34259.6}\right)\left(\frac{4000}{6.30}\right) = 26.80$$

For 5th to 4th floor:

$H_{Ed} = 92.08 + 451.116 \times 4 = 1896.54\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90 \times 4) = 43401.8\ kN$

$h = 4m$

$\delta = 6.63\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1896.54}{43401.8}\right)\left(\frac{4000}{6.63}\right) = 26.37$$

For 4th to 3rd floor:

$H_{Ed} = 92.08 + 451.116 \times 5 = 2347.66\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90 \times 5) = 52543.9\ kN$

$h = 4m$

$\delta = 6.82\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2347.66}{52543.9}\right)\left(\frac{4000}{6.82}\right) = 26.19$$

For 3rd to 2nd floor:

$H_{Ed} = 92.08 + 451.116 \times 6 = 2798.78\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90 \times 6) = 61686.1\ kN$

$h = 4m$

$\delta = 6.85\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2798.78}{61686.1}\right)\left(\frac{4000}{6.85}\right) = 26.51$$

For 2nd to 1st floor:

$H_{Ed} = 92.08 + 451.116 \times 7 = 3249.89\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90 \times 7) = 70828.3\ kN$

$h = 4m$

$\delta = 6.66\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3249.89}{70828.3}\right)\left(\frac{4000}{6.66}\right) = 27.56$$

For 1st floor to the ground:

$H_{Ed} = 92.08 + 451.116 \times 7 + 190.50 = 3440.39\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90 \times 8) = 79970.4\ kN$

$h = 6m$

$\delta = 7.01\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3440.39}{79970.4}\right)\left(\frac{6000}{7.01}\right) = 36.84$$

The worst case is $\alpha_{cr} = 11.1 > 10$, so first order analysis alone is sufficient. Second order effects and load amplifier do not need to be considered.

Combination 3

For the roof to 8th floor:

$H_{Ed} = 81.16\ kN$

$V_{Ed} = 0.25 \times 2835 \times 6.56 = 4648.16\ kN$

Also, V_{Ed} here is for each braced bay, so it must be divided by 4 (times 0.25).

$h = 4m$

$\delta = 4.59\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{81.16}{4648.16}\right)\left(\frac{4000}{4.59}\right) = 15.23$$

For 8th to 7th floor:

$H_{Ed} = 81.16 + 429.04 = 510.20\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67) = 9374.99\ kN$

$h = 4m$

$\delta = 5.08\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{510.20}{9374.99}\right)\left(\frac{4000}{5.08}\right) = 42.86$$

For 7th to 6th floor:

$H_{Ed} = 81.16 + 429.04 \times 2 = 939.23\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 2) = 14101.8\ kN$

$h = 4m$

$\delta = 5.55\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{939.23}{14101.8}\right)\left(\frac{4000}{5.55}\right) = 48.02$$

For 6th to 5th floor:

$H_{Ed} = 81.16 + 429.04 \times 3 = 1368.27\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 3) = 18828.7\ kN$

$h = 4m$

$\delta = 5.96\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1368.27}{18828.7}\right)\left(\frac{4000}{5.96}\right) = 48.78$$

For 5th to 4th floor:

$H_{Ed} = 81.16 + 429.04 \times 4 = 1797.31\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 4) = 23555.5\ kN$

$h = 4m$

$\delta = 6.28\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1797.31}{23555.5}\right)\left(\frac{4000}{6.28}\right) = 48.64$$

For 4th to 3rd floor:

$H_{Ed} = 81.16 + 429.04 \times 5 = 2226.35\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 5) = 28282.3\ kN$

$h = 4m$

$\delta = 6.46\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2226.35}{28282.3}\right)\left(\frac{4000}{6.46}\right) = 48.72$$

For 3rd to 2nd floor:

$H_{Ed} = 81.16 + 429.04 \times 6 = 2655.39\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 6) = 33009.1\ kN$

$h = 4m$

$\delta = 6.49\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2655.39}{33009.1}\right)\left(\frac{4000}{6.49}\right) = 49.60$$

For 2nd to 1st floor:

$H_{Ed} = 81.16 + 429.04 \times 7 = 3084.43\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 7) = 37736\ kN$

$h = 4m$

$\delta = 6.31\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3084.43}{37736}\right)\left(\frac{4000}{6.31}\right) = 51.78$$

For 1st floor to the ground:

$H_{Ed} = 81.16 + 429.04 \times 7 + 168.42 = 3252.85\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 8) = 42462.8\ kN$

$h = 6m$

$\delta = 6.62\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3252.85}{42462.8}\right)\left(\frac{6000}{6.62}\right) = 69.41$$

The worst case is $\alpha_{cr} = 15.23 > 10$, so first order analysis alone is sufficient. Second order effects and load amplifier do not need to be considered.

Combination 4

For the roof to 8th floor:

$H_{Ed} = 67.54\ kN$

$V_{Ed} = 0.25 \times 2835 \times 9.98 = 7072.36\ kN$

Also, V_{Ed} here is for each braced bay, so it must be divided by 4 (times 0.25).

$h = 4m$

$\delta = 0.89\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{67.54}{7072.36}\right)\left(\frac{4000}{0.89}\right) = 42.73$$

For 8th to 7th floor:

$H_{Ed} = 67.54 + 272.02 = 339.55\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20) = 16430.3\ kN$

$h = 4m$

$\delta = 1.34\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{339.55}{16430.3}\right)\left(\frac{4000}{1.34}\right) = 61.50$$

For 7th to 6th floor:

$H_{Ed} = 67.54 + 272.02 \times 2 = 611.57\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20 \times 2) = 25788.3\ kN$

$h = 4m$

$\delta = 1.79mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{611.57}{25788.3}\right)\left(\frac{4000}{1.79}\right) = 53.00$$

For 6th to 5th floor:

$H_{Ed} = 67.54 + 272.02 \times 3 = 883.58\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20 \times 3) = 35146.3\ kN$

$h = 4m$

$\delta = 2.23\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{883.58}{35146.3}\right)\left(\frac{4000}{2.23}\right) = 45.19$$

For 5th to 4th floor:

$H_{Ed} = 67.54 + 272.02 \times 4 = 1155.6\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20 \times 4) = 44504.2\ kN$

$h = 4m$

$\delta = 2.64\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1155.6}{44504.2}\right)\left(\frac{4000}{2.64}\right) = 39.28$$

For 4th to 3rd floor:

$H_{Ed} = 67.54 + 272.02 \times 5 = 1427.61\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20 \times 5) = 53862.2\ kN$

$h = 4m$

$\delta = 3.04\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1427.61}{53862.2}\right)\left(\frac{4000}{3.04}\right) = 34.87$$

For 3rd to 2nd floor:

$H_{Ed} = 67.54 + 272.02 \times 6 = 1699.63\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20 \times 6) = 63220.2\ kN$

$h = 4m$

$\delta = 3.41\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1699.63}{63220.2}\right)\left(\frac{4000}{3.41}\right) = 31.54$$

For 2nd to 1st floor:

$H_{Ed} = 67.54 + 272.02 \times 7 = 1971.64\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20 \times 7) = 72578.2\ kN$

$h = 4m$

$\delta = 3.74\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1971.64}{72578.2}\right)\left(\frac{4000}{3.74}\right) = 29.03$$

For 1st floor to the ground:

$$H_{Ed} = 67.54 + 272.02 \times 7 + 127.23 = 2098.87\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20 \times 8) = 81936.1\ kN$$

$$h = 6m$$

$$\delta = 4.00\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3252.85}{42462.8}\right)\left(\frac{6000}{4.00}\right) = 38.52$$

The worst case is $\alpha_{cr} = 29.03 > 10$, so first order analysis alone is sufficient. Second order effects and load amplifier do not need to be considered.

Combination 5

For the roof to 8th floor:

$$H_{Ed} = 98.52\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times 9.64 = 6833.16\ kN$$

Also, V_{Ed} here is for each braced bay, so it must be divided by 4 (times 0.25).

$$h = 4m$$

$$\delta = 1.56\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{98.52}{6833.16}\right)\left(\frac{4000}{1.56}\right) = 37.07$$

For 8th to 7th floor:

$$H_{Ed} = 98.52 + 492.78 = 591.29\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94) = 15298.1\ kN$$

$h = 4m$

$\delta = 2.37\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{591.29}{15298.1}\right)\left(\frac{4000}{2.37}\right) = 65.19$$

For 7th to 6th floor:

$H_{Ed} = 98.52 + 492.78 \times 2 = 1084.07\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94 \times 2) = 23763\ kN$

$h = 4m$

$\delta = 3.18mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1084.07}{23763}\right)\left(\frac{4000}{3.18}\right) = 57.39$$

For 6th to 5th floor:

$H_{Ed} = 98.52 + 492.78 \times 3 = 1576.84\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94 \times 3) = 32228\ kN$

$h = 4m$

$\delta = 3.97\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1576.84}{32228}\right)\left(\frac{4000}{3.97}\right) = 49.30$$

For 5th to 4th floor:

$H_{Ed} = 98.52 + 492.78 \times 4 = 2069.61\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94 \times 4) = 40692.9\ kN$

$h = 4m$

$\delta = 4.73\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2069.61}{40692.9}\right)\left(\frac{4000}{4.73}\right) = 43.00$$

For 4th to 3rd floor:

$$H_{Ed} = 98.52 + 492.78 \times 5 = 2562.39\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94 \times 5) = 49157.9\ kN$$

$$h = 4m$$

$$\delta = 5.45\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2562.39}{49157.9}\right)\left(\frac{4000}{5.45}\right) = 38.25$$

For 3rd to 2nd floor:

$$H_{Ed} = 98.52 + 492.78 \times 6 = 3055.16\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94 \times 6) = 57622.8\ kN$$

$$h = 4m$$

$$\delta = 6.12mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3055.16}{57622.8}\right)\left(\frac{4000}{6.12}\right) = 34.64$$

For 2nd to 1st floor:

$$H_{Ed} = 98.52 + 492.78 \times 7 = 3547.94\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94 \times 7) = 66087.8\ kN$$

$$h = 4m$$

$$\delta = 6.73\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3547.94}{66087.8}\right)\left(\frac{4000}{6.73}\right) = 31.90$$

For 1st floor to the ground:

$H_{Ed} = 98.52 + 492.78 \times 7 + 203.2 = 3751.14\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94 \times 8) = 74552.7\ kN$

$h = 6m$

$\delta = 7.13\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3751.14}{74552.7}\right)\left(\frac{6000}{7.13}\right) = 42.37$$

The worst case is $\alpha_{cr} = 31.90 > 10$, so first order analysis alone is sufficient. Second order effects and load amplifier do not need to be considered.

Combination 6

For the roof to 8th floor:

$H_{Ed} = 87.59\ kN$

$V_{Ed} = 0.25 \times 2835 \times 6.56 = 4648.16\ kN$

Also, V_{Ed} here is for each braced bay, so it must be divided by 4 (times 0.25).

$h = 4m$

$\delta = 1.48\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{87.59}{4648.16}\right)\left(\frac{4000}{1.48}\right) = 51.02$$

For 8th to 7th floor:

$H_{Ed} = 87.59 + 474.08 = 561.68\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67) = 9374.99\ kN$

$h = 4m$

$\delta = 2.26mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{561.68}{9374.99}\right)\left(\frac{4000}{2.26}\right) = 105.93$$

For 7th to 6th floor:

$$H_{Ed} = 87.59 + 474.08 \times 2 = 1035.76\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 2) = 14101.8\ kN$$

$$h = 4m$$

$$\delta = 3.04mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1035.76}{14101.8}\right)\left(\frac{4000}{3.04}\right) = 96.64$$

For 6th to 5th floor:

$$H_{Ed} = 87.59 + 474.08 \times 3 = 1509.84\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 3) = 18828.7\ kN$$

$$h = 4m$$

$$\delta = 3.80\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1509.84}{18828.7}\right)\left(\frac{4000}{3.80}\right) = 84.40$$

For 5th to 4th floor:

$$H_{Ed} = 87.59 + 474.08 \times 4 = 1983.93\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 4) = 23555.5\ kN$$

$$h = 4m$$

$$\delta = 4.53\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1983.93}{23555.5}\right)\left(\frac{4000}{4.53}\right) = 74.32$$

For 4th to 3rd floor:

$H_{Ed} = 87.59 + 474.08 \times 5 = 2458.01\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 5) = 28282.3\ kN$

$h = 4m$

$\delta = 5.23\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2458.01}{28282.3}\right)\left(\frac{4000}{5.23}\right) = 66.50$$

For 3rd to 2nd floor:

$H_{Ed} = 87.59 + 474.08 \times 6 = 2932.1\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 6) = 33009.1\ kN$

$h = 4m$

$\delta = 5.87\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2932.1}{33009.1}\right)\left(\frac{4000}{5.87}\right) = 60.49$$

For 2nd to 1st floor:

$H_{Ed} = 87.59 + 474.08 \times 7 = 3406.18\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 7) = 37736\ kN$

$h = 4m$

$\delta = 6.46\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3406.18}{37736}\right)\left(\frac{4000}{6.46}\right) = 55.89$$

For 1st floor to the ground:

$H_{Ed} = 87.59 + 474.08 \times 7 + 184.51 = 3590.69\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 8) = 42462.8\ kN$

$h = 6m$

$\delta = 6.82\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3590.69}{42462.8}\right)\left(\frac{6000}{6.82}\right) = 74..41$$

The worst case is $\alpha_{cr} = 51.02 > 10$, so first order analysis alone is sufficient. Second order effects and load amplifier do not need to be considered.

Therefore, all combinations can be analysed with first order effect and none of them need amplifier.

6.6.5. Design of bracing

From the calculation for α_{cr}, H_{Ed} for each floor is figured out and it is found that H_{Ed} between 1st floor and the ground is the largest. This can be used to find critical design load in diagonal bracing member.

Axial force in diagonal member

Combination 1

Horizontal force, 1st floor to ground $= 1968.65\ kN$

Diagonal member axial force in compression $= 1968.65 \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 1296.43\ kN$

Combination 2

Horizontal force, 1st floor to ground $= 3440.39\ kN$

Diagonal member axial force in compression $= 3440.39 \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 2265.63\ kN$

Combination 3

Horizontal force, 1st floor to ground $= 3252.85\ kN$

Diagonal member axial force in compression $= 3252.85 \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 =$

$2142.13\ kN$

Combination 1, 2 and 3 have the same structure of frame, so the buckling resistance is the same. The design force needed to be resist is largest one of this three combinations, which is $N_{Ed} = 2265.63\ kN$.**Combination 4**

Horizontal force, 1st floor to ground $= 2098.87\ kN$

Diagonal member axial force in compression $= 2098.87 \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 1141.75\ kN$

Combination 5

Horizontal force, 1st floor to ground $= 3751.14\ kN$

Diagonal member axial force in compression $= 3751.14 \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 2040.56\ kN$

Combination 6

Horizontal force, 1st floor to ground $= 3590.69\ kN$

Diagonal member axial force in compression $= 3590.69 \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 1953.28\ kN$

Combination 4, 5 and 6 have the same structure of frame, so the buckling resistance is the same. The design force needed to be resist is largest one of this three combinations, which is $N_{Ed} = 2040.56\ kN$.

6.6.6. Member checks

Buckling length

The effective length for member is:

$L_{cr} = 1.0 \times \sqrt{6^2 + 7^2} \times 10^3 = 9219.54mm$ for north-south bracing frames (combination 1, 2, 3)

$L_{cr} = 1.0 \times \sqrt{6^2 + 14^2} \times 10^3 = 15231.5mm$ for east-west bracing frames (combination 4, 5, 6)

Slenderness for flexural buckling

Combination 1, 2 and 3

For axial compression in members, non-dimensional slenderness $\bar{\lambda}$ is:

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = (\frac{L_{cr}}{i})(\frac{1}{\lambda_1}) \quad \text{for Class 1, 2 and 3 cross-sections}$$

BSEN1993-1-1 6.3.1.3 (1) Eqn. (6.50)

Where L_{cr} is the buckling length calculated before and i is radius of gyration about the axis. λ_1 can be calculated with equation: $\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.9\varepsilon$ and $\varepsilon = \sqrt{\frac{235}{f_y}}$

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/355} = 0.8136$$

All sections in the structure are S355, therefore $f_y = 355\ N/mm^2$

$$\lambda_1 = 93.9\varepsilon = 93.9 \times 0.81 = 76.3986$$

All the diagonal sections use hollow section, so radius of gyration about x and y are the same. For CHS 508×14.2, radius of gyration i is 175mm.

$$\bar{\lambda} = \left(\frac{L_{cr}}{i}\right)\left(\frac{1}{\lambda_1}\right) = \left(\frac{9219.54}{175}\right)\left(\frac{1}{76.3986}\right) = 0.69 > 0.2$$

BSEN1993-1-1 6.3.1.2(4)

For $\bar{\lambda} \leq 0.2$, the buckling effects can be ignored. However, $\bar{\lambda} > 0.2$, so reduction factor χ must be calculated for buckling resistance.

Combination 4, 5 and 6

$$\bar{\lambda} = \left(\frac{L_{cr}}{i}\right)\left(\frac{1}{\lambda_1}\right) = \left(\frac{15231.5}{175}\right)\left(\frac{1}{76.3986}\right) = 1.14 > 0.2$$

$\bar{\lambda}$ for the three combinations is also larger than 0.2, so reduction factor χ is also needed.

Design buckling resistance

Requirement for section:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0$$

BSEN1993-1-1 6.3.1.1(1) Eqn. (6.46)

Design buckling resistance is:

$$N_{b,Rd} = \frac{\chi\, Af_y}{\gamma_{M1}} \text{ (For Class 1, 2 and cross sections)}$$

6.3.1.1(3) Eqn. (6.47)

Where $\gamma_{M1} = 1.0$, $\chi = \frac{1}{\phi^2 + \sqrt{\phi^2 - \bar{\lambda}^2}} \leq 1.0$ and $\phi = 0.5 \times (1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2)$

A is section area and $f_y = 355\ N/mm^2$.

From Table 6.2 and 6.1 in EN 1993-1-1, the buckling curve for any hot finished hollow section is "a" and imperfection factor α for curve a is 0.21.

Table 6.2

Table 6.1

Combination 1, 2 and 3

$$\phi = 0.5 \times \left(1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right) = 0.5 \times (1 + 0.21 \times (0.69 - 0.2) + 0.69^2) = 0.79$$

BSEN1993-1-1 6.3.1.2(1)

$$\chi = \frac{1}{\phi^2 + \sqrt{\phi^2 - \bar{\lambda}^2}} = \frac{1}{0.79 + \sqrt{0.79^2 - 0.69^2}} = 0.85 < 1.0$$

$$N_{b,Rd} = \frac{\chi\, Af_y}{\gamma_{M1}} = \frac{0.85 \times 0.022 \times 10^6 \times 355}{1} \times 10^{-3} = 6658.62\ kN$$

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{2265.63}{6658.62} = 0.34 < 1.0$$

BSEN1993-1-1 Eqn. (6.47)

The design buckling resistance of the section is adequate for combination 1, 2 and 3 which are north-south directional bracing frames.

Combination 4, 5 and 6

$$\phi = 0.5 \times \left(1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right) = 0.5 \times (1 + 0.21 \times (1.14 - 0.2) + 1.14^2) = 1.25$$

BSEN1993-1-1 6.3.1.2(1)

$$\chi = \frac{1}{\phi^2 + \sqrt{\phi^2 - \bar{\lambda}^2}} = \frac{1}{1.25 + \sqrt{1.25^2 - 1.14^2}} = 0.57 < 1.0$$

$$N_{b,Rd} = \frac{\chi\, A f_y}{\gamma_{M1}} = \frac{0.57 \times 0.022 \times 10^6 \times 355}{1} \times 10^{-3} = 4447.52\ kN$$

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{2040.56}{4447.52} = 0.46 < 1.0$$

The design buckling resistance of the section if adequate for combination 4, 5 and 6 which are north-south directional bracing frames.

Verification of bracing adjacent to column splices

BS EN 1993-1-1, 5.3.3(4)

Combination 1

External loading

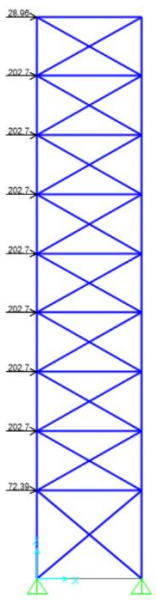


Figure 6.19 Bracing frame with only external load

Without equivalent horizontal force, figure 6.19 shows the external loading (wind load) on the frame. Then, axial forces in the bracing elements can be calculated. The forces are accumulated to the lower level.

Calculation process:

Ground to 8th floor= $28.96 \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 16.68\ kN$

8th floor to 7th floor= $(28.96 + 202.7) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 133.41\ kN$

7th floor to 6th floor= $(28.96 + 202.7 \times 2) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 250.14\ kN$

6th floor to 5th floor= $(28.96 + 202.7 \times 3) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 366.87\ kN$

5th floor to 4th floor= $(28.96 + 202.7 \times 4) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 483.60\ kN$

4th floor to 3rd floor= $(28.96 + 202.7 \times 5) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 600.33\ kN$

3rd floor to 2nd floor= $(28.96 + 202.7 \times 6) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 717.064\ kN$

2nd floor to 1st floor= $(28.96 + 202.7 \times 7) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 833.80\ kN$

1st floor to the ground $= (28.96 + 202.7 \times 7 + 72.39) \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 1001.15\ kN$

Local lateral force is:

$$\alpha_m N_{Ed}/1000$$

Where $\alpha_m = \sqrt{0.5(1 + \frac{1}{m})}$ and m is the number of columns to be restrained

$$\alpha_m = \sqrt{0.5(1 + \frac{1}{m})} = \sqrt{0.5(1 + \frac{1}{91})} = 0.50$$

Total axial force in columns $= 9.98 \times 2835 + 14.57 \times 2835 = 69591.1\ kN$

Local force for each bracing frame $= 0.25 \times 0.50 \times \frac{69591.1}{100} = 87.47\ kN$

The local force is assumed to be split equally between above and below bracing member, the resultant force in the diagonal member between 2nd floor and 1st floor levels is:

$$87.47 \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 50.37\ kN$$

Total maximum force= $833.30 + 50.37 = 884.17\ kN < 6658.62\ kN$

The local imperfection forces at splices can be carried by the bracing.

Combination 2

External loading

2nd floor to 1st floor= $(57.92 + 405.41) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 1667.59\ kN$

Lateral force

Total axial force in columns $= 9.64 \times 2835 + 12.90 \times 2835 = 63901.3\ kN$

Local force for each bracing frame $= 0.25 \times 0.50 \times \frac{63901.3}{100} = 80.31\ kN$

Resultant force in the diagonal member between 2nd floor and 1st floor levels

$$80.31 \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 46.25\ kN$$

Total maximum force= $1667.59 + 46.25 = 1713.84\ kN < 6658.62\ kN$

The local imperfection forces at splices can be carried by the bracing.

Combination 3

External loading

2nd floor to 1st floor= $(57.92 + 405.41) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 1667.59\ kN$

Lateral force

Total axial force in columns $= 6.56 \times 2835 + 6.67 \times 2835 = 37500\ kN$

Local force for each bracing frame $= 0.25 \times 0.50 \times \frac{37500}{100} = 47.13\ kN$

Resultant force in the diagonal member between 2nd floor and 1st floor levels

$$47.13 \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 27.14\ kN$$

Total maximum force= $1667.59 + 27.14 = 1694.73\ kN < 6658.62\ kN$

The local imperfection forces at splices can be carried by the bracing.

Combination 4

External loading

2nd floor to 1st floor= $(32.18 + 225.23) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 836.56\ kN$

Lateral force

Total axial force in columns $= 9.98 \times 2835 + 13.20 \times 2835 = 65721.3\ kN$

Local force for each bracing frame $= 0.25 \times 0.50 \times \frac{69721.3}{100} = 82.60\ kN$

Resultant force in the diagonal member between 2nd floor and 1st floor levels

$$82.60 \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 42.95\ kN$$

Total maximum force= $836.56 + 42.95 = 879.52\ kN < 6658.62\ kN$

The local imperfection forces at splices can be carried by the bracing.

Combination 5

External loading

2nd floor to 1st floor= $(64.35 + 450.45) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 1673.13\ kN$

Lateral force

Total axial force in columns $= 9.64 \times 2835 + 11.94 \times 2835 = 61192.4\ kN$

Local force for each bracing frame $= 0.25 \times 0.50 \times \frac{61192.4}{100} = 76.91\ kN$

Resultant force in the diagonal member between 2nd floor and 1st floor levels

$$76.91 \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 40.00\ kN$$

Total maximum force= $1673.13 + 40.00 = 1713.12\ kN < 6658.62\ kN$

The local imperfection forces at splices can be carried by the bracing.

Combination 6

External loading

2nd floor to 1st floor= $(64.35 + 450.45) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 1673.13\ kN$

Lateral force

Total axial force in columns $= 6.56 \times 2835 + 6.67 \times 2835 = 37500\ kN$

Local force for each bracing frame $= 0.25 \times 0.50 \times \frac{37500}{100} = 47.13\ kN$

Resultant force in the diagonal member between 2nd floor and 1st floor levels

$$47.13 \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 24.51\ kN$$

Total maximum force= $1673.13 + 24.51 = 1697.63\ kN < 6658.62\ kN$

The local imperfection forces at splices can be carried by the bracing.

Verification of restraint to columns provided by bracing

BS EN 1993-1-1, 5.3.2(5)

Equation for the process is:

$$H = \phi N_{Ed}$$

Where has been mentioned in 3.2.6.4 sway stiffness: $\phi = \phi_0 \alpha_h \alpha_m$ and $\phi_0 = \frac{1}{200}\alpha_h = 1.0\ \alpha_m = \sqrt{0.5(1+\frac{1}{m})} = \sqrt{0.5 \times (1+\frac{1}{91})} = 0.50$

$$\phi = \frac{1}{200} \times 0.5 = 0.00251$$

Combination 1

Total axial force in columns $= 9.98 \times 2835 + 14.57 \times 2835 \times 8 = 358703\ kN$

This means all the vertical forces on 9 stories in total.

Local force per bracing system $= 0.25 \times 0.0025 \times 358703 = 225.42\ kN$

Similarly, the local force is also assumed to be split equally as imperfection of column splices.

Resultant force in diagonal member

$$225.42 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 129.81\ kN$$

Axial force with external force applying between 1st floor and the ground

$$(28.96 + 202.7 \times 7 + 72.39) \times \frac{\sqrt{6^2 + 7^2}}{7} \times 0.5 = 1001.15\ kN$$

Total maximum force$= 1001.15 + 129.81 = 1130.97\ kN < 6658.62\ kN$

Therefore, the force can be carried by the frame.

Combination 2

Total axial force in columns $= 9.64 \times 2835 + 12.90 \times 2835 \times 8 = 319882\ kN$

This means all the vertical forces on 9 stories in total.

Local force per bracing system $= 0.25 \times 0.0025 \times 319882 = 201.02\ kN$

Similarly, the local force is also assumed to be split equally as imperfection of column splices.

Resultant force in diagonal member

$$201.02 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 115.76\ kN$$

Axial force with external force applying between 1st floor and the ground

$$(57.92 + 405.41 \times 7 + 144.79) \times \frac{\sqrt{6^2 + 7^2}}{7} \times 0.5 = 2002.31\ kN$$

Total maximum force= $2002.31 + 115.76 = 2118.08\ kN < 6658.62\ kN$

Therefore, the force can be carried by the frame.

Combination 3

Total axial force in columns $= 6.56 \times 2835 + 6.67 \times 2835 \times 8 = 169851\ kN$

This means all the vertical forces on 9 stories in total.

Local force per bracing system $= 0.25 \times 0.0025 \times 169851 = 106.74\ kN$

Similarly, the local force is also assumed to be split equally as imperfection of column splices.

Resultant force in diagonal member

$$106.74 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 61.47\ kN$$

Axial force with external force applying between 1st floor and the ground

$$(57.92 + 405.41 \times 7 + 144.79) \times \frac{\sqrt{6^2 + 7^2}}{7} \times 0.5 = 2002.31\ kN$$

Total maximum force= $2002.31 + 61.47 = 2063.78\ kN < 6658.62\ kN$

Therefore, the force can be carried by the frame.

Combination 4

Total axial force in columns $= 9.98 \times 2835 + 13.20 \times 2835 \times 8 = 327745\ kN$

This means all the vertical forces on 9 stories in total.

Local force per bracing system $= 0.25 \times 0.0025 \times 327745 = 205.96\ kN$

Similarly, the local force is also assumed to be split equally as imperfection of column splices.

Resultant force in diagonal member

$$205.96 \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 107.10\ kN$$

Axial force with external force applying between 1st floor and the ground

$$(32.18 + 225.23 \times 7 + 80.44) \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 918.89\ kN$$

Total maximum force= $918.89 + 107.10 = 1025.99\ kN < 6658.62\ kN$

Therefore, the force can be carried by the frame.

Combination 5

Total axial force in columns $= 9.64 \times 2835 + 11.94 \times 2835 \times 8 = 298211\ kN$

This means all the vertical forces on 9 stories in total.

Local force per bracing system $= 0.25 \times 0.0025 \times 298211 = 187.40\ kN$

Similarly, the local force is also assumed to be split equally as imperfection of column splices.

Resultant force in diagonal member

$$187.40 \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 97.45\ kN$$

Axial force with external force applying between 1st floor and the ground

$$(64.35 + 450.45 \times 7 + 160.88) \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 1837.78\ kN$$

Total maximum force= $1837.78 + 97.45 = 1935.23\ kN < 6658.62\ kN$

Therefore, the force can be carried by the frame.

Combination 6

Total axial force in columns $= 6.56 \times 2835 + 6.67 \times 2835 \times 8 = 169851\ kN$

This means all the vertical forces on 9 stories in total.

Local force per bracing system $= 0.25 \times 0.0025 \times 169851 = 106.74\ kN$

Similarly, the local force is also assumed to be split equally as imperfection of column splices.

Resultant force in diagonal member

$$106.74 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 55.51\ kN$$

Axial force with external force applying between 1st floor and the ground

$$(64.35 + 450.45 \times 7 + 160.88) \times \frac{\sqrt{6^2 + 7^2}}{7} \times 0.5 = 1837.78\ kN$$

Total maximum force$= 1837.78 + 55.51 = 1893.29\ kN < 6658.62\ kN$

Therefore, the force can be carried by the frame.

In conclusion, the section selected fulfil every limitation and the deflection limited is close to the requirement which means this is an effective choice. Also, resistance to the various imperfections and external load is sufficient for each bracing member.

7. Cantilever Truss System Design

7.1. Load Cases of Cantilevered Truss

Loads on the structure should include the following:

Table 7.1 Loading factors

Roof			**Floor**		
Topping	1	kN/m^2	Tiles(screed)	0.7	kN/m^2
Slab	2.84	kN/m^2	Slab	2.84	kN/m^2
Steel (assumed)	0.2	kN/m^2	Steel (assumed)	0.3	kN/m^2
Ceiling	0.5	kN/m^2	Partitions	1	kN/m^2
Services	1	kN/m^2	Services	1.5	kN/m^2
Imposed load	1.5	kN/m^2	Ceiling	0.5	kN/m^2
			Imposed load	4	kN/m^2

The imposed floor load is 4.0 kN/m^2 and the imposed roof load is 1.5 kN/m^2. The perimeter beams are also required to carry a cladding load of 1.30 kN/m^2 (area on elevation).

The wind loading can be assumed to generate a pressure on the structure of 1.30 kN/m^2. This can be taken as uniform for the whole height of the building.

The following load combinations need to be considered:

- 1.35Gk + 1.50Qk +EHF
- 1.35Gk + 1.50Qk + 0.75Wk + EHF
- 1.35Gk + 1.05Qk + 1.50Wk + EHF
- 1.0Gk + 1.50Wk + EHF

Assume Grade S355 Steel throughout. In addition, the structure performance in the event of fire is required to be 1.5 hours, assuming sprinklers are also used.

Hence, The bearing area of each column is:

(m^2)

Col. A	**Col. B**	**Col. C**	**Col. D**	**Col. E**	**Col. F**	**Col. G**
47.25	57.38	33.75	67.50	67.50	67.50	33.75

Cladding load of perimeter beam:

(kN)

Area A	Area B	Area C	Area D	Area E	Area F	Area G
36.4	44.2	61.1	70.2	70.2	70.2	32.75

Selection of the most unfavourable load conditions : 1.35Gk + 1.50Qk + 0.75Wk + EHF.

Roof load to each column:

(kN)

Col. A	Col. B	Col. C	Col. D	Col. E	Col. F	Col. G
508.84	617.87	410.84	751.48	751.48	751.48	372.57

Floor load to each column:

(kN)

Col. A	Col. B	Col. C	Col. D	Col. E	Col. F	Col. G
768.95	933.72	596.63	1123.1	1123.1	1123.1	558.36

Total load (Point load) on truss:

(kN)

Point A	Point B	Point C	Point D	Point E	Point F	Point G
3584.6	4352.8	2797.4	5243.7	5243.7	5243.7	2606.0

7.2. Proposal 1

7.2.1. Preliminary Truss Design

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

A truss is likely to be needed to bridge the area where no columns are allowed. The truss members can be made of fabricated steel sections, such as box sections, UKC sections or many others. Initially, it was thought to design the truss members as UKB sections, but the proposal was not adopted as the axial stiffness of UKB

sections are significantly smaller than UKC sections. Owing to this, UKC sections have been adopted.

In the calculation, the maximum compressive force in the truss chord member can quickly be determined as M/D, where M is the maximum bending moment, typically in the middle of the truss and D the depth of the truss.

$$N_1 = \frac{M}{D}, \textit{ Assume } \boldsymbol{N}_{b,Rd} = 0.70\boldsymbol{f}_y A$$

$$\therefore \frac{\boldsymbol{N}_1}{0.70\boldsymbol{f}_y} \Rightarrow \textit{ Area of cross section required}$$

(the 0.70 is effectively χ in figure 6.4EC3, buckling curves.)

The buckling load may then be determined using EC3 in a conventional manner. Additionally, the estimated deflection of the truss may be calculated. The truss's second moment of area I may be approximated by $AD^2/2$. This would tell you the truss's flexural deflection. The shear component of the deflection can be calculated by multiplying this amount by 1.25 to the deflection calculated.

As the truss depth D grows, the forces acting on the diagonal member decrease, thus it allows a smaller member size to be used.

Coursework Requirements

According to the coursework brief, the steel members should not be longer than 27m, with height greater than 4m and heavier than 26 tonnes. To avoid the needs of police escort during delivery, member width and length should also be limited to 2.9m and 18.3m respectively.

To guarantee that the diagonal member does not exceed 18.3 m in length, we picked a truss depth D of 12 m, which results in an 18.06 m diagonal member length.

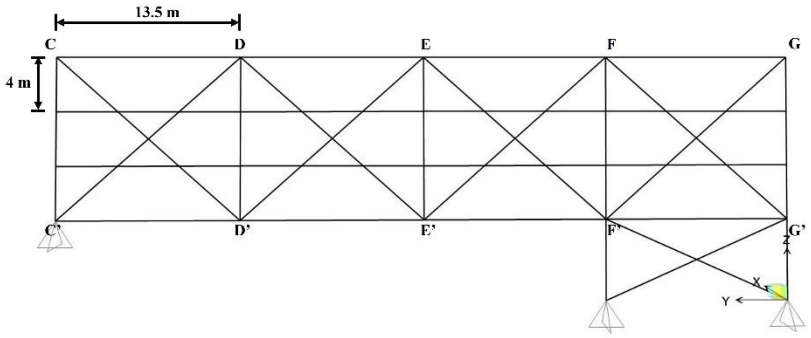


Figure 7.1 Truss - Proposal 1 - West View

Initial calculations were made manually to find member sizes which could then be adjusted to meet the final stress and deflection requirements if required.

The main truss is simplified to a simply supported beam model and the force diagram is shown below:

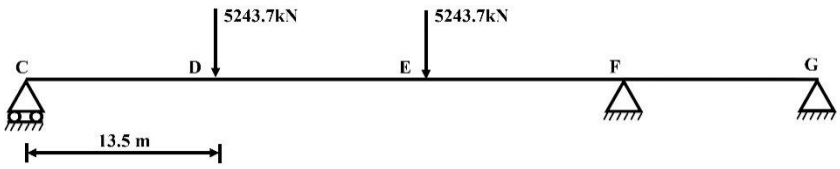


Figure 7.2 Force diagram of the simplified model - Proposal 1 – West View

$Maximum\ bending\ moment\ M_{max} = 70790.46\ kNm$

$$\therefore N_1 = \frac{M}{D} = 5899.21\ kN$$

$$\Rightarrow A = \frac{\boldsymbol{N}_1}{0.70\boldsymbol{f}_y} = 237.39\ cm^2$$

$And\ the\ deflection\ of\ the\ truss\ can\ also\ be\ determined\ in$

$an\ approximate\ way$:

$$The\ second\ moment: I = \frac{AD^2}{2} = 1.71 \times 10^{12}\ cm^4$$

$$Deflection\ caused\ by\ point\ load\ at\ D:$$

$$\delta_{mid,D} = -\vartheta(L/2) = \frac{P_D b_D (3L^2 - 4b_D^2)}{48EI} = 17.22\ mm\ (\downarrow)$$

$$Deflection\ caused\ by\ point\ load\ at\ E:$$

$$\delta_{mid,E} = -\vartheta(L/2) = \frac{P_E b_E (3L^2 - 4b_E^2)}{48EI} = 17.22\ mm\ (\downarrow)$$

$$Simply\ consider\ that\ the\ uniform\ distribute\ load\ q\ is\ equal$$

$$to\ the\ ratio\ of\ the\ point\ load\ on\ the\ column\ to\ the\ width\ of$$

$$the\ bearing\ area\ of\ the\ column\ load:$$

$$q = \frac{4P_{Col}}{b} = 332.76\ kN/m$$

$$Deflection\ caused\ by\ UDL:$$

$$\delta_{mid,UDL} = \frac{5qL^4}{384EI} = 32.48\ mm\ (\downarrow)$$

$$Hence, the\ total\ deflection\ is:$$

$$\delta_{total} = 66.92\ mm(\downarrow)$$

Application of the same method for the preliminary design of the truss on the northern side of the building. It should be noted that the truss on the northern side is shorter than the truss on the western side, so the members are relatively stiff. Therefore, the truss members analysis can be simplified as a cantilevered beam with a fixed section and a free end. The free end is subjected to a point load whose value is equal to the support reaction of the truss at that point.

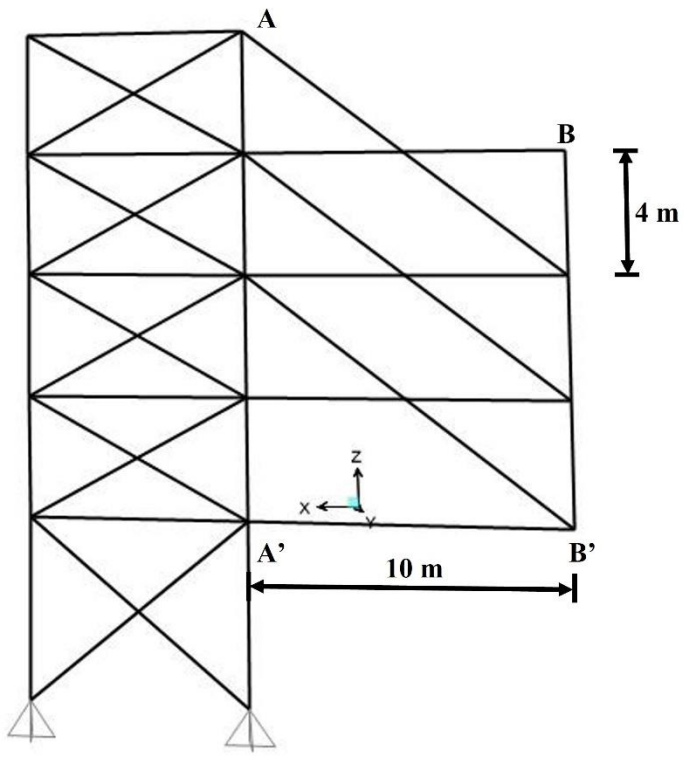


Figure 7.3 Truss - Proposal 1 – North View (North View)

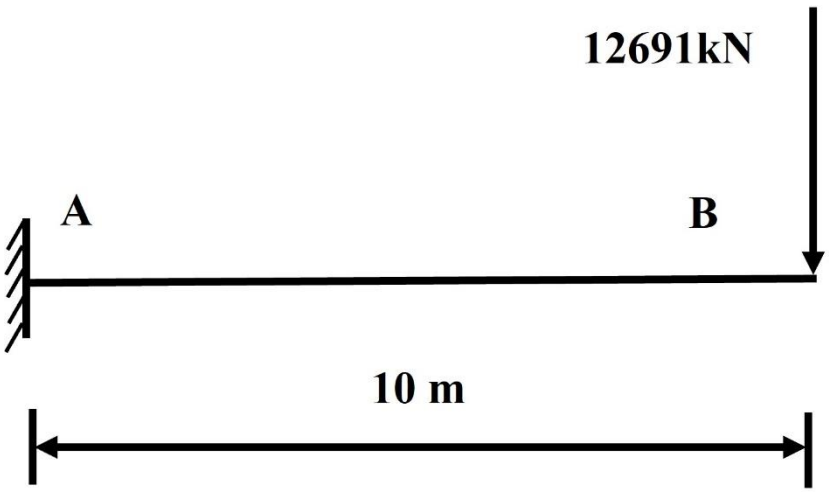


Figure 7.4 Force diagram of the simplified model – Proposal 1 - North View

$Maximum\ bending\ moment\ M_{max} = 126910\ kNm$

$$\therefore N_1 = \frac{M}{D} = 10575.83\ kN$$

$$\Rightarrow A = \frac{\boldsymbol{N}_1}{0.70\boldsymbol{f_y}} = 425.59\ cm^2$$

$And\ the\ deflection\ of\ the\ truss\ can\ also\ be\ determined\ in$

$an\ approximate\ way$:

$$The\ second\ moment: I = \frac{AD^2}{2} = 3.06 \times 10^{12}\ cm^4$$

$Deflection\ caused\ by\ point\ load$:

$$\delta_{C,P} = -\vartheta(L/2) = \frac{PL^3}{3EI} = 6.57\ mm\ (\downarrow)$$

Using the same simplification as UDL for truss on the west side, the point loads and bearing areas to which Column B and Column C are subjected are different, resulting in the evenly distributed load on the cantilever beam being divided into two different parts. Obviously the uniform load on the B side of the column (closer to the fixed junction) is larger. Therefore, average of the two parts of the UDL acting together can be assumed since this is a less favourable situation than the real one.

$$q = \left(\frac{4 * P_{Col,B}}{8.5} + \frac{4 * P_{Col,C}}{5}\right) \times \frac{1}{2} = 114.59\ kN/m$$

$Deflection\ caused\ by\ UDL$:

$$\delta_{C,UDL} = \frac{qL^4}{8EI} = 22.26\ mm\ (\downarrow)$$

$Hence, the\ total\ deflection\ is$:

$\delta_{total} = 28.84\ mm(\downarrow)$

As it is a truss in a cantilever region, the forces are more complex, an initial cross-section should be obtained in a careful manner. The design criteria should be consistent with the EC3's'simple design' approach, which considers connections to be pinned. The frame must be adequately braced in two orthogonal directions to provide global stability.

7.2.2. Truss West View Check

The changes to the section are shown in the table below:

Table 7.2 Section Properties - Proposal 1 - West View

Section Designation	Colour
UKC 356*406*235	
UKC 356*406*467	
UKC 356*406*634	

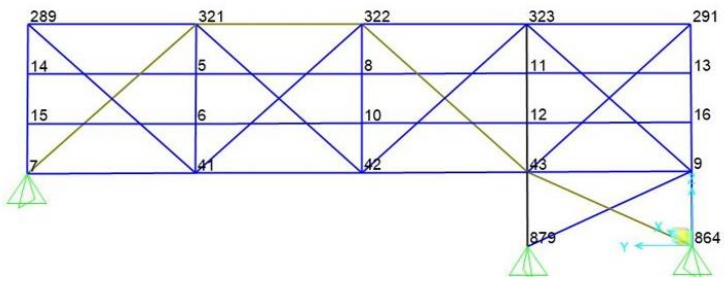


Figure 7.5 Section Properties - Proposal 1 - West View

Check the compression of the top & bottom chords, critical diagonals and critical vertical elements.

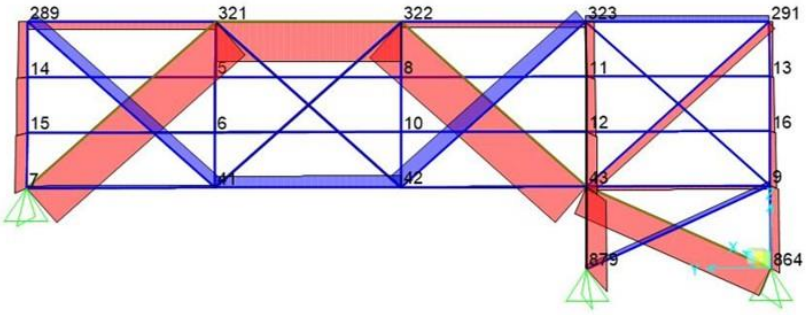


Figure 7.6 Axial Force - Proposal 1 – West View

Top & Bottom Chords Check

The most compressed element is 321-322, N_{Ed} = 1896.67 kN

To verify the adequacy of compression members, two types of checks are required:

1) Cross-section resistance
2) Member buckling resistance

For short compression members ($\bar{\lambda} \leq 0.2$), only the cross-sectional check is required, but for longer members, both checks should be carried out.

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 5.73$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 5.73$

$\therefore Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 15.8$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 15.8$

$\therefore Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Compression resistance of cross-section</u>

The design compression resistance of the cross-section

$$\mathrm{N_{c,Rd}} = \frac{\mathrm{Af_y}}{\gamma_{\mathrm{m0}}}\ for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{29900 \times 355}{1.0} = 10614.5\ kN\ > 1896.67\ kN\ \therefore OK$$

<u>Member Buckling Resistance in compression</u>

$$\mathrm{N_{b,Rd}} = \frac{\chi A \mathrm{f_y}}{\gamma_{\mathrm{Ml}}}\ fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\frac{\mathrm{Af_y}}{\mathrm{N_{cr}}}}$ for Class 1, 2 and 3 cross-section

Elastic Critical Force and non-dimensional slenderness for flexural buckling

For buckling about both the major (y-y) and minor (z-z) axes, L_{cr} should be taken as 1.0 times the actual (system) length of the member, since the truss has pinned end conditions.

$$N_{cr,y} = \frac{\pi^2 EI_y}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 791000000}{13500^2} = 8995.56 \times 10^3 \text{N} = 8995.56 \text{ kN}$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{29900 \times 355}{8995.56 \times 10^3}} = 1.09$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 310000000}{13500^2} = 3525.44 \times 10^3 \text{N}$$

$$= 3525.44 \text{ kN}$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{29900 \times 355}{3525.44 \times 10^3}} = 1.74$$

Selection of buckling curve and imperfection factor α

For a hot-rolled I section,

$$\frac{h}{b} = \frac{381}{394.8} = 0.97 < 1.2, and\ t_f = 30.2mm < 100mm$$

$$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$$

$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$

Buckling curves – major (y-y) axis

$\Phi_y = 0.5[1 + 0.34(1.09 - 0.2) + 1.09^2] = 1.24$

$$\chi_y = \frac{1}{1.24 + \sqrt{1.24^2 - 1.09^2}} = 0.54$$

$$\therefore N_{b,y,Rd} = \frac{0.54 \times 29900 \times 355}{1.0} = 5768.69 \times 10^3 N$$

$= 5768.69 kN$

$5768.69 > 1896.67$ kN

$\therefore Major\ axis\ flexural\ buckling\ resistance\ is\ OK$

Buckling curves – minor (z-z) axis

$\Phi_z = 0.5[1 + 0.49(1.74 - 0.2) + 1.74^2] = 2.38$

$$\chi_z = \frac{1}{2.38 + \sqrt{2.38^2 - 1.74^2}} = 0.25$$

$$\therefore N_{b,z,Rd} = \frac{0.25 \times 29900 \times 355}{1.0} = 2645.19 \times 10^3 N = 2645.19 kN$$

$2645.19 > 1896.67$ kN

$\therefore Minor\ axis\ flexural\ buckling\ resistance\ is\ OK$

The UKC 356*406*235 in grade S355 steel is suitable to support the design load N_{Ed} of 1896.67 kN. The resistance of this truss member is governed by buckling about the z-z axis, with $N_{b,z,Rd}$ = 2645.19 kN

Critical Diagonals Check

The most compressed element is 13-2, N_{Ed} = 11933.98 kN

To verify the adequacy of compression members, two types of checks are required:

1) Cross-section resistance
2) Member buckling resistance

For short compression members ($\bar{\lambda} \leq 0.2$), only the cross-sectional check is required, but for longer members, both checks should be carried out.

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 2.98$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 2.98$

$\therefore Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 8.11$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 8.11$

$\therefore Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Compression resistance of cross-section</u>

The design compression resistance of the cross-section

$$N_{c,Rd} = \frac{Af_y}{\gamma_{m0}} \; for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{59500 \times 355}{1.0} = 21122.50\ kN\ > 11933.98\ kN\ \therefore OK$$

<u>Member Buckling Resistance in compression</u>

$$N_{b,Rd} = \frac{\chi Af_y}{\gamma_{Ml}} \; fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}}$ for Class 1, 2 and 3 cross-section

<u>Elastic Critical Force and non-dimensional slenderness for flexural buckling</u>

For buckling about both the major (y-y) and minor (z-z) axes, L_{cr} should be taken as 0.33 times the actual (system) length of the member, since the truss has pinned end conditions and there are two lateral constraints at the third equivalence point of the member.

$$N_{cr,y} = \frac{\pi^2 EI_y}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 1830000000}{6020^2} = 104659.10 \times 10^3 \text{N}$$

$$= 104659.10\text{kN}$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{59500 \times 355}{104659.10 \times 10^3}} = 0.45$$

$$\text{N}_{\text{cr},z} = \frac{\pi^2 \text{EI}_z}{\text{L}_{\text{cr}}{}^2}$$

$$= \frac{\pi^2 \times 210000 \times 678000000}{6020^2} = 38775.35 \times 10^3 \text{N}$$

$$= 38775.35\text{kN}$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{59500 \times 355}{38775.35 \times 10^3}} = 0.74$$

<u>Selection of buckling curve and imperfection factor α</u>

For a hot-rolled I section,

$$\frac{\text{h}}{\text{b}} = \frac{436.6}{412.2} = 1.06 < 1.2, and\ t_f = 58mm < 100mm$$

$$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$$

$$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$$

Buckling curves – major (y-y) axis

$$\Phi_{\text{y}} = 0.5[1 + 0.34(0.45 - 0.2) + 0.45^2] = 0.64$$

$$\chi_y = \frac{1}{0.64 + \sqrt{0.64^2 - 0.45^2}} = 0.91$$

$$\therefore \mathrm{N_{b,y,Rd}} = \frac{0.91 \times 59500 \times 355}{1.0} = 19137.77 \times 10^3 \mathrm{N}$$

$= 19137.77\mathrm{kN}$

$19137.77 > 11933.98$ kN

$\therefore$ *Major axis flexural buckling resistance is OK*

Buckling curves – minor (z-z) axis

$$\Phi_z = 0.5[1 + 0.49(0.74 - 0.2) + 0.74^2] = 0.90$$

$$\chi_z = \frac{1}{0.90 + \sqrt{0.90^2 - 0.74^2}} = 0.70$$

$$\therefore \mathrm{N_{b,z,Rd}} = \frac{0.70 \times 59500 \times 355}{1.0} = 14806.93 \times 10^3 \mathrm{N}$$

$= 14806.93\mathrm{kN}$

$14806.93 > 11933.98$ kN

$\therefore$ *Minor axis flexural buckling resistance is OK*

The UKC 356*406*467 in grade S355 steel is suitable to support the design load $\mathrm{N_{Ed}}$ of 11933.98 kN. The resistance of this truss member is governed by buckling about the z-z axis, with $\mathrm{N_{b,z,Rd}}$ = 14806.93 kN.

Critical Vertical Element Check

The most compressed element is 321-322, $\mathrm{N_{Ed}}$ = 5060.19 kN

To verify the adequacy of compression members, two types of checks are required:

1) Cross-section resistance
2) Member buckling resistance

For short compression members ($\bar{\lambda} \leq 0.2$), only the cross-sectional check is required, but for longer members, both checks should be carried out.

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 5.73$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 5.73$

$\therefore\ Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 15.8$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 15.8$

$\therefore\ Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Compression resistance of cross-section</u>

The design compression resistance of the cross-section

$$N_{c,Rd} = \frac{Af_y}{\gamma_{m0}} \; for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{29900 \times 355}{1.0} = 10614.50\ kN\ > 5060.19\ kN\ \therefore OK$$

<u>Member Buckling Resistance in compression</u>

$$N_{b,Rd} = \frac{\chi Af_y}{\gamma_{Ml}} \; fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}}$ for Class 1, 2 and 3 cross-section

<u>Elastic Critical Force and non-dimensional slenderness for flexural buckling</u>

For buckling about both the major (y-y) and minor (z-z) axes, L_{cr} should be taken as 1.0 times the actual (system) length of the member, since the truss has pinned end conditions.

$$N_{cr,y} = \frac{\pi^2 EI_y}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 791000000}{4000^2} = 102465 \times 10^3 N = 102465 kN$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{29900 \times 355}{102465 \times 10^3}} = 0.32$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{L_{cr}{}^2}$$

$$= \frac{\pi^2 \times 210000 \times 310000000}{4000^2} = 40156.95 \times 10^3 N$$

$$= 40156.95 kN$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{29900 \times 355}{40156.95 \times 10^3}} = 0.51$$

<u>Selection of buckling curve and imperfection factor α</u>

For a hot-rolled I section,

$$\frac{h}{b} = \frac{381}{394.8} = 0.97 < 1.2, and\ t_f = 30.2mm < 100mm$$

$$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$$

$$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$$

Buckling curves – major (y-y) axis

$$\Phi_y = 0.5[1 + 0.34(0.32 - 0.2) + 0.32^2] = 0.57$$

$$\chi_y = \frac{1}{0.57 + \sqrt{0.57^2 - 0.32^2}} = 0.96$$

$$\therefore N_{b,y,Rd} = \frac{0.96 \times 29900 \times 355}{1.0} = 10147.85 \times 10^3 N$$

$$= 10147.85 kN$$

$$10147.85 > 5060.19\ kN$$

$\therefore$ *Major axis flexural buckling resistance is OK*

Buckling curves – minor (z-z) axis

$$\Phi_z = 0.5[1 + 0.49(0.51 - 0.2) + 0.51^2] = 0.71$$

$$\chi_z = \frac{1}{0.71 + \sqrt{0.71^2 - 0.51^2}} = 0.84$$

$$\therefore N_{b,z,Rd} = \frac{0.84 \times 29900 \times 355}{1.0} = 8863.74 \times 10^3 N = 8863.74 kN$$

$8863.74 > 5060.19$ kN

$\therefore$ *Minor axis flexural buckling resistance is OK*

The UKC 356*406*235 in grade S355 steel is suitable to support the design load N_{Ed} of 5060.19 kN. The resistance of this truss member is governed by buckling about the z-z axis, with $N_{b,z,Rd}$ = 8863.74 kN.

Lateral deflection check at serviceability limit state is needed, then the results are compared with a simple computer model for verification.

In this project, there is almost no horizontal deflection, so we are more interested in vertical deflection. The following diagram illustrates the limits of the Eurocode 3 on vertical deflection.

Table 7.3 Recommended limits for vertical deflections. Retrieved from Table NA.2 of UK National Annex to Eurocode 3: Design of Steel Structures (p. 6) by BSI London.

A1 Table NA.2 **Suggested limits for vertical deflections** A1

Vertical deflection	
Cantilevers	Length/180
Beams carrying plaster or other brittle finish	Span/360
Other beams (except purlins and sheeting rails)	Span/200
Purlins and sheeting rails	To suit the characteristics of particular cladding

As a general practice, the diameter of bolt holes is generally 2mm larger than the diameter of bolt for bolts of M22 or smaller, while a 3mm is usually adopted for larger bolts.

7.2.3. Deflection Check of West View

The cross-sectional area of members is shown in the table below:

Table 7.4 Section area of truss members

Section Designation	Colour	Area (cm^2)
UKC 356*406*235		299
UKC 356*406*467		595
UKC 356*406*634		808

Firstly, the member forces were evaluated by considering the horizontal and vertical components of the reaction at supports. The results are as follows after calculations:

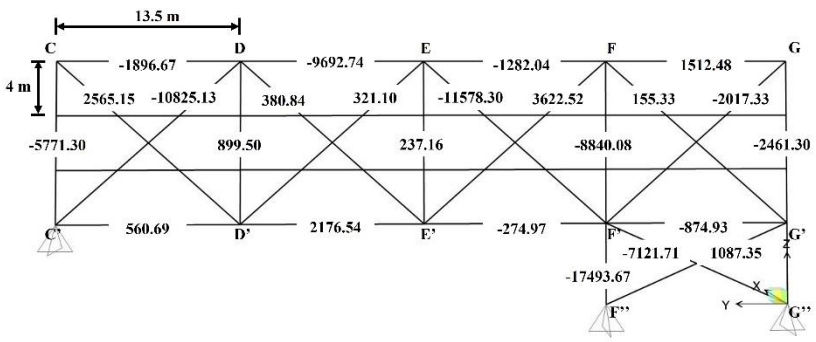


Figure 7.7 P forces (in kN) - Proposal 1 - West View

To calculate the vertical deflection at joint E', remove the externally applied load system and apply a unit load vertically only at joint E', as seen in the figure. Similar to the above, use the section or joint resolution approach to ascertain the magnitude and direction of the unknown member forces.

The deflection is given by:

$$v = \sum_{i=1}^{i=b} N_{1,i} \frac{F_i l_i}{ES_i}$$

Where:

$N_{1,\underline{i}}$ is the axial force produced in the member i by a unit force applied at the point where the deflection is required

l_i is the length of member i

S_i is the section area of the member i

b is the number of members with bolted connection(s)

$\frac{F_i l_i}{ES_i}$ is the variation in length of member I due to the slack recovery

$= \pm 4$mm according to whether the chord is in compression or tension

This is a significant additional deflection, compared with deflection due to the ULS combination.

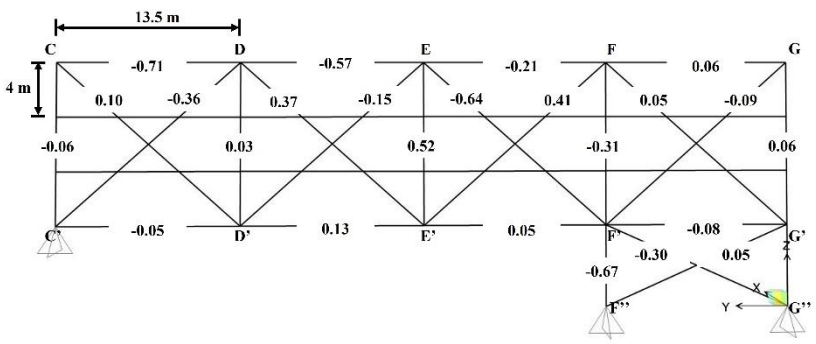


Figure 7.8 u forces - Proposal 1 - West View

The vertical deflection $\delta_{V,E'} = \sum \frac{PL}{AE} u$

The +ve sign indicates that the deflection is in the same direction as the applied unit load.

(PL*u)/AE instead of only (PL*u) has been computed for members with different geometric and mechanical properties, such as cross-sectional area and elastic modulus.

This is the better calculated in tabular form as shown in table below:

Table 7.5 Vertical deflection - Proposal 1 - West View

Member	Length (mm)	A (mm^2)	E (GPa)	AE (kN)	P-force (kN)	PL/AE (mm)	u	(PL/AE)*u (mm)
C’C	12000	29900	210	6279000	-5771.3	-11.03	-0.07	0.74
C’D	18060	59500	210	12495000	-10825.13	-15.65	-0.37	5.73
C’D’	13500	29900	210	6279000	560.69	1.21	-0.05	-0.06
CD	13500	29900	210	6279000	-1896.67	-4.08	-0.07	0.30
CD’	18060	29900	210	6279000	2565.15	7.38	0.10	0.74
D’D	12000	29900	210	6279000	899.5	1.72	0.04	0.06
D’E	18060	29900	210	6279000	321.1	0.92	-0.16	-0.14
D’E’	13500	29900	210	6279000	2176.54	4.68	0.14	0.65
DE	13500	59500	210	12495000	-9692.74	-10.47	-0.58	6.05
DE’	18060	29900	210	6279000	380.84	1.10	0.31	0.34
E’E	12000	29900	210	6279000	237.16	0.45	0.51	0.23
E’F	18060	29900	210	6279000	3622.52	10.42	0.43	4.52
E’F’	13500	29900	210	6279000	-274.97	-0.59	0.46	-0.27
EF	13500	29900	210	6279000	-1282.04	-2.76	0.24	-0.67
EF’	18060	59500	210	12495000	-11578.3	-16.74	-0.61	10.12
F’F	12000	80800	210	16968000	-8840.08	-6.25	-0.31	1.92
F’G	18060	29900	210	6279000	-2017.3	-5.80	-0.08	0.46
F’G’	13500	29900	210	6279000	-874.93	-1.88	-0.07	0.14
FG	13500	29900	210	6279000	1512.48	3.25	0.06	0.19
FG’	18060	29900	210	6279000	155.33	0.45	0.03	0.01
GG"	18000	29900	210	6279000	-2461.3	-7.06	0.05	-0.37
F"F'	6000	80800	210	16968000	-17493.67	-6.19	-0.67	4.14
F"G'	14770	29900	210	6279000	1087.35	2.56	0.05	0.13
F'G"	14770	59500	210	12495000	-7121.71	-8.42	-0.30	2.53
								37.49

From Table NA2 of EC3, the maximum deflection limit of the cantilever is Length/180, which equals to 55.56mm. Hand-calculated results meet the

requirements. In addition, the deflection from SAP2000 programme gives a value of 38.57mm. The error between the two is less than 3%, which is very close.

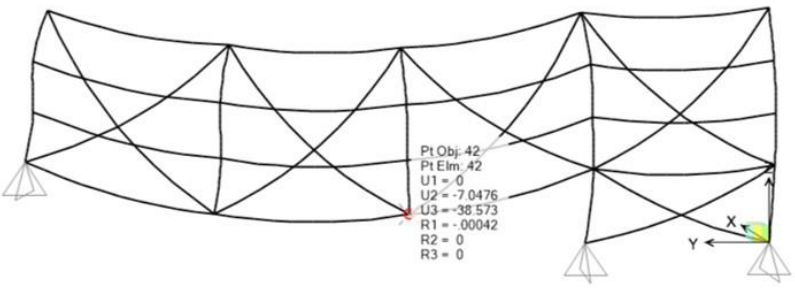


Figure 7.9 Max deflection - Proposal 1 - West View

The hand-calculated results are found to be smaller than that from the computer model. The deflection is small because the deflection of the middle two tiers of cross chord is not considered in the hand calculation and the effect of self-weight cannot be taken into account when calculating the u force.

7.2.4. Truss North View Check

The changes to the section are shown in the following table.

Table 7.6 Section Properties - Proposal 1 - North View

Section Designation	Colour
UKC 356*406*235	
UKC 356*406*467	

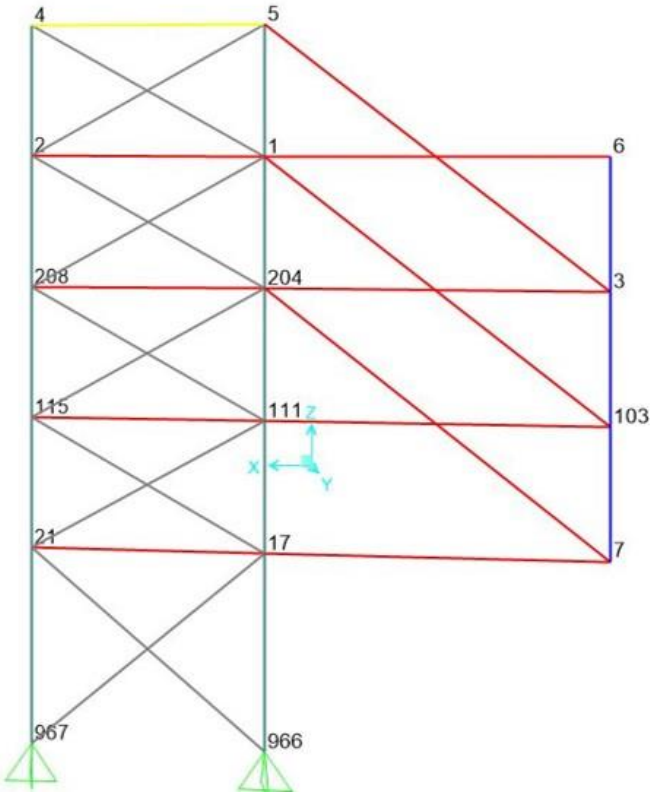


Figure 7.10 Section Properties - Proposal 1 - North View

The compression of the top & bottom chords, critical diagonals and critical vertical elements are then studied and analysed with reference to the model results from SAP2000.

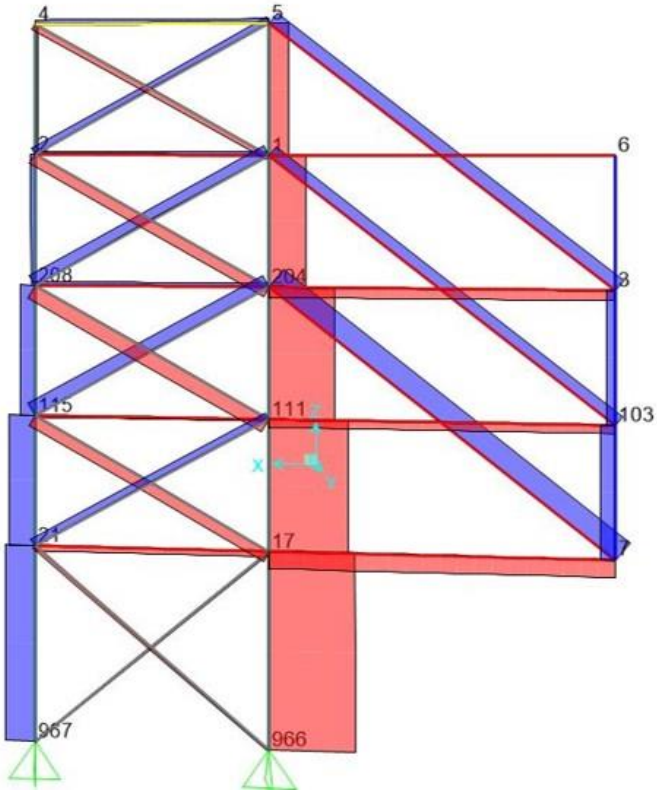


Figure 7.11 Axial Force - Proposal 1 – North View

Top & Bottom Chords Check

The most compressed element is 7-17 from the model, with N_{Ed} = 8759.28 kN

To verify the adequacy of compression members, two types of checks are required:

1) Cross-section resistance
2) Member buckling resistance

For short compression members ($\bar{\lambda} \leq 0.2$), only the cross-sectional check is required, but for longer members, both checks should be carried out.

Cross-section classification

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (Table 5.2, sheet 1)

$c/t = 2.98$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 2.98$

$\therefore\ Flange\ is\ Class\ 1$

Web – internal part in compression (Table 5.2, sheet 1)

$c/t = 8.11$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 8.11$

$\therefore\ Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

Compression resistance of cross-section

The design compression resistance of the cross-section

$$\mathrm{N_{c,Rd}} = \frac{\mathrm{Af_y}}{\gamma_{\mathrm{m0}}}\ for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{59500 \times 355}{1.0} = 21122.50\ kN\ > 8759.28\ kN\ \therefore OK$$

Member Buckling Resistance in compression

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{M1}} \; fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}}$ for Class 1, 2 and 3 cross-section

Elastic Critical Force and non-dimensional slenderness for flexural buckling

For buckling about both the major (y-y) and minor (z-z) axes, L_{cr} should be taken as 1.0 times the actual (system) length of the member, since the truss has pinned end conditions.

$$N_{cr,y} = \frac{\pi^2 E I_y}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 791000000}{10000^2} = 37928.89 \times 10^3 \text{N}$$

$$= 37928.89 \text{ kN}$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{29900 \times 355}{37928.89 \times 10^3}} = 0.75$$

$$N_{cr,z} = \frac{\pi^2 E I_z}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 310000000}{10000^2} = 14052.34 \times 10^3 \text{N}$$

$= 14052.34$ kN

$$\therefore \bar{\lambda}_z = \sqrt{\frac{29900 \times 355}{14052.34 \times 10^3}} = 1.23$$

<u>Selection of buckling curve and imperfection factor α</u>

For a hot-rolled I section,

$$\frac{\mathrm{h}}{\mathrm{b}} = \frac{436.6}{412.2} = 1.06 < 1.2, and\ t_f = 58mm < 100mm$$

$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$

$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$

Buckling curves – major (y-y) axis

$$\Phi_{\mathrm{y}} = 0.5[1 + 0.34(0.75 - 0.2) + 0.75^2] = 0.87$$

$$\chi_y = \frac{1}{0.87 + \sqrt{0.87^2 - 0.75^2}} = 0.76$$

$$\therefore \mathrm{N_{b,y,Rd}} = \frac{0.76 \times 29900 \times 355}{1.0} = 15988.86 \times 10^3 \mathrm{N}$$

$= 15988.86$ kN

$15988.86 > 8759.28$ kN

$\therefore Major\ axis\ flexural\ buckling\ resistance\ is\ OK$

Buckling curves – minor (z-z) axis

$$\Phi_{\mathrm{z}} = 0.5[1 + 0.49(1.23 - 0.2) + 1.23^2] = 1.50$$

$$\chi_z = \frac{1}{1.50 + \sqrt{1.50^2 - 1.23^2}} = 0.42$$

$$\therefore \mathrm{N_{b,z,Rd}} = \frac{0.42 \times 29900 \times 355}{1.0} = 8903.99 \times 10^3 \mathrm{N} = 8903.99 \text{ kN}$$

$8903.99 > 8759.28$ kN

$\therefore$ *Minor axis flexural buckling resistance is OK*

The UKC 356*406*467 in grade S355 steel is suitable to support the design load $\mathrm{N_{Ed}}$ of 8759.28 kN. The resistance of this truss member is governed by buckling about the z-z axis, with $\mathrm{N_{b,z,Rd}}$ = 8903.99 kN

Critical Diagonals Check

The inclined rod is only in tension, which means that the tension needs to be checked once the connection design is complete.

Critical Vertical Element Check

The most compressed element is 6-8 from the model in Figure 7.11, $\mathrm{N_{Ed}}$ = 36.28 kN

To verify the adequacy of compression members, two types of checks are required:

1) Cross-section resistance
2) Member buckling resistance

For short compression members ($\bar{\lambda} \leq 0.2$), only the cross-sectional check is required, but for longer members, both checks should be carried out.

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (Table 5.2, sheet 1)

$c/t = 5.73$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 5.73$

$\therefore\ Flange\ is\ Class\ 1$

Web – internal part in compression (Table 5.2, sheet 1)

$c/t = 15.8$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 15.8$

$\therefore\ Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Compression resistance of cross-section</u>

The design compression resistance of the cross-section

$$\mathrm{N_{c,Rd}} = \frac{A\mathrm{f_y}}{\gamma_{\mathrm{m0}}}\ for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{29900 \times 355}{1.0} = 10614.50\ kN\ > 36.28\ kN\ \therefore OK$$

<u>Member Buckling Resistance in compression</u>

$$\mathrm{N_{b,Rd}} = \frac{\chi \mathrm{Af_y}}{\gamma_{\mathrm{Ml}}}\ fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\frac{\mathrm{Af_y}}{\mathrm{N_{cr}}}}$ for Class 1, 2 and 3 cross-section

<u>Elastic Critical Force and non-dimensional slenderness for flexural buckling</u>

For buckling about both the major (y-y) and minor (z-z) axes, L_{cr} should be taken as 1.0 times the actual (system) length of the member, since the truss has pinned end conditions.

$$\mathrm{N_{cr,y}} = \frac{\pi^2 \mathrm{EI_y}}{\mathrm{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 791000000}{4000^2} = 102465 \times 10^3 \mathrm{N} = 102465\mathrm{kN}$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{29900 \times 355}{102465 \times 10^3}} = 0.32$$

$$\mathrm{N_{cr,z}} = \frac{\pi^2 \mathrm{EI}_z}{\mathrm{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 310000000}{4000^2} = 40156.95 \times 10^3 \mathrm{N}$$

$$= 40156.95\mathrm{kN}$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{29900 \times 355}{40156.95 \times 10^3}} = 0.51$$

<u>Selection of buckling curve and imperfection factor α</u>

For a hot-rolled I section,

$$\frac{\mathrm{h}}{\mathrm{b}} = \frac{381}{394.8} = 0.97 < 1.2, and\ t_f = 30.2mm < 100mm$$

$$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$$

$$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$$

Buckling curves – major (y-y) axis

$$\Phi_{\mathrm{y}} = 0.5[1 + 0.34(0.32 - 0.2) + 0.32^2] = 0.57$$

$$\chi_y = \frac{1}{0.57 + \sqrt{0.57^2 - 0.32^2}} = 0.96$$

$$\therefore \mathrm{N_{b,y,Rd}} = \frac{0.96 \times 29900 \times 355}{1.0} = 10147.85 \times 10^3\mathrm{N}$$

$$= 10147.85\mathrm{kN}$$

$$10147.85 > 36.28\ \mathrm{kN}$$

$$\therefore Major\ axis\ flexural\ buckling\ resistance\ is\ OK$$

Buckling curves – minor (z-z) axis

$$\Phi_{\mathrm{z}} = 0.5[1 + 0.49(0.51 - 0.2) + 0.51^2] = 0.71$$

$$\chi_z = \frac{1}{0.71 + \sqrt{0.71^2 - 0.51^2}} = 0.84$$

$$\therefore \mathrm{N_{b,z,Rd}} = \frac{0.84 \times 29900 \times 355}{1.0} = 8863.74 \times 10^3\mathrm{N} = 8863.74\mathrm{kN}$$

$$8863.74 > 36.28\ \mathrm{kN}$$

$\therefore$ *Minor axis flexural buckling resistance is OK*

The UKC 356*406*235 in grade S355 steel is suitable to support the design load N_{Ed} of 36.28 kN. The resistance of this truss member is governed by buckling about the z-z axis, with $N_{b,z,Rd}$ = 8863.74 kN.

7.2.5. Deflection Check of North View

The cross-sectional area of members is shown in the table below:

Table 7.7 Section area of truss members

Section Designation	Colour	Area (cm^2)
UKC 356*406*235		299
UKC 356*406*467		595

Firstly, the member forces are first evaluated by considering the horizontal and vertical components of reaction at both supports. By using the method of sections or joint resolution, the results are as follows:

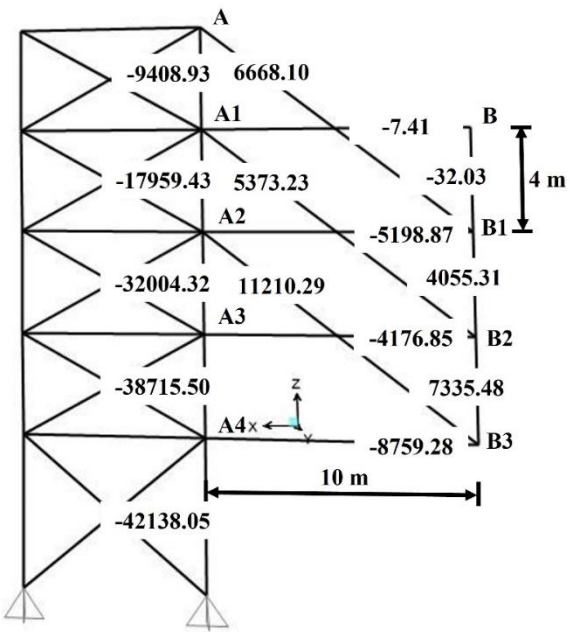


Figure 7.12 P forces (in kN) - Proposal 1 - North View

To determine the vertical deflection at joint B3, remove the externally applied load system and apply a unit load only in a vertical direction at joint B3 as shown in the figure. Use the method of sections or joint resolution as before to determine the magnitude and sense of the unknown member forces.

The deflection is given by:

$$v = \sum_{i=1}^{i=b} N_{1,i} \frac{F_i l_i}{E S_i}$$

Where:

$N_{1,\underline{i}}$ is the axial force produced in the member i by a unit force applied at the point where the deflection is required

l_i is the length of member i

S_i is the section area of the member i

b is the number of members with bolted connection(s)

$\frac{F_i l_i}{E S_i}$ is the variation in length of member I due to the slack recovery

$= \pm 4$mm according to whether the chord is in compression or tension

This is a significant additional deflection, compared with deflection due to the ULS combination.

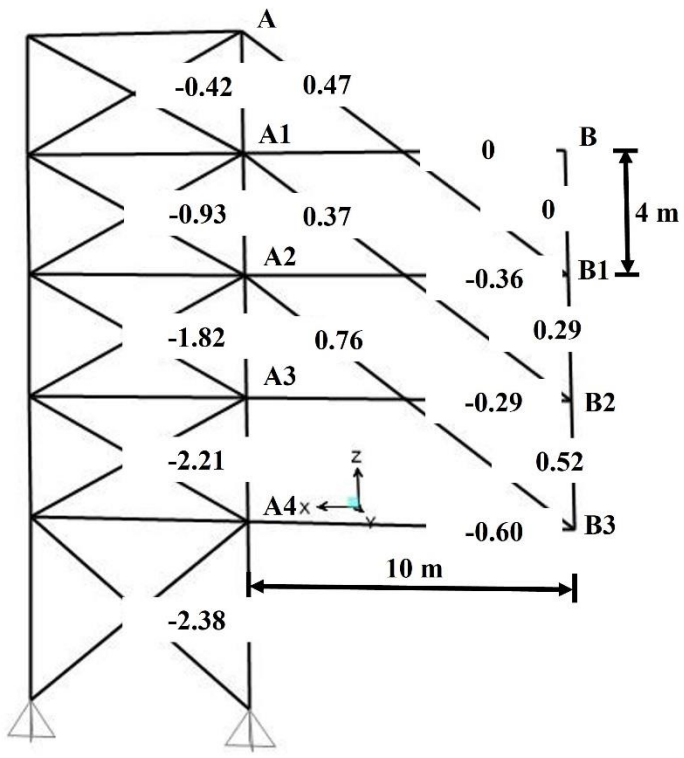


Figure 7.13 u forces - Proposal 1 - North View

The vertical deflection $\delta_{V,E'} = \sum \frac{PL}{AE} u$

The +ve sign indicates that the deflection is in the same direction as the applied unit load.

(PL*u)/AE instead of only (PL*u) has been computed for members with different geometric and mechanical properties, such as cross-sectional area and elastic modulus.

This is the better calculated in tabular form as shown in table below:

Table 7.8 Vertical deflection - Proposal 1 - North View

Member	Length (mm)	A (mm^2)	E (GPa)	AE (kN)	P-force (kN)	PL/AE (mm)	u	(PL/AE)*u (mm)
A1B	10000	59500	210	12495000	-7.41	-0.01	0.00	0.00

A2B1	10000	59500	210	12495000	-5198.87	-4.16	-0.36	1.50
A3B2	10000	59500	210	12495000	-4176.85	-3.34	-0.29	0.97
A4B3	10000	59500	210	12495000	-8759.28	-7.01	-0.60	4.21
BB1	4000	29900	210	6279000	-32.03	-0.02	0.00	0.00
B1B2	4000	29900	210	6279000	4055.31	2.58	0.29	0.75
B2B3	4000	29900	210	6279000	7335.48	4.67	0.52	2.43
AB1	12810	59500	210	12495000	6668.1	6.84	0.47	3.21
A1B2	12810	59500	210	12495000	5373.23	5.51	0.37	2.04
A2B3	12810	59500	210	12495000	11210.29	11.49	0.76	8.73
								23.84

From Table NA2 to EC3, the maximum deflection limit of the cantilever is Length/180, which equals to 55.56mm. Hand-calculated results meet the requirements. The result was then compared with those in SAP2000 model, which was 99.44mm. The reason for this huge difference is that there is an interaction between bracing and truss, and the effect of this interaction is significant. Owing to this, there is a significant difference as the truss members were calculated independently of each other in the calculations.

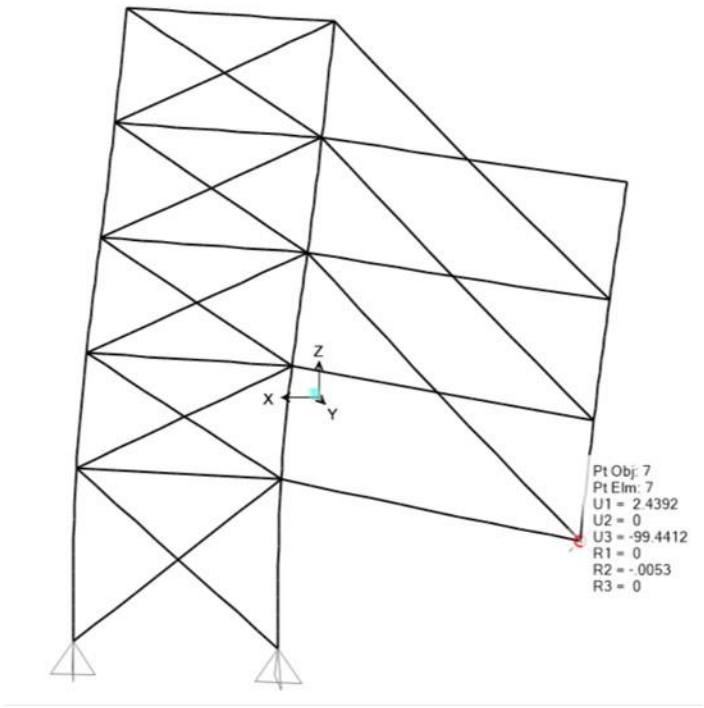


Figure 7.14 Max deflection - Proposal 1 – North View (from model)

7.3. Proposal 2

7.3.1. Preliminary Truss Design

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$N_1 = \frac{M}{D}, Assume\ \boldsymbol{N}_{b,Rd} = 0.70\boldsymbol{f}_y A$

$\therefore \frac{\boldsymbol{N}_1}{0.70\boldsymbol{f}_y} \Rightarrow\ Area\ of\ cross\ section\ required$

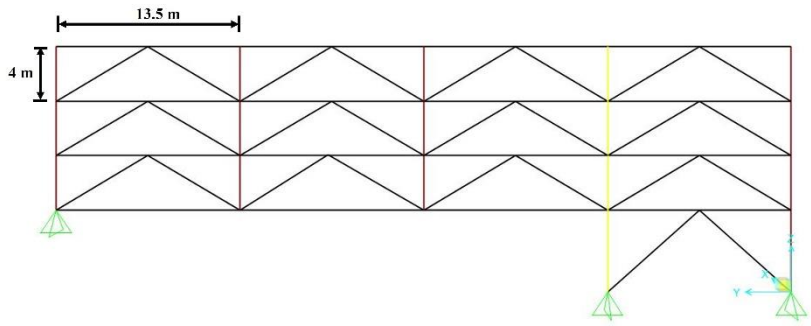


Figure 7.15 Truss - PROPOSAL 2 - West View

Initial calculations were made manually to find member sizes which could then be adjusted to meet the final stress and deflection requirements if required.

The main truss is simplified to a simply supported beam model and the force diagram is shown below:

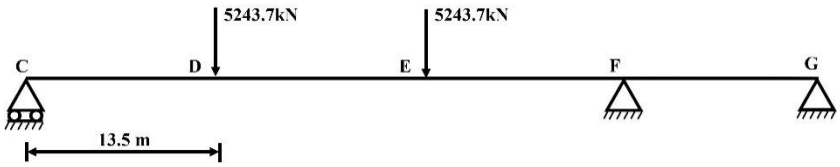


Figure 7.16 Force diagram of the simplified model - PROPOSAL 2 – West View

$Maximum\ bending\ moment\ M_{max} = 70790.46\ kNm$

$$\therefore N_1 = \frac{M}{D} = 5899.21\ kN$$

$$\Rightarrow A = \frac{\boldsymbol{N}_1}{0.70\boldsymbol{f_y}} = 237.39\ cm^2$$

And the deflection of the truss can also be determined in an approximate way:

$$The\ second\ moment: I = \frac{AD^2}{2} = 1.71 \times 10^{12}\ cm^4$$

Deflection caused by point load at D:

$$\delta_{mid,D} = -\vartheta(L/2) = \frac{P_D b_D(3L^2 - 4b_D^2)}{48EI} = 17.22\ mm\ (\downarrow)$$

Deflection caused by point load at E:

$$\delta_{mid,E} = -\vartheta(L/2) = \frac{P_E b_E(3L^2 - 4b_E^2)}{48EI} = 17.22\ mm\ (\downarrow)$$

Simply consider that the uniform distribute load q is equal to the ratio of the point load on the column to the width of the bearing area of the column load:

$$q = \frac{4P_{Col}}{b} = 332.76\ kN/m$$

Deflection caused by UDL:

$$\delta_{mid,UDL} = \frac{5qL^4}{384EI} = 32.48\ mm\ (\downarrow)$$

$Hence, the\ total\ deflection\ is$:

$\delta_{total} = 66.92\ mm(\downarrow)$

Application of the same method for the preliminary design of the truss on the north elevation of the building. It should be noted that the truss on the northern side is shorter than the truss on the western side, so the stiffness is relatively large. Owing to this, the design can be simplified into a cantilevered beam with a fixed section and a free end. The free end is subjected to a point load whose value is equal to the support reaction of the truss on the western side at that point.

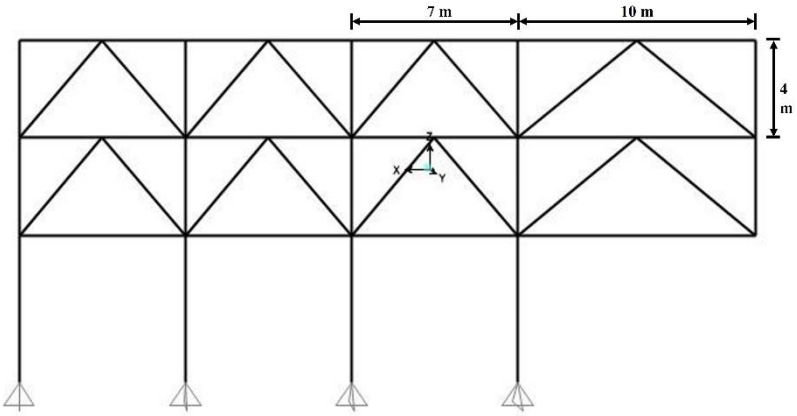


Figure 7.17 Truss - PROPOSAL 2 – North View

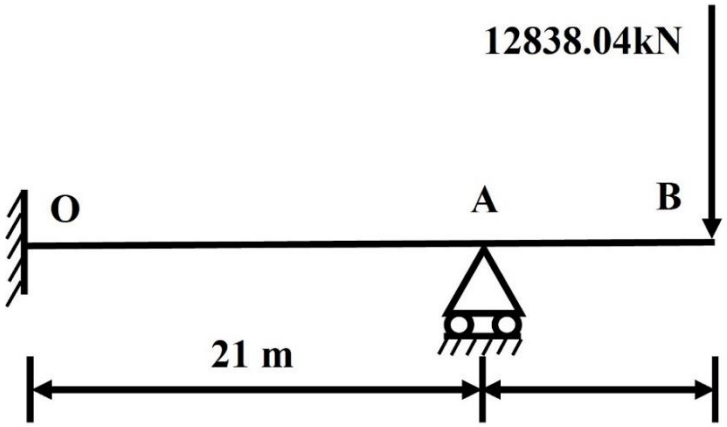


Figure 7.18 Force diagram of the simplified model - PROPOSAL 2 - North View

$Maximum\ bending\ moment\ M_{max} = 128380.4\ kNm$

$$\therefore N_1 = \frac{M}{D} = 16047.55\ kN$$

$$\Rightarrow A = \frac{\boldsymbol{N}_1}{0.70\boldsymbol{f_y}} = 645.78\ cm^2$$

$And\ the\ deflection\ of\ the\ truss\ can\ also\ be\ determined\ in$

$an\ approximate\ way$:

$$The\ second\ moment: I = \frac{AD^2}{2} = 2.07 \times 10^{12}\ cm^4$$

$Deflection\ caused\ by\ point\ load$:

$$\delta_{C,P} = -\vartheta(L/2) = \frac{PL^3}{8EI} = 3.70\ mm\ (\downarrow)$$

Using the same simplification as UDL for truss on the west side, the point loads and bearing areas to which Column B and Column C are subjected are different, resulting in the evenly distributed load on the cantilever beam being divided into two different parts. It is obvious that the uniform load on the B side of the column (closer to the fixed junction) is larger. Therefore, a mean of the two parts of the UDL acting together can be taken during design since this is a less favourable situation than the real one.

$q = 109.85\ kN/m$

$Deflection\ caused\ by\ UDL$:

$$\delta_{C,UDL} = \frac{qL^4}{8EI} = 0\ mm\ (\downarrow)$$

$Hence, the\ total\ deflection\ is$:

$\delta_{total} = 3.70\ mm(\downarrow)$

As it is not only a truss, but also a cantilever, it will be more complex in terms of forces, and careful considerations should be made for the choice of an appropriate cross-section.

The design criteria should satisfy the 'simple design' approach in which the connections are deemed to be pinned as described in cl. 5.1.2(2) of EC3. It requires the frame to be sufficiently braced in two orthogonal directions in order to achieve global stability.

7.3.1. Truss West View (West View) Check

The changes to the section are shown in the table below:

Table 7.9 Section Properties - PROPOSAL 2 - West View

Section Designation	Colour
UKC 356*406*235	
UKC 356*406*340	
UKC 356*406*634	

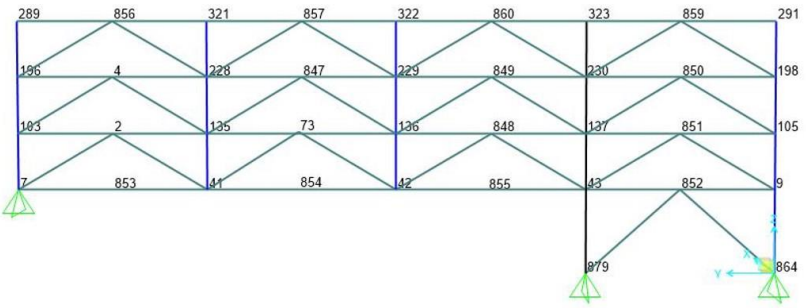


Figure 7.19 Section Properties - PROPOSAL 2 - West View

Check the compression of the top and bottom chords, critical diagonals and critical vertical elements.

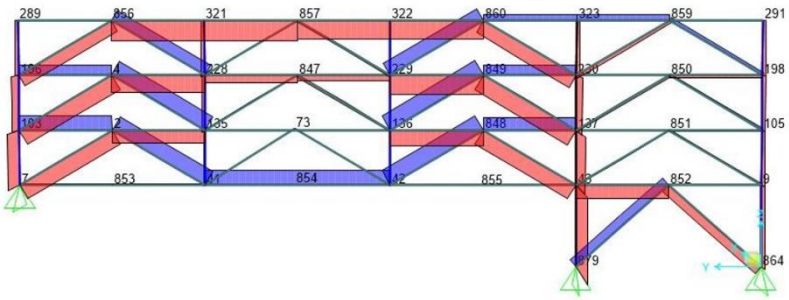


Figure 7.20 Axial Forces - Proposal 1 - West View

Top & Bottom Chords Check

The most compressed element is 15-982, N_{Ed} = 7407.56 kN

To verify the adequacy of compression members, two types of checks are required:

1) Cross-section resistance
2) Member buckling resistance

For short compression members ($\bar{\lambda} \leq 0.2$), only the cross-sectional check is required, but for longer members, both checks should be carried out.

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 4.03$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 4.03$

$\therefore Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 10.9$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 10.9$

$\therefore Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Compression resistance of cross-section</u>

The design compression resistance of the cross-section

$$\mathrm{N_{c,Rd}} = \frac{\mathrm{Af_y}}{\gamma_{\mathrm{m0}}}\ for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{43300 \times 355}{1.0} = 15371.5\ kN\ > 7407.56\ kN\ \therefore OK$$

<u>Member Buckling Resistance in compression</u>

$$\mathrm{N_{b,Rd}} = \frac{\chi A \mathrm{f_y}}{\gamma_{\mathrm{Ml}}}\ fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}}\ \text{ but }\ \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\frac{\mathrm{Af_y}}{\mathrm{N_{cr}}}}$ for Class 1, 2 and 3 cross-section

Elastic Critical Force and non-dimensional slenderness for flexural buckling

For buckling about both the major (y-y) and minor (z-z) axes, L_{cr} should be taken as 1.0 times the actual (system) length of the member, since the truss has pinned end conditions.

$$N_{cr,y} = \frac{\pi^2 EI_y}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 1230000000}{6750^2} = 55952.13 \times 10^3 \text{N}$$

$$= 55952.13 \text{ kN}$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{43300 \times 355}{55952.13 \times 10^3}} = 0.52$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 469000000}{6750^2} = 21334.59 \times 10^3 \text{N}$$

$$= 21334.59 \text{ kN}$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{43300 \times 355}{21334.59 \times 10^3}} = 0.85$$

Selection of buckling curve and imperfection factor α

For a hot-rolled I section,

$$\frac{h}{b} = \frac{406.4}{403} = 1.01 < 1.2, and\ t_f = 42.9mm < 100mm$$

$\therefore Use\ buckling\ curve\ b\ for\ y-y\ and\ curve\ c\ for\ z-z$

$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$

Buckling curves – major (y-y) axis

$\Phi_y = 0.5[1 + 0.34(0.52 - 0.2) + 0.52^2] = 0.69$

$$\chi_y = \frac{1}{0.69 + \sqrt{0.69^2 - 0.52^2}} = 0.87$$

$$\therefore N_{b,y,Rd} = \frac{0.87 \times 43300 \times 355}{1.0} = 13424.90 \times 10^3 N$$

$= 13424.90kN$

$13424.90 > 7407.56$ kN

$\therefore Major\ axis\ flexural\ buckling\ resistance\ is\ OK$

Buckling curves – minor (z-z) axis

$\Phi_z = 0.5[1 + 0.49(0.85 - 0.2) + 0.85^2] = 1.02$

$$\chi_z = \frac{1}{1.02 + \sqrt{1.02^2 - 0.85^2}} = 0.63$$

$$\therefore N_{b,z,Rd} = \frac{0.63 \times 43300 \times 355}{1.0} = 9708.02 \times 10^3 N = 9708.02kN$$

$9708.02 > 7407.56$ kN

$\therefore Minor\ axis\ flexural\ buckling\ resistance\ is\ OK$

The UKC 356*406*340 in grade S355 steel is suitable to support the design load N_{Ed} of 7407.56 kN. The resistance of this truss member is governed by buckling about the z-z axis, with $N_{b,z,Rd}$ = 9708.02 kN

Critical Diagonals Check

The most compressed element is 121-849, N_{Ed} = 6219.13 kN

Since the diagonals have the same cross-section with top & bottom chord, and only the axial load is considered. Thus, the check is not repeated.

The UKC 356*406*340 in grade S355 steel is suitable to support the design load N_{Ed} of 6219.13 kN. The resistance of this truss member is governed by buckling about the z-z axis, with $N_{b,z,Rd}$ = 9708.02 kN.

Critical Vertical Element Check

The most compressed element is 7-103, N_{Ed} = 5824.35 kN

To verify the adequacy of compression members, two types of checks are required:

1) Cross-section resistance
2) Member buckling resistance

For short compression members ($\bar{\lambda} \leq 0.2$), only the cross-sectional check is required, but for longer members, both checks should be carried out.

<u>Cross-section classification</u>

$$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$$c/t = 5.73$$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 5.73$

$\therefore\ Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 15.8$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 15.8$

$\therefore\ Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Compression resistance of cross-section</u>

The design compression resistance of the cross-section

$$\mathrm{N_{c,Rd}} = \frac{A\mathrm{f_y}}{\gamma_{\mathrm{m0}}}\ for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{29900 \times 355}{1.0} = 10614.50\ kN\ > 5824.35\ kN\ \therefore OK$$

<u>Member Buckling Resistance in compression</u>

$$\mathrm{N_{b,Rd}} = \frac{\chi \mathrm{Af_y}}{\gamma_{\mathrm{Ml}}}\ fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}}\ \text{ but }\ \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\frac{\mathrm{Af_y}}{\mathrm{N_{cr}}}}$ for Class 1, 2 and 3 cross-section

Elastic Critical Force and non-dimensional slenderness for flexural buckling

For buckling about both the major (y-y) and minor (z-z) axes, L_{cr} should be taken as 1.0 times the actual (system) length of the member, since the truss has pinned end conditions.

$$N_{cr,y} = \frac{\pi^2 EI_y}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 791000000}{4000^2} = 102465 \times 10^3 \text{N} = 102465\text{kN}$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{29900 \times 355}{102465 \times 10^3}} = 0.32$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 310000000}{4000^2} = 40156.95 \times 10^3 \text{N}$$

$$= 40156.95\text{kN}$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{29900 \times 355}{40156.95 \times 10^3}} = 0.51$$

Selection of buckling curve and imperfection factor α

For a hot-rolled I section,

$$\frac{h}{b} = \frac{381}{394.8} = 0.97 < 1.2, and\ t_f = 30.2mm < 100mm$$

$$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$$

$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$

Buckling curves – major (y-y) axis

$\Phi_y = 0.5[1 + 0.34(0.32 - 0.2) + 0.32^2] = 0.57$

$$\chi_y = \frac{1}{0.57 + \sqrt{0.57^2 - 0.32^2}} = 0.96$$

$$\therefore N_{b,y,Rd} = \frac{0.96 \times 29900 \times 355}{1.0} = 10147.85 \times 10^3 N$$

$= 10147.85 kN$

$10147.85 > 5824.35$ kN

$\therefore Major\ axis\ flexural\ buckling\ resistance\ is\ OK$

Buckling curves – minor (z-z) axis

$\Phi_z = 0.5[1 + 0.49(0.51 - 0.2) + 0.51^2] = 0.71$

$$\chi_z = \frac{1}{0.71 + \sqrt{0.71^2 - 0.51^2}} = 0.84$$

$$\therefore N_{b,z,Rd} = \frac{0.84 \times 29900 \times 355}{1.0} = 8863.74 \times 10^3 N = 8863.74 kN$$

$8863.74 > 5824.35$ kN

$\therefore Minor\ axis\ flexural\ buckling\ resistance\ is\ OK$

The UKC 356*406*235 in grade S355 steel is suitable to support the design load N_{Ed} of 5824.35 kN. The resistance of this truss member is governed by buckling about the z-z axis, with $N_{b,z,Rd}$ = 8863.74 kN.

Lateral deflection check at serviceability limit state is needed. And then we will compare it with a simple computer model.

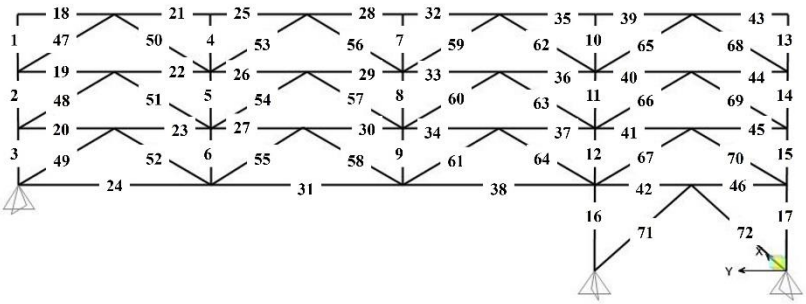


Figure 7.21 Member numbers – PROPOSAL 2 – West View

This is the better calculated in tabular form as shown in table below:

Table 7.10 Vertical deflection - PROPOSAL 2 - West View

Member	Length (mm)	A (mm2)	E (GPa)	AE (kN)	P-force (kN)	PL/AE (mm)	u	(PL/AE)*u (mm)
1	4000	29900	210	6279000	-2800.02	-1.78	0.00	0.00
2	4000	29900	210	6279000	-5819.93	-3.71	-0.05	0.19
3	4000	29900	210	6279000	-9680.75	-6.17	-0.14	0.86
4	4000	29900	210	6279000	-5093.66	-3.24	0.00	0.00
5	4000	29900	210	6279000	-3890.91	-2.48	-0.08	0.20
6	4000	29900	210	6279000	-1815.45	-1.16	-0.15	0.17
7	4000	29900	210	6279000	-5100.13	-3.25	0.10	-0.32
8	4000	29900	210	6279000	-3785.46	-2.41	0.36	-0.87
9	4000	29900	210	6279000	-1985.3	-1.26	0.78	-0.99
10	4000	18060	210	3792600	-5293.43	-5.58	0.00	0.00
11	4000	18060	210	3792600	-9936.01	-10.48	-0.26	2.72
12	4000	18060	210	3792600	-14912.51	-15.73	-0.56	8.81
13	4000	29900	210	6279000	-2613.99	-1.67	0.00	0.00
14	4000	29900	210	6279000	-2686.64	-1.71	0.03	-0.05
15	4000	29900	210	6279000	-2868.05	-1.83	0.07	-0.13
16	6000	18060	210	3792600	-19646.97	-31.08	-0.81	25.18
17	6000	43300	210	9093000	-3371.95	-2.22	0.11	-0.24
18	6750	43300	210	9093000	-11.51	-0.01	0.00	0.00
19	6750	43300	210	9093000	4018.22	2.98	0.08	0.24
20	6750	43300	210	9093000	5402.47	4.01	0.15	0.60
21	6750	43300	210	9093000	-7757.46	-5.76	-0.17	0.98

22	6750	43300	210	9093000	-6601.76	-4.90	-0.20	0.98
23	6750	43300	210	9093000	-5009.28	-3.72	-0.40	1.49
24	13500	43300	210	9093000	267.37	0.40	0.02	0.01
25	6750	43300	210	9093000	-7768.61	-5.77	-0.16	0.92
26	6750	43300	210	9093000	-2904.28	-2.16	0.11	-0.24
27	6750	43300	210	9093000	55.39	0.04	0.00	0.00
28	6750	43300	210	9093000	-7230.48	-5.37	-0.62	3.33
29	6750	43300	210	9093000	-2194.17	-1.63	-0.39	0.64
30	6750	43300	210	9093000	55.39	0.04	0.00	0.00
31	13500	43300	210	9093000	5409.68	8.03	0.31	2.49
32	6750	43300	210	9093000	-7214.76	-5.36	-0.62	3.32
33	6750	43300	210	9093000	-7176.76	-5.33	-0.53	2.82
34	6750	43300	210	9093000	-5976.46	-4.44	-0.22	0.98
35	6750	43300	210	9093000	1916.1	1.42	0.11	0.16
36	6750	43300	210	9093000	3955.8	2.94	0.37	1.09
37	6750	43300	210	9093000	4905.91	3.64	0.47	1.71
38	13500	43300	210	9093000	-54.18	-0.08	-0.03	0.00
39	6750	43300	210	9093000	1976.32	1.47	0.11	0.16
40	6750	43300	210	9093000	306.07	0.23	0.06	0.01
41	6750	43300	210	9093000	-106.88	-0.08	0.07	-0.01
42	6750	43300	210	9093000	-5186.25	-3.85	-0.31	1.19
43	6750	43300	210	9093000	10.15	0.01	0.00	0.00
44	6750	43300	210	9093000	-934.46	-0.69	-0.06	0.04
45	6750	43300	210	9093000	-583.11	-0.43	-0.06	0.03
46	6750	43300	210	9093000	-134.52	-0.10	-0.07	0.01
47	7850	43300	210	9093000	-4700.1	-4.06	-0.10	0.41
48	7850	43300	210	9093000	-6308.55	-5.45	-0.17	0.93
49	7850	43300	210	9093000	-6193.01	-5.35	-0.31	1.66
50	7850	43300	210	9093000	4314.8	3.72	0.09	0.34
51	7850	43300	210	9093000	6042.94	5.22	0.16	0.83
52	7850	43300	210	9093000	5912.43	5.10	0.31	1.58
53	7850	43300	210	9093000	-8.69	-0.01	-0.27	0.00
54	7850	43300	210	9093000	185.54	0.16	-0.30	-0.05
55	7850	43300	210	9093000	-32.41	-0.03	0.00	0.00
56	7850	43300	210	9093000	-635.52	-0.55	0.25	-0.14
57	7850	43300	210	9093000	-641.8	-0.55	0.29	-0.16
58	7850	43300	210	9093000	-35.29	-0.03	0.00	0.00
59	7850	43300	210	9093000	5180.78	4.47	0.41	1.83
60	7850	43300	210	9093000	6360.49	5.49	0.53	2.91
61	7850	43300	210	9093000	6256.27	5.40	0.40	2.16
62	7850	43300	210	9093000	-5499.78	-4.75	-0.43	2.04
63	7850	43300	210	9093000	-6588.38	-5.69	-0.53	3.01
64	7850	43300	210	9093000	-6406.63	-5.53	-0.40	2.21

65	7850	43300	210	9093000	-1193.64	-1.03	-0.06	0.06
66	7850	43300	210	9093000	-782.5	-0.68	-0.06	0.04
67	7850	43300	210	9093000	-409.71	-0.35	-0.08	0.03
68	7850	43300	210	9093000	1092.96	0.94	0.07	0.07
69	7850	43300	210	9093000	659.4	0.57	0.07	0.04
70	7850	43300	210	9093000	143.24	0.12	0.08	0.01
71	7850	43300	210	9093000	3292.92	2.84	0.16	0.45
72	7850	43300	210	9093000	-3476.97	-3.00	-0.16	0.48
								79.23

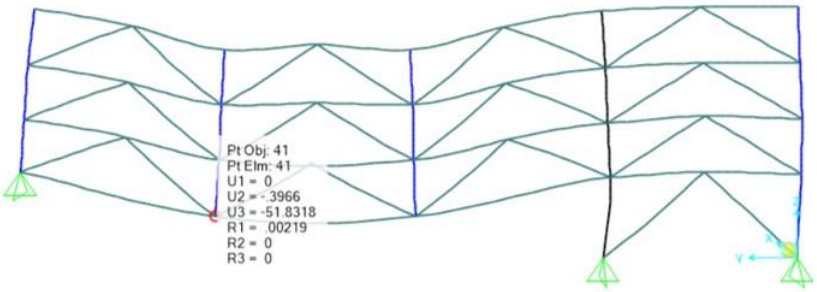


Figure 7.22 Max deflection - PROPOSAL 2 – West View

7.3.2. Truss North View Check

The changes to the section are shown in the table below:

Table 7.11 Section Properties – PROPOSAL 2 - North View

Section Designation	Colour
UKC 356*406*235	
UKC 356*406*467	

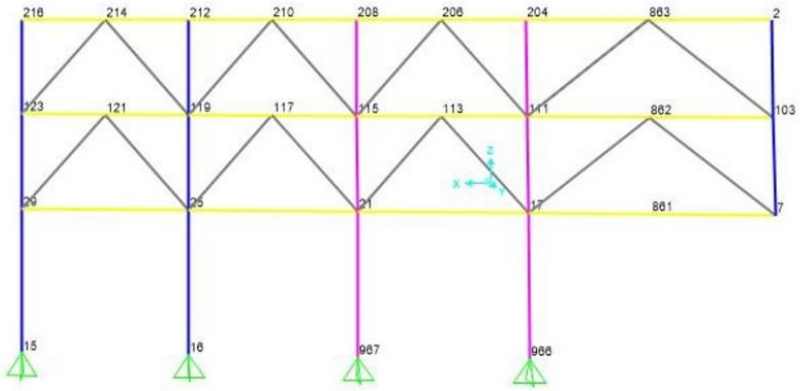


Figure 7.23 Section Properties - PROPOSAL 2 – North View

Check the compression of the top & bottom chords, critical diagonals and critical vertical elements.

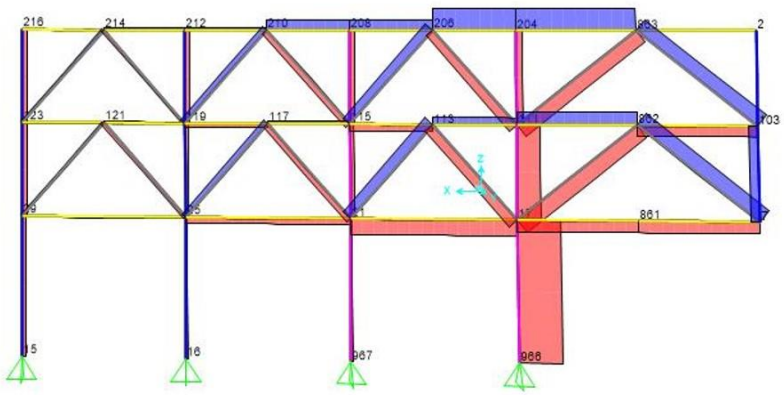


Figure 7.24 Axial Force - PROPOSAL 2 - North View

Critical Diagonals Check

The most compressed element is 321-857, N_{Ed} = 10007.56 kN

To verify the adequacy of compression members, two types of checks are required:

1) Cross-section resistance

2) Member buckling resistance

For short compression members ($\bar{\lambda} \leq 0.2$), only the cross-sectional check is required, but for longer members, both checks should be carried out.

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 4.03$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 4.03$

$\therefore\ Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 10.9$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 10.9$

$\therefore\ Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Compression resistance of cross-section</u>

The design compression resistance of the cross-section

$$\mathrm{N_{c,Rd}} = \frac{\mathrm{Af_y}}{\gamma_{\mathrm{m0}}}\ for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{43300 \times 355}{1.0} = 15371.5\ kN\ > 10007.56\ kN\ \therefore OK$$

<u>Member Buckling Resistance in compression</u>

$$\mathrm{N_{b,Rd}} = \frac{\chi A \mathrm{f_y}}{\gamma_{\mathrm{Ml}}}\ fro\ Class\ 1, 2\ and\ 3\ cross - section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}}\ \text{ but }\ \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\frac{\mathrm{Af_y}}{\mathrm{N_{cr}}}}$ for Class 1, 2 and 3 cross-section

<u>Elastic Critical Force and non-dimensional slenderness for flexural buckling</u>

For buckling about both the major (y-y) and minor (z-z) axes, L_{cr} should be taken as 1.0 times the actual (system) length of the member, since the truss has pinned end conditions.

$$\mathrm{N_{cr,y}} = \frac{\pi^2 \mathrm{EI_y}}{\mathrm{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 1230000000}{6400^2} = 62239.23 \times 10^3 \mathrm{N}$$

$$= 62239.23\ \mathrm{kN}$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{43300 \times 355}{62239.23 \times 10^3}} = 0.50$$

$$\mathrm{N_{cr,}}z = \frac{\pi^2 \mathrm{EI}_z}{\mathrm{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 469000000}{6400^2} = 23731.87 \times 10^3 \text{N}$$

$$= 23731.87 \text{ kN}$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{43300 \times 355}{23731.87 \times 10^3}} = 0.80$$

<u>Selection of buckling curve and imperfection factor α</u>

For a hot-rolled I section,

$$\frac{\text{h}}{\text{b}} = \frac{406.4}{403} = 1.01 < 1.2, and\ t_f = 42.9mm < 100mm$$

$$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$$

$$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$$

Buckling curves – major (y-y) axis

$$\Phi_{\text{y}} = 0.5[1 + 0.34(0.50 - 0.2) + 0.50^2] = 0.67$$

$$\chi_y = \frac{1}{0.67 + \sqrt{0.67^2 - 0.50^2}} = 0.89$$

$$\therefore \text{N}_{\text{b,y,Rd}} = \frac{0.89 \times 43300 \times 355}{1.0} = 13612.34 \times 10^3 \text{N}$$

$$= 13612.34\text{kN}$$

$$13612.34 > 10007.56 \text{ kN}$$

$$\therefore Major\ axis\ flexural\ buckling\ resistance\ is\ OK$$

Buckling curves – minor (z-z) axis

$\Phi_z = 0.5[1 + 0.49(0.80 - 0.2) + 0.80^2] = 0.97$

$$\chi_z = \frac{1}{0.97 + \sqrt{0.97^2 - 0.80^2}} = 0.97$$

$$\therefore N_{b,z,Rd} = \frac{0.97 \times 43300 \times 355}{1.0} = 10131.9 \times 10^3 N = 10131.9 kN$$

$10131.9 > 10007.56$ kN

$\therefore$ *Minor axis flexural buckling resistance is OK*

The UKC 356*406*340 in grade S355 steel is suitable to support the design load N_{Ed} of 10007.56 kN. The resistance of this truss member is governed by buckling about the z-z axis, with $N_{b,z,Rd}$ = 10131.9 kN

Critical Vertical Element Check

It has been found that the vertical members are purely tensile, therefore there is no need to carry out a compression check, only a tension check after the nodes have been designed.

Lateral deflection check at serviceability limit state is needed. And then we will compare it with a simple computer model.

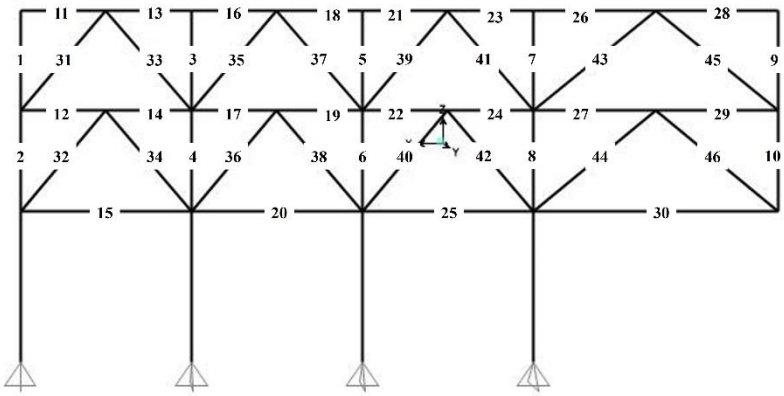


Figure 7.25 Member numbers - PROPOSAL 2 – North View

This is the better calculated in tabular form as shown in table below:

Table 7.12 Table showing the results of vertical element check

Member	Length (mm)	A (mm^2)	E (GPa)	AE (kN)	P-force (kN)	PL/AE (mm)	u	(PL/AE)*u (mm)
1	4000	29900	210	6279000	-3531.2	-2.25	0.00	0.00
2	4000	29900	210	6279000	-3327.48	-2.12	0.09	-0.19
3	4000	29900	210	6279000	-3480.36	-2.22	0.00	0.00
4	4000	29900	210	6279000	-2215.752	-1.41	0.09	-0.13
5	4000	153000	210	32130000	-3667.07	-0.46	0.00	0.00
6	4000	153000	210	32130000	-2712.03	-0.34	0.23	-0.08
7	4000	153000	210	32130000	-4613.08	-0.57	-0.02	0.01
8	4000	153000	210	32130000	-16358.87	-2.04	-0.88	1.79
9	4000	29900	210	6279000	-10.96	-0.01	0.00	0.00
10	4000	29900	210	6279000	6200.48	3.95	0.48	1.90
11	3500	36000	210	7560000	45.64	0.02	0.00	0.00
12	3500	36000	210	7560000	-885.08	-0.41	-0.08	0.03
13	3500	36000	210	7560000	1834.95	0.85	0.16	0.14
14	3500	36000	210	7560000	1052.67	0.49	0.75	0.37
15	7000	36000	210	7560000	-838.23	-0.78	-0.07	0.05
16	3500	36000	210	7560000	1854.98	0.86	0.16	0.14
17	3500	36000	210	7560000	-2587.38	-1.20	-0.17	0.20
18	3500	36000	210	7560000	7230.33	3.35	0.48	1.61
19	3500	36000	210	7560000	2349.14	1.09	0.15	0.16
20	7000	36000	210	7560000	-4247.31	-3.93	-0.31	1.22
21	3500	36000	210	7560000	7344.59	3.40	0.49	1.67
22	3500	36000	210	7560000	-4701.65	-2.18	-0.38	0.83
23	3500	36000	210	7560000	15524.82	7.19	1.20	8.62
24	3500	36000	210	7560000	4967.35	2.30	0.41	0.94
25	7000	36000	210	7560000	-11287.51	-10.45	-0.85	8.88
26	5000	36000	210	7560000	15428.24	10.20	1.19	12.14
27	5000	36000	210	7560000	8986.42	5.94	0.69	4.10
28	5000	36000	210	7560000	57.54	0.04	0.00	0.00
29	5000	36000	210	7560000	7702.55	5.09	-0.60	-3.06
30	1000	36000	210	7560000	-8344.09	-1.10	-0.65	0.72
31	5320	59500	210	12495000	1247.4	0.53	0.12	0.06
32	5320	59500	210	12495000	1357.9	0.58	0.12	0.07
33	5320	59500	210	12495000	-1471.2	-0.63	-0.12	0.08
34	5320	59500	210	12495000	-1585.88	-0.68	-0.12	0.08
35	5320	59500	210	12495000	4039.68	1.72	0.24	0.41
36	5320	59500	210	12495000	3640.23	1.55	0.24	0.37
37	5320	59500	210	12495000	-4124.3	-1.76	-0.24	0.42

38	5320	59500	210	12495000	-3857.65	-1.64	-0.25	0.41
39	5320	59500	210	12495000	6276.78	2.67	0.54	1.44
40	5320	59500	210	12495000	7165.36	3.05	0.59	1.80
41	5320	59500	210	12495000	-6145.04	-2.62	-0.53	1.39
42	5320	59500	210	12495000	-7517.31	-3.20	-0.61	1.95
43	6400	59500	210	12495000	-9798.58	-5.02	-0.75	3.76
44	6400	59500	210	12495000	-10775.65	-5.52	-0.83	4.58
45	6400	59500	210	12495000	9882.52	5.06	0.77	3.90
46	6400	59500	210	12495000	10593.91	5.43	0.82	4.45
								67.25

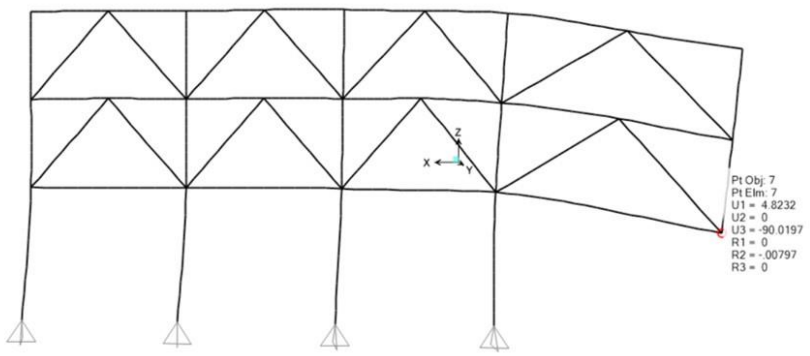


Figure 7.26 Max deflection - PROPOSAL 2 - North View

8. Beam-Beam Connections Design

In this section, the design of beam-to-beam connections will be explained in detail. Typical configuration of the connection with overall design procedures will be explained, followed by the relevant detailed calculations.

8.1. Typical Configuration of Connections

A typical configuration of beam-to-beam connection is shown in Figure 8.1.

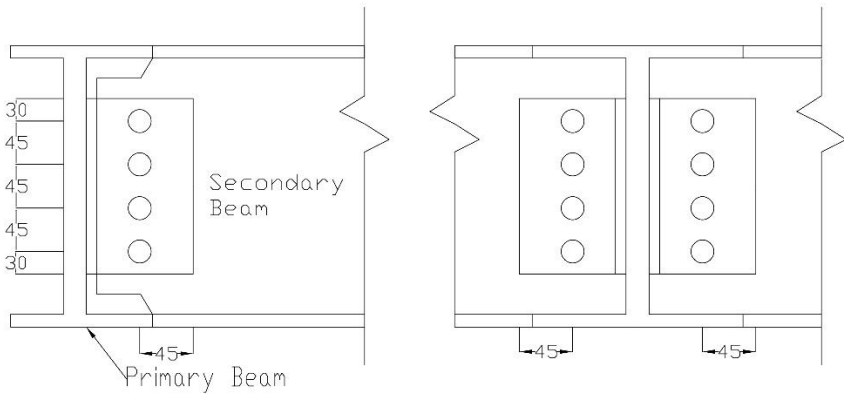


Figure 8.1: Figure showing the typical configuration of beam-to-beam connections

From the figure above, the secondary beam is connected to the primary beam using the bolts and cleat installed on their webs. Typical checks include the spacing between bolts, bearing and shear resistance of bolt group and the shear and bending resistance of the beam notched section. Details of the checks will be covered in the later section.

8.2. Actions considered during Design

Similar to the design of columns, only load case 1 ($1.35G_k + 1.5Q_k$ + EHF) was considered when calculating the actions for beams for maximum vertical forces. As the beam-beam connection was assumed to be a pinned-pinned connection, only the shear force at the end of secondary beam was considered.

8.3. Design Checks

There are a total of six main design checks for beam-to-beam connection design. Before conducting the checks, the proposed bolt spacings were first checked against Table 3.3 of BSEN1993-1-8. After fulfilling the relevant requirements, the main design checks were performed and details of each design check are summarized below:

8.3.1. Bearing Resistance of Bolt Group connecting Cleats to Main Beam Web (Check 1)

The bearing resistance of a single bolt can be calculated using the following equation:

$$F_{b1,Rd} = \frac{k_{1,1}\alpha_{b1}f_u dt}{\gamma_{M2}}$$

, where $k_{1,1}$, α_{b1} and γ_{M2} are constants, f_u, d and t are ultimate strength of steel, bolt diameter and thickness of steel respectively. To be conservative, the value of t was adopted as the smaller value between the thickness of beam web and web cleat.

As the value calculated above is the resistance per bolt, the total bearing resistance for the entire bolt group is given as:

$$V_{b1,Rd} = n_{b1}F_{b1,Rd}$$

, where n_{b1} is the total number of bolts per bolt group.

Should the resistance $V_{b1,Rd}$ be larger than the design shear force, the proposed design satisfies the design requirements.

8.3.2. Shear Resistance of Bolt Group connecting Cleats to Main Beam Web (Check 2)

The force in vertical direction on each bolt is given as:

$$F_{v1,Rd} = \frac{0.5f_{ub}A_s}{\gamma_{M2}}$$

, where f_{ub}, A_s and γ_{M2} are ultimate strength of bolt and stress area

of each bolt respectively.

As the value calculated above is the resistance per bolt, the total shear resistance for the entire bolt group is given as:

$$V_{v1,Rd} = n_{b1} F_{v1,Rd}$$

, where n_{b1} is the number of bolts per bolt group.

Should the resistance $V_{v1,Rd}$ be larger than the design shear force, the proposed design satisfies the design requirements.

8.3.3. Shear Resistance of Bolt Group connecting Cleats to Supported Beam Web (Check 3)

To determine the resultant shear force R on each bolt, the force in vertical F_y and horizontal F_x directions were computed using the following equations:

$$F_y = \frac{V_{Ed}}{n_{b2}}$$

$$F_x = \frac{M_{Ed} y_{max}}{\sum {y_i}^2}$$

$$R = \sqrt{{F_x}^2 + {F_y}^2}$$

, where n_{b2} is the number of bolts of bolt group on each side of the supported beam web.

M_{Ed} , y_{max} and y_i refer to the eccentricity moment, maximum vertical distance between bolt and centroid of bolt group and the vertical distance between bolt and centroid of bolt group for each bolt respectively.

Should the resultant force R be smaller than the shear resistance $2V_{v2,Rd}$ (due to double shear), the proposed design satisfies the design requirements.

8.3.4. Bearing Resistance of Bolt Group connecting Cleats to Supported Beam Web (Check 4)

Similar to Check 1, the bearing resistance of bolt group connecting to supported beam web $V_{b2,Rd}$ was calculated using similar procedures, except that the dimensions and properties of supported beam was used instead of those of main beam.

Should the resistance be larger than the design force, the design satisfies the requirements.

8.3.5. Shear Resistance of Beam Notched Section (Check 5)

In this check, the shear resistance of the beam notched section is verified. The check is divided into three parts, which are the resistance of net section, gross section and for block shear. The relevant shear resistance can be given by the set of equations below:

For gross section,

$$V_{Ed} \leq 2h_p t_p \frac{f_y}{\sqrt{3}\gamma_{M0}}$$

For net section,

$$V_{Ed} \leq 2\left(h_p - nd_0\right)t_p \frac{f_u}{\sqrt{3}\gamma_{M2}}$$

For block shear,

$$V_{Ed} \leq 2\left(\frac{f_u A_{nt}}{\gamma_{M2}} + \frac{f_y A_{nv}}{\sqrt{3}\gamma_{M0}}\right) = 2\left(\frac{f_u(e_2 - 0.5d_0)t_p}{\gamma_{M2}} + \frac{f_y\left(h_p - (n-0.5)d_0 - e_1\right)t_p}{\sqrt{3}\gamma_{M0}}\right)$$

, where V_{Ed} is the design shear force, γ_{M0} and γ_{M2} are constants, h_p, t_p, f_y and f_u are the height, thickness, yield strength and ultimate strength of the cleat, e_1 and e_2 are the end spacing of bolt group along direction of load and perpendicular to the direction of load and n and d_0 are the number of bolts per side of cleat and the

diameter of bolt hole respectively.

Should the three inequalities be satisfied, the design is checked to comply with the relevant requirements.

8.3.6. Bending Resistance of Beam Notched Section (Check 6)

The elastic bending resistance $M_{b,el,Rd}$ is given by the following equation:

$$M_{b,el,Rd} = \frac{W_b f_y}{\gamma_{M0}} = \frac{\left(\frac{2I_b}{h_p}\right) f_y}{\gamma_{M0}}$$

, where I_b, h_p and f_y are the second moment of area for net section of cleat, height of the cleat and yield strength of steel.

A reduction in bending resistance $M_{b,el,Rd}$ should be applied if the design shear force V_{Ed} is larger than half of the resistance V_{Rd} calculated in check 5. The reduction factor ρ and the reduced bending resistance $M_{b,el,Rd*}$ are given by the following equations:

$$\rho = \left(\frac{2V_{Ed}}{V_{Rd}} - 1\right)^2$$

$$M_{b,el,Rd*} = (1 - \rho) M_{b,el,Rd}$$

Should the bending resistance $M_{b,el,Rd*}$ be larger than the design moment given by $V_{Ed}(e)$, where e is the eccentricity of bolt group, the design satisfies the requirements.

8.4. Summary of Results

The results of the beam-to-beam connection design is summarized in the following figure.

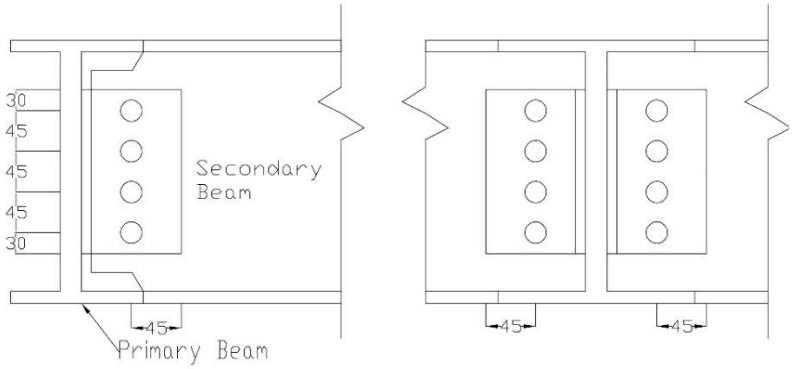


Figure 8.2: Figure showing the results of the beam-to-beam connection

In the calculation, 533x210x122 UB was adopted for both primary and secondary beams as per the design of beams in the earlier section. S355 2L90x90x9 cleats were proposed and all the bolts were assumed to be with Grade 8.8 and size M18.

8.5. Detailed Calculations

Based on the calculations, the proposed design satisfies all the requirements. The detailed calculations for the beam-to-beam connection are listed below:

Beam to Beam Connection

Primary Beam Size	533 x 210 x 122	UB
Secondary Beam Size	533 x 210 x 122	UB

Section Properties (Primary Beam)

Grade	355	
h_1	544.5	mm
b_1	211.9	mm
t_{w1}	12.7	mm
t_{f1}	21.3	mm
r_1	12.7	mm
A_1	155	cm^2

Section Properties (Secondary Beam)

Grade	355	
h_2	544.5	mm
b_2	211.9	mm
t_{w2}	12.7	mm
t_{f2}	21.3	mm
r_2	12.7	mm

A_2	155	cm^2

Splice Details

Cleats	2L 90 x 90 x 9 Angles	
Bolts	M18 8.8	8 nos.
Fittings Material	S355	

Design Actions

V_{Ed} =	249	kN

Check 1: Bearing Resistance of Bolt Group connecting Cleats to Beam Web

Spacings are desgined based on BSEN1993-1-8 Table 3.3:

Proposed e_1=	30	mm
Proposed e_2=	45	mm
Proposed p_1=	45	mm
Proposed p_2=	45	mm
Proposed Bolt Diameter d =	18	mm
Bolt Hole Diameter d_0=	20	mm

Therefore, proposed dimensions satisfies the following requirements:

$e_1>1.2d_0$, $e_2>1.2d_0$ and $p_1>2.2d_0$

Bolt Ultimate Strength $f_{u,b}$=	800	MPa
Beam Ultimate Strength f_u=	470	MPa
$\alpha_{d1}=\min(p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)$=	0.5	
$k_{1,1}=\min(2.8e_2/d_0-1.7,2.5)$=	2.5	

As thickness of cleats is smaller than that of beam web, use t=9mm to be conservative.

$F_{b1,Rd}=k_{1,1}\alpha_{d1}f_u dt/\gamma_{M2}$=	76.14	kN	
$V_{b1,Rd} = n_{b1}F_{b1,Rd}$ =	609.12	kN	**OK**

As it is larger than the design shear force, the proposed design satisfies the requirements.

Check 2: Shear Resistance of Bolt Group connecting Cleats to Main Beam Web

Bolt Stress Area A_s =	192	mm2	
$F_{v1,Rd}=0.5f_{ub}A_s/\gamma_{M2}$=	61.44	kN	
$V_{v1,Rd} = n_{b1}F_{b,Rd}$ =	491.52	kN	**OK**

As it is larger than the design shear force, the proposed design satisfies the requirements.

Check 3: Shear Resistance of Bolt Group connecting Cleats to Web of Supported Beam

Vertical Force per Bolt $F_y=V_{Ed}/n_{b2}$=	62.21	kN

Horizontal Force per Bolt $F_x=M_{Ed}y_{max}/\Sigma y_i^2$

F_x =	85.19	kN	
Resultant Force R =	105.49	kN	
For double shear, $F_{v2,Rd} = 2F_{v1,Rd}$ =	122.88	kN	**OK**

As it is larger than the resultant force, the proposed design satisfies the requirements.

<u>Check 4: Bearing Resistance of Bolt Group connecting Cleats to Web of Supported Beam</u>

$\alpha_{d1}=\min(e_1/(3d_0), p_1/(3d_0)-0.25, f_{ub}/f_u, 1)$=	0.5		
$k_{1,2}=\min(2.8e_2/d_0-1.7, 2.5)$=	2.5		

As beam web thickness is smaller than double of that of the two angles, use t=12.7mm to be conservative.

$F_{b2,Rd}=k_{1,2}\alpha_{d2}f_u dt/\gamma_{M2}$=	107.44	kN	**OK**

As it is larger than $F_{b1,Rd}$, the proposed design satisfies the requirements.

<u>Check 5: Shear Resistance of Beam Notched Section</u>

For net section, $V_{pl1,Rd}=A_{vnet}f_u/\sqrt{3}/\gamma_{M2}$=	317.05	kN	**OK**

As it is larger than the design shear force, the proposed design satisfies the requirements.

For gross section, $V_{pl2,Rd}=A_v f_y/\sqrt{3}/\gamma_{M0}$=	374.83	kN	**OK**

As it is larger than the design shear force, the proposed design satisfies the requirements.

For block shear, $V_{Rd,b}=[0.5A_{nt}f_u/\gamma_{M2}+A_{nv}f_y/\sqrt{3}/\gamma_{M0}]$

A_{nt} =	254	mm2	
A_{nv} =	1206.5	mm2	
$V_{Rd,b}$=	295.04	kN	**OK**

As it is larger than the design shear force, the proposed design satisfies the requirements.

<u>Check 6: Bending Resistance of Beam Notched Section</u>

Moment of Inertia I_b =	5275659	mm4	
$W_b = 2I_b/195$ =	54109.3	mm3	
$M_{b,el,Rd} = W_b f_y/\gamma_{M0}$=	19.21	kNm	
As $V_{Ed}>0.5V_{pl1,Rd}$,			
$\rho=(2V_{Ed}/V_{Rd}-1)^2$=	0.32		
$M_{b,el,Rd*} = (1-\rho)M_{b,el,Rd}$ =	12.97	kNm	
Eccentricity Moment $M_{Ed}=V_{Ed}(e)$=	12.78	kNm	**OK**

As it is smaller than $M_{b,el,Rd*}$, the proposed design satisfies the requirements.

9. Beam-Column Connections Design

The function of beam-column connection is to transfer shear forces from beam to column. There are three typical connections including web cleat connections, fin plate connections and end plate connections. For web cleat connections, two web cleats are bolted to both supporting and supported members. Fin plate connections has fin plate bolted on the supported beam and it is welded to supporting element with fillet weld.

In the design, all connections for connecting beam and column are end plate connections. The end plate is welded to the beam and bolted to the column. The use of the end plate can prevent curvature of the plate due to the existence of the bolts which is bearing the tension. Usually, the plate in this connection can be thinner than other types of connection which save material.

The height of the plate cannot be larger than the height of the beam. Therefore, there are two types of end plates: Full depth end plates and partial depth end plates. If $\frac{V_{Ed}}{V_{c,Rd}} < 0.75$, partial depth end plates are recommended for efficiency.

Access steel document SN013a-EN-EU

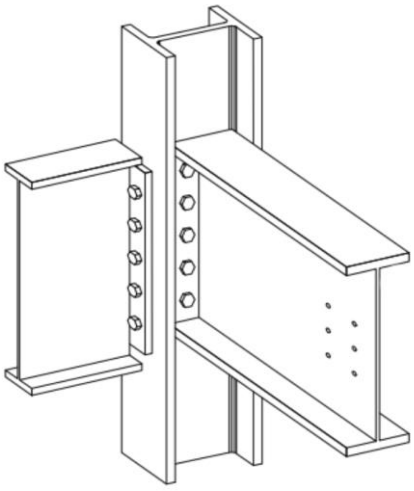


Figure 9.1 End plate connection for beam-column of different direction. Retrieved from "Joints in Steel Construction – Simple Joints to Eurocode 3" by The Steel Construction Institute (p. 10), 2014.

Except for shear force, partial depth end plates require tying resistance because all buildings must resist tying force in the standard. The thickness of the plate recommended in the project is 10mm when depth of supported beam is larger than

BS EN 1991-1-7

500mm.

The position of the end plates is close to the top flange so that it can provide positional restraint to the flange. Also, the spacing for holes should be limited with specific value.

Access steel document SN013a-EN-EU

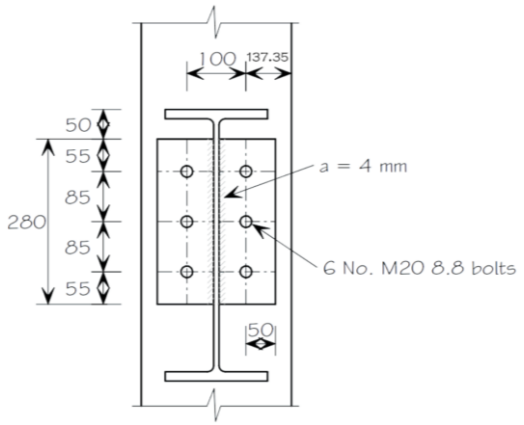


Figure 9.2 Positions of plate and holes for bolts. Retrieved from "Steel Building Design: Worked examples for students in accordance with Eurocodes and the UK National Annexes" by The Steel Construction Institute (p. 83), 2009.

Ductility check

Before checking the resistance of the connection, the ductility of connection must meet the design requirement for acting as pinned and it can be satisfied with column flange or the end plate within: $t_p \leq \frac{d}{2.8}\sqrt{\frac{f_{u,b}}{f_{y,p}}}$ or $t_{f,c} < \frac{d}{2.8}\sqrt{\frac{f_{u,b}}{f_{y,c}}}$

Resistance check

For joint shear resistance, bolts in shear, end plate in bearing and beam web in shear are three critical resistances to be checked. Tying resistances of end plate including bolts in tension and end plate in bending are checked. The tying force isn't calculated because normally has the same magnitude as the shear force.

The check for different connection positions and different columns is the same because the connection method is the same.

Shear resistance of a single bolt is

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

Access Steel document SN014a-EN-EU

A factor of 0.8 is introduced when calculating the shear resistance so that modest tension which doesn't calculated can be allowed. The total shear resistance should time number of bolts.

End plate in bearing means bearing resistance of a single bolt should be checked:

$$F_{v,Rd} = \frac{k_1 \alpha_b f_{u,p} d t_p}{\gamma_{M2}}$$

Where $\alpha_b = \min\left(\alpha_d, \frac{f_{u,b}}{f_{u,p}}, 1.0\right)$ and α_d is different for end bolts and inner bolts which may cause difference in the bearing resistance.

Only area of the beam web connected to the end plate has to be checked and the design plastic shear resistance is:

$$V_{pl,Rd} = V_{Rd,8} = \frac{A_v\left(f_y/\sqrt{3}\right)}{\gamma_{M0}}$$

A factor of 0.9 is introduced when calculating the plastic shear resistance.

Tension resistance of a single bolt is

$$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{Mu}}$$

The total tension resistance should time number of bolts.

Finally, end plate in bending is checked with

BS EN 1993-1-8 6.2.4 & Table 6.2

$$N_{Rd,u,2} = \min\left(F_{Rd,u,ep1}, F_{Rd,u,ep2}\right)$$

The critical load is smaller of two failure modes:

For mode 1: $F_{Rd,u,ep1} = F_{T,1,Rd} = \frac{(8n_p - 2e_w) M_{pl,1,Rd}}{2m_p n_p - e_w(m_p + n_p)}$

For mode 2: $F_{Rd,u,ep2} = F_{T,2,Rd} = \frac{(2M_{pl,2,Rd}+n_p \sum F_{t,Rd})}{m_p+n_p}$

Where $e_{c,2}$ is different for different size of columns and the least value is used when deciding n_p.

9.1. Calculation Progress

Size of the components for connection

Connection 1:

Column UKC 356×368×202 in S355 steel

Column UKC 356×368×1202 in S355 steel

Column UKC 356×368×235 in S355 steel

Column UKC 356×368×634 in S355 steel

Column UKC 356×368×467 in S355 steel

Beam UKB 533×210×122 in S355 steel

Beam details:

BS EN 1993-1-1 NA 2.4

$f_y = 355 \frac{N}{mm^2}$ $f_u = 470 \frac{N}{mm^2}$

$h_b = 544.5\ mm$ $b = 211.9\ mm$ $t_w = 12.7\ mm$ $t_f = 21.3\ mm$

BS EN 10025-2 Table 7

Column details:

$b = 374.7\ mm$ $h = 374.6\ mm$

Plate:

BS EN 1993-1-1 NA 2.15

$f_u = 470\ N/mm^2$

$$V_{c,Rd} \approx \frac{h_b \times t_w \times \left(\frac{f_y}{\sqrt{3}}\right)}{\gamma_{M0}} = \frac{544.5 \times 12.7 \times \left(\frac{355}{\sqrt{3}}\right)}{1.0 \times 10^{-3}} = 1417.33\ kN$$

Access steel document SN013a-EN-EU

From check for beam: $V_{Ed} = \frac{691.72}{2} = 345.86\ kN$

$345.86 < 0.75V_{c,Rd} = 0.75 \times 1417.33 = 1063.00\ kN$ so, a partial depth end plate is proposed. $h_b = 544.5\ mm > 500mm$ so, 10 mm endplate is proposed.

Minimum depth for end plate depth is $0.6h_b = 0.6 \times 544.5 = 326.7\ mm$, so proposed depth is 330 mm. Assuming M20 bolts, number of bolts= $\frac{V_{Ed}}{75} = \frac{345.86}{75} =$ 4.61, so proposed minimum number of bolts is 6.

Bolts details

The bolts are fully threaded, non-preloaded, M20 8.8, 60mm long as generally used in the UK.

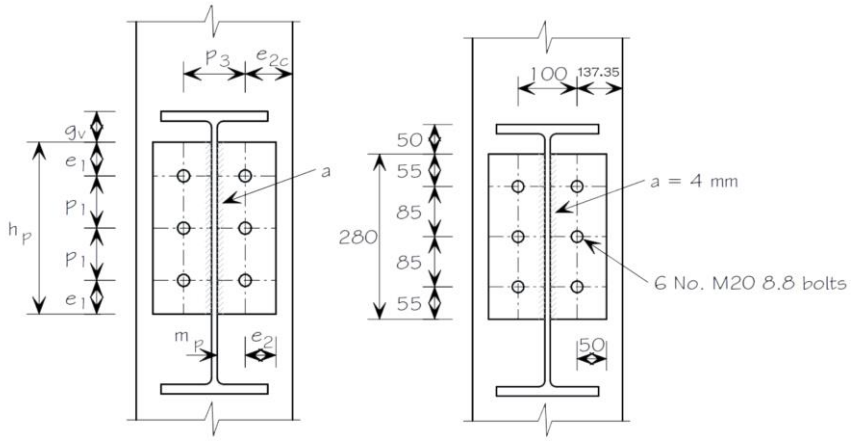


Figure 9.3 Position of holes for bolts. Retrieved from "Steel Building Design: Worked examples for students in accordance with Eurocodes and the UK National Annexes" by The Steel Construction Institute (p. 83), 2009.

Tensile stress area of bolt $A_s = 245\ mm^2$

Diameter of bolt $d = 20\ mm$

Diameter of the holes $d_0 = 22\ mm$

Diameter of the washer $d_w = 37\ mm$

Yield strength $f_{yb} = 640\ N/mm^2$

Ultimate tensile strength $f_{ub} = 800\ N/mm$^2

Position of bolts

$$e_1 = 55\ mm\ \ e_2 = 50\ mm\ \ p_1 = 85\ mm\ \ p_3 = 100\ mm$$

Table 9.1 Value of $e_{2,c}$

Column	$e_{2,c}$ (mm)
UKC 356×368×202	137.35
UKC 356×368×1202	185.5
UKC 356×368×235	147.4
UKC 356×368×634	162
UKC 356×368×467	156.1

BS EN 1993-1-8 3.5, Table 3.3

The least of $e_{2,c}$ is 137.35mm.

Limits for locations and spacing of bolts

Minimum $e_1 = 1.2d_0 = 1.2 \times 22 = 26.4\ mm < 55\ mm$

Minimum $e_2 = 1.2d_0 = 1.2 \times 22 = 26.4\ mm < 50\ mm$

Minimum $p_1 = 2.2d_0 = 2.2 \times 22 = 48.4\ mm < 85\ mm$

Minimum $p_3 = 2.4d_0 = 2.4 \times 22 = 52.8\ mm < 100\ mm$

Maximum $p_1 = 14t_p = 14 \times 10 = 140\ mm > 85\ mm$

BS EN 1993-1-8 4.7.3

Weld design

Throat $a \geq 0.39 \times t_w$ is required for full strength side welds.

Access Steel document SN014a-EN-EU

$$a > 0.39 \times 12.7 = 4.95\ mm$$

Adopt throat $a = 5\ mm$ leg $= 6\ \ mm$

BS EN 1993-1-1 6.1(1) Table 2.1

Patrial factors for resistance

BS EN 1993-1-8 NA 2.3 Table NA.1

$\gamma_{M0} = 1.0$

$\gamma_{M2} = 1.25\ (for\ shear)$

BS EN 1993-1-8 NA 2.3 Table NA.1

$\gamma_{M2} = 1.1\ (for\ bolts\ in\ tension)$

$\gamma_{Mu} = 1.1\ (for\ tying\ resistance)$ SN014a-EN-EU

Ductility check

The ductility of connection must meet the design requirement for acting as pinned and it can be satisfied with column flange or the end plate within: $t_p \leq \frac{d}{2.8}\sqrt{\frac{f_{u,b}}{f_{y,p}}}\ or\ t_{f,c} < \frac{d}{2.8}\sqrt{\frac{f_{u,b}}{f_{y,c}}}$

$$\frac{d}{2.8}\sqrt{\frac{f_{u,b}}{f_{y,p}}} = \left(\frac{20}{2.8}\right) \times \sqrt{\frac{800}{355}} = 10.72\ mm > t_p = 10\ mm$$

9.1.1. Joint shear resistance

Normally, 8 modes of failure shown in the table below are required to be checked.

Table 9.2 Shear resistance to be checked

Mode of failure	
Bolts in shear*	$V_{Rd,1}$
End plate in bearing*	$V_{Rd,2}$
Supporting member (column) in bearing	$V_{Rd,3}$
End plate in shear (gross section)	$V_{Rd,4}$
End plate in shear (net section)	$V_{Rd,5}$
End plate in block shear	$V_{Rd,6}$
End plate in bending	$V_{Rd,7}$
Beam web in shear*	$V_{Rd,8}$

However, only critical failures with * need to be checked in the connections of the project. 3.6.1 & Table 3.4

Bolts in shear

Shear resistance for shear planes assumed to pass through the threaded portion of the bolt is:

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

Access Steel document SN014a-EN-EU

A factor of 0.8 is introduced when calculating the shear resistance so that modest tension which doesn't calculated can be allowed.

For bolt class 8.8, $\alpha_v = 0.6$

$$F_{v,Rd} = 0.8 \times \frac{0.6 \times 800 \times 245}{1.25} \times 10^{-3} = 75.26\ kN$$

BS EN 1993-1-8 3.6.1 & Table 3.4

For 6 bolts, $V_{Rd,1} = 6 \times 75.26 = 451.58\ kN$

End plate in bearing

For a single bolt, $F_{v,Rd} = \frac{k_1 \alpha_b f_{u,p} d t_p}{\gamma_{M2}}$

Where $\alpha_b = \min\left(\alpha_d,\ \frac{f_{u,b}}{f_{u,p}}, 1.0\right)$

$$\alpha_d = \frac{e_1}{3d_0}\ for\ end\ bolts\ and\ \frac{p_1}{3d_0} - \frac{1}{4}\ for\ inner\ bolts$$

For end bolts

$$\alpha_b = \min\left(\frac{55}{3 \times 22}, \frac{800}{470}, 1.0\right) = \min(0.83, 1.70, 1.0) = 0.83$$

For inner bolts

$$\alpha_b = \min\left(\frac{85}{3 \times 22} - \frac{1}{4}, \frac{800}{470}, 1.0\right) = \min(1.04, 1.70, 1.0) = 1.0$$

Therefore $\alpha_b = 0.83$

$$k_1 = \min\left(\frac{2.8e_2}{d_0} - 1.7, 2.5\right) = \min\left(2.8 \times \frac{50}{22} - 1.7, 2.5\right) = \min(4.66, 2.5) = 2.5$$

For the end bolts

$$F_{v,Rd} = \frac{0.1 \times 0.83 \times 470 \times 20 \times 10}{1.25} \times 10^{-3} = 156.67\ kN$$

For the inner bolts

$$F_{v,Rd} = \frac{0.1 \times 1 \times 470 \times 20 \times 10}{1.25} \times 10^{-3} = 213.64\ kN$$

Number of end bolts is 2 and number of inner bolts is 4.

The bearing resistance of the bolts:

$$V_{Rd,2} = 2 \times 156.67 + 4 \times 213.64 = 1167.88\ kN$$

BS EN 1993-1-8:2005 3.7

Group of Fasteners

Shear resistance is $F_{v,Rd} = 75.26\ kN$ and bearing resistance is $F_{b,Rd} = 156.67\ kN$, $75.26\ kN < 156.67\ kN$ Therefore, the smallest design resistance is $75.26\ kN$.

Resistance of the group is $6 \times 75.26 = 451.58\ kN$

BS EN 1993-1-1 6.2.6(2)

Beam web in shear

Only web of the beam is checked:

Access Steel document SN014a-EN-EU

$$V_{pl,Rd} = V_{Rd,8} = \frac{A_v\left(\frac{f_y}{\sqrt{3}}\right)}{\gamma_{M0}}$$

A factor of 0.9 is introduced:

$$V_{Rd,8} = 0.9 \times \frac{(330 \times 12.7) \times \left(\frac{355}{\sqrt{3}}\right)}{1} \times 10^{-3} = 773.09\ kN$$

$> V_{Ed} 345.86\ kN\ OK$

9.1.2. Tying resistance of end plate

Table 9.3 Tying resistance to be checked

Mode of failure	
Bolts in tension	$N_{Rd,u,1}$
End plate in bearing*	$N_{Rd,u,2}$
Supporting member (column) in bearing	$N_{Rd,u,3}$
Beam web in tension	$N_{Rd,u,4}$

End plate in bearing is the only resistance to be checked in the design.

Bolts in tension

This is checked because end plate in bending needs tension capacity of the bolt group.

BS EN 1993-1-8 3.6.1 & Table 3.4

$$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{Mu}}$$

Where $k_2 = 0.9$, so $N_{Rd,u,1} = F_{t,Rd} = \frac{0.9 \times 800 \times 245}{1.1} \times 10^{-3} = 160.36\ kN$

For 6 bolts, $N_{Rd,u,1} = 6 \times 160.36 = 962.18\ kN$

End plate in bending

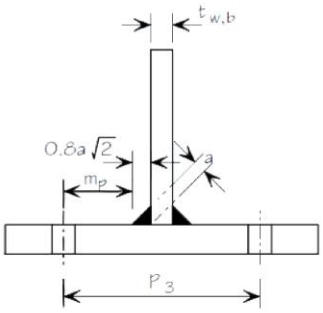


Figure 9.4 Plan view of details for end plate. Retrieved from "Steel Building Design: Worked examples for students in accordance with Eurocodes and the UK National Annexes" by The Steel Construction Institute (p. 87), 2009.

BS EN 1993-1-8 6.2.4 & Table 6.2

$$N_{Rd,u,2} = \min\left(F_{Rd,u,ep1}, F_{Rd,u,ep2}\right)$$

For mode 1: $F_{Rd,u,ep1} = F_{T,1,Rd} = \frac{(8n_p - 2e_w)M_{pl,1,Rd}}{2m_p n_p - e_w(m_p + n_p)}$

For mode 2: $F_{Rd,u,ep2} = F_{T,2,Rd} = \frac{(2M_{pl,2,Rd} + n_p \sum F_{t,Rd})}{m_p + n_p}$

Where $n_p = \min\left(e_2, e_{2,c}, 1.25m_p\right)$

$$m_p = \frac{p_3 - t_{w,b} - 2 \times 0.8a\sqrt{2}}{2} = \frac{100 - 12.7 - 2 \times 0.8 \times 5 \times \sqrt{2}}{2} = 37.99\ mm$$

$$e_w = \frac{d_w}{4} = \frac{37}{4} = 9.25\ mm$$

$$1.25m_p = 1.25 \times 37.99 = 47.49\ mm$$

$e_{c,2} = \frac{374.7-100}{2} = 137.35\ mm$ which is the least of all columns.

$$n_p = \min(50, 137.35, 47.49) = 47.49\ mm$$

$$M_{pl,1,Rd} = \frac{1}{4}\frac{\sum \ell_{eff,1} t_p^2 f_{y,p}}{\gamma_{M0}}\ (Mode\ 1)$$

$$M_{pl,2,Rd} = \frac{1}{4}\frac{\sum \ell_{eff,2} t_p^2 f_{y,p}}{\gamma_{M0}}\ (Mode\ 2)$$

Where ℓ_{eff} is equal to the length of the plate for simplicity: $\sum \ell_{eff,1} = \sum \ell_{eff,2} = h_p = 330\ mm$

$$M_{pl,1,Rd,u} = \frac{1}{4}\frac{\left(h_p t_p^2 f_{u,p}\right)}{\gamma_{Mu}} = \frac{1}{4} \times \frac{330 \times 10^2 \times 470}{1.1} \times 10^{-6} = 3.53\ kNm$$

BS EN 1993-1-8 6.2.6.5 & Table 6.6

Mode 1: $F_{T,1,Rd} = \frac{(8\times50-2\times9.25)\times3.53\times10^3}{(2\times37.99\times47.49)-9.25\times(37.99+47.49)} = 452.12\ kN$

Mode 2: $F_{T,1,Rd} = \frac{2\times3.53\times10^3+47.49\times962.18}{37.99+47.49} = 617.02\ kN$

$$F_{T,1,Rd} = \min(452.12, 617.02) = 452.12\ kN$$

Where $M_{pl,2,Rd,u} = M_{pl,1,Rd,u}$

Therefore, $N_{Rd,u,2} = 452.12\ kN$

Summary of the results

There are total 3 joint shear resistance and two tying resistance of end plate. Thickness of column flange is 21.3 mm which is larger than the thickness of end plate 10 mm, so bending doesn't need to be checked.

Table 9.4 Joint shear resistance

Mode of failure		
Bolts in shear*	$V_{Rd,1}$	$451.58\ kN$
End plate in bearing*	$V_{Rd,2}$	$1167.88\ kN$
Beam web in shear	$V_{Rd,8}$	$773.09\ kN$

The critical shear resistance is 451.58 kN.

Table 9.5 Tying resistance of end plate

Mode of failure		
Bolts in shear*	$N_{Rd,u,1}$	$962.18\ kN$
End plate in bearing*	$N_{Rd,u,2}$	$452.12\ kN$

The critical tying resistance is 452.12 kN.

10. Column Connections Design

In this section, the design of column connections will be summarized. The connections can basically be divided into two types, the column base connections at the ground floor and the column-column connections. Considerations and overall procedures for the design of each type of connections will be explained, followed by the relevant detailed calculations.

10.1. Column Base Connections Design

Column base connection connecting the columns to ground concrete foundation is discussed in this section.

10.1.1. Typical Configuration of Connections

The connection consists of base plate, welds and holding down bolts which connect the column to the concrete foundation. A plan view and cross section view of a typical column base connection are illustrated in Figure 10.1 respectively.

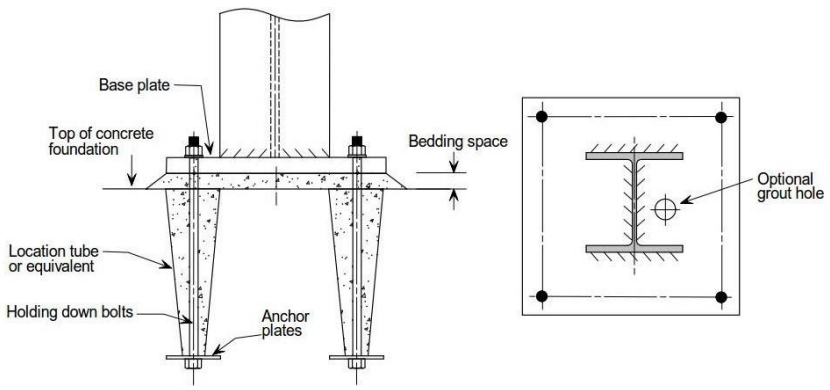


Figure 10.1: Figure showing the plan view of a typical column base connection. Retrieved from "Joints in Steel Construction – Simple Joints to Eurocode 3" by The Steel Construction Institute (p. 237), 2014.

10.1.2. Actions considered during Design

Axial and shear forces were obtained before the design of the connections. The axial forces were obtained from the design of columns. Similar to the design of columns, only load combination 1 ($1.35G_k + 1.5Q_k$ + EHF) was considered for the maximum vertical forces. On the other hand, shear forces were assumed to be the lateral force to be resisted by the connections due to the lateral wind actions and the associated EHF. To be conservative, the forces were calculated based on load combination 3 to give the maximum lateral forces ($1.35G_k + 1.05Q_k + 1.5W_k$ +

EHF).

10.1.3. Design Checks

As the columns connecting to the foundation have four different sizes, different sets of calculations were prepared in accordance with the different sizes and actions.

Figures 10.2 and 10.3 show the elevation and cross-section of the column base joints considered in this project.

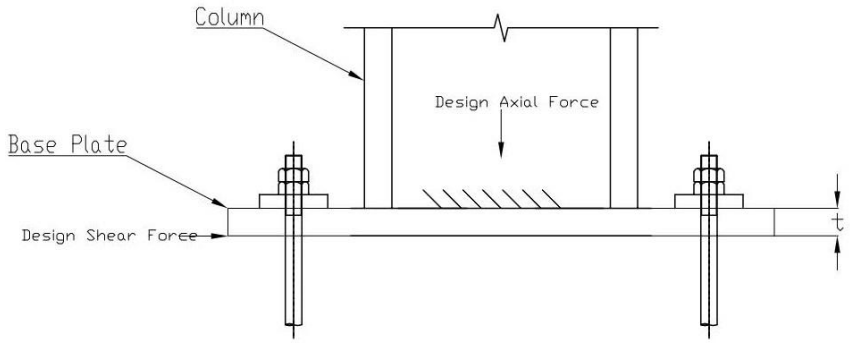


Figure 10.2: Figure showing the elevation of the column base joints considered in this project.

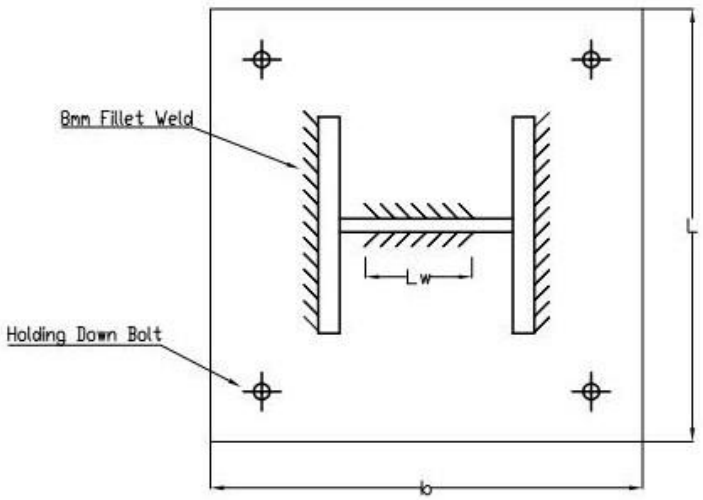


Figure 10.3: Figure showing the cross-section of the column base joints considered in this project.

For each column size, four design checks were conducted accordance with BSEN1993-1-8 Clause 6.2.5 and with reference to SCI Publication P358. The checks are explained in detail:

Check 1: Required Area Check

The aim of this check is to ensure provided base plate area (A_p) is larger than the required area (A_{req}) to withstand the design axial load (N_{Ed}). The equations used are given below:

$$A_p = h_p \times b_p \geq A_{req} = \frac{N_{Ed}}{f_{jd}}$$

$$f_{jd} = \beta_j \alpha f_{cd} = \frac{2}{3}(1.5) f_{cd} = f_{cd} = \frac{0.85 f_{ck}}{1.5}$$

, where h_p and b_p denote length and width of the base plate respectively. The values of coefficients β_j and α were taken as 2/3 and 1.5 respectively as per SCI Publication P358 (Steel Construction Institute, 2014).

Check 2: Effective Area check

In this check, the check is to ensure the effective area (A_{eff}) of base plate needed (A_{req}) is larger than the area required in Check 1. During the calculations, the design axial load is assumed to be spread uniformly across an effective area, which is around the perimeter of the column as stated in Figure 10.4.

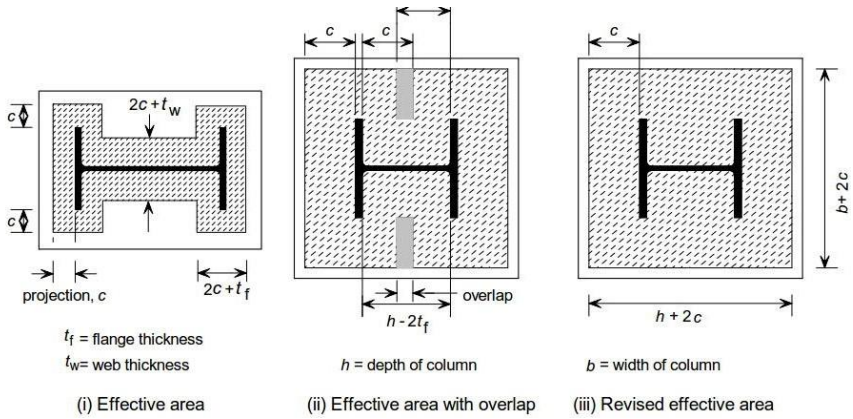


Figure 10.4: Figure showing the effective area around a column for aa column base connection. Retrieved from "Joints in Steel Construction – Simple Joints to Eurocode 3" by The Steel Construction Institute (p. 239), 2014.

Figure 10.4 also shows a scenario in which there is no overlap for the projected areas around the two flanges, that is $c \leq (h - 2t_f)/2$. The effective area can be given as follows:

$$A_{eff} = 4c^2 + P_{col}c + A_{col}$$

, where c, P_{col} and $A_{col}$are the projection, perimeter and area of the effective area respectively.

If there is an overlap, that is $c \geq (h - 2t_f)/2$, the equations used are as follows:

$$A_{eff} = 4c^2 + 2(h + b)c + hb$$

, where h and b are the height and width of the column cross section respectively.

Check 3: Plate Thickness

In accordance with BSEN1993-1-8 Clause 6.2.5(4), the plate thickness of the base plate is checked to ensure the provided thickness (t_p) is larger than the minimum requirement ($t_{p,min}$). The following equation is used:

$$t_{p,min} = c\sqrt{\frac{3f_{jd}\gamma_{M0}}{f_{yp}}}$$

, where $f_{yp}$is the yield strength of base plate.

Check 4: Capacity of Weld

In this check, the capacity of weld is verified. Mean stress method is adopted to be conservative for the calculation. The weld strength is assumed to be the shear strength of weld as weld is poor in pure shear. The following equation is used for the check:

$$V_{Ed} \leq F_{w,Rd}l_{w,eff} = (f_{vw,d}a)l_{w,eff} = \left(\frac{f_u/\sqrt{3}}{\beta_w\gamma_{M2}}a\right)l_{w,eff}$$

, where f_u , a and $l_{w,eff}$ are the ultimate strength of steel, throat thickness, effective length of weld in shear respectively.

10.1.4. Summary of Results

The results for the design of five different types of column base connections are listed in the table below. Note that the axial load from Proposal 2 was adopted for the design of base joints for columns at bracing bay (356 x 406 x 1202 UC) as it was more critical.

Table 10.1: Table summarizing the results of column base connections

For bottom internal columns (356 x 406 x 467 UC)	
Design Axial Force N_{Ed}	12491kN
Design Shear Force V_{Ed}	101kN
Base Plate Dimensions (Lxbxt)	1000mm x 1000mm x 120mm
Grade of Base Plate	S355
Weld Length along Shear L_w	100mm
Concrete Foundation Grade	C25/30

For bottom peripheral columns (356 x 406 x 235 UC)	
Design Axial Force N_{Ed}	6546kN
Design Shear Force V_{Ed}	101kN
Base Plate Dimensions (Lxbxt)	800mm x 800mm x 80mm
Grade of Base Plate	S355
Weld Length along Shear L_w	100mm
Concrete Foundation Grade	C25/30

For bottom columns at cantilever region (356 x 406 x 634 UC)	
Design Axial Force N_{Ed}	19022kN
Design Shear Force V_{Ed}	101kN
Base Plate Dimensions (Lxbxt)	1250mm x 1250mm x 150mm
Grade of Base Plate	S355
Weld Length along Shear L_w	100mm
Concrete Foundation Grade	C25/30

For bottom bracing columns – Proposal 1 (356 x 406 x 1202 UC)	
Design Axial Force N_{Ed}	21721kN
Design Shear Force V_{Ed}	101kN
Base Plate Dimensions (Lxbxt)	1250mm x 1250mm x 150mm
Grade of Base Plate	S355
Weld Length along Shear L_w	100mm
Concrete Foundation Grade	C25/30

10.1.5. Detailed Calculations

The detailed calculations for the four types of column base connections are shown:

Column Base Connection Calculations (Bottom Internal Columns)

Section	356 x 406 x 467	UC	

Section Properties

Grade	355		
h	436.6	mm	
b	412.2	mm	
t_w	35.8	mm	
t_f	58	mm	
r	15.2	mm	
Perimeter	2450.4	mm	
A	595	cm^2	
E	210	GPa	

Base Design using Resistance Table G.32 of SCI P358

N_{Ed}	12491	kN	
For a 1000 x 1000 x 120 baseplate with C25/30 concrete,			
Resistance obtained from table=	13500	kN	**OK**

Check 1: Required Area

According to BSEN1992-1-1 Table 3.1 and NA2,

Area of Base Plate $A_p = h_p \times b_p =$	1000000	mm^2	
$f_{cd}=\alpha_{cc}f_{ck}/\gamma_c =$	14.1667	MPa	
$f_{jd}=\alpha\beta_j f_{cd} =$	14.1667	MPa	
, where α=1.5, β_j=2/3			
Area Required =	881733.8	mm^2	**OK**

As it is larger than area of base plate, area required is sufficient

Check 2: Effective Area

Basic Requirement: $A_{eff}=A_{req}$

Effective area = $4c^2$+ Section Perimeter x c + Section Area

, where c is the cantilever outstand of effective area

c =	395.4	mm
$(h-2t_f)/2$ =	160.3	mm

There is overlap between flanges

Check effective area on base plate:

h+2c=	1227.5	mm	
b+2c=	1203.1	mm	**FURTHER CHECK**

As the effective area falls outside the base plate, assumption is not valid.

Calculation of Revised A_{eff} based on A_{eff}=(h+2c)(b+2c)

c=	257.3	mm

Re-check effective area on base plate:

h+2c=	951.3	mm	
b+2c=	926.9	mm	**OK**

As the effective area falls outside the base plate, assumption is not valid.

Check 3: Plate Thickness

According to BSEN1993-1-8 Clause 6.2.5(4),

Yield strength of plate with thickness	120	mm	
f_{yp}	335	MPa	
$t_{p,min}=c(3f_{jd}\gamma_{M0}/f_{yp})^{0.5}$	91.7	mm	**OK**

The plate thickness is larger than minimum value.

Check 4: Welds

Basic Requirement: $V_{Ed}<=F_{w,Rd}l_{w,eff}$

$F_{w,Rd}$ denotes the fillet weld resistance per unit length

$l_{w,eff}$ denotes the effective length of weld

According to Table 3.1 of BSEN1993-1-1,

For 120mm thick plate, f_u=	470	MPa
β_w=	0.9	
γ_{M2}=	1.25	(National Annex)
Propose weld leg length s =	8	mm
Throat thickness a = 0.7s =	5.6	mm
Using Mean-Stress Method,		
$F_{w,Rd}=F_{vw,d}a=af_u/3^{0.5}/\beta_w/\gamma_{M2}$=	1350.7	N/mm
Propose weld leg length along shear	100	mm
$l_{w,eff}=2(l-2s)$=	168	mm
$F_{w,Rd}l_{w,eff}$=	226.9	kN

Assume V_{Ed} is calculated from the lateral reaction at each column due to $1.5W_k$+EHF

V_{Ed}=	101.0	kN	**OK**

Therefore, $V_{Ed}<=F_{w,Rd}l_{w,eff}$

Column Base Connection Calculations (Bottom Peripheral Columns)

Section	356 x 406 x 235	UC

Section Properties

Grade	355	
h	381	mm
b	394.8	mm
t_w	18.4	mm
t_f	30.2	mm
r	15.2	mm

Perimeter	2304.4	mm	
A	299	cm^2	
E	210	GPa	

Base Design using Resistance Table G.32 of SCI P358

N_{Ed}	6546	kN	
For a 800 x 800 x 80 baseplate with C25/30 concrete,			
Resistance obtained from table=	8440	kN	**OK**

Check 1: Required Area

According to BSEN1992-1-1 Table 3.1 and NA2,

Area of Base Plate $A_p = h_p \times b_p =$	640000	mm^2	
$f_{cd}=\alpha_{cc}f_{ck}/\gamma_c =$	14.1667	MPa	
$f_{jd}=\alpha\beta_j f_{cd} =$	14.1667	MPa	

, where α=1.5, β_j=2/3

Area Required =	462038.5	mm^2	**OK**

As it is larger than area of base plate, area required is sufficient

Check 2: Effective Area

Basic Requirement: $A_{eff}=A_{req}$

Effective area = $4c^2$+ Section Perimeter x c + Section Area

, where c is the cantilever outstand of effective area

c =	149.0	mm
$(h-2t_f)/2$ =	160.3	mm

There is no overlap between flanges

Check effective area on base plate:

h+2c=	679.0	mm	
b+2c=	692.8	mm	**OK**

As the effective area falls within the base plate, assumption is valid.

Check 3: Plate Thickness

According to BSEN1993-1-8 Clause 6.2.5(4),

Yield strength of plate with thickness	80	mm	
f_{yp}	335	MPa	
$t_{p,min}=c(3f_{jd}\gamma_{M0}/f_{yp})^{0.5}$	53.1	mm	**OK**

The plate thickness is larger than minimum value.

Check 4: Welds

Basic Requirement: $V_{Ed}<=F_{w,Rd}l_{w,eff}$

$F_{w,Rd}$ denotes the fillet weld resistance per unit length

$l_{w,eff}$ denotes the effective length of weld

According to Table 3.1 of BSEN1993-1-1,

For 50mm thick plate, f_u=	470	MPa	
β_w=	0.9		
γ_{M2}=	1.25	(National Annex)	
Propose weld leg length s =	8	mm	
Throat thickness a = 0.7s =	5.6	mm	
Using Mean-Stress Method,			
$F_{w,Rd}=F_{vw,d}a=af_u/3^{0.5}/\beta_w/\gamma_{M2}$=	1350.7	N/mm	
Propose weld leg length along shear	100	mm	
$l_{w,eff}$=2(l-2s)=	168	mm	
$F_{w,Rd}l_{w,eff}$=	226.9	kN	
Assume V_{Ed} is calculated from the lateral reaction at each column due to 1.5W_k+EHF			
V_{Ed}=	101.0	kN	**OK**
Therefore, V_{Ed}<=$F_{w,Rd}l_{w,eff}$			

Column Base Connection Calculations (Columns at Cantilever Region)

Section	356 x 406 x 634	UC	

Section Properties

Grade	355		
h	474.6	mm	
b	424	mm	
t_w	47.6	mm	
t_f	77	mm	
r	15.2	mm	
Perimeter	2550	mm	
A	808	cm^2	
E	210	GPa	

Base Design using Resistance Table G.32 of SCI P358

N_{Ed}	19022	kN	
For a 1250 x 1250 x 150 baseplate with C25/30 concrete,			
Resistance obtained from table=	17600	kN	**FURTHER CHECK**

Check 1: Required Area

According to BSEN1992-1-1 Table 3.1 and NA2,

Area of Base Plate $A_p = h_p$ x b_p =	1562500	mm^2	
$f_{cd}=\alpha_{cc}f_{ck}/\gamma_c$ =	14.1667	MPa	
$f_{jd}=\alpha\beta_j f_{cd}$ =	14.1667	MPa	
, where α=1.5, β_j=2/3			
Area Required =	1342729	mm^2	**OK**

As it is larger than area of base plate, area required is sufficient

Check 2: Effective Area

Basic Requirement: $A_{eff}=A_{req}$

Effective area = $4c^2$+ Section Perimeter x c + Section Area

, where c is the cantilever outstand of effective area

c =	327.1	mm
$(h-2t_f)/2$ =	160.3	mm

There is overlap between flanges

Check effective area on base plate:

h+2c=	1128.7	mm	
b+2c=	1078.1	mm	**OK**

As the effective area falls within the base plate, assumption is valid.

Check 3: Plate Thickness

According to BSEN1993-1-8 Clause 6.2.5(4),

Yield strength of plate with thickness	150	mm	
f_{yp}	335	MPa	
$t_{p,min}=c(3f_{jd}\gamma_{M0}/f_{yp})^{0.5}$	116.5	mm	**OK**

The plate thickness is larger than minimum value.

Check 4: Welds

Basic Requirement: $V_{Ed}<=F_{w,Rd}l_{w,eff}$

$F_{w,Rd}$ denotes the fillet weld resistance per unit length

$l_{w,eff}$ denotes the effective length of weld

According to Table 3.1 of BSEN1993-1-1,

For 110mm thick plate, f_u=	470	MPa
β_w=	0.9	
γ_{M2}=	1.25	(National Annex)
Propose weld leg length s =	8	mm
Throat thickness a = 0.7s =	5.6	mm
Using Mean-Stress Method,		
$F_{w,Rd}=F_{vw,d}a=af_u/3^{0.5}/\beta_w/\gamma_{M2}$=	1350.7	N/mm
Propose weld leg length along shear	100	mm
$l_{w,eff}=2(l-2s)$=	168	mm
$F_{w,Rd}l_{w,eff}$=	226.9	kN

Assume V_{Ed} is calculated from the lateral reaction at each column due to $1.5W_k$+EHF

V_{Ed}=	101.0	kN	**OK**

Therefore, $V_{Ed}<=F_{w,Rd}l_{w,eff}$

Column Base Connection Calculations (Bracing System)

Section	356 x 406 x 1202	UC	

Section Properties

Grade	355		
h	580	mm	
b	471	mm	
t_w	95	mm	
t_f	130	mm	
r	15.4	mm	
Perimeter	2854	mm	
A	1531	cm^2	
E	210	GPa	

Base Design using Resistance Table G.32 of SCI P358

N_{Ed}	21721	kN	
For a 1250 x 1250 x 150 baseplate with C25/30 concrete,			
Resistance obtained from table=	17600	kN	**FURTHER CHECK**

Check 1: Required Area

According to BSEN1992-1-1 Table 3.1 and NA2,

Area of Base Plate $A_p = h_p \times b_p =$	1562500	mm^2	
$f_{cd}=\alpha_{cc}f_{ck}/\gamma_c =$	14.1667	MPa	
$f_{jd}=\alpha\beta_j f_{cd} =$	14.1667	MPa	

, where α=1.5, β_j=2/3 from BSEN1993-1-8 Clause 6.2.5(7)

Area Required =	1533247	mm^2	**OK**

As it is larger than area of base plate, area required is sufficient

Check 2: Effective Area

Basic Requirement: $A_{eff}=A_{req}$

Effective area = $4c^2$+ Section Perimeter x c + Section Area

, where c is the cantilever outstand of effective area

c =	330.5	mm	
$(h-2t_f)/2$ =	160	mm	

There is overlap between flanges

Check effective area on base plate:

h+2c=	1241.0	mm	
b+2c=	1132.0	mm	**OK**

As the effective area falls within the base plate, assumption is valid.

Check 3: Plate Thickness

According to BSEN1993-1-8 Clause 6.2.5(4),

Yield strength of plate with thickness	120	mm	
f_{yp}	335	MPa	
$t_{p,min}=c(3f_{jd}\gamma_{M0}/f_{yp})^{0.5}$	117.7	mm	**OK**

The plate thickness is larger than minimum value.

Check 4: Welds

Basic Requirement: $V_{Ed}<=F_{w,Rd}l_{w,eff}$

$F_{w,Rd}$ denotes the fillet weld resistance per unit length

$l_{w,eff}$ denotes the effective length of weld

According to Table 3.1 of BSEN1993-1-1,

For 110mm thick plate, f_u=	470	MPa	
β_w=	0.9		
γ_{M2}=	1.25	(National Annex)	
Propose weld leg length s =	8	mm	
Throat thickness a = 0.7s =	5.6	mm	
Using Mean-Stress Method,			
$F_{w,Rd}=F_{vw,d}a=af_u/3^{0.5}/\beta_w/\gamma_{M2}$=	1350.7	N/mm	
Propose weld leg length along shear	100	mm	
$l_{w,eff}=2(l-2s)$=	168	mm	
$F_{w,Rd}l_{w,eff}$=	226.9	kN	

Assume V_{Ed} is calculated from the lateral reaction at each column due to $1.5W_k$+EHF

V_{Ed}=	101.0	kN	**OK**

Therefore, $V_{Ed}<=F_{w,Rd}l_{w,eff}$

10.2. Column-Column Connections

In this section, the typical configuration of a column-column connection will be described, followed by the overall procedures of relevant design checks. Detailed design calculations will also be provided at the end.

10.2.1. Typical Configuration of Connections

The connection configuration generally follows a bearing type of column splice arrangement. The typical configuration of the connection can be shown clearly in Figure 10.5.

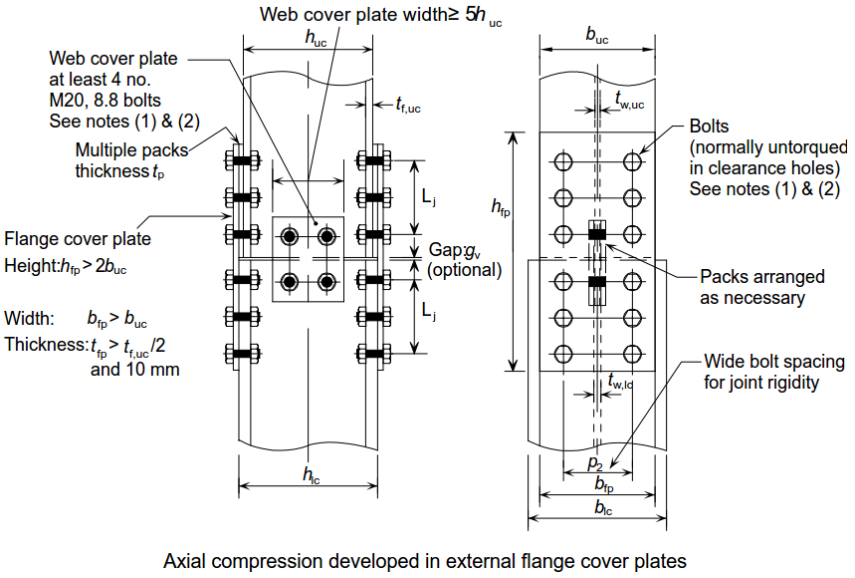


Figure 10.5: Figure showing the elevation of the connection across the width and height of the cross section (h_{uc}, h_{lc}, b_{uc} and b_{lc} refer to the cross-section height and width of upper column and lower column respectively). Retrieved from "Joints in Steel Construction – Simple Joints to Eurocode 3" by The Steel Construction Institute (p. 185), 2014.

In the bearing type of column splice arrangement, the load is assumed to transfer from upper column to lower column directly or through the division plate in the middle (Steel Construction Institute, 2021). The main benefit of adopting this arrangement is that less bolts are required in the connection, which results in a more economical and sustainable design (Steel Construction Institute, 2021).

10.2.2. Actions considered during Design

Similar to the design of columns and the column base connections shown above, the actions used for the column-column connections design were also obtained from load case 1 (i.e. $1.35G_k + 1.5Q_k$ + EHF) for the maximum vertical force.

10.2.3. Overall Design Procedures

The overall design procedures can be divided into two parts, which are the detailing of splice arrangement and the resistance check.

<u>Check 1: Detailing of Splice Arrangement</u>

According to SCI Publication P358, the following detailing arrangement is recommended.

For the external flange cover plate, its height h_{fp} and width b_{fp} should be larger

than $2b_{uc} + t_{dp}$ and b_{uc} respectively, where b_{uc} and t_{dp} denote the width of cross section of upper column and thickness of division plate respectively. The thickness of the flange plate should be given as the maximum of half of the upper column flange thickness $t_{f,uc}$, 10mm and $p_1/14$, where p_1 is the vertical spacing between bolts.

For the flange pack, its thickness t_{pa} is given by the following equation:

$$t_p = \frac{h_{lc} - h_{uc}}{2}$$

, where h_{lc} and h_{uc} are the cross-section height of lower column and upper column respectively.

For the division plate, the thickness t_{dp} is given by the following equation:

$$t_{dp} \geq \frac{[(h_{lc} - 2t_{f,lc}) - (h_{uc} - 2t_{f,uc})]}{2}$$

, where h_{lc} ,$t_{f,lc}$, h_{uc} and $t_{f,uc}$ refer to the lower column cross-section height and flange thickness, upper column cross-section height and flange thickness respectively.

For the web cleat, its length should be larger than half of the upper column cross-section height h_{uc}.

For the web pack, its thickness t_p is given by the following equation:

$$t_p = \frac{t_{w,lc} - t_{w,uc}}{2}$$

, where $t_{w,lc}$ and $t_{w,uc}$ are the thickness of web for lower column and upper column respectively.

Check 2: Resistance Check

For the cover plate to be able to resist the design load N_{Ed}, its design resistance N_{Rd} should exceed a quarter of the design load N_{Ed}, which is shown in the equation below:

$$N_{Rd} = \frac{2A_{fp}f_{y,fp}}{\gamma_{M0}} = 2A_{fp}f_{y,fp} \geq 0.25N_{Ed}$$

, where A_{fp} and $f_{y,fp}$ are the area and yield strength of the flange plate respectively.

For the bolt group, both shear resistance $F_{v,Rd}$ and bearing resistance $F_{b,Rd}$ were checked and the bolt group resistance $F_{Rd,fp}$was taken as the smaller one of the two to be more conservative. The shear and bearing resistance of bolt group are given by the following set of equations:

$$F_{b,Rd} = \frac{\beta_p \alpha_v f_{ub} A}{\gamma_{M2}} = \frac{\beta_p \alpha_v f_{ub} A}{1.25}$$

$$F_{b,Rd} = \frac{k_1 \alpha_d f_u d t_p}{\gamma_{M2}} = \frac{k_1 \alpha_d f_u d t_p}{1.25}$$

, where β_p and α_v are constants. f_{ub} and A are the ultimate strength and stressed tensile area of a bolt, k_1 and α_d are constants related to the bolt spacing as stated in Table 3.3 of BSEN1993-1-8. f_u is the ultimate strength of splice, d denotes the bolt diameter and t_{fp} refers to the flange plate thickness.

The bearing at column flange was not checked as the thickness of column flange is thicker, so that the effect should not be critical.

10.2.4. Summary of Results

As there are multiple combinations of column size and actions across different locations of the buildings, a total of nine different types of column-column connections were identified. Results can generally be divided into two groups based on the number of bolts. They are summarized in the following figures and tables.

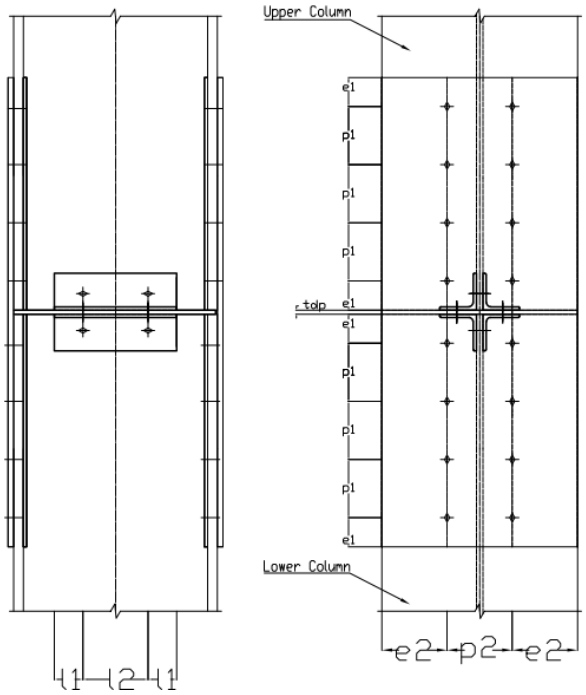


Figure 10.6: Figure showing the results of column-column connections (8 bolts per column)

Table 10.2: Table showing the results of column-column connections (8 bolts per column)

Type 1	
Upper Column Size	356 x 406 x 467 UC
Lower Column Size	356 x 406 x 467 UC
Flange Cover Plate	2/420 x 40 x 1000
Cleat	4/90 x 90 x 8 Angles x 220 Long
Division Plate	420 x 20 x 420
Bolts	M22 Grade 8.8
Type 2	
Upper Column Size	356 x 406 x 634 UC
Lower Column Size	356 x 406 x 634 UC
Flange Cover Plate	2/430 x 40 x 1150
Cleat	4/90 x 90 x 8 Angles x 250 Long
Division Plate	430 x 10 x 430
Bolts	M27 Grade 8.8
Type 3	
Upper Column Size	356 x 406 x 235 UC
Lower Column Size	356 x 406 x 634 UC

Flange Cover Plate	2/400 x 40 x 1220
Cleat	2/400 x 50 x 240
Division Plate	400 x 10 x 400
Bolts	M30 Grade 8.8

Type 4

Upper Column Size	356 x 406 x 1202 UC
Lower Column Size	356 x 406 x 1202 UC
Flange Cover Plate	2/480 x 40 x 1150
Cleat	4/90 x 90 x 8 Angles x 300 Long
Division Plate	480 x 10 x 480
Bolts	M27 Grade 8.8

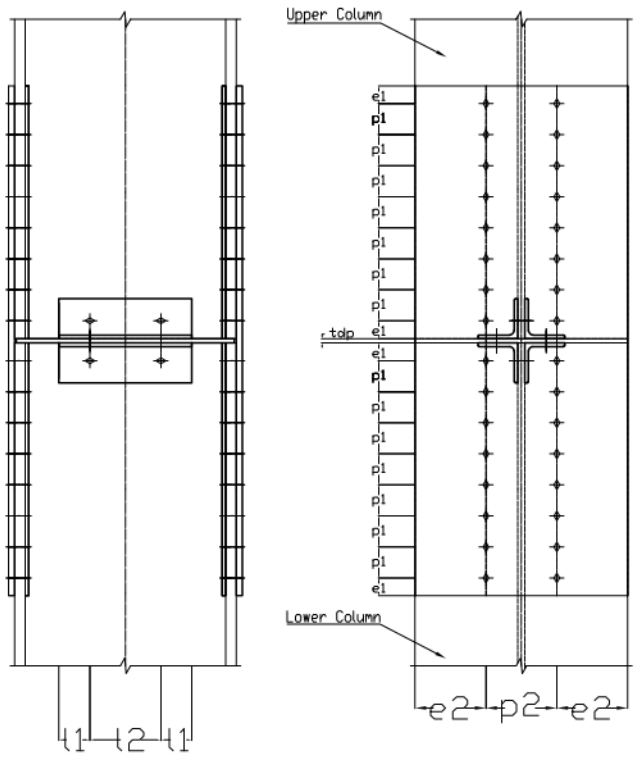


Figure 10.7: Figure showing the results of column-column connections (16 bolts per column)

Table 10.3: Table showing the results of column-column connections (16 bolts per column)

Type 5

Upper Column Size	356 x 368 x 202 UC
Lower Column Size	356 x 406 x 467 UC
Flange Cover Plate	2/380 x 40 x 820
Cleat	4/90 x 90 x 8 Angles x 200 Long

Division Plate	380 x 20 x 380
Bolts	M27 Grade 8.8

Type 6

Upper Column Size	356 x 368 x 202 UC
Lower Column Size	356 x 368 x 202 UC
Flange Cover Plate	2/380 x 40 x 800
Cleat	4/90 x 90 x 8 Angles x 200 Long
Division Plate	380 x 20 x 380
Bolts	M20 Grade 8.8

Type 7

Upper Column Size	356 x 406 x 235 UC
Lower Column Size	356 x 406 x 235 UC
Flange Cover Plate	2/400 x 20 x 800
Cleat	4/90 x 90 x 8 Angles x 200 Long
Division Plate	400 x 20 x 400
Bolts	M22 Grade 8.8

Type 8

Upper Column Size	356 x 368 x 202 UC
Lower Column Size	356 x 406 x 235 UC
Flange Cover Plate	2/380 x 40 x 800
Cleat	4/90 x 90 x 8 Angles x 200 Long
Division Plate	380 x 20 x 380
Bolts	M20 Grade 8.8

Type 9

Upper Column Size	356 x 368 x 202 UC
Lower Column Size	356 x 368 x 202 UC
Flange Cover Plate	2/380 x 40 x 800
Cleat	4/90 x 90 x 8 Angles x 200 Long
Division Plate	380 x 20 x 380
Grade of Materials	M20 Grade 8.8

10.2.5. Detailed Calculations

Detailed calculations for each type of column-column connections are listed below:

Column to Column Connection Calculations (Internal Columns 3rd Floor to 1st Floor)

Column Section Above	356 x 406 x 467	UC
Column Section Below	356 x 406 x 467	UC

Section Properties (Upper Column)

Grade	355	
h_{uc}	436.6	mm
b_{uc}	412.2	mm
$t_{w,uc}$	35.8	mm
$t_{f,uc}$	58	mm
r_{uc}	15.2	mm
A_{uc}	595	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	436.6	mm
b_{lc}	412.2	mm
$t_{w,lc}$	35.8	mm
$t_{f,lc}$	58	mm
r_{lc}	15.2	mm
A_{lc}	595	cm^2

Splice Details

Flange Cover Plate	2/420 x 40 x 1000
Cleats	4/90 x 90 x 8 Angles x 220 Long
Division Plate	420 x 20 x 420
Bolts	M22 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	7743	kN
N_{Ed} =	12491	kN

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height $h_{fp} >= 2b_{uc} + t_{dp}$=	844.4	mm
Width $b_{fp} >= b_{uc}$	412.2	mm

Vertical Bolt Spacing p_1=	60	mm	
$p_1/14$=	4.3	mm	
$t_{f,uc}$=	58	mm	
Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)			
t_{fp}>=	29	mm	
Therefore, propose h_{fp} =	1000	mm	
Therefore, propose b_{fp} =	420	mm	
Therefore, propose t_{fp} =	40	mm	
Division Plate			
tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm	
Therefore, propose t_{dp} =	20	mm	
Web Cleats			
Use 90 x 90 x 8 angles to accommodate M22 bolts.			
Length >= 0.5h_{uc}=	218.3	mm	
Therefore, propose length of	220	mm	
Check 2: Minimum Resistance			
Cover Plate			
Requirement: 0.25N_{Ed}<=N_{Rd}			
0.25N_{Ed}=	3122.8	kN	
N_{Rd}=2$A_{fp}f_{y,fp}/\gamma_{M0}$=	11928	kN	**OK**
Bolt Group			
Requirement: 0.25N_{Ed}<=2$F_{Rd,fp}$			
a) Shear Resistance per Bolt:			
Diameter of Bolts =	22	mm	
Total thickness of packing t_{pa} =	0	mm	**<d/3**
For t_{pa}>=d/3,β_p=9d/(8d+3t_{pa})=	1.125		
For t_{pa}<d/3, β_p=	1		
Therefore, β_p=	1		
α_v=	0.6	for 8.8 bolts	
Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	303	mm2	
γ_{M2}=	1.25		
$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}$=	116.4	kN	
b) Bearing Resistance per Bolt:			
Spacings are desgined based on BSEN1993-1-8 Table 3.3:			
Propose e_1=	35	mm	1000

Propose e_2=	130	mm	
Propose p_1=	60	mm	
Propose p_2=	160	mm	
Bolt Hole Diameter d_0=	24	mm	
$f_{u,p}$=	470	MPa	
$\alpha_d=\min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)$=	0.5		
$k_1=\min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)$=	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}$=	402.1	kN	
$\min(F_{v,Rd},F_{b,Rd})$=	116.4	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	16	nos.	
$F_{Rd,fp}=\min(F_{v,Rd},F_{b,Rd})(n)$ =	1861.6	kN	
As double shear is designed,			
$2F_{Rd,fp}$=	3723.3	kN	
$0.25N_{Ed}$=	3122.8	kN	**OK**

Therefore, the design satisfies the requirements.

Column to Column Connection Calculations (Internal Columns 4th Floor)

Column Section Above	356 x 368 x 202	UC
Column Section Below	356 x 406 x 467	UC

Section Properties (Upper Column)

Grade	355	
h_{uc}	374.6	mm
b_{uc}	374.7	mm
$t_{w,uc}$	16.5	mm
$t_{f,uc}$	27	mm
r_{uc}	15.2	mm
A_{uc}	257	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	436.6	mm
b_{lc}	412.2	mm
$t_{w,lc}$	35.8	mm
$t_{f,lc}$	58	mm
r_{lc}	15.2	mm
A_{lc}	595	cm^2

Splice Details

Flange Cover Plate	2/380 x 40 x 820
Flange Packs	2/380 x 40 x 240
Cleats	4/90 x 90 x 8 Angles x 200 Long
Web Packs	2/85 x 10 x 200
Division Plate	380 x 20 x 380
Bolts	M27 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	4252	kN
N_{Ed} =	6733	kN

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height h_{fp} >= $2b_{uc} + t_{dp}$=	769.4	mm
Width b_{fp} >= b_{uc}	374.7	mm
Vertical Bolt Spacing p_1=	110	mm
$p_1/14$=	7.9	mm
$t_{f,uc}$=	27	mm
Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)		
t_{fp}>=	13.5	mm
Therefore, propose h_{fp} =	820	mm
Therefore, propose b_{fp} =	380	mm
Therefore, propose t_{fp} =	40	mm

Flange Packs

$t_{pa}=(h_{lc}-h_{uc})/2$=	31	mm
Therefore, propose t_{pa} =	40	mm

Division Plate

tdp>=$[(h_{lc}-2t_{f,lc})-(h_{uc}-2t_{f,uc})]/2$=	0	mm
Therefore, propose t_{dp} =	20	mm

Web Cleats

Use 90 x 90 x 8 angles to accommodate M27 bolts.

Length >= $0.5h_{uc}$=	187.3	mm
Therefore, propose length of	200	mm

Web Packs

$t_p = (t_{w,lc}-t_{w,uc})/2$=	9.65	
Therefore, propose t_p =	10	mm

Check 2: Minimum Resistance

Cover Plate

Requirement: $0.25N_{Ed}<=N_{Rd}$

$0.25N_{Ed}=$	1683.2	kN	
$N_{Rd}=2A_{fp}f_{y,fp}/\gamma_{M0}=$	10792	kN	**OK**

Bolt Group

Requirement: $0.25N_{Ed}<=2F_{Rd,fp}$

a) Shear Resistance per Bolt:

Diameter of Bolts =	27	mm	
Total thickness of packing t_{pa} =	40	mm	**>d/3**
For $t_{pa}>=d/3, \beta_p=9d/(8d+3t_{pa})=$	0.7		
For $t_{pa}<d/3$, $\beta_p=$	1		
Therefore, $\beta_p=$	0.7		
$\alpha_v=$	0.6	for 8.8 bolts	
Bolt Ultimate Strength $f_{ub}=$	800	MPa	
Stress Area $A_s=$	459	mm2	
$\gamma_{M2}=$	1.25		
$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}=$	127.5	kN	

b) Bearing Resistance per Bolt:

Spacings are desgined based on BSEN1993-1-8 Table 3.3:

Propose $e_1=$	35	mm	820
Propose $e_2=$	110	mm	
Propose $p_1=$	110	mm	
Propose $p_2=$	160	mm	
Bolt Hole Diameter $d_0=$	30	mm	
$f_{u,p}=$	470	MPa	
$\alpha_d=\min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)=$	0.4		
$k_1=\min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)=$	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}=$	394.8	kN	
$\min(F_{v,Rd},F_{b,Rd})=$	127.5	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	8	nos.	
$F_{Rd,fp}=\min(F_{v,Rd},F_{b,Rd})(n)$ =	1019.8	kN	

As double shear is designed,

$2F_{Rd,fp}=$	2039.5	kN	
$0.25N_{Ed}=$	1683.2	kN	**OK**

Therefore, the design satisfies the requirements.

Column to Column Connection Calculations (Internal Columns 5th Floor to 8th Floor)

Column Section Above	356 x 368 x 202	UC
Column Section Below	356 x 368 x 202	UC

Section Properties (Upper Column)

Grade	355	
h_{uc}	374.6	mm
b_{uc}	374.7	mm
$t_{w,uc}$	16.5	mm
$t_{f,uc}$	27	mm
r_{uc}	15.2	mm
A_{uc}	257	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	374.6	mm
b_{lc}	374.7	mm
$t_{w,lc}$	16.5	mm
$t_{f,lc}$	27	mm
r_{lc}	15.2	mm
A_{lc}	257	cm^2

Splice Details

Flange Cover Plate	2/380 x 40 x 800
Cleats	4/90 x 90 x 8 Angles x 200 Long
Division Plate	380 x 20 x 380
Bolts	M22 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	4252	kN
N_{Ed} =	6733	kN

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height h_{fp} >= $2b_{uc} + t_{dp}$=	769.4	mm
Width b_{fp} >= b_{uc}	374.7	mm
Vertical Bolt Spacing p_1=	110	mm
$p_1/14$=	7.9	mm
$t_{f,uc}$=	27	mm

Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)

t_{fp}>=	13.5	mm	
Therefore, propose h_{fp} =	800	mm	
Therefore, propose b_{fp} =	380	mm	
Therefore, propose t_{fp} =	40	mm	
Division Plate			
tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm	
Therefore, propose t_{dp} =	20	mm	
Web Cleats			
Use 90 x 90 x 8 angles to accommodate M22 bolts.			
Length >= 0.5h_{uc}=	187.3	mm	
Therefore, propose length of	200	mm	
Check 2: Minimum Resistance			
Cover Plate			
Requirement: 0.25N_{Ed}<=N_{Rd}			
0.25N_{Ed}=	1683.2	kN	
N_{Rd}=2$A_{fp}f_{y,fp}/\gamma_{M0}$=	10792	kN	**OK**
Bolt Group			
Requirement: 0.25N_{Ed}<=2$F_{Rd,fp}$			
a) Shear Resistance per Bolt:			
Diameter of Bolts =	22	mm	
Total thickness of packing t_{pa} =	0	mm	**<d/3**
For t_{pa}>=d/3,β_p=9d/(8d+3t_{pa})=	1.125		
For t_{pa}<d/3, β_p=	1		
Therefore, β_p=	1		
α_v=	0.6	for 8.8 bolts	
Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	303	mm2	
γ_{M2}=	1.25		
$F_{v,Rd}$=$\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}$=	116.4	kN	
b) Bearing Resistance per Bolt:			
Spacings are desgined based on BSEN1993-1-8 Table 3.3:			
Propose e_1=	30	mm	800
Propose e_2=	110	mm	
Propose p_1=	110	mm	
Propose p_2=	160	mm	
Bolt Hole Diameter d_0=	24	mm	

$f_{u,p}$=	470	MPa	
α_d=min($e_1/(3d_0)$,$p_1/(3d_0)$-0.25,$f_{ub}/f_{u,p}$,1)=	0.4		
k_1=min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)=	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}$=	344.7	kN	
min($F_{v,Rd}$,$F_{b,Rd}$)=	116.4	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	8	nos.	
$F_{Rd,fp}$=min($F_{v,Rd}$,$F_{b,Rd}$)(n) =	930.8	kN	

As double shear is designed,

$2F_{Rd,fp}$=	1861.6	kN	
$0.25N_{Ed}$=	1683.2	kN	**OK**

Therefore, the design satisfies the requirements.

Column to Column Connection Calculations (Peripheral Columns 3rd Floor to 1st Floor)

Column Section Above	356 x 406 x 235	UC
Column Section Below	356 x 406 x 235	UC

Section Properties (Upper Column)

Grade	355	
h_{uc}	381	mm
b_{uc}	394.8	mm
$t_{w,uc}$	18.4	mm
$t_{f,uc}$	30.2	mm
r_{uc}	15.2	mm
A_{uc}	299	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	381	mm
b_{lc}	394.8	mm
$t_{w,lc}$	18.4	mm
$t_{f,lc}$	30.2	mm
r_{lc}	15.2	mm
A_{lc}	299	cm^2

Splice Details

Flange Cover Plate	2/400 x 20 x 800
Cleats	4/90 x 90 x 8 Angles x 200 Long
Division Plate	400 x 20 x 400
Bolts	M22 8.8

Fittings Material	S355		
Design Actions			
$N_{Ed,G}$ =	4083	kN	
N_{Ed} =	6546	kN	
Check 1: Recommended Detailing Practice			
External Flange Cover Plates			
Height h_{fp} >= $2b_{uc} + t_{dp}$=	809.6	mm	
Width b_{fp} >= b_{uc}	394.8	mm	
Vertical Bolt Spacing p_1=	110	mm	
$p_1/14$=	7.86	mm	
$t_{f,uc}$=	30.2	mm	
Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)			
t_{fp}>=	15.1	mm	
Therefore, propose h_{fp} =	800	mm	
Therefore, propose b_{fp} =	400	mm	
Therefore, propose t_{fp} =	40	mm	
Division Plate			
tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm	
Therefore, propose t_{dp} =	20	mm	
Web Cleats			
Use 90 x 90 x 8 angles to accommodate M22 bolts.			
Length >= 0.5h_{uc}=	190.5	mm	
Therefore, propose length of	200	mm	
Check 2: Minimum Resistance			
Cover Plate			
Requirement: $0.25N_{Ed}$<=N_{Rd}			
$0.25N_{Ed}$=	1636.39	kN	
N_{Rd}=$2A_{fp}f_{y,fp}/\gamma_{M0}$=	11360	kN	**OK**
Bolt Group			
Requirement: $0.25N_{Ed}$<=$2F_{Rd,fp}$			
a) Shear Resistance per Bolt:			
Diameter of Bolts =	22	mm	
Total thickness of packing t_{pa} =	0	mm	**<d/3**
For t_{pa}>=d/3,β_p=9d/(8d+3t_{pa})=	1.125		
For t_{pa}<d/3, β_p=	1		
Therefore, β_p=	1		

α_v=	0.6	for 8.8 bolts	
Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	303	mm2	
γ_{M2}=	1.25		
$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}$=	116.35	kN	

b) Bearing Resistance per Bolt:
Spacings are desgined based on BSEN1993-1-8 Table 3.3:

Propose e_1=	30	mm	800
Propose e_2=	120	mm	
Propose p_1=	110	mm	
Propose p_2=	160	mm	
Bolt Hole Diameter d_0=	24	mm	
$f_{u,p}$=	470	MPa	
$\alpha_d=\min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)$=	0.42		
$k_1=\min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)$=	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}$=	344.67	kN	
$\min(F_{v,Rd},F_{b,Rd})$=	116.35	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	8	nos.	
$F_{Rd,fp}=\min(F_{v,Rd},F_{b,Rd})(n)$ =	930.816	kN	
As double shear is designed,			
$2F_{Rd,fp}$=	1861.63	kN	
$0.25N_{Ed}$=	1636.39	kN	**OK**

Therefore, the design satisfies the requirements.

<u>Column to Column Connection Calculations (Peripheral Columns 4th Floor)</u>

Column Section Above	356 x 368 x 202	UC
Column Section Below	356 x 406 x 235	UC

<u>Section Properties (Upper Column)</u>

Grade	355	
h_{uc}	374.6	mm
b_{uc}	374.7	mm
$t_{w,uc}$	16.5	mm
$t_{f,uc}$	27	mm
r_{uc}	15.2	mm
A_{uc}	257	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	381	mm
b_{lc}	394.8	mm
$t_{w,lc}$	18.4	mm
$t_{f,lc}$	30.2	mm
r_{lc}	15.2	mm
A_{lc}	299	cm^2

Splice Details

Flange Cover Plate	2/380 x 40 x 800
Flange Packs	2/380 x 4 x 240
Cleats	4/90 x 90 x 8 Angles x 200 Long
Web Packs	2/85 x 1 x 200
Division Plate	380 x 20 x 380
Bolts	M20 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	2259	kN
N_{Ed} =	3546	kN

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height h_{fp} >= $2b_{uc} + t_{dp}$=	769.4	mm
Width b_{fp} >= b_{uc}	374.7	mm
Vertical Bolt Spacing p_1=	110	mm
$p_1/14$=	7.86	mm
$t_{f,uc}$=	27	mm
Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)		
t_{fp}>=	13.5	mm
Therefore, propose h_{fp} =	800	mm
Therefore, propose b_{fp} =	380	mm
Therefore, propose t_{fp} =	40	mm

Flange Packs

$t_{pa}=(h_{lc}-h_{uc})/2$=	3.2	mm
Therefore, propose t_{pa} =	4	mm

Division Plate

tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm
Therefore, propose t_{dp} =	20	mm

Web Cleats

Use 90 x 90 x 8 angles to accommodate M20 bolts

Length >= $0.5h_{uc}$=	187.3	mm	
Therefore, propose length of	200	mm	

Web Packs

$t_p = (t_{w,lc}-t_{w,uc})/2$=	0.95		
Therefore, propose t_p =	1	mm	

Check 2: Minimum Resistance

Cover Plate

Requirement: $0.25N_{Ed}<=N_{Rd}$

$0.25N_{Ed}$=	886.41	kN	
$N_{Rd}=2A_{fp}f_{y,fp}/\gamma_{M0}$=	10792	kN	**OK**

Bolt Group

Requirement: $0.25N_{Ed}<=2F_{Rd,fp}$

a) Shear Resistance per Bolt:

Diameter of Bolts =	20	mm	
Total thickness of packing t_{pa} =	4	mm	**<d/3**
For t_{pa}>=d/3,β_p=9d/(8d+3t_{pa})=	1.05		
For t_{pa}<d/3, β_p=	1		
Therefore, β_p=	1		
α_v=	0.6	for 8.8 bolts	
Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	245	mm2	
γ_{M2}=	1.25		
$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}$=	94.08	kN	

b) Bearing Resistance per Bolt:

Spacings are desgined based on BSEN1993-1-8 Table 3.3:

Propose e_1=	30	mm	800
Propose e_2=	110	mm	
Propose p_1=	110	mm	
Propose p_2=	160	mm	
Bolt Hole Diameter d_0=	22	mm	
$f_{u,p}$=	470	MPa	
$\alpha_d=\min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)$=	0.45		
$k_1=\min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)$=	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}$=	341.82	kN	

$\min(F_{v,Rd},F_{b,Rd})$=	94.08	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	8	nos.	
$F_{Rd,fp}=\min(F_{v,Rd},F_{b,Rd})(n)$ =	752.64	kN	

As double shear is designed,

$2F_{Rd,fp}$=	1505.28	kN	
$0.25N_{Ed}$=	886.41	kN	**OK**

Therefore, the design satisfies the requirements.

Column to Column Connection Calculations (Peripheral Columns 5th Floor to 8th Floor)

Column Section Above	356 x 368 x 202	UC
Column Section Below	356 x 368 x 202	UC

Section Properties (Upper Column)

Grade	355	
h_{uc}	374.6	mm
b_{uc}	374.7	mm
$t_{w,uc}$	16.5	mm
$t_{f,uc}$	27	mm
r_{uc}	15.2	mm
A_{uc}	257	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	374.6	mm
b_{lc}	374.7	mm
$t_{w,lc}$	16.5	mm
$t_{f,lc}$	27	mm
r_{lc}	15.2	mm
A_{lc}	257	cm^2

Splice Details

Flange Cover Plate	2/380 x 40 x 800
Cleats	4/90 x 90 x 8 Angles x 200 Long
Division Plate	380 x 20 x 380
Bolts	M20 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	2259	kN

N_{Ed} =	3546	kN	
Check 1: Recommended Detailing Practice			
External Flange Cover Plates			
Height h_{fp} >= $2b_{uc} + t_{dp}$=	769.4	mm	
Width b_{fp} >= b_{uc}	374.7	mm	
Vertical Bolt Spacing p_1=	110	mm	
$p_1/14$=	7.86	mm	
$t_{f,uc}$=	27	mm	
Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)			
t_{fp}>=	13.5	mm	
Therefore, propose h_{fp} =	800	mm	
Therefore, propose b_{fp} =	380	mm	
Therefore, propose t_{fp} =	40	mm	
Division Plate			
tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm	
Therefore, propose t_{dp} =	20	mm	
Web Cleats			
Use 90 x 90 x 8 angles to accommodate M20 bolts.			
Length >= 0.5h_{uc}=	187.3	mm	
Therefore, propose length of	200	mm	
Check 2: Minimum Resistance			
Cover Plate			
Requirement: 0.25N_{Ed}<=N_{Rd}			
0.25N_{Ed}=	886.41	kN	
N_{Rd}=2$A_{fp}f_{y,fp}/\gamma_{M0}$=	10792	kN	**OK**
Bolt Group			
Requirement: 0.25N_{Ed}<=2$F_{Rd,fp}$			
a) Shear Resistance per Bolt:			
Diameter of Bolts =	20	mm	
Total thickness of packing t_{pa} =	0	mm	**<d/3**
For t_{pa}>=d/3,β_p=9d/(8d+3t_{pa})=	1.125		
For t_{pa}<d/3, β_p=	1		
Therefore, β_p=	1		
α_v=	0.6	for 8.8 bolts	
Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	245	mm2	

γ_{M2}=	1.25		
$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}$=	94.08	kN	

b) Bearing Resistance per Bolt:
Spacings are desgined based on BSEN1993-1-8 Table 3.3:

Propose e_1=	30	mm	800
Propose e_2=	110	mm	
Propose p_1=	110	mm	
Propose p_2=	160	mm	
Bolt Hole Diameter d_0=	22	mm	
$f_{u,p}$=	410	MPa	
$\alpha_d=\min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)$=	0.45		
$k_1=\min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)$=	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}$=	298.18	kN	
$\min(F_{v,Rd},F_{b,Rd})$=	94.08	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	8	nos.	
$F_{Rd,fp}=\min(F_{v,Rd},F_{b,Rd})(n)$ =	752.64	kN	
As double shear is designed,			
$2F_{Rd,fp}$=	1505.28	kN	
$0.25N_{Ed}$=	886.41	kN	**OK**

Therefore, the design satisfies the requirements.

<u>Column to Column Connection Calculations (Cantilever 3rd Floor to 1st Floor)</u>

Column Section Above	356 x 406 x 634	UC
Column Section Below	356 x 406 x 634	UC

<u>Section Properties (Upper Column)</u>

Grade	355	
h_{uc}	474.6	mm
b_{uc}	424	mm
$t_{w,uc}$	47.6	mm
$t_{f,uc}$	77	mm
r_{uc}	15.2	mm
A_{uc}	808	cm^2

<u>Section Properties (Lower Column)</u>

Grade	355	
h_{lc}	474.6	mm
b_{lc}	424	mm

$t_{w,lc}$	47.6	mm
$t_{f,lc}$	77	mm
r_{lc}	15.2	mm
A_{lc}	808	cm^2

Splice Details

Flange Cover Plate	2/430 x 40 x 1150
Cleats	4/90 x 90 x 8 Angles x 250 Long
Division Plate	430 x 10 x 430
Bolts	M27 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	11866	kN
N_{Ed} =	19022	kN

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height h_{fp} >= $2b_{uc} + t_{dp}$=	858	mm
Width b_{fp} >= b_{uc}	424	mm
Vertical Bolt Spacing p_1=	70	mm
$p_1/14$=	5	mm
$t_{f,uc}$=	77	mm

Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)

t_{fp}>=	38.5	mm
Therefore, propose h_{fp} =	1150	mm
Therefore, propose b_{fp} =	430	mm
Therefore, propose t_{fp} =	40	mm

Division Plate

tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm
Therefore, propose t_{dp} =	10	mm

Web Cleats

Use 90 x 90 x 8 angles to accommodate M27 bolts.

Length >= 0.5h_{uc}=	237.3	mm
Therefore, propose length of	250	mm

Check 2: Minimum Resistance

Cover Plate

Requirement: 0.25N_{Ed}<=N_{Rd}

0.25N_{Ed}=	4755.5	kN

$N_{Rd}=2A_{fp}f_{y,fp}/\gamma_{M0}=$	12212	kN	**OK**

<u>Bolt Group</u>
Requirement: $0.25N_{Ed}<=2F_{Rd,fp}$

a) Shear Resistance per Bolt:

Diameter of Bolts =	27	mm	
Total thickness of packing t_{pa} =	0	mm	**<d/3**
For $t_{pa}>=d/3, \beta_p=9d/(8d+3t_{pa})=$	1.13		
For $t_{pa}<d/3$, $\beta_p=$	1		
Therefore, $\beta_p=$	1		
$\alpha_v=$	0.6	for 8.8 bolts	
Bolt Ultimate Strength $f_{ub}=$	800	MPa	
Stress Area $A_s=$	459	mm2	
$\gamma_{M2}=$	1.25		
$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}=$	176.26	kN	

b) Bearing Resistance per Bolt:
Spacings are desgined based on BSEN1993-1-8 Table 3.3:

Propose $e_1=$	40	mm	1150
Propose $e_2=$	140	mm	
Propose $p_1=$	70	mm	
Propose $p_2=$	150	mm	
Bolt Hole Diameter $d_0=$	30	mm	
$f_{u,p}=$	470	MPa	
$\alpha_d=\min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)=$	0.44		
$k_1=\min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)=$	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}=$	451.2	kN	
$\min(F_{v,Rd},F_{b,Rd})=$	176.256	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	16	nos.	
$F_{Rd,fp}=\min(F_{v,Rd},F_{b,Rd})(n)$ =	2820.10	kN	
As double shear is designed,			
$2F_{Rd,fp}=$	5640.19	kN	
$0.25N_{Ed}=$	4755.5	kN	**OK**

Therefore, the design satisfies the requirements.

Column to Column Connection Calculations (Cantilever 3rd Floor to 1st Floor)

Column Section Above	356 x 406 x 634	UC
Column Section Below	356 x 406 x 634	UC

Section Properties (Upper Column)

Grade	355	
h_{uc}	474.6	mm
b_{uc}	424	mm
$t_{w,uc}$	47.6	mm
$t_{f,uc}$	77	mm
r_{uc}	15.2	mm
A_{uc}	808	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	474.6	mm
b_{lc}	424	mm
$t_{w,lc}$	47.6	mm
$t_{f,lc}$	77	mm
r_{lc}	15.2	mm
A_{lc}	808	cm^2

Splice Details

Flange Cover Plate	2/430 x 40 x 1150
Cleats	4/90 x 90 x 8 Angles x 250 Long
Division Plate	430 x 10 x 430
Bolts	M27 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	11866	kN
N_{Ed} =	19022	kN

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height h_{fp} >= $2b_{uc} + t_{dp}$=	858	mm
Width b_{fp} >= b_{uc}	424	mm
Vertical Bolt Spacing p_1=	70	mm
$p_1/14$=	5	mm
$t_{f,uc}$=	77	mm

Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)

t_{fp}>=	38.5	mm	
Therefore, propose h_{fp} =	1150	mm	
Therefore, propose b_{fp} =	430	mm	
Therefore, propose t_{fp} =	40	mm	
Division Plate			
tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm	
Therefore, propose t_{dp} =	10	mm	
Web Cleats			
Use 90 x 90 x 8 angles to accommodate M27 bolts.			
Length >= 0.5h_{uc}=	237.3	mm	
Therefore, propose length of	250	mm	
Check 2: Minimum Resistance			
Cover Plate			
Requirement: 0.25N_{Ed}<=N_{Rd}			
0.25N_{Ed}=	4755.5	kN	
N_{Rd}=2$A_{fp}f_{y,fp}/\gamma_{M0}$=	12212	kN	**OK**
Bolt Group			
Requirement: 0.25N_{Ed}<=2$F_{Rd,fp}$			
a) Shear Resistance per Bolt:			
Diameter of Bolts =	27	mm	
Total thickness of packing t_{pa} =	0	mm	**<d/3**
For t_{pa}>=d/3,β_p=9d/(8d+3t_{pa})=	1.13		
For t_{pa}<d/3, β_p=	1		
Therefore, β_p=	1		
α_v=	0.6	for 8.8 bolts	
Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	459	mm2	
γ_{M2}=	1.25		
$F_{v,Rd}$=$\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}$=	176.26	kN	
b) Bearing Resistance per Bolt:			
Spacings are desgined based on BSEN1993-1-8 Table 3.3:			
Propose e_1=	40	mm	1150
Propose e_2=	140	mm	
Propose p_1=	70	mm	
Propose p_2=	150	mm	
Bolt Hole Diameter d_0=	30	mm	

$f_{u,p}$=	470	MPa	
α_d=min($e_1/(3d_0)$,$p_1/(3d_0)$-0.25,$f_{ub}/f_{u,p}$,1)=	0.44		
k_1=min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)=	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}$=	451.2	kN	
min($F_{v,Rd}$,$F_{b,Rd}$)=	176.256	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	16	nos.	
$F_{Rd,fp}$=min($F_{v,Rd}$,$F_{b,Rd}$)(n) =	2820.10	kN	
As double shear is designed,			
$2F_{Rd,fp}$=	5640.19	kN	
$0.25N_{Ed}$=	4755.5	kN	**OK**

Therefore, the design satisfies the requirements.

Column to Column Connection Calculations (Wind Bracing Bay Columns)

Column Section Above	356 x 406 x 1202	UC
Column Section Below	356 x 406 x 1202	UC

Section Properties (Upper Column)

Grade	355	
h_{uc}	580	mm
b_{uc}	471	mm
$t_{w,uc}$	95	mm
$t_{f,uc}$	130	mm
r_{uc}	15.4	mm
A_{uc}	1531	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	580	mm
b_{lc}	471	mm
$t_{w,lc}$	95	mm
$t_{f,lc}$	130	mm
r_{lc}	15.4	mm
A_{lc}	1531	cm^2

Splice Details

Flange Cover Plate	2/480 x 40 x 1150
Cleats	4/90 x 90 x 8 Angles x 300 Long
Division Plate	480 x 10 x 480
Bolts	M27 8.8

Fittings Material	S355		
Design Actions			
$N_{Ed,G}$ =	13550	kN	
N_{Ed} =	21721	kN	
Check 1: Recommended Detailing Practice			
External Flange Cover Plates			
Height h_{fp} >= $2b_{uc} + t_{dp}$=	952	mm	
Width b_{fp} >= b_{uc}	471	mm	
Vertical Bolt Spacing p_1=	70	mm	
$p_1/14$=	5	mm	
$t_{f,uc}$=	130	mm	
Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)			
t_{fp}>=	65	mm	
Therefore, propose h_{fp} =	1150	mm	
Therefore, propose b_{fp} =	480	mm	
Therefore, propose t_{fp} =	40	mm	
Division Plate			
tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm	
Therefore, propose t_{dp} =	10	mm	
Web Cleats			
Use 90 x 90 x 8 angles to accommodate M20 bolts.			
Length >= 0.5h_{uc}=	290	mm	
Therefore, propose length of	300	mm	
Check 2: Minimum Resistance			
Cover Plate			
Requirement: 0.25N_{Ed}<=N_{Rd}			
0.25N_{Ed}=	5430.25	kN	
N_{Rd}=2$A_{fp}f_{y,fp}/\gamma_{M0}$=	13632	kN	**OK**
Bolt Group			
Requirement: 0.25N_{Ed}<=2$F_{Rd,fp}$			
a) Shear Resistance per Bolt:			
Diameter of Bolts =	27	mm	
Total thickness of packing t_{pa} =	0	mm	**<d/3**
For t_{pa}>=d/3,β_p=9d/(8d+3t_{pa})=	1.125		
For t_{pa}<d/3, β_p=	1		
Therefore, β_p=	1		

α_v=	0.6	for 8.8 bolts	
Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	459	mm2	
γ_{M2}=	1.25		
$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}$=	176.26	kN	

b) Bearing Resistance per Bolt:
Spacings are desgined based on BSEN1993-1-8 Table 3.3:

Propose e_1=	40	mm	1150
Propose e_2=	160	mm	
Propose p_1=	70	mm	
Propose p_2=	150	mm	
Bolt Hole Diameter d_0=	30	mm	
$f_{u,p}$=	470	MPa	
$\alpha_d=\min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)$=	0.44		
$k_1=\min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)$=	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}$=	451.2	kN	
$\min(F_{v,Rd},F_{b,Rd})$=	176.26	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	16	nos.	
$F_{Rd,fp}=\min(F_{v,Rd},F_{b,Rd})(n)$ =	2820.10	kN	
As double shear is designed,			
$2F_{Rd,fp}$=	5640.19	kN	
$0.25N_{Ed}$=	5430.25	kN	**OK**

Therefore, the design saatisifies the requirements.

11. Wind Bracing Connections Design

Connections are needed for diagonal members and resistance of connection must be checked in tension members. The connection chosen for the project is typical connection with gusset plate. Then, diagonal member is connected to gusset plate with end plate. For circular hollow section (CHS), "T" shaped end connection in figure 11.1 is always used. The end of the hollow section is sealed with sealing plate and part of hollow section is welded to end plate. In this connection design, it is assumed that all memebers have the same centreline and there is no effect of eccentricities.

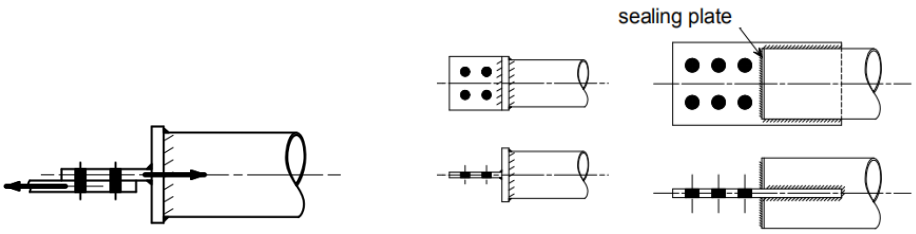


Figure 11.1 Connections details (Left: Gusset connection Right: End plate connection). Retrieved from "Joints in Steel Construction – Simple Joints to Eurocode 3" by The Steel Construction Institute (pp. 258-259), 2014.

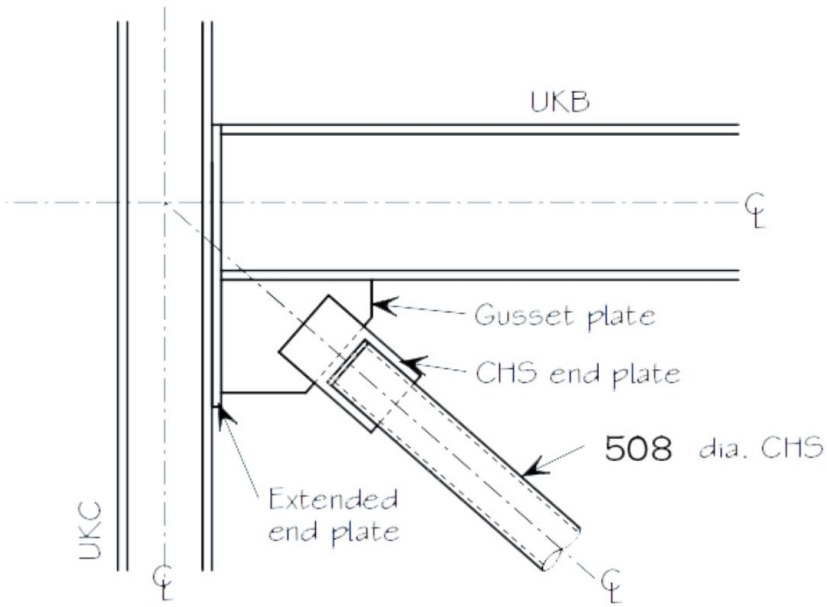


Figure 11.2 Connections details: General layout. Retrieved from "Steel Building Design: Worked examples for students in accordance with Eurocodes and the UK National Annexes" by The Steel Construction Institute (p. 76), 2009.

Axial force in the diagonal members for each bracing frame has been calculated in

3.2.6.5. For both north-south and east-west bracing systems, combination 2 leads to largest axial force.

North-south: $N_{Ed} = 2265.63\ kN$

East-west: $N_{Ed} = 2040.56\ kN$

For material efficiency, each bracing system will have different grade of bolts.

11.1. End plate

Generally, to attain ductility of the design requirement, plates should be relative thin. In the project, horizontal forces on the diagonal members have to be transferred to the column. Therefore, thickness of the plate is designed to be $21mm$ which is slightly thicker than normal plate.

11.2. Position of holes

The spacings of the holes has maximum and minimum limitation and figure 11.3 shows specific value of position of the holes.

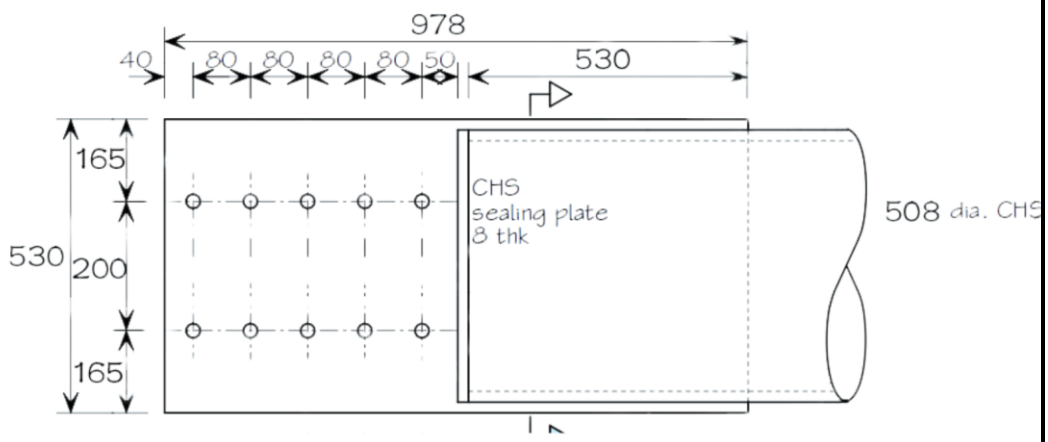


Figure 11.3 Position of holes for bolts. Retrieved from "Steel Building Design: Worked examples for students in accordance with Eurocodes and the UK National Annexes" by The Steel Construction Institute (p. 77), 2009.

BS EN 1993-1-8

Figure 3.1

11.3. Design of fillet weld

The hollow section should be welded to the end plate with fillet weld on both sides of the plate. Also, both top and bottom of the fitted end plate should be welded for increasing strength. In the design, the length of the fillet weld is assumed to be

6.5mm for north-south bracing and 6 mm for east-west bracing.

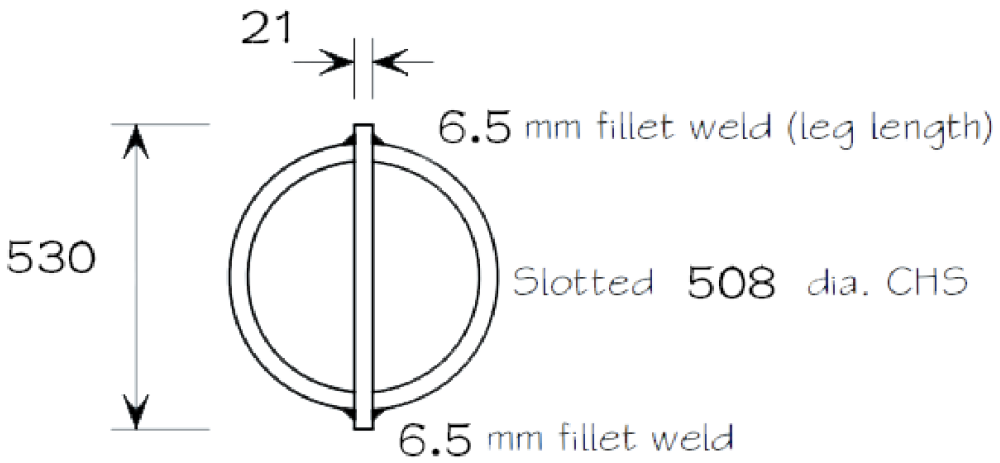


Figure 11.4 Details on connection between end plate and hollow section. Retrieved from "Steel Building Design: Worked examples for students in accordance with Eurocodes and the UK National Annexes" by The Steel Construction Institute (p. 77), 2009.

To check the shear resistance of fillet weld, simplified method of 4.5.3.3 in Eurocode is used:

BS EN 1993-1-8 4.5.3.3(3)

Design shear resistance: $f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_{M2}}$

BS EN 1993-1-8 4.5.3.3(3)

11.4. Local resistance of CHS wall

Shear resistance for the CHS wall is also needed:

$$V_{pl,Rd} = \frac{A_v(f_y/\sqrt{3})}{\gamma_{M0}}$$

11.5. Tensile resistance of end plate

There are two modes of failure including cross-sectional failure and block tearing failure. Cross-sectional failure in figure 11.5 happens on the cross section of the plate which means net area should be checked. For block tearing failure, two shaded areas in figure 11.6 and 11.7 may break with the rest of the area due to tension force. The failure usually happens on the holes for bolts. Resistance of the plate should

BS EN 1993-1-8 3.10.2

bear the tension force to prevent any failure happen.

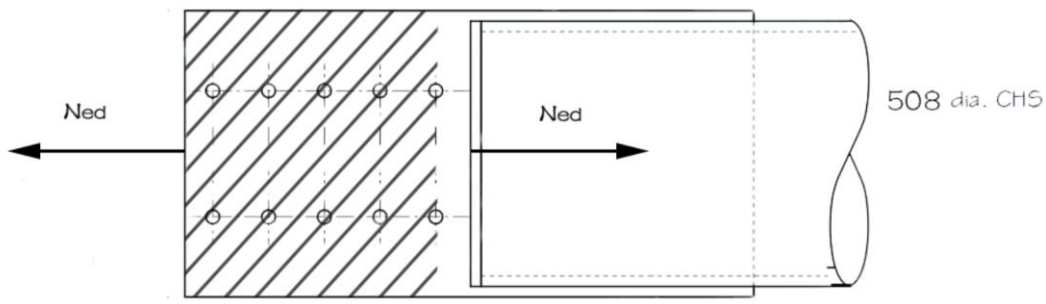


Figure 11.5 Cross-sectional failure. Retrieved from "Steel Building Design: Worked examples for students in accordance with Eurocodes and the UK National Annexes" by The Steel Construction Institute (p. 80), 2009.

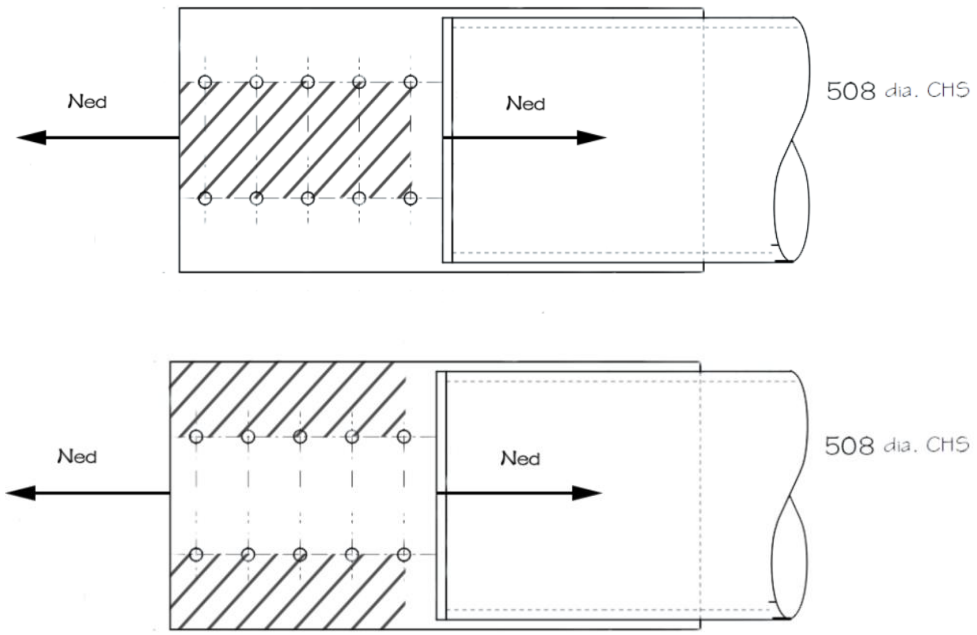


Figure 11.6 Block tearing failure. Retrieved from "Steel Building Design: Worked examples for students in accordance with Eurocodes and the UK National Annexes" by The Steel Construction Institute (p. 80), 2009.

11.5.1. Cross-sectional failure

The design tensile resistance of a cross-section with holes is smaller of $N_{pl,Rd}$ and

$N_{u,Rd}$

$$N_{pl,Rd} = \frac{A \times fy}{\gamma_{M0}}$$

BS EN 1993-1-1 Eqn. 6.6

Where A is the gross cross-sectional area

11.5.2. Block tearing failure

The bolts group of the connection is symmetrical about the concentric loading so the design block tearing resistance is:

$$V_{eff,1,Rd} = \frac{f_u A_{nt}}{\gamma_{M2}} + \left(1/\sqrt{3}\right)\frac{(f_u A_{nv})}{\gamma_{M0}}$$

Where A_{nt} is the net area subject to tension: $A_{nt} = \min((p_2 - d_0)\, t_p, 2(e_2 - 0.5d_0)t_p)$

and A_{nv} is the net area subject to shear: $A_{nv} = 2(3p_1 + e_1 - 2.5d_0)t_w$

11.6. Calculation process

Axial force in the diagonal members:

North-south: $N_{Ed} = 2265.63\ kN$

East-west: $N_{Ed} = 2040.56\ kN$

For material efficiency, each bracing system will have different grade of bolts.

North-south Direction

For connections between end plate and gusset plate, 10 No non-preloaded Class 10.9 M30 diameter bolts in 32 mm diameter clearance holes are chosen. The properties of the bolts are:

Cross section area $A = A_s = 561\ mm^2$

Clearance hole diameter $d_0 = 32\ mm$

Yield strength $f_{yb} = 900\ N/mm$^2

Ultimate tensile strength $f_{ub} = 1000\ N/mm^2$

East-west Direction

For connections between end plate and gusset plate, 10 No non-preloaded Class 8.8 M30 diameter bolts in 32 mm diameter clearance holes are chosen. The properties of the bolts are:

Cross section area $A = A_s = 561\ mm^2$

Clearance hole diameter $d_0 = 32\ mm$

Yield strength $f_{yb} = 640\ N/mm$^2

Ultimate tensile strength $f_{ub} = 800\ N/mm^2$

End plate

End plate section: Grade S355 $978 \times 530 \times 21\ mm$ (for all bracing)

BS EN 1993-1-1 NA 2.4

Thickness $t = 21\ mm$

Length $L = 530\ mm$

Yield strength $f_y = 355\ N/mm^2$

BS EN 10025-2 Table 7

Because $3 \leq t \leq 100\ mm$, Ultimate tensile strength $f_u = 470\ N/mm^2$

Position of holes:

$$e_1 = 40\ mm \quad e_2 = 165\ mm \quad p_1 = 80\ mm \quad p_2 = 200\ mm$$

Limitation of the spacings:

Minimum $e_1 = 1.2d_0 = 1.2 \times 32 = 38.4\ mm < 40\ mm$

Minimum $e_2 = 1.2d_0 = 1.2 \times 32 = 38.4\ mm < 170\ mm$

Minimum $p_1 = 2.2d_0 = 2.2 \times 32 = 70.4\ mm < 80\ mm$

Minimum $p_1 = 2.4d_0 = 2.4 \times 32 = 76.8\ mm < 200\ mm$

Maximum e_1 & e_2 : Larger of $8t = 8 \times 21 = 168\ mm\ or\ 125\ mm >$

$40\ mm$ & $165\ mm$

Maximum p_1 & p_2 : Smaller of $14t = 14 \times 21 = 294\ mm\ or\ 200\ mm > 80\ mm$ & $200mm$

Shear resistance of bolts

North-South

Shear resistance of a single bolt: $F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} = \frac{0.6 \times 1000 \times 561}{1.25} \times 10^{-3} = 269.28\ kN$

BS EN 1993-1-8 Table 3.4

Where for grade 8.8 bolts, $\alpha_v = 0.6$

Minimum No of bolts required: $\frac{N_{Ed}}{F_{v,Rd}} = \frac{2265.63}{269.28} = 8.41\ bolts$

East-west Direction

Shear resistance of a single bolt: $F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} = \frac{0.6 \times 800 \times 561}{1.25} \times 10^{-3} = 215.42\ kN$

Where for grade 8.8 bolts, $\alpha_v = 0.6$

Minimum No of bolts required: $\frac{N_{Ed}}{F_{v,Rd}} = \frac{2040.56}{215.42} = 9.47\ bolts$

Therefore, for each bracing frame, 10 bolts are provided in a single shear.

Bearing resistance of bolts

With grade 355 end plate mentioned before, $f_y = 355\ N/mm^2$ $f_u = 470\ N/mm^2$

BS EN 1993-1-8 Table 3.4

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}}$$

Where α_b is the least value of α_d, $\frac{f_{ub}}{f_{u,p}}$ and 1.0

For end bolts $\alpha_d = \frac{e_1}{3d_0} = \frac{40}{3 \times 32} = 0.42$

For inner bolts $\alpha_d = \frac{p_1}{3d_0} - 0.25 = \frac{80}{3\times32} - 0.25 = 0.58$

$$\frac{f_{ub}}{f_{u,p}} = \frac{1000}{470} = 2.13$$

For end bolts $\alpha_b = \min(0.42, 2.13, 1.0) = 0.42$

For inner bolts $\alpha_b = \min(0.58, 2.13, 1.0) = 0.58$

Therefore, for all bolts: $\alpha_b = 0.42$

For edge bolts k_1 is the smaller of $2.8\frac{e_2}{d_0} - 1.7\ or\ 2.5$

$$2.8\frac{e_2}{d_0} - 1.7 = 2.8 \times \frac{165}{32} - 1.7 = 12.74$$

$k_1 = \min(12.74, 2.5) = 2.5$

For inner bolts k_1 is the smaller of $1.4\frac{p_2}{d_0} - 1.7\ or\ 2.5$

$$1.4\frac{p_2}{d_0} - 1.7 = 1.4 \times \frac{200}{32} - 1.7 = 7.05$$

$$k_1 = \min(7.05, 2.5) = 2.5$$

Therefore, the least bearing resistance of a single bolt is:

$$F_{b,Rd} = \frac{2.5 \times 0.42 \times 470 \times 30 \times 21}{1.25} \times 10^{-3} = 246.75\ kN$$

Conservatively, resistance of all 10 bolts in bearing is: $10 \times 246.75 = 2467.5\ kN$

Group of fasteners

Comparing the shear resistance and bearing resistance of one bolt:

North-south: $F_{v,Rd} = 269.28\ kN > F_{b,Rd} = 246.75\ kN$

The design resistance of group should take the less one: $10 \times 246.75 = 2467.5\ kN$

BS EN 1993-1-8 3.7

East-west: $F_{v,Rd} = 215.42\ kN < F_{b,Rd} = 246.75\ kN$

The design resistance of group should take the less one: $10 \times 215.42 = 2154.2\ kN$

Design of fillet weld

To check the shear resistance of fillet weld, simplified method of 4.5.3.3 in Eurocode is used:

BS EN 1993-1-8 4.5

Design shear resistance: $f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_{M2}}$

BS EN 1993-1-8 4.5.3.3(3)

Where $\beta_w = 0.9$ for S355 steel.

BS EN 1993-1-8 Table 4.1

$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_{M2}} = \frac{470/\sqrt{3}}{0.9 \times 1.25} = 241.20\ N/mm^2$$

North-south Direction

Throat thickness of weld: $a = 0.7 \times leglength = 0.7 \times 6.5 = 4.55\ mm$

Design resistance of weld per unit length:

$$F_{vw,d} = f_{vw,d} \times a = 241.20 \times 4.55 = 1097.48\ N/mm$$

For four welds, each with an effective length of $l_{eff} = 530 - (2 \times 6.5) = 517\ mm$

Shear resistance: $4F_{w,Rd} l_{eff} = 4 \times 1097.48 \times 517 \times 10^{-3} = 2269.59\ kN > 2265.63\ kN$

OK

East-west

Throat thickness of weld: $a = 0.7 \times leglength = 0.7 \times 6 = 4.2\ mm$

Design resistance of weld per unit length:

$$F_{vw,d} = f_{vw,d} \times a = 241.20 \times 4.2 = 1013.06\ N/mm$$

For four welds, each with an effective length of $l_{eff} = 530 - (2 \times 6) = 518\ mm$

Shear resistance: $4F_{w,Rd} l_{eff} = 4 \times 1013.06 \times 518 \times 10^{-3} = 2099.06\ kN >$

$2040.56\ kN$

OK

Local resistance of CHS wall

BS EN 1993-1-1 6.2.6 (3)

For CHS 508×14.2, shear area: $A_v = 0.9dt$

Where d is the depth of the rectangular area and t is the thickness of the hollow section.

Total shear area= $4 \times 0.9 \times 530 \times 14.2 = 27093.6\ mm^2$

Shear resistance: $\frac{A_v(f_y/\sqrt{3})}{\gamma_{M0}} = \frac{27093.6 \times 355/\sqrt{3}}{1.0 \times 10^3} = 5553.09\ kN$

For north-south bracing frames: $5553.09\ kN > 2265.63\ kN$

For east-west bracing frame: $5553.09\ kN > 2040.56\ kN$

OK

Tensile resistance of end plate

BS EN 1993-1-8 3.10.2

There are two modes of failure including cross-sectional failure and block tearing failure.

Cross-sectional failure

BS EN 1993-1-1 6.2.3(1)

Basic requirement: $\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0$

The design tensile resistance of a cross-section with holes is smaller of $N_{pl,Rd}$ and $N_{u,Rd}$

$$N_{pl,Rd} = \frac{A \times fy}{\gamma_{M0}}$$

Where A is the gross cross-sectional area: $A = 21 \times 530 = 11130\ mm^2$

BS EN 1993-1-1 Eqn. 6.6

$$N_{pl,Rd} = \frac{A \times fy}{\gamma_{M0}} = \frac{11130 \times 355}{1.0} \times 10^{-3} = 3951.15\ kN$$

For north-south bracing frames: $3951.15\ kN > 2265.63\ kN$

For east-west bracing frame: $3951.15\ kN > 2040.56\ kN$

OK

BS EN 1993-1-1 6.2.2.2

$$N_{u,Rd} = \frac{0.9 \times A_{net} \times f_u}{\gamma_{M2}}$$

Where $A_{net} = 11130 - (2 \times 32 \times 21) = 9786\ mm^2$

$$N_{u,Rd} = \frac{0.9 \times A_{net} \times f_u}{\gamma_{M2}} = \frac{0.9 \times 9786 \times 470}{1.25} \times 10^{-3} = 3311.58\ kN$$

For north-south bracing frames: $3311.58\ kN > 2265.63\ kN$

For east-west bracing frame: $3311.58\ kN > 2040.56\ kN$

OK

Block tearing failure

BS EN 1993-1-8 3.10.2 (2)

The bolts group of the connection is symmetrical about the concentric loading so the design block tearing resistance is:

$$V_{eff,1,Rd} = \frac{f_u A_{nt}}{\gamma_{M2}} + \left(1/\sqrt{3}\right)\frac{(f_u A_{nv})}{\gamma_{M0}}$$

Where A_{nt} is the net area subject to tension: $A_{nt} = \min((p_2 - d_0)\, t_p, 2(e_2 - 0.5d_0)t_p)$

$$(p_2 - d_0)t_p = (200 - 32) \times 21 = 3528\ mm^2$$

$$2(e_2 - 0.5 \times d_0)t_p = 2 \times (165 - 0.5 \times 32) \times 21 = 6528\ mm^2$$

$$A_{nt} = \min(3528, 6528) = 3528\ mm^2$$

and A_{nv} is the net area subject to shear:

$$A_{nv} = 2(3p_1 + e_1 - 2.5d_0)t_w = 2 \times (3 \times 80 + 40 - 2.5 \times 32) \times 21$$
$$= 8400\ mm^2$$

$$V_{eff,1,Rd} = \frac{f_u A_{nt}}{\gamma_{M2}} + \left(1/\sqrt{3}\right)\frac{(f_u A_{nv})}{\gamma_{M0}} = \frac{470 \times 3528}{1.25 \times 10^3} + \left(1/\sqrt{3}\right)\frac{470 \times 8400}{1 \times 10^3}$$
$$= 3048.19\ kN$$

For north-south bracing frames: $3048.19\ kN > 2265.63\ kN$

For east-west bracing frame: $3048.19\ kN > 2040.56\ kN$

OK

In conclusion, with 10 bolts of different grades used on two types of bracing bay. The connections including bolts, end plate and fillet weld can bear the tensile force from the diagonal member.

12. Cantilever Truss System Connections Design

12.1. General Considerations

It is essential to understand the modelling assumptions before designing the connections. For example, a pinned connection allows rotation at the support, while a fixed connection does not. As a result, a pinned connection does not transfer bending moment at the support, which a fixed connection does. However, global rotation should be allowed by permitting relative horizontal displacement.

Truss members can be connected to posts in multiple ways as shown in the figure below.

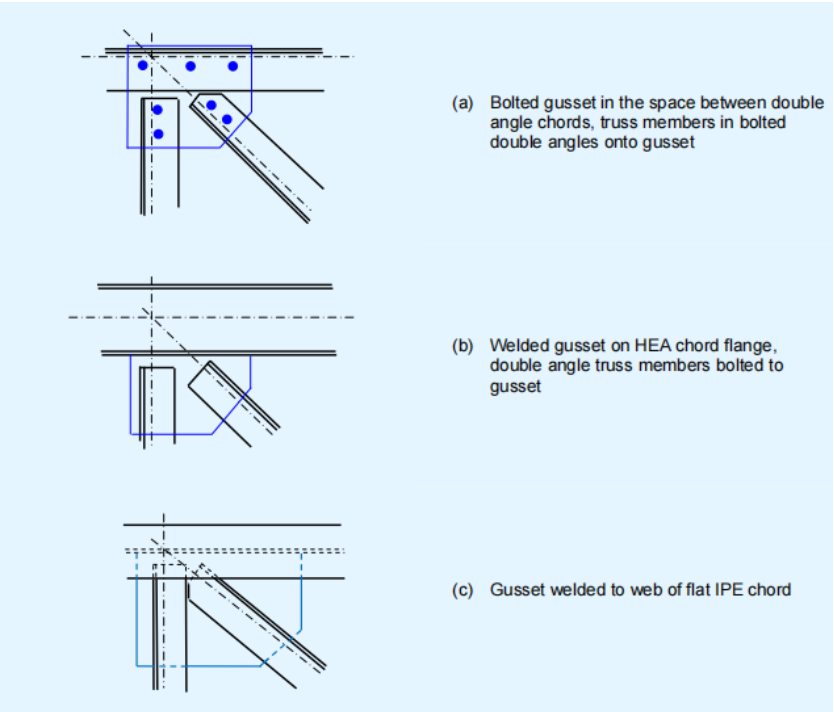


Figure 12.1 Truss connections on chord. Retrieved from Figure 5.5 of Single-Story Steel Buildings Part5: Detailed Design of Trusses, p. 5-49.

Apart from the truss-post connections as mentioned above, it is also common to have connections linking different truss members to form a continuous member. A typical configuration is shown below:

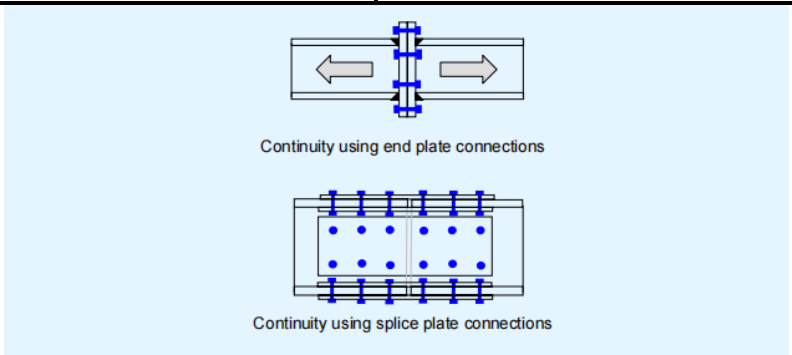


Figure 12.2 Chord Continuity. Retrieved from Figure 5.4 of Single-Story Steel Buildings Part5: Detailed Design of Trusses, p.5-48.

12.2. Design Processes

After designing the geometry of the joints, the design of bolts and welds are then conducted in accordance with BSEN1993-1-8. Buckling of gusset plates should also be checked. Connections to chords in this report is also designed as slip-resistant in accordance with BSEN1993-1-8 to avoid excessive deformation.

12.3. Design of a truss node with gusset – Proposal 1

The truss includes types of joints: splice joints by bolted cover plates, T joint and KT joints. This appendix gives the detailed design of a KT joint located on the upper chord, as shown in figure below:

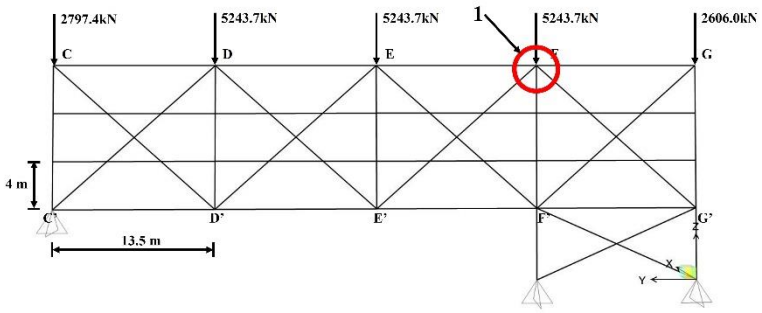
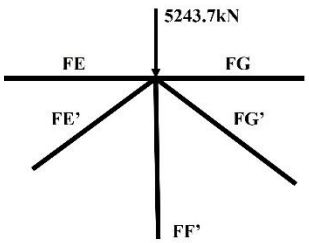


Figure 12.1 Location of the KT joint

The values of the internal forces in the truss members (see table below) result from a gravity load case. This load case corresponds to a ULS combination of actions, determined according to EN 1990.

Table 12.1 KT joint - Internal forces in the truss members - Proposal 1 - West View



Member	N (kN)
Chord FE	-1282.04
Chord FG	1512.48
Diagonal FE'	3622.52
Diagonal FG'	155.33
Diagonal FF'	-8840.08

General presentation of KT joint

The KT joint studied consists of the following connections: the gusset to web chord welded connection and the flanges to gusset bolted connection. Both connections should be verified according to the rules from EN 1993-1-1 and EN 1993-1-8.

Gusset plate to web chord welded connection

The gusset to web chord welded connection is a fillet welded connection formed by a plate welded perpendicular to the chord web. The two fillet welds are identical and designed by considering axial forces transferred from all the three I members connected to the joint.

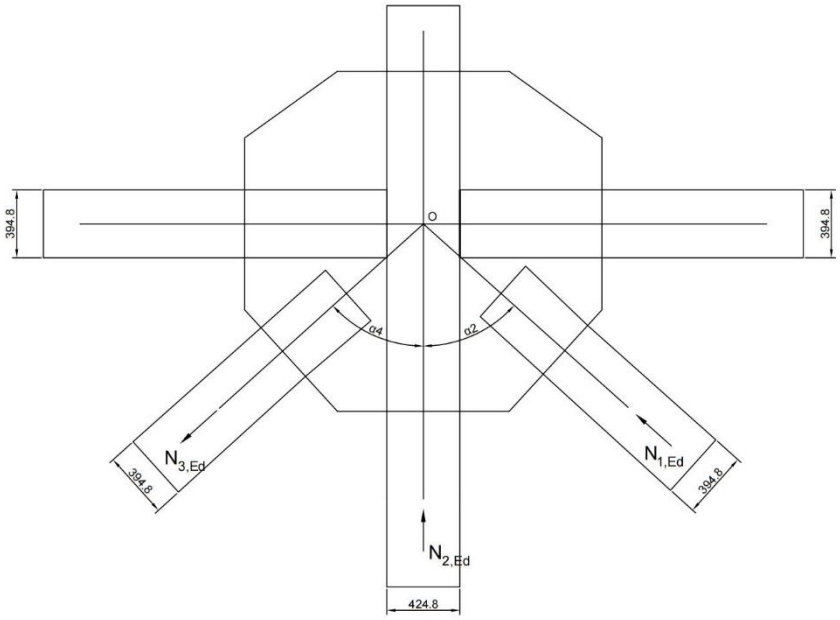


Figure 12.2 Gusset plate to web chord welded connection

The longitudinal axes of all three angle members intersect on the chord axis at the point O in the web

The moment resulting from the eccentricity e_Z should be considered as the gusset plane is not positioned symmetrically about the normal OY to the web plane.

The moment resulting from the eccentricity $e_Y = t_w/2$ can be neglected.

Global coordinate system

The YOZ Plane is that of the gusset plate

The XOZ Plane is that of the chord web

Geometric data

Gusset plate thickness	$t_g = 60\text{mm}$
Flange thickness of DC, DD', DE'	$t_f = 30.2\text{mm}$
Flange thickness of DC', DE	$t_f = 58\text{mm}$

Thickness of shims $t_s = 27.8$mm

Material data

Steel grade	S355
Yield strength	$f_y = 355\ N/mm^2$
Ultimate tensile strength	$f_u = 510\ N/mm^2$

Partial Factor

Resistance of weld: $\gamma_{M2} = 1{,}25$ (recommended value)

Internal force in the truss members

All axial forces are applied in the gusset plate XOZ plane:

Compression axial force at an I to normal OY of $\alpha_1 = 90°$

$N_{1,Ed} = -9692.74$ kN

Compression axial force at an I to normal OY of $\alpha_2 = 48.37°$

$N_{1,Ed} = -382.46$ kN

Tension axial force at an I to normal OY of $\alpha_3 = 0°$

$N_{1,Ed} = 900.69$ kN

Compression axial force at an I to normal OY of $\alpha_4 = -48.37°$

$N_{1,Ed} = -10826.52$ kN

Compression axial force at an I to normal OY of $\alpha_5 = -90°$

$N_{1,Ed} = -1896.67$ kN

Stresses in the gusset cross-section in front of welds

The design approach is based on a linear-elastic analysis, which is conservative in assessing the capacity of the joint.

<u>Design forces in the gusset plate at the chord web face</u>

The effects of the small eccentricity e_Y from the chord axis will be neglected. The gusset plate section is verified for the following forces:

$N_{g,Ed}$ Axial force at an eccentricity of $e_Z = 30\text{mm}$ to the centreline of the gusset plate

$V_{g,Ed}$ shear force

$$With: N_{g,Ed} = \sum_{i=1}^{3} N_i \cos(\alpha_i)$$

$$V_{g,Ed} = \sum_{i=1}^{3} N_i \sin(\alpha_i)$$

And $M_{g,Ed}$, the moment resulting from the eccentricity, $M_{g,Ed} = e_Z N_{g,Ed}$

Then: $N_{g,Ed} = -6545.34\text{kN}$

$V_{g,Ed} = 10.09\text{kN}$

$M_{g,Ed} = -13.09\text{kNm}$

, where $N_{g,Ed}$ is calculated based on local point load and self-weight of truss member.

<u>Normal stress</u>

Assuming a uniform distribution of the load in the section, the normal stress is:

$$\sigma_{g,max} = \frac{N_{g,Ed}}{A_g} + \frac{M_{g,Ed}}{\frac{I_g}{v}}$$

Where: A_g is the cross-section area

I_g is the second moment of cross-section

v is the position of the end fibre

Then: $\sigma_{g,max} = 195.77\ \mathrm{N/mm^2}$

<u>Shear stress</u>

The shear mean stress is:

$$\tau_g = \frac{V_{g,Ed}}{A_g}$$

Then:

$$\tau_g = 0\ \mathrm{N/mm^2}$$

The combination of axial and shear stresses in the gusset plate section can be checked using the Von Mises criterion.

Design resistance of the fillet weld

A fillet weld's design resistance should be assessed using either the directional or simplified method. To achieve a more optimum result, directional method is used in this design.

<u>Directional method</u>

With: $\sigma_\perp$ the normal stress to the throat plane

$\tau_\perp$ the shear stress (in the plane of throat) perpendicular to the axis of the weld

$\tau_{//}$ the shear stress (in the plane of throat) parallel to the axis of the weld

The normal stress $\sigma_{//}$ in the weld is not considered in the design.

On the throat section of the weld, the force per unit length are:

$$a\sigma_{\perp} = \frac{\sigma_{g,max} e_g}{n_a} \sin\left(\frac{\alpha_a}{2}\right) = -138.43\text{N/mm.mm}$$

$$a\tau_{\perp} = \frac{\sigma_{g,max} e_g}{n_a} \cos\left(\frac{\alpha_a}{2}\right) = -138.43\text{N/mm} \cdot \text{mm}$$

$$a\tau_{//} = \frac{\tau_g e_g}{n_a} = -0 \text{ N/mm.mm}$$

The design resistance of the fillet weld will be sufficient if the following conditions are both fulfilled:

$$\sigma_w = \left[\sigma_{\perp}^2 + 3\left(\tau_{\perp}^2 + \tau_{//}^2\right)\right]^{0,5} \leq f_u/(\beta_w \gamma_{M2})$$

$$\sigma_{\perp} \leq 0{,}9 f_u/\gamma_{M2}$$

Where:
β_w is the correlation factor for fillet weld

$$\beta_w = 0.8$$

These conditions can be rewritten in the following forms:

$$(a\sigma_w)/a \leq f_u/(\beta_w \gamma_{M2})$$

$$(a\tau_{\perp})/a \leq 0{,}9 f_u/\gamma_{M2}$$

From these conditions, a minimum value for the effective throat thickness is derived.

$$a_{1,min} = a\sigma_w / \left[\frac{f_u}{(\beta_w \gamma_{M2})}\right] = 0.54\text{mm}$$

$a_{2,min} = a\sigma_{\perp} / \left(\frac{0{,}9 f_{\mathrm{u}}}{\gamma_{\mathrm{M2}}}\right) = 0.38\mathrm{mm}$

$a_{min} = max\left(a_{1,min}; a_{2,min}\right) = 0.54\mathrm{mm}$

The following requirements must be satisfied:

$a \geq 3\mathrm{mm}$

$l_{\mathrm{eff}} \geq max(30\mathrm{mm}; 6a)$ with $l_{\mathrm{eff}} = L_{\mathrm{w}} - 2a$

An effective throat thickness of 4 mm is then sufficient.

12.4. I to gusset bolted connection

All axial forces are applied in the gusset plate XOZ plane:

Compression axial force at an I to normal OY of $\alpha_1 = 90°$

$N_{1,Ed} = -9692.74$ kN

Compression axial force at an I to normal OY of $\alpha_2 = 48.37°$

$N_{1,Ed} = -382.46$ kN

Tension axial force at an I to normal OY of $\alpha_3 = 0°$

$N_{1,Ed} = 900.69$ kN

Compression axial force at an I to normal OY of $\alpha_4 = -48.37°$

$N_{1,Ed} = -10826.52$ kN

Compression axial force at an I to normal OY of $\alpha_5 = -90°$

$N_{1,Ed} = -1896.67$ kN

Basic Data

Material data

Steel grade	S355
Yield strength	$f_y = 355\, N/mm^2$
Ultimate tensile strength	$f_u = 510\, N/mm^2$

Gusset plate

Thickness	$t_g = 30mm$
Length	$L_g = 20mm$
Width	$H_g = 250$

I members

N1	UKC 356*406*467
N2	UKC 356*406*235
N3	UKC 356*406*235
N4	UKC 356*406*467
N5	UKC 356*406*235

Bolted connections data

Category of bolted connections	Category C
Bolt Class	Class 10.9
Yield strength	$f_{yb} = 900\, N/mm^2$
Ultimate tensile strength	$f_{ub} = 1000\, N/mm^2$
Nominal bolt diameter	$\mathrm{d} = 24mm$
Hole diameter	$d_0 = 26mm$

Partial Factors (Recommended valuers)

Structural steel	$\gamma_{\mathrm{M0}} = 1.00$
Structural steel	$\gamma_{\mathrm{M1}} = 1.00$

Structural steel	$\gamma_{M2} = 1.25$
Bolts	$\gamma_{M2} = 1.25$
Bolts	$\gamma_{M3} = 1.25$

12.4.1. Global checking of gross cross-section of the gusset plate

The gross cross-sections of the gusset plates to check are located on the figure as follows:

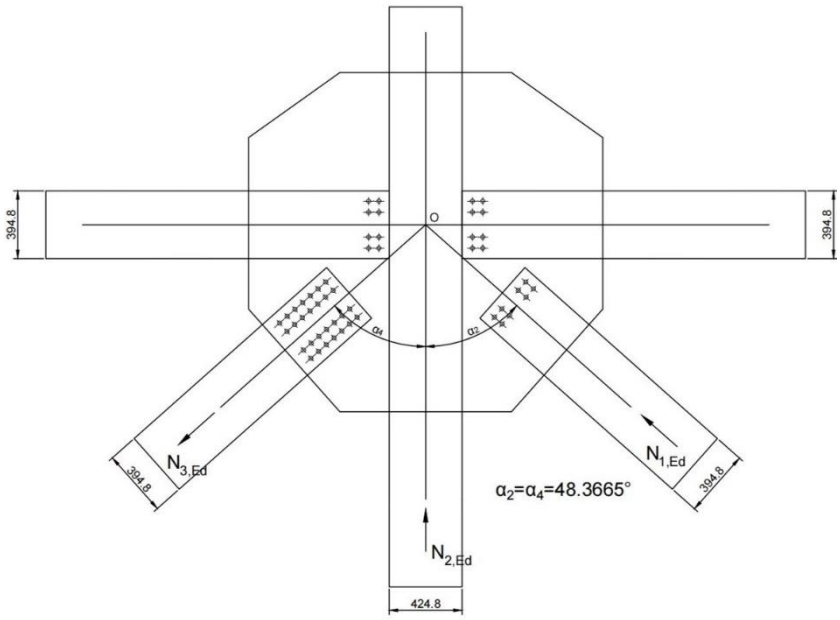


Figure 12.3 Location of the gross cross-sections of the gusset plate

Checking of gross cross-section 1

With A_{g1} cross-sectional area 1 $A_{gl} = H_g t_g = 5200\text{mm}^2$

Shera resistance

$V_{g1,Ed}$

$= max\left(N_{1,Ed}\cos\alpha_1\,;N_{2,Ed}\cos\alpha_2\,;N_{3,Ed}\cos\alpha_3\,;N_{4,Ed}\cos\alpha_4\,;N_{5,Ed}\cos\alpha_5\right)$

$= 899.5\text{kN}$

$V_{\text{g1,pl,Rd}} = A_{\text{gl}} f_{\text{y}} / \left(\gamma_{\text{M0}} \sqrt{3}\right) = 1065.79\text{kN}$

$V_{\text{g1,Ed}} < V_{\text{gl,pl,Rd}} \Rightarrow \text{OK}$

Axial force resistance

$N_{\text{gl,Ed}} = \sum_{\text{i=1}}^{5} N_{\text{i,Ed}} \sin\left(\alpha_{\text{i}}\right) = 10.09\text{kN}$

$N_{\text{g1,pl,Rd}} = A_{\text{g1}} f_{\text{y}} / \gamma_{\text{M0}} = 1846.00\text{kN}$

$N_{\text{g1,Ed}} < N_{\text{g1,pl,Rd}} \Rightarrow \text{OK}$

Checking of gross cross-section 2

With A_{g2} cross-sectional area 2 $A_{\text{g2}} = H_{\text{g}} t_{\text{g}} = 12000\text{mm}^2$

Shera resistance

$$V_{\text{g2,Ed}} = \sum_{\text{i=1}}^{5} N_{\text{i,Ed}} \sin\left(\alpha_{\text{i}}\right) = 10.09\text{kN}$$

$V_{\text{g2,pl,Rd}} = A_{\text{g2}} f_{\text{y}} / \left(\gamma_{\text{M0}} \sqrt{3}\right) = 2459.51\text{kN}$

$V_{\text{g2,Ed}} < V_{\text{g2,pl,Rd}} \Rightarrow \text{OK}$

Axial force resistance

$N_{\text{gl,Ed}} = \sum_{\text{i=1}}^{5} N_{\text{i,Ed}} \sin(\alpha_{\text{i}}) = 10.09\text{kN}$

$N_{\text{g1,pl,Rd}} = A_{\text{g1}} f_{\text{y}} / \gamma_{\text{M0}} = 1846.00\text{kN}$

$N_{\text{g1,Ed}} < N_{\text{g1,pl,Rd}} \Rightarrow \text{OK}$

12.4.2. Connection N3

The shear connection in compression is designed as Category C.

The sizes of the components and the positioning of the holes are shown on the figure as follows:

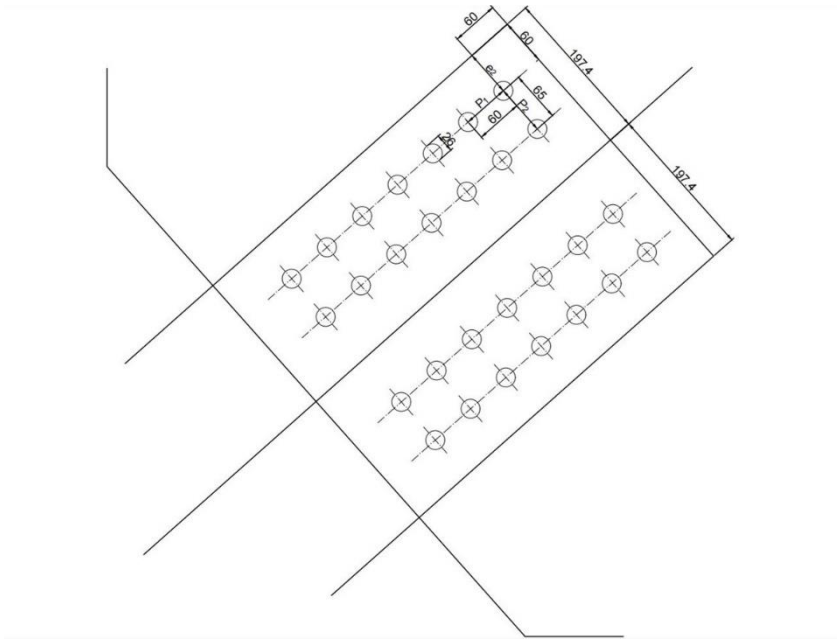


Figure 12.4 Connection sizes and positioning

With: $N_{3,\mathrm{Ed}}$ Axial compression force at an eccentricity of e_{N3} to the centre of gravity of the joint

$M_{3,\mathrm{N,Ed}}$ Bending moment resulting from the eccentricity, $M_{3,\mathrm{N,Ed}} = e_{\mathrm{N3}} N_{3,\mathrm{Ed}}$

For the gusset:

$N_{3,\mathrm{g,Ed}} = 899.5kN$

$e_{\mathrm{N3}} = 65mm$

$M_{3,\mathrm{N,Ed}} = e_{\mathrm{N3}} N_{3,\mathrm{Ed}} = 58.47kNm$

For each flange:

$N_{3,g,Ed} = 449.75kN$

$M_{3,N,Ed} = e_{N3}N_{3,Ed} = 29.23kNm$

<u>Checking of flange resistance of gross cross-section longitudinal stress</u>

Assuming a uniform distribution of the load in the section, the longitudinal stress is:

$$\sigma_i = \frac{N_{3,a,Ed}}{A_{3,a}} + \frac{M_{3,a,Ed}}{I_{3,a}/\boldsymbol{v}}$$

Where: $A_{3,a}$ is the section area of I

$A_{3,a} = 29900\text{mm}^2$

$I_{3,a}$ is the second moment of area of I

$I_{3,a} = 7.91 \times 10^8\text{mm}^4$

v position of considered end fibre, which equals to 65mm

Then the normal stresses is:

$\sigma = 17.44\text{N/mm}^2$ (tension)

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (Table 5.2, sheet 1)

$c/t = 5.73$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 5.73$

$\therefore\ Flange\ is\ Class\ 1$

Web – internal part in compression (Table 5.2, sheet 1)

$c/t = 15.8$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 15.8$

$\therefore\ Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Resistance of net cross-section</u>

From 6.2.5 (5) of EN 1993-1-1, the fastener holes in tension zone need not be allowed for, provided that the following limit is satisfied for the complete tension zone:

$$\frac{A_{\mathrm{t,net}} 0{,}9 f_{\mathrm{u}}}{\gamma_{\mathrm{M2}}} \geq \frac{A_{\mathrm{t}} f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}}$$

Here, the holes are in the tension zone.

The following criterion should be fulfilled:

$N_{3,\mathrm{a,Ed}} \leq N_{3,\mathrm{a,c,Rd}} = \frac{A_{3,\mathrm{a}} f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}}$

With: $A_{3,\mathrm{a}} = 29900\mathrm{mm}^2$:

$N_{3,\mathrm{a,Ed}} = 899.5 < N_{3,\mathrm{a,c,Rd}} = 10614.5\mathrm{kN}$

Buckling resistance

The buckling resistance is verified in this section.

For the determination of the gross cross-section of gusset plate, a diffusion of 45° of the axial force $N_{g,Ed}$ is assumed.

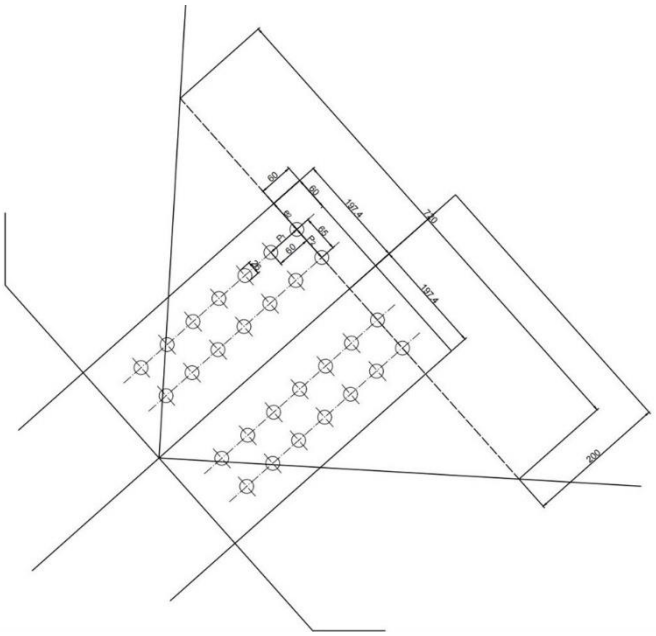


Figure 12.5 Diffusion by 45° of the axial force

The following criteria must be satisfied:

$$\sigma_{x,Ed} = \frac{N_{3,g,Ed}}{A_{3,g}} \pm \frac{M_{3,g,Ed}}{I_{3,g}/v} \leq \frac{f_y}{\gamma_{M0}}$$

With: $A_{3,g} = 240 \times t_g = 7200\text{mm}^2$

$I_{3,g} = t_g \times 240^3/12 = 34560000\text{mm}^4$

$M_{3,g,Ed} = 0$

Then: $\sigma_{\mathrm{x,Ed}} = 30.08 < \frac{f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}} = 355\mathrm{N/mm^2}$

Buckling resistance

The gusset is made similar to an embedded column of characteristics:

Area	$A_{3,\mathrm{g}} = 7200\mathrm{mm}^2$
Height	$h_{\mathrm{c}} = 115\mathrm{mm}$
Second moment of area	$I_{\mathrm{c},zz} = 34560000\mathrm{mm}^4$

We should satisfy:

$N_{3,g,Ed} < N_{3,g,b,\mathrm{Rd}} = \frac{\chi A_{3,g} f_y}{\gamma_{M1}}$

Where χ is the reduction factor for the relevant buckling curve

With a buckling length of $2h_{\mathrm{c}}$, the slenderness is given by:

$\bar{\lambda} = \sqrt{\frac{4h_{\mathrm{c}}^2 A_{\mathrm{c}} f_{\mathrm{y}}}{\pi^2 E I_{\mathrm{c}}}} = 1.37$

The buckling curve to use is curve c and the imperfection is:

$\alpha = 0.49$

$\Phi = 0.5\left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right] = 1.73$

$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = 0.36$

Then: $N_{3,\mathrm{g,Ed}} = 899.5 < N_{3,\mathrm{g,b,Rd}} = 917.74\mathrm{kN}$

12.5. Bolts arrangement

The bolts are fully threaded, non-preloaded, M24 10.9, as generally used in the UK

Yield strength	$f_{yb} = 900\, N/mm^2$
Ultimate tensile strength	$f_{ub} = 1000\, N/mm^2$
Nominal bolt diameter	$\mathrm{d} = 24mm$
Hole diameter	$d_0 = 26mm$
Tensile stress area of bolt	$A_s = 353mm^2$

Spacing requirements

$\mathrm{e}_1 \geq 2\mathrm{d}_0 \rightarrow 60\mathrm{mm} \geq (2 \times 26) = 52\mathrm{mm}$

$\mathrm{e}_2 \geq 2\mathrm{d}_0 \rightarrow 60\mathrm{mm} \geq (2 \times 26) = 52\mathrm{mm}$

$p_1 \geq 2.2\mathrm{d}_0 \rightarrow 60\mathrm{mm} \geq (2.2 \times 26) = 57.2\mathrm{mm}$

$p_2 \geq 2.4\mathrm{d}_0 \rightarrow 65\mathrm{mm} \geq (2.4 \times 26) = 62.4\mathrm{mm}$

Cross-section of flange

UKC 356*406*235

Depth h	381 mm
Width b	394.8 mm
Thickness of flange t_f	$t_f = 30.2mm$

UKC 356*406*467

Depth h	436.6 mm
Width b	412.2 mm
Thickness of flange t_f	$t_f = 58mm$

As the height of the two I-members are not identical, spacers are added to the flange with a thickness of $t_s = \frac{t_{f,1} - t_{f,2}}{2} = 27.8mm$

Partial Factors (Recommended valuers)

$\gamma_{M0} = 1.00$
$\gamma_{M1} = 1.00$
$\gamma_{M2} = 1.25$ (for shear)
$\gamma_{M2} = 1.1$ (for bolts in tension)

$\gamma_{Mu} = 1.1$ The partial factor for resistance γ_{Mu} is used for the tying resistance. Plastic deformation is expected, so elastic check is not appropriate in this case.

Bolt shear resistance

Assuming the shear plane passes through the threaded portion of the bolt, the shear resistance $F_{v,Rd}$ of a single bolt is given by:

$F_{v,Rd} = \frac{0.5 \cdot f_{ub} \cdot A_s}{\gamma_{M2}}$ for bolt class 10.9

A reduction factor of 0.8 is adopted to account for the tension present in the bolts.

$$F_{v,Rd} = 0.8 \times \frac{0.5 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = 0.8 \times \frac{0.5 \cdot 1000 \cdot 353}{1.25} \cdot 10^{-3} = 112.96\text{kN}$$

Bolt tension resistance

The design tension resistance of a bolt is given by:

$$F_{t,Rd} = \frac{0.9 \cdot f_{ub} \cdot A_s}{\gamma_{M2}}$$

$$F_{t,Rd} = \frac{0.9 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = \frac{0.9 \cdot 1000 \cdot 353}{1.25} = 254.16\ kN$$

Minimum bolts numbers

$$n = \frac{F_{v,Ed}}{F_{v,Rd}} = \frac{3623.66}{112.96} = 32.08$$

Bearing resistance

The design bearing resistance of a bolt is given by:

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}}$$

Where: α_b is the smallest of $\alpha_d, f_{ub}/f_u \; and \; 1$

In the direction of load transfer:

For end bolts: $\alpha_d = \frac{e_1}{3d_0} \; and \; for \; inner \; bolts: \alpha_d = \frac{p_1}{3d_0} - \frac{1}{4}$

$\alpha_b = min\left(\frac{60}{3\times24}; \frac{1000}{510}; 1.0\right) = 0.77$ for end bolts

$\alpha_b = min\left(\frac{60}{3\times24} - 0.25; \frac{1000}{510}; 1.0\right) = 0.52$ for inner bolts

Perpendicular to the direction of load transfer:

For edge bolts: $k_1 \; is \; the \; smallest \; of \; 2.8\frac{e_2}{d_0} - 1.7 \; or \; 2.5$,

For inner bolts: $k_1 \; is \; the \; smallest \; of \; 1.4\frac{p_2}{d_0} - 1.7 \; or \; 2.5$ (where d is bolt diameter and d_0 is hole diameter)

And t is the thickness of the plate, which equals to 15 mm.

$k_1 = min\left(2.8\frac{60}{24} - 1.7; 2.5\right) = 2.5$ for edge bolts

$k_1 = min\left(1.4\frac{65}{24} - 1.7; 2.5\right) = 1.8$ for inner bolts

Therefore, for the edge bolts,

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}} = \frac{2.5 \cdot 0.77 \cdot 510 \cdot 24 \cdot 30}{1.25} = 564.92\ kN$$

For the inner bolts,

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}} = \frac{1.8 \cdot 0.52 \cdot 510 \cdot 24 \cdot 30}{1.25} = 274.55\ \text{kN}$$

The bearing resistance of the bolts $= 2 \times 564.92 + 34 \times 274.55 = 10464.64\ kN$

Group of Fasteners

Because the shear resistance of the fasteners (112.96 kN) is less than the bearing resistance, the resistance of the group of fasteners must be taken as the number of fasteners multiplied by the smallest design resistance of the individual fasteners – in this case 112.96 kN

Resistance of the group $= 36 \times 112.96 = 4066.56\ kN \geq V_{Ed} = 3623.96\text{kN}$ OK

Plate strength

Considering 2 critical sections:

Net section:

$$V_{Rd,n} = 2 \cdot \frac{A_{v,net} \cdot f_u}{\sqrt{3} \cdot \gamma_{M2}} = 2 \cdot \frac{11520 \cdot 510}{\sqrt{3} \cdot 1.25} \cdot 10^{-3} = 5427.28\text{kN} \geq V_{Ed}$$
$$= 3623.96\text{kN}$$

Gross section:

$$V_{Rd,g} = 2 \cdot \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = 2 \cdot \frac{18000 \cdot 355}{\sqrt{3} \cdot 1.0} \cdot 10^{-3} = 7378.54\text{kN} \geq V_{Ed}$$
$$= 3623.96\text{kN}$$

Block shear:

$$V_{Rd,b} = 2 \cdot \left[\frac{0.5 \cdot A_{nt} \cdot f_u}{\gamma_{M2}} + \frac{A_{nv} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}}\right]$$

$$= 2 \cdot \left[\frac{0.5 \cdot (45 - 9) \cdot 9 \cdot 510}{1.25} + \frac{(220 - 35 + 9 - 4 \cdot 18) \cdot 9 \cdot 355}{\sqrt{3} \cdot 1.0}\right] \cdot 10^{-3}$$

$$= 7755.02 \text{ kN} \geq V_{Ed} = 3623.96\text{kN OK}$$

Weld Strength

Use mean stress method to check weld strength. This method considers the weld strength as being equal to the shear strength, independent of the direction of the force acting on it. Since the weld is weakest in pure shear the mean stress method always gives results on the safe side.

$$\frac{F}{a \cdot l} \leq f_{vw} \text{ with } f_{vw} = \frac{f_u}{\sqrt{3 \cdot \beta_w \cdot \gamma_{M2}}}$$

Where: l = 800 mm, a = 20 mm, $\beta_w = 0.9$ for S355

Then, $f_{vw} = \frac{510}{\sqrt{3 \cdot 0.9 \cdot 1.25}} = 277.61\ kN$

$F = f_{vw} a l = 277.61 \times 800 \times 20 = 4441.74\ kN \geq V_{Ed} = 3623.96\text{kN OK}$

For the rest of the node design, the same steps as above are followed and the following table is made to calculate the number of bolts required:

12.6. Spreadsheet of bolts numbers – Proposal 1

For ease of representation, we have labelled the rods, which correspond to the following diagram：

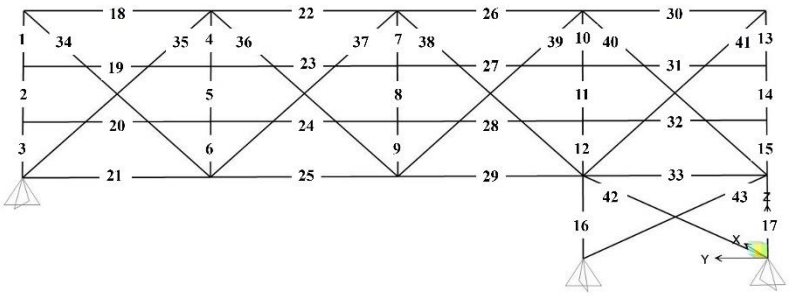


Figure 12.6 Member numbers – Proposal 1 - West View

The calculation for the other nodes is not significantly different from that described in the previous section, and the number of bolts required for each node is given below：

Table 12.2 Number of bolts - Proposal 1 – West View

Member	Axial Force kN	F_v,Ed kN	Number of bolts n
1	-5146.31	2573.155	24
2	-5771.52	2885.76	28
3	-6397.92	3198.96	32
4	2069.26	1034.63	12
5	899.14	449.57	4
6	-269.32	134.66	4
7	1406.95	703.475	8
8	237.07	118.535	4
9	-931.85	465.925	8
10	-7648.5	3824.25	36
11	-8839.38	4419.69	40
12	-10022.33	5011.165	48
13	-1972.85	986.425	12
14	2457.68	1228.84	12
15	-3046.74	1523.37	16
16	-17495.33	8747.665	80
17	-4230.13	2115.065	20
18	-1896.67	948.335	12

19	-50.95	25.475	4
20	45.53	22.765	4
21	569.69	284.845	4
22	-9692.74	4846.37	44
23	-86.11	43.055	4
24	78.73	39.365	4
25	2716.54	1358.27	16
26	-1282.04	641.02	8
27	-72.61	36.305	4
28	68.9	34.45	4
29	-274.97	137.485	4
30	1512.48	756.24	8
31	-3.62	1.81	4
32	-0.46	0.23	4
33	-874.93	437.465	4
34	2564.38	1282.19	12
35	-10824.33	5412.165	48
36	-381.78	190.89	4
37	-321.4	160.7	4
38	-11577.47	5788.735	52
39	3623.7	1811.85	20
40	-154.68	77.34	4
41	-2016.55	1008.275	12
42	-7114.42	3557.21	32
43	1087.23	543.615	8

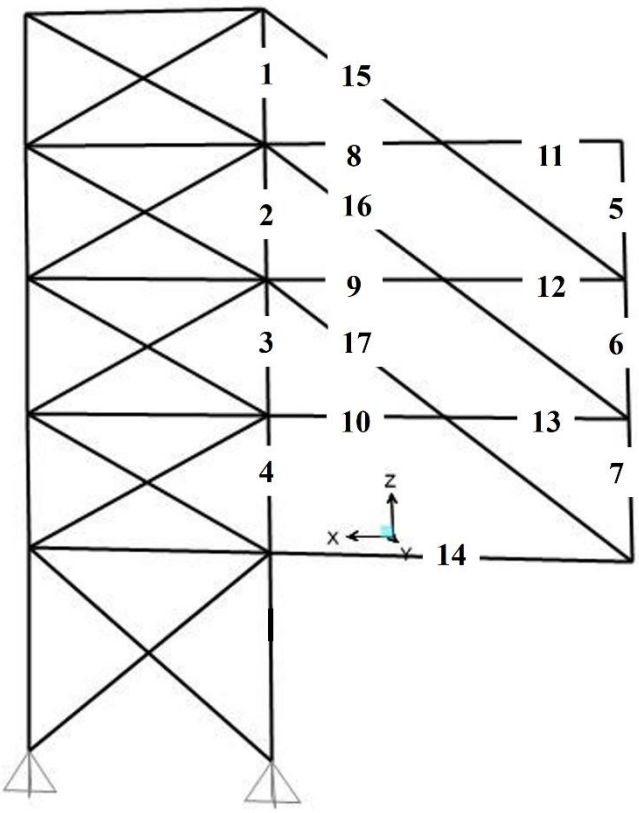


Figure 12.7 Member numbers - Proposal 1 – North View

The number of bolts required for each node is given below：

Member	Axial Force kN	F_v,Ed kN	Number of bolts n
1	-9415.12	4707.56	44
2	-17951.75	8975.875	80
3	-32002.77	16001.385	144
4	-38713.17	19356.585	172
5	-32.14	16.07	4
6	4055.33	2027.665	20
7	7336.06	3668.03	36
8	-7.41	3.705	4
9	-5198.87	2599.435	24

10	-4176.85	2088.425	20
11	-7.41	3.705	4
12	-5198.87	2599.435	24
13	-4176.85	2088.425	20
14	-8759.28	4379.64	40
15	6671.32	3335.66	32
16	5377.5	2688.75	24
17	11211.54	5605.77	52

12.7. Spreadsheet of bolts numbers – Proposal 2

As the same as analysis above, we got this:

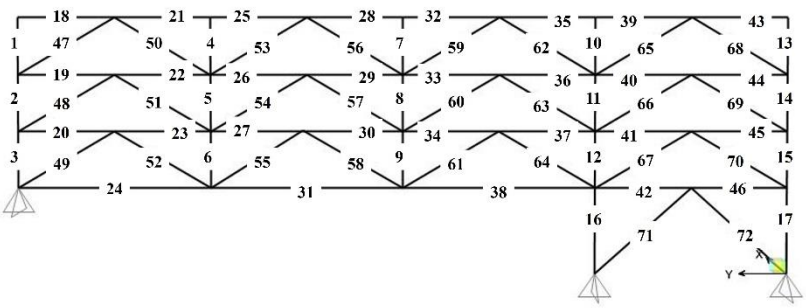


Figure 12.8 Member numbers - Proposal 2 – West View

The number of bolts required for each node is given below：

Table 12.3 Number of bolts - Proposal 2 - West View

Member	Axial Force kN	F_v,Ed kN	Number of bolts n
1	-2800.02	1400.01	16
2	-5819.93	2909.965	28
3	-9680.75	4840.375	44
4	-5093.66	2546.83	24
5	-3890.91	1945.455	20

6	-1815.45	907.725	12
7	-5100.13	2550.065	24
8	-3785.46	1892.73	20
9	-1985.3	992.65	12
10	-5293.43	2646.715	24
11	-9936.01	4968.005	44
12	-14912.5	7456.255	68
13	-2613.99	1306.995	12
14	-2686.64	1343.32	12
15	-2868.05	1434.025	16
16	-19647	9823.485	88
17	-3371.95	1685.975	16
18	-11.51	5.755	4
19	4018.22	2009.11	20
20	5402.47	2701.235	24
21	-7757.46	3878.73	36
22	-6601.76	3300.88	32
23	-5009.28	2504.64	24
24	267.37	133.685	4
25	-7768.61	3884.305	36
26	-2904.28	1452.14	16
27	55.39	27.695	4
28	-7230.48	3615.24	36
29	-2194.17	1097.085	12
30	55.39	27.695	4
31	5409.68	2704.84	24
32	-7214.76	3607.38	32
33	-7176.76	3588.38	32
34	-5976.46	2988.23	28
35	1916.1	958.05	12
36	3955.8	1977.9	20
37	4905.91	2452.955	24
38	-54.18	27.09	4
39	1976.32	988.16	12

40	306.07	153.035	4
41	-106.88	53.44	4
42	-5186.25	2593.125	24
43	10.15	5.075	4
44	-934.46	467.23	8
45	-583.11	291.555	4
46	-134.52	67.26	4
47	-4700.1	2350.05	24
48	-6308.55	3154.275	28
49	-6193.01	3096.505	28
50	4314.8	2157.4	20
51	6042.94	3021.47	28
52	5912.43	2956.215	28
53	-8.69	4.345	4
54	185.54	92.77	4
55	-32.41	16.205	4
56	-635.52	317.76	4
57	-641.8	320.9	4
58	-35.29	17.645	4
59	5180.78	2590.39	24
60	6360.49	3180.245	32
61	6256.27	3128.135	28
62	-5499.78	2749.89	28
63	-6588.38	3294.19	32
64	-6406.63	3203.315	32
65	-1193.64	596.82	8
66	-782.5	391.25	4
67	-409.71	204.855	4
68	1092.96	546.48	8
69	659.4	329.7	4
70	143.24	71.62	4
71	3292.92	1646.46	16
72	-3476.97	1738.485	16

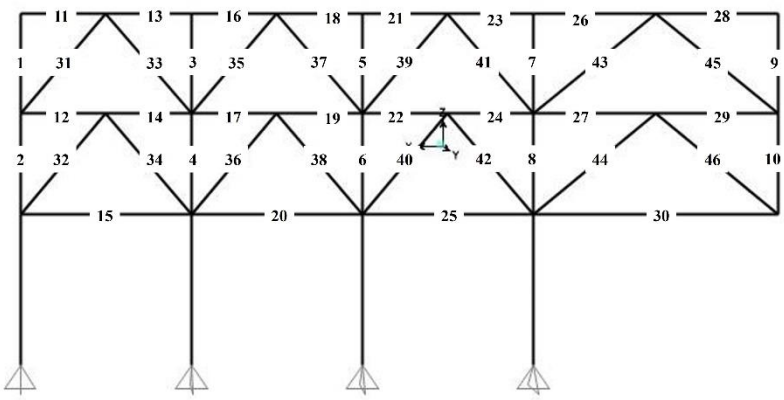


Figure 12.9 Member numbers - Proposal 2 - North View

The number of bolts required for each node is given below：

Member	Axial Force kN	F_v,Ed kN	Number of bolts n
1	-3531.2	1765.6	16
2	-3327.48	1663.74	16
3	-3480.36	1740.18	16
4	-2215.752	1107.876	12
5	-3667.07	1833.535	20
6	-2712.03	1356.015	16
7	-4613.08	2306.54	24
8	-16358.87	8179.435	76
9	-10.96	5.48	4
10	6200.48	3100.24	28
11	45.64	22.82	4
12	-885.08	442.54	4
13	1834.95	917.475	12
14	1052.67	526.335	8
15	-838.23	419.115	4
16	1854.98	927.49	12

17	-2587.38	1293.69	12
18	7230.33	3615.165	36
19	2349.14	1174.57	12
20	-4247.31	2123.655	20
21	7344.59	3672.295	36
22	-4701.65	2350.825	24
23	15524.82	7762.41	72
24	4967.35	2483.675	24
25	-11287.51	5643.755	52
26	15428.24	7714.12	72
27	8986.42	4493.21	40
28	57.54	28.77	4
29	7702.55	3851.275	36
30	-8344.09	4172.045	40
31	1247.4	623.7	8
32	1357.9	678.95	8
33	-1471.2	735.6	8
34	-1585.88	792.94	8
35	4039.68	2019.84	20
36	3640.23	1820.115	20
37	-4124.3	2062.15	20
38	-3857.65	1928.825	20
39	6276.78	3138.39	28
40	7165.36	3582.68	32
41	-6145.04	3072.52	28
42	-7517.31	3758.655	36
43	-9798.58	4899.29	44
44	-10775.65	5387.825	48
45	9882.52	4941.26	44
46	10593.91	5296.955	48

13. Parametric Analysis

Other than the two proposals mentioned above, some alternative proposals have also been studied, which will be mentioned in this section.

In Proposal one, the truss system on the north-east side of the building only covers from the cantilever region to the first column. Before proposal two has been firmed, the extension of the bottom truss system all along the length of the northeast side was once considered. This was because the maximum axial load found on the column was significantly larger than other columns. Therefore, a parametric analysis was conducted. Based on the results, the truss system was found to be less effective when its length increased. A possible reason was that the axial stiffness (EA/L) of the column was much larger than that of the truss. As a result, loads were transferred to the nearest column instead of via the truss system to the columns far away from the cantilever region. Owing to this, this scheme was not chosen as one of the proposals as it could not achieve a much better load distribution while requiring much more materials to build the truss.

As an alternative to Proposal two, the wind bracing was once considered to put on the external surface like Proposal one. However, this scheme was finally not adopted due to the issues of connection between truss and bracing system. As the wind bracing and bottom truss system was on the same plane, the two systems would clash with each other, which made it difficult to construct. It might not be practical when the members of both systems were close to each other, such that there might not be sufficient clearance between them for construction and future maintenance. In addition, the design objective was to transfer the force of the cantilever region via the bottom truss system to the adjacent columns. When the bracing system was on the external surface with the bottom truss system, some of the forces might be transferred to the bracing system. Owing to this, a larger cross-section might be required for the bracing members, which made it less sustainable. The interaction between the two systems also made the analysis more complicated, so this scheme was not adopted.

13.1. Aborted Proposals

Some interesting and typical proposals with different layouts of truss, grid systems are listed below and aborted due to inefficient transfer loads or failure to resist loads.

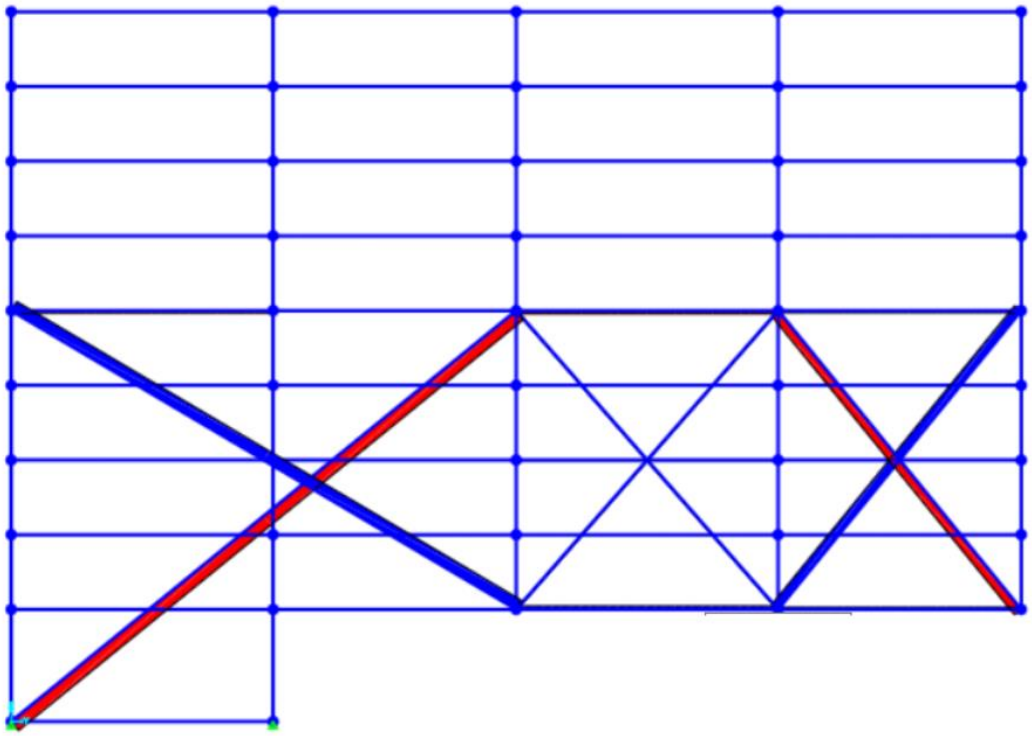


Figure 13.1 Long truss system in West face

As shown above, along with a truss system with a large angle, can efficiently transfer loads to the ground. The reason the proposal is to be aborted is that the compression member in compression (red truss member in LHS) is too long and will fail due to buckling behavior even though very large cross-sections are deployed.

However, it has been found that the intersection with beams can provide lateral restrains and the effective length can be reduced.

Therefore, it is a proposal worth investigating.

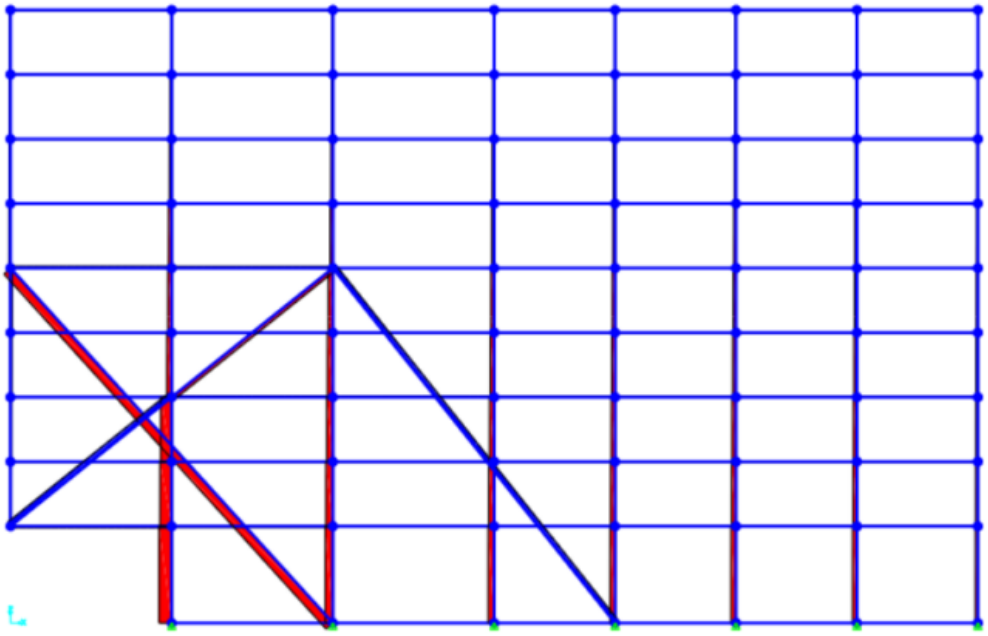


Figure 13.2 Long truss system in North face

It is the same long truss system as introduced before. A large angle of the truss can provide an efficient loading path.

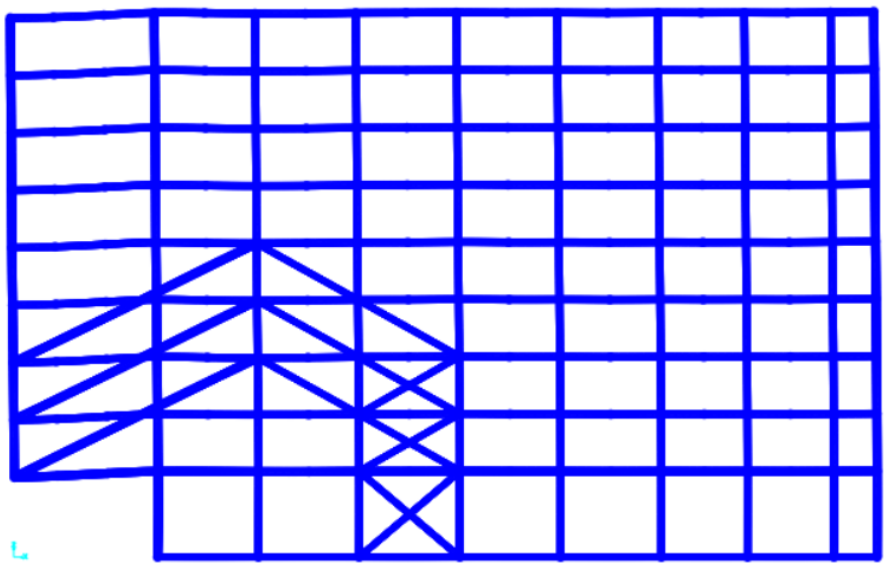


Figure 13.3 Aborted truss system in north face

This truss system is similar to the one provided in proposal 1 in north face. By arranging like this, it can be meeting the requirement for deflection and axial compression.

However, it is a very inefficient system resulting from the very shallow angle of diagonal members. Furthermore, the bracing system can be shared with a truss system to increase structural efficiency. For those reasons, this proposal has been aborted.

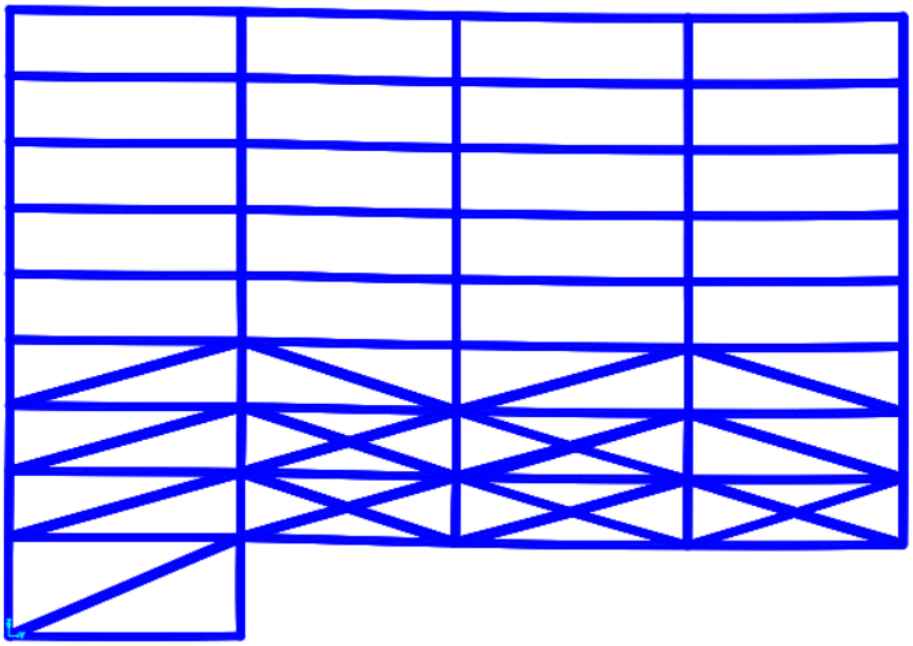


Figure 13.4 Aborted truss system in the west face

The span of the secondary beam is 13.5m, hence if every diagonal truss member is just across one floor the angle between diagonal and horizontal truss member will be very shallow. Therefore, it is an inefficient proposal.

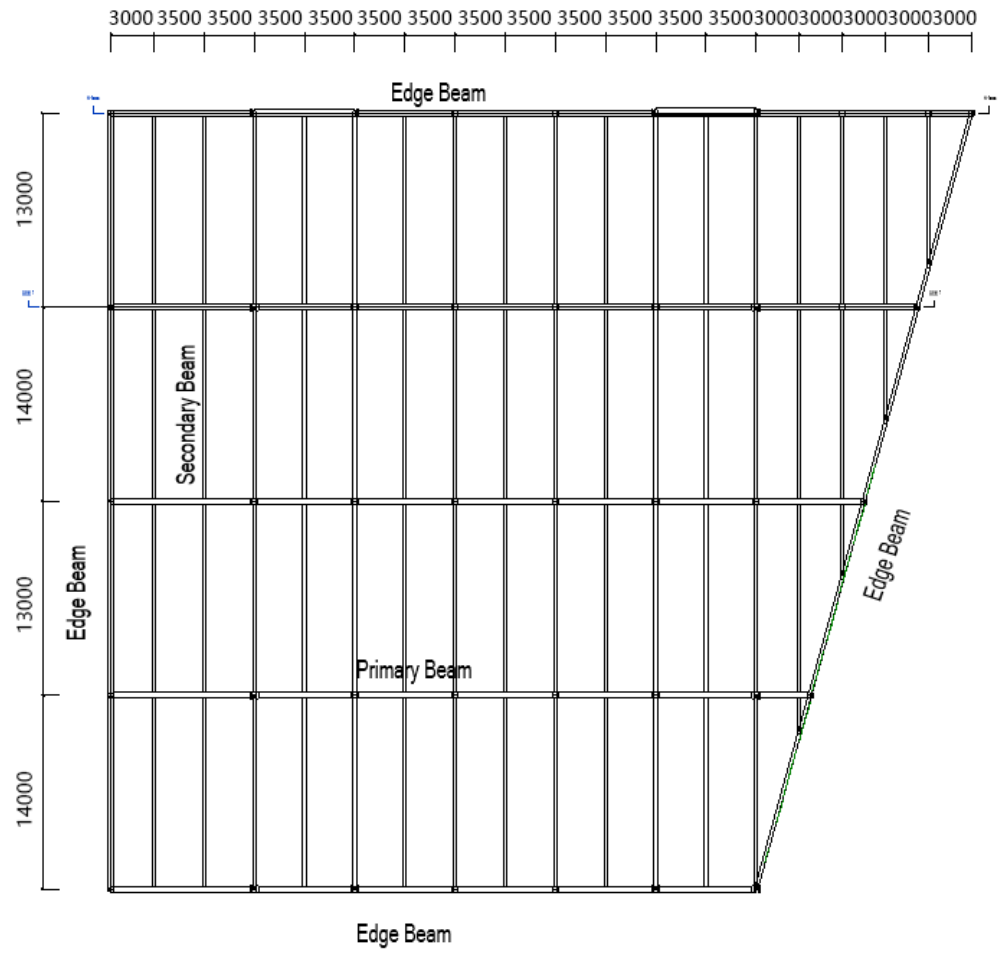


Figure 13.5 Aborted grid layout

The beam layout shown above was intended to meet the minimum span for the cantilever, therefore, the first span of the secondary beam is 14 meters, and the rest of the spans are 13m, 14m, and 13 m.

But it is a very non-uniform span layout which will lead to difficult construction and non-uniform loading distribution which may lead to additional tilting moments.

Therefore, for optimizing the design, the minimum span for the cantilever has to be sacrificed and the plan of 13.5m for each span is deployed.

13.2. Potentially better Proposals

As shown in proposal 2, there is no diagonal truss connected to the ground floor, and by analyzing the loading path, it can be found that the vertical forces transfer from a column in the cantilever region will be transferred to the first and second

floors hence to the ground by the columns.

However, if the truss system is provided as shown below, in the north face of the building, an additional x truss is provided on the ground floor and the truss is provided on the 4th floor.

For truss, it can be more consistent between the two faces, and the x truss on the ground floor can transfer the loads to the ground rather than transferred to upper floors. Therefore, the loading path is shortened and the truss system is more efficient.

Hence, it may be a better truss version and worth investigating if there is more time.

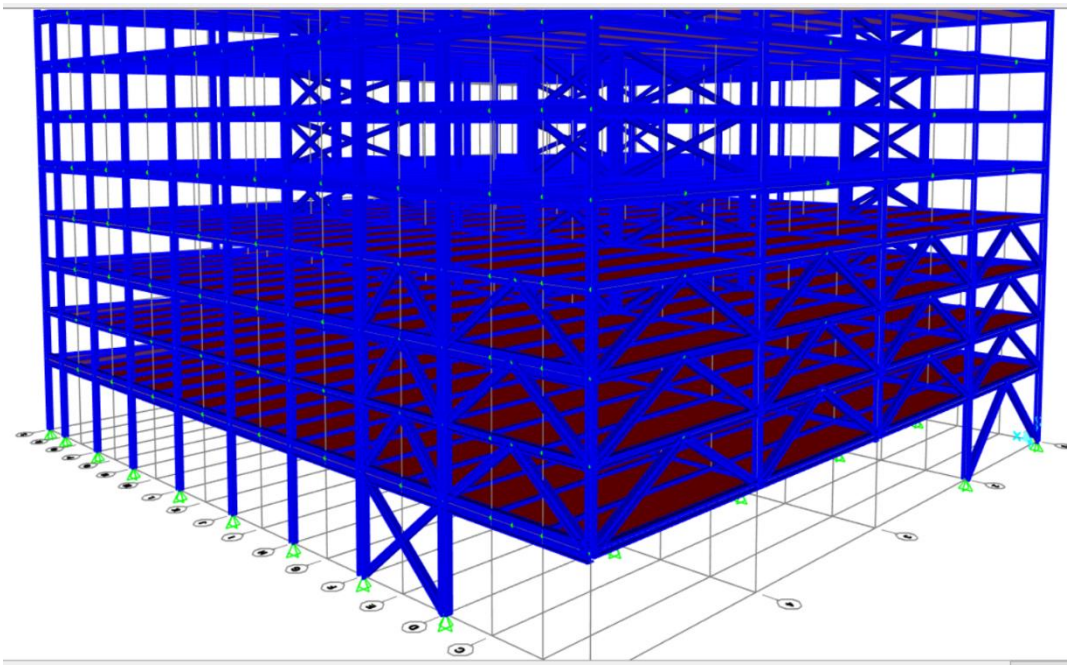


Figure 13.6 Potentially better proposal in the global view

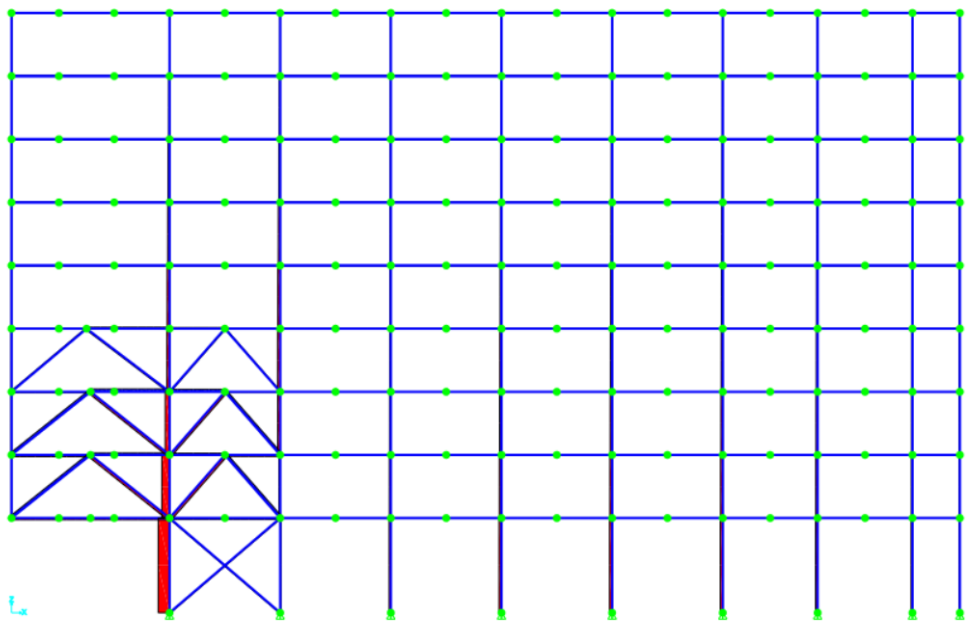


Figure 13.7 Potentially better proposal in the north to west view

In addition, continuous truss members running along between the first floor to the upper floors can also be considered in the future. This is because the loads can be better transferred along the diagonal truss members to the adjacent columns by forming a continuous load path. However, the effective length of the truss members may need to be better controlled by adding some lateral restraints along the members to avoid buckling.

14. Conclusion

In this project, many past cases for cantilever steel buildings have been investigated. Based on the theoretical and practical knowledge in Steel Building design, many possible arrangements for truss and bracing systems, columns, and beams were studied. Computational and hand calculations were carried out with the design. In the early stage, structural modeling software such as SAP2000 was used to analyze many possible proposals for the steel building structure under the requirement of minimum spacing of primary and secondary beams, vertical, and lateral loadings, efficient loading paths to name only a few.

Furthermore, once a variety of choices of possible proposals were narrowed (obsoleted most of the proposals by computational modeling), detailed handling calculation and structural modeling were carried out based on the Eurocode and UK National Annex.

Comprehensive, meticulous, and rules-based calculations for composite slabs, composite beams and columns were investigated. Moreover, for a more efficient cantilever and bracing system for resisting vertical and lateral loadings, the different cross-sections of members and distinct loading paths truss were arranged in the system. Connections resistance and construction feasibility were also considered, and detailed calculations were provided.

Last but not least, even though complete and detailed calculations and drawings have included in this report, many possible proposals and improvements in more efficient loading paths system, more economical cross-sections selection and arrangement, which are also worth investigating in the future.

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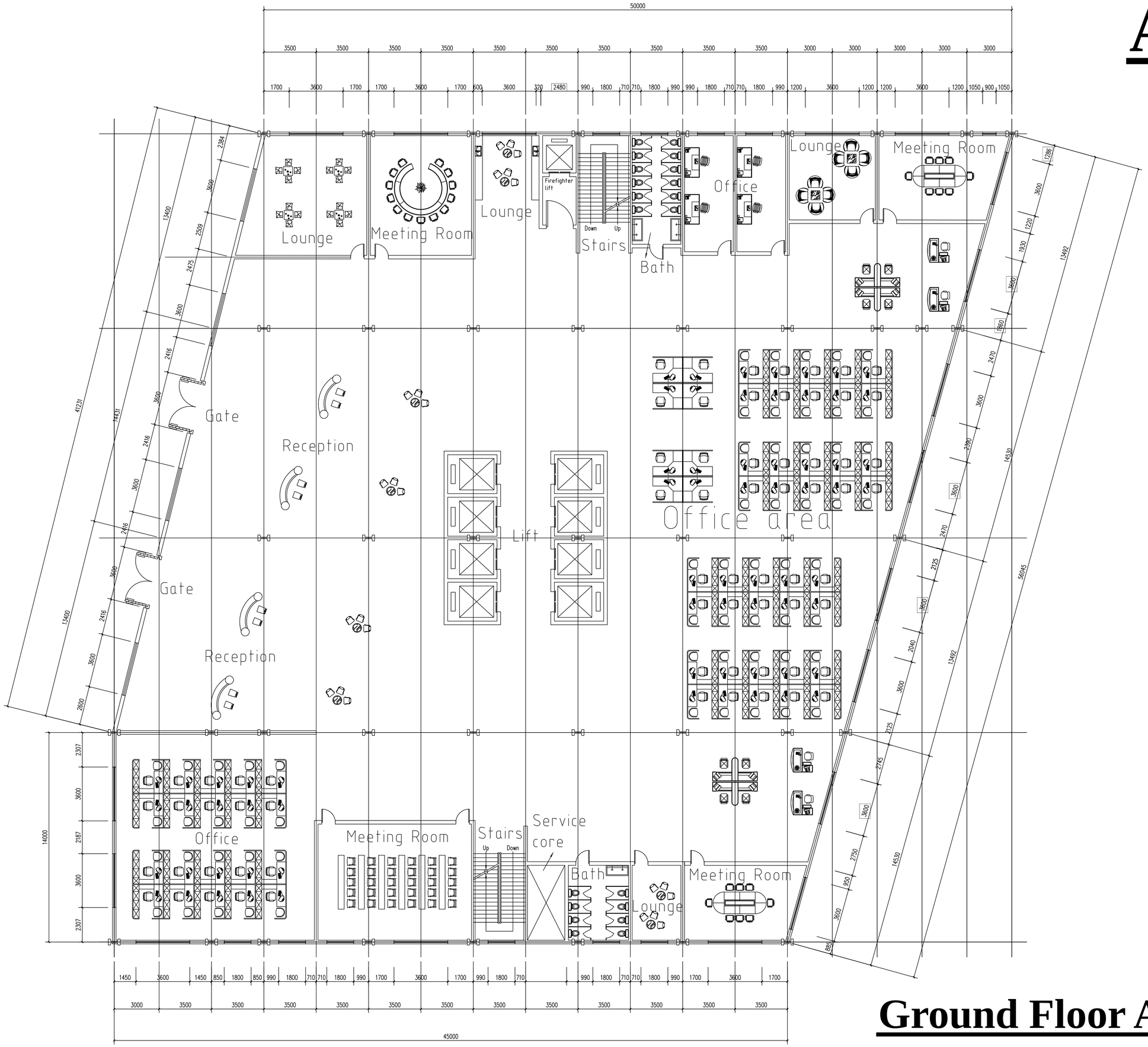
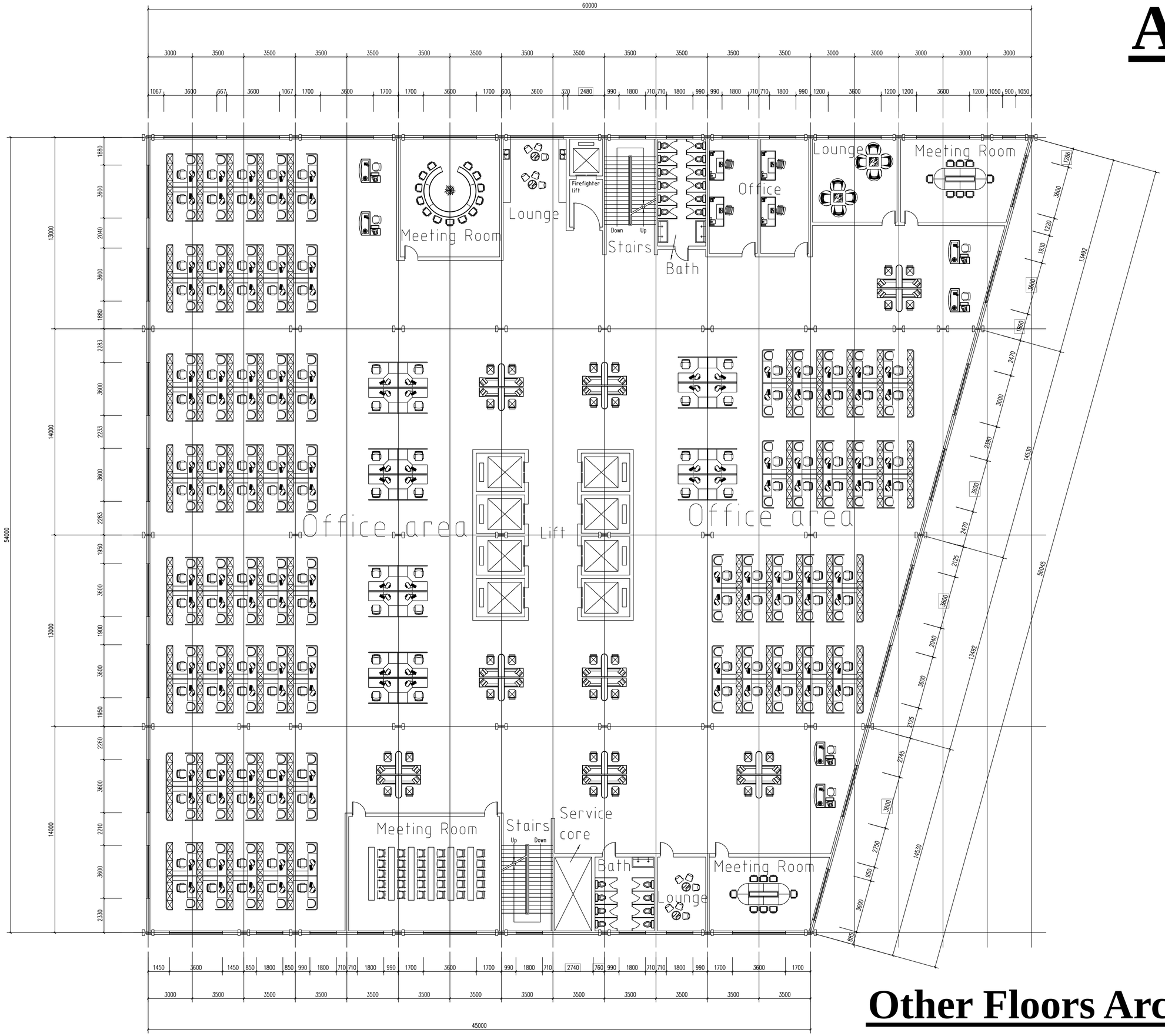
Appendix A1 (Page A1-1)



Ground Floor Architectural Layout

Appendix A1 (Page A1-2)



Other Floors Architectural Layout

Appendix A2

Drawings of Structural General Layout, Plan and Elevation Views

For Proposal 1

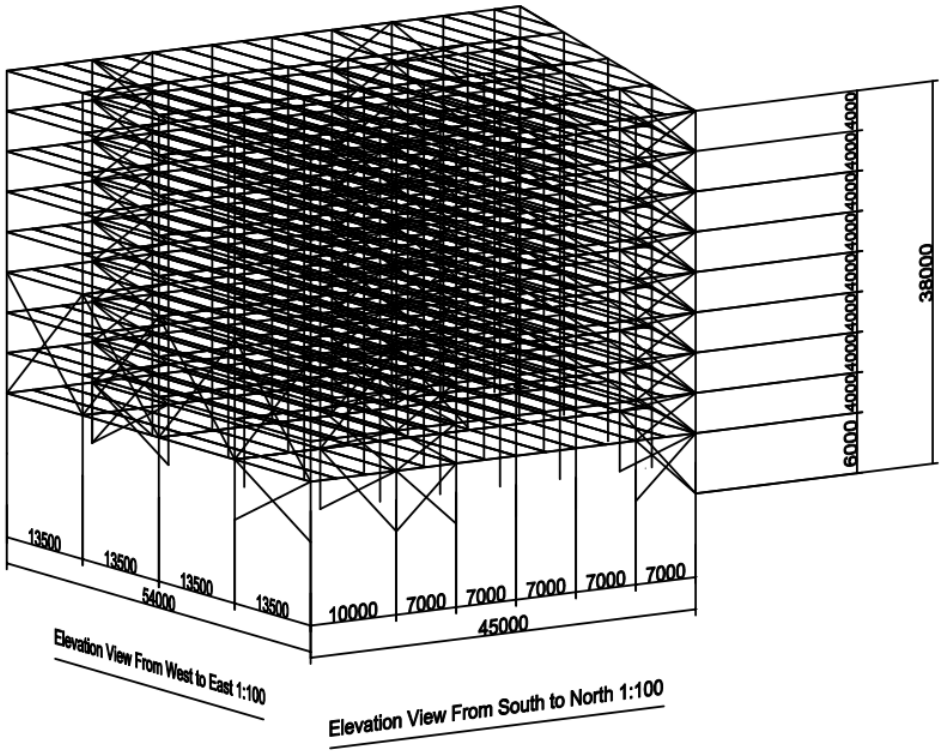


Figure A2.1 An overall View for Proposal one with dimensions

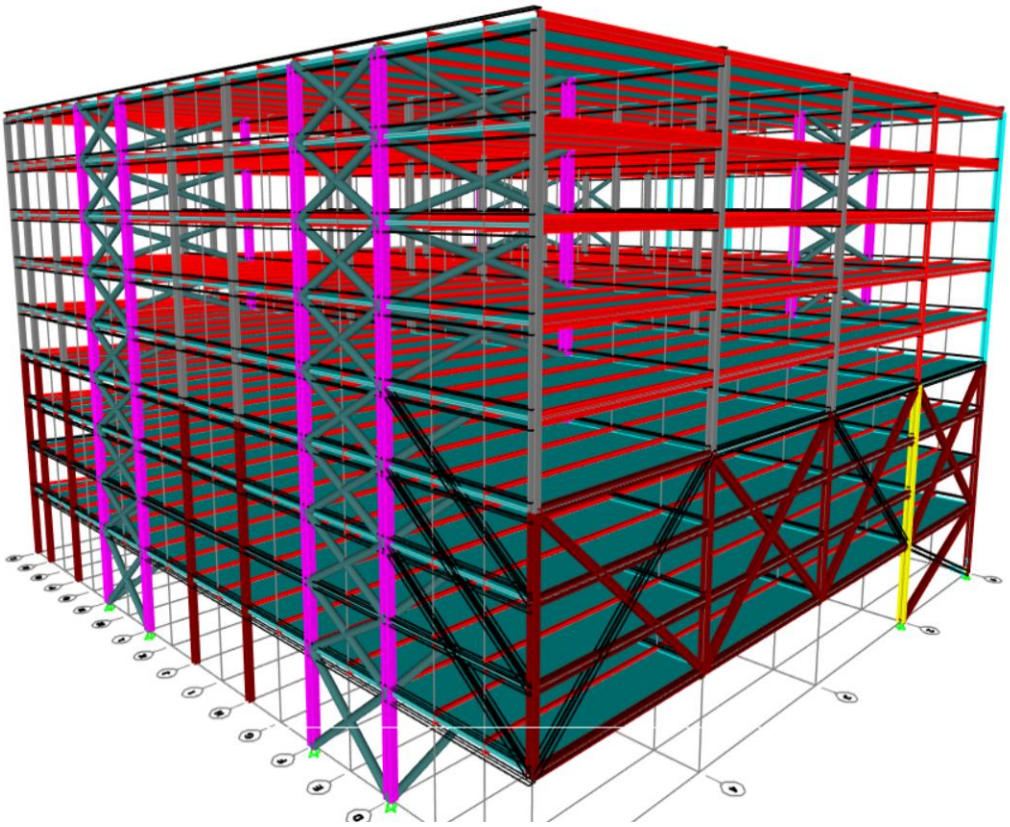


Figure A2.2 Global Structural Model for Proposal 1 (SAP2000)

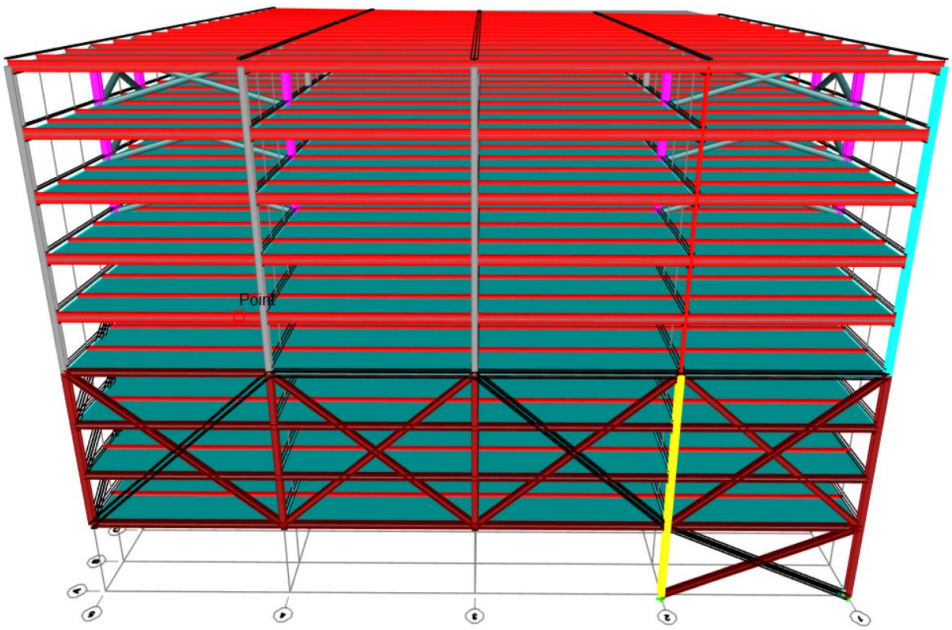


Figure A2.3 Elevation Structural Model for Proposal 1 (SAP2000)

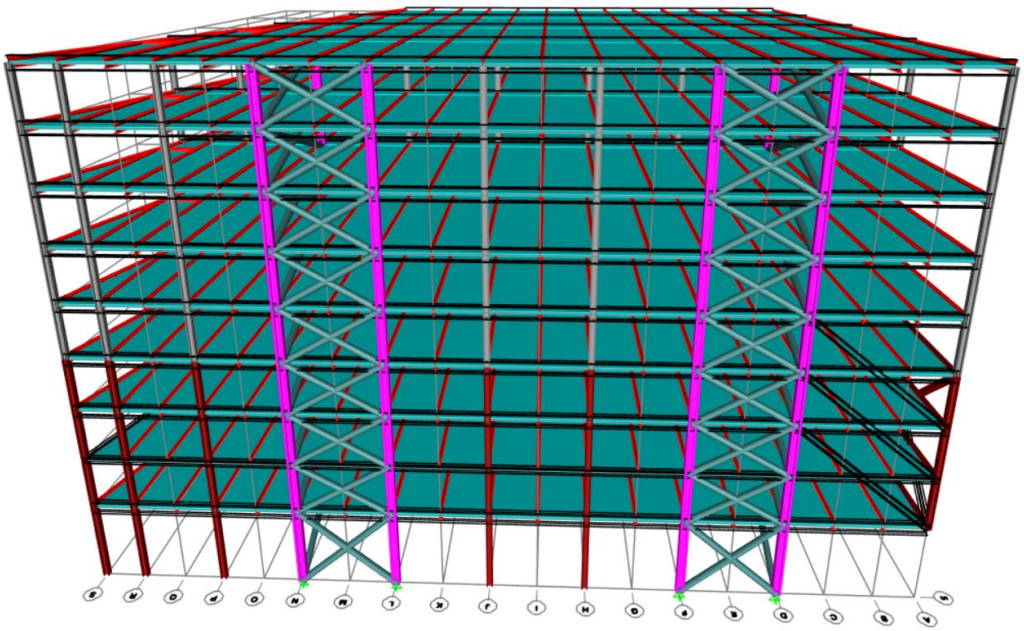


Figure A2.4 Elevation View from West to East for Proposal 1 (SAP2000)

For Proposal 2

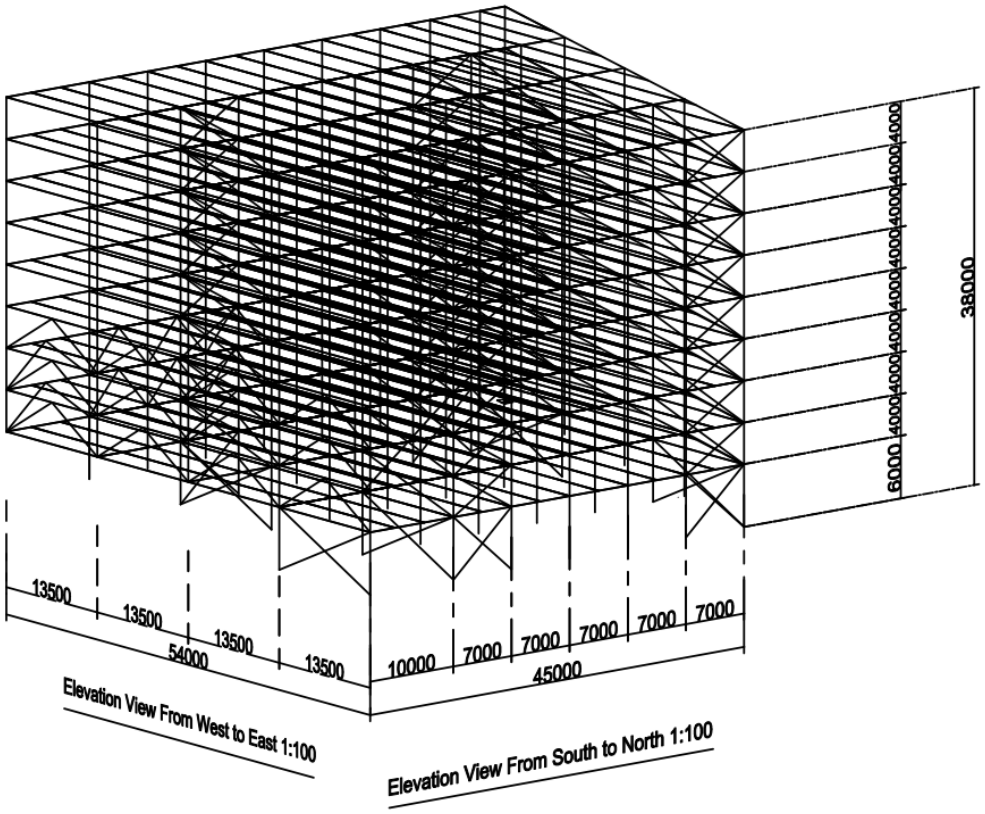


Figure A2.5 Global Structural Model for Proposal 2 (Autodesk)

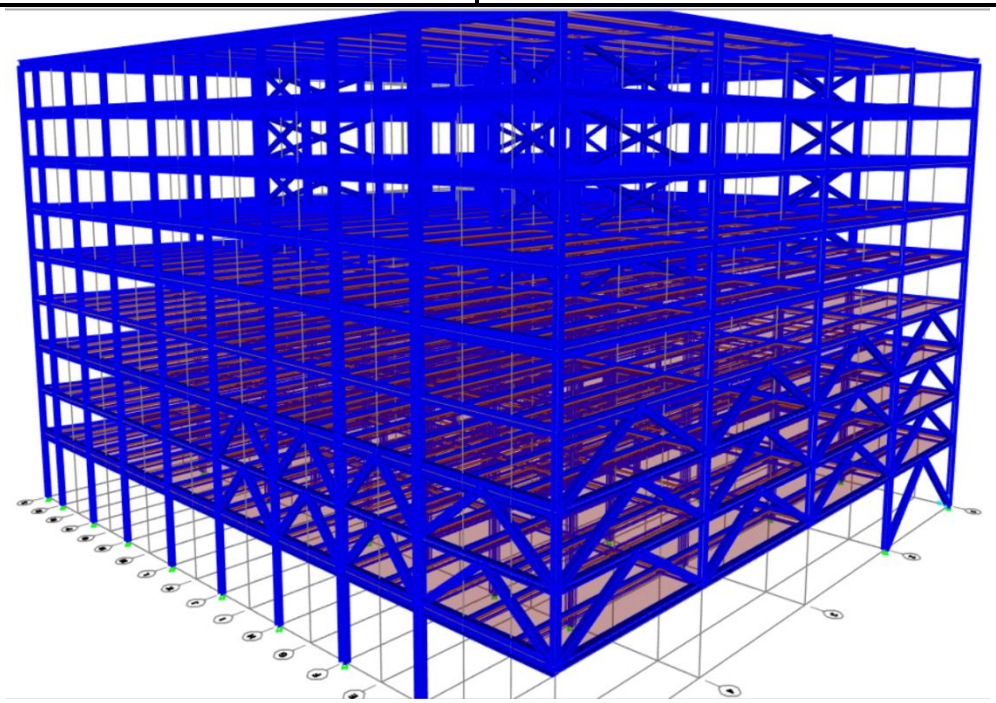


Figure A2.6 Global Structural Model for Proposal 2 (SAP2000)

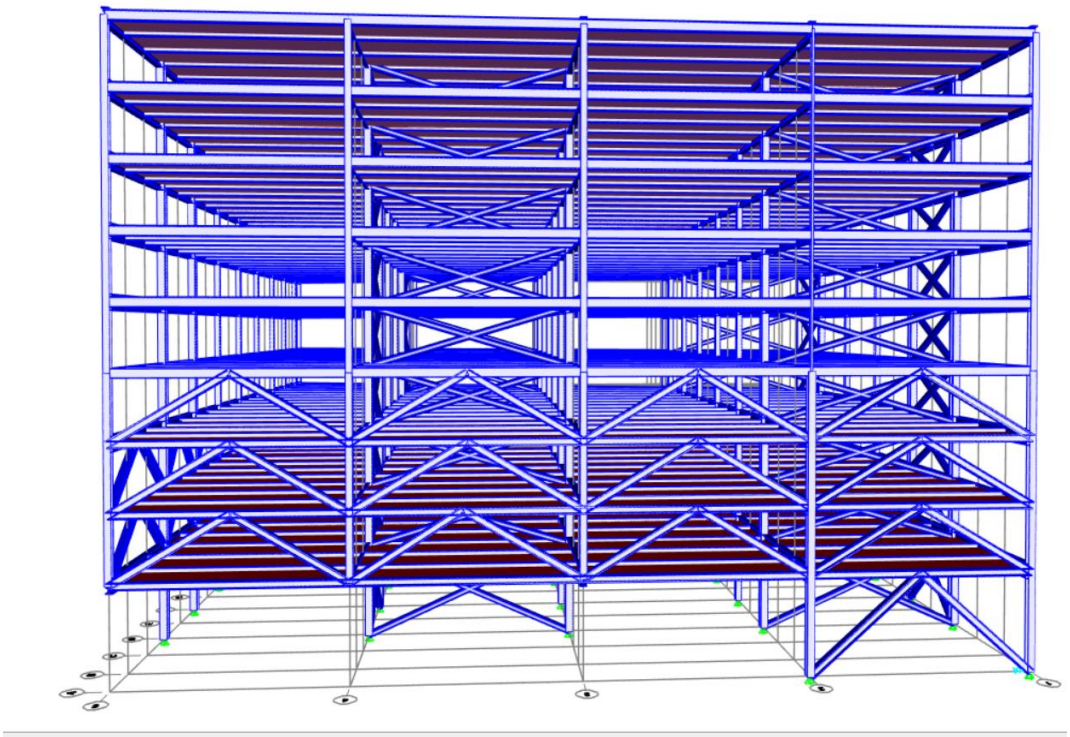


Figure A2.7 Elevation Structural Model for Proposal 2 (SAP2000)

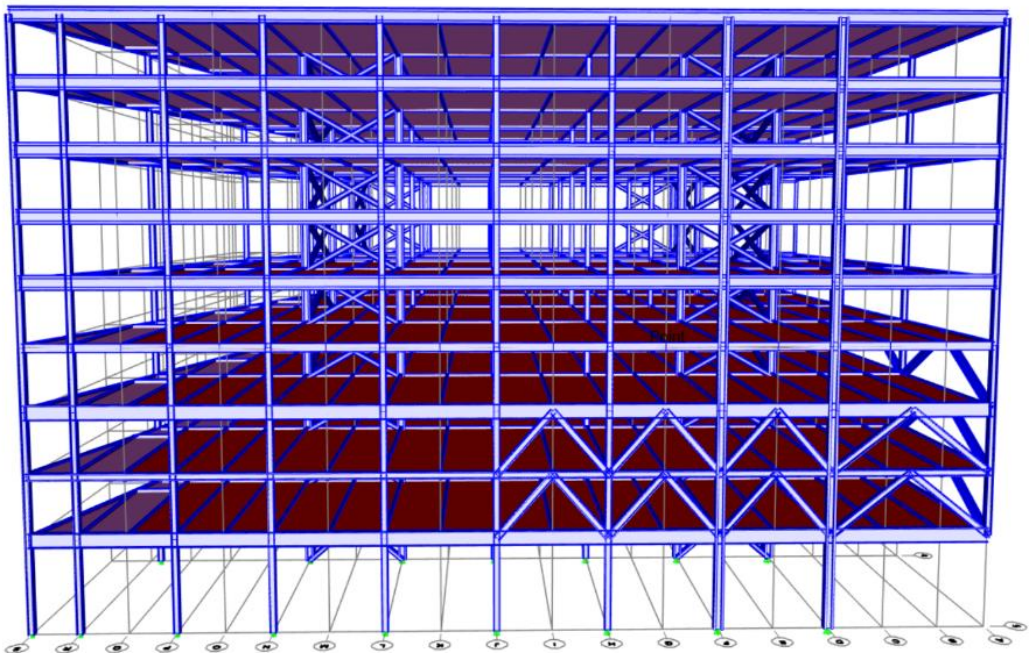


Figure A2.8 Elevation View from West to East (SAP2000)

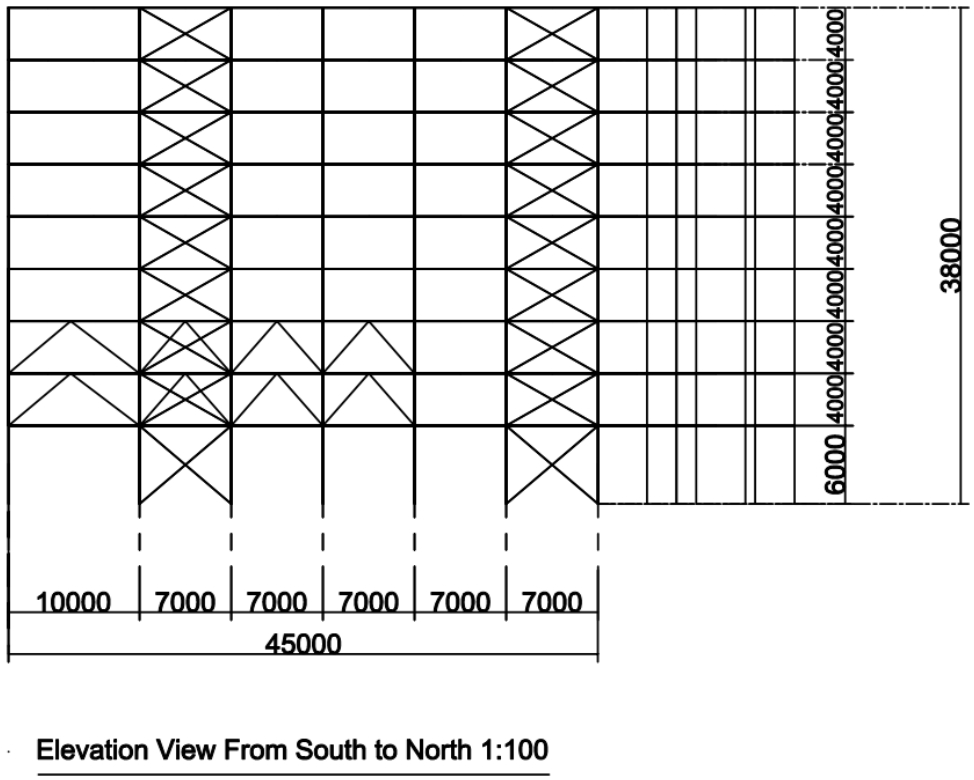
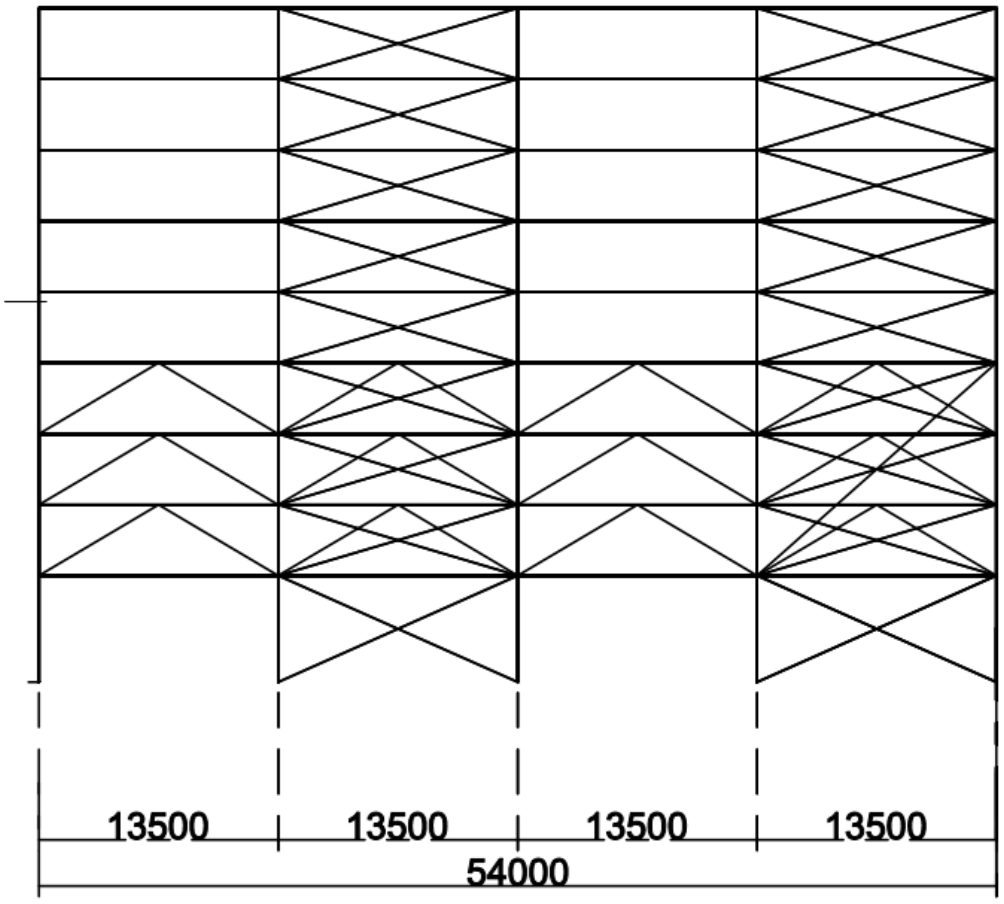


Figure A2.9 Elevation View from South to North 1:100



Elevation View From West to East 1:100

Figure A2.10 Elevation View from West to East 1:100

Appendix A3

Drawing of Structural Stability System

For Proposal 1

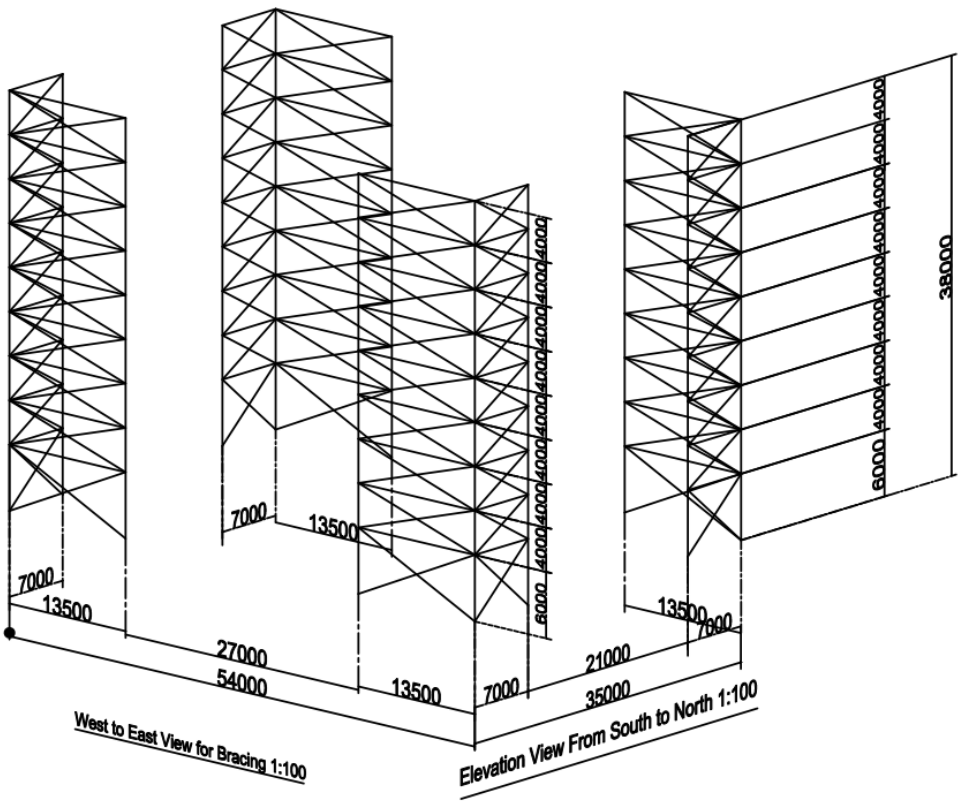


Figure A3.1 3D View for Global Bracing System in Proposal 1 (Autodesk CAD)

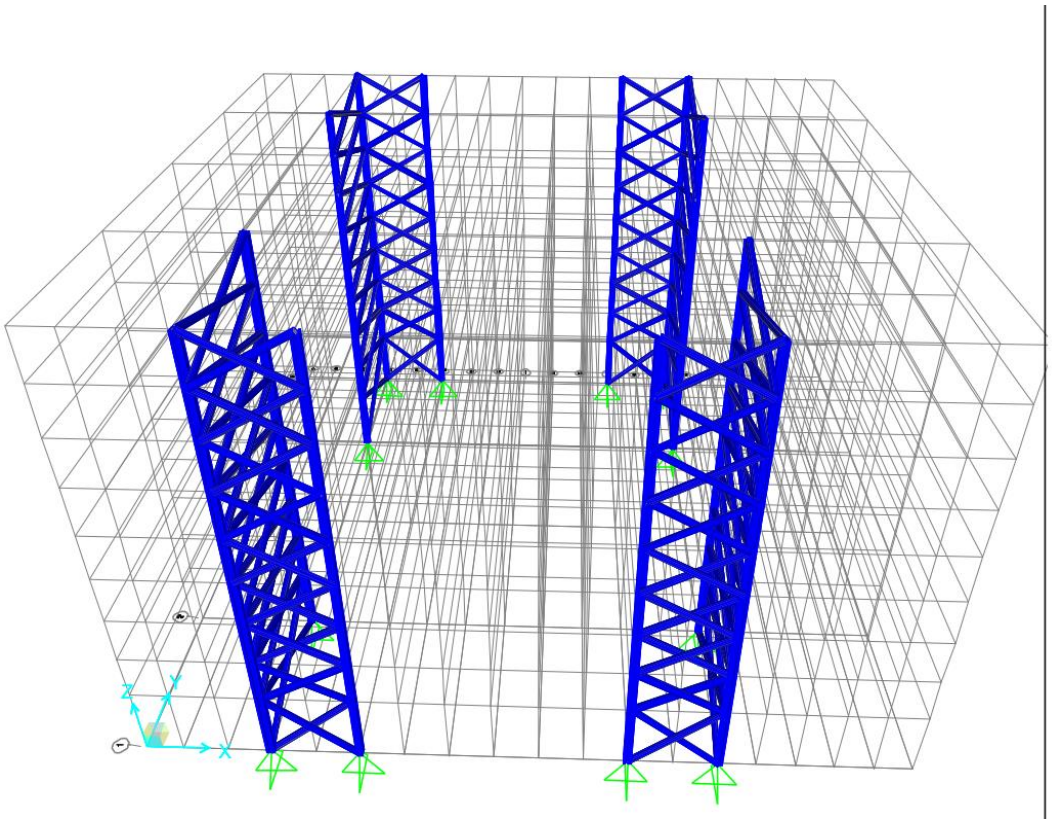


Figure A3.2 Global layout of the bracing system (in SAP2000)

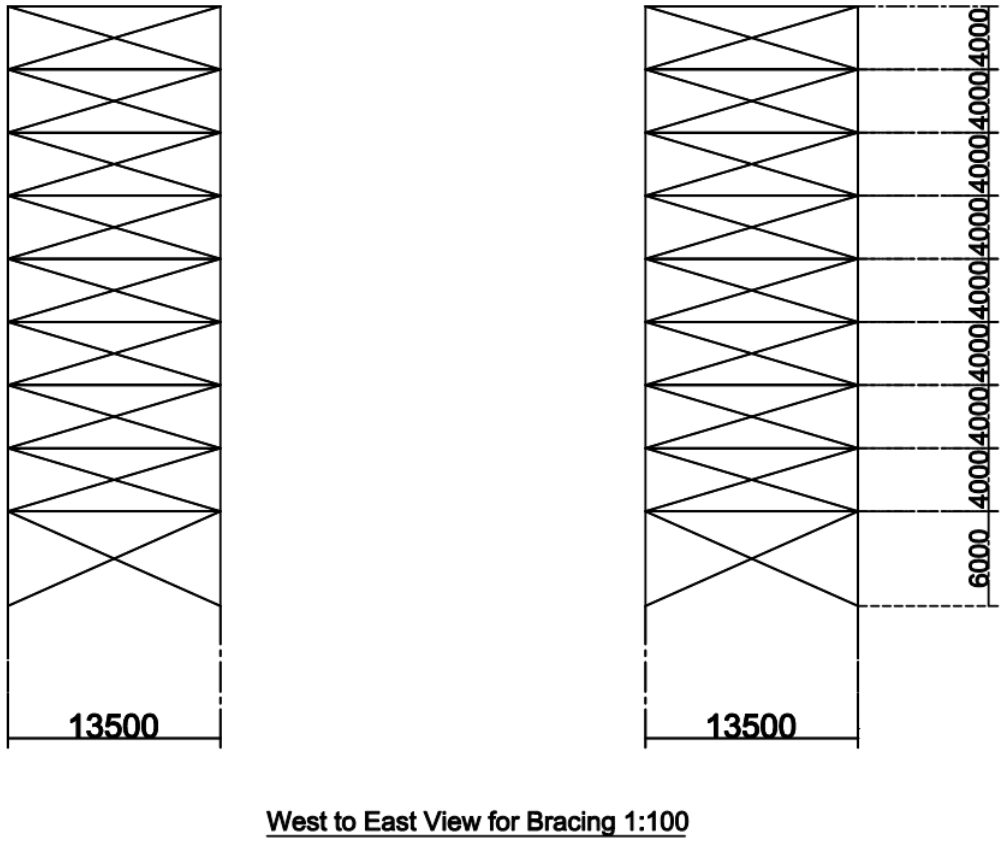


Figure A3.3 Elevation View for Bracing from West to East (Autodesk CAD)

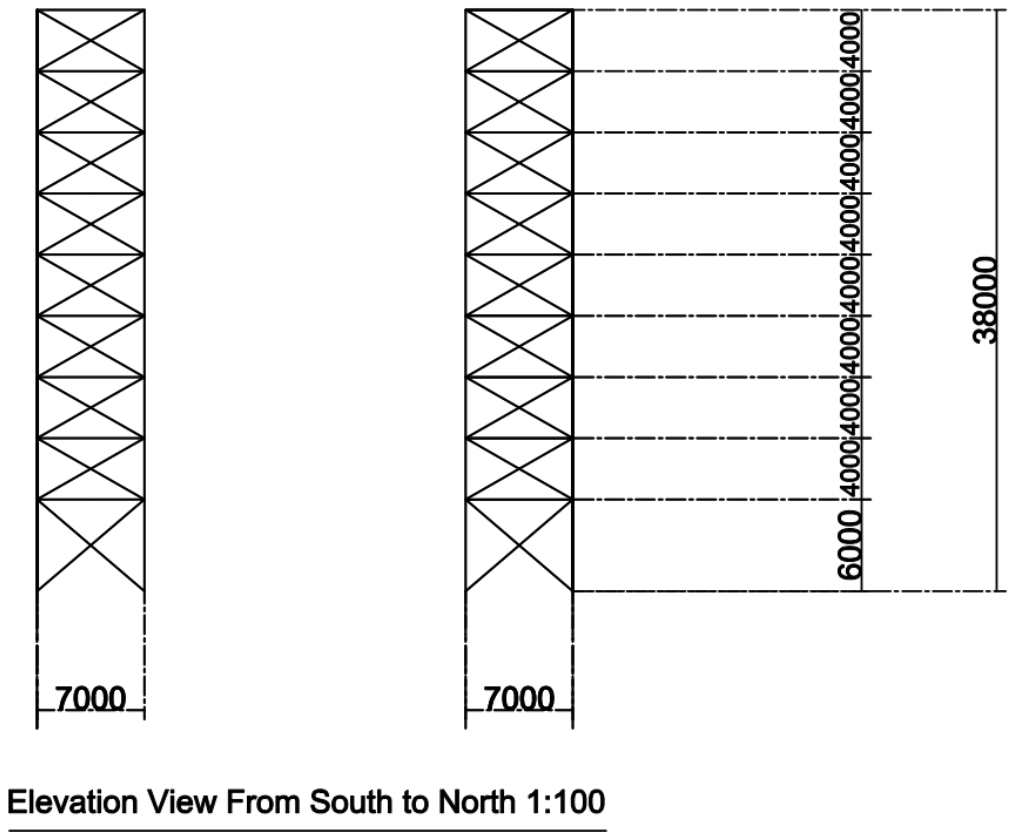


Figure A3.4 Elevation View for Bracing from South to North (Autodesk CAD)

For Proposal 2

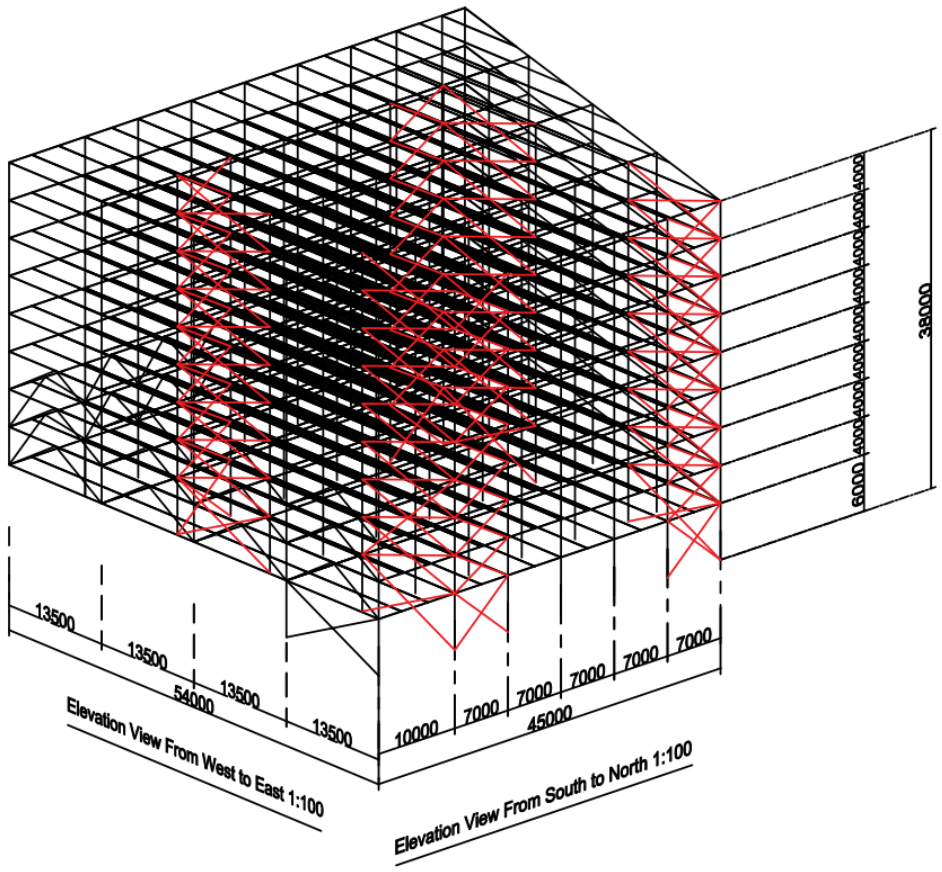


Figure A3.5 Global View for Bracing in the Structure (Highlighted Stability System)

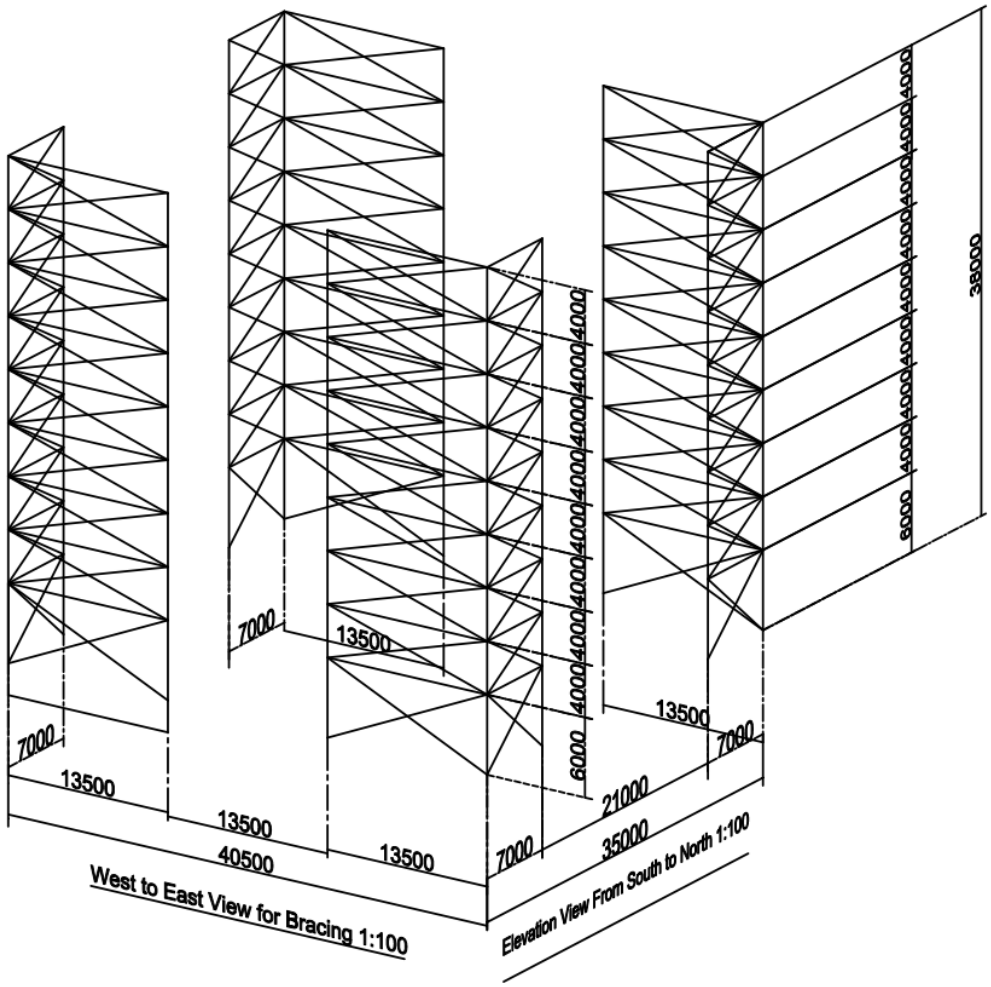


Figure A3.6 Overview for isolated bracing system

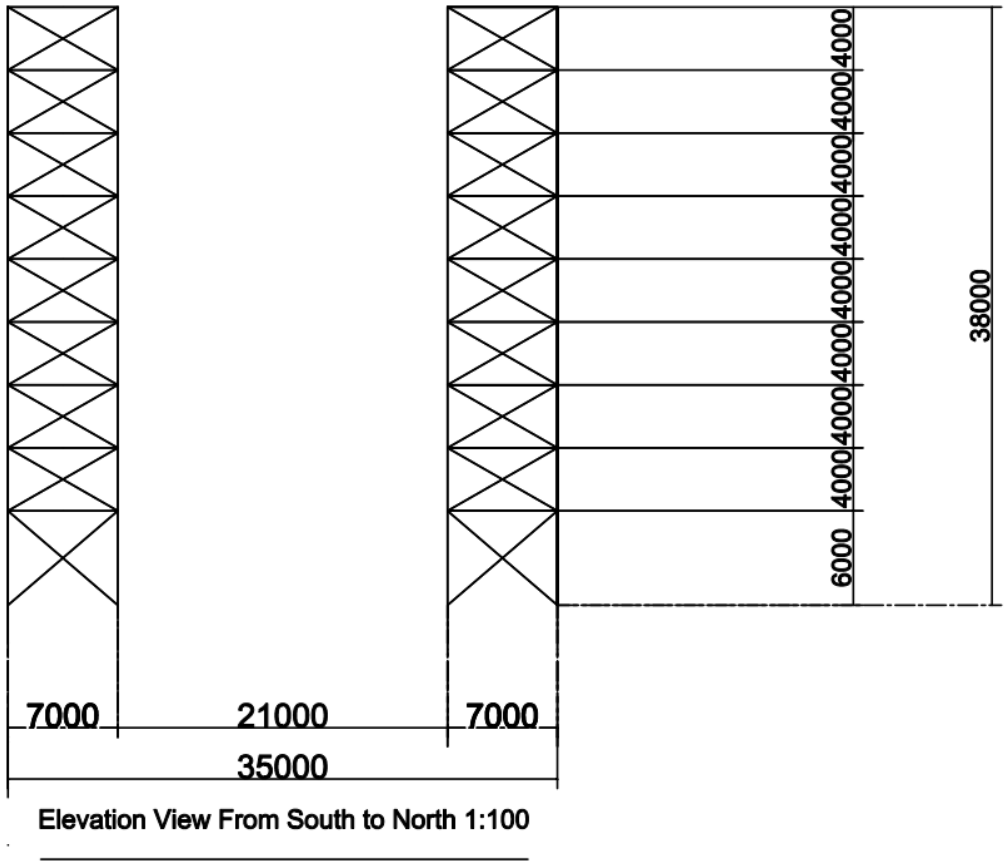


Figure A3.7 Elevation View from South to North for Bracing System

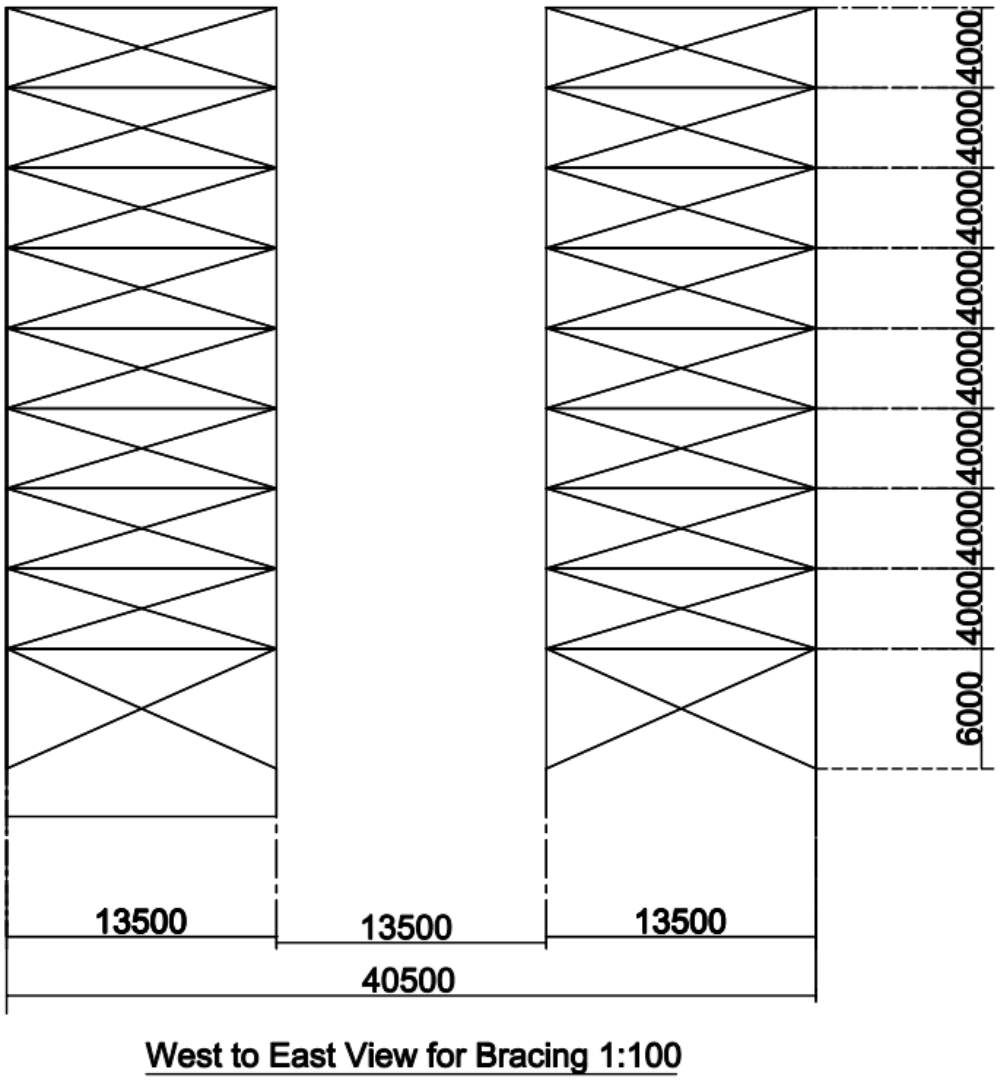


Figure A3.8 Elevation View from West to East for Bracing System

Appendix A4

Drawing of Typical Plan View with Critical Structural Floor Zone and Columns Layout (Structural)

Typical Plan View and Critical Structural Floor Zone in Proposal 1 and 2

Table A4.1 Cross-Section selection and represented colors in SAP2000

Location of the Columns	Cross-section Selection (UKB)	Represented Colors in SAP2000
Primary Beam from level 1 to Level Roof	533×210×122	
Secondary Beam from level 1 to Level Roof	533×210×122	
Edge Beam from level 5 to Level Roof	533×210×122	
Edge Beam from level 1 to Level 4	Depends on Truss	

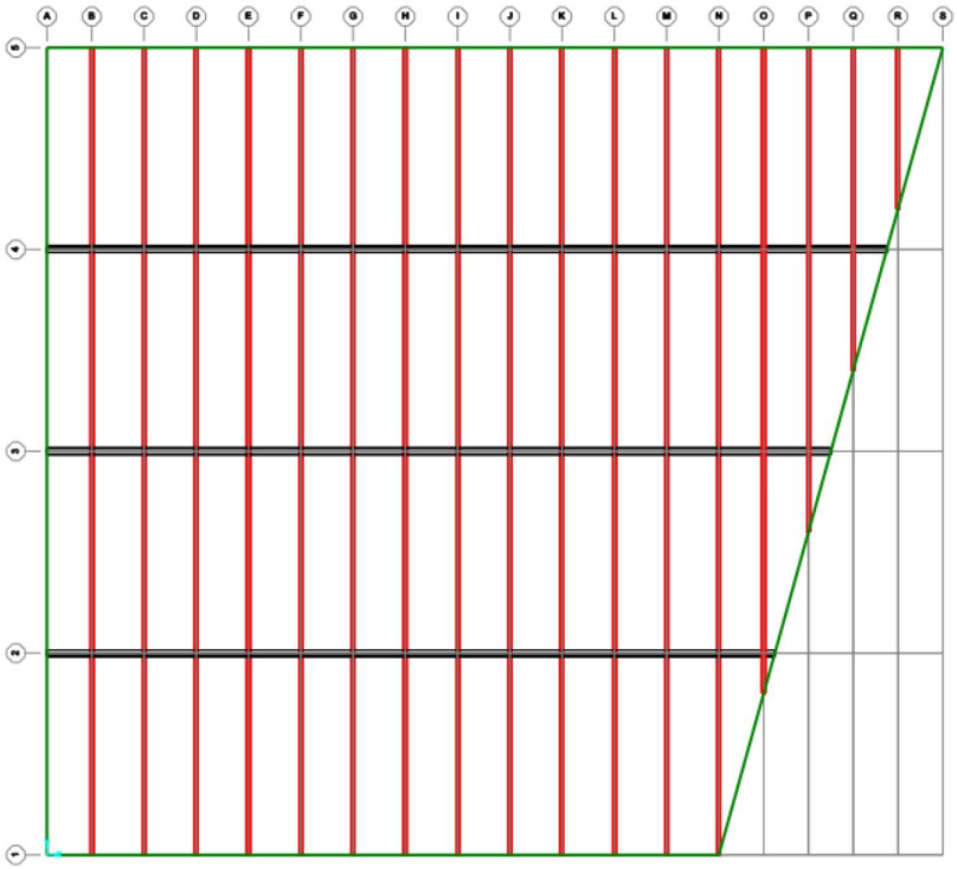


Figure A4.1 Primary and Secondary Beam Arrangement

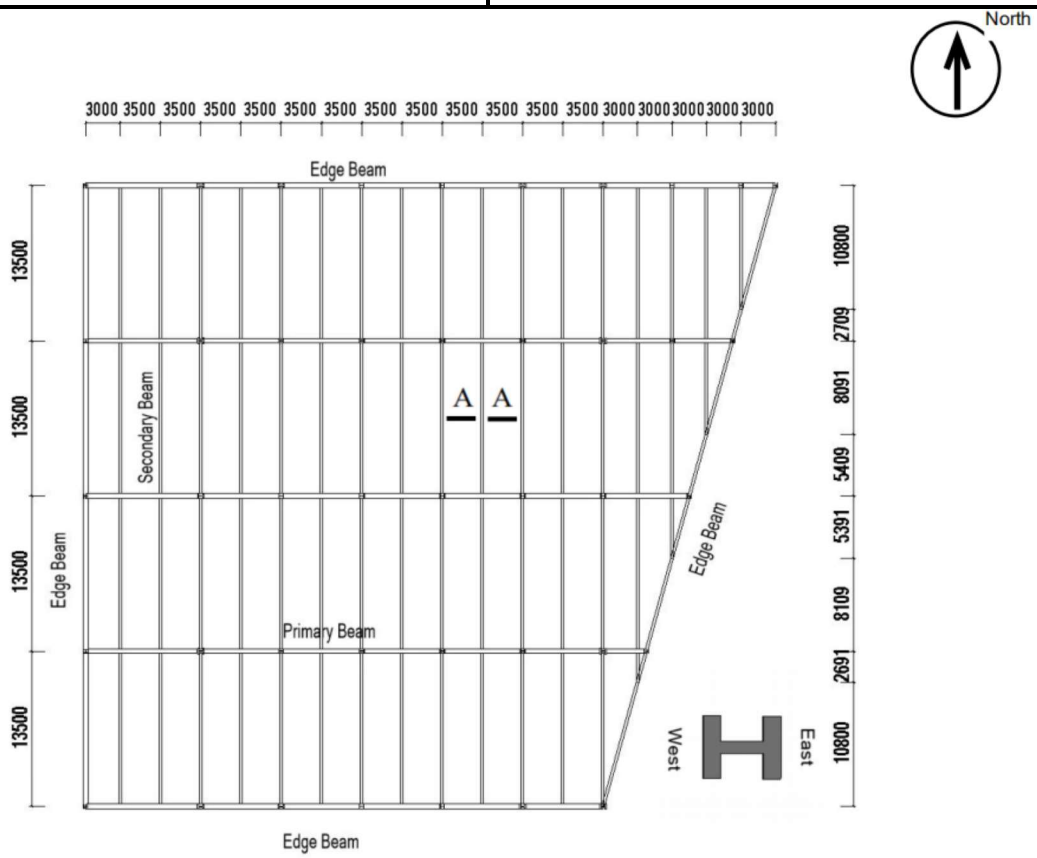


Figure A4.2 Beam Arrangement with critical section (Revit)

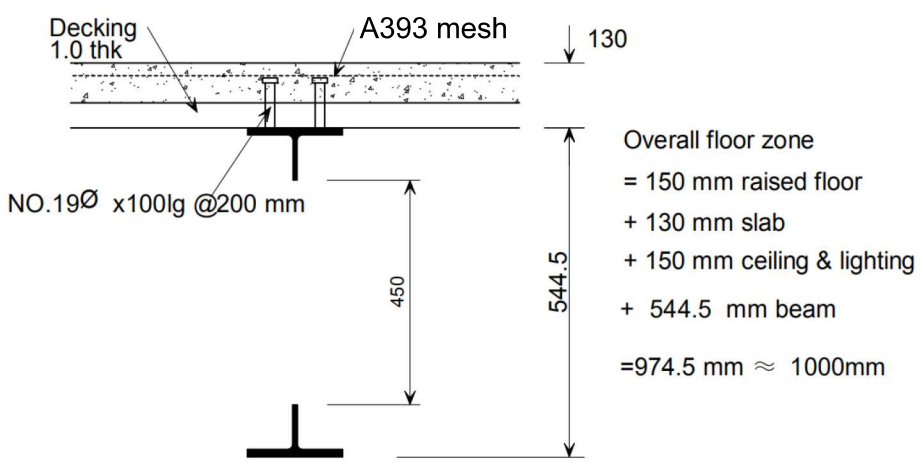


Figure A4.3 Details with critical section A-A

Columns in Proposal 1

Table A4.2 Column Cross-section

Location of the Columns	Cross-section Selection (UKC)	Represented Colors in SAP2000
Typical Internal Columns from 4th Floor to Roof	356×368×202	
Typical internal Columns from 4th Floor to Ground	356×406x467	
Typical Peripheral Columns from 4th Floor to Roof	356×368× 202	
Typical Peripheral Columns from 4th Floor to Ground	356×406×235	
Columns for Wind Bracing Bays from Roof to Ground	356*406*1202	
Columns near the cantilever region from 4th Floor to Ground	356×406×634	

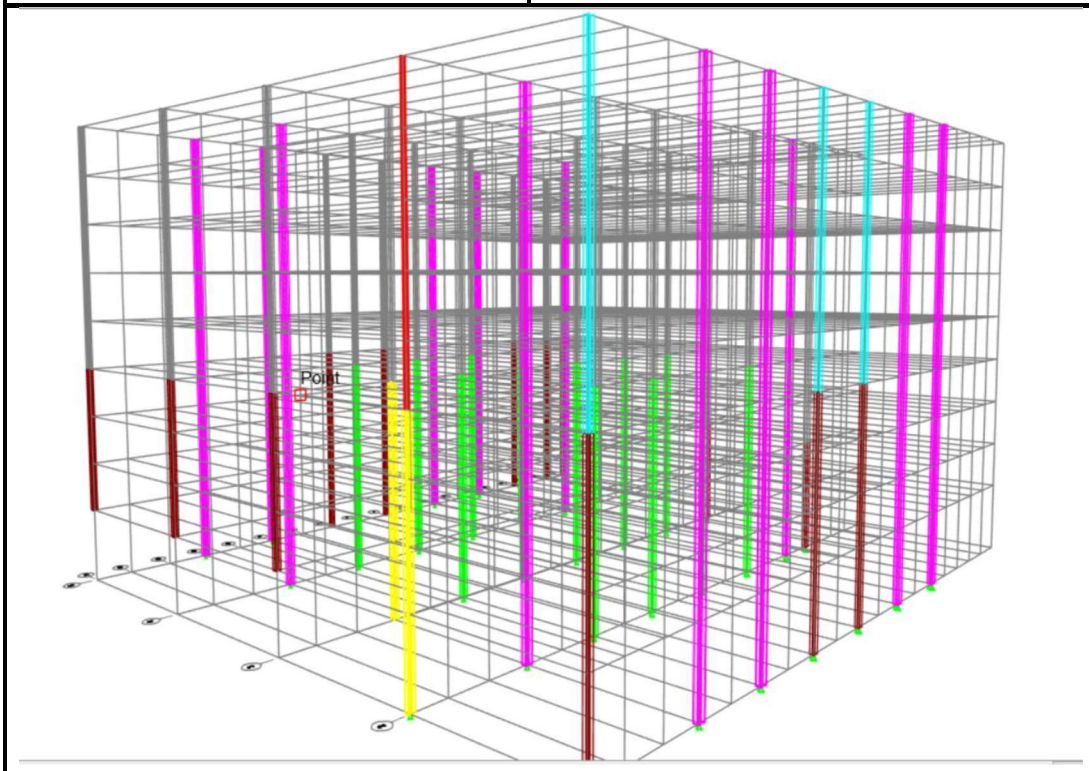


Figure A4.4 An overview for the layout of Columns (SAP2000)

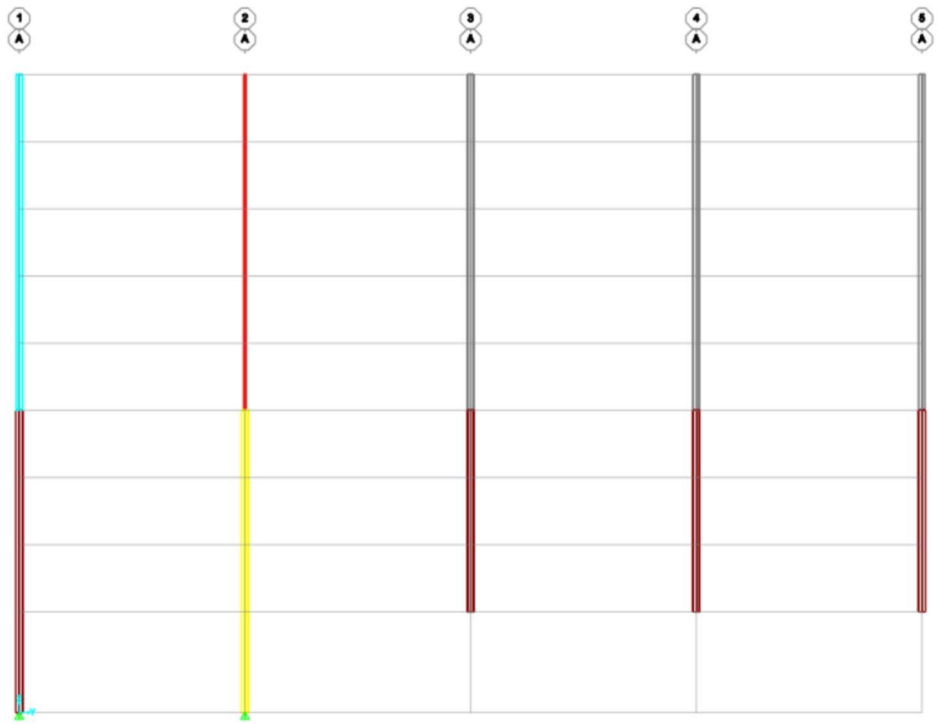


Figure A4.5 Column cross-section layout in the face A (SAP2000)

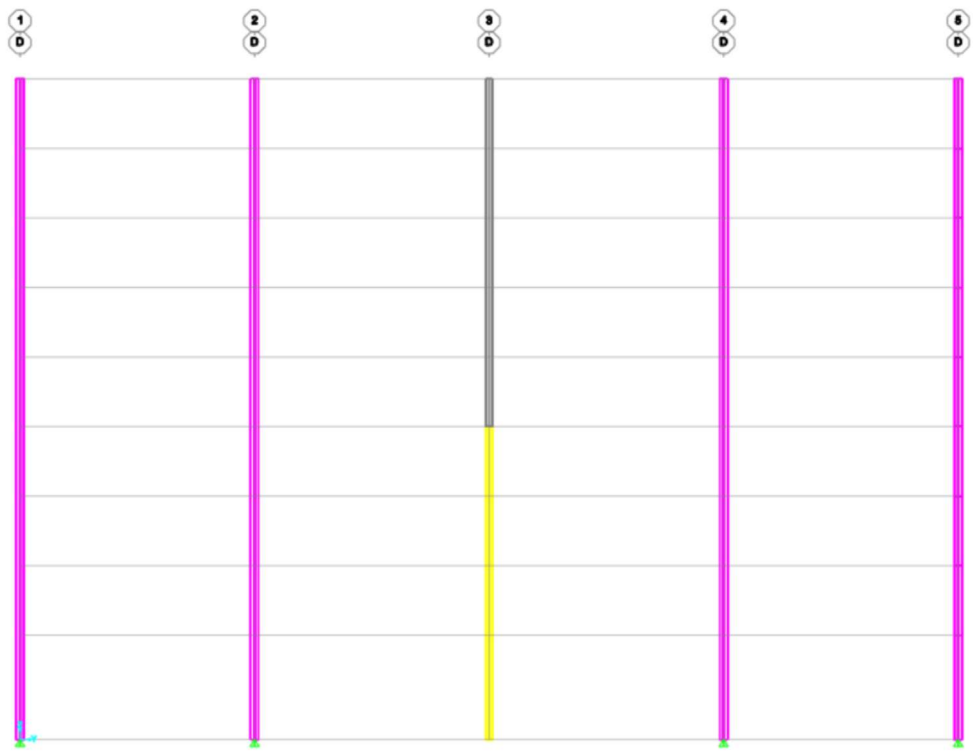


Figure A4.6 Column cross-section layout in the face D (SAP2000)

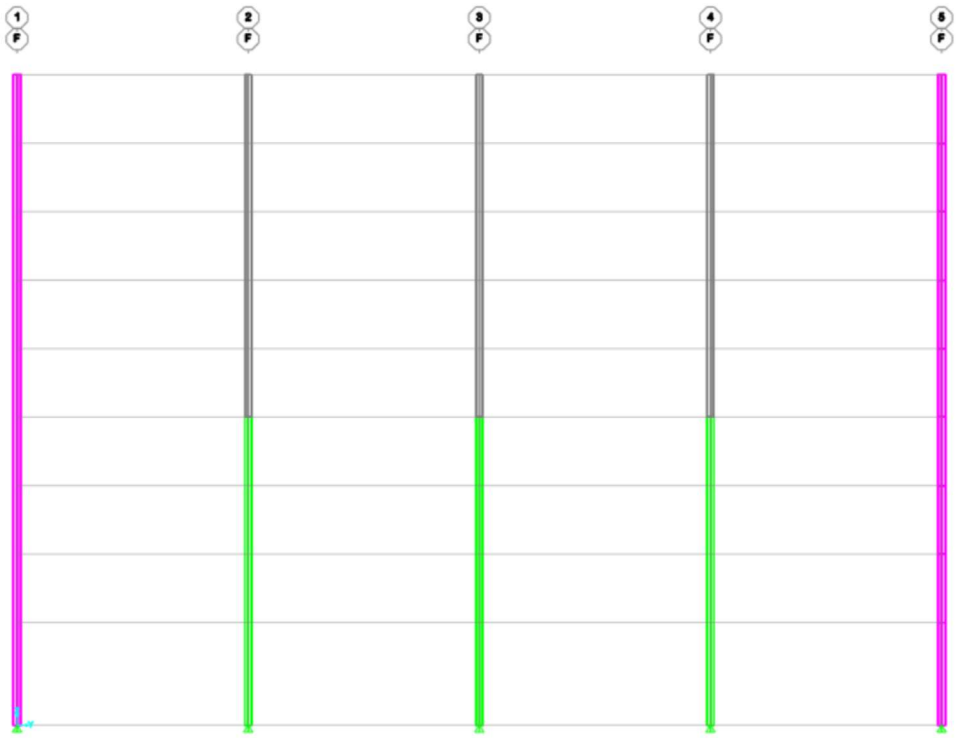


Figure A4.7 Column cross-section layout in the face F (SAP2000)

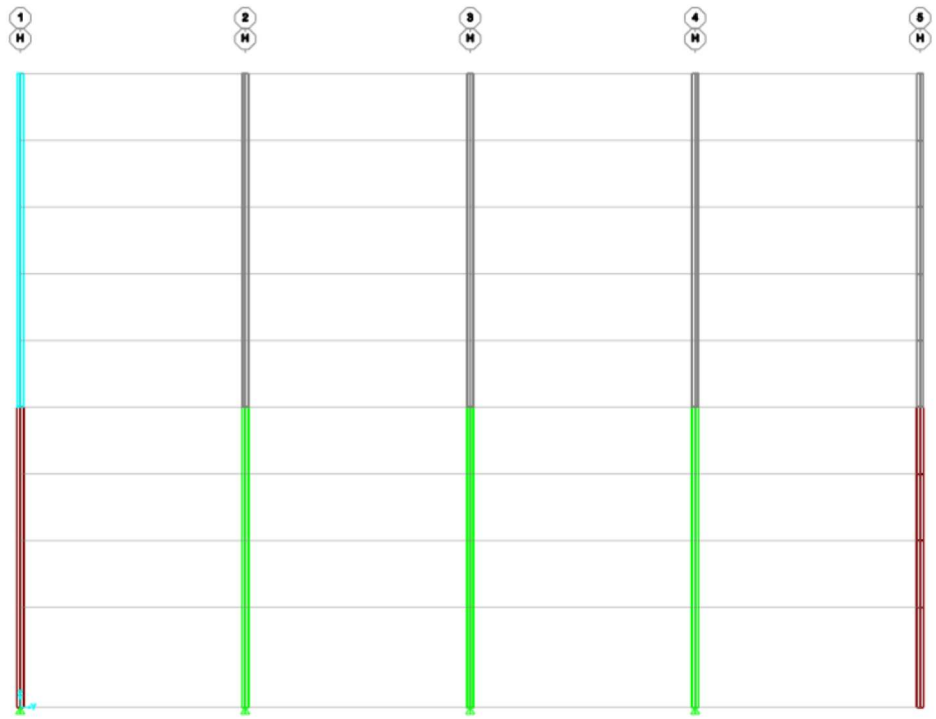


Figure A4.8 Column cross-section layout in the face H (SAP2000)

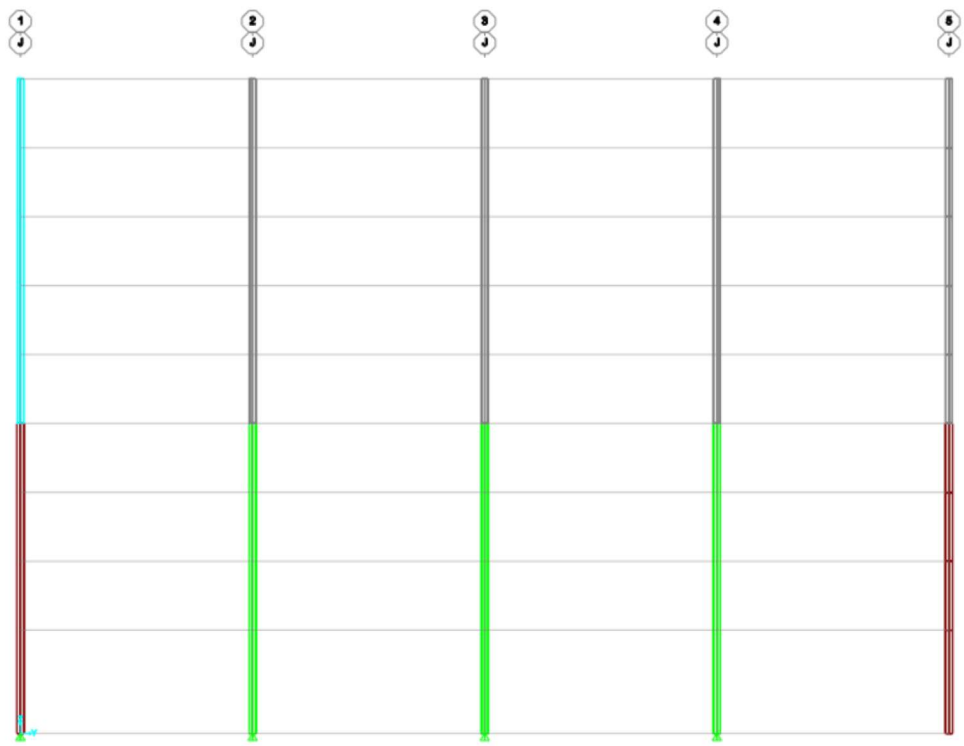


Figure A4.9 Column cross-section layout in the face J (SAP2000)

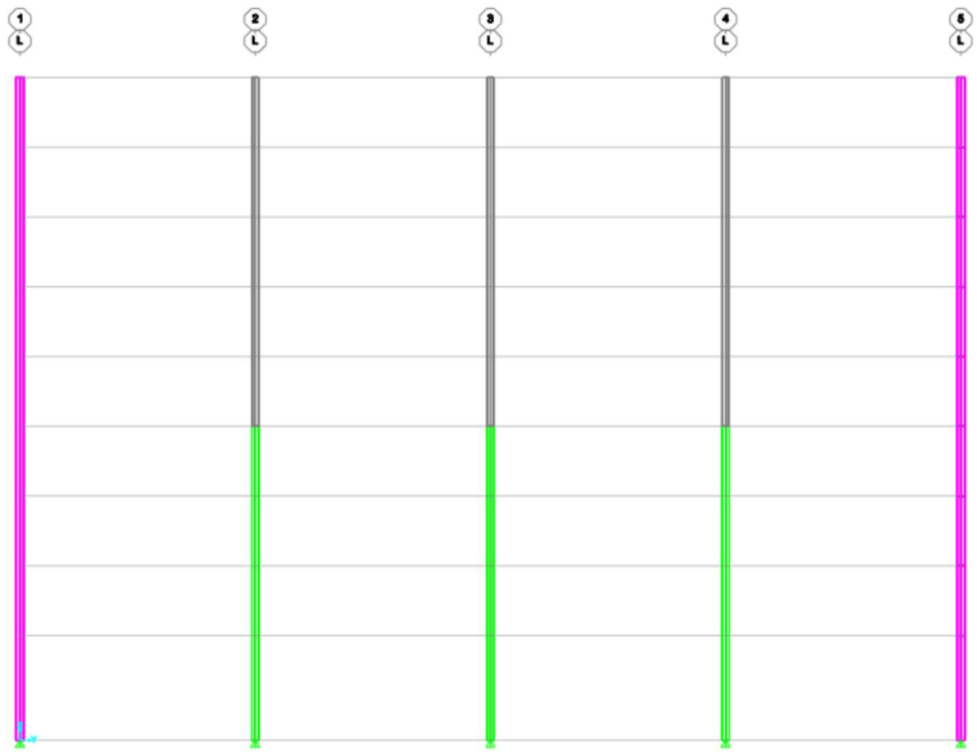


Figure A4.10 Column cross-section layout in the face L (SAP2000)

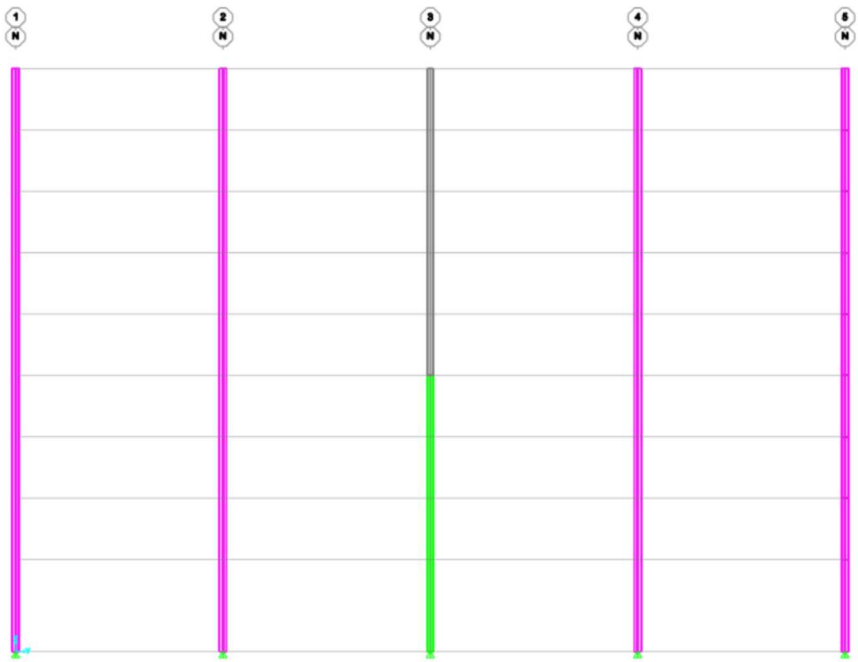


Figure A4.11 Column cross-section layout in the face H (SAP2000)

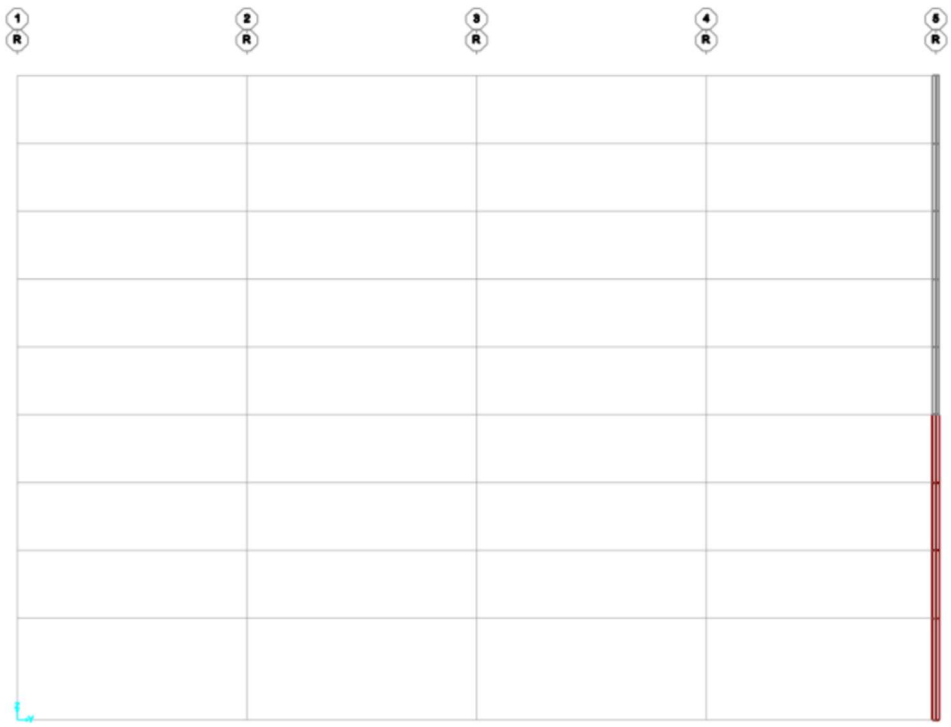


Figure A4.12 Column cross-section layout in the face R (SAP2000)

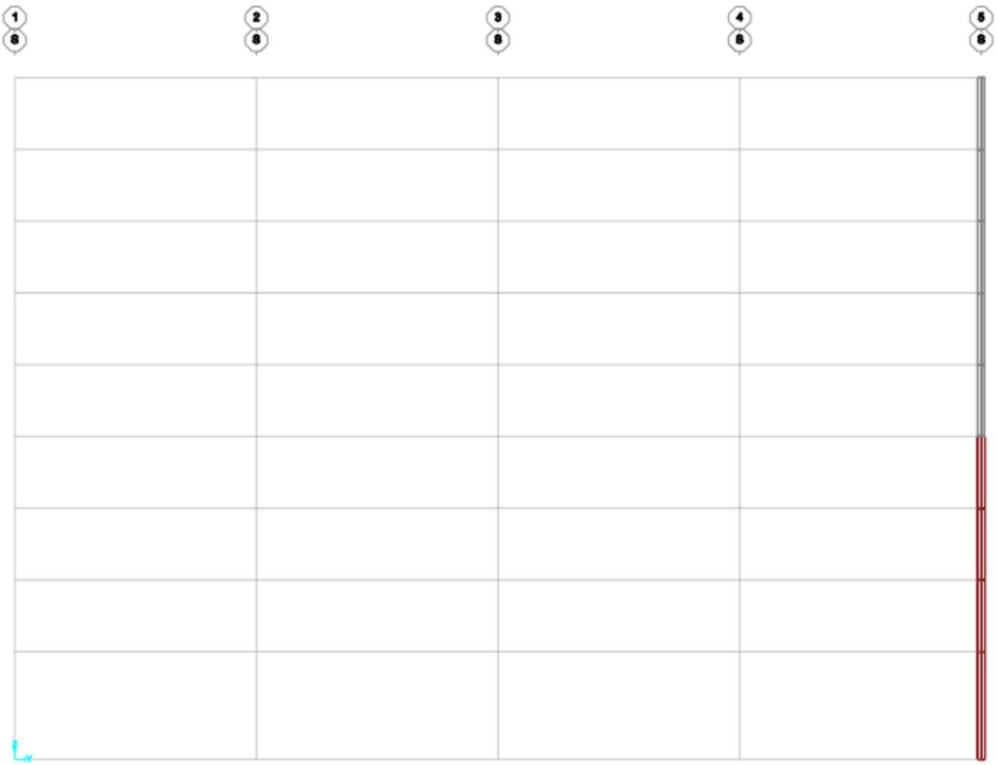


Figure A4.13 Column cross-section layout in the face S (SAP2000)

For Columns in Proposal 2

Table A4.3 Cross-section properties in SAP2000

Location of the Columns	Cross-section Selection (UKC)	Represented Colors in SAP2000
Typical Internal Columns from 4th Floor to Roof	356×368×202	
Typical internal Columns from 4th Floor to Ground	356×406x467	
Typical Peripheral Columns from 4th Floor to Roof	356×368× 202	
Typical Peripheral Columns from 4th Floor to Ground	356×406×235	
Columns for Wind Bracing Bays from Roof to Ground	356*406*1202	
Columns near the cantilever region from 4th Floor to Ground	356×406×634	

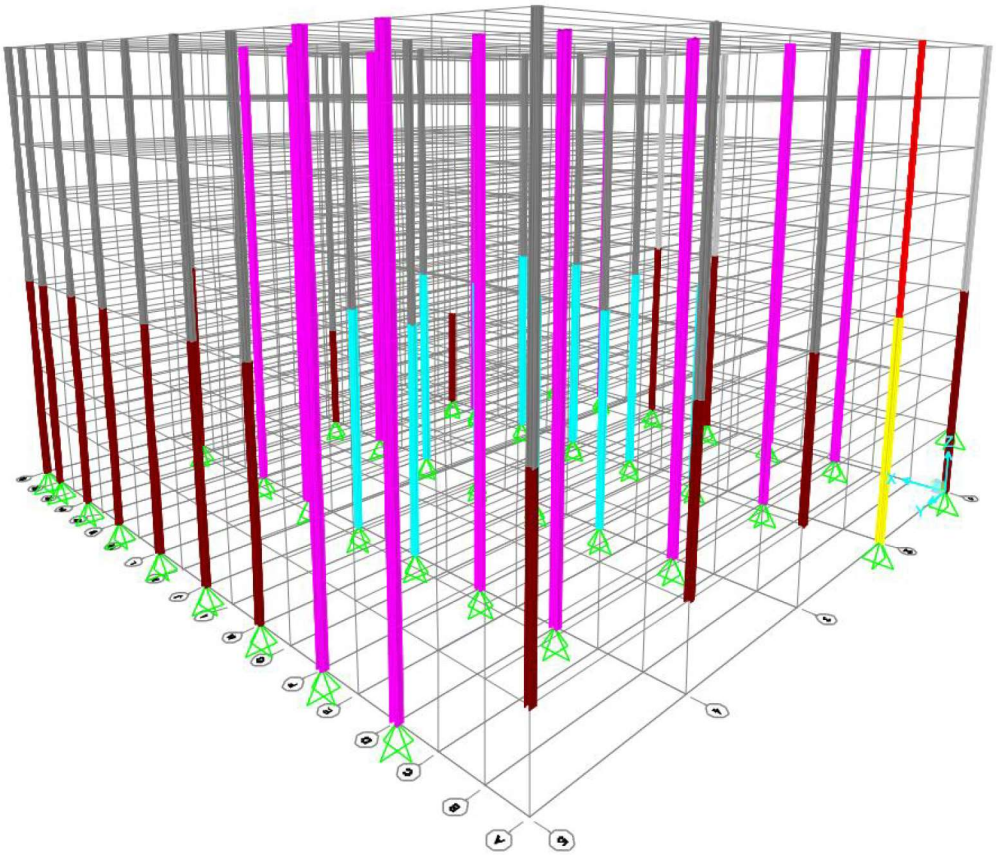


Figure A4.14 Global View for Columns (SAP2000)

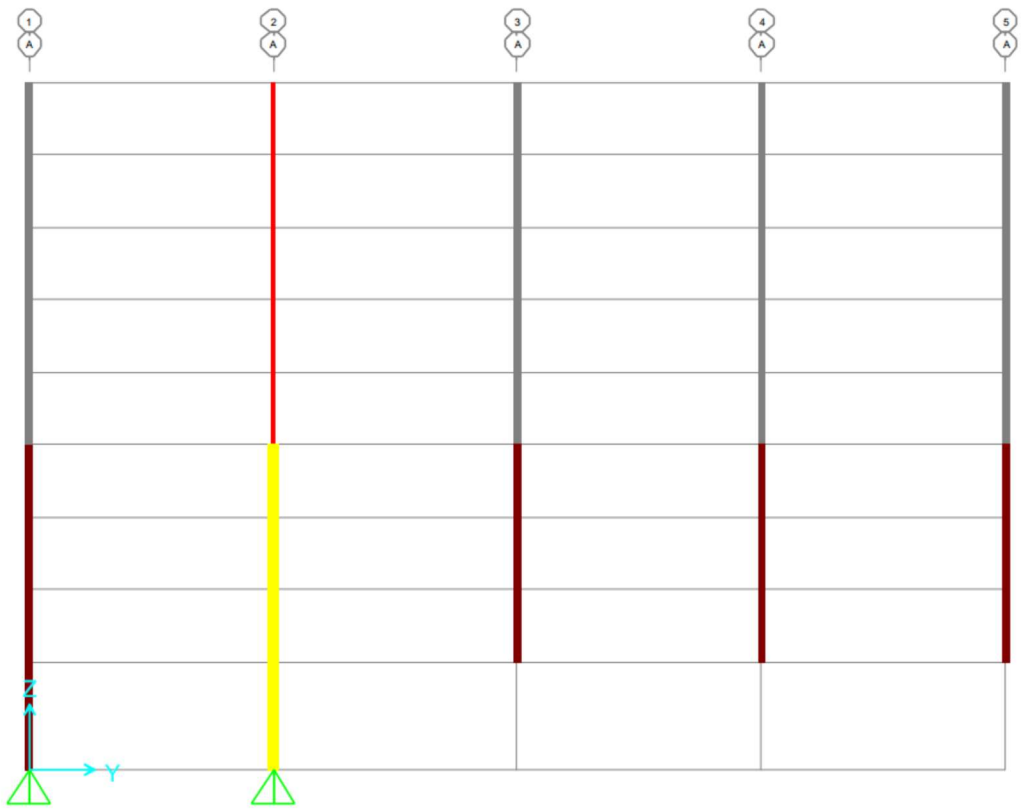


Figure A4.15 Column cross-section layout in the face A (SAP2000)

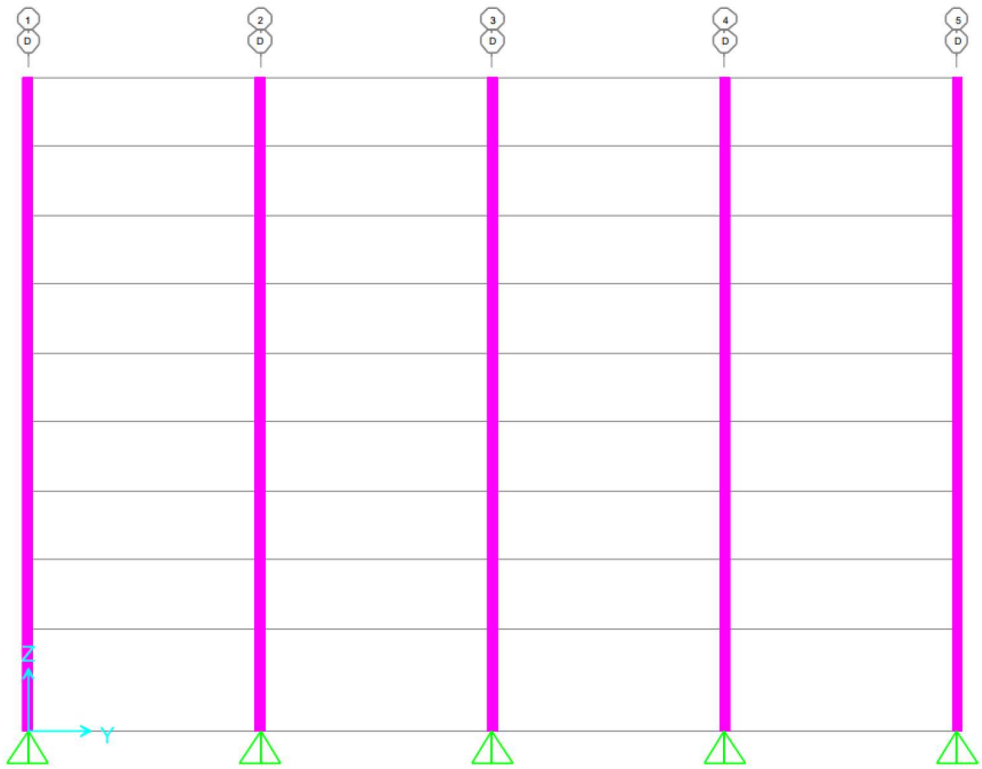


Figure A4.16 Column cross-section layout in the face D (SAP2000)

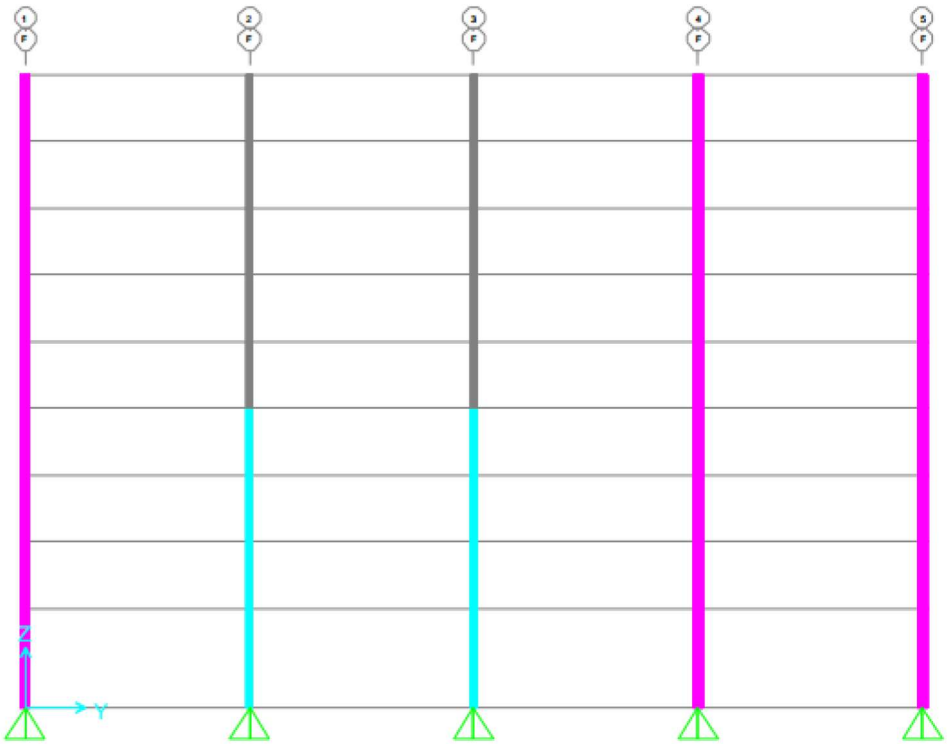


Figure A4.17 Column cross-section layout in the face F (SAP2000)

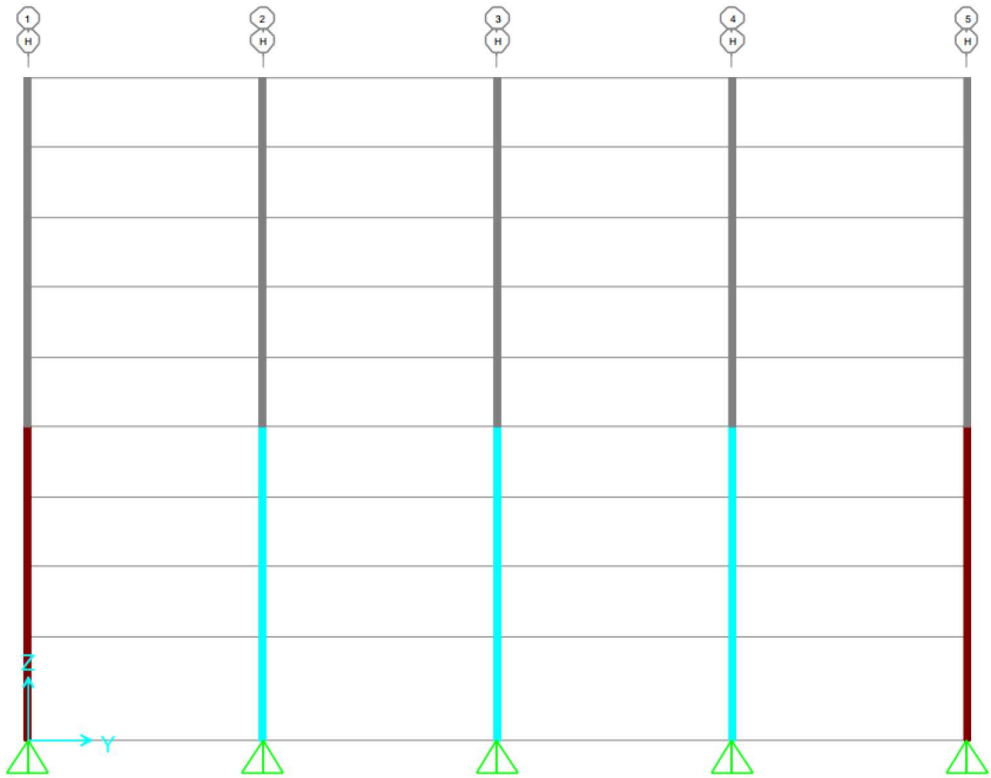


Figure A4.18 Column cross-section layout in the face H (SAP2000)

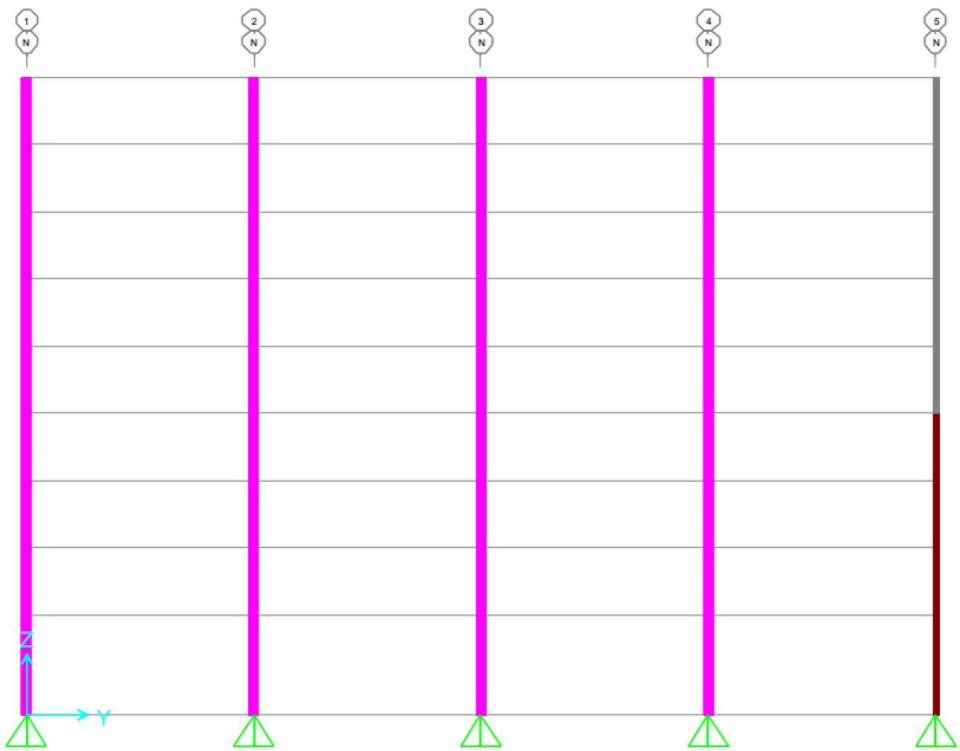


Figure A4.19 Column cross-section layout in the face N (SAP2000)

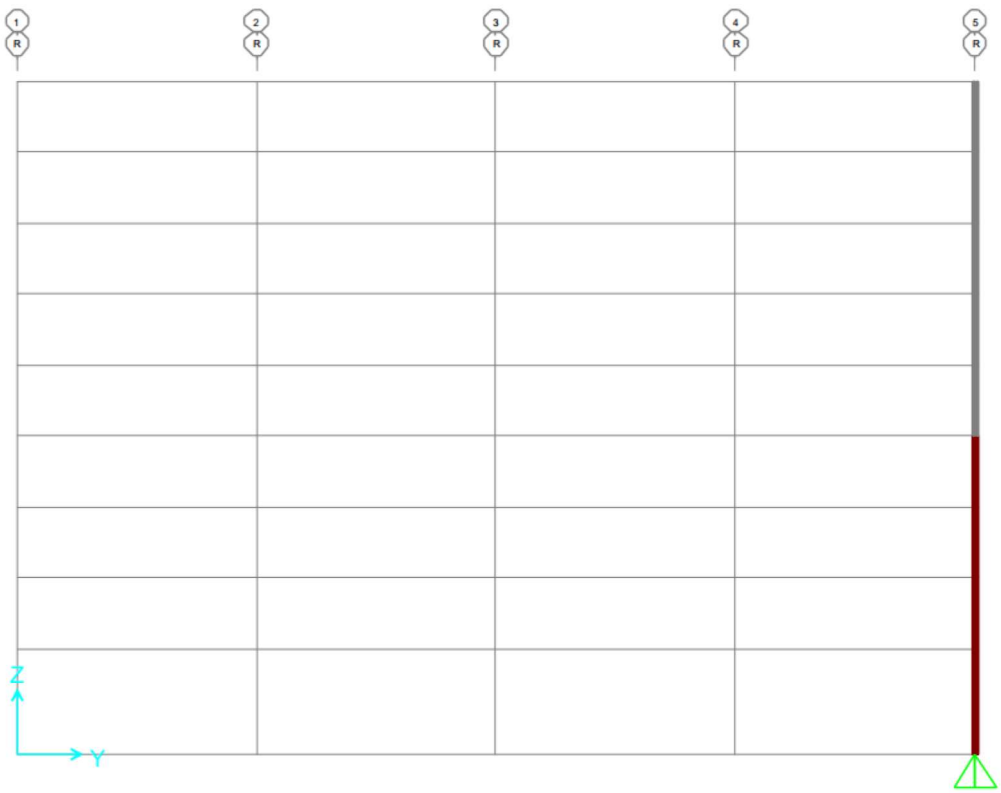


Figure A4.20 Column cross-section layout in the face R (SAP2000)

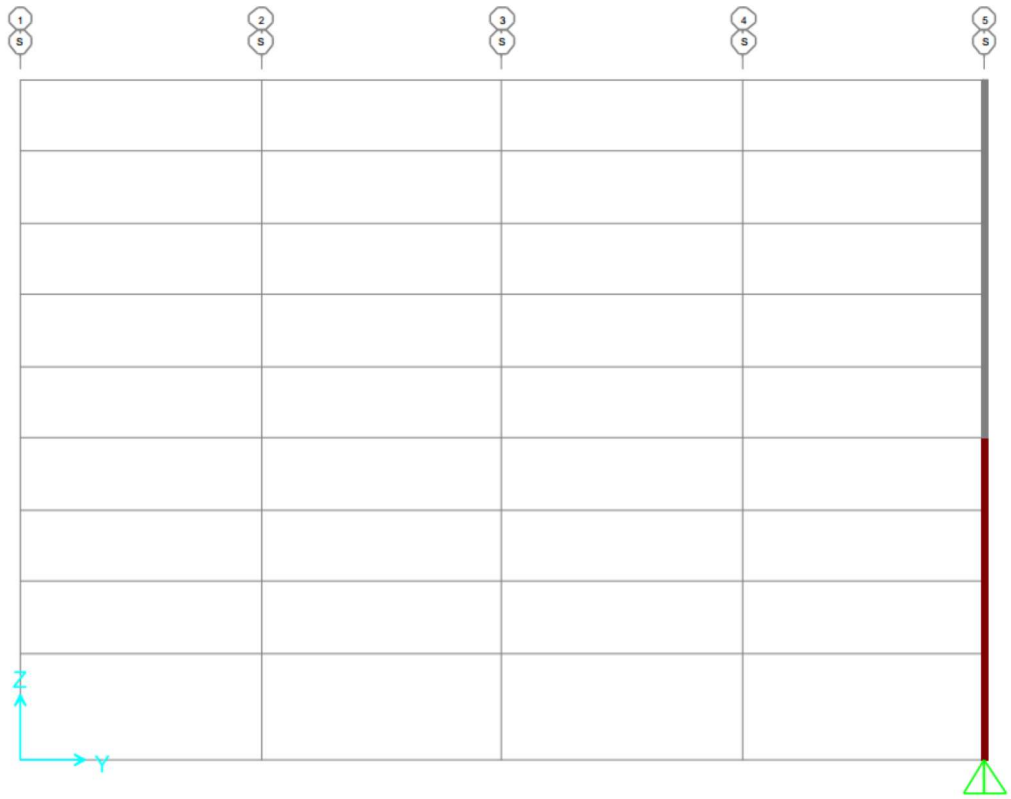


Figure A4.21 Column cross-section layout in the face S (SAP2000)

Appendix A5

Drawings of Beam-Beam Connections

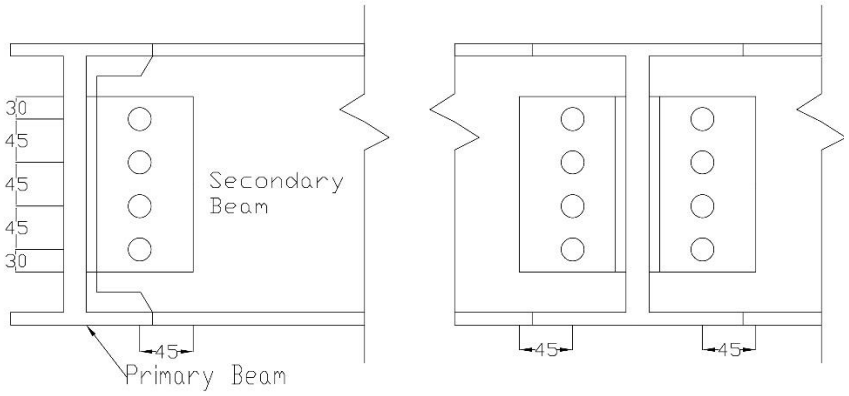


Figure A5.1: Figure showing the configuration of beam-to-beam connections

Appendix A6

Drawings of Beam-Column Connections

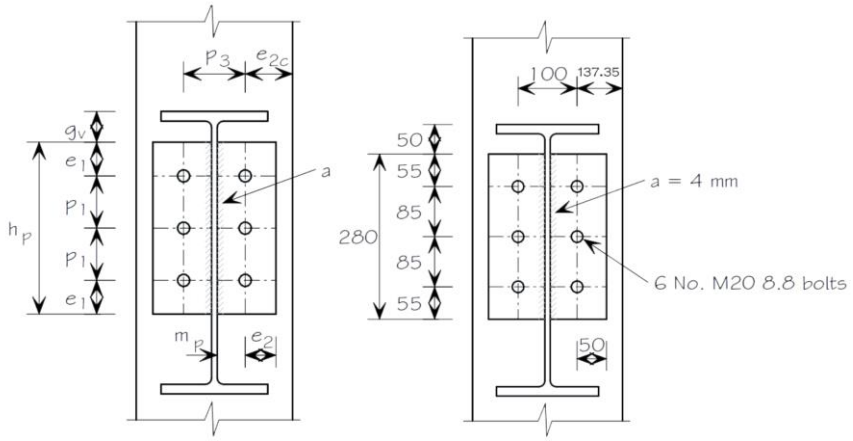


Figure A6.1 Positions of plate and holes for bolts.4

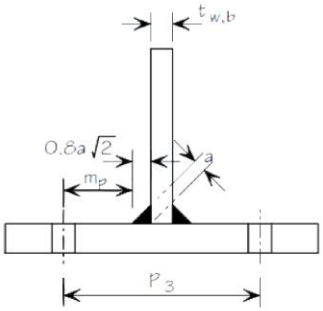


Figure A6.2 Plan view of details for end plate

Appendix A7

Drawing of Wind Bracing Connections

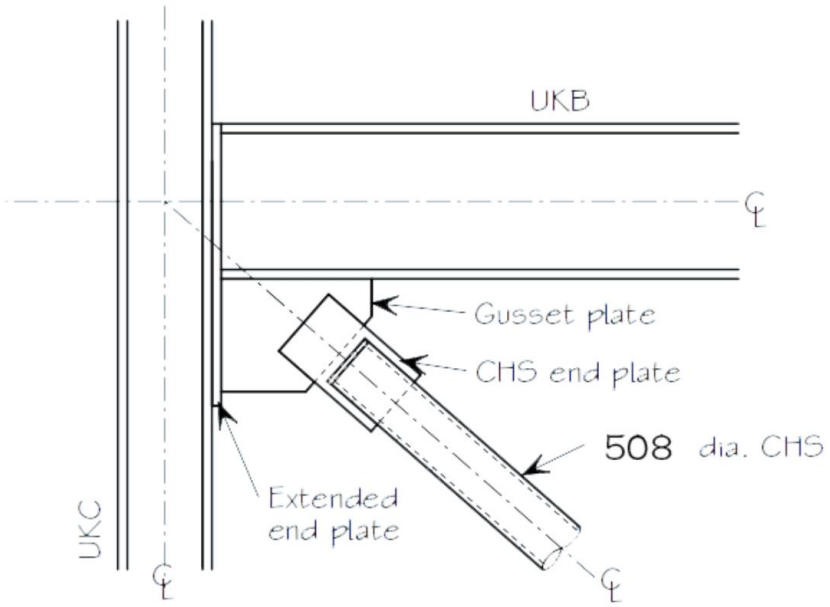


Figure A7.1 Connections details: General layout. Referenced from "Steel Building Design: Worked examples for students in accordance with Eurocodes and the UK National Annexes" by The Steel Construction Institute (p. 76), 2009.

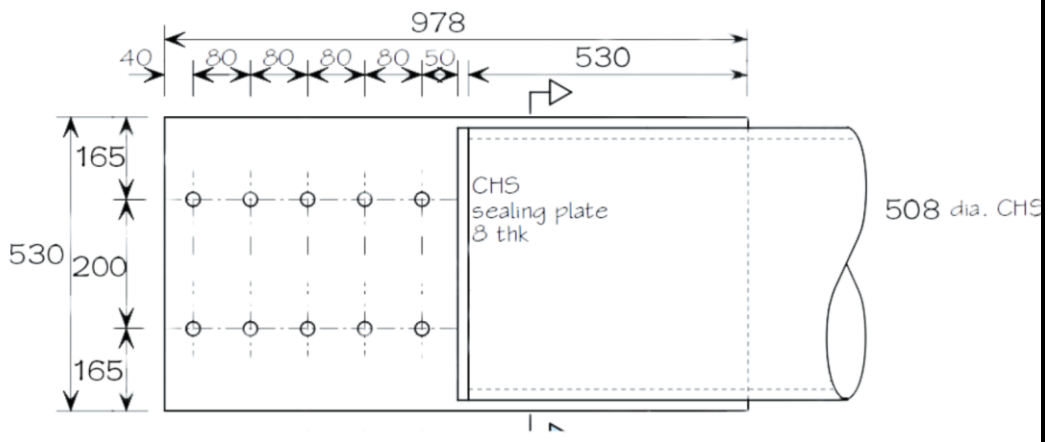


Figure A7.2 Position of holes for bolts. Referenced from "Steel Building Design: Worked examples for students in accordance with Eurocodes and the UK National Annexes" by The Steel Construction Institute (p. 77), 2009.

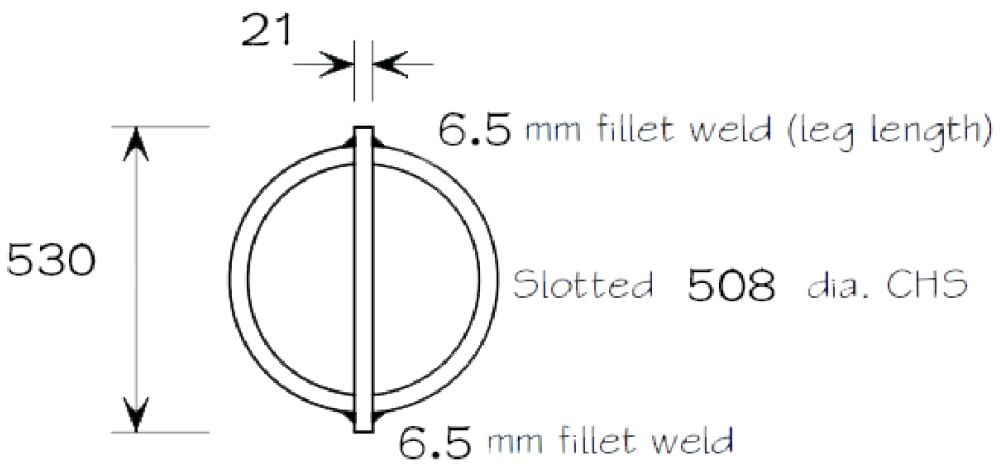


Figure A7.3 Details on connection between end plate and hollow section. Referenced from "Steel Building Design: Worked examples for students in accordance with Eurocodes and the UK National Annexes" by The Steel Construction Institute (p. 77), 2009.

Appendix A8

Drawing of Cantilever Truss System Connections

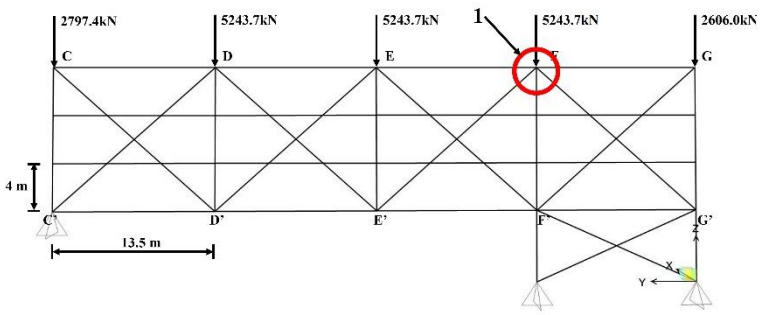


Figure A8.1 Location of the KT joint

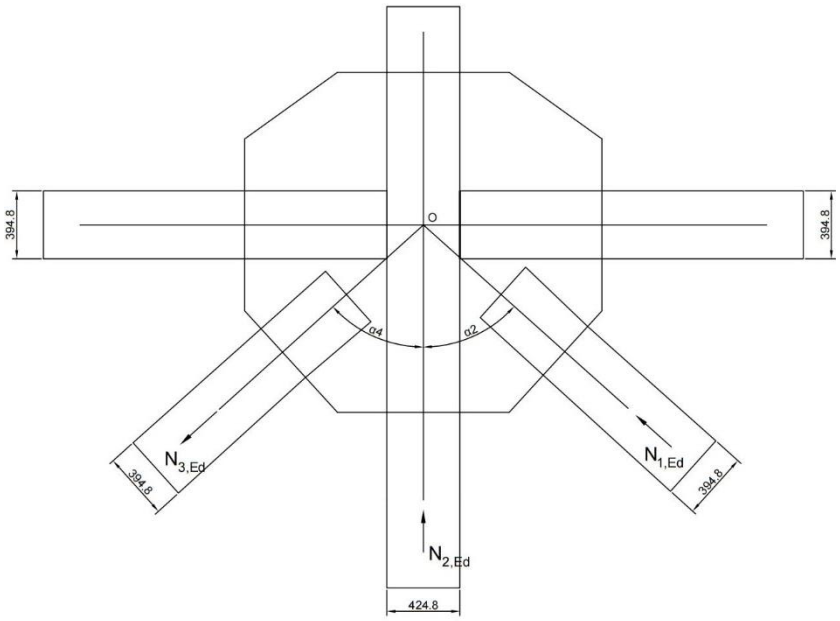


Figure A8.2 Gusset plate to web chord welded connection

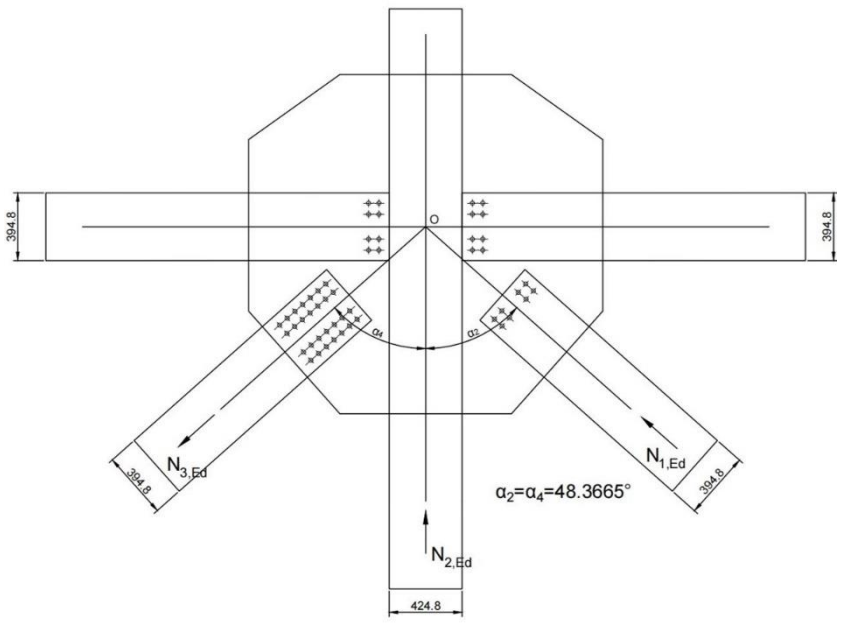


Figure A8.3 Location of the gross cross-sections of the gusset plate

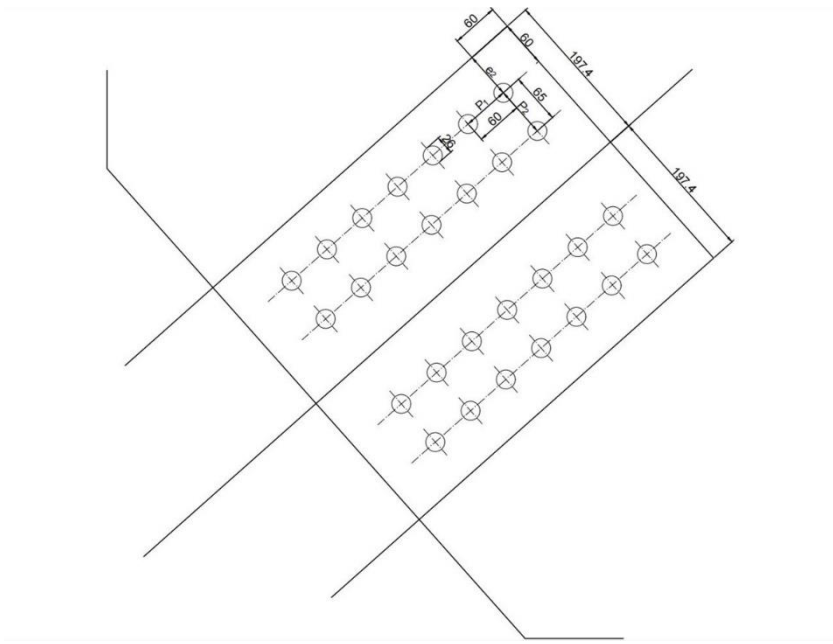


Figure A8.4 Connection sizes and positioning

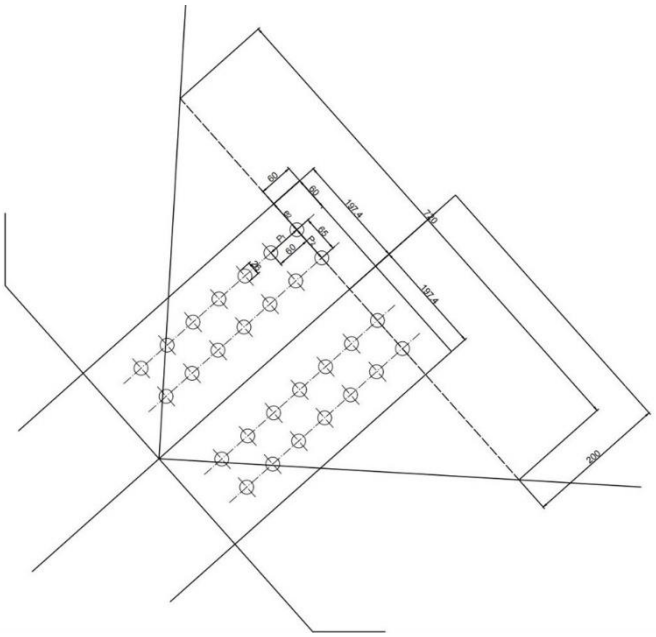


Figure A8.5 Diffusion by 45° of the axial force

Appendix A9

Composite Slabs Design

A9.1 Summary of Results

To summarise, a double-span composite slab with depth 130mm and 1.2mm of steel profile using ComFlor 60 (bar fire method/unpropped double-span slab) is proposed. A393 mesh is proposed for the slab to meet the requirements related to concrete cracking.

A9.2 Detailed Calculations

Propose ComFlor 60 Bar Fire Method/Unpropped double-span slab,

<u>From ComFlor 60 manufacturer's data:</u>

Total Depth of Slab t=130mm

Depth of Profile h_p=1.2mm

Span L=3.5m

Cross Section Area A_{pe}=1721mm^2/m

Second Moment of Area I_p=132.91cm^4/m

Yield Strength of Deck f_{yp}=350MPa

Design Bending Resistance (Sagging) M_{Rd}=15.21kNm/m

Height of NA above soffit=33.85mm

Assuming a concrete class C25/30,

Density (wet)=26kN/m^3

Density (dry)=25kN/m^3

Cylinder Strength f_{ck}=25MPa

Concrete Design Strength=0.85(25)/1.5=14.17MPa

Elastic Modulus E_{cm}=31GPa

Actions

Concrete Self Weight (Wet)=2.46kPa

Concrete Self Weight (Dry)=2.36kPa

Permanent Actions

Load (Construction Stage)=0.14kPa

Load (Operation Stage)=0.7+0.3+1+1.5+0.5+2.36=6.5kPa

Variable Actions

Load (Construction Stage)=Construction Load+Concrete SW (Wet)

=1.5+ 2.46=3.96kPa

Load (Operation Stage)=4kPa

ULS Partial Factors (Consider Load Case 1 to be Conservative)

γ_G=1.35

γ_Q=1.5

Load (ULS)

Construction Stage :0.14(1.35)+3.96(1.5)=6.13kPa

Operation Stage: 6.36(1.35)+4(1.5)=14.78kPa

Design Moment and Shear Force (ULS)

Construction Stage

$M_{Ed}=FL^2/8=6.1(3.5)^2/8=9.39$kNm/m width

$V_{Ed}=FL/2=6.1(3.5)/2=10.73$kN/m width

Operation Stage

$M_{Ed}=FL^2/8=14.6(3.5)^2/8=22.62$kNm/m width

$V_{Ed}=FL/2=14.6(3.5)/2=25.86$kN/m width

ULS Moment Design Check (Construction Stage)

According to BSEN1993-1-3 Clause 6.1.1,

$M_{Ed}/M_{Rd}=10.53/15.21=0.62<=1$ **OK**

SLS Deflection Check (Construction Stage)

Load under SLS=0.14+2.96=2.6kPa

According to BSEN 1994-1-1 Clause 9.3.2(2),

$\delta_s=5FL^4/(384EI)=18.20\text{mm}>13\text{mm}$ **FURTHER CHECK**

Therefore, ponding cannot be ignored and the thickness of concrete increases by $0.7\delta_s$.

Revised Load under SLS=2.6+0.7(18/1000)(26)=2.93kPa

Revised $\delta_s=5FL^4/(384EI)=20.52$mm

$\delta_{s,max}=L/130=3500/130=26.92\text{mm}<=30\text{mm}$ **OK**

According to BSEN 1994-1-1 NA2.15,

$\delta_s=20.5\text{mm}<=\delta_{s,max}=26.9\text{mm}$, so it is OK.

ULS Composite Slab Check (Operation Stage)

Bending Resistance

According to BSEN1994-1-1 Clause 6.2.1.2,

Max. compressive design force in concrete (assuming pna is below solid part of

slab):

$N_c=f_{cd}(A_c)=14.2(130-60)=991.67kN/m$

Max. tensile resistance of profiled steel sheet:

$N_p=f_{yp,d}(A_p)=350(1721)/1000=602.35kN/m$

As $N_p<N_c$, netural axis lies above profiled sheeting.

According to BSEN1994-1-1 Clause 9.7.2(5),

Depth of concrete in compression:

$x_{pl}=A_{pe}(f_{yp,d})/(bf_{cd})=1721(350)/1000/14.2=42.52mm$

Assuming full shear connection:

$M_{pl,Rd}=A_pf_{yd}(d_p-x_{pl}/2)=45.11kNm/m$

$M_{Ed}/M_{pl,Rd}=22.33/45.11=0.50<=1$ **OK**

<u>Longitudinal Shear Resistance</u>

From BSEN1994-1-1 Clause 9.7.3,

$V_{l,Rd}=bd_p(mA_p/bL_s+k)/\gamma_{vs}$

From manufacturer's data,

m=157.2MPa

k=0.12MPa

For uniform load across span length,

$L_s=L/4=3500/4=875mm$

$V_{l,Rd}=33.26kN/m$

$V_{Ed}/V_{l,Rd}=25.5/33.3=0.78<=1$ **OK**

<u>Vertical Shear Resistance</u>

$V_{v,Rd}=v_{min}b_sd_p$

According to BSEN1992-1-1 NA2 Table NA.1

$v_{min}=0.035k^{1.5}f_{ck}^{0.5}$

, where $k=1+(200/d_p)^{0.5}=2.44>2$ **FURTHER CHECK**

Therefore, take k=2

$v_{min}=0.035(2)^{1.5}(25)^{0.5}=0.49MPa$

$V_{v,Rd}=0.049\ (130-33.9)=47.59kN/m$

$V_{Ed}/V_{v,Rd}=25.5/47.6=0.54<=1$ **OK**

<u>SLS Composite Slab Check (Operation Stage)</u>

<u>Cracking of Concrete</u>

According to BSEN1994-1-1 Clause 9.8.1(2), minimum reinforcement of 0.4% of concrete area should be provided.

Therefore, provide $0.4\%(130-60)(1000)=280mm^2$

Therefore, provide A393 instead.

Appendix A10

A10.1 Beam section design results

Table A10.1: Table showing the UKB sections for beams adopted for the design

Type	Section
primary beam (floor)	533x210x122UKB
secondary beam (floor)	533x210x122UKB
edge beam (floor)	533x210x122UKB
primary beam (roof)	533x210x122UKB
secondary beam (roof)	533x210x122UKB
edge beam (roof)	533x210x122UKB

A10.2 Detailed Calculation

Design data

Beam span	L = 14.0 m
Beam spacing	s = 3.5 m
Total slab depth	h = 130 mm
Depth of concrete above profile	h_c = 70 mm
Deck profile height	h_p = 60 mm
Width of the bottom of the trough	b_{bot} = 120 mm
Width of the top of the trough	b_{top} = 170 mm approx

Shear connectors

Diameter	d = 19 mm
Overall height before welding	h_{sc} = 100 mm
Height after welding	95 mm

Materials

Structural Steel:

For grade S355 and maximum thickness (t) less than 40 mm

Yield strength	f_y = 355 N/mm^2
Ultimate strength	f_u = 510 N/mm^2

Steel reinforcement:

Yield strength	f_{yk} = 500 N/mm^2

Concrete:

Normal weight concrete strength class C25/30

Density	26 kN/m³ (wet)
	25 kN/m³ (dry)
Cylinder strength	f_{ck} = 25 N/mm^2
Secant modulus of elasticity	E_{cm} = 31 kN/mm^2

Actions

Use the data from slab part.
Ultimate Limit State
Combination of actions for Ultimate Limit State
The design value of combined actions are:
Construction stage:
Distributed load (0.925×1.35×0.44)+(1.5×3.96) = 6.49 kN/m^2
Total load F_d = 5.26×14.0×3.5 = 317.98 kN
Composite stage:
Distributed load (0.925×1.35×6.50)+(1.5×4.00) = 14.12 kN/m2
Total load Fd = 14.12×14.0×3.5 = 691.73 kN

Design values of moment and shear force at ULS

Construction stage

Maximum design moment (at mid span)

$$M_{y,Ed} = \frac{F_d L}{8} = \frac{317.98 \times 14.0}{8} = 556.5 \text{ kNm}$$

Composite stage

Maximum design moment (at mid span)

$$M_{y,Ed} = \frac{F_d L}{8} = \frac{691.73 \times 14.0}{8} = 1210.5 \text{ kNm}$$

Maximum design shear force (at supports)

$$V_{E_d} = \frac{F_d}{2} = \frac{691.7}{2} = 345.9 \text{ kN}$$

Partial factors for resistance

Structural steel	γ_{M0} = 1.0
Concrete	γ_c = 1.5
Reinforcement	γ_s = 1.15
Shear connectors	γ_v = 1.25
Longitudinal shear	γ_{vs} = 1.25

Trial section

The plastic modulus that is required to resist the construction stage maximum design bending moment is determined as:

$$W_{pl,y} = \frac{M_{y,Ed}\gamma_{MO}}{f_y} = \frac{556.5 \times 10^3 \times 1.0}{355} = 1568 \text{ cm}^3$$

From the tables of section properties try section 533×165×75 UKB, S355, which has $W_{pl,y}$ = 2100 cm^3

Depth of cross-section	h_a = 529.1 mm
Width of cross-section	b = 165.9 mm
Depth between fillets	d = 476.5 mm
Web thickness	t_w = 9.7 mm
Flange thickness	t_f = 13.6 mm
Radius of root fillet	r = 12.7 mm

Cross-section area	$A_a = 95.2$ cm^2
Plastic section modulus (y-y)	$W_{pl,y} = 1810$ cm^3
$t_f < 40$ mm, therefore	$f_y = 355$ N/mm^2
Modulus of elasticity	$E = 210000$ N/mm^2
Second moment of area (y-y)	$I_a = 41100$ cm^4

Section classification

For section classification the coefficient e is:

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81$$

Outstand flange: flange under uniform compression

$$c = \frac{(b - t_w - 2r)}{2} = \frac{(165.9 - 9.7 - 2 \times 12.7)}{2} = 65.4 \text{ mm}$$

$$\frac{c}{t_f} = \frac{65.4}{13.6} = 4.81$$

The limiting value for Class 1 is $\frac{c}{t} \leq 9\varepsilon = 9 \times 0.81 = 7.32$

4.81 < 7.32

Therefore, the flange outstand in compression is Class 1.

Internal compression part: web under pure bending

c = d = 476.5 mm

$$\frac{c}{t_w} = \frac{476.5}{9.7} = 49.12$$

The limiting value for Class 1 is $\frac{c}{t} \leq 72\varepsilon = 72 \times 0.81 = 58.58$

49.12 < 58.58

Therefore, the web in pure bending is Class 1.

Therefore, the section is Class 1 under pure bending.

<u>Composite stage member resistance checks</u>

Concrete

Design value of concrete compressive strength $f_{cd} = \alpha_{cc} \times f_{ck}/\gamma_c$

$\alpha_{cc} = 0.85$

$f_{cd} = 0.85 \times \frac{25}{1.5} = 14.2\ N/mm^2$

Compression resistance of concrete slab

At mid-span the effective width of the compression flange of the composite beam is determined from:

$b_{eff} = b_0 + \sum b_{ei}$

$b_{ei} = \frac{L_e}{8} = \frac{L}{8} = \frac{14}{8} = 1.75$ m ($L_e = L$ for simply supported beams)

Assume a single line of shear studs, therefore, $b_0 = 0$ m

$b_{eff} = 0 + (2 \times 1.75) = 3.50m \leq 3.5m$ (beam spacing)

Compression resistance of concrete slab is determined from:

$$N_{c,\,\text{slab}} = f_{cd} b_{eff} h_c$$

where h_c is the depth of the solid concrete above the decking

$N_{c,slab} = 14.2 \times 3500 \times 70 \times 10^{-3} = 3471$ kN

Tensile resistance of steel section

$$N_{pl,a} = f_d A_a = \frac{f_y A_a}{\gamma_{MO}}$$

$$N_{pl,a} = \frac{355 \times 95.2 \times 10^2}{1.0} \times 10^{-3} = 3380 \text{ kN}$$

Location of neutral axis

Since $N_{pl,a} < N_{c,slab}$ the plastic neutral axis lies in the concrete flange.

Design bending resistance with full shear connection

As the plastic neutral axis lies in the concrete flange, the plastic resistance moment of the composite beam with full shear connection is:

$$M_{pl,Rd} = N_{pl,a}\left[\frac{h_s}{2} + h - \frac{N_{pl,a}}{N_{c,slab}} \times \frac{h_c}{2}\right]$$

$$M_{pl,Rd} = 3380\left[\frac{529.1}{2} + 130 - \frac{3380}{3471} \times \frac{70}{2}\right] \times 10^{-3} = 1218 \text{ kNm}$$

Bending moment at mid span $M_{y,Ed} = 1210.5$ kNm

$$\frac{M_{y,Ed}}{M_{pl,Rd}} = \frac{1210.5}{1218} = 0.99 < 1.0$$

Therefore, the design bending resistance of the composite beam is adequate, assuming full shear connection.

Shear connector resistance

The design shear resistance of a single shear connector in a solid slab is the smaller of:

$$P_{Rd} = \frac{0.29\alpha d^2\sqrt{f_{ck}E_{cm}}}{\gamma_v} \text{ and}$$

$$P_{Rd} = \frac{0.8 f_u(\pi d^2/4)}{\gamma_v}$$

$$\frac{h_{\mathrm{SC}}}{d} = \frac{100}{19} = 5.26$$

As $\frac{h_{\mathrm{Sc}}}{d} > 4.0 \qquad \alpha = 1.0$

$$P_{\mathrm{Rd}} = \frac{0.29 \times 1.0 \times 19^2\sqrt{25 \times 31 \times 10^3}}{1.25} \times 10^{-3} = 73.7 \text{ kN}$$

or

$$P_{\mathrm{Rd}} = \frac{0.8 \times 450 \times \left(\pi \times \frac{19^2}{4}\right)}{1.25} \times 10^{-3} = 81.7 \text{ kN}$$

As 73.7 kN < 81.7 kN $P_{\mathrm{Rd}} = 73.7$ kN

Influence of deck shape

Deck crosses the beam (i.e., ribs transverse to the beam)

One stud per trough, $n_r = 1.0$

Reduction factor

$$k_t = \left(\frac{0.7}{\sqrt{n_p}}\right)\left(\frac{b_0}{h_p}\right)\left(\frac{h_{sc}}{h_p} - 1\right) \leq 1.0$$

For trapezoidal profiles, b_0 is the average width of a trough, taken here as (120 + 170) ÷ 2 = 145 mm

$$k_t = \left(\frac{0.7}{\sqrt{1}}\right) \times \left(\frac{145}{60}\right) \times \left(\frac{100}{60} - 1\right) = 1.13 \text{ but not more than } 1.0$$

Therefore, as $k_t = 1.0$ no reduction in shear connector resistance is required.

Therefore, $P_{Rd} = 73.7$ kN

Number of shear studs in half span

Use one shear connector per trough, therefore,

Stud spacing along beam = 175 mm

Centre line to centre line span of 7 m should be reduced to allow for the primary beam width or the column width (assume 254 mm).

$n = \frac{7000-(254/2)}{300} = 39$ stud shear connectors per half span

Degree of shear connection

Total resistance of 39 shear connectors

$R_q = 39P_{Rd} = 39 \times 73.7 = 2875.5$kN

$$\frac{R_q}{N_{pl,a}} = \frac{2875.5}{3380} = 0.85 < 1.0$$

As this is less than 1.0, this beam has partial shear connection.

Therefore, the minimum shear connection requirements must be checked, and the moment resistance reassessed.

Minimum shear connection requirements

The minimum shear connection requirement is calculated from:
(for $L_e < 25$m):

$$\eta \geq 1 - \left(\frac{355}{f_y}\right)(0.75 - 0.03L_e), \qquad \eta \geq 0.4$$

For a simply supported beam, L_e is equal to the span.

$$\eta \geq 1 - \left(\frac{355}{355}\right)(0.75 - 0.03 \times 14) = 0.67, \qquad \eta \geq 0.4$$

Therefore, the degree of shear connection must be at least 0.67. As shown above, there are a sufficient number of shear connectors to achieve this.

Design bending moment resistance with partial shear connection

The design bending moment can conservatively be calculated using:

$$M_{Rd} = M_{pl,a,Rd} + \left(M_{pl,Rd} - M_{pl,a,Rd}\right)\eta$$

$M_{pl,a,Rd}$ is the plastic moment resistance of the steel section:

$$M_{pl,a,Rd} = f_{yd}W_{pl,y} = 355 \times 1810 \times 10^{-3} = 642.6 \text{ kNm}$$

Therefore, the design bending moment resistance is:

$$M_{Rd} = 642.6 + (1218 - 642.6) \times 0.85 = 1132.4 \text{ kNm}$$

$$\frac{M_{\mathrm{y,Ed}}}{M_{\mathrm{Rd}}} = 1 \leq 1.0$$

Therefore, the design bending resistance of the composite beam with partial shear connection is adequate.

<u>Shear buckling resistance of the uncased web</u>

For unstiffened webs if $\frac{h_w}{t} > \frac{72}{\eta}\varepsilon$ the shear buckling resistance of the web should be checked.

Where:

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} = 0.81$$

$\eta = 1.0$ (conservative)

$$h_w = h_a - 2t_f = 529.1 - (2 \times 13.6) = 501.9 \text{ mm}$$

$$\frac{72}{\eta}\varepsilon = \left(\frac{72}{1.0}\right) \times 0.81 = 58.58$$

$$\frac{h_w}{t} = \frac{h_w}{t_w} = \frac{501.9}{9.7} = 51.74$$

As 51.74 < 58.58 the shear buckling resistance of the web does not need to be checked.

Resistance to vertical shear

Vertical shear resistance of the composite beam is:

$$V_{pl,Rd} = V_{pl,a,Rd} = \frac{A(f_y/\sqrt{3})}{\gamma_{MO}}$$

For rolled I and H sections loaded parallel to the web:

$A_v = A - 2bt_f + t_f(t_w + 2r)$ but not less than $\eta h_w t_w$

$A_v = 9520 - (2 \times 165.9 \times 13.6) + 13.6 \times [9.7 + (2 \times 12.7)]$

$A_v = 5485\ \text{mm}^2$

$\eta = 1.0$ (Conservatively)

$\eta h_w t_w = 1.0 \times 501.9 \times 9.7 = 4868\ \text{mm}^2$

$5485\ \text{mm}^2 > 4868\ \text{mm}^2$

Therefore, $A = 5485\ \text{mm}^2$

$$V_{pl,Rd} = 5485 \times \frac{355}{\sqrt{3} \times 1.0} \times 10^{-3} = 1124\ \text{kN}$$

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{76.5}{247} = 0.31 < 1.0$$

Therefore, the design resistance to vertical shear is adequate.

As there is no shear force at the point of maximum bending moment (mid span) no reduction (due to shear) in bending resistance is required.

Design of the transverse reinforcement

For simplicity, neglect the contribution of the decking and check the resistance of the reinforced concrete flange to splitting.

The area of reinforcement (A_{sf}) can be determined using the following equation:

$$\frac{A_{sf} f_{yd}}{s_f} > \frac{V_{Ed} h_f}{\cot\theta_f} \quad \text{therefore,} \quad \frac{A_{sf}}{s_f} > \frac{V_{Ed} h_f}{f_{yd} \cot\ \theta_f}$$

where:

h_f is the depth of concrete above the metal decking, therefore,

$h_f = h_c = 70$ mm

S_f is the spacing of the transverse reinforcement

$$f_{yd} = \frac{f_y}{\gamma_s} = \frac{500}{1.15} = 435\text{N/mm}^2$$

For compression flanges $26.5° \leq \theta_f \leq 45°$

The longitudinal shear stress is the stress transferred from the steel beam to the concrete. This is determined from the minimum resistance of the steel, concrete and shear connectors. In this example, with partial shear connection, that maximum force that can be transferred is limited by the resistance of the shear connectors over half of the span, and is given by $R_q = 2875.5$ kN

This force must be transferred over each half-span. As there are two shear planes (one on either side of the beam, running parallel to it), the longitudinal shear stress is:

$$v_{Ed} = \frac{R_q}{2h_f \Delta x} = \frac{2875.5 \times 1000}{2 \times 70 \times 7000} = 2.93 \text{ N/mm}^2$$

For minimum area of transverse reinforcement assume θ = 26.5°

$$\frac{A_{sf}}{s_f} \geq \frac{V_{Ed} h_f}{f_{yd} \cot \theta_f} = \frac{2.93 \times 70}{435 \times \cot 26.5} \times 10^3 = 235.5 \text{ mm}^2/\text{m}$$

Therefore, provide A393 mesh reinforcement (393 mm^2 /m) in the slab.

<u>Crushing of the concrete compression strut</u>

The model for resisting the longitudinal shear assumes compression struts form in the concrete slab

Verify that:

$$v_{Ed} \leq v f_{cd} \sin\ \theta_f \cos\ \theta_f$$

where:

$$v = 0.6\left[1 - \frac{f_{ck}}{250}\right]$$

$$v = 0.6\left[1 - \frac{25}{250}\right] = 0.54$$

$$v f_{cd} \sin \theta_f \cos \theta_f = 0.54 \times 14.2 \times \sin 26.5 \times \cos 26.5 = 3.06 \text{ N/mm}^2$$

$$v_{Ed} = 2.93 \text{ N/mm}^2 < 3.06 \text{ N/mm}^2$$

Therefore, the crushing resistance of the compression strut is adequate.

<u>Serviceability limit state</u>

Deflection

Check deflection in service under imposed load only assuming full composite action.

Check for position of neutral axis:

$$A_a(z_g - h_c) \leq \frac{b_{eff} h_c^2}{2n}$$

$$Z_g = 529.1/2 + 130 = 394.55\text{mm}$$

Note: n takes $2n_0$

$$\frac{b_{eff} h_c^2}{2n} = \frac{3500 \times 70^2}{2 \times 2 \times 6.77} = 632917$$

$$A_a(z_g - h_c) = 9520 \times (394.55 - 70) = 3089716 > 632917$$

Therefore, neutral axis depth exceeds h_c

From $A_a(z_g - x) = \frac{b_{eff}h_c(x-h_c/2)}{n}$, x = 159 mm

$$I = I_a + A_a(z_g - x)^2 + \frac{b_{eff}h_c}{n}\left[\frac{h_c^2}{12} + \left(x - \frac{h_c}{2}\right)^2\right] = 1.22 \times 10^9 \text{ mm}^4$$

$$\delta_c = \frac{5wL^4}{384EI} = \frac{5 \times 4 \times 3.5 \times 14000\text{^}4}{384 \times 210000 \times 1.22 \times 10\text{^}9} = 27.23 \text{ mm}$$

Allowable = span/360 = 39 mm

Check deflection with partial shear connection expression:

$$\delta = \delta_c + 0.3(1 - \eta)(\delta_s - \delta_c)$$

$$\delta_s = \frac{5 \times 4 \times 3.5 \times 14000^4}{384 \times 21000 \times 41100 \times 10000} = 81.14 \text{ mm}$$

$$\delta = \delta_c + 0.30 \times (1 - 0.85)(81.14 - 27.23) = 30 \text{ mm}$$

Therefore, OK.

Appendix A11

Steel Columns Design

A11.1 Summary of Results

Table A11.1: Table showing the proposed column sizes

Region	Column Size (UKC)
Internal Columns	
Roof to 4/F	356 x 368 x 202 (Grey)
G/F to 4/F	356 x 406 x 467 (Green)
Peripheral Columns	
Roof to 4/F	356 x 368 x 202 (Grey)
G/F to 4/F	356 x 406 x 235 (Brown)
Cantilever Region	
Roof to 4/F	356 x 368 x 202 (Grey)
G/F to 4/F	356 x 406 x 634 (Yellow)
Wind Bracing Bay	
All Floors	356 x 406 x 1202 (Purple)

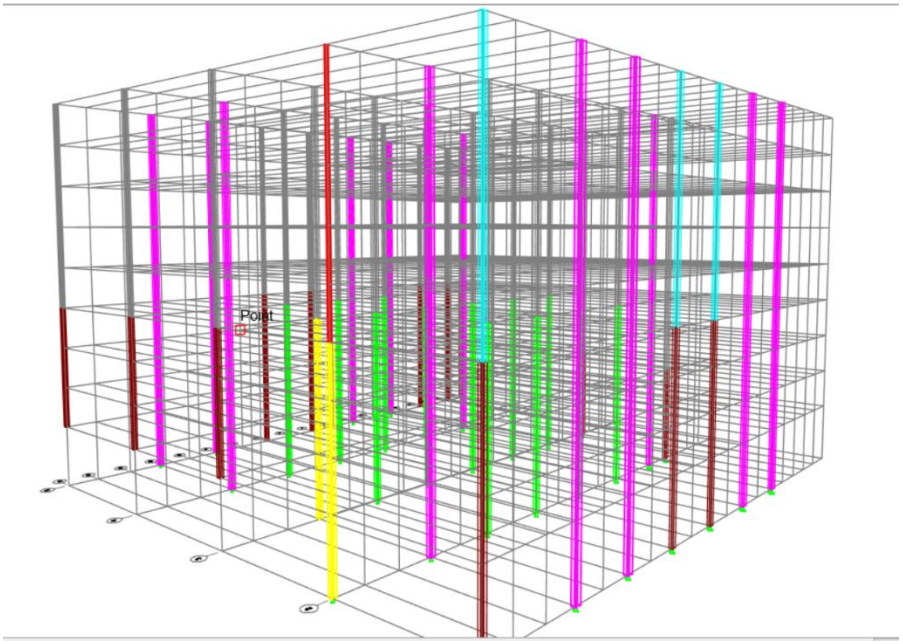


Figure A11.1: Figure showing the Proposed Column Sizes (Proposal 1)

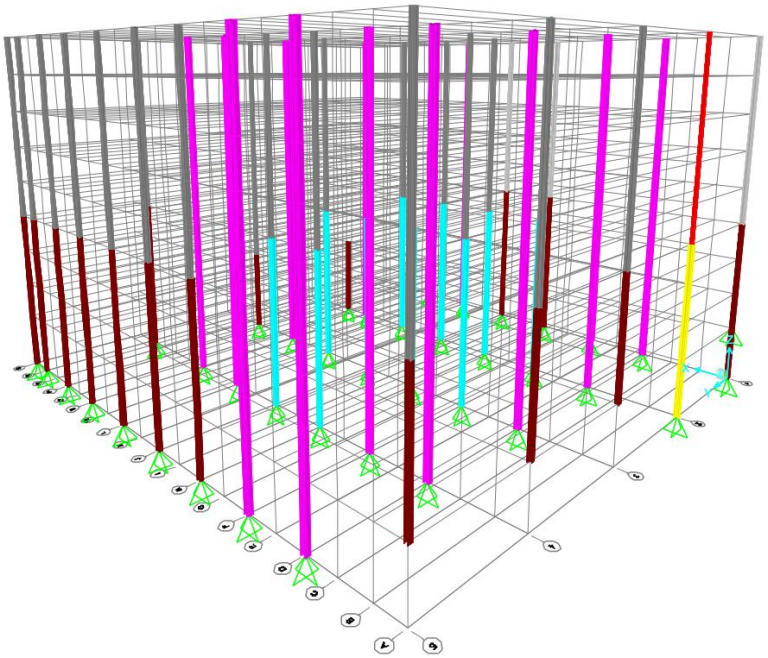


Figure A11.2: Figure showing the Proposed Column Sizes (Proposal 2)

A11.2 Calculation Results

The detailed calculation for each type of columns mentioned above are detailed from the next page. Note that the axial load of Proposal 2 was adopted for check for column UKC 356 x 406 x 1202 as it was more critical.

<u>Design Load for Internal Columns (Roof - 4th Floor)</u>

The slab is designed as one-way slab.

Tributary Area =	94.5	m^2
Assume ComFlor 60 Composite Slab of	150	mm
Weight of ComFlor Slab =	2.84	kPa
Assume 533x210x122 secondary beam,		
Mass of Sec. Beam =	122	kg/m
Length of Sec. Beam supported by a column=	27.0	m/column
SW of Sec. Beam on each column =	32	kN/column
Assume 533x210x122 primary beam,		
Mass of Primary Beam supported by a column=	122	kg/m
Length of Primary Beam =	7	m/column
SW of Primary Beam on each column =	8	kN/column

As the axial load is the largest at the bottom of columns

Max. Dead Load on a Column = 3150 kN

Max. Live Load on a Column = 1654 kN

Load Case

Load Case 1: 1.35 G_k + 1.5 Q_k

It is the most critical load case as the vertical forces are the greatest.

Factored Dead Load on a Column = 4252 kN

Factored Live Load on a Column = 2481 kN

Design of Compressive Columns

Section 356 x 368 x 202 UC

Section Properties

Grade	355	
f_y	355	MPa
h	374.6	mm
b	374.7	mm
t_w	16.5	mm
t_f	27	mm
r	15.2	mm
A	257	cm^2
E	210	GPa

Axis	y-y	z-z	
I (cm^4)	66300	23700	
Wpl (cm3)	3970	1920	*Class 1-2*
Wel (cm3)	3540	1260	*Class 3*

Classification of Steel Sections (Section 5.5.2 of BSEN1993-1-1)

ε 0.81

For Web, $c=h-2t_f-2r$

c 290.2 mm

c/t 17.59 = 21.62 ε

Class 1 web.

For Flange $c=b/2-t_w/2-r$

c 163.9

c/t 6.07 = 7.46 ε

Class 1 flange.

Overall Class 1 section.

Design Load

N_{Ed}	6733	kN
$M_{y,Ed}$	0	kNm
$M_{z,Ed}$	0	kNm

Cross-Section Resistance (Section 6.2.4 of BSEN1993-1-1)

$N_{c,Rd} = Af_y/\gamma_{M0}$ = 9123.5 kN

Because $N_{c,Rd}$ >= $N_{c,Ed}$ **OK**

Buckling Resistance (Section 6.3.1 of BSEN1993-1-1)

Axis	y-y	z-z	
L_{cr} (m)	4	4	
$N_{cr} = \pi^2EI/L^2$(kN)	85884.06	30700.6	
$\lambda = (Af_y/N_{cr})^{0.5}$	0.33	0.55	
Buckling Curve	b	c	*Table 6.2*
α	0.34	0.49	
$\Phi = 0.5[1+\alpha(\lambda-0.2)+\lambda^2)$	0.57	0.73	
$\chi = 1/[\Phi+(\Phi^2-\lambda^2)^{0.5}]$	0.95	0.82	
$N_{b,Rd} = \chi Af_y/\gamma_{M1}$ (kN)	8708.56	7457.62	

Because $N_{b,Rd}$ >= $N_{b,Ed}$ **OK**

Design Load for Internal Columns (Bottom - 4th Floor)

The slab is designed as one-way slab.

Tributary Area =	94.5	m^2
Assume ComFlor 60 Composite Slab of	150	mm
Weight of ComFlor Slab =	2.84	kPa

Assume 533x210x122 secondary beam,

Mass of Sec. Beam =	122	kg/m
Length of Sec. Beam supported by a column=	27.0	m/column
SW of Sec. Beam on each column =	32	kN/column

Assume 533x210x122 primary beam,

Mass of Primary Beam supported by a column=	122	kg/m
Length of Primary Beam =	7	m/column
SW of Primary Beam on each column =	8	kN/column

As the axial load is the largest at the bottom of columns

Max. Dead Load on a Column = 5735 kN

Max. Live Load on a Column = 3166 kN

Load Case

Load Case 1: 1.35 G_k + 1.5 Q_k

It is the most critical load case as the vertical forces are the greatest.

Factored Dead Load on a Column = 7743 kN

Factored Live Load on a Column = 4749 kN

Design of Compressive Columns

Section 356 x 406 x 467 UC

Section Properties

Grade	355	
f_y	335	MPa
h	436.6	mm
b	412.2	mm
t_w	35.8	mm
t_f	58	mm
r	15.2	mm
A	595	cm^2
E	210	GPa

Axis	y-y	z-z	
I (cm^4)	183000	67800	
Wpl (cm3)	10000	5030	*Class 1-2*
Wel (cm3)	8380	3290	*Class 3*

Classification of Steel Sections (Section 5.5.2 of BSEN1993-1-1)

ε 0.84

For Web, $c=h-2t_f-2r$

c 290.2 mm

c/t 8.11 = 9.68 ε

Class 1 web.

For Flange $c=b/2-t_w/2-r$

c 173

c/t 2.98 = 3.56 ε

Class 1 flange.

Overall Class 1 section.

Design Load

N_{Ed}	12491	kN
$M_{y,Ed}$	0	kNm
$M_{z,Ed}$	0	kNm

Cross-Section Resistance (Section 6.2.4 of BSEN1993-1-1)

$N_{c,Rd} = Af_y/\gamma_{M0}$ = 19932.5 kN

Because $N_{c,Rd}$ >= $N_{c,Ed}$ **OK**

Buckling Resistance (Section 6.3.1 of BSEN1993-1-1)

Axis	y-y	z-z	
L_{cr} (m)	6	6	
$N_{cr} = \pi^2EI/L^2$(kN)	105358.0	39034.2	
$\lambda = (Af_y/N_{cr})^{0.5}$	0.43	0.71	
Buckling Curve	b	c	*Table 6.2*
α	0.34	0.49	
$\Phi = 0.5[1+\alpha(\lambda-0.2)+\lambda^2)$	0.63	0.88	
$\chi = 1/[\Phi+(\Phi^2-\lambda^2)^{0.5}]$	0.91	0.72	
$N_{b,Rd} = \chi Af_y/\gamma_{M1}$ (kN)	18177.59	14264.43	

Because $N_{b,Rd}$ >= $N_{b,Ed}$ **OK**

Design Load for Peripheral Columns (Roof - 4th Floor)

The slab is designed as one-way slab.

Tributary Area =	49	m^2
Assume ComFlor 60 Composite Slab of	150	mm
Weight of ComFlor Slab =	2.84	kPa

Assume 533x210x122 secondary beam,

Mass of Sec. Beam =	122	kg/m
Length of Sec. Beam supported by a column=	14.0	m/column
SW of Sec. Beam on each column =	17	kN/column

Assume 533x210x122 primary beam,

Mass of Primary Beam supported by a column=	122	kg/m
Length of Primary Beam =	7	m/column
SW of Primary Beam and Cladding on each column =	45	kN/column

As the axial load is the largest at the bottom of columns

Max. Dead Load on a Column = 1674 kN

Max. Live Load on a Column = 858 kN

Load Case

Load Case 1: 1.35 G_k + 1.5 Q_k

It is the most critical load case as the vertical forces are the greatest.

Factored Dead Load on a Column = 2259 kN

Factored Live Load on a Column = 1286 kN

Design of Compressive Columns

Section 356 x 368 x 202 UC

Section Properties

Grade	355	
f_y	355	MPa
h	374.6	mm
b	374.7	mm
t_w	16.5	mm
t_f	27	mm
r	15.2	mm
A	257	cm^2
E	210	GPa

Axis	y-y	z-z	
I (cm^4)	66300	23700	
Wpl (cm3)	3970	1920	*Class 1-2*
Wel (cm3)	3540	1260	*Class 3*

Classification of Steel Sections (Section 5.5.2 of BSEN1993-1-1)

ε 0.81

For Web, $c=h-2t_f-2r$

c 290.2 mm

c/t 17.59 = 21.62 ε

Class 1 web.

For Flange $c=b/2-t_w/2-r$

c 163.9

c/t 6.07 = 7.46 ε

Class 1 flange.

Overall Class 1 section.

Design Load

N_{Ed}	3546	kN
$M_{y,Ed}$	0	kNm
$M_{z,Ed}$	0	kNm

Cross-Section Resistance (Section 6.2.4 of BSEN1993-1-1)

$N_{c,Rd} = Af_y/\gamma_{M0}$ = 9123.5 kN

Because $N_{c,Rd}$ >= $N_{c,Ed}$ **OK**

Buckling Resistance (Section 6.3.1 of BSEN1993-1-1)

Axis	y-y	z-z	
L_{cr} (m)	6	6	
$N_{cr} = \pi^2EI/L^2$(kN)	38170.70	13644.7	
$\lambda = (Af_y/N_{cr})^{0.5}$	0.49	0.82	
Buckling Curve	b	c	*Table 6.2*
α	0.34	0.49	
$\Phi = 0.5[1+\alpha(\lambda-0.2)+\lambda^2)$	0.67	0.99	
$\chi = 1/[\Phi+(\Phi^2-\lambda^2)^{0.5}]$	0.89	0.65	
$N_{b,Rd} = \chi Af_y/\gamma_{M1}$ (kN)	8111.72	5939.74	

Because $N_{b,Rd}$ >= $N_{b,Ed}$ **OK**

Design Load for Peripheral Columns (Bottom - 4th Floor)

The slab is designed as one-way slab.

Tributary Area =	49	m^2
Assume ComFlor 60 Composite Slab of	150	mm
Weight of ComFlor Slab =	2.84	kPa

Assume 533 x 210 x 122 secondary beam,

Mass of Sec. Beam =	122	kg/m
Length of Sec. Beam supported by a column=	14.0	m/column
SW of Sec. Beam on each column =	17	kN/column

Assume 533x312x273 primary beam,

Mass of Primary Beam supported by a column=	273.3	kg/m
Length of Primary Beam =	7	m/column
SW of Primary Beam on each column =	55	kN/column

As the axial load is the largest at the bottom of columns

Max. Dead Load on a Column = 3025 kN

Max. Live Load on a Column = 1642 kN

Load Case

Load Case 1: 1.35 G_k + 1.5 Q_k

It is the most critical load case as the vertical forces are the greatest.

Factored Dead Load on a Column = 4083 kN

Factored Live Load on a Column = 2462 kN

Design of Compressive Columns

Section 356 x 406 x 235 UC

Section Properties

Grade	355	
f_y	355	MPa
h	381	mm
b	394.8	mm
t_w	18.4	mm
t_f	30.2	mm
r	15.2	mm
A	299	cm^2
E	210	GPa

Axis	y-y	z-z	
I (cm^4)	79100	31000	
Wpl (cm3)	4690	2380	*Class 1-2*
Wel (cm3)	4150	1570	*Class 3*

Classification of Steel Sections (Section 5.5.2 of BSEN1993-1-1)

ε 0.81

For Web, $c=h-2t_f-2r$

c 290.2 mm

c/t 15.77 = 19.38 ε

Class 1 web.

For Flange $c=b/2-t_w/2-r$

c 173

c/t 5.73 = 7.04 ε

Class 1 flange.

Overall Class 1 section.

Design Load

N_{Ed}	6546	kN
$M_{y,Ed}$	0	kNm
$M_{z,Ed}$	0	kNm

Cross-Section Resistance (Section 6.2.4 of BSEN1993-1-1)

$N_{c,Rd} = Af_y/\gamma_{M0} =$ 10614.5 kN

Because $N_{c,Rd}$ >= $N_{c,Ed}$ **OK**

Buckling Resistance (Section 6.3.1 of BSEN1993-1-1)

Axis	y-y	z-z	
L_{cr} (m)	6	6	
$N_{cr} = \pi^2EI/L^2$(kN)	45540.00	17847.5	
$\lambda = (Af_y/N_{cr})^{0.5}$	0.48	0.77	
Buckling Curve	b	c	*Table 6.2*
α	0.34	0.49	
$\Phi = 0.5[1+\alpha(\lambda-0.2)+\lambda^2)$	0.66	0.94	
$\chi = 1/[\Phi+(\Phi^2-\lambda^2)^{0.5}]$	0.89	0.68	
$N_{b,Rd} = \chi Af_y/\gamma_{M1}$ (kN)	9465.61	7220.51	

Because $N_{b,Rd}$ >= $N_{b,Ed}$ **OK**

Design Load for Columns near Cantilever Region (Proposal One)

There are three columns in cantilever region.
Assuming the loads are transferred to the two adjacent columns, each column takes 2.5 times of the normal load of a peripheral column. Based on the assumption, the load is 16365kN.
However, the loads were found to be spread to the row of columns behind the cantilever region.
To be conservative, a larger load from SAP2000 (19022kN) was taken for the analysis.

Design of Compressive Columns

Section 356 x 406 x 634 UC

Section Properties

Grade	355	
f_y	335	MPa
h	474.6	mm
b	424	mm
t_w	47.6	mm

t_f	77	mm
r	15.2	mm
A	808	cm^2
E	210	GPa

Axis	y-y	z-z	
I (cm^4)	275000	98100	
Wpl (cm3)	14200	7110	*Class 1 and 2*
Wel (cm3)	11600	4630	*Class 3*

<u>Classification of Steel Sections (Section 5.5.2 of BSEN1993-1-1)</u>

ε	0.84		
For Web,	$c=h-2t_f-2r$		
c	290.2	mm	
c/t	6.10	= 7.28 ε	7.28
Class	**1**	**web.**	
For Flange	$c=b/2-t_w/2-r$		
c	173		
c/t	2.25	= 2.68 ε	2.68
Class	**1**	**flange.**	
Overall Class	**1**	**section.**	

<u>Design Load</u>

N_{Ed}	19022	kN
$M_{y,Ed}$	0	kNm
$M_{z,Ed}$	0	kNm

<u>Cross-Section Resistance (Section 6.2.4 of BSEN1993-1-1)</u>

$N_{c,Rd} = Af_y/\gamma_{M0}$ =	27068	kN	
Because $N_{c,Rd}$	>=	$N_{c,Ed}$	**OK**

<u>Buckling Resistance (Section 6.3.1 of BSEN1993-1-1)</u>

Axis	y-y	z-z	
L_{cr} (m)	6	6	
$N_{cr} = \pi^2EI/L^2$(kN)	158324.90	56478.81	
$\lambda = (Af_y/N_{cr})^{0.5}$	0.41	0.69	
Buckling Curve	b	c	*Table6.2*
α	0.34	0.49	
$\Phi = 0.5[1+\alpha(\lambda-0.2)+\lambda^2)$	0.62	0.86	
$\chi = 1/[\Phi+(\Phi^2-\lambda^2)^{0.5}]$	0.92	0.73	
$N_{b,Rd} = \chi Af_y/\gamma_{M1}$ (kN)	24921.23	19744.9	

Because $N_{b,Rd}$ >= $N_{b,Ed}$ **OK**

Design Load for Columns at Cantilever and Wind Bracing (Proposal 2)

There are three columns in cantilever region.
Assuming the loads are transferred to the two asdjacent columns, each column takes 2.5 times of the normal load of a peripheral column.
Based on the assumption, the load is 16365kN and the value from SAP2000 is 21721kN.
To be conservative, take load as 21721kN for analysis.

Design of Compressive Columns

Section 356 x 406 x 1202 UC

Section Properties

Grade	355	
f_y	335	MPa
h	580	mm
b	471	mm
t_w	95	mm
t_f	130	mm
r	15.4	mm
A	1531	cm^2
E	210	GPa

Axis	y-y	z-z	
I (cm^4)	664000	229000	
Wpl (cm3)	30000	15200	*Class 1-2*
Wel (cm3)	22900	9710	*Class 3*

Classification of Steel Sections (Section 5.5.2 of BSEN1993-1-1)

ε	0.84	
For Web,	$c=h-2t_f-2r$	
c	289.2	mm
c/t	3.04	= 3.63 ε
Class	**1**	**web.**
For Flange	$c=b/2-t_w/2-r$	
c	172.6	
c/t	1.33	= 1.59 ε
Class	**1**	**flange.**
Overall Class	**1**	**section.**

Design Load

N_{Ed}	21721	kN
$M_{y,Ed}$	0	kNm
$M_{z,Ed}$	0	kNm

Cross-Section Resistance (Section 6.2.4 of BSEN1993-1-1)

$N_{c,Rd} = Af_y/\gamma_{M0} =$ 51288.5 kN

Because $N_{c,Rd}$ >= $N_{c,Ed}$ **OK**

Buckling Resistance (Section 6.3.1 of BSEN1993-1-1)

Axis	y-y	z-z	
L_{cr} (m)	6	6	
$N_{cr} = \pi^2EI/L^2$(kN)	382282.68	131841.4	
$\lambda = (Af_y/N_{cr})^{0.5}$	0.37	0.62	
Buckling Curve	b	c	*Table 6.2 of BSEN1993-1-1*
α	0.34	0.49	
$\Phi = 0.5[1+\alpha(\lambda-0.2)+\lambda^2)$	0.60	0.80	
$\chi = 1/[\Phi+(\Phi^2-\lambda^2)^{0.5}]$	0.94	0.77	
$N_{b,Rd} = \chi Af_y/\gamma_{M1}$ (kN)	48172.33	39555.86	

Because $N_{b,Rd}$ >= $N_{b,Ed}$ **OK**

Appendix 12

Calculation Spreadsheets for Wind Bracing

Actions of the Whole Building

North-south direction bracing:

Roof vertical loads:

Permanent action $g_{k,r} = 7.09\ kN/m^2$

Variable action $g_{k,r} = 1.5\ kN/m^2$

Floors vertical loads:

Permanent action $g_{k,r} = 7.21\ kN/m^2$

Variable action $g_{k,r} = 4 + 1.3 = 5.3\ \ kN/m^2$

Horizontal loads for both roof and floors:

Wind pressure $= 1.3\ kN/m^2$

Overall wind force coefficient $= 1.1$

The projected area of the vertical face of building (east-west direction) with consideration for bracing:$= 54 \times 38 = 2052\ m^2$

Characteristic wind load on the face of the building: $= 1.3 \times 1.1 \times 2052 = 2934.36\ kN$

Since four bracings is designed to resist the horizontal loads, the load on each bracing should be divided by $4 = 2934.36 \div 4 = 733.59\ kN$

East-west direction bracing:

Roof vertical loads:

Permanent action $g_{k,r} = 7.09\ kN/m^2$

Variable action $g_{k,r} = 1.5\ kN/m^2$

Floors vertical loads:

Permanent action $g_{k,r} = 7.21\ kN/m^2$

Variable action $g_{k,r} = 4\ kN/m^2$

Horizontal loads for both roof and floors:

Wind pressure $= 1.3\ kN/m^2$

Overall wind force coefficient $= 1.1$

The projected area of the vertical face of building (east-west direction) with consideration for bracing:$= 54 \times 60 = 2280\ m^2$

Characteristic wind load on the face of the building: $= 1.3 \times 1.1 \times 2280 = 3260.4\ kN$

Since four bracings is designed to resist the horizontal loads, the load on each bracing should be divided by $4 = 3260.4 \div 4 = 815.1\ kN$

Factors on actions

According to the National Annex to BS EN 1990. From NA.2.2.3.2 and Table NA.A1.2:

BS EN 1990 A1.3.1(4)

Partial factors:

BS EN 1990 Table NA.A1.2(B)

Permanent actions (Unfavourable) $\gamma_{\mathrm{Gj,sup}} = 1.35$

Reduction factor for unfavourable permanent actions (6.10b) $\xi = 0.925$

Variable actions (Unfavourable) $\gamma_{\mathrm{Q,1}} = 1.5$

Imposed load on floors and wind load $\gamma_{\mathrm{Q,i}} = 1.5$

Factors on accompanying actions:

BS EN 1990 Table NA.A1.1

Imposed loads on buildings for Category B: Office areas $\psi_0 = 0.7$

Wind loads on buildings $\psi_0 = 0.5$

Snow loads (Altitude < 1000 m above sea level) $\psi_0 = 0.5$

Note that for favourable actions:

Permanent actions (Favourable) $\gamma_{\mathrm{Gj,sup}} = 1$

Variable actions (Favourable) $\gamma_{\mathrm{Q,1}} = 0$

Imposed load on floors and wind load $\gamma_{\mathrm{Q,i}} = 0$

ULS with combination of actions

Design values of actions

Wind is assumed to be the leading action when checking resistance of bracing.

There are two directions bracing frames and three combinations for each of them. Combination 1, 2, 3 is for north-south direction and combination 4, 5, 6 is for east-west direction.

Combination 1 of North-South direction (1.35Gk + 1.50Qk + 0.75Wk + EHF) with 6.10b

Roof vertical loads:

$$0.925 \times 1.35G + 1.5 \times 0.5Q_{imp} = 0.925 \times 1.35 \times 7.09 + 1.5 \times 0.5 \times 1.5$$
$$= 9.98\ kN/m^2$$

Floor vertical loads:

$$0.925 \times 1.35G + 1.5 \times 0.7Q_{imp} = 0.925 \times 1.35 \times 7.21 + 1.5 \times 0.5 \times 5.3$$
$$= 14.57\ kN/m^2$$

Horizontal loads:

Design value of total horizontal wind load per bracing system for combination 1 is:

$$0.75 \times 733.59 = 550.19\ kN$$

Roof level= $\frac{4\times0.5}{38} \times 550.19 = 28.96\ kN$

2nd to 8th floor level= $\frac{2\times4\times0.5}{38} \times 550.19 = 202.70\ kN$

1st floor level= $\frac{(4+6)\times 0.5}{38} \times 550.19 = 72.39\ kN$

Combination 2 of North-South direction (1.35Gk + 1.05Qk + 1.50Wk + EHF) with 6.10b

Roof vertical loads:

$$0.925 \times 1.35G + 1.05 \times 0.5Q_{imp} = 0.925 \times 1.35 \times 7.09 + 1.05 \times 0.5 \times 1.5 = 9.64\ kN/m^2$$

Floor vertical loads:

$$0.925 \times 1.35G + 1.05 \times 0.7Q_{imp} = 0.925 \times 1.35 \times 7.21 + 1.05 \times 0.5 \times 5.3 = 12.90\ kN/m^2$$

Horizontal loads:

Design value of total horizontal wind load per bracing system for combination 2 is:

$$1.5 \times 733.59 = 1100.39kN$$

Design value of wind load:

Roof level= $\frac{4\times 0.5}{38} \times 1100.39 = 57.915\ kN$

2nd to 8th floor level= $\frac{2\times 4\times 0.5}{38} \times 1100.39 = 405.41\ kN$

1st floor level= $\frac{(4+6)\times 0.5}{38} \times 1100.39 = 144.79\ kN$

Combination 3 of North-South direction (1.0Gk + 1.50Wk + EHF) with 6.10b

Roof vertical loads:

$$0.925 \times G + 0 \times 0.5Q_{imp} = 0.925 \times 1 \times 7.09 + 0 \times 0.5 \times 1.5 = 6.56\ kN/m^2$$

Floor vertical loads:

$$0.925 \times G + 0 \times 0.7Q_{imp} = 0.925 \times 1 \times 7.21 + 0 \times 0.5 \times 5.3 = 6.67\ kN/m^2$$

Horizontal loads:

Design value of total horizontal wind load per bracing system for combination 3 is:

$$1.5 \times 733.59 = 1100.39kN$$

Design value of wind load:

Roof level= $\frac{4\times0.5}{38} \times 1100.39 = 57.915\ kN$

2nd to 8th floor level= $\frac{2\times4\times0.5}{38} \times 1100.39 = 405.41\ kN$

1st floor level= $\frac{(4+6)\times0.5}{38} \times 1100.39 = 144.79\ kN$

Wind leading combination calculated above can be used to check bracing system and a clear table is listed below:

Table A12.1 Loads with different combinations of north-south direction

	$G\ (kN/m^2)$	Q_{imp} (kN/m^2)	LC1 (kN/m^2)	LC2 (kN/m^2)	LC3 (kN/m^2)
Roof	7.09	1.5	9.98	9.64	6.56
Floor	7.21	5.3	14.57	12.90	6.67

Vertical design forces on columns of braced frame will be calculated in the column section.

Combination 4 of East-West direction (1.35Gk + 1.50Qk + 0.75Wk + EHF) with 6.10b

Roof vertical loads:

$$0.925 \times 1.35G + 1.5 \times 0.5Q_{imp} = 0.925 \times 1.35 \times 7.09 + 1.5 \times 0.5 \times 1.5$$
$$= 9.98\ kN/m^2$$

Floor vertical loads:

$$0.925 \times 1.35G + 1.5 \times 0.7Q_{imp} = 0.925 \times 1.35 \times 7.21 + 1.5 \times 0.5 \times 4$$
$$= 13.20\ kN/m^2$$

Horizontal loads:

Design value of total horizontal wind load per bracing system for combination 4 is:

$$0.75 \times 815.1 = 611.33\ kN$$

Roof level= $\frac{4\times0.5}{38} \times 611.33 = 32.18\ kN$

2nd to 8th floor level= $\frac{2\times4\times0.5}{38} \times 611.33 = 225.23\ kN$

1st floor level= $\frac{(4+6)\times0.5}{38} \times 611.33 = 80.44\ kN$

Combination 5 of East-West direction (1.35Gk + 1.05Qk + 1.50Wk + EHF) with 6.10b

Roof vertical loads:

$$0.925 \times 1.35G + 1.05 \times 0.5Q_{imp} = 0.925 \times 1.35 \times 7.09 + 1.05 \times 0.5 \times 1.5 = 9.64\ kN/m^2$$

Floor vertical loads:

$$0.925 \times 1.35G + 1.05 \times 0.7Q_{imp} = 0.925 \times 1.35 \times 7.21 + 1.05 \times 0.5 \times 4 = 11.94\ kN/m^2$$

Horizontal loads:

Design value of total horizontal wind load per bracing system for combination 5 is:

$$1.5 \times 815.1 = 1222.65kN$$

Design value of wind load:

Roof level= $\frac{4\times0.5}{38} \times 1222.65 = 64.35\ kN$

2nd to 8th floor level= $\frac{2\times4\times0.5}{38} \times 122 = 405.45\ kN$

1st floor level= $\frac{(4+6)\times0.5}{38} \times 1100.39 = 160.88\ kN$

Combination 6 of East-West direction (1.0Gk + 1.50Wk + EHF) with 6.10b

Roof vertical loads:

$$0.925 \times G + 0 \times 0.5Q_{imp} = 0.925 \times 1 \times 7.09 + 0 \times 0.5 \times 1.5 = 6.56\ kN/m^2$$

Floor vertical loads:

$$0.925 \times G + 0 \times 0.7Q_{imp} = 0.925 \times 1 \times 7.21 + 0 \times 0.5 \times 5.3 = 6.67\ kN/m^2$$

Horizontal loads:

Design value of total horizontal wind load per bracing system for combination 6 is:

$$1.5 \times 815.1 = 1222.65\ kN$$

Design value of wind load:

Roof level= $\frac{4\times0.5}{38} \times 1222.65 = 64.35\ kN$

2nd to 8th floor level= $\frac{2\times4\times0.5}{38} \times 1222.65 = 450.45\ kN$

1st floor level= $\frac{(4+6)\times0.5}{38} \times 1222.65 = 160.88\ kN$

Wind leading combination calculated above can be used to check bracing system and a clear table is listed below:

Table A12.2 Loads with different combinations of east-west direction

	$G\ (kN/m^2)$	$Q_{imp}\ (kN/m^2)$	LC1 (kN/m^2)	LC2 (kN/m^2)	LC3 (kN/m^2)
Roof	7.09	1.5	9.98	9.64	6.56
Floor	7.21	4	13.20	11.94	6.67

Sway stiffness

Combination 1

Vertical load on floor and on roof is different because they have different loads, but the area of each level is the same. For per bracing plane, the design value must be divided by 4. This is because all the four bracing planes are in the orthogonal direction, the horizontal force is assumed to be equally divided by these four bracing systems.

EHF per floor, per bracing plane:

Roof: $0.005 \times (0.25 \times 2835) \times 9.98 = 35.36\ kN$

Floor: $0.005 \times (0.25 \times 2835) \times 14.57 = 51.63\ kN$

Total horizontal loads H_{Ed} which is equal to wind load add EHF:

Roof: $28.96 + 35.36 = 64.32\ kN$

2nd floor to 8th floor: $202.70 + 51.63 = 254.33\ kN$

1st floor: $72.39 + 51.63 = 124.02\ kN$

Combination 2

EHF per floor, per bracing plane:

Roof: $0.005 \times (0.25 \times 2835) \times 9.64 = 34.17\ kN$

Floor: $0.005 \times (0.25 \times 2835) \times 12.90 = 45.71\ kN$

Total horizontal loads H_{Ed} which is equal to wind load add EHF:

Roof: $57.92 + 34.17 = 92.08\ kN$

2nd floor to 8th floor: $405.41 + 45.71 = 451.12\ kN$

1st floor: $144.79 + 45.71 = 190.50\ kN$

Combination 3

EHF per floor, per bracing plane:

Roof: $0.005 \times (0.25 \times 2835) \times 6.56 = 23.24\ kN$

Floor: $0.005 \times (0.25 \times 2835) \times 6.67 = 23.63\ kN$

Total horizontal loads H_{Ed} which is equal to wind load add EHF:

Roof: $57.92 + 23.24 = 81.16\ kN$

2nd floor to 8th floor: $405.41 + 23.63 = 429.04\ kN$

1st floor: $144.79 + 23.63 = 168.42\ kN$

Combination 4

EHF per floor, per bracing plane:

Roof: $0.005 \times (0.25 \times 2835) \times 9.98 = 35.36\ kN$

Floor: $0.005 \times (0.25 \times 2835) \times 13.20 = 46.79\ kN$

Total horizontal loads H_{Ed} which is equal to wind load add EHF:

Roof: $32.18 + 35.36 = 67.54\ kN$

2nd floor to 8th floor: $225.23 + 46.79 = 272.02\ kN$

1st floor: $80.44 + 46.79 = 127.23kN$

Combination 5

EHF per floor, per bracing plane:

Roof: $0.005 \times (0.25 \times 2835) \times 9.64 = 34.17\ kN$

Floor: $0.005 \times (0.25 \times 2835) \times 11.94 = 42.32\ kN$

Total horizontal loads H_{Ed} which is equal to wind load add EHF:

Roof: $64.35 + 35.36 = 98.52\ kN$

2nd floor to 8th floor: $450.45 + 42.32 = 492.78\ kN$

1st floor: $160.88 + 42.32 = 203.2\ kN$

Combination 6

EHF per floor, per bracing plane:

Roof: $0.005 \times (0.25 \times 2835) \times 6.56 = 23.24\ kN$

Floor: $0.005 \times (0.25 \times 2835) \times 6.67 = 23.63\ kN$

Total horizontal loads H_{Ed} which is equal to wind load add EHF:

Roof: $64.35 + 23.24 = 87.59\ kN$

2nd floor to 8th floor: $450.45 + 23.63 = 474.08\ kN$

1st floor: $160.88 + 23.63 = 184.51\ kN$

After calculating all the load needed for deflection, the size of bracing member can be selected:

Assumed column: UKC $356 \times 406 \times 1202$

Assumed beam: UKC $914 \times 419 \times 388$

Assumed bracing: CHS 508×14.2

Deflection with elastic analysis

Table A12.3 Combination 1

Storey	Storey height (m)	Inertia I (m^2)	Moment (kNm)	$\delta\theta_i$ (rads)	Storey inclination	Storey drift (mm)	Shear drift (mm)	Total deflection (mm)	Deflection added together (mm)
9	4	3.75	128638.59	6.53E-07	0.00068	2.72	0.074	2.80	32.26
8	4	3.75	894574.93	4.54E-06	0.00068	2.72	0.294	3.09	29.46
7	4	3.75	2677829.58	1.36E-05	0.00068	2.70	0.294	3.37	26.37
6	4	3.75	5478402.549	2.78E-05	0.00066	2.65	0.294	3.61	23.00
5	4	3.75	9296293.822	4.72E-05	0.00063	2.54	0.294	3.79	19.39
4	4	3.75	14131503.41	7.18E-05	0.00059	2.35	0.294	3.90	15.60
3	4	3.75	19984031.3	0.00010	0.00052	2.06	0.294	3.90	11.71
2	4	3.75	26853877.5	0.00014	0.00041	1.66	0.294	3.79	7.81
1	6	3.75	36449071.68	0.00028	0.00028	1.67	0.215	4.02	4.02

Deflection of combination 1 is 32.26mm which should be smaller than the limitation 63mm. To verify the accuracy of hand calculation, SAP2000 is used for getting results. U1 in figure A12.1 euqals to 30.65mm which means deflection on the horizontal direction. The error between hand calculation and sofware is 1.61 and it has about 5.25% error rate.

$$\frac{32.26 - 30.65}{30.65} \times 100\% = 5.25\%$$

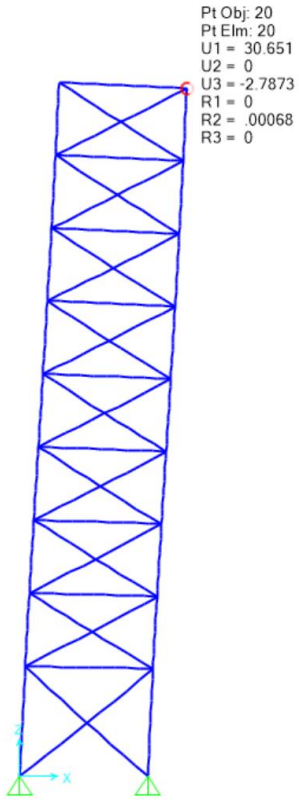


Figure A12.1 Software results for deflection of combination 1

Table A12.4 Combination 2

Storey	Storey height (m)	Inertia I (m^2)	Moment (kNm)	$\delta\theta_i$ (rads)	Storey inclination	Storey drift (mm)	Shear drift (mm)	Total deflection (mm)	Deflection added together (mm)
9	4	3.75	184161.562	9.35E-07	0.0012	4.75	0.11	4.86	56.35
8	4	3.75	1454716.26	7.39E-06	0.0012	4.75	0.52	5.37	51.50
7	4	3.75	4529734.106	2.30E-05	0.0012	4.72	0.52	5.87	46.12
6	4	3.75	9409215.099	4.78E-05	0.0012	4.62	0.52	6.30	40.26
5	4	3.75	16093159.24	8.17E-05	0.0011	4.43	0.52	6.63	33.96
4	4	3.75	24581566.53	0.00012	0.0010	4.11	0.52	6.82	27.33
3	4	3.75	34874436.97	0.00018	0.0009	3.61	0.52	6.85	20.51
2	4	3.75	46971770.55	0.00024	0.0007	2.90	0.52	6.66	13.67
1	6	3.75	63792721.86	0.00049	0.0005	2.92	0.33	7.01	7.01

Total horizontal deflection is 56.35mm and software result is 53.57mm. The error is $\frac{56.35-53.57}{53.57} \times 100\% = 5.19\%$ which is less than 10%. 56.35mm<63mm so that the total deflection is less than the limit.

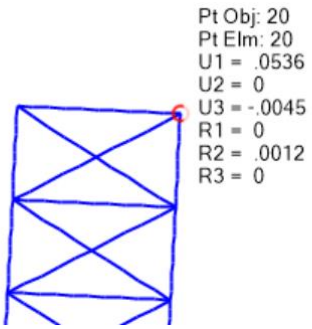


Figure A12.2 Software results for deflection of combination 2

Table A12.5 Combination 3

Storey	Storey height (m)	Inertia I (m^2)	Moment (kNm)	$\delta\theta_i$ (rads)	Storey inclination	Storey drift (mm)	Shear drift (mm)	Total deflection (mm)	Deflection added together (mm)
9	4	3.75	162311.60	8.24E-07	0.0011	4.49	0.093934	4.59	53.33
8	4	3.75	1345013.1	6.83E-06	0.0011	4.49	0.496591	5.079	48.75
7	4	3.75	4243871.22	2.16E-05	0.0011	4.46	0.496591	5.55	43.67
6	4	3.75	8858885.96	4.50E-05	0.0011	4.37	0.496591	5.96	38.12
5	4	3.75	15190057.32	7.71E-05	0.0010	4.19	0.496591	6.28	32.16
4	4	3.75	23237385.3	0.00012	0.00097	3.89	0.496591	6.46	25.89
3	4	3.75	33000869.9	0.00017	0.00085	3.41	0.496591	6.49	19.42
2	4	3.75	44480511.11	0.00023	0.00069	2.74	0.496591	6.31	12.94
1	6	3.75	60407925.48	0.00046	0.00046	2.76	0.291513	6.622	6.62

Total horizontal deflection is 53.33mm and software result is 50.70mm. The error is $\frac{53.33-50.70}{50.70} \times 100\% = 5.19\%$ which is less than 10%. 53.33mm<63mm so that the total deflection is less than the limit.

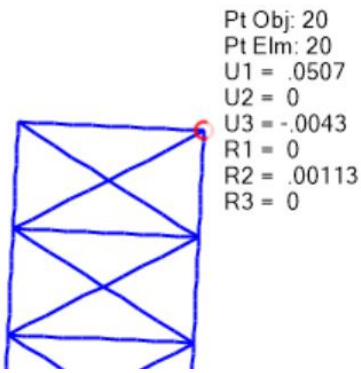


Figure A12.3 Software results for deflection of combination 3

Table A12.6 Combination 4

Storey	Storey height (m)	Inertia I (m^2)	Moment (kNm)	$\delta\theta_i$ (rads)	Storey inclination	Storey drift (mm)	Shear drift (mm)	Total deflection (mm)	Deflection added together (mm)
9	4	15.00	135073.59	1.84416E-07	0.000195503	0.782011063	0.111949135	0.893960199	23.08
8	4	15.00	949250.50	1.29601E-06	0.000195318	0.7812734	0.450892399	1.344114934	22.19
7	4	15.00	2851486.83	3.89313E-06	0.000194022	0.776089354	0.450892399	1.789823287	20.84
6	4	15.00	5841782.61	7.97578E-06	0.000190129	0.760516819	0.450892399	2.22514315	19.05
5	4	15.00	9920137.82	1.3544E-05	0.000182153	0.728613685	0.450892399	2.644132415	16.83
4	4	15.00	15086552.47	2.05977E-05	0.000168609	0.674437844	0.450892399	3.040848973	14.19
3	4	15.00	21341026.55	2.91369E-05	0.000148012	0.592047188	0.450892399	3.409350715	11.14
2	4	15.00	28683560.06	3.91617E-05	0.000118875	0.475499607	0.450892399	3.743695533	7.73
1	6	15.00	38923446.18	7.97132E-05	7.97132E-05	0.478279491	0.24359725	3.990072667	3.99

Total horizontal deflection is 23.14mm and software result is 21.60mm. The error is $\frac{23.14-21.60}{21.60} \times 100\% = 7.13\%$ which is less than 10%. 23.14mm<63mm so that the total deflection is less than the limit.

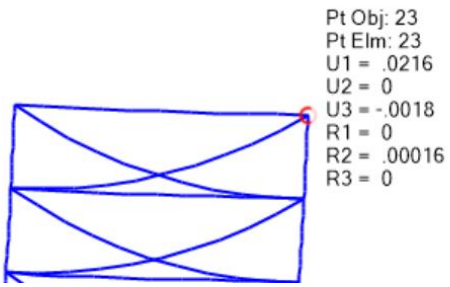


Figure A12.4 Software results for deflection of combination 4

Table A12.7 Combination 5

Storey	Storey height (m)	Inertia I (m^2)	Moment (kNm)	$\delta\theta_i$ (rads)	Storey inclination	Storey drift (mm)	Shear drift (mm)	Total deflection (mm)	Deflection added together (mm)
9	4	15.00	197031.56	2.69007 E-07	0.000348127	1.392507198	0.163299965	1.555807163	41.24
8	4	15.00	1576644.15	2.15259 E-06	0.000347858	1.39143117	0.816824429	2.371555563	39.68
7	4	15.00	4927355.68	6.72732 E-06	0.000345705	1.382820803	0.816824429	3.179769625	37.31
6	4	15.00	10249166.14	1.39932 E-05	0.000338978	1.355911536	0.816824429	3.969684787	34.13
5	4	15.00	17542075.54	2.39502 E-05	0.000324985	1.299938806	0.816824429	4.730536486	30.16
4	4	15.00	26806083.87	3.65983 E-05	0.000301035	1.204138051	0.816824429	5.45156016	25.43
3	4	15.00	38041191.14	5.19376 E-05	0.000264436	1.057744708	0.816824429	6.121991246	19.98
2	4	15.00	51247397.35	6.99681 E-05	0.000212499	0.849994215	0.816824429	6.731065182	13.86
1	6	15.00	69596691.14	0.000142531	0.000142531	0.855183014	0.389058587	7.125312568	7.13

Total horizontal deflection is 41.34mm and software result is 38.70mm. The error is $\frac{41.34-38.7}{21.60} \times 100\% = 6.82\%$ which is less than 10%. 41.34mm<63mm so that the total deflection is less than the limit.

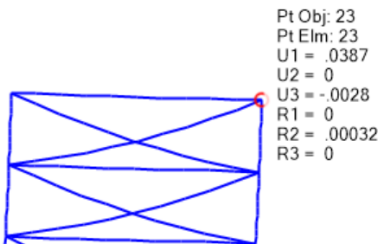


Figure A12.5 Software results for deflection of combination 5

Table A12.8 Combination 6

Storey	Storey height (m)	Inertia I (m^2)	Moment (kNm)	$\delta\theta_i$ (rads)	Storey inclination	Storey drift (mm)	Shear drift (mm)	Total deflection (mm)	Deflection added together (mm)
9	4	15.00	175181.60	2.39175 E-07	0.000333084	1.332334623	0.145190691	1.477525314	39.49
8	4	15.00	1473713.1	2.01206 E-06	0.000332844	1.331377922	0.785842886	2.262411498	38.02
7	4	15.00	4668581.22	6.37401 E-06	0.000330832	1.323329682	0.785842886	3.040206145	35.75
6	4	15.00	9759785.96	1.3325 E-05	0.000324458	1.297833634	0.785842886	3.800552982	32.71
5	4	15.00	16747327.32	2.28651 E-05	0.000311133	1.244533506	0.785842886	4.5330957 4	28.91
4	4	15.00	25631205.3	3.49943 E-05	0.000288268	1.153073029	0.785842886	5.227478149	24.38
3	4	15.00	36411419.9	4.97125 E-05	0.000253274	1.013095931	0.785842886	5.873343937	19.15
2	4	15.00	49087971.11	6.70198	0.0002035	0.8142	0.7858	6.4603368	13.28

1	6	15.00	66672397.98	E-05 0.00013 6542	61 0.0001365 42	45942 0.8192 50188	42886 0.3532 72466	34 6.8186135 45	6.82

Total horizontal deflection is 39.59mm and software result is 37.10mm. The error is $\frac{39.59-37.10}{37.10} \times 100\% = 6.71\%$ which is less than 10%. 39.59mm<63mm so that the total deflection is less than the limit.

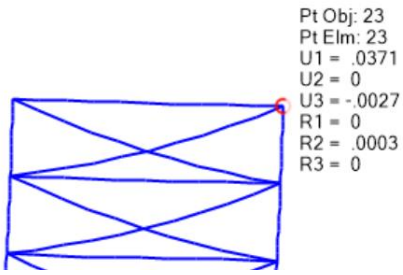


Figure A12.6 Software results for deflection of combination 6

α_{cr} has to be calculated for all combinations.

Combination 1

For the roof to 8th floor:

$H_{Ed} = 64.32\ kN$

$V_{Ed} = 0.25 \times 2835 \times 9.98 = 7072.36\ kN$

Also, V_{Ed} here is for each braced bay, so it must be divided by 4 (times 0.25).

$h = 4m$

$\delta = 2.80\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{64.32}{7072.36}\right)\left(\frac{4000}{2.8}\right) = 13.00$$

For 8th to 7th floor:

$H_{Ed} = 64.32 + 254.33 = 318.65\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57) = 17397.8\ kN$

$h = 4m$

$\delta = 3.09\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{318.65}{17397.8}\right)\left(\frac{4000}{3.09}\right) = 23.71$$

For 7th to 6th floor:

$H_{Ed} = 64.32 + 254.33 \times 2 = 572.98\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57 \times 2) = 27723.2\ kN$

$h = 4m$

$\delta = 3.37\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{572.98}{27723.2}\right)\left(\frac{4000}{3.37}\right) = 24.56$$

For 6th to 5th floor:

$H_{Ed} = 64.32 + 254.33 \times 3 = 827.31\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57 \times 3) = 38048.6\ kN$

$h = 4m$

$\delta = 3.61\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{827.31}{38048.6}\right)\left(\frac{4000}{3.61}\right) = 24.11$$

For 5th to 4th floor:

$H_{Ed} = 64.32 + 254.33 \times 4 = 1081.64\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57 \times 4) = 48374\ kN$

$h = 4m$

$\delta = 3.79\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1081.64}{48374}\right)\left(\frac{4000}{3.79}\right) = 23.60$$

For 4th to 3rd floor:

$$H_{Ed} = 64.32 + 254.33 \times 5 = 1335.97\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57 \times 5) = 58699.4\ kN$$

$$h = 4m$$

$$\delta = 3.90\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1335.97}{58699.4}\right)\left(\frac{4000}{3.90}\right) = 23.37$$

For 3rd to 2nd floor:

$$H_{Ed} = 64.32 + 254.33 \times 6 = 1590.3\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57 \times 6) = 69024.9\ kN$$

$$h = 4m$$

$$\delta = 3.90\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1590.3}{69024.9}\right)\left(\frac{4000}{3.90}\right) = 23.61$$

For 2nd to 1st floor:

$$H_{Ed} = 64.32 + 254.33 \times 7 = 1844.63\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57 \times 7) = 79350.3\ kN$$

$$h = 4m$$

$$\delta = 3.79\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1844.63}{79350.3}\right)\left(\frac{4000}{3.79}\right) = 24.53$$

For 1st floor to the ground:

$H_{Ed} = 64.32 + 254.33 \times 7 + 124.02 = 1968.65\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 14.57 \times 8) = 89675.7\ kN$

$h = 6m$

$\delta = 4.02\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1968.65}{89675.7}\right)\left(\frac{6000}{4.02}\right) = 32.80$$

The worst case is $\alpha_{cr} = 13.00 > 10$, so first order analysis alone is sufficient. Second order effects and load amplifier do not need to be considered.

Combination 2

For the roof to 8th floor:

$H_{Ed} = 92.08\ kN$

$V_{Ed} = 0.25 \times 2835 \times 9.64 = 6833.16\ kN$

Also, V_{Ed} here is for each braced bay, so it must be divided by 4 (times 0.25).

$h = 4m$

$\delta = 4.86\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{92.08}{6833.16}\right)\left(\frac{4000}{4.86}\right) = 11.10$$

For 8th to 7th floor:

$H_{Ed} = 92.08 + 451.116 = 543.20\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90) = 15975.3\ kN$

$h = 4m$

$\delta = 5.37\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{543.20}{15975.3}\right)\left(\frac{4000}{5.37}\right) = 25.31$$

For 7th to 6th floor:

$$H_{Ed} = 92.08 + 451.116 \times 2 = 994.31\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90 \times 2) = 25117.5\ kN$$

$$h = 4m$$

$$\delta = 5.87\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{994.31}{25117.5}\right)\left(\frac{4000}{5.87}\right) = 27.00$$

For 6th to 5th floor:

$$H_{Ed} = 92.08 + 451.116 \times 3 = 1445.43\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90 \times 3) = 34259.6\ kN$$

$$h = 4m$$

$$\delta = 6.30\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1445.43}{34259.6}\right)\left(\frac{4000}{6.30}\right) = 26.80$$

For 5th to 4th floor:

$$H_{Ed} = 92.08 + 451.116 \times 4 = 1896.54\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90 \times 4) = 43401.8\ kN$$

$$h = 4m$$

$$\delta = 6.63\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1896.54}{43401.8}\right)\left(\frac{4000}{6.63}\right) = 26.37$$

For 4th to 3rd floor:

$H_{Ed} = 92.08 + 451.116 \times 5 = 2347.66\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90 \times 5) = 52543.9\ kN$

$h = 4m$

$\delta = 6.82\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2347.66}{52543.9}\right)\left(\frac{4000}{6.82}\right) = 26.19$$

For 3rd to 2nd floor:

$H_{Ed} = 92.08 + 451.116 \times 6 = 2798.78\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90 \times 6) = 61686.1\ kN$

$h = 4m$

$\delta = 6.85\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2798.78}{61686.1}\right)\left(\frac{4000}{6.85}\right) = 26.51$$

For 2nd to 1st floor:

$H_{Ed} = 92.08 + 451.116 \times 7 = 3249.89\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90 \times 7) = 70828.3\ kN$

$h = 4m$

$\delta = 6.66\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3249.89}{70828.3}\right)\left(\frac{4000}{6.66}\right) = 27.56$$

For 1st floor to the ground:

$H_{Ed} = 92.08 + 451.116 \times 7 + 190.50 = 3440.39\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 12.90 \times 8) = 79970.4\ kN$

$h = 6m$

$\delta = 7.01\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3440.39}{79970.4}\right)\left(\frac{6000}{7.01}\right) = 36.84$$

The worst case is $\alpha_{cr} = 11.1 > 10$, so first order analysis alone is sufficient. Second order effects and load amplifier do not need to be considered.

Combination 3

For the roof to 8th floor:

$H_{Ed} = 81.16\ kN$

$V_{Ed} = 0.25 \times 2835 \times 6.56 = 4648.16\ kN$

Also, V_{Ed} here is for each braced bay, so it must be divided by 4 (times 0.25).

$h = 4m$

$\delta = 4.59\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{81.16}{4648.16}\right)\left(\frac{4000}{4.59}\right) = 15.23$$

For 8th to 7th floor:

$H_{Ed} = 81.16 + 429.04 = 510.20\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67) = 9374.99\ kN$

$h = 4m$

$\delta = 5.08\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{510.20}{9374.99}\right)\left(\frac{4000}{5.08}\right) = 42.86$$

For 7th to 6th floor:

$H_{Ed} = 81.16 + 429.04 \times 2 = 939.23\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 2) = 14101.8\ kN$

$h = 4m$

$\delta = 5.55\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{939.23}{14101.8}\right)\left(\frac{4000}{5.55}\right) = 48.02$$

For 6th to 5th floor:

$H_{Ed} = 81.16 + 429.04 \times 3 = 1368.27\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 3) = 18828.7\ kN$

$h = 4m$

$\delta = 5.96\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1368.27}{18828.7}\right)\left(\frac{4000}{5.96}\right) = 48.78$$

For 5th to 4th floor:

$H_{Ed} = 81.16 + 429.04 \times 4 = 1797.31\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 4) = 23555.5\ kN$

$h = 4m$

$\delta = 6.28\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1797.31}{23555.5}\right)\left(\frac{4000}{6.28}\right) = 48.64$$

For 4th to 3rd floor:

$H_{Ed} = 81.16 + 429.04 \times 5 = 2226.35\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 5) = 28282.3\ kN$

$h = 4m$

$\delta = 6.46\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2226.35}{28282.3}\right)\left(\frac{4000}{6.46}\right) = 48.72$$

For 3rd to 2nd floor:

$H_{Ed} = 81.16 + 429.04 \times 6 = 2655.39\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 6) = 33009.1\ kN$

$h = 4m$

$\delta = 6.49\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2655.39}{33009.1}\right)\left(\frac{4000}{6.49}\right) = 49.60$$

For 2nd to 1st floor:

$H_{Ed} = 81.16 + 429.04 \times 7 = 3084.43\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 7) = 37736\ kN$

$h = 4m$

$\delta = 6.31\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3084.43}{37736}\right)\left(\frac{4000}{6.31}\right) = 51.78$$

For 1st floor to the ground:

$H_{Ed} = 81.16 + 429.04 \times 7 + 168.42 = 3252.85\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 8) = 42462.8\ kN$

$h = 6m$

$\delta = 6.62\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3252.85}{42462.8}\right)\left(\frac{6000}{6.62}\right) = 69.41$$

The worst case is $\alpha_{cr} = 15.23 > 10$, so first order analysis alone is sufficient. Second order effects and load amplifier do not need to be considered.

Combination 4

For the roof to 8th floor:

$H_{Ed} = 67.54\ kN$

$V_{Ed} = 0.25 \times 2835 \times 9.98 = 7072.36\ kN$

Also, V_{Ed} here is for each braced bay, so it must be divided by 4 (times 0.25).

$h = 4m$

$\delta = 0.89\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{67.54}{7072.36}\right)\left(\frac{4000}{0.89}\right) = 42.73$$

For 8th to 7th floor:

$H_{Ed} = 67.54 + 272.02 = 339.55\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20) = 16430.3\ kN$

$h = 4m$

$\delta = 1.34\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{339.55}{16430.3}\right)\left(\frac{4000}{1.34}\right) = 61.50$$

For 7th to 6th floor:

$H_{Ed} = 67.54 + 272.02 \times 2 = 611.57\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20 \times 2) = 25788.3\ kN$

$h = 4m$

$\delta = 1.79mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{611.57}{25788.3}\right)\left(\frac{4000}{1.79}\right) = 53.00$$

For 6th to 5th floor:

$H_{Ed} = 67.54 + 272.02 \times 3 = 883.58\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20 \times 3) = 35146.3\ kN$

$h = 4m$

$\delta = 2.23\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{883.58}{35146.3}\right)\left(\frac{4000}{2.23}\right) = 45.19$$

For 5th to 4th floor:

$H_{Ed} = 67.54 + 272.02 \times 4 = 1155.6\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20 \times 4) = 44504.2\ kN$

$h = 4m$

$\delta = 2.64\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1155.6}{44504.2}\right)\left(\frac{4000}{2.64}\right) = 39.28$$

For 4th to 3rd floor:

$H_{Ed} = 67.54 + 272.02 \times 5 = 1427.61\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20 \times 5) = 53862.2\ kN$

$h = 4m$

$\delta = 3.04\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1427.61}{53862.2}\right)\left(\frac{4000}{3.04}\right) = 34.87$$

For 3rd to 2nd floor:

$H_{Ed} = 67.54 + 272.02 \times 6 = 1699.63\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20 \times 6) = 63220.2\ kN$

$h = 4m$

$\delta = 3.41\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1699.63}{63220.2}\right)\left(\frac{4000}{3.41}\right) = 31.54$$

For 2nd to 1st floor:

$H_{Ed} = 67.54 + 272.02 \times 7 = 1971.64\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20 \times 7) = 72578.2\ kN$

$h = 4m$

$\delta = 3.74\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1971.64}{72578.2}\right)\left(\frac{4000}{3.74}\right) = 29.03$$

For 1st floor to the ground:

$H_{Ed} = 67.54 + 272.02 \times 7 + 127.23 = 2098.87\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.98 + 13.20 \times 8) = 81936.1\ kN$

$h = 6m$

$\delta = 4.00\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3252.85}{42462.8}\right)\left(\frac{6000}{4.00}\right) = 38.52$$

The worst case is $\alpha_{cr} = 29.03 > 10$, so first order analysis alone is sufficient. Second order effects and load amplifier do not need to be considered.

Combination 5

For the roof to 8th floor:

$H_{Ed} = 98.52\ kN$

$V_{Ed} = 0.25 \times 2835 \times 9.64 = 6833.16\ kN$

Also, V_{Ed} here is for each braced bay, so it must be divided by 4 (times 0.25).

$h = 4m$

$\delta = 1.56\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{98.52}{6833.16}\right)\left(\frac{4000}{1.56}\right) = 37.07$$

For 8th to 7th floor:

$H_{Ed} = 98.52 + 492.78 = 591.29\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94) = 15298.1\ kN$

$h = 4m$

$\delta = 2.37\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{591.29}{15298.1}\right)\left(\frac{4000}{2.37}\right) = 65.19$$

For 7th to 6th floor:

$H_{Ed} = 98.52 + 492.78 \times 2 = 1084.07\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94 \times 2) = 23763\ kN$

$h = 4m$

$\delta = 3.18mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1084.07}{23763}\right)\left(\frac{4000}{3.18}\right) = 57.39$$

For 6th to 5th floor:

$H_{Ed} = 98.52 + 492.78 \times 3 = 1576.84\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94 \times 3) = 32228\ kN$

$h = 4m$

$\delta = 3.97\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1576.84}{32228}\right)\left(\frac{4000}{3.97}\right) = 49.30$$

For 5th to 4th floor:

$H_{Ed} = 98.52 + 492.78 \times 4 = 2069.61\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94 \times 4) = 40692.9\ kN$

$h = 4m$

$\delta = 4.73\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2069.61}{40692.9}\right)\left(\frac{4000}{4.73}\right) = 43.00$$

For 4th to 3rd floor:

$H_{Ed} = 98.52 + 492.78 \times 5 = 2562.39\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94 \times 5) = 49157.9\ kN$

$h = 4m$

$\delta = 5.45\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2562.39}{49157.9}\right)\left(\frac{4000}{5.45}\right) = 38.25$$

For 3rd to 2nd floor:

$H_{Ed} = 98.52 + 492.78 \times 6 = 3055.16\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94 \times 6) = 57622.8\ kN$

$h = 4m$

$\delta = 6.12mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3055.16}{57622.8}\right)\left(\frac{4000}{6.12}\right) = 34.64$$

For 2nd to 1st floor:

$H_{Ed} = 98.52 + 492.78 \times 7 = 3547.94\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94 \times 7) = 66087.8\ kN$

$h = 4m$

$\delta = 6.73\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3547.94}{66087.8}\right)\left(\frac{4000}{6.73}\right) = 31.90$$

For 1st floor to the ground:

$H_{Ed} = 98.52 + 492.78 \times 7 + 203.2 = 3751.14\ kN$

$V_{Ed} = 0.25 \times 2835 \times (9.64 + 11.94 \times 8) = 74552.7\ kN$

$h = 6m$

$\delta = 7.13\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3751.14}{74552.7}\right)\left(\frac{6000}{7.13}\right) = 42.37$$

The worst case is $\alpha_{cr} = 31.90 > 10$, so first order analysis alone is sufficient.

Second order effects and load amplifier do not need to be considered.

Combination 6

For the roof to 8th floor:

$H_{Ed} = 87.59\ kN$

$V_{Ed} = 0.25 \times 2835 \times 6.56 = 4648.16\ kN$

Also, V_{Ed} here is for each braced bay, so it must be divided by 4 (times 0.25).

$h = 4m$

$\delta = 1.48\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{87.59}{4648.16}\right)\left(\frac{4000}{1.48}\right) = 51.02$$

For 8th to 7th floor:

$H_{Ed} = 87.59 + 474.08 = 561.68\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67) = 9374.99\ kN$

$h = 4m$

$\delta = 2.26mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{561.68}{9374.99}\right)\left(\frac{4000}{2.26}\right) = 105.93$$

For 7th to 6th floor:

$H_{Ed} = 87.59 + 474.08 \times 2 = 1035.76\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 2) = 14101.8\ kN$

$h = 4m$

$\delta = 3.04mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1035.76}{14101.8}\right)\left(\frac{4000}{3.04}\right) = 96.64$$

For 6th to 5th floor:

$$H_{Ed} = 87.59 + 474.08 \times 3 = 1509.84\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 3) = 18828.7\ kN$$

$$h = 4m$$

$$\delta = 3.80\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1509.84}{18828.7}\right)\left(\frac{4000}{3.80}\right) = 84.40$$

For 5th to 4th floor:

$$H_{Ed} = 87.59 + 474.08 \times 4 = 1983.93\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 4) = 23555.5\ kN$$

$$h = 4m$$

$$\delta = 4.53\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{1983.93}{23555.5}\right)\left(\frac{4000}{4.53}\right) = 74.32$$

For 4th to 3rd floor:

$$H_{Ed} = 87.59 + 474.08 \times 5 = 2458.01\ kN$$

$$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 5) = 28282.3\ kN$$

$$h = 4m$$

$$\delta = 5.23\ mm$$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2458.01}{28282.3}\right)\left(\frac{4000}{5.23}\right) = 66.50$$

For 3rd to 2nd floor:

$H_{Ed} = 87.59 + 474.08 \times 6 = 2932.1\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 6) = 33009.1\ kN$

$h = 4m$

$\delta = 5.87\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{2932.1}{33009.1}\right)\left(\frac{4000}{5.87}\right) = 60.49$$

For 2nd to 1st floor:

$H_{Ed} = 87.59 + 474.08 \times 7 = 3406.18\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 7) = 37736\ kN$

$h = 4m$

$\delta = 6.46\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3406.18}{37736}\right)\left(\frac{4000}{6.46}\right) = 55.89$$

For 1st floor to the ground:

$H_{Ed} = 87.59 + 474.08 \times 7 + 184.51 = 3590.69\ kN$

$V_{Ed} = 0.25 \times 2835 \times (6.56 + 6.67 \times 8) = 42462.8\ kN$

$h = 6m$

$\delta = 6.82\ mm$

$$\alpha_{cr} = \left(\frac{H_{Ed}}{V_{Ed}}\right)\left(\frac{h}{\delta_{H,Ed}}\right) = \left(\frac{3590.69}{42462.8}\right)\left(\frac{6000}{6.82}\right) = 74..41$$

The worst case is $\alpha_{cr} = 51.02 > 10$, so first order analysis alone is sufficient. Second order effects and load amplifier do not need to be considered.

Therefore, all combinations can be analysed with first order effect and none of them

need amplifier.

Design of bracing

From the calculation for α_{cr}, H_{Ed} for each floor is figured out and it is found that H_{Ed} between 1st floor and the ground is the largest. This can be used to find critical design load in diagonal bracing member.

Axial force in diagonal member

Combination 1

Horizontal force, 1st floor to ground $= 1968.65\ kN$

Diagonal member axial force in compression $= 1968.65 \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 1296.43\ kN$

Combination 2

Horizontal force, 1st floor to ground $= 3440.39\ kN$

Diagonal member axial force in compression $= 3440.39 \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 2265.63\ kN$

Combination 3

Horizontal force, 1st floor to ground $= 3252.85\ kN$

Diagonal member axial force in compression $= 3252.85 \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 2142.13\ kN$

Combination 1, 2 and 3 have the same structure of frame, so the buckling resistance is the same. The design force needed to be resist is largest one of this three combinations, which is $N_{Ed} = 2265.63\ kN$.

Combination 4

Horizontal force, 1st floor to ground $= 2098.87\ kN$

Diagonal member axial force in compression $= 2098.87 \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 1141.75\ kN$

Combination 5

Horizontal force, 1st floor to ground $= 3751.14\ kN$

Diagonal member axial force in compression $= 3751.14 \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 2040.56\ kN$

Combination 6

Horizontal force, 1st floor to ground $= 3590.69\ kN$

Diagonal member axial force in compression $= 3590.69 \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 1953.28\ kN$

Combination 4, 5 and 6 have the same structure of frame, so the buckling resistance is the same. The design force needed to be resist is largest one of this three combinations, which is $N_{Ed} = 2040.56\ kN$.

Member checks

Buckling length

The effective length for member is:

$L_{cr} = 1.0 \times \sqrt{6^2 + 7^2} \times 10^3 = 9219.54mm$ for north-south bracing frames (combination 1, 2, 3)

$L_{cr} = 1.0 \times \sqrt{6^2 + 14^2} \times 10^3 = 15231.5mm$ for east-west bracing frames (combination 4, 5, 6)

Slenderness for flexural buckling

Combination 1, 2 and 3

For axial compression in members, non-dimensional slenderness $\bar{\lambda}$ is:

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} = \left(\frac{L_{cr}}{i}\right)\left(\frac{1}{\lambda_1}\right) \quad \text{for Class 1, 2 and 3 cross-sections}$$

BSEN1993-1-1 6.3.1.3 (1) Eqn. (6.50)

Where L_{cr} is the buckling length calculated before and i is radius of gyration about

the axis. λ_1 can be calculated with equation: $\lambda_1 = \pi\sqrt{\frac{E}{f_y}} = 93.9\varepsilon$ and $\varepsilon = \sqrt{\frac{235}{f_y}}$

$$\varepsilon = \sqrt{235/f_y} = \sqrt{235/355} = 0.8136$$

All sections in the structure are S355, therefore $f_y = 355\ N/mm^2$

$$\lambda_1 = 93.9\varepsilon = 93.9 \times 0.81 = 76.3986$$

All the diagonal sections use hollow section, so radius of gyration about x and y are the same. For CHS 508×14.2, radius of gyration i is 175mm.

$$\bar{\lambda} = \left(\frac{L_{cr}}{i}\right)\left(\frac{1}{\lambda_1}\right) = \left(\frac{9219.54}{175}\right)\left(\frac{1}{76.3986}\right) = 0.69 > 0.2$$

For $\bar{\lambda} \leq 0.2$, the buckling effects can be ignored. However, $\bar{\lambda} > 0.2$, so reduction factor χ must be calculated for buckling resistance.

BSEN1993-1-1 6.3.1.2(4)

Combination 4, 5 and 6

$$\bar{\lambda} = \left(\frac{L_{cr}}{i}\right)\left(\frac{1}{\lambda_1}\right) = \left(\frac{15231.5}{175}\right)\left(\frac{1}{76.3986}\right) = 1.14 > 0.2$$

$\bar{\lambda}$ for the three combinations is also larger than 0.2, so reduction factor χ is also needed.

Design buckling resistance

Requirement for section:

$$\frac{N_{Ed}}{N_{b,Rd}} \leq 1.0$$

BSEN1993-1-1 6.3.1.1(1) Eqn. (6.46)

Design buckling resistance is:

$$N_{b,Rd} = \frac{\chi\, A f_y}{\gamma_{M1}} \text{ (For Class 1, 2 and cross sections)}$$

BSEN1993-1-1 6.3.1.1(3) Eqn. (6.47)

Where $\gamma_{M1} = 1.0$, $\chi = \frac{1}{\phi^2 + \sqrt{\phi^2 - \bar{\lambda}^2}} \leq 1.0$ and $\phi = 0.5 \times (1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2)$

A is section area and $f_y = 355\ N/mm^2$.

BSEN1993-1-1 Table 6.2

From Table 6.2 and 6.1 in EN 1993-1-1, the buckling curve for any hot finished hollow section is "a" and imperfection factor α for curve a is 0.21.

BSEN1993-1-1 Table 6.1

Combination 1, 2 and 3

$$\phi = 0.5 \times \left(1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right) = 0.5 \times (1 + 0.21 \times (0.69 - 0.2) + 0.69^2) = 0.79$$

BSEN1993-1-1 6.3.1.2(1)

$$\chi = \frac{1}{\phi^2 + \sqrt{\phi^2 - \bar{\lambda}^2}} = \frac{1}{0.79 + \sqrt{0.79^2 - 0.69^2}} = 0.85 < 1.0$$

$$N_{b,Rd} = \frac{\chi\, A f_y}{\gamma_{M1}} = \frac{0.85 \times 0.022 \times 10^6 \times 355}{1} \times 10^{-3} = 6658.62\ kN$$

BSEN1993-1-1 Eqn. (6.47)

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{2265.63}{6658.62} = 0.34 < 1.0$$

The design buckling resistance of the section is adequate for combination 1, 2 and 3 which are north-south directional bracing frames.

Combination 4, 5 and 6

$$\phi = 0.5 \times \left(1 + \alpha\left(\bar{\lambda} - 0.2\right) + \bar{\lambda}^2\right) = 0.5 \times (1 + 0.21 \times (1.14 - 0.2) + 1.14^2) = 1.25$$

$$\chi = \frac{1}{\phi^2 + \sqrt{\phi^2 - \bar{\lambda}^2}} = \frac{1}{1.25 + \sqrt{1.25^2 - 1.14^2}} = 0.57 < 1.0$$

BSEN1993-1-1 6.3.1.2(1)

$$N_{b,Rd} = \frac{\chi\, A f_y}{\gamma_{M1}} = \frac{0.57 \times 0.022 \times 10^6 \times 355}{1} \times 10^{-3} = 4447.52\ kN$$

$$\frac{N_{Ed}}{N_{b,Rd}} = \frac{2040.56}{4447.52} = 0.46 < 1.0$$

The design buckling resistance of the section if adequate for combination 4, 5 and 6 which are north-south directional bracing frames.

Verification of bracing adjacent to column splices

Combination 1

BS EN 1993-1-1, 5.3.3(4)

External loading

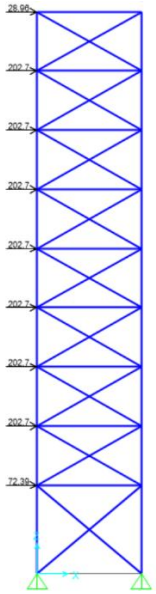


Figure A12.7 Bracing frame with only external load

Without equivalent horizontal force, figure A12.7 shows the external loading (wind load) on the frame. Then, axial forces in the bracing elements can be calculated. The forces are accumulated to the lower level.

Calculation process:

Ground to 8th floor= $28.96 \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 16.68\ kN$

8th floor to 7th floor= $(28.96 + 202.7) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 133.41\ kN$

7th floor to 6th floor= $(28.96 + 202.7 \times 2) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 250.14\ kN$

6th floor to 5th floor= $(28.96 + 202.7 \times 3) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 366.87\ kN$

5th floor to 4th floor= $(28.96 + 202.7 \times 4) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 483.60\ kN$

4th floor to 3rd floor= $(28.96 + 202.7 \times 5) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 600.33\ kN$

3rd floor to 2nd floor= $(28.96 + 202.7 \times 6) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 717.064\ kN$

2nd floor to 1st floor= $(28.96 + 202.7 \times 7) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 833.80\ kN$

1st floor to the ground $= (28.96 + 202.7 \times 7 + 72.39) \times \frac{\sqrt{6^2+7^2}}{7} \times 0.5 = 1001.15\ kN$

Local lateral force is:

$$\alpha_m N_{Ed}/1000$$

Where $\alpha_m = \sqrt{0.5(1 + \frac{1}{m})}$ and m is the number of columns to be restrained

$$\alpha_m = \sqrt{0.5(1 + \frac{1}{m})} = \sqrt{0.5(1 + \frac{1}{91})} = 0.50$$

Total axial force in columns $= 9.98 \times 2835 + 14.57 \times 2835 = 69591.1\ kN$

Local force for each bracing frame $= 0.25 \times 0.50 \times \frac{69591.1}{100} = 87.47\ kN$

The local force is assumed to be split equally between above and below bracing member, the resultant force in the diagonal member between 2nd floor and 1st floor levels is:

$$87.47 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 50.37\ kN$$

Total maximum force= $833.30 + 50.37 = 884.17\ kN < 6658.62\ kN$

The local imperfection forces at splices can be carried by the bracing.

Combination 2

External loading

2nd floor to 1st floor= $(57.92 + 405.41) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 1667.59\ kN$

Lateral force

Total axial force in columns $= 9.64 \times 2835 + 12.90 \times 2835 = 63901.3\ kN$

Local force for each bracing frame $= 0.25 \times 0.50 \times \frac{63901.3}{100} = 80.31\ kN$

Resultant force in the diagonal member between 2nd floor and 1st floor levels

$$80.31 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 46.25\ kN$$

Total maximum force$= 1667.59 + 46.25 = 1713.84\ kN < 6658.62\ kN$

The local imperfection forces at splices can be carried by the bracing.

Combination 3

External loading

2nd floor to 1st floor$= (57.92 + 405.41) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 1667.59\ kN$

Lateral force

Total axial force in columns $= 6.56 \times 2835 + 6.67 \times 2835 = 37500\ kN$

Local force for each bracing frame $= 0.25 \times 0.50 \times \frac{37500}{100} = 47.13\ kN$

Resultant force in the diagonal member between 2nd floor and 1st floor levels

$$47.13 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 27.14\ kN$$

Total maximum force$= 1667.59 + 27.14 = 1694.73\ kN < 6658.62\ kN$

The local imperfection forces at splices can be carried by the bracing.

Combination 4

External loading

2nd floor to 1st floor$= (32.18 + 225.23) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 836.56\ kN$

Lateral force

Total axial force in columns $= 9.98 \times 2835 + 13.20 \times 2835 = 65721.3\ kN$

Local force for each bracing frame $= 0.25 \times 0.50 \times \frac{69721.3}{100} = 82.60\ kN$

Resultant force in the diagonal member between 2nd floor and 1st floor levels

$$82.60 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 42.95\ kN$$

Total maximum force$= 836.56 + 42.95 = 879.52\ kN < 6658.62\ kN$

The local imperfection forces at splices can be carried by the bracing

Combination 5

External loading

2nd floor to 1st floor$= (64.35 + 450.45) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 1673.13\ kN$

Lateral force

Total axial force in columns $= 9.64 \times 2835 + 11.94 \times 2835 = 61192.4\ kN$

Local force for each bracing frame $= 0.25 \times 0.50 \times \frac{61192.4}{100} = 76.91\ kN$

Resultant force in the diagonal member between 2nd floor and 1st floor levels

$$76.91 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 40.00\ kN$$

Total maximum force$= 1673.13 + 40.00 = 1713.12\ kN < 6658.62\ kN$

The local imperfection forces at splices can be carried by the bracing.

Combination 6

External loading

2nd floor to 1st floor$= (64.35 + 450.45) \times \frac{\sqrt{4^2+7^2}}{7} \times 0.5 = 1673.13\ kN$

Lateral force

Total axial force in columns $= 6.56 \times 2835 + 6.67 \times 2835 = 37500\ kN$

Local force for each bracing frame $= 0.25 \times 0.50 \times \frac{37500}{100} = 47.13\ kN$

Resultant force in the diagonal member between 2nd floor and 1st floor levels

$$47.13 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 24.51\ kN$$

Total maximum force$= 1673.13 + 24.51 = 1697.63\ kN < 6658.62\ kN$

The local imperfection forces at splices can be carried by the bracing.

Verification of restraint to columns provided by bracing

Equation for the process is:

$$H = \phi N_{Ed}$$

BS EN 1993-1-1, 5.3.2(5)

Where has been mentioned in 3.2.6.4 sway stiffness: $\phi = \phi_0 \alpha_h \alpha_m$ and $\phi_0 = \frac{1}{200} \alpha_h = 1.0\ \alpha_m = \sqrt{0.5(1 + \frac{1}{m})} = \sqrt{0.5 \times (1 + \frac{1}{91})} = 0.50$

$$\phi = \frac{1}{200} \times 0.5 = 0.00251$$

Combination 1

Total axial force in columns $= 9.98 \times 2835 + 14.57 \times 2835 \times 8 = 358703\ kN$

This means all the vertical forces on 9 stories in total.

Local force per bracing system $= 0.25 \times 0.0025 \times 358703 = 225.42\ kN$

Similarly, the local force is also assumed to be split equally as imperfection of column splices.

Resultant force in diagonal member

$$225.42 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 129.81\ kN$$

Axial force with external force applying between 1st floor and the ground

$$(28.96 + 202.7 \times 7 + 72.39) \times \frac{\sqrt{6^2 + 7^2}}{7} \times 0.5 = 1001.15\ kN$$

Total maximum force= $1001.15 + 129.81 = 1130.97\ kN < 6658.62\ kN$

Therefore, the force can be carried by the frame.

Combination 2

Total axial force in columns $= 9.64 \times 2835 + 12.90 \times 2835 \times 8 = 319882\ kN$

This means all the vertical forces on 9 stories in total.

Local force per bracing system $= 0.25 \times 0.0025 \times 319882 = 201.02\ kN$

Similarly, the local force is also assumed to be split equally as imperfection of column splices.

Resultant force in diagonal member

$$201.02 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 115.76\ kN$$

Axial force with external force applying between 1st floor and the ground

$$(57.92 + 405.41 \times 7 + 144.79) \times \frac{\sqrt{6^2 + 7^2}}{7} \times 0.5 = 2002.31\ kN$$

Total maximum force= $2002.31 + 115.76 = 2118.08\ kN < 6658.62\ kN$

Therefore, the force can be carried by the frame.

Combination 3

Total axial force in columns $= 6.56 \times 2835 + 6.67 \times 2835 \times 8 = 169851\ kN$

This means all the vertical forces on 9 stories in total.

Local force per bracing system $= 0.25 \times 0.0025 \times 169851 = 106.74\ kN$

Similarly, the local force is also assumed to be split equally as imperfection of column splices.

Resultant force in diagonal member

$$106.74 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 61.47\ kN$$

Axial force with external force applying between 1st floor and the ground

$$(57.92 + 405.41 \times 7 + 144.79) \times \frac{\sqrt{6^2 + 7^2}}{7} \times 0.5 = 2002.31\ kN$$

Total maximum force= $2002.31 + 61.47 = 2063.78\ kN < 6658.62\ kN$

Therefore, the force can be carried by the frame.

Combination 4

Total axial force in columns $= 9.98 \times 2835 + 13.20 \times 2835 \times 8 = 327745\ kN$

This means all the vertical forces on 9 stories in total.

Local force per bracing system $= 0.25 \times 0.0025 \times 327745 = 205.96\ kN$

Similarly, the local force is also assumed to be split equally as imperfection of column splices.

Resultant force in diagonal member

$$205.96 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 107.10\ kN$$

Axial force with external force applying between 1st floor and the ground

$$(32.18 + 225.23 \times 7 + 80.44) \times \frac{\sqrt{6^2 + 7^2}}{7} \times 0.5 = 918.89\ kN$$

Total maximum force= $918.89 + 107.10 = 1025.99\ kN < 6658.62\ kN$

Therefore, the force can be carried by the frame.

Combination 5

Total axial force in columns $= 9.64 \times 2835 + 11.94 \times 2835 \times 8 = 298211\ kN$

This means all the vertical forces on 9 stories in total.

Local force per bracing system $= 0.25 \times 0.0025 \times 298211 = 187.40\ kN$

Similarly, the local force is also assumed to be split equally as imperfection of column splices.

Resultant force in diagonal member

$$187.40 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 97.45\ kN$$

Axial force with external force applying between 1st floor and the ground

$$(64.35 + 450.45 \times 7 + 160.88) \times \frac{\sqrt{6^2 + 7^2}}{7} \times 0.5 = 1837.78\ kN$$

Total maximum force$= 1837.78 + 97.45 = 1935.23\ kN < 6658.62\ kN$

Therefore, the force can be carried by the frame.

Combination 6

Total axial force in columns $= 6.56 \times 2835 + 6.67 \times 2835 \times 8 = 169851\ kN$

This means all the vertical forces on 9 stories in total.

Local force per bracing system $= 0.25 \times 0.0025 \times 169851 = 106.74\ kN$

Similarly, the local force is also assumed to be split equally as imperfection of column splices.

Resultant force in diagonal member

$$106.74 \times \frac{\sqrt{4^2 + 7^2}}{7} \times 0.5 = 55.51\ kN$$

Axial force with external force applying between 1st floor and the ground

$$(64.35 + 450.45 \times 7 + 160.88) \times \frac{\sqrt{6^2 + 7^2}}{7} \times 0.5 = 1837.78\ kN$$

Total maximum force= $1837.78 + 55.51 = 1893.29\ kN < 6658.62\ kN$

Therefore, the force can be carried by the frame.

In conclusion, the section selected fulfil every limitation and the deflection limited is close to the requirement which means this is an effective choice. Also, resistance to the various imperfections and external load is sufficient for each bracing member.

Appendix A13

Cantilever Truss System Design

A13.1 Load Cases of Cantilevered Truss

The bearing area of each column is:

(m^2)

Col. A	Col. B	Col. C	Col. D	Col. E	Col. F	Col. G
47.25	57.38	33.75	67.50	67.50	67.50	33.75

Cladding load of perimeter beam:

(kN)

Area A	Area B	Area C	Area D	Area E	Area F	Area G
36.4	44.2	61.1	70.2	70.2	70.2	32.75

Roof load to each column:

(kN)

Col. A	Col. B	Col. C	Col. D	Col. E	Col. F	Col. G
508.84	617.87	410.84	751.48	751.48	751.48	372.57

Floor load to each column:

(kN)

Col. A	Col. B	Col. C	Col. D	Col. E	Col. F	Col. G
768.95	933.72	596.63	1123.1	1123.1	1123.1	558.36

Total load (Point load) on truss:

(kN)

Point A	Point B	Point C	Point D	Point E	Point F	Point G
3584.6	4352.8	2797.4	5243.7	5243.7	5243.7	2606.0

A13.2 Proposal 1

Preliminary Truss Design

$N_1 = \frac{M}{D}$, *Assume* $\boldsymbol{N}_{b,Rd} = 0.70\boldsymbol{f}_y A$

$\therefore \frac{\boldsymbol{N}_1}{0.70\boldsymbol{f}_y} \Rightarrow$ *Area of cross section required*

$Maximum\ bending\ moment\ M_{max} = 70790.46\ kNm$

$\therefore N_1 = \frac{M}{D} = 5899.21\ kN$

$\Rightarrow A = \frac{\boldsymbol{N}_1}{0.70\boldsymbol{f}_y} = 237.39\ cm^2$

$And\ the\ deflection\ of\ the\ truss\ can\ also\ be\ determined\ in$

$an\ approximate\ way$:

$The\ second\ moment: I = \frac{AD^2}{2} = 1.71 \times 10^{12}\ cm^4$

$Deflection\ caused\ by\ point\ load\ at\ D$:

$\delta_{mid,D} = -\vartheta(L/2) = \frac{P_D b_D(3L^2 - 4b_D^2)}{48EI} = 17.22\ mm\ (\downarrow)$

$Deflection\ caused\ by\ point\ load\ at\ E$:

$\delta_{mid,E} = -\vartheta(L/2) = \frac{P_E b_E(3L^2 - 4b_E^2)}{48EI} = 17.22\ mm\ (\downarrow)$

$Simply\ consider\ that\ the\ uniform\ distribute\ load\ q\ is\ equal$

$to\ the\ ratio\ of\ the\ point\ load\ on\ the\ column\ to\ the\ width\ of$

$the\ bearing\ area\ of\ the\ column\ load$:

$$q = \frac{4P_{Col}}{b} = 332.76\ kN/m$$

$Deflection\ caused\ by\ UDL$:

$$\delta_{mid,UDL} = \frac{5qL^4}{384EI} = 32.48\ mm\ (\downarrow)$$

$Hence, the\ total\ deflection\ is$:

$$\delta_{total} = 66.92\ mm(\downarrow)$$

$Maximum\ bending\ moment\ M_{max} = 126910\ kNm$

$$\therefore N_1 = \frac{M}{D} = 10575.83\ kN$$

$$\Rightarrow A = \frac{\boldsymbol{N}_1}{0.70\boldsymbol{f_y}} = 425.59\ cm^2$$

$And\ the\ deflection\ of\ the\ truss\ can\ also\ be\ determined\ in$

$an\ approximate\ way$:

$$The\ second\ moment: I = \frac{AD^2}{2} = 3.06 \times 10^{12}\ cm^4$$

$Deflection\ caused\ by\ point\ load$:

$$\delta_{C,P} = -\vartheta(L/2) = \frac{PL^3}{3EI} = 6.57\ mm\ (\downarrow)$$

$$q = (\frac{4 * P_{Col,B}}{8.5} + \frac{4 * P_{Col,C}}{5}) \times \frac{1}{2} = 114.59\ kN/m$$

$Deflection\ caused\ by\ UDL$:

$\delta_{C,UDL} = \frac{qL^4}{8EI} = 22.26\ mm\ (\downarrow)$

$Hence, the\ total\ deflection\ is$:

$\delta_{total} = 28.84\ mm(\downarrow)$

Truss WEST VIEW Check

Top & Bottom Chords Check

The most compressed element is 321-322, N_{Ed} = 1896.67 kN

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 5.73$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 5.73$

$\therefore\ Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 15.8$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 15.8$

$\therefore\ Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

Compression resistance of cross-section

The design compression resistance of the cross-section

$$N_{c,Rd} = \frac{Af_y}{\gamma_{m0}} \; for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{29900 \times 355}{1.0} = 10614.5\ kN\ > 1896.67\ kN\ \therefore OK$$

Member Buckling Resistance in compression

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{Ml}} \; fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} \text{ for Class 1, 2 and 3 cross-section}$$

Elastic Critical Force and non-dimensional slenderness for flexural buckling

$$N_{cr,y} = \frac{\pi^2 EI_y}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 791000000}{13500^2} = 8995.56 \times 10^3 \text{N} = 8995.56 \text{ kN}$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{29900 \times 355}{8995.56 \times 10^3}} = 1.09$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 310000000}{13500^2} = 3525.44 \times 10^3 \text{N}$$

$$= 3525.44 \text{ kN}$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{29900 \times 355}{3525.44 \times 10^3}} = 1.74$$

<u>Selection of buckling curve and imperfection factor α</u>

For a hot-rolled I section,

$$\frac{\text{h}}{\text{b}} = \frac{381}{394.8} = 0.97 < 1.2, and\ t_f = 30.2mm < 100mm$$

$$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$$

$$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$$

Buckling curves – major (y-y) axis

$$\Phi_{\text{y}} = 0.5[1 + 0.34(1.09 - 0.2) + 1.09^2] = 1.24$$

$$\chi_y = \frac{1}{1.24 + \sqrt{1.24^2 - 1.09^2}} = 0.54$$

$$\therefore \text{N}_{\text{b,y,Rd}} = \frac{0.54 \times 29900 \times 355}{1.0} = 5768.69 \times 10^3 \text{N}$$

$$= 5768.69\text{kN}$$

$$5768.69 > 1896.67 \text{ kN}$$

$$\therefore Major\ axis\ flexural\ buckling\ resistance\ is\ OK$$

Buckling curves – minor (z-z) axis

$$\Phi_{\text{z}} = 0.5[1 + 0.49(1.74 - 0.2) + 1.74^2] = 2.38$$

$$\chi_z = \frac{1}{2.38 + \sqrt{2.38^2 - 1.74^2}} = 0.25$$

$$\therefore \mathrm{N_{b,z,Rd}} = \frac{0.25 \times 29900 \times 355}{1.0} = 2645.19 \times 10^3 \mathrm{N} = 2645.19 \mathrm{kN}$$

$2645.19 > 1896.67$ kN

$\therefore$ *Minor axis flexural buckling resistance is OK*

Critical Diagonals Check

The most compressed element is 13-2, $\mathrm{N_{Ed}}$ = 11933.98 kN

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 2.98$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 2.98$

$\therefore Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 8.11$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 8.11$

$\therefore Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Compression resistance of cross-section</u>

The design compression resistance of the cross-section

$$N_{c,Rd} = \frac{Af_y}{\gamma_{m0}} \; for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{59500 \times 355}{1.0} = 21122.50\ kN\ > 11933.98\ kN\ \therefore OK$$

<u>Member Buckling Resistance in compression</u>

$$N_{b,Rd} = \frac{\chi Af_y}{\gamma_{Ml}} \; fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}}$ for Class 1, 2 and 3 cross-section

<u>Elastic Critical Force and non-dimensional slenderness for flexural buckling</u>

$$N_{cr,y} = \frac{\pi^2 EI_y}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 1830000000}{6020^2} = 104659.10 \times 10^3 \text{N}$$

$$= 104659.10\text{kN}$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{59500 \times 355}{104659.10 \times 10^3}} = 0.45$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{L_{cr}{}^2}$$

$$= \frac{\pi^2 \times 210000 \times 678000000}{6020^2} = 38775.35 \times 10^3 N$$

$$= 38775.35 kN$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{59500 \times 355}{38775.35 \times 10^3}} = 0.74$$

<u>Selection of buckling curve and imperfection factor α</u>

For a hot-rolled I section,

$$\frac{h}{b} = \frac{436.6}{412.2} = 1.06 < 1.2, and\ t_f = 58mm < 100mm$$

$$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$$

$$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$$

Buckling curves – major (y-y) axis

$$\Phi_y = 0.5[1 + 0.34(0.45 - 0.2) + 0.45^2] = 0.64$$

$$\chi_y = \frac{1}{0.64 + \sqrt{0.64^2 - 0.45^2}} = 0.91$$

$$\therefore N_{b,y,Rd} = \frac{0.91 \times 59500 \times 355}{1.0} = 19137.77 \times 10^3 N$$

$$= 19137.77 kN$$

$$19137.77 > 11933.98\ kN$$

$$\therefore Major\ axis\ flexural\ buckling\ resistance\ is\ OK$$

Buckling curves – minor (z-z) axis

$\Phi_z = 0.5[1 + 0.49(0.74 - 0.2) + 0.74^2] = 0.90$

$$\chi_z = \frac{1}{0.90 + \sqrt{0.90^2 - 0.74^2}} = 0.70$$

$$\therefore \mathrm{N_{b,z,Rd}} = \frac{0.70 \times 59500 \times 355}{1.0} = 14806.93 \times 10^3 \mathrm{N}$$

$= 14806.93\mathrm{kN}$

$14806.93 > 11933.98$ kN

$\therefore Minor\ axis\ flexural\ buckling\ resistance\ is\ OK$

Critical Vertical Element Check

The most compressed element is 321-322, $\mathrm{N_{Ed}}$ = 5060.19 kN

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 5.73$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 5.73$

$\therefore Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 15.8$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 15.8$

$\therefore Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Compression resistance of cross-section</u>

The design compression resistance of the cross-section

$$\mathrm{N_{c,Rd}} = \frac{A\mathrm{f_y}}{\gamma_{\mathrm{m0}}}\ for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{29900 \times 355}{1.0} = 10614.50\ kN\ > 5060.19\ kN\ \therefore OK$$

<u>Member Buckling Resistance in compression</u>

$$\mathrm{N_{b,Rd}} = \frac{\chi \mathrm{Af_y}}{\gamma_{\mathrm{Ml}}}\ fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\frac{\mathrm{Af_y}}{\mathrm{N_{cr}}}}$ for Class 1, 2 and 3 cross-section

<u>Elastic Critical Force and non-dimensional slenderness for flexural buckling</u>

$$\mathrm{N_{cr,y}} = \frac{\pi^2 \mathrm{EI_y}}{\mathrm{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 791000000}{4000^2} = 102465 \times 10^3\mathrm{N} = 102465\mathrm{kN}$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{29900 \times 355}{102465 \times 10^3}} = 0.32$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 310000000}{4000^2} = 40156.95 \times 10^3 N$$

$$= 40156.95 kN$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{29900 \times 355}{40156.95 \times 10^3}} = 0.51$$

<u>Selection of buckling curve and imperfection factor α</u>

For a hot-rolled I section,

$$\frac{h}{b} = \frac{381}{394.8} = 0.97 < 1.2, and\ t_f = 30.2mm < 100mm$$

$$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$$

$$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$$

Buckling curves – major (y-y) axis

$$\Phi_y = 0.5[1 + 0.34(0.32 - 0.2) + 0.32^2] = 0.57$$

$$\chi_y = \frac{1}{0.57 + \sqrt{0.57^2 - 0.32^2}} = 0.96$$

$$\therefore N_{b,y,Rd} = \frac{0.96 \times 29900 \times 355}{1.0} = 10147.85 \times 10^3 N$$

$$= 10147.85 kN$$

$10147.85 > 5060.19$ kN

$\therefore$ *Major axis flexural buckling resistance is OK*

Buckling curves – minor (z-z) axis

$$\Phi_z = 0.5[1 + 0.49(0.51 - 0.2) + 0.51^2] = 0.71$$

$$\chi_z = \frac{1}{0.71 + \sqrt{0.71^2 - 0.51^2}} = 0.84$$

$$\therefore N_{b,z,Rd} = \frac{0.84 \times 29900 \times 355}{1.0} = 8863.74 \times 10^3 N = 8863.74 kN$$

$8863.74 > 5060.19$ kN

$\therefore$ *Minor axis flexural buckling resistance is OK*

Deflection Check of WEST VIEW

This is the better calculated in tabular form as shown in table below:

Table A13.1 Vertical deflection - PROPOSAL 1 - WEST VIEW

Member	Length (mm)	A (mm^2)	E (GPa)	AE (kN)	P-force (kN)	PL/AE (mm)	u	(PL/AE)*u (mm)
C'C	12000	29900	210	6279000	-5771.3	-11.03	-0.07	0.74
C'D	18060	59500	210	12495000	-10825.13	-15.65	-0.37	5.73
C'D'	13500	29900	210	6279000	560.69	1.21	-0.05	-0.06
CD	13500	29900	210	6279000	-1896.67	-4.08	-0.07	0.30
CD'	18060	29900	210	6279000	2565.15	7.38	0.10	0.74
D'D	12000	29900	210	6279000	899.5	1.72	0.04	0.06
D'E	18060	29900	210	6279000	321.1	0.92	-0.16	-0.14
D'E'	13500	29900	210	6279000	2176.54	4.68	0.14	0.65
DE	13500	59500	210	12495000	-9692.74	-10.47	-0.58	6.05
DE'	18060	29900	210	6279000	380.84	1.10	0.31	0.34
E'E	12000	29900	210	6279000	237.16	0.45	0.51	0.23
E'F	18060	29900	210	6279000	3622.52	10.42	0.43	4.52
E'F'	13500	29900	210	6279000	-274.97	-0.59	0.46	-0.27
EF	13500	29900	210	6279000	-1282.04	-2.76	0.24	-0.67
EF'	18060	59500	210	12495000	-11578.3	-16.74	-0.61	10.12
F'F	12000	80800	210	16968000	-8840.08	-6.25	-0.31	1.92

F'G	18060	29900	210	6279000	-2017.3	-5.80	-0.08	0.46
F'G'	13500	29900	210	6279000	-874.93	-1.88	-0.07	0.14
FG	13500	29900	210	6279000	1512.48	3.25	0.06	0.19
FG'	18060	29900	210	6279000	155.33	0.45	0.03	0.01
GG"	18000	29900	210	6279000	-2461.3	-7.06	0.05	-0.37
F"F'	6000	80800	210	16968000	-17493.67	-6.19	-0.67	4.14
F"G'	14770	29900	210	6279000	1087.35	2.56	0.05	0.13
F'G"	14770	59500	210	12495000	-7121.71	-8.42	-0.30	2.53
								37.49

Truss NORTH VIEW Check

Top & Bottom Chords Check

The most compressed element is 7-17, N_{Ed} = 8759.28 kN

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 2.98$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 2.98$

$\therefore Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 8.11$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 8.11$

$\therefore Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Compression resistance of cross-section</u>

The design compression resistance of the cross-section

$$N_{c,Rd} = \frac{Af_y}{\gamma_{m0}} \; for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{59500 \times 355}{1.0} = 21122.50\ kN\ > 8759.28\ kN\ \therefore OK$$

<u>Member Buckling Resistance in compression</u>

$$N_{b,Rd} = \frac{\chi Af_y}{\gamma_{Ml}} \; fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}}$ for Class 1, 2 and 3 cross-section

<u>Elastic Critical Force and non-dimensional slenderness for flexural buckling</u>

$$N_{cr,y} = \frac{\pi^2 EI_y}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 791000000}{10000^2} = 37928.89 \times 10^3 \text{N}$$

$$= 37928.89 \text{ kN}$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{29900 \times 355}{37928.89 \times 10^3}} = 0.75$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 310000000}{10000^2} = 14052.34 \times 10^3 N$$

$$= 14052.34 \text{ kN}$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{29900 \times 355}{14052.34 \times 10^3}} = 1.23$$

<u>Selection of buckling curve and imperfection factor α</u>

For a hot-rolled I section,

$$\frac{h}{b} = \frac{436.6}{412.2} = 1.06 < 1.2, and\ t_f = 58mm < 100mm$$

$$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$$

$$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$$

Buckling curves – major (y-y) axis

$$\Phi_y = 0.5[1 + 0.34(0.75 - 0.2) + 0.75^2] = 0.87$$

$$\chi_y = \frac{1}{0.87 + \sqrt{0.87^2 - 0.75^2}} = 0.76$$

$$\therefore N_{b,y,Rd} = \frac{0.76 \times 29900 \times 355}{1.0} = 15988.86 \times 10^3 N$$

$$= 15988.86 \text{ kN}$$

$$15988.86 > 8759.28 \text{ kN}$$

$$\therefore Major\ axis\ flexural\ buckling\ resistance\ is\ OK$$

Buckling curves – minor (z-z) axis

$\Phi_z = 0.5[1 + 0.49(1.23 - 0.2) + 1.23^2] = 1.50$

$$\chi_z = \frac{1}{1.50 + \sqrt{1.50^2 - 1.23^2}} = 0.42$$

$$\therefore \mathrm{N_{b,z,Rd}} = \frac{0.42 \times 29900 \times 355}{1.0} = 8903.99 \times 10^3 \mathrm{N} = 8903.99 \text{ kN}$$

$8903.99 > 8759.28$ kN

$\therefore Minor\ axis\ flexural\ buckling\ resistance\ is\ OK$

Critical Diagonals Check

The inclined rod is only in tension, which means that the tension needs to be checked once the connection design is complete.

Critical Vertical Element Check

The most compressed element is 6-8, $\mathrm{N_{Ed}}$ = 36.28 kN

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 5.73$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 5.73$

$\therefore Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 15.8$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 15.8$

$\therefore Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Compression resistance of cross-section</u>

The design compression resistance of the cross-section

$$\mathrm{N_{c,Rd}} = \frac{A\mathrm{f_y}}{\gamma_{\mathrm{m0}}}\ for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{29900 \times 355}{1.0} = 10614.50\ kN\ > 36.28\ kN\ \therefore OK$$

<u>Member Buckling Resistance in compression</u>

$$\mathrm{N_{b,Rd}} = \frac{\chi \mathrm{Af_y}}{\gamma_{\mathrm{Ml}}}\ fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\frac{\mathrm{Af_y}}{\mathrm{N_{cr}}}}$ for Class 1, 2 and 3 cross-section

<u>Elastic Critical Force and non-dimensional slenderness for flexural buckling</u>

$$\mathrm{N_{cr,y}} = \frac{\pi^2 \mathrm{EI_y}}{\mathrm{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 791000000}{4000^2} = 102465 \times 10^3\text{N} = 102465\text{kN}$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{29900 \times 355}{102465 \times 10^3}} = 0.32$$

$$\text{N}_{\text{cr},z} = \frac{\pi^2 \text{EI}_z}{\text{L}_{\text{cr}}{}^2}$$

$$= \frac{\pi^2 \times 210000 \times 310000000}{4000^2} = 40156.95 \times 10^3\text{N}$$

$$= 40156.95\text{kN}$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{29900 \times 355}{40156.95 \times 10^3}} = 0.51$$

<u>Selection of buckling curve and imperfection factor α</u>

For a hot-rolled I section,

$$\frac{\text{h}}{\text{b}} = \frac{381}{394.8} = 0.97 < 1.2, and\ t_f = 30.2mm < 100mm$$

$$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$$

$$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$$

Buckling curves – major (y-y) axis

$$\Phi_{\text{y}} = 0.5[1 + 0.34(0.32 - 0.2) + 0.32^2] = 0.57$$

$$\chi_y = \frac{1}{0.57 + \sqrt{0.57^2 - 0.32^2}} = 0.96$$

$$\therefore \text{N}_{\text{b,y,Rd}} = \frac{0.96 \times 29900 \times 355}{1.0} = 10147.85 \times 10^3\text{N}$$

$= 10147.85\text{kN}$

$10147.85 > 36.28\ \text{kN}$

$\therefore\ Major\ axis\ flexural\ buckling\ resistance\ is\ OK$

Buckling curves – minor (z-z) axis

$\Phi_z = 0.5[1 + 0.49(0.51 - 0.2) + 0.51^2] = 0.71$

$$\chi_z = \frac{1}{0.71 + \sqrt{0.71^2 - 0.51^2}} = 0.84$$

$$\therefore\ N_{b,z,Rd} = \frac{0.84 \times 29900 \times 355}{1.0} = 8863.74 \times 10^3\text{N} = 8863.74\text{kN}$$

$8863.74 > 36.28\ \text{kN}$

$\therefore\ Minor\ axis\ flexural\ buckling\ resistance\ is\ OK$

Deflection Check of NORTH VIEW

This is the better calculated in tabular form as shown in table below:

Table A13.2 Vertical deflection - PROPOSAL 1 - NORTH VIEW

Member	Length (mm)	A (mm^2)	E (GPa)	AE (kN)	P-force (kN)	PL/AE (mm)	u	(PL/AE)*u (mm)
A1B	10000	59500	210	12495000	-7.41	-0.01	0.00	0.00
A2B1	10000	59500	210	12495000	-5198.87	-4.16	-0.36	1.50
A3B2	10000	59500	210	12495000	-4176.85	-3.34	-0.29	0.97
A4B3	10000	59500	210	12495000	-8759.28	-7.01	-0.60	4.21
BB1	4000	29900	210	6279000	-32.03	-0.02	0.00	0.00
B1B2	4000	29900	210	6279000	4055.31	2.58	0.29	0.75
B2B3	4000	29900	210	6279000	7335.48	4.67	0.52	2.43
AB1	12810	59500	210	12495000	6668.1	6.84	0.47	3.21
A1B2	12810	59500	210	12495000	5373.23	5.51	0.37	2.04
A2B3	12810	59500	210	12495000	11210.29	11.49	0.76	8.73
								23.84

A13.3 Proposal 2

Preliminary Truss Design

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$N_1 = \frac{M}{D}$, *Assume* $\boldsymbol{N}_{b,Rd} = 0.70\boldsymbol{f}_y A$

$\therefore \frac{\boldsymbol{N}_1}{0.70\boldsymbol{f}_y} \Rightarrow$ *Area of cross section required*

$Maximum\ bending\ moment\ M_{max} = 70790.46\ kNm$

$\therefore N_1 = \frac{M}{D} = 5899.21\ kN$

$\Rightarrow A = \frac{\boldsymbol{N}_1}{0.70\boldsymbol{f}_y} = 237.39\ cm^2$

$And\ the\ deflection\ of\ the\ truss\ can\ also\ be\ determined\ in$

$an\ approximate\ way$:

$The\ second\ moment: I = \frac{AD^2}{2} = 1.71 \times 10^{12}\ cm^4$

$Deflection\ caused\ by\ point\ load\ at\ D$:

$\delta_{mid,D} = -\vartheta(L/2) = \frac{P_D b_D(3L^2 - 4b_D^2)}{48EI} = 17.22\ mm\ (\downarrow)$

$Deflection\ caused\ by\ point\ load\ at\ E$:

$\delta_{mid,E} = -\vartheta(L/2) = \frac{P_E b_E(3L^2 - 4b_E^2)}{48EI} = 17.22\ mm\ (\downarrow)$

$Simply\ consider\ that\ the\ uniform\ distribute\ load\ q\ is\ equal$

$to\ the\ ratio\ of\ the\ point\ load\ on\ the\ column\ to\ the\ width\ of$

$the\ bearing\ area\ of\ the\ column\ load$:

$$q = \frac{4P_{Col}}{b} = 332.76\ kN/m$$

$Deflection\ caused\ by\ UDL$:

$$\delta_{mid,UDL} = \frac{5qL^4}{384EI} = 32.48\ mm\ (\downarrow)$$

$Hence, the\ total\ deflection\ is$:

$$\delta_{total} = 66.92\ mm(\downarrow)$$

$Maximum\ bending\ moment\ M_{max} = 128380.4\ kNm$

$$\therefore N_1 = \frac{M}{D} = 16047.55\ kN$$

$$\Rightarrow A = \frac{\boldsymbol{N}_1}{0.70\boldsymbol{f_y}} = 645.78\ cm^2$$

$And\ the\ deflection\ of\ the\ truss\ can\ also\ be\ determined\ in$

$an\ approximate\ way$:

$$The\ second\ moment: I = \frac{AD^2}{2} = 2.07 \times 10^{12}\ cm^4$$

$Deflection\ caused\ by\ point\ load$:

$$\delta_{C,P} = -\vartheta(L/2) = \frac{PL^3}{8EI} = 3.70\ mm\ (\downarrow)$$

$$q = 109.85\ kN/m$$

$Deflection\ caused\ by\ UDL$:

$$\delta_{C,UDL} = \frac{qL^4}{8EI} = 0\ mm\ (\downarrow)$$

$Hence, the\ total\ deflection\ is$:

$$\delta_{total} = 3.70\ mm(\downarrow)$$

Truss WEST VIEW Check

Top & Bottom Chords Check

The most compressed element is 15-982, N_{Ed} = 7407.56 kN

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 4.03$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 4.03$

$\therefore\ Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 10.9$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 10.9$

$\therefore\ Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Compression resistance of cross-section</u>

The design compression resistance of the cross-section

$$N_{c,Rd} = \frac{Af_y}{\gamma_{m0}} \; for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{43300 \times 355}{1.0} = 15371.5\ kN\ > 7407.56\ kN\ \therefore OK$$

<u>Member Buckling Resistance in compression</u>

$$N_{b,Rd} = \frac{\chi A f_y}{\gamma_{Ml}} \; fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$$\bar{\lambda} = \sqrt{\frac{Af_y}{N_{cr}}} \text{ for Class 1, 2 and 3 cross-section}$$

<u>Elastic Critical Force and non-dimensional slenderness for flexural buckling</u>

$$N_{cr,y} = \frac{\pi^2 EI_y}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 1230000000}{6750^2} = 55952.13 \times 10^3 \text{N}$$

$$= 55952.13 \text{ kN}$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{43300 \times 355}{55952.13 \times 10^3}} = 0.52$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{{L_{cr}}^2}$$

$$= \frac{\pi^2 \times 210000 \times 469000000}{6750^2} = 21334.59 \times 10^3 N$$

$$= 21334.59 \text{ kN}$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{43300 \times 355}{21334.59 \times 10^3}} = 0.85$$

<u>Selection of buckling curve and imperfection factor α</u>

For a hot-rolled I section,

$$\frac{h}{b} = \frac{406.4}{403} = 1.01 < 1.2, and\ t_f = 42.9mm < 100mm$$

$$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$$

$$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$$

Buckling curves – major (y-y) axis

$$\Phi_y = 0.5[1 + 0.34(0.52 - 0.2) + 0.52^2] = 0.69$$

$$\chi_y = \frac{1}{0.69 + \sqrt{0.69^2 - 0.52^2}} = 0.87$$

$$\therefore N_{b,y,Rd} = \frac{0.87 \times 43300 \times 355}{1.0} = 13424.90 \times 10^3 N$$

$$= 13424.90 kN$$

$$13424.90 > 7407.56 \text{ kN}$$

$$\therefore Major\ axis\ flexural\ buckling\ resistance\ is\ OK$$

Buckling curves – minor (z-z) axis

$$\Phi_z = 0.5[1 + 0.49(0.85 - 0.2) + 0.85^2] = 1.02$$

$$\chi_z = \frac{1}{1.02 + \sqrt{1.02^2 - 0.85^2}} = 0.63$$

$$\therefore N_{b,z,Rd} = \frac{0.63 \times 43300 \times 355}{1.0} = 9708.02 \times 10^3 N = 9708.02 kN$$

$9708.02 > 7407.56$ kN

$\therefore$ *Minor axis flexural buckling resistance is OK*

Critical Diagonals Check

The most compressed element is 121-849, N_{Ed} = 6219.13 kN

The UKC 356*406*340 in grade S355 steel is suitable to support the design load N_{Ed} of 6219.13 kN. The resistance of this truss member is governed by buckling about the z-z axis, with $N_{b,z,Rd}$ = 9708.02 kN.

Critical Vertical Element Check

The most compressed element is 7-103, N_{Ed} = 5824.35 kN

<u>Cross-section classification</u>

$$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$$c/t = 5.73$$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 5.73$

$\therefore\ Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 15.8$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 15.8$

$\therefore\ Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Compression resistance of cross-section</u>

The design compression resistance of the cross-section

$$\mathrm{N_{c,Rd}} = \frac{A\mathrm{f_y}}{\gamma_{\mathrm{m0}}}\ for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{29900 \times 355}{1.0} = 10614.50\ kN\ > 5824.35\ kN\ \therefore OK$$

<u>Member Buckling Resistance in compression</u>

$$\mathrm{N_{b,Rd}} = \frac{\chi \mathrm{Af_y}}{\gamma_{\mathrm{Ml}}}\ fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \text{ but } \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$\bar{\lambda} = \sqrt{\frac{\mathrm{Af_y}}{\mathrm{N_{cr}}}}$ for Class 1, 2 and 3 cross-section

<u>Elastic Critical Force and non-dimensional slenderness for flexural buckling</u>

$$N_{cr,y} = \frac{\pi^2 EI_y}{L_{cr}{}^2}$$

$$= \frac{\pi^2 \times 210000 \times 791000000}{4000^2} = 102465 \times 10^3 N = 102465kN$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{29900 \times 355}{102465 \times 10^3}} = 0.32$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{L_{cr}{}^2}$$

$$= \frac{\pi^2 \times 210000 \times 310000000}{4000^2} = 40156.95 \times 10^3 N$$

$$= 40156.95kN$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{29900 \times 355}{40156.95 \times 10^3}} = 0.51$$

<u>Selection of buckling curve and imperfection factor α</u>

For a hot-rolled I section,

$$\frac{h}{b} = \frac{381}{394.8} = 0.97 < 1.2, and\ t_f = 30.2mm < 100mm$$

$$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$$

$$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$$

Buckling curves – major (y-y) axis

$$\Phi_y = 0.5[1 + 0.34(0.32 - 0.2) + 0.32^2] = 0.57$$

$$\chi_y = \frac{1}{0.57 + \sqrt{0.57^2 - 0.32^2}} = 0.96$$

$$\therefore \mathrm{N_{b,y,Rd}} = \frac{0.96 \times 29900 \times 355}{1.0} = 10147.85 \times 10^3 \mathrm{N}$$

$$= 10147.85 \mathrm{kN}$$

$$10147.85 > 5824.35 \text{ kN}$$

$$\therefore \textit{Major axis flexural buckling resistance is OK}$$

Buckling curves – minor (z-z) axis

$$\Phi_z = 0.5[1 + 0.49(0.51 - 0.2) + 0.51^2] = 0.71$$

$$\chi_z = \frac{1}{0.71 + \sqrt{0.71^2 - 0.51^2}} = 0.84$$

$$\therefore \mathrm{N_{b,z,Rd}} = \frac{0.84 \times 29900 \times 355}{1.0} = 8863.74 \times 10^3 \mathrm{N} = 8863.74 \mathrm{kN}$$

$$8863.74 > 5824.35 \text{ kN}$$

$$\therefore \textit{Minor axis flexural buckling resistance is OK}$$

This is the better calculated in tabular form as shown in table below:

Table A13.3 Vertical deflection - PROPOSAL 2 - WEST VIEW

Member	Length (mm)	A (mm^2)	E (GPa)	AE (kN)	P-force (kN)	PL/AE (mm)	u	(PL/AE)*u (mm)
1	4000	29900	210	6279000	-2800.02	-1.78	0.00	0.00
2	4000	29900	210	6279000	-5819.93	-3.71	-0.05	0.19
3	4000	29900	210	6279000	-9680.75	-6.17	-0.14	0.86
4	4000	29900	210	6279000	-5093.66	-3.24	0.00	0.00
5	4000	29900	210	6279000	-3890.91	-2.48	-0.08	0.20
6	4000	29900	210	6279000	-1815.45	-1.16	-0.15	0.17
7	4000	29900	210	6279000	-5100.13	-3.25	0.10	-0.32
8	4000	29900	210	6279000	-3785.46	-2.41	0.36	-0.87
9	4000	29900	210	6279000	-1985.3	-1.26	0.78	-0.99
10	4000	18060	210	3792600	-5293.43	-5.58	0.00	0.00

11	4000	18060	210	3792600	-9936.01	-10.48	-0.26	2.72
12	4000	18060	210	3792600	-14912.51	-15.73	-0.56	8.81
13	4000	29900	210	6279000	-2613.99	-1.67	0.00	0.00
14	4000	29900	210	6279000	-2686.64	-1.71	0.03	-0.05
15	4000	29900	210	6279000	-2868.05	-1.83	0.07	-0.13
16	6000	18060	210	3792600	-19646.97	-31.08	-0.81	25.18
17	6000	43300	210	9093000	-3371.95	-2.22	0.11	-0.24
18	6750	43300	210	9093000	-11.51	-0.01	0.00	0.00
19	6750	43300	210	9093000	4018.22	2.98	0.08	0.24
20	6750	43300	210	9093000	5402.47	4.01	0.15	0.60
21	6750	43300	210	9093000	-7757.46	-5.76	-0.17	0.98
22	6750	43300	210	9093000	-6601.76	-4.90	-0.20	0.98
23	6750	43300	210	9093000	-5009.28	-3.72	-0.40	1.49
24	13500	43300	210	9093000	267.37	0.40	0.02	0.01
25	6750	43300	210	9093000	-7768.61	-5.77	-0.16	0.92
26	6750	43300	210	9093000	-2904.28	-2.16	0.11	-0.24
27	6750	43300	210	9093000	55.39	0.04	0.00	0.00
28	6750	43300	210	9093000	-7230.48	-5.37	-0.62	3.33
29	6750	43300	210	9093000	-2194.17	-1.63	-0.39	0.64
30	6750	43300	210	9093000	55.39	0.04	0.00	0.00
31	13500	43300	210	9093000	5409.68	8.03	0.31	2.49
32	6750	43300	210	9093000	-7214.76	-5.36	-0.62	3.32
33	6750	43300	210	9093000	-7176.76	-5.33	-0.53	2.82
34	6750	43300	210	9093000	-5976.46	-4.44	-0.22	0.98
35	6750	43300	210	9093000	1916.1	1.42	0.11	0.16
36	6750	43300	210	9093000	3955.8	2.94	0.37	1.09
37	6750	43300	210	9093000	4905.91	3.64	0.47	1.71
38	13500	43300	210	9093000	-54.18	-0.08	-0.03	0.00
39	6750	43300	210	9093000	1976.32	1.47	0.11	0.16
40	6750	43300	210	9093000	306.07	0.23	0.06	0.01
41	6750	43300	210	9093000	-106.88	-0.08	0.07	-0.01
42	6750	43300	210	9093000	-5186.25	-3.85	-0.31	1.19
43	6750	43300	210	9093000	10.15	0.01	0.00	0.00
44	6750	43300	210	9093000	-934.46	-0.69	-0.06	0.04
45	6750	43300	210	9093000	-583.11	-0.43	-0.06	0.03
46	6750	43300	210	9093000	-134.52	-0.10	-0.07	0.01
47	7850	43300	210	9093000	-4700.1	-4.06	-0.10	0.41
48	7850	43300	210	9093000	-6308.55	-5.45	-0.17	0.93
49	7850	43300	210	9093000	-6193.01	-5.35	-0.31	1.66
50	7850	43300	210	9093000	4314.8	3.72	0.09	0.34
51	7850	43300	210	9093000	6042.94	5.22	0.16	0.83
52	7850	43300	210	9093000	5912.43	5.10	0.31	1.58
53	7850	43300	210	9093000	-8.69	-0.01	-0.27	0.00
54	7850	43300	210	9093000	185.54	0.16	-0.30	-0.05

55	7850	43300	210	9093000	-32.41	-0.03	0.00	0.00
56	7850	43300	210	9093000	-635.52	-0.55	0.25	-0.14
57	7850	43300	210	9093000	-641.8	-0.55	0.29	-0.16
58	7850	43300	210	9093000	-35.29	-0.03	0.00	0.00
59	7850	43300	210	9093000	5180.78	4.47	0.41	1.83
60	7850	43300	210	9093000	6360.49	5.49	0.53	2.91
61	7850	43300	210	9093000	6256.27	5.40	0.40	2.16
62	7850	43300	210	9093000	-5499.78	-4.75	-0.43	2.04
63	7850	43300	210	9093000	-6588.38	-5.69	-0.53	3.01
64	7850	43300	210	9093000	-6406.63	-5.53	-0.40	2.21
65	7850	43300	210	9093000	-1193.64	-1.03	-0.06	0.06
66	7850	43300	210	9093000	-782.5	-0.68	-0.06	0.04
67	7850	43300	210	9093000	-409.71	-0.35	-0.08	0.03
68	7850	43300	210	9093000	1092.96	0.94	0.07	0.07
69	7850	43300	210	9093000	659.4	0.57	0.07	0.04
70	7850	43300	210	9093000	143.24	0.12	0.08	0.01
71	7850	43300	210	9093000	3292.92	2.84	0.16	0.45
72	7850	43300	210	9093000	-3476.97	-3.00	-0.16	0.48
								79.23

Truss NORTH VIEW Check

Critical Diagonals Check

The most compressed element is 321-857, N_{Ed} = 10007.56 kN

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 4.03$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 4.03$

$\therefore Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 10.9$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 10.9$

$\therefore Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Compression resistance of cross-section</u>

The design compression resistance of the cross-section

$$\mathrm{N_{c,Rd}} = \frac{\mathrm{Af_y}}{\gamma_{\mathrm{m0}}}\ for\ Class\ 1, 2\ and\ 3\ cross-section$$

$$= \frac{43300 \times 355}{1.0} = 15371.5\ kN\ > 10007.56\ kN\ \therefore OK$$

<u>Member Buckling Resistance in compression</u>

$$\mathrm{N_{b,Rd}} = \frac{\chi A \mathrm{f_y}}{\gamma_{\mathrm{Ml}}}\ fro\ Class\ 1, 2\ and\ 3\ cross-section$$

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}}\ \text{but}\ \chi \leq 1{,}0$$

Where $\Phi = 0{,}5\left[1 + \alpha(\bar{\lambda} - 0{,}2) + \bar{\lambda}^2\right]$

$$\bar{\lambda} = \sqrt{\frac{\mathrm{Af_y}}{\mathrm{N_{cr}}}}\ \text{for Class 1, 2 and 3 cross-section}$$

<u>Elastic Critical Force and non-dimensional slenderness for flexural buckling</u>

$$N_{cr,y} = \frac{\pi^2 EI_y}{L_{cr}{}^2}$$

$$= \frac{\pi^2 \times 210000 \times 1230000000}{6400^2} = 62239.23 \times 10^3 \text{N}$$

$$= 62239.23 \text{ kN}$$

$$\therefore \bar{\lambda}_y = \sqrt{\frac{43300 \times 355}{62239.23 \times 10^3}} = 0.50$$

$$N_{cr,z} = \frac{\pi^2 EI_z}{L_{cr}{}^2}$$

$$= \frac{\pi^2 \times 210000 \times 469000000}{6400^2} = 23731.87 \times 10^3 \text{N}$$

$$= 23731.87 \text{ kN}$$

$$\therefore \bar{\lambda}_z = \sqrt{\frac{43300 \times 355}{23731.87 \times 10^3}} = 0.80$$

<u>Selection of buckling curve and imperfection factor α</u>

For a hot-rolled I section,

$$\frac{h}{b} = \frac{406.4}{403} = 1.01 < 1.2, and\ t_f = 42.9mm < 100mm$$

$$\therefore Use\ buckling\ curve\ b\ for\ y - y\ and\ curve\ c\ for\ z - z$$

$$For\ curve\ buckling\ curve\ b, \alpha = 0.34, and\ \alpha = 0.49\ for\ curve\ c$$

Buckling curves – major (y-y) axis

$$\Phi_y = 0.5[1 + 0.34(0.50 - 0.2) + 0.50^2] = 0.67$$

$$\chi_y = \frac{1}{0.67 + \sqrt{0.67^2 - 0.50^2}} = 0.89$$

$$\therefore \mathrm{N_{b,y,Rd}} = \frac{0.89 \times 43300 \times 355}{1.0} = 13612.34 \times 10^3 \mathrm{N}$$

$$= 13612.34\mathrm{kN}$$

$$13612.34 > 10007.56 \text{ kN}$$

$$\therefore \textit{Major axis flexural buckling resistance is OK}$$

Buckling curves – minor (z-z) axis

$$\Phi_z = 0.5[1 + 0.49(0.80 - 0.2) + 0.80^2] = 0.97$$

$$\chi_z = \frac{1}{0.97 + \sqrt{0.97^2 - 0.80^2}} = 0.97$$

$$\therefore \mathrm{N_{b,z,Rd}} = \frac{0.97 \times 43300 \times 355}{1.0} = 10131.9 \times 10^3 \mathrm{N} = 10131.9\mathrm{kN}$$

$$10131.9 > 10007.56 \text{ kN}$$

$$\therefore \textit{Minor axis flexural buckling resistance is OK}$$

Critical Vertical Element Check

We found that the vertical members are purely tensile, so there is no need to carry out a compression check, only a tension check after the nodes have been designed.

This is the better calculated in tabular form as shown in table below:

Table 13.4 Table showing the results of vertical element check

Member	Length (mm)	A (mm^2)	E (GPa)	AE (kN)	P-force (kN)	PL/AE (mm)	u	(PL/AE)*u (mm)
1	4000	29900	210	6279000	-3531.2	-2.25	0.00	0.00
2	4000	29900	210	6279000	-3327.48	-2.12	0.09	-0.19
3	4000	29900	210	6279000	-3480.36	-2.22	0.00	0.00
4	4000	29900	210	6279000	-2215.752	-1.41	0.09	-0.13

5	4000	153000	210	32130000	-3667.07	-0.46	0.00	0.00
6	4000	153000	210	32130000	-2712.03	-0.34	0.23	-0.08
7	4000	153000	210	32130000	-4613.08	-0.57	-0.02	0.01
8	4000	153000	210	32130000	-16358.87	-2.04	-0.88	1.79
9	4000	29900	210	6279000	-10.96	-0.01	0.00	0.00
10	4000	29900	210	6279000	6200.48	3.95	0.48	1.90
11	3500	36000	210	7560000	45.64	0.02	0.00	0.00
12	3500	36000	210	7560000	-885.08	-0.41	-0.08	0.03
13	3500	36000	210	7560000	1834.95	0.85	0.16	0.14
14	3500	36000	210	7560000	1052.67	0.49	0.75	0.37
15	7000	36000	210	7560000	-838.23	-0.78	-0.07	0.05
16	3500	36000	210	7560000	1854.98	0.86	0.16	0.14
17	3500	36000	210	7560000	-2587.38	-1.20	-0.17	0.20
18	3500	36000	210	7560000	7230.33	3.35	0.48	1.61
19	3500	36000	210	7560000	2349.14	1.09	0.15	0.16
20	7000	36000	210	7560000	-4247.31	-3.93	-0.31	1.22
21	3500	36000	210	7560000	7344.59	3.40	0.49	1.67
22	3500	36000	210	7560000	-4701.65	-2.18	-0.38	0.83
23	3500	36000	210	7560000	15524.82	7.19	1.20	8.62
24	3500	36000	210	7560000	4967.35	2.30	0.41	0.94
25	7000	36000	210	7560000	-11287.51	-10.45	-0.85	8.88
26	5000	36000	210	7560000	15428.24	10.20	1.19	12.14
27	5000	36000	210	7560000	8986.42	5.94	0.69	4.10
28	5000	36000	210	7560000	57.54	0.04	0.00	0.00
29	5000	36000	210	7560000	7702.55	5.09	-0.60	-3.06
30	1000	36000	210	7560000	-8344.09	-1.10	-0.65	0.72
31	5320	59500	210	12495000	1247.4	0.53	0.12	0.06
32	5320	59500	210	12495000	1357.9	0.58	0.12	0.07
33	5320	59500	210	12495000	-1471.2	-0.63	-0.12	0.08
34	5320	59500	210	12495000	-1585.88	-0.68	-0.12	0.08
35	5320	59500	210	12495000	4039.68	1.72	0.24	0.41
36	5320	59500	210	12495000	3640.23	1.55	0.24	0.37
37	5320	59500	210	12495000	-4124.3	-1.76	-0.24	0.42
38	5320	59500	210	12495000	-3857.65	-1.64	-0.25	0.41
39	5320	59500	210	12495000	6276.78	2.67	0.54	1.44
40	5320	59500	210	12495000	7165.36	3.05	0.59	1.80
41	5320	59500	210	12495000	-6145.04	-2.62	-0.53	1.39
42	5320	59500	210	12495000	-7517.31	-3.20	-0.61	1.95
43	6400	59500	210	12495000	-9798.58	-5.02	-0.75	3.76
44	6400	59500	210	12495000	-10775.65	-5.52	-0.83	4.58
45	6400	59500	210	12495000	9882.52	5.06	0.77	3.90
46	6400	59500	210	12495000	10593.91	5.43	0.82	4.45
								67.25

Appendix A14

Beam-Beam Connections Design

A14.1 Summary of Results

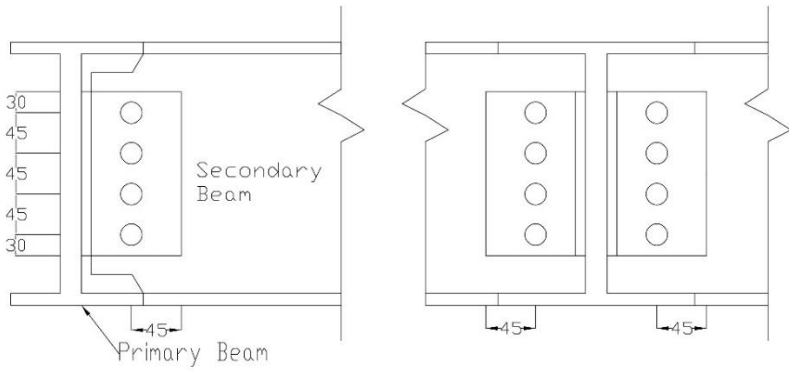


Figure A14.1: Figure showing the results of the beam-to-beam connection

In the calculation, 533x210x122 UB was adopted for both primary and secondary beams as per the design of beams in the earlier section. S355 2L90x90x9 cleats were proposed and all the bolts were assumed to be with Grade 8.8 and size M18.

A14.2 Detailed Calculations

Based on the calculations, the proposed design satisfies all the requirements. The detailed calculations for the beam-to-beam connection are listed below:

Beam to Beam Connection

Primary Beam Size	533 x 210 x 122	UB
Secondary Beam Size	533 x 210 x 122	UB

Section Properties (Primary Beam)

Grade	355	
h_1	544.5	mm
b_1	211.9	mm
t_{w1}	12.7	mm
t_{f1}	21.3	mm
r_1	12.7	mm
A_1	155	cm^2

Section Properties (Secondary Beam)

Grade	355	
h_2	544.5	mm
b_2	211.9	mm
t_{w2}	12.7	mm
t_{f2}	21.3	mm
r_2	12.7	mm
A_2	155	cm^2

Splice Details

Cleats	2L 90 x 90 x 9 Angles	
Bolts	M18 8.8	8 nos.
Fittings Material	S355	

Design Actions

V_{Ed} =	249	kN

Check 1: Bearing Resistance of Bolt Group connecting Cleats to Beam Web

Spacings are desgined based on BSEN1993-1-8 Table 3.3:

Proposed e_1=	30	mm
Proposed e_2=	45	mm
Proposed p_1=	45	mm
Proposed p_2=	45	mm
Proposed Bolt Diameter d =	18	mm
Bolt Hole Diameter d_0=	20	mm

Therefore, proposed dimensions satisfies the following requirements:

$e_1>1.2d_0$, $e_2>1.2d_0$ and $p_1>2.2d_0$

Bolt Ultimate Strength $f_{u,b}$=	800	MPa
Beam Ultimate Strength f_u=	470	MPa
$\alpha_{d1}=\min(p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)$=	0.5	
$k_{1,1}=\min(2.8e_2/d_0-1.7,2.5)$=	2.5	

As thickness of cleats is smaller than that of beam web, use t=9mm to be conservative.

$F_{b1,Rd}=k_{1,1}\alpha_{d1}f_u dt/\gamma_{M2}$=	76.14	kN
$V_{b1,Rd} = n_{b1}F_{b1,Rd}$ =	609.12	kN **OK**

As it is larger than the design shear force, the proposed design satisfies the requirements.

Check 2: Shear Resistance of Bolt Group connecting Cleats to Main Beam Web

Bolt Stress Area A_s =	192	mm2
$F_{v1,Rd}=0.5f_{ub}A_s/\gamma_{M2}$=	61.44	kN

$V_{v1,Rd} = n_{b1}F_{b,Rd} =$ 491.52 kN **OK**

As it is larger than the design shear force, the proposed design satisfies the requirements.

Check 3: Shear Resistance of Bolt Group connecting Cleats to Web of Supported Beam

Vertical Force per Bolt $F_y=V_{Ed}/n_{b2}=$ 62.21 kN

Horizontal Force per Bolt $F_x=M_{Ed}y_{max}/\Sigma y_i^2$

$F_x =$ 85.19 kN

Resultant Force R = 105.49 kN

For double shear, $F_{v2,Rd} = 2F_{v1,Rd} =$ 122.88 kN **OK**

As it is larger than the resultant force, the proposed design satisfies the requirements.

Check 4: Bearing Resistance of Bolt Group connecting Cleats to Web of Supported Beam

$\alpha_{d1}=\min(e_1/(3d_0), p_1/(3d_0)-0.25, f_{ub}/f_u, 1)=$ 0.5

$k_{1,2}=\min(2.8e_2/d_0-1.7, 2.5)=$ 2.5

As beam web thickness is smaller than double of that of the two angles, use t=12.7mm to be conservative.

$F_{b2,Rd}=k_{1,2}\alpha_{d2}f_u dt/\gamma_{M2}=$ 107.44 kN **OK**

As it is larger than $F_{b1,Rd}$, the proposed design satisfies the requirements.

Check 5: Shear Resistance of Beam Notched Section

For net section, $V_{pl1,Rd}=A_{vnet}f_u/\sqrt{3}/\gamma_{M2}=$ 317.05 kN **OK**

As it is larger than the design shear force, the proposed design satisfies the requirements.

For gross section, $V_{pl2,Rd}=A_v f_y/\sqrt{3}/\gamma_{M0}=$ 374.83 kN **OK**

As it is larger than the design shear force, the proposed design satisfies the requirements.

For block shear, $V_{Rd,b}=[0.5A_{nt}f_u/\gamma_{M2}+A_{nv}f_y/\sqrt{3}/\gamma_{M0}]$

$A_{nt} =$ 254 mm2

$A_{nv} =$ 1206.5 mm2

$V_{Rd,b}=$ 295.04 kN **OK**

As it is larger than the design shear force, the proposed design satisfies the requirements.

Check 6: Bending Resistance of Beam Notched Section

Moment of Inertia $I_b =$ 5275659 mm4

$W_b = 2I_b/195 =$ 54109.3 mm3

$M_{b,el,Rd} = W_b f_y/\gamma_{M0}=$ 19.21 kNm

As $V_{Ed}>0.5V_{pl1,Rd}$,

$\rho=(2V_{Ed}/V_{Rd}-1)^2=$ 0.32

$M_{b,el,Rd*} = (1-\rho)M_{b,el,Rd} =$ 12.97 kNm

Eccentricity Moment $M_{Ed}=V_{Ed}(e)=$ 12.78 kNm **OK**

As it is smaller than $M_{b,el,Rd*}$, the proposed design satisfies the requirements.

Appendix A15

Calculation Spreadsheets for Beam-Column Connections

Size of the components for connection

Connection 1:

Column UKC 356×368×202 in S355 steel1144

Column UKC 356×368×1202 in S355 steel

Column UKC 356×368×235 in S355 steel

Column UKC 356×368×634 in S355 steel

Column UKC 356×368×467 in S355 steel

Beam UKB 533×210×122 in S355 steel

BS EN 1993-1-1 NA 2.4

Beam details:

$f_y = 355 \frac{N}{mm^2}$ $f_u = 470 \frac{N}{mm^2}$

$h_b = 544.5\ mm$ $b = 211.9\ mm$ $t_w = 12.7\ mm$ $t_f = 21.3\ mm$

Column details:

BS EN 10025-2 Table 7

$b = 374.7\ mm$ $h = 374.6\ mm$

Plate:

$f_u = 470\ N/mm^2$

BS EN 1993-1-1 NA 2.15

$$V_{c,Rd} \approx \frac{h_b \times t_w \times \left(\frac{f_y}{\sqrt{3}}\right)}{\gamma_{M0}} = \frac{544.5 \times 12.7 \times \left(\frac{355}{\sqrt{3}}\right)}{1.0 \times 10^{-3}} = 1417.33\ kN$$

Access steel document SN013a-EN-EU

From check for beam: $V_{Ed} = \frac{691.72}{2} = 345.86\ kN$

$345.86 < 0.75V_{c,Rd} = 0.75 \times 1417.33 = 1063.00\ kN$ so, a partial depth end

plate is proposed. $h_b = 544.5\ mm > 500mm$ so, $10\ mm$ endplate is proposed.

Minimum depth for end plate depth is $0.6h_b = 0.6 \times 544.5 = 326.7\ mm$, so proposed depth is $330\ mm$. Assuming M20 bolts, number of bolts$= \frac{V_{Ed}}{75} = \frac{345.86}{75} = 4.61$, so proposed minimum number of bolts is 6.

Bolts details

The bolts are fully threaded, non-preloaded, M20 8.8, 60mm long as generally used in the UK.

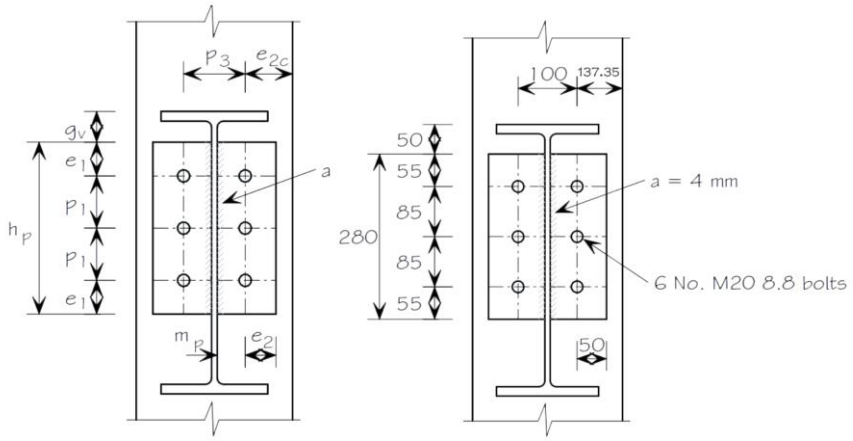


Figure A15.1 Position of holes for bolts. Retrieved from "Steel Building Design: Worked examples for students in accordance with Eurocodes and the UK National Annexes" by The Steel Construction Institute (p. 83), 2009.

Tensile stress area of bolt $A_s = 245\ mm^2$

Diameter of bolt $d = 20\ mm$

Diameter of the holes $d_0 = 22\ mm$

Diameter of the washer $d_w = 37\ mm$

Yield strength $f_{yb} = 640\ N/mm^2$

Ultimate tensile strength $f_{ub} = 800\ N/mm$^2

Position of bolts

$$e_1 = 55\ mm\ \ e_2 = 50\ mm\ \ p_1 = 85\ mm\ \ p_3 = 100\ mm$$

Table A15.1 Value of $e_{2,c}$

Column	$e_{2,c}$ (mm)
UKC 356×368×202	137.35
UKC 356×368×1202	185.5
UKC 356×368×235	147.4
UKC 356×368×634	162
UKC 356×368×467	156.1

The least of $e_{2,c}$ is 137.35mm.

BS EN 1993-1-8 3.5, Table 3.3

Limits for locations and spacing of bolts

Minimum $e_1 = 1.2d_0 = 1.2 \times 22 = 26.4\ mm < 55\ mm$

Minimum $e_2 = 1.2d_0 = 1.2 \times 22 = 26.4\ mm < 50\ mm$

Minimum $p_1 = 2.2d_0 = 2.2 \times 22 = 48.4\ mm < 85\ mm$

BS EN 1993-1-8 4.7.3

Minimum $p_3 = 2.4d_0 = 2.4 \times 22 = 52.8\ mm < 100\ mm$

Maximum $p_1 = 14t_p = 14 \times 10 = 140\ mm > 85\ mm$

Access Steel document SN014a-EN-EU

Weld design

Throat $a \geq 0.39 \times t_w$ is required for full strength side welds.

$$a > 0.39 \times 12.7 = 4.95\ mm$$

BS EN 1993-1-1 6.1(1) Table 2.1

Adopt throat $a = 5\ mm$ leg $= 6\ \ mm$

Patrial factors for resistance

BS EN 1993-1-8 NA 2.3 Table NA.1

$\gamma_{M0} = 1.0$

BS EN 1993-1-8 NA 2.3 Table NA.1

$\gamma_{M2} = 1.25\ (for\ shear)$

$\gamma_{M2} = 1.1\ (for\ bolts\ in\ tension)$

SN014a-EN-EU

$\gamma_{Mu} = 1.1\ (for\ tying\ resistance)$

Ductility check

The ductility of connection must meet the design requirement for acting as pinned and it can be satisfied with column flange or the end plate within: $t_p \leq \frac{d}{2.8}\sqrt{\frac{f_{u,b}}{f_{y,p}}}$ $or\ t_{f,c} < \frac{d}{2.8}\sqrt{\frac{f_{u,b}}{f_{y,c}}}$

$$\frac{d}{2.8}\sqrt{\frac{f_{u,b}}{f_{y,p}}} = \left(\frac{20}{2.8}\right) \times \sqrt{\frac{800}{355}} = 10.72\ mm > t_p = 10\ mm$$

Joint shear resistance

Normally, 8 modes of failure shown in the table below are required to be checked.

Table A15.2 Shear resistance to be checked

Mode of failure	
Bolts in shear*	$V_{Rd,1}$
End plate in bearing*	$V_{Rd,2}$
Supporting member (column) in bearing	$V_{Rd,3}$
End plate in shear (gross section)	$V_{Rd,4}$
End plate in shear (net section)	$V_{Rd,5}$
End plate in block shear	$V_{Rd,6}$
End plate in bending	$V_{Rd,7}$
Beam web in shear*	$V_{Rd,8}$

BS EN 1993-1-8 3.6.1 & Table 3.4

However, only critical failures with * need to be checked in the connections of the project.

Bolts in shear

Access Steel document SN014a-EN-EU

Shear resistance for shear planes assumed to pass through the threaded portion of the bolt is:

$$F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}}$$

A factor of 0.8 is introduced when calculating the shear resistance so that modest tension which doesn't calculated can be allowed.

For bolt class 8.8, $\alpha_v = 0.6$

$$F_{v,Rd} = 0.8 \times \frac{0.6 \times 800 \times 245}{1.25} \times 10^{-3} = 75.26\ kN$$

BS EN 1993-1-8 3.6.1 & Table 3.4

For 6 bolts, $V_{Rd,1} = 6 \times 75.26 = 451.58\ kN$

End plate in bearing

For a single bolt, $F_{v,Rd} = \frac{k_1 \alpha_b f_{u,p} d t_p}{\gamma_{M2}}$

Where $\alpha_b = \min\left(\alpha_d,\ \frac{f_{u,b}}{f_{u,p}}, 1.0\right)$

$$\alpha_d = \frac{e_1}{3d_0}\ for\ end\ bolts\ and\ \frac{p_1}{3d_0} - \frac{1}{4}\ for\ inner\ bolts$$

For end bolts

$$\alpha_b = \min\left(\frac{55}{3 \times 22}, \frac{800}{470}, 1.0\right) = \min(0.83, 1.70, 1.0) = 0.83$$

For inner bolts

$$\alpha_b = \min\left(\frac{85}{3 \times 22} - \frac{1}{4}, \frac{800}{470}, 1.0\right) = \min(1.04, 1.70, 1.0) = 1.0$$

Therefore $\alpha_b = 0.83$

$$k_1 = \min\left(\frac{2.8e_2}{d_0} - 1.7, 2.5\right) = \min\left(2.8 \times \frac{50}{22} - 1.7, 2.5\right) = \min(4.66, 2.5) = 2.5$$

For the end bolts

$$F_{v,Rd} = \frac{0.1 \times 0.83 \times 470 \times 20 \times 10}{1.25} \times 10^{-3} = 156.67\ kN$$

BS EN 1993-1-8:2005 3.7

For the inner bolts

$$F_{v,Rd} = \frac{0.1 \times 1 \times 470 \times 20 \times 10}{1.25} \times 10^{-3} = 213.64\ kN$$

Number of end bolts is 2 and number of inner bolts is 4.

The bearing resistance of the bolts:

$$V_{Rd,2} = 2 \times 156.67 + 4 \times 213.64 = 1167.88\ kN$$

BS EN 1993-1-1 6.2.6(2)

Group of Fasteners

Shear resistance is $F_{v,Rd} = 75.26\ kN$ and bearing resistance is $F_{b,Rd} = 156.67\ kN$, $75.26\ kN < 156.67\ kN$ Therefore, the smallest design resistance is $75.26\ kN$.

Access Steel document SN014a-EN-EU

Resistance of the group is $6 \times 75.26 = 451.58\ kN$

Beam web in shear

Only web of the beam is checked:

$$V_{pl,Rd} = V_{Rd,8} = \frac{A_v\left(\frac{f_y}{\sqrt{3}}\right)}{\gamma_{M0}}$$

A factor of 0.9 is introduced:

$$V_{Rd,8} = 0.9 \times \frac{(330 \times 12.7) \times \left(\frac{355}{\sqrt{3}}\right)}{1} \times 10^{-3} = 773.09\ kN$$

$> V_{Ed} 345.86\ kN\ OK$

Tying resistance of end plate

Table A15.3 Tying resistance to be checked

Mode of failure	
Bolts in tension	$N_{Rd,u,1}$
End plate in bearing*	$N_{Rd,u,2}$
Supporting member (column) in bearing	$N_{Rd,u,3}$
Beam web in tension	$N_{Rd,u,4}$

BS EN 1993-1-8 3.6.1 & Table 3.4

End plate in bearing is the only resistance to be checked in the design.

Bolts in tension

This is checked because end plate in bending needs tension capacity of the bolt

group.

$$F_{t,Rd} = \frac{k_2 f_{ub} A_s}{\gamma_{Mu}}$$

Where $k_2 = 0.9$, so $N_{Rd,u,1} = F_{t,Rd} = \frac{0.9 \times 800 \times 245}{1.1} \times 10^{-3} = 160.36\ kN$

For 6 bolts, $N_{Rd,u,1} = 6 \times 160.36 = 962.18\ kN$

End plate in bending

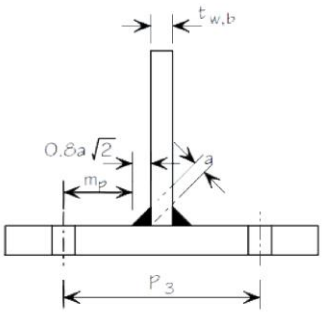


Figure A15.2 Plan view of details for end plate. Retrieved from "Steel Building Design: Worked examples for students in accordance with Eurocodes and the UK National Annexes" by The Steel Construction Institute (p. 87), 2009.

BS EN 1993-1-8 6.2.4 & Table 6.2

$$N_{Rd,u,2} = \min\left(F_{Rd,u,ep1}, F_{Rd,u,ep2}\right)$$

For mode 1: $F_{Rd,u,ep1} = F_{T,1,Rd} = \frac{(8n_p - 2e_w)M_{pl,1,Rd}}{2m_p n_p - e_w(m_p + n_p)}$

For mode 2: $F_{Rd,u,ep2} = F_{T,2,Rd} = \frac{(2M_{pl,2,Rd} + n_p \sum F_{t,Rd})}{m_p + n_p}$

Where $n_p = \min\left(e_2, e_{2,c}, 1.25m_p\right)$

$$m_p = \frac{p_3 - t_{w,b} - 2 \times 0.8a\sqrt{2}}{2} = \frac{100 - 12.7 - 2 \times 0.8 \times 5 \times \sqrt{2}}{2} = 37.99\ mm$$

$$e_w = \frac{d_w}{4} = \frac{37}{4} = 9.25\ mm$$

$$1.25m_p = 1.25 \times 37.99 = 47.49\ mm$$

$e_{c,2} = \frac{374.7-100}{2} = 137.35\ mm$ which is the least of all columns.

$$n_p = \min(50, 137.35, 47.49) = 47.49\ mm$$

$$M_{pl,1,Rd} = \frac{1}{4}\frac{\sum \ell_{eff,1} t_p^2 f_{y,p}}{\gamma_{M0}}\ (Mode\ 1)$$

$$M_{pl,2,Rd} = \frac{1}{4}\frac{\sum \ell_{eff,2} t_p^2 f_{y,p}}{\gamma_{M0}}\ (Mode\ 2)$$

Where ℓ_{eff} is equal to the length of the plate for simplicity: $\sum \ell_{eff,1} = \sum \ell_{eff,2} = h_p = 330\ mm$

BS EN 1993-1-8 6.2.6.5 & Table 6.6

$$M_{pl,1,Rd,u} = \frac{1}{4}\frac{\left(h_p t_p^2 f_{u,p}\right)}{\gamma_{Mu}} = \frac{1}{4} \times \frac{330 \times 10^2 \times 470}{1.1} \times 10^{-6} = 3.53\ kNm$$

Mode 1: $F_{T,1,Rd} = \frac{(8\times50-2\times9.25)\times3.53\times10^3}{(2\times37.99\times47.49)-9.25\times(37.99+47.49)} = 452.12\ kN$

Mode 2: $F_{T,1,Rd} = \frac{2\times3.53\times10^3+47.49\times962.18}{37.99+47.49} = 617.02\ kN$

$$F_{T,1,Rd} = \min(452.12, 617.02) = 452.12\ kN$$

Where $M_{pl,2,Rd,u} = M_{pl,1,Rd,u}$

Therefore, $N_{Rd,u,2} = 452.12\ kN$

Summary of the results

There are total 3 joint shear resistance and two tying resistance of end plate. Thickness of column flange is 21.3 mm which is larger than the thickness of end plate 10 mm, so bending doesn't need to be checked.

Table A15.3 Joint shear resistance

Mode of failure		
Bolts in shear*	$V_{Rd,1}$	$451.58\ kN$
End plate in bearing*	$V_{Rd,2}$	$1167.88\ kN$
Beam web in shear	$V_{Rd,8}$	$773.09\ kN$

The critical shear resistance is 451.58 kN.

Table A15.4 Tying resistance of end plate

Mode of failure		
Bolts in shear*	$N_{Rd,u,1}$	$962.18\ kN$
End plate in bearing*	$N_{Rd,u,2}$	$452.12\ kN$

The critical tying resistance is 452.12 kN.

Appendix A16

Column Connections Calculations

A16.1 Column Base Connections Design

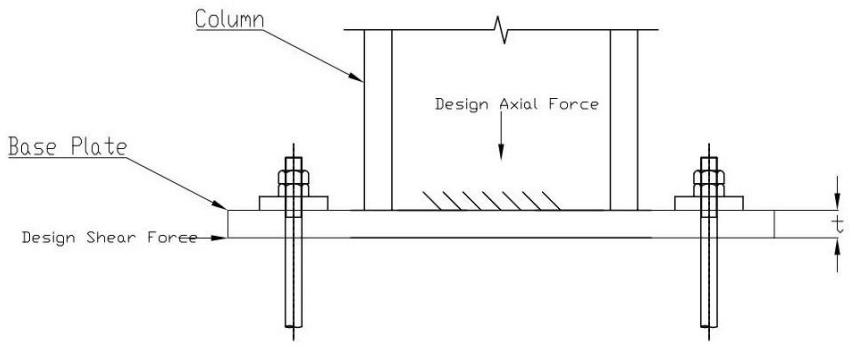


Figure A16.1: Figure showing the elevation of the column base joints considered in this project.

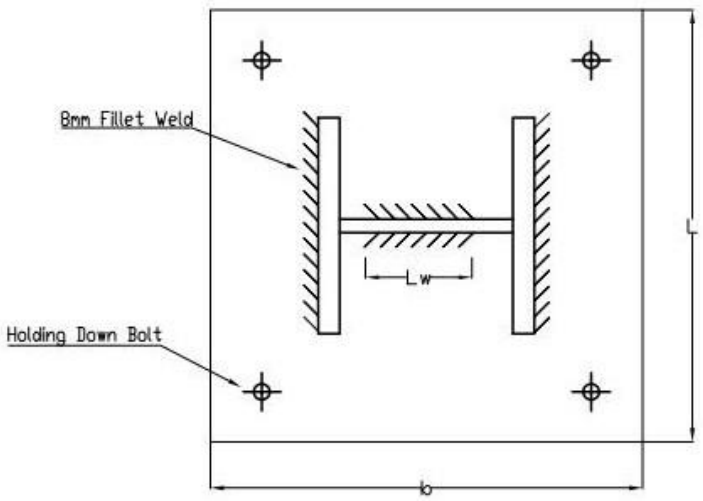


Figure A16.2: Figure showing the cross-section of the column base joints considered in this project.

The results for the design of five different types of column base connections are listed in the table below. Note that the axial load from Proposal 2 was adopted for the design of base joints for columns at bracing bay (356 x 406 x 1202 UC) as it was more critical.

Table A16.1: Table summarizing the results of column base connections

For bottom internal columns (356 x 406 x 467 UC)	
Design Axial Force N_{Ed}	12491kN
Design Shear Force V_{Ed}	101kN
Base Plate Dimensions (Lxbxt)	1000mm x 1000mm x 120mm
Grade of Base Plate	S355
Weld Length along Shear L_w	100mm
Concrete Foundation Grade	C25/30

For bottom peripheral columns (356 x 406 x 235 UC)	
Design Axial Force N_{Ed}	6546kN
Design Shear Force V_{Ed}	101kN
Base Plate Dimensions (Lxbxt)	800mm x 800mm x 80mm
Grade of Base Plate	S355
Weld Length along Shear L_w	100mm
Concrete Foundation Grade	C25/30

For bottom columns at cantilever region (356 x 406 x 634 UC)	
Design Axial Force N_{Ed}	19022kN
Design Shear Force V_{Ed}	101kN
Base Plate Dimensions (Lxbxt)	1250mm x 1250mm x 150mm
Grade of Base Plate	S355
Weld Length along Shear L_w	100mm
Concrete Foundation Grade	C25/30

For bottom bracing columns – Proposal 1 (356 x 406 x 1202 UC)	
Design Axial Force N_{Ed}	21721kN
Design Shear Force V_{Ed}	101kN
Base Plate Dimensions (Lxbxt)	1250mm x 1250mm x 150mm
Grade of Base Plate	S355
Weld Length along Shear L_w	100mm
Concrete Foundation Grade	C25/30

Column Base Connection Calculations (Bottom Internal Columns)

Section	356 x 406 x 467	UC	

Section Properties

Grade	355		
h	436.6	mm	
b	412.2	mm	
t_w	35.8	mm	
t_f	58	mm	
r	15.2	mm	
Perimeter	2450.4	mm	
A	595	cm^2	
E	210	GPa	

Base Design using Resistance Table G.32 of SCI P358

N_{Ed}	12491	kN	
For a 1000 x 1000 x 120 baseplate with C25/30 concrete,			
Resistance obtained from table=	13500	kN	**OK**

Check 1: Required Area

According to BSEN1992-1-1 Table 3.1 and NA2,

Area of Base Plate $A_p = h_p \times b_p =$	1000000	mm^2	
$f_{cd}=\alpha_{cc}f_{ck}/\gamma_c =$	14.1667	MPa	
$f_{jd}=\alpha\beta_j f_{cd} =$	14.1667	MPa	

, where α=1.5, β_j=2/3

Area Required =	881733.8	mm^2	**OK**

As it is larger than area of base plate, area required is sufficient

Check 2: Effective Area

Basic Requirement: $A_{eff}=A_{req}$

Effective area = $4c^2$+ Section Perimeter x c + Section Area

, where c is the cantilever outstand of effective area

c =	395.4	mm
$(h-2t_f)/2$ =	160.3	mm

There is overlap between flanges

Check effective area on base plate:

h+2c=	1227.5	mm	
b+2c=	1203.1	mm	**FURTHER CHECK**

As the effective area falls outside the base plate, assumption is not valid.

Calculation of Revised A_{eff} based on A_{eff}=(h+2c)(b+2c)

c=	257.3	mm

Re-check effective area on base plate:

h+2c=	951.3	mm	
b+2c=	926.9	mm	**OK**

As the effective area falls outside the base plate, assumption is not valid.

Check 3: Plate Thickness

According to BSEN1993-1-8 Clause 6.2.5(4),

Yield strength of plate with thickness	120	mm	
f_{yp}	335	MPa	
$t_{p,min}=c(3f_{jd}\gamma_{M0}/f_{yp})^{0.5}$	91.7	mm	**OK**

The plate thickness is larger than minimum value.

Check 4: Welds

Basic Requirement: $V_{Ed}<=F_{w,Rd}l_{w,eff}$

$F_{w,Rd}$ denotes the fillet weld resistance per unit length

$l_{w,eff}$ denotes the effective length of weld

According to Table 3.1 of BSEN1993-1-1,

For 120mm thick plate, f_u=	470	MPa
β_w=	0.9	
γ_{M2}=	1.25	(National Annex)
Propose weld leg length s =	8	mm
Throat thickness a = 0.7s =	5.6	mm
Using Mean-Stress Method,		
$F_{w,Rd}=F_{vw,d}a=af_u/3^{0.5}/\beta_w/\gamma_{M2}$=	1350.7	N/mm
Propose weld leg length along shear	100	mm
$l_{w,eff}$=2(l-2s)=	168	mm
$F_{w,Rd}l_{w,eff}$=	226.9	kN

Assume V_{Ed} is calculated from the lateral reaction at each column due to $1.5W_k$+EHF

V_{Ed}=	101.0	kN	**OK**

Therefore, $V_{Ed}<=F_{w,Rd}l_{w,eff}$

Column Base Connection Calculations (Bottom Peripheral Columns)

Section	356 x 406 x 235		UC

Section Properties

Grade	355		
h	381	mm	
b	394.8	mm	
t_w	18.4	mm	
t_f	30.2	mm	
r	15.2	mm	
Perimeter	2304.4	mm	
A	299	cm^2	
E	210	GPa	

Base Design using Resistance Table G.32 of SCI P358

N_{Ed}	6546	kN	
For a 800 x 800 x 80 baseplate with C25/30 concrete, Resistance obtained from table=	8440	kN	**OK**

Check 1: Required Area

According to BSEN1992-1-1 Table 3.1 and NA2,

Area of Base Plate $A_p = h_p \times b_p =$	640000	mm^2	
$f_{cd}=\alpha_{cc}f_{ck}/\gamma_c =$	14.1667	MPa	
$f_{jd}=\alpha\beta_j f_{cd} =$	14.1667	MPa	
, where α=1.5, β_j=2/3			
Area Required =	462038.5	mm^2	**OK**

As it is larger than area of base plate, area required is sufficient

Check 2: Effective Area

Basic Requirement: $A_{eff}=A_{req}$

Effective area = $4c^2$+ Section Perimeter x c + Section Area

, where c is the cantilever outstand of effective area

c =	149.0	mm	
$(h-2t_f)/2$ =	160.3	mm	

There is no overlap between flanges

Check effective area on base plate:

h+2c=	679.0	mm	
b+2c=	692.8	mm	**OK**

As the effective area falls within the base plate, assumption is valid.

Check 3: Plate Thickness

According to BSEN1993-1-8 Clause 6.2.5(4),

Yield strength of plate with thickness	80	mm

f_{yp}	335	MPa	
$t_{p,min}=c(3f_{jd}\gamma_{M0}/f_{yp})^{0.5}$	53.1	mm	**OK**

The plate thickness is larger than minimum value.

Check 4: Welds

Basic Requirement: $V_{Ed}<=F_{w,Rd}l_{w,eff}$

$F_{w,Rd}$ denotes the fillet weld resistance per unit length

$l_{w,eff}$ denotes the effective length of weld

According to Table 3.1 of BSEN1993-1-1,

For 50mm thick plate, f_u=	470	MPa	
β_w=	0.9		
γ_{M2}=	1.25	(National Annex)	
Propose weld leg length s =	8	mm	
Throat thickness a = 0.7s =	5.6	mm	
Using Mean-Stress Method,			
$F_{w,Rd}=F_{vw,d}a=af_u/3^{0.5}/\beta_w/\gamma_{M2}$=	1350.7	N/mm	
Propose weld leg length along shear	100	mm	
$l_{w,eff}$=2(l-2s)=	168	mm	
$F_{w,Rd}l_{w,eff}$=	226.9	kN	

Assume V_{Ed} is calculated from the lateral reaction at each column due to $1.5W_k$+EHF

V_{Ed}=	101.0	kN	**OK**

Therefore, $V_{Ed}<=F_{w,Rd}l_{w,eff}$

Column Base Connection Calculations (Columns at Cantilever Region)

Section	356 x 406 x 634	UC	

Section Properties

Grade	355		
h	474.6	mm	
b	424	mm	
t_w	47.6	mm	
t_f	77	mm	
r	15.2	mm	
Perimeter	2550	mm	
A	808	cm^2	
E	210	GPa	

Base Design using Resistance Table G.32 of SCI P358

N_{Ed}	19022	kN	
For a 1250 x 1250 x 150 baseplate with C25/30 concrete,			
Resistance obtained from table=	17600	kN	**FURTHER CHECK**

Check 1: Required Area

According to BSEN1992-1-1 Table 3.1 and NA2,

Area of Base Plate $A_p = h_p \times b_p =$	1562500	mm^2	
$f_{cd}=\alpha_{cc}f_{ck}/\gamma_c =$	14.1667	MPa	
$f_{jd}=\alpha\beta_j f_{cd} =$	14.1667	MPa	
, where α=1.5, β_j=2/3			
Area Required =	1342729	mm^2	**OK**

As it is larger than area of base plate, area required is sufficient

Check 2: Effective Area

Basic Requirement: $A_{eff}=A_{req}$

Effective area = $4c^2$+ Section Perimeter x c + Section Area

, where c is the cantilever outstand of effective area

c =	327.1	mm	
$(h-2t_f)/2$ =	160.3	mm	

There is overlap between flanges

Check effective area on base plate:

h+2c=	1128.7	mm	
b+2c=	1078.1	mm	**OK**

As the effective area falls within the base plate, assumption is valid.

Check 3: Plate Thickness

According to BSEN1993-1-8 Clause 6.2.5(4),

Yield strength of plate with thickness	150	mm	
f_{yp}	335	MPa	
$t_{p,min}=c(3f_{jd}\gamma_{M0}/f_{yp})^{0.5}$	116.5	mm	**OK**

The plate thickness is larger than minimum value.

Check 4: Welds

Basic Requirement: $V_{Ed}<=F_{w,Rd}l_{w,eff}$

$F_{w,Rd}$ denotes the fillet weld resistance per unit length

$l_{w,eff}$ denotes the effective length of weld

According to Table 3.1 of BSEN1993-1-1,

For 110mm thick plate, f_u=	470	MPa	
β_w=	0.9		
γ_{M2}=	1.25	(National Annex)	
Propose weld leg length s =	8	mm	
Throat thickness a = 0.7s =	5.6	mm	
Using Mean-Stress Method,			
$F_{w,Rd}=F_{vw,d}a=af_u/3^{0.5}/\beta_w/\gamma_{M2}$=	1350.7	N/mm	
Propose weld leg length along shear	100	mm	
$l_{w,eff}$=2(l-2s)=	168	mm	
$F_{w,Rd}l_{w,eff}$=	226.9	kN	

Assume V_{Ed} is calculated from the lateral reaction at each column due to $1.5W_k$+EHF

V_{Ed}=	101.0	kN	**OK**

Therefore, $V_{Ed}<=F_{w,Rd}l_{w,eff}$

Column Base Connection Calculations (Bracing System)

Section	356 x 406 x 1202	UC	

Section Properties

Grade	355		
h	580	mm	
b	471	mm	
t_w	95	mm	
t_f	130	mm	
r	15.4	mm	
Perimeter	2854	mm	
A	1531	cm^2	
E	210	GPa	

Base Design using Resistance Table G.32 of SCI P358

N_{Ed}	21721	kN	
For a 1250 x 1250 x 150 baseplate with C25/30 concrete,			
Resistance obtained from table=	17600	kN	**FURTHER CHECK**

Check 1: Required Area

According to BSEN1992-1-1 Table 3.1 and NA2,

Area of Base Plate $A_p = h_p$ x b_p =	1562500	mm^2	
$f_{cd}=\alpha_{cc}f_{ck}/\gamma_c$ =	14.1667	MPa	
$f_{jd}=\alpha\beta_j f_{cd}$ =	14.1667	MPa	

, where α=1.5, β_j=2/3 from BSEN1993-1-8 Clause 6.2.5(7)

Area Required =	1533247	mm^2	**OK**

As it is larger than area of base plate, area required is sufficient

Check 2: Effective Area

Basic Requirement: $A_{eff}=A_{req}$

Effective area = $4c^2$+ Section Perimeter x c + Section Area

, where c is the cantilever outstand of effective area

c =	330.5	mm
$(h-2t_f)/2$ =	160	mm

There is overlap between flanges

Check effective area on base plate:

h+2c=	1241.0	mm	
b+2c=	1132.0	mm	**OK**

As the effective area falls within the base plate, assumption is valid.

Check 3: Plate Thickness

According to BSEN1993-1-8 Clause 6.2.5(4),

Yield strength of plate with thickness	120	mm	
f_{yp}	335	MPa	
$t_{p,min}=c(3f_{jd}\gamma_{M0}/f_{yp})^{0.5}$	117.7	mm	**OK**

The plate thickness is larger than minimum value.

Check 4: Welds

Basic Requirement: $V_{Ed}<=F_{w,Rd}l_{w,eff}$

$F_{w,Rd}$ denotes the fillet weld resistance per unit length

$l_{w,eff}$ denotes the effective length of weld

According to Table 3.1 of BSEN1993-1-1,

For 110mm thick plate, f_u=	470	MPa	
β_w=	0.9		
γ_{M2}=	1.25	(National Annex)	
Propose weld leg length s =	8	mm	
Throat thickness a = 0.7s =	5.6	mm	

Using Mean-Stress Method,

$F_{w,Rd}=F_{vw,d}a=af_u/3^{0.5}/\beta_w/\gamma_{M2}$=	1350.7	N/mm	
Propose weld leg length along shear	100	mm	
$l_{w,eff}$=2(l-2s)=	168	mm	
$F_{w,Rd}l_{w,eff}$=	226.9	kN	

Assume V_{Ed} is calculated from the lateral reaction at each column due to $1.5W_k$+EHF

V_{Ed}=	101.0	kN	**OK**

Therefore, $V_{Ed}<=F_{w,Rd}l_{w,eff}$

A16.2 Column-Column Connections

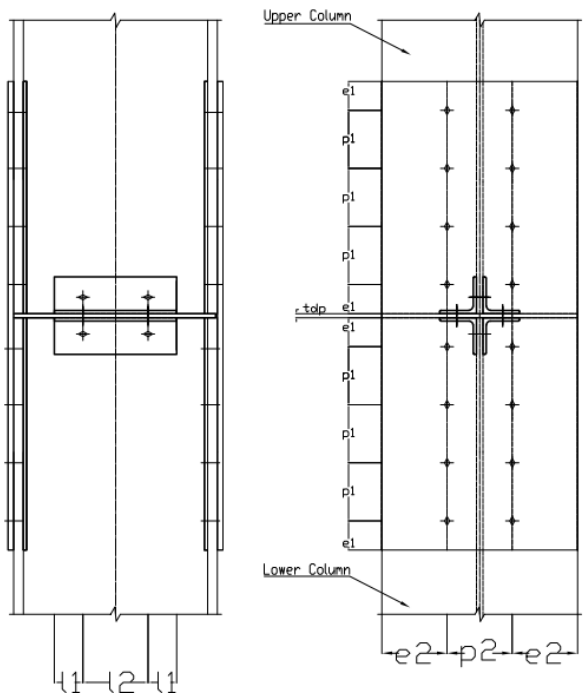


Figure A16.3: Figure showing the results of column-column connections (8 bolts per column)

Table A16.2: Table showing the results of column-column connections (8 bolts per column)

Type 1	
Upper Column Size	356 x 406 x 467 UC
Lower Column Size	356 x 406 x 467 UC
Flange Cover Plate	2/420 x 40 x 1000
Cleat	4/90 x 90 x 8 Angles x 220 Long
Division Plate	420 x 20 x 420
Bolts	M22 Grade 8.8
Type 2	
Upper Column Size	356 x 406 x 634 UC
Lower Column Size	356 x 406 x 634 UC
Flange Cover Plate	2/430 x 40 x 1150
Cleat	4/90 x 90 x 8 Angles x 250 Long
Division Plate	430 x 10 x 430
Bolts	M27 Grade 8.8

Type 3

Upper Column Size	356 x 406 x 235 UC
Lower Column Size	356 x 406 x 634 UC
Flange Cover Plate	2/400 x 40 x 1220
Cleat	2/400 x 50 x 240
Division Plate	400 x 10 x 400
Bolts	M30 Grade 8.8

Type 4

Upper Column Size	356 x 406 x 1202 UC
Lower Column Size	356 x 406 x 1202 UC
Flange Cover Plate	2/480 x 40 x 1150
Cleat	4/90 x 90 x 8 Angles x 300 Long
Division Plate	480 x 10 x 480
Bolts	M27 Grade 8.8

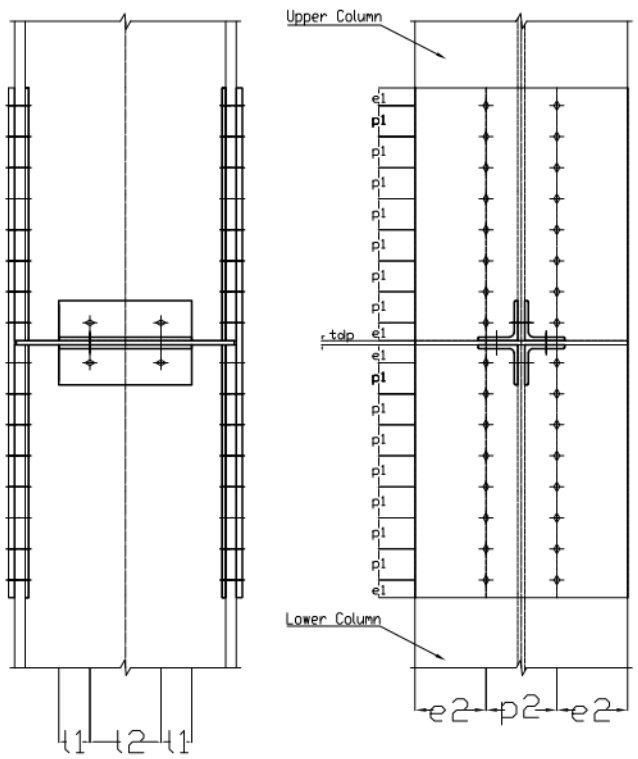


Figure A16.4: Figure showing the results of column-column connections (16 bolts per column)

Table A16.3: Table showing the results of column-column connections (16 bolts per column)

Type 5	
Upper Column Size	356 x 368 x 202 UC
Lower Column Size	356 x 406 x 467 UC
Flange Cover Plate	2/380 x 40 x 820
Cleat	4/90 x 90 x 8 Angles x 200 Long
Division Plate	380 x 20 x 380
Bolts	M27 Grade 8.8

Type 6	
Upper Column Size	356 x 368 x 202 UC
Lower Column Size	356 x 368 x 202 UC
Flange Cover Plate	2/380 x 40 x 800
Cleat	4/90 x 90 x 8 Angles x 200 Long
Division Plate	380 x 20 x 380
Bolts	M20 Grade 8.8

Type 7	
Upper Column Size	356 x 406 x 235 UC
Lower Column Size	356 x 406 x 235 UC
Flange Cover Plate	2/400 x 20 x 800
Cleat	4/90 x 90 x 8 Angles x 200 Long
Division Plate	400 x 20 x 400
Bolts	M22 Grade 8.8

Type 8	
Upper Column Size	356 x 368 x 202 UC
Lower Column Size	356 x 406 x 235 UC
Flange Cover Plate	2/380 x 40 x 800
Cleat	4/90 x 90 x 8 Angles x 200 Long
Division Plate	380 x 20 x 380
Bolts	M20 Grade 8.8

Type 9	
Upper Column Size	356 x 368 x 202 UC
Lower Column Size	356 x 368 x 202 UC
Flange Cover Plate	2/380 x 40 x 800
Cleat	4/90 x 90 x 8 Angles x 200 Long
Division Plate	380 x 20 x 380
Grade of Materials	M20 Grade 8.8

Detailed calculations for each type of column-column connections are listed below:

Column to Column Connection Calculations (Internal Columns 3rd Floor to 1st Floor)

Column Section Above	356 x 406 x 467	UC
Column Section Below	356 x 406 x 467	UC

Section Properties (Upper Column)

Grade	355	
h_{uc}	436.6	mm
b_{uc}	412.2	mm
$t_{w,uc}$	35.8	mm
$t_{f,uc}$	58	mm
r_{uc}	15.2	mm
A_{uc}	595	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	436.6	mm
b_{lc}	412.2	mm
$t_{w,lc}$	35.8	mm
$t_{f,lc}$	58	mm
r_{lc}	15.2	mm
A_{lc}	595	cm^2

Splice Details

Flange Cover Plate	2/420 x 40 x 1000
Cleats	4/90 x 90 x 8 Angles x 220 Long
Division Plate	420 x 20 x 420
Bolts	M22 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	7743	kN
N_{Ed} =	12491	kN

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height $h_{fp} >= 2b_{uc} + t_{dp}$=	844.4	mm
Width $b_{fp} >= b_{uc}$	412.2	mm
Vertical Bolt Spacing p_1=	60	mm
$p_1/14$=	4.3	mm

$t_{f,uc}$=	58	mm	
Thickness t_{fp}>=max($t_{f,uc}$/2, 10mm, p_1/14)			
t_{fp}>=	29	mm	
Therefore, propose h_{fp} =	1000	mm	
Therefore, propose b_{fp} =	420	mm	
Therefore, propose t_{fp} =	40	mm	
Division Plate			
tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm	
Therefore, propose t_{dp} =	20	mm	
Web Cleats			
Use 90 x 90 x 8 angles to accommodate M22 bolts.			
Length >= 0.5h_{uc}=	218.3	mm	
Therefore, propose length of	220	mm	
Check 2: Minimum Resistance			
Cover Plate			
Requirement: 0.25N_{Ed}<=N_{Rd}			
0.25N_{Ed}=	3122.8	kN	
N_{Rd}=2$A_{fp}f_{y,fp}/\gamma_{M0}$=	11928	kN	**OK**
Bolt Group			
Requirement: 0.25N_{Ed}<=2$F_{Rd,fp}$			
a) Shear Resistance per Bolt:			
Diameter of Bolts =	22	mm	
Total thickness of packing t_{pa} =	0	mm	**<d/3**
For t_{pa}>=d/3,β_p=9d/(8d+3t_{pa})=	1.125		
For t_{pa}<d/3, β_p=	1		
Therefore, β_p=	1		
α_v=	0.6	for 8.8 bolts	
Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	303	mm2	
γ_{M2}=	1.25		
$F_{v,Rd}$=$\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}$=	116.4	kN	
b) Bearing Resistance per Bolt:			
Spacings are desgined based on BSEN1993-1-8 Table 3.3:			
Propose e_1=	35	mm	1000
Propose e_2=	130	mm	
Propose p_1=	60	mm	

Propose p_2=	160	mm	
Bolt Hole Diameter d_0=	24	mm	
$f_{u,p}$=	470	MPa	
α_d=min($e_1/(3d_0)$,$p_1/(3d_0)$-0.25,$f_{ub}/f_{u,p}$,1)=	0.5		
k_1=min($2.8e_2/d_0$-1.7,$1.4p_2/d_0$-1.7,2.5)=	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}$=	402.1	kN	
min($F_{v,Rd}$,$F_{b,Rd}$)=	116.4	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	16	nos.	
$F_{Rd,fp}$=min($F_{v,Rd}$,$F_{b,Rd}$)(n) =	1861.6	kN	
As double shear is designed,			
$2F_{Rd,fp}$=	3723.3	kN	
$0.25N_{Ed}$=	3122.8	kN	**OK**

Therefore, the design satisfies the requirements.

Column to Column Connection Calculations (Internal Columns 4th Floor)

Column Section Above	356 x 368 x 202	UC
Column Section Below	356 x 406 x 467	UC

Section Properties (Upper Column)

Grade	355	
h_{uc}	374.6	mm
b_{uc}	374.7	mm
$t_{w,uc}$	16.5	mm
$t_{f,uc}$	27	mm
r_{uc}	15.2	mm
A_{uc}	257	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	436.6	mm
b_{lc}	412.2	mm
$t_{w,lc}$	35.8	mm
$t_{f,lc}$	58	mm
r_{lc}	15.2	mm
A_{lc}	595	cm^2

Splice Details

Flange Cover Plate	2/380 x 40 x 820
Flange Packs	2/380 x 40 x 240

Cleats	4/90 x 90 x 8 Angles x 200 Long	
Web Packs	2/85 x 10 x 200	
Division Plate	380 x 20 x 380	
Bolts	M27 8.8	
Fittings Material	S355	

Design Actions

$N_{Ed,G}$ =	4252	kN
N_{Ed} =	6733	kN

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height h_{fp} >= $2b_{uc} + t_{dp}$=	769.4	mm
Width b_{fp} >= b_{uc}	374.7	mm
Vertical Bolt Spacing p_1=	110	mm
$p_1/14$=	7.9	mm
$t_{f,uc}$=	27	mm
Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)		
t_{fp}>=	13.5	mm
Therefore, propose h_{fp} =	820	mm
Therefore, propose b_{fp} =	380	mm
Therefore, propose t_{fp} =	40	mm

Flange Packs

$t_{pa}=(h_{lc}-h_{uc})/2$=	31	mm
Therefore, propose t_{pa} =	40	mm

Division Plate

tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm
Therefore, propose t_{dp} =	20	mm

Web Cleats

Use 90 x 90 x 8 angles to accommodate M27 bolts.

Length >= $0.5h_{uc}$=	187.3	mm
Therefore, propose length of	200	mm

Web Packs

$t_p = (t_{w,lc}-t_{w,uc})/2$=	9.65	
Therefore, propose t_p =	10	mm

Check 2: Minimum Resistance

Cover Plate

Requirement: $0.25N_{Ed}<=N_{Rd}$

$0.25N_{Ed}$=	1683.2	kN	
$N_{Rd}=2A_{fp}f_{y,fp}/\gamma_{M0}$=	10792	kN	**OK**

<u>Bolt Group</u>
Requirement: $0.25N_{Ed}<=2F_{Rd,fp}$

a) Shear Resistance per Bolt:

Diameter of Bolts =	27	mm	
Total thickness of packing t_{pa} =	40	mm	**>d/3**
For $t_{pa}>=d/3,\beta_p=9d/(8d+3t_{pa})$=	0.7		
For $t_{pa}<d/3$, β_p=	1		
Therefore, β_p=	0.7		
α_v=	0.6	for 8.8 bolts	
Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	459	mm2	
γ_{M2}=	1.25		
$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}$=	127.5	kN	

b) Bearing Resistance per Bolt:
Spacings are desgined based on BSEN1993-1-8 Table 3.3:

Propose e_1=	35	mm	820
Propose e_2=	110	mm	
Propose p_1=	110	mm	
Propose p_2=	160	mm	
Bolt Hole Diameter d_0=	30	mm	
$f_{u,p}$=	470	MPa	
$\alpha_d=\min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)$=	0.4		
$k_1=\min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)$=	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}$=	394.8	kN	
$\min(F_{v,Rd},F_{b,Rd})$=	127.5	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	8	nos.	
$F_{Rd,fp}=\min(F_{v,Rd},F_{b,Rd})(n)$ =	1019.8	kN	
As double shear is designed,			
$2F_{Rd,fp}$=	2039.5	kN	
$0.25N_{Ed}$=	1683.2	kN	**OK**

Therefore, the design satisfies the requirements.

Column to Column Connection Calculations (Internal Columns 5th Floor to 8th Floor)

Column Section Above	356 x 368 x 202	UC
Column Section Below	356 x 368 x 202	UC

Section Properties (Upper Column)

Grade	355	
h_{uc}	374.6	mm
b_{uc}	374.7	mm
$t_{w,uc}$	16.5	mm
$t_{f,uc}$	27	mm
r_{uc}	15.2	mm
A_{uc}	257	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	374.6	mm
b_{lc}	374.7	mm
$t_{w,lc}$	16.5	mm
$t_{f,lc}$	27	mm
r_{lc}	15.2	mm
A_{lc}	257	cm^2

Splice Details

Flange Cover Plate	2/380 x 40 x 800
Cleats	4/90 x 90 x 8 Angles x 200 Long
Division Plate	380 x 20 x 380
Bolts	M22 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	4252	kN
N_{Ed} =	6733	kN

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height h_{fp} >= $2b_{uc} + t_{dp}$=	769.4	mm
Width b_{fp} >= b_{uc}	374.7	mm
Vertical Bolt Spacing p_1=	110	mm
$p_1/14$=	7.9	mm
$t_{f,uc}$=	27	mm
Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)		
t_{fp}>=	13.5	mm

Therefore, propose h_{fp} =	800	mm	
Therefore, propose b_{fp} =	380	mm	
Therefore, propose t_{fp} =	40	mm	
Division Plate			
tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm	
Therefore, propose t_{dp} =	20	mm	
Web Cleats			
Use 90 x 90 x 8 angles to accommodate M22 bolts.			
Length >= 0.5h_{uc}=	187.3	mm	
Therefore, propose length of	200	mm	
Check 2: Minimum Resistance			
Cover Plate			
Requirement: 0.25N_{Ed}<=N_{Rd}			
0.25N_{Ed}=	1683.2	kN	
N_{Rd}=2$A_{fp}f_{y,fp}/\gamma_{M0}$=	10792	kN	**OK**
Bolt Group			
Requirement: 0.25N_{Ed}<=2$F_{Rd,fp}$			
a) Shear Resistance per Bolt:			
Diameter of Bolts =	22	mm	
Total thickness of packing t_{pa} =	0	mm	**<d/3**
For t_{pa}>=d/3,β_p=9d/(8d+3t_{pa})=	1.125		
For t_{pa}<d/3, β_p=	1		
Therefore, β_p=	1		
α_v=	0.6	for 8.8 bolts	
Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	303	mm2	
γ_{M2}=	1.25		
$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}$=	116.4	kN	
b) Bearing Resistance per Bolt:			
Spacings are desgined based on BSEN1993-1-8 Table 3.3:			
Propose e_1=	30	mm	800
Propose e_2=	110	mm	
Propose p_1=	110	mm	
Propose p_2=	160	mm	
Bolt Hole Diameter d_0=	24	mm	
$f_{u,p}$=	470	MPa	

$\alpha_d=\min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)=$	0.4		
$k_1=\min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)=$	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}=$	344.7	kN	
$\min(F_{v,Rd},F_{b,Rd})=$	116.4	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	8	nos.	
$F_{Rd,fp}=\min(F_{v,Rd},F_{b,Rd})(n)=$	930.8	kN	

As double shear is designed,

$2F_{Rd,fp}=$	1861.6	kN	
$0.25N_{Ed}=$	1683.2	kN	**OK**

Therefore, the design satisfies the requirements.

Column to Column Connection Calculations (Peripheral Columns 3rd Floor to 1st Floor)

Column Section Above	356 x 406 x 235	UC
Column Section Below	356 x 406 x 235	UC

Section Properties (Upper Column)

Grade	355	
h_{uc}	381	mm
b_{uc}	394.8	mm
$t_{w,uc}$	18.4	mm
$t_{f,uc}$	30.2	mm
r_{uc}	15.2	mm
A_{uc}	299	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	381	mm
b_{lc}	394.8	mm
$t_{w,lc}$	18.4	mm
$t_{f,lc}$	30.2	mm
r_{lc}	15.2	mm
A_{lc}	299	cm^2

Splice Details

Flange Cover Plate	2/400 x 20 x 800
Cleats	4/90 x 90 x 8 Angles x 200 Long
Division Plate	400 x 20 x 400
Bolts	M22 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	4083	kN	
N_{Ed} =	6546	kN	

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height h_{fp} >= $2b_{uc} + t_{dp}$=	809.6	mm	
Width b_{fp} >= b_{uc}	394.8	mm	
Vertical Bolt Spacing p_1=	110	mm	
$p_1/14$=	7.86	mm	
$t_{f,uc}$=	30.2	mm	
Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)			
t_{fp}>=	15.1	mm	
Therefore, propose h_{fp} =	800	mm	
Therefore, propose b_{fp} =	400	mm	
Therefore, propose t_{fp} =	40	mm	

Division Plate

tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm	
Therefore, propose t_{dp} =	20	mm	

Web Cleats

Use 90 x 90 x 8 angles to accommodate M22 bolts.

Length >= 0.5h_{uc}=	190.5	mm	
Therefore, propose length of	200	mm	

Check 2: Minimum Resistance

Cover Plate

Requirement: $0.25N_{Ed}$<=N_{Rd}

$0.25N_{Ed}$=	1636.39	kN	
$N_{Rd}=2A_{fp}f_{y,fp}/\gamma_{M0}$=	11360	kN	**OK**

Bolt Group

Requirement: $0.25N_{Ed}$<=$2F_{Rd,fp}$

a) Shear Resistance per Bolt:

Diameter of Bolts =	22	mm	
Total thickness of packing t_{pa} =	0	mm	**<d/3**
For t_{pa}>=d/3,β_p=9d/(8d+3t_{pa})=	1.125		
For t_{pa}<d/3, β_p=	1		
Therefore, β_p=	1		
α_v=	0.6	for 8.8 bolts	

Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	303	mm2	
γ_{M2}=	1.25		
$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}$=	116.35	kN	
b) Bearing Resistance per Bolt:			
Spacings are desgined based on BSEN1993-1-8 Table 3.3:			
Propose e_1=	30	mm	800
Propose e_2=	120	mm	
Propose p_1=	110	mm	
Propose p_2=	160	mm	
Bolt Hole Diameter d_0=	24	mm	
$f_{u,p}$=	470	MPa	
$\alpha_d=\min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)$=	0.42		
$k_1=\min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)$=	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}$=	344.67	kN	
$\min(F_{v,Rd},F_{b,Rd})$=	116.35	kN	
As column flange is thicker, it is less critical. Therefore, no further check is needed.			
Propose Number of Bolts n=	8	nos.	
$F_{Rd,fp}=\min(F_{v,Rd},F_{b,Rd})(n)$ =	930.816	kN	
As double shear is designed,			
$2F_{Rd,fp}$=	1861.63	kN	
$0.25N_{Ed}$=	1636.39	kN	**OK**
Therefore, the design satisfies the requirements.			

Column to Column Connection Calculations (Peripheral Columns 4th Floor)

Column Section Above	356 x 368 x 202	UC
Column Section Below	356 x 406 x 235	UC

Section Properties (Upper Column)

Grade	355	
h_{uc}	374.6	mm
b_{uc}	374.7	mm
$t_{w,uc}$	16.5	mm
$t_{f,uc}$	27	mm
r_{uc}	15.2	mm
A_{uc}	257	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	381	mm
b_{lc}	394.8	mm
$t_{w,lc}$	18.4	mm
$t_{f,lc}$	30.2	mm
r_{lc}	15.2	mm
A_{lc}	299	cm^2

Splice Details

Flange Cover Plate	2/380 x 40 x 800
Flange Packs	2/380 x 4 x 240
Cleats	4/90 x 90 x 8 Angles x 200 Long
Web Packs	2/85 x 1 x 200
Division Plate	380 x 20 x 380
Bolts	M20 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	2259	kN
N_{Ed} =	3546	kN

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height $h_{fp} >= 2b_{uc} + t_{dp}$=	769.4	mm
Width $b_{fp} >= b_{uc}$	374.7	mm
Vertical Bolt Spacing p_1=	110	mm
$p_1/14$=	7.86	mm
$t_{f,uc}$=	27	mm

Thickness t_{fp}>=max($t_{f,uc}$/2, 10mm, p_1/14)			
t_{fp}>=	13.5	mm	
Therefore, propose h_{fp} =	800	mm	
Therefore, propose b_{fp} =	380	mm	
Therefore, propose t_{fp} =	40	mm	
Flange Packs			
t_{pa}=(h_{lc}-h_{uc})/2=	3.2	mm	
Therefore, propose t_{pa} =	4	mm	
Division Plate			
tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm	
Therefore, propose t_{dp} =	20	mm	
Web Cleats			
Use 90 x 90 x 8 angles to accommodate M20 bolts			
Length >= 0.5h_{uc}=	187.3	mm	
Therefore, propose length of	200	mm	
Web Packs			
t_p = ($t_{w,lc}$-$t_{w,uc}$)/2=	0.95		
Therefore, propose t_p =	1	mm	
Check 2: Minimum Resistance			
Cover Plate			
Requirement: 0.25N_{Ed}<=N_{Rd}			
0.25N_{Ed}=	886.41	kN	
N_{Rd}=2$A_{fp}f_{y,fp}/\gamma_{M0}$=	10792	kN	**OK**
Bolt Group			
Requirement: 0.25N_{Ed}<=2$F_{Rd,fp}$			
a) Shear Resistance per Bolt:			
Diameter of Bolts =	20	mm	
Total thickness of packing t_{pa} =	4	mm	**<d/3**
For t_{pa}>=d/3,β_p=9d/(8d+3t_{pa})=	1.05		
For t_{pa}<d/3, β_p=	1		
Therefore, β_p=	1		
α_v=	0.6	for 8.8 bolts	
Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	245	mm2	
γ_{M2}=	1.25		

$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}=$	94.08	kN	

b) Bearing Resistance per Bolt:
Spacings are desgined based on BSEN1993-1-8 Table 3.3:

Propose $e_1=$	30	mm	800
Propose $e_2=$	110	mm	
Propose $p_1=$	110	mm	
Propose $p_2=$	160	mm	
Bolt Hole Diameter $d_0=$	22	mm	
$f_{u,p}=$	470	MPa	
$\alpha_d=\min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)=$	0.45		
$k_1=\min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)=$	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}=$	341.82	kN	
$\min(F_{v,Rd},F_{b,Rd})=$	94.08	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	8	nos.	
$F_{Rd,fp}=\min(F_{v,Rd},F_{b,Rd})(n)=$	752.64	kN	

As double shear is designed,

$2F_{Rd,fp}=$	1505.28	kN	
$0.25N_{Ed}=$	886.41	kN	**OK**

Therefore, the design satisfies the requirements.

<u>Column to Column Connection Calculations (Peripheral Columns 5th Floor to 8th Floor)</u>

Column Section Above	356 x 368 x 202	UC
Column Section Below	356 x 368 x 202	UC

<u>Section Properties (Upper Column)</u>

Grade	355	
h_{uc}	374.6	mm
b_{uc}	374.7	mm
$t_{w,uc}$	16.5	mm
$t_{f,uc}$	27	mm
r_{uc}	15.2	mm
A_{uc}	257	cm^2

<u>Section Properties (Lower Column)</u>

Grade	355	
h_{lc}	374.6	mm
b_{lc}	374.7	mm
$t_{w,lc}$	16.5	mm

$t_{f,lc}$	27	mm	
r_{lc}	15.2	mm	
A_{lc}	257	cm^2	

Splice Details

Flange Cover Plate	2/380 x 40 x 800
Cleats	4/90 x 90 x 8 Angles x 200 Long
Division Plate	380 x 20 x 380
Bolts	M20 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	2259	kN	
N_{Ed} =	3546	kN	

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height h_{fp} >= $2b_{uc} + t_{dp}$=	769.4	mm	
Width b_{fp} >= b_{uc}	374.7	mm	
Vertical Bolt Spacing p_1=	110	mm	
$p_1/14$=	7.86	mm	
$t_{f,uc}$=	27	mm	
Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)			
t_{fp}>=	13.5	mm	
Therefore, propose h_{fp} =	800	mm	
Therefore, propose b_{fp} =	380	mm	
Therefore, propose t_{fp} =	40	mm	

Division Plate

tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm	
Therefore, propose t_{dp} =	20	mm	

Web Cleats

Use 90 x 90 x 8 angles to accommodate M20 bolts.

Length >= 0.5h_{uc}=	187.3	mm	
Therefore, propose length of	200	mm	

Check 2: Minimum Resistance

Cover Plate

Requirement: $0.25N_{Ed}$<=N_{Rd}

$0.25N_{Ed}$=	886.41	kN	
$N_{Rd}=2A_{fp}f_{y,fp}/\gamma_{M0}$=	10792	kN	**OK**

Bolt Group

Requirement: $0.25N_{Ed}<=2F_{Rd,fp}$

a) Shear Resistance per Bolt:

Diameter of Bolts =	20	mm	
Total thickness of packing t_{pa} =	0	mm	**<d/3**
For $t_{pa}>=d/3, \beta_p=9d/(8d+3t_{pa})=$	1.125		
For $t_{pa}<d/3$, $\beta_p=$	1		
Therefore, $\beta_p=$	1		
$\alpha_v=$	0.6	for 8.8 bolts	
Bolt Ultimate Strength $f_{ub}=$	800	MPa	
Stress Area $A_s=$	245	mm2	
$\gamma_{M2}=$	1.25		
$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}=$	94.08	kN	

b) Bearing Resistance per Bolt:

Spacings are desgined based on BSEN1993-1-8 Table 3.3:

Propose $e_1=$	30	mm	800
Propose $e_2=$	110	mm	
Propose $p_1=$	110	mm	
Propose $p_2=$	160	mm	
Bolt Hole Diameter $d_0=$	22	mm	
$f_{u,p}=$	410	MPa	
$\alpha_d=\min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)=$	0.45		
$k_1=\min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)=$	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}=$	298.18	kN	
$\min(F_{v,Rd},F_{b,Rd})=$	94.08	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	8	nos.	
$F_{Rd,fp}=\min(F_{v,Rd},F_{b,Rd})(n)$ =	752.64	kN	
As double shear is designed,			
$2F_{Rd,fp}=$	1505.28	kN	
$0.25N_{Ed}=$	886.41	kN	**OK**

Therefore, the design satisfies the requirements.

Column to Column Connection Calculations (Cantilever 3rd Floor to 1st Floor)

Column Section Above	356 x 406 x 634	UC
Column Section Below	356 x 406 x 634	UC

Section Properties (Upper Column)

Grade	355	
h_{uc}	474.6	mm
b_{uc}	424	mm
$t_{w,uc}$	47.6	mm
$t_{f,uc}$	77	mm
r_{uc}	15.2	mm
A_{uc}	808	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	474.6	mm
b_{lc}	424	mm
$t_{w,lc}$	47.6	mm
$t_{f,lc}$	77	mm
r_{lc}	15.2	mm
A_{lc}	808	cm^2

Splice Details

Flange Cover Plate	2/430 x 40 x 1150
Cleats	4/90 x 90 x 8 Angles x 250 Long
Division Plate	430 x 10 x 430
Bolts	M27 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	11866	kN
N_{Ed} =	19022	kN

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height h_{fp} >= $2b_{uc} + t_{dp}$=	858	mm
Width b_{fp} >= b_{uc}	424	mm
Vertical Bolt Spacing p_1=	70	mm
$p_1/14$=	5	mm
$t_{f,uc}$=	77	mm

Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)

t_{fp}>=	38.5	mm

Therefore, propose h_{fp} =	1150	mm	
Therefore, propose b_{fp} =	430	mm	
Therefore, propose t_{fp} =	40	mm	
Division Plate			
tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm	
Therefore, propose t_{dp} =	10	mm	
Web Cleats			
Use 90 x 90 x 8 angles to accommodate M27 bolts.			
Length >= 0.5h_{uc}=	237.3	mm	
Therefore, propose length of	250	mm	
Check 2: Minimum Resistance			
Cover Plate			
Requirement: 0.25N_{Ed}<=N_{Rd}			
0.25N_{Ed}=	4755.5	kN	
N_{Rd}=2$A_{fp}f_{y,fp}/\gamma_{M0}$=	12212	kN	**OK**
Bolt Group			
Requirement: 0.25N_{Ed}<=2$F_{Rd,fp}$			
a) Shear Resistance per Bolt:			
Diameter of Bolts =	27	mm	
Total thickness of packing t_{pa} =	0	mm	**<d/3**
For t_{pa}>=d/3,β_p=9d/(8d+3t_{pa})=	1.13		
For t_{pa}<d/3, β_p=	1		
Therefore, β_p=	1		
α_v=	0.6	for 8.8 bolts	
Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	459	mm2	
γ_{M2}=	1.25		
$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}$=	176.26	kN	
b) Bearing Resistance per Bolt:			
Spacings are desgined based on BSEN1993-1-8 Table 3.3:			
Propose e_1=	40	mm	1150
Propose e_2=	140	mm	
Propose p_1=	70	mm	
Propose p_2=	150	mm	
Bolt Hole Diameter d_0=	30	mm	
$f_{u,p}$=	470	MPa	

α_d=min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/$f_{u,p}$,1)=	0.44		
k_1=min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)=	2.5		
$F_{b,Rd}$=$k_1\alpha_d f_u d t_{fp}/\gamma_{M2}$=	451.2	kN	
min($F_{v,Rd}$,$F_{b,Rd}$)=	176.256	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	16	nos.	
$F_{Rd,fp}$=min($F_{v,Rd}$,$F_{b,Rd}$)(n) =	2820.10	kN	

As double shear is designed,

2$F_{Rd,fp}$=	5640.19	kN	
0.25N_{Ed}=	4755.5	kN	**OK**

Therefore, the design satisfies the requirements.

Column to Column Connection Calculations (Cantilever 3rd Floor to 1st Floor)

Column Section Above	356 x 406 x 634	UC
Column Section Below	356 x 406 x 634	UC

Section Properties (Upper Column)

Grade	355	
h_{uc}	474.6	mm
b_{uc}	424	mm
$t_{w,uc}$	47.6	mm
$t_{f,uc}$	77	mm
r_{uc}	15.2	mm
A_{uc}	808	cm^2

Section Properties (Lower Column)

Grade	355	
h_{lc}	474.6	mm
b_{lc}	424	mm
$t_{w,lc}$	47.6	mm
$t_{f,lc}$	77	mm
r_{lc}	15.2	mm
A_{lc}	808	cm^2

Splice Details

Flange Cover Plate	2/430 x 40 x 1150
Cleats	4/90 x 90 x 8 Angles x 250 Long
Division Plate	430 x 10 x 430
Bolts	M27 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	11866	kN	
N_{Ed} =	19022	kN	

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height h_{fp} >= $2b_{uc} + t_{dp}$=	858	mm	
Width b_{fp} >= b_{uc}	424	mm	
Vertical Bolt Spacing p_1=	70	mm	
$p_1/14$=	5	mm	
$t_{f,uc}$=	77	mm	
Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)			
t_{fp}>=	38.5	mm	
Therefore, propose h_{fp} =	1150	mm	
Therefore, propose b_{fp} =	430	mm	
Therefore, propose t_{fp} =	40	mm	

Division Plate

tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm	
Therefore, propose t_{dp} =	10	mm	

Web Cleats

Use 90 x 90 x 8 angles to accommodate M27 bolts.

Length >= 0.5h_{uc}=	237.3	mm	
Therefore, propose length of	250	mm	

Check 2: Minimum Resistance

Cover Plate

Requirement: $0.25N_{Ed}$<=N_{Rd}

$0.25N_{Ed}$=	4755.5	kN	
$N_{Rd}=2A_{fp}f_{y,fp}/\gamma_{M0}$=	12212	kN	**OK**

Bolt Group

Requirement: $0.25N_{Ed}$<=$2F_{Rd,fp}$

a) Shear Resistance per Bolt:

Diameter of Bolts =	27	mm	
Total thickness of packing t_{pa} =	0	mm	**<d/3**
For t_{pa}>=d/3,β_p=9d/(8d+3t_{pa})=	1.13		
For t_{pa}<d/3, β_p=	1		
Therefore, β_p=	1		
α_v=	0.6	for 8.8 bolts	

Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	459	mm2	
γ_{M2}=	1.25		
$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}$=	176.26	kN	
b) Bearing Resistance per Bolt:			
Spacings are desgined based on BSEN1993-1-8 Table 3.3:			
Propose e_1=	40	mm	1150
Propose e_2=	140	mm	
Propose p_1=	70	mm	
Propose p_2=	150	mm	
Bolt Hole Diameter d_0=	30	mm	
$f_{u,p}$=	470	MPa	
$\alpha_d=\min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)$=	0.44		
$k_1=\min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)$=	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}$=	451.2	kN	
$\min(F_{v,Rd},F_{b,Rd})$=	176.256	kN	
As column flange is thicker, it is less critical. Therefore, no further check is needed.			
Propose Number of Bolts n=	16	nos.	
$F_{Rd,fp}=\min(F_{v,Rd},F_{b,Rd})(n)$ =	2820.10	kN	
As double shear is designed,			
$2F_{Rd,fp}$=	5640.19	kN	
$0.25N_{Ed}$=	4755.5	kN	**OK**
Therefore, the design satisfies the requirements.			
<u>Column to Column Connection Calculations (Wind Bracing Bay Columns)</u>			
Column Section Above	356 x 406 x 1202		UC
Column Section Below	356 x 406 x 1202		UC
<u>Section Properties (Upper Column)</u>			
Grade	355		
h_{uc}	580	mm	
b_{uc}	471	mm	
$t_{w,uc}$	95	mm	
$t_{f,uc}$	130	mm	
r_{uc}	15.4	mm	
A_{uc}	1531	cm^2	
<u>Section Properties (Lower Column)</u>			
Grade	355		

h_{lc}	580	mm
b_{lc}	471	mm
$t_{w,lc}$	95	mm
$t_{f,lc}$	130	mm
r_{lc}	15.4	mm
A_{lc}	1531	cm^2

Splice Details

Flange Cover Plate	2/480 x 40 x 1150
Cleats	4/90 x 90 x 8 Angles x 300 Long
Division Plate	480 x 10 x 480
Bolts	M27 8.8
Fittings Material	S355

Design Actions

$N_{Ed,G}$ =	13550	kN
N_{Ed} =	21721	kN

Check 1: Recommended Detailing Practice

External Flange Cover Plates

Height h_{fp} >= $2b_{uc} + t_{dp}$=	952	mm
Width b_{fp} >= b_{uc}	471	mm
Vertical Bolt Spacing p_1=	70	mm
$p_1/14$=	5	mm
$t_{f,uc}$=	130	mm
Thickness t_{fp}>=max($t_{f,uc}/2$, 10mm, $p_1/14$)		
t_{fp}>=	65	mm
Therefore, propose h_{fp} =	1150	mm
Therefore, propose b_{fp} =	480	mm
Therefore, propose t_{fp} =	40	mm

Division Plate

tdp>=[(h_{lc}-2$t_{f,lc}$)-(h_{uc}-2$t_{f,uc}$)]/2=	0	mm
Therefore, propose t_{dp} =	10	mm

Web Cleats

Use 90 x 90 x 8 angles to accommodate M20 bolts.

Length >= 0.5h_{uc}=	290	mm
Therefore, propose length of	300	mm

Check 2: Minimum Resistance

Cover Plate

Requirement: $0.25N_{Ed}$<=N_{Rd}

$0.25N_{Ed}$=	5430.25	kN	
$N_{Rd}=2A_{fp}f_{y,fp}/\gamma_{M0}$=	13632	kN	**OK**

Bolt Group

Requirement: $0.25N_{Ed}<=2F_{Rd,fp}$

a) Shear Resistance per Bolt:

Diameter of Bolts =	27	mm	
Total thickness of packing t_{pa} =	0	mm	**<d/3**
For $t_{pa}>=d/3,\beta_p=9d/(8d+3t_{pa})$=	1.125		
For $t_{pa}<d/3$, β_p=	1		
Therefore, β_p=	1		
α_v=	0.6	for 8.8 bolts	
Bolt Ultimate Strength f_{ub}=	800	MPa	
Stress Area A_s=	459	mm2	
γ_{M2}=	1.25		
$F_{v,Rd}=\beta_p\alpha_v f_{ub}A_s/\gamma_{M2}$=	176.26	kN	

b) Bearing Resistance per Bolt:

Spacings are desgined based on BSEN1993-1-8 Table 3.3:

Propose e_1=	40	mm	1150
Propose e_2=	160	mm	
Propose p_1=	70	mm	
Propose p_2=	150	mm	
Bolt Hole Diameter d_0=	30	mm	
$f_{u,p}$=	470	MPa	
$\alpha_d=\min(e_1/(3d_0),p_1/(3d_0)-0.25,f_{ub}/f_{u,p},1)$=	0.44		
$k_1=\min(2.8e_2/d_0-1.7,1.4p_2/d_0-1.7,2.5)$=	2.5		
$F_{b,Rd}=k_1\alpha_d f_u d t_{fp}/\gamma_{M2}$=	451.2	kN	
$\min(F_{v,Rd},F_{b,Rd})$=	176.26	kN	

As column flange is thicker, it is less critical. Therefore, no further check is needed.

Propose Number of Bolts n=	16	nos.	
$F_{Rd,fp}=\min(F_{v,Rd},F_{b,Rd})(n)$ =	2820.10	kN	

As double shear is designed,

$2F_{Rd,fp}$=	5640.19	kN	
$0.25N_{Ed}$=	5430.25	kN	**OK**

Therefore, the design satisfies the requirements.

Appendix A17

Calculation Spreadsheets for Wind Bracing Connections

Axial force in the diagonal members:

North-south: $N_{Ed} = 2265.63\ kN$

East-west: $N_{Ed} = 2040.56\ kN$

For material efficiency, each bracing system will have different grade of bolts.

North-south

For connections between end plate and gusset plate, 10 No non-preloaded Class 10.9 M30 diameter bolts in 32 mm diameter clearance holes are chosen. The properties of the bolts are:

Cross section area $A = A_s = 561\ mm^2$

Clearance hole diameter $d_0 = 32\ mm$

Yield strength $f_{yb} = 900\ N/mm$^2

Ultimate tensile strength $f_{ub} = 1000\ N/mm^2$

East-west

For connections between end plate and gusset plate, 10 No non-preloaded Class 8.8 M30 diameter bolts in 32 mm diameter clearance holes are chosen. The properties of the bolts are:

Cross section area $A = A_s = 561\ mm^2$

Clearance hole diameter $d_0 = 32\ mm$

Yield strength $f_{yb} = 640\ N/mm$^2

Ultimate tensile strength $f_{ub} = 800\ N/mm^2$

End plate

End plate section: Grade S355 $978 \times 530 \times 21\ mm$ (for all bracing)

Thickness $t = 21\ mm$ — BS EN 1993-1-1 NA 2.4

Length $L = 530\ mm$

Yield strength $f_y = 355\ N/mm^2$

Because $3 \leq t \leq 100\ mm$, Ultimate tensile strength $f_u = 470\ N/mm^2$ — BS EN 10025-2 Table 7

Position of holes:

$$e_1 = 40\ mm \quad e_2 = 165\ mm \quad p_1 = 80\ mm \quad p_2 = 200\ mm$$

Limitation of the spacings:

Minimum $e_1 = 1.2d_0 = 1.2 \times 32 = 38.4\ mm < 40\ mm$

Minimum $e_2 = 1.2d_0 = 1.2 \times 32 = 38.4\ mm < 170\ mm$

Minimum $p_1 = 2.2d_0 = 2.2 \times 32 = 70.4\ mm < 80\ mm$

Minimum $p_1 = 2.4d_0 = 2.4 \times 32 = 76.8\ mm < 200\ mm$

Maximum e_1 & e_2 : Larger of $8t = 8 \times 21 = 168\ mm\ or\ 125\ mm > 40\ mm$ & $165\ mm$

Maximum p_1 & p_2 : Smaller of $14t = 14 \times 21 = 294\ mm\ or\ 200\ mm > 80\ mm$ & $200mm$

Shear resistance of bolts

North-South

Shear resistance of a single bolt: $F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} = \frac{0.6 \times 1000 \times 561}{1.25} \times 10^{-3} = 269.28\ kN$ — BS EN 1993-1-8 Table 3.4

Where for grade 8.8 bolts, $\alpha_v = 0.6$

Minimum No of bolts required: $\frac{N_{Ed}}{F_{v,Rd}} = \frac{2265.63}{269.28} = 8.41\ bolts$

East-west

Shear resistance of a single bolt: $F_{v,Rd} = \frac{\alpha_v f_{ub} A}{\gamma_{M2}} = \frac{0.6 \times 800 \times 561}{1.25} \times 10^{-3} = 215.42\ kN$

Where for grade 8.8 bolts, $\alpha_v = 0.6$

Minimum No of bolts required: $\frac{N_{Ed}}{F_{v,Rd}} = \frac{2040.56}{215.42} = 9.47\ bolts$

Therefore, for each bracing frame, 10 bolts are provided in a single shear.

Bearing resistance of bolts

With grade 355 end plate mentioned before, $f_y = 355\ N/mm^2$ $f_u = 470\ N/mm^2$

BS EN 1993-1-8 Table 3.4

$$F_{b,Rd} = \frac{k_1 \alpha_b f_u d t}{\gamma_{M2}}$$

Where α_b is the least value of α_d, $\frac{f_{ub}}{f_{u,p}}$ and 1.0

For end bolts $\alpha_d = \frac{e_1}{3d_0} = \frac{40}{3 \times 32} = 0.42$

For inner bolts $\alpha_d = \frac{p_1}{3d_0} - 0.25 = \frac{80}{3 \times 32} - 0.25 = 0.58$

$$\frac{f_{ub}}{f_{u,p}} = \frac{1000}{470} = 2.13$$

For end bolts $\alpha_b = \min(0.42, 2.13, 1.0) = 0.42$

For inner bolts $\alpha_b = \min(0.58, 2.13, 1.0) = 0.58$

Therefore, for all bolts: $\alpha_b = 0.42$

For edge bolts k_1 is the smaller of $2.8\frac{e_2}{d_0} - 1.7$ or 2.5

$$2.8\frac{e_2}{d_0} - 1.7 = 2.8 \times \frac{165}{32} - 1.7 = 12.74$$

$k_1 = \min(12.74, 2.5) = 2.5$

For inner bolts k_1 is the smaller of $1.4\frac{p_2}{d_0} - 1.7\ or\ 2.5$

$$1.4\frac{p_2}{d_0} - 1.7 = 1.4 \times \frac{200}{32} - 1.7 = 7.05$$

$$k_1 = \min(7.05, 2.5) = 2.5$$

Therefore, the least bearing resistance of a single bolt is:

$$F_{b,Rd} = \frac{2.5 \times 0.42 \times 470 \times 30 \times 21}{1.25} \times 10^{-3} = 246.75\ kN$$

Conservatively, resistance of all 10 bolts in bearing is: $10 \times 246.75 = 2467.5\ kN$

Group of fasteners

Comparing the shear resistance and bearing resistance of one bolt:

BS EN 1993-1-8 3.7

North-south: $F_{v,Rd} = 269.28\ kN > F_{b,Rd} = 246.75\ kN$

The design resistance of group should take the less one: $10 \times 246.75 = 2467.5\ kN$

East-west: $F_{v,Rd} = 215.42\ kN < F_{b,Rd} = 246.75\ kN$

The design resistance of group should take the less one: $10 \times 215.42 = 2154.2\ kN$

Design of fillet weld

BS EN 1993-1-8 4.5

To check the shear resistance of fillet weld, simplified method of 4.5.3.3 in Eurocode is used:

BS EN 1993-1-8 4.5.3.3(3)

Design shear resistance: $f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_{M2}}$

BS EN 1993-1-8 Table 4.1

Where $\beta_w = 0.9$ for S355 steel.

$$f_{vw,d} = \frac{f_u/\sqrt{3}}{\beta_w \gamma_{M2}} = \frac{470/\sqrt{3}}{0.9 \times 1.25} = 241.20\ N/mm^2$$

North-south

Throat thickness of weld: $a = 0.7 \times leglength = 0.7 \times 6.5 = 4.55\ mm$

Design resistance of weld per unit length:

$$F_{vw,d} = f_{vw,d} \times a = 241.20 \times 4.55 = 1097.48\ N/mm$$

For four welds, each with an effective length of $l_{eff} = 530 - (2 \times 6.5) = 517\ mm$

Shear resistance: $4F_{w,Rd}l_{eff} = 4 \times 1097.48 \times 517 \times 10^{-3} = 2269.59\ kN > 2265.63\ kN$

OK

East-west

Throat thickness of weld: $a = 0.7 \times leglength = 0.7 \times 6 = 4.2\ mm$

Design resistance of weld per unit length:

$$F_{vw,d} = f_{vw,d} \times a = 241.20 \times 4.2 = 1013.06\ N/mm$$

For four welds, each with an effective length of $l_{eff} = 530 - (2 \times 6) = 518\ mm$

Shear resistance: $4F_{w,Rd}l_{eff} = 4 \times 1013.06 \times 518 \times 10^{-3} = 2099.06\ kN > 2040.56\ kN$

OK

Local resistance of CHS wall

BS EN 1993-1-1 6.2.6 (3)

For CHS 508×14.2, shear area: $A_v = 0.9dt$

Where d is the depth of the rectangular area and t is the thickness of the hollow section.

Total shear area$= 4 \times 0.9 \times 530 \times 14.2 = 27093.6\ mm^2$

Shear resistance: $\frac{A_v(f_y/\sqrt{3})}{\gamma_{M0}} = \frac{27093.6 \times 355/\sqrt{3}}{1.0 \times 10^3} = 5553.09\ kN$

For north-south bracing frames: $5553.09\ kN > 2265.63\ kN$

For east-west bracing frame: $5553.09\ kN > 2040.56\ kN$

OK

Tensile resistance of end plate

BS EN 1993-1-8 3.10.2

There are two modes of failure including cross-sectional failure and block tearing failure.

Cross-sectional failure

BS EN 1993-1-1 6.2.3(1)

Basic requirement: $\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0$

The design tensile resistance of a cross-section with holes is smaller of $N_{pl,Rd}$ and $N_{u,Rd}$

$$N_{pl,Rd} = \frac{A \times fy}{\gamma_{M0}}$$

Where A is the gross cross-sectional area: $A = 21 \times 530 = 11130\ mm^2$

BS EN 1993-1-1 Eqn. 6.6

$$N_{pl,Rd} = \frac{A \times fy}{\gamma_{M0}} = \frac{11130 \times 355}{1.0} \times 10^{-3} = 3951.15\ kN$$

For north-south bracing frames: $3951.15\ kN > 2265.63\ kN$

For east-west bracing frame: $3951.15\ kN > 2040.56\ kN$

OK

$$N_{u,Rd} = \frac{0.9 \times A_{net} \times f_u}{\gamma_{M2}}$$

Where $A_{net} = 11130 - (2 \times 32 \times 21) = 9786\ mm^2$

BS EN 1993-1-1 6.2.2.2

$$N_{u,Rd} = \frac{0.9 \times A_{net} \times f_u}{\gamma_{M2}} = \frac{0.9 \times 9786 \times 470}{1.25} \times 10^{-3} = 3311.58\ kN$$

For north-south bracing frames: $3311.58\ kN > 2265.63\ kN$

For east-west bracing frame: $3311.58\ kN > 2040.56\ kN$

OK

Block tearing failure

The bolts group of the connection is symmetrical about the concentric loading so the design block tearing resistance is:

BS EN 1993-1-8 3.10.2 (2)

$$V_{eff,1,Rd} = \frac{f_u A_{nt}}{\gamma_{M2}} + \left(1/\sqrt{3}\right)\frac{(f_u A_{nv})}{\gamma_{M0}}$$

Where A_{nt} is the net area subject to tension: $A_{nt} = \min((p_2 - d_0)\, t_p, 2(e_2 - 0.5d_0)t_p)$

$$(p_2 - d_0)t_p = (200 - 32) \times 21 = 3528\, mm^2$$

$$2(e_2 - 0.5 \times d_0)t_p = 2 \times (165 - 0.5 \times 32) \times 21 = 6528\, mm^2$$

$$A_{nt} = \min(3528, 6528) = 3528\, mm^2$$

and A_{nv} is the net area subject to shear:

$$A_{nv} = 2(3p_1 + e_1 - 2.5d_0)t_w = 2 \times (3 \times 80 + 40 - 2.5 \times 32) \times 21 = 8400\, mm^2$$

$$V_{eff,1,Rd} = \frac{f_u A_{nt}}{\gamma_{M2}} + \left(1/\sqrt{3}\right)\frac{(f_u A_{nv})}{\gamma_{M0}} = \frac{470 \times 3528}{1.25 \times 10^3} + \left(1/\sqrt{3}\right)\frac{470 \times 8400}{1 \times 10^3} = 3048.19\, kN$$

For north-south bracing frames: $3048.19\, kN > 2265.63\, kN$

For east-west bracing frame: $3048.19\, kN > 2040.56\, kN$

OK

In conclusion, with 10 bolts of different grades used on two types of bracing bay. The connections including bolts, end plate and fillet weld can bear the tensile force from the diagonal member.

Appendix A18

Cantilever Truss System Connections Design

Design of a truss node with gusset – PROPOSAL 1

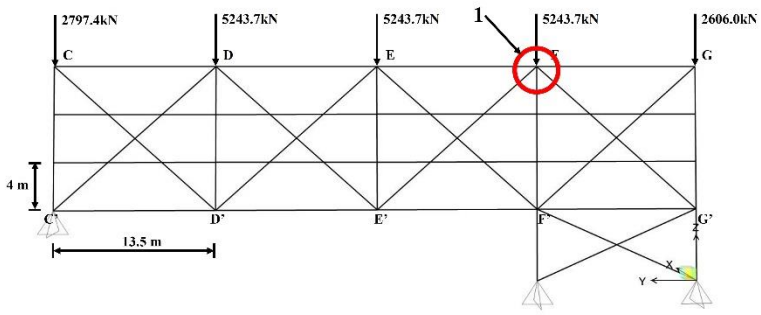
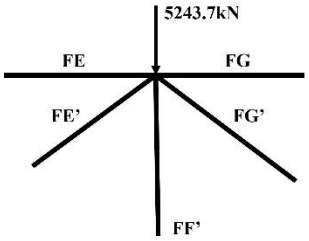


Figure A18.1 Location of the KT joint

Table A18.1 KT joint - Internal forces in the truss members - PROPOSAL 1 - WEST VIEW

	Member	N (kN)
	Chord FE	-1282.04
	Chord FG	1512.48
	Diagonal FE'	3622.52
	Diagonal FG'	155.33
	Diagonal FF'	-8840.08

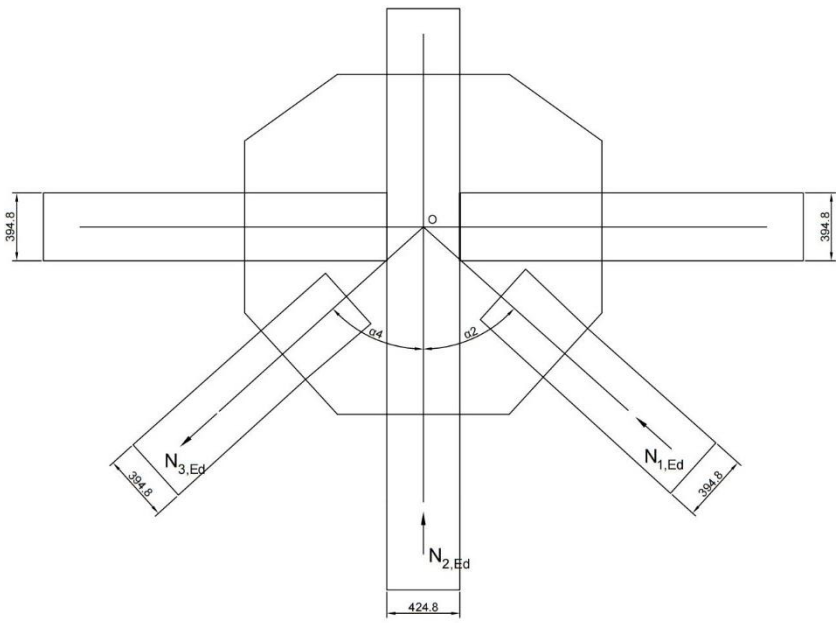


Figure A18.2 Gusset plate to web chord welded connection

Internal force in the truss members

All axial forces are applied in the gusset plate XOZ plane:

Compression axial force at an I to normal OY of $\alpha_1 = 90°$

$N_{1,Ed} = -9692.74$ kN

Compression axial force at an I to normal OY of $\alpha_2 = 48.37°$

$N_{1,Ed} = -382.46$ kN

Tension axial force at an I to normal OY of $\alpha_3 = 0°$

$N_{1,Ed} = 900.69$ kN

Compression axial force at an I to normal OY of $\alpha_4 = -48.37°$

$N_{1,Ed} = -10826.52$ kN

Compression axial force at an I to normal OY of $\alpha_5 = -90°$

$N_{1,Ed} = -1896.67$ kN

Stresses in the gusset cross-section in front of welds

<u>Normal stress</u>

Assuming a uniform distribution of the load in the section, the normal stress is:

$$\sigma_{g,max} = \frac{N_{g,Ed}}{A_g} + \frac{M_{g,Ed}}{\frac{I_g}{v}}$$

$$\tau_g = \frac{V_{g,Ed}}{A_g}$$

Then:

$$\tau_g = 0 \text{ N/mm}^2$$

One usually checks the combination of axial and shear stresses in the gusset plate section using the Von Mises criterion.

Design resistance of the fillet weld

<u>Directional method</u>

On the throat section of the weld, the force per unit length are:

$$a\sigma_\perp = \frac{\sigma_{g,max} e_g}{n_a} \sin\left(\frac{\alpha_a}{2}\right) = -138.43 \text{N/mm.mm}$$

$$a\tau_\perp = \frac{\sigma_{g,max} e_g}{n_a} \cos\left(\frac{\alpha_a}{2}\right) = -138.43 \text{N/mm} \cdot \text{mm}$$

$$a\tau_{//} = \frac{\tau_g e_g}{n_a} = -0 \text{ N/mm.mm}$$

The design resistance of the fillet weld will be sufficient if the following conditions are both fulfilled:

$$\sigma_{\mathrm{w}} = \left[\sigma_{\perp}^2 + 3\left(\tau_{\perp}^2 + \tau_{//}^2\right)\right]^{0,5} \leq f_{\mathrm{u}}/(\beta_{\mathrm{w}}\gamma_{\mathrm{M2}})$$

$$\sigma_{\perp} \leq 0{,}9 f_{\mathrm{u}}/\gamma_{\mathrm{M2}}$$

$$a_{1,min} = a\sigma_{\mathrm{w}} / \left[\frac{f_{\mathrm{u}}}{(\beta_{\mathrm{w}}\gamma_{\mathrm{M2}})}\right] = 0.54\mathrm{mm}$$

$$a_{2,min} = a\sigma_{\perp} / \left(\frac{0{,}9 f_{\mathrm{u}}}{\gamma_{\mathrm{M2}}}\right) = 0.38\mathrm{mm}$$

$$a_{min} = max\left(a_{1,min}; a_{2,min}\right) = 0.54\mathrm{mm}$$

The following requirements must be satisfied:

$a \geq 3\mathrm{mm}$

$l_{\mathrm{eff}} \geq max(30\mathrm{mm}; 6a)$ with $l_{\mathrm{eff}} = L_{\mathrm{w}} - 2a$

An effective throat thickness of 4 mm is then sufficient.

7.1. I to gusset bolted connection

All axial forces are applied in the gusset plate XOZ plane:

Compression axial force at an I to normal OY of $\alpha_1 = 90°$

$N_{1,Ed} = -9692.74$ kN

Compression axial force at an I to normal OY of $\alpha_2 = 48.37°$

$N_{1,Ed} = -382.46$ kN

Tension axial force at an I to normal OY of $\alpha_3 = 0°$

$N_{1,Ed} = 900.69$ kN

Compression axial force at an I to normal OY of $\alpha_4 = -48.37°$

$N_{1,Ed} = -10826.52$ kN

Compression axial force at an I to normal OY of $\alpha_5 = -90°$

$N_{1,Ed} = -1896.67$ kN

Global checking of gross cross-section of the gusset plate

The gross cross-sections of the gusset plates to check are located on the figure as follows:

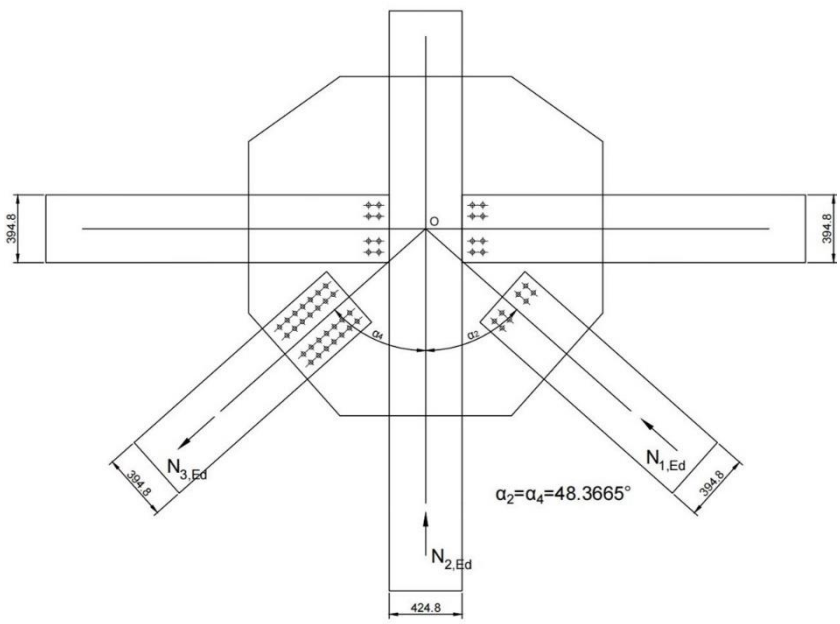


Figure A18.3 Location of the gross cross-sections of the gusset plate

Checking of gross cross-section 1

With A_{g1} cross-sectional area 1 $A_{gl} = H_g t_g = 5200\text{mm}^2$

Shera resistance

$V_{g1,Ed}$

$= max\left(N_{1,\mathrm{Ed}} \cos \alpha_1 ; N_{2,\mathrm{Ed}} \cos \alpha_2 ; N_{3,\mathrm{Ed}} \cos \alpha_3 ; N_{4,\mathrm{Ed}} \cos \alpha_4 ; N_{5,\mathrm{Ed}} \cos \alpha_5\right)$

$= 899.5\mathrm{kN}$

$V_{\mathrm{g1,pl,Rd}} = A_{\mathrm{gl}} f_{\mathrm{y}} / \left(\gamma_{\mathrm{M0}} \sqrt{3}\right) = 1065.79\mathrm{kN}$

$V_{\mathrm{g1,Ed}} < V_{\mathrm{gl,pl,Rd}} \Rightarrow \mathrm{OK}$

Axial force resistance

$N_{\mathrm{gl,Ed}} = \sum_{\mathrm{i}=1}^{5} N_{\mathrm{i,Ed}} \sin\left(\alpha_{\mathrm{i}}\right) = 10.09\mathrm{kN}$

$N_{\mathrm{g1,pl,Rd}} = A_{\mathrm{g1}} f_{\mathrm{y}} / \gamma_{\mathrm{M0}} = 1846.00\mathrm{kN}$

$N_{\mathrm{g1,Ed}} < N_{\mathrm{g1,pl,Rd}} \Rightarrow \mathrm{OK}$

Checking of gross cross-section 2

With A_{g2} cross-sectional area 2 $A_{\mathrm{g2}} = H_{\mathrm{g}} t_{\mathrm{g}} = 12000\mathrm{mm}^2$

Shera resistance

$$V_{\mathrm{g2,Ed}} = \sum_{\mathrm{i}=1}^{5} N_{\mathrm{i,Ed}} \sin\left(\alpha_{\mathrm{i}}\right) = 10.09\mathrm{kN}$$

$V_{\mathrm{g2,pl,Rd}} = A_{\mathrm{g2}} f_{\mathrm{y}} / \left(\gamma_{\mathrm{M0}} \sqrt{3}\right) = 2459.51\mathrm{kN}$

$V_{\mathrm{g2,Ed}} < V_{\mathrm{g2,pl,Rd}} \Rightarrow \mathrm{OK}$

Axial force resistance

$N_{\mathrm{gl,Ed}} = \sum_{\mathrm{i}=1}^{5} N_{\mathrm{i,Ed}} \sin(\alpha_{\mathrm{i}}) = 10.09\mathrm{kN}$

$N_{\mathrm{g1,pl,Rd}} = A_{\mathrm{g1}} f_{\mathrm{y}} / \gamma_{\mathrm{M0}} = 1846.00\mathrm{kN}$

$N_{\mathrm{g1,Ed}} < N_{\mathrm{g1,pl,Rd}} \Rightarrow \mathrm{OK}$

Connection N3

The shear connection in compression is designed as Category C.

The sizes of the components and the positioning of the holes are shown on the figure as follows:

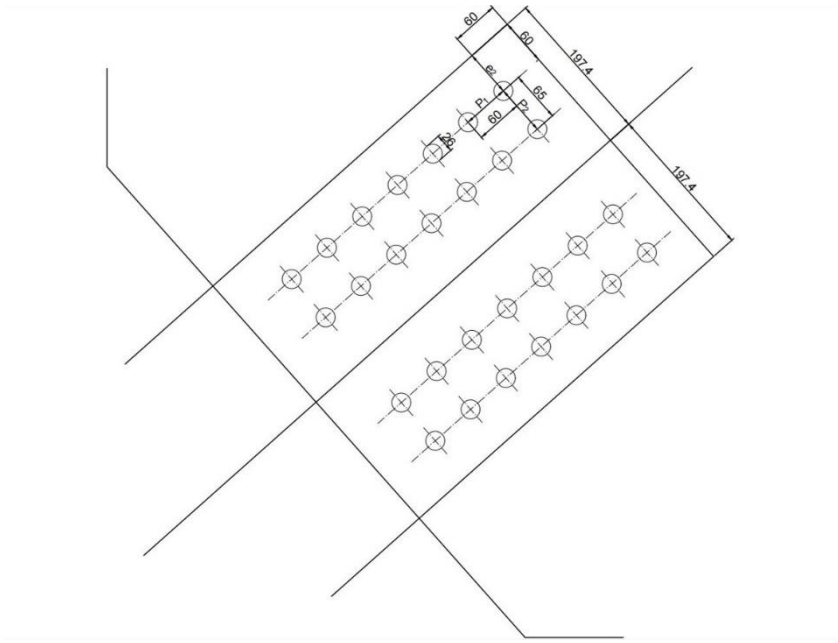


Figure A18.4 Connection sizes and positioning

$M_{3,\mathrm{N,Ed}}$ Bending moment resulting from the eccentricity, $M_{3,\mathrm{N,Ed}} = e_{\mathrm{N3}} N_{3,\mathrm{Ed}}$

For the gusset:

$N_{3,\mathrm{g,Ed}} = 899.5kN$

$e_{\mathrm{N3}} = 65mm$

$M_{3,\mathrm{N,Ed}} = e_{\mathrm{N3}} N_{3,\mathrm{Ed}} = 58.47kNm$

For each flange:

$N_{3,\mathrm{g,Ed}} = 449.75kN$

$M_{3,\mathrm{N,Ed}} = e_{\mathrm{N3}} N_{3,\mathrm{Ed}} = 29.23 kNm$

<u>Checking of flange resistance of gross cross-section longitudinal stress</u>

Assuming a uniform distribution of the load in the section, the longitudinal stress is:

$$\sigma_{\mathrm{i}} = \frac{N_{3,\mathrm{a,Ed}}}{A_{3,\mathrm{a}}} + \frac{M_{3,\mathrm{a,Ed}}}{I_{3,\mathrm{a}}/\boldsymbol{v}}$$

Then the normal stresses is:

$\sigma = 17.44 \mathrm{N/mm^2}$ (tension)

<u>Cross-section classification</u>

$Yield\ strenth\ f_y = 355\ MPa, Young's\ modulus\ E = 210\ GPa$

$$\varepsilon = \sqrt{\frac{235}{f_y}} = \sqrt{\frac{235}{355}} \approx 0.814$$

Flange – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 5.73$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 5.73$

$\therefore\ Flange\ is\ Class\ 1$

Web – internal part in compression (EC3 Table 5.2, sheet 1)

$c/t = 15.8$

$Limit\ for\ Class\ 1\ flange = 33\varepsilon = 26.85 > 15.8$

$\therefore\ Web\ is\ Class\ 1$

Overall cross-section classification is therefore Class 1 (under pure compression)

<u>Resistance of net cross-section</u>

$$\frac{A_{\mathrm{t,net}} 0{,}9 f_{\mathrm{u}}}{\gamma_{\mathrm{M2}}} \geq \frac{A_{\mathrm{t}} f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}}$$

Here, the holes are in the tension zone.

The following criterion should be fulfilled:

$N_{3,\mathrm{a,Ed}} \leq N_{3,\mathrm{a,c,Rd}} = \frac{A_{3,\mathrm{a}} f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}}$

With: $A_{3,\mathrm{a}} = 29900\mathrm{mm}^2$:

$N_{3,\mathrm{a,Ed}} = 899.5 < N_{3,\mathrm{a,c,Rd}} = 10614.5\mathrm{kN}$

<u>Buckling resistance</u>

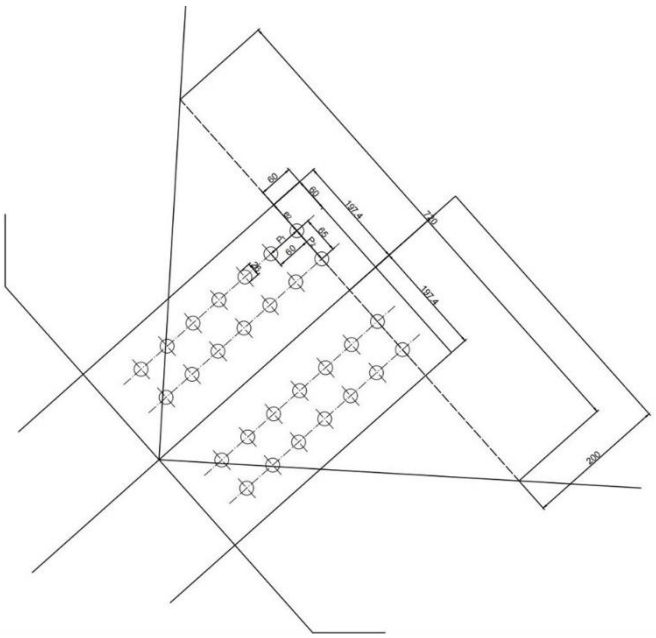


Figure A18.5 Diffusion by 45° of the axial force

The following criteria must be satisfied:

$$\sigma_{\mathrm{x,Ed}} = \frac{N_{3,\mathrm{g,Ed}}}{A_{3,\mathrm{g}}} \pm \frac{M_{3,\mathrm{g,Ed}}}{I_{3,\mathrm{g}}/v} \leq \frac{f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}}$$

With: $A_{3,\mathrm{g}} = 240 \times t_{\mathrm{g}} = 7200\mathrm{mm}^2$

$I_{3,\mathrm{g}} = t_g \times 240^3/12 = 34560000\mathrm{mm}^4$

$M_{3,\mathrm{g,Ed}} = 0$

Then: $\sigma_{\mathrm{x,Ed}} = 30.08 < \frac{f_{\mathrm{y}}}{\gamma_{\mathrm{M0}}} = 355\mathrm{N/mm}^2$

Buckling resistance

The gusset is made similar to an embedded column of characteristics:

Area	$A_{3,\mathrm{g}} = 7200\mathrm{mm}^2$
Height	$h_{\mathrm{c}} = 115\mathrm{mm}$
Second moment of area	$I_{\mathrm{c},zz} = 34560000\mathrm{mm}^4$

We should satisfy:

$$N_{3,g,Ed} < N_{3,g,b,\mathrm{Rd}} = \frac{\chi A_{3,g} f_y}{\gamma_{M1}}$$

Where χ is the reduction factor for the relevant buckling curve

With a buckling length of $2h_c$, the slenderness is given by:

$$\bar{\lambda} = \sqrt{\frac{4h_{\mathrm{c}}^2 A_{\mathrm{c}} f_{\mathrm{y}}}{\pi^2 E I_{\mathrm{c}}}} = 1.37$$

The buckling curve to use is curve c and the imperfection is:

$\alpha = 0.49$

$\Phi = 0.5\left[1 + \alpha(\bar{\lambda} - 0.2) + \bar{\lambda}^2\right] = 1.73$

$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} = 0.36$

Then: $N_{3,g,Ed} = 899.5 < N_{3,g,b,Rd} = 917.74\text{kN}$

Bolts arrangement

The bolts are fully threaded, non-preloaded, M24 10.9, as generally used in the UK

Spacing requirements

$e_1 \geq 2d_0 \rightarrow 60\text{mm} \geq (2 \times 26) = 52\text{mm}$

$e_2 \geq 2d_0 \rightarrow 60\text{mm} \geq (2 \times 26) = 52\text{mm}$

$p_1 \geq 2.2d_0 \rightarrow 60\text{mm} \geq (2.2 \times 26) = 57.2\text{mm}$

$p_2 \geq 2.4d_0 \rightarrow 65\text{mm} \geq (2.4 \times 26) = 62.4\text{mm}$

Cross-section of flange

Because the two I-members are not the same height, we need to add spacers to the Flange with a thickness of

$$t_s = \frac{t_{f,1} - t_{f,2}}{2} = 27.8mm$$

Bolt shear resistance

Assuming the shear plane passes through the threaded portion of the bolt, the shear resistance $F_{v,Rd}$ of a single bolt is given by:

$F_{v,Rd} = \frac{0.5 \cdot f_{ub} \cdot A_s}{\gamma_{M2}}$ for bolt class 10.9

$$F_{v,Rd} = 0.8 \times \frac{0.5 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = 0.8 \times \frac{0.5 \cdot 1000 \cdot 353}{1.25} \cdot 10^{-3} = 112.96\text{kN}$$

Bolt tension resistance

The design tension resistance of a bolt is given by:

$$F_{t,Rd} = \frac{0.9 \cdot f_{ub} \cdot A_s}{\gamma_{M2}}$$

$$F_{t,Rd} = \frac{0.9 \cdot f_{ub} \cdot A_s}{\gamma_{M2}} = \frac{0.9 \cdot 1000 \cdot 353}{1.25} = 254.16\ kN$$

Minimum bolts numbers

$$n = \frac{F_{v,Ed}}{F_{v,Rd}} = \frac{3623.66}{112.96} = 32.08$$

Bearing resistance

The design bearing resistance of a bolt is given by:

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}}$$

Where: α_b is the smallest of $\alpha_d, f_{ub}/f_u\ and\ 1$

In the direction of load transfer:

For end bolts: $\alpha_d = \frac{e_1}{3d_0}\ and\ for\ inner\ bolts: \alpha_d = \frac{p_1}{3d_0} - \frac{1}{4}$

$\alpha_b = min\left(\frac{60}{3\times24}; \frac{1000}{510}; 1.0\right) = 0.77$ for end bolts

$\alpha_b = min\left(\frac{60}{3\times24} - 0.25; \frac{1000}{510}; 1.0\right) = 0.52$ for inner bolts

Perpendicular to the direction of load transfer:

For edge bolts: $k_1\ is\ the\ smallest\ of\ 2.8\frac{e_2}{d_0} - 1.7\ or\ 2.5$,

For inner bolts: $k_1\ is\ the\ smallest\ of\ 1.4\frac{p_2}{d_0} - 1.7\ or\ 2.5$ (where d is bolt diameter and d_0 is hole diameter)

And t is the thickness of the plate, which equals to 15 mm.

$k_1 = min\left(2.8\frac{60}{24} - 1.7; 2.5\right) = 2.5$ for edge bolts

$k_1 = min\left(1.4\frac{65}{24} - 1.7; 2.5\right) = 1.8$ for inner bolts

Therefore, for the edge bolts,

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}} = \frac{2.5 \cdot 0.77 \cdot 510 \cdot 24 \cdot 30}{1.25} = 564.92\ kN$$

For the inner bolts,

$$F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_u \cdot d \cdot t}{\gamma_{M2}} = \frac{1.8 \cdot 0.52 \cdot 510 \cdot 24 \cdot 30}{1.25} = 274.55\ \text{kN}$$

The bearing resistance of the bolts $= 2 \times 564.92 + 34 \times 274.55 = 10464.64\ kN$

Group of Fasteners

Resistance of the group $= 36 \times 112.96 = 4066.56\ kN \geq V_{Ed} = 3623.96\text{kN}$ OK

Plate strength

Considering 2 critical sections:

Net section:

$$V_{Rd,n} = 2 \cdot \frac{A_{v,net} \cdot f_u}{\sqrt{3} \cdot \gamma_{M2}} = 2 \cdot \frac{11520 \cdot 510}{\sqrt{3} \cdot 1.25} \cdot 10^{-3} = 5427.28\text{kN} \geq V_{Ed} = 3623.96\text{kN}$$

Gross section:

$$V_{Rd,g} = 2 \cdot \frac{A_v \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}} = 2 \cdot \frac{18000 \cdot 355}{\sqrt{3} \cdot 1.0} \cdot 10^{-3} = 7378.54\text{kN} \geq V_{Ed} = 3623.96\text{kN}$$

Block shear:

$$V_{Rd,b} = 2 \cdot \left[\frac{0.5 \cdot A_{nt} \cdot f_u}{\gamma_{M2}} + \frac{A_{North\ View} \cdot f_y}{\sqrt{3} \cdot \gamma_{M0}}\right]$$

$$= 2 \cdot \left[\frac{0.5 \cdot (45 - 9) \cdot 9 \cdot 510}{1.25} + \frac{(220 - 35 + 9 - 4 \cdot 18) \cdot 9 \cdot 355}{\sqrt{3} \cdot 1.0}\right] \cdot 10^{-3}$$

$$= 7755.02\ \text{kN} \geq V_{Ed} = 3623.96\text{kN OK}$$

Weld Strength

$$\frac{F}{a \cdot l} \leq f_{vw} \text{ with } f_{vw} = \frac{f_u}{\sqrt{3 \cdot \beta_w \cdot \gamma_{M2}}}$$

Where: l = 800 mm, a = 20 mm, $\beta_w = 0.9$ for S355

Then, $f_{vw} = \frac{510}{\sqrt{3 \cdot 0.9 \cdot 1.25}} = 277.61\ kN$

$$F = f_{vw} a l = 277.61 \times 800 \times 20 = 4441.74\ kN \geq V_{Ed} = 3623.96\text{kN OK}$$

7.2. Spreadsheet of bolts numbers – PROPOSAL 1

The calculation for the other nodes is not significantly different from that described in the previous section, and the number of bolts required for each node is given below:

Table A18.1 Number of bolts - PROPOSAL 1 – WEST VIEW

Member	Axial Force kN	F_v,Ed kN	Number of bolts n
1	-5146.31	2573.155	24
2	-5771.52	2885.76	28
3	-6397.92	3198.96	32
4	2069.26	1034.63	12
5	899.14	449.57	4
6	-269.32	134.66	4
7	1406.95	703.475	8

8	237.07	118.535	4
9	-931.85	465.925	8
10	-7648.5	3824.25	36
11	-8839.38	4419.69	40
12	-10022.33	5011.165	48
13	-1972.85	986.425	12
14	2457.68	1228.84	12
15	-3046.74	1523.37	16
16	-17495.33	8747.665	80
17	-4230.13	2115.065	20
18	-1896.67	948.335	12
19	-50.95	25.475	4
20	45.53	22.765	4
21	569.69	284.845	4
22	-9692.74	4846.37	44
23	-86.11	43.055	4
24	78.73	39.365	4
25	2716.54	1358.27	16
26	-1282.04	641.02	8
27	-72.61	36.305	4
28	68.9	34.45	4
29	-274.97	137.485	4
30	1512.48	756.24	8
31	-3.62	1.81	4
32	-0.46	0.23	4
33	-874.93	437.465	4
34	2564.38	1282.19	12
35	-10824.33	5412.165	48
36	-381.78	190.89	4
37	-321.4	160.7	4
38	-11577.47	5788.735	52
39	3623.7	1811.85	20
40	-154.68	77.34	4
41	-2016.55	1008.275	12
42	-7114.42	3557.21	32
43	1087.23	543.615	8

The number of bolts required for each node is given below：

Table A18.2 Number of bolts - PROPOSAL 1 - NORTH VIEW

Member	Axial Force kN	F_v,Ed kN	Number of bolts n
1	-9415.12	4707.56	44
2	-17951.75	8975.875	80
3	-32002.77	16001.385	144
4	-38713.17	19356.585	172
5	-32.14	16.07	4
6	4055.33	2027.665	20
7	7336.06	3668.03	36
8	-7.41	3.705	4
9	-5198.87	2599.435	24
10	-4176.85	2088.425	20
11	-7.41	3.705	4
12	-5198.87	2599.435	24
13	-4176.85	2088.425	20
14	-8759.28	4379.64	40
15	6671.32	3335.66	32
16	5377.5	2688.75	24
17	11211.54	5605.77	52

Spreadsheet of bolts numbers – PROPOSAL 2

The number of bolts required for each node is given below：

Table A18.3 Number of bolts - PROPOSAL 2 - WEST VIEW

Member	Axial Force kN	F_v,Ed kN	Number of bolts n
1	-2800.02	1400.01	16
2	-5819.93	2909.965	28
3	-9680.75	4840.375	44
4	-5093.66	2546.83	24
5	-3890.91	1945.455	20
6	-1815.45	907.725	12
7	-5100.13	2550.065	24
8	-3785.46	1892.73	20
9	-1985.3	992.65	12
10	-5293.43	2646.715	24

11	-9936.01	4968.005	44
12	-14912.5	7456.255	68
13	-2613.99	1306.995	12
14	-2686.64	1343.32	12
15	-2868.05	1434.025	16
16	-19647	9823.485	88
17	-3371.95	1685.975	16
18	-11.51	5.755	4
19	4018.22	2009.11	20
20	5402.47	2701.235	24
21	-7757.46	3878.73	36
22	-6601.76	3300.88	32
23	-5009.28	2504.64	24
24	267.37	133.685	4
25	-7768.61	3884.305	36
26	-2904.28	1452.14	16
27	55.39	27.695	4
28	-7230.48	3615.24	36
29	-2194.17	1097.085	12
30	55.39	27.695	4
31	5409.68	2704.84	24
32	-7214.76	3607.38	32
33	-7176.76	3588.38	32
34	-5976.46	2988.23	28
35	1916.1	958.05	12
36	3955.8	1977.9	20
37	4905.91	2452.955	24
38	-54.18	27.09	4
39	1976.32	988.16	12
40	306.07	153.035	4
41	-106.88	53.44	4
42	-5186.25	2593.125	24
43	10.15	5.075	4
44	-934.46	467.23	8
45	-583.11	291.555	4

46	-134.52	67.26	4
47	-4700.1	2350.05	24
48	-6308.55	3154.275	28
49	-6193.01	3096.505	28
50	4314.8	2157.4	20
51	6042.94	3021.47	28
52	5912.43	2956.215	28
53	-8.69	4.345	4
54	185.54	92.77	4
55	-32.41	16.205	4
56	-635.52	317.76	4
57	-641.8	320.9	4
58	-35.29	17.645	4
59	5180.78	2590.39	24
60	6360.49	3180.245	32
61	6256.27	3128.135	28
62	-5499.78	2749.89	28
63	-6588.38	3294.19	32
64	-6406.63	3203.315	32
65	-1193.64	596.82	8
66	-782.5	391.25	4
67	-409.71	204.855	4
68	1092.96	546.48	8
69	659.4	329.7	4
70	143.24	71.62	4
71	3292.92	1646.46	16
72	-3476.97	1738.485	16

The number of bolts required for each node is given below：

Table A18.4 Number of bolts - PROPOSAL 2 - NORTH VIEW

Member	Axial Force kN	F_v,Ed kN	Number of bolts n
1	-3531.2	1765.6	16
2	-3327.48	1663.74	16
3	-3480.36	1740.18	16

4	-2215.752	1107.876	12
5	-3667.07	1833.535	20
6	-2712.03	1356.015	16
7	-4613.08	2306.54	24
8	-16358.87	8179.435	76
9	-10.96	5.48	4
10	6200.48	3100.24	28
11	45.64	22.82	4
12	-885.08	442.54	4
13	1834.95	917.475	12
14	1052.67	526.335	8
15	-838.23	419.115	4
16	1854.98	927.49	12
17	-2587.38	1293.69	12
18	7230.33	3615.165	36
19	2349.14	1174.57	12
20	-4247.31	2123.655	20
21	7344.59	3672.295	36
22	-4701.65	2350.825	24
23	15524.82	7762.41	72
24	4967.35	2483.675	24
25	-11287.51	5643.755	52
26	15428.24	7714.12	72
27	8986.42	4493.21	40
28	57.54	28.77	4
29	7702.55	3851.275	36
30	-8344.09	4172.045	40
31	1247.4	623.7	8
32	1357.9	678.95	8
33	-1471.2	735.6	8
34	-1585.88	792.94	8
35	4039.68	2019.84	20
36	3640.23	1820.115	20
37	-4124.3	2062.15	20
38	-3857.65	1928.825	20

39	6276.78	3138.39	28
40	7165.36	3582.68	32
41	-6145.04	3072.52	28
42	-7517.31	3758.655	36
43	-9798.58	4899.29	44
44	-10775.65	5387.825	48
45	9882.52	4941.26	44
46	10593.91	5296.955	48