Structural Design of North Side of Breda Central Station

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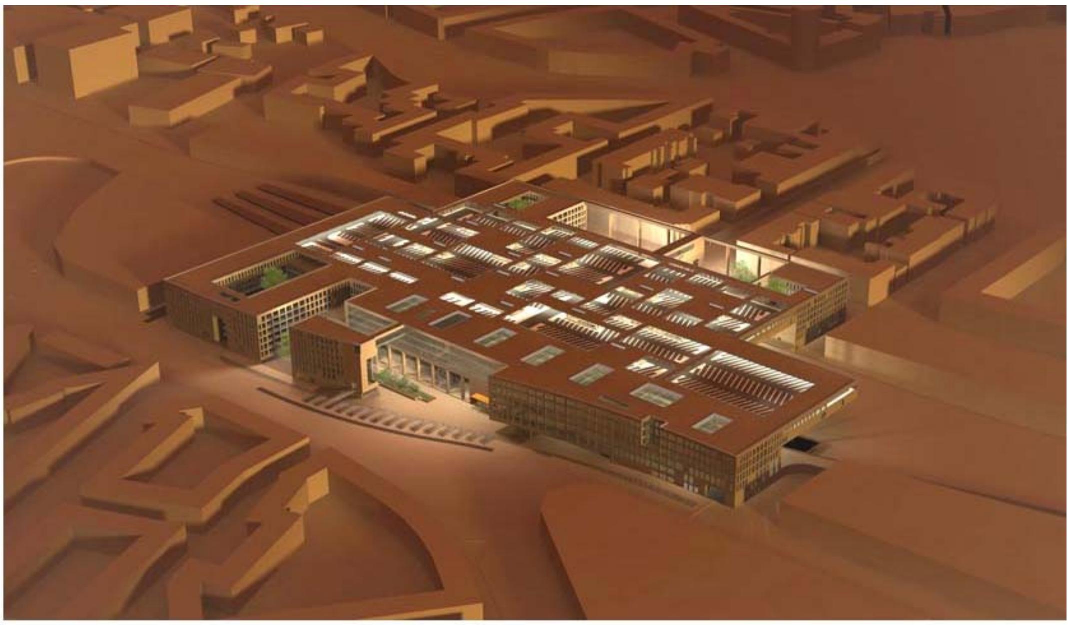

Delft University of Technology

Master's Thesis

Structural Design of North Side of Breda Central Station

MASTER OF SCIENCE THESIS

Civil Engineering

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Preface

Graduation Committee

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This report as the master's thesis is the completion of my MSc study in Building Engineering at Delft University of Technology, Faculty of Civil Engineering and Geosciences. The subject of the master thesis is structural design of north side of New Breda Central Station. Breda Central Station has been selected as one of the six new key projects of the redevelopment of major stations in the Netherlands. DHV is in charge of the part of the preliminary design and the total final design of the terminal complex. For both DHV and me, it is interesting to create an alternative structural design for the north side of the station complex which is a multi-function part composed of offices, bus terminal and underground commercial area with almost 300-meter length and less than 30-meter width. The objective of the thesis is to reduce the amount of the columns on the bus terminal to get more open space in accordance with the rest of the station. The research and design was supported by DHV Building and Industry and performed from January 2009 to September 2009.

This thesis could not be accomplished without the help of the supervisors, colleagues, friends and my family. I would like to give my acknowledgement to all of them.

Firstly I would like to thank all the members of my graduation committee for their contribution to this project, and for their comments and advices during the meetings. I would give my sincere gratitude to Ir. Rene Hosptaken who provides me the opportunity to work on this interesting project and his patience and face-to-face meeting regularly. Next, I would like give my most appreciation to Prof.Dipl.-Ing.J.N.J.A.Vamberský, the chairmen of my committee, Ir. Sander Pasterkamp from Section of Structural and Building Engineering and Ir. Henk Mihl from Faculty of Architecture for their careful, patient and critical guidance throughout the period. I would also like to extend my thanks to Ir. Henk Mihl for his recommendation of all the books and articles which inspire me a lot.

Furthermore I would thank all my colleagues, Ying Ying Ip, Matthijs Toussaint, Wietse Kragting, Laurent Schouten, etc in DHV Building and Industry who have supported me, taught me and also made me feel at home in the office.

A lot of thanks go to my good friends Luyuan Li as well as Jason Yan, Li Liu, Jingyi Liu and so on, for their selfless help with the encouragements and discussions. Thanks to all of my friends who spent and accompanied all the happiness and perplexity with me during the two years study in the Netherlands.

Last but not least, acknowledgement is given to my family especially my great mother who has supported me throughout my life and forever. Special thanks go to Felix Wang for his daily support and encouragements.

Yirui Yao
August, 2009
Delft

Summary

Breda Central Station, one of the six new key projects in the Netherlands, designed by Koen van Velsen, has a vision of all the functions under one single roof. The visualization of the architectural design shows large open space in major public area such as the train platforms and the concourses. However, there are a lot of columns on the bus platform in the north of the new station. The goal of this master's thesis is to research the consequences for the design of diminishing the columns to get more open space at the bus terminal with 3-storey offices above it. Big challenge of this project lies in the multi-functional north side including offices, bus terminal and underground commercial areas, which requires different column spacing and ceiling heights at different levels. This increases the difficulties of the goal of reducing the columns.

The new station complex is a 6-storey high building with a roof covering all the functions at about 24m high. The length of the station from east to west is approximate 280m and the width from north to south is approximate 180m. Under the one single roof, the multi-function station consists of: four apartment blocks in the north and south, the main entrance in the south, the railway terminal in the middle, the office and bus terminal in the north, the bicycle storage and commercial area underground, and the car parking on the top. With the requirements of getting enough light into the station complex, plenty of openings and windows were applied both on the facades and the roof. Due to the station complex's large scale and the characteristics of each part, some logical cuts had been chosen to separate the station into several parts by expansion joints. The 27.5m wide north part of the station complex (scope of this thesis) contains the underground commercial area, the bus terminal at level 1 and 2, and the offices with partial curved cantilever at level 3 to 5.

Firstly, several widely used structural stabilizing systems (frame structure, wall or core structure and space structure) and floor systems (prefabricated floor, cast-in-situ floor, composite floor) in the Netherlands had been studied, each with advantages and disadvantages. With the program of requirements, the goal of this thesis project, and the guidance of the structural solutions, five alternatives had been designed for the north side of Breda CS, which included two arch support structures, one frame structure with braced cores, one truss structure and one space frame structure. The contributions and features of these alternatives had been analyzed respectively. To select a most suitable one for this project, MCA (multi criteria analysis) were applied to the selection under different parties involved in the project. It came out that the truss structure was the most suitable structural concept among all the proposals to elaborate. The chance of the reducing the columns on the bus terminal, the light weight structure, and simple truss behavior were the most important motivations for this choice.

The elaboration of the final structure started with the division of the structural model into 10 modules to simplify the structural model and design process. However, the north part would still be constructed continuously without expansion joints. The slim floor system had been selected with regard to the weight, floor thickness and flexibilities. The pattern of the truss had been determined by the pattern optimization and unit check. The results of the single module and the complete truss structure in the offices were verified effectively. On the bus terminal level, the tree column structure was designed to save more columns besides the contribution of the above trusses. The geometry of the tree column structure was determined by form finding to an optimal. The partial cantilever office as an additional part had been realized by diagonals. Frame structure had been chosen for the underground due to its functional requirements.

With the results of the final structure from SCIA ESA PT, conclusions could be drawn that the stiffness, stability and strength were generally verified effectively and sufficiently. Deformation of the structure, force distribution and most of the member stresses confirmed to the codes and rules of thumb. Only several members had yielded checked by von Mises stress which required

optimization of the strength. The detail results remain unclear until accurate calculations are made in the further study.

The comparison between the new structure and current structure showed a clear result that the goal of this thesis had been achieved. The number of the columns on the bus terminal had been reduced by 70% from 122 to 36, and the structural area had been reduced by 75% from 28.5m^2 to 7.1m^2. In addition, despite the offices with truss structure was 40% heavier, the weight of the whole structure had been saved to 1/3 of the current one by using steel instead of concrete. In summary, the new design reduces the number of the columns on the bus terminal and creates more open space for the public successfully.

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1 Introduction

In this chapter, the project background of new Breda Central Station, one of the six new generation key projects in the Netherlands, will be introduced in the first section, and then the general requirements of different functions of the new station complex from the clients will be mentioned. According to that, problem statement and objective definition of this master's thesis will be studied and determined.

1.1 Project Background

The Netherlands is set to join the Europe high-speed railway network which will create great opportunities for the station on the network's routes. With new facilities, outstanding architecture, the high-speed railway station areas will be transformed into attractive places to live and work. These major station redevelopment projects in the Netherlands launched in 1997, are officially called New Key Projects (Nieuwe Sleutel Projecten in Dutch), focusing on the proposed stations and its surroundings of the stops of the future HST (High Speed Train). Originally, five HST stations in the Netherlands were selected into this New Key Projects in 1997, which were Rotterdam Central and The Hague Central on the southern line, Utrecht Central and Arnhem Central on the eastern line and Amsterdam South Aix connected both lines. Breda Central on the southern line was added as the sixth one in 1998 into the project. It is strategically located between the Dutch Randstad and the Belgian Rhombus – Antwerp, Brussels, Mechelen and Ghent.

Fig 1-1 Six NSP in the Netherlands [1]

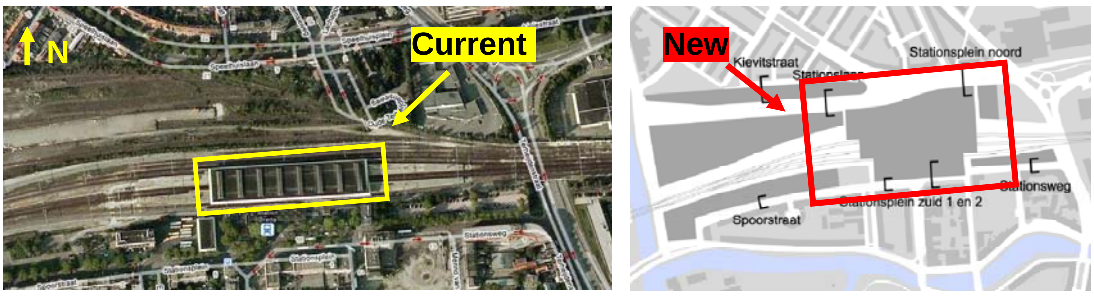


Fig 1-2 Breda CS location [2]

The new Breda Central Station known as a new Public Transport Terminal Complex will serve as a future HST-shuttle stop which reduces the travel time to Amsterdam, Rotterdam and Antwerp by 50%. Moreover, the new terminal complex will also be the icon of the centre of Breda as the Church of Our Lady in the historic city centre.

The new Public Transport Terminal Complex is going to be the interchange of international, national, regional and local transport connections: of international arrivals and departures to and from the Public Transport Terminal Breda-CS, which will consist of:

– the railway station
– the bus station

- the bicycle stalls
- the car parks belonging to the Public Transport Terminal
- the real estate developments on the edges of the Public Transport Terminal
- the commercial amenities within the Public Transport Terminal and at ground level of the real estate development
- the public space in front of the terminal

For the terminal complex itself, this will result in an integrated terminal bringing all the function and modes of transport (including train tracks, bus station, elevated car park, pedestrian tunnel, and bicycle link and installs) under one roof. However, it still has to make the travelers who use the station to find their way around easily, quickly and safely. Before that, there will be a newly built platform to accommodate the growth in passenger amounts, in accordance with a level that matches the NSP quality.

The terminal complex occupies an area of about 280m×180m and is about 20m high. It can be divided into three parts from north to south, of which there are the north square and concourse, bus terminal, office space and a dwelling on the north side, the platform and car park in the middle, and south square and concourse and two dwellings on the south side.

Selection of Design Team
27 architectural firms registered for the tender of New Breda Central Station project in 2004, and seven of them were selected to make a spatial vision to present. In the end, the vision of the team Koen van Velsen / Atelier Quadrat as a progressive, unconventional, imaginative and instructive one was selected by the selection committee. The vision of the architect that the future public transport terminal would have a significant improvement of the urban connection of the station of the city and the link between North- and South- Breda. The future train passengers would be able to get a view of the city and the function such as living and working in the station quarter. The new Breda CS is the engine of the station quarter and will be developed into a major location including business center of international style. And DHV is executing the structural design.

Breda CS Project Information

Cooperating Parties:	Gemeente Breda
	Provincie Noord-Brabant
	Ministerie van Verkeer en Waterstaat
	Ministerie VROM
	NS
	ProRail
Architect:	Architectencombinatie Koen van Velsen/Quadrat
Engineering:	Breda AAA (a combination among DHV, Movares and NACO)
Design commences:	2006
Construction period:	2008-2011

1.2 New Breda Central Station

For all the functions locating together in the terminal complex under one roof, clear division and traffic flow should be made. The urban development outline is listed below.

Passage
The passage underground with pedestrian way links the platforms and station entrances on both sides. Because the station will also serve as a HST-shuttle of international arrivals and departures as well as local transport connections, a good traffic flow for the passengers is essential. In addition, there will be some commercial uses like retail in the passage as well.

Platform hall
The platforms and rail tracks will locate in the central of the Breda CS. The platform hall consists of 3 platforms. Passengers must be given a pleasant place to wait and transparent material must be used so that the waiting rooms, balustrades and elevators do not obscure sightlines.

Car park
Above the platform hall, there will be a car park containing 700 places on the top of the station complex. Meanwhile, the roof requires as much as daylight as possible and also admit enough light to illuminate the platforms underneath.

The south square and concourse
The southern square is a sunny and open space facing the city center side as an entrance to the station. In the concourse, there also locates elevators and staircases which connect the car parking on the roof and bicycle stall underground.

Apartments
There are three dwelling groups in all, among which two locate on the south side with 60 apartments and one locates on the north side with 68 apartments.

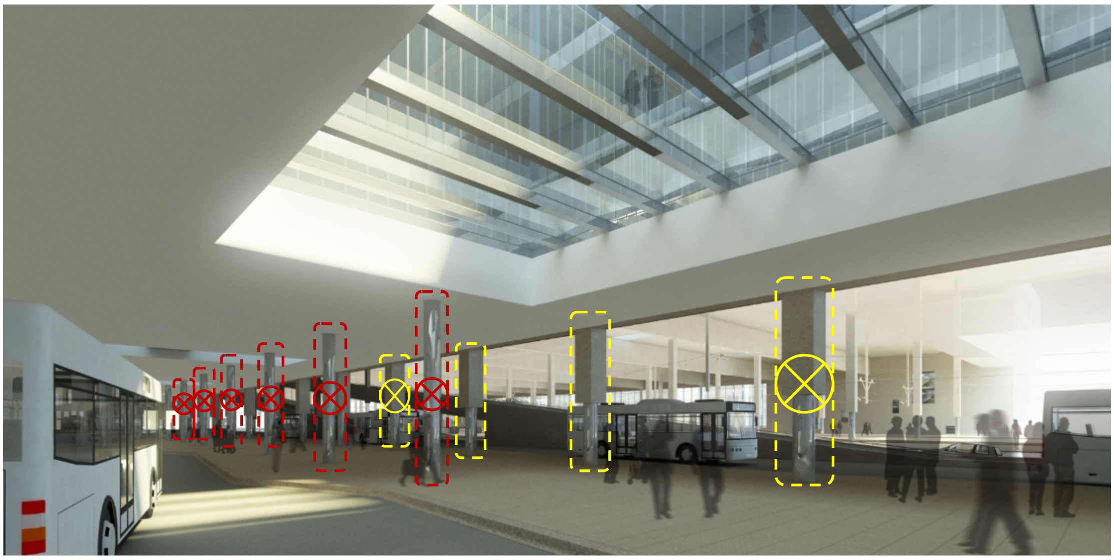
The north square and concourse
The terminal's north side is a mix-function area that possesses a passenger tunnel, commercial use and bicycle stall underground, a bus terminal on the first and second level, and office areas above the bus terminal from level 3 to 5. Besides that, there is a dwelling block on the east side. Similar to the requirements of the middle part, the north building also requires abundant daylight into the building.

Bus terminal
As the requirement of the clients, the bus terminal will locate at the first and second level direct to the train platform. It will accommodate about 16 departure points and 2 arrival points, and 3 buffer areas.

Office area
The office spaces situates above the bus terminal along the whole length of the building for about 20000 m^2. The requirements of as many rentable areas with abundant daylight into the building as possible result in the high demand of architectural and structural design.

1.3 Problem statement

Open space on train platforms



Lots of columns on bus platform

The design of the Public Transport Terminal Complex made by van Velsen has a vision of all the modes of transport, residential accommodation, office part and commercial facilities are combined under one single roof.

The public area of a railway station as a public facility is in general quite spatial, so is the roof over the south concourse elevated to a height of about 20m above the ground. Also the north concourse has a 10m high entrance ceiling. The visualization of the architectural design shows that the train platforms have very open space while there are quite a lot of columns on the bus platform with lower ceilings. In order to create more open space on the bus platform, in this thesis it is researched what the consequences will be for the design to diminish the columns on the bus terminal with 3-storey offices above it.

Big challenge in this thesis is the multi-function of that station complex part includes office, bus terminal and commercial areas, each requiring different column spacing and ceiling heights. This increases the difficulties of reducing the columns and realizing more open and flexible space on the bus terminal.

1.4 Objective definition

So by formulating the problems above, there comes some objectives for this master thesis project,

1. Compose the Program of requirements of the office building, and then study the most frequently used structural systems for multi-storey buildings in the Netherlands.
2. Create several alternative structural systems which fulfill the demands of the new design.
3. By using Multi Criteria Analysis, choose and design one of the alternatives which is the most efficient structure by all the parties involved in a project.
4. Design and elaborate the selected structural system, optimize it and reviewing the current design by comparing the cost, feasibility, buildability and etc.

2 Architectural Design of Breda CS

In order to create a picture of the multi-function station complex, in this chapter, the general architectural design for the whole station complex will be described, followed by a more detailed description of the north side which is the scope of this thesis. The features of the design of the north side and their consequences to the structural solution will be introduced.

2.1 General Design

The architect made a flat and generally regular shape design for the station complex based on the program of requirements and to blend with the buildings around it.

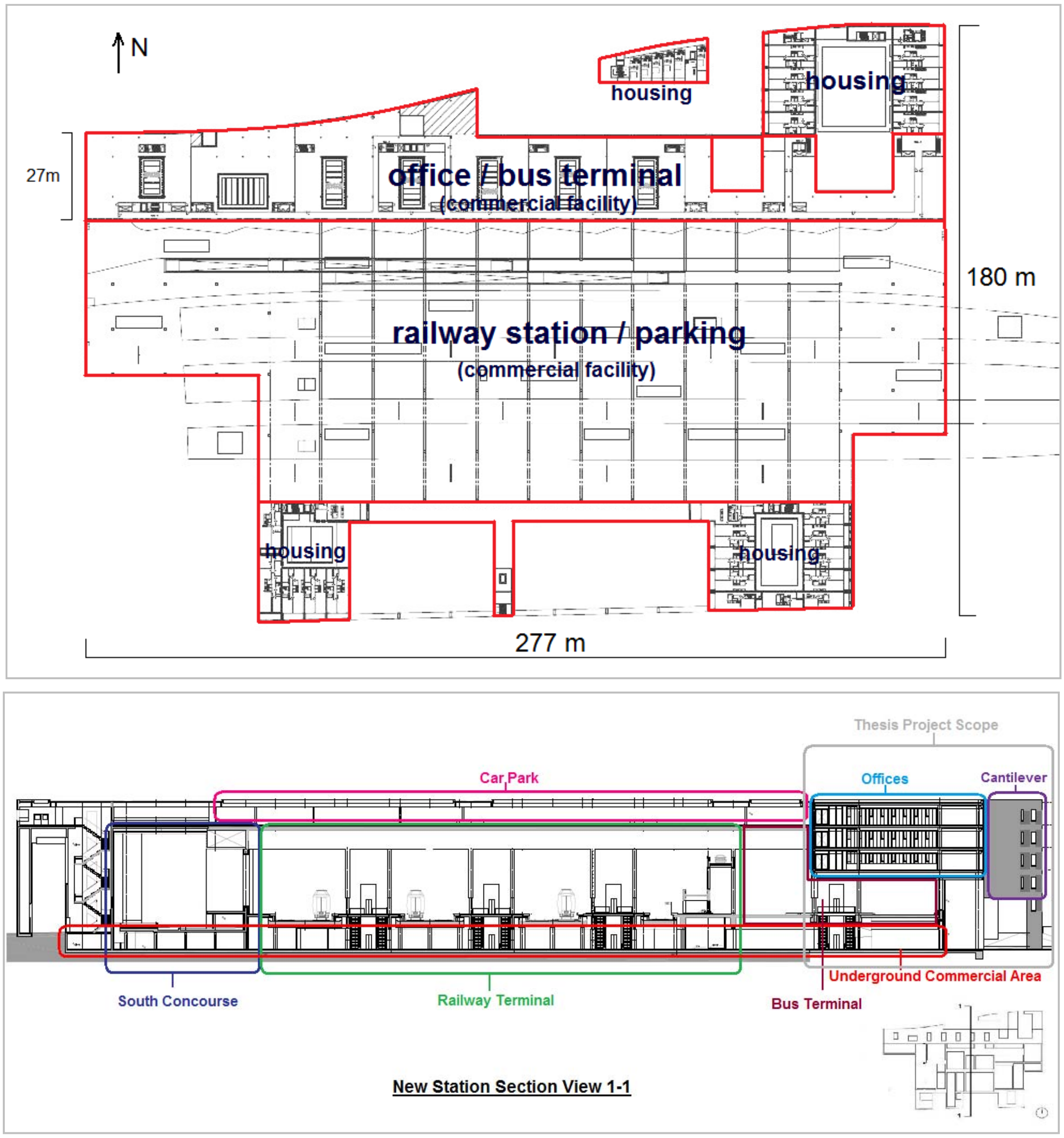


Fig 2-1 New station plan view and section view

The station complex is a six-storey-high building, and the roof covering all the functions in the complex stands at 23.8 meters high. The maximum length of the station from east to west is 277.2 meters and maximum width from north to south is about 180 meters. In the architectural design, under the requirements of enough light pouring into the station, there are plenty of openings and windows applied, not only on the façades, but also on the roof which acts as a "fifth façade" of the building. More images and the location of different functions can be seen in Appendix 3. To harmonize with the adjacent buildings around the station complex, the materials on façades like brick are chosen to accommodate to this, and meanwhile let the façades look like a whole. At the lower level of the station, light transparent material is used to the entrances.

Due to the characteristics of each part of the station complex, some rather logical cuts in the building blocks can be chosen where expansion joints will be located. Therefore, the whole station complex would mainly be separated into seven individual parts which are four housing blocks, the office and bus terminal as one block, the roof cover platform and the south entrance. Each of them will have its own stability which means they don't depend on the other building part for structural integrity.

This thesis focuses on the north side of the station complex including office, bus terminal and commercial area underground. Usually housing block is used as stabilizing cores in the structure; however, the apartment in the north side will not be included in the independent north part. Two factors determine the decision. If the apartment is used as a stabilizing core, firstly additional expansion joints are needed due to the super long length, and secondly the location of the apartment, at the corner, causes negative effect on the structure which will result in large deformation on the other side, and even rotation of the building. Therefore, there will be no structural relation between the north side (scope of the thesis) and apartment. Moreover, in the original design, on the south facade of the offices, the parking deck is supported on it which causes additional massive loads on the office structure. In this thesis, however, assumption is made that this will not be taken into account. Possible solutions will be considered afterwards.

2.2 The north side of Breda CS

2.2.1 Multi Functions

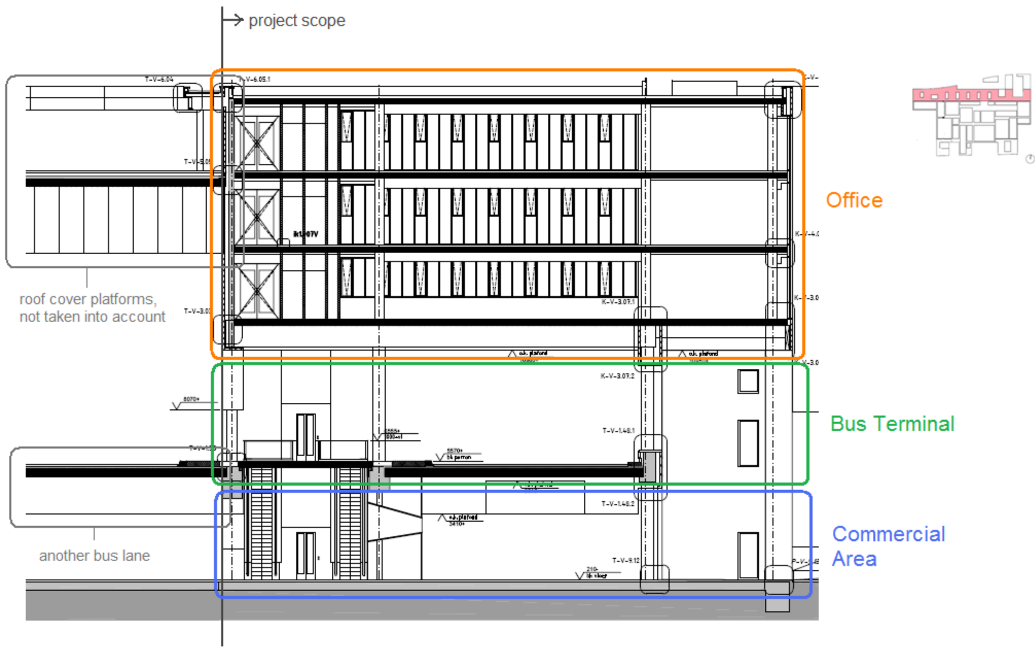
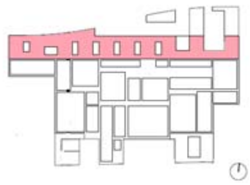


Fig 2-2 section view of north side

The north part of the station complex (scope of this thesis) contains:

_Commercial area	(level -1, 0)
_Bus terminal	(level 1, 2)
_Offices	(level 3, 4, 5)

This is so called “stacking of different functions” which often leads to design difficulties, such as the column and other structural elements arrangement. It’s always a challenge to find an optimum of the structure.



Commercial Area

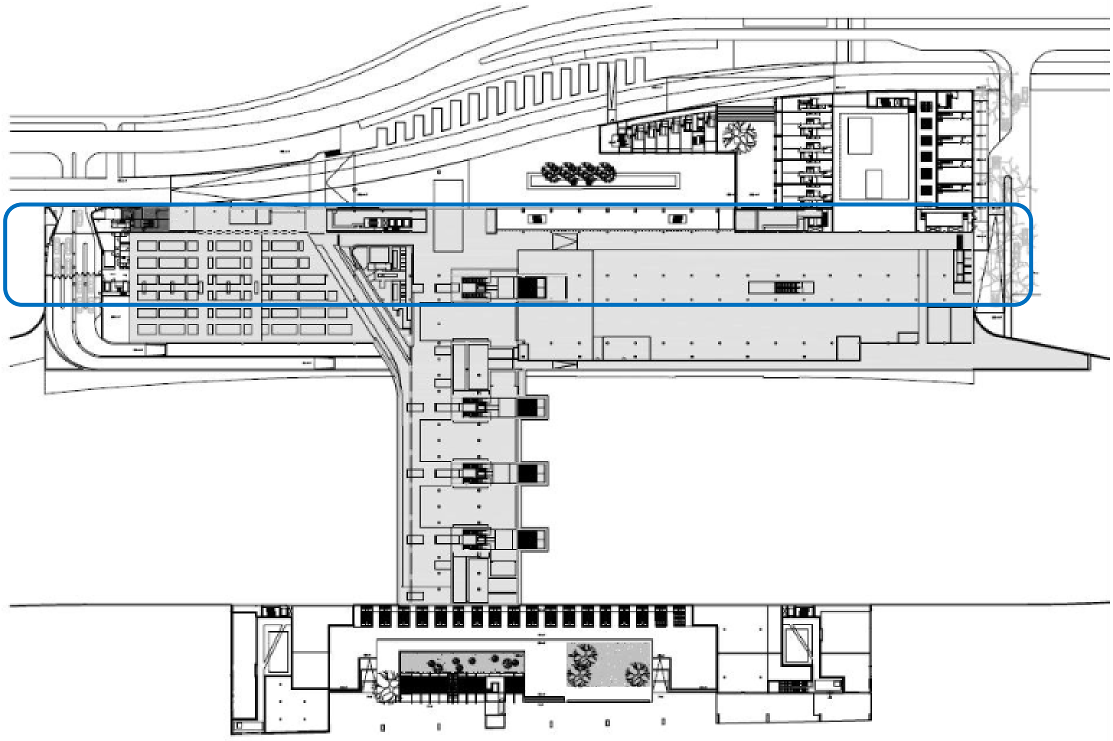


Fig 2-3 Plan view level -1, 0 **(□ commercial area)**

Within the multi-funtion north side of the new station complex, the underground commercial area also consists of kinds of facilities which are characterized as,

- The underground commercial area consists of varies functions including entrance for car parking, bicycle installs, bicycle lane, retail stores and etc.
- All of these functions have different requirements of the ceiling height with large ducts for ventilation, sprinker, and etc. The floor-to-floor height of the underground is 5490mm while the floor-to-ceiling height is 4780mm.
- Also high distributed live loads due to bicycles, shop storages and so on are applied there.
- Moreover, due to the demand of the developer and change of the lease market, the tenants of the retail stores usually change every 3-5 years and different tenants require different spatial arrangements for their shops, therefore the structure of the commercial area has to be as much flexible as possible. Shear walls and other similar solid structure are then not suitable to design.

Bus Terminal

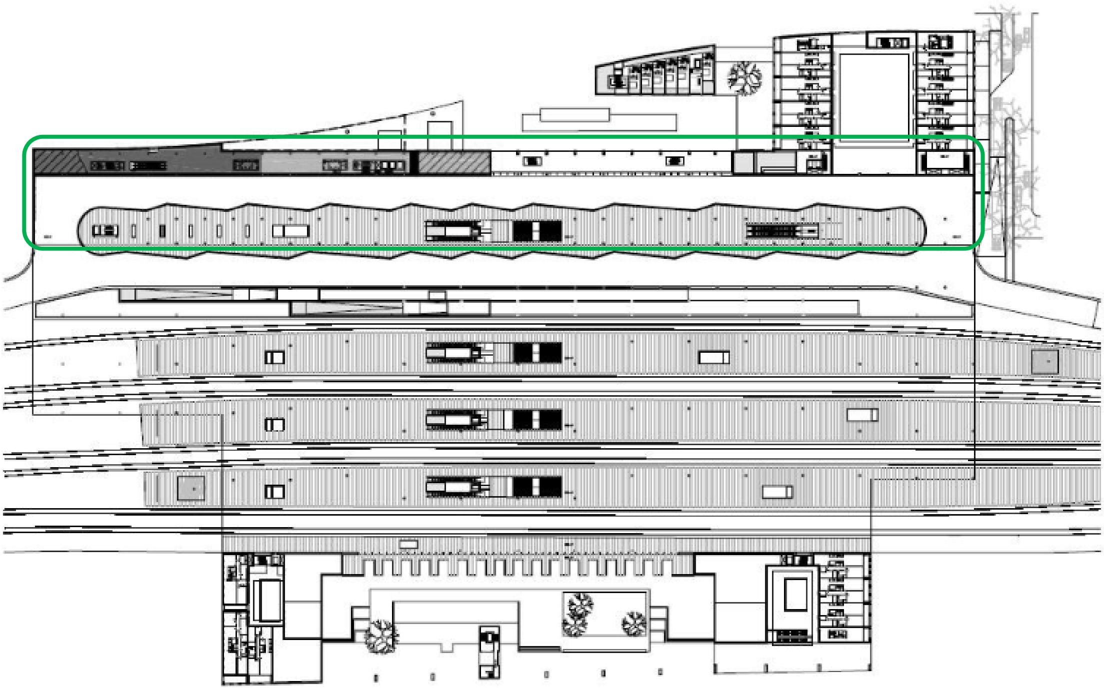


Fig 2-4 Plan view level 1-2 **(□ bus terminal)**

The bus terminal locates at the north side of the station complex from level 1 to level 2. The platform is about 7.2m wide and the lane for bus going through is about 13m wide. The rest area of this level contains partial office and some traffic facilities such as staircases and elevators. The sawtooth shape platform makes use of the characteristics of the bus operation to get more space. Unlike other area of the station complex which large open space has been designed, there are many columns locating on the bus level. This results in the goal of this thesis to diminish the number of columns there. The flexible spatial space not only harmonizes with the station complex, but also is welcome by the public. The features of the bus terminal is,

- It is not a separate structure but has to support three-storey offices above it which increases the difficulty of the design. In order to reduce the number of columns which means reduce the amount of the structure bearing the offices, the offices have to be designed as light weight structure.
- The arrangement of the structure on the bus terminal is also limited because there must be enough lane area for the busses to go through and make a turn. Thus, the blue area in Fig 2-5 has to be fully freed without any column or any other vertical structure, and the remaining columns can only locate in the yellow area.
- The two ends of the building have a closed image that it's possible to design load bearing structure there to transfer vertical and/or horizontal loads to the foundation.
- The floor-to-floor height of the bus terminal is 7.2m. Considering the goal of this thesis and the normal bus height of 2.8-3.5m, the floor-to-ceiling clearance should be as high as possible. This might influence the structural possibilities.

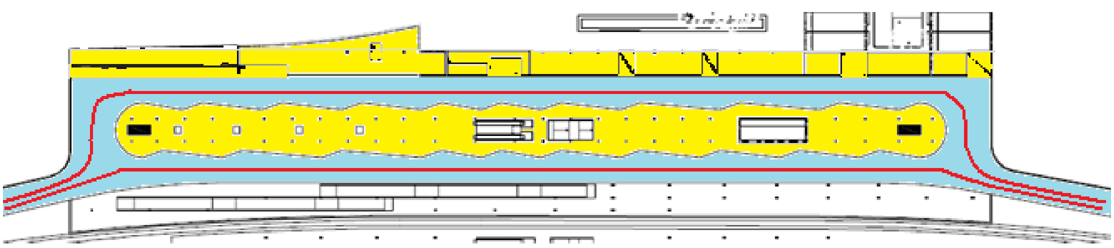


Fig 2-5 Bus route in the terminal (red line)

Office

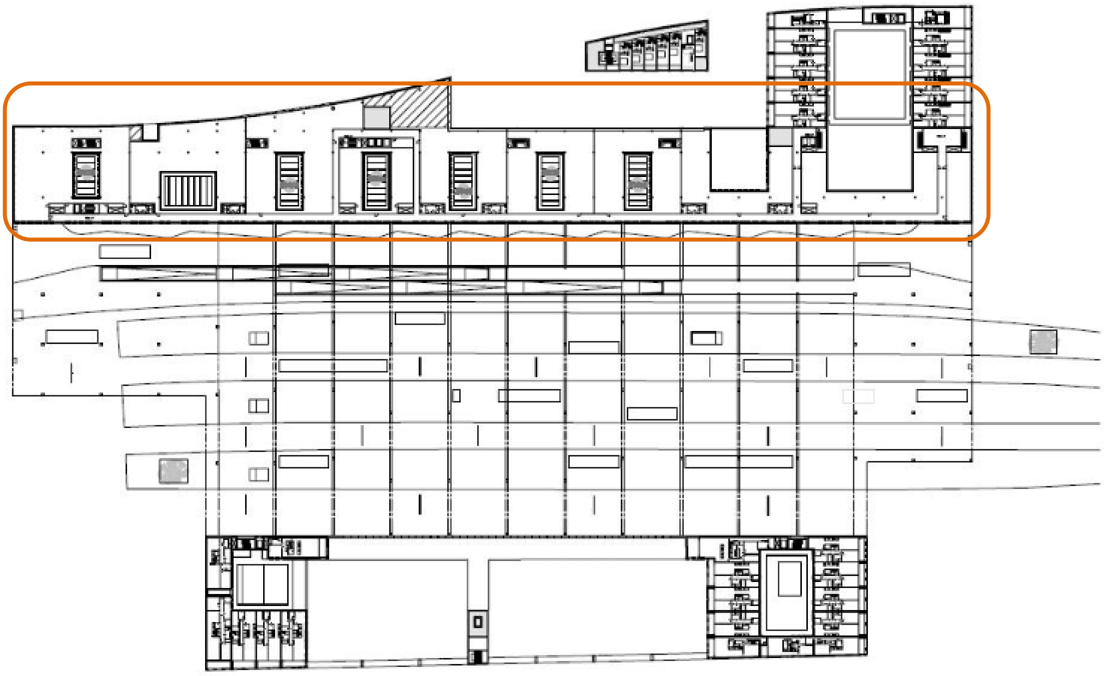


Fig 2-6 Plan view level 3-4-5 **(□ office)**

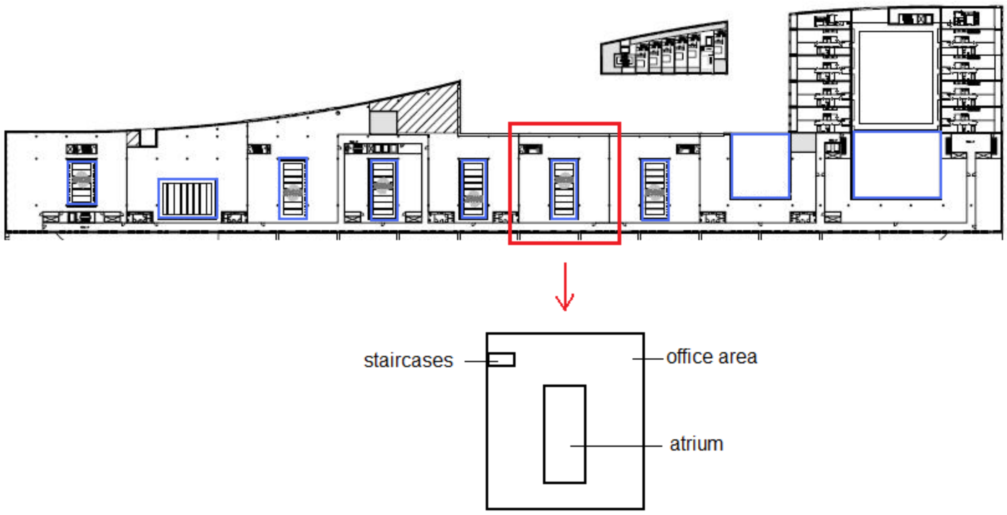


Fig 2-7 Atrium and cuts

The floor-to-floor height of the offices is 3.6m while the floor-to-ceiling is 2.7m, which means there is 0.9m space for floor thickness and installations. This affects the choice of floor system. There are 7 atria (openings) along the building which were designed to bring daylight into the office. Besides that, there are also 2 large cuts locating in the east of the building. These atria and large cuts have large influence on the design of the building. The floor system of a building usually acts diaphragm action like a deep beam to resist the horizontal loads, however, these openings from level 3 to level 5 breaks the diaphragm action to some extends.

On the other hand, these atria distribute relative evenly along the building which could be used as vertical stabilizing cores for the structure. But considering the light that has to be brought in, the stabilizing cores cannot utilize relatively closed wall structure, only open bracings can be used.

Nevertheless, these openings only extend to the bottom of the office part at level 3 and the projection of them locate on the lane of bus terminal, which signifies that the stabilizing cores are not able to reach the foundation. This extremely weakens the use of these cores.

These fixed atria in the office also influence the arrangement of the office. The circulation has to be designed longitudinally along the building while the office area will then be arranged around the atria.

2.2.2 Other Features

Façade

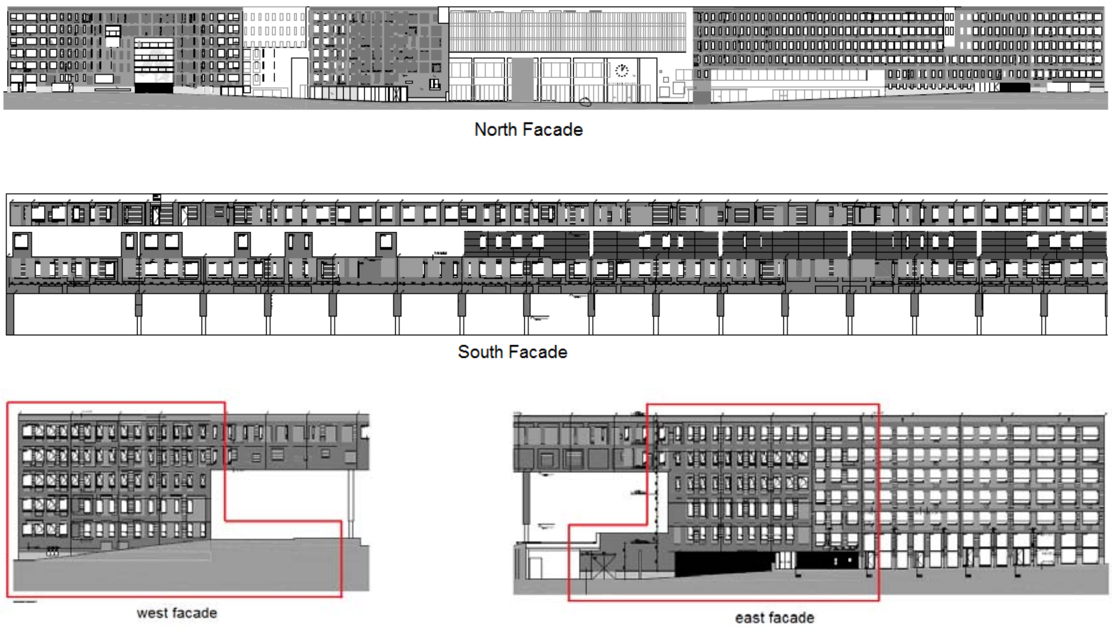


Fig 2-8 Facades

There are 4 façades of this part in which the south one is directly connected to the station terminal hall. The other three façades have amounts of rectangular openings on them due to the needs of enough light and serving as the vestibule of the city. On the east and west side, the elevation stands flat, but on the north side, due to the complicated geometry, the façade has variant differences.

Cantilever
On the north side, the facade from the 2nd floor up curving in an outwards direction forms a cantilever. According to the design of the architects, one column is set to support it, therefore a proper structural solution such as light weight floor and facade has to be made. Meanwhile, the deflection of the cantilever also has to be well controlled to prevent the materials like brick on the facades from being damaged.

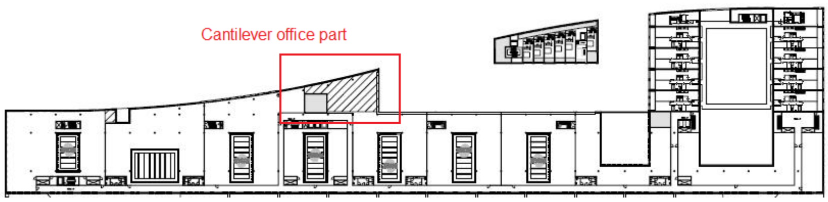


Fig 2-9 Cantilever office part

3 Program of Requirements

This chapter will introduce the different requirements for the structural design. The local conditions, functional and technical requirements from the client and government will be described. These requirements are so objective that the design of the structure is strongly limited by them and has to follow them perfectly.

3.1 Local Condition

Project Location
Breda Central Station locates in the inner city of Breda in a strategic position approximate five to ten minutes' walking distance to the city center and other historical sites. The station links the two different worlds on the north and south. The village-like urban area is set on the north side and the south side faces the inner city of Breda.

On the south side of the station quarter many existing buildings locate around, including a building listed as a monument. The distances from the station at some places is only about ten meters, which means there is limited space for construction. Also the influence of the construction activities on these buildings has to be measured during the design and construction (See Appendix 1). Fortunately on the north side, more spaces are owned by NS so that there is sufficient space for construction.

The zoning plan (*bestemmingsplan*) is the key planning document that contains information regarding planning rights and restrictions. Several requirements can be concluded from the plan,

- The existing station has to remain operating during the whole construction period;
- The current track layout (5.1 and 5.2) is a physical boundary;
- There are a lot of new developments all around the station complex (2.1-2.9) which means these areas could be used as construction site. Nevertheless, time might be a restriction since the developments could be started before the construction of the new station finishes;
- To the north and south side of the complex, existing buildings are not far away from it which limits the construction, especially on the south side, there is very little space left with the existing buildings of 10-20m distance.

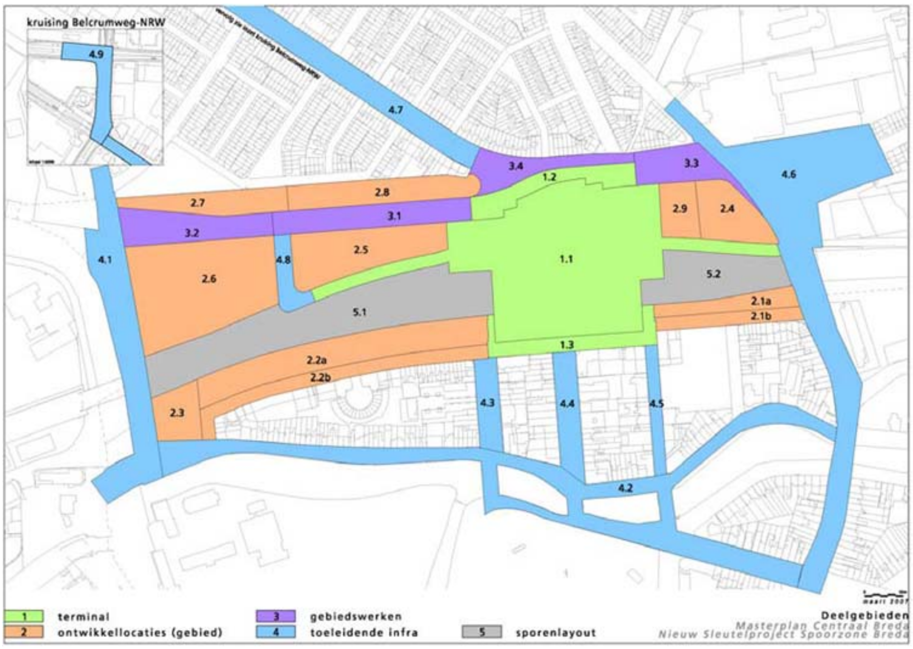


Fig 3-1 Masterplan Central Breda [2]

Soil Condition

The soil condition and water level of the site determines the way of construction of foundation. CPT (Cone Penetration Test) has been done to determine the soil conditions for the project.

Top level (t.o.v. NAP)	Soil Type	γ/γ_{sat} [kN/m³]	Φ' (°)	c' [kN/m²]	$K_{1;rep}$ [kN/m³]	$K_{2;rep}$ [kN/m³]	$K_{3;rep}$ [kN/m³]
2.5	03 sand moderate coarse	18/20	30	0	15000	7500	3750
2	05 very sandy loam	19/20	27.5	0	4000	2000	1000
0.8	03 sand moderate coarse	18/20	30	0	15000	7500	3750
0	04 very sandy clay	18/18	25	1	2000	1000	500
-0.5	03 sand moderate coarse	18/20	30	0	15000	7500	3750
-5	04 very sandy clay	18/18	25	1	2000	1000	500
-5.7	07 sand, solid	19/21	30	0	30000	15000	7500
-13	03 sand moderate coarse	18/20	30	0	15000	7500	3750

γ volume weight of soil — γ_{sat} volume weight of saturated soil
c' cohesion — Φ' angle of internal friction
K_{rep} horizontal bedding constant

Table 3-1 Indicated soil condition on the north side [3]

The current level of outside station is +2.5m NAP and +4.8m NAP for railway platform, while the level of new north square of Breda CS is going to be -0.21m NAP in the end which is below the ground water table of 1.1m NAP. So the basement floor has to be waterproof and to resist the upwards water pressure.

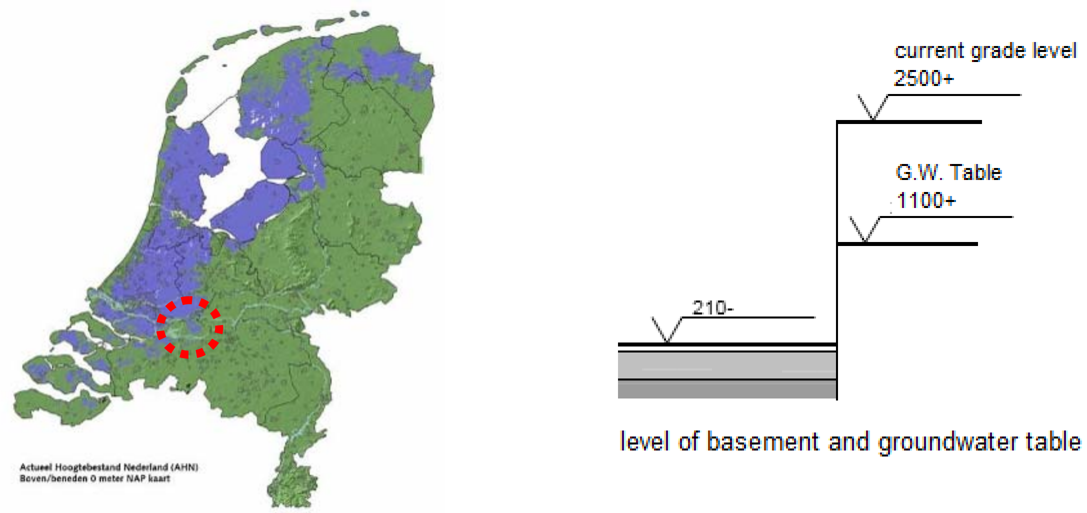


Fig 3-2 NL NAP map (blue: below 0m; green: above 0m, RWS) & G.W. Table Breda CS

Climate

In this project, due to the different functions, including open concourse, station terminal, bus terminal, office and apartments, in the station complex, all the situations should be in accordance with the program of requirements and codes.

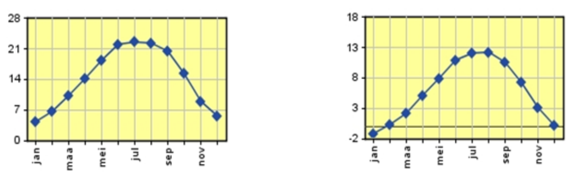


Fig 3-3 Maximum and minimum temperature of Breda (www.parool.nl)

	Winter	*Summer*
Outside temperature	-10 ºC (min.)	28 ºC (max.)
Wind speed	7 m/s	7 m/s
Humidity	90%	60%
Office temperature	21 ºC	≤ 25 ºC

Table 3-2 Climate requirements of office

Situation	*Temperature (ºC)*	
	Instantaneously	Extremely
Summer – outside not direct sun irradiation direct sun irradiation - very light color - light color - dark color	 17 17 17 17	 30 50 60 75
Summer – inside	17	25
Winter – outside	4	-25
Winter – inside	17	20
Underground structure	10	10
[a] white, light grey, yellow, crème [b] ocher, beige, gray, green, light blue [c] black, blue, brown, red		

Table 3-3 NEN 6702 8.8.2 Table 12 – Temperature

3.2 Functional Requirement

Fire Safety

The starting point of the fire safety is the Bouwbesluit. In general, the fire resistance for a public building situating more than 13 m above the ground requires 90 min.

Section	*Time*
Office and Bus terminal	120 min (30 min reduction)
Platform Hall	90 min (30 min reduction)
Tunnel / South Entrance	30 min (parking roof 90 min)
Apartments	120 min

Table 3-4 Fire safety requirements [3]

Sound

Since the office area and apartments are just next to the station, and is sensitive to noise nuisance, measures must be taken to reduce the level of noise to enable to minimize the disturbance.

The nuisance caused by the building work during the construction to the local residents and current station users should be limited and counteracted as little as possible too by means of some temporary measures.

Vibration

As the office area and apartments are directly linked to the railway station and are under one roof together, vibration problems caused by train and construction work will probably happen. Research has indicated that the housing blocks on the south side need to be isolated by vibration dampers, while the vibrations occurring in the north office part are within the limitations.

3.3 Technical requirements

Codes

The structural design of the project will be in accordance with Bouwbesluit and Eurocodes

General : Eurocode 1; Dutch National Annex; NEN 6702

Steel : Eurocode 3; Dutch National Annex

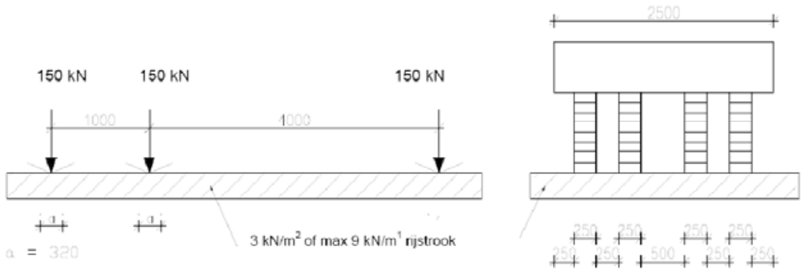
Concrete : Eurocode 2; Dutch National Annex

Loads

Dead loads: Including parking deck, bus terminal deck, office floors and roof, ramp, basement floor, glass floor, facades, etc

Live loads: Including parking deck, bus terminal deck, office floors and roof, ramp, basement floor, glass floor, facades, etc

Traffic loads: In accordance with Eurocode 1 Part 2 and NEN 6723 (on the bus terminal)



	Uniformly Distributed loads	One car	
		Axle weight	Each Axle load distribution on
Class 45	3 kN/m²	3×150 kN	4 wheels

Fig 3-4 Appendix A of NEN6723

Wind loads: Breda locates in the category III according to Eurocode 1 Part 1-4 and NEN 6702 which determines the calculation of wind loads.

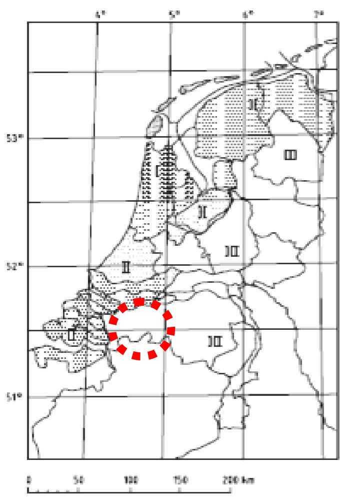


Fig 3-5 Wind category of the Netherlands

4 Structural Options for Multi-storey Buildings

The structural solution design concepts will comply with the program of requirements and architectural design. Before that, several feasible and suitable structural systems for multi-storey buildings are discussed.

For multi-storey buildings, force-flows transfer both vertically and horizontally. The forces are transferred vertically mainly through the stabilizing system, while horizontally through the beams and floor system.

4.1 Stabilizing System

The stabilizing system in multi-storey buildings is usually called the structural system which can be divided into three categories: Frame Structure, Wall Structure, and Space Structure. Their characteristics and suitability will be described individually in the following parts. In practical projects, composite stabilizing system utilizing the advantages of different systems may chosen by the engineers.

4.1.1 Frame Structure

Frames of different size and complexity represent one of the most frequent uses of flexible structural solution. The common used predominant forms of the frame structure are, steel or concrete frames. In structural principle, vertical loads on the roof and floors are transmitted by bending and shear into the columns which, in turn, transfer load into the foundations by means of bending, compressive and shear actions. Horizontal loads, like wind or earthquake load, have to be transferred into the foundation and depending on the frame geometry and the relative magnitudes of the horizontal loads and vertical loads, tension may be induced in some columns that uplift on the foundation.

For the purpose of design and analysis, the frame structure has been traditionally considered to be belonged to two different categories based on the construction method which are rigid jointed (continuous construction) or pin jointed (single construction).

Rigid Frame
One means of stabilizing the beam and column frame is by using moment resisting joints, which can transfer moments between members compared to other frame structure. And the members therefore may flex or curve due to the bending and shear stresses. The common used predominant forms of rigid frame are,
- Steel sections with welded joints or high-strength bolted joints for connections
- Cast-in-situ monolithic continuous concrete and the extended reinforcement achieving the member continuity to realize the rigid joints

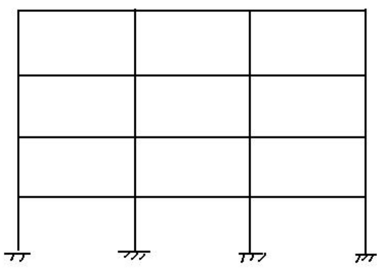
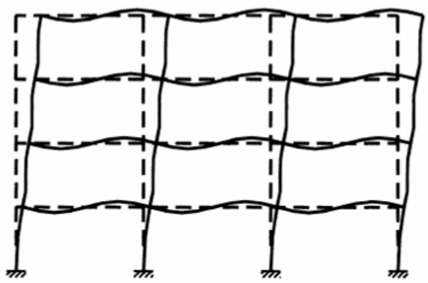



Fig 4-1 rigid frame and its lateral resistance mechanism

Rigid frame offers an advantage for architectural plan as a bracing system. Absence of other stabilizing elements, for instance, shear wall or diagonal bracing clear up the inside spaces and outside façade.

One of the benefits of rigid frame structure is to resist seismic load due to movements of the frame which consumes the energy by dynamic loads. However, either for steel rigid frame or concrete frame, lateral deflections should be concerned as a key issue during design. Too many deflections may damage the rigid welds or bolts (and finishing) in steel structure and cause extensive cracking of concrete and damage to anchored reinforcement.

Advantages	- Stiffer than hinged frame under same condition - No diaphragm action of floor - Complete open structure with flexibility - Resist seismic load
Disadvantages	- Fixed connections increase cost and time - Large deflection may damage connections

Table 4-1 Summary of rigid frame

Semi-rigid Frame

In some cases, rigid frame may not be the optimum solution due to its larger and heavier connections. Semi rigid frame which behaves between fully rigid and simple connection helps with the problems caused by rigid frame. It not only transfers the vertical shear stress but also the moments through column-to-beam connections that offer potential restraint to the end moment and affect sufficient reduction in the mid-span moment of the beam. Since it benefits the advantages of rigid frame and simple frame, it has a relative complex behavior.

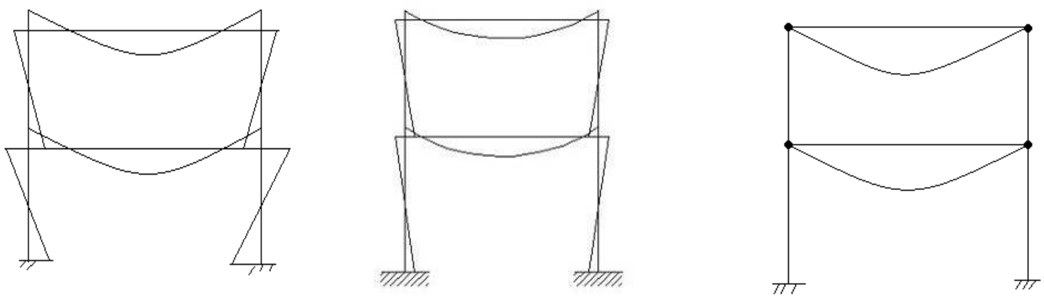


Fig 4-2 internal moment under gravity load in rigid, semi-rigid, and pinned frame

Advantages	- Reduce mid-span moment compared to hinged frame - Flexible than rigid frame
Disadvantages	- Complex behavior, difficult to analysis

Table 4-2 Summary of semi rigid frame

Pinned Frame

A pinned frame has members connected by pin joints. Since the joints are supposed to be incapable of transmitting moments, lateral stability requires the use of bracing or other stiff members because a rectangular bay with pinned beam to column connection possess no later stiffness. Therefore pinned frame usually also works together with shear wall or core elements to increase the lateral stiffness and at mean time free the connections from rigidity. The only exception to this condition is when the feet of the columns are rigidly fixed to a solid foundation so that they can function as vertical cantilevers. Pinned frame is widely used in multi-storey buildings because of its fast and simple connections which reduce the costs of material and labor efficiently. The disadvantage of this kind of structure is, for buildings in earthquake zones, it may be not rigid enough to resist large horizontal loads. Pinned frame can be achieved by variant materials like concrete, steel, timber and etc.

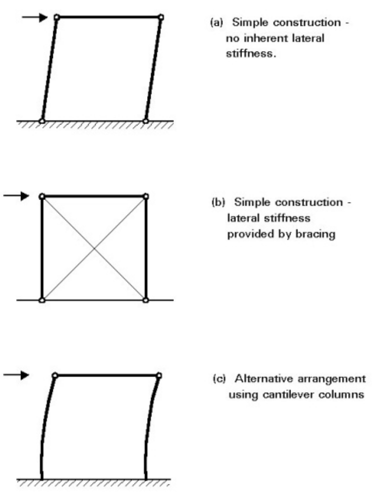


Fig 4-3 pinned joint frame [15]

Advantages	- Simple and fast connections - No moment transferred at the joints
Disadvantages	- Too flexible, not rigid - Not suitable in earthquake zones

Table 4-3 Summary of pinned frame

According to Eurocode 3, steel frame structure is classified into two kinds of system in order to provide guidance on the most appropriate type of analysis to use in particular cases, which are braced or unbraced frame, and sway or non-sway frame.

Braced Frame

A frame may be classified as braced if its sway resistance is supplied by a bracing system with a response to in-plane horizontal loads which is sufficiently stiff for it to be acceptably accurate to assume that all horizontal loads are resisted by the bracing system. The contribution of the bracing system has to reduce the horizontal displacement by at least 80%, otherwise the frame is considered as unbraced[15]. Bracing introduced in pinned or rigid frame helps to reduce and balance the deflection of frame structure caused by bending and shear. Most of the bracing elements are achieved by truss members, such as diagonals which carry the lateral forces in axial action predominantly. Compared to the wall structure, the bracing part in the frame just acts as the shear walls either locating separately or together to form a core. Moreover, the bracing parts also forms diagram actions under wind loads and other horizontal loads.

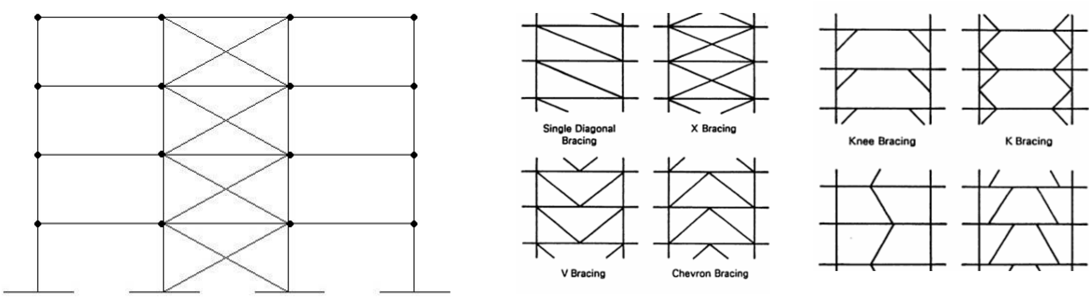


Fig 4-4 Braced frame and different forms of bracing

In this project, braced frame might be a good solution because it frees the layout of the plan which is important to both office area and bus terminal, and the bracing elements stiffening the structure can be set in variant positions in the structure.

Advantages	- Provide lateral stiffness effectively - And free the inside space at the same time - Frame can be designed with pinned connections - Bracing is less visible than shear wall, suitable in buildings needs more lights - Fast construction
Disadvantages	- Complicated joints at bracing and other elements

Table 4-4 Summary of braced frame

Frame – Shear wall or Shear core

Frame-shear wall structure is the combination of frame structure and shear wall structure which owns the advantages of these two structures, flexible in plan arrangement and stiff in lateral direction. It is quite commonly used in high rise buildings because of its virtues. The frames are generally analyzed with the assumption that they are pin-joined structures since stability is provided by the shear walls or cores, so that columns and beams don't need to have moment resisting capacity for the stability.

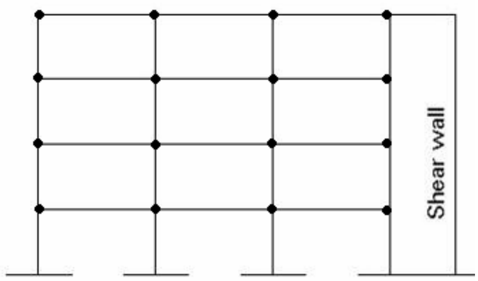


Fig 4-5 Frame-shear wall

Shear walls or cores can locate in different positions in the building plan, such as in the center or at the side of the building which confirms to the principle that it's better to set them symmetrically and continuously to prevent the building from torsion. This kind of structure is recommended to be used in medium- and high-rises to provide good stability and seldom used in low-rise buildings due to its lack of flexibility.

Advantages	- Provide lateral stiffness effectively - Frame can be designed with pinned connections
Disadvantages	- Cast-in-situ shear wall increases cost and time - Less flexible than braced frame structure

Table 4-5 Summary of frame-shear wall/core structure

4.1.2 Wall or Core Structure

Shear Wall

Shear wall structure is one of the most basic stabilizing structures throughout the history. Walls as structural elements transfer both vertical and horizontal loads. In small scale structures, no columns are needed, and the walls, as multi function, structural and space dividing at the same time save construction materials. And too shear wall have rigid connections that increase the lateral stiffness for the structure. For shear walls grouped together can be regarded as core structure which is rather stiffer since it provides stiffness in both directions.

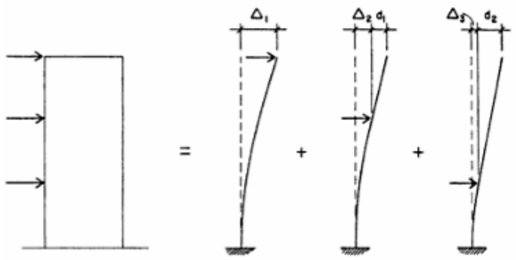


Fig 4-6 Deflection of a multi-storey shear wall [4]

Mostly concrete as solid material is used in shear wall or core structure. However, steel plate shear wall as new system is also an option because of its smaller thickness and light weight compared to concrete wall and other advantages of steel.

The layout of the shear walls had better be symmetrical and in general continuous throughout the entire building height, nevertheless according to the design of this project, it's difficult to arrange continuous and symmetrical shear walls.

Advantages	- Provide lateral stiffness effectively - Combine multi-function together, structural element and wall
Disadvantages	- Not flexible in layout, not good for future renovation - Cast-in-situ construction increase cost and time

Table 4-6 Summary of shear wall structure

Shear Core Structure

This kind of core structure has the similar behavior as shear wall structure but combine the stabilizing walls together mostly in the center or at the side of a building. These stabilizing walls locate together to resist the horizontal loads from both two directions. Since they are rigidly connected to each other which mean each wall can acts as a "flange" to the wall perpendicular to it when the later one is resisting the loads from its direction. Besides that, the concentrated stabilizing core cannot be easily or impossible to renovate in the future, so services like elevators and toilets are usually arranged at the core location.

Shear core structures usually work together with frame structures, or loading-bearing facades which free the inside space and benefit the advantages for mechanical installations. And to stabilize the building, the torsion stiff core has to be combined with rigid floor slabs acting as a diaphragm.

Advantages	- Open façades - Provide lateral stiffness effectively in both directions - Combine multi-function together, structural element and wall
Disadvantages	- Not flexible in layout, not good for future renovation - Cast-in-situ construction increase cost and time

	- Work well with other system, like frame or load-bearing facades to form composite structure

Table 4-7 Summary of core structure

4.1.3 Space Structure

Truss / Space Frame

Space structure is defined as a 3-dimmensional structural system transferring the forces spatially, mostly formed by trusses or space frames. The structural components are assembled in triangular shapes with small scale elements. Loads go through the light weight linear elements mainly in tension and compression, so truss and space frame structure are able to accomplish large spans and variable shapes. It is seldom use d in multi-storey buildings but often in single storey large span buildings like stadiums or exhibition halls. However it might be interesting to investigate the feasibility of application in this multi-storey building project which also requires large open space.

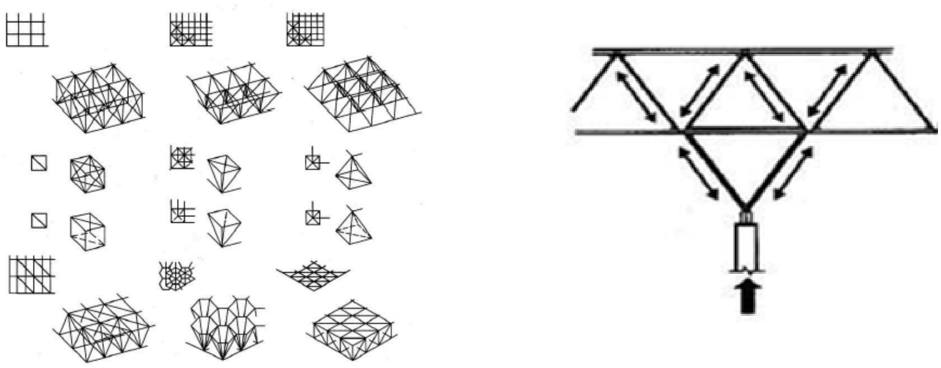


Fig 4-7 Types and forces in space frame

Fig 4-8 Space frame structure projects

Advantages	- Only tension and compression forces in the members, no bending - Suitable for large span structure - Small scale components, easy to erect
Disadvantages	- Complicated connections - Mostly used in single storey building and roof structure

Table 4-8 Summary of space structure

4.2 Floor System

The choice of floor system in the structural design stage not only influences the horizontal path of the forces, but also has impact on the construction speed of the whole project. Following common used floor system in multi-storey buildings will be described including their characteristics.

4.2.1 Prefabrication Floor

Prefabrication floor due to its elements being fabricated in the factory in advance and simple connections which results in fast construction speed is widely used in many buildings, especially those buildings with regular floor plan. The most two common prefabrication floor systems are hollow core slabs and timber floor.

Hollow Core Slabs

Precast hollow core slabs are floor elements having voids in the slabs. They have the benefits of simply connections to the other structural elements, such as columns and beams which save the costs of labor and connection material at the same time. This lowest cost floor system and the benefits mentioned above result in the most sustainable concrete floor. Moreover, the span of HCS can reach to 16 m, and the load/span ratio is very favorable.

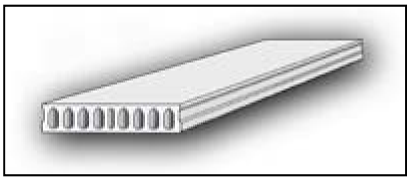


Fig 4-9 hollow core slab sample

Of course, disadvantages exist in hollow core slabs, for instance, they spans only in one direction and need linear supports. And to make all the slabs work together, cast-in-situ concrete topping is always needed to provide integrity.

Timber Floor

Timber floor system is commonly used in residential buildings for both light weight and aesthetic reasons. Traditional timber floor is laid on bearers and joists, and the price of the timber floor is determined by the quality of the timber which means how many natural variations (knots etc) there are. However, timber is a combustible material and will warp if there is no adequate ventilation so timber floor could only be laid by a skilled crew. By considering the features of timber floor, it can be conclude that timber floor system is not suitable to be used in this project, thus it will not be considered as an option for the floor system.

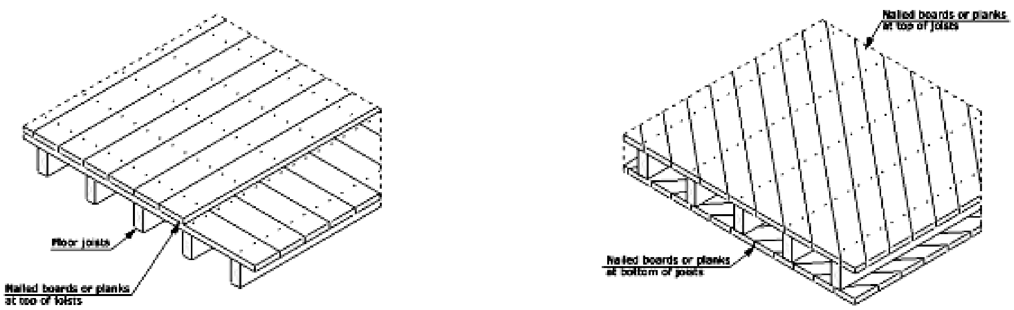


Fig 4-10 Timber floor

4.2.2 Cast-in-situ Floor

Cast-in-situ floor system is used when there is irregular plan or rigid connection required. Although it costs more time and labor during construction, and sometimes relies on the weather, it gives the best structural integrity and stiffness to the building. Concrete is the well known material that can be cast in situ.

Concrete Flat Slab

Cast-in-situ concrete floor has the form of reinforced concrete floor or prestressed concrete floor. Prestressed concrete floor compared to reinforced one because of its prestressed tendons can resist larger loads. It is always used in bridge deck so that may suitable to the bus terminal of Breda CS. Both of these two kinds of cast-in-situ concrete floor have the advantages that they are able to control the floor deflection best, and adapt to any kind of building of plan layout. While the disadvantages of cast-in-situ concrete are also obvious that it influences the speed of the construction the formwork can only be continued after the concrete is hardened.

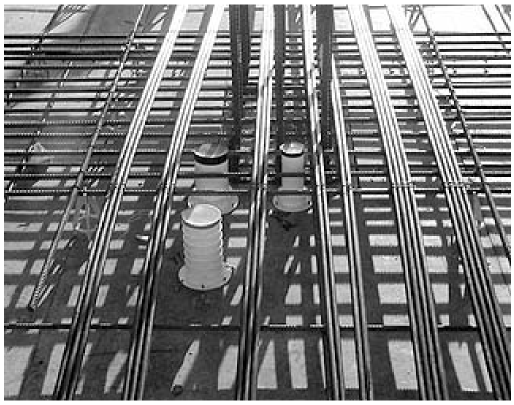


Fig 4-11 Post-tensioned concrete floor

For cast-in-situ concrete floor construction, different large panel formwork systems are widely used today by contractors in the Netherlands for constructing standardized housing or office blocks, and are broadly classified as table forms (with wall form) and tunnel forms. And compared with traditional timber formwork, metal panel formwork has several advantages.

Table Forms: Standard modules of housing blocks are relatively large in span and large table forms are widely used for assembly time reduction, fewer joints and better surface finishes. The table method uses separate vertical forms for walls and horizontal table forms for floor slabs. Normally, the table form will work together with the wall form, which is combined with the slab form so that the wall and slabs can be formed monolithically in one casting operation, and the number of joints between panels is minimized. The work is done in two stages. First, the walls are cast, and forms are stripped, the tables are then positioned, and the horizontal slabs are cast.

Tunnel Forms: The half tunnel is composed of vertical and horizontal panels set at right angles and supported by struts and props. The walls and slabs are cast in a single operation. Like the wall-forms and table forms, this reduces not only the number of joints, but also the assembly time. Therefore, the casting of walls and slabs can be completed in the one day.

4.2.3 Composite Floor

Composite floor system using different materials and forms optimizes the structural behavior of them to get a best solution for the floor.

Composite plank floor (Breedplaat in Dutch)

The principle of the composite plank floor is, rectangular slabs are laid between supports and used as permanent form-work for an in situ concrete topping. Composite action depends on the shear transfer in the horizontal joints between the precast plank and cast-in-situ concrete topping. There are several advantages and disadvantages of this kind of floor which are, it has smooth and rapid finish; the lattice girder has the function of bonding the precast and cast-in-situ concrete, providing flexural reinforcement, increasing vertical stiffness in temporary condition and etc. However, due to the combination of precast and cast-in-situ concrete, the self-weight is relative high and it required propping till the concrete topping is hardened as other cast-in-situ concrete floor. Another disadvantage of composite plank floor is that temporary supports are needed during construction which has impact on the other installations and construction time.

Fig 4-12 Composite plank floor (Dycore)

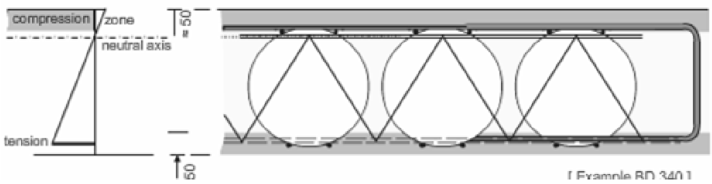
BubbleDeck

A bubble deck slab behaves like a solid slab with true biaxial behavior, which compromises a biaxial carrying hollow slab in which plastic balls acts the purpose of eliminating concrete. The floor removes the non working dead load of concrete while maintaining the biaxial strength.

Bubble deck system has lots of advantages for both architectural and structural consideration,

- Light weight, flat, open space without beams needed, reduce building height
- Large span and long cantilevers but less deflection
- Free form of shape and flexible
- Less foundation, no problem with water penetration
- Less material, and low installation and operation cost

Bubble deck could be a good solution for the office building part of Breda CS because all of its characteristics have benefits to the requirements of the project. However, the high cost for the floor system itself should be well considered and controlled.



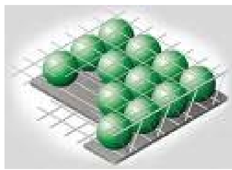


Fig 4-13 Bubble deck [6]

Slim Floor

Slim floor is one of the optimum composite floor systems which combines the steel beam, deck with the concrete floor. Aim to get the minimum floor height and protect the steel beam from corrosion and fire, steel beam and concrete deck are set in the same plane to work together and allow the horizontal mechanical services through. In this floor system, both precast concrete slabs (hollow core slabs, double tee slabs and etc) and cast-in-situ reinforced concrete slabs can be used to fulfill different floor requirements.

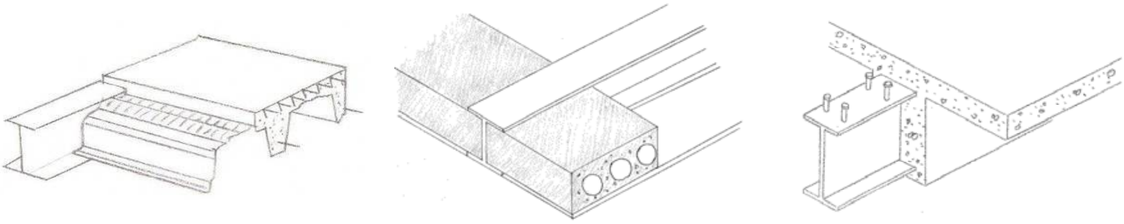


Fig 4-14 Slim Floor Samples

Steel Deck Concrete Floor

Steel deck concrete floor is composed of a steel wide flange shape beam attached to a steel deck with concrete slab above. This floor system is stronger than the separate parts by rigidly joining the composite materials together. And the composite action will better utilize the advantages of the properties of each material, the concrete is supposed to mainly take the compression forces while the steel takes the tension forces. It also solves the problem of using concrete floor in steel frames.

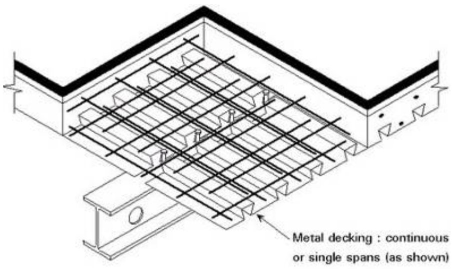


Fig 4-15 Steel deck concrete floor

Compared to the slim floor system, it has the disadvantage of larger floor thickness since the concrete floor is put above the steel beam. And shear studs are needed between the steel and concrete part to resist the shear forces. In a word, this kind of floor is merely used as composite floor system when the plan layout is irregular and the horizontal forces needs to be transferred in two directions.

4.2.4 Summary of floor system

The floor systems described above are frequently used in multi-storey buildings under different condition. From that, it can be concluded that every type of floor has its advantages and disadvantages. Some of them have light weight; some of them realize large span, and some of them can be erected fast. Therefore, it is important to determine a criterion to choose a suitable one for this project. Considering the objective of this thesis which is to achieve more space on bus level, the weight and depth of the floor has been regarded as the criteria. The following table summarizes the characteristics of the floor systems mentioned above.

Floor System	Cost	Weight (kN/m²)	Span (m)	Depth (mm)	Flexibility (mechanical voids)	Construction speed (temporary support)	Sustainability	Remarks
Hollow core slab	very cheap	2,5~5	4~16	200-500	bad	excellent	excellent	- simple connection - span in one direction - concrete topping needed
Timber floor (Prefab)	expensive	<1,5	2~8	145-450	good	good	excellent	- combustible - skilled crews needed
RC Flat slab	middle	5~10	4~12	Varies	excellent	poor	poor	- on site time and labor cost
Post-tensioned flat slab	cheap	4~8	7~15	Varies	middle	poor	poor	- small deflection - work as diaphragm - on site time and labor cost
Composite plank floor	middle	5~10	5~15	50-100 (deck)	good	middle	poor	- temporary support needed - lattice reinforcement have lots of functions
Bubble deck	expensive	3,7~7,3	7~18	230~450	good	middle	middle	- no beam needed - thin floor
Slim floor	very expensive	2,21~4,44	5~9	225 (deck) 290~360 (slab)	middle	good	middle	- thin floor - good combination between steel and concrete - no shear studs needed
Steel deck concrete floor	very expensive	1,78~5,61	5~9	110~400 (deck)	middle	good	middle	- thick floor - good combination between steel and concrete

Table 4-9 Summary of floor system

5 Alternatives

In this Chapter, several alternatives will be described as the structural solution for the north side of Breda Central Station. The main goal of these alternatives which is also the objective of the project are to realize large span in the offices or bus terminal, so as to reduce the structure and supports on the bus level as a consequence of more open space and clearance, and to save the use of amount of the materials at the same time. Each of them will be described in concept.

5.1 Alternative 1 - Arch support structure 1

Goal of this alternative
Inspired by arch bridge, the alternative was planned to reduce the number of columns on the bus terminal level by using large span arch structure.

Structural Geometry
Since there is limited space and height on the bus level, the supported deck type and suspended deck type appealed themselves as inefficient in this case. Therefore, the arch was designed at the office part to realize the large span. The plan was to design a bridge that could withstand the dead load and live load of a 3 storey office building.

Fig 5-1 the broadgate exchange house, UK (courtesy of SOM)

Four longitudinal arch groups spacing 7.2m, 13m and 6.2m are tied by transverse floor girders. The span to depth ratio of the broadgate exchange house is 87m to 7-storey high, for a reasonable estimation, the span of the arch in this project could be approximate 40m. Thus, 6 arches, each spans 46.2m, are arranged in each group. These 24 arches carry the load down to 48 supports at level 2. The parabolic shape has been selected for the arch as it's the most efficient shape for the uniform loading configuration. The arches are segmental components of wide flange beams with a fixed connection every 4.2 meters. A series of equal point loads are imposed on the arches by vertical hanger columns at 4.2m spacing. The angle of the arch which is,
$\bar{y} = -\frac{10.8}{23.1^2} * 2x = -\frac{10.8}{23.1^2} * 2 * (-23.1) = 0.935 \rightarrow \alpha = 43.07°$.

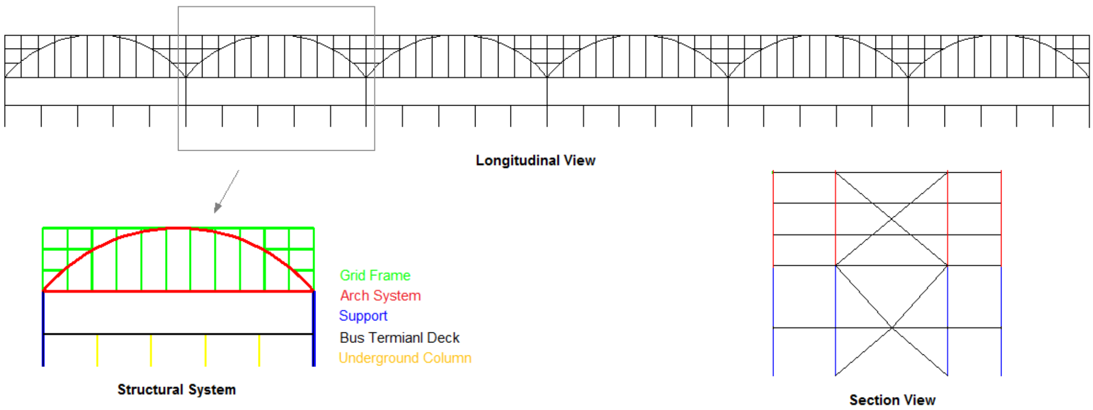


Fig 5-2 Elevations and Structure (Alt.1)

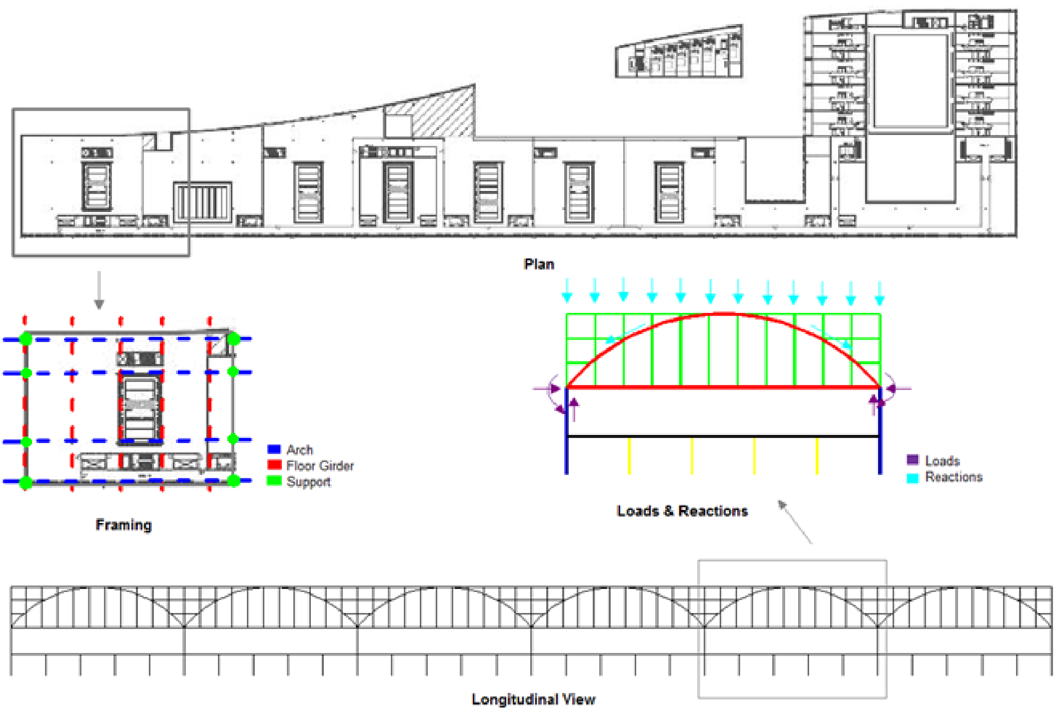


Fig 5-3 Framing and Loads (Alt.1)

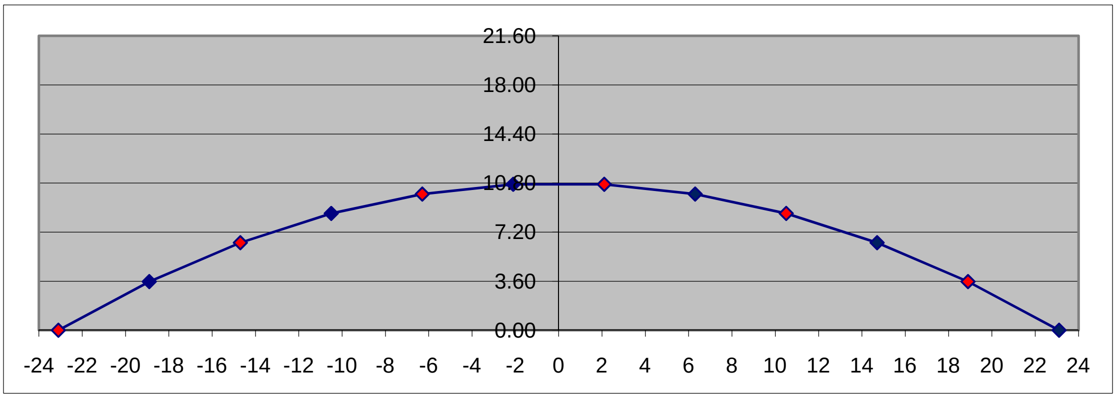


Fig 5-4 Column spacing of the arch

Force distribution and Stability

The vertical load proceeds from the floor beams to the slender columns, to the arches and finally to their supports. The arches match the moment diagram for uniform point loads and these loads are carried as axial compression forces in the arch with a minimum flexural bending. The thrust of the arches is supposed to be taken by the ties at the bottom of the arches. Vertical trusses locating at the sections of the end of the arches provide lateral stiffness under horizontal loads, and also resist the out-of-plane movements of the arches. The floor diaphragms transfer the shear forces to these trusses to guarantee the stability of the whole structure. The underground structure utilized rigid frame system with the spacing of 9.24m to support the bus decks, and provide stability for the lower part of the building.

Estimated Loads:	$w = (1.2 \times 5.0 + 1.5 \times 2.5) \times 10.5 = 102.375\ \mathrm{kN/m}$
Moment:	$M = wl^2/8 = 102.375 \times 46.2^2/8 = 27314\ \mathrm{kN \cdot m}$
Horizontal Reaction:	$H = M/h = 27314/10.8 = 2529\ \mathrm{kN}$
Vertical Reaction:	$R = wL/2 = 102.375 \times 46.2/2 = 2365\ \mathrm{kN}$
Internal Normal Force:	$F \approx 3039\ \mathrm{kN}$ (by STAAD Pro)

Supports
The arches carrying heavy loads are supported on large composite bearings at bus level so that these supports are able to act as springs which permit lateral, rotational and some vertical movements. Due to large compression forces going to the supports and foundations, all supports have to resist horizontal loads and preventing moments as well resulting in larger scale and higher cost. The supports on the bus level transfer the loads to the foundation eventually.

+ Advantages
- Provide more space in bus terminal and underground area;
- Aesthetic view, fulfill the requirement of being the landmark of Breda

- Disadvantages
- Large arches and supports required because the arch structure has to bear heavy loads;
- Have to prevent moment from arch;
- More attention should be paid to the connections and supports of this composite structure;
- The spacing of the columns forming point loads on the arch is considered to use 4.2m, which results in small grid in the longitudinal direction;
- Differ from the architectural design

⊿ Difficulties
- Intentioned arch action;
- Arches have to support both upper office part and withstand the loads on the bus terminal at the same time which results in large scale;
- The span and distance between the arches are very large as well;
- The supports and connections are the most important and cost parts;
- The scale of structural elements (arch, support, etc) may be large;
- Arches may unstable under asymmetrical loads and cause in-plane buckling

5.2 Alternative 2 - Arch support structure 2

Goal of this alternative

To create larger column-free space in transverse direction at bus terminal level, also utilize the aesthetic appeal by using arch as alternative 1, but to reduce the number and span of the arches.

Fig 5-5 Moscone Convention Center, USA

Structural Geometry

In this alternative, arches were arranged in transverse direction, setting 16.8m apart along the 277.2m length of the building. Each arch was up to approximately 7.2m high at its crown and spanned 26.4m across the bus terminal to get column-free space there, and meanwhile supporting the bus terminal and the office area above. Double tee slab is a light weight solution to long span concrete floor. So it was used for the floor system in this alternative, spanning 16.8m in longitudinal direction, in accordance with the arrangement of the arches. This also created more flexible space for the offices above the bus terminal. Slender columns were designed to connect between the arch and above office to realize uniform loads on the arches. The underground structure remained the same rigid frame structure as alternative 1.

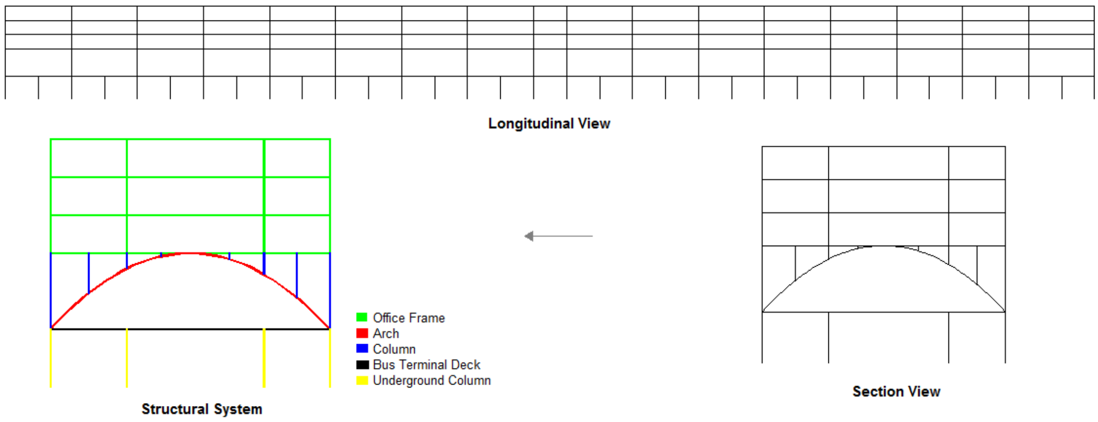


Fig 5-6 Elevations and Structure (Alt.2)

Force distribution and Stability

The vertical forces would be firstly transferred from the floor slabs to the beams and then the columns in the offices. After that, the columns connected the offices and arches would withstand the forces to the arches, and eventually to the underground. These arches was planned to be made of prestressed concrete or steel to carry the heavy loads from 3-storey offices. The office structure was designed as light weight structure by steel. The horizontal wind load would be carried by the columns and arches instead of floor diaphragm.

Estimated Loads:	$w = (1.2 \times 5.0 + 1.5 \times 2.5) \times 10.5 = 102.375\ \mathrm{kN/m}$
Moment:	$M = wl^2/8 = 102.375 \times 26.4^2/8 = 8919\ \mathrm{kN \cdot m}$
Horizontal Reaction:	$H = M/h = 8919/7.2 = 1239\ \mathrm{kN}$
Vertical Reaction:	$R = wL/2 = 102.375 \times 26.4/2 = 1351.35\ \mathrm{kN}$
Internal Normal Force:	$F \approx 1520\ \mathrm{kN}$ (by STAAD Pro)

Supports
Due to large compression forces going to the supports and foundations, all supports have to resist horizontal loads and prevent moments as well, resulting in larger scale and higher cost. But the situation might be better compared to alternative 1.

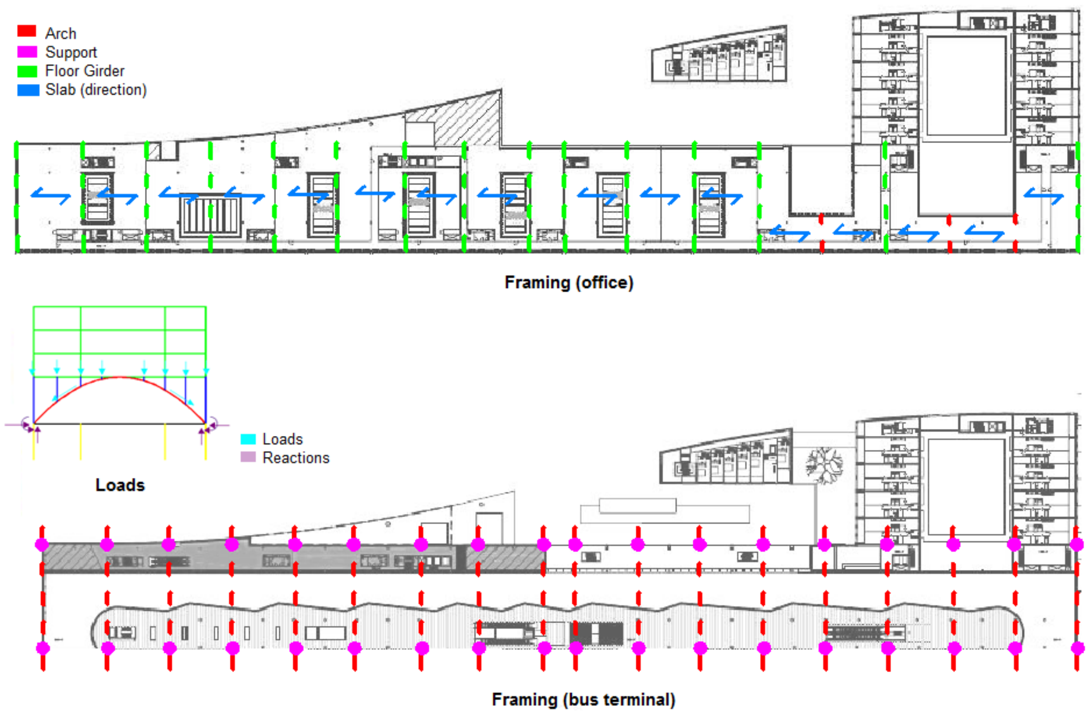


Fig 5-7 Framing and Loads (Alt.2)

+ Advantages
- Provide more space on the bus terminal and offices;
- Aesthetic view, fulfill the requirement of being the landmark of Breda
- Reasonable arch spans and distances

- Disadvantages
- Uniform point load pattern cannot perfectly achieved in this alternative;
- More attention should be paid to the connections and supports of this composite structure;
- Transverse arches will disturb the function and cause unusable area;
- Also differ from architectural design

⊿ Difficulties
- Intentioned arch action has to be avoid;
- Arches have to support both upper office part and withstand the loads on the bus terminal at the same time;
- The supports and connections are the most important and cost parts;
- The scale of structural elements (arch, support, etc) may be large;
- Arches may unstable under asymmetrical loads and cause in-plane buckling

5.3 Alternative 3 - Frame with braced core

Goal of this alternative
Reduce the columns in the offices and bus terminal and realize pin-connected frame structure by utilizing the atria (openings) as stabilizing cores which provides main stability for the whole structure.

Fig 5-8 Da Vinci, NL

Structural Geometry
Stabilizing cores are always chosen to be used in frame structures to provide the stability. There are two widely used forms of stabilizing core which are composed of shear walls or frames with bracings together. The purpose of the architectural design of several openings was to let more daylight into the building, so that closed shear wall cores were not suitable. As a solution, braced frame as an alternative was designed to work as the cores around the openings. The rest of the structure was the frame around the façades that got larger inside space as well. Due to the large open space in the offices, primary (longitudinal) and secondary (transverse) beam were used to transfer the load to the core and frame structure.

Force Distribution and Stability
The braced cores working as stabilizing system provided stability in both directions for the structure. The vertical loads were transferred from the floors to the beams, and then to the core and frame columns downwards. The horizontal loads were also transferred from the facades to the cores and eventually down to the foundation. The connection in the frame structure of the offices was designed as hinged joint which made use of the benefits of the stiff cores. This simplified structural behavior of the frame structure , and the simple connections saved the construction time and labour at the same time.

However, due to the functional requirements on the bus terminal that no disturbrance is allowed along the bus route, which means that all the stabilizing cores are not able to reach the bus level or foundation. The massive forces in these cores would result in large columns to support them. This strongly weakened the effect of the core system and the goal of the project.

+ Advantages
- More stable than pure frame structure;
- Pinned connections can be used in frame part;
- Reduce the amount of columns inside to get larger space

- Disadvantages
- Less flexible at core position for future renovations;
- To avoid disturbing the bus terminal, braced cores are not able to reach the bus level or foundation. Large moments occurs in the support columns which will lead to large scale of these columns

⊿ Difficulties
- Find a solution that will neither disturb the bus terminal nor cause unpleasant structural behavior;
- Determine the numbers and geometry of braced cores;

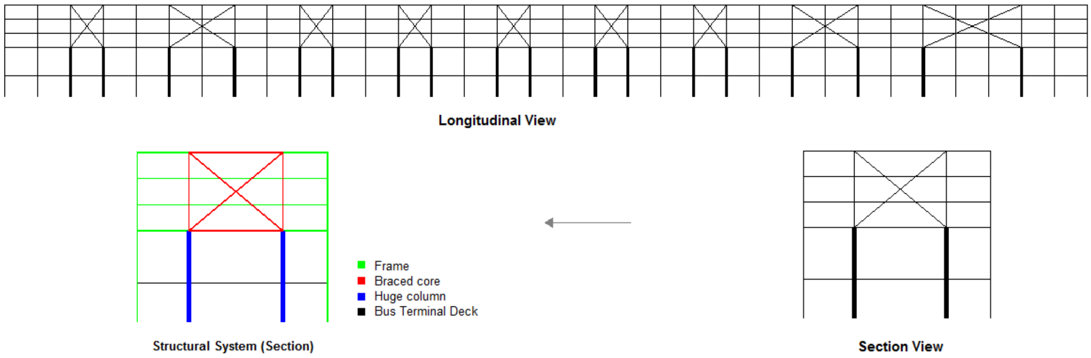


Fig 5-9 Elevations and Structure (Alt.3)

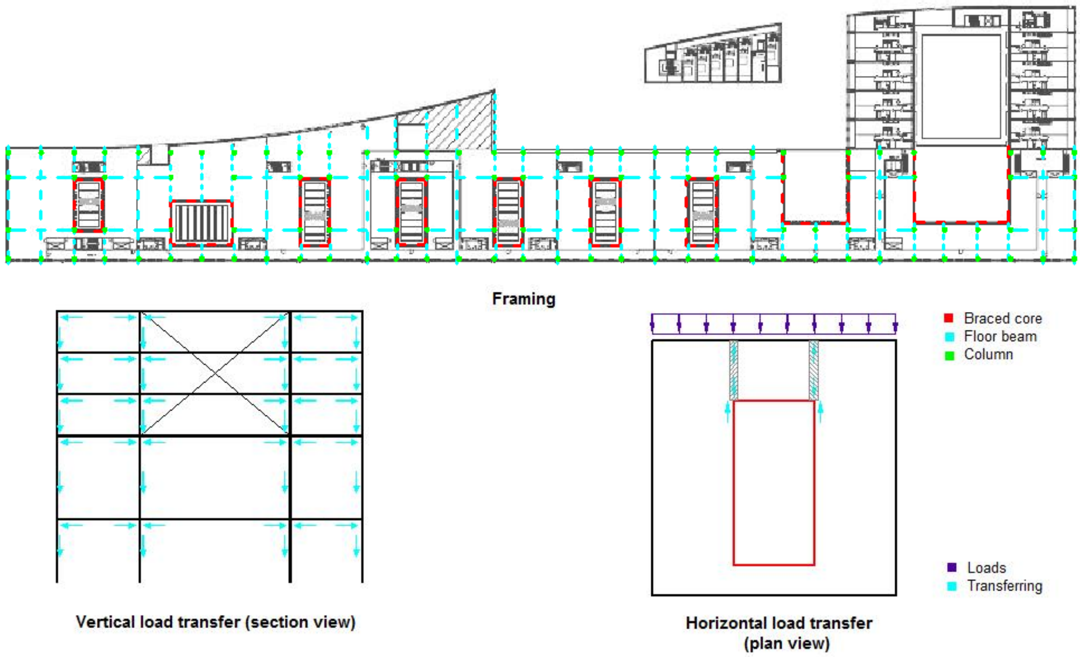


Fig 5-10 Framing and Loads (Alt.3)

5.4 Alternative 4 - Truss Structure

Goal of this alternative

To diminish the columns on the bus level by using truss structure with large span in the offices. And also to create a light weight structure with least influence on the architectural view.

Fig 5-11 Berlin Central Station

Structural Geometry

To reduce the columns on the bus terminal means to reduce the support of the office part. Normally the span-to-depth ratio of the truss structure is approximate 10-15, so that for a span of 25.2m, the truss would be 1.68-2.52m deep. There is limited floor-to-ceiling on the bus terminal, therefore, it was decided that the truss would be designed over 3-storey office. The 25.2m span trusses would locate longitudinally, spacing 7.2m, 13m and 6.2m according to the architectural design. Light weight floor system was going to be used to make the office part as light as possible. According to the truss span, support structures on the bus terminal located every 25.2m as well. Ideas like tree column structure were designed to reduce more columns on that level.

Force Distribution and Stability

The forces of the truss structure are assumed to be applied to the joints only, but not long the members. So each member of the truss structure is in compression or tension only, shear, bending moment and other stresses are considered as zero practically. The flow of the vertical loads firstly transfers from the transverse floor girders to the trusses, and secondly down to the supports on the bus terminal and underground structure, and finally to the foundations. The floor girders fixed to the columns alongside them forms rigid structure to resist the horizontal loads and provide lateral stability. The longtudinal stability will be provided by the trusses.

+ Advantages

- Trusses increase the distance between the supports, so the number of the supports reduces within the certain length;
- The tension and compression only truss members results in simple structural behavior;
- Less structural and construction disturbance at bus level ;

- Disadvantages

- Connections and joints become difficult and expensive;
- Scale of structures on the bus level could be huge;
- Steel structure is expensive in the 'first' cost;
- Structure has to be fixed transversely to resist moment and provide lateral stability;

⊿ **Difficulties**

- Determine the loads on trusses and the scale of them;
- Design of the structure on the bus level to support the trusses

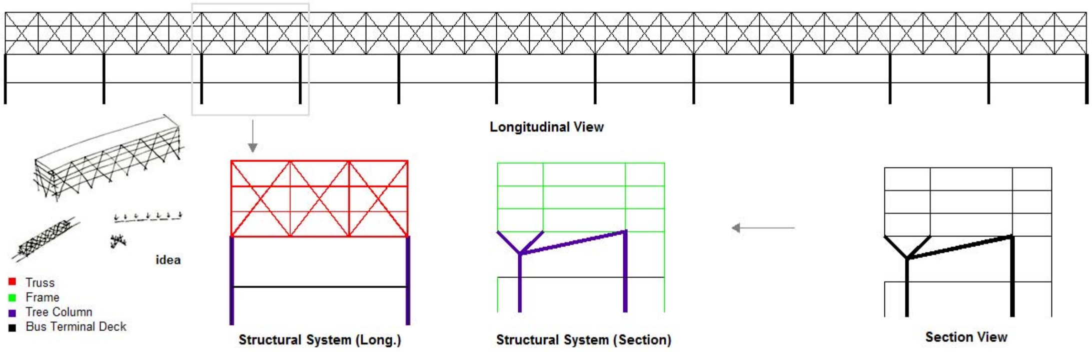


Fig 5-12 Elevations and Structure (Alt.4)

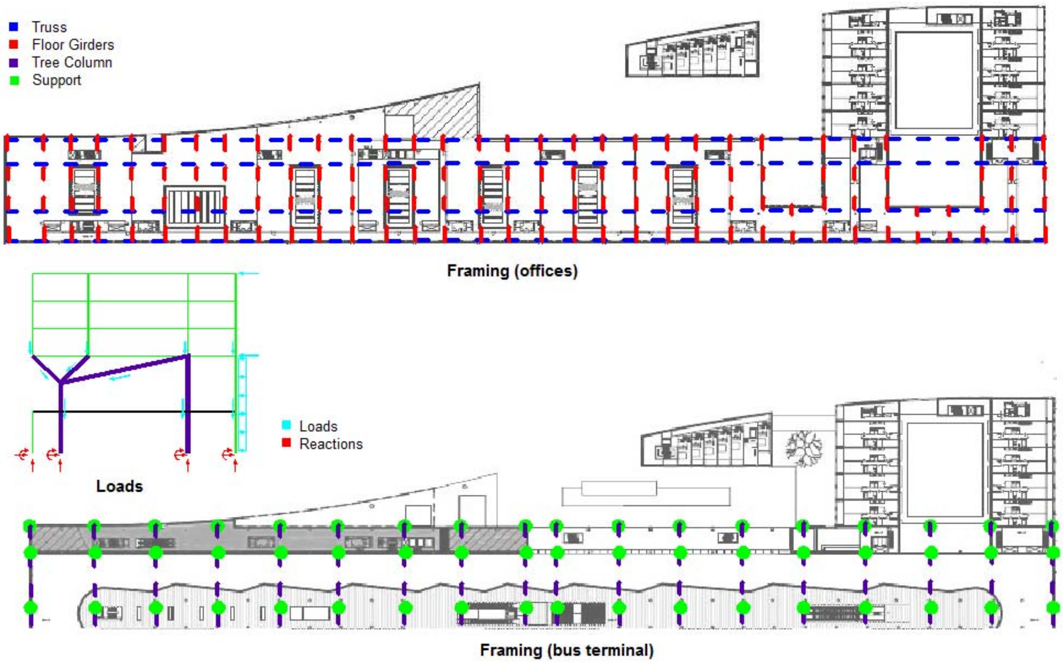


Fig 5-13 Framing and Loads (Alt.4)

5.5 Alternative 5 - Space frame support structure

Goal of this alternative

To create large open space on bus terminal with fewest supports by using space frame structure.

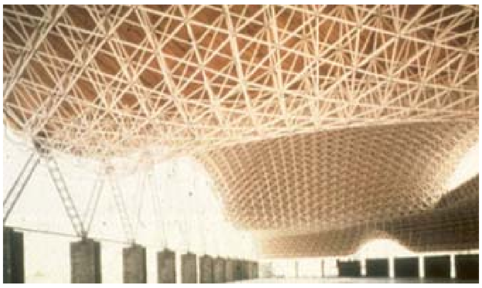


Fig 5-14 Palafolls Sport Hall, Spain

Structural Geometry

The above office block remained as the frame structure, while the space frames which located at the bottom of it would free the space of bus terminal of columns. In order to have fully open space for the bus terminal, the dimension of the space frame was 26.4×277.2m which was divided into 10 parts, 7 parts with 26.4×25.2 m and 3 parts with 26.4×33.6m at the end of the building. The common span-to-depth ratio of space frame is 15:1, so the depth of the space frame is suggested to be 1.8m. However, due to the heavy loads from the above office applied on the space frame, the depth was then considered to be 2.4m. And from the top view, the grid size was designed as 2.4×2.8 m. Large columns on the bus level and underground would support the space frame and above offices.

Force Distribution and Stability

The vertical loads firstly transfer along the frame structure to the bottom of the office block, and then loads go into the nodes of the space frame beneath without bending and shear. After that, forces in the space frame will be transferred to four corner supports at each space frame and then to the columns and the foundation in the end. The bus level will be supported by the additional columns at the underground level which means the office block and bus terminal are structurally separated. The horizontal loads are resisted by the columns in the offices alongside the space frame. The space frame provides the stability in both directions.

+ Advantages

- Free space without columns under space frame;
- Space frame transfer the loads in both directions;
- In spite of few supports at bus level, office has no need to use large span

- Disadvantages

- Space frame bear too heavy loads (office block);
- Less flexible for future renovations;
- Difficulty and high cost in connections between space frame and other parts;
- Heavy self weight

⊿ Difficulties

- Control the structural behavior of the space frame, e.g. deflection, buckling;
- Space frame is usually used as roof structure for large span; it is rare that other structures lay on it, special attention has to be paid;
- Design of joints, especially the connections between space frame with above offices;

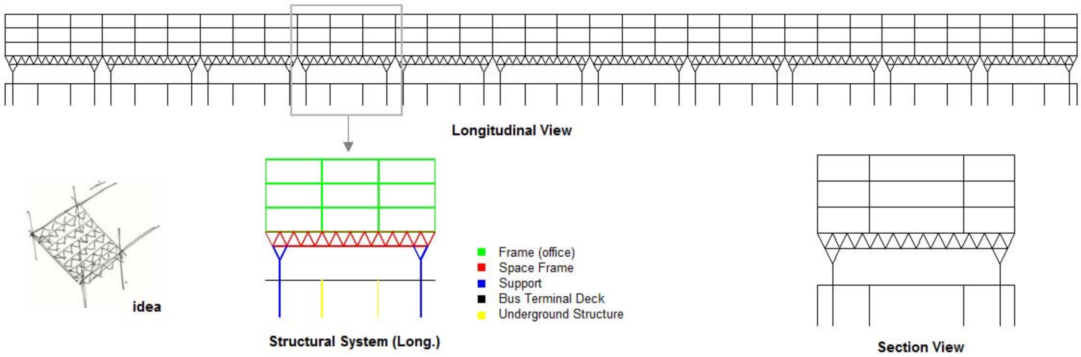


Fig 5-15 Elevations and Structure (Alt.5)

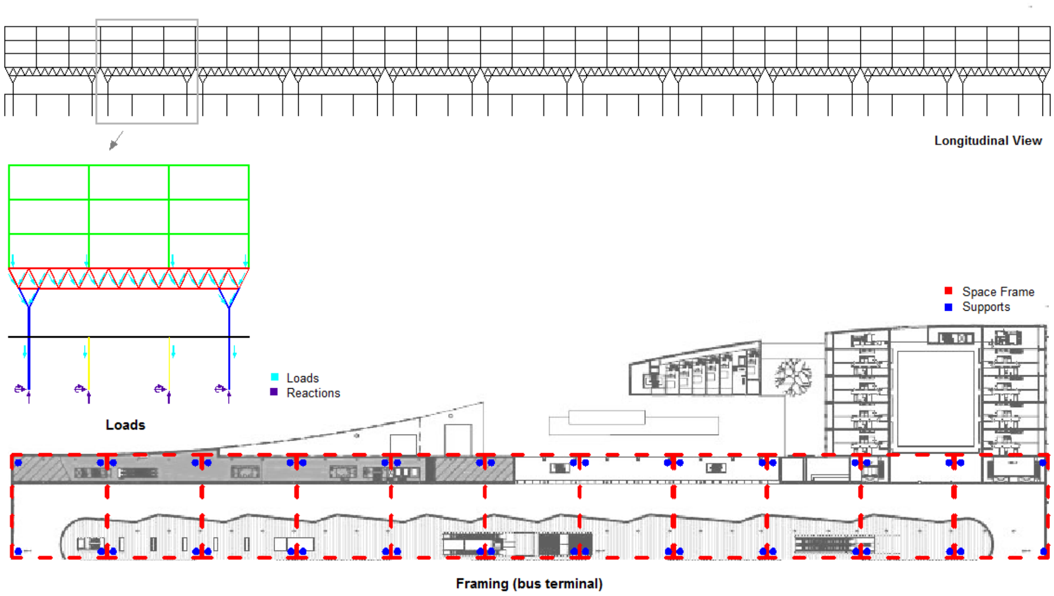


Fig 5-16 Framing and Loads (Alt.5)

6 Selection of Alternatives

In order to choose a suitable alternative for the final design, all the alternatives generated above will be evaluated according to MCA (multi criteria analysis) as the guidance. In this chapter, all the alternatives will be evaluated based on several criteria and after that, the alternative with highest score will be chosen as the highly recommended proposal of this project.

6.1 Multi Criteria Analysis

Multi Criteria Analysis is based on a series of considerations and decision factors which values each of them by different parties. It is one of the best and common used ways to compare and choose alternatives.

Five main criteria: cost, aesthetics function and structure and technique are considered as the most important influence factors for a building project. However, different parties which are client, architect, engineer, contractor and users care different criteria from their point of views. The follow table shows which criteria they care most when selecting.

	Cost	Aesthetics	Function	Structure	Technique
Client	×	×			
Architect		×	×		
Engineer				×	
Contractor					×
Users		×	×		

Table 6-1 Criteria to different parties

The alternatives were then evaluated by different parties. Each of them weighted the criteria they concerned mostly for all the alternatives, for instance, the client weighted Cost and Aesthetics while the architect weighted the Aesthetics and Function. The mark ranged from 1-5 from weak to strong.

Client	**Cost**			**Aesthetics**			**Total**
	Material	Construction	Operation and Maintenance	Shape	Material Appearance	Facades	
Alter. 1	3	3	4	4	4	5	23
Alter. 2	3	4	4	4	4	3	22
Alter. 3	4	4	5	3	3	3	22
Alter. 4	4	4	4	4	4	4	24
Alter. 5	3	5	4	4	3	4	23

Architect	**Aesthetics**			**Function**			**Total**
	Shape	Material Appearance	Facades	Area Flexibility	Clearance of circulation	Physical requirement	
Alter. 1	3	3	3	4	4	4	21
Alter. 2	3.5	3	3.5	3	3	4	20
Alter. 3	3	3.5	3.5	4	4	4	22
Alter. 4	3	4	3	4	3.5	4	21.5
Alter. 5	3	3.5	4	4	3.5	4	22

Engineer	**Structure**					**Total**
	Stability	Vertical/Horizontal Stiffness	Structure weight/Foundation mass	Connection aspect	Mechanical Services	
Alter. 1	3	4	4	4	4	19
Alter. 2	4	5	4	4	4	21
Alter. 3	2	4	4	4	4	18
Alter. 4	4	4	5	4	5	22
Alter. 5	4	5	2	2	4	17

Contractor	**Technique**			**Total**
	Construction Time	Labor Requirement	Difficulty of connection	
Alter. 1	3	3	4	10
Alter. 2	3	3	4	10
Alter. 3	5	4	4	13
Alter. 4	5	4	5	14
Alter. 5	4	4	3	11

User	**Aesthetics**			**Function**			**Total**
	Shape	Material Appearance	Facades	Area Flexibility	Clearance of circulation	Physical requirement	
Alter. 1	4	4	4	4	4	5	25
Alter. 2	4	4	3	5	4	4	24
Alter. 3	3	4	3	3	4	4	21
Alter. 4	4	4	4	5	4	4	25
Alter. 5	3	3	4	5	4	3	23

Table 6-2 Alternatives valued by different parties individually

After determining the value of each alternative by different parties involved in the project, the total scores were added together to know the most welcome one. From the result, the 4th alternative, truss structure, got a relative high score than others. Attentions have to be paid that the result of the MCA just shows a relative optimal alterative among all the parties, however, the main purpose of this thesis is the structural design, and therefore, this will be only regarded as a guidance of the selection. The reason of the final choice will be analyzed in next section.

	Client	**Architect**	**Engineer**	**Contractor**	**User**	**Total**
Alter. 1 Arch 1	23	21	19	10	25	98
Alter. 2 Arch 2	22	20	21	10	24	97
Alter. 3 Braced cores	22	22	18	13	21	96
Alter. 4 Truss	24	21.5	22	14	25	**106.5**
Alter. 5 Space frame	22	23	17	11	23	96

Table 6-3 Total score of every alternative

6.2 Conclusion and selection

Alternative 1: Arch support structure 1

The first alternative seems to be a creative concept since arches along longitudinal direction provide more space on the bus level because of the large span realized. And the results of the MCA also indicated that this alternative was appreciated by client and users mostly. However, disadvantages and difficulties appear in this alternative as well. It is not that workable because it causes relative large disturbance of the architectural design. And from structural point of view, arch structure works well under uniformly distributed loads than under concentrated loads, although solution has been made to this disadvantage by using slender columns transferring the loads to the arches evenly. According to the scale of the building, heavy loads will result in large horizontal forces in the supports; the size of the supports will then become larger. Moreover, the influence of the asymmetrical loads on arch structure is large and will probably cause in-plane buckling. It can be summarized that there are more disadvantages and difficulties than advantages in this alternative, so this alternative will not be elaborated.

Alternative 2: Arch support structure 2

Besides realizing smaller span of the arch, the goal of this arch structure was to get totally column-free space on the bus terminal. When turning the arch direction from longitudinal to transversal, the span and distance of the arch become smaller so that the scale of the arches could decrease. Thus, from this point, and also from the rough calculation of the force distribution in alternative 1 and 2, the second one is prior to alternative 1. However, other problems arise from this structure. In order to achieve uniformly distributed loads on the arches, columns have been designed to connect the arches and the office above it; it causes negative impact on the view of bus terminal. And the in-plane stability of the arches is also a weak point of this alternative. In a word, this alternative realizes large column-free space on the bus terminal and aesthetic architectural appeal, however negative structural behavior reduces the value of this alternative, so the same consequence is that this alternative will not be selected and elaborated.

Alternative 3: Frame with braced cores

The result of MCA shows that this alternative is valued by low score, especially by the engineer. The goal of this alternative is to utilize the atrium along the building as stabilizing cores to provide the stability of the structure and then could simplify the rest frame structure to be hinged connected. And it also complies with the architectural design by using the bracings. However, the difficulties such as the vertical location of these stabilizing cores and influence on the bus terminal are the main decision factors. The braced cores at these atriums have promising effects on the office structure but it is not possible to extend them to the lower bus level where no disturbance is allowed. Thus, the columns supporting the braced cores on the bus level will have to bear large bending moment which results in large dimensions. Therefore, these discontinuous stabilizing cores are not good structural solution in this project. These points decreased the practical value of it, and therefore this alternative will not be chosen as well.

Alternative 4: Truss structure

The alternative of truss structure was regards as a promising alternative from the results of MCA and my opinion. The truss structure sets out the advantages of itself and preponderance over other alternatives.

One of the main advantages of it is the office structure realizes large span by using truss which reduces the number of the columns and other support structure on bus terminal. This fulfills the goal of the alternative and the project, and results in light weight structure too. Of course, it has to be investigated how light it can realize.

A second big advantage is, compared to the arch structures in alternative 1 and 2 which also achieve large span in the offices, truss structure has more favorable behavior than them because the principle of truss structure is that bending will not governing the structure since the truss members only bear tension and compression, while the arch has more complicated structural behavior and is very sensitive to the loads applied on it. So it is interesting to make more detail calculation to study the efficiency of the force distribution of this alternative.

Moreover, the trusses longitudinally along the building provide the stability in that direction, and the stability in the other direction is supposed to be provided by rigid connected columns and the structure on the bus terminal. This is also considered as an advantage compared to the last alternative, space frame, although space frame provides stability in both directions and achieves large span on the bus level, it leads to heavy self weight and cause structural difficulties of supporting massive load above it.

From the MCA results, the truss structure only got relative lower score by architect party compared to alternative 3 and 5, the reason might be the truss diagonals would probably cause some effect on the office facades. Nevertheless, from my point of view, this will not produce large influence on the function and other aspects of the design so that it is possible to persuade the architect. For this master thesis, it is still a quite interesting alternative that will be elaborated.

Alternative 5: Space frame support structure

The space frame alternative tried to implement the large span and flexible area on bus terminal which was welcomed by the parties, such as architect, client and users, who mainly focus on the aesthetics and function aspects. However, this was not appreciated by the technical parties because space frame has heavy self weight and will increase the difficulty of erection. And another important influence factor is, space frame structure as introduced in the former chapter is normally used in the roof structure which mainly bears the self weight only. But the space frame in this alternative also has to support a three-storey office block; this will probably cause unexpected effect on the structure and result in large scale of elements. The depth of the space frame is likely to be great to several meters or even one-storey high, and the total height of the building will become larger consequently. (An estimated calculation of the space frame with relevant conclusions can be found in appendix 4). Considering these aspects, this alternative will not be chosen either.

In short, the fourth alternative, truss structure, is going to be elaborated as the new structure of Breda CS.

7 Case Study

In this chapter, five relevant projects with large span truss structure or huge columns bearing massive loads would be studied. The inspiration and significance to the design of this project would be concluded afterwards.

7.1 Relevant Projects

Berlin Central Station

The newly built Europe's largest train station – Berlin Central station has a 320 meters long glass building running from east to west crossed by a 160-meter long and 40-meter wide passenger building which runs from north to south. Besides its extremely long structure and huge amount of glass used, the construction of the new station which could even not stop operation for several days was a big challenge for both the designers and contractors. Thus a very important step in the construction of this new railway station was the steel skeletons of two office bridges being tilted from their erection position to the final horizontal level.

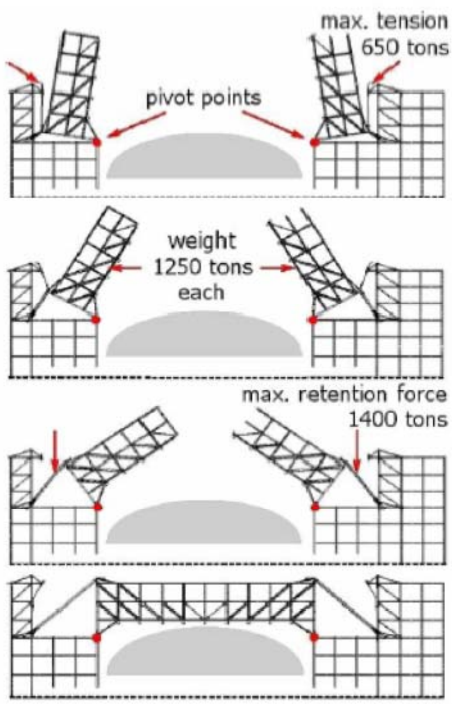


Fig 7-1 Sequence of work (by VSL)

The bridge accommodates four levels of offices and they span 87m over the glass roof covers the railway lines and platforms. The owner imposed that the assembly work could not be done over ongoing rail traffic. This limit the structural solution for the offices over it. The main contractor opted for assembling each bridge in two halves in the vertical position on top of the four office towers adjacent to the station area. The four bridge halves had to be tilted to horizontal position within two weekend period. Large truss structures were used in the office part that achieved the large sapn over 87m across the passenger building.

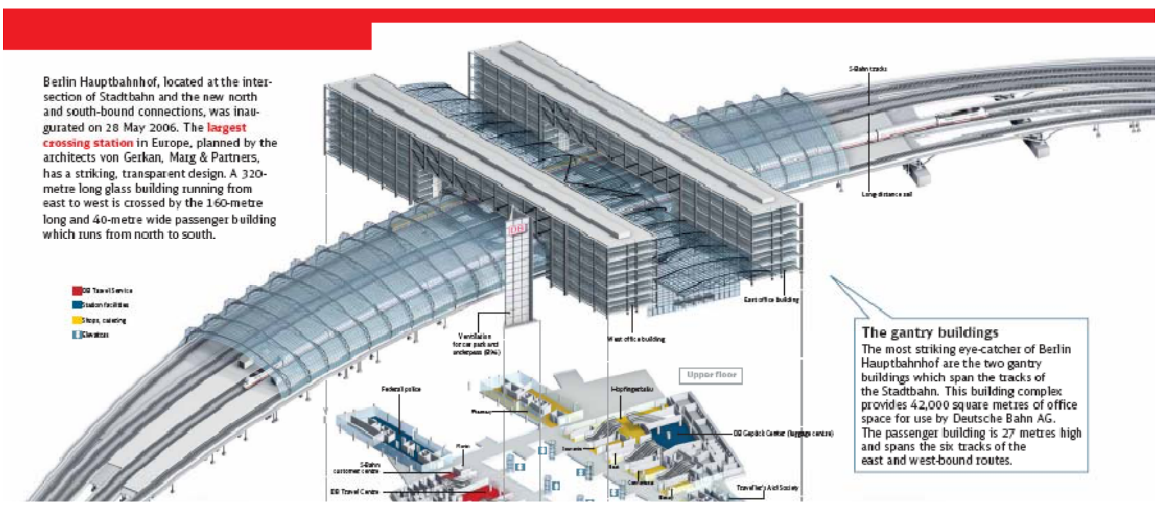


Fig 7-2 Berlin Central Station(by DB station & service AG)

De Brug, Netherlands

De Brug, Rotterdam a very special office building because it is located beyond historical buildings of an international company, namely Unilever, at a level of 17 m beyond ground level. This historical building is a factory and an office building. The factory is closed for a very short time for maintenance during the erection time. Again limited by the sourroundings and existing conditions, the building could not run the risk to be built directly on the factory below it. In addition, there were not many possibilities to support the building vertically so that long spans were necessary. And the building was decided to be built in a temporary position and then be transported to the definite location after that. The big spans were realized by four truss beams over four floor levels, two located in the facades and two located inside of the building. The dimension of the large truss beam was 130 meters long, 32.4 meters wide and 14.4 meters high. To result in a light weight structure, floor system with steel floor beams, steel decks and concrete was used.

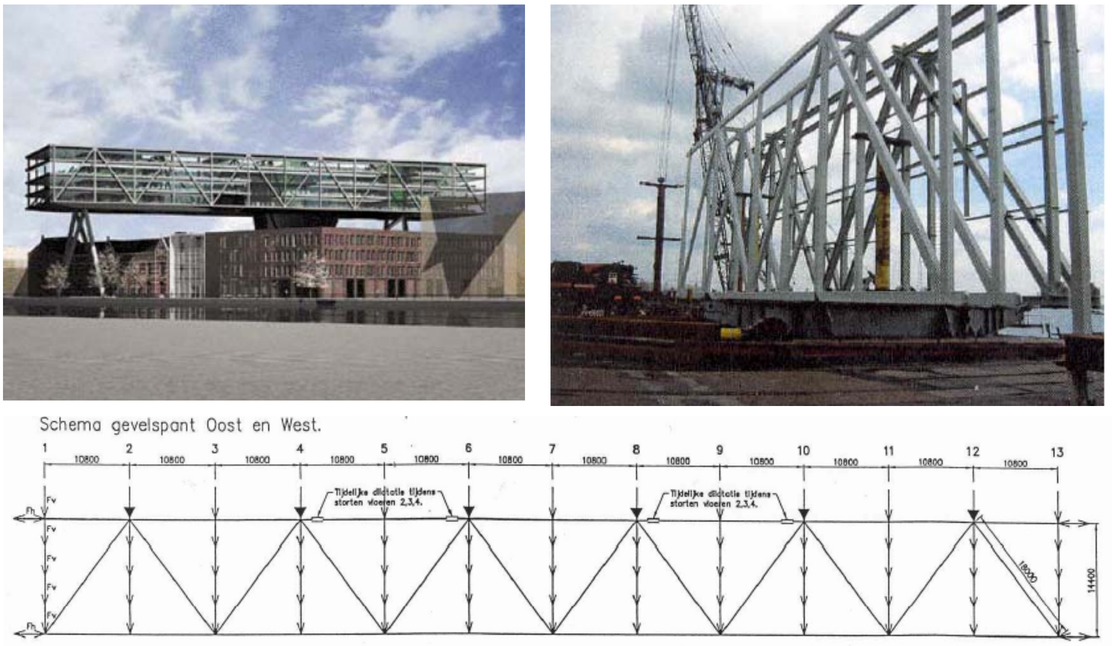


Fig 7-3 De Brug, Netherlands [23]

The building was built up next to its definitive location at a parking place on temporary supports, but also on definitive supports. To transport the complete building to its definitive location, a special rail track was realised around the factory. From the building yard to the definitive location the building was transported first with special transport lorries and the last part at one side on these lorries and the other side on the rail track. The transport was done during a factory stop in a weekend.

Port Authority Bus Terminal, USA

Located in the heart of New York City, the Port Authority Bus Terminal is the world's busiest bus terminal, the region's primary ground transportation facility, and the largest bus terminal in the United States. Opened on December 15, 1950, the terminal is located one block west of Times Square, occupying the blocks between Eighth and Ninth Avenues, from 40th to 42nd Streets. It is an integral part of the revitalized Times Square and theater district areas, and a vital connection for the region's workers, travelers and visitors. To realize the free space in the bus level, large truss structure was used as well.

Fig 7-4 Port Authority Bus Terminal, USA

Hotel du Departement, France

The Hotel du Departement (Regional Government Centre) in Marseilles, France could be read as an amalgamation of at least four distinct architectural forms. The most obvious contrast between forms occurs within the first three level of office blocks where exposed three-storey X-columns align longitudinally along each side. They dominate the lower storeys both on the exterior and in the atrium. These were described by one reviewer that their unexpected geometries ricocheting through the glazed atrium like sculptures. While the structural form of this does not relate to any other architectural view, they function as transfer structures for gravity loads. The X-columns locate on a 5.4m office module at third floor level and extend to a 10.8m grid at ground floor level to enlarge the basement for car parking. The architects deliberately expose the X-columns on the exterior by moving the buidling envelop into the building behind the structure. And these X-columns enrich both the interior and exterior of the building.

These X-columns inspire the idea of the thesis project that extra large columns might be used to support the three-storey offices above the bus terminal. And at mean time, it simple geometry would enrich the architectural view.

Fig 7-5 Hotel du Departement, France

Southern Cross Station, Australia

The Southern Cross Station in Melbourne has metamorphosed into a modern aiport sytle facility that operates as a catalyst for urban renewal. Similar to the other relevant projects, the redevelopment of the southern cross staion was undertaken while maintaining the station as a fully operational facility throughout all stage of the works. This imposed significant restrictions for the construction methods available and the working hours during which various activities could be carried out. The roof system acted as an one-way system during the construction, enabling erection over the operating station with minimum use of temporary propping. In the end, the structure utilizes the wave-form geometry to act as a series of domes effectively spanning two-way to the columns, achieving greater design efficiency.

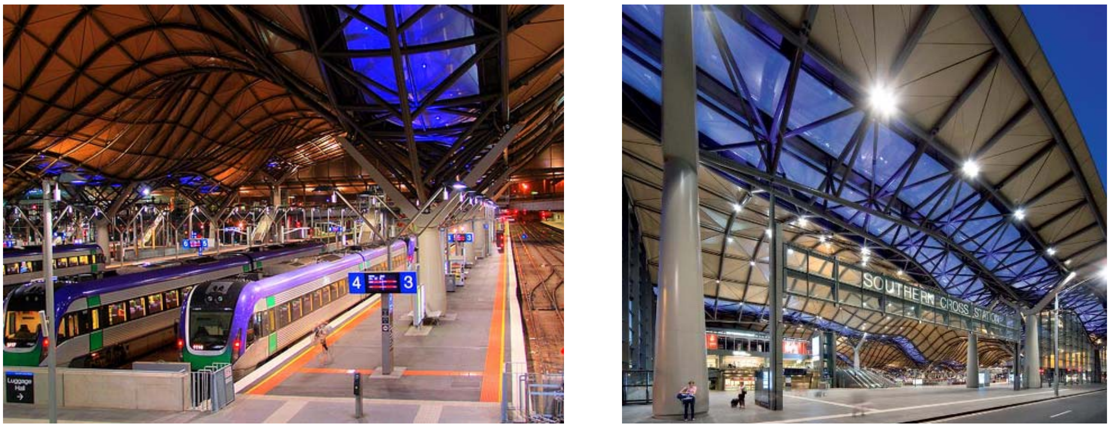


Fig 7-18 Southern Cross Station, Australia

The main spine truss running north-south above each alternative platform support arches at four meter centers spanning in the east-west direction up to 40 meters. The complex wave-form geometry relies upon the stiffness of each of the individual roof members, footing and columns to carry the loads and provide lateral stability to the total roof. The spine trusses were connected to the columns which have a fixed end by a rigid pile cap and group of piles. The columns are rigid in both direction to form a stable system.[1]

7.2 Summary

All these projects have similar traits either realizing large span by truss structure or huge columns bearing heavy loads. By studying the structural concept of these railway station, bus terminal and building projects, several conclusion can be made to inspire the design of Breda CS.

- The construction method of Berlin CS was the highlight of this project, however, the structual system was more interesting to be studied. The two office buildings of Berlin Central Station span 87m over the track of the railway station without any supports in between. Large truss structure has been designed to fulfill the requirement which also gives the idea to the offices of Breda Central Station to implement more flexible area and fewer supports on the bus terminal.
- Both truss structurs of Berlin Central Station and De Brug project have the height over 4 storeys high and hundreds of meters long which indicates that such truss structure is feasible to be used in the office building of Breda CS with the scale of 3-storey high, 277m long and 27m wide. With a view to the super long length of this project, it was suggested to divided the building into several individual parts to realize a feasible span of the truss.
- The slim floor system was used in De Brug building due to the weight of that system. This could also be applied in this project not only because of the weight of the floor, but also it was able to reduce the floor thickness and allow openings in the floor compared to other systems.
- The large truss used in the Port Authority bus terminal as load bearing facade implied that large free space for the bus passing through can be achieved by this.
- Since large span has been realized in the offices by utilizing the truss structure, so fewer supports could be set on the bus terminal of Breda CS. Massive columns designed the Regional Government Central in France and the station roof of the Southern Cross Station in Australia inspired that these kinds of structures might be good solutions used in this project. These huge fixed columns not only transfer the vertical loads but also provide stability to the whole structure.

[1] The new Southern Cross Station, 2006, steel Australia

8 Structural Design Concept

In the previous chapters, truss structure had been chosen to elaborate. The influence factor on the decision making was illustrated by the MCA and the analysis of the author. The major motivations were to get rid of the columns on the bus terminal in harmony with the rest of the station complex with vast space. In this chapter, the structural design concept would be described first based on the requirements and goal of the project, after that structural geometry, boundary conditions, load cases would be determined as the starting for the following truss structure design in next chapter.

8.1 Structural Concept

The north side of the station complex consists of variant functions such as offices, bus terminal and underground tunnel with commercial facilities. This might lead to the structure into several parts due to the different requirements of these multi functions.

For the office area, most of the time, the client wants the rentable areas as many as possible which usually results in a clear and regular plan view of the structure with fewer disturbances of the area. So making the facade bear the loads or the structure locate at the outside of the office area can be a good option, like truss facade. Moreover, as the office part locates on the bus terminal, it desires the structure as light as possible to minimize the size of the structure on the bus level to get more space.

For the bus terminal, since the busses have to drive on the lanes and the passengers will congregate on the platform, therefore no column or any other supporting structure are possible to be set there. So there is limited structure position on the bus level which leads to large span structure of office part.

For the underground tunnel and commercial facilities, the functional requirements which influence the structural solution are for every several years, the tenants change, which means that flexible arrangement is promising. Two dimensional plan structural elements like shear walls or cores are not favorable solutions.

According to these requirements, the study and analysis of the different alternatives in the previous chapter showed that the most efficient structural concept of this project was the truss structure which was able to realize large span and light weight structure so that fewer columns or other supports could be set on the bus level.

8.2 Structural geometry and model

8.2.1 Geometry and Dimensions

According to the architectural design, the structural dimension of the north side of Breda Central Station is, 277.2m×26.4m×23.49m (-0.21mNAP ~ +23.280m NAP)

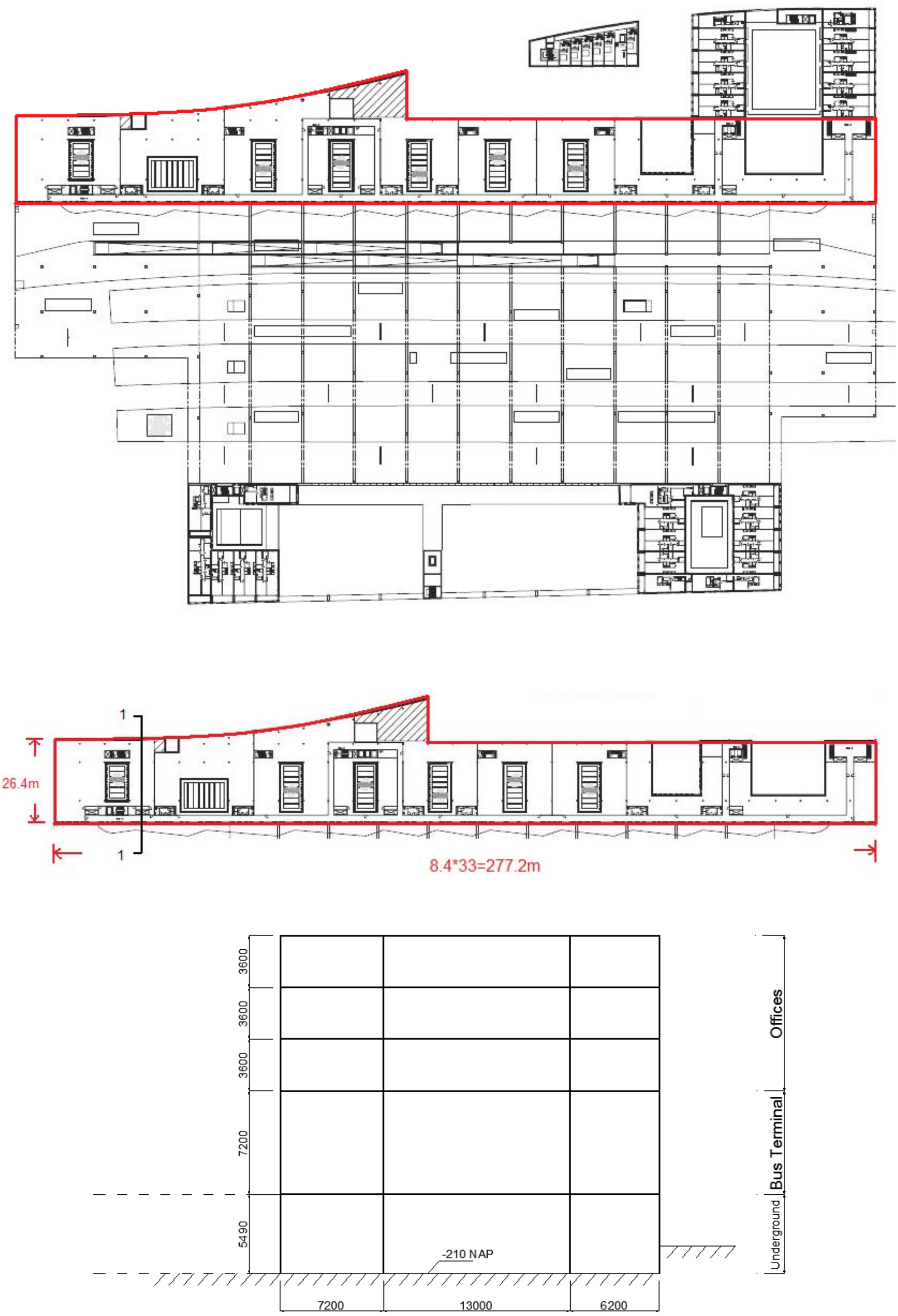


Fig 8-1 Dimension of north side

8.2.2 Supports and Boundary conditions

As discussing before, the north part of the Breda Central Station is designed to be an individual part of the station complex by expansion joints with the rest structure. Thus, it is supported by itself providing its own stability. The main load bearing structure that transfers the loads to the concrete foundation is the columns on the piles.

The boundary conditions of this project are,

- On the north, west, and east side are the facades of the station complex; while on the south side this part faces towards the station terminal.
- The car parking at the fifth floor level and its support structure: truss girders are designed to connect to the office area. To simplify this thesis, assumption has been made that the car parking would not supported by the office area any more, other structural solution would be used to support the car parking.

8.2.3 Structural Model

Single module

As mentioned before, the truss structure was selected as the solution for this project. And according to the geometry of the building, seven openings are set up evenly every 16.8m along the building (Fig 8-2). To simplify the design process, the elaboration of the structure will firstly be divided into single module (25.2m×26.4m) as the basic structural unit shown below. The determination of size of the module is according to the location of the openings and the partition wall. At two ends of the structure, module with 33.6m×26.4m and 16.8m×26.4m will be considered. After the final structure is determined, the whole structure with the full length will be modeled, calculated and analyzed.

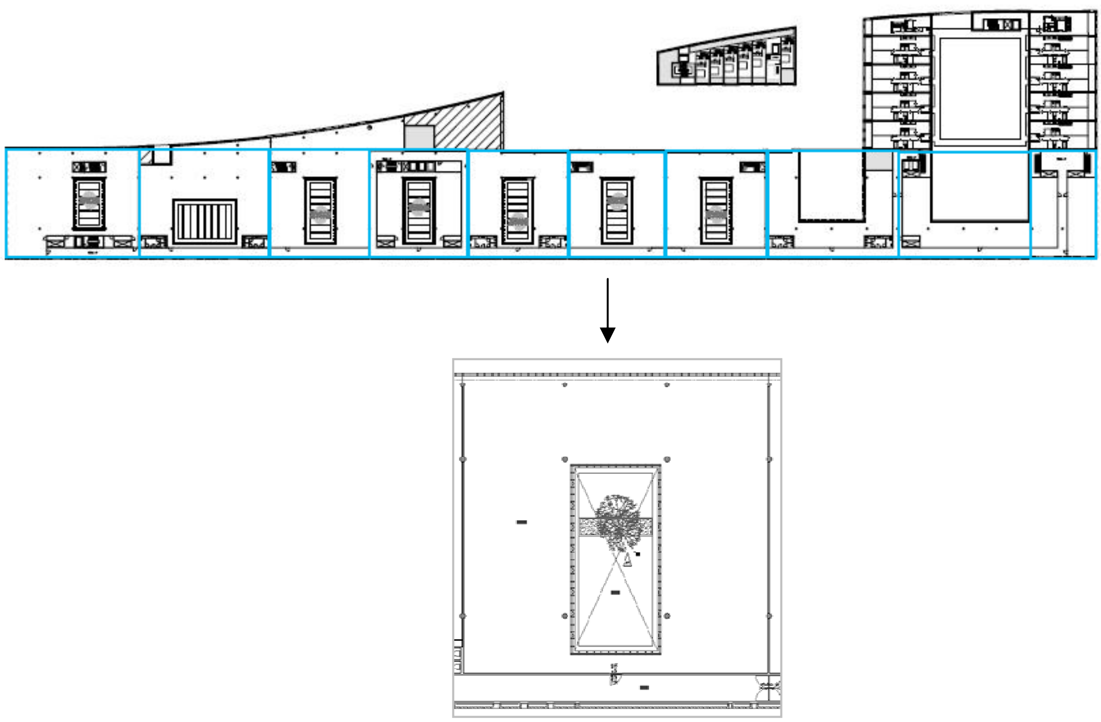


Fig 8-2 Single Module

Expansion joint

As a general rule of thumb, the maximum dimension of a building block without expansion joints should not exceed 60m. However, in this thesis, no other expansion joint would be placed between the modules for several considerations listed below.

- The building was supposed to have fixed columns.
- The temperature change of the office part is less than 5℃ the whole year.
- Also the building would be designed by steel structure but not concrete, so the shrinkage in concrete would not happen in steel structures.

In summary, this structure is going to be built continuously, so no other expansion joint would be placed, except those had been set up along the north and south facades to separate this part from the rest of the station complex.

Other consideration and assumptions

- For the general structural design, the facility rooms and transportation like staircases and elevators which have no structural function in this project will not be included in the structural model, but are supposed to be considered in the detail design stage.
- The dimension of the openings was changed a little bit to make it easier for the general structural analysis. Influence on this would be taken into account for the analysis and should follow the architectural design in the detail design stage. Thus, the structural geometry of the module is shown in Fig 8-3.

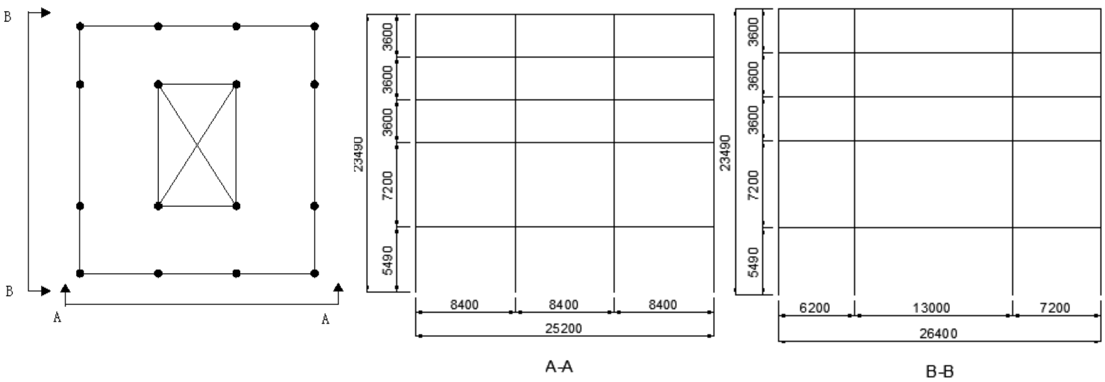
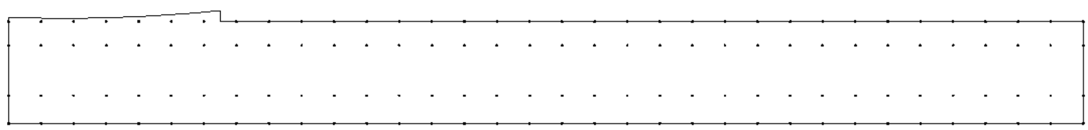


Fig 8-3 Structural model of single module

8.3 Overall Stability

In the architectural design, the columns on the bus terminal were arranged by 4 longitudinal rows and 34 transverse rows. The way to diminish the amount of the columns was by using truss structure with long span. The direction of the large truss could be either longitudinal or transverse or even both which also determined the stability of the structure.



Transverse stability

Trusses along transverse direction could free the whole bus terminal by means of supporting the office part only at the facades or at two intermediate supports cantilevering part of the offices. The basic model was supported by four supports along the frame. To get rid of some of the columns on the bus terminal, some of the supports were removed and trusses were introduced in to achieve the span. Two basic truss structures with different support position have been modeled. After that, several improved structures were studied.

The horizontal beams were 300ASB249, and the rest members (columns and bracings) were HE280B. The stiffness of the support used 214 MNm/rad.

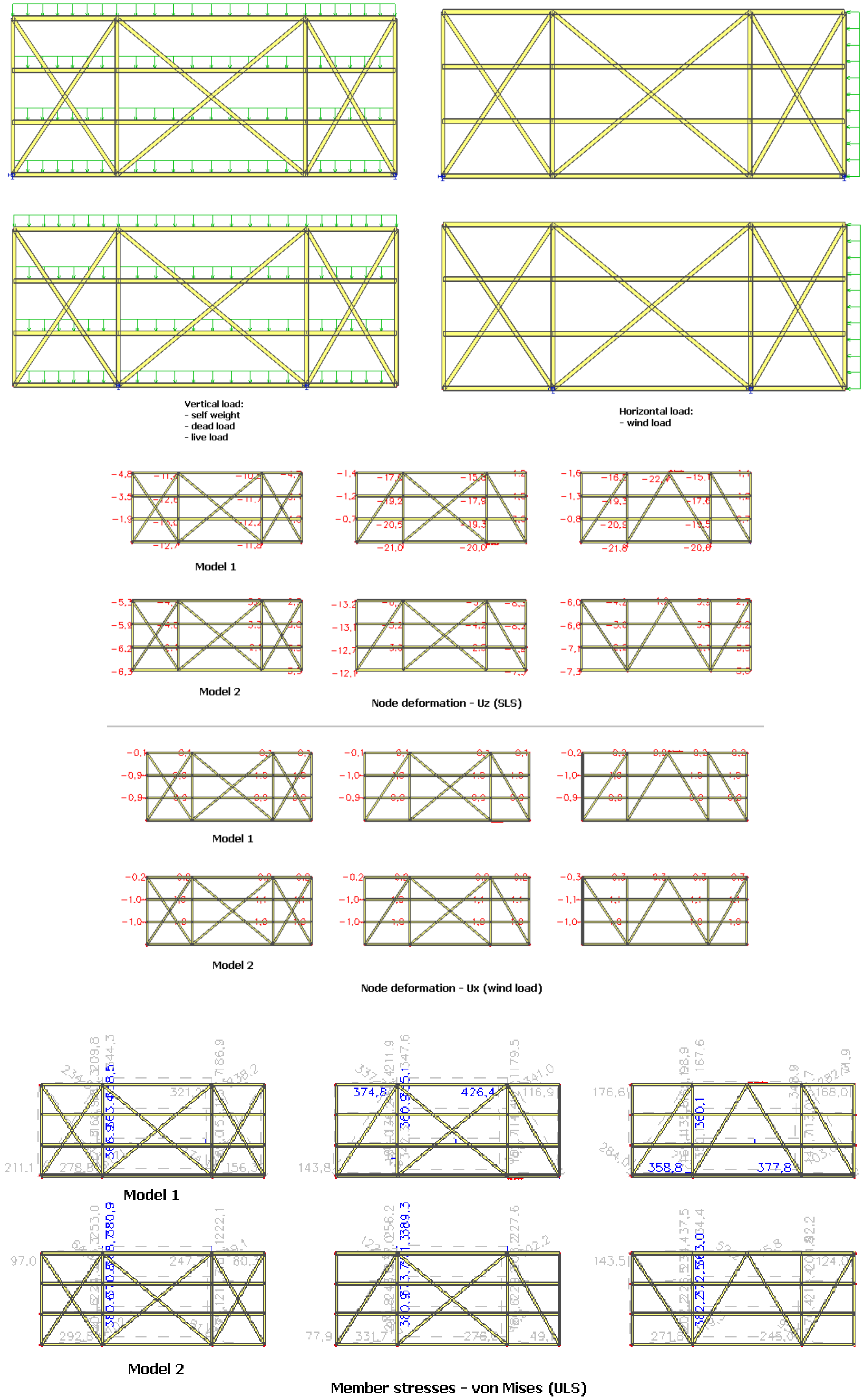


Fig 8-4 2D transverse truss models and results

From the results of the two basic truss structures (model 1 and 2) and the improved ones, several conclusions can be drawn.

- 2D transverse truss structure achieved the possibility of supporting the office part only at the facades (model 1) or forming cantilever (model 2). The main frame bay located in transverse direction provides stability in this direction. While in this structure, the stability in longitudinal direction was supposed to be provided by the rigid connection between these truss frame bays since the wind load along longitudinal direction was much smaller than that along transverse direction.
- Trusses in this direction fulfilled the requirements of the design and provided the stability of the domain direction of the building which seemed to be an effective solution for the structure.
- The selection of the member section was basically sufficient since the deformation of the nodes was promising while some of the member stresses have exceeded the yield value.
- However, the geometry of the truss required it had to locate at every 8.4m according to structural requirements and the floor span which was not possible for the office demanding the more flexible space the better. Neither clients nor architects, and of course me as the structural designer would prefer big diagonals locating every 8.4m in the office as obstacles, because the diagonals in the truss interrupted the circulation and arrangement of the offices just like irremovable shear walls.
- Of course, the pattern of the truss was able to be optimized to fewer diagonals (see the improved structures besides model 1 and 2). Under the same condition which was the same member section and load cases, the results of these improved structure showed that the deformation and force distribution didn't differ a lot from the original structure.
- But it only optimized the pattern of the truss, would not change the spacing of these transverse trusses unless truss was placed longitudinally as well, which means the interruption to the insulation of the office area still exists. In a word, although trusses designed in transverse direction had advantages for the bus terminal by providing more transverse clearance, it caused negative influences on the offices. The more space and clearance on the bus level could not be achieved at the expense of fewer flexible areas in the offices.
- Therefore, truss structure would not be placed transversely. The transverse stability would be provided by rigid-connected beams and columns.

Longitudinal stability

As the truss structure was going to be set in longitudinal direction, therefore it would provide the stability in that direction too. Based on the structural module determined in last section, the span of the longitudinal truss was mainly 25.2m, and 33.6m at the end of the building. The design and optimization of the truss structure would be described in next chapter.

8.4 Floor System

To estimate the dead loads of the floors acting on the structure, the floor system <u>in the offices</u> had to be assumed generally in advance. The rules of thumb of choosing the floor system complied with the goal to get light weight structure and small floor thickness. And regarding the floor plan of the architectural design, hollow core slabs and slim floor system were regarded as good options for this project. The characteristic of these two floor system has been discussed in chapter 4.

By comparing them, the slim floor system is selected for the project rather than hollow core slabs due to several reasons.

· With the same height of the floors, for instance, 320mm, the weight of the ComFlor 225 is about 3.66 kN/m^2 while the weight of hollow core slab (©Dycore) is about 4.3 kN/m^2 which means that under same floor height, slim floor system is lighter than hollow core slab system or under the same weight, slim floor system can achieve smaller thickness.

- Secondly, the slim floor system containing the steel deck and concrete floor above made use of both of the materials in their features. And this concrete floor as an integrity would perform diaphragm action to provide the lateral stability of the structure and transfer the horizontal loads.
- Last but not the least, due to the certain amount of openings in the office area, the ComFlor is much more flexible than the hollow core slabs, openings are possible in this kind of floor system, but not in hollow core slabs.

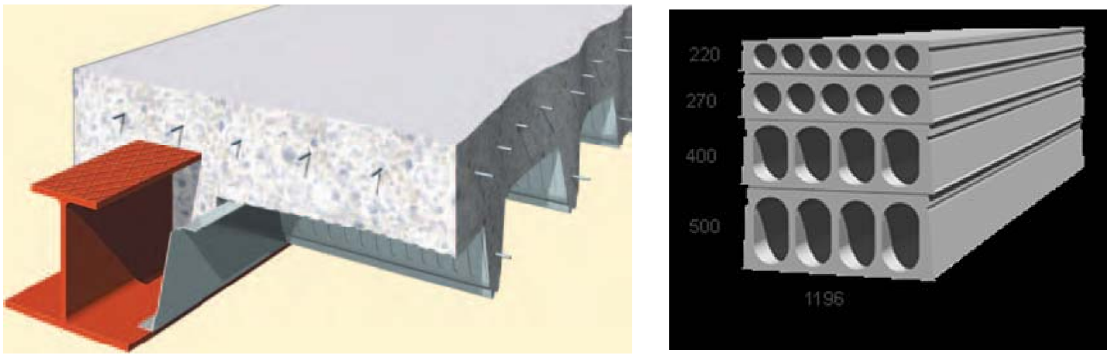


Fig 8-5 slim floor system (©Corus) and hollow core slab

Therefore, the floors in the office area were supposed to use ComFlor 225 (©Corus) with the floor thickness of 290mm (detail information see Appendix 9). The thickness was an assumption for the general design stage, and the feasibility of using this thickness should be checked and designed under the detail design stage.

8.5 Load Cases

The determination of the load cases were according to the Eurocode 1: Action on structures EN 1991-1 and Netherlands National Annex to Eurocode 1. For determining the loads on the structure, several aspects should be considered and calculated first.

Wind load from both longitudinal and transverse direction had to be considered. However, due to the extremely long length of the building, the calculation of the wind load indicated that the wind load from the north dominated, thus wind load from north direction was calculated in this project, detail calculation of wind load can be found in Appendix 5.

For the facades, according to the architectural design and the structural solution, it is supposed to use light weight bricks outside the truss structure, so the weight of the facade is defined about 0.5kN/m^2.

<table>
<tr><td colspan="3">Permanent Load</td></tr>
<tr><td>- Self Weight</td><td colspan="2">- Dead Load [kN/m^2]</td></tr>
<tr><td rowspan="7">According to the elements’ properties</td><td>Floors</td><td>2.31</td></tr>
<tr><td>Screed (50mm)</td><td>1.0</td></tr>
<tr><td>Installations and false ceiling</td><td>0.5</td></tr>
<tr><td>Light weight separations</td><td>0.8</td></tr>
<tr><td>Total (level 3-5)</td><td>4.6</td></tr>
<tr><td>Roof</td><td>3.8</td></tr>
<tr><td>Facade</td><td>0.5</td></tr>
<tr><td colspan="3">Variable Load</td></tr>
<tr><td>- Wind Load (north – south direction) [kN/m^2]</td><td colspan="2">- Live Load [kN/m^2]</td></tr>
<tr><td>North: 0.72</td><td>Office area</td><td>2.5</td></tr>
</table>

South: -0.45 East & West: -1.08 / -0.72	Bus terminal platform	5.0
	Roof	1.0
	Traffic load (see Section 3.3)	vk450

Table 8-1 Load cases

Load combination

The load combination should comply with the requirement of Eurocode 1 under both SLS (serviceability limit stage) and ULS (ultimate limit stage). The factor of the all the load cases under SLS is 1 while those under ULS varies from 1.0 to 1.5 according to the code. However, to simplify the design, one combination under ULS has been calculated for all the models where safety factor for the permanent load uses 1.2 and for the variable load uses 1.5.

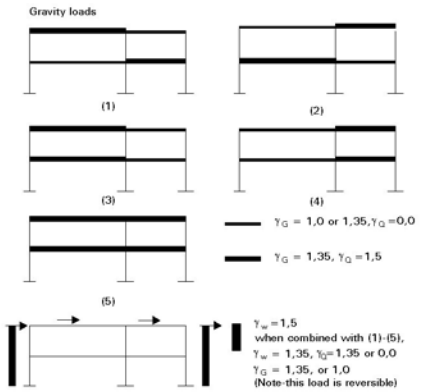


Fig 8-6 Load factors in Eurocode 3

8.6 Floor Beam Design

The spacing between the floor girders was 8.4m which means the ComFlor span was also 8.4m, and referring to the product information from Corus (see Appendix 6), temporary supports were needed during construction. The size of the floor girders and possible columns that supported these floor girders was then estimated.

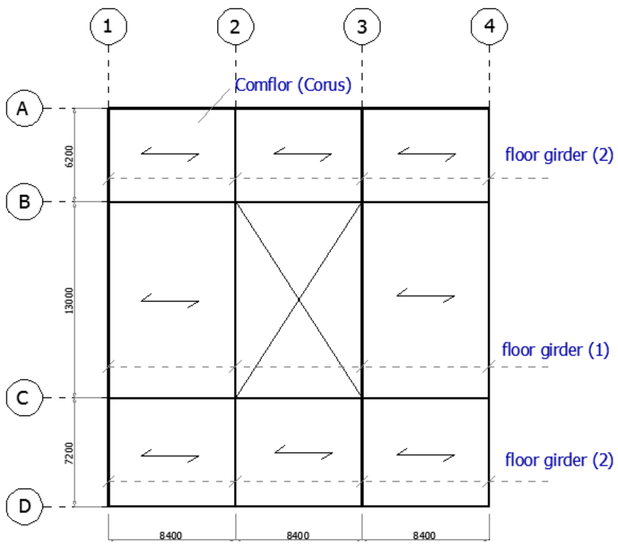


Fig 8-7 Floor system indication

Floor girder (1)

$$q_d = (4.6 + 1) * 1.2 * 8.4 = 56.4 \text{ kN/m}$$

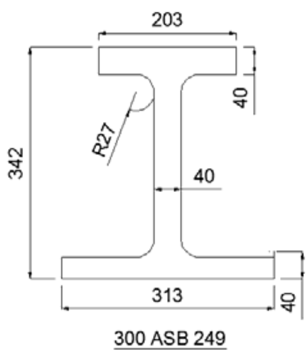
$$q_v = 2.5 * 1.5 * 8.4 = 21 \text{kN/m}$$

The boundary condition of the floor girder was designed as rigid connected, so

$$M_u = \frac{1}{12} q_u l^2 = \frac{1}{12} * (56.4 + 21) * 13^2 = 1238 \text{ kN} \cdot \text{m}$$

$$W = \frac{M_u}{\gamma_{M0} * f_y} = \frac{1238 * 10^6}{1 * 355} = 3.49 * 10^6 \text{ mm}^3$$

Select 300ASB249, $W = 3530 \text{ cm}^3$



$I_y = 52900\ cm^4$
W= 249 kg/m
A = 318 cm²

Deflection check

$$\delta = \frac{ql^4}{384EI} = \frac{72 * 13^4}{384 * 210 * 10^6 * 5.29 * 10^{-4}} = 48.2\text{mm} < 0.004l = 52mm, ok!$$

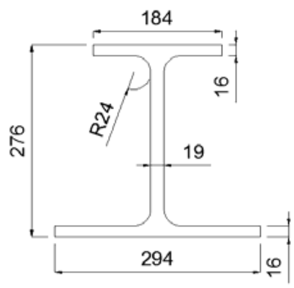
Floor girder (2)

$$M_u = \frac{1}{12} q_u l^2 = \frac{1}{8} * 87.9 * 7.2^2 = 379.7\ \text{kN} \cdot \text{m}$$

$$W = \frac{M_u}{\gamma_{M0} * f_y} = \frac{379.7 * 10^6}{1 * 355} = 1.07 * 10^6\ \text{mm}^3$$

Select 280ASB100, $W = 1290\ \text{cm}^3$

$I_y = 1550\ cm^4$
W= 100 kg/m
A = 128 cm²



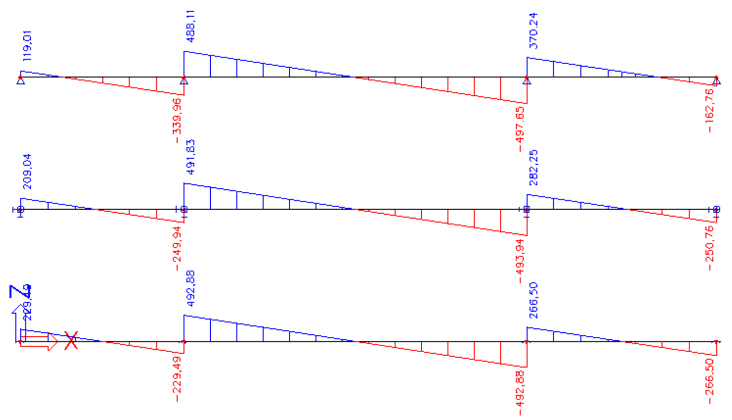
280 ASB 100

Deflection check

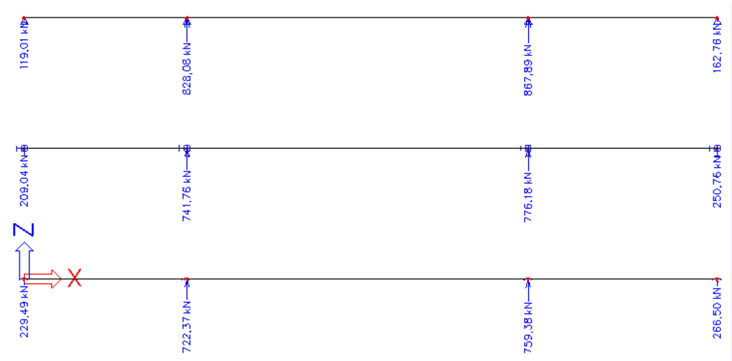
$$\delta = \frac{ql^4}{384EI} = \frac{66 * 7.2^4}{384 * 210 * 10^6 * 1.55 * 10^{-4}} = 14.2\text{mm} < 0.004l = 28.8mm, ok!$$

Reactions of the floor girders

In order to get to know the influence of the support condition on the structural behavior, three continuous beams with same sections and loads supported by different supports: hinged, flexible and fixed, were modeled in ESA PT to compare the results.



Vz



Rz

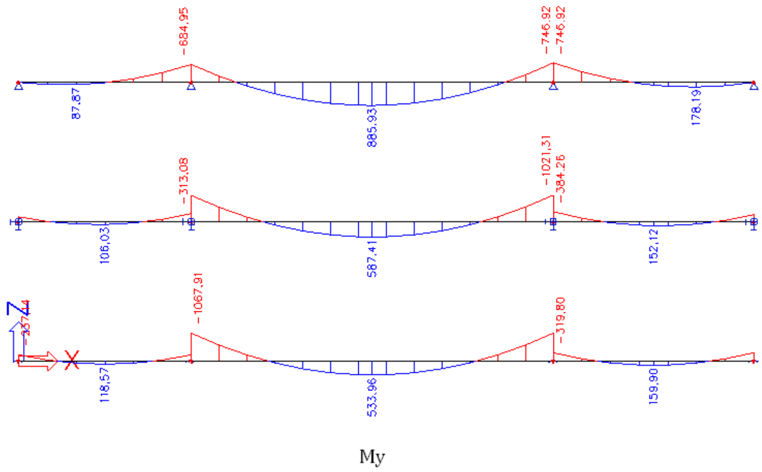


Fig 8-8 Results of continuous beam with different support condition

The results indicated that the shear forces were almost the same among the three different models, and the reaction forces had slight difference, while the support condition mainly determined the capability of transferring the moments which resulted in large difference in bending moments. Therefore, this implied the importance of determining the stiffness of the connections.

The stiffness of the frame is determined by influence of connection flexibility on elastic frame stability. The (M-ϕ) response is a linear straight line which is called the beam line as show in Figure 8-9 (b). In real frames the end moment restraint is provided by the rigidity of the connection S = M/ ϕ as shown in Figure (c) and therefore the actual end moment and the end rotation of the beam is given by the intersection of the beam line with the connection characteristic as shown in Figure (b). And for practical design situations the actual non-linear connection behavior has to be approximated.

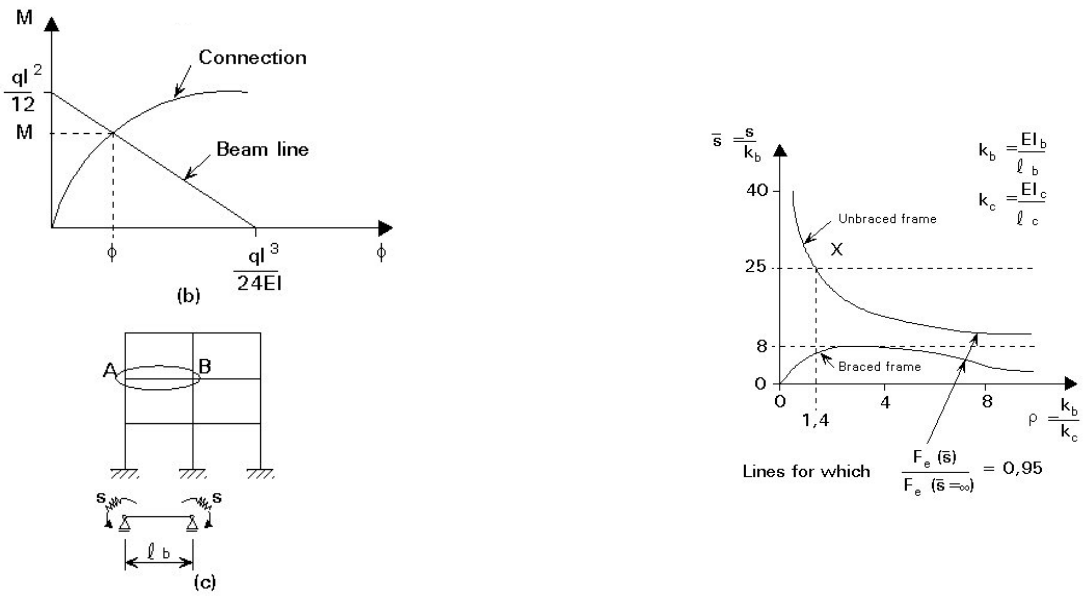


Fig 8-9 Beam-line and connection behavior & Influence of connection rigidity on frame behavior [15]

In order to determine if the connection flexibility S^{-1} needs to be included in the overall frame analysis it is important to examine its influence on the behaviour of the frame[2]. The figure

[2] Background Document 6.09 to Eurocode 3, Beam to Column Connections, Commission of the European Communities, 1989

presents the relationship between the relative connection-to-beam rigidity and the relative beam-to-column rigidity r in order that the flexibility of the connection reduces the Euler buckling load of the rigid frame by 5%.

It can be concluded from the Eurocode 3 that $s/k_b = 25$ is a sufficiently safe boundary value for practical frame for the rotation stiffness of beam-to-column connection in unbraced frames in order to consider them as rigid.

$$s = \bar{s} * k_b = 25 * \frac{210 * 10^6 * 5.29 * 10^6}{13} = 214 \text{ MNm/rad}$$

So the stiffness of the connection used in the structural model was 214 MNm/rad.

The contiuous beam model was calculated in ESA PT with the support stiffness of 214 MNm/rad. Based on the results of the model, the columns that transfer the loads vertically towards down was estimated as HHS355.6×355.6×15.9 as a start which had equal stiffness in both directions.

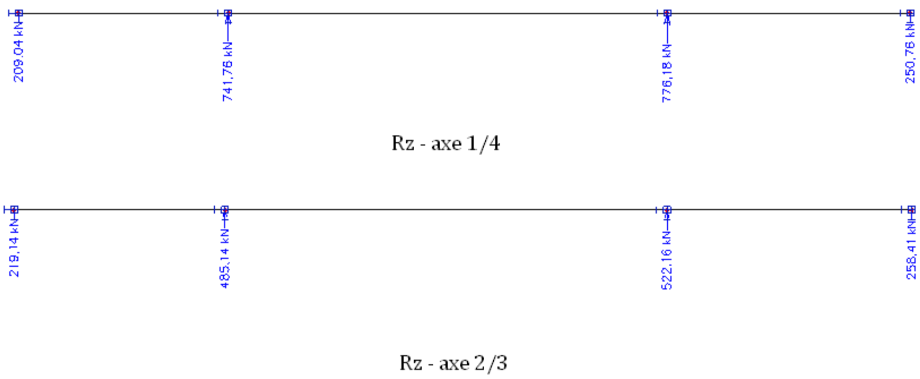


Fig 8-10 reation forces of continuous beam

9 Structural Design Calculation

After determining the truss direction, load cases and element sections in last chapter, the design process of the structure will be described in this chapter. The individual north side of the station complex will be vertically divided to three parts, office structure, bus terminal structure and underground commercial area due to their different functional and technical requirements. The truss structure in the office part, the tree column structure on the bus level and the structure of the underground commercial area has been designed and optimized respectively. After that, the entire structure of the north side of the Breda CS will be modeled and calculated.

9.1 Office Structure

As said before, the main purpose of this design was to gain more free space and clearance on the bus level by reducing the columns and more aesthetic architectural view regarding to the design. Truss structure considered and valued as an alternative being able to realize large span in the office area resulting in half or even one third of the columns in the bus terminal precedes other structure.

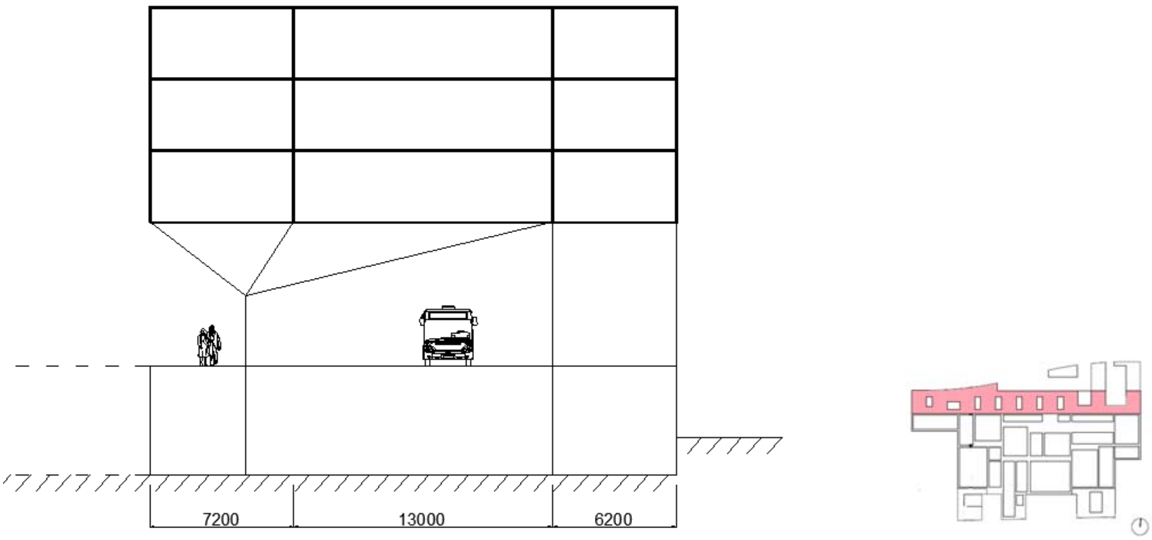


Fig 9-1 Indication of the section view

9.1.1 2D Longitudinal Truss

By discarding the transverse truss option in chapter 8, longitudinal truss by turning the truss span the other way around was selected as the main structure in the office part. To design the members in the truss, the reaction forces in the continuous beam that resulted from the floor girders to it were applied. Fig 9-2 showed the forces transferred to the truss each floor level.

Note:

- As principle in the single module, the forces in axe 1 and 4 should be about half of the forces in axe 2 and 3, however, considering the real situation that the structure was going to be built continuously, so that the structure along axe 1 and 4 had to carry the loads from both of the adjacent module. Therefore, the load bearing area in the module was also selected 8.4m wide as the intermediate ones.
- In addition, it had been decided that the wind load in this project would be considered from north, so the design of the truss structure would consider vertical forces only. <u>The wind load would be introduced in the 3D model</u> afterwards.

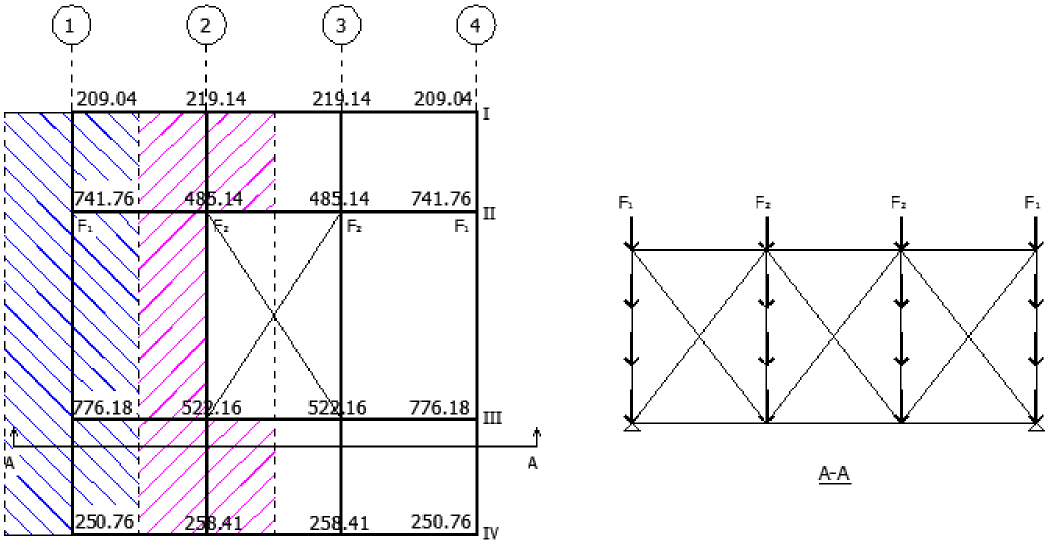


Fig 9-2 loads transferred to the truss

Truss Design

There were four trusses located longitudinally and truss III was chosen to be elaborated because it carried the most loads compared to the others.

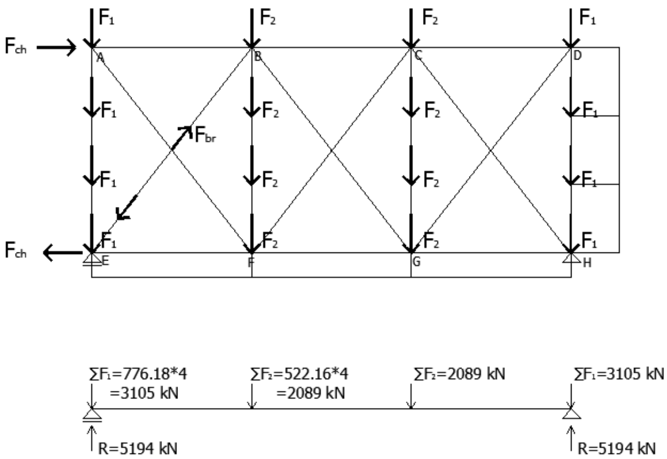
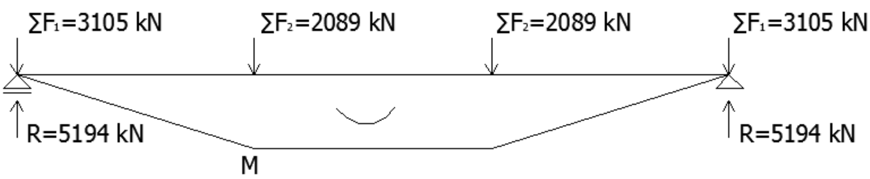


Fig 9-3 Forces in truss

Estimate the top/bottom chord sections,



$$M = (5194 - 3105) * 8.4 = 17548 \text{ kN} \cdot \text{m}$$

$$F_{ch} = \frac{M}{h} = \frac{17548}{10.8} = 1625 \text{ kN}$$

$$A_{ch} = \frac{F_{ch}}{f_y} = \frac{1625 * 10^3}{355} = 4577 \text{ mm}^2$$

For safety and other consideration, select beam HE240A, A_{ch}=76.8 cm^2 ;

Estimate the column sections,

$$A_{col} = \frac{3F_1}{f_y} = \frac{3 * 776 * 10^3}{355} = 6558 \text{ mm}^2$$

Then, select beam HHS355.6×355.6×15.9, A_{col}=216 cm^2

Estimate the bracing sections,

Analyze the corner A for the bracing forces,

$$F_{br} \times \frac{8.4}{\sqrt{8.4^2 + 10.8^2}} = F_{ch} = 1625 \text{ kN}$$

$$F_{br} = 2647 \text{ kN}, A_{br} = \frac{2647 * 10^3}{355} = 7456 \text{ mm}^2$$

Select beam HE260A, $A_{br} = 86.8 \text{ cm}^2$

Summary

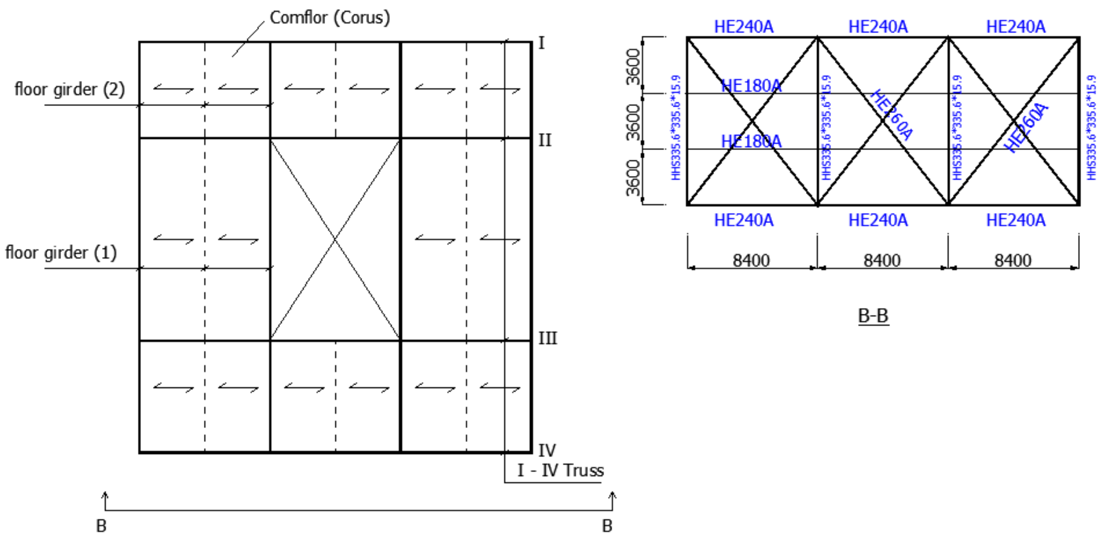


Fig 9-4 Overview of the structural elements of single block

Elements	***Designation***
Floor girder(1)	300ASB249
Floor girder(2)	280ASB100
Additional floor beam	HE180A (to support the floor only)
Chord	HE240A
Column	HHS355.6×355.6×15.9
Bracing	HE260A

Table 9-1 Truss elements summary

Pattern Optimization

After determining the basic properties of the truss members, several truss patterns were modeled to get an optimized structure. The first one had the diagonals in every panel, and the second one called Howe removed the tension members while the third one called Pratt removed the compression members.

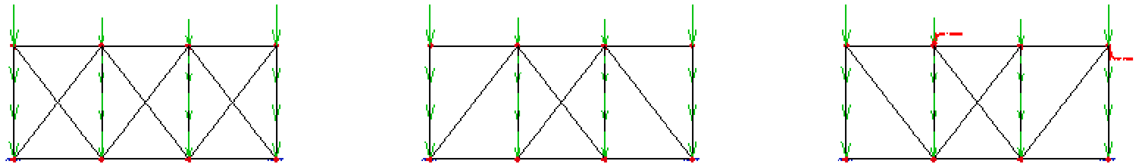


Fig 9-5 Alternative of truss pattern

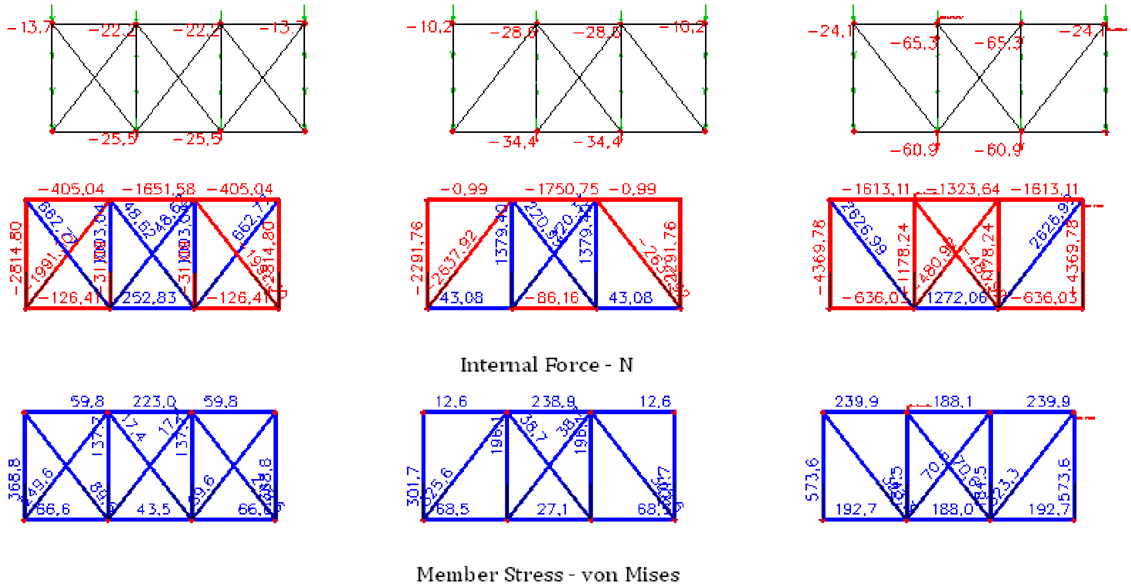


Fig 9-6 Results of pattern optimization

The aim of Howe and Pratt truss was to reduce the number of the members to a minimum for not only saving the materials but also obtainning more clear overview from the facades. And from the results of these three different truss pattern, we found that,

- Under same load condition and same member section, three different pattern showed different behavior in deformation, internal force and member stresses.
- The Howe truss had similar behavior as the original truss while the Pratt truss behaved worse than those two with large node defomation, normal forces and high stresses.
- The node deformation in Pratt truss was more than twice of the other two trusses, and also exceeded the limitation.
- The internal normal force in the Howe truss showed a most reasonable distribution than the other two. Especially, extremely high compression force occurred in the outside columns of the Pratt truss.
- The member stresses in Howe truss also behaved better than the other two trusses.

The results implied that the Pratt truss pattern with the best structural behavior and fewer member amounts was the most effective one among these three, thus this pattern was chosen for the following design.

Member Optimization by Unit Check

However, the disadvantage of the pattern optimization under same element properties is the effectiveness of the structure could not be garanteed which means that the unit check of these three models might be not optimized. The consequence could be that, after unit check, the total amount of the steel used in model 1 might be less than that of model 2 even it had more elements. The unit check is the actual stresses in the members divided by the yield strength of the material (σ/σ_y) which is usually accepted by <1, and the closer to 1 the better.

Hence, member optimization by unit check was done for mdoel 1 and model 2. Since model 3 had too unfavourable behavior that member optimization would not help it to reduce the deformation and amount of steel simultaneously, so member optimaztion would not be done to this model. The prerequisite of the optimization was the largest deformation of the nodes in all the models were the same, making a criteria of about 34mm since the recommended limit of element deformation is less than 0.004L (0.004×8.4=33.6mm).

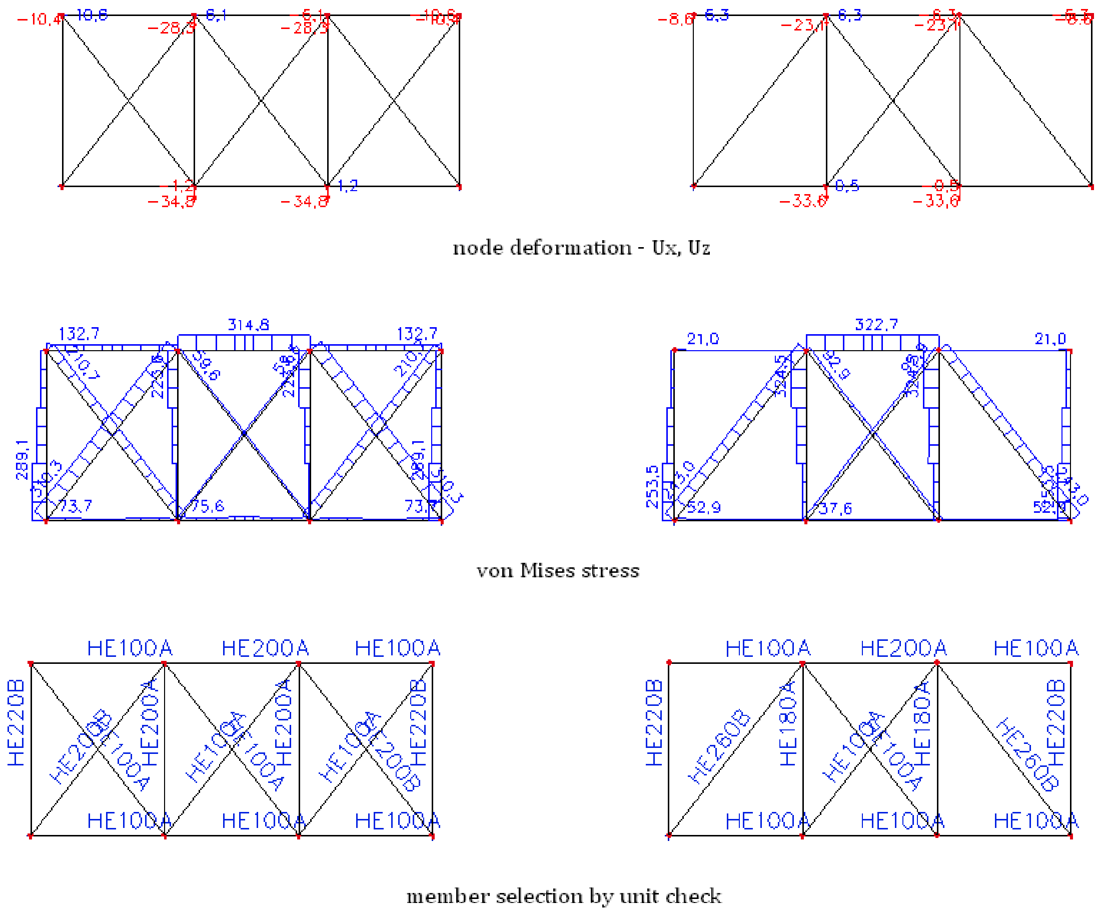


Fig 9-7 Unit check

After optimizing the members by unit check, the total amount used in model 1 was about 59.82kN (5982kg) while that used in model 2 was about 62.42kN (6242kg), that the assumption has been proved that model 1 has more members but use less amount of material on the contrary. Although model 1 is now lighter than model 2, the difference between them is very slight, and the use of model 2 that has fewer members and connections is not only for light weight, but also for providing more clearance. This is usually preferred by architects and users. For these reasons, model 2 was still chosen as the pattern of the truss structure.

In addition, no matter in model 1 or model 2, the forces in the two diagonals in the middle frame were always very small. This made it interesting to see what if these two diagonals had been removed. A test model was made and the results implied that the deformation and stresses didn't change a lot. However, this could not be concluded that these two diagonals were able to be removed. Becaues when adding unexpected asymmetric loads or horizontal loads on this structure, large deformation happened due to this structure geometry. This also demonstrated the effect of the zero-force member in the truss structure. Consequencely removing them was not reasonable and acceptable.

9.1.2 3D Truss Model (single module)

After determining the truss type in the longitudinal direction by means of 2D models, 3D truss model was input into ESA PT according to all the members designed previously to calculate the structure. The lateral stability is supposed to be provided by the floor system with the diaphragm action, and then the lateral forces will be transferred to vertical stiff elements such as columns. Additional measures have to be taken if the horizontal deflection is too large due to the insufficiency of the stiff columns. For 3D model, it is important to determine the load type and simulate the diaphragm action of the floor system before the calculation because different load type may results in different behavior of the structure. To find a relative accurate and feasible load type, three models were calculated by ESA PT and the comparison of the results is followed.

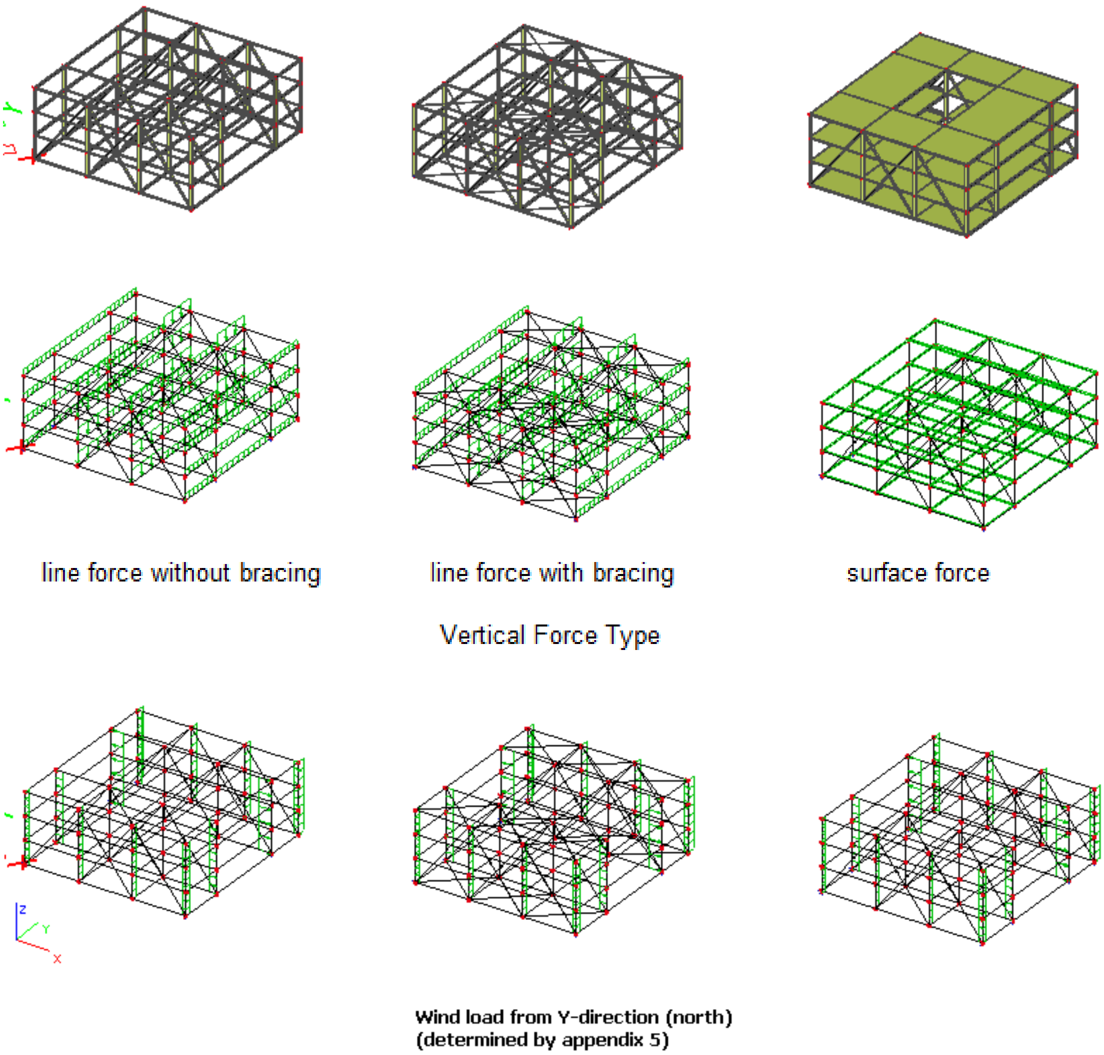


Fig 9-8 three 3D truss models

Model information

- Same dimension: 25.2m×26.4m×10.8m;
- Same member property and same support condition;
- Load type: different vertical load type;
- Supports Flexibility: 214 MNm/rad
- Load cases:

Load Cases		Values	Type	Direction
LC1	Self weight	default	line force	-Z
LC2	Dead loads	4.1 kN/m²	line force	-Z
LC3	Live loads	2.5 kN/m²	line force	-Z
LC4	Wind loads	0.9 kN/m² (basic)	line force	±X; ±Y

	Model 1	**Model 2**	**Model 3**
Deformation of nodes (SLS)			
Ux (mm)	±3.3	±2.9	+0.3
Uy (mm)	-96.8	-6.5	-4.2
Uz (mm)	-19.5	-19.1	-15.1
Reactions (ULS)			
R_1 (kN)	1745.17	1757.22	1652.27
R_2 (kN)	3834.46	3840.22	3939.60
R_3 (kN)	3702.95	3713.69	3805.94
R_4 (kN)	1369.64	1366.37	1254.72
Internal forces on beam (ULS)			
My (kN*m)	+316.03	+315.98	+284.11
	-585.49	-568.83	-284.11
N (kN)	+1640.84	+1603.78	+1233.74
	-2750.02	-2735.00	-2509.87
Beam Deformation (SLS)			
Rel Uz	1/489	1/489	1/1150
Von Mises Stress (ULS)			
σ (N/mm²)	355.1	231	217.5

Table 9-2 Single module results under three different load types

Conclusion

The purpose of the above comparison was to make clear the effect of the slab in the floor. Since in the reality, all the vertical loads were firstly loaded on the floors and then be transferred to the beams and columns or other structural elements. But for normal design process, frame structure was modeled by line forces acting on beams to simplify the design, while the slab as a whole performs diaphragm action to resist the horizontal forces. Without slabs or floors, large horizontal deformation happened according to the results of Model 1 since there were no structural elements designed to resist the horizontal loads. From this point of view, in order to perform the diaphragm action by the floor system, model with slabs had to be used.

However, there are still some problems with this model. First of all, as the forces were added on the slabs as surface load, but not on the beam directly, then the internal forces like bending moments of the beams could not be observed. The results of the bending moment of the beams in

Model 3 are relatively much smaller than Model 1 and 2. Secondly, by using slim floor system, the floor is a composite one that contained steel decks and concrete which means the slabs are not isotropy. At some locations, the concrete is fully poured through the entire height while at some locations where steel decks are, the concrete doesn't occupy the full height. Although the self weight of the slabs in the model were added by imposed forces calculated according to the real information, but not calculated by the program automatically, the program still computed the slabs as isotropy elements that the forces distribution of the results didn't work accurately.

Considering the situation discussed above, the second model with horizontal bracing looks like a good solution for the model. On one hand, it simplifies the model by means of using bracings instead of the floor system to resist the horizontal forces, and on the other hand, it avoided the problem caused by the slabs that bending moment in the beams could still be observed. The only thing has to be done with these horizontal bracings is, in the detailed design stage, the floors has to be designed strong and stable enough to comply with the structural behavior of the bracings. Here, for the concept design, these bracings were used HE100A sections.

In addition, the results of model 2 shows that the horizontal deformation is acceptable compared to the general limitation (h/500=10800/500=21.6mm) which means that the columns are strong enough to bring the horizontal loads from horizontal stiff members to the foundation. This would be checked again in the final structure to see if it still works, otherwise, other stiff elements have to be designed to resist lateral loads.

9.1.3 3D Truss (whole structure)

In order to determine whether the defined truss structure was useful to the whole long office part, a whole structure combined by the above single module was modeled in ESA PT. The node and beam deflection, and force distribution would be observed from the result to check the design. The conclusion would be described afterwards. Detail results can be found in Appendix 7.

Structural Geometry

To model the whole office part, five 25.2×26.4m middle truss modules, four 33.6×26.4m end truss modules and one 16.8×26.4m truss module at east side has been combined to each other which could be seen in the figure below. All of them had the same height of 10.8m which is the total height of the three-storey offices. The choice of the dimension these modules were according to the location of the atrium and large cuts from the architectural design. Each truss module had one atrium or cut in it. The truss pattern followed that in the single module. Viewing from the whole structure, there were four 277.2 meters longitudinal trusses along west-east direction with the support spacing of 25.2-meter and 33.6-meter at different locations. The beams were used ASB floor girders to work together with the Comflor system, and the columns were HHS355.6×355.6×15.9 as mentioned before. The truss diagonals were HE260B which could bear axial forces only. The horizontal HE100A bracings at roof and bottom represent the behavior of the floor diaphragms to transfer the horizontal loads to the vertical columns. Finally, four load cases were added to the model based on the reality, which were self weight of all the members, dead load and live load from the function requirements, and wind load defined in the appendix. Two load combinations, SLS and ULS with different load factors were made to check the different results under global elastic analysis.

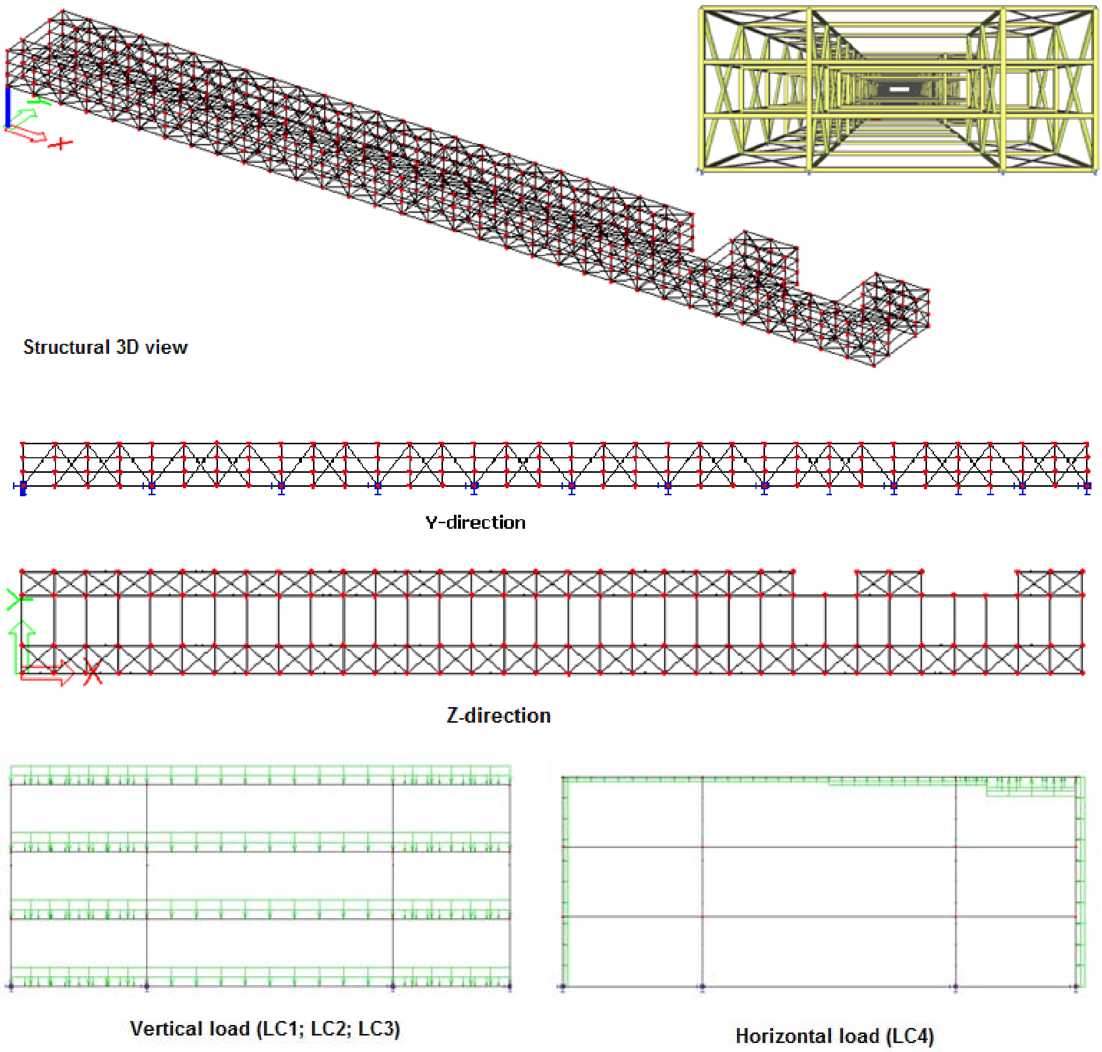


Fig 9-9 Structural model of the entire office

Results and conclusion

Several conclusions can be drawn from the results of ESA PT under linear calculation.

- The deflection of the nodes in the entire structure is acceptable under serviceability limit stage of combination load cases. The maximum horizontal deflection in y-direction is 8.4mm and in x-direction is 11.1mm which are smaller than the limitation of the structure integrally ($0.002\times10800=21.6$mm).
- The horizontal node deflection under wind load only is also under control with 7.7mm.
- Largest compressive axial force occurs in the truss diagonal. And the stress has exceeded the yield strength of the selected steel material, also buckling might happen due to large axial forces. Therefore the effectiveness of stiffness of the member had to be taken into account for further calculation, measures like enlarging the section of the member or changing the geometry of the structure might be taken.
- The largest beam deflection occurred in z-direction due to bending in the floor beams of 1/239. For the general design, it was acceptable compared to the suggested limitation 1/250, for the further detail design stage, this is supposed to be controlled.
- Considering the vertical columns are loaded by combined actions such as compression, bending and shear, and have to provide the stability as well, the member are used HHS without weak or strong axis to provide stability in both directions.
- From the results and check above, this 3D truss structure for the office part could be concluded a feasible one but has to be optimized in the final structure.

9.2 Bus Terminal Structure

The function of level 1 and 2 of the north side of the station complex is mainly bus terminal and small part of office on the north of it. Besides the main function of the bus terminal which enough flexible space is essential for the traffic flow and passenger circulation, the bus terminal structure also has to carry the whole office above it. The realization of the truss structure in the office has succeeded in reducing about 2/3 of the columns on the bus terminal. The further reduction of the number of columns there could be achieved by using tree column structure.

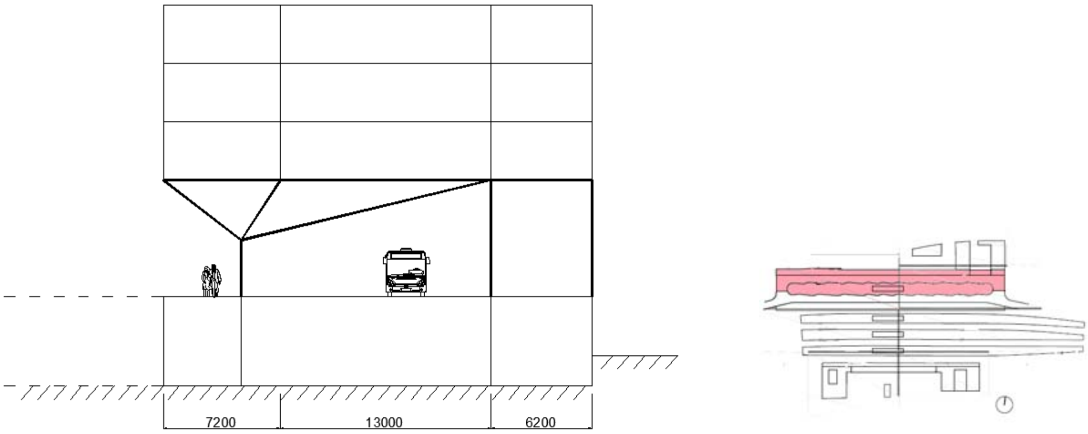


Fig 9-10 Bus terminal architectural plan view and structural section view

9.2.1 General Structure

As a start to estimate the section of bus terminal structure, rigid frame which columns placed at the supports of the truss structures were set. These columns were fixed at the end and provided stability in both directions.

The loads acting on the 2D frame were taken from the largest reaction forces of the entire 3D truss structure of the office part (loads on bottom floor of the offices were not considered, these would be added on the tree column structure directly). The largest reaction forces occurred at where Figure 9-11 indicated. This would be used for the design of the structure on the bus level.

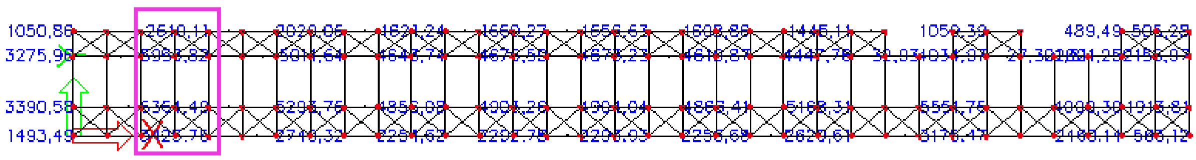


Fig 9-11 Reaction forces Rz in 3D truss whole model

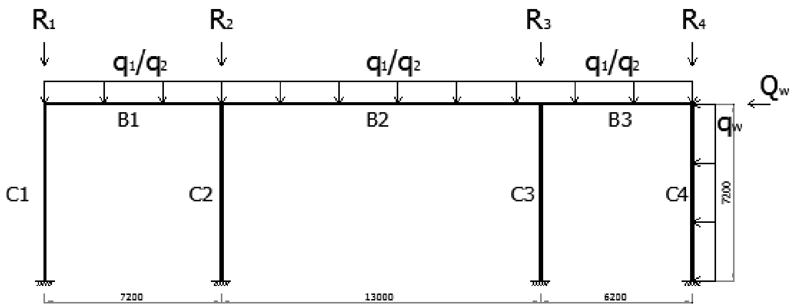


Fig 9-12 Structural model of frame on bus terminal

Load Case			Members	
Point load	R_1 [kN]	3426	B1	300ASB249
	R_2 [kN]	6364	B2	300ASB249
	R_3 [kN]	5954	B3	300ASB249
	R_4 [kN]	2619	C1	HE600M
Uniformly distributed load	q_1 [kN/m]	34.44	C2	HE600M
	q_2 [kN/m]	21	C4	HE600M
	q_w [kN/m]	6.05	C4	HE600M
	Q_w [kN]	65.34		

Table 9-3 Load cases and member section

9.2.2 Tree Column Structure

The idea of the tree column originated from the rigid frame formed by vertical columns. When trying to reduce one row of the columns on the bus terminal, tree column shape came into mind as an option. Several alternatives were made to compare and optimize the structure. The most direct way to reduce one row of the columns was to use inclined beams supported at one point. However, structure like this shape under the uniformly distributed loads and point loads behaved unstably from the result. Method to make the structure more reliable was to add bracing to increase the stiffness of the structure.

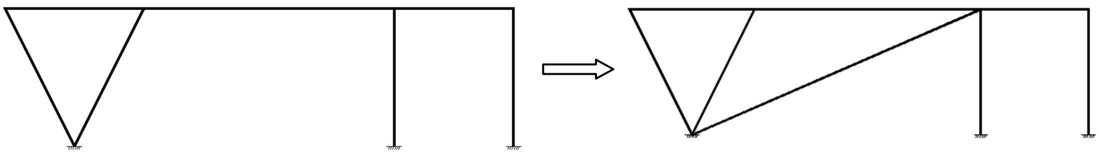


Fig 9-13 Tree column idea

9.2.3 Form Finding

From structural point of view, after adding one bracing in the structure, the results looked better. However, considering the reality, it was not possible to be accepted because this bracing obstruct the bus route and passenger walkway in that area. Then ideas were made to lift the intersection of these columns so that the passengers and busses would not be interrupted by the structure. Several models made by lifting and moving the intersection to different height were observed for the influence and force distribution. By comparing the different tree columns, deflections, internal forces and stresses were looked into to get an optimized shape of the tree column.

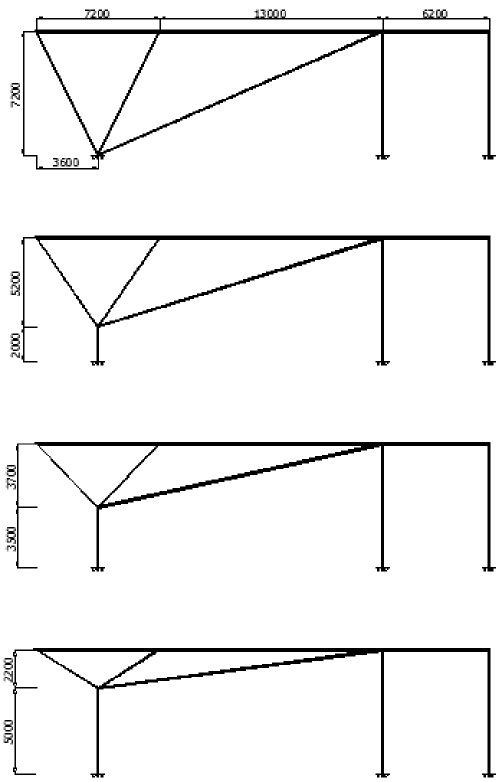


Fig 9-14 Different height of the intersection (lifting it vertically)

Results

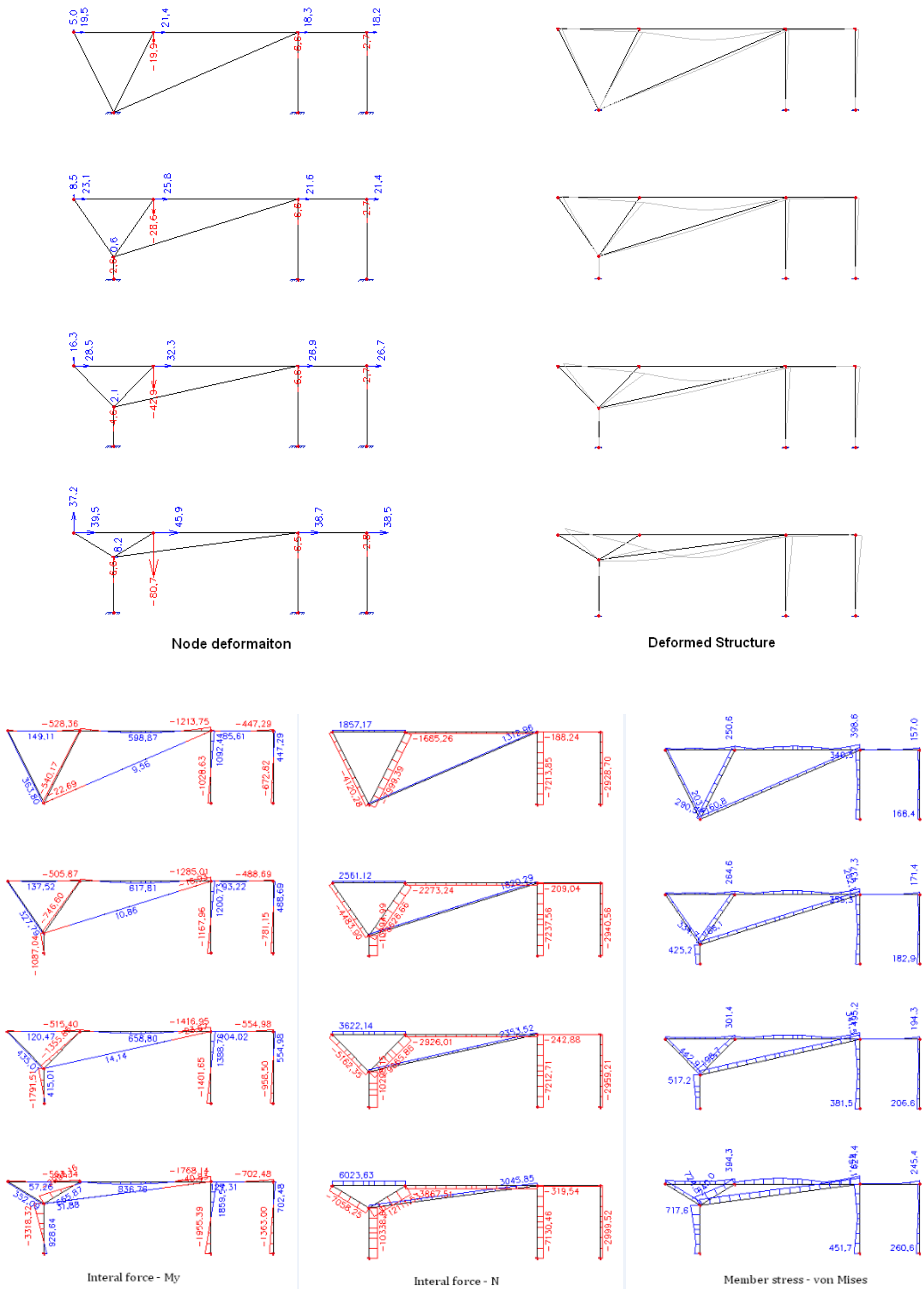


Fig 9-15 Results of form finding 1

The results of these models with different height of the intersection of the tree columns implied that by increasing the length of the vertical column (the trunk of the tree), the structure behaved less stable and the forces distribution also became worse. Under same load cases and with same elements, when lifting the intersection, the structure would probably cause failure. The best situation was the first one that intersected the two inclined column (the branch of the tree column) at the support to form a most stable structure. However, this would cause reality problem that it interrupted the operation of the bus terminal. Therefore, considering the service requirement, the 'trunk' column was adjusted to 3.5m high to fulfill both the functional and structural requirements.

After determining the height of the intersection, the influence of the horizontal position of it would also be studied to get an optimized geometry of the tree column.

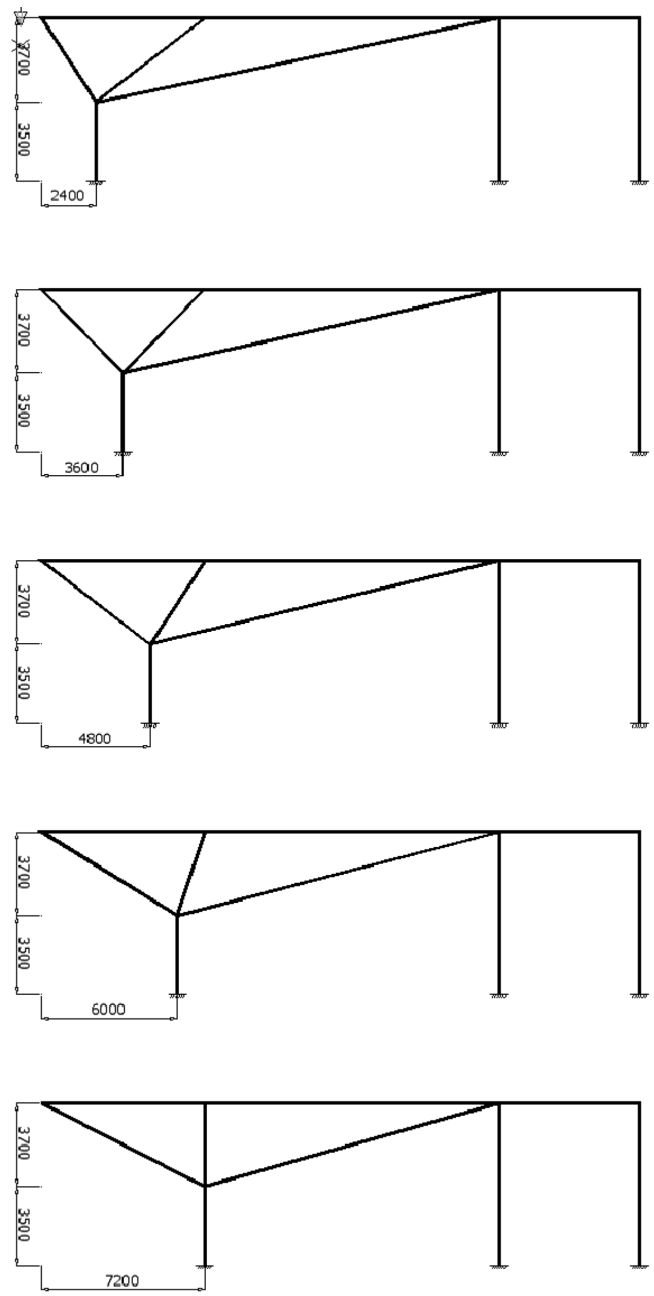


Fig 9-16 Different horizontal position of the intersection (moving it horizontally)

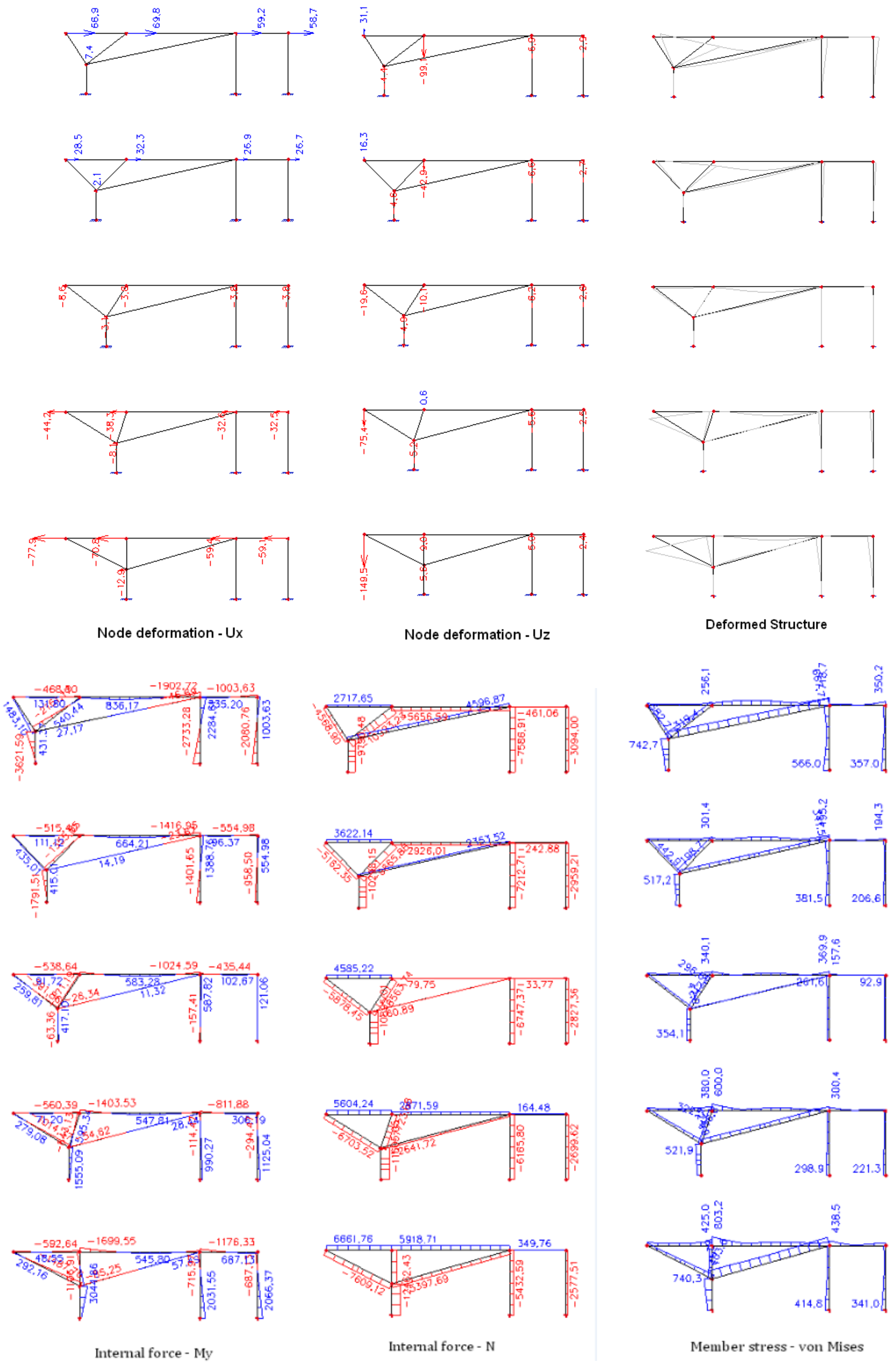


Fig 9-17 Results of form finding 2

9.2.4 Results and Conclusion

Several conclusions could be drawn based on these results.

- When moving the intersection of the tree column, the structure behaved quite differently.
- Both the horizontal and vertical deformation of the nodes in the structure was strongly influenced by the intersection position. Under the serviceability limit stage and the combined effects of all the load cases, the horizontal deformation changed from positive to negative when moving the intersection from left to right. The zero horizontal deformation might happen when the distance between the intersection and the left point of the structure was 4.7 to 4.8m.
- The vertical deformation of the nodes also varied from model to model. However, differing from horizontal deformation, the vertical deformation didn't decrease gradually when moving the intersection from left to right. There was a minimum value of the vertical deformation that we can found that it still located at the point where minimum horizontal deformation occurred. Thus the third model among these seemed to be an optimum geometry of the structure with regarded to the deformation.
- By observing the force distribution, this optimum model also showed the most effective force distribution for the structure that fully used the capacity of the elements.
- Large compression forces occurred in the element C2, C4 and C5. And the capacities of these members were more or less sufficient by the selected sections.

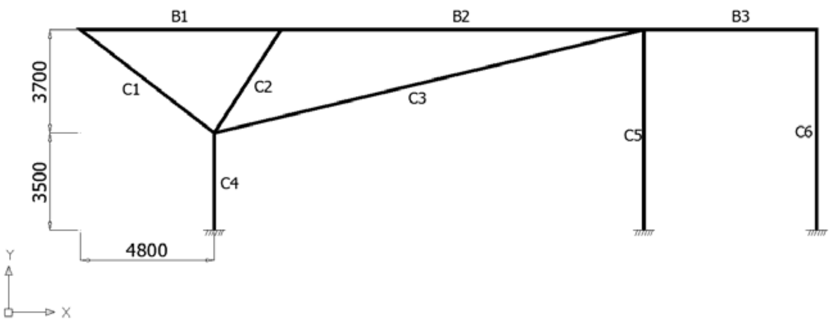
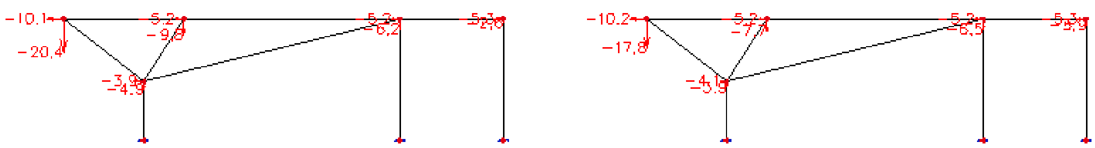


Fig 9-18 Optimum geometry

Following the tree column structure was improved by the purpose of aesthetic appeal and better force distribution by unit check. In the results, the stresses of B2 had exceeded the capacity of 355N/mm^2 that the member was yield and caused structure failure. Thus, the section of B2 has been changed from 300ASB249 to HE300M by optimization to upgrade the load bearing capacity. In addition, in order to get a pleasing view and consider the protection of the steel members, tree column elements C1, C2, C3 and C4 had been changed from HE beam sections to CHS (circular hollow sections) by unit check with the close properties so that the load bearing capacity would not change a lot.



Node deformation

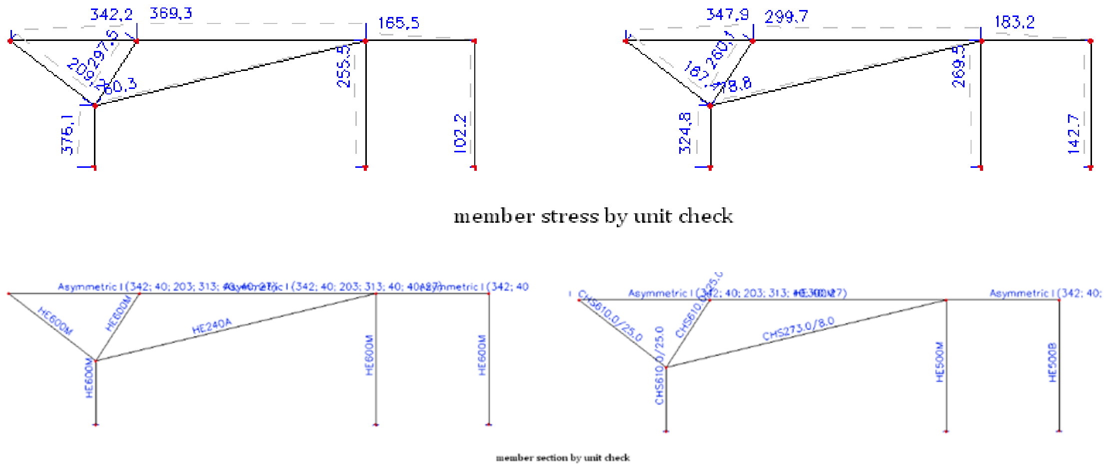


Fig 9-19 Optimizing tree column structure by unit check

Fig 9-20 Origin (left) and defined (right) tree column structure

Summary of Tree Column Structure

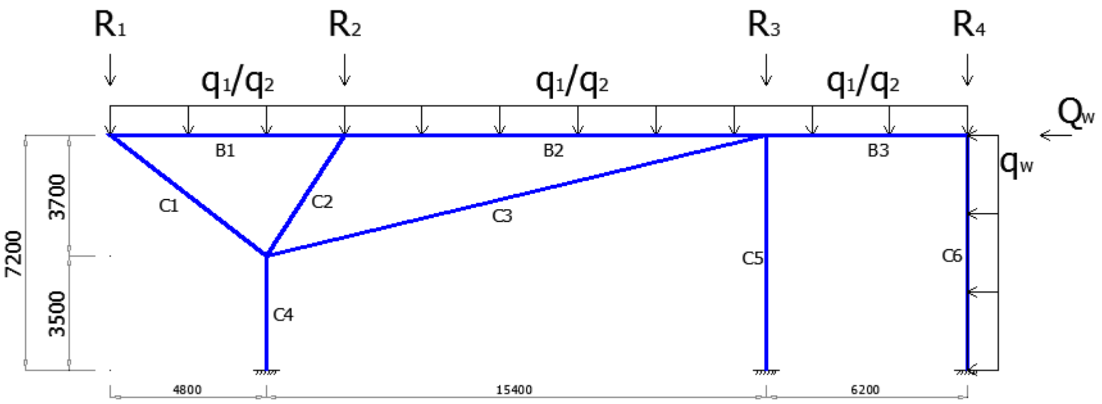


Fig 9-21 Structural model of tree column structure

Load Case			**Members**	
LC1	Self weight		B1	280ASB100
LC2[kN]	R_1	3426	B2	HE300M
	R_2	6364	B3	300ASB249
	R_3	5954	C1	CHS610/25
	R_4	2619	C2	CHS610/25
LC3 [kN/m]	q_1	34.44	C3	CHS273/8
LC4 [kN/m]	q_2	21	C4	CHS610/25
LC5	q_w[kN/m]	6.05	C5	HE500M
	Q_w[kN]	65.34	C6	HE500B

Table 9-4 Tree column structure load cases and member sections

9.3 Underground Structure – Commercial area

The underground structure has to support the whole loads from the offices and bus terminal. The futures and requirements of this area are,

- Every three to five years, the tenant of the commercial facility will change;
- According to the changeable tenants, the commercial area should be flexible enough for these requirements;
- The multi function in the area also demands the clearance in circulation and sights.

Considering these factors, the most feasible structure in the area was frame structure without solid structure like shear walls or cores. The column arrangement in the underground thus would follow the architectural design and the new structure above.

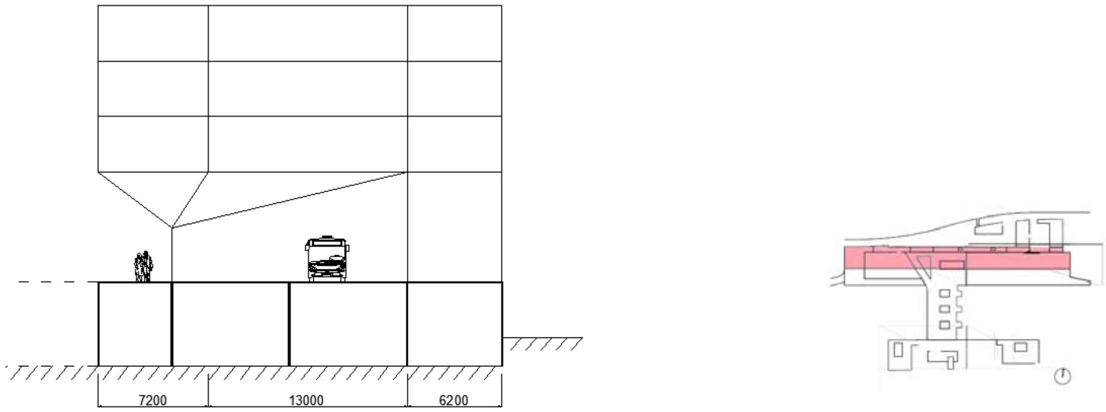


Fig 9-22 Underground architectural plan view and structural section view

Structural Design

The traffic loads on the bus terminal comply with the appendix of Dutch code NEN6723, load class 45 as mentioned before. Based on the bridge design consideration, the floor of the bus terminal where the busses drive were determined to use prestressed solid floor (Dycore BV) with the thickness of about 400mm directing in north-south. The rest of the floor where in the offices and platform was supposed to use plank floor with the same thickness of adjacent prestressed solid floor plate.

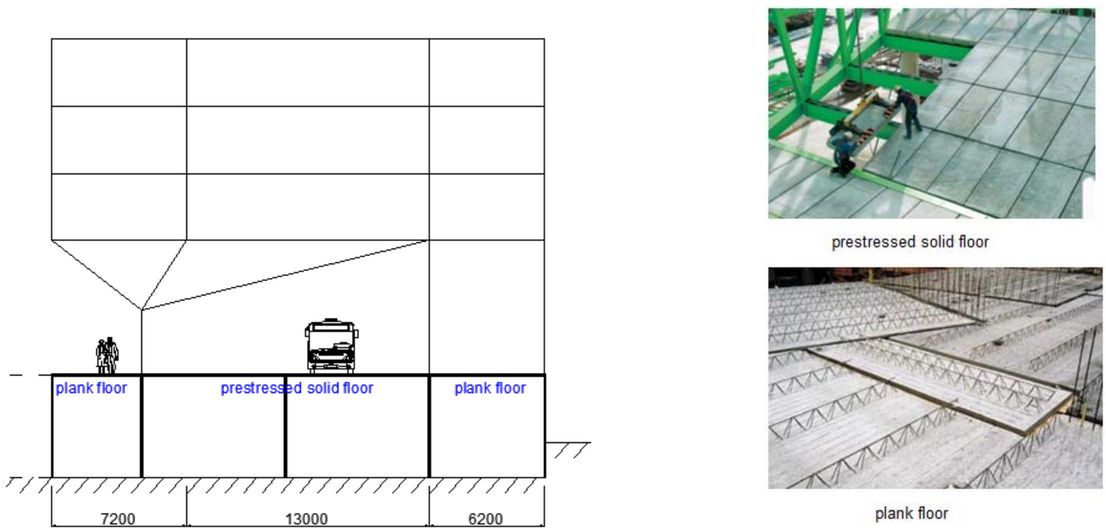


Fig 9-23 floor system of level 1

According to the selected floor system of the bus level, the load cases acting on the underground structure could be defined as,

	Platform (Axe C-D)	Bus Lane (Axe B-C)	Offices (Axe A-B)
Dead Load [kN/m^2]	4	9	4.6
Live Load [kN/m^2]	5	VK 450	2.5

Table 9-5 Load case of underground structure

Besides the column continuing from the above tree column on the bus terminal, another four rows of columns were set every 8.4-meter according to the direction and span of the slab. The beams that support these slabs were designed as HE450B in the middle row and HE360B in the rest area based on the loads defined above.

The lateral stability will be provided by these floor slabs by diaphragm action and horizontal forces will be then transferred to the columns and foundations eventually. A model combining all the parts of the building would be modeled and calculated in ESA PT to check these design.

9.4 Cantilever office part

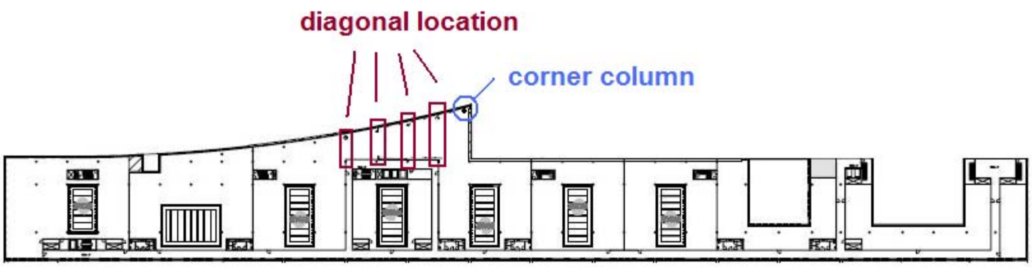
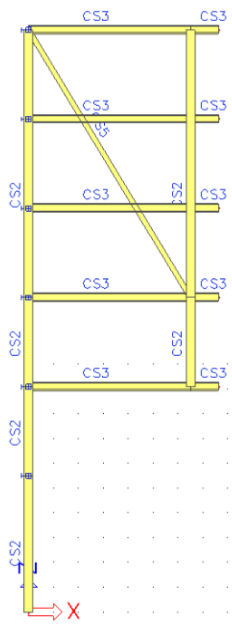


Fig 9-24 cantilever office part

In the architectural design, there was a curved office part at the west side from partial level 1 to 5. This had not been included in the previous integral structural design since idea was made that this part would be cantilevered from the main structure. The architect only designed one column to support this part, so the rest of area had to be cantilevered. For a structural solution, the only column was designed to be placed at the corner of this curved office area which had the largest span about 9m of the cantilever. The rest of the cantilever part was designed as 2D cantilever frame structure. A 2D frame structure with cantilever about 7.6m was modeled in ESA PT.



The arrangement and dimension of the structural elements, such as beams and columns followed the architectural plan. The beam members utilized ASB (CS2) and the column utilized CHS350/350/22 (CS2) which was the same as in the main structure. For those where large cantilever position, diagonals with the section of QHO200×16 (CS5) were used to provide lateral stability of the cantilever part and resist the vertical deformation in z-direction.

The results of the model showed that the general selection of the structural geometry and member section were feasible. The member stress didn't exceed the yield stress and the deformation of the nodes and beams were within the limit range.

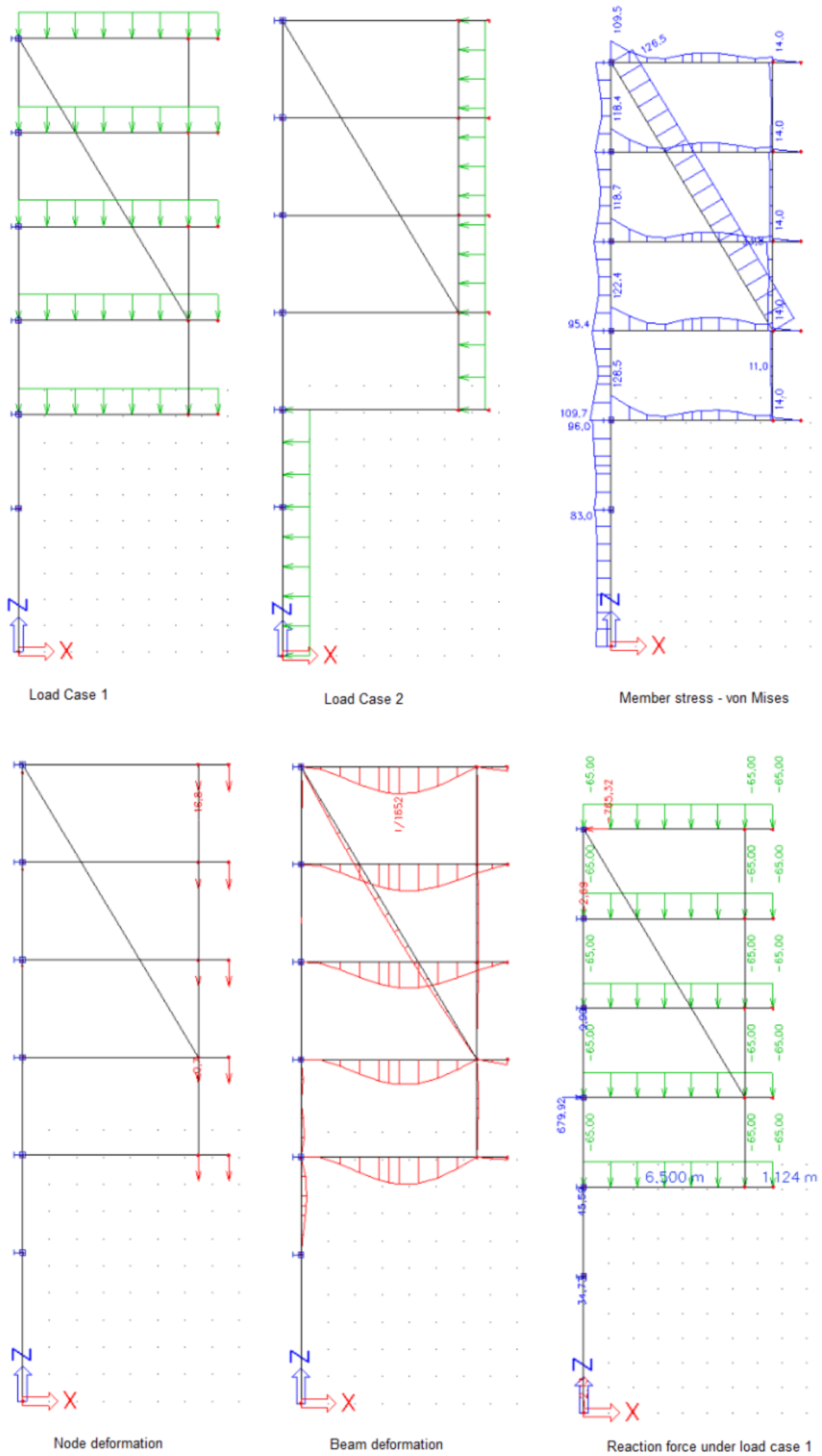


Fig 9-25 Results of 2D cantilever frame

9.5 The entire north side structure

9.5.1 General Check and Member Optimization

After having studied and designed the different parts of the north side, it was also necessary to make an entire model of them all to get an insight view of its behavior. A model of the entire north side part of Breda Central Station was then modeled and calculated in ESA PT with all the members sections, load cases defined in the previous chapters.

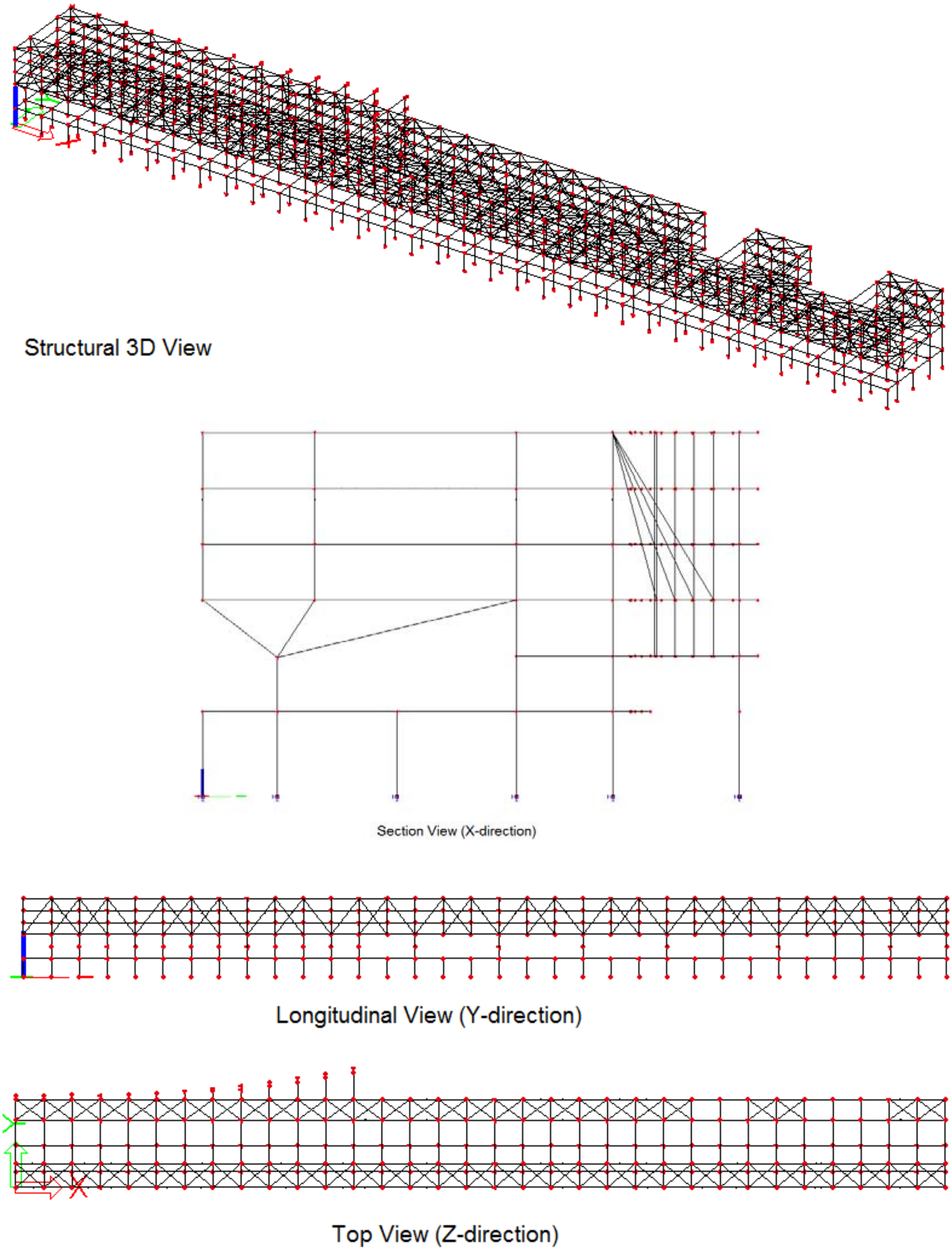
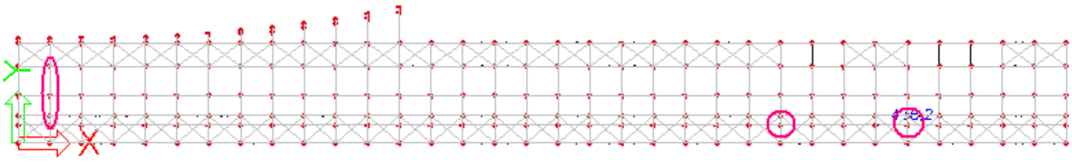


Fig 9-26 Structural model

Several conclusions could be drawn from the results in appendix 7.

- The results showed that the entire structure worked in general, no exaggerated deformation, internal forces and stresses occur. However, for some members, large internal forces and von Mises stress had been found, which meant those elements should be optimized to a suitable section.
- From the results, it could be discovered that members which had been fully loaded and where large internal forces and stresses occurred at some of the connection between floor girders and truss columns, and in the bottom of some tree columns.
- To avoid changing the geometry of the structure which might increase the difficulty and complex of the construction and structural behavior, measures of enlarge the section of these member with larger strength and stiffness had been taken. For those critical members, truss columns changed from HHS355.6/335.6/15.9 to HHS406.4/406.4/15.9, and tree columns changed from CHS610/25 to CHS610/40. The optimized structure was then recalculated in ESA PT.



member stress where exceeded yield stress

After optimizing the critical members, a new model was calculated again. Checks were then done based on the results. Detail results are shown in the appendix 7.

- The optimized structure indicates that maximum von Mises stresses are under the yield strength which means the all the members will not fail under global elastic stage.
- The deflection of the nodes is also under control that the horizontal deformation under SLS stage is 16.9mm and -19.4mm in x-direction and 16.3mm and -24.6 mm in y-direction. Both of them are smaller than the limitation of h/500≈45mm.
- The maximum relative beam deformation, 1/254, which is smaller than the 1/250 limitation is also acceptable.

9.5.2 Buckling Check

Besides the general check of deformation, internal forces and member stress of the structure, buckling was also a critical part governing the structural behavior. According to Eurocode 3, the method of analysis of non-sway braced steel structure is suggested to use first order elastic analysis. So the buckling check would also be under first order elastic analysis, with regardless to the second order effect.

The maximum normal force occurs in the column underground with the value of 11938kN. Buckling check has been done for this CHS610/25 column which shows that the compression force has not exceeded the buckling force. Although buckling doesn't happen in the member where the maximum normal force is, other members may probably buckle due to different member properties. Thus, a buckling check table shown below has been made to check the maximum internal forces in every kind of cross section.

Cross Section	**Maximum Internal Force (ULS)**		**Length**
	N- [kN]	N+ [kN]	L [m]
HHS355.6/335.6/15.9	-2093.84	1363.54	10.8
HHS406.4/406.4/15.9	-2467.70	1947.27	10.8
CHS610/25	-11974.87	-	3.5
CHS610/40	-11966.62	-	5.49

HE260B	-3473.64	1137.61	13.68
HE300B	-3451.87	-	13.68
300ASB249	-481.66	4700.35	13 / 7.2
HE500M	-6581.05	-	5.49
HE500B	-3540.95	71.99	5.49

Table 9-6 maximum internal force of different cross section

Cross section	L [m]	I [m⁴]	N_{cr} [kN]	A [m²]	λ	B-Curve	χ	N_{bd} [kN]
SHS350/350/19	10.8	4.34E-04	3.09E+04	0.0242	0.528	c	0.83	7131
SHS350/350/22	10.8	5.29E-04	3.76E+04	0.0309	0.540	c	0.82	8995
CHS610/25	3.5	1.97E-03	1.33E+06	0.0459	0.111	c	1	16295
CHS610/40	5.49	2.92E-03	8.03E+05	0.0716	0.178	c	1	25418
HE260B	13.68	1.49E-04	6.60E+03	0.01184	0.798	b	0.73	3547
HE300B	13.68	2.52E-04	1.12E+04	0.01491	0.688	b	0.79	4182
300ASB249	7.2	5.91E-04	9.45E+04	0.0344	0.359	b	0.935	11418
HE500M	5.49	1.62E-03	4.46E+05	0.0344	0.166	b	1	12212
HE500B	5.49	1.07E-03	2.94E+05	0.0239	0.170	b	1	8485

Table 9-7 member in compression buckling manual calculation

Cross Section	N- [kN]	N_{bd} [kN]	Check
HHS355.6/335.6/15.9	-2093.84	7131	ok
HHS406.4/406.4/15.9	-2467.70	8995	ok
CHS610/25	-11974.87	16295	ok
CHS610/40	-11966.62	25418	ok
HE260B	-3473.64	3547	ok
HE300B	-3451.87	4182	ok
300ASB249	-481.66	11418	ok
HE500M	-6581.05	12212	ok
HE500B	-3540.95	8485	ok

Table 9-8 member in compression buckling check

The above buckling check indicates that all the members in compression have been verified against buckling. Attention has to be paid that the buckling check for member in compression was made under first order elastic stage according to Eurocode 3.

10 Final Structure

In this chapter, the final structure combined with all the parts studied in the previous chapters would be elaborated in ESA PT. The cantilever part at the west side of the station complex which is supported by additional beams and columns is also included. The results would show if the structure works or not.

10.1 Structural System

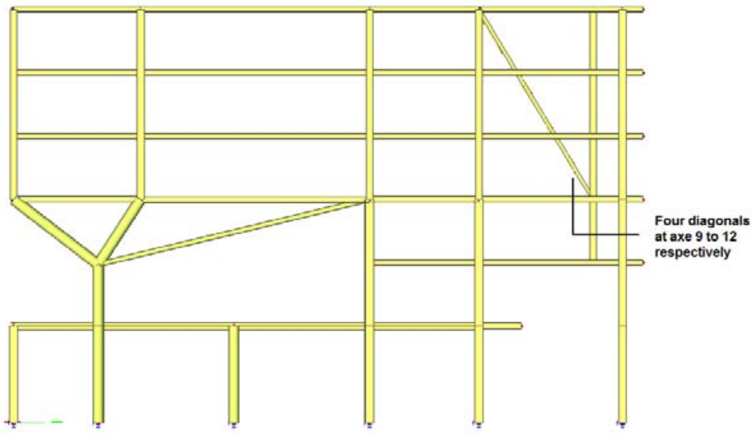
The north side of the Breda Central Station complex has multi function of commercial service and passenger tunnel at underground continuing to the ground floor level, bus terminal at level 1 to 2, and offices at level 3 to 5. The multi function arrangement and complicated architectural design of this part result in highly demand structural solution. The entire dimension of this part of the station is 277.2m×26.4 m(structurally) in plan, and 23.49m high (from NAP -0.21m to 23.28m). According to the previous study, the structure has been divided into three parts in accordance with the function.

The structural concept is strongly affected by the goal of the structural solution of this thesis which is to achieve as much as possible space and fewer supports on bus terminal to provide more clearance to the passengers and more aesthetic view for the public. Therefore, truss structure is used in the office part to realize large span in the longitudinal direction. Four 277.2-meter long planar trusses locate along east-west direction spacing 6.2m, 13m and 7.2 complying with the architectural plan. Two of them locate on the facades and two of them transverse through the body of the offices and are partially expressed internally through atriums. These super long trusses are divided into single modules with the length of 25.2m (8.4m×3) and 33.6m (8.4m×4) at different positions.

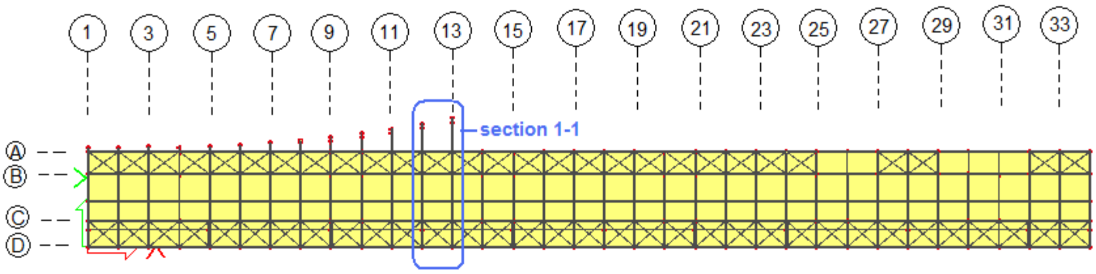
Vertically rigidly connected columns provide lateral stiffness for the wind load and lateral stability for the trusses. To minimize the weight of the office structure, slim floor system has been used combined with ASB floor girders and Comflor floor system, and it also performs diaphragm action to resist the horizontal loads. In the ESA PT model, the floor system was substituted by horizontal bracings. Therefore, the gravity loads will proceed from floor girders to the trusses and then to the support structure on the bus level. The horizontal loads will transfer from truss on the facade to the floor system and to the columns vertically.

On bus terminal level, tree column structure at the position where the trusses in the offices span is the main structure that supports the offices above. The transverse located tree column structure not only provides the lateral stability of the whole building, but also reduces the number of the columns in the bus terminal for a clear and appeal view. These tree columns also result in light weight compared to the space frame concept.

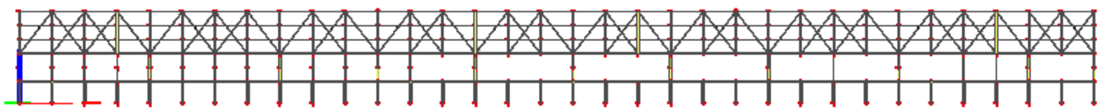
As traffic loads act on the bus level, solid prestressed floor system is used in that area. And due to the flexible requirement of the underground commercial use, frame has been selected for the underground structure to support the above bus terminal and offices.



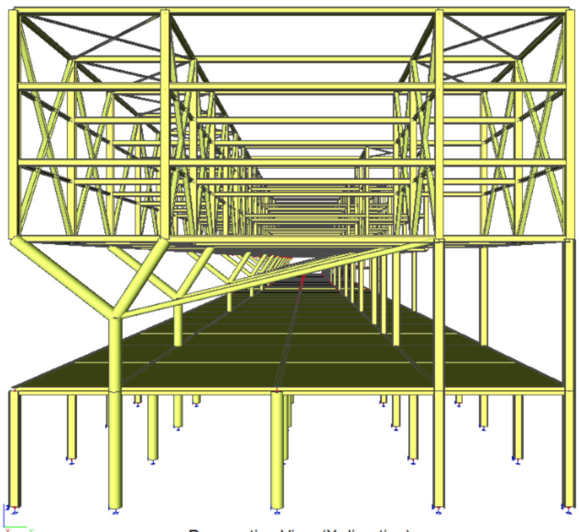
Section 1-1



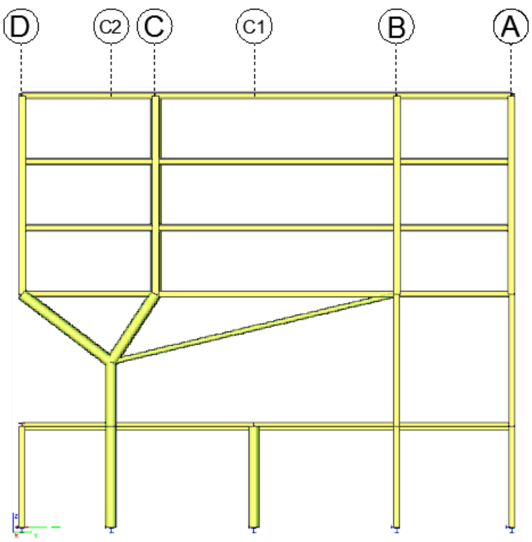
Top View (Z-direction)



Longitudinal View (Y-direction)



Prespective View (X-direction)



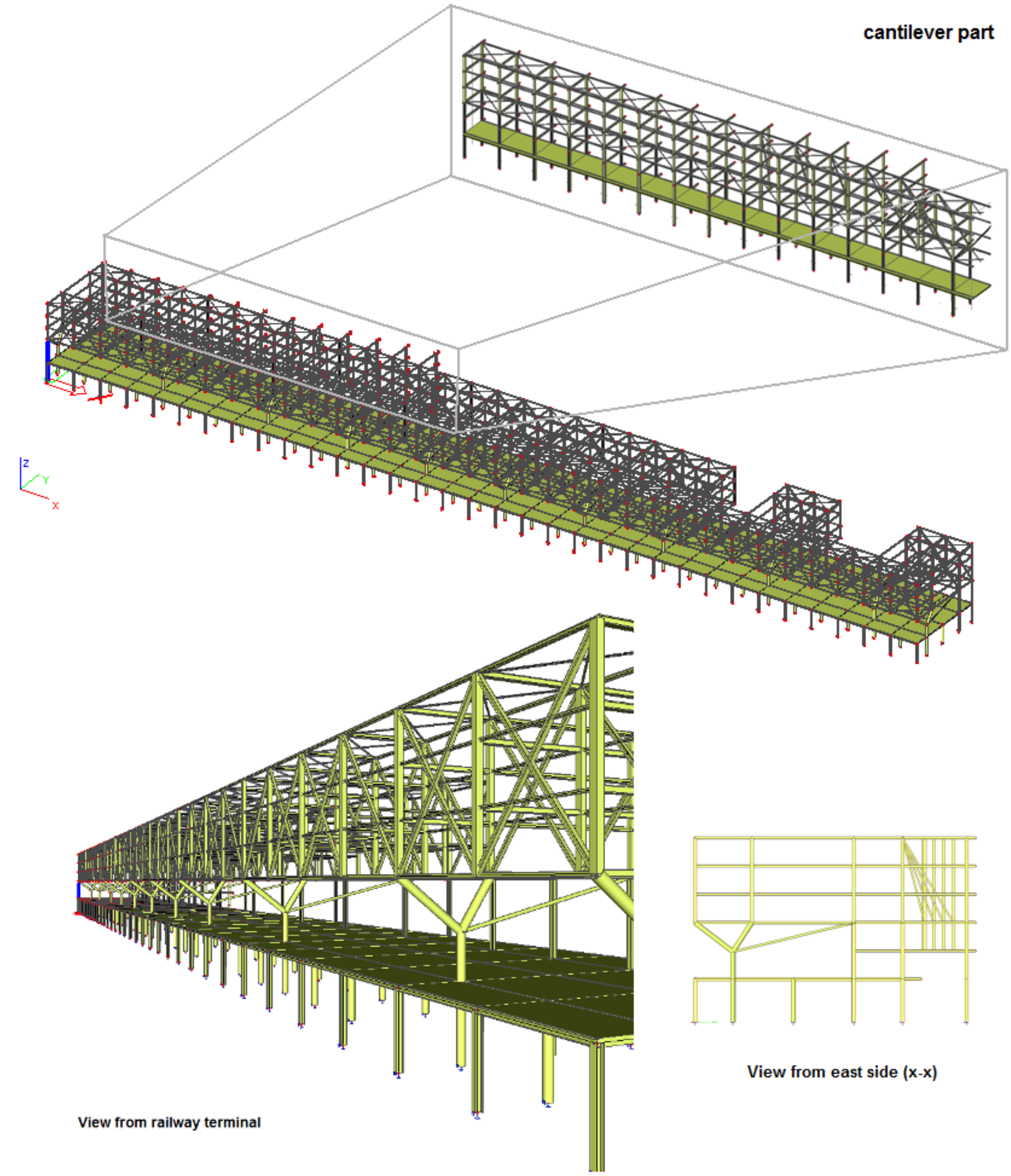


Fig 10-1 Final structure ESA PT model

10.2 Load Cases

Four general load cases have been considered and combined in the model in accordance with that defined in Chapter 7.

Self weight - based on the structural element properties computed by program
Dead load - transformed from surface force to line force on beam in model
Live load - partial transformed from surface force to line force on beam in model; the rest keeps surface force on 2D member in model
Wind load - transformed from surface force to line force on vertical columns and roof beams in model

Load Case	*Force Type*	*Location*	*Value*	*Unit*	*Direction*
LC1 Self Weight	Line Force	all	see Table 8.1	kN/m	Z
LC2 Dead Load	Line Force	all		kN/m	Z
LC3 Live Load	Line Force	office		kN/m	Z
	Surface Force	bus terminal		kN/m²	Z
LC4 Wind Load	Line Force	east & west facade		kN/m	X
	Line Force	north & south facade		kN/m	Y
	Line Force	roof		kN/m	Z

Table 10-1 Load cases in the model

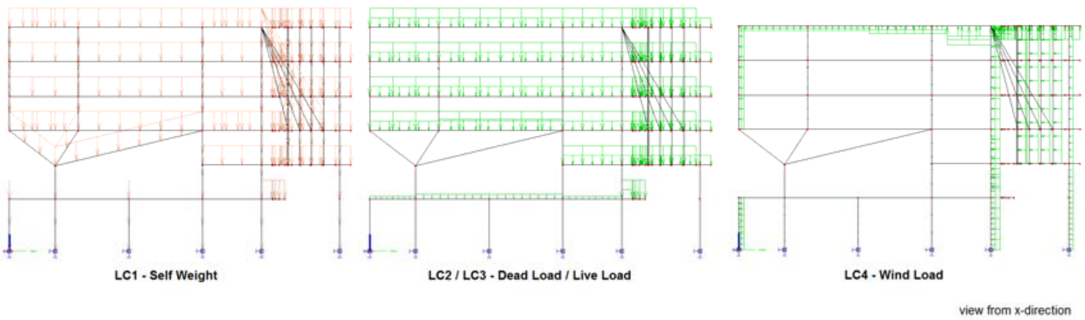
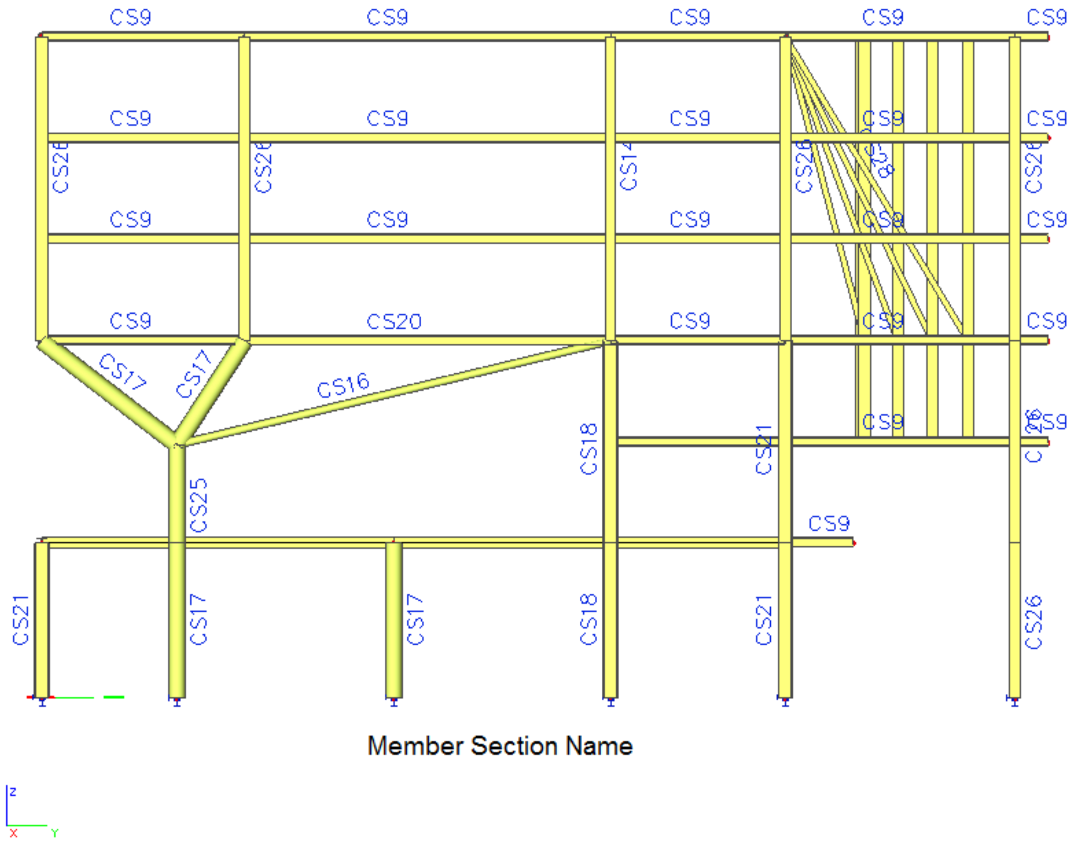


Fig 10-2 Load cases schematization

10.3 Members

All the floor girders utilize 300ASB249 beams for slim floor system, except the 13m long intermediate beam at level 3 due to the behavior of the tree column structure on the bus terminal. These beams have been changed to HE300M to fulfill the structural requirements of tree column structure on the bus level. The top and bottom chords of the trusses are constructed using HE240A and the diagonals are HE260B. The columns in the trusses have been used the rectangular hollow sections HHS355.6/355.6/15.9 as mentioned before to be stiffer in both directions. The horizontal bracings at the roof and bottom of office level use HE100A to represent the diaphragm action of the floor system.

The structures on the bus terminal, tree columns, have CHS610/25 for the short inclined beams and columns beneath them, and CHS273/8 for the long inclined beams. The rest of the columns along axis A and B utilize HE500M and HE500B respectively. The floors span in north-south direction on the bus terminal using solid prestressed floors and plank floor system. This results in the beams on the bus level directing east-west. The beams in the middle row (axis C1) will use HE450B and the rest rows of the beams will use HE360B. The columns that support these beams will use CHS610/25 for axis C1 and C2, HE500B for axis A and D, and HE500M for axis B, mainly continuing with the above structure. As the bottom of the whole building, these columns are supposed to be fixed at the bottom.



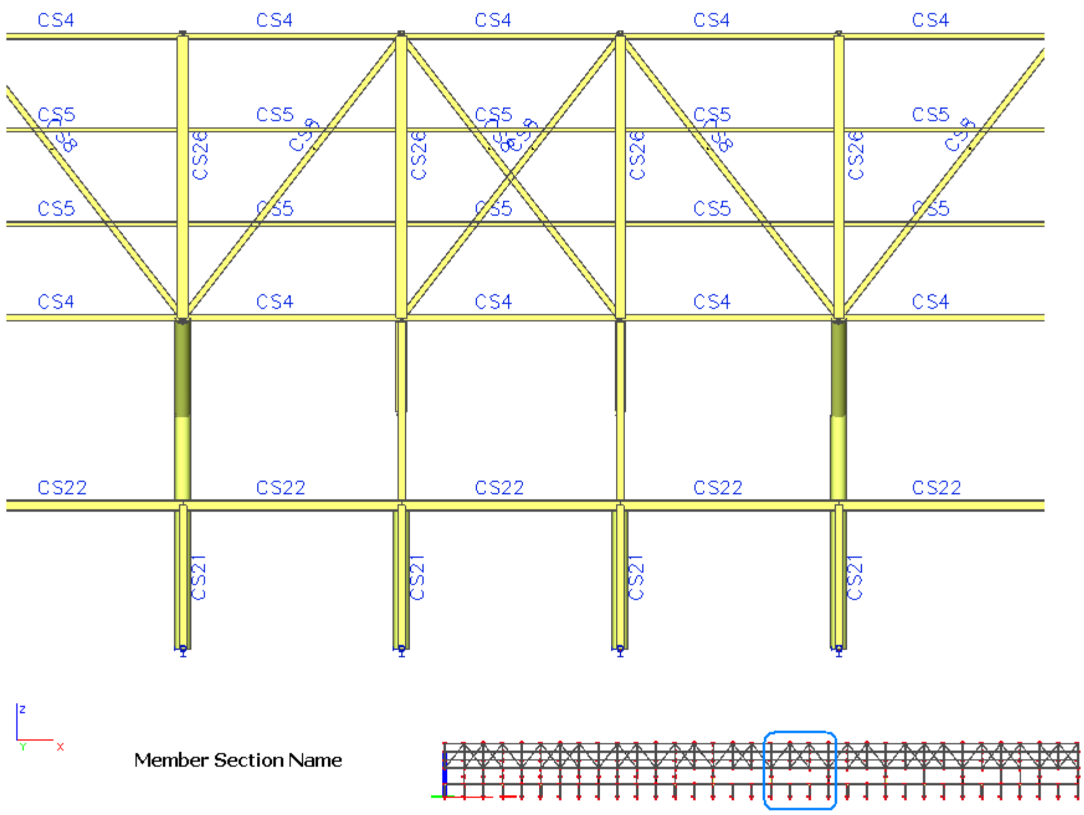
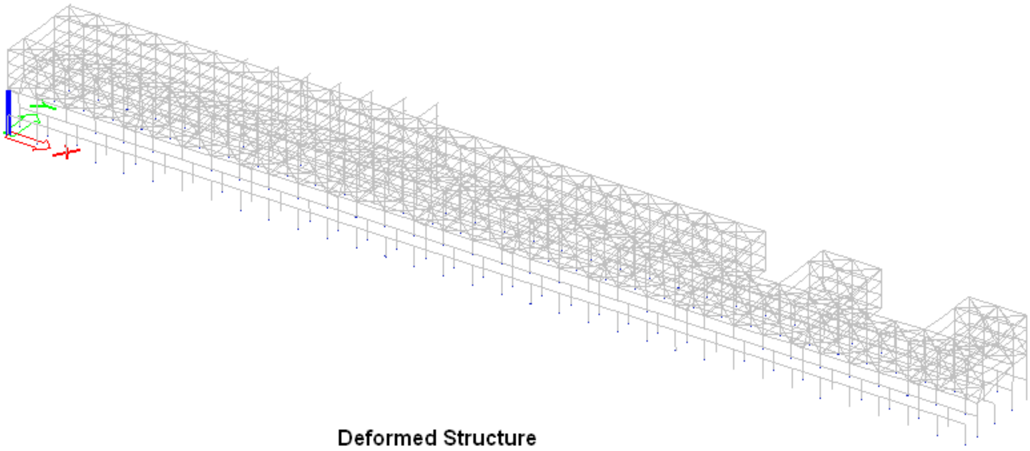


Fig 10-3 Member sections

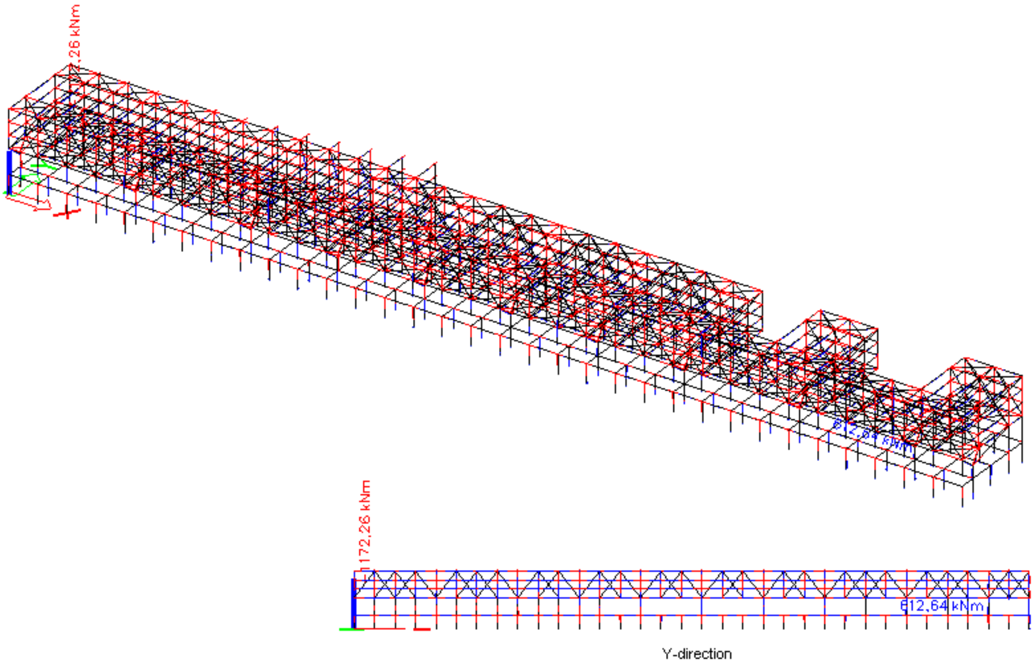
	Structural Part	*Length [m]*	*Name*	*Designation*	
Office	Truss Chord	8.4	CS4	HE240A	
	Truss Diagonal	13.68	CS8	HE260B	
	Truss Column	10.8	CS14/CS26	HHS355.6/355.6/15.9	HHS406.4/406.4/15.9
	Floor Girder	6.2/13/7.2	CS9/CS20	300ASB249	HE300M
	End Beam	8.4	CS5	HE180A	
	Horizontal bracings	10.4/11.1	CS15	HE100A	
	Diagonal @cantilever	14.7~15.8	CS29	QHO200×16	
	Column @cantilever	3.6	CS26	HHS406.4/406.4/15.9	
Bus Terminal	Tree Column	4.41/6.06	CS17/CS25	CHS610/25	CHS610/40
	Column	7.2	CS18	HE500M	
		7.2	CS26	HHS406.4/406.4/15.9	
Underground Commercial Area	Beam	8.4	CS22	HE360B	
		8.4	CS23	HE450B	
	Column	5.49	CS17	CHS610/25	
		5.49	CS18	HE500M	
		5.49	CS21	HE500B	
		5.49	CS26	HHS406.4/406.4/15.9	

Table 10-2 Summary of member designation

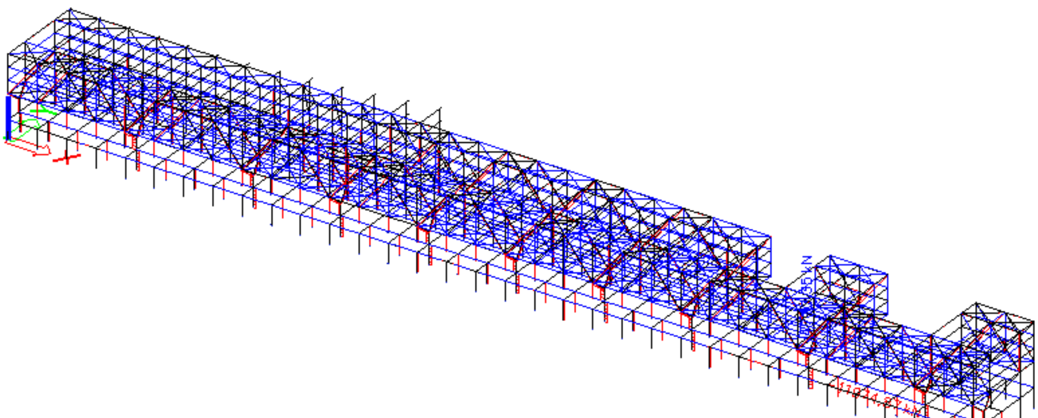
10.4 Calculation Results



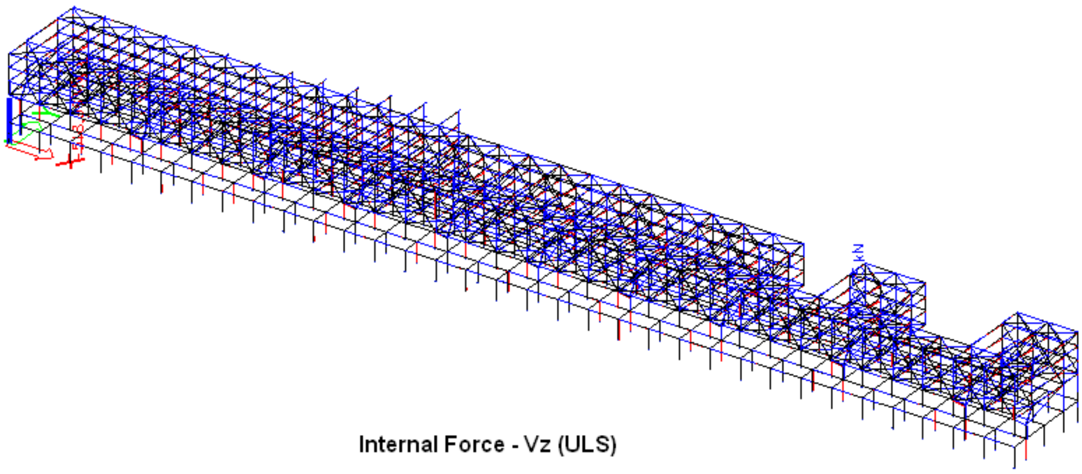
Deformed Structure



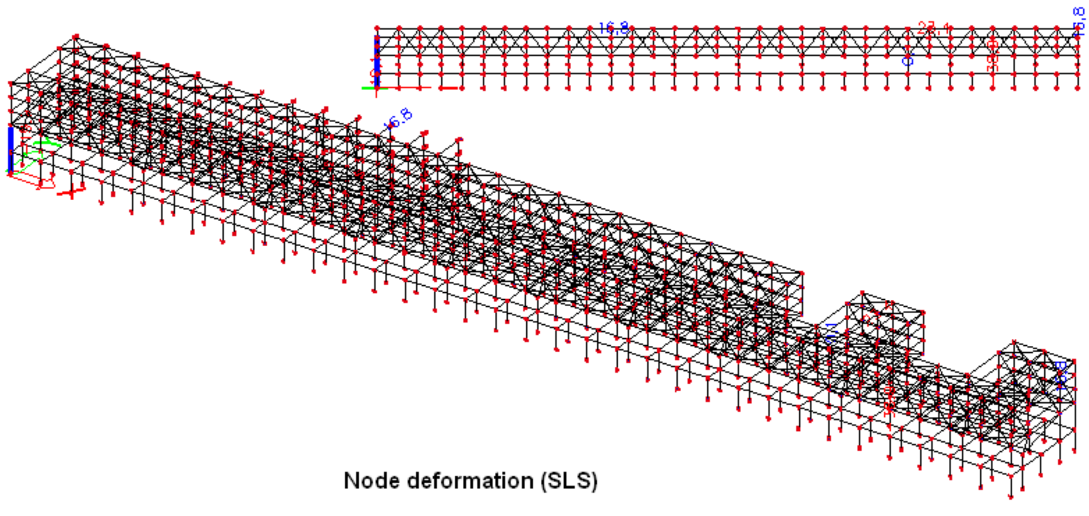
Internal Force - My (ULS)



Internal Force - N (ULS)



Internal Force - Vz (ULS)



Node deformation (SLS)

1. Load cases

Name	Description	Action type	LoadGroup	Load type	Spec	Duration	Master load case
LC2	Dead Load	Permanent	LG1	Standard			
LC4	Wind Load	Variable	LG3	Static	Standard	Long	None
LC1	Self Weight	Permanent	LG1	Standard			
LC3	Live Load	Variable	LG2	Static	Standard	Long	None

2. Combinations

Name	Description	Type	Load cases	Coeff. [1]
SLS	SLS	Linear - serviceability	LC2 - Dead Load	1,00
			LC4 - Wind Load	1,00
			LC1 - Self Weight	1,00
			LC3 - Live Load	1,00
ULS	ULS	Linear - ultimate	LC2 - Dead Load	1,20
			LC4 - Wind Load	1,50
			LC1 - Self Weight	1,20
			LC3 - Live Load	1,50

3. Internal forces on member

Linear calculation,Extreme : Global, System : Principal
Selection : All
Combinations : ULS

Member	Case	dx [m]	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
B4962	ULS/1	0,000	**-11974,87**	-24,40	-17,75	0,26	26,34	-15,30
B4277	ULS/1	0,000	**4700,35**	0,05	100,84	-0,06	-81,05	-5,76
B4911	ULS/1	0,000	-10238,21	**-567,43**	17,75	15,13	46,23	747,76
B556	ULS/1	0,000	-471,09	**247,81**	-3,79	1,40	1,38	-458,27
B4076	ULS/1	13,000	232,57	1,34	**-518,96**	0,14	**-1172,26**	8,51
B4280	ULS/1	0,000	35,34	-0,01	**509,75**	-0,02	-1131,42	0,78
B4952	ULS/1	0,000	-4359,61	-154,63	134,55	**-165,60**	-526,09	258,71
B4951	ULS/1	0,000	-2220,48	54,71	17,54	**183,22**	61,24	-191,56
B4899	ULS/1	3,500	-9453,75	-328,21	153,13	-36,95	**612,64**	-812,67
B4911	ULS/1	3,500	-10261,36	-567,43	17,75	15,13	108,35	**-1238,26**
B4923	ULS/1	0,000	-9317,89	-536,10	1,01	28,45	5,96	**798,87**

4. Deformation of nodes

Linear calculation,Extreme : Global
Selection : All
Combinations : SLS

Node	Case	Ux [mm]	Uy [mm]	Uz [mm]
N1591	SLS/2	**-19,1**	-5,2	-6,1
N1654	SLS/2	**16,8**	-7,0	-5,4
N1625	SLS/2	9,1	**-23,4**	-2,9
N370	SLS/2	-1,1	**16,8**	-9,6
N1596	SLS/2	8,2	-19,1	**-38,9**
N1866	SLS/2	0,0	-1,4	**0,1**

5. Stress

Linear calculation,Extreme : Global
Selection : All
Combinations : ULS

Member	Case	dx [m]	Normal - [MPa]	Normal + [MPa]	Shear [MPa]	von Mises [MPa]	Fatigue [MPa]	Kappa [1]
B4076	ULS	13,000	**-335,8**	351,1	**85,6**	351,1	0,0	0,00
B624	ULS	0,000	**0,0**	102,9	4,6	102,9	0,0	0,00
B553	ULS	1,800	-29,6	**0,0**	11,1	32,4	0,0	0,00
B3242	ULS	0,000	-294,5	**364,9**	45,5	**364,9**	0,0	0,00
B2800	ULS	0,000	-110,2	0,0	**0,0**	110,2	0,0	0,00
B5432	ULS	1,100	0,0	0,0	0,0	**0,0**	0,0	0,00
B553	ULS	0,000	-94,0	43,7	11,1	94,0	**0,0**	**0,00**

6. Relative deformation

Linear calculation,Extreme : Global, System : Principal
Selection : All
Combinations : SLS

Case - combination	Member	dx [m]	uy [mm]	Rel uy [1/xx]	uz [mm]	Rel uz [1/xx]
SLS/2	B4952	0,000	**-11,8**	**1/374**	0,0	0
SLS/2	B4894	0,000	**10,7**	1/1478	0,0	0
SLS/2	B4892	0,000	8,0	**1/549**	0,0	0
SLS/2	B4074	6,067	0,2	1/10000	**-53,9**	1/510
SLS/2	B5360	0,000	0,0	0	**23,7**	**1/535**
SLS/2	B4533	6,500	0,2	1/10000	-46,9	**1/277**

10.5 Compare with current design

10.5.1 Structure Weight Comparison

Besides designing a new structure for the north side of Breda Central Station for goal of winning more free space and clearance at the bus terminal and appeal view, comparison the weight of the whole structure and the amount of the steel used in the current design and in this thesis is also an interesting point.

The self weight of the final structure model (with cantilever) is 22624.7kN got from ESA PT, in which the weight of the offices and bus terminal structure is about 17280kN. Compared to the self weight of space frame and rigid frame structure designed in Appendix 4 which is already 39806kN, excluding the large columns supporting the space frame on the bus terminal. If estimating the space frame were supported by CHS610/25 columns, the total weight of the space frame structure would be about 40836kN. Compared to that of 17280kN of the truss structure, it was almost 2.5 times. This demonstrates that the truss structure with tree column is much more effective and lighter.

Then comparison between the truss and tree column structure and the current design by DHV is made.

Current Structure (DHV)

- Steel amount of 3D model calculated by MicroStation V8
  $\sum W = 8.13 \times 10^5$ kg
  In which excluding those members (program problem): a) columns ϕ457*12.5 and ϕ355.6*10; b) diagonal ϕ244.5*8; c) beam k200*150*8; d) all the strips. Considering these members manually, the total amount of steel is about,
  $\sum W = 8.13 \times 10^5 + 4 \times 10^5 = 12.13 \times 10^5$ kg

- Concrete amount of 3D model calculated by MicroStation V8
  $\sum V_c = 2.78 \times 10^{12}$ mm^3
  In which only concrete beams and columns were counted, excluding the concrete floors.
  $\sum W_c = 2.78 \times 10^{12} \times 10^{-9} \times 2400 = 6.672 \times 10^6$ kg

- Total structural elements amount
  $\sum W_{total} = \sum W_s + \sum W_c = 7.885 \times 10^6$ kg

New structure in this thesis

- Self weight of office part in ESA PT
  $\sum W_o = 16423$ kN
- Self weight of final structure model with cantilever in ESA PT
  $\sum W = 22625$ kN

Considering this model is just the general integral design while the model of DHV in MicroStation is already under detail design stage, therefore an adjustment factor 1.2 has been used for a relative fair comparison.

$$\sum W_{o,t} = 1.2 \times 16423 = 19708 \text{ kN} = 1.9708 \times 10^6 \text{ kg}$$
$$\sum W_{total} = 1.2 \times 22625 = 27150 \text{ kN} = 2.715 \times 10^6 \text{ kg}$$

All the floor weight in these two designs was not included in the comparison for two reasons. First, in the general design stage, the area of the slim floor system used in the new design has not been defined yet, so it’s not easy to get a relative accurate weight of the floor system; second, the use of the slim floor system (2.79+0.17=2.96 kN/m^2 for normal concrete with 290mm thick slab) is supposed to be lighter than the current composite plank floor system (3.1 kN/m^2). So the floor weight will not influence the result of the comparison. Moreover, other dead loads have not been included in the comparison either, since they are the same in these two designs.

From the results above, several conclusions can be drawn,

	Current structure [kg]	*New structure (truss + tree column) [kg]*
Steel structure (offices)	1.213×10^6	1.9708×10^6
Rest structure	6.672×10^6	0.7442×10^6
Entire building	7.885×10^6	2.7150×10^6

Table 10-3 Structure weight comparison

Although the new structure of the office part is a bit heavier than the current structure, the total weight of the entire building is much lighter for almost 3 times. This indicates that the steel structure has the great advantage in weight. However, one of the reasons of using the concrete is its low cost compared to the steel structure, it doesn't make sense to compare the weight of the two structural solutions only. Therefore, it's also suggested to calculate and compare the total cost of the building. Due to lack of the sources of the unit cost of concrete, it's difficult to estimate the total cost of these two designs. This should be done by the cost management, and has not been included in this thesis.

10.5.2 Structural Area Comparison

As the goal of this thesis is to reduce the amount of columns on the bus terminal, so another important aspect to review the new structure by the current structure is to compare the structural area between these two structures. Therefore, the projection of the main structural elements, columns, on the bus terminal and underground commercial area of these two structures has been counted.

Note: For column section like HE500B, the structural area was considered as a rectangular area by width×height (300×500). Similarly, the structural area of the circular hollow column was counted by full circular area instead of section area.

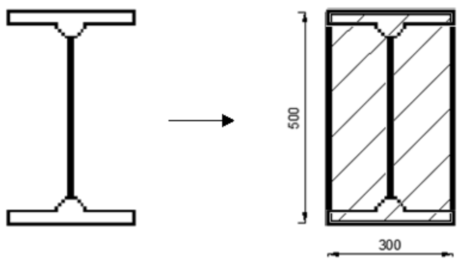


Fig 10-4 column counted structural area

Current Structure

Bus Terminal			
	Member Section	Number	Subtotal Area (m^2)
	ϕ457	14	0.164
	ϕ600	40	11.31
	ϕ500	30	5.89
	600×400	11	2.64
	550×800	2	0.88
	350×1000	1	0.35
	550×550	24	7.26
	Σ	**122**	**28.49**

Underground			
	Member Section	Number	Subtotal Area (m^2)
	ϕ550	2	0.475
	ϕ457	10	1.772
	ϕ600	11	3.11
	ϕ400	3	0.377
	ϕ650	38	12.61
	ϕ450	3	0.477
	550×800	3	1.32
	550×550	17	5.143
	600×400	11	2.64
	600×600	32	11.52
	700×550	2	0.77
	400×400	1	0.16
	650×650	2	0.845
	400×700	1	0.28
	1000×1100	1	1.1
	Σ	**137**	**42.60**
Total Σ		**259**	**71.09**

Table 10-4 Structural area of DHV structure

New Structure

Bus Terminal			
	Member Section	Number	Subtotal Area (m^2)
	ϕ610	11	3.22
	HE500B	14	2.1
	HE500M	11	1.76
	Σ	**36**	**7.08**
Underground			
	Member Section	Number	Subtotal Area (m^2)
	ϕ610	68	19.87
	HE500B	68	10.20
	HE500M	34	5.45
	Σ	**170**	**35.53**
Total Σ		**206**	**42.60**

Table 10-5 Structural area of new structure

11 Conclusions and Recommendations

This chapter presents the main conclusions and recommendations for this thesis as an alternative design of the north side of the new Breda Central Station, which aims to reduce the amount of columns on the bus terminal with the goal of getting more open space. The conclusions are the summary of the most important ones that have been drawn in this thesis. After that, recommendations based on the conclusions for the thesis and further study will follow.

11.1 Conclusions

Alternatives
Based on the architectural design of the Breda CS, the program of requirements and the goal of this thesis, five structural alternatives have been designed initially. To select a most suitable one among these alternatives, MCA (multi criteria analysis) has been applied to as guidance for the selection.

The 1st and 2nd alternative – arch structure have not been selected since, although they provide clear images to reduce the columns on the bus level by increasing the span of the office part, the disturbance to the architectural design and sensitive force distribution cause much more difficulties. The 3rd alternative – frame structure with braced cores utilizes the atria in the offices for structural use which shows a good combination between the architectural design and structural solution. Nevertheless, the combination is not so perfect that the braced cores are not able to reach the foundation due to the functional requirements of the design which greatly weakens this alternative, so that it was not chosen either. The 5th alternative – space frame is a quite creative one to realize the large open space on bus terminal by least supports, and also the general calculation in appendix 4 indicates the feasibility of space frame carrying massive imposed loads – three-storey offices on it. However, it was still not selected as the final structure since its extremely heavy self weight and the complicity of the connections and buildability.

The 4th alternative – truss structure was selected to be elaborated as the final structure because of its own advantages and preponderance over the other alternatives. It not only reduces the columns on the bus terminal by using truss structure in the offices but also realizes the light weight structure compared to the space frame. Besides this, the principle of the truss that only tension and compression governing the structure shows advantages than the arch structure which is very sensitive to the loads, and will not result in large scale supports as well. Thus, it was concluded this alternative the most suitable one with regard to the objective and design.

Final Truss Structure
The direction of the truss has been determined firstly. The initial idea was to set the truss structure in the offices transversely so that the bus terminal could be totally freed in the section or achieve partially cantilever. However, this has been negated because considering the scale of the structure and the span of the floor; the transverse trusses have to be arranged every 8.4m that is not welcomed by either the clients or the users. The big diagonals locate greatly influences the flexibility of the offices just like irremovable shear walls. Therefore, the direction of the truss was then determined to be set longitudinally. Although this cannot fully free the bus terminal as the transverse one, it is solved by optimizing the structure on the bus terminal. The pattern of the truss has been defined by pattern optimization and member optimization. The Howe truss without tension members at two sides of the truss behaves better than Pratt truss without compression members and similar to the original one but saves more members. The member optimization by unit check between original truss and Howe truss shows that the original truss is able to use less material (lighter weight) than Howe truss although it has fewer members. But the Howe pattern was still chosen because it's only a little bit heavier than the other one, but it provides more clearance.

The slim floor system has been chosen for this project due to several considerations including the weight, the floor thickness and the flexibility for the openings. In addition, in the structural model, the loads acting on the structure was applied by means of line force, so the floor slab was not modeled to simplify the design. Horizontal bracings were introduced in the top and bottom of the office part to substitute the floor slab to resist the horizontal loads. The results indicate that the assumption and estimated section of the bracings are sufficient.

Due to the extremely long length of the building and the span of the truss, the design of the structure has been divided into several modules. This was made to simplify the design and model in the computer program, while the structure was then designed to be built continuously without expansion joints considered. And the results of the design of the single module and complete structure afterwards have been verified effectively that no exaggerated deformation, internal forces and stresses have been found. But some member strengths are reached at several locations. These elements are then optimized.

The tree column structure on the bus terminal was designed to save more columns there besides the contribution of the truss structure in the offices. The geometry of the tree column structure was determined by form finding. When lifting the intersection of the tree column vertically which increased the length of the vertical column (trunk of the tree), the structure behaved less stable and the force distribution became worse. The minimum length of the trunk was determined as 3.5m by the functional requirements for the passengers and busses to go through. The vertical and horizontal deformation was strongly influenced by moving the intersection of the tree column horizontally. The horizontal deformation changed from positive to negative when moving the intersection from left to right. The vertical deformation also varied from the different position of the intersection, but it didn't decrease gradually when moving the intersection from left to right like horizontal deformation. Fortunately the critical point of the horizontal and vertical deformation happen to be at the almost the same point. So the horizontal position of the intersection of the tree column was then defined at 4.8m from the left side. The results of these fixed (moment-resisting) tree column structures have been verified effectively, only several elements have to be optimized since the stresses of them have yield.

The curved cantilever offices towards outside is also a part of the structure that only one column is designed by the architect to support it at the corner. The rest of this part remains cantilever by using diagonals across three-storey to control the deformation. And the results indicate that the main building structure can resist the horizontal loads caused by the diagonals.

The results of the final structure under linear elastic calculation show that the structure is effective and sufficient generally. The deformation of the structure under serviceability limit stage is within the limitation and the internal forces and member stresses of most of the members are strong enough. Only the strength of some crucial members has reached, and these members have been optimized to a stronger section then. The final results indicated that after optimizing the sections, the deformation and the maximum internal forces increase a bit instead, but they are still under control. The maximum negative normal stress has decreased under the yield value, while the maximum positive normal stress still exceeds approximate 3% where in the floor girders. Although the ASB floor girder in the slim floor have been used largest profiles, the structure can be considered as safe in conceptual design stage.

Comparison
The weight of office structure is about 40% heavier than the current design. Nevertheless, due to designing the whole structure in steel and reducing the columns on the bus terminal, the total weight of the entire north side structure is only 1/3 of the current structure. This shows great advantage in using steel instead of concrete. The concrete has the advantage of low cost, however, because cost of the concrete structure is determined by the unit cost of different profiles and lack of the sources, the comparison between the costs of these two structures cannot be concluded in this these.

On the other hand, by comparing the structural area of these two structures, the goal of this thesis has reached. Besides more open space on the bus terminal, the structural area of the columns reduces by 40% in the new structure, especially in the bus terminal by 75%. The number of the columns has also reduced 20% in total and 70% in the bus terminal.

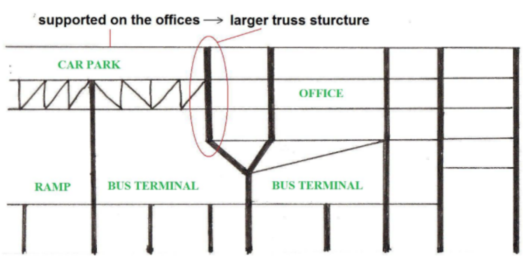
Reflection to the whole station complex

The topology of the Breda CS has the image of the whole station complex under one single roof at the same height. This requires the every part of the enclosed multi-functional building being in harmony with each other. As a public transport with 60,000 passengers on an average working day, the new station complex has to provide enough open space and daylight for the travelers to find their way around it easily, quickly and safely. The architectural design meets the needs of this as a whole where large open space can be found in the railway platforms, south concourse and the station square on both the front and back side. However, the bus terminal at level 1 which is also within the range of the whole station has many columns and low ceiling height. This master's thesis aims to reduce the rows of columns on the bus terminal and creative more open space for the public in a structural way.

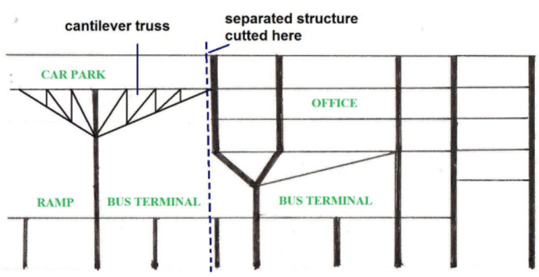
Several aspects influence and determine the choice of the structural solution for that part. Like other NSP stations, Breda CS also has railway tracks and platforms at level 1, so as the bus terminal of it. This makes it difficult to reducing the barrier to a certain degree compared to the tracks at ground level or subsurface. In the meanwhile, there locates offices on the bus terminal so it results in the bus terminal to be an intermediate level which has to have as few supports as possible yet. To reduce the columns on the bus terminal means to reduce the supports to the office above it. But the overhead office building is also a route-derived structure that corresponds to the grid imposed by the bus route. This dictates the office grid, and at the same time, the office grid is also affected by the car park adjacent to it. That's why the truss structure is an optimal solution because it works as a transfer structure which converts an unfavorable grid dictated by the platforms and tracks into a feasible or even a standard grid.

By the research, it has been concluded that the truss is more effective to be set longitudinally than transversely. But to create open space at the bus level as much as possible, moment resisting tree column structure is designed to reduce the columns there in transverse direction. Nevertheless, although this north part of the new station complex has been isolated as an independent structural part, it still has to consider the integrity of the whole station. The car park on the fifth floor which is originally supported by the offices has not been taken into account in this thesis. The way of supporting the car park above the railway platform will also have impact on the design, therefore, some possible solutions for it is considered as a concept below.

- The first option is to follow the current design that one end of the car park is connected to and supported by the office part. Consequently, the scale of the truss on the south side and the tree column might become larger due to the extra loads.
- The second option will follow the scheme of the tree column structure on the bus terminal in a large scale to support the car park deck, since that area is also a part of the bus terminal.



Option 1



Option 2

In conclusion, the new design reduces the number of the columns on the bus terminal and creates more open space for the public successfully. And the more flexible space makes it possible for future renovation. Moreover, getting rid of the deep concrete beams by steel structure also reduces the ceiling height at bus level. However, it cannot be said that the current design is not recommended because with regard to the design process, buildability and construction aspects including cost, time, labor and etc, the current design is a traditional and economical solution which is always welcome nowadays.

11.2 Recommendations

For alternatives

The improvement of the unselected alternatives is an interesting point to further study. The 1st to 3rd alternative could be discussed with the architect and client together to improve the functional feasibility. The structural difficulties of the 5th alternative space frame with fewer functional problems can be optimized that the dimension of the space frame could be reduced to 20.2m wide, so that the weight of the space frame could reduce as well. The geometry of the space frame could improve to large planar truss girders in grids rather than 3D space frame to both reduce the weight and simplify the connections.

For optimizing the final structure

Although the study has expatiated on the feasibility and structural behavior of the final structure, optimizing the structure by unit check is also promising for the following design stage. Although the von Mises stress is over safe in the principle, measures have to taken to the members in the final structure whose stresses have exceeded the yield stress. Every structural member could be optimized to nearly fully loaded but without regardless of the cost and construction convenience. With the optimization of the structure, deflection of the structure may increase which has to be taken into account and controlled at the same time.

In this thesis, one wind load case has only been considered mainly from the outside of the station from north direction. Positive and negative wind pressures have been involved in the calculation. However, wind load from other direction which is not the governing direction should also be considered in the detailed design. The stability of the structure could either be provided by the current rigid structure or strengthen the two closed ends of the building to transfer the wind load to the foundation.

By using the ASB girders for the slim floor system which has an asymmetric section, it is often structurally more efficient and architecturally desirable to use an RHS (Rectangular Hollow Section) instead at the perimeter of the building which can resist torsion forces caused by eccentric loading on the beam. The RHSFB (Rectangular Hollow Slimflor Beam) comprises a rectangular hollow section with a 15mm thick S355 steel plate welded to the underside of the RHS.

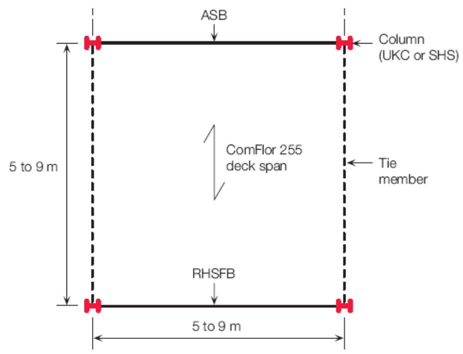
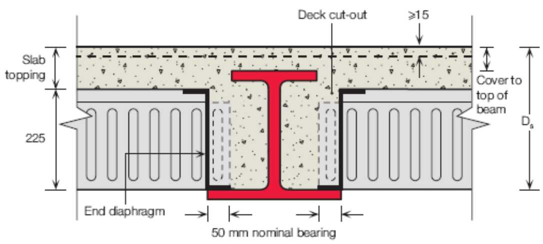


Fig 11-1 Slimdek – beam layout [8]



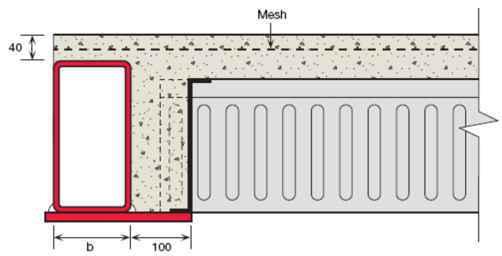
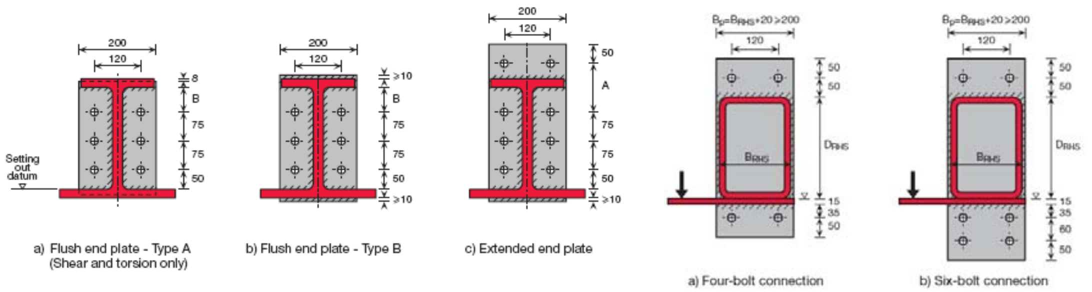


Fig 11-2 Section view of ASB and RHSFB [8]

For further design stage

In the general structural design stage, the effect of the slim floor system used in this project has not been modeled directly, but has been substituted by horizontal bracing members. With regards of this, the structural design of the slim floor will also be a crucial part of the design. In addition, the beam in the floor system has to be designed for two load situations, one of which has not been studied in this thesis project is that during construction when the concrete has been placed but the restraining effect caused by solidification of the concrete is not present. Other design aspects including fire resistance, punching shear, crack control for the floor slab and lateral torsion buckling, moment capacity, shear resistance, vibration control, and etc for the beams also have to be well considered and designed.

This thesis has not reached the connection design, but several ideas and recommendations are given to this part. The design and detailing of end plate connections to ASB or RHSFB in the braced frames should take into account: the width of the beam and column flanges, the requirements for shear, torsion resistance, as well as those to tension, and connections to HHS columns. Some end plate connection samples are shown in Fig 11-3. The flush type A end plates are usually used in shear-resisting connections with torsion resistance, while flush type B or extended end plates are used in moment-resisting connections.



Dimension	A	B
ASB280	110	44
ASB300	140	62

Fig 11-3 Detailing rules for end plate connections to ASBs and RHSFs [8]

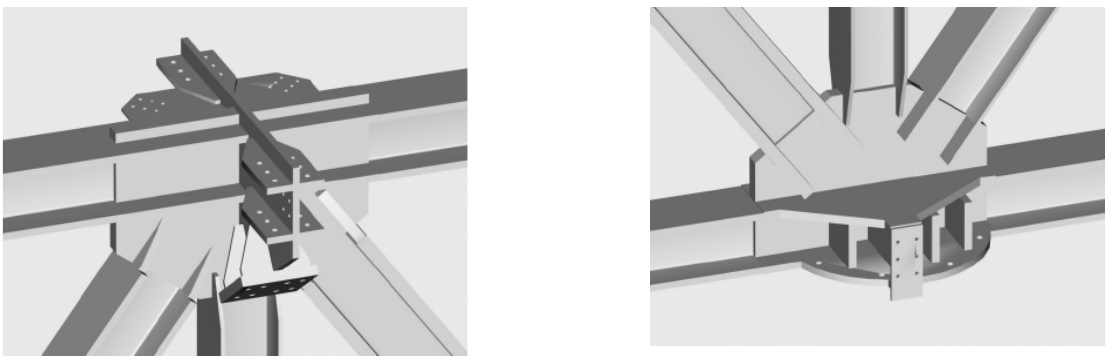


Fig 11-4 Connection recommendation for truss structure [23]

Appendix

Appendix 1 Function Requirement and Arrangement

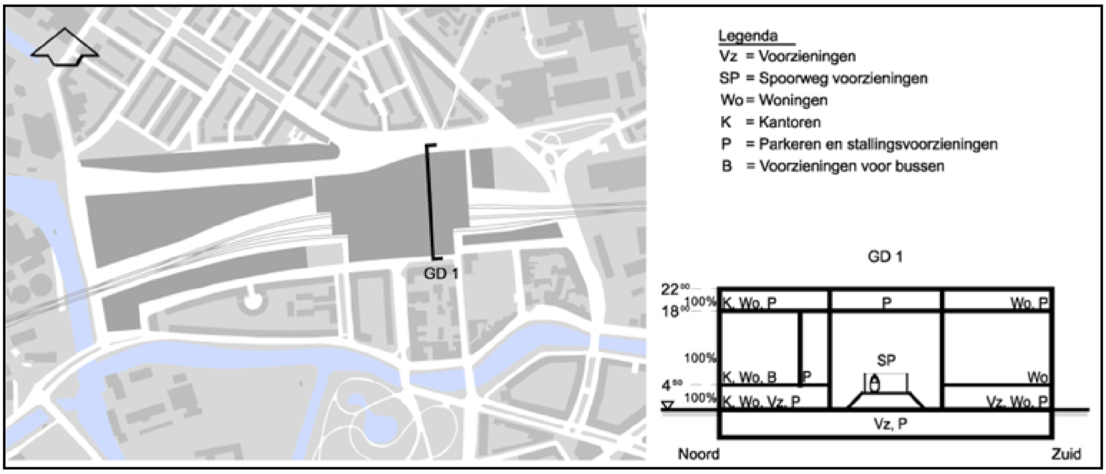
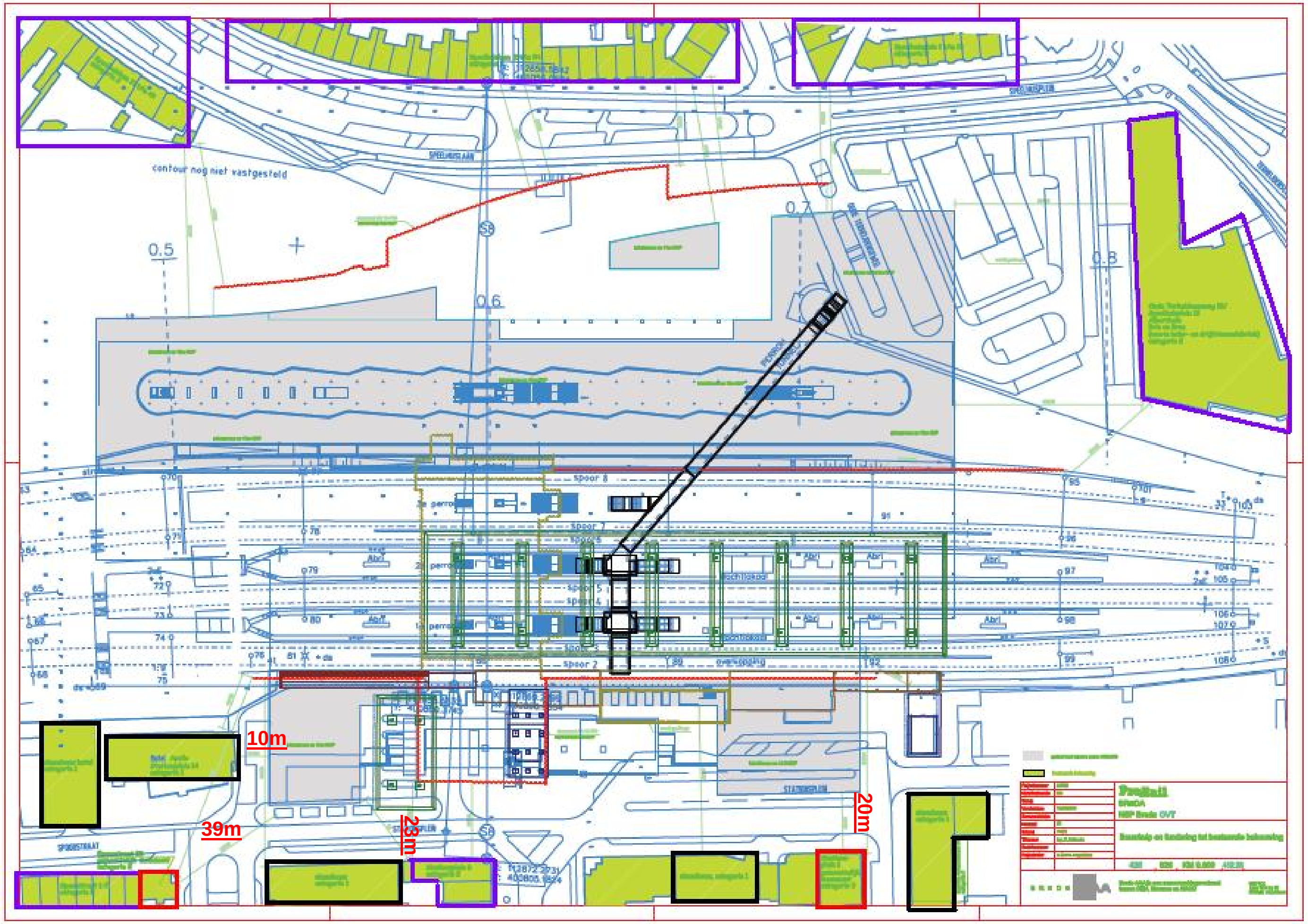


Fig A1-1 Function requirement and arrangement [2]

Function	*Figure*	*Location (level)*
Passage	± 800 m²	-1
Bicycle (way & install)	± 4,200 (guarded 2800, unguarded 1400)	-1
Commercial function NS	± 8,500 m²	-1, 0
Square	± 4025 m² (south); ± 2875 m² (north)	0
Bus terminal	± 7,500 m² (1 bus platform for 20 busses)	1, 2
Train	3 platforms	1 - 4
Office	± 20,000 m² (north)	3 - 5
Apartments	± 20,000 m² (60 on south; 68 on north)	0 - 5
Parking	700 places (432 parking and P&R 268)	5 (ramp starts from 0)

Table A1 Functional arrangements

Appendix 2 Overview of Breda Central Station



Building Categoray:

Black: I

Purple: II

Red: III

Fig A2-1 Location boudaries of Breda Central Station [3]

Appendix 3 New Breda Central Station

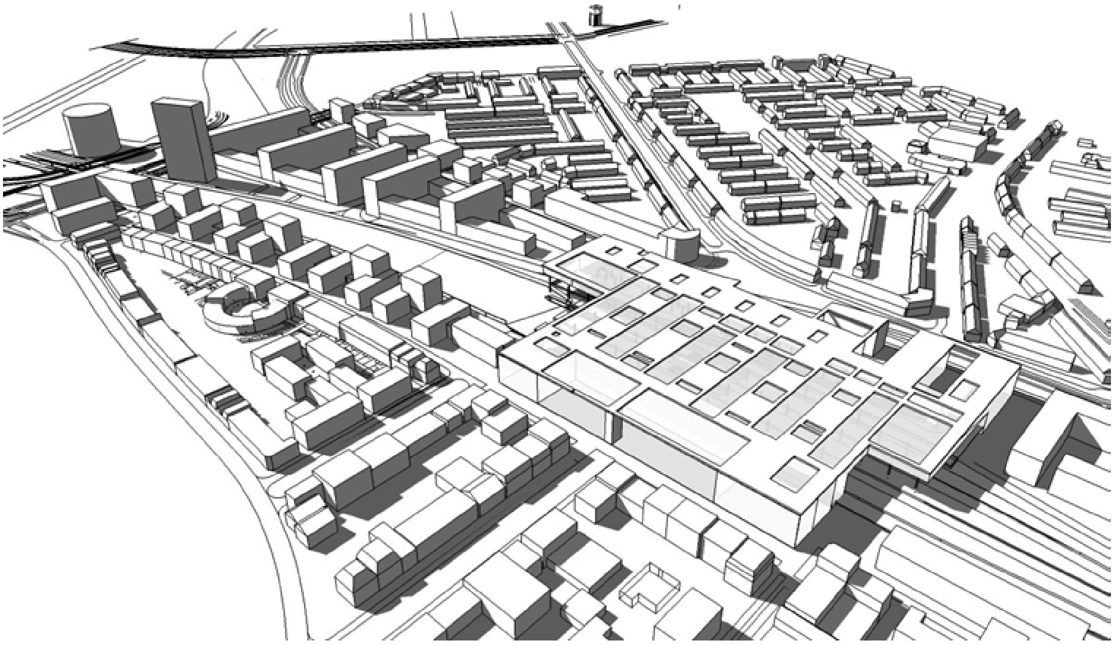


Fig A3-1 Example of an urban form [2]

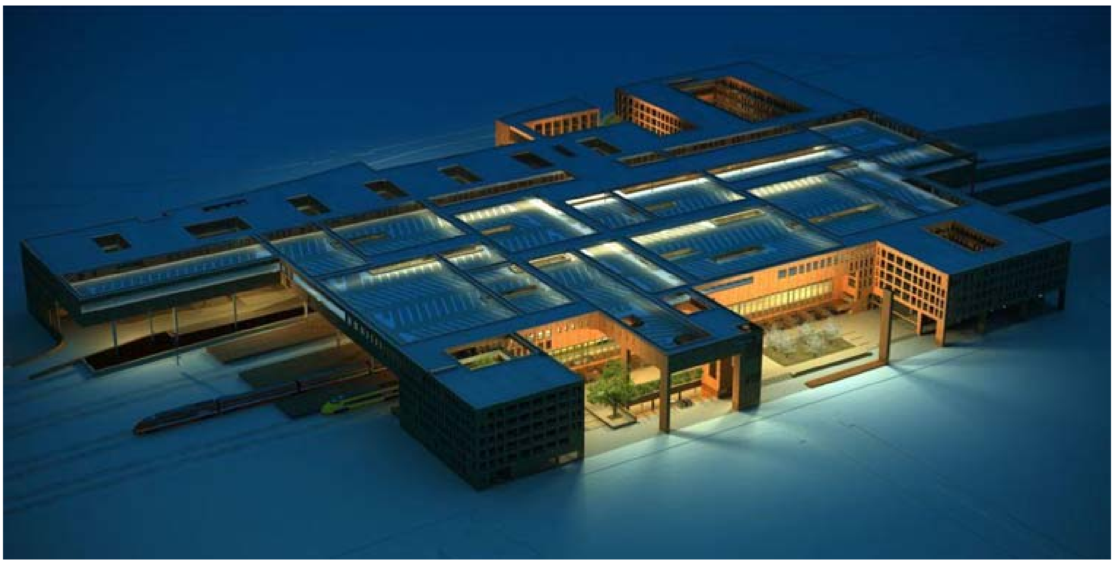
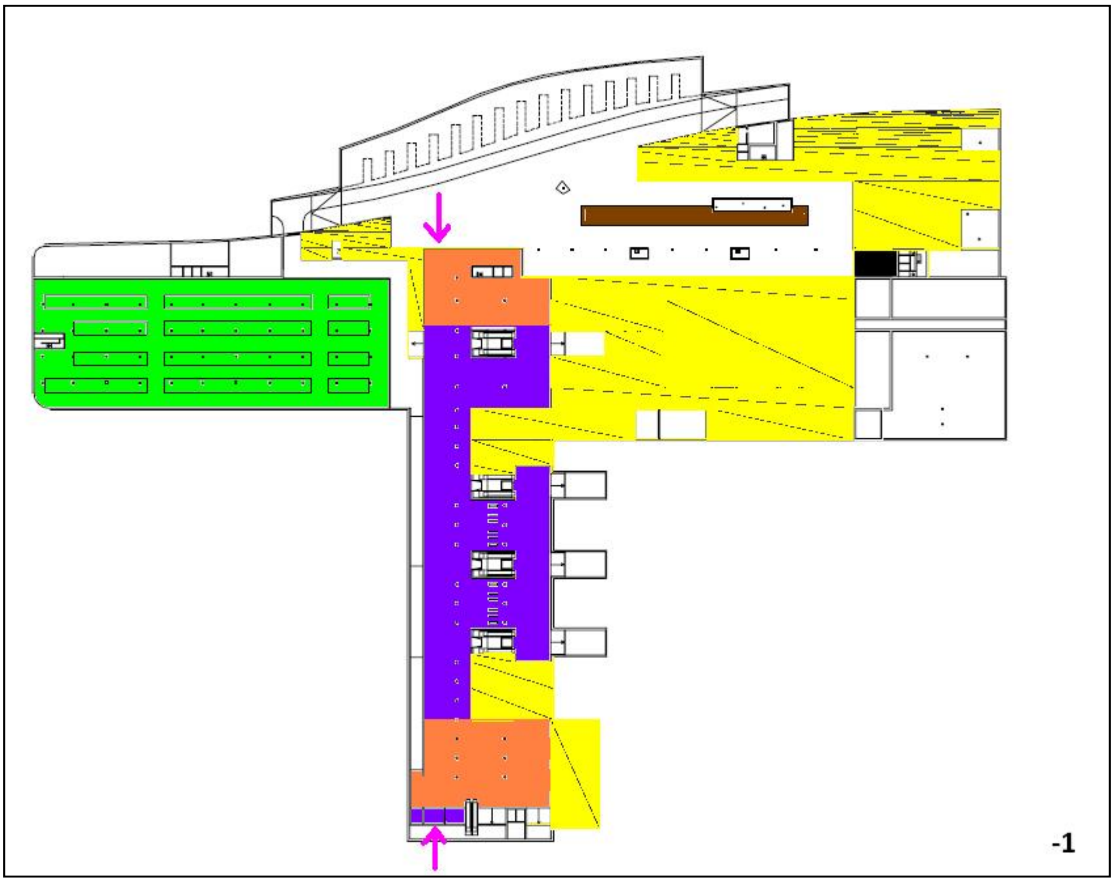


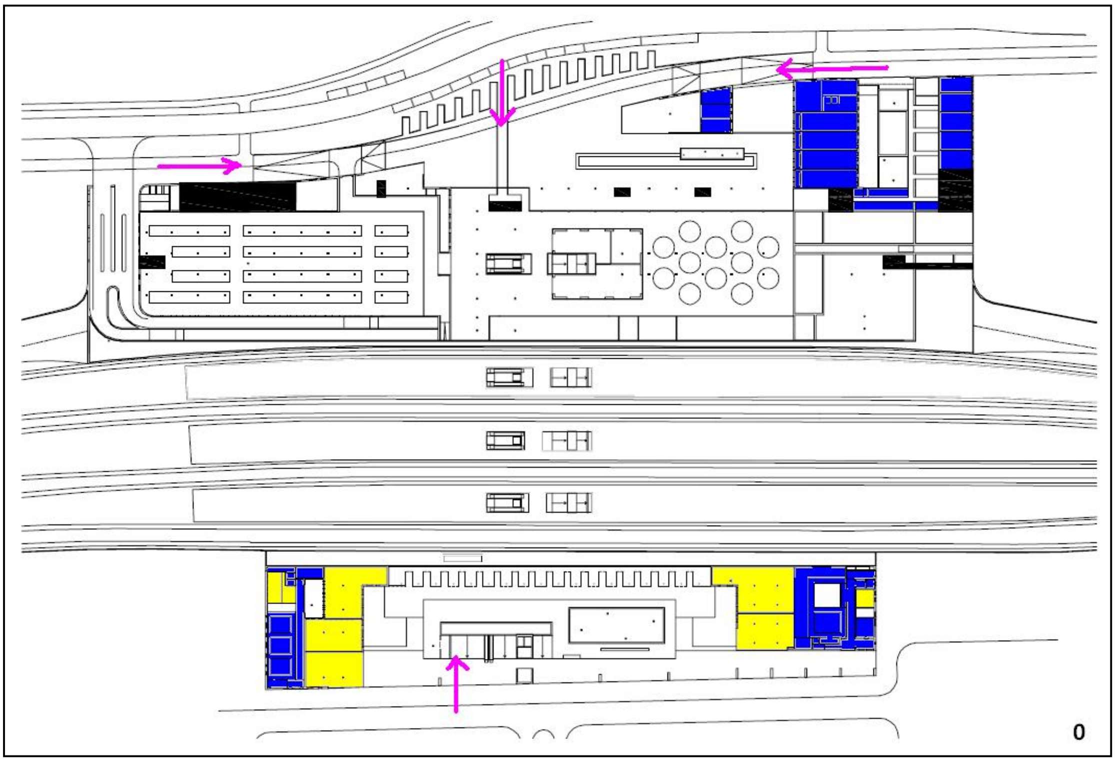
Fig A3-2 3D model of new station complex [7]

Fig A3-3 new Breda CS rendering [7]



Legenda:

- Office
- Car Park
- Apartment
- Commerical
- Lake
- Bus Terminal
- Lobby & Passage
- Ticketsale
- Bike Storage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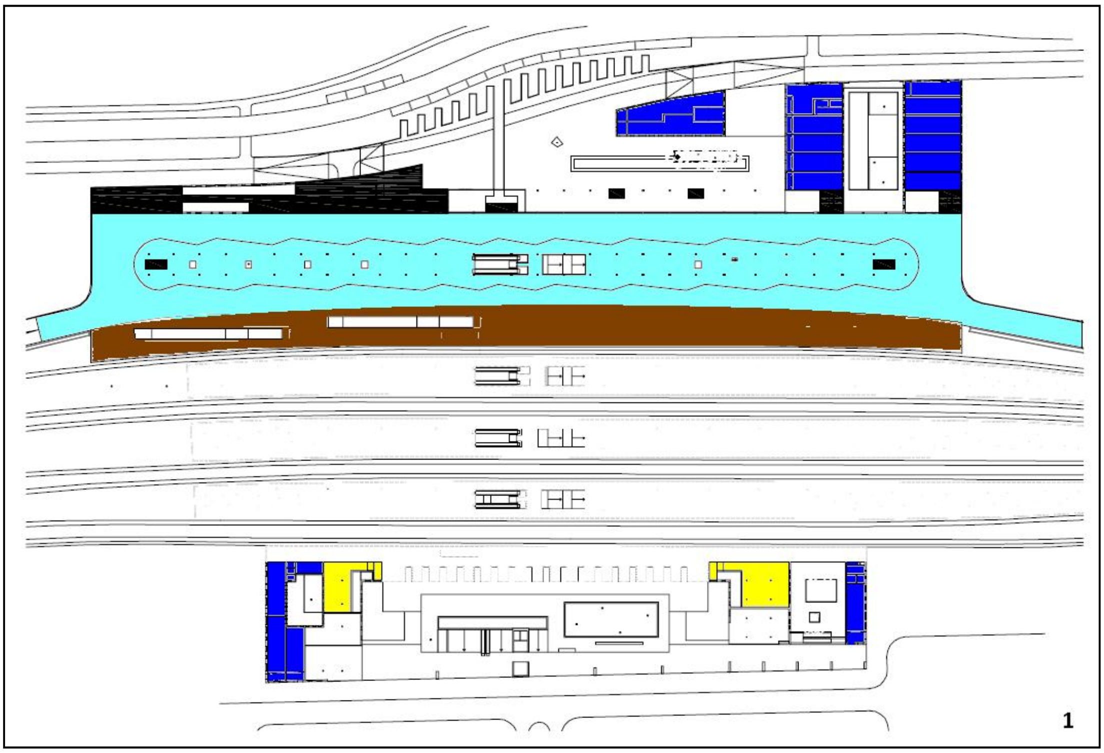
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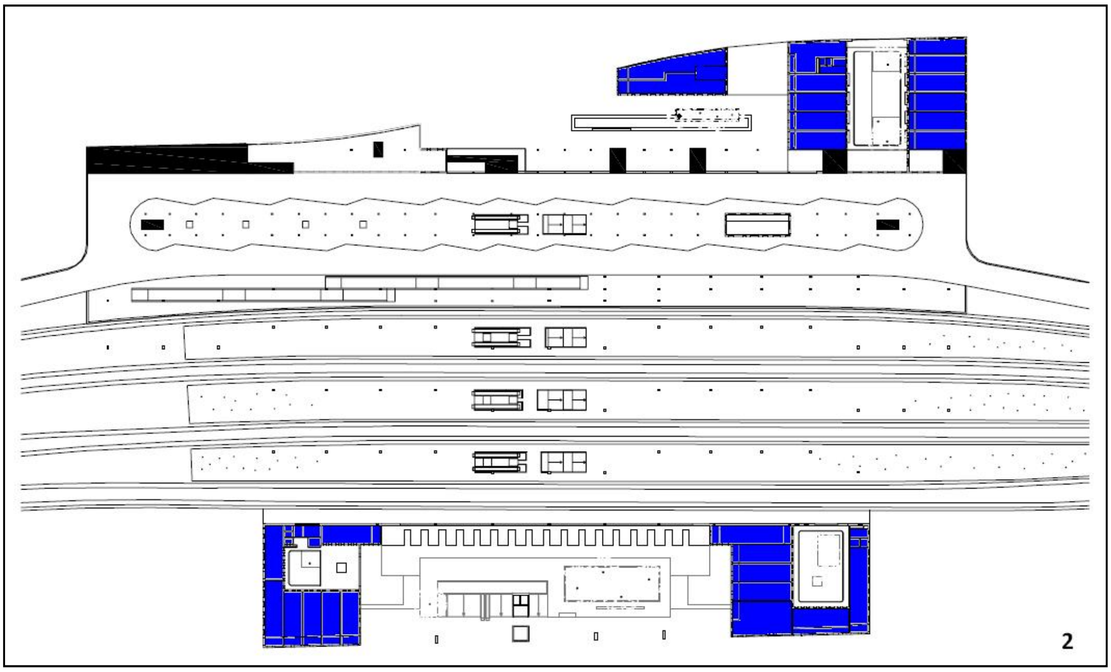

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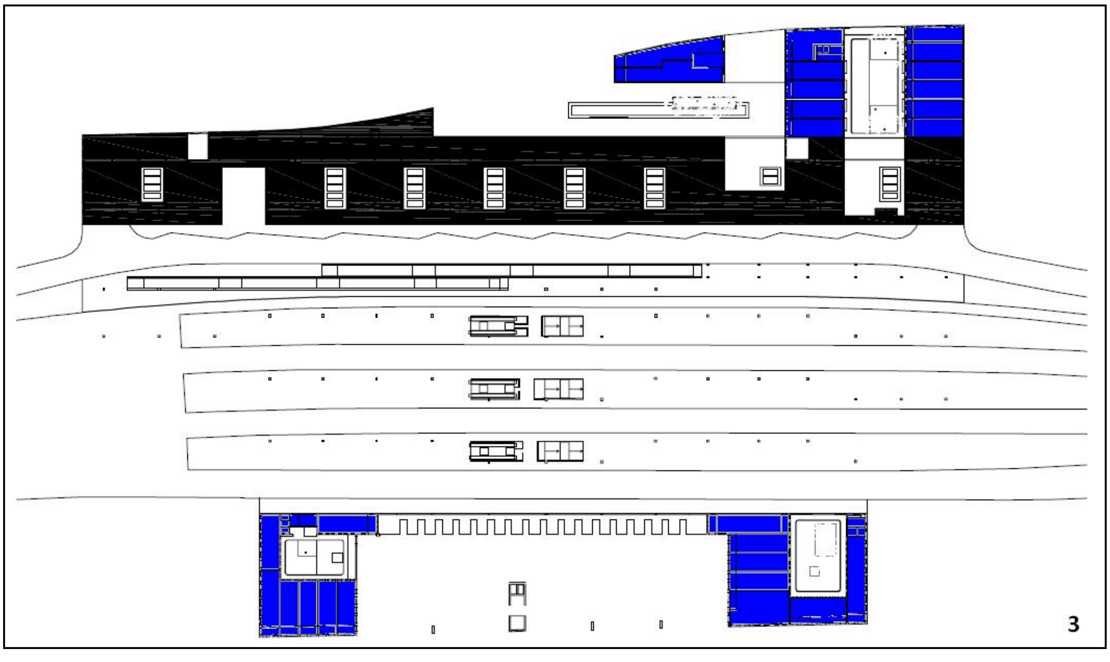

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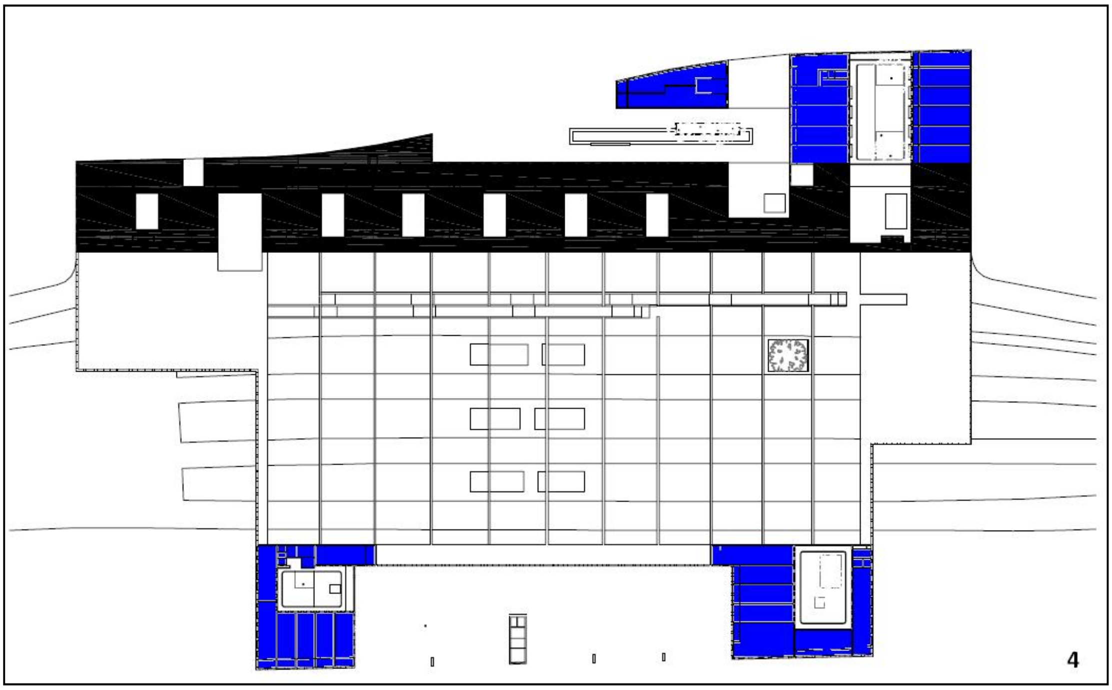


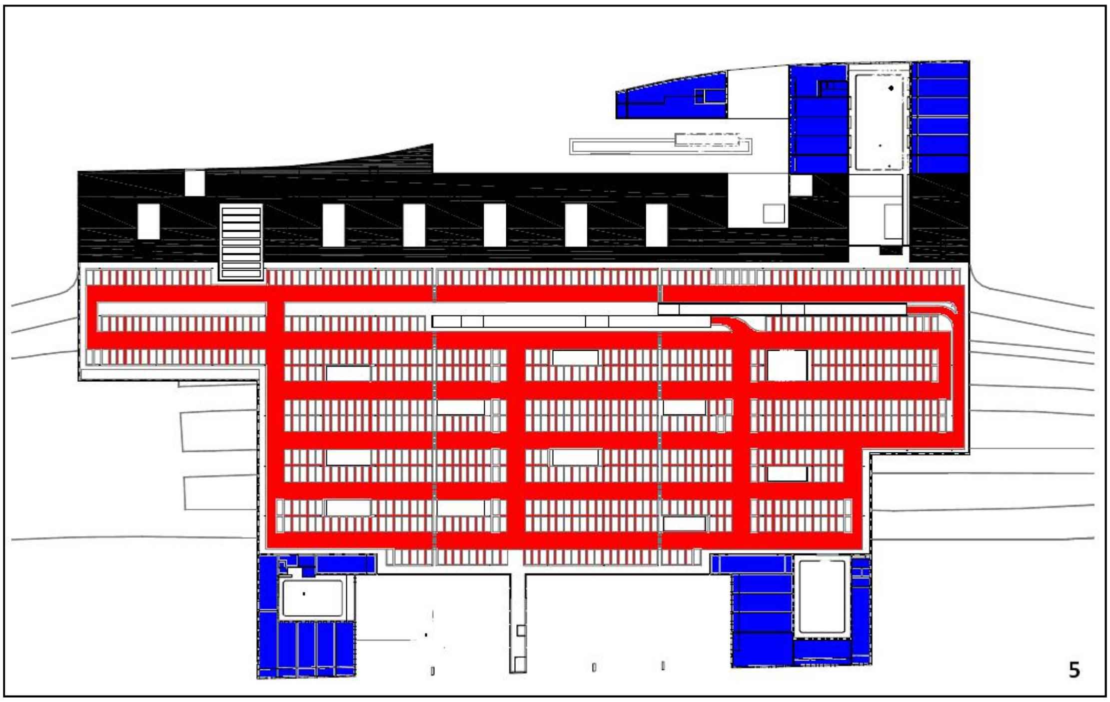


Fig A3-4 Design of Breda CS

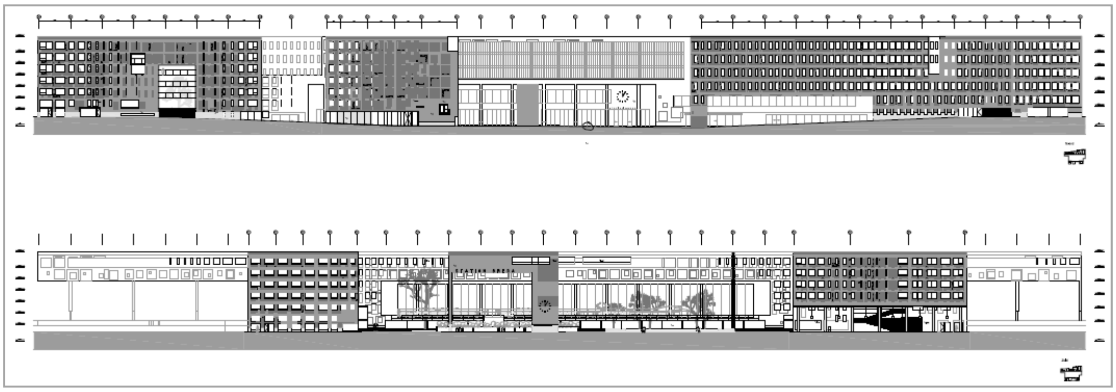


Fig A3-5 North and South Elevation [3]

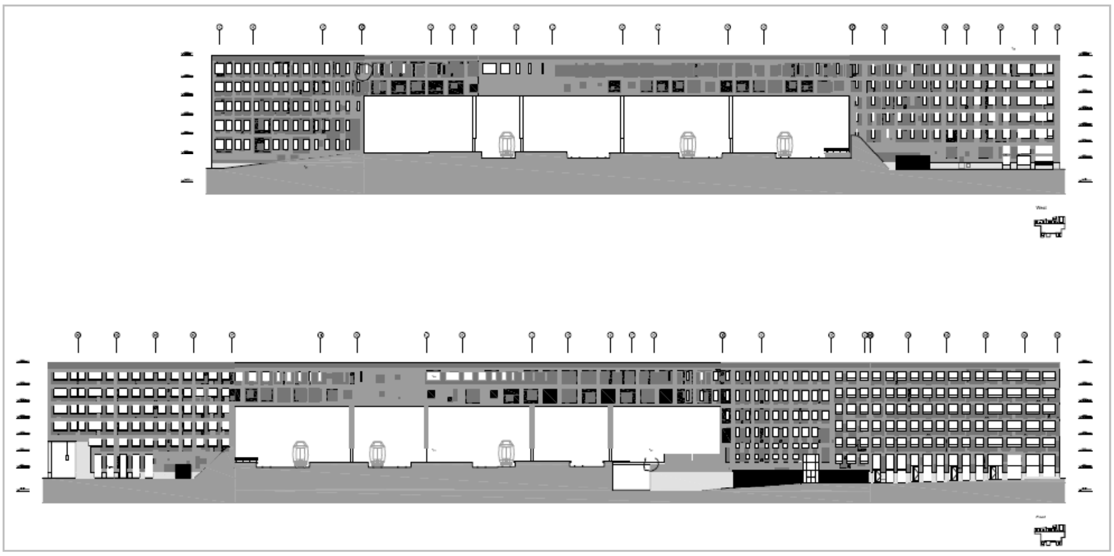


Fig A3-6 West and East Elevation [3]

Appendix 4 Space frame design calculation

In order to achieve the goal of this thesis which is to reduce the columns on the bus terminal to get more open space, space frame has been considered as an alternative. Although it was not chosen for the final structure due to several reasons, it is still interesting to research the scale and structural consequence that a space frame bears a three-storey office part above it. Therefore, in this chapter, an estimated calculation of the space frame structure will be studied. Linear elastic calculation was done in SCIA ESA PT to check the deformation, force distribution and stresses.

Appendix 4.1 General Conditions

Space frame structure mentioned in alternative 5 is the structure usually used in large span structures which in this project could help to reduce the supports on the bus terminal level and provide stability in both directions at the same time. In addition, setting fewer supports on the bus terminal by using space frame means offices above the bus terminal don't need to have fewer supports; that's to say, for the office part, regular structure such as rigid frame can be used.

Goal
Space frame is usually used as the roof structure for lager span buildings, but there is not so often to carry other structures on it. The goal of this part study is to get an insight into the space frame structure with three-storey offices on it.

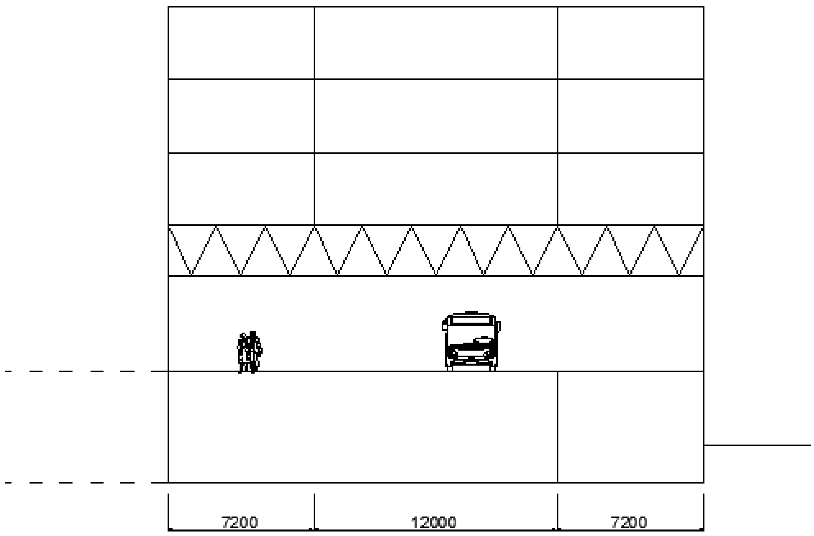


Fig A4-1 Indication of space frame structure on the bus terminal (section view)

Dimension
The design of the space frame is also divided into modules mentioned in chapter 8, so that the space frame has the width of 26.4m and the length of 25.2m/33.6m (three/four 8.4m column spacing). The general dimension of the grids comply with the span to depth ratio of 15:1 which means the depth of the space frame can be set as about 1.8m, however, considering the space frame has to carry not only itself but also the upper offices, so the depth of the space frame design as 2.4m as a start. Considering the dimension of the space frame and the chord angle of it, the grid dimension uses 2.4m×2.8m.

Supports
Four corner supports are applied in every space frame to realize the minimum supports in the certain area.

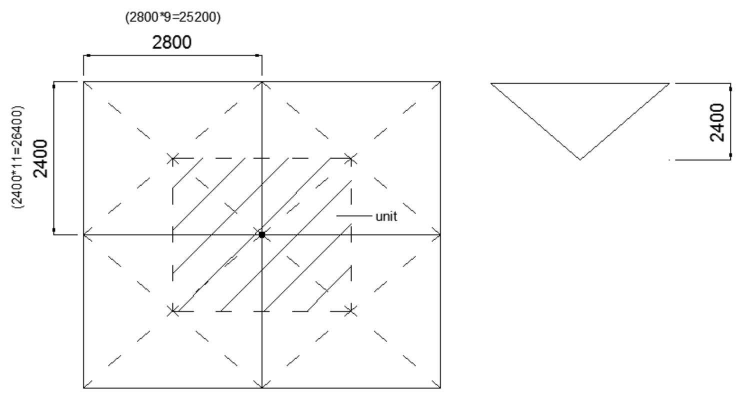


Fig A4-2 Grid dimension of space frame

Appendix 4.2 Design calculation

To estimate the sections of the space frame, the loads of the office part were calculated to apply on the space frame. Five different rigid frame models were made in ESA PT in which the properties of the elements had been determined in Chapter 8.

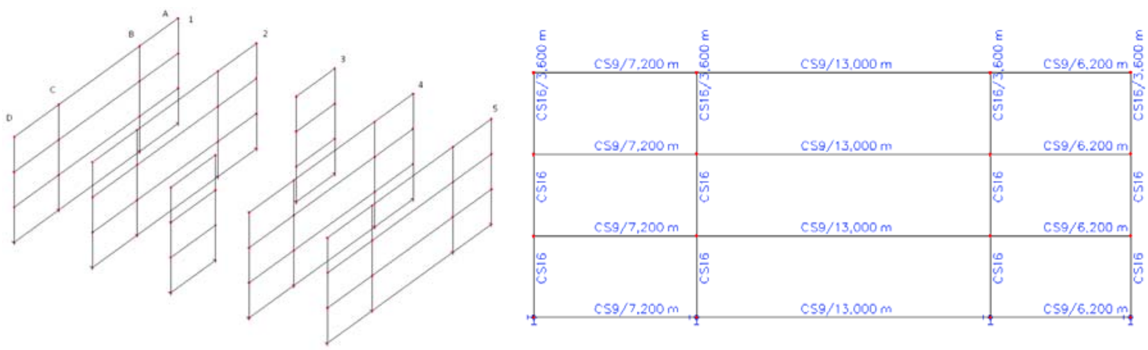


Fig A4-3 Frame structure of offices

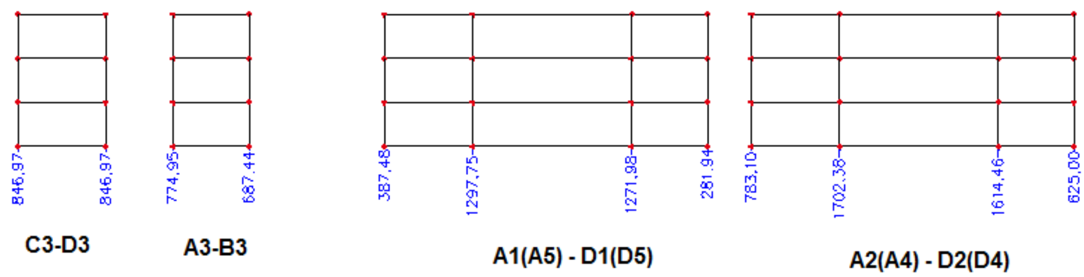


Fig A4-4 Reactions of the frames

Distributed point loaded space frame

The goal of this model is to estimate the scale of the space frame and make a first impression of its behavior.

Total loads of office area (reaction force Rz from rigid frame in Fig A4-4)

$$R_{z,total} = 15928\text{kN}, q = \frac{15928}{25.2 * 26.4} = 24\ \text{kN/m}^2$$

$$R_{unit} = \frac{15928}{80 + 8 + 10 + 1} = 161\ \text{kN (point load)}$$

Self weight + 20% connection weight: $R \approx 1\ \text{kN/m}^2$

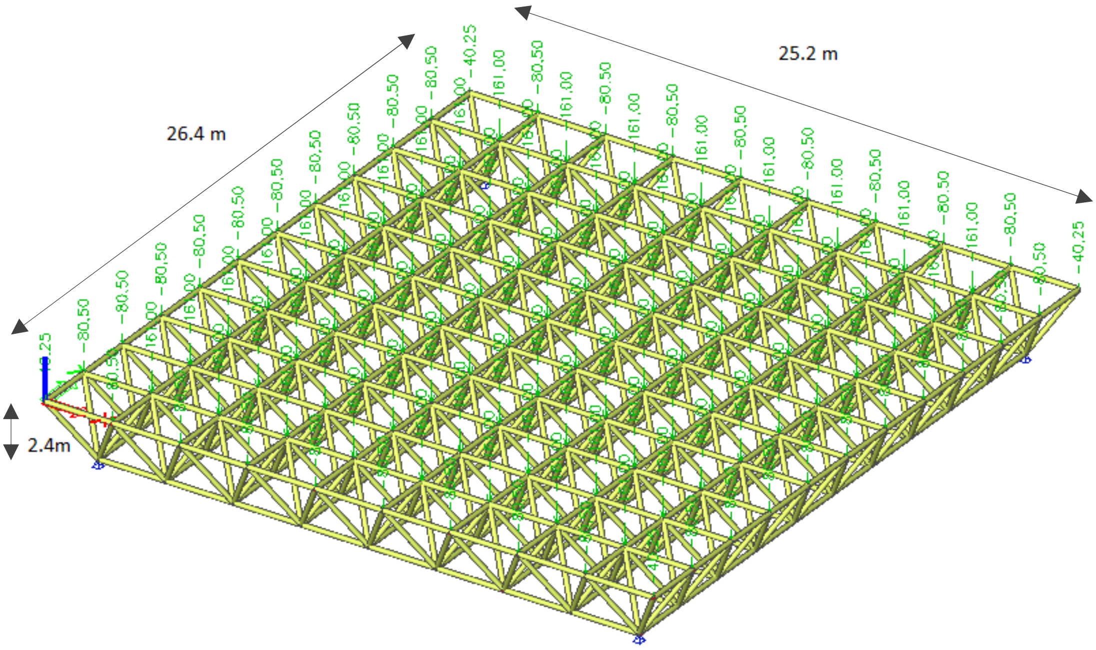
$$q = (24 + 1) * 2.4 = 60\ \text{kN/m}$$

$$F_{ch} = \frac{\frac{1}{8} * q * l^2}{h} = \frac{\frac{1}{8} * 60 * 26.4^2}{2.4} = 1984.5\ \text{kN}$$

$$d = \frac{F_{ch}}{f_y * \pi * t} = \frac{1984.5 * 10^3}{355 * \pi * 12.5} = 182.3\ \text{mm}$$

Select bar element: CHS193.7/12.5 for first design.

Structural Model



Space Frame under uniformly distributed point loads

Dimension:	25.2×26.4m
Element:	CHS193.7/12.5
Grid dimension:	2.8×2.4×2.4m

Load cases

	Load Case	Value
LC1	Self weight	Default
LC2	Point load	161 kN
ULS	LC1*1.2 LC2*1.0 (for a reduction)	

Note: A reduction factor was used for LC2 for accuracy since the forces from the rigid frame applied on the space frame model were already got under ULS.

Results

4. Internal forces on member

Linear calculation,Extreme : Global, System : Principal
Selection : All
Combinations : ULS

Member	Case	dx [m]	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
B852	ULS/1	3,027	**-3062,17**	2,80	5,58	0,29	5,93	4,14
B1317	ULS/1	0,000	**1656,21**	1,29	2,40	0,62	6,11	-1,39
B653	ULS/1	0,000	-1287,91	**24,54**	-10,58	-3,59	14,41	-28,46
B1322	ULS/1	2,400	-2786,74	20,87	**-28,46**	0,48	-38,06	27,07
B1313	ULS/1	0,000	-2786,74	-20,87	**28,46**	-0,48	**-38,06**	27,07
B1331	ULS/1	0,000	484,34	-1,35	-3,34	**-3,72**	7,87	1,47
B1324	ULS/1	0,000	484,34	1,35	5,24	**3,72**	-2,42	-1,77
B1313	ULS/1	2,400	-2786,74	-20,87	26,56	-0,48	**27,96**	-23,01
B722	ULS/1	0,000	-1287,91	24,54	12,48	-3,59	-13,26	**-30,44**

5. Deformation of nodes

Linear calculation,Extreme : Global
Selection : All
Combinations : ULS

Node	Case	Ux [mm]	Uy [mm]	Uz [mm]
N319	ULS/1	**-12,9**	15,7	-23,2
N235	ULS/1	**12,9**	**15,7**	-23,2
N244	ULS/1	12,9	**-15,7**	-23,2
N380	ULS/1	0,0	0,0	**-100,7**
N1	ULS/1	12,0	15,0	**14,4**

Sum of loads and reactions.

	[kN]	X	Y	Z
Loadcase 1	loads	0.0	0.0	-1465.5
	reactions in nodes	0.0	0.0	1465.5
	reactions on lines	0.0	0.0	0.0
	contact 1D	0.0	0.0	0.0
	contact 2D	0.0	0.0	0.0
Loadcase 2	loads	0.0	0.0	-15939.0
	reactions in nodes	0.0	0.0	15939.0
	reactions on lines	0.0	0.0	0.0
	contact 1D	0.0	0.0	0.0
	contact 2D	0.0	0.0	0.0

6. Stress

Linear calculation,Extreme : Global
Selection : All
Combinations : ULS

Member	Case	dx [m]	Normal - [MPa]	Normal + [MPa]	Shear [MPa]	von Mises [MPa]	Fatigue [MPa]	Kappa [1]
B1313	ULS	0,000	**-545,1**	0,0	8,7	**545,2**	0,0	0,00
B282	ULS	0,000	**0,0**	189,5	1,9	189,5	0,0	0,00
B12	ULS	0,000	-230,4	**0,0**	8,5	230,6	0,0	0,00
B1317	ULS	2,400	0,0	**264,3**	1,3	264,3	0,0	0,00
B704	ULS	1,200	-198,8	0,0	**0,0**	198,8	0,0	0,00
B645	ULS	0,000	-289,8	0,0	**12,4**	290,2	0,0	0,00
B1017	ULS	3,027	-3,5	3,9	0,6	**4,0**	0,0	0,00
B11	ULS	0,000	-32,8	33,5	7,5	34,8	**0,0**	**0,00**

Check and Conclusion

The linear calculation under the two load cases indicate that,

- The maximum deformation of the space frame under SLS occurred in the middle of the structure is within the limitation,
  $U_{z,max}$=100.7 mm < 0.004L=0.004×26400=105.6 mm
- Extremely large compression force (3602kN) occurred in the bars around the supports due to the large reaction forces.

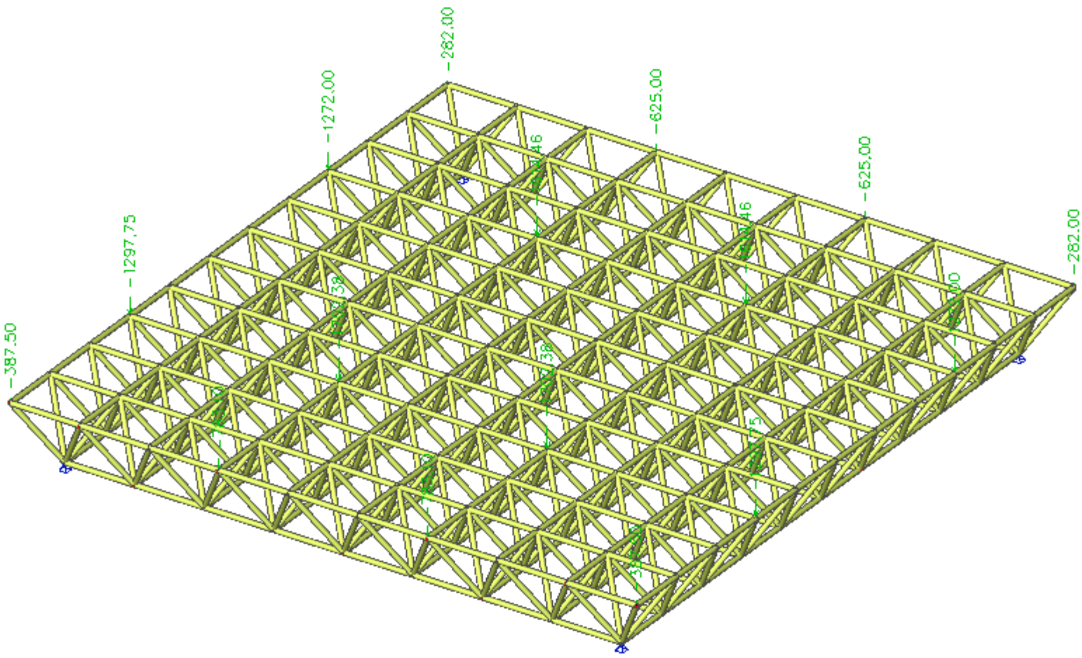
- The dimension of the governing bars is not sufficient enough by checking the stresses in them which has yielded and also might buckle due to the large compression forces.
- However, many of the other bars are not fully loaded, so the structure has to be optimized.
- Moreover, the designed and expected load condition is that the space was only connected to the above office part at the column positions, which means the load transferred from the offices should be applied on the space frame by 16 definitive point loads.

Definitive point loaded space frame

The goal of this model was to calculate the space frame under designed condition by applying 16 point loads on it, and to compare the results with the former one to get to know which one was more effective. This could affect the way of the connection between the space frame and the above office structure.

Note: Due to the grid dimension of the space frame and its connection to the office part. The location of the columns in offices at axis B were moved 1m towards south so that it could align to the nodes on the space frame.

Structural Model



Space Frame under 16 definitive point loads

Dimension: 25.2×26.4m
Element: CHS193.7/12.5
Grid dimension: 2.8×2.4×2.4m

Load cases

	Load Case	Value
LC1	Self weight	Default
LC2	Point load	16 point loads
ULS	LC1*1.2 LC2*1.0	

Results

4. Internal forces on member

Linear calculation,Extreme : Global, System : Principal
Selection : All
Combinations : ULS

Member	Case	dx [m]	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
B1101	ULS/1	3,027	**-2838,13**	2,78	5,02	0,17	5,57	4,13
B1105	ULS/1	0,000	**1431,96**	3,92	-0,87	0,90	-1,12	-4,89
B901	ULS/1	0,000	-1275,09	**-26,93**	12,53	4,87	-13,51	**33,61**
B978	ULS/1	0,000	-1275,09	**26,93**	12,53	-4,87	-13,51	**-33,61**
B1592	ULS/1	2,400	-2780,67	-16,19	**-26,83**	0,38	-36,49	-21,25
B1583	ULS/1	0,000	-2817,59	17,07	**27,84**	-0,36	**-37,80**	-22,30
B988	ULS/1	0,000	-1905,83	10,80	25,96	**-6,74**	-20,95	-9,10
B889	ULS/1	0,000	-1905,83	-10,80	25,96	**6,74**	-20,95	9,10
B889	ULS/1	2,400	-1905,83	-10,80	24,07	6,74	**39,08**	-16,81

5. Deformation of nodes

Linear calculation,Extreme : Global
Selection : All
Combinations : ULS

Node	Case	Ux [mm]	Uy [mm]	Uz [mm]
N349	ULS/1	**-12,2**	14,8	-21,8
N265	ULS/1	**12,2**	**14,8**	-21,8
N255	ULS/1	11,2	**-14,7**	-20,8
N412	ULS/1	0,0	0,3	**-82,9**
N254	ULS/1	11,1	14,1	**12,3**

Sum of loads and reactions.

	[kN]	X	Y	Z
Loadcase 1	loads	0.0	0.0	-1465.5
	reactions in nodes	0.0	0.0	1465.5
	reactions on lines	0.0	0.0	0.0
	contact 1D	0.0	0.0	0.0
	contact 2D	0.0	0.0	0.0
Loadcase 2	loads	0.0	0.0	-15928.4
	reactions in nodes	0.0	0.0	15928.4
	reactions on lines	0.0	0.0	0.0
	contact 1D	0.0	0.0	0.0
	contact 2D	0.0	0.0	0.0

6. Stress

Linear calculation,Extreme : Global
Selection : All
Combinations : ULS

Member	Case	dx [m]	Normal - [MPa]	Normal + [MPa]	Shear [MPa]	von Mises [MPa]	Fatigue [MPa]	Kappa [1]
B1583	ULS	0,000	**-538,6**	0,0	8,4	**538,7**	0,0	0,00
B891	ULS	0,600	**0,0**	43,8	8,5	45,7	0,0	0,00
B882	ULS	0,000	-178,9	**0,0**	9,7	179,2	0,0	0,00
B1105	ULS	3,027	0,0	**230,7**	2,5	230,8	0,0	0,00
B995	ULS	1,400	-324,0	0,0	**0,0**	324,0	0,0	0,00
B889	ULS	0,000	-342,9	0,0	**17,7**	343,7	0,0	0,00
B935	ULS	1,800	0,0	1,0	0,1	**1,0**	0,0	0,00
B881	ULS	0,000	-56,1	90,6	7,6	91,3	**0,0**	**0,00**

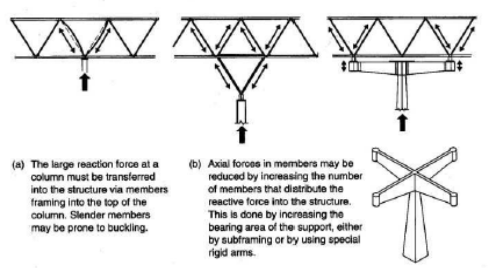
Check and Conclusion

The results of this model indicate that,

- Under same total loads, this model loaded by 16 large point loads even behaves better than the last one loaded by uniformly distributed point loads. Less deformation, smaller internal forces and stress are got from this model.

- The maximum deformation 82.9mm is smaller than 100.7mm because loads are not applied on the weakest position (center) of the space frame.
- The maximum compression force is also smaller but still occurred in the elements near the supports.
- The maximum stress of the elements exceeds the yield value too, and other elements are not fully loaded as well.
- Therefore a positive conclusion can be drawn that due to the better behavior of the second model, the connection between the space frame and above office structure is suggested to use the 16 definitive points where the columns in the office locate, rather than every joint on the top of the space frame. This also saves the amount of the connections, cost and erection time.
- However the results also show that the force distribution and stresses in this model still cause failure and buckling problems, so the structure has to be improved.

Although failure and buckling occur in the structure, the structure cannot be said not safe or not sufficient, because the failure and buckling happen in the elements near the support with large reaction forces, while other bars in the space frame has not been fully loaded. So measures could be taken to the supports to optimize the structure. From the literature of Schodek, single supports at the space frame are very unfavorable. The large axial forces in the bars could be reduced by increasing the number of the elements or the bearing area of the supports.



Sub-frame supported space frame

The optimization of the model was done by sub-framing. In addition, to simplify the connection of the space frame, hinge joints are introduced into it to get benefits. Two models were made to see the influence of optimizing the element sections on the force distribution and deflection. In the left model, all the elements dimension are CHS193.7/12.5, while in the right model, the diagonals have been optimized to CHS139.7/8 and sub-frame to CHS219.1/16.

Load cases

	Load Case	Value
LC1	Self weight	Default
LC2	Point load	16 point loads
ULS	LC1*1.2 LC2*1.0	

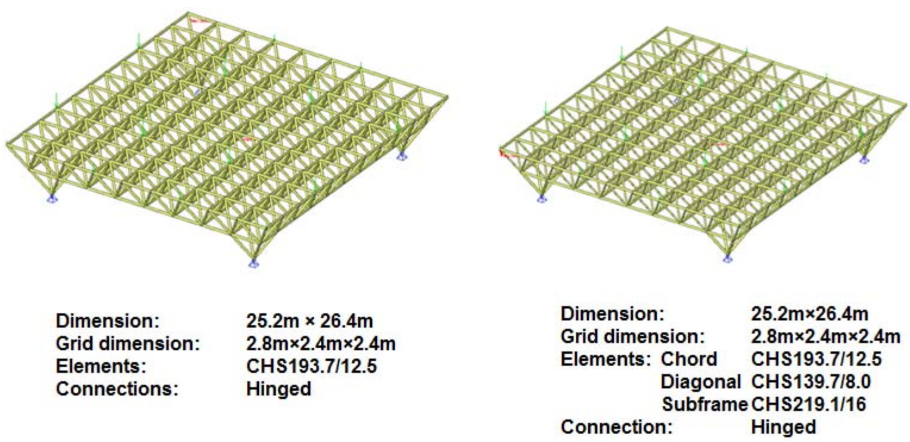
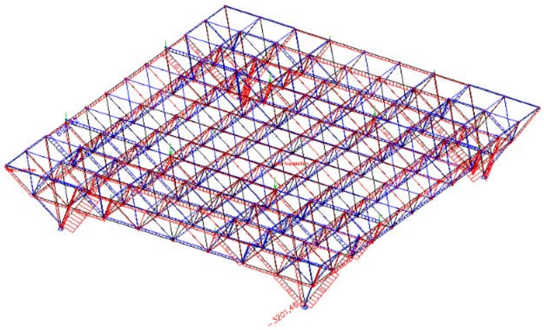
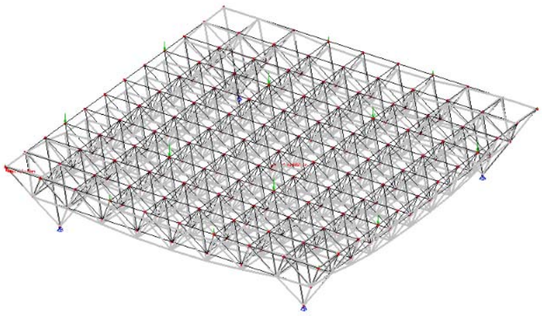


Fig 8-5 Sub-frame supported space frame models

Result



Internal Force - N



Deformed Structure

4. Internal forces on member

Linear calculation,Extreme : Global, System : Principal
Selection : All
Combinations : ULS

Member	Case	dx [m]	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
B1686	ULS/1	3,027	**-3201,46**	0,00	-1,04	0,00	0,00	0,00
B1110	ULS/1	0,000	**898,42**	0,00	0,34	0,00	0,00	0,00
B1690	ULS/1	0,000	-1691,69	**0,00**	1,04	0,00	0,00	0,00
B1687	ULS/1	0,000	-1691,69	**0,00**	1,04	0,00	0,00	**0,00**
B991	ULS/1	2,800	166,02	0,00	**-1,11**	0,00	0,00	0,00
B991	ULS/1	0,000	166,02	0,00	**1,11**	0,00	0,00	0,00
B1121	ULS/1	0,000	-17,47	0,00	0,34	**0,00**	0,00	0,00
B1474	ULS/1	0,000	-17,47	0,00	0,34	**0,00**	0,00	0,00
B1685	ULS/1	3,027	232,07	0,00	-1,04	0,00	**0,00**	0,00
B1677	ULS/1	1,513	-1654,23	0,00	0,00	0,00	**0,79**	0,00
B1687	ULS/1	3,027	-1694,40	0,00	-1,04	0,00	0,00	**0,00**

6. Deformation of nodes

Linear calculation,Extreme : Global
Selection : All
Combinations : ULS

Node	Case	Ux [mm]	Uy [mm]	Uz [mm]
N337	ULS/1	**-4,1**	3,9	-9,2
N277	ULS/1	**4,1**	3,9	-9,2
N257	ULS/1	2,4	**-6,0**	-11,1
N264	ULS/1	2,7	**5,5**	-11,2
N412	ULS/1	0,0	-0,2	**-37,1**
N256	ULS/1	1,5	-5,4	**2,7**

Sum of loads and reactions.

	[kN]	X	Y	Z
Loadcase 1	loads	0.0	0.0	-1089.2
	reactions in nodes	0.0	0.0	1089.2
	reactions on lines	0.0	0.0	0.0
	contact 1D	0.0	0.0	0.0
	contact 2D	0.0	0.0	0.0
Loadcase 2	loads	0.0	0.0	-15928.3
	reactions in nodes	0.0	0.0	15928.3
	reactions on lines	0.0	0.0	0.0
	contact 1D	0.0	0.0	0.0
	contact 2D	0.0	0.0	0.0

5. Stress

Linear calculation,Extreme : Global
Selection : All
Combinations : ULS

Member	Case	dx [m]	Normal - [MPa]	Normal + [MPa]	Shear [MPa]	von Mises [MPa]	Fatigue [MPa]	Kappa [1]
B1107	ULS	1,513	**-402,3**	0,0	0,0	**402,3**	0,0	0,00
B881	ULS	0,000	**0,0**	20,0	0,3	20,0	**0,0**	**0,00**
B883	ULS	0,000	-55,7	**0,0**	0,3	55,7	0,0	0,00
B1110	ULS	1,513	0,0	**273,8**	0,0	273,8	0,0	0,00
B1687	ULS	1,513	-167,6	0,0	**0,0**	167,6	0,0	0,00
B991	ULS	0,000	0,0	23,3	**0,3**	23,3	0,0	0,00
B1295	ULS	3,027	0,0	0,0	0,2	**0,4**	0,0	0,00

Check and Conclusion

- After optimizing the elements in the structure, the self weight has been reduced.
- The optimization also shows a promising result that much smaller deformation in the middle of the structure is found instead in this lighter structure.
- And smaller maximum tension force is got compared with former model.
- However, the maximum compression force and member stresses in the sub-frame are still too large out of control, so that other measures have to be taken.
- From the design calculation and analysis above, one can find that the space frame is an effective structure that is able to free the space on bus terminal and doesn't require large span in the offices. And the dimension of the space frame could also be realized by reasonable sizes without influence on the bus and passenger route.
- Moreover, the space frame provided the stability in both directions.
- However, as said before, space frame is normally used in the roof structure of large span buildings which means it only carries the roof loads and self-weight. Although the design calculation above indicate that it’s possible to carry additional heavy loads on the space frame, it will results in huge amount of members with quite large self weight and the connections between the space frame and above office part will be the most difficult and crucial point of the design.
- From the results of the ESA PT, the total weight of offices and space frame of the whole building (277.2m long) were about 39806kN, excluding the weight of the supports of the space frame which are probably 4.8m huge columns due to the large loads in the space frame. The total weight of offices and bus terminal then would possibly exceed 40000kN. This is an exceedingly heavy structure compared to the truss structure.
- In summary, the scale of the space frame is feasible while considering the cost, buildability and weight of the space frame, it is not feasible to select any more.

Appendix 5 Wind Load Calculation

The wind load was calculated based on
Eurocode 1: Actions on structures — General actions — Part 1-4: Wind actions

Expression (5.1)

$$W_e = q_p(Z_e) \cdot C_{pe}$$

$q_p(Z_e)$ - peak velocity pressure at height Z, according to National Annex, Table NB 4
$P_{rep} = C_{dim} \times C_{index} \times C_{eq} \times \varphi_1 \times p_w = 0.91 \times 1.28 \times 1 \times 0.81 \times 0.95 = 0.9\ kN/m^2 = q_p(Z_e)$
$q_p(Z_e) = 0.9\ kN/m^2$ (for Breda)

C_{pe} – pressure coefficient for the external pressure, section 7

Wind load on vertical walls (east-west direction)

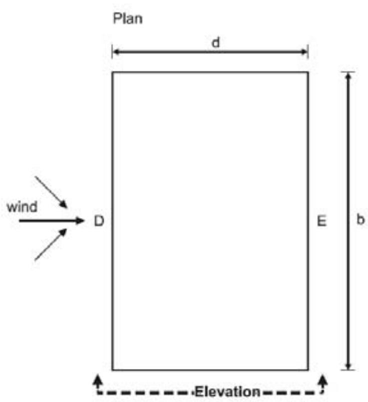
e=the smaller of [b, 2h], in which b=26.4m, 2h=46m
e=26.4m < d=277.2m

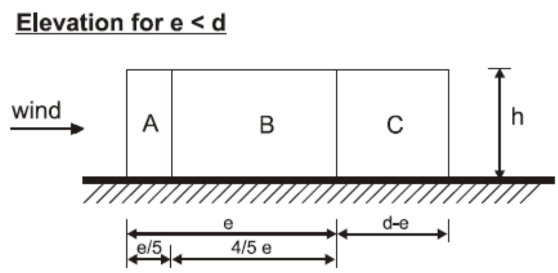
According to Table 7.1

Zone	A	B	C	D	E
h/d<0.25	-1.2	-0.8	-0.5	+0.7	-0.3

Table A5-1 C_{pe} ***– pressure coefficient for the external pressure***

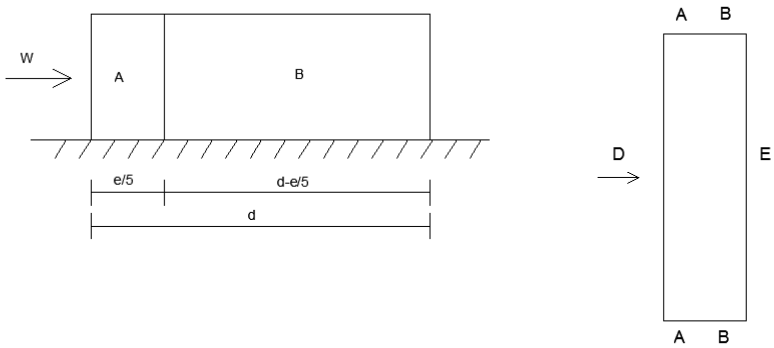
Surface D (east/west facade in the project):
$W_e = 0.7 * 0.9 = 0.63\ kN/m^2$





Wind load on vertical walls (north-south direction)

e=the smaller of [b, 2h], in which b=277.2m, 2h=46m
e=46m > d=26.4m



e/5=9.2m
d-e/5=17.2m
h/d=23/26.4=0.87

Fig A5-1 Eurocode 1-4, figure 7.5 key for vertical walls

According to Table 7.1

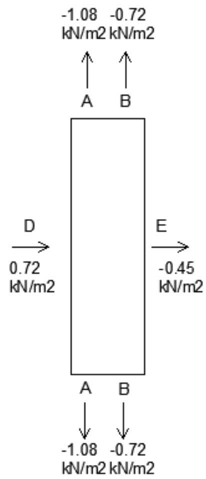
Zone	A	B	D	E
h/d=1	-1.2	-0.8	+0.8	-0.5

Table A5-2 C_{pe} ***– pressure coefficient for the external pressure***

The whole wind force acting on east/west facade
$F_{w,e} = 0.63 * 26.4 * 23 = 382.5 \text{ kN}$

The whole wind force acting on the north facade
$F_{w,e} = 0.72 * 277.2 * 23 = 4590 \text{ kN}$

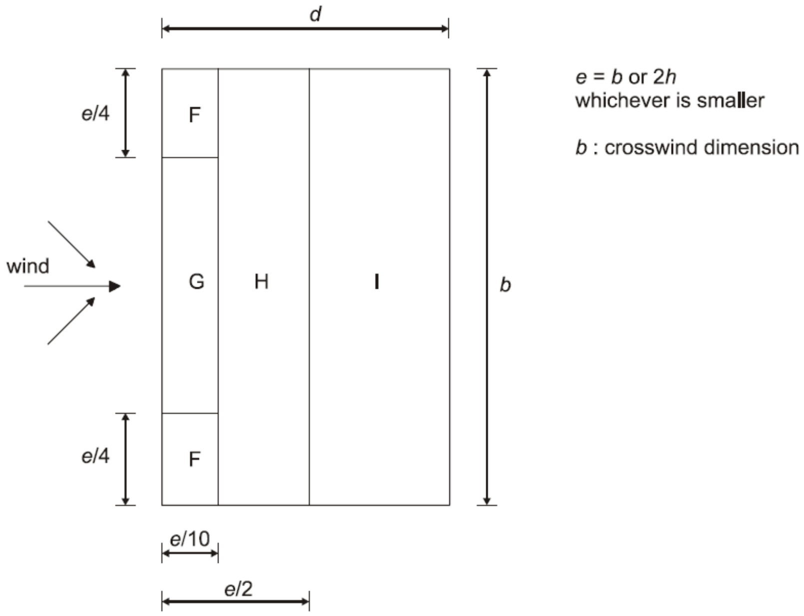
Then it can be concluded that compared to the wind force in east-west direction, which in north-south direction is the domain so that wind load from that direction would be considered in the design.



Surface D: $W_e = 0.8 * 0.9 = 0.72 \text{ kN/m}^2$
Surface E: $W_e = -0.5 * 0.9 = -0.45 \text{ kN/m}^2$
Surface A: $W_e = -1.2 * 0.9 = -1.08 \text{ kN/m}^2$
Surface B: $W_e = -0.8 * 0.9 = -0.72 \text{ kN/m}^2$

According to section 5.3(4), wind friction forces will be disregarded.

Wind load on roof



In which,
e=46m
e/4=11.5m
e/10=4.6m
e/2=23m
d=26.4m
b=277.2m

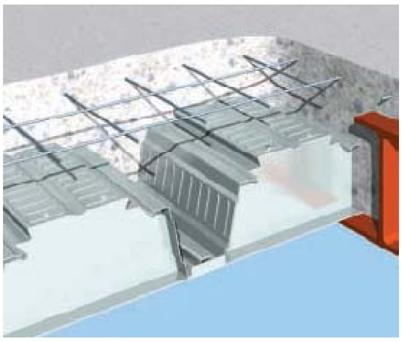
According to Table 7.2 in Eurocode 1-4

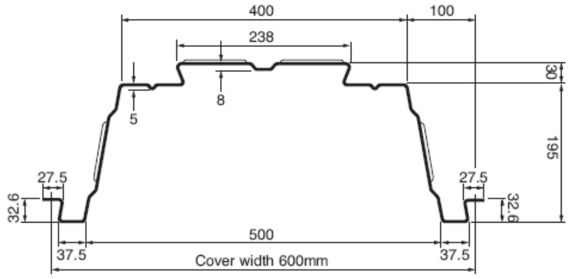
Roof Type	F	G	H	I
Shape eaves	-1.8	-1.2	-0.7	±0.2

Table A5-3 C_{pe} ***– pressure coefficient for the flat roof***

Surface F: $W_e = -1.8 * 0.9 = -1.62 \text{ kN/m}^2$
Surface G: $W_e = -1.2 * 0.9 = -1.08 \text{ kN/m}^2$
Surface H: $W_e = -0.7 * 0.9 = -0.63 \text{ kN/m}^2$
Surface I: $W_e = \pm 0.2 * 0.9 = \pm 0.18 \text{ kN/m}^2$

Appendix 6 ComFlor® 225 Floor System (©Corus)





ComFlor® 225 Composite Slab - Volume & Weight

Slab Depth (mm)	Concrete volume (m^3/m^2)	Weight of Concrete (kN/m^2) Normal weight Concrete Wet	Normal weight Concrete Dry	Lightweight Concrete Wet	Lightweight Concrete Dry
285	0.116	2.74	2.68	2.17	2.05
290	0.121	2.85	2.79	2.26	2.14
295	0.126	2.97	2.91	2.35	2.23
300	0.131	3.09	3.02	2.45	2.32
305	0.136	3.21	3.14	2.54	2.41
310	0.141	3.32	3.26	2.63	2.49
320	0.151	3.56	3.49	2.82	2.67
350	0.181	4.27	4.18	3.38	3.20
380	0.211	4.97	4.87	3.94	3.73
400	0.231	5.44	5.33	4.31	4.08

Volume & weight table notes

1. Deck and beam deflection (i.e. ponding) is not allowed for in the table.
2. Deck and mesh weight is not included in the weight of concrete figures.
3. Density of concrete is taken as:
   Normal weight (wet) 2400 kg/m³
   Normal weight (dry) 2350 kg/m³
   Lightweight (wet) 1900 kg/m³
   Lightweight (dry) 1800 kg/m³

Section Properties (per metre width)

Nominal thickness (mm)	Design thickness (mm)	Profile weight (kN/m^2)	Area of steel (mm^2/m)	Height to neutral axis (mm)	Moment of inertia (cm^4/m)	Ultimate Moment capacity (kNm/m) Sagging	Hogging
1.25	1.21	0.17	2118	107.00	968.00	30.80	30.80

ComFlor 225 span tables

Table 3.1 60 min fire resistance, Light Weight Concrete (LWC), 2.5 kN/m² LL

Slab Depth (mm)		290	300	310	320	330	340	350	360
Slab Weight (kN/m²)		2.7	2.89	3.08	3.27	3.46	3.65	3.84	4.03
Bar dia (mm)	Props	Span limit (m) for s = simply supported, p = partial continuity							
16	Unpropped	6.3	6.2	6.1	6.0	5.8	5.7	5.6	5.5
	1	7.0 s **L** 7.6 p **L**	7.0 s **L** 7.6 p **L**	7.1 s **L** 7.7 p **L**	7.1 s **L** 7.8 p **L**	7.2 s **L** 7.7 p **L**	7.2 s **L** 7.4 p **L**	7.1 **L**	6.8 **L**
	2	7.0 s **L** 7.6 p **L**	7.0 s **L** 7.7 p **L**	7.1 s **L** 7.7 p **L**	7.1 s **L** 7.8 p **L**	7.2 s **L** 7.9 p **L**	7.2 s **L** 7.9 p **L**	7.3 s **L** 7.9 p **L**	7.3 s **L** 8.0 p **L**
20	Unpropped	6.3	6.2	6.1	6.0	5.8	5.7	5.6	5.5
	1	7.5 s **L** 7.9 p **L**	7.6 s **L** 8.1 p **L**	7.6 s **L** 8.2 p **L**	7.7 s **L** 8.0 p **L**	7.7 **L**	7.4 **L**	7.1 **L**	6.8 **L**
	2	7.5 s **L** 8.0 p **L**	7.6 s **L** 8.1 p **L**	7.6 s **L** 8.2 p **L**	7.7 s **L** 8.3 p **L**	7.8 s **L** 8.5 p **L**	7.9 s **L** 8.6 p **L**	7.9 s **L** 8.7 p **L**	8.0 s **L** 8.7 p **L**
25	Unpropped	6.3	6.2	6.1	6.0	5.8	5.7	5.6	5.5
	1	7.7 s 8.2 p **L**	7.8 s 8.3 p **L**	7.9 s **L** 8.4 p **L**	8.0 **L**	7.7 **L**	7.4 **L**	7.1	6.8
	2	7.7 s 8.2 p **L**	7.8 s 8.3 p **L**	7.9 s **L** 8.5 p **L**	8.0 s **L** 8.6 p **L**	8.0 s **L** 8.7 p **L**	8.1 s **L** 8.8 p **L**	8.2 s **L** 8.9 p **L**	8.3 s **L** 9.0 p **L**
32	Unpropped	6.3	6.2	6.1	6.0	5.8	5.7	5.6	5.5
	1	8.0 s 8.5 p	8.1 s 8.6 p **L**	8.2 s 8.4 p **L**	8.0	7.7	7.4	7.1	6.8
	2	8.0 s 8.5 p	8.2 s 8.7 p **L**	8.2 s 8.8 p **L**	8.3 s 8.9 p **L**	8.4 s **L** 9.1 p **L**	8.5 s **L** 9.2 p **L**	8.6 s **L** 9.3 p **L**	8.6 s **L** 9.4 p **L**

Appendix 7 Results of ESA PT model

There are three documents produced by ESA PT attached in this chapter section, which are the results of

- 3D truss whole structure – entire office part
- Final Structure 1 – entire north side of Breda CS
- Final Structure 3 – optimized entire north side of Breda CS

Project	3D Truss Whole
Part	-
Description	entire office part

1. Load cases

Name	Description	Action type	LoadGroup	Load type	Spec	Duration	Master load case
LC1	Self Weight	Permanent	LG1	Standard			
LC2	Dead Load	Permanent	LG1	Standard			
LC3	Live Load	Variable	LG2	Static	Standard	Long	None
LC4	Wind Load	Variable	LG3	Static	Standard	Long	None

2. Combinations

Name	Description	Type	Load cases	Coeff. [1]
SLS	SLS	Linear - serviceability	LC2 - Dead Load	1,00
			LC4 - Wind Load	1,00
			LC1 - Self Weight	1,00
			LC3 - Live Load	1,00
ULS	ULS	Linear - ultimate	LC2 - Dead Load	1,20
			LC4 - Wind Load	1,50
			LC1 - Self Weight	1,20
			LC3 - Live Load	1,50

3. Internal forces on member

Linear calculation, Extreme : Global, System : Principal
Selection : All
Combinations : ULS

Member	Case	dx [m]	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
B4168	ULS/1	13,682	**-4596,89**	0,00	0,00	0,00	0,00	0,00
B4185	ULS/1	0,000	**2592,19**	-249,28	30,23	3,58	-13,84	496,75
B4185	ULS/1	7,200	1111,17	**-296,75**	-26,69	-0,83	30,24	474,66
B4070	ULS/1	0,000	2455,25	**283,21**	29,14	-4,32	-14,21	-556,56
B860	ULS/1	13,000	0,00	0,00	**-492,46**	0,00	-1066,99	0,00
B3999	ULS/1	0,000	90,02	-0,06	**500,16**	0,01	**-1069,04**	0,33
B4878	ULS/1	0,000	393,19	77,95	7,29	**-5,05**	-4,62	-118,68
B3575	ULS/1	0,000	380,91	73,09	-4,60	**4,18**	2,73	-108,82
B4076	ULS/1	6,500	367,97	-0,17	4,12	0,01	**630,36**	0,09
B4185	ULS/1	10,800	1104,64	-296,75	-26,69	-0,83	-65,83	**-593,64**
B4070	ULS/1	10,800	1032,39	261,80	-24,85	0,07	-61,65	**527,69**

4. Deformation of nodes

Linear calculation, Extreme : Global
Selection : All
Combinations : SLS

Node	Case	Ux [mm]	Uy [mm]	Uz [mm]
N1556	SLS/2	**-5,3**	-8,0	-28,8
N1580	SLS/2	**11,1**	-7,2	-31,9
N1549	SLS/2	-2,0	**-8,4**	-13,7
N138	SLS/2	0,0	**0,0**	**0,0**
N1534	SLS/2	0,2	-5,4	**-39,6**

Author	Y Yao	Date	08. 05. 2009
Version	SCIA.ESA PT 7.1.170		

5. Relative deformation

Linear calculation,Extreme : Global, System : Principal
Selection : All
Combinations : SLS

Case - combination	Member	dx [m]	uy [mm]	Rel uy [1/xx]	uz [mm]	Rel uz [1/xx]
SLS/2	B4070	9,600	**-1,9**	**1/5659**	-1,0	1/10000
SLS/2	B3515	8,400	**3,3**	**1/3288**	0,0	1/10000
SLS/2	B4075	6,500	0,0	1/10000	**-65,0**	1/406
SLS/2	B3499	8,400	0,0	0	**14,4**	1/10000
SLS/2	B3527	7,000	0,0	1/10000	-54,4	**1/239**
SLS/2	B4323	8,400	0,0	0	12,8	**1/1312**

6. Stress

Linear calculation,Extreme : Global
Selection : All
Combinations : ULS

Member	Case	dx [m]	Normal - [MPa]	Normal + [MPa]	Shear [MPa]	von Mises [MPa]	Fatigue [MPa]	Kappa [1]
B4168	ULS	13,682	**-388,3**	0,0	0,0	388,3	0,0	0,00
B624	ULS	0,000	**0,0**	144,5	8,4	144,5	0,0	0,00
B553	ULS	2,400	-78,2	**0,0**	22,9	78,2	0,0	0,00
B4070	ULS	0,000	-145,9	**396,4**	32,8	**396,5**	0,0	0,00
B2800	ULS	0,000	-133,1	0,0	**0,0**	133,1	0,0	0,00
B3999	ULS	0,000	-300,6	390,7	**45,0**	391,4	0,0	0,00
B2820	ULS	10,525	0,0	0,0	0,0	**0,0**	0,0	0,00
B553	ULS	0,000	-237,6	157,5	22,9	237,6	**0,0**	**0,00**

Project	Final Structure
Part	-
Description	entire north side

1. Load cases

Name	Description	Action type	LoadGroup	Load type	Spec	Duration	Master load case
LC2	Dead Load	Permanent	LG1	Standard			
LC4	Wind Load	Variable	LG3	Static	Standard	Long	None
LC1	Self Weight	Permanent	LG1	Standard			
LC3	Live Load	Variable	LG2	Static	Standard	Long	None

2. Combinations

Name	Description	Type	Load cases	Coeff. [1]
SLS	SLS	Linear - serviceability	LC2 - Dead Load	1,00
			LC4 - Wind Load	1,00
			LC1 - Self Weight	1,00
			LC3 - Live Load	1,00
ULS	ULS	Linear - ultimate	LC2 - Dead Load	1,20
			LC4 - Wind Load	1,50
			LC1 - Self Weight	1,20
			LC3 - Live Load	1,50

3. Internal forces on member

Linear calculation, Extreme : Global, System : Principal
Selection : All
Combinations : ULS

Member	Case	dx [m]	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
B4962	ULS/1	0,000	**-11928,39**	-15,85	-14,00	0,09	20,30	-30,07
B4277	ULS/1	0,000	**4716,98**	0,13	99,60	-0,06	-84,06	-5,34
B4911	ULS/1	0,000	-10196,17	**-556,36**	14,92	11,99	36,41	796,07
B556	ULS/1	0,000	-475,15	**269,42**	-4,63	1,50	1,46	-501,64
B4076	ULS/1	13,000	215,30	1,51	**-524,20**	0,19	**-1198,22**	9,62
B4280	ULS/1	0,000	32,05	0,03	**506,76**	-0,03	-1103,42	0,32
B4952	ULS/1	0,000	-4381,76	-134,18	125,52	**-147,81**	-515,42	218,07
B4951	ULS/1	0,000	-2199,51	48,08	21,54	**172,74**	46,59	-175,01
B4076	ULS/1	6,067	215,30	1,51	0,16	0,19	**618,45**	-0,86
B4911	ULS/1	3,500	-10211,01	-556,36	14,92	11,99	88,63	**-1151,18**
B4923	ULS/1	0,000	-9267,42	-529,26	0,94	22,54	5,24	**832,35**

4. Deformation of nodes

Linear calculation, Extreme : Global
Selection : All
Combinations : SLS

Node	Case	Ux [mm]	Uy [mm]	Uz [mm]
N1591	SLS/2	**-20,3**	-5,7	-6,8
N1654	SLS/2	**16,9**	-8,1	-5,5
N1625	SLS/2	9,0	**-26,2**	-2,8
N370	SLS/2	-1,1	**15,3**	-10,0
N1533	SLS/2	-8,6	-12,6	**-41,7**

Node	Case	Ux [mm]	Uy [mm]	Uz [mm]
N1866	SLS/2	0,0	-1,4	**0,1**

5. Stress

Linear calculation,Extreme : Global
Selection : All
Combinations : ULS

Member	Case	dx [m]	Normal - [MPa]	Normal + [MPa]	Shear [MPa]	von Mises [MPa]	Fatigue [MPa]	Kappa [1]
B4905	ULS	3,500	**-436,1**	0,0	22,4	**436,2**	0,0	0,00
B624	ULS	0,000	**0,0**	124,0	5,5	124,0	0,0	0,00
B553	ULS	1,800	-34,7	**0,0**	10,7	36,5	0,0	0,00
B4074	ULS	13,000	-299,5	**371,4**	46,3	371,4	0,0	0,00
B2800	ULS	0,000	-108,8	0,0	**0,0**	108,8	0,0	0,00
B5386	ULS	0,000	-203,5	194,1	**96,9**	212,1	0,0	0,00
B5434	ULS	1,434	0,0	0,0	0,0	**0,0**	0,0	0,00
B553	ULS	0,000	-103,9	46,3	10,7	103,9	**0,0**	**0,00**

6. Relative deformation

Linear calculation,Extreme : Global, System : Principal
Selection : All
Combinations : SLS

Case - combination	Member	dx [m]	uy [mm]	Rel uy [1/xx]	uz [mm]	Rel uz [1/xx]
SLS/2	B4952	0,000	**-12,5**	**1/352**	0,0	0
SLS/2	B4897	6,061	**10,8**	1/563	0,0	0
SLS/2	B4892	0,000	8,4	**1/523**	0,0	0
SLS/2	B4074	6,067	0,2	1/10000	**-55,2**	1/498
SLS/2	B5360	0,000	0,0	0	**24,8**	1/513
SLS/2	B5380	3,155	0,4	1/7947	-12,2	**1/258**
SLS/2	B5365	0,000	0,0	0	22,8	**1/501**

1. Load cases

Name	Description	Action type	LoadGroup	Load type	Spec	Duration	Master load case
LC2	Dead Load	Permanent	LG1	Standard			
LC4	Wind Load	Variable	LG3	Static	Standard	Long	None
LC1	Self Weight	Permanent	LG1	Standard			
LC3	Live Load	Variable	LG2	Static	Standard	Long	None

2. Combinations

Name	Description	Type	Load cases	Coeff. [1]
SLS	SLS	Linear - serviceability	LC2 - Dead Load	1,00
			LC4 - Wind Load	1,00
			LC1 - Self Weight	1,00
			LC3 - Live Load	1,00
ULS	ULS	Linear - ultimate	LC2 - Dead Load	1,20
			LC4 - Wind Load	1,50
			LC1 - Self Weight	1,20
			LC3 - Live Load	1,50

3. Internal forces on member

Linear calculation,Extreme : Global, System : Principal
Selection : All
Combinations : ULS

Member	Case	dx [m]	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
B4962	ULS/1	0,000	**-11974,87**	-24,40	-17,75	0,26	26,34	-15,30
B4277	ULS/1	0,000	**4700,35**	0,05	100,84	-0,06	-81,05	-5,76
B4911	ULS/1	0,000	-10238,21	**-567,43**	17,75	15,13	46,23	747,76
B556	ULS/1	0,000	-471,09	**247,81**	-3,79	1,40	1,38	-458,27
B4076	ULS/1	13,000	232,57	1,34	**-518,96**	0,14	**-1172,26**	8,51
B4280	ULS/1	0,000	35,34	-0,01	**509,75**	-0,02	-1131,42	0,78
B4952	ULS/1	0,000	-4359,61	-154,63	134,55	**-165,60**	-526,09	258,71
B4951	ULS/1	0,000	-2220,48	54,71	17,54	**183,22**	61,24	-191,56
B4899	ULS/1	3,500	-9453,75	-328,21	153,13	-36,95	**612,64**	-812,67
B4911	ULS/1	3,500	-10261,36	-567,43	17,75	15,13	108,35	**-1238,26**
B4923	ULS/1	0,000	-9317,89	-536,10	1,01	28,45	5,96	**798,87**

4. Deformation of nodes

Linear calculation,Extreme : Global
Selection : All
Combinations : SLS

Node	Case	Ux [mm]	Uy [mm]	Uz [mm]
N1591	SLS/2	**-19,1**	-5,2	-6,1
N1654	SLS/2	**16,8**	-7,0	-5,4
N1625	SLS/2	9,1	**-23,4**	-2,9
N370	SLS/2	-1,1	**16,8**	-9,6
N1596	SLS/2	8,2	-19,1	**-38,9**

Node	Case	Ux [mm]	Uy [mm]	Uz [mm]
N1866	SLS/2	0,0	-1,4	**0,1**

5. Stress

Linear calculation,Extreme : Global
Selection : All
Combinations : ULS

Member	Case	dx [m]	Normal - [MPa]	Normal + [MPa]	Shear [MPa]	von Mises [MPa]	Fatigue [MPa]	Kappa [1]
B4076	ULS	13,000	**-335,8**	351,1	**85,6**	351,1	0,0	0,00
B624	ULS	0,000	**0,0**	102,9	4,6	102,9	0,0	0,00
B553	ULS	1,800	-29,6	**0,0**	11,1	32,4	0,0	0,00
B3242	ULS	0,000	-294,5	**364,9**	45,5	**364,9**	0,0	0,00
B2800	ULS	0,000	-110,2	0,0	**0,0**	110,2	0,0	0,00
B5432	ULS	1,100	0,0	0,0	0,0	**0,0**	0,0	0,00
B553	ULS	0,000	-94,0	43,7	11,1	94,0	**0,0**	**0,00**

6. Relative deformation

Linear calculation,Extreme : Global, System : Principal
Selection : All
Combinations : SLS

Case - combination	Member	dx [m]	uy [mm]	Rel uy [1/xx]	uz [mm]	Rel uz [1/xx]
SLS/2	B4952	0,000	**-11,8**	**1/374**	0,0	0
SLS/2	B4894	0,000	**10,7**	1/1478	0,0	0
SLS/2	B4892	0,000	8,0	**1/549**	0,0	0
SLS/2	B4074	6,067	0,2	1/10000	**-53,9**	1/510
SLS/2	B5360	0,000	0,0	0	**23,7**	**1/535**
SLS/2	B4533	6,500	0,2	1/10000	-46,9	**1/277**

Reference

[1] Stan Majoor and Dick Schuiling, 2007, New Key Projects for station redevelopment in the Netherlands, Railway Development
[2] Gemeente Breda, 2007, Bestemmingsplan Stationskwartier, available at http://www.breda.nl
[3] Project Breda Central Station, 2009, DHV B.V. Building and Industry
[4] James E. Ambrose and Dimitry Vergun, 1999, Design for Earthquakes, Wiley
[5] General Theory of Bubbledeck, available at http://www.bubbledeck.com
[6] Mohsen Mostafavi, 2006, Structure as Space: Engineering And Architecture in the works of Jnrg Conzett, Architectural Assoc
[7] Visualizations courtesy of A2 Studio
[8] Corus, 2007, Slimdeck Manual, available at http://www.corusconstruction.com
[9] Andrew W. Charleson, 2005, Structure As Architecture: A Source Book For Architects And Structural Engineers, Elsevier Science & Technology Books
[10] Daniel Lewis Schodek, 2007, Structures, Prenticee Hall, 6th Edition
[11] James Ambrose, 1994, Design of Building Trusses, Wiley
[12] Cecil Balmond, 2002, Informal, Prestel USA
[13] ESDEP Courses, University of Leuven, available at http://www.esdep.org
[14] Alessia Ferrarini, 2005, Railway Station: From the Gare de L'Est to Penn Station
[15] Stefan Plonyi, 1993, Interpreting the Supporting Structures, of Architecture, Lotus 79, After High Tech
[16] T.Y. Lin, 1983, Moscone Convention Center - an Underground storey, available at ftp://imgs.ebuild.com/woc/C830103.pdf
[17] Jason A. Cook, 2006, Structural Steel Framing Options for Mid- an High-rise Buildings, Master’s Thesis, Massachusetts Institute of Technology
[18] Dr. Ing. Christian Muller and Dr. Ing. Matthias Oppe, 2008, Concept design and design examples for multi-storey buildings
[19] Steelwork Design Guide to BS 5950-1:2000, Volume 1, Section Properties and Member Capacities
[20] European Wide Flange Beam Section Properties and Member Capacities
[21] R. Abspoel, 2006, CT4281 Building Structures 2, steel part
[22] Unknown author, 2006, The new Southern Cross Station, steel Australia, March 2006
[23] Jack Stroud Foster and Roger Greeno, 2007, Structure and Fabric, Part 1, Pearson Prentice Hall, 7th Edition
[24] J Y Richard Liew, 2004, Buildable Design of Multi-storey and Large-span Steel Structures, Steel Structures 4 (2004) 53-70
[25] T. Graafland, 2008, The Structural Design of the Rotterdam CS Southern Hall, Master’s Thesis, Delft University of Technology
[26] Construction Innovation Forum, 2007 Nova Award Nomination 22, Post-tensioned Steel Trusses for Long Span Roofs

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