DELFT UNIVERSITY OF TECHNOLOGY

FACULTY OF CIVIL ENGINEERING AND GEOSCIENCES
DEPARTMENT OF BUILDING ENGINEERING

Adding stories on top of the existing building by using steel structures

Case study: Dillenburgsingel project in Leidschendam

Master of Science Thesis
By

Leopold Uwimana

Delft, June- 2011

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This research is done for the partial fulfillment of requirements for the Master of Science degree at Delft University of Technology
TU-Delft, Delft, the Netherlands

Delft

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To Marie Claire INEZA

Preface

For the completion of the Delft Civil Engineering study, this thesis has been written. The knowledge gained during the Civil Engineering study had to be applied on a certain technical subject. For this thesis, it has been chosen to make an alternative structural design for adding stories on existing building by using a case study of Dellenbourgsingle in Leidschendam. The engineering office SmitWesterman of Gouda, created a structural design for a building of fifteen stories by demolishing the existing building of six stories. Thereafter SmitWesterman, wished to create another alternative by creating more stories and even cantilevering the existing building which would not be demolished. This design project was therefore done as Master project at Building section of Delft University of Technology .In this report, the first part of the thesis, a study will be done to find feasible structural concepts for the Southern Hall.

The success of the thesis would not have been possible without the contribution and guidance of the examination committee. To this regard, I would like to express my sincere gratitude to my supervisor Prof.dipl.ing. J.N.J.A. Vambersky, who gave the opportunity to work on this project,. His guidance, inspiring discussion, great patience and the freedom he gave in the research helped to complete the project. I also thank Prof. R. Nijsse who took over the role of Prof Vambersky to guide me until the end of the project.

Many thanks to Ir. R, Abspoel for his major contributions to the completion of this thesis. He gave most generously his time any time I need it within the sole objective of getting it right and the trouble he took to provide me with necessary instructions and to discuss my problems throughout the entire project.

I am grateful to Ing. R.Stark of SmithWesterman for his tremendous contribution in providing the topic as well as the required data and various information.

Furthermore, I would like to thank Ing. H.Everts for his assistance in foundation design by providing me with valuable information pertaining to the fulfilment of this research.

I owe much to my Parents, my brother B. Uwamahoro, my fiancée M.C.Ineza my uncle E. Muzindutsi and the rest of my family for their long-distance encouragement and financial support during difficulty times.

Leopold Uwimana

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Summary

This study intends to develop a design method of adding more stories on existing building by using a case study of construction project in Leischendam where 9-stories are to be added on existing 6-storey residential building. A low riser building is to be changed into multistory building. The state-of-art of the project was designed by the Engineering office of SmitWesterman and implemented by demolishing a section of the existing building where additional stories were constructed. The initial idea was to add 9-stories on existing 6 to form a structure of 15 story-high without demolition. Even though this idea was abandoned during implementation but it was forwarded to the University to make further research and formulate possible design alternatives.
This report, in its chapter 2, describes the existing building by estimating the capacity of superstructure as well as the foundation in existing building by using data obtained during the design in 1968. The reason for this checking is to know if, the existing building can take a part of weight from the new added structure.

It has been obtained that the super structure of the existing building was designed to take only the load of 6-stories. However, the foundation piles show a very high capacity but cannot be used directly by the additional building which brings to the ground to much load exceeding the capacity of the existing piles. The changed form of the new structure (cantilevering existing building), brings also too much weight to the foundation and it is practically impossible to add more piles to the existing foundation without demolishing the existing structure.

The new structural concept will be developed in Chapters 4 and 6, whereby steel frames and Hollow core slab units will be used to make a new structure. The existing building will be bridged; it is not supposed to take any vertical load from the new structure but it can be facilitate carrying lateral loads by connecting it to the new structure.

After choosing the structural system, the design alternatives were developed in chapter 6. Based on Multi Criteria Analysis in Chapter 7, the alternative with continuous parallel girders and continuous columns was decided to be the final design. The reason is the simplicity in erection and connection, easy construction of cantilevers. Other details of the selected design will be made in Chapters 8 to 12 in a combination with calculations in Appendices 4 to 10.

The new structure is not totally on top of the existing building, there is a part that starts from the ground level which will be used to take much of the horizontal loading. In chapter 8 and Appendix 7, lateral load bearing system, a concrete core accommodating stair case and lift shaft, showed less capacity to take alone all horizontal loadings. It was decided to supplement the concrete core with three braced steel frames.

The foundation was also studied in Chapter 12 and Appendix 11. The weight from the new building is directed to the sides of the existing building. The choice of an appropriate method to install piles is of big importance since a good method should not cause damage in existing building. Fundex piles were selected to be used in this project, they are soil displacement method and do not cause vibrations which can induce further settlements of the already settled existing structure.

The final part of the thesis, Chapter 13, consists of the conclusions and recommendations .It was concluded that it is possible to change a low riser building into multi-storey building . In the case study used, a system built up with continuous twin girders cantilevering at each floor level on top of the existing building, continuous steel columns and concrete core with braced frames for stability, is the most suitable structural system for this case.

It is recommended that when adding more stories on existing building; the shape of additional structure must match with that of the existing structure. Steel frames should be used due their low weight to avoid more stresses in foundation of the existing structure. Both structures should be connected laterally to enhance lateral stability. Soil displacement foundation should be used to avoid vibration that can cause damage in the existing building. It is also very important to conduct a further research in dynamic behavior of the added structure when more and more stories are to be added. Thermal and sound properties should also be studied, interaction between existing and new foundation can also be studied because this research deals mostly with structural design of the superstructure. Connection devices between existing and new structures must be developed.

PART 1: CONCEPT STUDY

1. INTRODUCTION

1.1. Research background

1.1.1 Technical characteristics

Adding stories to existing building relates to the construction area where existing low riser building is to be transformed into high-riser building.

The addition of additional stories to existing buildings is a new theme in development of urban construction all over the world [12]. With the increase of urban population, cities are bound to expand but actual area of individual city cannot expand at will. It is therefore necessary to confine the development within the scope of the city properly. This requires raising the height of buildings in the city, especially where the existing buildings are very low in height, where contradiction between reality and requirement is most prominent.

Up to now, there are two solutions to this contradiction:

i. Demolishing the existing low riser buildings and construct new high- riser buildings at the site. In doing so , not only the problem of moving people to other places, as well as the disposal of waste from the construction site , but also that some of the buildings are forced down without reaching there service maturity.

ii. Raising the height of the existing buildings. At present, raising the height of buildings comprises mainly two processes:

- The existing building is retained, and few stories are added to it; the weight of the additional stories are to be supported by the existing building structures. However, as the bearing capacity of the existing building structure is quite limited, only one or two story can be added at most.
- More stories are added by means of pure frame; the weight of the added stories cannot be transmitted to the foundation via the frame structure, because the frame structure has not taken into consideration in its design the precise route of force transmission of the added structure of the stories.

Adding stories on existing buildings has been widely done by using steel structure in maintenance and reconstruction of buildings. Steel is preferred due to these advantages: [19]

- Good vibration performance
- Light weight
- Instant construction
- New structure and its architectural plane are not limited by the old building

There are many projects of adding stories on existing building, but no related technical researches have been done. [19] The reason is that the mechanical properties and stress mode of these structures are not deeply searched and therefore relevant theory and criterion are not formed. There are only some professional standards and construction experience can be used by the designer and constructor.
Therefore the working performance, analytical method and calculation mode of these types of structure must be researched, especially for the new and old structure foundation and integral performance to resist lateral loads.

1.1.2 Case study

The case study consists of the construction project located at Dellenburgsingel in Leischendam; it is 6-stories apartment building. As more rooms are needed, a part of this building needs to be changed into multistory building; from 6 to 15 stories.

The number of floors to be added is 9, so, supporting the new structure to the old one may not be possible.
As there are two solution of providing these additional floors to the existing building, namely demolishing the existing building and starting a multistory building and adding stories on existing building . The two scenarios

have to be analyzed and the most promising one will be implemented.The first option consists of demolishing the existing low riser apartment buildings since they are old and make a new beginning for the high riser apartment buildings.

The second option is to provide the additional stories on the top of the existing apartment buildings. The existing building will not be demolished and the new building will be added on top of it. This method will obviously be chosen because it is the most important objective of this thesis.
The major problem is to know if the old building can be connected to the new one or look for other structural solutions.

The structural behavior of the old building and its foundation is a key factor to know whether the new and the old structures can be integrated or not. In the beginning of this thesis work, the capacity of the existing structure will be investigated. Therefore the structural solutions will be to make; either integrated or separate design. For both measures the light weight structures have to be considered since no excess loads are to be transferred to the existing building and no excess stresses are allowed near the existing foundations.

After investigating the capacity of the existing building, the major part of this thesis will be started where some structural concepts using light weight structures will be proposed and the most feasible concept will be designed and detailed.

Furthermore, the last part of this thesis is the study of foundation method that will be designed for the chosen structural concept. This is of great importance since one has to look the interaction between the old and the new structures when they are separated. For combined or integrated design one has also to look for the technique required to increase the bearing capacity of the old foundation structure.

1.2 Research objectives

The objectives of this study are to generate structural design and techniques to be used when adding more floors on existing residential building without demolishing the existing building by using a case study of Dillenburgsingel project in Leischendam. As the number of additional floors is big (9) an other purpose of this research to provide a method for adding additional stories to an existing low riser building for raising its height in which the weight of the newly added multistory building portion is transmitted to the ground base via a weight supporting frame structure connected to the existing building, rather than supported by the existing building, so that the routes of force transmission of the added integral structure in any circumstance will not be confused.

Furthermore, Dillenburgsingel project as a residential 6-stories building, it has many other buildings constructed in similar way in Leischendam and elsewhere in other cities, which can use this techniques of adding stories in order to generate more homes in future without demolishing existing buildings.

Finally, this research also intends to develop possibilities of using lightweight materials (steel structures) as solution for additional stories on existing residential buildings.

1.3 Problem definition

In this study, the major issue to be solved is to know how an existing low rise building can be extended to a multistory building. This study would have a significant effect on already built residential areas whereby more homes can be built in extremely limited place without demolishing the existing buildings. The study will be done by using the case study of Dillenburgsingel project in Leidschendam.

Apart from the possibility of adding story, the structural system and construction material would not be the same as the traditional way of construction. When making additional stories on top of the existing building, the traditional concrete building for apartments is not a good solution due to its excessive weight that can induce further settlement of the existing foundation. Steel frames would be selected since they can have large spans and lightweight.

1.4 Research methodology

This research consists of adding stories on existing building. As the new and old structure serve the same function, their function must be obviously integrated but their structures can be either integrated or separated. This research start with checking the capacity of old building and thereafter, structural design concept will be made for the new added structure by either being integrated with or not integrated with the old building.

- **Integrated design**: The capacity of existing building will be checked for both superstructure and foundation. If the existing structure shows to be able to carry some loads acting on new structure, light weight materials would be used to avoid excess load being imposed on existing building.

- **Separated design**: If the capacity of existing structure cannot carry load from additional structure, the new structure will be designed differently from the existing one. However, adding excessive load near the existing building foundation can create unwanted deformation; hence, lightweight materials will be selected.

- **Partially integrated**: This can be implemented in case the old building presents weakness in some structural aspects.

For a chosen possibility, various alternatives will be generated. All alternatives will be judged and compared from functional, aesthetical and technical point of view in order to validate a method to be used when adding extra storey on existing residential buildings. The alternative, which is the most promising, will be selected for further design (top and foundation structures). Finally, the conclusion and recommendations for the final structure will be made.

1.5 Scope

This study will provide a structural system and design of frame of the additional building. Furthermore, this study will cover the foundation design for the new structure. In this study only steel frame will be used as lightweight material, therefore there will not be an investigation on differences between steel and concrete frame. An investigation can be done in further studies whereby concrete frames will be designed and then a comparison with steel frame (focused in this study) can be made

In addition, there will be no comparison between proposed alternative and current construction in concrete done by demolishing the old building. The aim of this project is simply providing possibilities and structural design required to add more stories on existing building by using lightweight materials (steel). Acoustic and thermal design will not be covered in this project, however their effect will be considered in making structural design of the frame as well as the foundation.

2. OVERVIEW OF THE EXISTING BUILDING

In this part, the existing building will be described in terms of its foundation bearing capacity, the capacity of columns and the capacity of the building in resisting lateral loads. The design alternatives will be made according to the behavior of the existing building.

2.1 Environment

The building is located in the northeast part of Leischendam municipality between Prins Johan Willem Frisolaan and Prinsenhof streets respectively in its north side and its south side. At its east, side there is Dillenburgsingel and Prins Frederiklaan to the west. The figure 2-1 shows the location of the building block of this study.

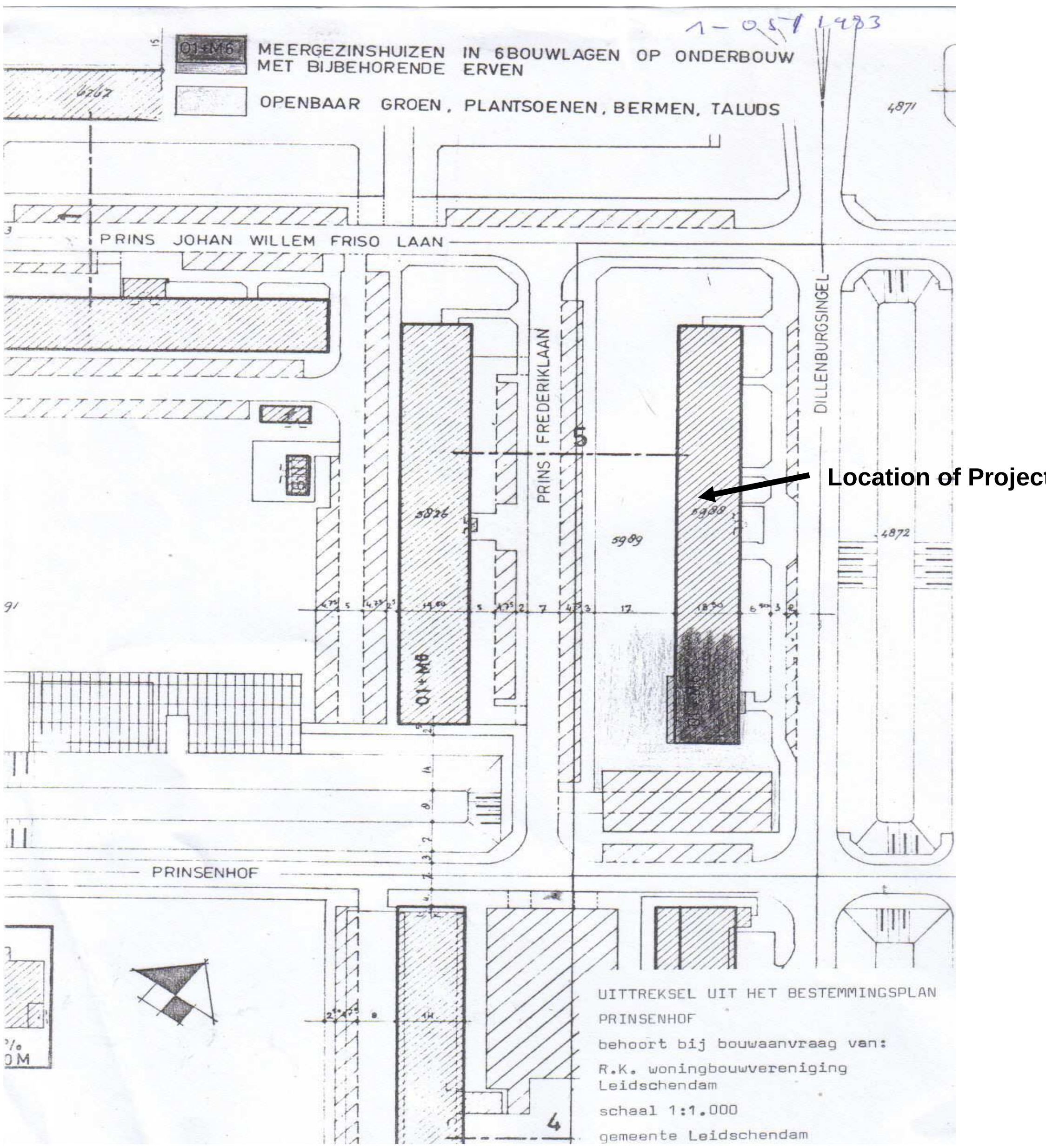


Figure 2- 1: existing spatial location of the building in this study [20]

The distance between the edges of the building and Dilenburgsingel is 10m; while the distance to the Prins Frederiklaan is 20m. The piping systems and other public utilities are located at Prins Frederiklaan side- Figure2-2.

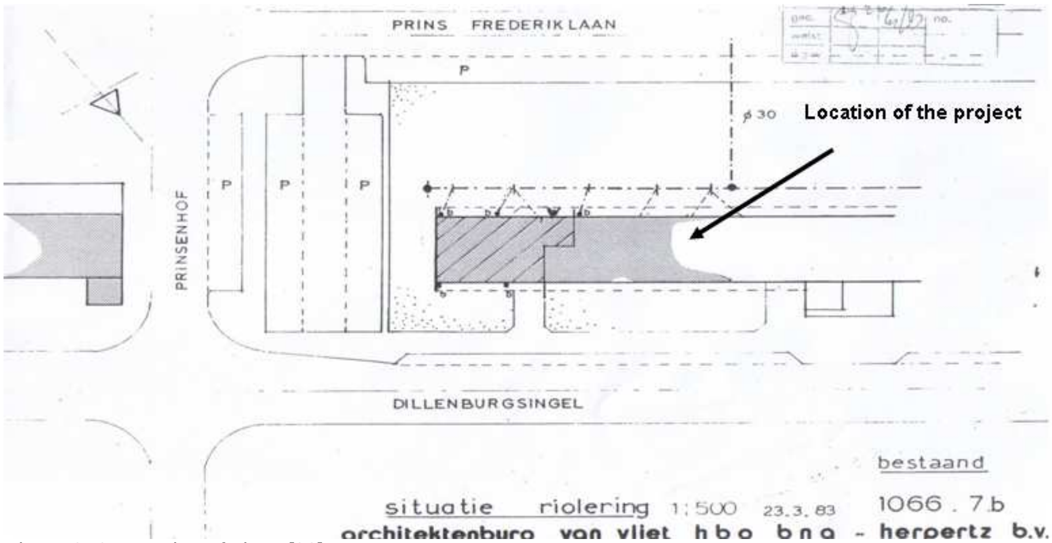
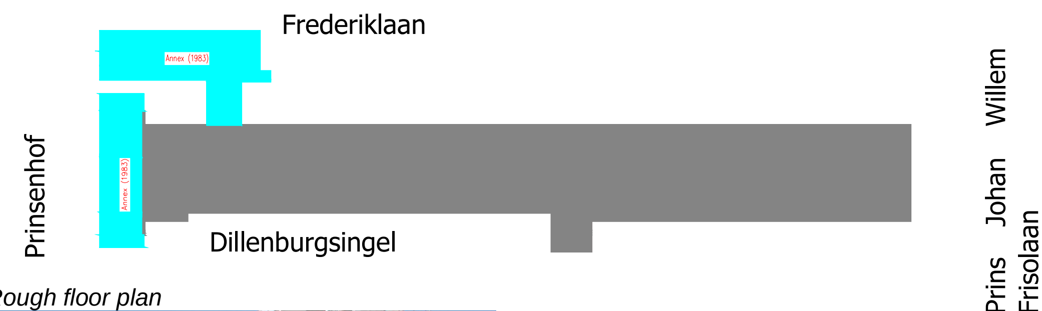


Figure 2- 2: Location of pipes [20]

The old building was constructed in 1968 according to the building archives from the municipality of Leischendam .In 1983 two low rise annexes were added at one of its ends as they are shown in blue in figure 2-3. The building is one of the similar apartment buildings constructed in that area.



(a) Rough floor plan

(b) Rear façade at Frederiklaan after demolishing southern part

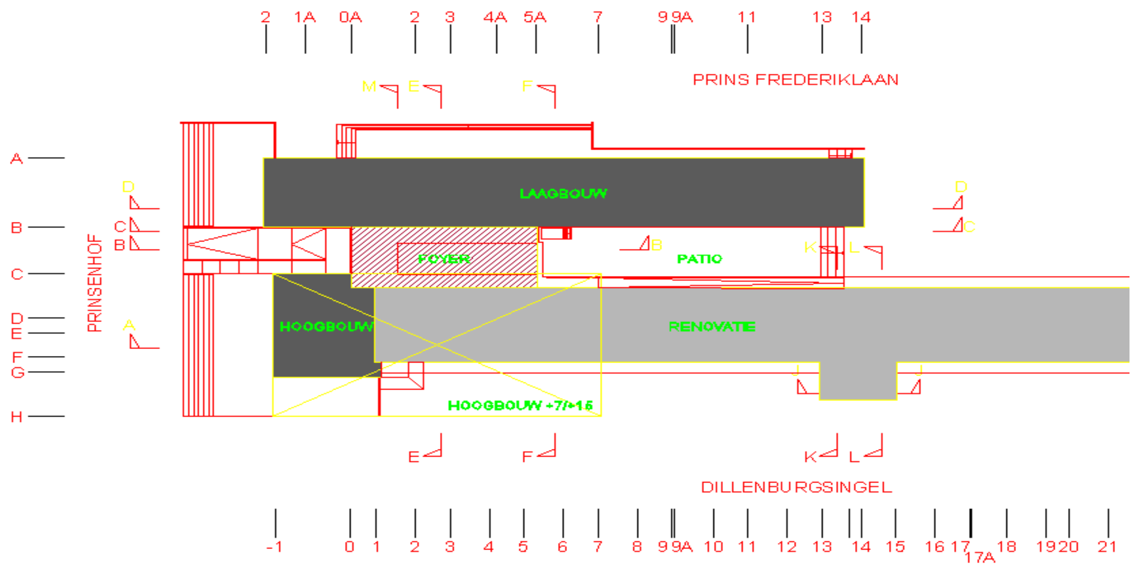
(c) Part of front façade at Dillenburgsingel

Figure 2- 3: Existing situation of the building

The old building is 6-storey ; at this moment the building is undergoing a complete renewal. On the south side nine more apartment floors will be added.

The old building consists of main load bearing structures stabilized by elevator shafts. The additional 9-stories extend beyond the front facade of the by creating a cantilever of nearly 6 meters.

The current building can be divided into three parts according to the figure2-4; the north-west side(see figure2-1), from axis 7 to axis 21. The second part is the south-east; from axis 1 to axis 7. The third part in extreme south-east from axis -1 to axis 1.



(a) General floor plan

(b) Impression of final structure

Figure 2- 4: New and Existing situation of the building [25]

The northern part
This part consists of the existing 6-storey apartment building. It's getting renovated for both internally and externally. This part is not a part of this study.

The southern part

This part consists of 6-storey building on which additional 9-floors will be added; in transverse direction it's limited by axis C to H. This part will be the main focus of this study when conducting a research into the possibility of using light weight material as solution method to provide more stories to the existing building. This project will be used as case study to make an optimal design complying with structural and functional requirements of residential building.

The extreme south part.

This part will also be at the same level as the southern part;15-storey. It starts at ground level and houses the stair case and elevator shafts. It will not be taken into account during this study.

2.2. Soil condition and ground water

Various tests were conducted in order to find the soil properties. Six testing points were set up on site where the building is standing. The cone penetration test (SPT) was done to determine the soil bearing capacity and to know the depth at which the piles will be driven. Figure figure2-5 shows the tested points; D1 to D6.

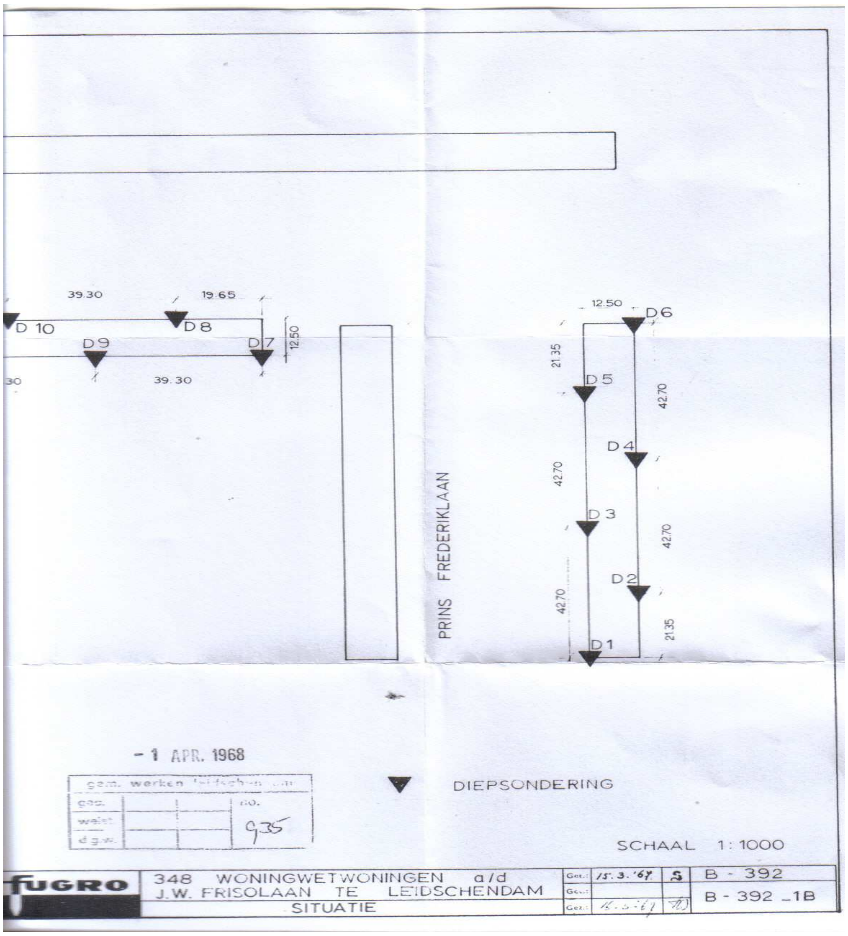
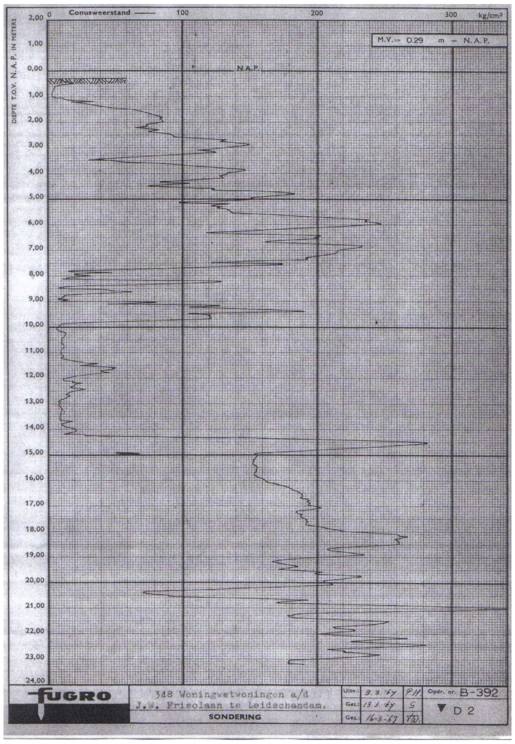


Figure 2-5 Soil testing location [20]

The cone penetration test was conducted mostly up to 22 meters deep, and cone resistance was plotted against the depth. Figure 2-6 shows the plot generated at point D2. For all points, the sufficient capacity was reached at 15m; that is why it was decided to drive all piles up to 15m-figure2-7



Opdr. No. B-392 blz. 3.

TABEL PAALPUNTNIVEAUS

Blok	Sondering No.	Paalpuntniveau in m - N.A.P.
Frederiklaan	D1	15,0
	D2	15,0
	D3	15,0
	D4	15,0
	D5	15,0
	D6	15,0
J.W. Frisolaan	D7	15,0
	D8	15,0
	D9	15,0
	D10	15,0
	D11	16,0
	D12	16,5
	D13	15,0
	D14	16,0
	D15	14,5
	D16	15,5
	D17	15,0
	D18	15,0
	D19	15,0
	D20	15,0
	D21	15,0
	D22	15,0
	D23	15,0
	D24	15,0
	D25	15,5
	D26	16,5
	D27	17,5
	D28	14,5

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Figure 2- 6: Cone Resistance at point D2 [20]

Figure 2- 7 : Location of Pile point [20]

Furthermore, the information about the ground water and soil type is provided. According to the soil profile of the site at different points the ground water table is at 1.5m deep. In 1m deep, there are two layers; fine sand and peat. Going deeper the type of soil is fine sand-figure 2-8.

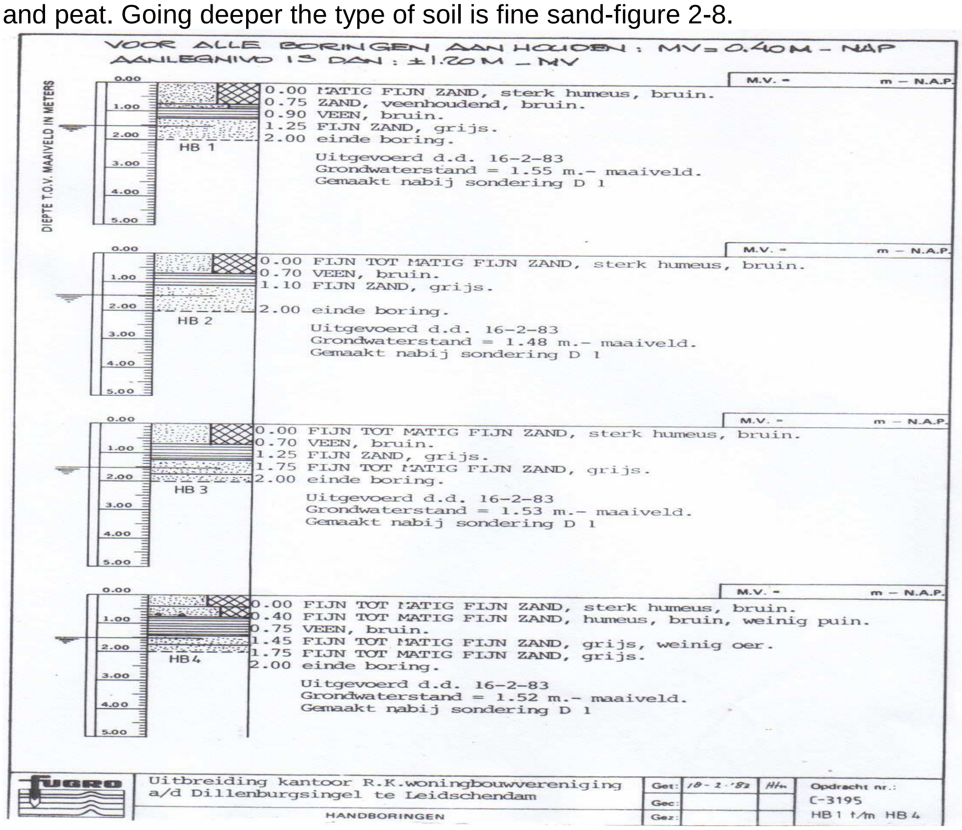


Figure 2- 8 Ground water table and soil type [20]

2.3. Structural members of the existing building

2.3.1. Walls

The existing building is six storeys high. Its structural system is made by concrete frame with in-fill of concrete blocks. These frames are located in all transverse axes (see figure 2-9 and 2-10) to carry the gravity loads as well as lateral loads.

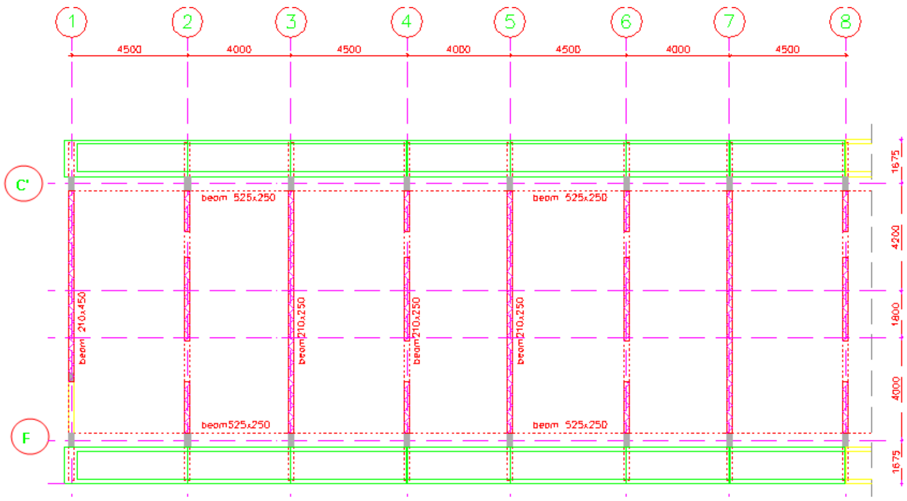
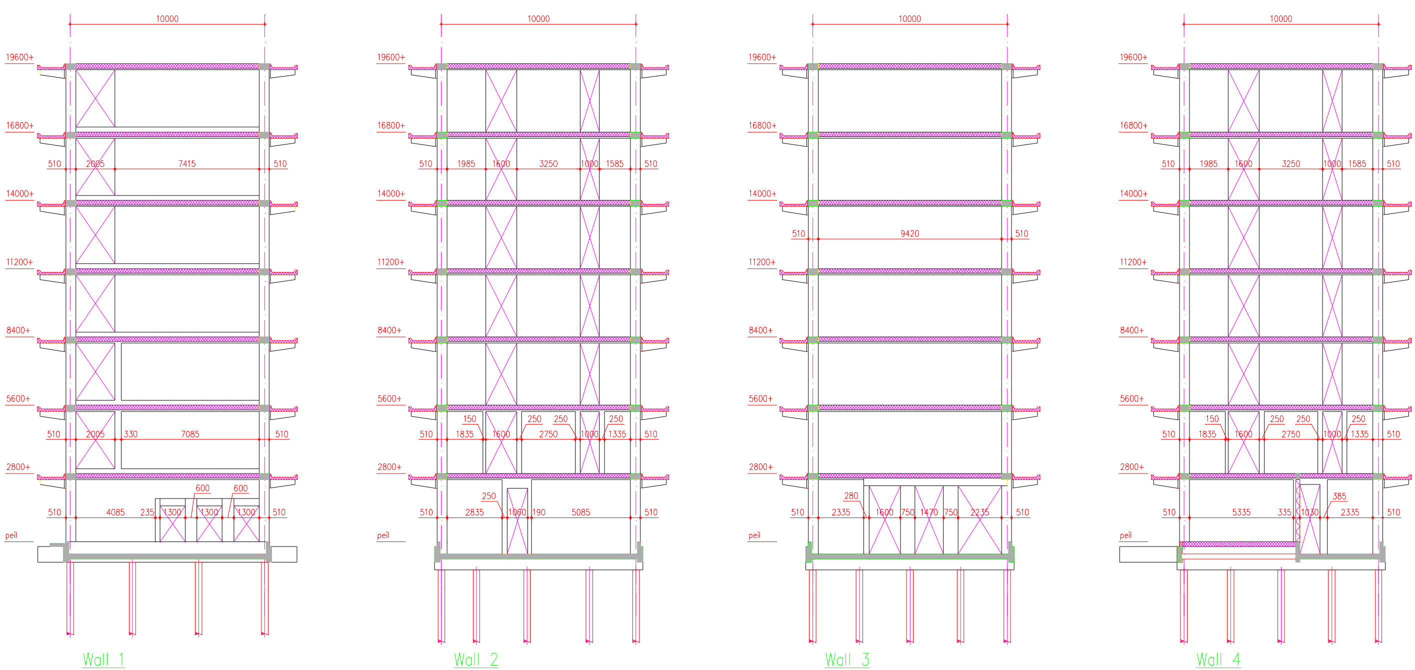


Figure 2- 1: Floor plan of the existing building (1st to 6th floor) [25]



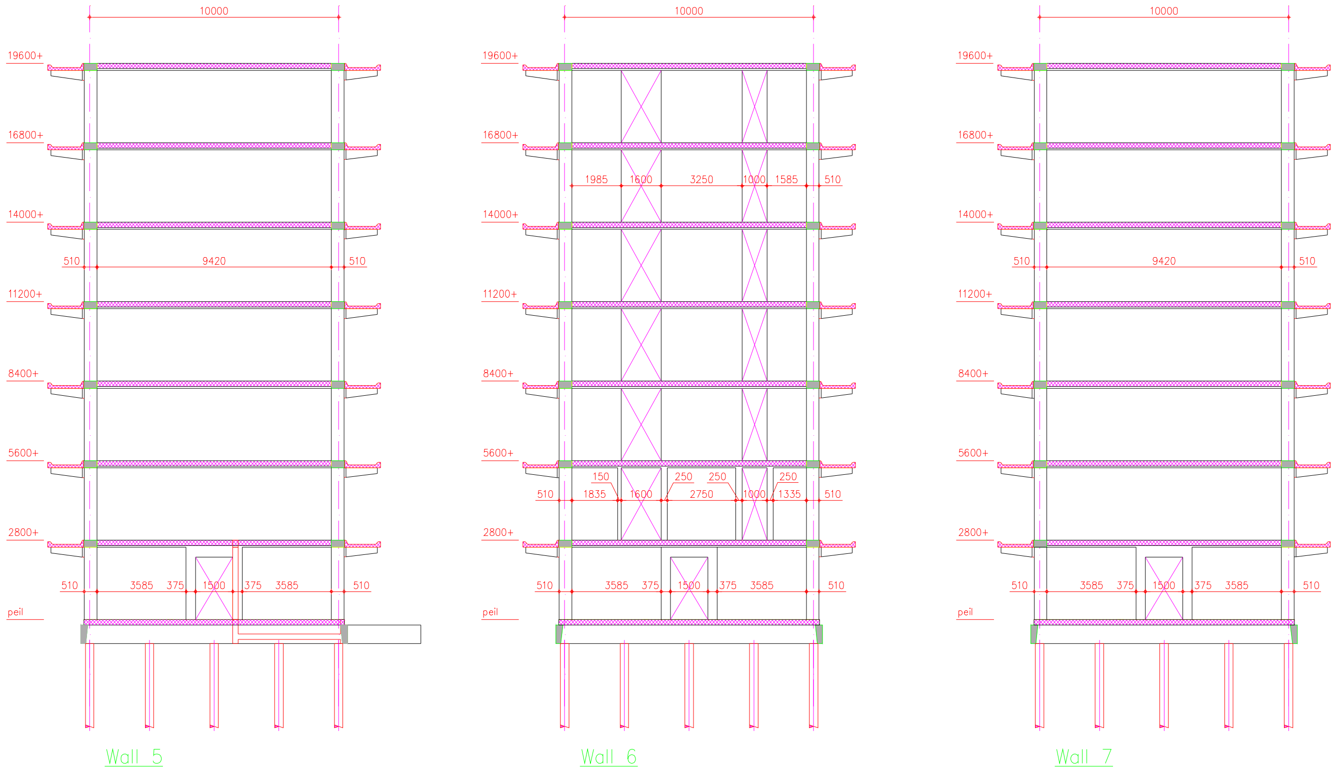


Figure 2- 2: Cross- sections of the existing building in axes 1to 7 [25]

The masonry in-fill are made of concrete block-work, so called “Muwi”-system. These are large concrete bricks with two holes. The holes in the bricks are filled with concrete and reinforcement bars (see figure2-11)

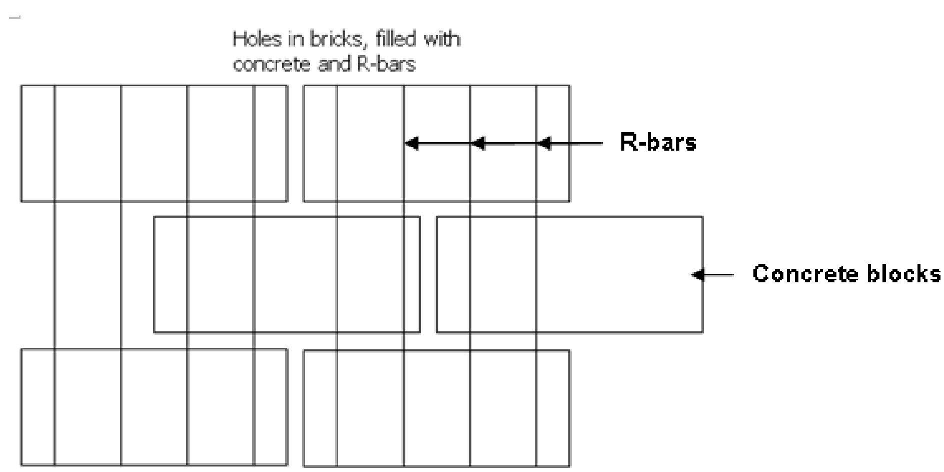


Figure 2- 3: In-fill Block work in frame I of existing building

2.3.2. Columns and beams

In existing building, columns are located where the transverse axes intersect with the edge longitudinal axis C' and F as it appears on figure 2-10.They have a height of 510mm and a width of 210mm .

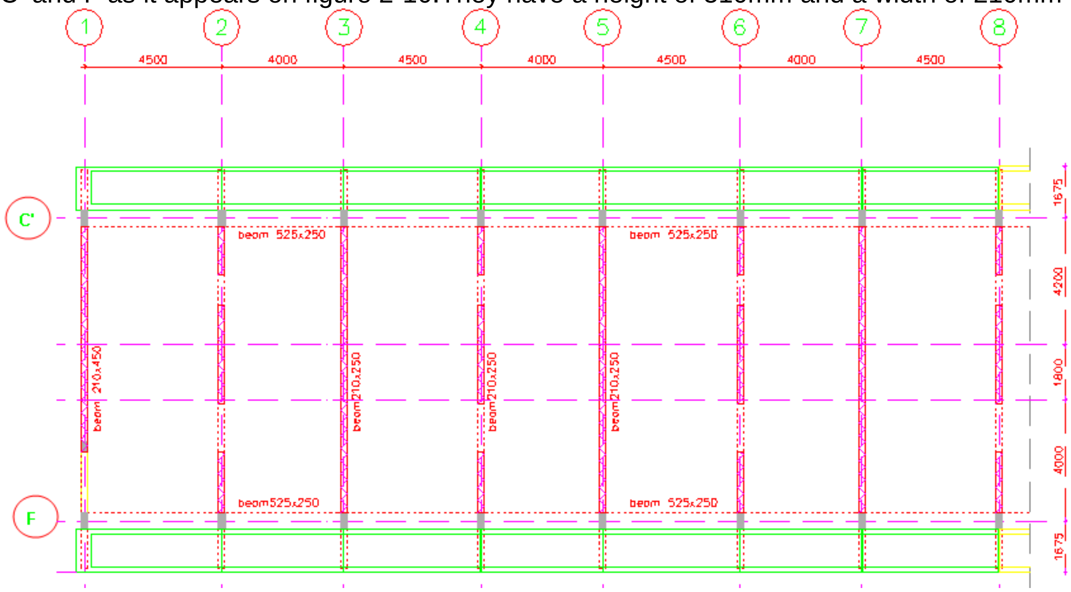


Figure 2- 4: existing building, 2nd to roof floor plan of existing building. [20]

There are two continuous facade beam for each floor on axis C' and F. Their width is 525mm and heights of 250mm.They support the façade and a part of load coming from floor system.

The floor beams were reinforced differently since they do not carry the same loads. The detailed calculations and drawings from archives [20] show that these members were designed according to the dead and live load carried by each beam.
Transverse beam have a width of 210mm and a height of 250mm, they run transversally and join two columns located on both sides of the building.

The facade beams are located at axis C' and F, they carry the load from the façade as well as a part of load from the floor system. Their height is 250mm whereas their width is 525mm. They are reinforced beams and their reinforcement was designed according to the load applied on it, according to the detailed calculations and drawings found from the building archives [20].

2.3.3. Floor system

The ground floor of this building is "Beam-block floor'' type. It's a composite floor made with the following components:

- ✓ Precast joists (main bearing component) placed parallel to each other; they are made with prestressed concrete. The figure 2-13 shows the cross section of the beam-block floor where the joists are inverted T-beams. These joists are supported by the facade beams.

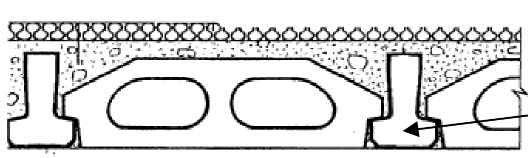


Figure 2- 5: Floor system: beam- block system

- ✓ Prefabricated infill blocks, placed between the joists. The blocks are made of light weight concrete. They are shown between the joists on figure 2-10.
- ✓ In-situ concrete filling combined with an integral concrete topping.

The floor units are spanning longitudinally and they are supported by the floor beams placed directly to the bearing walls. The floor beams have the cantilever portions which protrude the facade walls in order to support the balconies.

Other floors are solid floor made in reinforced concrete; they are designed according to the loads applied on it both permanent and live loads. The applied load allowed calculating the moments, the reaction forces, the shear forces as well as reinforcing steel. The concrete used is K225 and the reinforcement is QR_n40.

2.3.4. Sub- structure (foundation) composition

The foundation structure consists of the reinforced foundation beam and the piles underneath the foundation beam.

They are placed in transversal direction under the bearing walls-figure2-14.
Two foundation beams are also located in longitudinal direction at axis C’ and F.

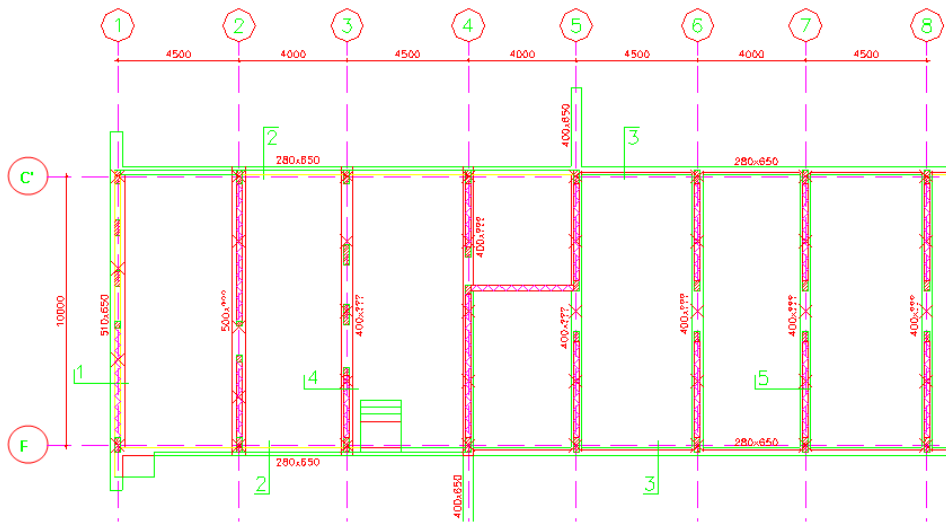


Figure 2- 6: Foundation plan [25]

The total load of the building is transferred to the foundation soil by piles. The length of these piles is 15m, this length depends on the cone penetration test done in that zone where the piles are put. As this study focuses only from axis 1 to axis 7, the total number of piles in this section of the building is 34 as it is shown in figure 2-15.

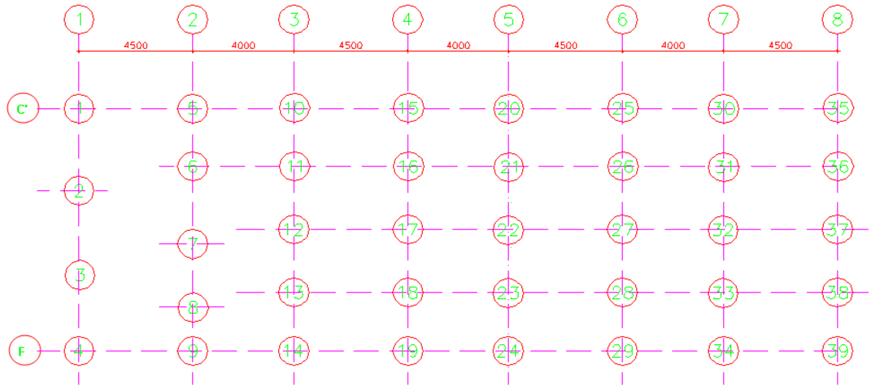


Figure 2- 7: Pile plan [25]

2.4 Capacity of structural elements of the old building.

The structural design and detailing for this building was done by "Construction company *A. van ECK N.V"* in 1967. The results of the detailed design are found in the building archives [20] from the Leidschendam city. The document is titled *"348 Galerijwoningen te Leidschendam"*. According to the calculation documents, the loads which have been used in designing the structural members are represented in tables2-1a, b, and c.

This section will emphasize on capacity of columns, foundation piles and lateral stiffness which can help in stabilizing the new building.

- **Load on floors**

Table2- 1: Loading of the existing building [20]

(a) Loading on floors

	Type of load	Load[kg/m^2]	Total load [kg/m^2]
Roof	Own weight	240	400
	Finishing and isolation	50	
	Gravel	60	
	Snow	50	
6th floor	Own weight	240	550
	Finishing and isolation	60	
	Partition	50	
	Live load	200	
5th floor		550-20	530
4th floor		550-40	510
3rd floor		550-60	490
2nd floor		550-80	470
1st floor		550-100	450
Ground floor		550-120	430
q-total			3830

(b): Loading on balconies and corridors

	Type of load	Load[kg/m^2]	Total load [kg/m^2]
Roof	Own weight	270	510
	Light concrete	180	
	Finishing and isolation	50	
	Gravel	60	
	Snow	50	
6th floor	Own weight	270	470
	Live load	200	
5th floor		470-20	450
4th floor		470-40	430
3rd floor		470-60	410
2nd floor		470-80	390
1st floor		470-100	370
q-total			3030

(c) Loading on stair cases

	Type of load	Load[kg/m^2]	Total load [kg/m^2]
Roof			510
6th floor	Own weight	335	590
	Finishing and isolation	55	
	Live load	200	
5th floor		590-20	570
4th floor		590-40	550
3rd floor		590-60	530
2nd floor		590-80	510
1st floor		590-100	490
Ground floor	Own weight	240	380
	Finishing and isolation	60	
	Live load	80	
q-total			4130

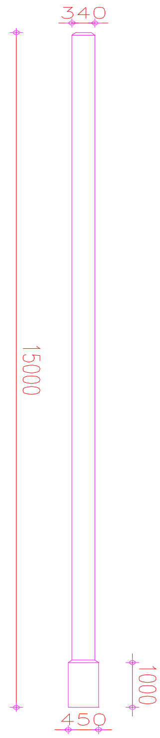


Figure 2- 8: Existing pile

2.4.1 Foundation

The foundation consists of foundation beam and piles. The beams are made of reinforced concrete whose reinforcements are designed based on load coming from the above structure.

The load from the beams is transferred to the soil through the concrete piles-figure
According to [20], after finding out the loading for each component using the loads from table2-1a, b, c; the total loads on piles were calculated. Those loads on piles are summarized in table2-2.and were used in designing the existing foundation. All piles used have a square cross section 340x340 mm, 15m long and a square point 450x450 mm with a maximum load of 70 ton (700KN).

Table2- 2 : Total loading on piles [20]

# Pile	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Load[Kg]	66005	69170	68770	68600	61450	64120	48345	69625	64020	59880	60955	63080	69260	69260	69260	54600	58780
# Pile	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34
Load[Kg]	53355	59880	59880	60955	63080	69260	69260	69260	54600	58780	53355	59880	59880	60955	63080	69260	69260

In Appendix 11, the maximum bearing capacity of the existing pile was recalculated. It has been found that the bearing capacity of old piles is 1736 KN while the maximum load on pile is 700KN.

The piles use only 40% of its capacity. However, this doesn't mean that the old building can bear the new structure because the number of pile are few (only 5 per wall) and the new structure will bring too much load concentrated on one side of the building due to cantilever part (Figure 2-17).

Figure2- 17: New structure with cantilever part (in actual design)[25]

2.4.2. Columns

The supporting columns are located in front and rear façades; they carry the load from the floor, façade beams and consoles carrying the balconies. The reinforcement for these columns is determined according to the normal forces and moment created by the loading system. The reinforcement area increases downward as function of the normal forces. The detailed calculations and drawings are also found in the building archives stated before. [20]

The capacity of frame will be basically depend on columns strength; for this reason only columns are checked since additional load to the structure can be conducted through the existing column.

- **Applied axial forces from gravity load**

The gravity loads in the columns come from the tributary areas; these loads were collected from design archives [20] and are shown in table 2-3. A large amount of vertical load is taken by the masonry in-fill.
Axial forces in the columns and beams resulting from the horizontal loading should be estimated by simple static analysis of the analogous braced frame considering each infill as a diagonal strut. These two kind of axial forces are added to form the design force for each column.

Table2- 3 Load, moments and reinforcement of existing columns [20]

N_{max} in Kg

M_{max} in Kgm

Floor	Member	Vertical load in Kg, moment in Kgm and reinforcement per column																				
		Wall 1			Wall 2			Wall 3			Wall 4			Wall 5			Wall 6			Wall 7		
		N_{max}	M_{max}	R-bar	N_{max}	M_{max}	R-bar	N_{max}	M_{max}	R-bar	N_{max}	M_{max}	R-bar	N_{max}	M_{max}	R-bar	N_{max}	M_{max}	R-bar	N_{max}	M_{max}	R-bar
Under 6th floor	Col.C'	17060	1347	6ϕ10	19055	1048	6ϕ10	25580	1017	6ϕ10	25580	1017	6ϕ10	24030	1049	6ϕ10	25580	1017	6ϕ10	16510	1147	6ϕ10
	Col. F	17060	1347	6ϕ10	19055	1048	6ϕ10	27295	1017	6ϕ10	25580	1017	6ϕ10	24030	1049	6ϕ10	25580	1017	6ϕ10	18220	1147	6ϕ10
Under 5th floor	Col.C'	17060	1347	6ϕ10	19055	1048	6ϕ10	25580	1017	6ϕ10	25580	1017	6ϕ10	24030	1049	6ϕ10	25580	1017	6ϕ10	16510	1147	6ϕ10
	Col. F	17060	1347	6ϕ10	19055	1048	6ϕ10	27295	1017	6ϕ10	25580	1017	6ϕ10	24030	1049	6ϕ10	25580	1017	6ϕ10	18220	1147	6ϕ10
Under 4th floor	Col.C'	22600	1136	6ϕ10	25580	1017	6ϕ10	25580	1017	6ϕ10	25580	1017	6ϕ10	24030	1049	6ϕ10	25580	1017	6ϕ10	22150	1475	6ϕ12
	Col. F	27215	1556	6ϕ12	30180	1449	6ϕ14	27295	1017	6ϕ10	25580	1017	6ϕ10	24030	1049	6ϕ10	25580	1017	6ϕ10	24435	1475	6ϕ12
Under 3rd floor	Col.C'	28320	1346	6ϕ12	32090	1307	6ϕ12	32090	1307	6ϕ12	32090	1307	6ϕ12	30440	1348	6ϕ12	32090	1307	6ϕ12	27810	1806	6ϕ14
	Col. F	34460	1766	6ϕ16	38230	1629	6ϕ18	34375	1307	6ϕ12	32090	1307	6ϕ12	30440	1348	6ϕ12	32090	1307	6ϕ12	25525	1806	6ϕ14
Under 2nd floor	Col.C'	34050	1555	6ϕ14	38645	1598	6ϕ16	38645	1598	6ϕ16	38645	1598	6ϕ16	36650	1647	6ϕ16	38645	1598	6ϕ16	33465	2135	6ϕ18
	Col. F	41725	1975	6ϕ20	46320	1960	6ϕ20	41500	1598	6ϕ18	38645	1598	6ϕ16	36650	1647	6ϕ16	38645	1598	6ϕ16	36320	2135	6ϕ18
Load Under 1st floor	**Col.C'**	**39790**	**1765**	**6ϕ18**	**45170**	**1925**	**6ϕ20**	**45170**	**1925**	**6ϕ20**	**45170**	**1925**	**6ϕ20**	**42820**	**2033**	**6ϕ20**	**45170**	**1925**	**6ϕ20**	**39020**	**2465**	**6ϕ22**
	Col. F	**48990**	**2185**	**6ϕ22**	**54370**	**2250**	**6ϕ24**	**48600**	**1925**	**6ϕ22**	**45170**	**1925**	**6ϕ20**	**42820**	**2033**	**6ϕ20**	**45170**	**1925**	**6ϕ20**	**42450**	**2465**	**6ϕ22**
Total Moment	Col.C'		8496			7943			7881			7881			8175			7881			10175	
	Col. F		10176			9384			7881			7881			8175			7881			10175	
	Total moment		**18672**			**17327**			**15762**			**15762**			**16350**			**15762**			**20350**	

- **Applied force due to horizontal load**

As the frame with in-fill masonry behaves as braced frame, the axial load in columns due to horizontal load can be calculated as follow:

$F_c = \pm \frac{M_H}{\frac{1}{2}b}$; $M_H = \frac{1}{2} q_w . H^2$; $q_w = q_{w,rep} . L \Rightarrow 1.28,6 = 28,6 KN/m$; as there are 7 frames, one takes $28,6/7 = 4KN/m$

$$M_H = \frac{1}{2} q_w . H^2 \Rightarrow \frac{1}{2} 4.19,6^2 = 768KNm$$, therefore $$F_c = \pm \frac{768}{\frac{1}{2}10} = \pm 154KN$$

Where F_c : axial force as a result of horizontal load

M_H: Moment due to horizontal load

b and L: the width and the length of building

H : height of building

$q_{w,rep}$: Representative wind pressure

q_w : wind load

Table2- 4" Axial load due to lateral moment and moment from vertical load

Lateral axis	Total moment due to vertical load [KNm]	Moment due to horizontal load [KNm]	Total moment [KNm]	Width [m]	Equivalent axial load [KN]
1	187	768	955	10	191
2	173	768	941	10	188
3	158	768	926	10	185
4	158	768	926	10	185
5	164	768	932	10	186
6	158	768	926	10	185
7	204	768	972	10	194

- **Axial load capacity of columns**

In a reinforced concrete column, both longitudinal steel and concrete assist in carrying the load. The links (stirrups) prevent the longitudinal bars from buckling.
According to Euro Code 2 [23], the ultimate load capacity of column is given by the following expression

$$N = \alpha f_{cd} A_c + A_{st} f_{yd}$$

$\alpha = 0{,}85$, $f_{cd} = \dfrac{f_{ck}}{1{,}5} \Rightarrow \dfrac{20}{1{,}5} = 13{,}3 N/mm^2$ and $f_{yd} = \dfrac{f_{yk}}{1{,}15} \Rightarrow \dfrac{400}{1{,}15} = 348 N/mm^2$

Where N is the ultimate load capacity
F_{cu}: ultimate strength of concrete (K300 or 20N/mm^2)
A_c: area of concrete (A tot – Ast)
A_{st}: Total area of longitudinal reinforcement
f_y: Yield strength of steel(Q40 or 400N/mm^2)

The capacity for all columns on ground level can be calculated and compared to the applied load. The applied load is found in table 2-3 under first floor. The calculations are summarised in table 2-4:

Table2- 5: Calculation of the capacity of the existing columns

Longitudinal Axis	Transversal Axis	f_{cd} N/mm^2	A_c mm^2	A_{sc} mm^2	f_{yd} N/mm^2	Ultimate capacity[KN]	Applied load load [KN]	Applied load Due to M_H KN	Applied load Total applied load[KN]	% of used capacity
C'	1	13,3	107100	1526	348	1741,8	397,9	191,0	588,9	34
	2	13,3	107100	1884	348	1866,4	451,7	188,0	639,7	34
	3	13,3	107100	1884	348	1866,4	451,7	185,0	636,7	34
	4	13,3	107100	1884	348	1866,4	451,7	185,0	636,7	34
	5	13,3	107100	1884	348	1866,4	428,2	186,0	614,2	33
	6	13,3	107100	1884	348	1866,4	451,7	185,0	636,7	34
	7	13,3	107100	2280	348	2004,2	390,2	194,0	584,2	29
F	1	13,3	107100	2280	348	2004,2	489,9	191,0	680,9	34
	2	13,3	107100	2713	348	2154,9	543,7	188,0	731,7	34
	3	13,3	107100	2280	348	2004,2	486,0	185,0	671,0	33
	4	13,3	107100	1884	348	1866,4	451,7	185,0	636,7	34
	5	13,3	107100	1884	348	1866,4	428,2	186,0	614,2	33
	6	13,3	107100	1884	348	1866,4	451,7	185,0	636,7	34
	7	13,3	107100	2280	348	2004,2	424,5	194,0	618,5	31

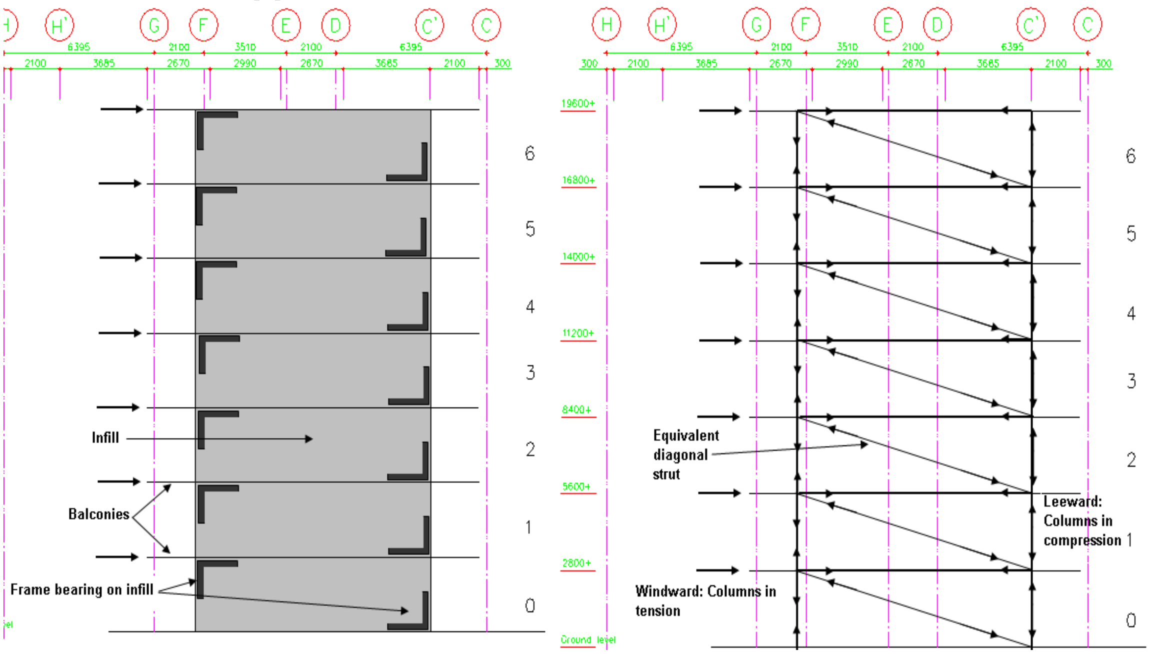
According to the results, most of the columns use less than 40% of their capacity. The capacity left can take moderate loads but not as much as 9 floors.

2.4.3 Stability of the existing building

The existing building consists of the reinforced concrete frames with in-fills of concrete block-work. In addition to functioning as partitions, exterior walls, the in-fills may also serve structurally to brace the frame against horizontal loading. The frame is designed for gravity load only and, the in-fills are presumed to contribute sufficiently to the lateral strength of the structure for it to withstand the horizontal loading.

2.4.3.1 Behaviour of in-filled frames

The use of a masonry in-fill to brace a frame combines some of the desirable structural characteristics of each while overcoming some of their deficiencies. The high in-plane rigidity of the masonry wall significantly stiffens the otherwise relatively flexible frame, while the ductile frame contains the brittle masonry, after cracking, up to load and displacements much larger than it could achieve without the frame. The result is, therefore, a relatively stiff and tough bracing system. The wall braces the frame partly by its in-plane shear resistance and partly by its behaviour as a diagonal bracing strut in the frame. Figure 2-18 illustrate these modes of behaviour. [6]



(a) Interactive behaviour of frame and infill *(b). Analogous braced frame*

Figure 2- 18: Frame and infill behaviour [6]

When the frame is subjected to horizontal loading, it deforms with double curvature bending of the columns and girders. The translation of the upper part of the column in each story and the shortening of the leading diagonal of the frame causes the column to lean against the wall as well as to compress the wall against its diagonal. It is roughly analogous to a diagonally braced frame (Fig.2-18 b).
Three potential modes of failure of the wall arise as a result of its interaction with the frame, and these are illustrated in Fig.2-19. The first is the shear failure stepping down through the joints of the masonry, and precipitated by the horizontal shear stresses in the bed joints. The second is the a diagonal cracking of the wall through the masonry along a line, or lines, parallel to the leading diagonal, and caused by tensile stresses perpendicular to the leading diagonal.
In the third mode of failure, a corner of the infill at one of the end of the diagonal strut may be crushed against the frame due to the high compressive stresses in the corner.

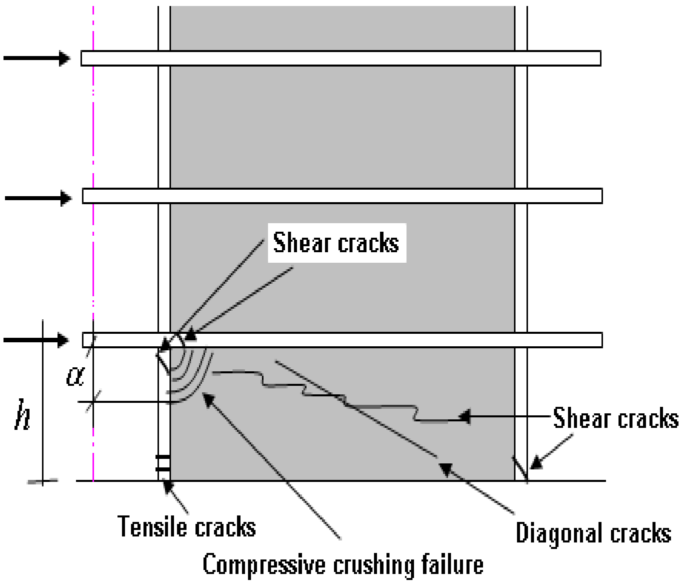


Figure 2-19 : Mode of infill and frame failure [6]

2.4.3.2 Checking the capacity of the in-fill

Two modes of infill failure may cause collapse of the structure. The first is shear failure stepping down diagonally through the bed joint of the masonry and the second is by spalling and crushing of the masonry in the corners of the infill. The lesser of the two strengths should be taken as the critical value

Shear failure: The shear failure of the structure based on the shear failure of the infill should be estimated from:

$$Q_s = \frac{f_{bs} L.t}{1,43 - \mu\left(0,8h/L - 0,2\right)}$$

In which f_{bs} :allowable values of the bond shear stress

μ : Coefficient of internal friction.

L : Length of in-fill

t : Thickness of in-fill

h : Height of each floor

Compressive failure: If the infill is bounded by a reinforced concrete frame the shear strength relating to a compressive failure of the infill should be estimated from $Q_c = 2 f_m \cos^2\theta \sqrt[4]{Iht^3}$

Where: f_m is allowable compressive stress in in-fill

θ is inclination of equivalent diagonal

From Appendix 10, $Q_s = 221KN$ and $Q_c = 2435KN$.

The infill is just adequate to carry the external shear on the basis of the shear failure criterion (strength =221KN compared with the shear due to applied load of 71,4KN) and more than adequate on the basis of compressive failure criterion (strength = 2435KN).

2.4.3.3 Checking the deflection

A conservative estimate of the horizontal deflection of an in-filled frame would be given by the calculated deflection of the equivalent pin jointed braced frame as summing each infill to be replaced by a diagonal strut with a cross sectional area equal to the product of one-tenth of its diagonal length and its thickness [6] . There are two kinds of deflection; namely flexural deflection as well as shear deflection.

(i). Flexural component

This component depends on bending stiffness of frame. At each floor level the deflection is calculated and finally added up. The procedure is shown in Appendix 10 and any component prior to calculation are filled in a table for calculation.

After calculation the deflection at each floor, the final deflection due to flexure is the sum of floor deflection:

$$\delta_{f;tot} = \sum \delta_{f;i}$$

It has been obtained that the final horizontal drift due to flexure is $\delta_{f;tot} = 0,16mm$

(ii). Shear component

The shear due to horizontal force participates in increasing the deflection. Its magnitude depends up on physical behaviour of in-fill and a little participation of frame members. The bracing member is obtained by making an equivalent diagonal strut and its equivalent cross section is given by this formula: $A_d = \frac{1}{10} L_d.t$

Where L_d : length of equivalent diagonal

A_d Cross section of equivalent diagonal

t Thickness of in-fill

As the infill with frame behave as braced bent, the deflection for each transverse frame is calculated as follows

$$\delta_{s;i} = \frac{Q_i}{E}\left(\frac{L_d^{\ 3}}{L_g^{\ 2} A_d} + \frac{L_g}{A_g}\right) \quad [6]$$

Where: $\delta_{s;i}$: shear deflection at each floor level

L_g : length of beam above in-fill

Q_i : shear force due to lateral load at each floor level

E : Modulus of elasticity on in-fill

The total deflection due to shear is the summation of deflection at every floor level

$\delta_{s;tot} = \sum \delta_{s;i}$, From Appendix 10, it has been obtained that $\delta_{s;tot} = 4,6mm$

The total drift is the summation of both flexural and shear drifts:

$$\delta_{total} = \delta_{f;tot} + \delta_{s;tot} \Rightarrow 0,16 + 4,6 = 4,8mm$$

Normally as the building height is 19600mm, the allowable horizontal deflection should be

$\delta_{\max} = \frac{H}{500} = \frac{19600}{500} = 39mm >> 4,8mm$; the structure is more stable.

2.5 Conclusion on existing building

The old building is made of concrete frame braced by block-work in-fill. This in-fill contributes in carrying floor loads as well as lateral loads. This contribution makes the frame columns to carry fewer loads compared to their load capacity. The new structure is 9-storey high and it cannot rest on existing building, in addition to that the new structure is placed eccentrically(cantilever) with respect to the existing building, this effect bring too much loads on one side of the existing building whose frame cannot tolerate such load.
From the following figure (Figure2-20) some conclusions can be drawn;

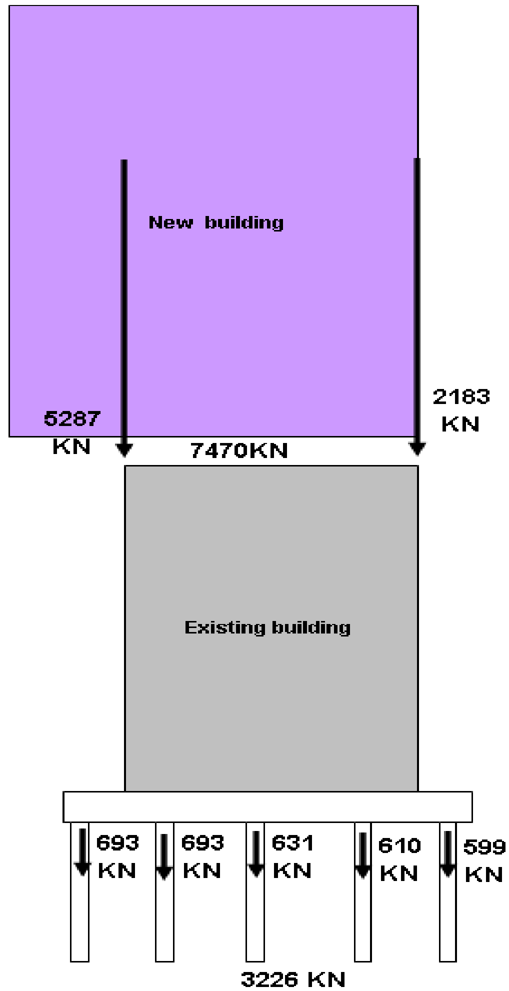


Figure 2-110 : Service load in new and existing building for axis 5

By taking the axis 5, the total load of the existing building is 3226KN distributed over 5 piles. The existing foundation pile capacity was estimated to 1736KN, as the maximum load applied to each pile is nearly 700KN (70 tons); 60% of the capacity is not used. However, for the same axis 5, the load from the new structure with service load is 7470 KN. Therefore, the total load that would go to the same foundation would be 10696 KN, consequently, 5 piles cannot handle this load and hence the new structure should have its own foundation since providing more piles to the existing foundation is practically impossible.

The only thing the existing building can do is to help the new one in restraining the lateral loads since its frames with their in-fills are very stiff in their planes. It can be done by connecting rather than supporting the new structure to the existing one in lateral direction.

3. REFERENCE PROJECTS

In This section, some projects having similar properties to the case study of this thesis will be described in order to adopt structural concept that can be compatible with the case study of this thesis.

3.1 Actual design of Smitwesterrman

3.1.1 General description

In this section, the current design made by the design office of SmitWesterman will be described so that one can see the difference between this design and the final concept that will be obtained at the end of the thesis. The actual construction is a residential facility of 15 floor levels above the ground level with a total height of 47m. It is located in Leidschendam at Dillenburgsingel Street.

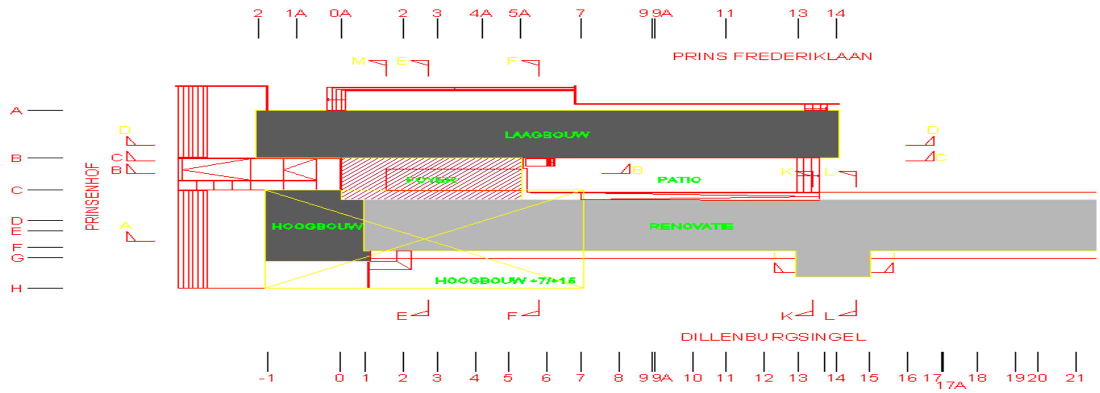
This project exactly the case study of this thesis. The designers of this project wanted to put nine more floors to the existing six floor building, but they skipped the concept by designing another similar structure by starting from the foundation.

They thought that its probably possible to put nine more floors on existing building that is why they forwarded the projected(case) to the university so that anyone interested can make possible designs of adding more story to the existing building.

In this project, structure is mainly made of concrete and the whole building seems to be unique since the existing building was completely demolished. This construction method differs from the objectives of this thesis whereby the additional structures can be made without bringing down the existing building. Various data of this project such as drawing will be consulted in the work of this thesis because the final products should be functionary the same but structurally different.

Previously, there was an old building of 6 floor levels constructed from axes 1 up to 21 (Figure 3-1b) with 10m wide (without considering balconies on both sides of the building), thereafter, it was decided to increase the number of floors up to 15 and increasing the width of the building to 20m from 7^{th} floor and above. The raised part of the building has a length of 37, 1 m in which 25,5 m coinciding with the old building (from axis 1 to 7) - Figure 3-1b and an other part of 8,5 m which is added next to the existing building to accommodate the elevators stair cases and additional rooms. In summary the 15 –storey constructed is 37,1m long and 14m wide (from ground to the 6^{th} floor) and 20 m from 7^{th} to 15^{th} floor with a total height of 47,2m.

a. Pictures of the final structure



b. Overview of the old and new building in plan

Figure 3-1: Overview of the new building as it is integrated with the old building [25]

To construct this building, the old building from axis 1 to 7 was simply demolished so that the construction starts by making a new foundation and constructing walls capable to withstand the load of the new structure. Figure 3-2 shows the old building after demolishing where the multistory building will be constructed.

Figure 3-2: The remaining structure from axes 7 to 21 after demolishing from 1 to 7

3.1.2 Functional concept

The new structures consist of two specific parts; the first 6 floors which are in line with the old building and 9 floors with lateral cantilevers of 6m beyond the existing building. From axis 1 to 7 of the first 6 floors, the building consists of 3 apartments arranged similarly to the existing building and can be accessed through the staircase or levitator (from axis -1 to 0) and balconies on front facade. The part between -1 and 0 accommodates elevators, staircase and one apartment. (Figures 3-3 and 3-4).

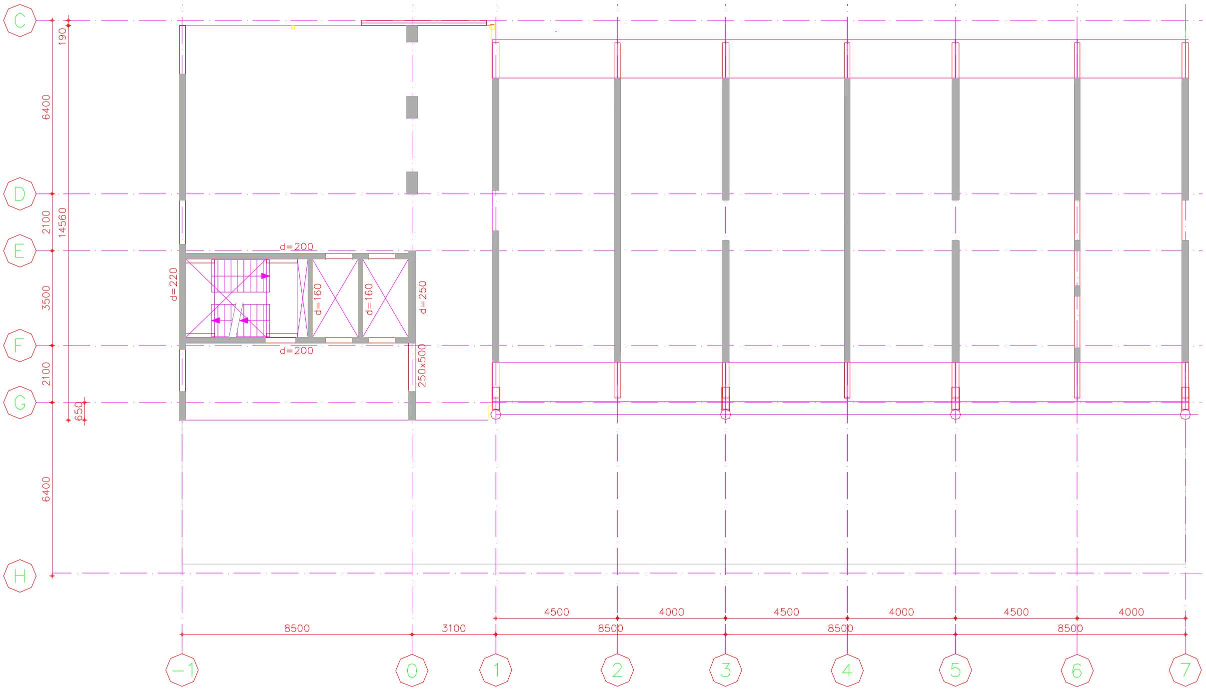
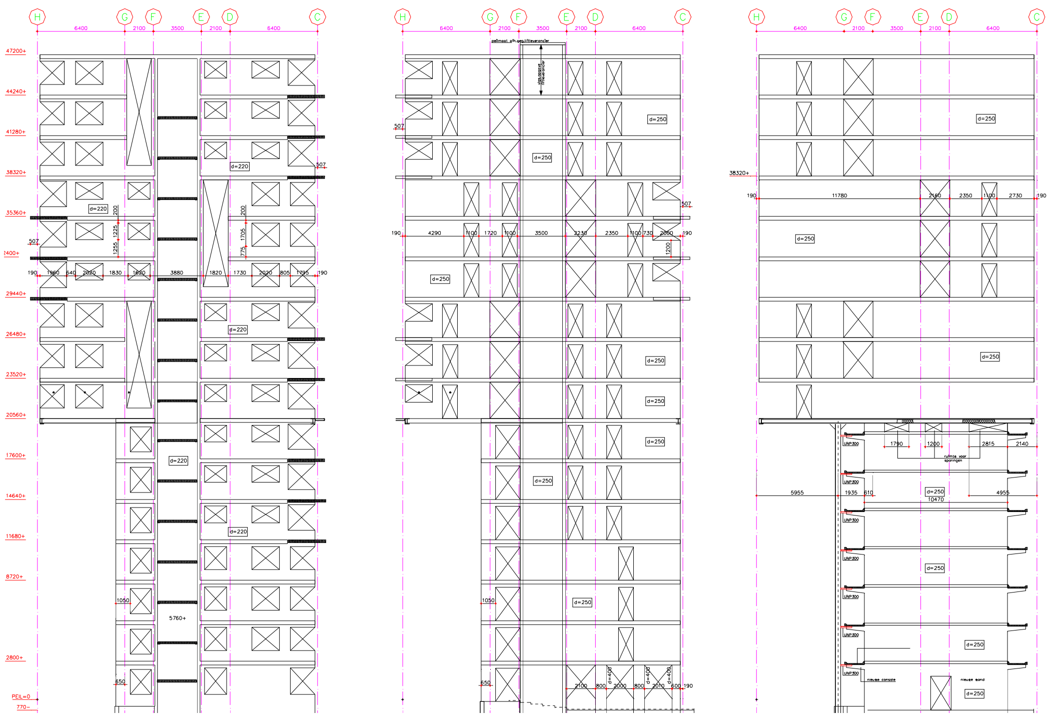
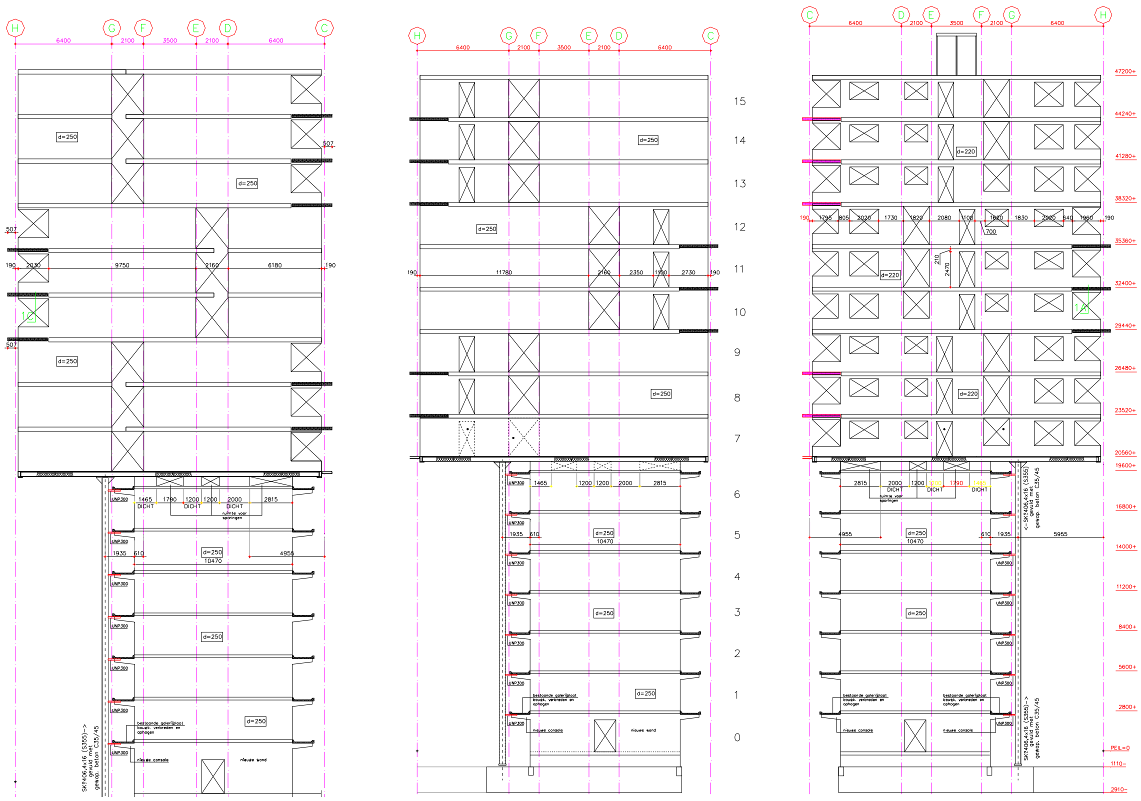


Figure 3-3: Plan of floor levels 1 to 6



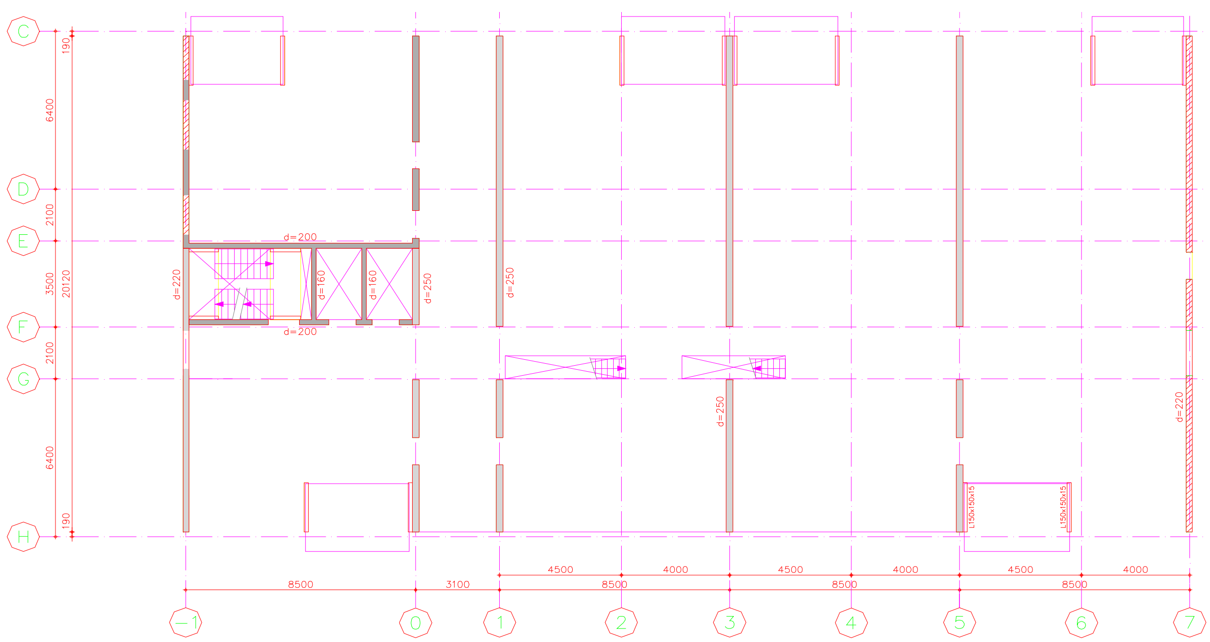
Sections in axes -1, 0, and 1



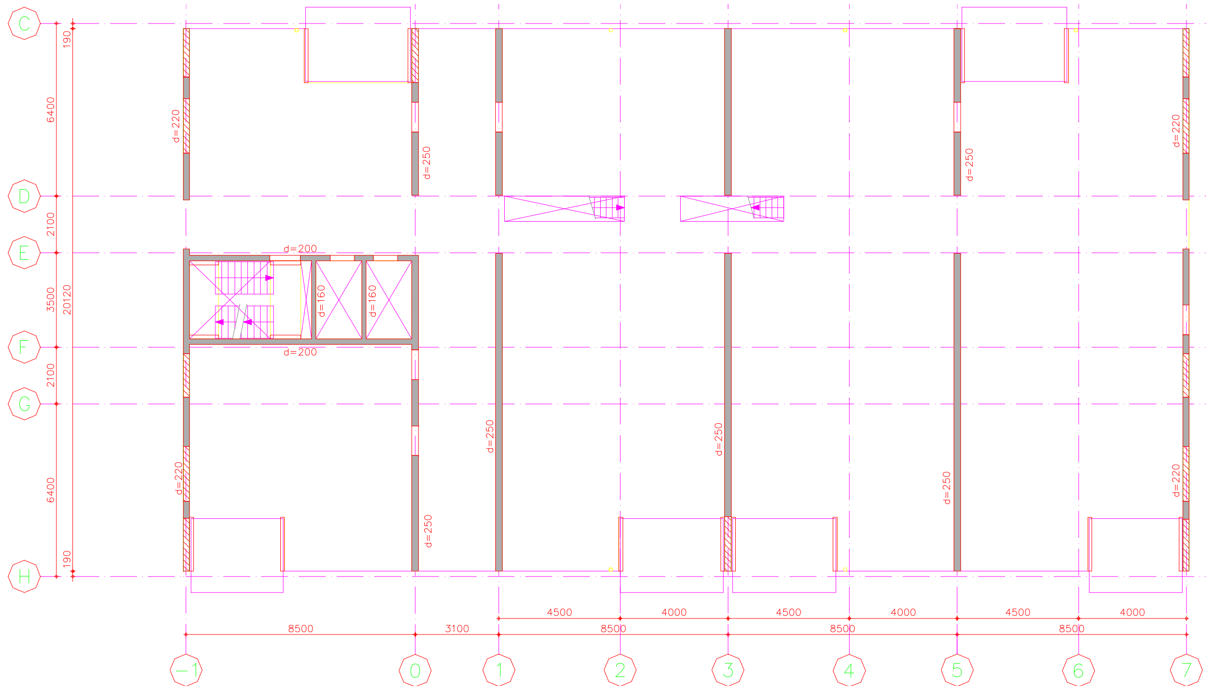
Sections in axes 3, 5, and 7

Figure 3-4: Cross section of the new building [25]

From 7th to 15th floor, the new building has 5 apartments and their access corridors are inside the building. For floor levels 7,8,9,13,14 and 15, this corridor is between F and G (Figure3-5a) , whilst it is between axes D and E for floor levels 10,11 and 12(Figure 3-5b)



(a) Plan of floor level 7,8,9,13,14,15



(b) Plan of floor levels 10, 11, and 12

Figure 3-5: Floor plan of the new building from 7th to 15th floor [25]

3.1.3 Structural concept

Stability

This structure, as it starts from the ground, concrete walls were constructed from the ground level up to 7th floor in axes 1to 7(Figure 3-5a).. From 7th to 15th floor, only walls 1, 3, 5 and 7 continue up to the roof (Figure 3-5b). All these walls together with the concrete core stabilize the building in lateral direction, but in longitudinal direction only the core assume the stability.

Floor system

Solid slabs made of precast planks make floors; these planks are placed in longitudinal direction with a span of 4,5 and 8, 5 m (Figure 3-3 to 3-5). These floor units, due to their relatively big mass, they resist sound and vibration; they also have an important resistance to fire and therefore good for residential buildings.

The precast planks are 100mm thick with lattice girders to receive additional reinforcements. After placing the floor units, reinforcements are fixed and concrete is poured, the final structural depth becomes 280mm. Because the weight of the floor units and concrete topping, temporary supports are obviously necessary.

This building, has big cantilever of 6 m (from 7th floor), due to an increase of the width of the floor plan from 14m for the first 6 floors up to 20m for the other floor levels; this brings complicated erection.

3.1.4 Erection and construction method

In the first six floors, the work starts by constructing concrete walls at every 4 or 4, 5 m (Figure3-5a) up to one floor height. After that, concrete planks (floor units) are put into position and concrete is cast. These activities are repeated up to 6th floor.

From 7th floor, big cantilevers come into place, small off-sets of the walls are arranged on 6th floor in order to support the 9 floor to be put on already constructed 6-floors. The supporting members (wall offsets) and the front steel columns are placed symmetrically in the floor plan so that the cantilever part could be balanced on both sides of the building. A table -like structure is therefore created on 7th floor, then 9-floors in form of box are put on that table. The 9-floor box is made of prefabricated concrete walls of 250mm thick and floor system, which consists of hollow core slabs of 260mm thick. (Figure 3-6)

(a) 7th floor *(b) Floor erection progress* *(c) End of erection*

Figure 3-6: Erection of 7th floor [25]

3.1.5 Useful point from this project that can be used in the thesis

The final product of this thesis should more or less be the same as the product of this project in terms of functionality even though the structural systems may be different.

The system used in this project could be adopted in the thesis where the whole structure starts from the ground; namely the section between axes -1 and 0. The part between axes 0 and 7, an other system must be obtained.

3.2 Kennedy tower in Eindhoven

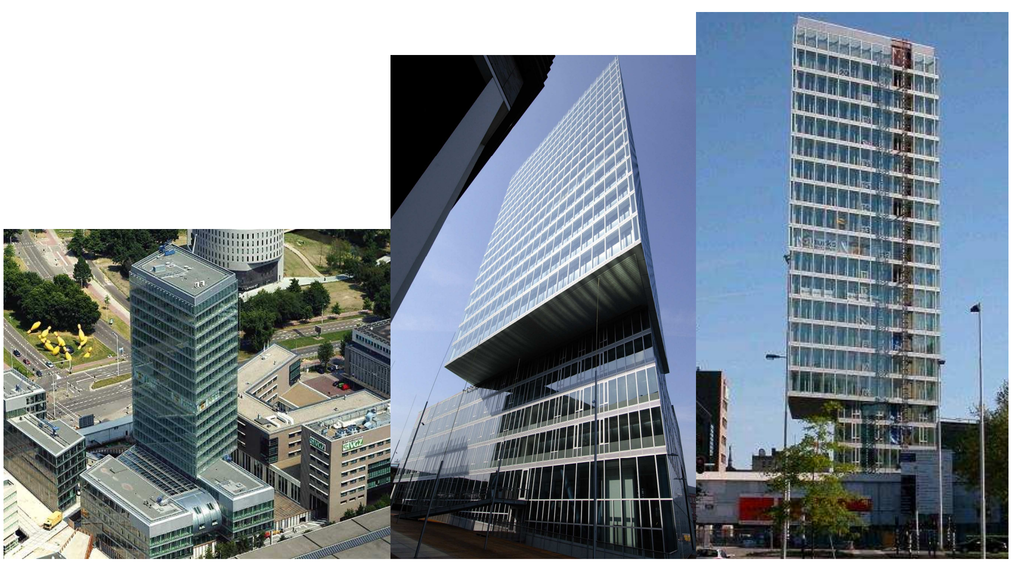


Figure3-8: Picture of Kennedy tower in Eindhoven [32] and [33]

Location:	Eindhoven, Netherlands
Principal	Ontwikkelingscombinatie Eindhoven
Start	2001, January
Commissioning	2002, December
Architect:	Van Aken Architecten
Designer	Adviesburo Tielemans
Contractor	Geveke/CFE, Rotterdam
Steel construction:	Victor Buyck, Eeklo
Completion	1999
Tonnage	800 ton
Height	83m(21 floors)
Total area	30,000 m².
Ground surface	21,6m x 36m

3.2.1 General description and Functional concept

The Kennedy Tower in the heart of Eindhoven. With its central location, highly flexible and customizable floor plans and modern, transparent latest offers over 80 m tall office tower overlooking long-term returns for owners and users. The desire for an unobstructed view and a view of the twenty-story office brought the design team of contractors, architects and engineers at a steel main structure of columns and struts.

The Kennedy Tower is a landmark in the area. With its glass walls, the location of the tower in a "double-skin facade" model makes it a landmark in the area, ensuring the transparency on the one hand reflects the technological, innovative appearance, and also a neutral transitional form between the very different architectural buildings of the Kennedy Business Center. The disposition of the tower relative to the substructure is challenging and takes a huge overhang of the tension. The choice of a steel construction of the tower makes this possible and has the added advantage that the problem can disappear from a sealed concrete core, the building is really transparent and provides great freedom in designing the office floors. [34]

The Kennedy Tower is designed as a technological and programmatic quality and flexible office complex for lease. The total includes 30,000 m² of gross floor area. The atrium is the central building which housed a few auxiliary functions for the various businesses, including catering facilities, a business lobby and lounge area, child care, and corporate fitness facilities.

3.2.2 Structural concept and construction method

For the sake of transparency of the support structure is 82.8 meters high tower columns and braces. The construction was very consciously integrated into the design by selecting a Y-shaped structure, with each diagonal line of the Y still provides the stability of three stories (Figure 3-9).

Figure3-9: Kennedy tower during construction [35]

Circular hollow sections which are flame retardant were used to form frame of this building. Stabilizing the frame using bracing instead of conventional concrete core resulted in structural design that is more transparent and more flexible.

The composite slabs on floors were used because they facilitate various office formats (figure3-10).
These floor are also lighter compared to other kind of floors and can be constructed quickly.

Figure 3-10: Floor Construction

3.2.3 Useful point from this project that can be used in the thesis

In this project, steel structures were used and the structure has a huge cantilever. These aspects of this project are similar to the project of this thesis; therefore, the stability system, the support of cantilevers can be adopted and used in the work of this thesis.

3.3 Hogeschool voor muziek en dans Rotterdam (HMDR)

Figure3-10: Artistic impression of HMDR [36]

Location:	Kruisplein, Rotterdam
Architect:	Jan Hoogstad, Rotterdam
Designer	Aronsohn Raadgevende Ingenieurs, Rotterdam
Contractor	Geveke/CFE, Rotterdam
Steel construction:	Victor Buyck, Eeklo
Start	
Commission	
Height	48 m(9 floors)
Total area	10,000 m².
Ground surface	(Min)13,5m x 75m and (Max) 20 m x 75m

3.3.1 General description

This building is located in Rotterdam next to the theater “De Doelen”. It is a conservatorium for education of musicians and dancers. It is characterized by its huge overhang which is 5-stories high. It has much column free area required for dance halls and classes. All spaces are made sound proof due to musical requirements

3.3.2 Functional concept

This building has many different rooms depending on different users. The building is also complex due to its integration with theatre “De Doelen”. HMDR is constructed in Steel , wheras “De Doelen” is in concrete.
On ground level the building is divided into two sections; the entrance for the HMDR and entrance for “De Doelen”. In both sections, there are central core , elevators and staircases, Large escalators at the back side of the building provides the users of HMDR to a public entrance at the fifth floor level. The HMDR uses elevators for lifting heavy instruments (Figure 3-11)

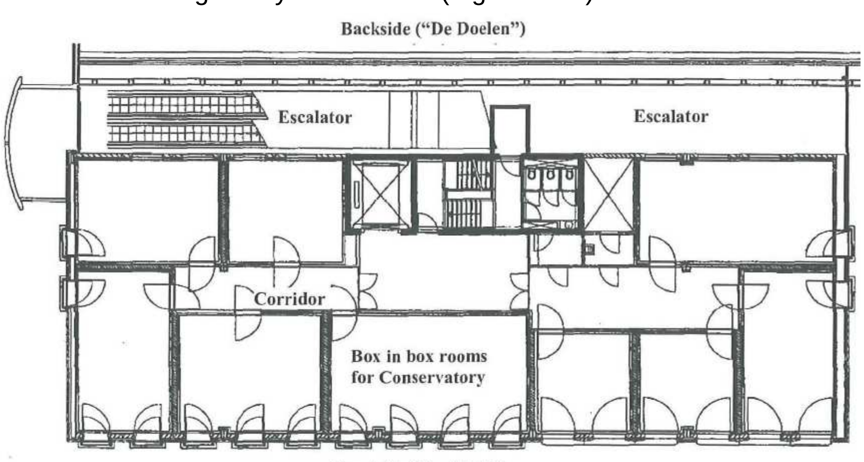


Figure 3-11: Functional plan of the second floor [37]

The Building has these rooms:

- 1 recording room
- 1 music hall
- 1 theatre studio
- 10 dance studios
- 40 music rooms
- 20 study rooms

All these rooms have different specifications and differ in height and area. Some rooms require column- free areas. The type of room influences the functional as well as structural concept of the building.

3.3.3 Structural concept

To stabilize this structure was challenging due to slender cores in lateral direction. The stability was obtained by using table-like structure composed by the 5th floor and 2 shear walls in each building(De Doelen and HMDR)(Figure 3-12)18],[37]

Horizontal loads are restrained by the concrete cores and table structure depending on stiffness of the cores and transverse stiffness of table structure.

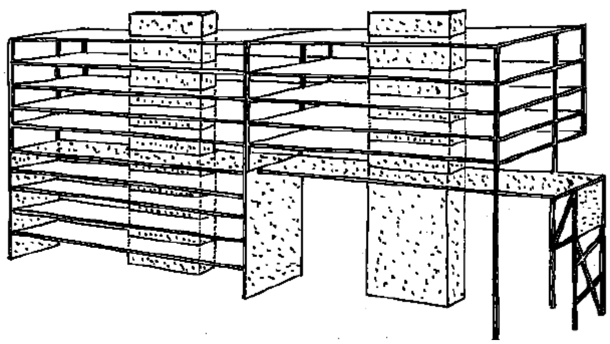


Figure3-12: Table structure in each building [18],[37]

Because table structure is very stiff in lateral direction, core seems to be supported on 5th floor; this results in reduced shear force and bending moments in foundation(Figure 3-13).

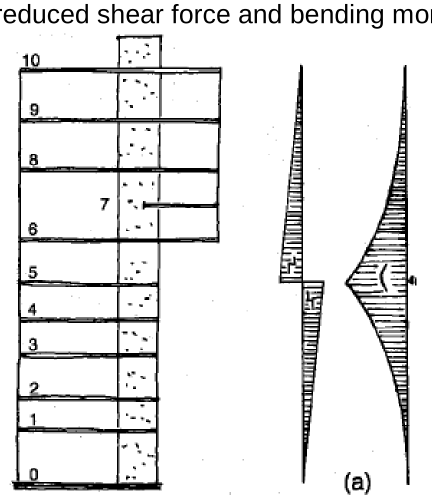


Figure3-13: Moment and shear force diagram when core is connected to 5th floor [37]

Floor system is made by prefabricated planks , because a big mass is needed to avoid noise and vibration during dancing or singing .The width of the first 4 floors is 13m and 20m for above floors. The planks are laid in longitudinal direction.

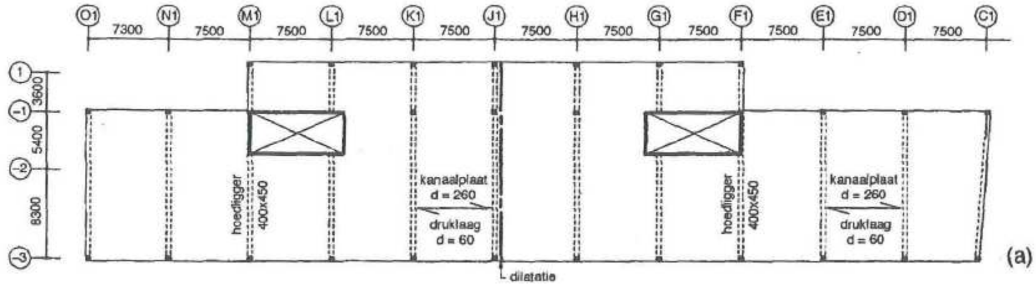


Figure 3-14: Floor plan for upper floors (from 5th and upper) [18]

3.3.4 Erection and construction method

Beyond 5th floor the floor width increase from 13 to 20m . Beyond De Doelen , additional 7 m floor is executed. It was decided to use double continuous steel beams with cantilevers at each floor level[18],[37]. Temporary supports were needed during erection, and latter the structure takes the whole loads by itself.

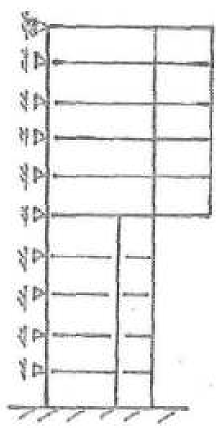


Figure 3-15: Vertical section showing design model [18]

3.3.5 Useful points from this project that can be used in the thesis

The common point of this project and the work of the thesis is the construction of the cantilever above a certain number of floors. Double beam system used in this project can also be used in this thesis since the spans and cantilevers in this project as well as the work in the thesis are respectively 14m and 6 m.
The stability system will differ because of lack of the central cores in this thesis.

3.4 Red Apple Rotterdam


Figure 3-16: Red apple pictures [38]

Location:	Rotterdam, Netherlands
Principal	Woning Stichting Rotterdam
Start	
Commissioning	2009
Architect:	KCAP Architects&Planners Design Studio Jan des Bouvrie, Naarden Dirk Jan Peters
Designer	
Contractor	
Steel construction:	
Tonnage	
Height	124 m(21 floors)
Total area	35,000 m².
Ground surface	

3.4.1 General description and Functional concept

In Rotterdam, June 5, 2009 - KCAP Architects&Planners completes residential high rise development The Red Apple in Rotterdam; 124 metres high The Red Apple rises skyward at the head of Wijnhaven Island, located between Rotterdam's city centre and the river Maas.

The Red Apple is a highly varied architectural development with 231 apartments, offices, retail and restaurants on a total gross floor area of 35.000 m2.
The Wijnhaven Island occupies a strategic spot within the city. It is being redeveloped using a dynamic transformation model, which provides development guidelines that ensure a balance between new and existing construction as well as the preservation of fine views and sufficient incidence of daylight throughout the area. “By applying this model, its redevelopment substantially increases the area's capacity and improves the residential and environmental quality while preserving important qualities of the existing situation” states architect Han van den Born (KCAP).

The Red Apple complex is situated on a visually prominent location: at the tip of the Wijnhaven Island, with water on three sides and views across the river and the “Old Harbor'”. The converging lines of Wijnhaven

Island meet at The Red Apple, outlining a five-sided volume, part of which cantilevers beyond the substructure. In this block building, apartments of various sizes are grouped around a central atrium.

The southwest corner of the site is occupied by a slender apartment tower with a spacious glazed lobby as ground-floor entrance and live/work loft spaces on top. The levels 8 through to 40 contain apartments of various sizes. All apartments of the two volumes are diagonally oriented and offer a maximum of transparency via floor-to-ceiling glass and provide extraordinary views.

3.4.3 Structural concept

The interesting part is the low riser building with huge cantilever; it looks like a box placed on top of supports. This big cantilever is achieved by using 2-stories truss girder in which its vertical members are columns of the top structure and the bottom and top chords are floor beam (Figure3-17) [38]

Figure 3-17: Supporting structure [39]

The building is stabilized by braced steel frames because concrete core is not possible since where it should be , there is an glass atrium which takes 3-stories.

3.4.4 Erection and construction method

Lower building is erected first since it contains columns which support the top part of the building. Temporary supports are necessary during construction so that the young structure gets stable as soon as steel elements are erected (Figure 3-18).

3-18: Red Apple during construction [39]

3.4.5 Useful points from this project that can be used in the thesis

The important section of this project with respect to the work of this thesis is the low rise building in which a bloc of 9-floors is put on top of the structure previously constructed. But both structures share the supporting columns; this is not the case in the work of this thesis. The only thing, which can be leant from this project, is stability system and handling the gravity loads.

3.5 Conclusion

In SmithWesterrman, design was one of the options to raise the height of Dillenburgsingle building. In contrast to the objectives of this thesis, their structure was achieved by demolishing the old building and the structure is made mainly by concrete. However, the construction method used between axes -1 and 0 can be used in this project.

All the other remaining projects are made in steel and all have long cantilevers. The way these structures are made can help in determining the structural system required for Dillenburgsingle case study. The difference between the structural system of the above-mentioned projects and the case in this thesis is the presence of existing building which should stay untouched during construction of additional floors.

4. STRUCTURAL SYSTEM TO BE USED IN THE CASE STUDY

4.1 The design

To be able to design a structural solution for the given problem, it was necessary to get an insight in the vision of the principal as well as that of the architects. These two visions have been used to create the design in which the shape is the most essential element. The shape has been designed to emphasize the ideas of the architect.

4.2 Vision

The assignment given to architect was to create a design that is more attractive from both architectural and engineering point of views. The new design should provide more homes without putting down the already existing building and without using the existing terrain to erect a more extended building.

The new design must be congruent to the existing built environment. The point of great attention in this design is the location of the supporting columns in such a way that they don't affect the existing building

- The structural design should also meet aesthetical, functional, and technical requirements.
- The loads from the new building should be conveyed to the ground by the columns erected on sides of the existing building.
- The new alternative designs should have more relevant advantages in comparison to the current design whereby the existing building is demolished in order to make a new beginning

4.3 Boundary conditions

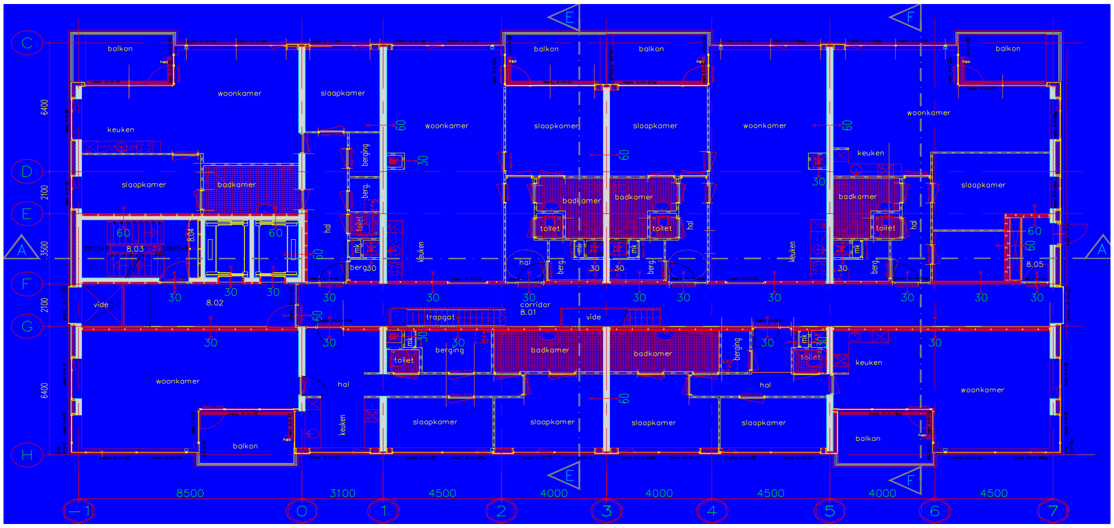
- Space to the main street (Dillenburgsingel) is small, no columns should be put there; cantilever is necessary to get required spaces.
- The new building should be carefully connected to the existing without transferring vertical, load to the existing building.
- On the rear façade of the building there's a low building and piping system; so, the position of columns should not be shifted.

The deformations of the new building should be limited since any excess deformation will put load on existing building.

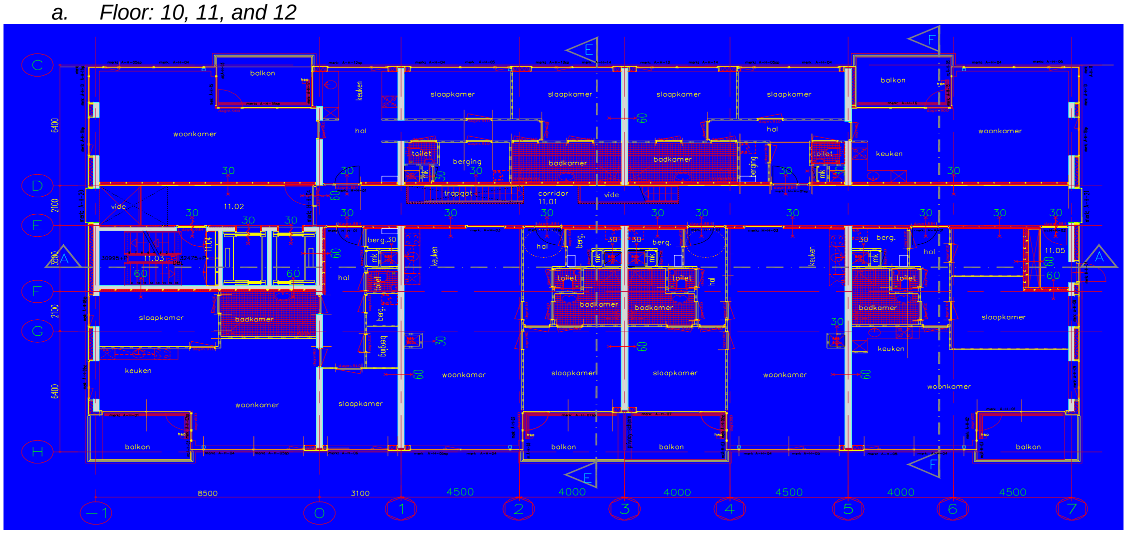
4.4 Interaction between floor framing and the building function

In current situation, the structure was implemented by using load bearing concrete walls as major supporting and bracing members, these walls are separated by a distance of 8.5m and located on odd axes- Figure 4-1. (see also 3.1)

If the even axes could be considered, there would be a possibility to increase the number of supporting columns since the underneath structure will not contribute in carrying the load of the added structure. The even axes correspond to the location of partition walls and can cover the structural members.
The advantage of introducing many frames is to reduce the size of structural members and to spread the vertical loads through many points to avoid large foundation load near the existing foundations. Finally, the inter-distance between two steel frames will be either 4,5m or 4,0m (Figure4-1).

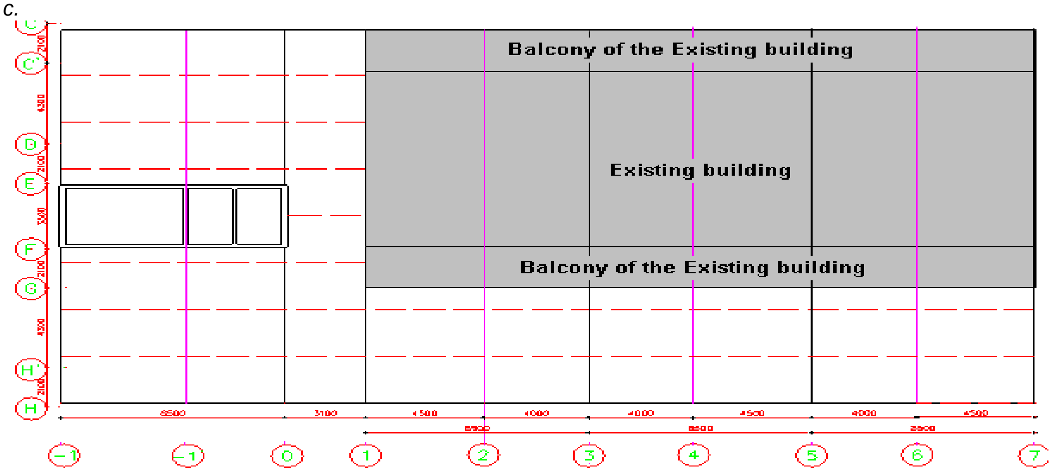


a. Floor: 10, 11, and 12



b. Floors: 7, 8, 9, 13, 14 and 15

c.



(c). Floor plan from 7th up 15th floor

Figure 4- 1: Furnished floor plans showing the function and division of spaces

4.5 The design parameters (Program of requirements)

To be able to design a structural solution for the given project, it was necessary to get an insight in a number of requirements:

- Structural demand or technical requirement: The new structure will stand on top of the existing without resting on it. The new structure has a cantilever part extending the existing building. The new structure should be as light as possible to reduce the weight on foundation so that no further stress to the existing foundation structure. It must resist the wind load and having a very limited deflection in order to not hinder the existing building.
- Aesthetic requirement: The set of the new and existing building should be aesthetically pleasing. There should be harmony between the new and the existing façade so that they can look like one structure.
- Functional requirements: The new and existing buildings are apartment facilities. They must always keep people comfortable in terms sound and heating requirements.
- Economy: the new structure should be executed at affordable price.

All of these requirements are presented in table of program of requirements.
The requirements can be divided in Aesthetical, Functional and structural or Technical requirements. Besides that, the goals, boundary conditions and assumptions are described.

Table 4-1: Program of requirements

Aesthetic demands		
Nr demand	Name	Description
A.1	Shape	The new structure has unique character due to cantilever section, integrated to the existing building.
A.2	Cantilever	The new structure cantilevers 6m from the edges of the existing structure.
A.3.1	Height	The entire building height is 47m.
A.3.2	Height	The height of the existing building is 19.6m and a sufficient clearance should be provided between new and existing structures.
A.4	Supporting	Columns for new structure should be put along both sides of the existing building.
A.5	Floor thickness	The structural height of the floor fhas to be reduced to a minimum.
A.6	Façade material	The facade should be made of steel sandwich panels and glass for windows. This reduces the weight of the structure.
A.7	Façade	The façade should have an open character and must provide comfort to occupants

Functional demands		
Nr demand	Name	Description
F.1	Perm. Load	The additional structure has to be able to resist all permanent loads
F.2	Var. Load	The additional structure has to be able to resist variable loads
F.3	Stability	The additional structure has to be able to resist all horizontal loads and be stable in all directions
F.4	Acoustics	No loud sound are allowable to be transferred from one floor to an other
F.5	Temperature	Temperature inside room should be comfortable to the occupants.
F.6	Durability	The structure has to be durable and reliable.
F.7	Columns	The columns supporting the new structure should not hinder the facade of the existing structure
F.8	Ventilation	The new building should have natural ventilation
F.9	Facilities and Fire re	Fire detection and suppression equipments, the building should have a fire resistance of at least 120minutes.
F.10	Integration	Services, functions of existing and new buildings should be integrated.
F.11	Cleaning façade	There must be a possibility to clean the façade on both sides

Technical or structural requirement		
• **Loads**		
Nr. req.	Name	Description
T.1	Permanent and Variable loads and their factor . in U.L.S and S.L.S	NEN 6702:2001 (art.7&8)
T.2	Load comb.	NEN 6702:2001 (art. 6,4)
T.3	Snow load	Prep= Ci*Psn;rep Psn;rep = 0,7 kN/m2 NEN 6702:2001(art.8)
T.4	Wind load	NEN 6702:2001 Annex A(see attachment)
• **Deflections**		
T.5	Hor. Deflection	The allowable horizontal displacement of the entire buildings may not exceed: 1/500 h. (NEN 6702:2001 art10)
T.6	Vert. Deflection	The vertical deflection of the horizontal member has to be smaller then 0.004*Lrep (NEN 6702:2001 art.10)
• **Boundaries**		
T.7	With existing building	The new structure have to be connected to the existing one.
T.8	With existing building	The connection between the two parts must be in a such a way that no vertical load would be transferred to the existing structure.
T.9	With existing building	The new structure has resist horizontal loads with help of the existing structure.
T.10	With street	Space at Dillenburgsingel is small , the new building will cantilever the existing one.

4.6 Choice of structural system.

The ability to form and shape a high-rise building is influenced by the structural system. This influence becomes progressively significant as the height of the building increases.
Buildings up to about 20 stories can be shaped without undue influence of the structure. In a range of 20 to 40 stories, one must start identifying a specific structural system, its composition and efficiency, and the flexibility for shaping offered by the system.
The primary structural concern is to develop enough lateral wind stiffness. In this case study the total number of storey is 15 made of existing six stories and nine stories which are added on top of the existing building. To restrain both vertical and horizontal loads, a building of less than twenty storey should be made of semi or rigid frames [6]. However, in the part, which is on top of the existing building, the load from the added structure should be carried separately from the load path of the existing building; the existing structure should be bridged. Therefore, this make it complicated to make a full rigid frame. As a concrete core is built for vertical transport between axis -1 and 0, It can restrain the horizontal load alone or with collaboration of braced frames which can be constructed in transversal direction of the building. Other techniques can also be introduced through design alternatives.

As the additional structure has to be light, steel frames can be used as major structural elements. Steel framed construction is a very popular structural form for multi-storey buildings in many countries as it provides great flexibility. In the Netherlands, even though steel building for residential purposes are not common, they can be used as a means of light weight structure to be used to add more store to the existing building.

4.6.1 Factors considered in choosing structural system for the case study

The design should ensure a coordinated approach including structure, envelope, services and finishes. The principal decisions regarding structure relate to column layout, foundation conditions, integration of building services, and external wall construction.

a. Position of columns

The position of columns will be influenced by functional and structural requirements. In this case study whereby the existing building would be bridged, the existing building should be columns free which support additional structure, and the columns should be placed along sides of the existing building(seefigure4-5) [12].

In current construction, due to heavy weight of the added top structure in concrete, the supporting members consist of concrete walls.

In this thesis project whereby the existing building will be maintained; the supporting members should not be located directly on top of the existing building. Therefore, the supports have to be located along the sides of the existing building in axis C and. This choice has been made to keep a distance of 14m between supports in transverse direction and therefore the ratio between cantilever and field will be within the allowable limits of 0, 42. [13]. Furthermore, this method was only one to keep the existing building free from additional weight of the added structure.

In current design, the supporting walls are located in axes -1, 0, 1, 3, 5, and 7. In this project due to the long span between supporting columns in transversal direction, and too slender columns there will be too much load which leads to column instability.
To avoid that, one can add other series of columns in axis -1', 2, 4, and 6(see figure4-2).

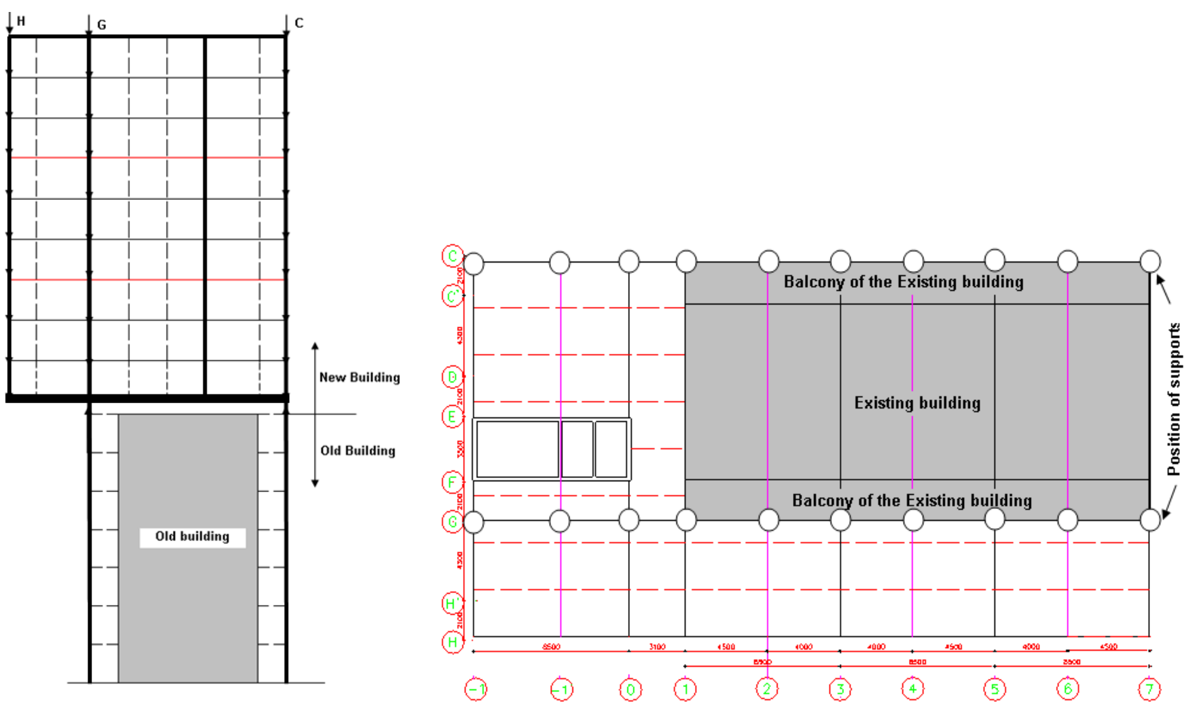


Figure 4-2: Position of columns in a cross section and plan

b. Construction materials

The new structure will stand on top of the existing building but without transferring weight to it. Therefore, the lightweight materials are needed, that's why steel elements are suggested to make the frame of the additional structure and sandwich panels for cladding and partition. The walls must provide quality space to meet the needs of occupants.
The acoustic and thermal performances should meet the requirements of building regulation. Therefore a proper insulation should be provided between the inner and outer skin of the wall panel.
The separation wall consists of:

- Two layers of sound resistance plasterboard(1)
- Horizontal resilient bars (2)
- Light steel studs section (3)
- Mineral wool between studs(4)

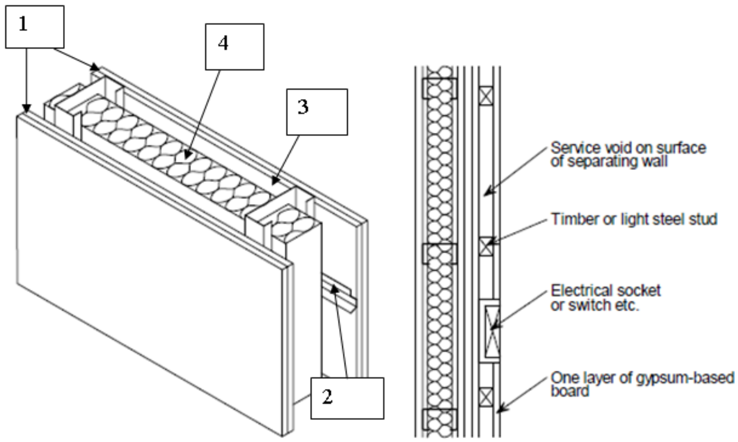


Figure 4- 3: Wall solution

For aesthetical reason, the supporting columns may be made of steel hollow section since they are visible and filled with concrete to increase their bearing and fire resistance.

c. Foundations

The design of the superstructure should take account of foundation conditions. In inner city and difficult sites, the time and cost of constructing the foundations has a major effect on the viability of a project. Although the weight of the frame is relatively small compared with floors and walls, a steel frame can be significantly lighter than a comparable reinforced concrete frame. Further reductions in weight can be achieved by using light floor construction such as composite metal deck floors, Hollow core slabs and lightweight concrete.

In this project, the location of the existing building dictates the grid of columns. The supporting columns must be locates along the sides of the existing building. The spacing of columns will be decided by taking into account the vertical load from the additional structure.

d. Floor system

There are different types of floor used in multistory buildings. Unlikely the Netherlands, in many countries most steel buildings use composite (shallow or deep) floor system. For both, various structural components - slab, beams and columns - are brought together to form the complete building structure. Many floor types will be described and the most advantageous will be selected in chapter 5 to be used in this particular project.

e. Structural principles

The structure must be designed to carry safely the applied loadings. The structure must have adequate strength and stiffness to resist the applied loads due to gravity and wind. The function of the structure in resisting vertical loads and horizontal loads due to wind is generally considered separately.

The principal floor loadings are due to the self weight of the building and its occupancy. These are referred to as 'dead' and 'superimposed' (or 'imposed') loads respectively.

The floor loadings to be supported by the structure have two components:

- The permanent or dead loading comprising the self-weight of the flooring and the supporting structure together with the weight of finishes, raised flooring, ceiling, air-conditioning ducts and equipment.

- The superimposed loading, which is the load that the floor is likely to sustain during its life and will depend on the use. This loading is provided by the code depending on the function of the building.

The design of the floor structure is concerned mainly with vertical loads. The criteria determining member sizes depend on floor span. The criteria determining the choice of a member size in a floor system varies with the span. [6] (See Figure 4-4)

The minimum size is fixed by practical considerations such as fitting practical connections. As the span increases, the size will be determined by the bending strength of the member and, for longer spans, by the rigidity necessary to prevent excessive deflection under superimposed load or excessive sensitivity to induced vibrations.

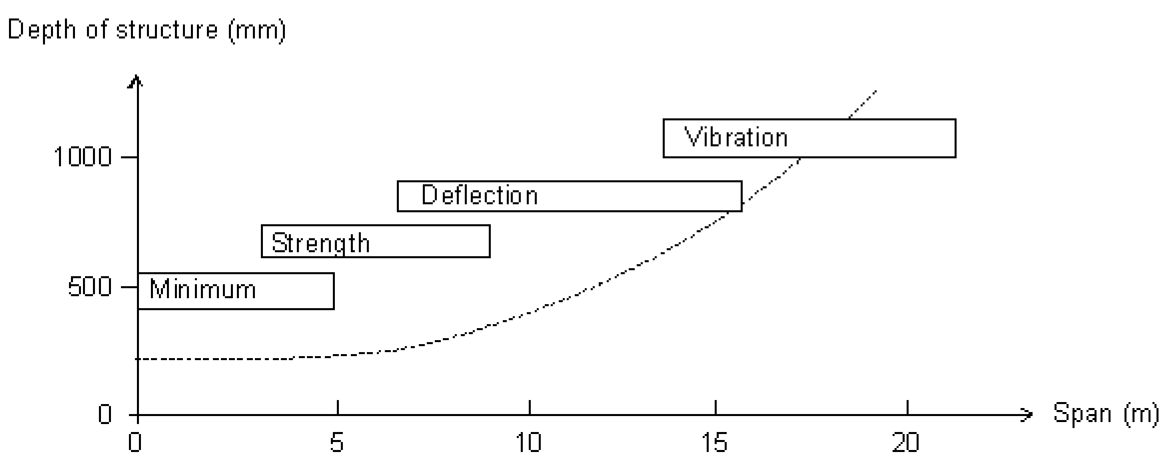


Figure 4- 4: Structural criteria governing choice of floor beam

f. Stability with respect to the horizontal loads.

Steel buildings have to be rigid enough in lateral direction to resist wind and other lateral loads. Most of multi-storey buildings are designed on the basis that wind forces acting on the external cladding are transmitted to the floors which form horizontal diaphragms, transferring the lateral load to rigid elements and then to the ground.

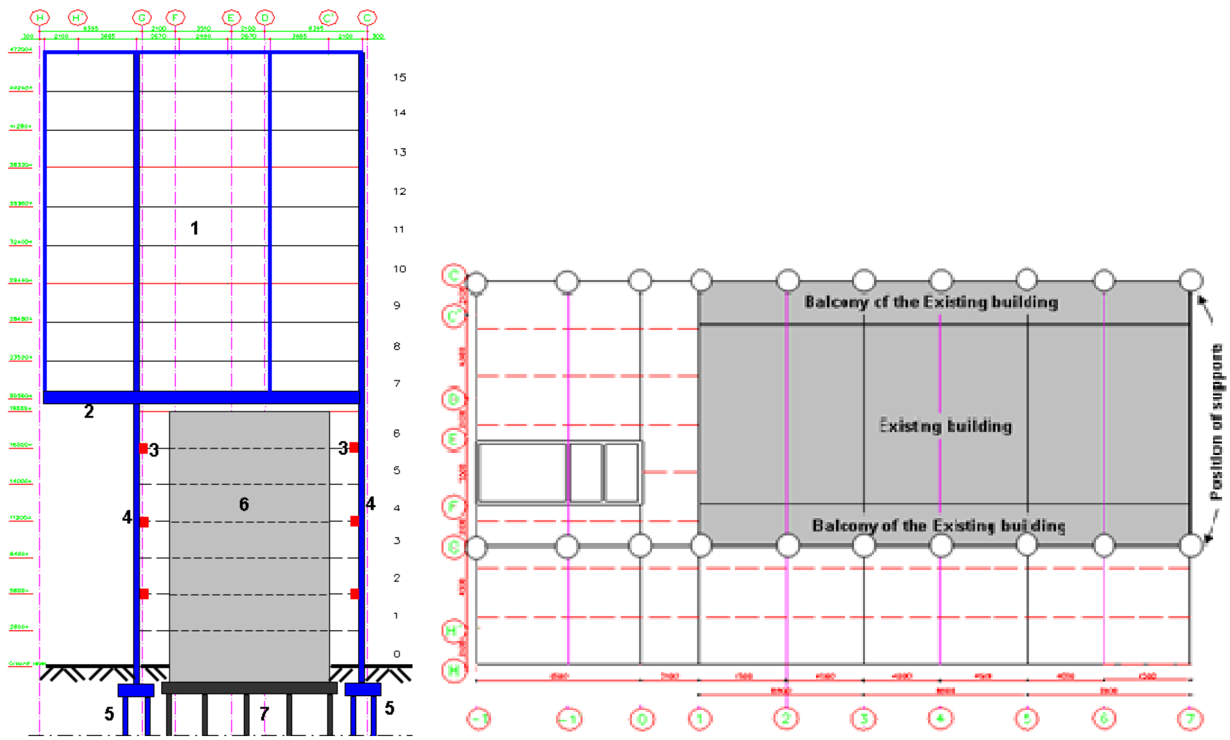
In this project, the stability of the building can be assured either by the concrete core alone (located between axis -1 and 0) or by combining it with braced frames.

All these will depend on flexural and torsional stiffness of the core. The appropriate method of stabilizing will be provided in chapter 8 and Appendix 8

4.6.2 Decision on structural system to be used in this case study

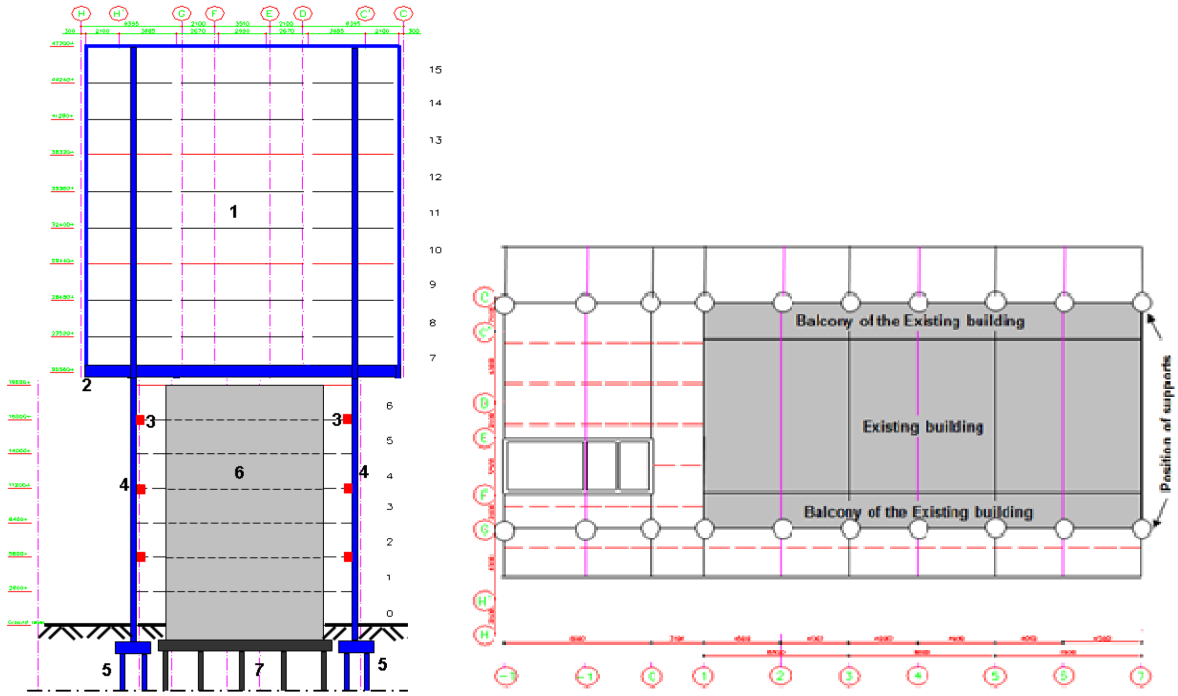
The existing building should not receive any load from the new added structure, therefore external supporting frame in transverse direction will be used. Two systems can be introduced; firstly with only one cantilever and secondly a cantered structure with two short cantilevers on both sides of the structure (see figure 4-7).

With these methods, it is possible to add more stories to the existing building without demolishing the existing building. In Chapter 2, it has been obtained that the existing building can stabilize the new structure once both structures are connected, therefore the frame of the new building should act independently but pivotally connected to the existing building, the weight of the new building is not supported by the existing building except a part of horizontal load . This method of raising the height of low-storied building solves a number of problems in relation to rebuilding buildings and avoids great waste.



(a). Section and plan with long cantilevers

1: New building frame
2: New building transfer platform
3: Connectors to reduce buckling length with a longitudinal beam
4. Columns of the additional structure
5. Foundation of the additional structure
6: Old building
7: Foundation of the old building
6: longitudinal beam at each level



(b). Section and plan with cantered frame (short cantilevers)

Figure4- 5: schematic drawing for calculation

The framework with the added structure centered on top of the existing building seems to be more attractive form structural and aesthetical points of view. With this system, the negative moments in cantilevers are reduced and hence the size of girders is reduced. However on the rear façade of the building (near the axis C), another low riser building will be constructed(see figure4-6). To provide more space between the two structures the existing design with long cantilever on front side of the building should be kept (Figure 4-5a) and the building looks structurally or architecturally pleasant as the new building is fully integrated to the existing one.

This shape complies with the vision of architect whereby the new building should have long cantilevers on one side of the building. Cantilevers will be most influencing factor of the design. This system will be coupled with an appropriate floor system in order to get the final concept to be implemented.

Figure 4- 6: Shape overview of the new structure

4.6.3 Construction techniques

The frame of the added structure is constructed above the roof of the existing building in a such away that the weight of the added structure should be supported by additional structure itself and should be transmitted to the ground base without passing through the existing frame. A clearance is provided between a bottom surface of new structure girders and a top surface of existing building; the clearance should be bigger than an amount of sinking of additional structure.

In order to enhance the stability of the newly added story-adding supporting frame columns, they are connected to each other lengthwise along the existing building at the level of every two corresponding floors place thereof by means of the steel beams [12]. In transverse direction these supporting columns should also be connected to the floor diaphragm of the existing building to reduce the buckling length of columns.

It is understood that the structure of the existing building is basically stable after many years of use, so that it may be deemed a non-deforming rigid body. The additional structure, at an early stage of use, may present sinking of foundation and self-deformation of the structure members after being subjected to load, resulting in a vertical relative displacement, between the new and existing buildings. Consequently, if no appropriate measures are taken, or no specific connections are provided between the existing and new buildings, but rather, a conventional rigid connection is still employed, the weight of the new building would surely press down on the existing building resulting in a confusion of route of force transmission and the increase of stress within the existing and new structures.

For that reason , certain clearance between the bottom surface of the additional frame supporting girders an the top surface of the existing building, particularly as the connectors between new and existing structures employed between the new and existing buildings are oriented pivoted bearings. The connection between new and existing frame enhances the stability of the new structure and the integrity with the existing building [12].

The ground base on site is not rock bedding; isolated foundation under columns and strip foundation are not possible. Pile foundation is the only possible option but piles will be installed new existing building, drilled or bored piles can be used. It is not allowed to use driven piles to avoid disturbance of the already settled foundation of existing building. [12]

5. FLOOR SYSTEM

After deciding on appropriate frame, the next step is to choose an economic structural floor system to satisfy all the design constraints. The choice depends on the span of the floor and function of the building. According to high-rise [6], for spans up to 12-15m, simple universal beams with composite deck or precast floor are likely to be most economical. Above 15m, composite steel trusses or fabricated girders will be necessary. As the span increases, the depth and weight of the structural floor increases, and for spaces in excess of 15m, span depth predominates because of the need to achieve adequate rigidity. Figure 5-1 shows the economical way of choosing the floor system based on span.

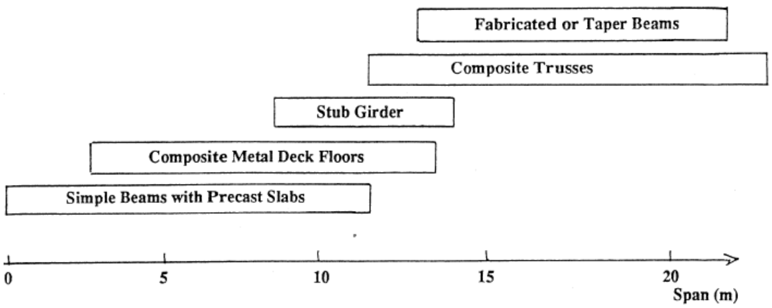


Figure5- 1: Choice of floor system according to span

As the maximum span for floor units is 4, 5 m (between axes 1 and 7) and 8,5m between axes -1 and 0, composite floor and precast floor units seem to be adequate solution.

5.1 System with composite deck slabs

The recent growth in the use of structural steel work for multi-storey buildings has been largely the result of development in the design and construction of structural floors consisting of simply supported universal beams acting compositely with a thin concrete slab supported on a metal deck. In this system, a metal decking spans between secondary steel beams. Steel studs are welded through the decking onto the flange of the beam below to form a connection between steel beam and concrete slab. The obtained slab is shallow with reduced volume of concrete which lead to a very light structure.

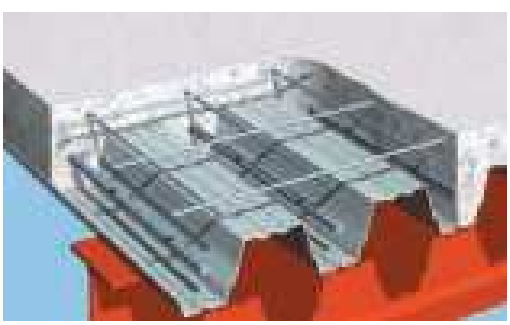


Figure5- 2: Composite floor system (Shallow)[11]

The other type of composite system is the use of deep floor system (Slimdek) whereby the steel sheeting is put on bottom flange of the main beam, and the secondary beams are not necessary (Figure5-3).

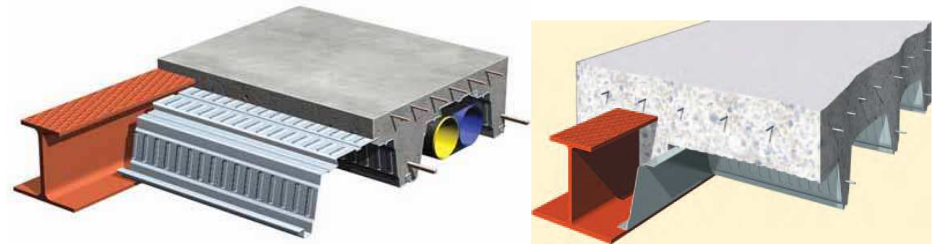


Figure5- 3: Composite floor system (deep floor system)[11]

The deep floor components form a stable structure once installed; the decking sheets are fixed to the frame to provide lateral stability and end diaphragms not only ensure that the concrete is contained during placement but resist vertical loading and allow the full shear capacity of the deck to be realized during the construction stage.

It has the ability to accommodate services between the ribs and lead to substantial savings in the cost of services

Advantages of shallow floor system

- Steel deck acts as permanent shuttering which can eliminate the need for slab reinforcement and propping of construction while concrete develops strength..
- Lightweight construction reduces frame loading and the foundation cost.
- It allows simple and rapid construction technique
- To provide a working platform for construction, to act as formwork for the concrete slab, to constitute bottom reinforcement for the slab.
- The shear connectors make the connection between steel and concrete.
- Easy connection among members.
- No temporary support during construction.

Advantages of deep floor system

- Reduction of structural height
- The composite action is achieved without shear studs
- The construction is lightweight and services can be integrated between the decking ribs passing though opening in the slab.
- The integration of building services is provided by the voids left in deep decking system.
- These floor have a large span
- Fireproofing is very easy since the area of steel exposed to fire is limited.
- They have a high sound insulation.
- The secondary beams are not necessary.

Disadvantages of shallow floor system

- During fire outbreak, all steel beam will face fire
- Fire protection is expensive due to the large steel area to be protected
- It results in slightly high structural height
- Service integration may require perforation of structural members
- The structural height is bid due to superimposition of steel profiles

Disadvantages of deep floor system

- Deep floor system results in heavy structure compared to the shallow system
- To avoid more weight, short span may be effective
- The inspection of connection point is not easy
- The deep decking sheet is placed on the bottom flange; it results in a very thick slab.
- The temporary props are necessary during construction stage.

5.2 System made by Universal beams with precast concrete floors(Hollow core slab).

Hollow core slab units are made of either reinforced or prestressed concrete with longitudinal cores of which the main function is to reduce the weight of the floor-Figure 5-4. The elements are available in different depth in order to satisfy various performances needs for span and loading. The prestressed units are the most used in multistory building. They are normally 120mm wide and up to 20m long. The actual unit width is usually 3 to 6 mm less than the normal size to allow for construction tolerances. Prestressed hollow core units are mostly use in public building such as office, hospital, schools etc. However they rare also used in apartment buildings because of the favorable cost rate and the fast erection. Prestressed hollow core units are manufactured using either a long line extrusion or a slip forming process. The degree of prestressing, strand pattern and depth of unit are the main design parameters.

Figure5- 4: Precast and prestressed floor units

Advantages

- Fewer floor beam since precast floor units can span up to 6 to 8m without difficulty and no propping is required.
- Shallow floor construction can be obtained by supporting precast floor units on shelf angles.
- Fast construction because no time is needed for curing and the development of concrete strength.
- High fire and sound resistance

Disadvantages

- Composite action not ready achieved without a structural floor screed (structural topping).
- Heavy floor units are difficult to erect in many locations and require use of a tower crane which may have implications on the construction program.

5.3 System with Bubble Deck floor

Bubble Deck is a flooring method of virtually eliminating concrete from the middle of a floor slab not performing any structural function, thereby dramatically reducing structural dead weight.

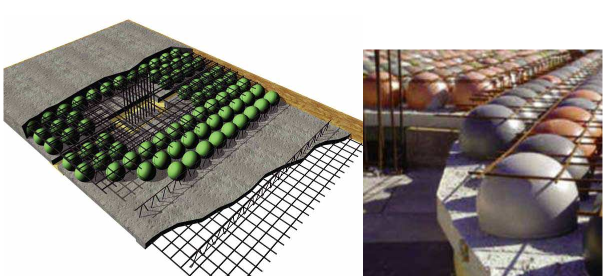


Figure5- 5: Bubble deck floor

Void formers in the middle of a flat slab eliminates 35% of a slabs self-weight removing constraints of high dead loads and short spans. Incorporation of recycled plastic bubbles as void formers permits 50% longer spans between columns. Combination of this with a flat slab construction approach spanning in two directions; the slab is connected directly to in-situ concrete columns without any beams

Advantages

- Design freedom: flexible layout easily adapts to irregular and curved plan layouts.
- Reduced dead weight: 35% removed allowing smaller foundation sizes.
- Longer spans between columns: up to 50% further than traditional structures.
- Down stand beams eliminated: quicker and cheaper erection of walls and services.
- Load bearing walls eliminated: permit lightweight building envelopes.
- High fire resistance

Disadvantages

- Due to void, the bending and punching shear resistance are reduced
- Services cannot be integrated the structural height

5.4 System with Lattice Girder Floors (Reinforced composite floor plates)

These floor units are used as permanent precast concrete formwork to In-situ concrete slabs. This type of composite floor is equally suited to use in almost every building type; masonry, steel or concrete structures and particularly those with progressive collapse considerations.

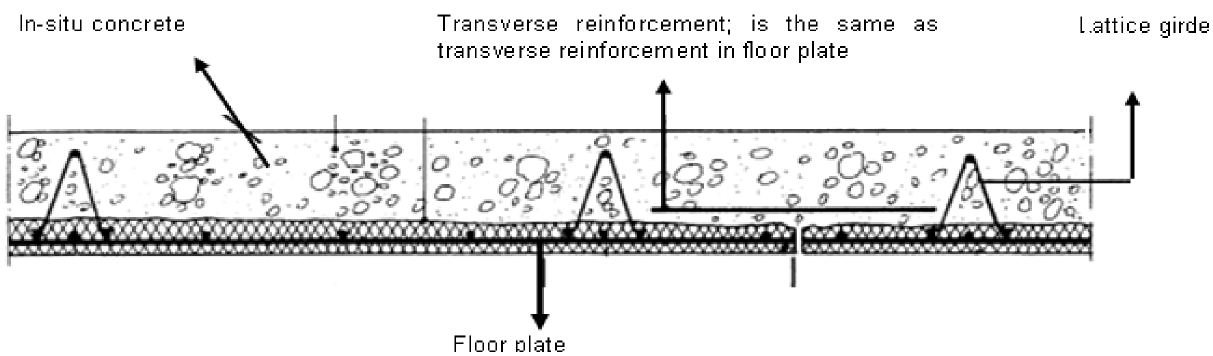


Figure5- 6: Lattice girder floor

These floor slabs comprise a 50 to 70mm thick precast concrete soffit slab, containing individually designed main and transverse reinforcement cast on steel moulds giving a good soffit finish ready for direct decoration after normal preparatory treatments.
Lattice girders are cast into the slab to provide a mechanical fixing between the precast unit and the In-situ concrete. These girders provide rigidity to the unit during transportation and their placement onto the prepared bearings.

Advantage*:*

- The main advantages of this system , compared to the traditional cast-in-situ floors are , that, apart from the props, no mould have to be used and most of the reinforcement is incorporated in the precast plate.
- The construction time is reduced and the good finishes of the whole floor are achieved due to the smooth floor soffit.
- It behaves as normal concrete floor and therefore it has a high sound and fire resistance

Disadvantage

These kinds of floor are heavy and therefore the cost of supporting members as well as the foundation is much more.

5.5 Selection of floor system

Selection of floor system is based on structural load, method of construction, accommodation of services, fire safety, erection speed and comfort of occupants.

In this case study, the building should have sufficient sound insulation since it is used for residential purposes, therefore sufficient mass should be considered when selecting the floor type and eventually the erection speed.

Solid slab (lattice girder) and Bubble deck will not be selected. The first is very heavy whereas the second is not strong enough for out- of -plane actions. Both floor system cannot accommodate services and require too much site works.
In composite floor system, one can see that the span between support columns is 14m; deep floor system cannot be used due its high depth and consequently enormous weight.

The system with shallow floor reduces floor weight and the secondary beams minimize the overall deflection of the whole floor. However, the structural height becomes too big and field works are enormous.

Finally, Hollow core slab units present lot of advantages and can be adapted to the entire structure which will facilitate the construction works.

6. DESIGN ALTERNATIVES BASED ON FLOOR FRAMING SYSTEM

In this chapter, various alternatives will be made based on floor framing system from axes 1 to 7. This is a governing part of the building; it is where the new structure will be put on top of the existing building. The part form axes -1 to 0 can be designed by taking into account the presence of the core and load transfer system.

The floor structure forms horizontal rigid planes. They stiffen and join the vertical building structures, allowing the building to respond to forces as closed unit. The floor framing transmits gravity and lateral loads to the columns and / or walls.
The layout of the floor structure is of great importance because it determines the direction of flow of wind and gravity forces, thus influences the geometry of the building skeleton.

This part will discuss the floor structures from the following point of view:

- The floor framing as related to common structural building systems and as a distributor of gravity forces.
- The floor framing as a distributor of lateral forces

The new building; the additional parts to the sides and the top of the existing building has a floor area of 37,1m x 19,9m (Figure 7-1). From the layout of the new building, an optimum framing should be in transverse direction so that the frame put there may have two possibilities of being used for carrying gravity loads as well as lateral loads.

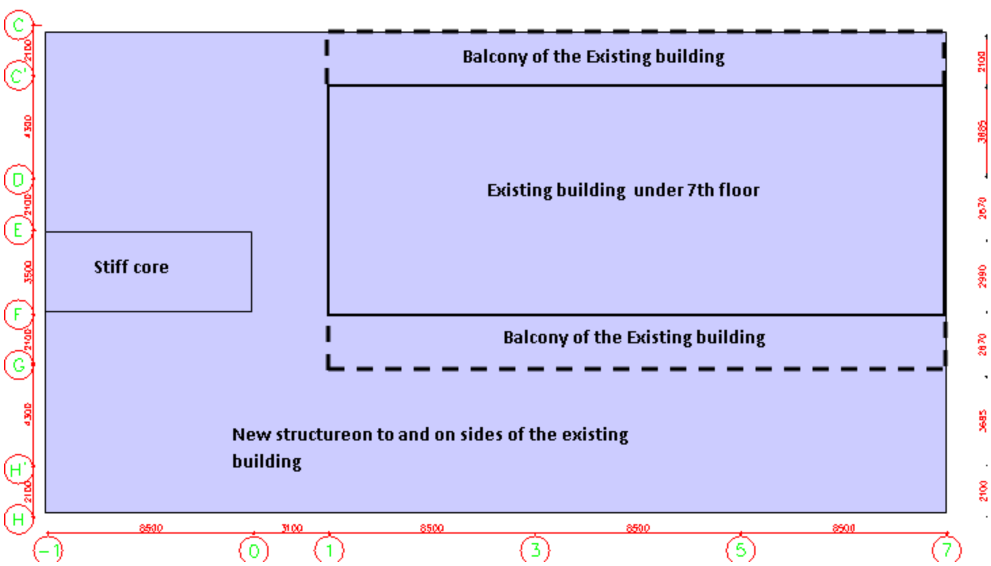


Figure 6- 1: Floor area of the new building (from 7th to 15th floor)

The most effective floor framing system is transverse framing [9]. A number of alternatives will be produced and in each alternative the number of intermediate supports is varied which will increase the possibilities of making the floor structure of the new building.

In transverse frame systems, the gravity forces are distributed to the internal frame spanning the width of the building. Thus, the frame will resist not only the gravity but also the primary lateral forces. This capability of restraining lateral forces, shows that the transverse or cross frames are the most efficient to form the floor structure.

6.1 Transverse frame made of continuous girders cantilevering at each floor level

Goals of using continuous parallel beams system

- The goal of this alternative is to create a light weight structure, easy and few connections and increasing the erection speed.
- To reduce fabrication and erection complexities by reducing the total number of steel members in steel frame
- To reduce the weight of steel beams by using continuity
- To reduce the complexities of connections between structural members and between structural members and services

The combination of these aims produces a steel frame that can be constructed quickly and cheaply. The resulting building has great flexibility of service and planning, allowing speedy erection of the frame and reducing the overall building cost

A general arrangement of this method of framing is shown in figure 6-2. To avoid conflict between service and structure, it has two planning zones, one above the other. The services are then arranged parallel to the structural members, permitting a high degree of servicing within the structural depth.

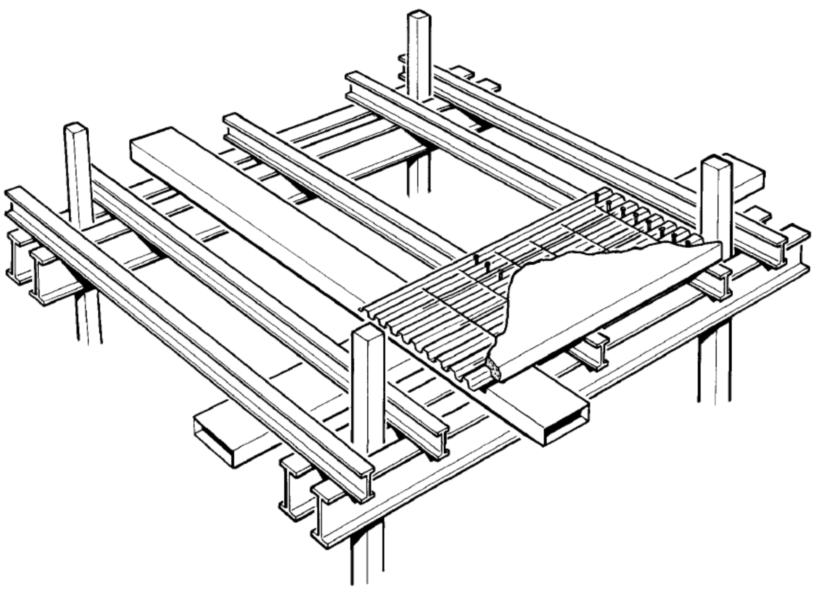


Figure6-2: Parallel grillage system showing service zones

To make continuous girders and continuous columns, the girder at each grid is executed as double girder [8], one at each side of the column. Continuous columns will increase the erection speed. To reduce the moments in the mid span of girders, small cantilevers can be welded to the columns. The floor beams can be simply connected to these cantilevers. This effect can introduce additional moments in the columns and continuous columns can easily restrain these moments.
In lateral cross section of the building, there are only two supporting columns. As both columns and girders must be continuous, two brackets are welded on both sides of the columns in lateral direction to make double-beam system.

The whole frame is composed by the beams, which carry the slabs and columns for taking the floor loads to the foundation. Column spacing depends on building function; they must be arranged in the plan depending on the space to be provided in the building as well as to the area where the foundation will be located.

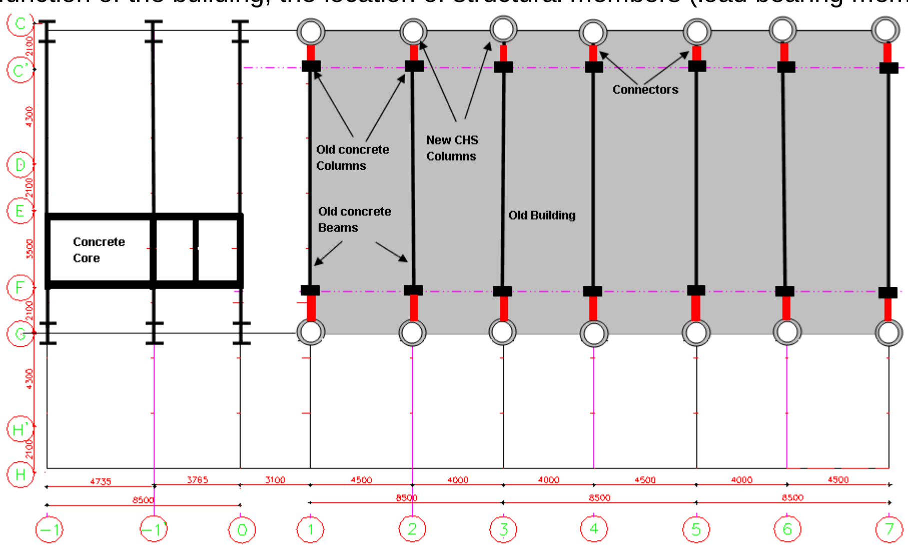
The main supporting columns are put along the sides of the existing building because it is not possible to put columns through the existing building [12]. The means of load transfer will be discussed and must match with the floor system.

This structural solution consists of floor girders with cantilevers at each floor level. The resulting girder height, according to the method of additional deflection [24] will be around 550mm, since the span is 14115mm and cantilever of 5,785 m.

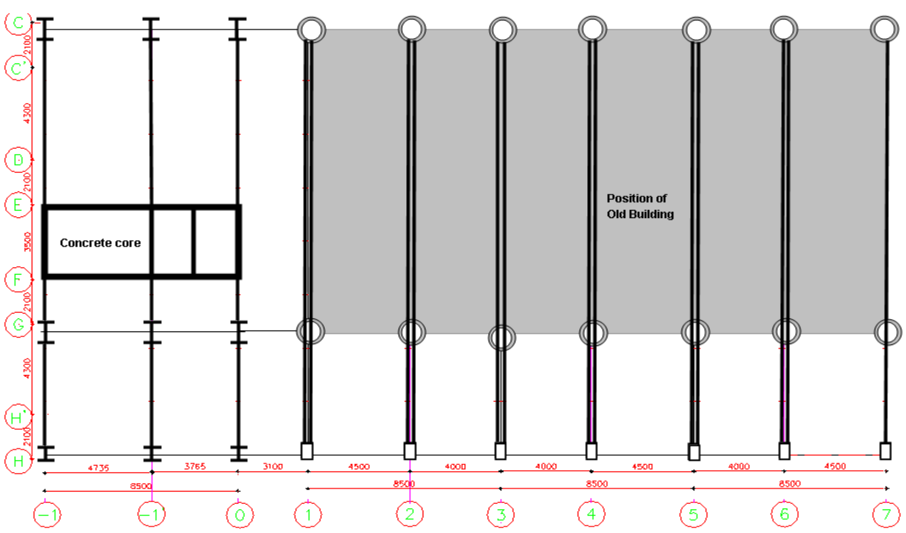
Shape

Apparently, shape of the structure is adapted to the shape designed by the architect .But some internal spaces such as corridor will be altered due to the presence of bracing members in braced frames. Another change is the increase of the number of frames; for the new design frame in axis -1', 2, 4 and 6 (Figure 6-3) are inserted in order to reduce the load carried by columns and therefore to have less impacts on adjacent existing foundations.

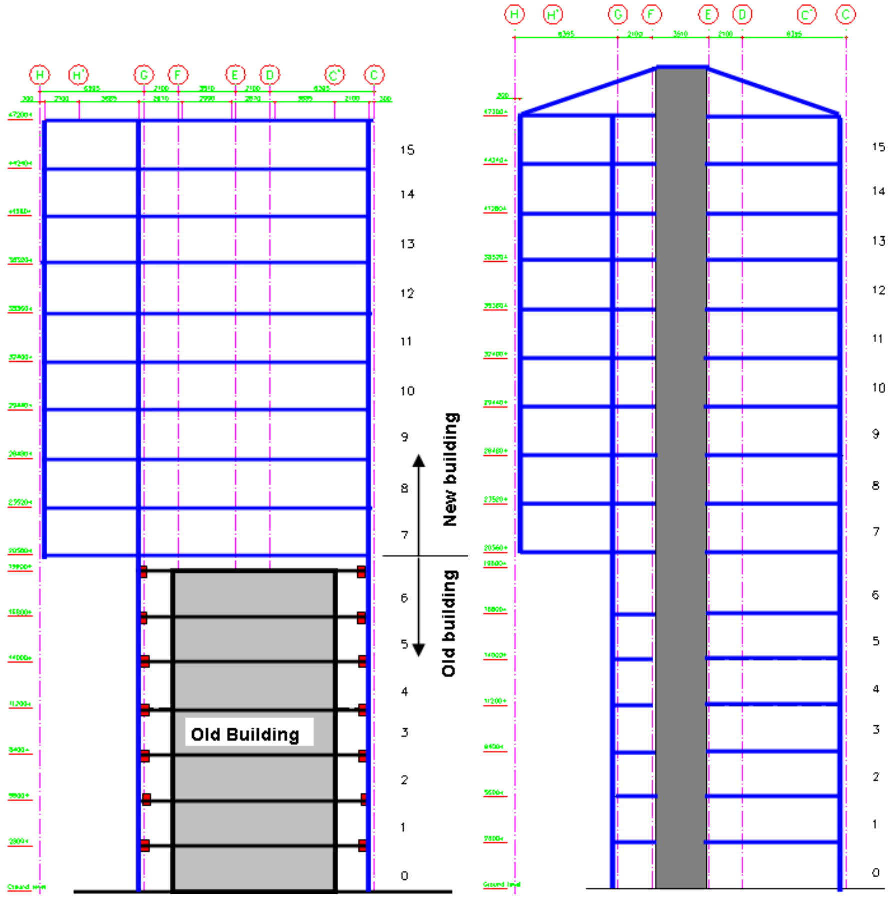
In longitudinal direction, the floor is divided into 4, 5m and 4, 0 m grids (Figure 6-3), with respect to the subdivision of the apartment. In transverse direction, the subdivision depends on the available spaces, the function of the building, the location of structural members (load bearing members) and bracing systems.



(a) Plan for Floors 1 to 6



(b) Plan for Floors 7 to 15



(c). Section in axes 1 to 7 (d) Section in axes -1 to 0

Figure6- 3: Transverse frame made of continuous double girder

Supporting

The structure will be supported by columns located near the existing building and bring the load of the superstructure to the new foundation located next to the existing foundation. The columns are arranged by following the grid of the existing building, the front columns will be heavily loaded due to the long cantilever load. In longitudinal direction, columns are arranged in axes G and C (Figure 6-3). In part between axes -1 and 0, vertical load is carried by columns and concrete core.

Stability

The stability of the whole structure can be assured by concrete core, braced frames and existing building.
When braced frames are used It is important that the structure possesses sufficient stiffness; by vertical braced frame which consist of beams, columns, and stiff diagonal braces that perform like shear walls.
Joining the columns in longitudinal direction and connection columns to the existing building's slab at each floor level, will make the new structure more stable. The columns work together and resist lateral movement and lateral displacement by bending. With diagonal bracings, the horizontal loads are resisted through tension and compression forces in the braces.

Force distribution

In frames 1 to 7, the load coming from the floor is transferred to the spine beams and then to columns and finally to the foundation.
The mechanical model for spine of this alternative is shown in Figure 6-4 where two scenarios (firstly with variable load on cantilever and secondary variable load on part between supports) can be analysed in order

to calculate the required beam size to be used. In this method the collaboration of the entire frame is not considered only unfavourable situation of individual beam is taken into account. However by coupling the edges of the beam with a column, the system may respond much better to the loading.

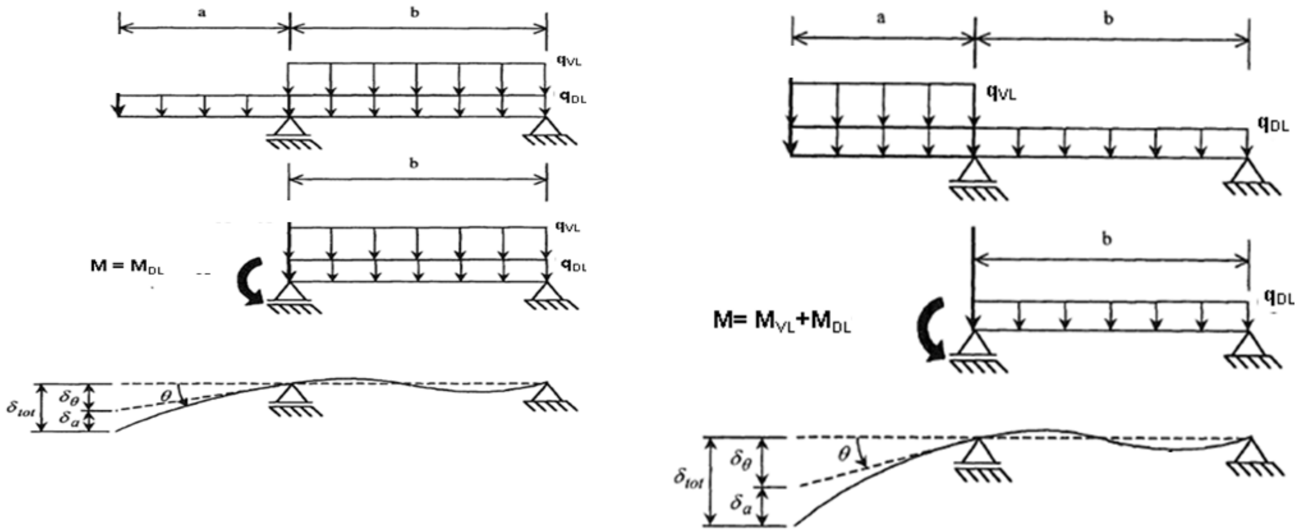


Figure6- 4: Spine beam models

From Appendix 3 and 6; it has been obtained that when live load is placed between supports a big beam section would be required .The moment of inertia equal to: $I = \dfrac{5,8.10^{10}}{\delta_{span}}$

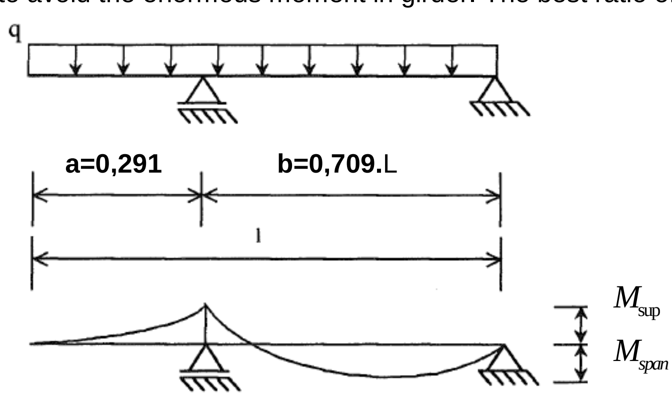
Where: I = required moment of inertia
δ_{span} = deflection in mid-span

Secondly when live load is on cantilever, a small size can be used; the section depends on this moment of inertia: $I = \dfrac{3,27.10^{10}}{\delta_{span}}$.

The worst case scenario has to be considered; that is why the first case will be taken into account when deciding on beam section to be used.

- **Required cantilever to field ratio.**

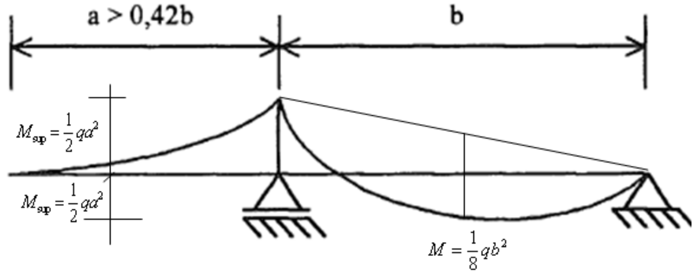
It possible to make a continuous girder by resting it to the console welded on one side of the column. The cantilever induces negative moment in beam, therefore the span and cantilever should have an optimum ratio to avoid the enormous moment in girder. The best ratio of cantilever to span is 0, 42 [13]



$$M_{sup} = \frac{1}{2}qa^2 \Rightarrow \frac{1}{2}q(0,291l)^2 = 0,0423ql^2$$

$$M_{span} = \frac{1}{8}qb^2 - \frac{1}{2}M_{sup} \Rightarrow \frac{1}{8}q(0,709l)^2 - \frac{1}{2}0,0423 = 0,23ql^2$$

Favorable cantilever-field ratio



Cantilever beam system

The maximum field moment in the girder is M_{span}= 1/8 q.b^2 and the support moment is $M_{support}$ =1/2 q.a^2.

$M_{span} = \frac{1}{8}qb^2 - \frac{1}{2}M_{sup}$ or $M_{span} = \frac{1}{8}qb^2 - \frac{1}{2}qa^2$

$$M_{span} = M_{sup} \Rightarrow M_{sup} = \frac{1}{8}qb^2 - \frac{1}{2}M_{sup}$$

As, $\frac{3}{2}M_{sup} = \frac{1}{8}qb^2 \Rightarrow \frac{3}{2}\frac{1}{2}qa^2 = \frac{1}{8}qb^2 \Rightarrow \frac{3}{4}a^2 = \frac{1}{8}b^2$

$$a^2 = \frac{1}{6}b^2 \Rightarrow a = 0,408b$$

The actual ratio of cantilever to span is $\frac{a}{b} = \frac{5785}{14115} \Rightarrow a = 0,409b$; this is slightly bigger than $0,408b$ obtained before but less than $0,42b$ considered as the maximum cantilever length [13]

In frames -1 to 0, the presence of concrete core brings problem in handling cantilever. Suspended system can be used to carry the cantilever part (Figure 6-3d). The roof can be used to build system which transfers a heavy bending structure. But the height of building increases and the erection is more complicated.

Connections

Strong building connections allow forces and displacements to be transferred between vertical and horizontal building elements. Furthermore, strong connections increase the overall building strength and stiffness by allowing all of the building elements to act together as a unit. Inadequate connections represent a weak link in the load path of the building and are a common cause of damage and collapse when building is subjected to the huge horizontal loading. In this project, the important connections are the following:

- Beams to columns connection
- Beam to floor slab connection
- Columns to foundation connection
- Connection of columns of new structure to the existing building

Connections between beams and columns are made by bolting through the beam web to bracket welded to the columns as shown in Figures 6-5. For a double spine beam, the brackets are symmetrical about the column major axis.)

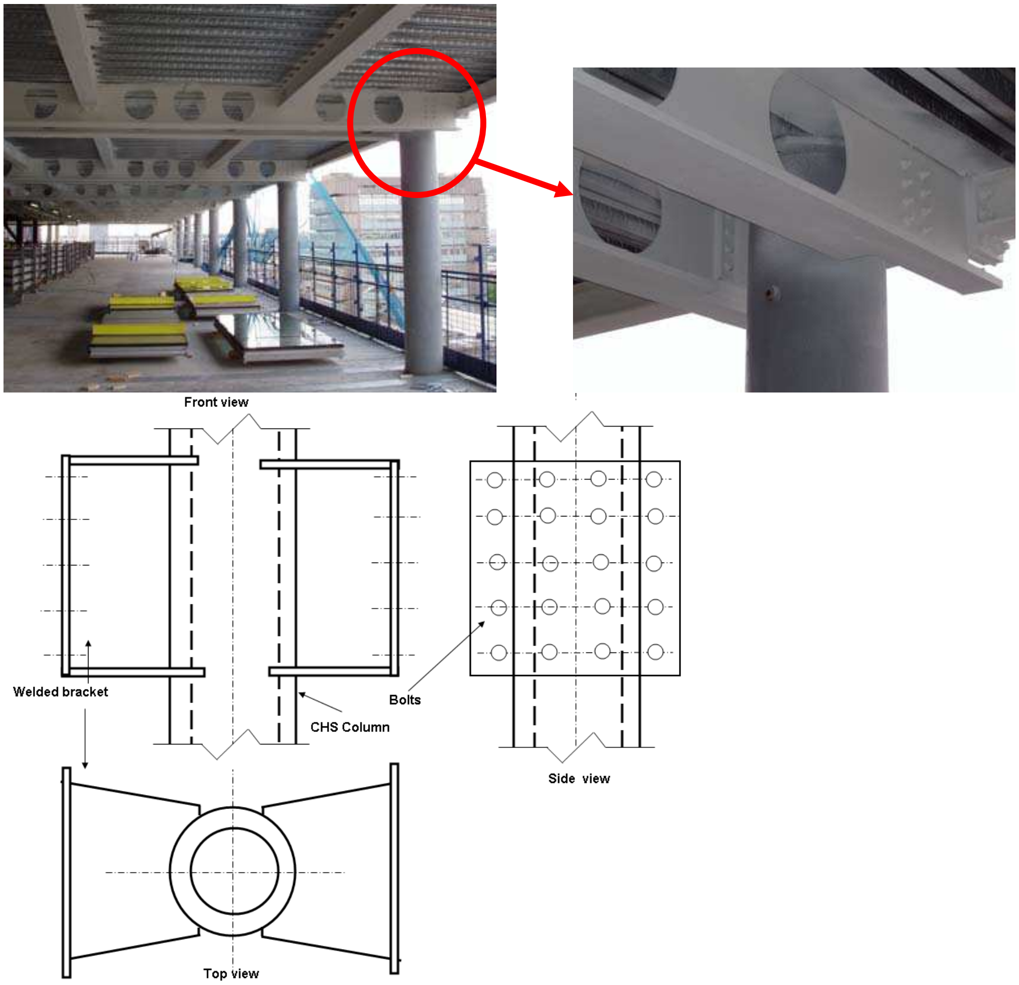


Figure6- 5: Connection of spine beams to columns

Fixing beams on brackets welded on two sides of the columns ,results in continuous beam, therefore the columns will be continuous too.
For erection purposes, a temporary support is put at the edges of the cantilever and then a column joining the edges of cantilevers for all floors can be put.

There are three possibility of connecting the beam edges to the edge column:
-By hinge connection
- Moment connection
- Moment connection plus additional hinge.
The latter is optimum due to the small deflection.
To reduce the moments in the mid spans, of the girders small cantilevers are welded to the columns. The floor beams are simply connected to these cantilevers.

Advantage and disadvantages of continuous beam system

– High speed of construction
– Few connection
– Clear flow of forces
– But to stabilize the building may be complicated because of the lack of concrete core and the bracings may have large dimensions.
– Composite floor system is not a good solution for such particular structure because on cantilever side the compressive stresses will be in steel girder and tensile stresses will be in the reinforcement of the concrete. The negative moment capacity will not increase much compared to the moment capacity of the steel girder.

Main difficulties

- The main difficulty of the design is to control the horizontal displacements and prevent large displacements. For example directly on top of the existing building; the wind may create large deflections.
- Stabilizing the structures along the existing building will be a point of attention. The horizontal displacement can be controlled by stabilizing structures
- The cantilever construction will be difficult because its large length; especially between axes -1 and 0
- The construction of new foundation near the existing one , requires too much attention
- Connection of the existing building's floor to the columns of the new building to reduce the buckling length of columns seems to be complicated.

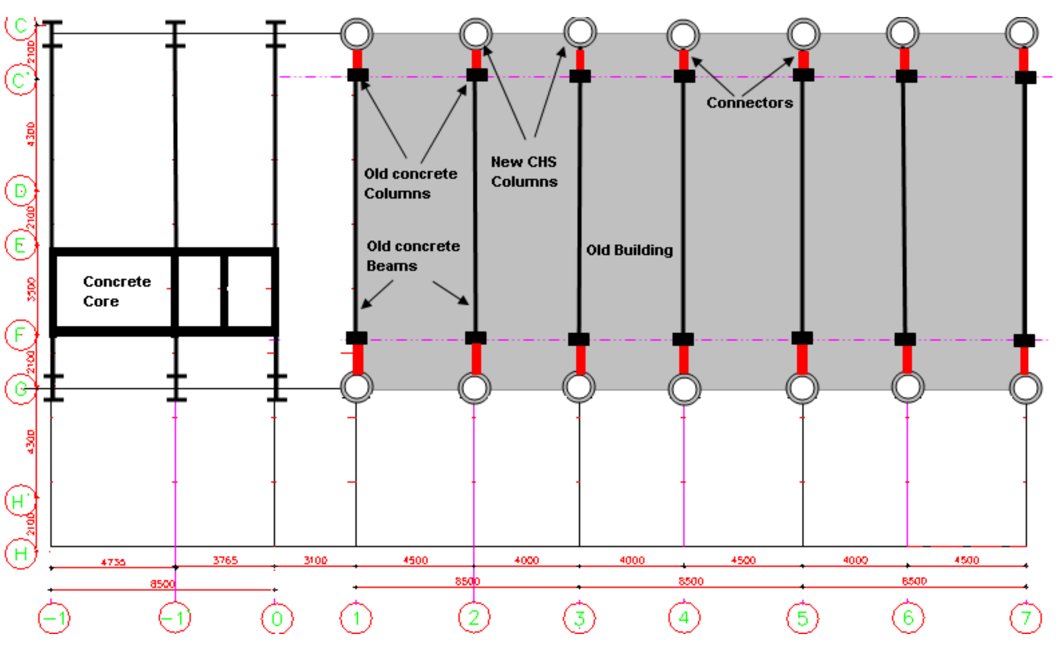
6.2 Transverse framing with one intermediate support

Goals of this alternative

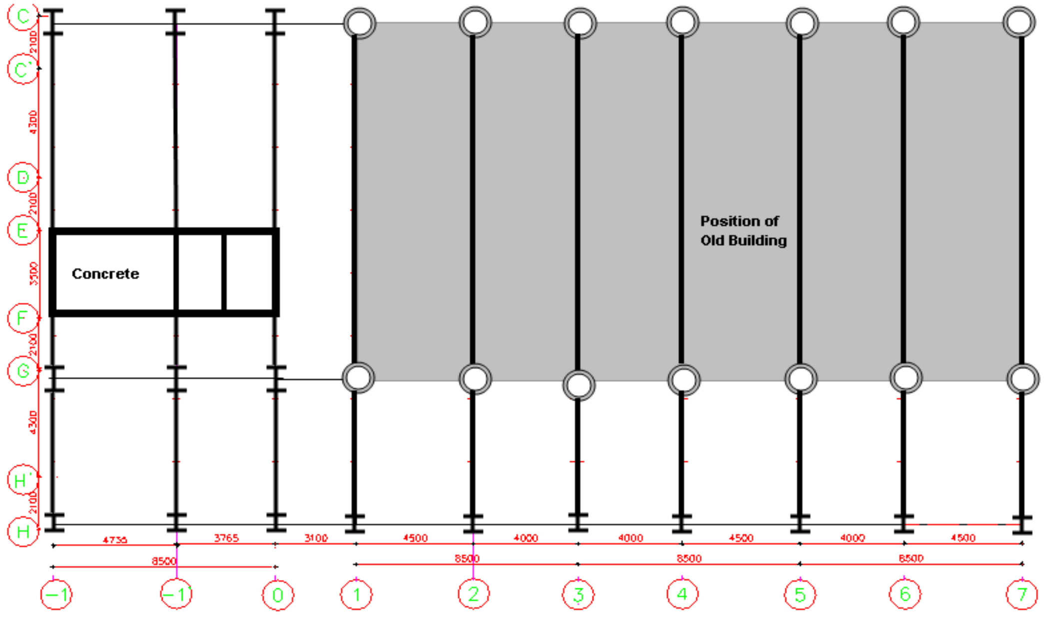
The goal of this alternative is to create a light weight structure and facilitating construction process by avoiding temporary support under cantilevers part. In the added structure, the cantilever beams are avoided since the whole new structure stack on top of a general cantilever made at the beginning of the new structure. The beams sizes are also reduced since they are no longer continuous.

Shape

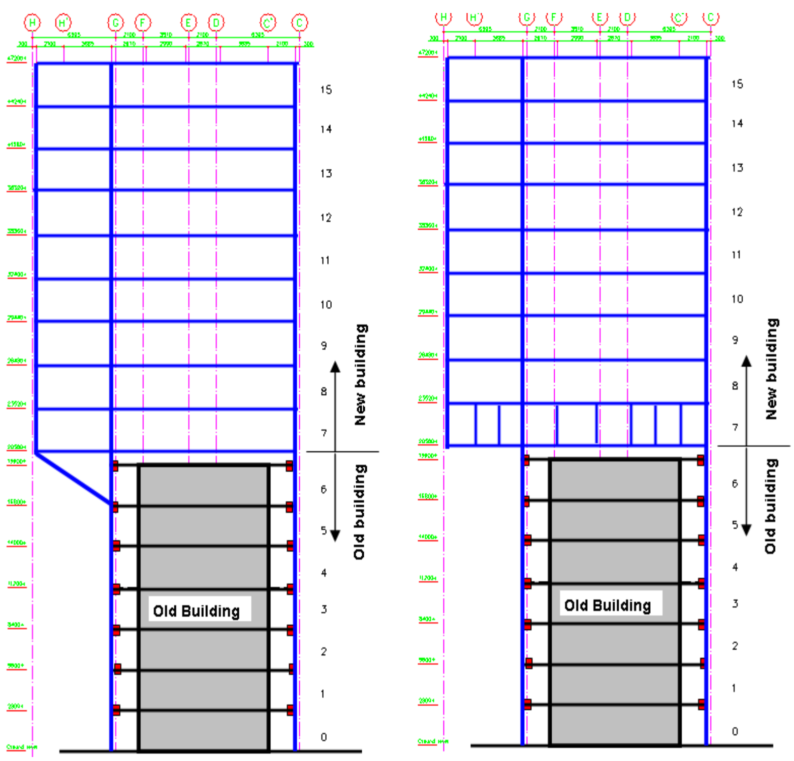
The shape of this alternative is similar to the existing design; however the presence of cantilevers where the new structure starts makes a small difference from the existing design (Figure 6-6a).
In this system, the transverse girders are no longer continuous; they are simply supported to some supporting columns.
At the beginning of the new building (between 7th and 8th floors) there must be a system to take the upper floor's weight by stacking in order to avoid the cantilever structures in upper floors. Therefore, consoles (Figure6-6c) or a storey high truss girders (Figure6-6b) should be provided at 7th floor to support the rest of the building and transfer the load to the side columns. This will facilitate the erection process because there will be no need of temporary support to the cantilever side.

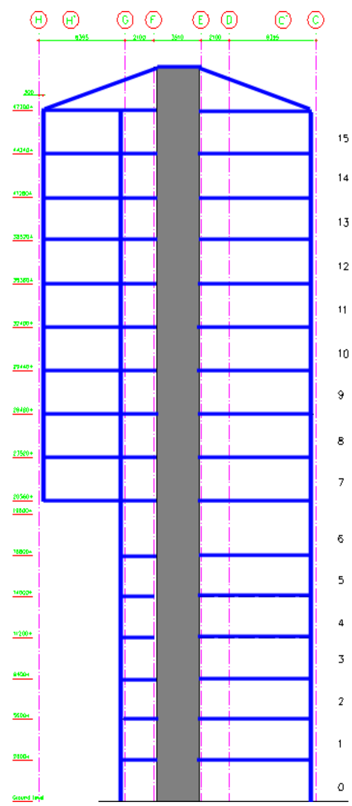


(a). Floor plan from 1st to 6th floor



(b). Floor plan from 7th to 15th floor





(c) Cross section in axes 1 to 7 with structural cantilever or transfer girder

(d) Cross section in axes -1 to 0

Figure6- 6: Frames with one intermediate support

The method with structural cantilever is good from structural point of view because the remaining floors stack on it. However, there will be to much horizontal force which cannot be handled by existing building and therefore this method is discarded. Transfer truss can only be considered.

Supporting

The new structure will be supported by columns located near the existing building and bring the load of the new structure to the new foundation located next to the existing foundation. The columns are arranged by following the grid of the existing building, the front columns will be heavily loaded due to the long cantilever load. In longitudinal direction, columns are arranged in axes G and C (Figure 6-6).
The additional structure is supported by transfer truss girders located at each transversal axis between 7th and 8th floors .These truss girders transfer the load to the supporting columns and then to the foundation.

- **Choice of truss girder type**

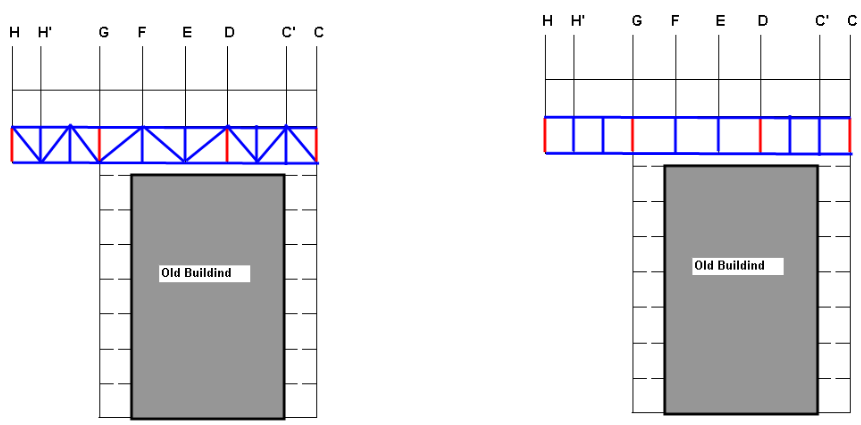
The truss girders are capable to carry enormous load and take them to the side columns. This method is very useful because what is needed is to bridge the existing building. The trusses are put between 7th and 8th floor and are one storey high-Figure 6-6, they are located at every transversal axis

The trusses have to be stiff enough and should not obstruct the building spaces such as rooms, opening, and corridors.
The web members should coincide with the axes where the columns for wall framing will be located.

As the top and bottom chords carry the slabs; so they act as composite trusses. Although the fabrication cost of truss girders is greater than any other forms of construction, they do have the following advantages.

- They do not require any special fabrication equipment.
- They offer plenty of spaces to accommodate services

The principal disadvantage, other than the increased fabrication cost, is that they are difficult to protect from fire. Sprayed protection systems are messy while the alternative of wrapping is labor intensive.
The trusses with triangulations, cause hindrance in some locations on 7th floor; some openings are closed by the oblique struts. To avoid this, Vierendeel truss was proposed to be the solution for the truss girder-Figure 6-7. However, Vierendeel girder brings some complications in joint manufacture since they have to resist enormous bending moments.



(a) Normal truss (b) Vierendeel truss

Figure6-7: Truss girder between 7th and 8th floor for axes 1 to 7

Stability

As the previous alternative, the horizontal load can be taken by the concrete core working together with braced frames put in one or several transverse axes and existing building. The longitudinal stability can be assured by the concrete core only.

Force distribution

From axes 1 to 7, the floor load is taken to the floor beam and then to the main beams which channel it to the columns. The columns take the load to the transfer girder and finally to the supporting columns and to the

foundation. From the structural point of view, this is the best solution, the floor levels above the cantilever, the other floors can easily stack on the first floor of the added structure.
The loads go directly to the foundation. The erection is easy because of the stack from the bottom to the top.
The beams can be modelled as simple supported beam (Figure 6-8) and hence the small sizes would be adequate for this system.

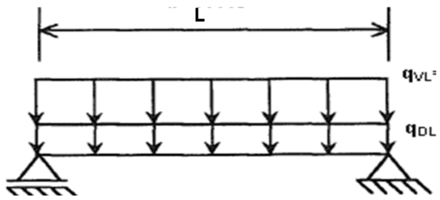


Figure6- 8: Beam model for system with one intermediate support

In axes -1 to 0, the loads are similarly transferred as in previous alternative because the structure in that part remains unchanged.

Connections

Simple connections between beams and columns would be used. If a general cantilever is to be used, special attention would be paid for connection the 7th floor main beams to the supporting columns. If transfer girder would be used, between 7th and 8th floors, twin transfer girders would be used in order to make them continuous and they will be connected to the supporting columns using the brackets welded to the columns.

Advantage

- Continuous girders are avoided in top floors
- Small beam size can be used since they are not continuous
- The whole cantilever part stands on structural transfer truss made at the beginning of the new structure and hence during construction no temporary support will be used.

Disadvantages

- The transfer truss girder systems seem to be not economical in terms of fabrication, connections and fire protection
- The support columns on cantilever side will not be continuous and will require complicated connections.

Main difficulties

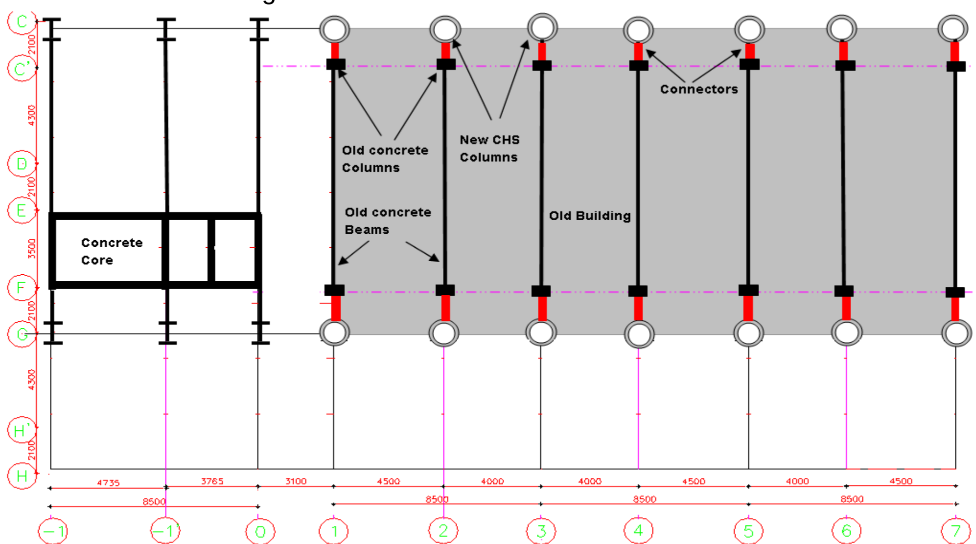
- The control and limiting horizontal deflection
- Huge negative moment in transfer truss girder
- The cantilever construction will be difficult because its large length
- The construction of new foundation near the existing one , requires too much attention
- Connection of the existing building's floor to the columns of the new building
- Fabrication of transfer truss girders and their fire protection measures

6.3 Transverse floor framing with two or more intermediate supports

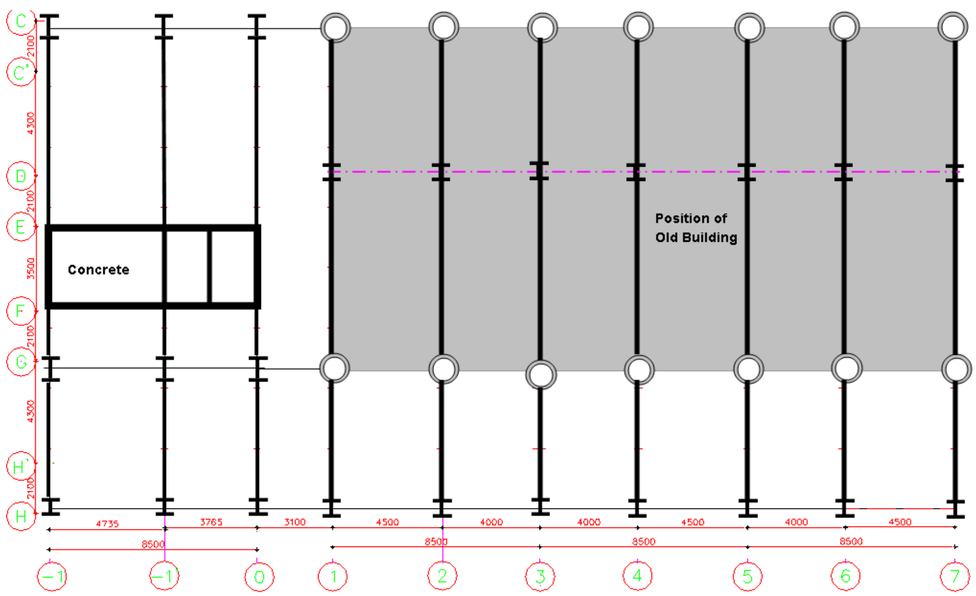
Goals
The goal of this alternative is the construction of a light structure by using many columns fixed on transfer girders. Short spans beams with small depth will be used.

Shape

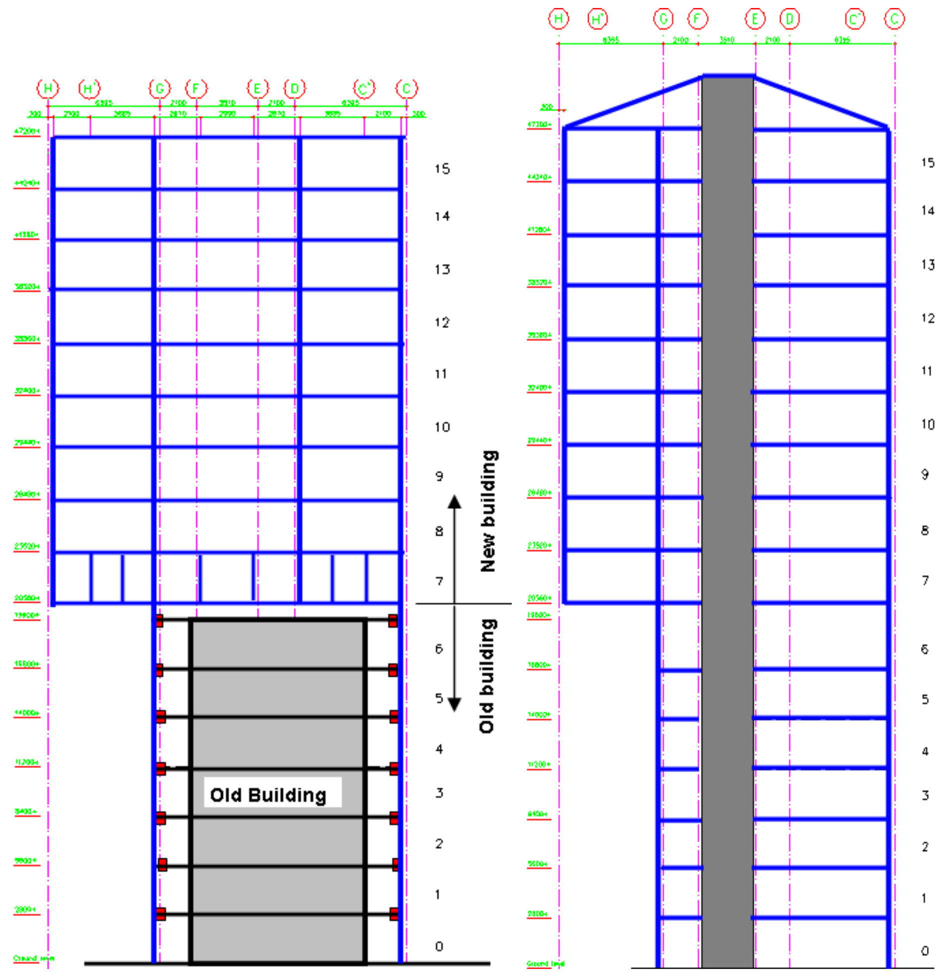
In this alternative, the solution to carry the cantilever load is by using transfer truss girders in the first storey of the added structure (Figure 6-9). The floors above stack on those girders for both cantilever portions as well as the parts between the supporting columns. The loads are taken by the girder and carried directly to the columns which ferry them to the foundation. The shape for this structure is the same as the existing shape from architectural design.



(a). Floor plan from 1st to 6th floor



(b). Floor plan from 7th to 15th floor



(c). Cross section in axes 1 to 7

(c). Cross section in axes -1 to 0

Figure6- 9: Steel frame supported by truss girder (stabilizing frame)

More columns can be now erected on truss girder to make rigid frames. This approach is a good solution to construct the top structure without neither cantilevers nor temporary supports, but rigid frame maybe needed to stabilize the frame which will increase the cost due to rigid connections.

Supporting

The additional structure is supported by transfer truss girders located at each transversal axis between 7th and 8th floors .These truss girders transfer the load to the supporting columns and then to the foundation.

Stability

The stability for this system is similar to the previous cases whereby concrete core with braced or rigid frames can be used to restrain horizontal loading.

Force distribution

Loads from floors are transferred to the floor beams connected to the columns. The columns in their turn, take the loads to the transfer truss girders and then to the supporting columns and finally to the foundation.

Connections

In top stories simple or rigid connections (depending on number of columns used) are used between beams and columns. The top and bottom chords of the transfer truss girders are also the main beams for the floor system; the transfer girders must be rigidly connected to the columns.

Advantage

- This is a better method to carry the cantilever load
- The entire building lay on transfer girders and therefore from 8th floor , the main beam are simply connected to the columns.
- Cantilevers are avoided on the rest of the building
- Member size can be reduced since many columns can be erected
- The construction is easy because the additional structure is supported by the transfer truss girders

Disadvantages

- The fabrication of truss girder may be very expensive
- Using truss girder on 7^{th} floor can obstruct some path ways
- In Vierendeel truss, as the joints have to resist huge moments on 7^{th} floor, enormous members will be required to make the truss which makes the system more expensive.
- Continuous truss girders are required to make cantilevers and hence columns in axis G are no longer continuous.

Main difficulties

- The main difficulty is based on bracing method for stabilizing the building against horizontal loading and controlling the horizontal deflection
- Connection of columns to transfer girders and connection of support columns to transfer girders
- Huge negative moment in transfer truss girder
- The cantilever construction will be difficult because its large length
- The construction of new foundation near the existing one , requires too much attention
- Connection of the existing building's floor to the columns of the new building
- Fabrication of transfer truss girders and their fire protection measures

6.4 Transverse floor framing with two concrete walls in axis -1 and 0

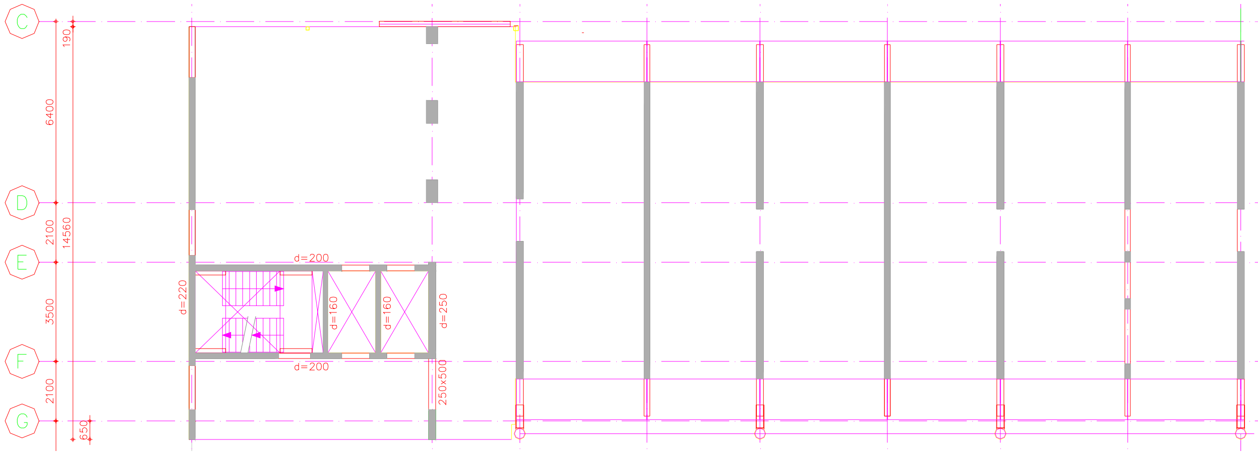
Goals

The main goal for this system is to facilitate the construction of cantilever section where the concrete core is located and to enhance the stability of the whole building.

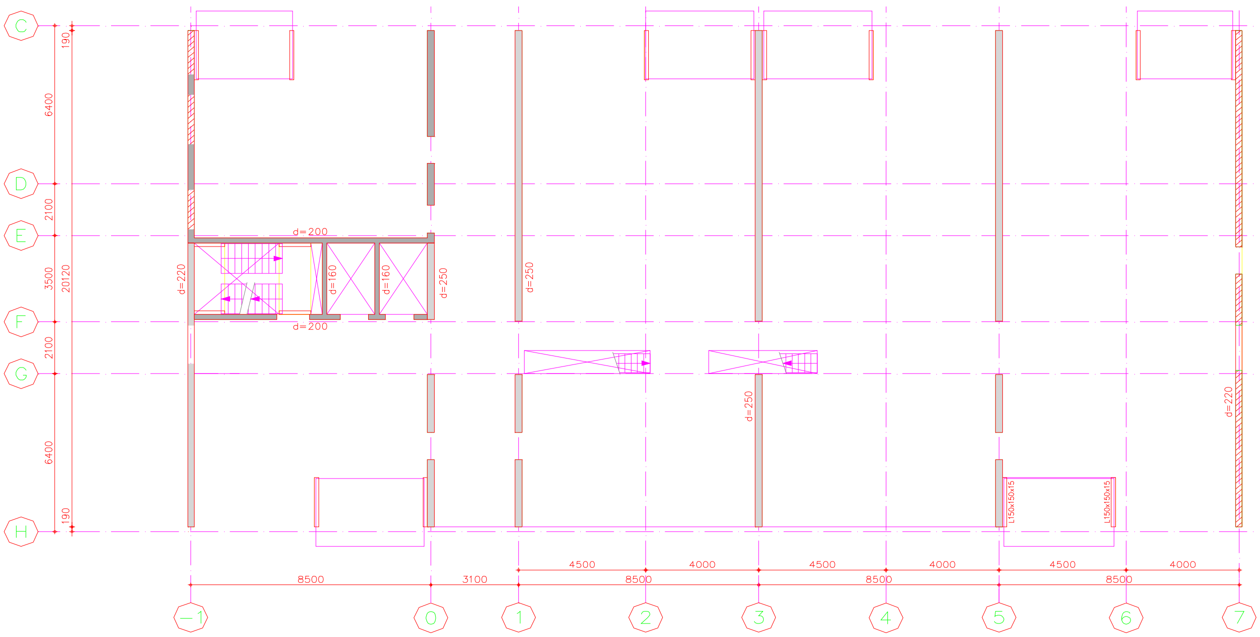
Functional concept

This part of the building has three functions, namely:

- Providing access to both new and existing building and new apartments. It has a concrete core which accommodates the staircase and the lift shaft.
- New rooms are housed in this part
- To stabilize the building



a. Plan of the floor 1 to 6.



a. Plan of the floors 7 to 15.

Figure 6-10: Floor plans for first 6 floor and top floors

Shape

The shape of the building is the same as the wish of the architect. In axes -1 and 0, concrete walls are constructed and from 7th floor, these walls are extended to make cantilevers. As these walls are considerdas deep beams they are stiff enough in their plane and therefore a good method to make cantilevers.
From axis 1 to axis 7, the system may be like to what have been discussed in the first three alternatives. However the double beam system at each floor level seems to be very efficient.

Structural concept.

In previous alternatives, this part between axes -1 and 0 is made by the concrete core of 3,5 x 8,5 m and steel frames.

Handling cantilevers with the presence of concrete core led to the use hanging system (Figure 6-11). This system is not easy to make and requires increasing the building height. The route of forces is so long and then the consumption of steel becomes high.

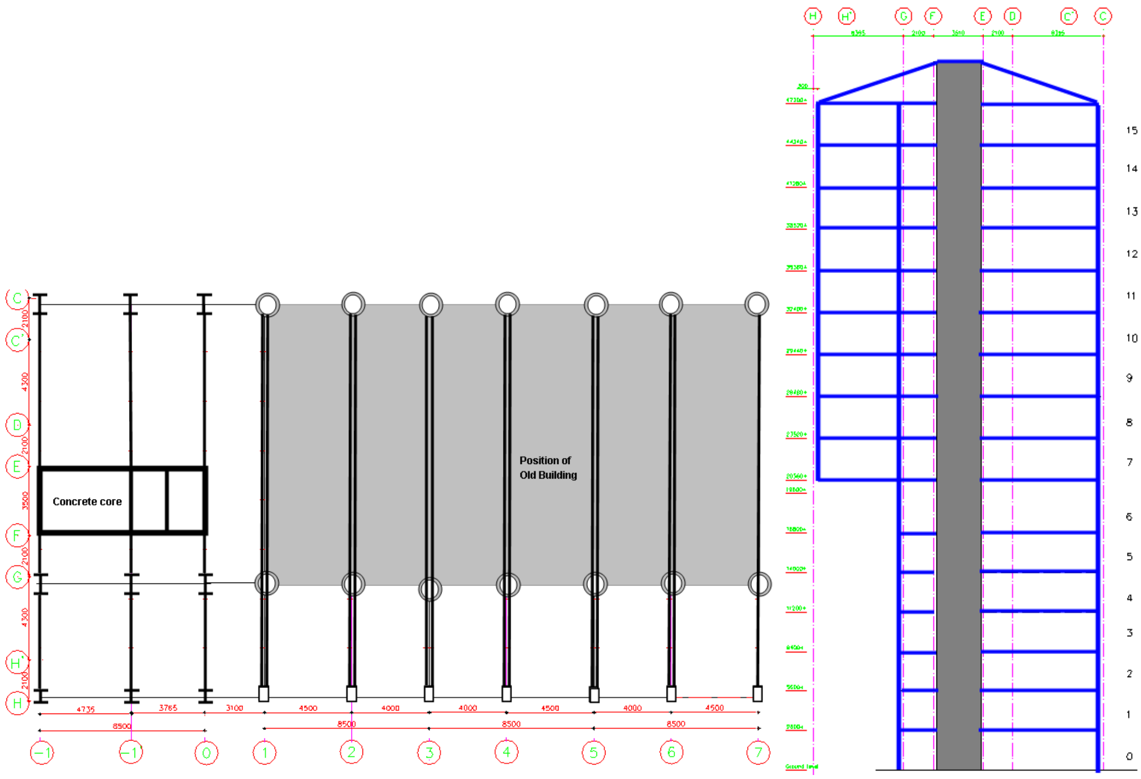
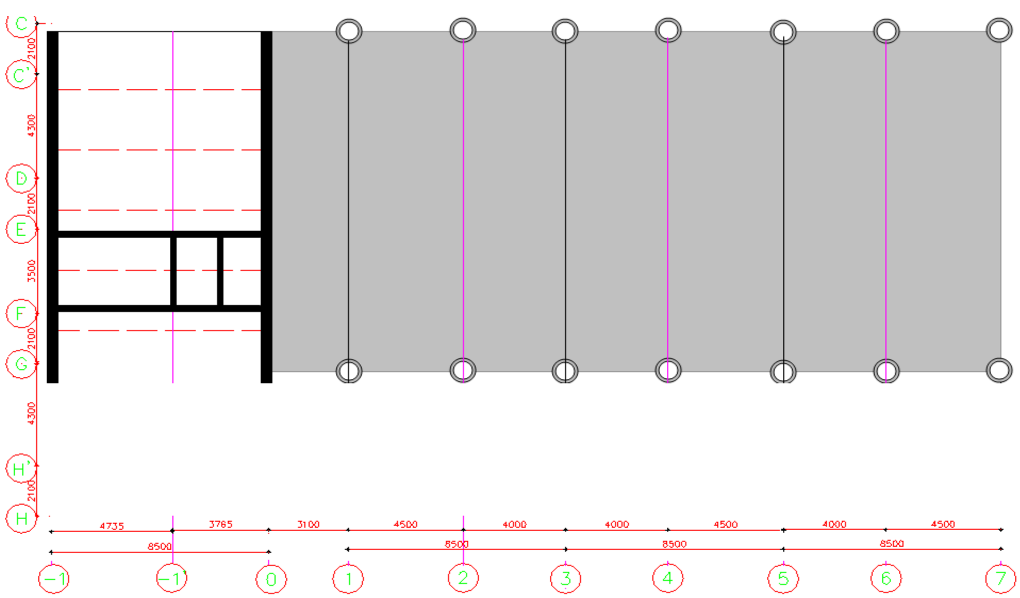


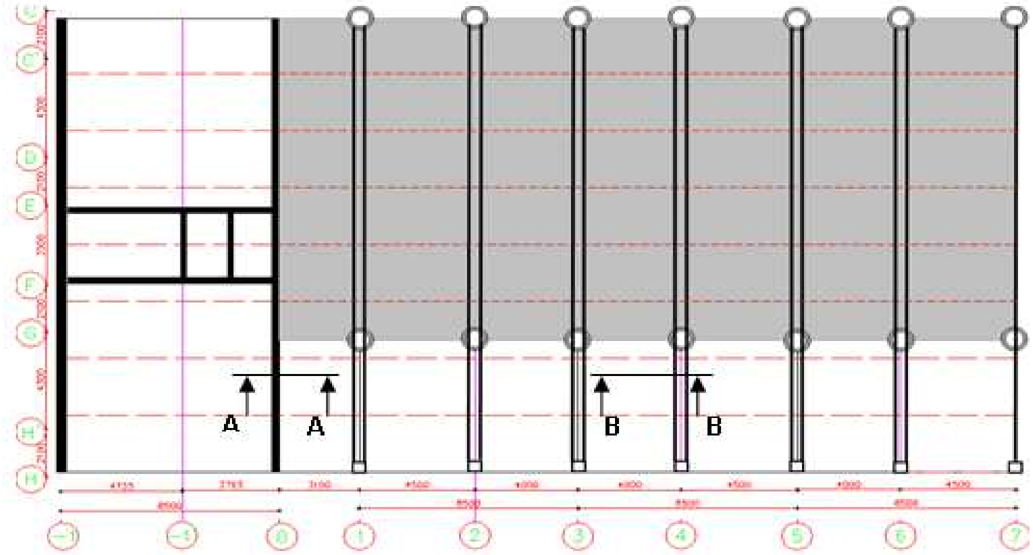
Figure6- 11: Floor system with double girder for floor level 7 to 15 and cross section in axes -1, -1' and 0

The remedy for this issue is to use concrete walls in axis -1 and 0. As these walls have a big number of openings (Figure6-14), they cannot be used to take the horizontal loads; they will only take the vertical loads and providing cantilevers at each floor level from 7th to 15th floor. Horizontal loads will be restrained by the concrete core as well as the braced frames between axes 1 and 7.

The easy way of making the floor system, is by using hollow core slab units which can be simply put on consoles made on walls. (See figure6-12 and 6-13).



a. Floor plan for ground level to floor 6



b .Floor plan for top floors.

Figure 6- 12: Floor plans

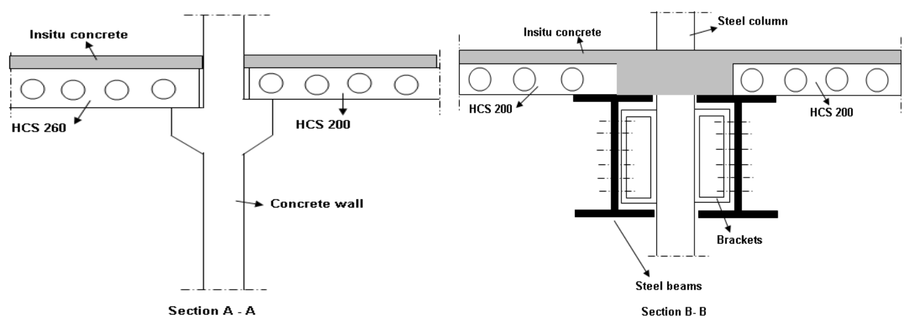
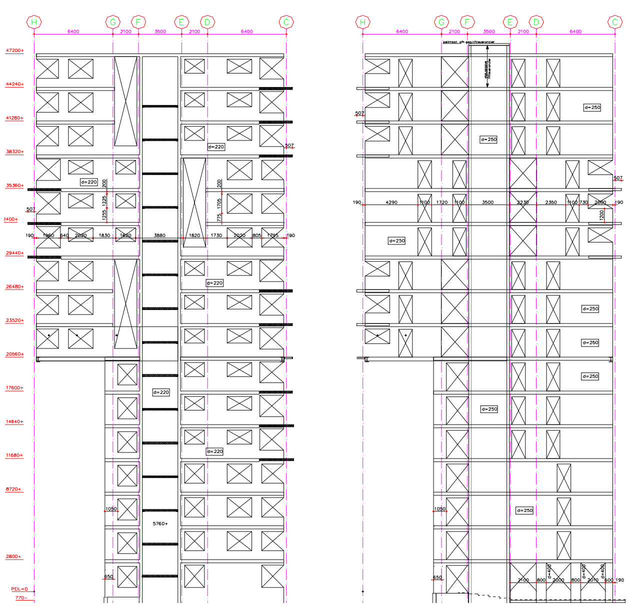


Figure 6- 13: Construction details of the floor system



Wall -1

Wall 0

Figure 6-14: Cross section showing openings in walls

According to VBI [29] , the HCS260 (Hollow core slab of 260mm deep) should be used between axes -1 and 0 since the floor span is closer to 10m , the floor units will be laid on consoles provided on concrete walls. For the remaining part of the building HCS200 will be used, these units will be put on top of the spine beams and finally, the floor units will be joined by the structural topping to make floor diaphragm.

Supporting

The supporting system for this system is the same as what has been used in the first alternatives for the axes 1 to 7, but from axes -1 to 0, floor load are taken by concrete walls to the foundation.

Stability

The major stabilizing member is the concrete core, but braced frames are required on the other side of the building to avoid torsional deformations. The existing building as it is stiff in lateral direction; it can help in stabilizing the new structure.

Force distribution and connection

From axis 1 to axis 7, the force distribution and connections are the same as the force distribution in the first alternatives. In the part with walls, the floor units transfer their loads to the concrete walls which carry them to the foundation. At each floor level the wall can be modelled as a deep beam, very stiff in their plane and cannot easily be deflected when they are cantilevered. The floor units are simply laid on consoles provided on walls.

Advantages

- The resulting structure is more stiff to restrain horizontal forces
- Good method to make cantilevers between axes -1 and 0
- Concrete walls will facilitate making first 6-floors between axes -1 and 0
- No need of hanging system

Disadvantages

- The use of concrete wall limits the flexibility to make future modifications
- Wall are heavy and require more foundation piles

7. EVALUATION OF ALTERNATIVES

Firstly, a short summary about the goals and the difficulties of the elaborated alternatives will be given. After that, the principle of the Multi-Criteria Analysis (MCA) will be described. The selection, based on the rating of the alternatives, will then be made.

7.1 Summary of alternatives

Alternative 1*: Structure with continuous parallel girders*

With a system of continuous parallel girders method, the girders run from one side of the existing building to the other and continue to the cantilevers. The girders are fixed on brackets welded on both side on the column which make the column to be also continuous. The use of hollow core slab system provides less weight which can be carried safely by the girders. The system is easy to erect, the sound as well as heat resistance are achieved due to the concrete layer. The stability, horizontal or/and vertical deflections, large foundation forces from the new structure are the governing factors of the design.

Alternative 2*: Structure with one intermediate support.*

Unlikely the previous system, this time the girders are simply supported because the cantilevers are supported by transfer girders made between 7^{th} and 8^{th} floors. There is no need of using double beam system since the cantilevers are supported and stack on transfer girders. The transfer girder may be in form of Vierendeel girders since oblique struts cause hindrance of some spaces. The fabrication of Vierendeel girder may be too expensive due to its joints capable to resist large moments brought by the top structure.

Alternative 3*: Structure with two or more intermediate supports*

This alternative is close to the previous one, and this also requires a transfer girder to support the entire added structure. The use of one story high transfer girder between 7th and 8th floors allows adding more columns. Then the result is to have moderate sizes of columns and transverse beams. It is also possible to make rigid frame once more columns are erected. Vierendeel transfer girder may also be preferred instead of normal truss.

Alternative 4*: Structure made of concrete walls (axis -1 and 0) and steel frame (axis 1 to 7)*

The sections between axes -1 and 0 in previous alternatives, the gravity loads are transferred to the foundation by hanging systems. But when concrete walls would be used, these walls can make cantilevers at each floor level since they are incorporated to the core no hanging system would be required.

7.2 Decision factors (criteria)

To decide which alternative will be elaborated, various views from different stakeholders (parts interested in the project) have to be analysed. The possible stake holders are the following:

- the architect
- the structural engineer
- the user(occupants)
- the investor
- the contractor
- the Municipality
- the people on the street(passer-by)

Each part will be interested in different elements which have to be taken into account when deciding. There are five main elements that influence the decision namely aesthetical value, efficiency, technical value, functional requirements and costs.

Aesthetical value

Aesthetical value is one of the sensitive points for the final results of the MCA. The architectural design is a key factor when looking at Aesthetic. For the architectural design it is chosen to compare the alternatives with the vision of the architects. The alternatives that are of the same shape of the architectural design therefore have a high score. This point of departure results in the fact that architectural value is based on the opinion of the architect, the municipality the passer- by people. This is the reference point of the architectural value-factor.

Efficiency

To be able to compare the alternatives in such a way that the efficiency of the design according to shape and force distribution, the factor efficiency is added to the criteria. This factor shows that the design seems to be efficient or not and can give a critical view on the architectural design. The parties that are interested in this factor are the structural engineer and, the contractor and to a lesser extend the architect.

Technical value

The technical value the designs is based on the technical difficulties and technical properties of each design. This factor is added to the MCA to be able to compare the technical difficulties of the designs.

Functional requirements

The main point of interest of residential building is the occupants. The structure must meet all the requirements namely thermal and sound comfort, privacy etc. Therefore the functional requirements are of great influence during initiation and design; especially for the engineer, municipality and the .

Costs

The costs of the structures are also an important factor. To estimate these costs is of course very difficult. Based on the insight in cost rates, estimation is made to what extend the alternatives differ in costs and where savings can be made. These estimations are roughly made but give an insight in the rates between the alternatives.
For example the costs of the connections are based on the amount and difficulties of the connections.

7.3 The Multi-Criteria Analysis (MCA)

To make a well founded decision, a Multi-Criteria Analysis (MCA) will be done. The MCA is a tool to get insight in the suitability of the alternatives. This tool is based on the main factors influencing the suitability and gives an insight in the main differences between alternatives. From this MCA a few conclusions can be made. The main advantages and disadvantages of each of the alternatives are displayed in a surveyable way. The main subjects of this MCA are aesthetical value, efficiency, technical value, functional requirements and costs.

These factors are not all of the same importance. To determine this importance it is useful to determine the most important parties influencing the design and their main interests. The passengers for example are not really interested in the technical requirements of the design, but are very interested in the functionality of the design.

In analyzing the views of the possible parties in this project; there was no direct interview held with them , the author put himself in a place of each party order to determine the field of his/her interest.

By using MCA, the members do not have to agree on the relative importance of the Criteria or the rankings of the alternatives. Each member enters his or her own judgments, and makes a distinct, identifiable contribution to a jointly reached conclusion. [21], [26]

Weighting the Scores According to the Weights Assigned to the Criteria

The next step in the MCA process involves 'prioritizing' the criteria by assigning different rankings or weights. The weights can be assigned by the analyst, the decision maker or they can be based on the views of the stakeholders [22]. The importance of a criterion is called a weight; this is a matter of preference of stakeholders involved in the whole process of the project. All these views concerning possible criteria can be summarized in priority matrix (table 7-1). These priorities are ranked in terms of ordinal expression such as better, worse, favorable, less favorable etc. [26]
In the following table (table 7-1), a global interest rating is done. The average value of each factor is calculated to make an estimation of the global interest.

Table 7- 1: Priority matrix according to [26]

Party\Aspect	Aesth. Value	Efficiency	Technical value	Functional value	Costs
Architect	10	7	8	8	4
Structural Engineer	6	10	10	7	7
Municipality	7	6	7	10	10
Users(occupants)	6	0	4	10	7
Passer-by	5	0	0	0	0
Investor	7	6	7	10	10
Contractor	4	8	10	4	7
Average	**6,4**	**5,3**	**6,6**	**7,0**	**6,4**

From this table, the functionality and technical value of the new structure seems to be the most important. The residents comfort, the feasibility or implementation of the structure are the basis of the design. The functionality and technique of the design are therefore important for all parties. The aesthetical design is also a very important factor. For the architect, municipality and the investor it is of great influence. The average weight factors will be used in Appraisal matrix.

The discussed influence-factors can be divided in sub-factors that influence the value of the mentioned influence-factors [27]. The importance of these sub-factors, with regard to the main factors, is rated in the weight factor column.

Each alternative is then rated by giving a value for each sub-factor regarding to the alternative. The higher the value, the better the alternative scores for the sub-factor. It is determined to what extend the alternative “scores” for each sub-factor.

By calculating the average score (multiplied by the value of the main influence factor) the total score for each alternative, regarding to the main factor, can be calculated. By adding up these scores a total score for the design can be determined. [26], [27]

Table 7- 2: Appraisal Matrix according to [26], [27]

MCA	Weight factor	Concept 1		Concept 2		Concept 3		Concept 4	
		with Continuous parallel bea		ure with one intermediate s		with two or more intermediate		with concrete walls in axis	
1. Aesthetical value	**6.4**								
Shape/disturbance	**5**	3	15	3	15	3	15	3	15
Material appearance	**4**	4	16	2	8	4	16	4	16
Height of building	**3**	4	12	2	6	3	9	3	9
Facade appearance	**5**	4	20	3	15	4	20	4	20
Apartments	**3**	3	9	3	9	3	9	3	9
Average score			*14.4*		*10.6*		*13.8*		*13.8*
Total score AD			**92**		**68**		**88**		**88**
2. Efficiency	**5.3**								
Weight of the structure	**3**	2	6	4	12	2	6	4	12
Shape/force distribution	**4**	3	12	4	16	3	12	4	16
Average score			*9.0*		*14.0*		*9.0*		*14.0*
Total score E			**48**		**74**		**48**		**74**
3. Technical	**6.6**								
Ammount of connections	**4**	4	16	2	8	3	12	5	20
connections	**4**	3	12	3	12	3	12	4	16
Foundation construction	**5**	3	15	3	15	3	15	3	15
Loads on foundation	**4**	1	4	4	16	2	8	3	12
Durability	**5**	5	25	4	20	5	25	5	25
Stability	**5**	4	20	3	15	4	20	4	20
Vertical stiffness	**5**	3	15	5	25	4	20	4	20
Horizontal stiffness	**5**	3	15	3	15	3	15	3	15
Service integration	**4**	2	8	4	16	2	8	5	20
Execution/Erection	**4**	3	12	2	8	5	20	4	16
Average score			*14.2*		*15.0*		*15.5*		*17.9*
Total score T			**94**		**99**		**102**		**118**
4. Functional	**7.0**								
Thermal requirements	**5**	5	25	4	20	4	20	5	25
Acoustics requirements	**5**	5	25	4	20	3	15	4	20
Fire safety	**4**	4	16	2	8	3	12	4	16
Apartment layout	**5**	5	25	5	25	5	25	5	25
Access to new building	**3**	4	12	4	12	4	12	4	12
Connecting facilities	**4**	3	12	3	12	3	12	3	12
Average score			*19.2*		*16.2*		*16.0*		*18.3*
Total score F			**134**		**113**		**112**		**128**
5, Costs	**6.4**								
Execution costs	**4**	4	16	3	12	5	20	4	16
Connection costs	**4**	4	16	2	8	4	16	5	20
Material costs	**4**	3	12	4	16	3	12	3	12
Average score			*14.7*		*12.0*		*16.0*		*16.0*
Total score C			**94**		**77**		**102**		**102**
Total score			**462**		**431**		**453**		**511**

The total score of each alternative is sensitive for changes in the valuation of all factors. The difference between alternatives has to be relatively large to be able to tell anything about the suitability of the alternatives.
The differences in these scores look relatively small. Therefore some conclusions can be made by this MCA.

7.4 Conclusion

The final score of the alternatives have to be interpreted with care. The differences among alternatives are relatively small but some alternatives seem to be less suitable for this particular project.

The alternatives based on frameworks with one, two or more intermediate supports do not have a high score (Concept 2 and 3). The technical difficulties, functional requirements and the costs are the most important disadvantages of these alternatives. The major problem is based between axes -1 and 0 where hanging system would be required to transfer the loads to the foundation; hanging system pauses a lot of difficulties and consequently alternatives 2 and 3 would not be chosen.

The alternative whereby, the framework between axes 1 and 7 are made of continuous girders (Concept1). scored also less. The cause is also technical difficult where the concrete core is located. Hanging system would be needed and hence this alternative would not be chosen.

Finally a frame made of continuous steel columns and continuous double beam system in axes 1 to 7 and concrete walls in axes -1 and 0 has a highest score. It is a combination of concept 1(axes 1 to 7) and concrete walls in axes -1 and 0
The high score is due to technical solution of making cantilevers incorporated to the walls.
The bending moments in concrete core reduce and the response to the horizontal load improves.

PART 2: STRUCTURAL DESIGN OF THE SELECTED ALTERNATIVE

8. STABILITY SYSTEM OF THE SELECTED ALTERNATIVE

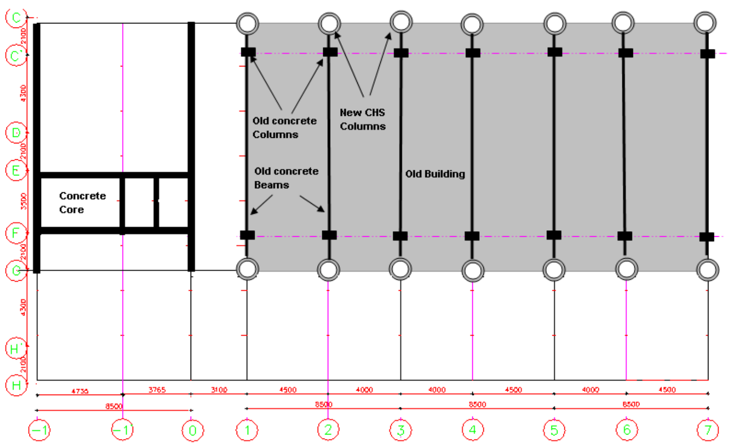
In this chapter, the stability for the selected structure will be designed. The process starts by verifying the capacity of concrete core alone and the core plus additional braced frames, and finally, the optimum method will be selected to stabilise the new structure.

Although the primary function of the structure is to support vertical loads, it must incorporate a lateral stability system to resist horizontal forces, which commonly are wind loads, imperfection and, in some countries, earthquakes. The earthquake loads are disregarded in this project.

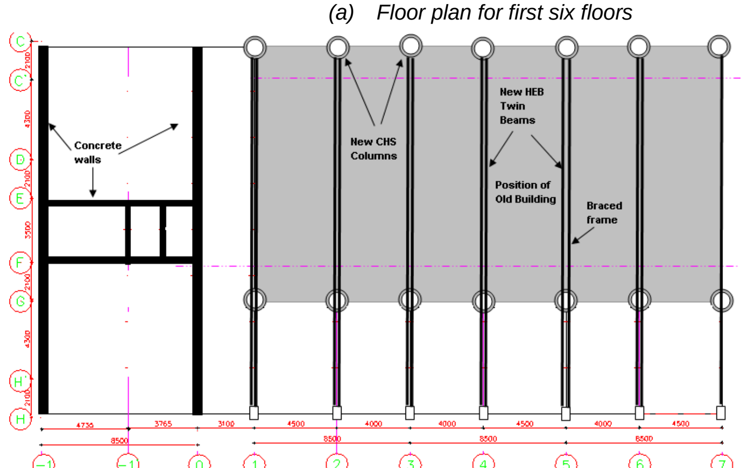
8.1 Stabilizing structures

8.1.1 Concrete Core

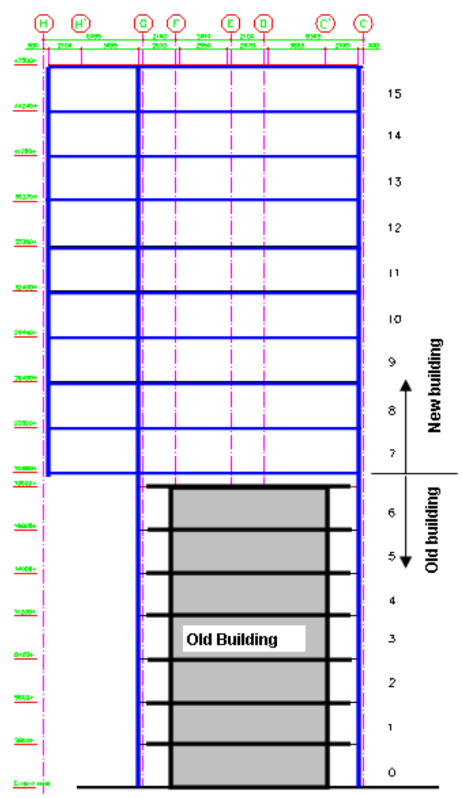
Elevator shafts (cores) are primary components for resisting both horizontal and gravity loading in multistory buildings structures. Reinforced concrete cores, comprise an assembly of connected shear walls forming a box section with openings that may be partially closed by beams or floor slabs. The moment of inertia of reinforced concrete core is large so that it is often adequate in itself to carry the whole of lateral loading.
Usually when stiff cores are used to stabilize the buildings, they should preferably be located in the centre of the building, since it can expand freely in both directions. The torsion stability is then provided by the torsional rigidity of the core. In this project, the central core solution is impossible because of the old building. Therefore the core is built eccentrically in plan (Figure8-1) ; in this concept ,the core is the only one structure which should stabilize the entire building because the frames 1 to 7 old are not connected to the old building(see figure8-1 c)



(a) Floor plan for first six floors



(b) Floor plan for 7th to 15th floor

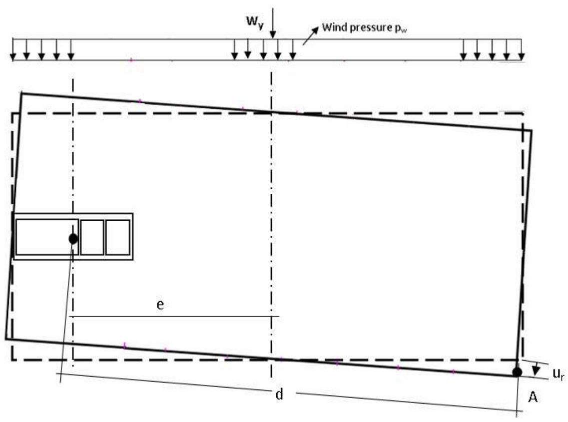


(c) Cross section in axis1 to 7, it shows no connection between

Figure 8- 1: Floor plans and section when only core is used for stability

Distribution of horizontal load over the core

In this case, concrete core is the only one stabilizing element considered. As it is located eccentrically with respect to the application point of horizontal load; there will be both rotation and translation modes of deformation.



FigureA8- 2: Possible movement (translation and rotation) of the building due to lateral load

It has been shown that the deflection on top of the building (opposite side of the core; point A on figure 8-56) can be calculated using the following formula: $u_A = \frac{wh^4}{8EI} + \frac{6}{5}\frac{(w.e)h^2}{EI_t}.d$

Where: d = distance from core center to the point A

e = is the eccentricity of core with respect to the center of rotation

w = horizontal load

h = the height of the building

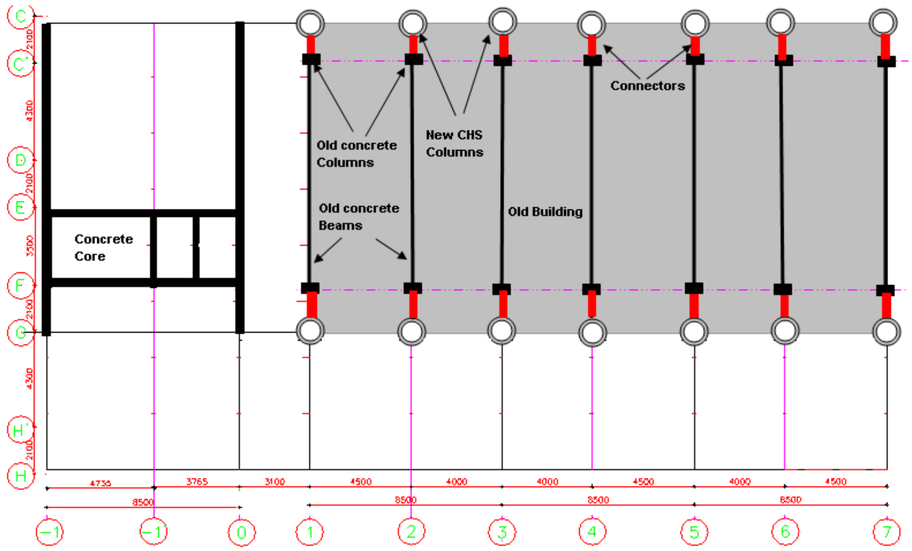
I and I_t = respectively moment of inertia and polar moment of inertia of the core

E = modulus of elasticity of concrete core

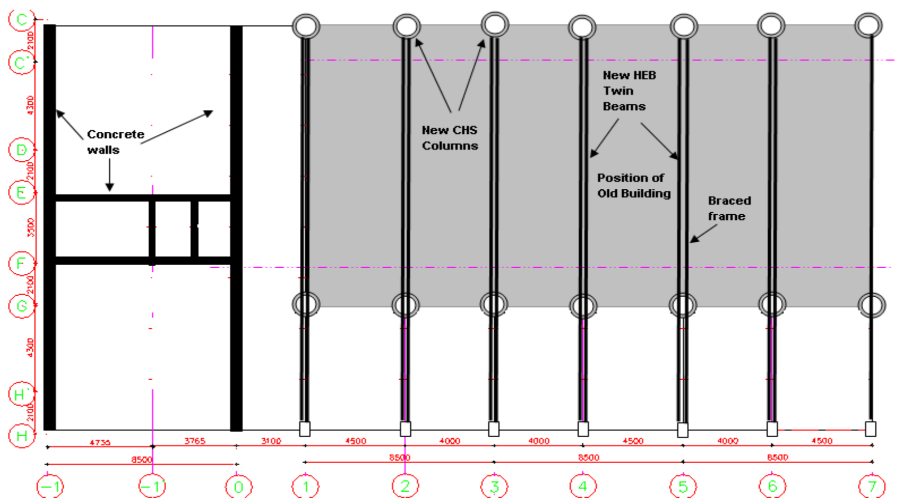
The resulting horizontal deformation is much bigger; the Appendix 7 shows that, the core deflects by 144mm. The attempt to increase the wall sizes of core also gave a deflection of 122 mm. This was due to its eccentricity in floor plan and its big slenderness ratio of $\lambda = \frac{h_{building}}{b_{core}} \Rightarrow \lambda = \frac{47200}{3500} \approx 13{,}5$; therefore some other techniques should be introduced in order to find a better system which can stabilize this building.

8.1.2 Concrete Core with old building

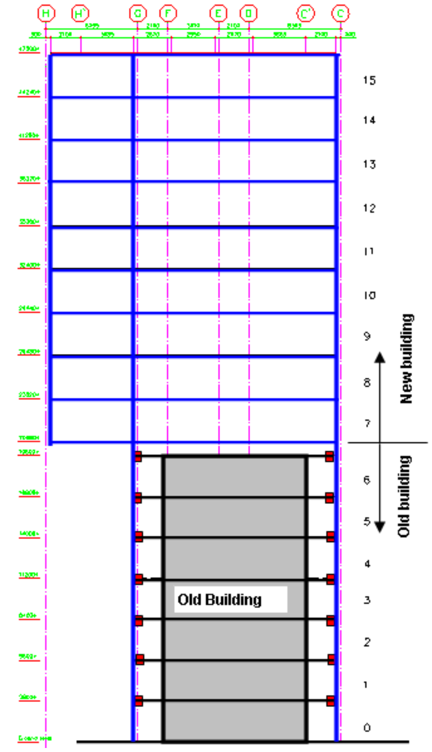
The Appendix 10 (Capacity of old building) shows that the old building is stiff in lateral direction. This property can be used to enhance the stability of the whole structure.
In lower floors, in lateral axes 1 to 7, the stability can be improved by connecting all frames to the old building at every each floor level (Figure 8-3).



(a) Floor plan for first six floors



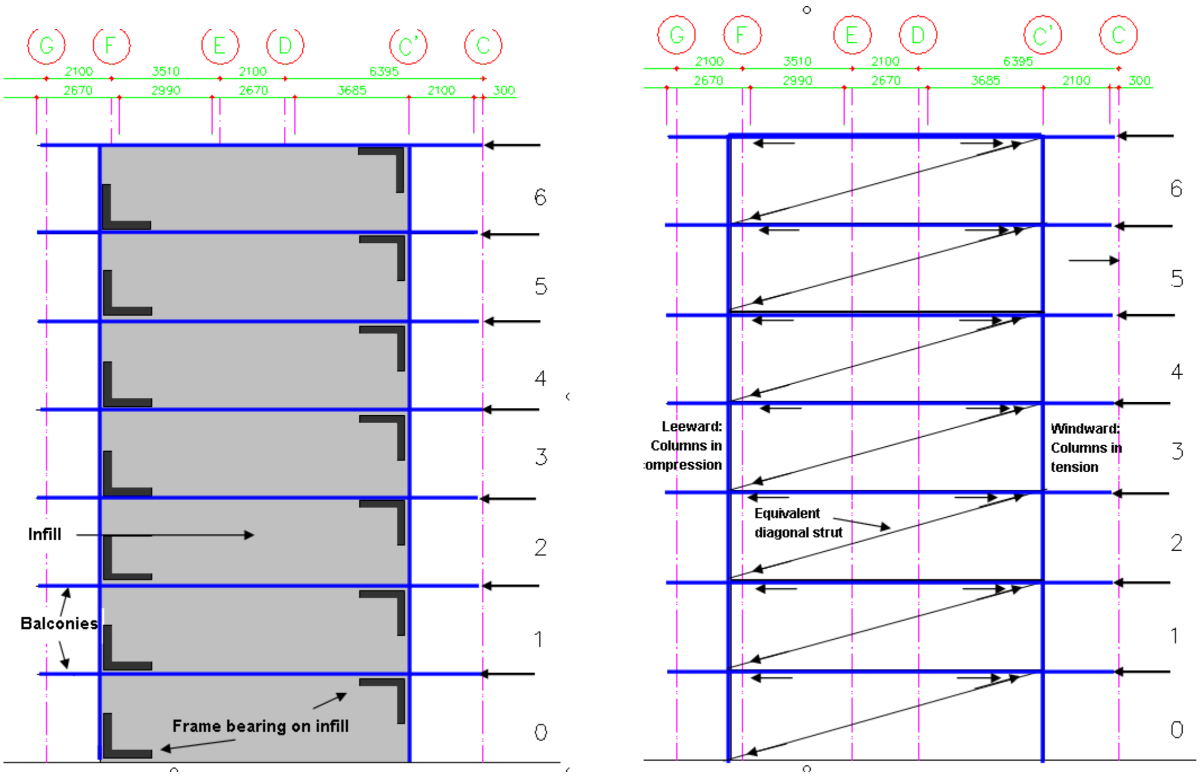
(a) Floor plan for 7th to 15th floor



(c) Cross section 1 to 7

Figure 8- 3: Floor plan showing concrete core and connection of new structure to the old building

As the frames of the old building are filled with the masonry, they behave like braced frames [19], and therefore the set of frame and masonry in-fill contributes to the overall stability of the added structure.
The frame of the old building can be modeled as braced frame with single diagonal [6] (see figure 8-4). The masonry in-fill play a big role in strengthening the old structure and this advantage help to stabilize the new structure. The equivalent area of diagonal representing the in-fill is given by $\frac{1}{10} L.t$, where L is the length of diagonal and" t" is the thickness of wall. [6]



(a) Frame with masonry in-fill *(b) In-fill represented by diagonal strut*

Figure 8-4: Masonry in-fill and its representation in old building

Distribution of horizontal load over stabilizing elements

The horizontal loads are distributed to the concrete core and to the frames of old building in the first 6-floors. As the old building is stiff in lateral direction, the loads restrained by the core is little but from 7^{th} floor upward, the core take the entire horizontal load by itself.

- **Upper part**

In the uper part of the building , the horiontal loads are only restrained by the core. As the core is eccentric, it is bubjected to translation as well as rotations forces.

$$F_{y,transl.;i} = \frac{I_{y,i}}{\sum_{i=1}^{n} I_{y,i}} .W_y \text{ and } F_{y,rotation,i} \frac{I_{y,i}.\overline{x}_i}{\sum_{i=1}^{n}\left(I_{y,i}.\overline{x}_i^{\,2} + I_{x,i}.\overline{y}_i^{\,2}\right)} .M_T$$

$I_{y,i}$ and $I_{x;i}$: moment of inertia of bracing structures i, in Y and X- direction

W_y : Total lateral forces action on floor level

$F_{y,transl.;i}$ and $F_{y,rotation.;i}$: Translation and rotation force acting on bracing structure i

$\overline{x}_i$ and $\overline{y}_i$: Distance from rotation center to structure I, in X and Y – direction.

M_T :Tortional moment $M_T = W_y.e$; where e is the eccentricity

The rotation center coincides with the centroid of the core and then , the eccentricity with respect to the horizontal load becomes 14,3m (Figure8-5).

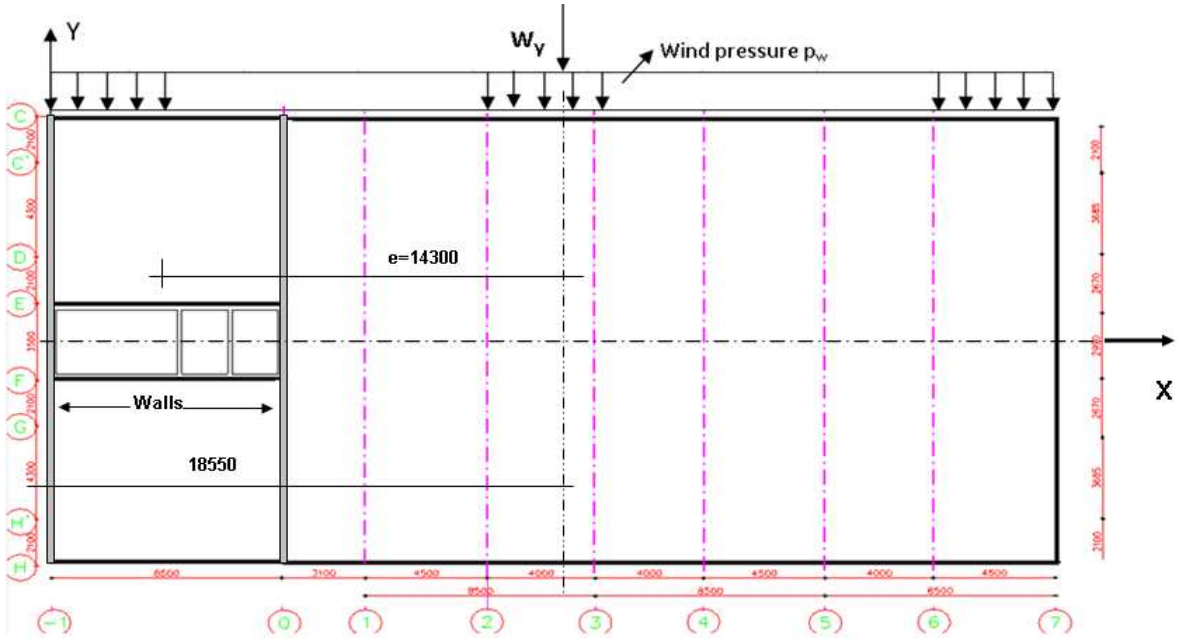


Figure 8-5: Floor plan 7th to 15th showing only the core as bracing structure

The resulting forces in core are enormous (see Appendix7) in such a way that they cannot be restrained by the core only.

- **Lower part**

In the lower part of the building, the horizontal loads are taken by concrete core and existing building .The rotation center goes far from the core and the eccentricity reduces (figure8-6)

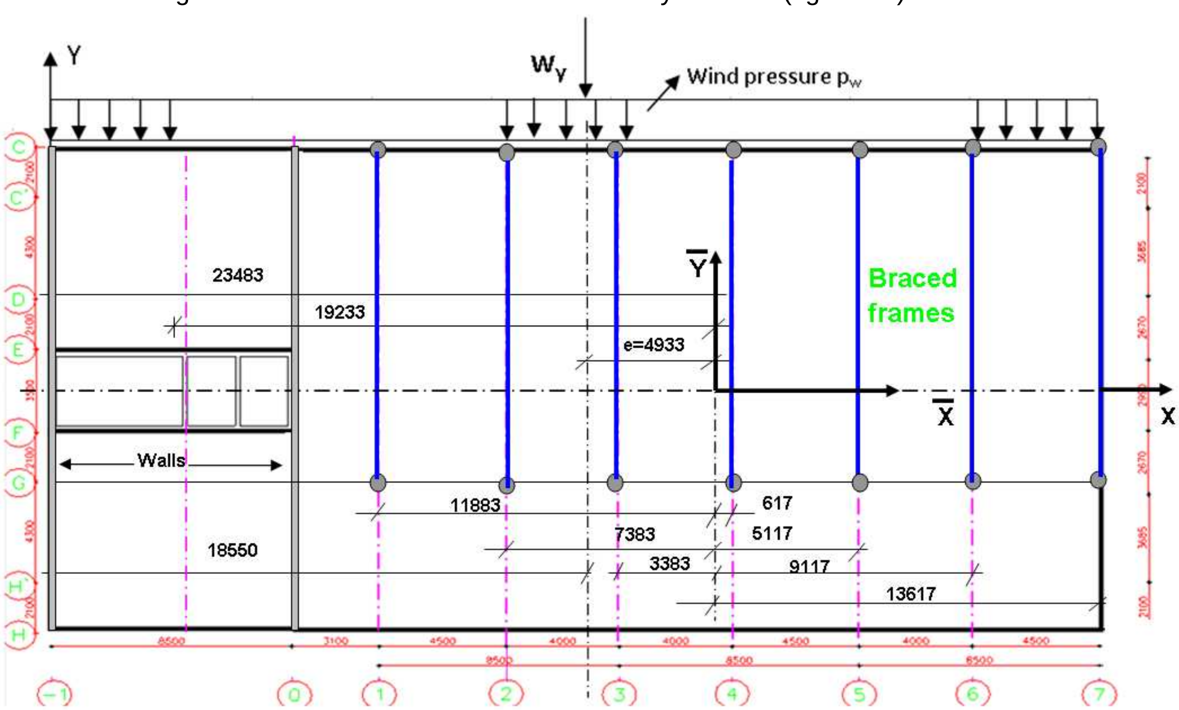


Figure 8-6: Location of rotation centre in plan (core and old building frames frame) in floor 0 to 7

The horizontal loads are shared among frames of old building and to the core. The core in this part is not heavily loaded. The calculations are made in Appendix 7 and the resulting contribution in restraining horizontal loads is shown in the flowing table.

Table 8-1: Forces acting on bracing members

Upper

	Translation KN	Rotation KN	Total force KN	Height m	UDL KN/m
W5	1137	48	1185	26,64	44
Core	881	48	929	26,64	35

Lower

	Translation KN	Rotation KN	Total force KN	Height m	UDL N/m
W1	276	193	469	20.56	23
W2	276	120	396	20.56	19
W3	276	55	331	20.56	16
W4	276	10	286	20.56	14
W5	276	83	359	20.56	17
W6	276	148	424	20.56	21
W7	276	221	497	20.56	24
Core	84	94	178	20.56	9

Conclusion

By connecting, the new building to the old one makes the lower part stiff enough to restrain horizontal loads. However, the upper part is still flexible since only core can take all horizontal loads. Additional structure may be required to help the core to take the lateral loads.

8.1.3 Concrete core with one braced steel frames and old building

As the core is located on one end of the building, it will be subjected to deformation due to translation and rotation; the braced bents are needed to help the core.

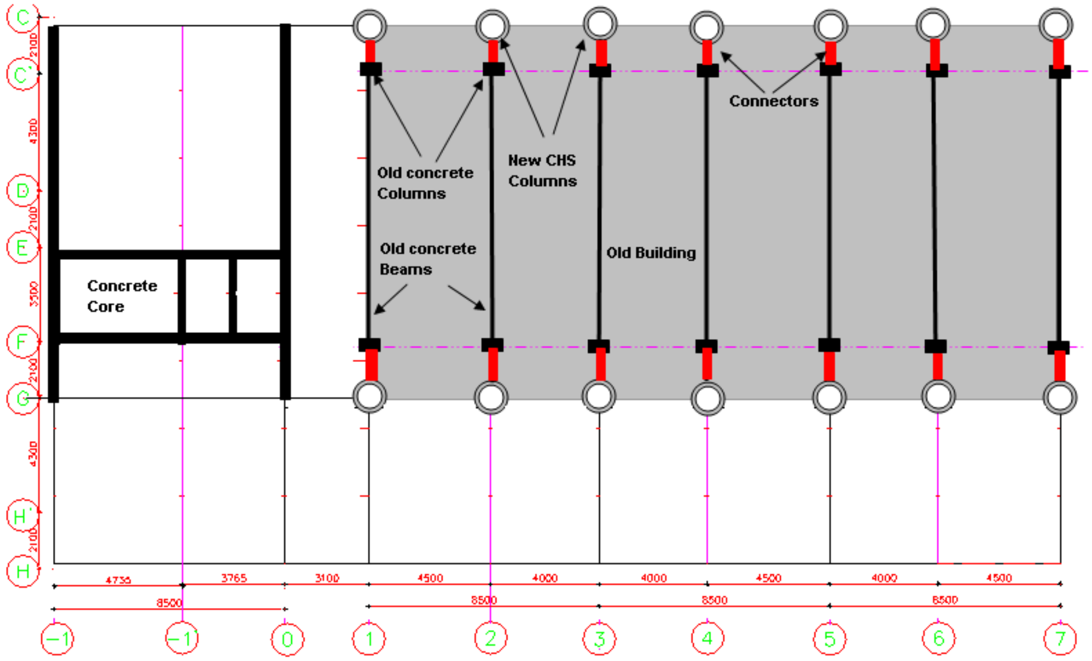
To brace the edge wall (wall at axis 7) as it is suggested by most of the cases is inappropriate since this wall has many openings (Figure8-9). Axis 6 passes through the living rooms, bracing members can cause hindrance. Finally, the wall located in axis 5 will be braced because the bracing elements will not interfere with the other functions of the building (figure8-7and 8-8).

The bracing members are 3-story high and join supporting columns. In cantilever part, due to vertical load bracings are put to take the load directly to the column and deflection is reduced in stabilizing frame.
This method of bracing avoids the weak parts in the frame, it acts as a 3- storey high truss .This will cross the voids provided for corridors and the horizontal forces will be taken to the side supporting columns.

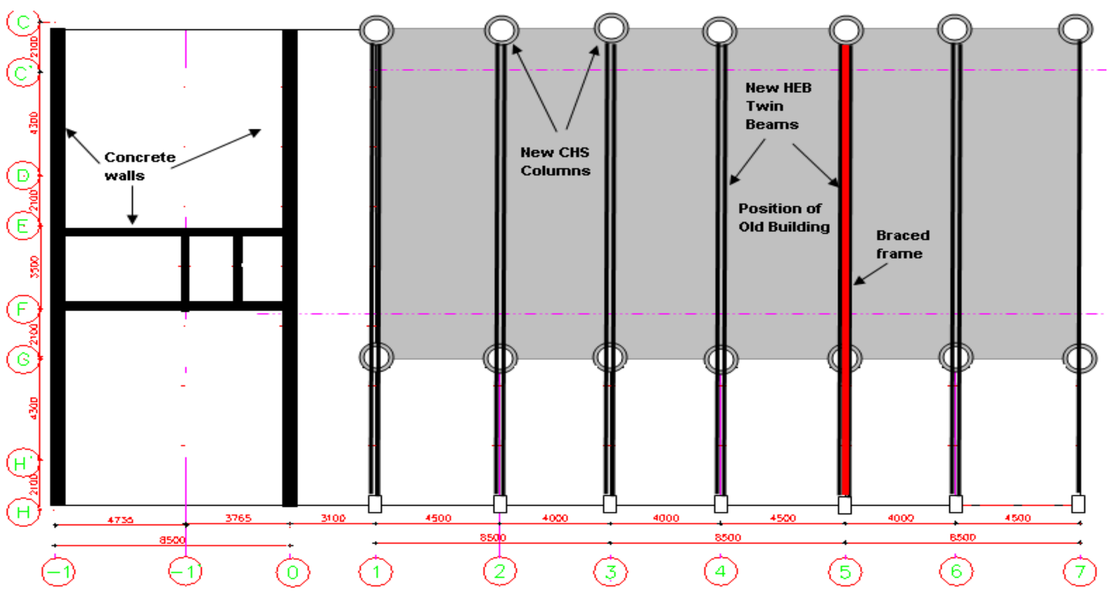
The added structures will have 3-trusses, which are interconnected and arranged in such a way, that the corridors places interchange upwards. Therefore, there is cooperation among members which brace the whole structure. The figure 8-8 shows possible arrangement of bracings and how the openings are interchanged. (See spaces for corridors in green).

In the longitudinal direction, it is not necessary to have the braced bays because the concrete core is stiff enough to resist any kind of horizontal force acting in longitudinal direction.

The frames carrying only the gravity load (axis 1, 2, 3, 4, 6, and 7) will not require additional bracing in the upper structure since the bracing in lower part minimized considerably the overall drift of the building. The lower part must be braced by connecting it to the old building as it has been stated before (Figure 8-8).

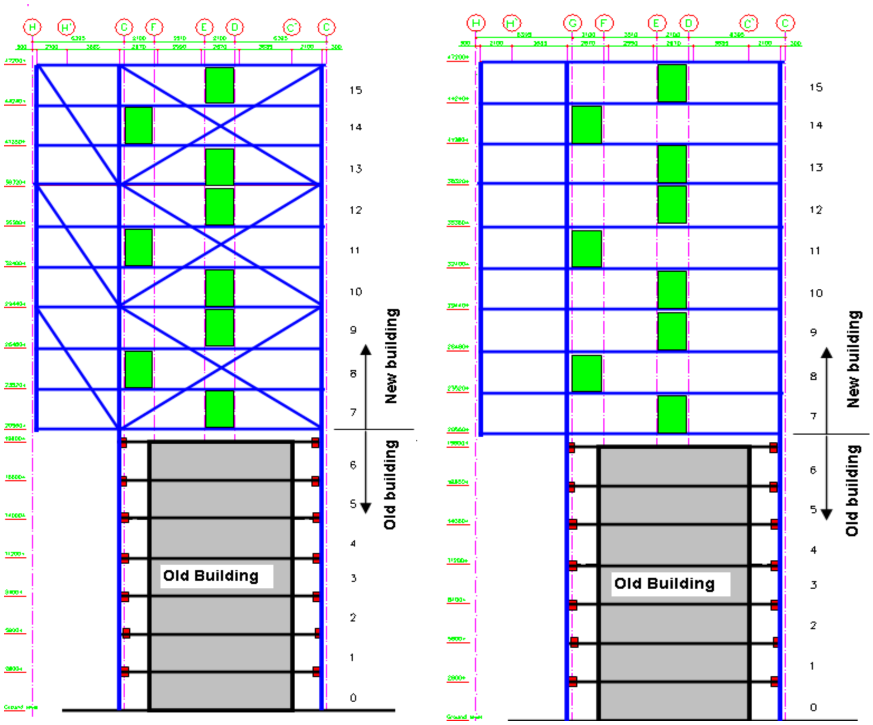


(a). First 6 floors



(b). floor 7 to 15

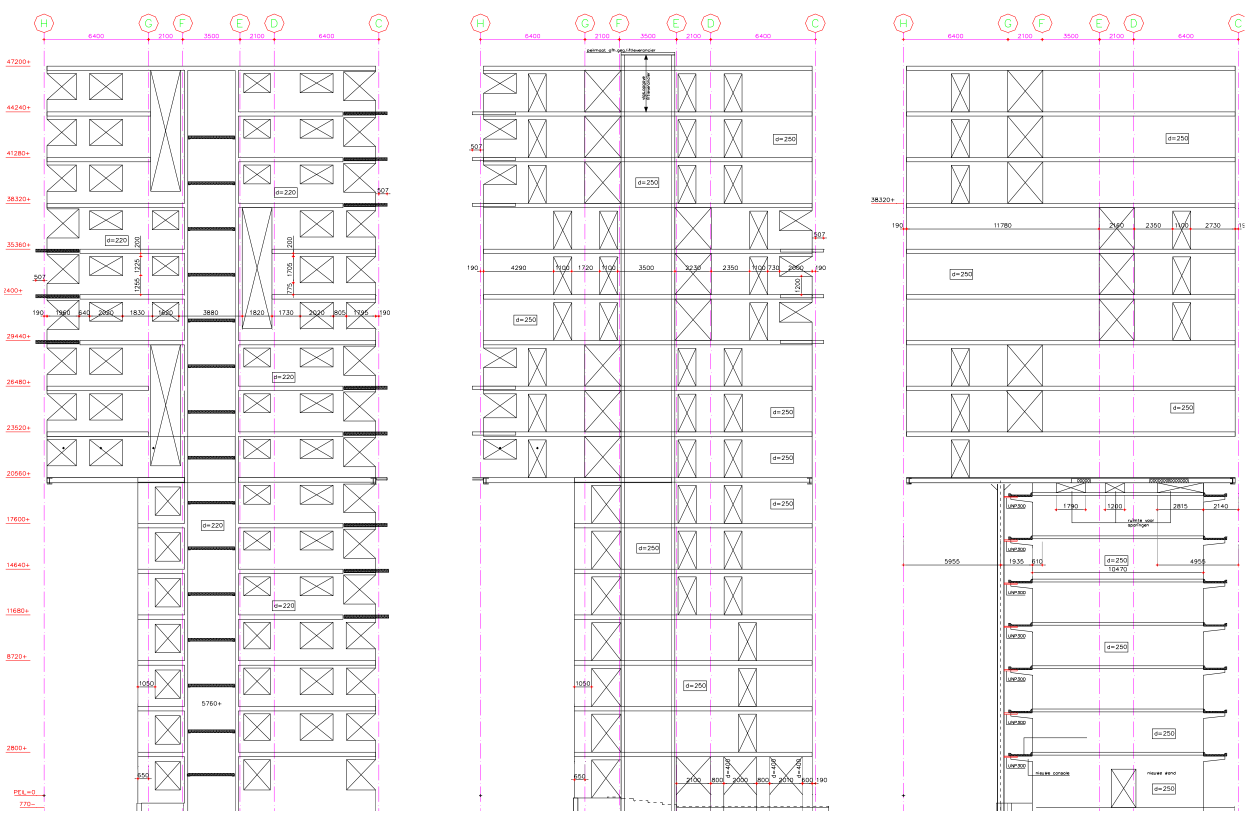
Figure 8- 7: Floor plan showing the braced bays



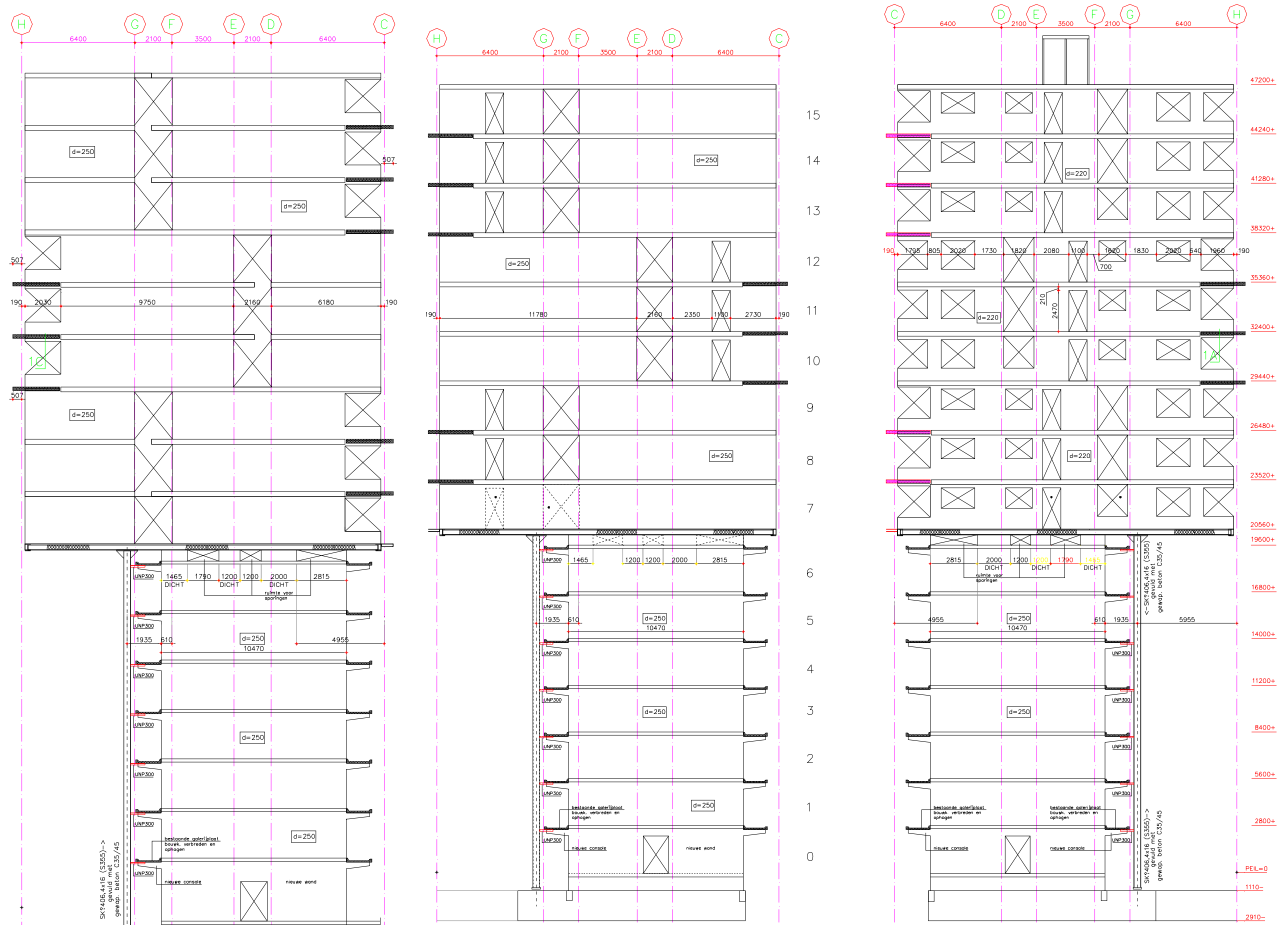
New design in axis 5 *New design in axis 1,2,3,4,6,7*

(c). Cross sections

Figure 8- 8: Location of major openings (corridors) in braced and non braced frames in new design



Wall -1 Wall 0 Wall 1



Wall 3 Wall 5 Wall 7

Figure 8-9: Location of major openings (corridors) in actual design from axes -1 to 7

From Appendix 7, the rotation center is near the application point of the horizontal load(Figure8-10) and the resulting force in bracing structure are shown in table 8-2.

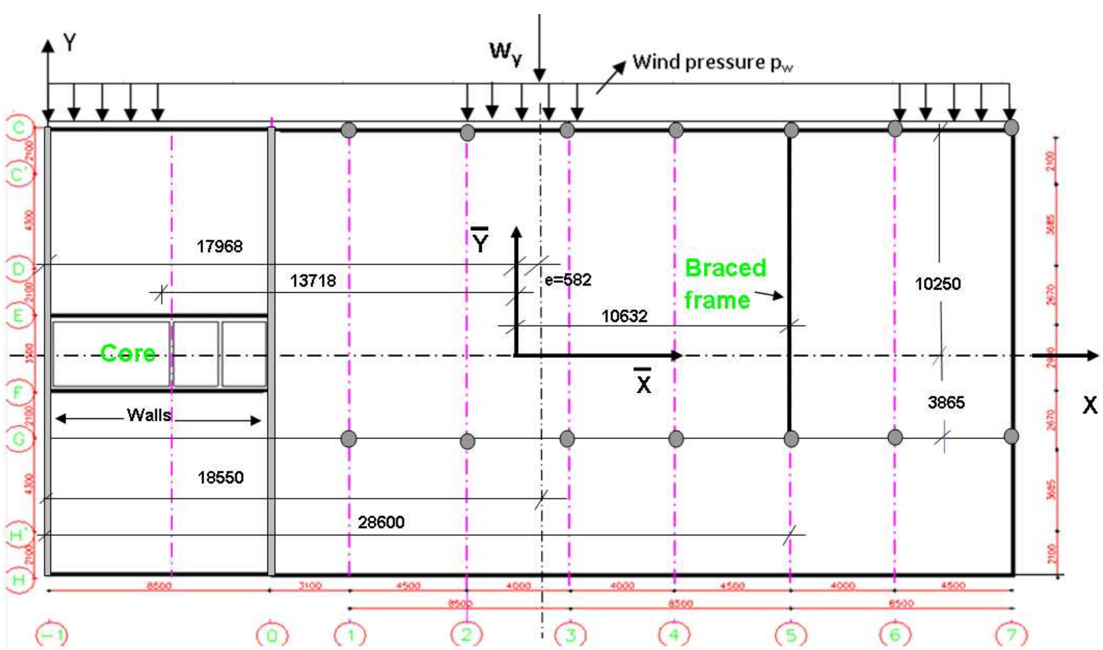


Figure 8-10: Floor plan 7th to 15th showing the core and braced frame 5 as bracing structures

Table 8-2: Forces acting on bracing members

Upper

	Translation KN	Rotation KN	Total force KN	Height m	UDL KN/m
W5	1137	48	1185	26,64	44
Core	881	48	929	26,64	35

Lower

	Translation KN	Rotation KN	Total force KN	Height m	UDL N/m
W1	276	193	469	20.56	23
W2	276	120	396	20.56	19
W3	276	55	331	20.56	16
W4	276	10	286	20.56	14
W5	276	83	359	20.56	17
W6	276	148	424	20.56	21
W7	276	221	497	20.56	24
Core	84	94	178	20.56	9

Conclusion

In this system, the contribution of the braced frame is big which may lead to further deflection. To reduce individual contribution of one braced frame, two similar braced frames can be added in axes 1 and 3 (Figure 8-11). Their cross sections are similar to that of frame in axis five (see Figure 8-7)

8.1.4 Concrete core with three steel frames and old building

The lower part is unchanged, only the upper part receive two additional braced frames to share horizontal load with the core and the frame previously located in axis 5.(Figure8-7) In these axes, there is no interference between structural members and the building function.

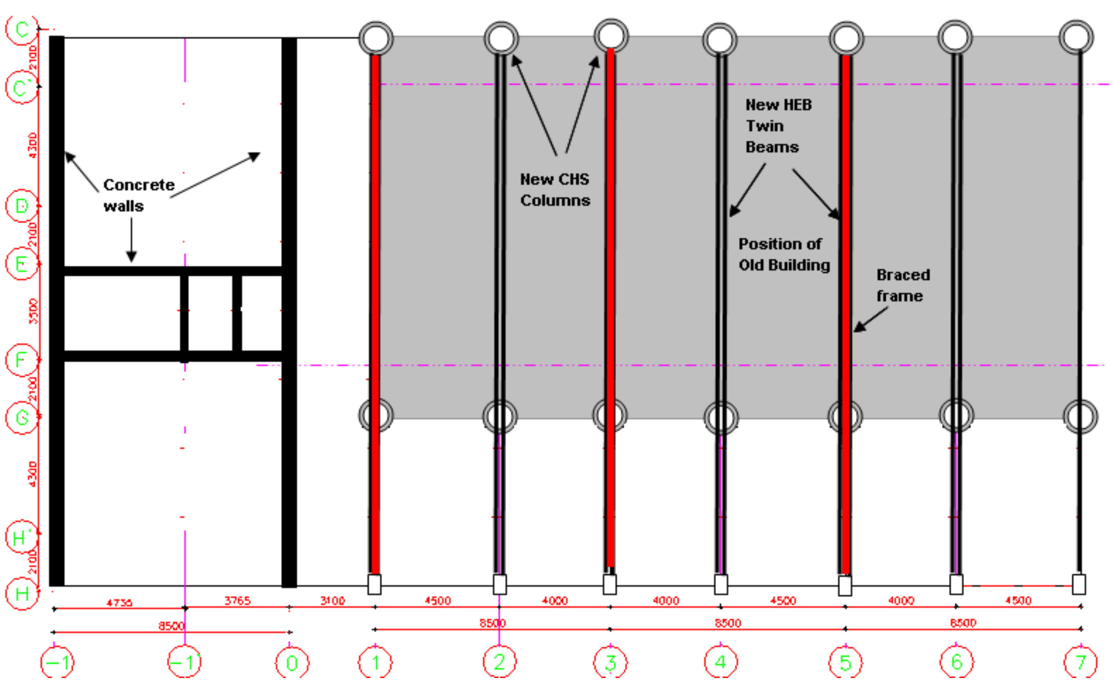


Figure 8-11: Floor plan showing concrete core and 3 braced frames (floor7th to 15th floor)

From Appendix 7, one can see that the eccentricity is small (Figure8-11)and an other advantage is the reduction in forces applied to these bracing structures(table8-3)

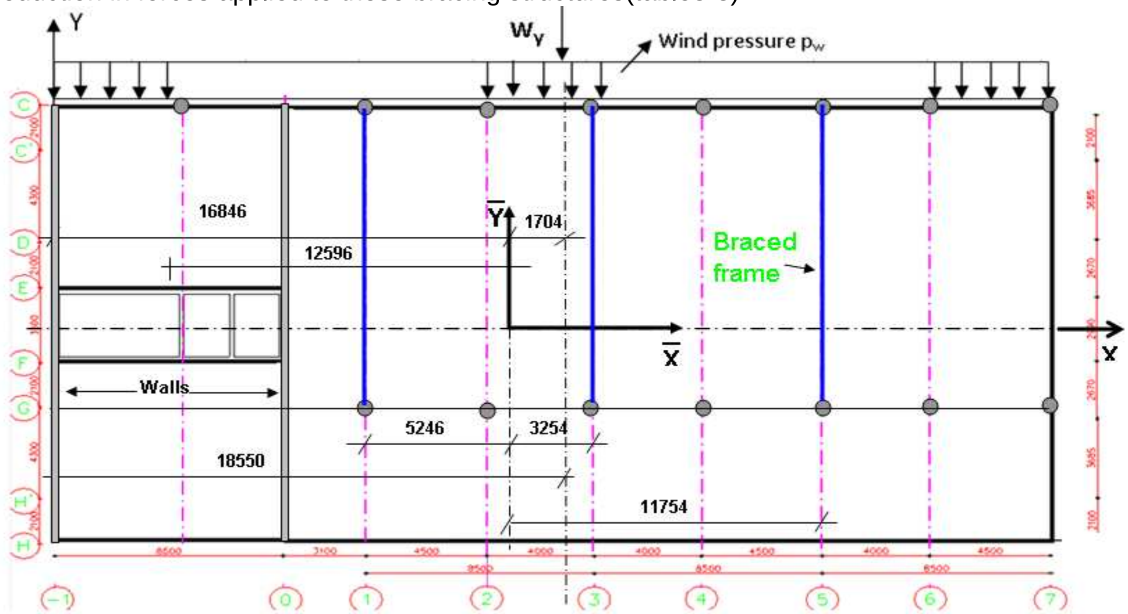


Figure 8-11: Floor plan 7th to 15th showing the core and 3-braced frames as bracing structures

Table 8-3: Total forces acting on bracing members

Upper part

	Translation KN	Rotation KN	Total force KN	Height m	UDL N/m
W1	535	60	595	26,64	22
W3	535	37	572	26,64	21
W5	535	135	670	26,64	25
Core	414	112	526	26,64	20

Lower part

The characteristics of the lower part is the same as what obtained in previous option (see A7.4)

	Translation KN	Rotation KN	Total force KN	Height m	UDL N/m
W1	276	193	469	20.56	23
W2	276	120	396	20.56	19
W3	276	55	331	20.56	16
W4	276	10	286	20.56	14
W5	276	83	359	20.56	17
W6	276	148	424	20.56	21
W7	276	221	497	20.56	24
Core	84	94	178	20.56	9

The calculations shown in Appendix 7 and 8, show that the deflections reduce very much in such a way that the deflection of the core is 3 mm, whereas the steel frame deflect by 39 mm.

8.1.5 Conclusion

Concrete core alone cannot satisfy the limitation of horizontal deflection. The good results are abstained when it works together with old building and 3-braced frames. This structure will be used in further design.

9. DESIGN OF STRUCTURAL ELEMENTS OF THE FINAL STRUCTURE

In this Chapter, the main structural members will be described. This elaboration has been done to show various details such as columns, beams, bracings and floor elements.

Goal: The goal was to get an insight the behaviour of the new frame in order to design different structural members.

9.1 Column design

The sizes of columns are estimated based on the maximum vertical load that they carry. By the use of rules of thumb [10], the column sections in each part of the building can be obtained. In appendix 4, the vertical loads were calculated for each column in order to estimate the column section.

However, the steel columns designed in appendix 4 have relatively large dimensions and are not protected against fire. The way of reducing their size by keeping or enhancing the capacity and fire protection is by using composite columns. The design is made according to Euro Code 4 [5] and the Structural Hollow Sections (SHS) filled with concrete were selected because they are the most efficient of all structural steel sections in resisting compression. It is also a method of fire protection of the steel sections. Table9-1 and figure 9-1 show the composite column as new approach to reduce the steel section as it was calculated in appendix 5.b. The he edge columns will not be filled with concrete to avoid overweight, they will be insulated for fore proofing. The reinforcement used are as follow:

- For CHS 406,4 x8 $8\phi20$ should be used
- For CHS 457 x16 $6\phi25$ should be used

Table 9 - 2: Composite columns with required longitudinal reinforcement steel

Axis	1	2	3	4	5	6	7
C	CHS 406,4 x 8	CHS 406,4 x 8	CHS 406,4 x 8	CHS 406,4 x 8	CHS 406,4 x 8	CHS 406,4 x 8	CHS 406,4 x 8
G	CHS 457x16	CHS 457x16	CHS 457x16	CHS 457x16	CHS 457x16	CHS 457x16	CHS 457x16
H	RHS 200x150x5	RHS 200x150x5	RHS 200x150x5	RHS 200x150x5	RHS 200x150x5	RHS 200x150x5	RHS 200x150x5

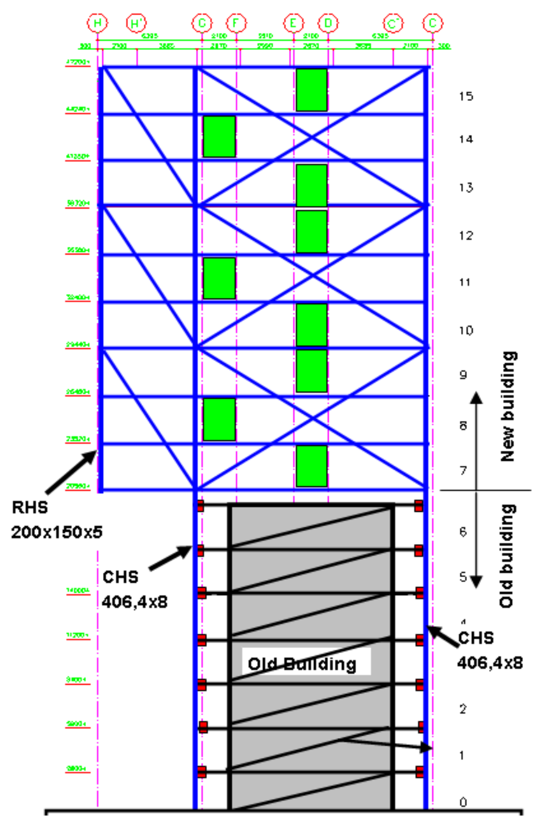


Figure 9 - 1: Section in axis 1, 3, 5 showing column sizes

9.2 Spine beams design

The spine beams run in lateral direction of building and are loaded with Uniform distributed load from floor units, the average spacing between spine beams is 4,25m. Two parallel beams are used and are supported by the brackets welded on both side of the column.
For beam design the following have to be considered:

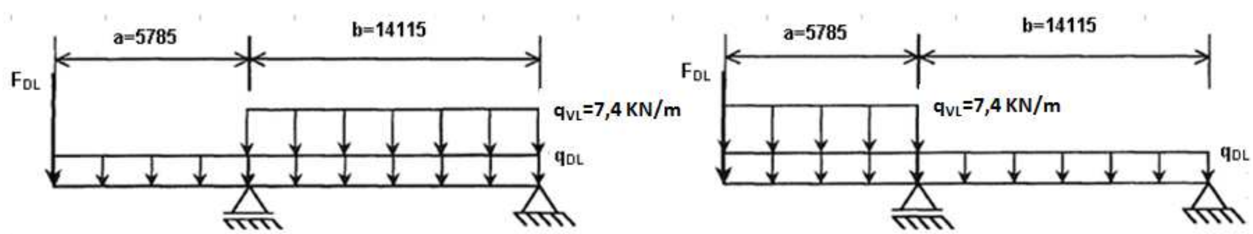
(i). Ultimate limit state on critical span: strength
(ii). Serviceability limit state; Deflection

Loading: Dead load: 7, 14/ m^2
Variable load: 1, 75 KN/m^2

Table9- 2: Load on spine beam

	Load [KN/m^2]	Beam spacing[m]	UDL [KN/m]	Point load [KN]	Design load SLS [KN/m]	Design load ULS [KN/m]	
						Factor	Load
DL	7,14	4,25	30,35		67,3	1,2	81
LL	1,75	4,25	7,44		18,6	1,5	28
Façade	2KN/m		4,25	8,5KN	8,5KN	1,2	10,2KN

From Appendix 3 and 6, the beam can be designed after checking the possible deflection in mid-span by varying the position of variable loads. Two loading condition were established (Figure 10-2) and it has been obtained that the high bending stiffness is needed when the live loads are placed between supports.



$$I = \frac{-2,732.10^{10}}{\delta_{Cant}} \qquad I = \frac{5,8.10^{10}}{\delta_{span}} \qquad I = \frac{2,032.10^{10}}{\delta_{Cant}} \qquad I = \frac{3,27.10^{10}}{\delta_{span}}$$

Figure9-2: Continuous beam model

From Appendix 3, one can see that the beam section was selected by limiting the deflection to 10mm[24] and therefore two beam sections of HE700M would be used in parallel for all internal frames (axis 1 to 6).

9.3 Bracing design

Along the old building, from axis 1 to 7, the new building is supported by tall columns of about 21m high from the ground level (Figure 10-1). It results in relative slenderness λ_{rel} of 3,7 whilst the limit given by the EC4 is 2; this leads to instability of those tall columns.

To enhance the stability or reducing the columns buckling length of the newly added stories, the columns should be connected to the frame of the existing building at every floor level [12].

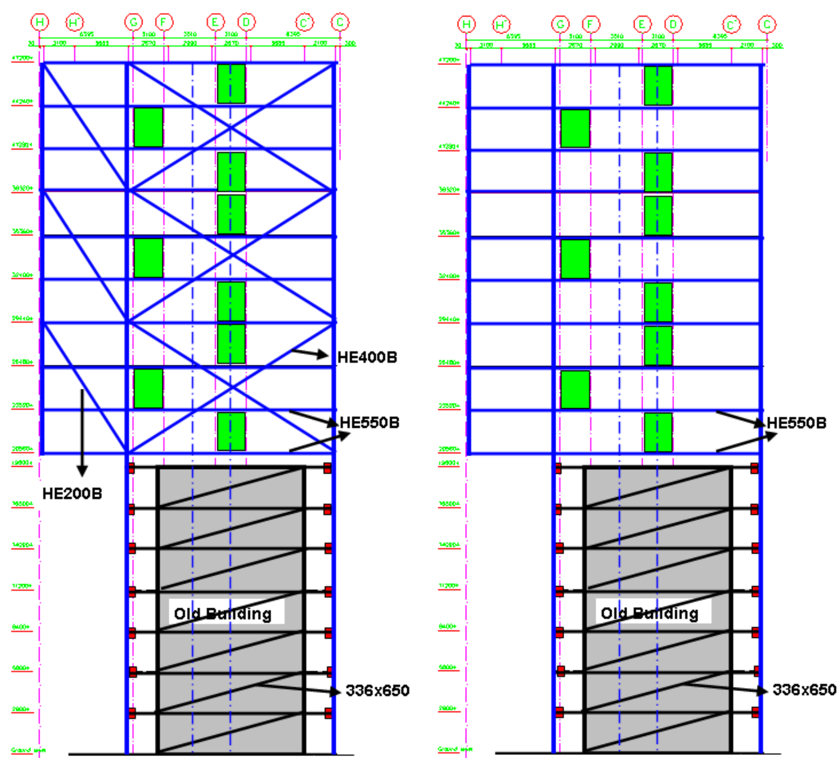
The connectors of columns to the old building as well as cross bracing in upper floors are dimensioned according to the shear force caused by lateral loads. From Appendix 8, it has bees selected to use two channel section UPN 220x80 on both side of concrete beam of old building. These channels are connected to the new column's brackets by bolts.

The in- fill inside the frame of the existing building has a big lateral stiffness and can be used to carry lateral loads and to reduce the buckling length of columns. Therefore, In lateral direction up to the 7th floor (from axis 1 to 7), the steel frames of the new structure can be stabilized by the old building (Figure 9-2). The in-fill will be modeled as single bracing in each floor level (Figure 9-3) with its equivalent area equal to

$A_d = \frac{1}{10} L_d .t$ [6]. During calculation, an equivalent concrete section of 650 x 336 mm was used.

Where A_d is area, L_d is the diagonal length and t is wall thickness.

In braced frames, from 7th to 15th floor the in made by HE400B as it was calculated in Appendix 8. In order to reduce the cantilever deflection of the beams in the frame restraining lateral loads, the cantilevers part should be braced by 3-strorey bracings of HE200B.(Figure 9-3).



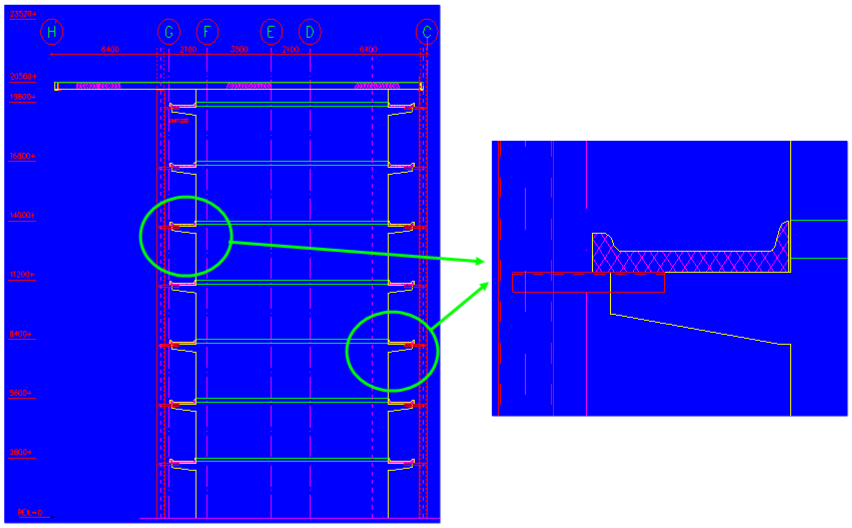
(a). Lateral section in axis1,3, 5   (b) Lateral section in axis 2,4,6 and 7

Figure 9- 3: Lateral Connection of supporting columns to the existing building (frames in axes 1 to 7)

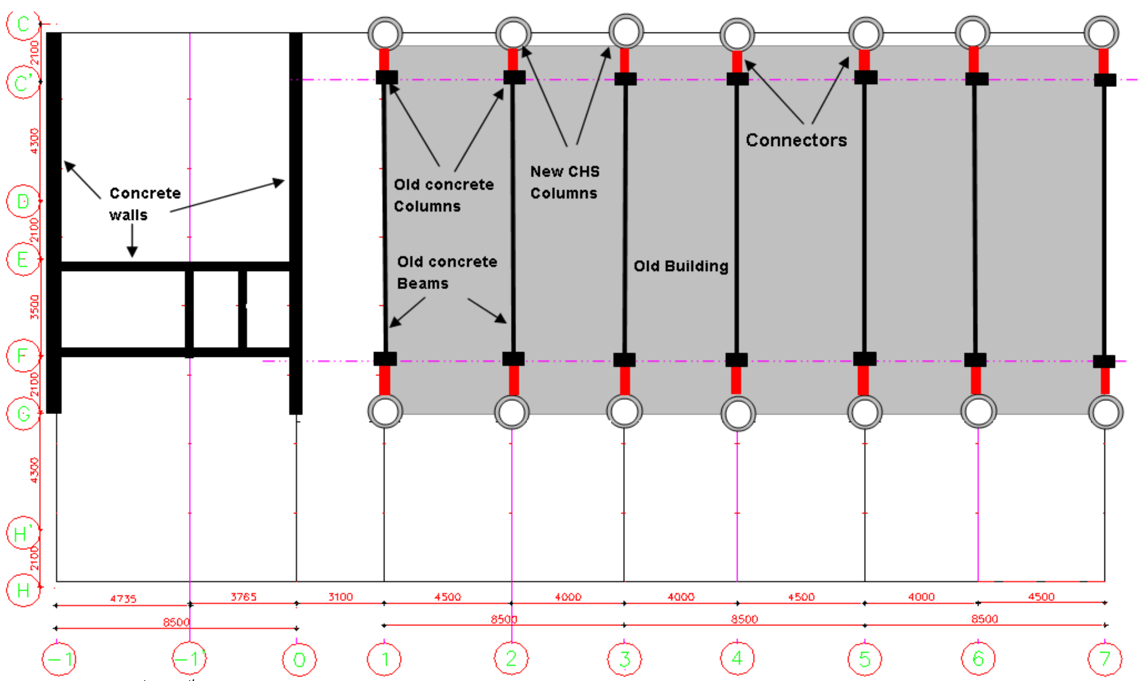
9.4 Important connections

9.4.1 Connection of columns to the old building

This is the major connection since it determines the stability of the new building. The columns are connected to the old building by using twin UPN 220x80 profiles bolted on both sides of the cantilever beam carrying the balcony(of the old building) to the column of the new structure(Figure9-4).



(a). Section in Axis 1,2,3,4,5,6 and 7



(b) Plan from 1st to 6th floor

Figure 9- 4: Bracing the new structure by using old building from 1st to 6th floor

9.4.2 Connection of beams to columns

The parallel beams should be connected to the column, but this task is not easy compared to the situation where H- columns are used. Two simple brackets are welded on both sides of the column and the beams are bolted to their edges(Figure 9-5).[29]

Figure 9- 5: Connection of parallel beams to H-column.

In case of CHS, special bracket must be welded on both side of column to attach beams to column. The beam's webs should be bolted to those brackets which transmit the load from beam to columns(Figure9-6). The brackets should be sized depending on shear force acting in it.

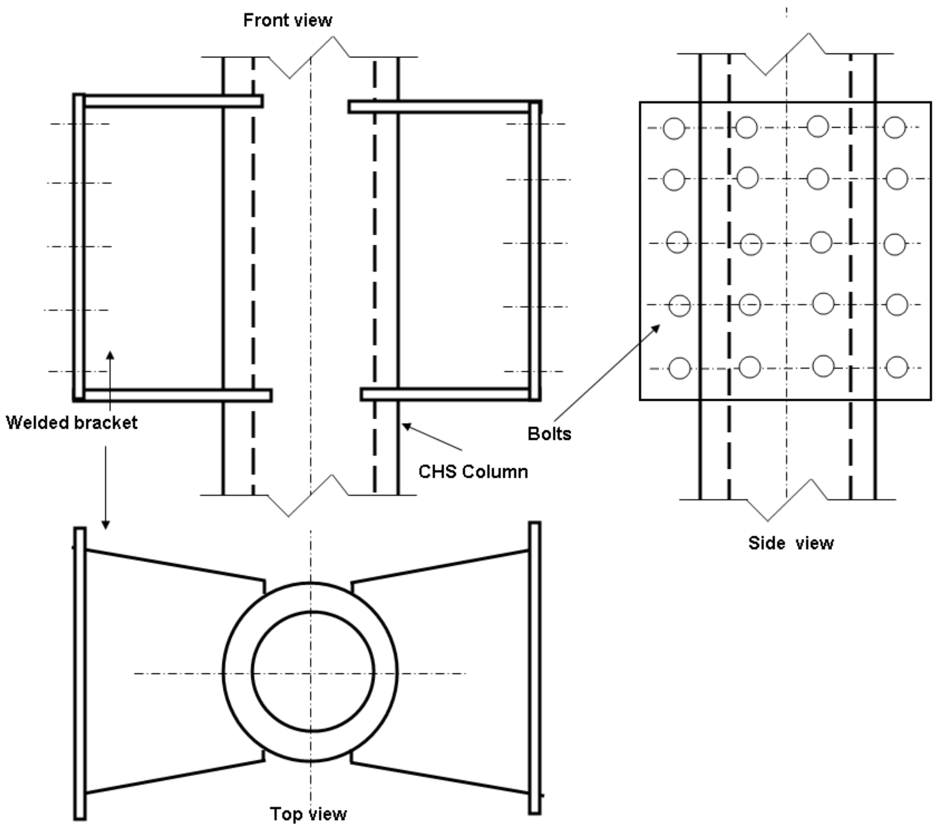


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Figure 9- 6: Connection of parallel beams to CHS-column.

9.4.3 Connection among beams

The spine beams are placed laterally so that they pass beside the columns thus avoiding an intersection with the columns. For internal frames, parallel beams are used whereas, in external frames single beam are used. Spine beams are placed on both sides of the column with 20-40mm gap between the face of the column and the edge of the spine flange. [8]
A diaphragm beam (similar to spine beams) must be bolted between spine beams, at regular interval to avoid lateral buckling of beams or for restraining the bottom flanges of two parallel beams. (Figure9-7). The top flanges of the spines are braced together and held in position by the hollow core slabs.

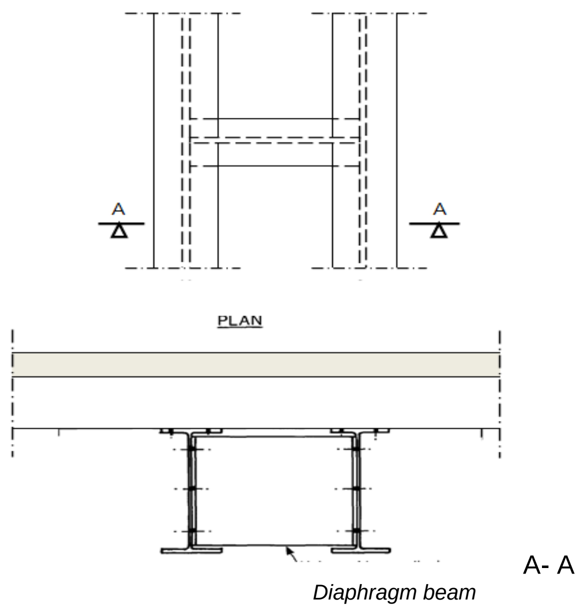


Figure 9- 7: Connection between parallel beams

9.5 Fabrication and erection of steel structure

The erection of steel structure starts a little bit after starting erection of concrete core because the steel structure must be stabilized immediately by the core as well as the existing building.
With a large reduction in number of lifts and increased repetition, combined with a system in which beams are landed on rather than suspended between supports, erection is much simpler and faster. Alignment of bolt holes can be safely achieved.
End plates and beam notches are no longer required; beam lengths now relate to grid centerline, as opposed to the distance between the faces of column or beams. Beams lengths are therefore the same even though the supporting columns or beam section may have changed. Fabrication is simplified and repetition of members increases. The external and internal spine beams should have the same section which increases the degree of repetition.

9.6 Lateral stability of frames and beams

Lateral stability in both horizontal directions (X and Y) is provided by the floor units after being connected to each other by concrete topping. The floor units transmit the horizontal shear forces to the vertical bracing and then to the concrete core and braced frames.

10. PRELIMINARY ANALYSIS OF THE FINAL STRUCTURE

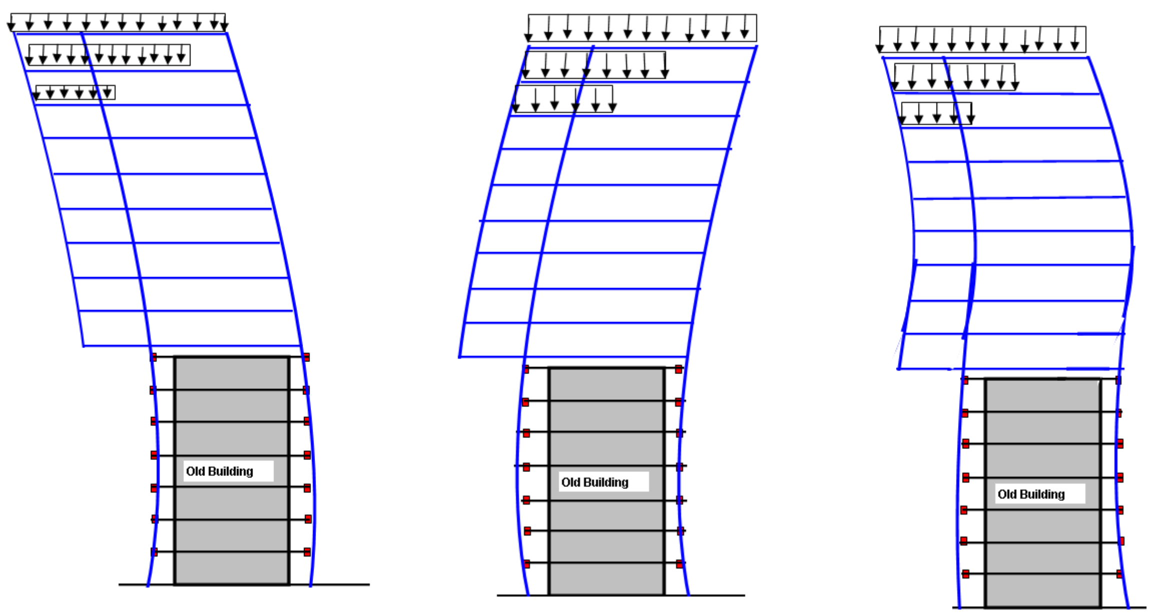
10.1 Braced steel frame

10.1.1 First order deflection

By increasing the height of building, the overall stiffness reduces and consequently the structure becomes subjected to instability. Checking on the effect of this reduction in stability is more important as far as this project is concerned.

In considering the stability, the structure is considered as whole and individual members that make up the building must be examined. However, the stability of individual members will not be covered in this project since it is provided by national design code requirement. This discussion on stability looks after the whole structure rather than individual members.

In its overall behavior, a multistory building resembles a cantilever column of moderate slenderness ratio. It differs from normal structural column, which is flexural element by including the possibility of dominant shear flexibility. Consequently, the potential modes of overall buckling include flexural mode but alternatively, a shear mode or possibly, a combination of these two modes (Figure 10-1).



(a) Flexural buckling *(b) Shear buckling* *(c) Combined shear-flexural buckling*

Figure 10-1: Buckling modes of whole structure

These mode shapes may not occur not only in transverse buckling of the structure but also in torsional or transverse-torsional forms of buckling.

The frame in axis 5 was selected for calculation because its upper portion receives more horizontal loads compared to the other frame. The results for this frame will predict the behavior of other frames.

(i). Flexural component

The procedure for obtaining the flexural component of drift is to first calculate for the structure the external moment diagram. Then to calculate for the different vertical regions of the bent, the second moment of area I of the column sectional areas about their common centroid.

$I = \sum_{i=1}^{n} A_i.L_i^2$ Where Ai, is the sectional area of steel i, and Li is the distance from the common centroid to the steel i.

The storey deflection in story i, u_{if} due to flexure of the structure is then obtained from; $u_{if} = h_i.\theta_{if}$ in which h_i is the height of story i, and θ_{if} is the inclination of story i, which is equal to the area under the $\frac{M}{EI}$ curve between the base of the structure and the mid height of the story i.

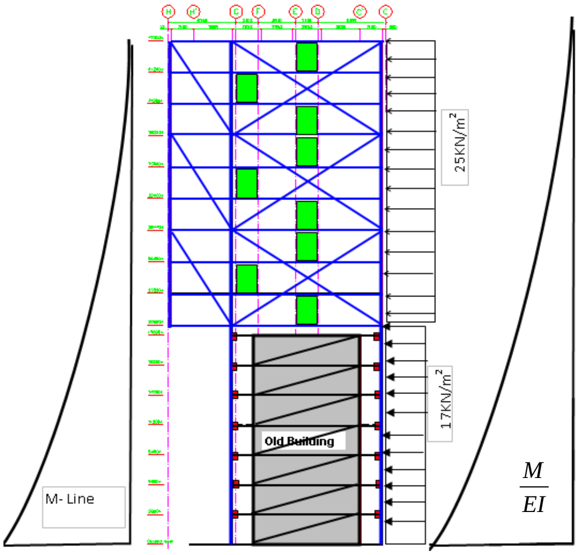


Figure 10- 2: Bending moment due to lateral load

The total deflection at floor n, due to flexure is then given by the sum of the story drifts from the first to n[th] stories.

$$u_{f,tot} = \sum_{i=1}^{n} u_{if}$$

Calculation procedures:

1. Compute the moment of inertia of the column sectional area about their common centroid. The part contributing to the lateral load resistance is the one starting from the ground. It includes 2-support columns from the ground level to the 15[th] floor.

This values is put in table 12-3 column 7.

Compute the value of the external moment at each story and enter the value in column 6.

2. Determine for each story the value of $\frac{hM}{EI}$, and enter the results in column 9.
3. Determine for each story i, the accumulation of $u.\theta_{if}$ from the ground level up to and including story i, θ_{if} and record it in column10. Such accumulated values give the inclination of each story i due to flexure θ_{if} .
4. Record the product of h_i and θ_{if} in column 11; $h_i.\theta_{if}$ is the drift in story i, u_{if} , due to flexure.
5. At each level, the value of the lateral drift is required, evaluate the accumulation of the story drift , u_{if} ,from the ground level up and considered n [th] floor to give the drift u_{total} due to flexure. Enter these in column 12.

Table10- 1: Evaluation of flexural component for the braced frame in axis 5(See Appendix 8)

Story	Story heigh mm	Cum.height Ground -roof m	Cum. height roof-ground m		Lateral load KN/m	External moment Nm	Frame inertia $x10^{12}mm^4$	Mod. Elast E $x10^4N/mm^2$	Storey Inclination $\delta\theta i=h*M/EI$ rads	Cumul.storey inclinastion θ_{if} Rads	storey drift mm	Cumul. Drift mm
Roof	2960	47200	0		25	0	16	21	0	6,14E-05	0,18	2,00
15	2960	44240	2960		25	1,1E+08	16	21	9,648E-08	6,14E-05	0,18	1,82
14	2960	41280	5920		25	4,4E+08	16	21	3,859E-07	6,13E-05	0,18	1,64
13	2960	38320	8880		25	9,9E+08	16	21	8,683E-07	6,09E-05	0,18	1,46
12	2960	35360	11840		25	1,8E+09	16	21	1,544E-06	6,01E-05	0,18	1,28
11	2960	32400	14800		25	2,7E+09	16	21	2,412E-06	5,85E-05	0,17	1,10
10	2960	29440	17760		25	3,9E+09	16	21	3,473E-06	5,61E-05	0,17	0,93
9	2960	26480	20720		25	5,4E+09	16	21	4,728E-06	5,27E-05	0,16	0,76
8	2960	23520	23680		25	7E+09	16	21	6,175E-06	4,79E-05	0,14	0,60
7	2960	20560	26640		25	8,9E+09	16	21	7,815E-06	4,17E-05	0,12	0,46
Roof 6	2800	19600	27600	0	17	9,5E+09	41	22,5	2,887E-06	3,39E-05	0,00	0,34
6	2800	16800	30400	2800	17	1,1E+10	41	22,5	3,473E-06	3,10E-05	0,09	0,34
5	2800	14000	33200	5600	17	1,4E+10	41	22,5	4,1E-06	2,76E-05	0,08	0,25
4	2800	11200	36000	8400	17	1,6E+10	41	22,5	4,767E-06	2,35E-05	0,07	0,17
3	2800	8400	38800	11200	17	1,8E+10	41	22,5	5,474E-06	1,87E-05	0,05	0,11
2	2800	5600	41600	14000	17	2,1E+10	41	22,5	6,222E-06	1,32E-05	0,04	0,06
1	2800	2800	44400	16800	17	2,3E+10	41	22,5	7,011E-06	7,01E-06	0,02	0,02
Ground floor		0	47200	19600	17	2,6E+10	41	22,5	0	0,00E+00	0,00	0,00

$$u_{f,tot} = \sum_{i=1}^{n} u_{f,i} = 2mm$$

(ii). Shear component

The shear component of the story drift is story i, u_{is} is a function of the external shear and the properties of braces and girder in that story [6]. The shear component of the total drift at floor level n, $u_{s,total}$ is equal to the sum of the story shear component of the drift from the ground to n[th] stories, that is

$$u_{s,tot} = \sum_{i=1}^{n} u_{is}$$

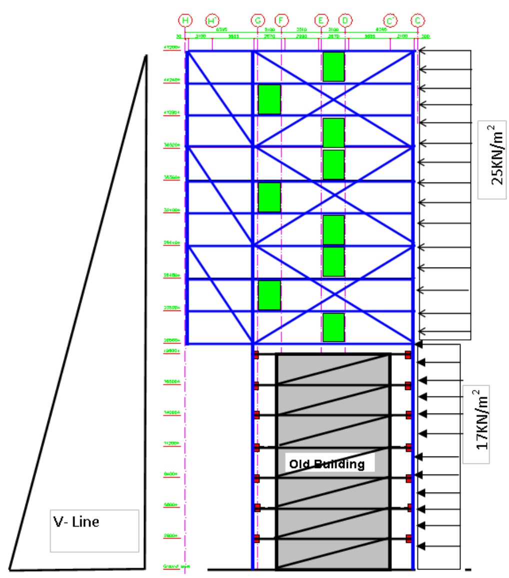


Figure 10- 3: Steel frame and lateral loads

The upper part of the frame is braced by cross bracings whereas the lower part is braced by the existing building whereby the bracings are modelled as single bracings representing the masonry in-fill. [6]
The corresponding horizontal drifts at each floor level displace are given by the following relations:

- For cross bracing: $u_{7-15} = \frac{Q}{2E}\left[\frac{L_{BR}{}^3}{L_B{}^2.A_{BR}}\right]$
- For single bracing: $u_{0-7} = \frac{Q}{E}\left[\frac{L_{BR}{}^3}{L_B{}^2.A_{BR}} + \frac{L_B}{A_B}\right]$

L_{BR} and A_{BR} are respectively is the length and the area of the bracing
L is the length of girder
E is modulus of elasticity.
Q: The shear force
L_B and A_B are respectively is the length and the area of the upper beam

By using the above formulae, the total deflection of the building as a result of shear can be calculated in form of table. (See Table 10-2)

Table10-2: Shear component of deflection for the braced frame in axis 5 (See Appendix 8)

Story	Lateral load KN/m	Cumul.height Ground -roof mm	Shear Q KN	L_{BR} mm	L_B mm	Ad mm²	A_B mm²	I_B mm⁴	E_{BR} N/mm²	Storey drift(mm)	Cumulat. Drift mm
Roof	25	0	0	16676	14115	19800	76600	2,73E+09	210000	0,00	20,51
15	25	2,96	74	16676	14115	19800	76600	2,73E+09	210000	0,21	20,51
14	25	5,92	148	16676	14115	19800	76600	2,73E+09	210000	0,41	20,30
13	25	8,88	222	16676	14115	19800	76600	2,73E+09	210000	0,62	19,89
12	25	11,84	296	16676	14115	19800	76600	2,73E+09	210000	$\delta_i = \frac{Q}{2E}\left(\frac{d^3}{L^2 A_d}\right)$ 0,83	19,27
11	25	14,8	370	16676	14115	19800	76600	2,73E+09	210000	1,04	18,44
10	25	17,76	444	16676	14115	19800	76600	2,73E+09	210000	1,24	17,40
9	25	20,72	518	16676	14115	19800	76600	2,73E+09	210000	1,45	16,16
8	25	23,68	592	16676	14115	19800	76600	2,73E+09	210000	1,66	14,71
7	25	26,64	666	16676	14115	19800	76600	2,73E+09	210000	1,86	13,05
Roof 6	17	27,6	1135,2	10385	10000	218400	52500	2,73E+08	225000	1,22	11,19
6	17	30,4	1182,8	10385	10000	218400	52500	2,73E+08	225000	1,27	9,97
5	17	33,2	1230,4	10385	10000	218400	52500	2,73E+08	225000	$\delta_i = \frac{Q}{E}\left(\frac{d^3}{L^2 A_d} + \frac{L}{A_g}\right)$ 1,32	8,70
4	17	36	1278	10385	10000	218400	52500	2,73E+08	225000	1,37	7,38
3	17	38,8	1325,6	10385	10000	218400	52500	2,73E+08	225000	1,42	6,00
2	17	41,6	1373,2	10385	10000	218400	52500	2,73E+08	225000	1,48	4,58
1	17	44,4	1420,8	10385	10000	218400	52500	2,73E+08	225000	1,53	3,10
Ground	17	47,2	1468,4	10385	10000	218400	52500	2,73E+08	225000	1,58	1,58

$$u_{s,tot} = \sum_{i=1}^{n} u_{s,i} = 20,5mm$$

Finally, the total deflection of the braced frame is the sum of deflection due to shear and that of flexure.

$$u_{tot;1st} = u_{f,tot} + u_{s,tot} \Rightarrow 2 + 20,5 = 22,5mm$$

10.1. 2 Second order effect of gravity loading

The total gravity load of the building is small portion of the load that would be required to cause overall buckling. Therefore, the possibility of collapse in this way is little. The more serious stability consideration concerns the second order effects of gravity loading acting on transverse displacements caused by horizontal loading, or acting on initial vertical misalignment in the structure. The vertical eccentricity of gravity load and the presence of cantilever causes increase in transverse displacements and in member moments.

- **Overall buckling analysis of frame**

Determination of overall buckling is necessary due to the flowing reasons:

- It indicates an upper bound for the critical gravity load,
- It allows an assessment of relative vulnerability of the building to transverse buckling or tortional buckling,

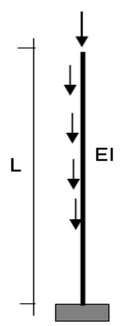
- It may be used in a structure for which an approximate P-Delta analysis is appropriate to evaluate an amplification factor for the displacements and moments.

- **The P-Delta effect**

A first –order computer analysis of a building structure for simultaneous application of gravity and horizontal loads results in deflections and forces that are a direct superposition of the results for the two types of loading considered separately [6]. The interaction between the effect of gravity loading and horizontal loading is not determined by the analysis.
When horizontal load acts on a building and causes it to drift, the resulting eccentricity of the gravity load from the exes of the walls (Bents) and columns produces additional external moments to which the structure responds by drifting further. The additional drift induces additional internal moments sufficient to equilibrate the gravity load moment. This effect of gravity load “P” acting on already displaced frame is called P-Delta effect.

- **Buckling force**

Mostly, all stabilizing systems can be schematized as a flexural cantilever. The buckling length L_0 of a fully restrained flexural cantilever of height L and constant stiffness EI, depends on the origin of forces.(Figure 10-4). As the number of floor is more than 5, the buckling length $L_0 = 1,12L$



The buckling force $P_{E_1} = \dfrac{\pi^2 EI}{L_0{}^2}$

Figure 10-4: Buckling load for stabilizing structure which are fully restrained [14]

In reality the stabilizing elements are always partly restrained, the degree of fixity depends on the stiffness of foundation. This degree of fixity is expressed by the spring constant C. The buckling force P_{E2} of a partly restrained cantilever EI is given by : $P_{E2} = \dfrac{C}{L}$. As the frame is more than 5 stories, $P_{E2} = \dfrac{C}{0,5L}$

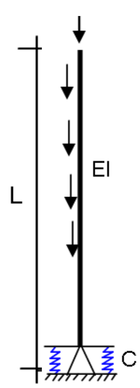


Figure 10-5: Buckling load for stabilizing structures which are partly restrained [14]

The total buckling force of partly restrained stabilizing element P_E is determined as follows:

$$\frac{1}{P_E}=\frac{1}{P_{E_1}}+\frac{1}{P_{E_2}}$$

- **Amplification factor**

A vertical cantilever displaced laterally by a uniform distributed horizontal load is subjected to the increase of horizontal displacement when a concentrated vertical load is placed at free end of the cantilever [14] (Figure10-6). The rate at which the horizontal displacement increases is known as amplification factor.

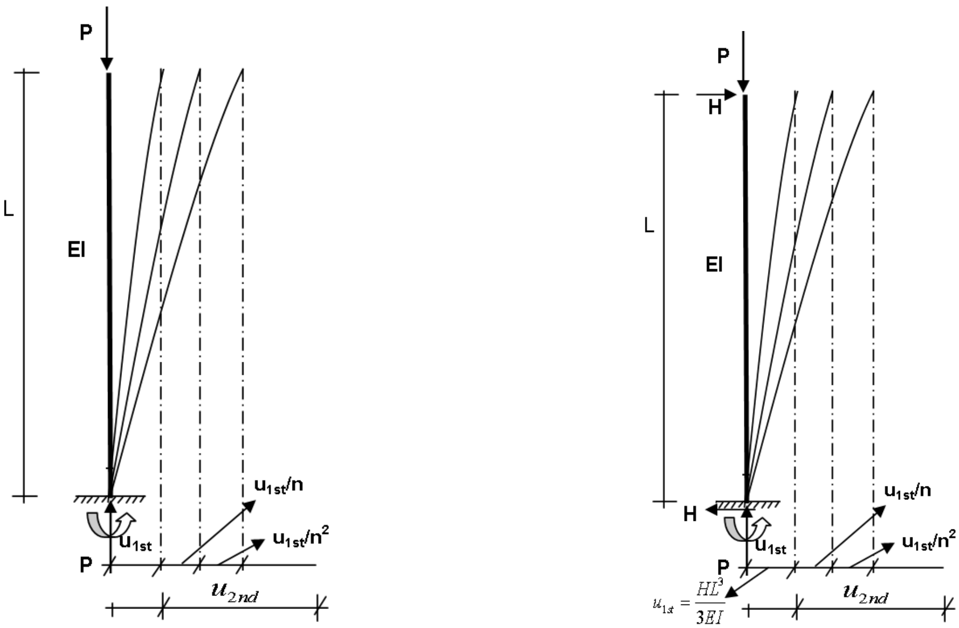


Figure 10-6: Second order effect for partly restrained bracing structure [14]

If initial deflection of the top of a column is u_{1st} , then the deflection due to the second order u_{2nd} is as follow:

$$u_{2nd}=\frac{u_{1st}}{n}+\frac{u_{1st}}{n^2}+\frac{u_{1st}}{n^3}+\ldots\Rightarrow u_{2nd}=\frac{1}{n-1}u_{1st},$$

The buckling force is reached when n=1 and the first term $\frac{u_{1st}}{n}=u_{1st}$ and the total added deflection becomes infinitely large. Therefore $P_E=n\gamma P\Rightarrow n=\frac{P_E}{\gamma P}$

- The first order deflection $u_{1st}=\frac{HL^3}{3EI}$
- The second order deflection $u_{2nd}=\frac{1}{n-1}u_{1st}$
- Total deflection $u_{tot}=u_{1st}+\frac{1}{n-1}u_{1st}\Rightarrow u_{tot}=u_{1st}\frac{n}{n-1}u_{1st}$

The total moment $M_t=HL+\frac{n}{n-1}u_{1st}.P$, then $HL=P_E.u_{1st}$; hence $M_t=\frac{n}{n-1}HL$

- **Calculation results**

The calculations in Appendix9 show that $n=\frac{P_E}{\gamma P}$, $n=115$. The amplification factor $\frac{n}{n-1}=1{,}01$; this shows that the second degree effect is small.

The second degree deflection: $u_{2nd} = \frac{1}{n-1} u_{1st}$, $u_{2nd} = \frac{1}{115-1} 22,5 = 0,20mm$

10.1.3 Effect of foundation rotation

A flexible foundation will affect the overall stability of a building by reducing the effective lateral stiffness of the vertical cantilevers structure.
It increases the deflection from horizontal load and hence increases the second order effect.
The horizontal deflection of flexural cantilever subjected to uniform distributed load is given by this relation.

$$u_{1st} = \frac{wL^3}{8EI} \text{ [14]}$$

If the foundation rotational flexibility is defined as the rotation per unit moment is C, then the deflection is increased to:

$$u_{rot} = \frac{wL^3}{8EI} + \frac{wL^2}{2C} \text{ [14]}$$

- **Determination of spring constant C for pile foundation**

Constant C depends on elastic deformation of pile and moment distribution on pile group. Each pile in group receives a load P_n generated by the moment

$$P_n = \frac{M}{I_p}.e$$

Elastic deformation of a pile is as follow:

$$\Delta L = \frac{P_n.1,5L_p}{E_{p.}A_p}$$

Where : I_p = the second moment of area of the pile group
e = distance of individual pile from centroidal axis of the pile group
L_p =Length of the pile
E_p = Modulus of elasticity of the pile
A_p = Cross section area of the pile
$1,5$ = factor covering the deformation of pile foot

Then the rotation angle $\varphi = \frac{\Delta L}{e}$ and $C = \frac{M}{\varphi}$

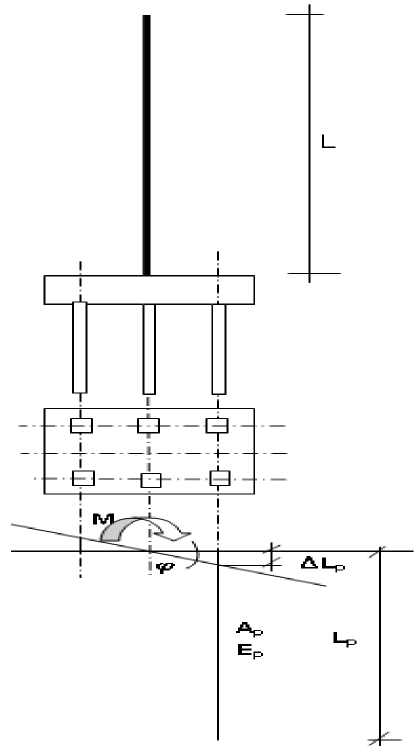


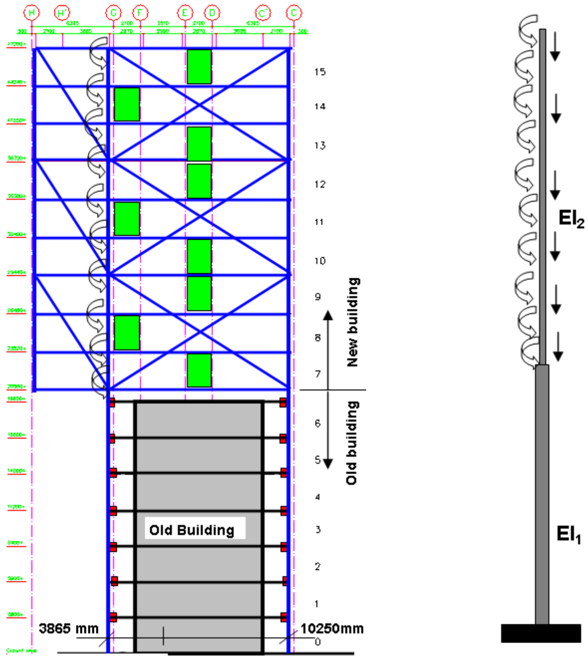
Figure 10-7: Pile foundation [14]

- **Calculation results**

The calculations done in Appendix 9 , show the total moment in foundation due to lateral load and eccentricity is $50765KNm$ and the spring constant of piles is $208.10^6 KNm/rad$. Therefore, the deflection due to foundation rotation becomes negligible since it is given by this formula: $u_{rot} = \frac{M}{C}$.

10.1.4 Effect of moment due to cantilevers

As the new structure has the big cantilevers, these make the new building to be eccentrically placed with respect to the supporting elements. Therefore further horizontal deformations will occur.



(a) Stabilizing frame (b) Stabilizing frame schematized as Vertical cantilever

Figure 10-8: Stabilizing frame showing moment loading from cantilevers

The deformation δ_M of cantilever loaded with a bending moment is given by this formula:

$u_M = \frac{\sum M_i L_i^2}{2EI}$, where M_i is the bending moment in cantilever level i and L_i is the height from foundation to the cantilever level i. The results from Appendix 9 , show that the frame can drift horizontally by $2mm$

10.1.5 Conclusion

The procedures stated above were meant to check any kind of failure that can occur in the new structure. The final horizontal drift is the summation of the drifts caused by:

- First order deflection (u_{1st})
- Second order deflection (u_{2nd})
- Deflection due to the rotation of foundation (u_{rot})
- Deflection due to moments in cantilevers (u_M)

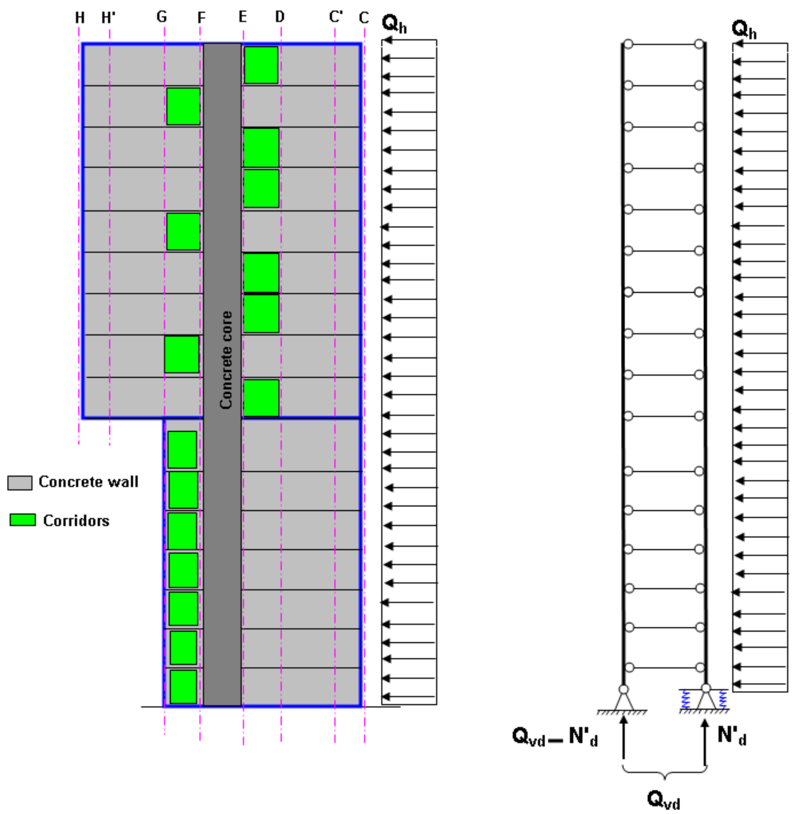
$$u_{Total} = u_{1st} + u_{2nd} + u_{rot} + u_M$$

All these deflection components were calculated in Appendix 9 and it has been obtained that the total deflection of the braced frame $u_{Total} = 22,5 + 0,2 + 0 + 2 = 24,7mm$

10.2 Core

In this building the concrete core is provided and constructed just next to the existing building,; this section of the building starts from the ground.
This core is 8, 5 m long and 3, 5 m wide; it houses the stair case and elevators. Its main role is to restrain both horizontal load and a part of gravity load acting to the building. It is composed by an assembly of connected shear walls forming a box section with openings closed partially by floor slabs.

The core in this building plays a role of bracing element and is schematized as partly restrained vertical cantilever. The walls around it are schematized as column with pin jointed end. These walls have a large number of openings (corridors, windows and door) in such a way that they cannot be used to restrain lateral loads. They are only considered for vertical loads.



N'_d = Vertical load acting on core

Q_{vd} = Total vertical load of the building

Q_{hd} = Total horizontal load of the building

Figure 10-10: Core and wall with main openings for corridors in axis -1 and 0 and schematization

- **Calculation of core**

A further schematization can be done (see figure 9-11). The core is represented as a partly restrained column subjected to wind loads, imperfections Q_{hd} and vertical loads made by core own weight and loads which are directly supported by the core N'_d. The rest of the vertical loads $Q_{vd} - N'_d$ are carried to the foundation by columns. These loads carried by columns can have influence on second degree moment and shear force of the core.

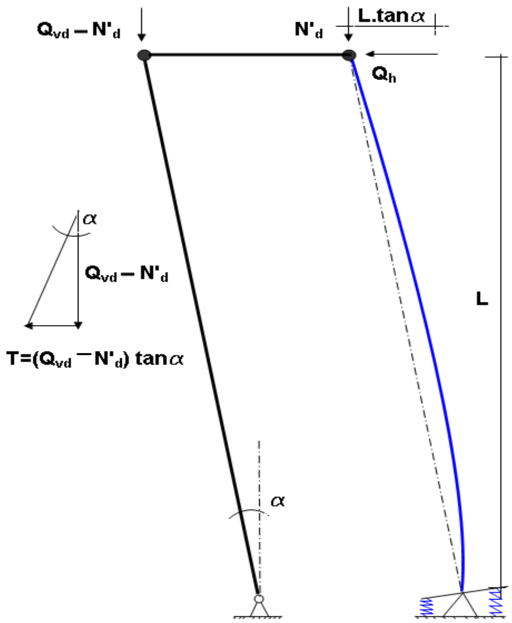


Figure10-11: Schematization of core and supporting wall [14]

The partly restrained core is subjected to horizontal force Q_{hd} and normal force N'_d . The walls(represented as one column hinged at the top and base are attached to the core by floor slab and are loaded with $Q_{vd} - N'_d$.

By horizontal force, the displacement of the whole system is $L\tan\alpha$.

By the angular rotation α , a tensile force $T = \left(Q_{vd} - N'_d\right)\tan\alpha$ occurs.

At the base of the restrained core the second-degree moment $M_{core;2nd}$ is:

$$M_{core;2nd;1} = T.\alpha \Rightarrow L\left(Q_{vd} - N'_d\right)\tan\alpha$$

The normal force N'_d causes:

$$M_{core;2nd;2} = N'_d\, L\tan\alpha$$

Therefore the total second-degree moment is $M_{core;2nd} = Q_{vd} L\tan\alpha$

- **Determination of the horizontal deflection of the core.**

In Appendix 9, the calculations were done in the same way as for the braced frame. One can see that the total deflection is mainly composed by the 1st order deflection and the effect of cantilever deformation . The second order effects as well as the foundation rotation are negligible since the foundation is so stiff. The final deflection becomes 37,1 mm, which is less than 59mm; the maximum allowable horizontal deflection as it was calculated in Appendix 7.

11. FINAL ANALYSIS OF THE FINAL STRUCTURE

In this part, the final analysis of steel frames will be done by using Matrix frame software. This helps to have clear results of member forces and deflections, support reactions of the whole structure and the overall deflection of the structure. It was decided to analyze the frame in axis 5 since it receives more lateral loads compared to the other frames
In preliminary analysis, composite supporting columns were used. In this analysis only steel sections will be used since the software doesn't have composite sections in its database.

11.1 Input data

11.1.1 Loading

The loads acting on the frame consist of:

- Dead loads : 7,14 KN/m^2 and 2KN/m from front and back facade
- Live loads : 1,75 KN/m^2
- Lateral load acting on East side: 25 KN/m on the upper section and 17KN/m on the lower section
- Lateral load acting on West side: 25 KN/m on the upper section and 17KN/m on the lower section

Load combination

Table 11-1: Used load combination

PC	1	2	3	4
LC	SLS	SLS	ULS	ULS
Self weight	1,0	1,0	1,2	1,2
Variable load	1,0	1,0	1,5	1,5
Lateral load from East	1,0		1,5	
Lateral load from west		1,0		1,5

11.1.2 Materials

- Supporting columns in axis G : CHS 559 x 23,8
- Supporting columns in axis C: CHS 457 x 16
- Façade columns in axis H : RHS 200 x 100x 5 x5
- Girders : HE 700 B
- Cross bracing between G and C: HE 400 B
- Single bracing between H and G: HE 200 B
- Brackets connecting the girders to the columns: UPN 220 x 80
- Concrete beams of the existing building : 510 x 210
- Equivalent concrete bracing of the existing building : 336 x 650

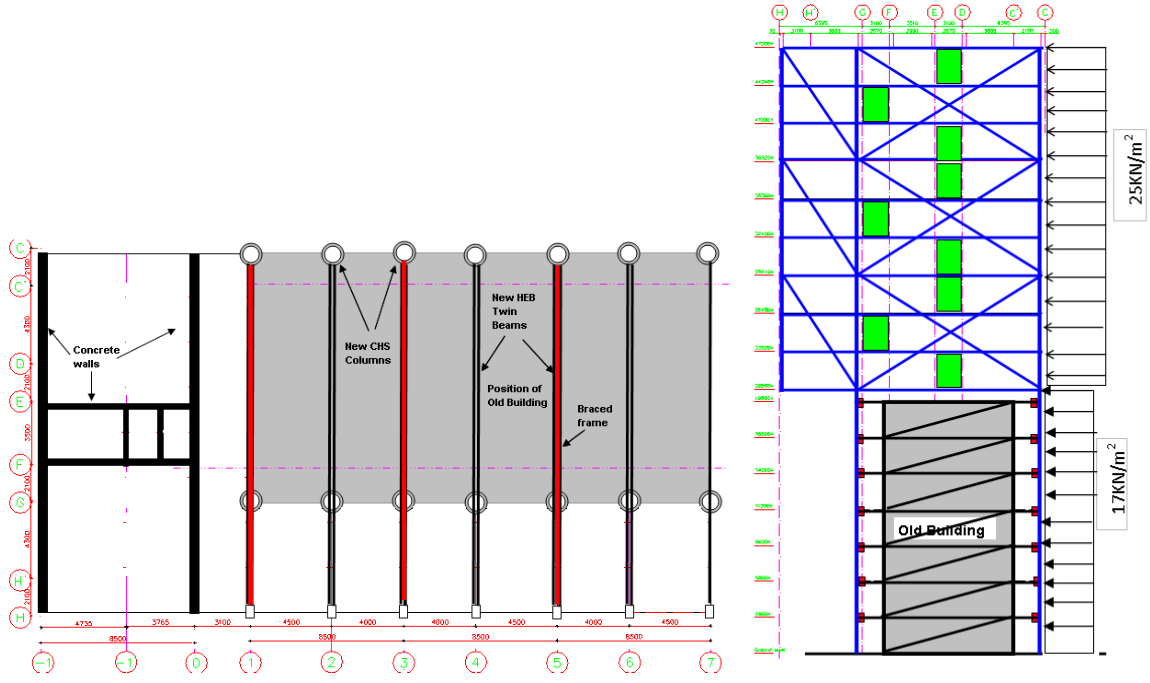


Figure 11-1: Plan for floor 7th to 15 and section in axes 1,3 and 5

11.1.3 Spring stiffness of bracing structure

The stability is provided by concrete core and braced frames; the distribution of lateral loads is based on assumption that floors(Horizontal diaphragms are infinitely stiff in their plane. Each floor is supported by stabilizing structures which are schematized as springs. (Figure 11-2).

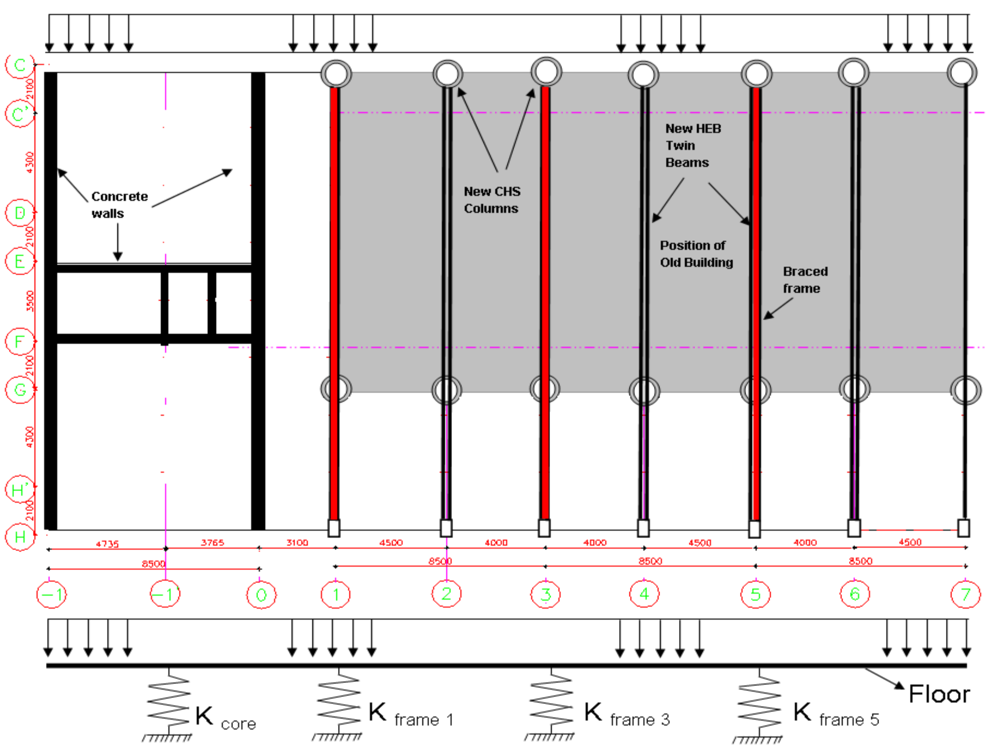


Figure 11-2: Schematization of floor and stabilizing structures

(i) Concrete core

$K = \frac{M}{u}$ Where M = Bending moment

u = Horizontal deflection

Assume the lateral load is 50KN/m, therefore the bending moment is $M = \frac{ql^2}{2} \Rightarrow \frac{50.47,2^2}{2} = 55696KNm$

The deflection $u = \frac{ql^4}{8EI} \Rightarrow \frac{50.47200^4}{8.36000.12,4.10^{12}} = 69,5mm$

$$K = \frac{M}{u} \Rightarrow \frac{55696}{69,5.10^{-3}} = 801381KNm/m$$

(ii) Braced Steel frames (axes 1,3,and 5)

When a steel frame is braced , it becomes stiff. The stiffness constant(spring constant) at each floor level can be calculated by the following formulae:

$K = \frac{E.A_b.L^2}{\left(h^2+L^2\right)^{1,5}}$ for single diagonal and $K = 2.\frac{E.A_b.L^2}{\left(h^2+L^2\right)^{1,5}}$ for cross bracing.[6]

Where E = Modulus of elasticity of the bracing

L = the beam length

h = The floor height

A_b = Cross section of the bracing

The spring constant of the lower part (old building)

$$K = \frac{E.A_b.L^2}{\left(h^2+L^2\right)^{1,5}} \Rightarrow \frac{15000.10^3.(336.650).10^{-6}.10^2}{\left(2,8^2+10^2\right)^{1,5}} = 292532KNm/m$$

During modeling of this part(old building), this spring stiffness will not be used because the diagonal is imaginary or equivalent diagonal. Only the equivalent diagonal will be considered in order to curb various mistakes which would occur.

The spring constant of the upper part

$$K = 2.\frac{E.A_b.L^2}{\left(h^2+L^2\right)^{1,5}} \Rightarrow 2.\frac{210000.10^3.(19800)10^{-6}.14,115^2}{\left(2,960^2+14,115^2\right)^{1,5}} = 552328KNm/m$$

(iii) Foundation spring constant

The foundation is also flexible, the set of piles is characterized by a constant which can amplify the horizontal deflection. From Appendix 9, the spring stiffness of foundation was calculated and it value is $C = 208.10^6 KNm/rad$

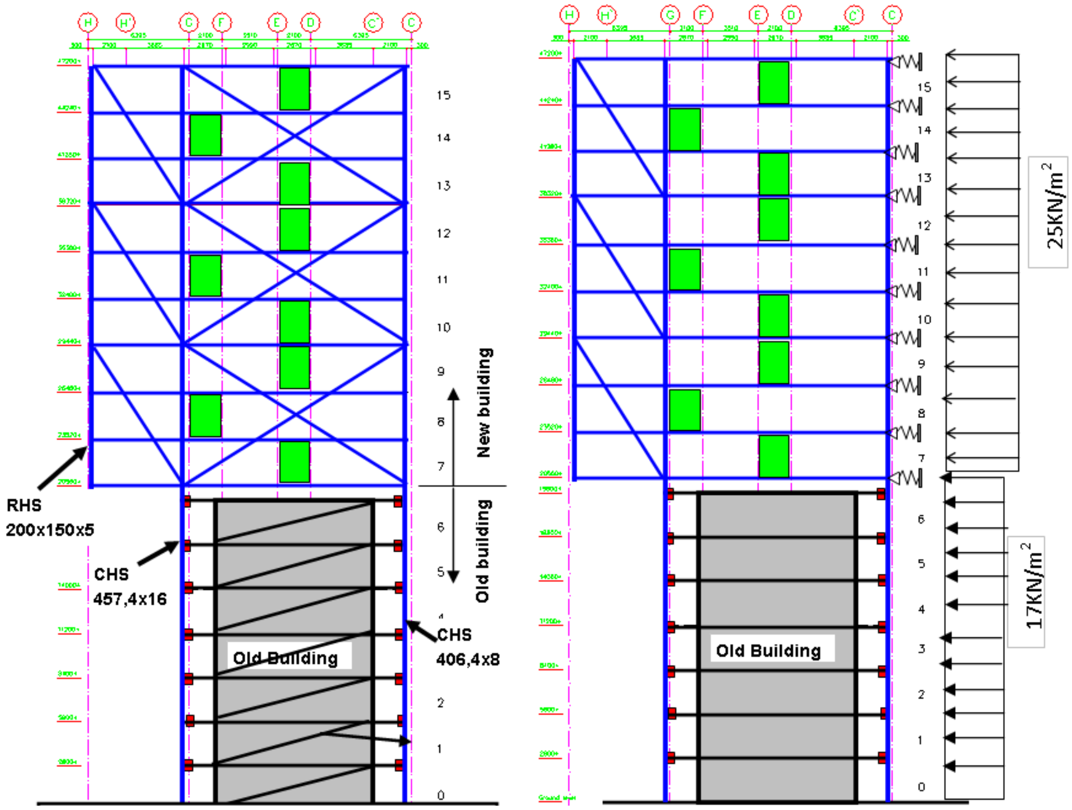
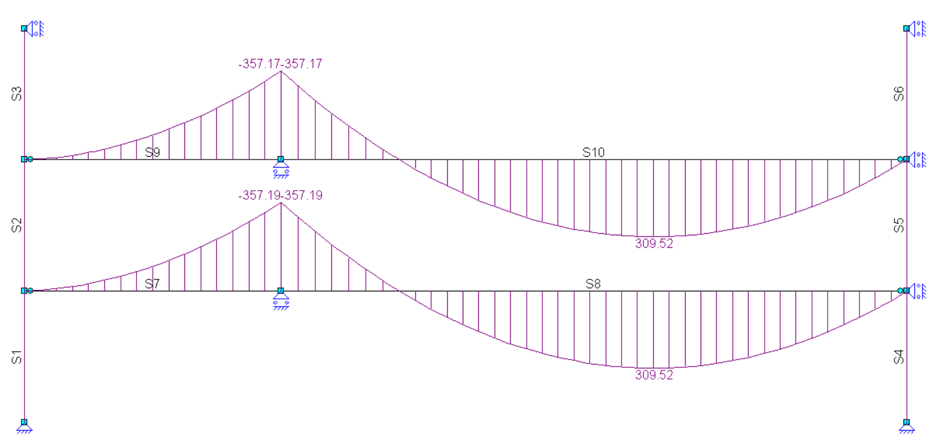


Figure 11-3: Calculation model of section in axes 1,3 and 5 with bracing structures replaced by srping supports at each floor leve

11.1.4 Connection between girders and edge columns

Hinge or rigid connection can be used to connect the beams to the columns. It is therefore necessary to check a method with more benefits so that it can be adopted when modeling the frame.

(i) Hinge connection



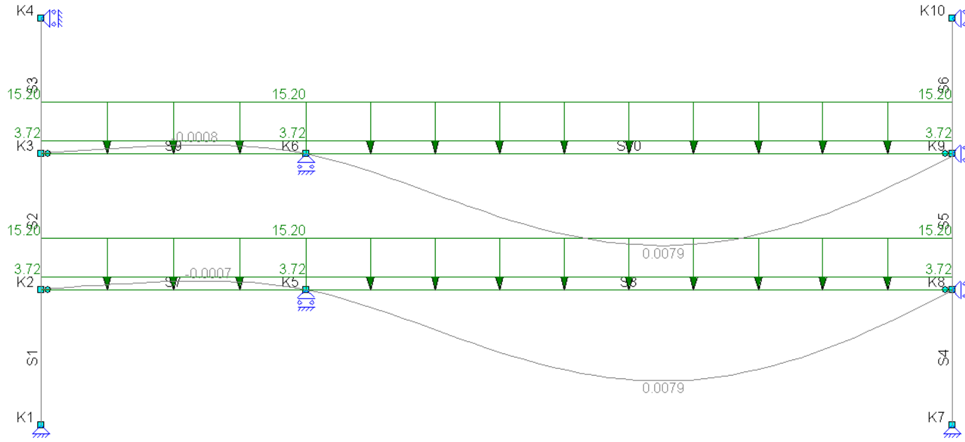


Figure 11-4: Moment distribution and deflection for hinge connection

(ii) Rigid connection

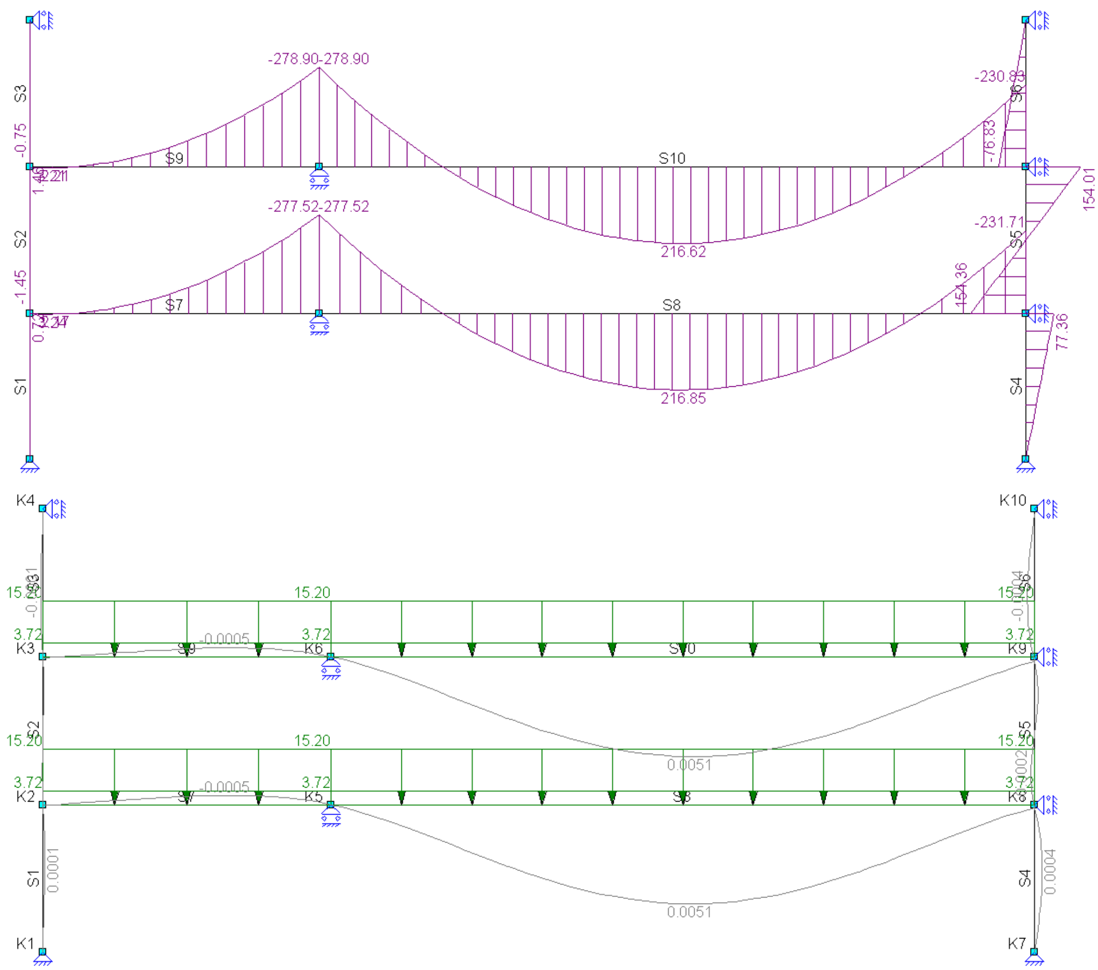


Figure 11-5: Moment distribution and deflection for rigid connection

From the above moment and deflection diagrams, one can see that the moments and deflection in mid span reduce considerably when floor beams are clamped to the columns. Therefore, rigid connections would be used in modeling the frame in order to have minimum deflections

11.2 Braced frame modeling and results

All the data stated above are put in Matrix frame software. It was decided to use 3-D model, because the representation of the double beam system can be easy. As the frame in axis 5 takes much of lateral loads; it was decided to use it in analysis; other frames in axes 1 and 3 will have more or less the same behavior as the frame in axis 5.

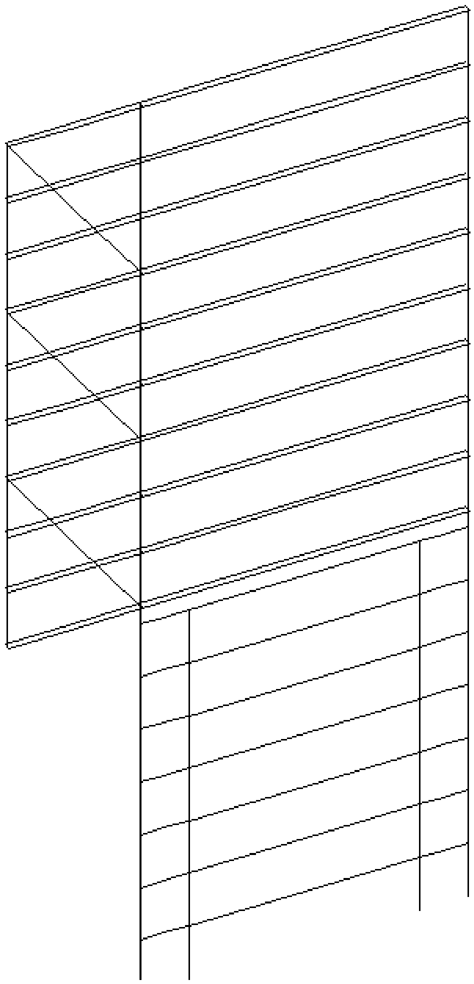
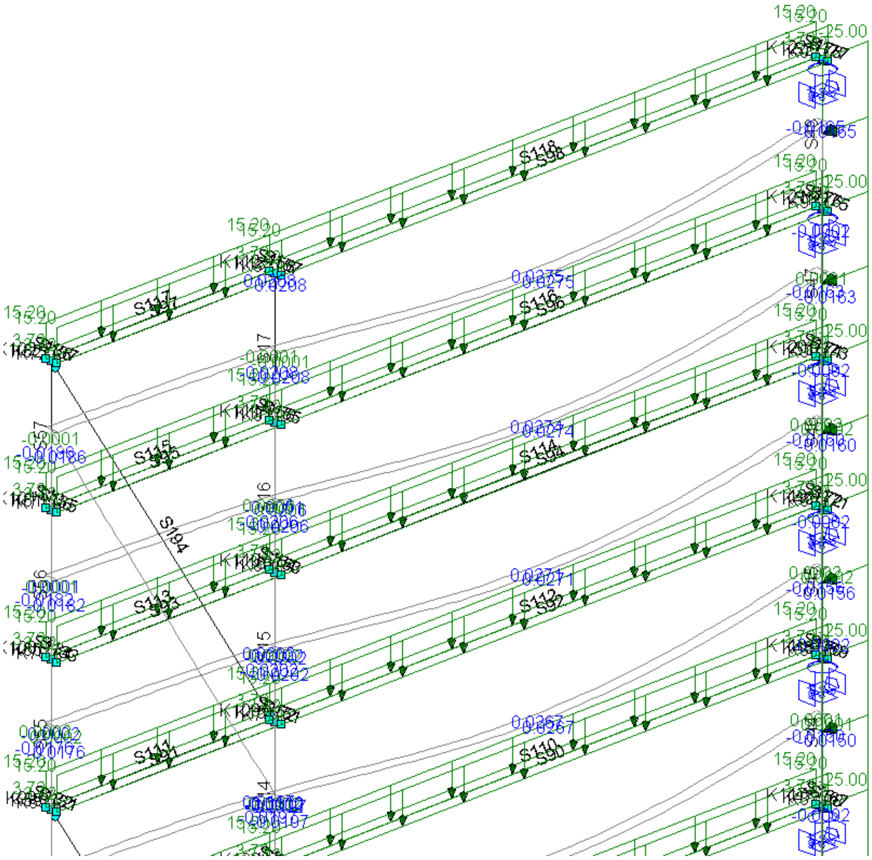


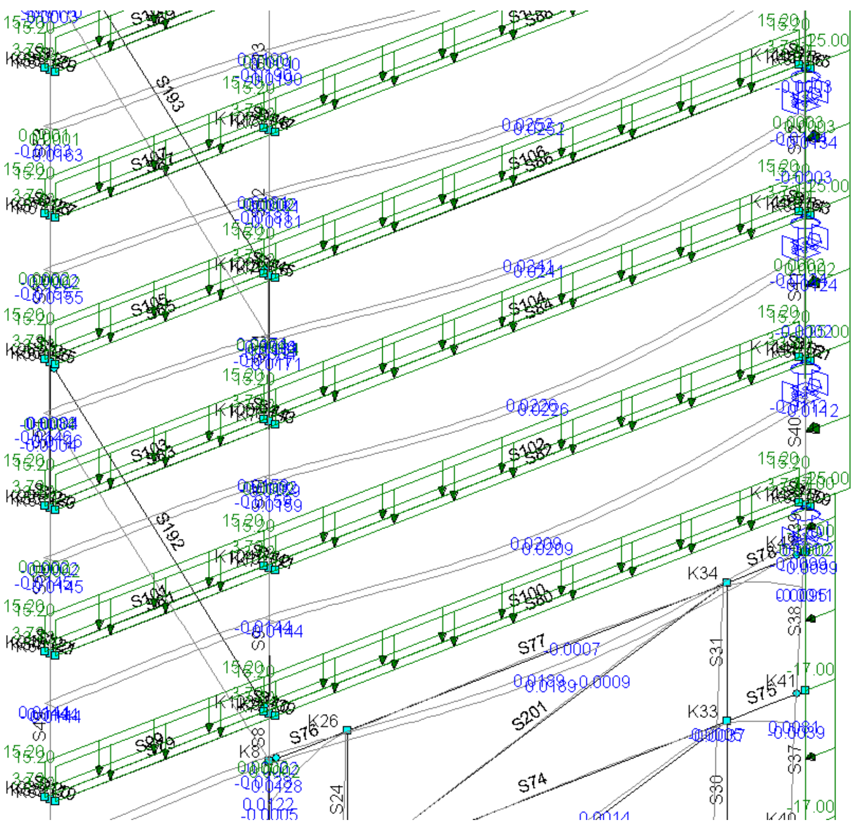
Figure 11-6: 3-D model of the frame

11.2.1 Deflection

The horizontal and vertical deflections of the frame are checked by applying service load to the frame, load combinations 1 and 2 are considered. The results (Figure 11-3) show that (load combination 1 and 2) the structure will not sway laterally due to high spring stiffness of the bracing structures. However, the maxima vertical deflection of girders and their cantilevers are respectively 27 mm and 18 mm. (Figure 11-7 and 11-8)
These deflections are less than the allowable maximum deflection for girders and their cantilevers, which are respectively 56mm and 23mm.

- **Load case 1**





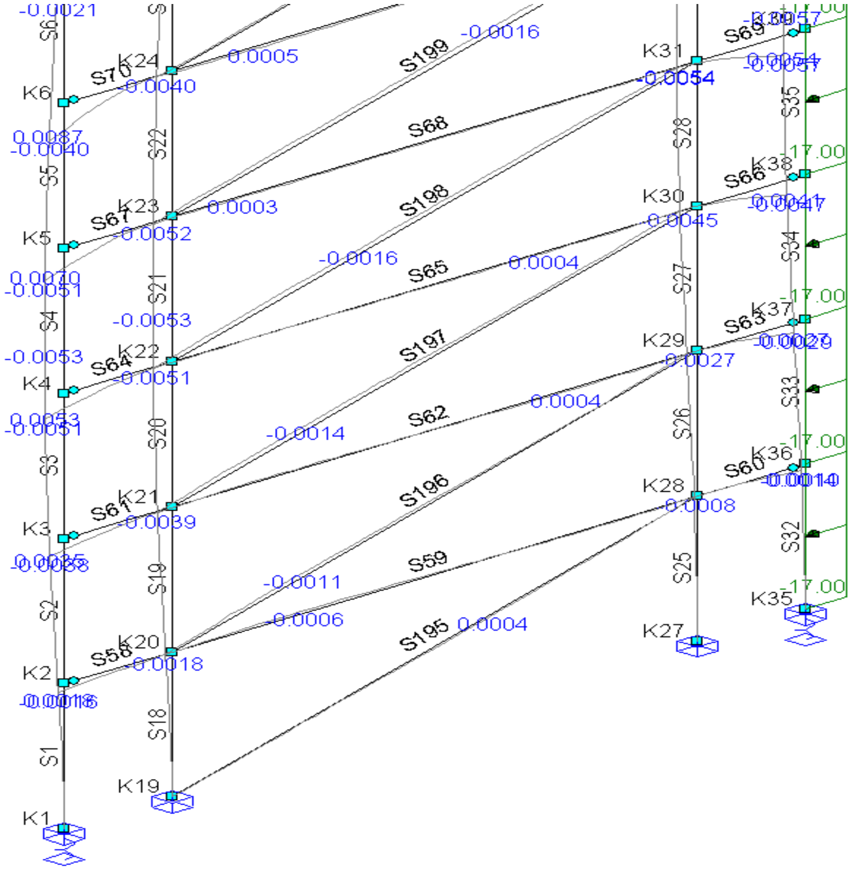
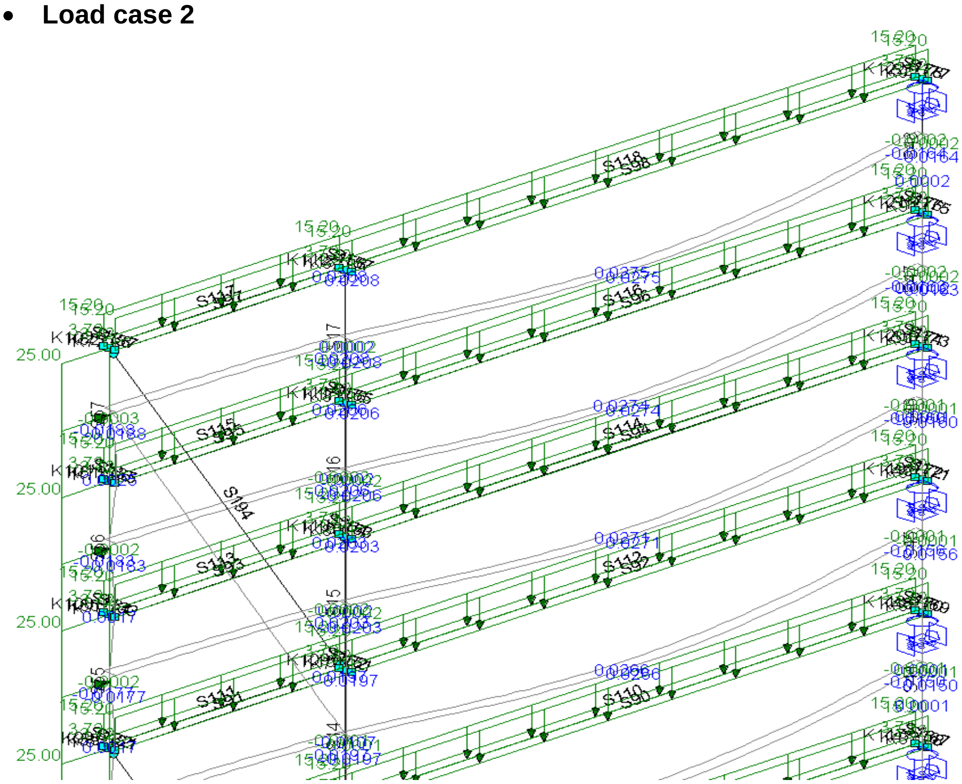


Figure 11-7: 3-D Model showing a deflected braced frame for load case 1

- **Load case 2**



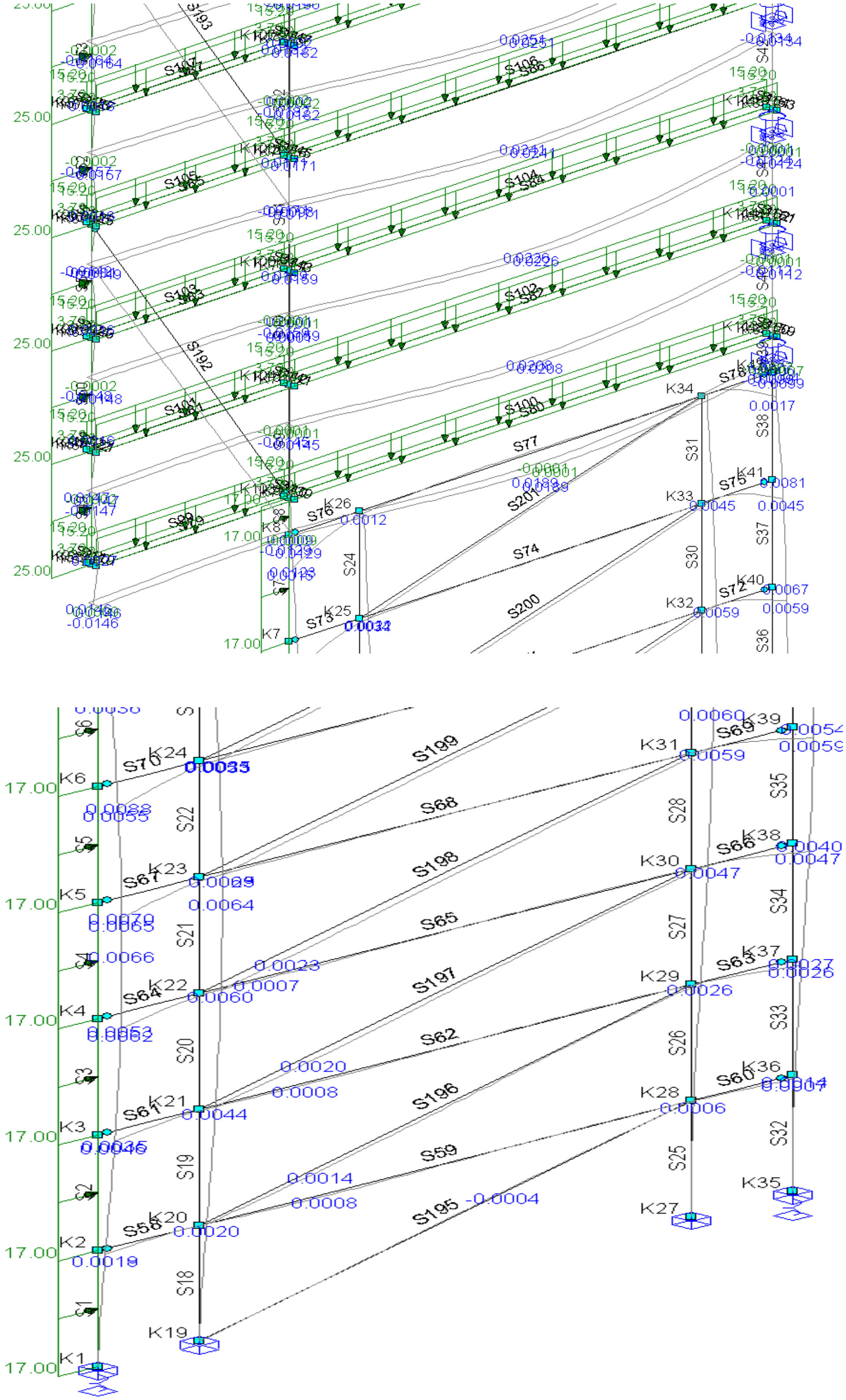


Figure 11-8: 3-D Model showing a deflected braced frame for load case 2

11.2.2 Support reactions

The support reactions depend on load combinations; each combination has its own support reaction. One can see that in ultimate limit state; load combination 5, is where maximum support reactions are obtained (figure11-4 and table 11-2). These reactions together with the vertical load from lateral moment are used to design the supporting columns and the foundation piles.

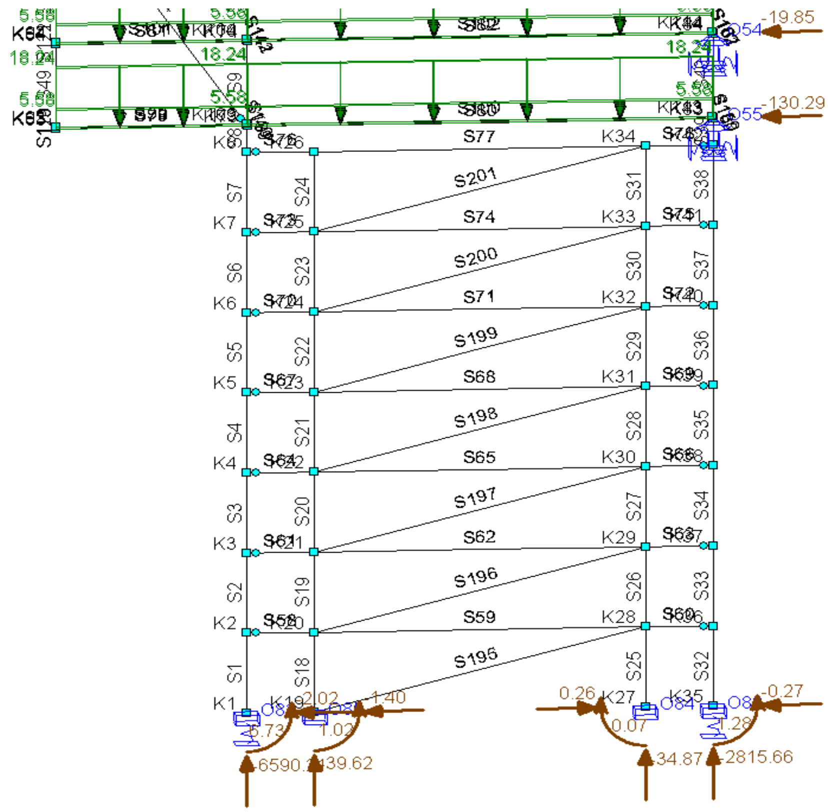


Figure11- 9: Support reaction in load combination 5

Table11- 2: Support reaction from 5 load combinations

1. In New building

Load combination	Support G [KN]		Support C [KN]	
	Compression	Tension	Compression	Tension
PC1	5231		2238	
PC2	5239		2233	
PC3	3780		1619	
PC4	3792		1612	
PC5	6590		2816	
2. In old building				
PC1	85			24
PC2	21		37	
PC3	103			58
PC4	8		33	
PC5	40		35	

As the existing building helps the new structure to carry the lateral loads, the former will eventually receives moderate loads.(Table 11-2) This can be carried safely since it has obtained that the foundation of the existing building has large security margin.

11.3 Modeling of Un-braced frame carrying only the gravity load

The frames carrying only the gravity load are those, which do not take part in resisting the lateral forces. There would not be deformation due to the lateral load. The frames taken into account are frames located in axes 2, 4, 6 and 7.
The connection of the frame to the old building and to the bracing structures by the floor system will relieve the structure from swaying toward cantilever sides (Figure 11-10)

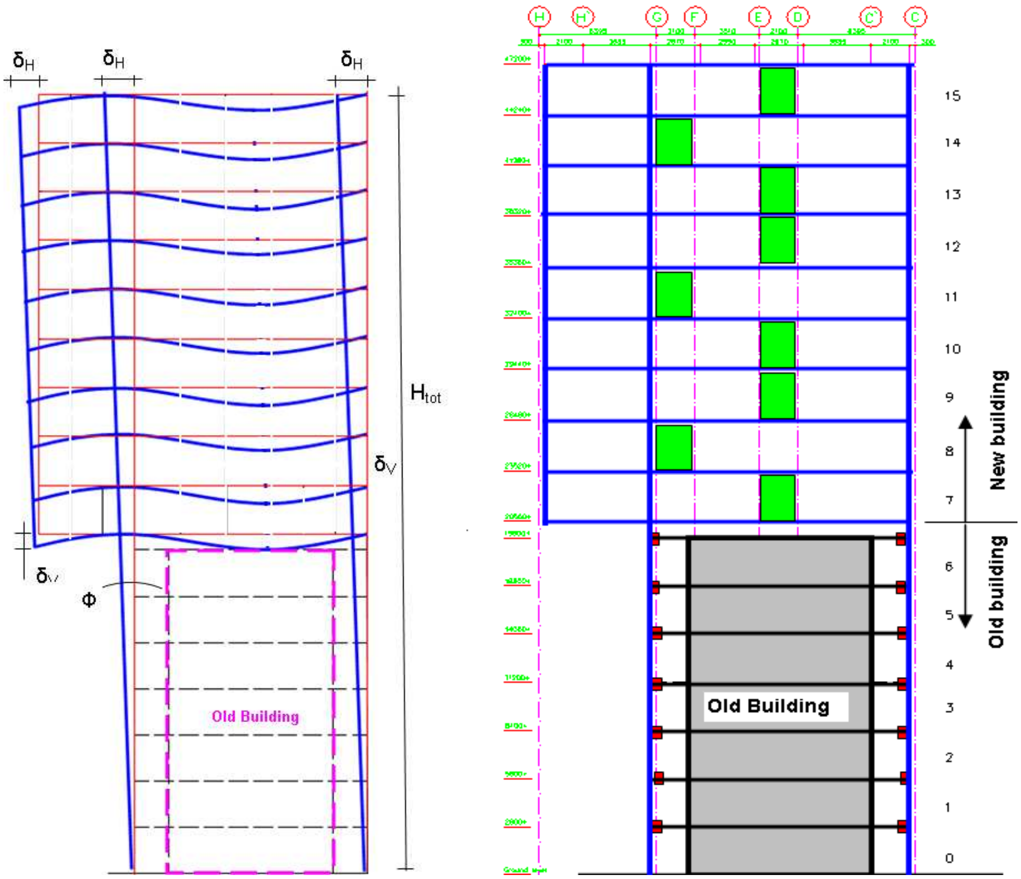
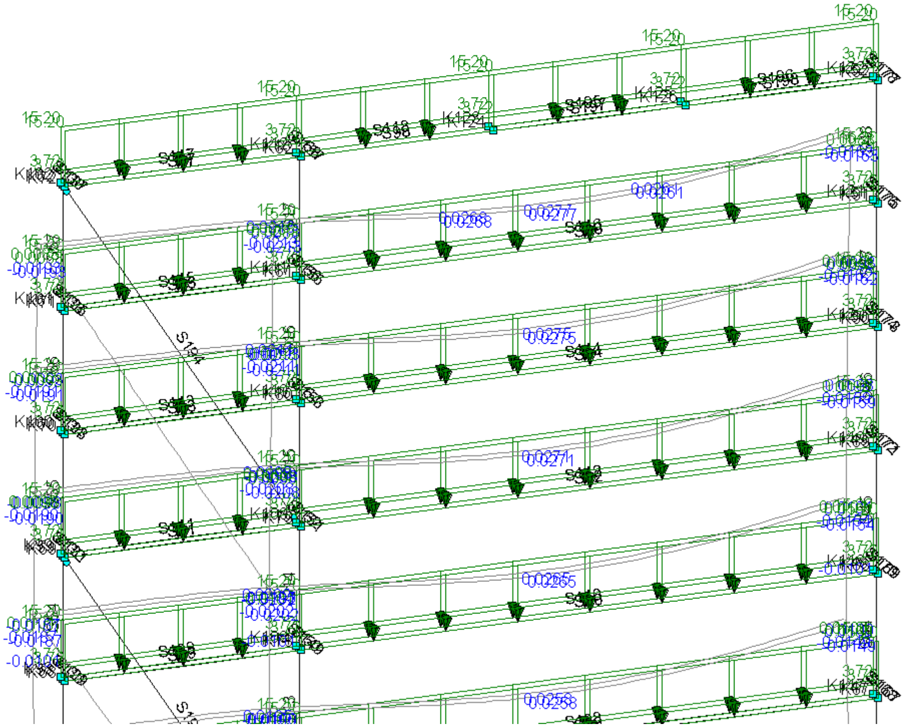
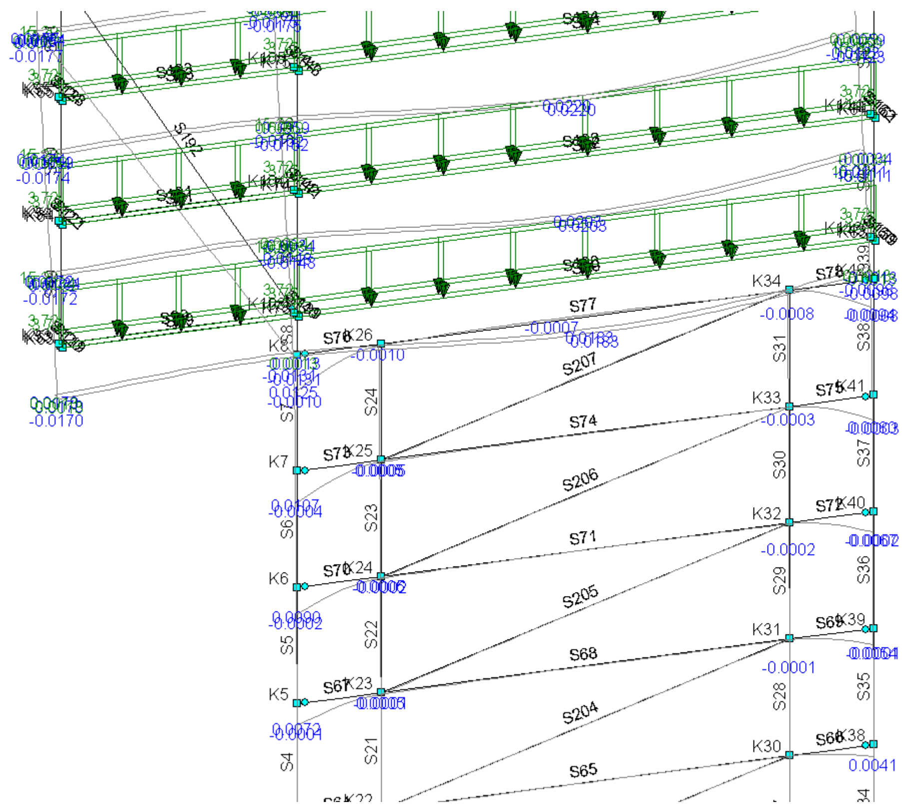


Figure 11- 10: Managing buckling and imperfection of frame taking gravity loads.

As the new structure starts at 20,56m high, columns are subjected to buckling, but when they are connected to the old building at each storey, the buckling length becomes 2,8m and then , the structure is more stable.
The result of this method is the deflection of beams only in vertical direction. When beams and edge columns are connected by hinges, it has been obtained that the deflection of beams is 18 mm and 38 mm for cantilever and between supports (Figure 11-11).



(a) Upper floors



(b) Lower floors

Figure 11- 11: Deflection of beams when there are pin connections between beams and edge columns

11.4. Reduction of vertical deflection of beams

Although the horizontal deflection is negligible in models because of lateral spring stiffness of the bracings, the beams have a considerable vertical deflection in their mid-spans. The vertical deflection can be minimized welding small cantilevers to the columns and then connecting them to the beams.(Figure11-12)

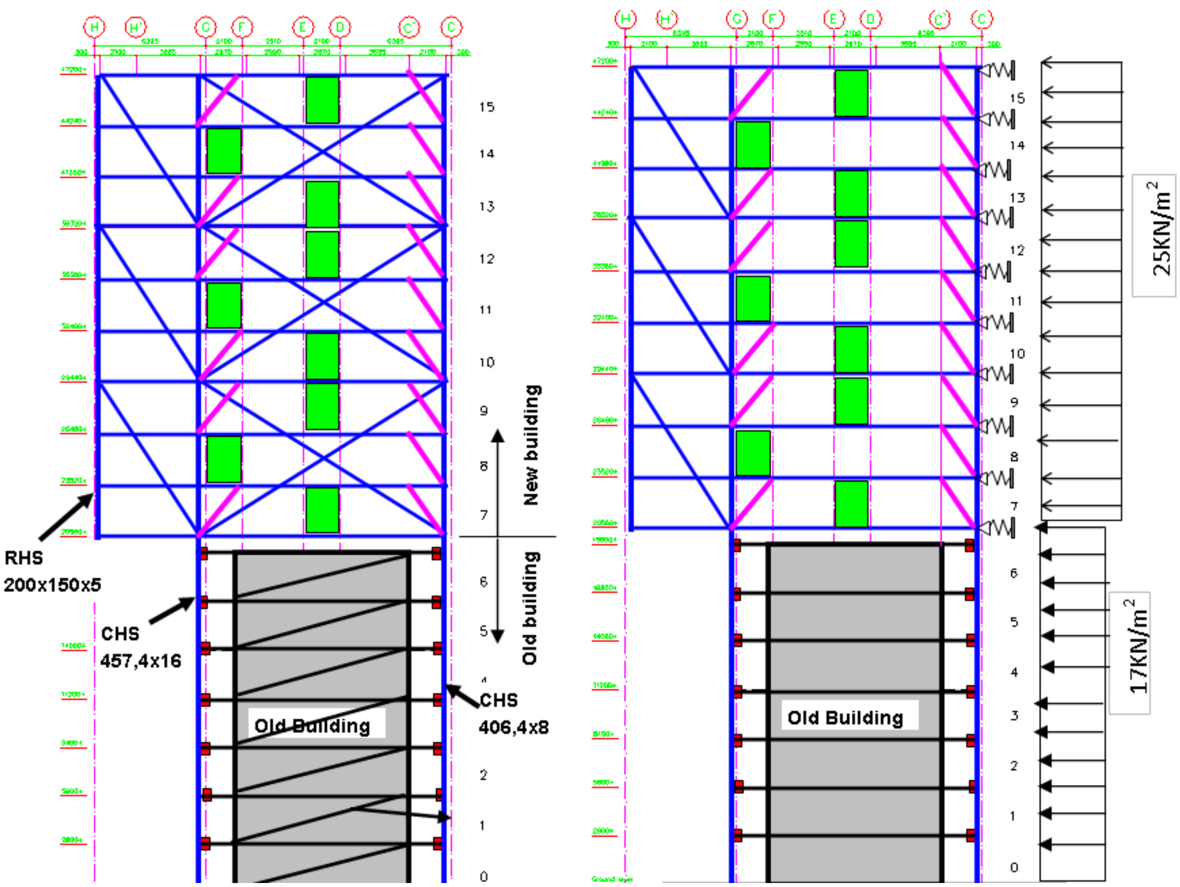


Figure11- 12: Braced and non-braced frames with welded cantilevers to reduce the vertical deflection of beams

The results from computer model show that the maximum vertical deflection of beams has reduced from 28 to 20 mm for middle fields and from 18 to 15 mm for cantilevers. These cantilever are supposed to be welded on sides of columns where the beams meet columns; parallel struts would be used.(See figure 11-13)

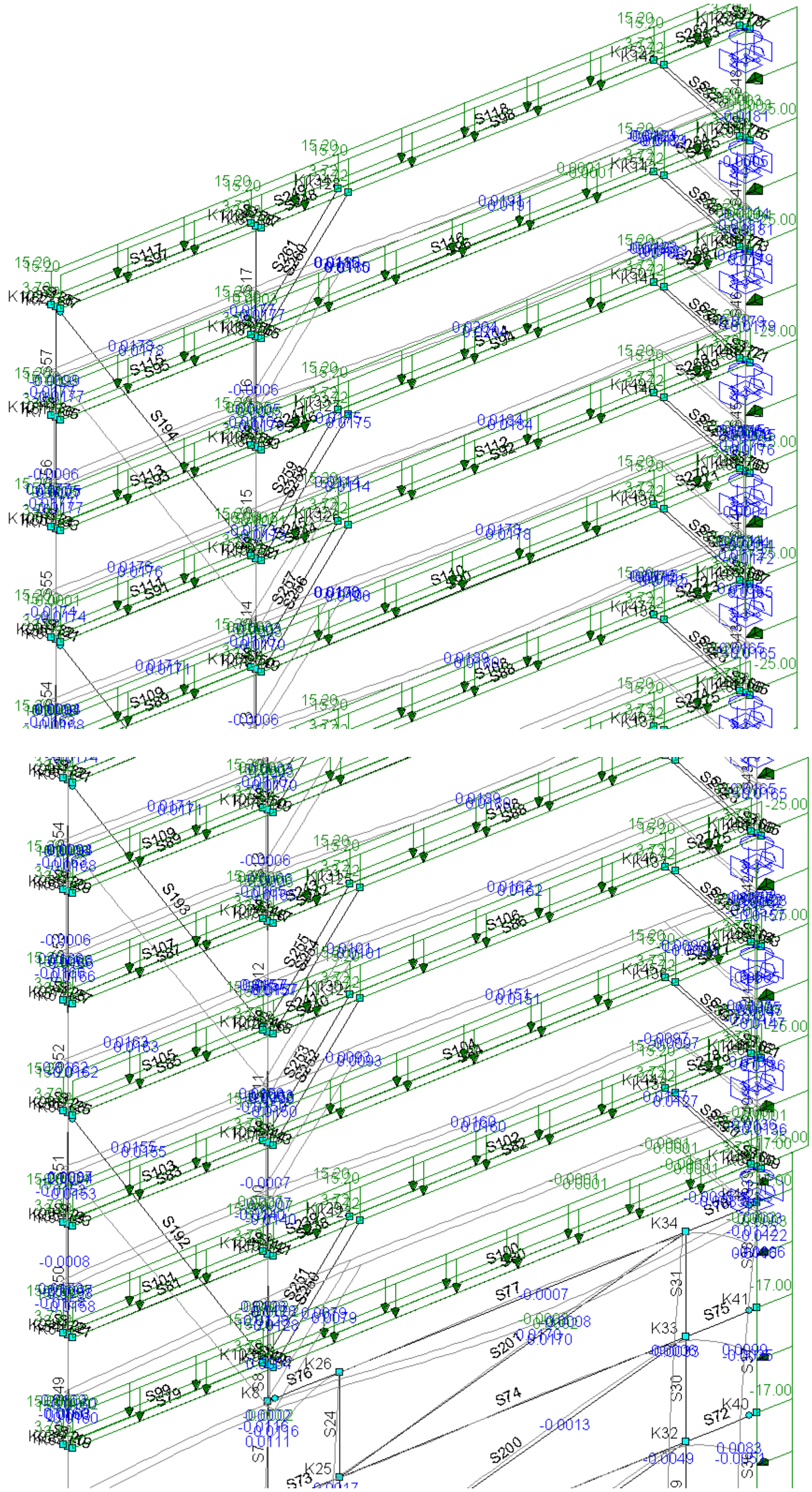


Figure11- 13: Frame model with cantilevers to reduce vertical deflection of beams

However, in braced frames, these struts, will be crossing with the bracings and aesthetically unpleasant. Therefore another means of bracing can be suggested even though its behavior will not be covered in this report(See figure11-14).

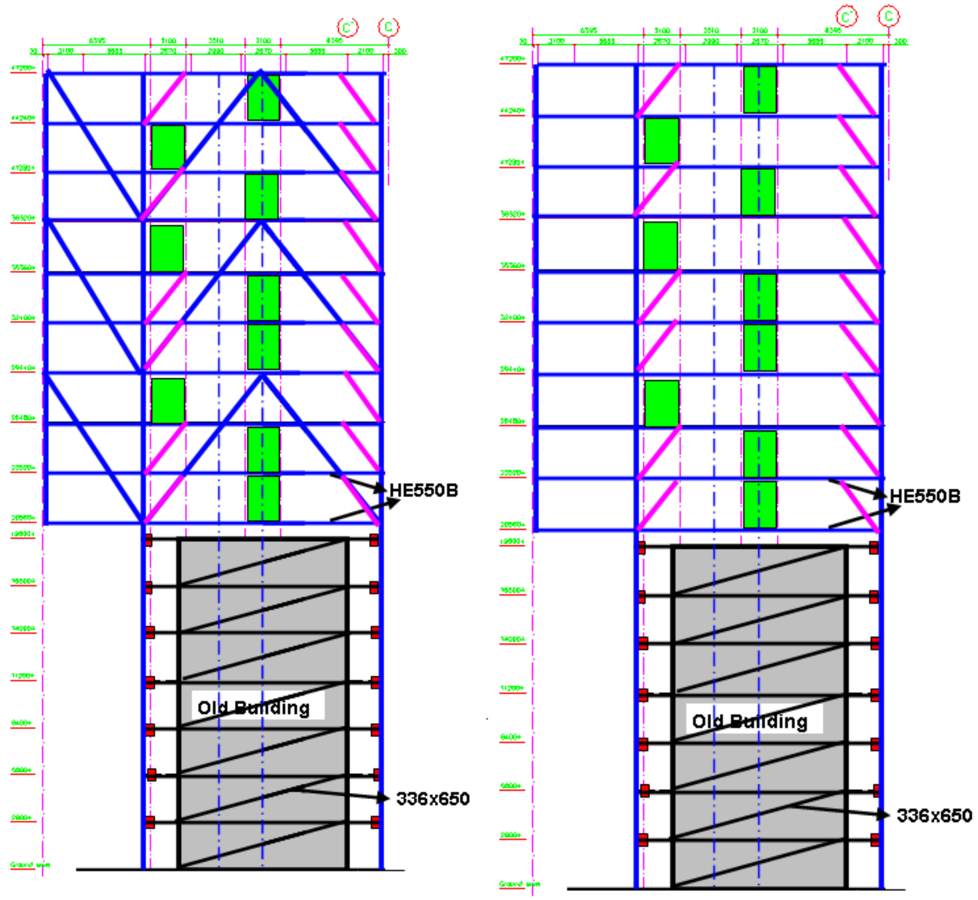


Figure11- 14: Another mode of bracing (K-Bracing) to match with cantilevers that reduce beam deflection

11.4 Conclusion

In preliminary analysis, a certain amount of horizontal deflection was registered, but in computer modeling, the consideration of the lateral spring stiffness prevented the frame from drifting. This is what happens in reality.

What matters now is the vertical deformation of beams. The maximum values obtained was 27 mm in mid spans and 18 in cantilevers; even though these values are less that the maximum allowable values, they have to be minimized since the beam spans are so large.

By using short cantilevers connecting beams and columns, the vertical deflection of beams reduces considerably. The new values are 20mm in mid span and 15mm in cantilevers.

However, these cantilevers would interfere with cross bracings. To avoid one can brace the structure by K-bracing so that cantilevers and bracings go parallel.

The loads carried by the existing structure for different load combination are so small , therefore there would not be fear for failure of the existing structure since its columns have enough reserve capacity to carry moderate loads coming to them.

12. DESIGN OF FOUNDATION

12.1 Location of the foundation piles

The foundation piles should be placed in soil with enough bearing capacity. The cone penetration test (CPT) done to the site (Figure 12-1) show that the soil with enough resistance starts 15 m below the datum (NAP -15m) .The foundation piles of the existing building are installed directly at NAP -15m [20].

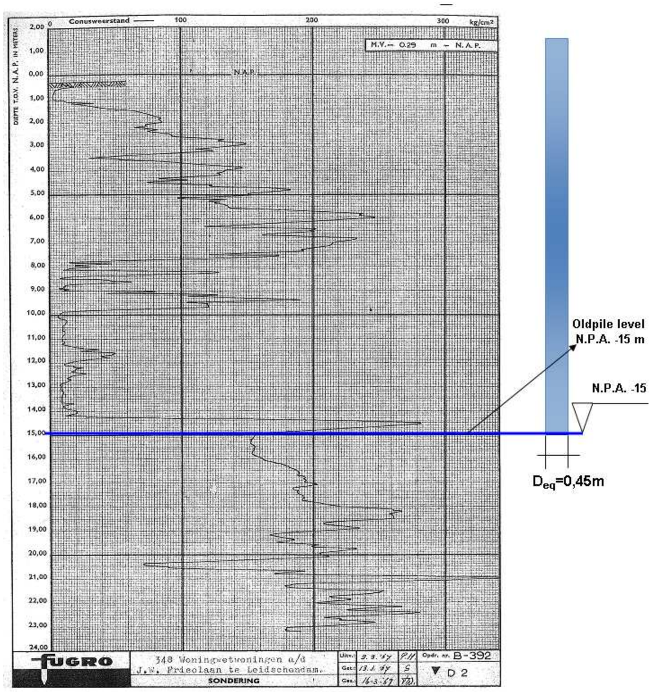


Figure 12- 1: CPT of the soil profile [20]

To avoid the soil failure envelope being developed under the already settled existing structure, the new foundation piles should be placed at the same depth as the existing ones; NAP-15m. The new building foundation piles are placed in 2m next to the existing ones, in order to respect the minimum inter-distance between piles (Figure 11-3). Figure11-2 shows the floor plan where new foundation piles should be located.

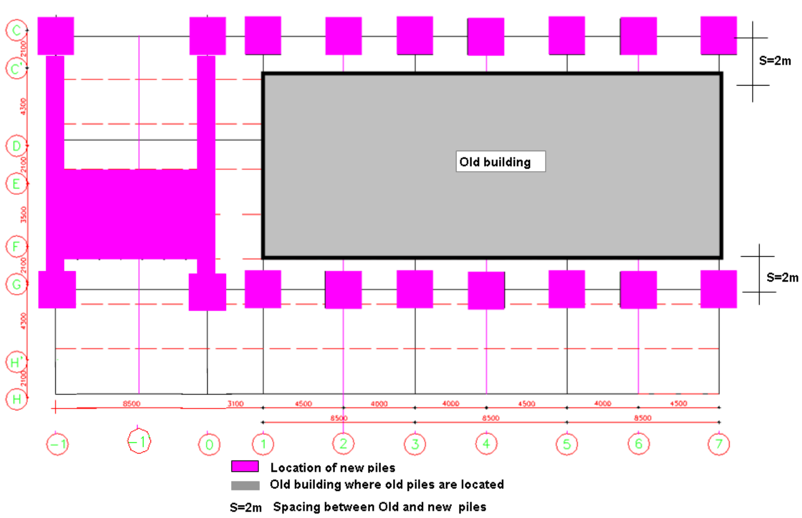


Figure 12- 3: Location of new and existing piles in plan

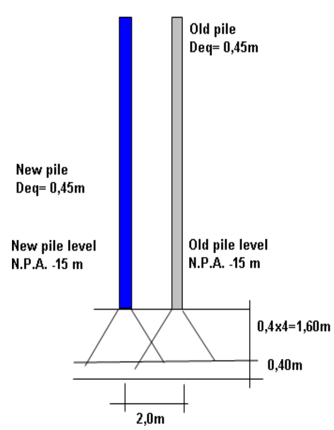


Figure 12- 2: Location of the new piles with respect to the existing piles

12.2 Choice of type of foundation

The choice of foundation method is also very important as far as this project is concerned. The new foundation is placed closer to the existing foundation. What matters is the disturbance of the already settled and stabilized foundation soil of the existing building. Without careful choice of appropriate foundation method, some damages may occur to the existing building.
It is very important to choose a method, which is free from vibration, with soil displacement, instead of removing soil.
Fundex piles are best alternative to construct the foundation for the new building. There are various sizes; namely 380,450 & 520 mm diameter; in this project the piles of 380mm diameter can chosen

12.3 Description of Fundex piles [29]

The Fundex pile section is cast in place reinforced concrete pile, installed by torque and down pressure, completely free of vibration and with no pile driving noise. It is a soil displacement pile over its full pile length. The method of installation permits no possibility of discontinuities of the pile shaft because a sealant prevents intrusion of soil or water into the incomplete pile. There are no limitations on the depth of the reinforcing cage or rebar size within the pile. There is no central tendon required, and concrete is not soil mixed, but rather pure structural concrete.
Fundex piles are also soil displacement cast-in-place piles. While the operation appears similar to a drilling job, the Fundex tip displaces the soil laterally, bringing no spoils to the ground surface. This soil-displacement quality greatly enhances the pile's capacity over comparably sized conventional drilled shafts.

A cast iron boring tip is fitted to a drilling mandrel by means of a chuck assembly. The mandrel is installed by using torque and down-pressure, producing very low noise levels and no vibration. Upon reaching the bearing layer - or upon sufficiently penetrating soil strata in the case of friction pile design - a reinforcing cage is suspended within the mandrel, and concrete is placed. The drill mandrel is then extracted by oscillation thereby leaving the tip, concrete and cage behind in place. In this way, true full section cast-in-place concrete piles are constructed.

12.3.1 Application of Fundex piles

Fundex piles are suitable for projects on which

- Drilled shafts may be appropriate, but drill spoils are to be avoided
- Space restricts the use of larger diameter drilled shafts, and higher unit friction values are required to develop capacity
- The project space is too confined for conventional precast pile driving
- Noise or vibration is not permissible
- There is a high variability in required pile depths
- Are too remote from precast pile fabricators to economically ship precast piles to the site.
- Auger cast pile construction presents quality control issues

12.3.2 Advantages of Fundex piles

- Suitable for projects inaccessible to precast piles due to space restrictions
- Roughened amplitude of concrete surface affords higher unit friction than smooth pipe or concrete pile
- Installation produces no pile driving noise or vibration
- No danger to adjacent buildings during drilling. Soil displacement so no decrease of cone resistance and no harm to existing piles.
- The only noise comes from the power pack only.

- Running sands, caving or very poor soils, or the presence of a high water table do not affect the installation of Fundex pile.
- Because Fundex piles are displacement piles, the Fundex pile affords a higher unit friction capacity than a comparable drilled shaft
- No materials are conveyed to the surface or are allowed to migrate to deeper strata- a consideration if contaminants are known to exist at a site. Drill spoil off haul is virtually eliminated.
- Fundex method allows reinforcing cages of any length, and constructs the structural shaft without necking or soil intrusion
- Fundex piles are full section soil displacement

12.4 Method of installation

The Fundex piles are constructed in four phases namely, drilling, placing reinforcement and concrete, removal of drilling tube, and finishing. (See figure 12-4)

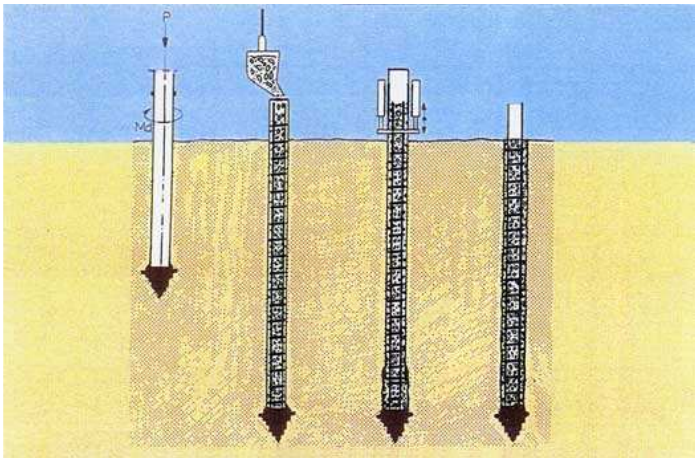


Figure 12- 4: Installation of pile [29]

Phase 1
A crawler is mounted hydraulic piling rig fitted with an hydraulic drill table with 1, 5 m stroke Hydraulic pull down. The drill point complete with a bayonet joint, is attached to a drill tube which in turn is clamped in the drill table. To commence drilling the drill table is raised to its maximum height and the drill tube is clamped. The drill tube is drilled into the ground with a pull down force on the axis and a torque. The ground is displaced, as drilling takes place.

Phase 2
After ensuring that the drill tube is dry, the reinforcing is placed in position and sufficient concrete for the complete pile is poured into the drill tube.

Phase 3
The drill tube is extracted with upward and downward oscillating movements. The drill-point forms an enlarged pile base and stays in the soil.

Phase 4
The pile is completed when the reinforcing cage is placed in position.
When the foundation level is reached, the reinforcement is placed over the anchor- cage in the pile-point, which guarantees sufficient anchor length. The drill-tube can be extracted after placing of concrete. Due to the shape of the pile-point, the Fundex piles are very suitable for tension piles.

12.5 Pile capacity and required number of piles

The capacity of the new foundation pile was calculated in Appendix 11, the calculations were made according to the Dutch code NEN6743 [3]. From the appendix 11, the maximum capacity of one foundation pile 1090KN.

The number of piles required to support the structure depends on loading applied to the structure such as permanent loads, variable load and lateral load. According to Appendix 12, the part between -1 has a total number of piles of 78. The steel frames from axis 1 to 7 will have the same number of piles since the resulting reaction force from load combination 5 (ULS) was used as the design load of foundation. The re will be 7 and 3 piles per pile cap respectively for longitudinal axes G and C . Table 12-1 and figure 12-5 give the summary of all pile to be installed.

Table 12- 1: Required number of piles

Structure		Self weight[KN]	Load from slab[KN]	Load from wind[KN]	Stair case [KN]	Total load[KN]	Pile capacity[KN]	Required number of piles
Wall -1	Wall	3434	11619		439	15492	1090	15
	Cantilever	570	2979			3549		4
	core			5364		5364		5
Wall 0	Wall	3766	15856			19622	1090	18
	Cantilever	616	4062			4678		4
	Core			5364		5364	1090	5
Small wall -1’		843		4291	439	5573	1090	6
Small wall -1’’		843		4291		5134	1090	5
Longitudinal wall E		2311		5347		7658	1090	8
Longitudinal wall F		2311		5347		7658	1090	8
Steel frame axis G								7x7
Steel frame axis C								3x7
Total number of piles								**148**

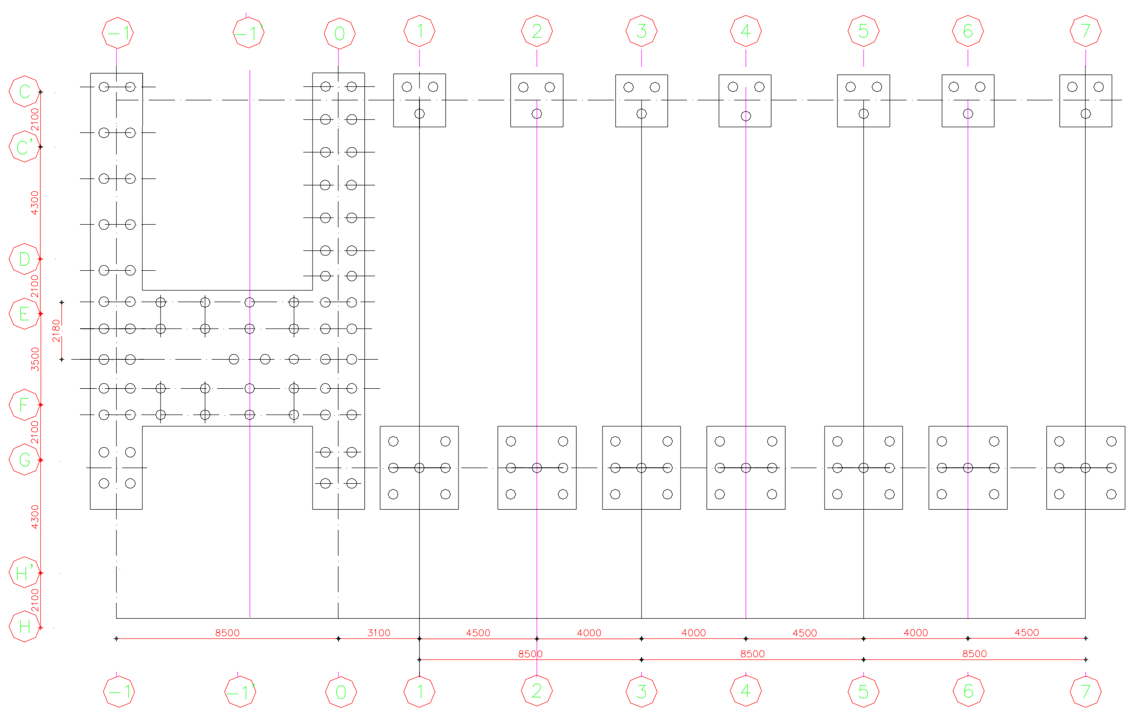


Figure 12- 5: Floor plan showing the required number of piles

For frames receiving the gravity load, the load is shared equally among piles. However, for the stabilizing structures the load from the lateral moment is added to the gravity load. The distribution of lateral load to the piles depends on location of piles; the edge piles receive more load than the inner piles. [24]

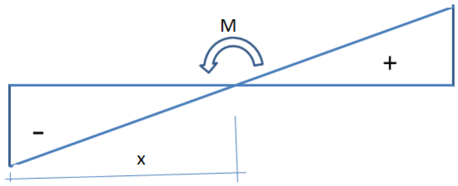


Figure 12- 6: Lateral load on foundation

The load on pile due to lateral moment is determined by using the following formula.

$P_{later.} = \frac{M.x}{I}$ With $M = \frac{1}{2} q_{later.} h^2$

X: distance from the building center to the edge pile

$$I = \sum_{1}^{i\max} n.A_i.x_i^2$$

A_i is the area of pile

n: is the number of pile

X_i is distance from the building center to the i^{th} pile

$$P_i = \frac{P}{n} + \frac{M_x.x}{I_x} + \frac{M_y.y}{I_y}$$

Where P_i : load on single pile

n : Number of piles

M_x and M_y : Lateral moments in X and y- directions

I_x and I_y : Moments of inertia in X and y- directions

x and y : Distance from the center of the building to pile I, in X and y- directions.

13. CONCLUSIONS AND RECOMMENDATIONS

13.1 Conclusions

The objective of this thesis was to formulate possible design alternatives to be used when adding more stories to the existing building. In this thesis the Dillenmburgsingle project designed by SmitsWesterman design office was used as a case study. The major problem was to know if it is possible to extend a low riser existing building into multi-storey building. This thesis gave a positive answer to this question whereby additional stories were designed by separating the frames of the new structure from that of existing building but functionary integrated. The load of the additional structure was reduced by using steel structure. This was very important as far as this project is concerned because the load to be transferred in the vicinity of the existing building must be limited to avoid disturbance in the existing foundation. It was concluded that two structures (existing and new) can be laterally connected for stability reason rather than putting directly additional stories on top of the existing building.

Different conclusions taken throughout this thesis will be summarised in this chapter.

13.1.1 Existing Building

- **Frame**.

The frame of the existing building was designed according to the applied load with safety measures but no additional future structure was planned. As the frame is stiffened by the masonry in-fill, this will attract more vertical load and columns carry less load compared to their load carrying capacity. As the two structures are connected laterally, these columns are able to handle portion of load brought by lateral loading.

- **Stability**:

The lateral stiffness of the existing is provided by the concrete frame with masonry in-fill. It has been obtained that the structure is stiff enough since the frames are provided at every 4 and 4,5m, for this, each frame receive less lateral load. Therefore, the existing building can be used to restrain a part of the lateral load by connecting the existing beams to the columns of the new structure.

- **Foundation**

The foundation of the existing building is made of concrete piles 340x340mm shaft and a tip of 450x450. Each pile was driven up to the depth of 15m (NAP-15m) and loaded with maximum load of 700KN (70Tons). The estimation of the pile capacity revealed that, each pile has a maximum capacity of 1736 KN, which means that only 40% of its capacity is utilised.

However, exploiting the remaining capacity becomes impossible since the superstructure cannot withstand additional loading. In addition too much load is concentrated to the cantilever side which would require additional piles which is practically impossible to do when the existing building is still standing. Unused capacity can be used to carry lateral loads to the soil.

Even though some columns can erected inside the existing building, but each frame of the new building brings to the ground a load of 9407KN (6658 + 2749) which can be taken by 10 Fundex piles. This is quite impossible job because the foundation beams for existing building carry 5-piles; no place left for additional piles. In addition to that it is totally impossible to make additional foundation inside the existing building without demolishing it. The simplest method to make the foundation for the new building was to separate it from the existing one.

13.1.2 New structure

- **Structural system and construction materials**

It was decided to extend the existing building form 6-stories to 15-stories by separating the new structures from the existing building. The frame of the new building is designed by bridging the existing building.

Therefore, the supporting elements should be put to the sides of the existing building.
This approach was also decided depending on the presence of the big cantilevers made on one side of the building. The cantilevers are 6m long, and by respecting the maximum cantilever to field ratio of 0, 42, the field span should be at least 14 m. This length can only be obtained on two balconies edges of the existing building. It is in this place, new supporting elements can be erected.

The span is then long; to have a successful design, light weight materials such as steel should be used for columns and walls. It was decided to use steel frame since traditional concrete structure can be structurally less efficient for such big spans and also bringing large weight near existing foundation could induce damages. Precast floor units will also be used due to its significant reduced weight and easy erection.

- **Alternatives**

Four alternatives were discussed, but the most efficient method to use is a system of two parallel continuous girders cantilevering at each floor level ; they are put on each side of the column. The system is much better to handle the cantilevers and provide the continuity of both columns and girders, making the simple connection and speedy erection. Around the core concrete walls can be used to provide the cantilevers because the presence of the core makes it impossible to have sufficient cantilever to span ratio. As this part is not to be put on top of the existing building, its weight doesn't matter and cannot have effect to the existing foundation.

- **Stability**

Different methods of stabilizing the new structure were analysed in chapter 8 and Appendix7.
All cases were checked, it has been concluded that, due to high slenderness ratio of the core, the stability cannot be fulfilled once core is used alone. The only successful method is by using the core and three braced frames in axes 1, 3 and 5. The existing building also play a significant role since all frames of the new structure are laterally connected to the frame of the existing building.

- **Foundation**

The new foundation is to be constructed near the already existing foundation. To avoid disturbance du to vibration caused by driven piling method, soil displacement method can be used. Fundex piles type (380mm shaft diameter) were selected to be used in this project; each pile with a capacity of 1090KN.

13.2. Recommendations

- **Shape**

The shape of any structure influences the force distribution in a structure. Therefore, it is important to be able to discuss shape considerations with an architect when extending an existing building. An additional building with a long cantilever is more challenging to the structural engineer. It is important when deciding to add amore story on top of the existing building to follow the shape of the existing building or by providing small cantilevers on both sides of the existing building.

- **Construction materials and framing**

Whenever additional structure is decided to be put on existing building, light weight materials should be used. It facilitate the construction of cheap foundations and the load from the new structure would not affect the foundation of the existing structure.
When deciding to add more story cantilevering the existing building, the efficient method to use continuous columns and girder. This is an easy method to make cantilevers and for big span, two beams share gravity load and take it to the columns without failure. Continuous system facilitates the erection by limiting the number of connections and therefore less horizontal drift can be achieved.

- **Stability and supporting**

In this thesis, overall stability of the new building can be achieved by concrete core associated with three steel frames on the other side of the building. As the new structure starts at high level (20m high), the support columns are so slender. The stability should be enhanced by connecting new and existing structures at each floor level .
The frame of the new structure is mainly made of two composite columns erected on the sides of the existing building.

- **Foundation:**

Soil displacement foundation is a good method to make foundation near the existing structure. This is to avoid vibration which can create cracks in existing building when driven piles would be used.
The piles should be placed on the sides of the existing building with respect to the location of the columns.

- **Future study**

In this thesis, the main field of interest was based on structural design of a 15-storey building, existing 6-storey and additional 9-storeys. In case more and more story need to be constructed, wind pressure to the building becomes strong and the whole structure would be subjected the significant dynamic loading since it is made of light weight materials.
Therefore it is recommended to make another case of tall building whereby dynamic behaviour should be analysed.

The second point for future study is to analyse thermal as wells sound comfort of occupants of such particular building with light weight materials cladding. As there were no sufficient data of soil, interaction between existing and new foundation can be provided in further research.

The third study can be made to determine proper means of lateral connection between new and existing structure. Shock absorber devices should be used to avoid possible damage when new structure moves towards the existing structure.

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APPENDICES

APPENDIX 1. HOLLOW CORE SLABS

A1.1 Introduction

Hollow core slabs have been selected to be used as floor units. They have longitudinal openings of the main purpose is to reduce the weight of the floor.
Although these floor units are mainly used in public buildings, they are also used in apartment buildings and social housing because of the interesting cost rate and the fast erection. Their depth varies between 120 to 420 mm but higher depths can also be found in other countries.

A1.2 Function

These floors will have different functions namely:

- Resisting gravity load
- Transmitting horizontal forces to the bracing system
- Acting as sound and fire barrier.

Taking gravity load and transferring horizontal loads is fulfilled when a mortar is filled between slab elements and when floor units are tied by steel reinforcement ties.

A1.3 Design information

The sizes of floor units are selected based on span length [31]. From axes -1 to 0 it has been decided to use floor units of 260mm deep, whereas 200mm is used for the part between axes 0 and 7. The floor units can work as units when a structural topping of 50mm is put during construction. The table A1-1 shows various design information required in order to make a design of the whole structure.

Table A1- 1: Design information of floor units [31]

Floor type	Self weight [KN/m^2]	Structural topping 50mm [KN/m^2]	Environ mental class	Fire resistance [min]	Concrete class	Moment of inertia [x 10^6 mm ^{4}]	Concrete cross section [mm^2]
HCS 200	3,02	0,12	XC1 , XC3	Max. 120	C45/55	677,42	143823
HCS 260	3,76	0,12	XC1 , XC3	Max. 120	C45/55	1434,98	177829

From the table above , one can see that after pouring the structural topping the resulting floor self weight is as follow:

- For HCS 200 , the self weight is **3,14 KN/m^2**
- For HCS 260 , the self weight is **3,88 KN/m^2**

APPENDIX 2. LOAD ON BUILDING

A 2.1 Vertical load

A 2.1.1 Floor slab, slab treatment and variable load

The vertical load is a combination of self weight of structural elements and variable loads. The floor load is obtained from appendix 1. The variable load used for this building was selected from the requirement of NEN6702 [1] .

A 2.1.2 Steel load

Steel load is obtained from steel section as they are calculated in Appendix 4,6,8 and 9.This consists of applying the vertical load from the slab to the intermediate frames (Figure A2-1), the axial force in each column is obtained then used to calculate the member size and hence the load. Finally the total steel load acting on intermediate frame is presented in table A2-2.

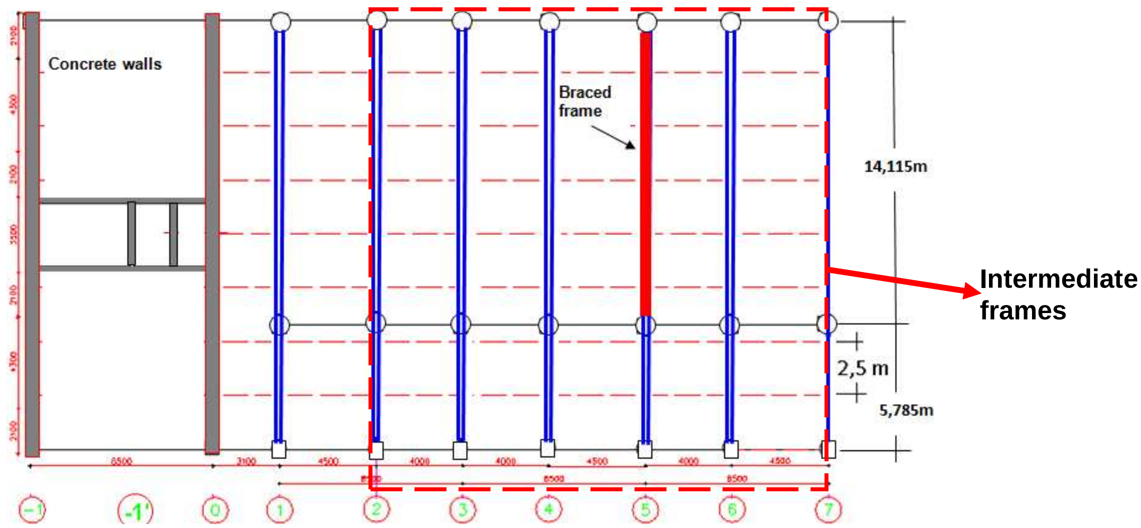


Figure A2- 1: Floor plan

Table A2- 1: Load of steel members per floor per transverse frame

Axis	H	G	C	
Type	RHS200x100x5	CHS559x23,8	CHS457x16	Total [KN]
G[KN/m]	0,22	3,14	1,74	
L[m]	2,96	2,96	2,96	
Weight[KN]	0,65	9,29	5,15	**15,10**

	Spine beams	Cross Bracing	Bracing in cantilever	Total [KN]
Type	HE550B	HE400B	HE200B	
G[KN/m]	1,99	1,55	0,61	
L[m]	19,9	(16,7 x2)/3=11,13	(10,6)/3=3,5	
Number	2	1		
Weight[KN]	79,2	17,3	2,14	**98,64**

The total steel load on transversal frame is 15,1+ 98,64= 113,74 KN
Steel load per floor per unit area :

Floor : $\frac{113,74}{19,9.4,25} \simeq 1,3KN/m^2$

Let's assume that the total steel weight is 1,5KN/m²

The total load from floor slab, floor treatment and steel member used in design is presented in table A2-1.

Table A2- 2 Load on structure from axes 0 to 7

Weight calculation				
Structure	**Type of load**	**Elements**	**Load per membe [KN/m2]**	**Total load per floor [KN/m2]**
Roof				
Roof	Dead load (DL)			
		Element	Load[KN/m2]	
		Roof floor	3,14	
		Floor finishing,Ceiling, insulation&installation	1,50	
		Steel	1,50	
		Total DL	**6,14**	
	Variable load	Snow (VL)	**1,00**	
	Total load		**7,14**	**7,14**
Floor slab				
7th floor to 15th floor	Dead load (DL)			
		Floor slab	3,14	
		Floor finishing,Ceiling, insulation&installation	1,50	
		Partition	1,00	
		Steel	1,50	
		Total DL	**7,14**	
	Variable load	VL	**1,75**	
	Total load		**8,90**	**8,90**
Gallerie and Balcons				
7th floor to 15th floor	Dead load (DL)	Floor slab	3,14	
		Floor finishing,Ceiling, insulation&installation	1,50	
		Total DL	**4,64**	
	Variable load(VL)	VL	**2,00**	
	Total load		**6,64**	**6,64**
Stair case				
7th floor to 15th floor	Dead load (DL)	slab	3,14	
		finishing	1,00	
		DL	**4,14**	
	Variable load	VL	2,00	
	Total load		**6,14**	**6,14**
Fire escape				
7th floor to 15th floor	Dead load (DL)	Total DL	1,13	
	Variable load	VL	2,00	
	Total load		**3,13**	**3,13**
Facade				
7th floor to 15th floor	Dead load		2KN/m	

Between axes -1 and 0, there is increase in self weight; therefore the total self weight is 3,88KN/m2 plus other floor treatment which give 7, 9 KN/m²

A2.2 Horizontal load

A.2.2.1 Wind load

The wind pressure acting on building sides was calculated by using NEN 6702;2007 Annex A [1]

$$P_{rep} = C_{\dim}.C_{index}.C_{eq}.\varphi_1.p_w$$

Whereby: P_{rep}: representative wind pressure

C_{di}: Dimension factor; for this building it's 0,89

C_{index}: Wind pressure and suction factors; 0.8+0.4 = 1, 2

C_{eq} ; pressure settlement factor; = 1

φ_1 : dynamic amplification factor = 1

p_w: wind pressure

$$p_w = 1 + 7l_z .0,5\rho .V_w^2(z)$$

$$V_w = 2,5.u^*.\ln(z-d_w/z_o)\ ,\ l_{(z)} = \frac{k}{\ln(z-d_w/z_o)}\ ,\ \ln(z-d_w/z_o);\ 1+7l_7\ ;\ \ 0,5.1,25.v^2$$

$l_{(z)}$: is the turbulence intensity at a height z above the adjacent ground;
ρ: is the density of air; = 1.25 kg/m3;
z_0: is the roughness length in m; 0.7
d_w : is the displacement height in meters;35
u *: is friction velocity, in m / s; 2.82
k: is a factor; 0,9
z : height
The formula can be put in excel sheet to calculate the representative wind pressure at any height (see tableA2-3)

Table A2- 3: Calculation of Wind pressure

h [m]	ln(z-dw/zo)	V_w	0,9/(ln(z-dv	1+7lz	0.5*1.25*V	p_w	p_{rep}
4	-0,34	-2,37	-2,67	-17,72	3,52	-0,06	-0,07
5	0,76	5,37	1,18	9,27	18,04	0,17	0,18
6	1,27	8,97	0,71	5,95	50,34	0,30	0,32
7	1,61	11,35	0,56	4,91	80,46	0,40	0,42
8	1,86	13,12	0,48	4,39	107,56	0,47	0,50
9	2,06	14,53	0,44	4,06	132,01	0,54	0,57
10	2,23	15,71	0,40	3,83	154,27	0,59	0,63
11	2,37	16,72	0,38	3,66	174,72	0,64	0,68
12	2,50	17,60	0,36	3,52	193,64	0,68	0,73
13	2,61	18,39	0,35	3,42	211,28	0,72	0,77
14	2,71	19,09	0,33	3,33	227,81	0,76	0,81
15	2,80	19,73	0,32	3,25	243,37	0,79	0,84
16	2,88	20,32	0,31	3,19	258,09	0,82	0,88
17	2,96	20,86	0,30	3,13	272,05	0,85	0,91
18	3,03	21,37	0,30	3,08	285,35	0,88	0,94
19	3,10	21,84	0,29	3,03	298,05	0,90	0,97
20	3,16	22,28	0,28	2,99	310,20	0,93	0,99
21	3,22	22,69	0,28	2,96	321,86	0,95	1,02
22	3,27	23,08	0,27	2,92	333,07	0,97	1,04
23	3,33	23,46	0,27	2,89	343,86	0,99	1,06
24	3,38	23,81	0,27	2,87	354,28	1,02	1,08
25	3,42	24,14	0,26	2,84	364,34	1,03	1,10
26	3,47	24,46	0,26	2,82	374,08	1,05	1,12
27	3,51	24,77	0,26	2,79	383,51	1,07	1,14
28	3,56	25,07	0,25	2,77	392,67	1,09	1,16
29	3,60	25,35	0,25	2,75	401,55	1,11	1,18
30	3,63	25,62	0,25	2,73	410,19	1,12	1,20
31	3,67	25,88	0,25	2,72	418,60	1,14	1,21
32	3,71	26,13	0,24	2,70	426,78	1,15	1,23
33	3,74	26,37	0,24	2,68	434,76	1,17	1,25
34	3,77	26,61	0,24	2,67	442,54	1,18	1,26
35	3,81	26,84	0,24	2,65	450,14	1,20	1,28
36	3,84	27,06	0,23	2,64	457,56	1,21	1,29
37	3,87	27,27	0,23	2,63	464,82	1,22	1,30
38	3,90	27,48	0,23	2,62	471,91	1,23	1,32
39	3,93	27,68	0,23	2,60	478,86	1,25	1,33
40	3,95	27,88	0,23	2,59	485,66	1,26	1,35
41	3,98	28,07	0,23	2,58	492,32	1,27	1,36
42	4,01	28,25	0,22	2,57	498,85	1,28	1,37
43	4,03	28,43	0,22	2,56	505,25	1,29	1,38
44	4,06	28,61	0,22	2,55	511,54	1,31	1,39
45	4,08	28,78	0,22	2,54	517,71	1,32	1,41
46	4,11	28,95	0,22	2,53	523,76	1,33	1,42
47	**4,13**	**29,11**	**0,22**	**2,53**	**529,71**	**1,34**	**1,43**
48	**4,15**	**29,27**	**0,22**	**2,52**	**535,56**	**1,35**	**1,44**
49	4,17	29,43	0,22	2,51	541,31	1,36	1,45
50	4,20	29,58	0,21	2,50	546,96	1,37	1,46
51	4,22	29,73	0,21	2,49	552,52	1,38	1,47
52	4,24	29,88	0,21	2,49	557,99	1,39	1,48
53	4,26	30,02	0,21	2,48	563,38	1,40	1,49
54	4,28	30,16	0,21	2,47	568,68	1,41	1,50

The results can be put in graph of height versus representative wind pressure. The figure A2-2 shows that relationship.

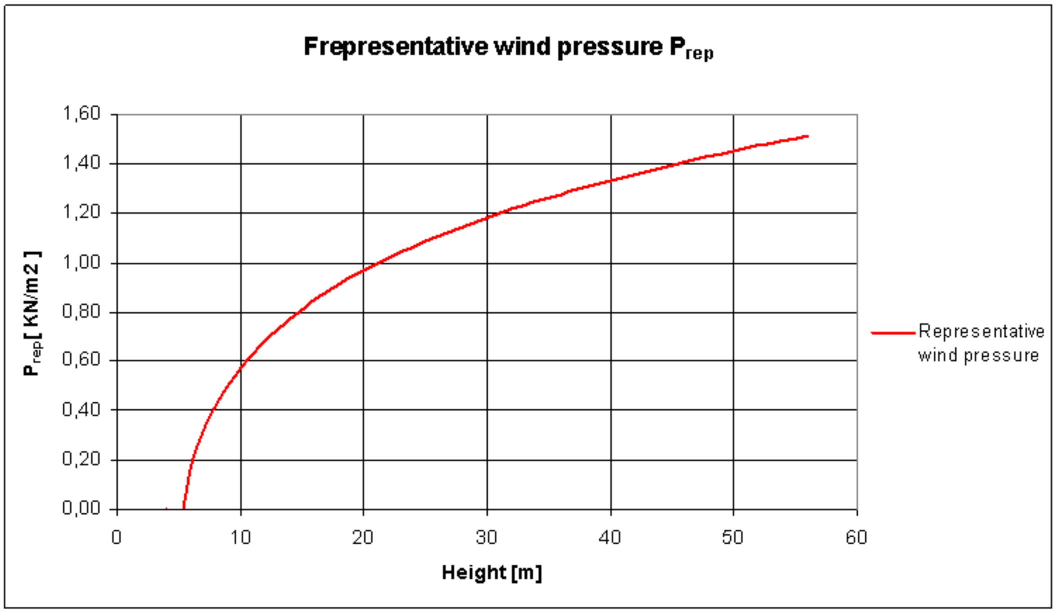


Figure A2- 2: Wind pressure against building height

From table A2-3 and figure A2-2, one can see that the representative wind pressure p_{rep} on top of the building(around 48m) is equal to 1,44 KN/m^2; This figure will be used in calculation as lateral load on structure due to wind pressure.

A2.2.2 Allowance for imperfection

The vertical load cause the frame to sway. An equivalent horizontal load acting on each story is established as a function of vertical load-Figure A2-3

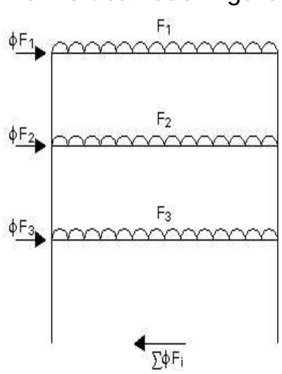


Figure A2- 3: Equivalent horizontal forces due to sway imperfection

$$H_i = \phi F_i$$

$$\phi = K_c . K_s . \phi_o$$

H_i. is equivalent horizontal force due to vertical load on story i.
F_i is vertical load on story i.
n_s is the number of storeys
n_c is the number of columns per plane
with Φ_0 = 1/325, the maximum for a system with 2-columns

$$K_c = \sqrt{\left(0,5 + \frac{1}{n_c}\right)} \leq 1 \Rightarrow \sqrt{\left(0,5 + \frac{1}{2}\right)} = 1$$

$$K_s = \sqrt{\left(0,2 + \frac{1}{n_s}\right)} \leq 1 \Rightarrow \sqrt{\left(0,5 + \frac{1}{9}\right)} = 0,56$$

Φ =1,0. 0, 56. 1/325 = 0,002

For the roof Q_{DL} =(5 . 4,25).19,9 = 423 KN
Equivalent horizontal force for roof floor is 423. 0.002 = 0,85 KN
For other floors Q_{DL} =(6,33 . 4,25).19,9 = 535 KN
Equivalent horizontal force for other floor: 535. 0,002 = 1KN

Let's consider the maximum force (1KN) and then is put in terms of uniform distributed load we have : (1+1)/2,96 = 0,7KN/m, Let's round it up and use 1,0 KN/m

The total horizontal force acting on the building per unit length will be the wind load plus the force due to imperfection depend on building sizes(see figure A2-4).The lateral load used in design is calculated in -table 2-4.

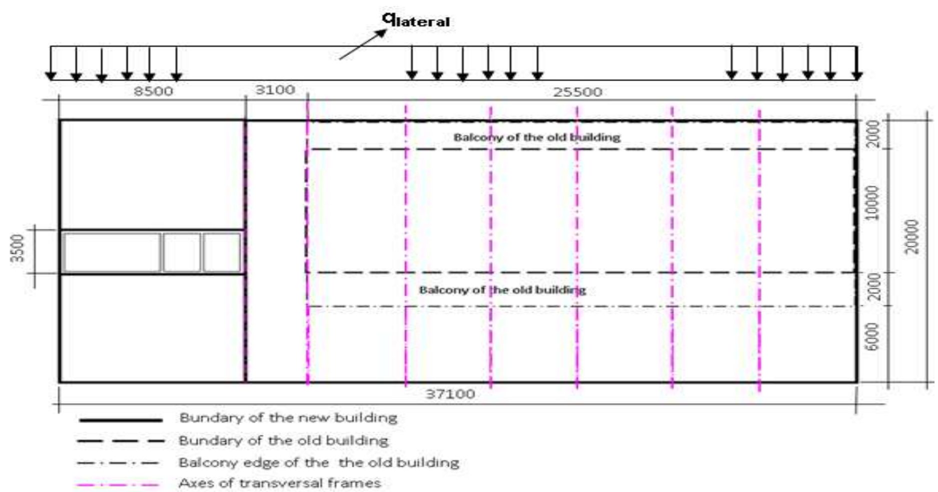


Figure A2- 4: Constructive model for wind load
Table A2- 4: Total horizontal load on building

Load	Load in SLS	Load in ULS
w	Line load [KN/m]	
	(1,44.37,1)+1=54,4	54,4.1,5 = 81,6
V	Shear force [KN]	
	54,4.47,2=2568	2568.1,5=3852

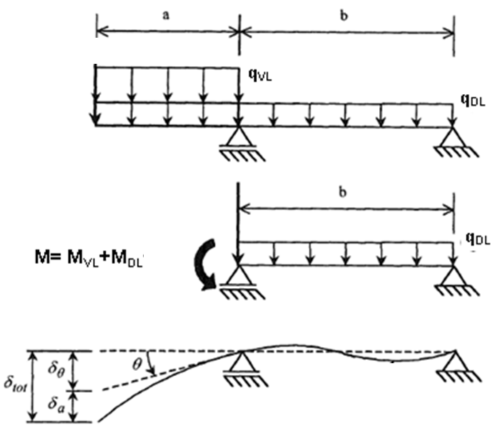
APPENDIX 3. DETERMINATION OF GIRDER SECTION

The total deflection of beam is the result from dead load and live load. The variable load can be positioned to the beam in various position and then the section of beam will be designed according to the critical load case.

$$\delta_{tot} = \delta_{DL} + \delta_{VL}$$

A3.1. Variable load on cantilever

1



- **Deflection of cantilever**

$$\delta_{Cant} = \left(\left(\frac{M_{VL} b}{3EI} + \frac{M_{DL} b}{3EI} - \frac{q_{DL} b^3}{24EI} \right).a + \frac{q_{VL} a^4}{8EI} + \frac{q_{DL} a^4}{8EI} \right)$$

$q_{DL} = 30,4KN/m$ and $q_{VL} = 7,4KN/m$ then $q = 37,8KN/m$

$$M = M_{DL} + M_{VL} = \frac{q.a^2}{2} \Rightarrow \frac{37,8.6000^2}{2} = 680,4.10^6\,Nm$$

$$\delta_{Cant} = \left(\left(\frac{680,4.10^6.14000}{3.21.10^4.I} - \frac{30,4.14000^3}{24.21.10^4.I} \right).6000 + \frac{37,8.6000^4}{8.21.10^4.I} \right)$$

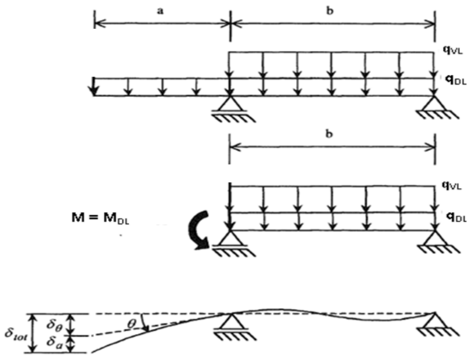
$$\left(\left(\frac{15,12.10^6}{I} - \frac{16,6.10^6}{I} \right).6000 + \frac{2,92.10^{10}}{I} \right) \Rightarrow \delta_{tot} = \frac{2,032.10^{10}}{I}$$

- **Deflection in mid span**

$$\delta_{span} = \frac{5q_{DL} b^4}{384EI} - \frac{Mb^2}{16EI},\ M = \frac{q.a^2}{2} \Rightarrow \frac{37,8.6000^2}{2} = 680,4.10^6\,Nm$$

$$\delta_{span} = \frac{5.30,4.14000^4}{384.21.10^4 I} - \frac{680,4.10^6.14000^2}{16.21.10^4 I} \Rightarrow \frac{3,27.10^{10}}{I}$$

A3.2 Variable load on beam part between supports



- **Deflection of cantilever**

$$\delta_{Cant} = \left(\frac{M_{DL} b}{3EI} - \frac{q_{DL} b^3}{24EI} - \frac{q_{VL} b^3}{24EI} \right).a + \frac{q_{DL} a^4}{8EI}$$

$$M_{DL} = \frac{q_{DL}.a^2}{2} \Rightarrow \frac{30,4.6000^2}{2} = 547,2.10^6\,Nm$$

$$\delta_{Cant} = \left(\left(\frac{547,2.10^6.14000}{3.21.10^4.I} - \frac{30,4.14000^3}{24.21.10^4.I} - \frac{7,4.14000^3}{24.21.10^4.I} \right).6000 + \frac{30,4.6000^4}{8.21.10^4.I} \right)$$

$$\delta_{Cant} = \left(\left(\frac{12,16.10^6}{I} - \frac{16,6.10^6}{I} - \frac{4,03.10^6}{I} \right).6000 + \frac{2,35.10^{10}}{I} \right)$$

$\delta_{Cant} = \dfrac{-2,732.10^{10}}{I}$, upwards deflection

Mid span

$$\delta_{span} = \frac{5q.b^4}{384EI} - \frac{Mb^2}{16EI}, \quad M = \frac{q_{DL}.a^2}{2} \Rightarrow \frac{30,4.6000^2}{2} = 547,2.10^6\,Nm$$

$$\delta_{span} = \frac{5.37,8.14000^4}{384.21.10^4 I} - \frac{547,2.10^6.14000^2}{16.21.10^4 I} \Rightarrow \frac{5,8.10^{10}}{I}$$

A3.2 The required moment of inertia

The results from the two cases of loading show that when variable loads are in mid span a bid beam section is required. As the deflection is given by this relation $\delta_{span} = \frac{5,8.10^{10}}{I}$, the required moment of inertia will be

$$I = \frac{5,8.10^{10}}{\delta_{span}}$$

APPENDIX 4. STEEL SECTION FOR COLUMN

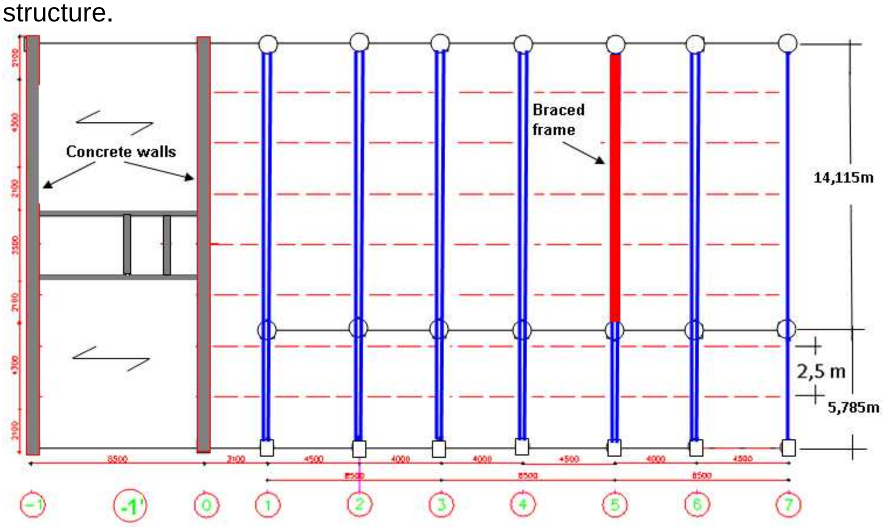
When looking at the new structure from the 7th floor, the sizes of the structural members are estimated according to the ultimate loads acting on them. Table A4-1 shows the loads used to estimate the size of structural members. By using the hollow core slabs [28], the dead load will be 7,14 KN/m2 floors. The variable loads are 1KN/m2 on the roof and 1,75KN/m2 on the apartment floors according to NEN 6702[1] and the wind load of 54,4 KN/m as it was calculated in Appendix2.

Table A4- 1: Load acting on building

Roof						
	Load [KN/m2]	Spine beams Length [m]	Spine beams spacing [m]	Total load [KN/m2]	Total point load with factor	
Dead load	7,14	19,9	4,25	30,3	1,2	36,36
Variable load	1,0	19,9	4,25	4,25	1,5	6,38
Floor						
Dead load	7,14	19,9	4,25	30,3	1,2	36,36
Variable load	1,75	19,9	4,25	7,44	1,5	11,16

Facade					
	Load [KN/m]	Length [m]	Total point load [KN]	Total point load with factor 1,2	Total distributed load with factor 1,5
Wind load	54,4				20,4
Facade load	2	4,25	8,5	10,2	

The heavily loaded columns are middle columns in axes 1to 6 (see figure A4-1). The columns size are estimated by using the rules of thumb [10], depending on the axial force due to both self and variable loads each of them bears. The diagonal (bracings) dimensions are estimated from the shear force acting on structure.



Floor system spans: 4000 and 4500mm   *Beam spans : 14115mm and 5785mm cantilever*

Figure A4- 1: Floor plan showing column position and spine beams

The columns sizes are depend on ultimate load from floors dead loads and live load acting on those floors. The dead weight is a combination of floor weight, steel frames, separating walls and floor finishes.
The table 4-1 shows the input loads, and by using the matrix frame software the load acting on each column is known.

A4.1 Design load

The model of 4-colums is made and after putting the ultimate loads stated above, the axial load in each column is obtained –figureA4-2. Those loads are used to estimate the size of columns . However the supporting columns are designed according to the total reaction forces in supports.

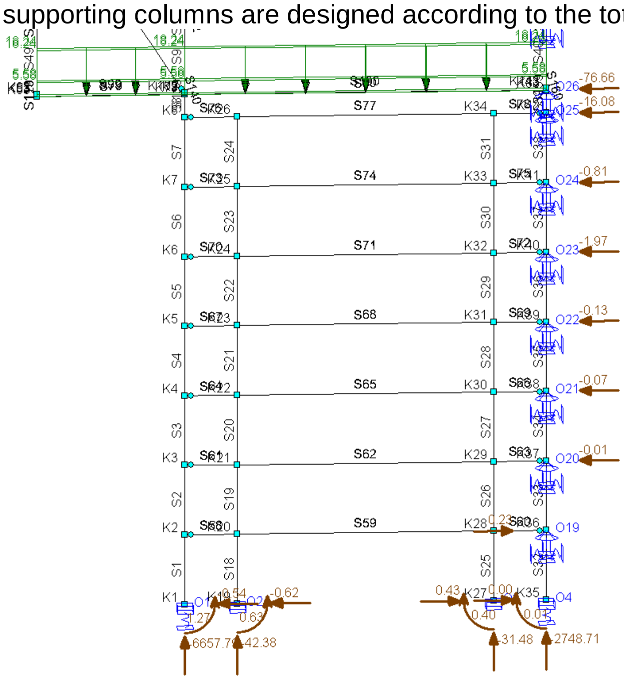


Figure A4- 1 Ultimate axial load in columns

	G	C
	Braced frame	Braced frame
Axial force[KN] = Reaction force	**6658**	**2749**

A4. 2. Estimation of column sizes

The columns sizes can be estimated by using the rules of thumb of steel [10]. The obtained size can also be checked if it fits all the stability requirements.
The circular hollow section was selected for supporting columns. It is an attractive sections the strength and stiffness in both directions are the same, in addition the circular hollow section is much better placed to torque the I or H-sections. The columns sizes are estimated and shown in in table A4-2.

- **Axis G (support columns)**

L_{buc}= 2,80m
Axial load F = 6658 KN
Steel class S235 with yield strength of 235N/mm^2
The required steel section is :

$$A_{st} = \frac{F}{f_y} \Rightarrow A_{st} = \frac{6658.10^3}{235} = 28332mm^2$$

Choose CHS 559x23,8 D=559mm t=23,8mm
A = 400cm2
Iy=143563cm4
i = 18,9cm
Wy= 6822cm3
G= 3,14KN/m
Stability Verification

$$\lambda_y = \frac{L_{buc}}{i_y} \qquad i_y = \sqrt{\frac{I_y}{A}} \qquad \lambda_e = \pi\sqrt{\frac{E_d}{f_{y,d}}} \Rightarrow 3,14\sqrt{\frac{210000}{235}} = 94 \qquad \lambda_{rel} = \frac{\lambda_y}{\lambda_e} \Rightarrow \frac{L_{buc}}{i_y.94}$$

$$\lambda_{rel} = \frac{L}{i.94} \Rightarrow \lambda = \frac{2800}{189.94} \approx 0,2 \Rightarrow \omega_{buc} = 1,0$$

$$\frac{F}{\omega_{buc}.A.f_y} \leq 1,0 \Rightarrow \frac{6936.10^3}{1,0.40000.235} = 0,74 \leq 1,0 \text{ , OK}$$

- **Axis C (support columns)**

Axis G (support column)
L_{buc}= 2,80m
Axial load F = 2749 KN
Steel class S235 with yield strength of 235N/mm^2
The required steel section is :

$$A_{st} = \frac{F}{f_y} \Rightarrow A_{st} = \frac{2868.10^3}{235} = 11698mm^2$$

Choose CHS 457x16, D=457mm t=16mm
A = 222cm2
Iy=53959cm4
i = 15,6cm
Wy= 3113cm3
G= 1,74 KN

$$\lambda_{rel} = \frac{L}{i.94} \Rightarrow \lambda = \frac{2800}{156.94} \approx 0,2 \Rightarrow \omega_{buc} = 1,0$$

$$\frac{F}{\omega_{buc}.A.f_y} \leq 1,0 \Rightarrow \frac{2868.10^3}{1,0.22200.235} = 0,55 \leq 1,0 \text{ , OK}$$

- **Axis H (Facade columns)**

Axis G (support column)
L_{buc}= 2,96m
Axial load F = 8,5 KN (Facade load)
Steel class S235 with yield strength of 235N/mm^2
The required steel section is :

$$A_{st} = \frac{F}{f_y} \Rightarrow A_{st} = \frac{8,5.10^3}{235} = 36mm^2$$

Choose RHS 200x100x5
A = 28,7cm2
Iy=1495cm4
i = 7,21cm

Wy= 408cm3
G= 0,22 KN
Since this column doesn't carry too much load it can be left without concrete filling and can be coated with insulation for fire protection.

Table A4- 2: Summary of columns size specifications

	Axis 1 to 7		
	G	C	H
Axial load from DL[KN]	7408	3252	8,5KN
Column type	CHS559 x 23,8	CHS457 x 16	RHS200 x 100x5
h or D[mm]	559	457	200
b or D [mm]	559	457	100
A [mm2]	40000	22200	2870
Iy .104[mm4]	143563	71295	1495
i [mm]	189	156	72,1
Wy [103mm3]	6822	3113	408
G[KN/m]	3,14	1,74	0,22

APPENDIX 5: DESIGN OF COMPOSITE COLUMNS

The steel columns designed in appendix 4 have relatively large dimensions. The way of reducing their size and to have the same capacity is by using composite columns. The design is made according to Euro Code 4 [5] and the Structural Hollow Sections (SHS) filled with concrete were selected because they are the most efficient of all structural steel sections in resisting compression. It is also a method of fire protection of the steel sections.
The columns are located on sides of the old building in order to carry the new structure put on top of the old building. These column will be visible and the more pleasant shape will be circular hollow section (CHS) filled with concrete. The columns in axis 1,3 and 5 will be used to design all columns in axes 1 to 7.

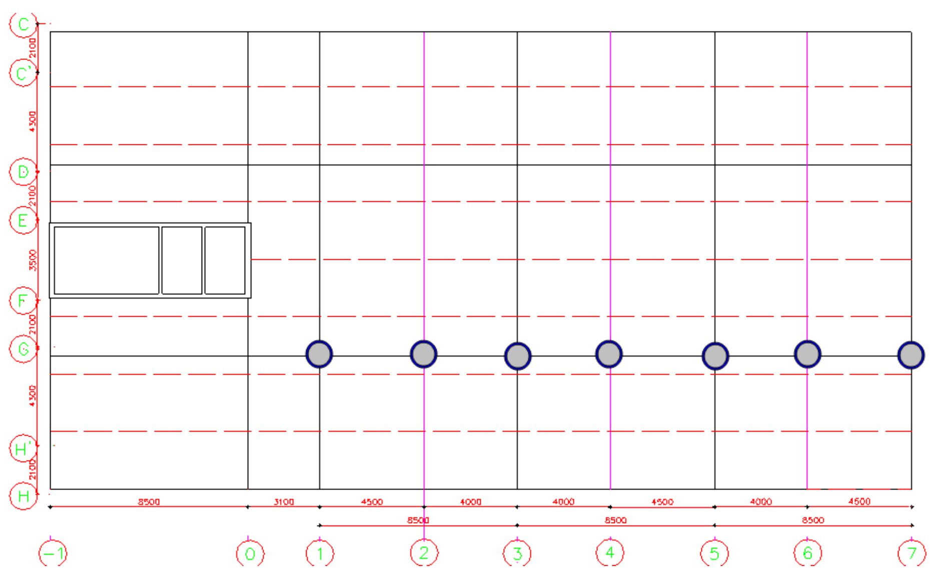
A5.1. Design of supporting columns in axis G, frame 1 to 7

These are heavily loaded columns in this section of the building. It is assumed that the circular hollow sections are to be with70% preliminary steel contribution to the load carrying capacity. The design load is 7408 KN according to the appendix 4.

(i). Material properties.

Steel
It is assumed that the rectangular hollow sections are to be with70% preliminary steel contribution to the load carrying capacity. The required steel section is

$$A_a = \frac{0.7.F}{f_y} \Rightarrow \frac{0,7.6658.10^3}{235} = 19832 mm^2$$



FigureA5- 1 Location of column section in axes G

A circular hollow section of 457 mm diameter and 16mm thickness is chosen (CHS 457x16)
The columns have to be checked for local buckling, the circular hollow sections must be satisfy this requirement:

$$\frac{h}{t} \leq 90\varepsilon^2$$

t : is the wall thickness of the steel hollow section in mm.
h: is the larger outer dimension of the rectangular hollow section in mm.

$\varepsilon = \sqrt{\frac{235}{f_y}}$ $\varepsilon = \sqrt{\frac{235}{235}} = 1,0$

f_y : is the yield strength of the steel section in N/mm².

$\frac{457}{16} \simeq 28,6$
< 90, OK
Therefore the column has the following properties:

CHS 457, 4 x 16
$f_{y;d}$ = 235 N/mm2 ($\gamma_s = 1,0$)
Ed = 210000 N/mm2
Aa = 22200 mm2, Ia = 53959. 104 mm4

Concrete.

Area of concrete:

$$A_c = \pi \frac{\left(457-(16.2)\right)^2}{4} = 141791 mm^2$$

Concrete moment of inertia:

$$I_c = \pi \frac{\left(457-(16.2)\right)^4}{64} - I_s \Rightarrow I_c = (16.10^8 - I_s) mm^4$$

The concrete strength is C25/30

$f_{ck} = 25 N/mm^2$ ($\gamma_c = 1,5$)

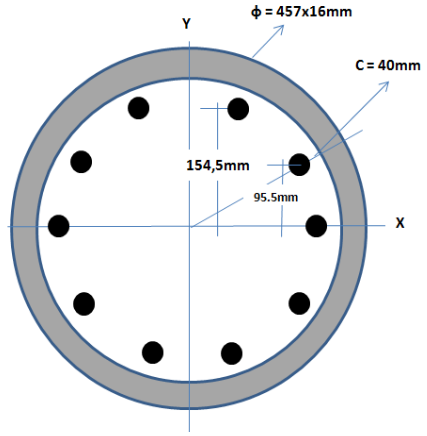
$E_{cm} = 30500 N/mm^2$ ($\gamma_s = 1,35$)

Longitudinal reinforcement steel

According to EC4 [5], the longitudinal reinforcement steel depends on concrete area and has the following limitation:

$0\% \leq \frac{A_s}{A_c} = 6\%$ $A_s = \frac{2}{100}.141791 = 2836 mm^2$

Assume As = 2% Ac ;
Use $10\phi 20 \Rightarrow A_s = 3142 mm^2$; equivalent to 2,2% which is always within the limit.



FigureA5- 2: Column section in axis G

Reinforcing steel moment of inertia

$$I_s = 6.\frac{\pi.d^4}{64} + 2\frac{A_s}{6}.2.r^2$$,

$$I_s = 10.\frac{\pi.20^4}{64} + 2.\frac{3142}{10}.2.95,5^2 + 2.\frac{3142}{10}.2.154,5^2 = 41,5.10^6 mm^4$$

f_{sk} = 500 N/mm2 $(\gamma_s = 1,15)$

Ed = 210000 N/mm2

As = 2946 mm2, Is = 45,62. 106 mm4

(ii). Calculation of composite column capacity.

$$N_{pl} = A_a \frac{f_{yk}}{\gamma_a} + A_c.\frac{f_{ck}}{\gamma_c} + A_s.\frac{f_{sk}}{\gamma_s}$$

$$N_{pl} = 22200\frac{235}{1,0} + 141791.\frac{25}{1,5} + 3142.\frac{500}{1,15} = 8946KN$$

$$N_{pl,R} = A_a.f_{yk} + A_c.f_{ck} + A_s.f_{sk}$$;

$$N_{pl} = 22200.235 + 141791.25 + 3142.500 = 10333KN$$

(iii). Check steel contribution factor

$$\delta = \frac{A_a.f_y}{N_{pl}} \Rightarrow$$

$$\delta = \frac{22200.235}{8946} = 0,58\,; \quad 0,2 < \delta < 0,9$$; OK

(iv). Column buckling resistance

The effective bending stiffness

The effective bending stiffness is given by this formula:

$$(EI)_e = E_a.I_a + 0,8E_{cd}.I_c + E_s.I_s$$

$$\left(EI\right)_e = 21.10^4.53959.10^4 + 0,8.\frac{30500}{1,35}\left(16.10^8 - 41,5.10^6\right) + 21.10^4.41,5.10^6 = 1,5.101^4 Nmm^2$$

The critical load:

$$N_{cr} = \frac{\pi^2.(EI)_e}{l^2} \Rightarrow \frac{\pi^2.1,5.10^{14}}{2800^2} \simeq 189MN$$

$$\lambda_{rel} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \Rightarrow \sqrt{\frac{10333}{189000}} = 0,23 < 2 \quad \omega_{buc} = 1,0$$

Influence of long term deformation (shrinkage and creep).
For concrete filled hollow section , the relative slenderness has the following limitations

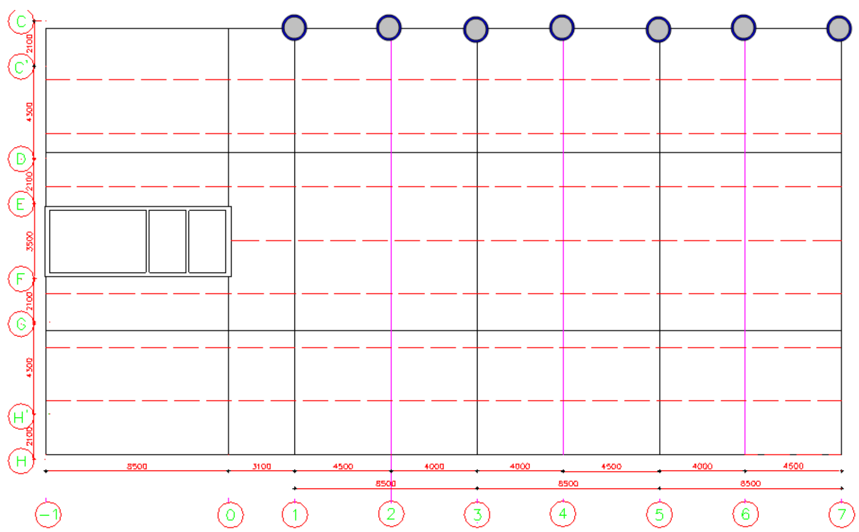
$$\lambda_{rel} = \frac{0,8}{(1-\delta)} \Rightarrow \frac{0,8}{(1-0,59)} = 1,95$$; 0,2< 1,95 ; neglect shrinkage and creep.

For $\lambda_{rel} = 0,5$; $\omega_{buc} = 0,92$

$N_{Rd} = \omega_{buc}.N_{Pl} \Rightarrow 0,92.8946 = 8230KN$; Hence $> N_{Sd} = 6658KN$

A5.2. Design of supporting columns in axis C , frame 1 to 7.

These are also the supporting columns but not heavy as for the axis G. It is assumed that the circular hollow sections are to be used with70% preliminary steel contribution to the load carrying capacity. The design load is 2749 KN according to the appendix 4.



FigureA5- 3 *Column location in axis C*

(i). Material properties.
Steel

The required steel section is $A_a = \frac{0.7.F}{f_y} \Rightarrow \frac{0,7.2749.10^3}{235} = 8189mm^2$

A circular hollow section of 406,4 mm diameter and 8mm thickness is chosen (CHS 406,4 x 8 , A = 10000mm2)
The columns has to be checked for local buckling , the circular hollow sections must be satisfy this requirement : $\frac{h}{t} \leq 90\varepsilon^2$
t : is the wall thickness of the steel hollow section in mm.
h: is the larger outer dimension of the rectangular hollow section in mm.

$$\varepsilon = \sqrt{\frac{235}{f_y}} \quad \varepsilon = \sqrt{\frac{235}{235}} = 1,0$$

f_y : is the yield strength of the steel section in N/mm².

$\frac{406,4}{8} \simeq 51$ < 90 , OK

Therefore the column has the following properties:
CHS406,4 x8

$f_{y;d}$ = 235 N/mm2 ($\gamma_s = 1,0$)
Ed = 210000 N/mm2
Aa = 10000 mm2, Ia = 19874. 104 mm4

Concrete.

Area of concrete: $A_c = \pi \frac{\left(406,4-(8.2)\right)^2}{4} = 119644 mm^2$

Concrete moment of inertia: $I_c = \pi \frac{\left(406,4-(8.2)\right)^4}{64} - I_s \Rightarrow I_c = (11,4.10^8 - I_s) mm^4$

The concrete strength is C25/30

$f_{ck} = 25 N/mm^2$ ($\gamma_c = 1,5$)

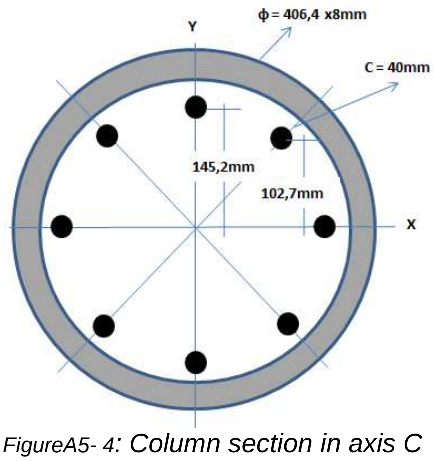
$E_{cm} = 30500 N/mm^2$ ($\gamma_s = 1,35$)

Longitudinal reinforcement steel

According to EC4 , the longitudinal reinforcement steel depends on concrete area and has the following limitation: $0\% \leq \frac{A_s}{A_c} = 6\%$

Assume As = 2% Ac ; $A_s = \frac{2}{100}.119644 = 2393 mm^2$

Use $8\phi 20 \Rightarrow A_s = 2512 mm^2$; equivalent to 2% which is always within the limit.



FigureA5- 4: Column section in axis C

Reinforcing steel moment of inertia: $I_s = 8.\frac{\pi.d^4}{64} + 2\frac{A_s}{8}.2.r_1^2 + \frac{A_s}{8}.2.r_2^2$,

$$I_s = 8.\frac{\pi.20^4}{64} + 2.\frac{2512}{8}.2.102,7^2 + 1.\frac{2512}{8}.2.145,2^2 = 26,6.10^6$$

f_{sk} = 500 N/mm2 ($\gamma_s = 1{,}15$)
Ed = 210000 N/mm2
As = 2512 mm2, Is = 26,6. 106 mm4

(ii). Calculation of composite column capacities.

$$N_{pl} = A_a \frac{f_{yk}}{\gamma_a} + A_c.\frac{f_{ck}}{\gamma_c} + A_s.\frac{f_{sk}}{\gamma_s}$$

$$N_{pl} = 10000\frac{235}{1{,}0} + 119644.\frac{25}{1{,}5} + 2512.\frac{500}{1{,}15}$$

$$N_{pl} = 2350 + 1994 + 1093 = 5437KN$$

$$N_{pl,R} = A_a.f_{yk} + A_c.f_{ck} + A_s.f_{sk}\ ;$$

$$N_{pl,Rk} = 10000.235 + 119644.25 + 2512.500 = 6597KN$$

(iii). Check steel contribution factor

$$\delta = \frac{A_a.f_y}{N_{pl}} \Rightarrow \frac{10000.235}{5437.10^3} = 0{,}43 \qquad 0{,}2 < \delta < 0{,}9\ \text{; Ok}$$

(iv). Column buckling resistance
The effective bending stiffness

The effective bending stiffness is given by this formula: $(EI)_e = E_a.I_a + 0{,}8E_{cd}.I_c + E_s.I_s$

$$(EI)_e = 21.10^4.19874.10^4 + 0{,}8.\frac{30500}{1{,}35}\left(11{,}4.10^8 - 26{,}6.10^6\right) + 21.10^4.26{,}6.10^6$$

$$(EI)_e = 4{,}2.10^{13} + 2.10^{13} + 5{,}6.10^{12} = 6{,}76.10^{13}\ Nmm^2$$

The critical load:

$$N_{cr} = \frac{\pi^2.(EI)_e}{l^2} \Rightarrow \frac{\pi^2.6{,}76.10^{13}}{2800^2} \simeq 85MN$$

$$\lambda_{rel} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \Rightarrow \sqrt{\frac{6597}{85000}} = 0{,}3 < 2 \quad \omega_{buc} = 1{,}0$$

Influence of long term deformation (shrinkage and creep).
For concrete filled hollow section , the relative slenderness has the following limitations

$$\lambda_{rel} = \frac{0{,}6}{(1-\delta)} \Rightarrow \frac{0{,}6}{(1-0{,}43)} = 1{,}05$$

0,3< 1,05 ; neglect shrinkage and creep

$\lambda_{rel} = 0{,}3$ and $\omega_{buc} = 0{,}98$

.

For $\lambda_{rel} = 0{,}5$; $\omega_{buc} = 0{,}92$

$$N_{Rd} = \omega_{buc}.N_{pl} \Rightarrow 0{,}98.5437 = 5328KN \qquad > N_{Rd} = 2749KN$$

APPENDIX 6. DESIGN OF SPINE BEAM

The spine beams run in lateral direction of building and are loaded with Uniform distributed load from floor units, the average spacing between spine beams is 4,25m.
Fro beam design the following have to be considered:

(i). Ultimate limit state on critical span: strength
(ii). Serviceability limit state; Deflection

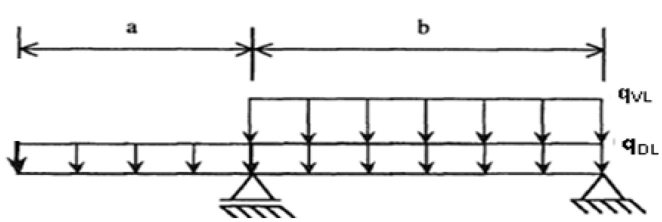
Loading: Dead load: 7, 14/ m^2
Variable load: 1, 75 KN/m^2

TableA6- 1: Load on spine beam

	Load [KN/m^2]	Beam spacing[m]	UDL [KN/m]	Point load [KN]	Design load SLS [KN/m]	Design load ULS [KN/m]	
						Factor	Load
DL	7,14	4,25	30,35		67,3	1,2	81
LL	1,75	4,25	7,44		18,6	1,5	28
Façade	2KN/m		4,25	8,5KN	8,5KN	1,2	10,2KN

A6.1 Design by additional deflection

Total deflection in mid-span can be used to estimate the bending stiffness of the girder. The formulas are established in appendix 3 and the design formula for moment of inertia was obtained when live load is in mid span.



$$I = \frac{5,8.10^{10}}{\delta_{span}}$$

(a) Continuous girder with variable load in mid field

FigureA6- 1: Moment of inertia of continuous beam

The deflection should be limited to 10 mm [24]. Therefore the moments of inertia of the required steel section is: $I = \frac{5,8.10^{10}}{\delta_{span}} \Rightarrow \frac{5,8.10^{10}}{10} = 5,8.10^9 mm^4$ **or** $580000 cm^4$

Two parallel girders of HE700M can be used for intermediate frames.

$I = 329300 cm^4$ each

HE 700M ; $I_y = 329300 cm^4$; $W_y = 9198 cm^3$; $W_{y,pl} = 10540 cm^3$; G=2x1,99= 3,01KN/m

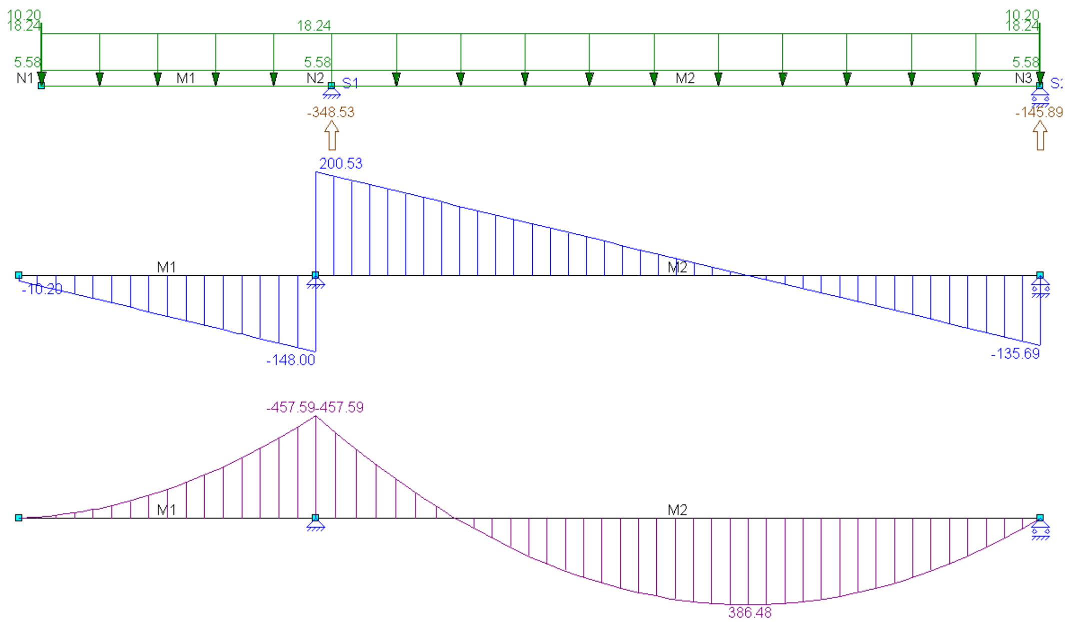
A6.2 Verification of Spine by matrix frame model

The loads on spine beam are shown in Table A61, where the dead load from the slab is 30,35KN/m and point load at the beam edge of 8,5KN from the facade
Variable load: 7,44 KN/m

As the girders are double beam; therefore the loads will be halved (see figure A6-2). The dead load was combined with live load by changing the position of the latter and the following combinations were obtained:

- Live load on cantilever only
- Live load in middle span
- Live load on both middle span and cantilever

The third case gave higher external reactions and was considered for design; the girder section was checked at ultimate limit state for the third load combination as the results are given in figure A62



FigureA6- 2: External forces at ULS on spine beam in axis

The maximum moment is located on cantilever part, the cantilever will be governing and the size which will be obtained will be the same for entire girder.
The obtained maximum bending moment is 458 KNm.
Assume the steel grade to be S235

A6.3 Checking the cross-section of the girder

Therefore the required minimum section modulus $W_y = \frac{M}{f_y} \Rightarrow \frac{458.10^6}{235} = 1949 cm^3 < 9198 cm^3$

The moment capacity of the beam: $M_{pl} = W_{y,pl} \cdot f_y \Rightarrow 9198.10^3.235 = 2162 KNm > 458 KNm$:

$M_{sd} = 458 KNm < M_{pl} = 2162 KNm$

The shear resistance of the steel section : $V_{pl} = \frac{f_y}{\sqrt{3}}.(1,04 h_w.t_w) = \frac{235}{\sqrt{3}}.(1.04.636.21) = 1885 KN$

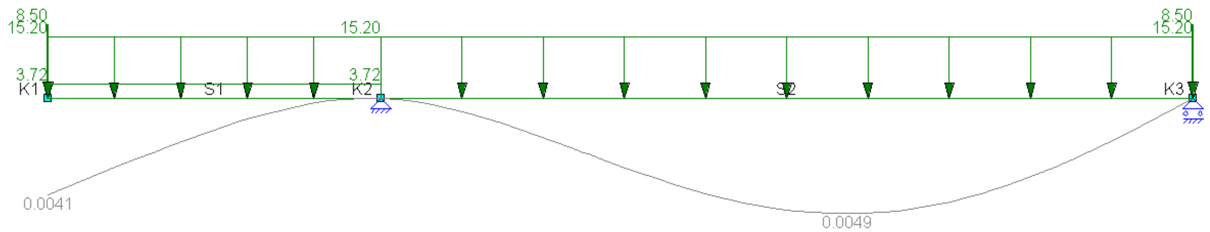
The applied shear force $V_{Rd} = 349 KN < 0,5 V_{pl}$, no interaction with moment.

A6.4 Checking the deflection of the girder

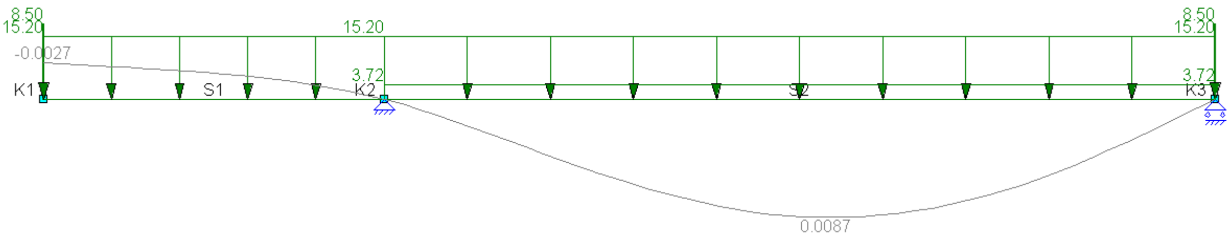
The maximum deflection of the spine beam at cantilever should be $\delta_{\max;cant} = \frac{L}{250} \Rightarrow \frac{5785}{250} = 23mm$ and in the middle span $\delta_{\max;\,field} = \frac{L}{250} \Rightarrow \frac{14115}{250} = 56mm$

In serviceability limit state, the maximum deflection of the cantilever is obtained in load case 1; 10mm (Figure A6-3,this is less than the allowable deflection of 23mm.
In the middle span , load case 2 give a deflection of 21mm(Figure A8-3) which is less than 56mm, the allowable maximum deflection in middle field.

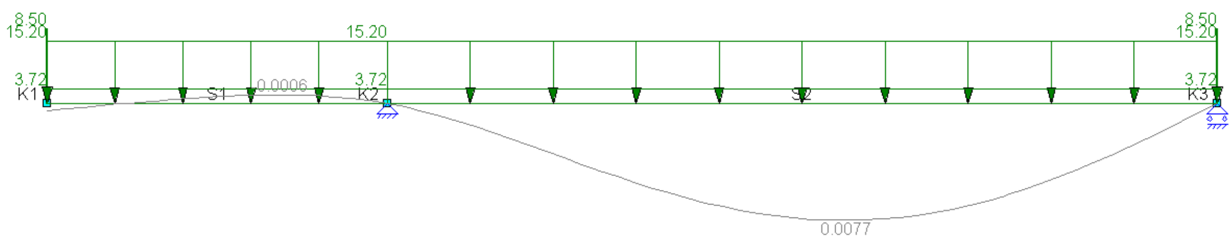
LC1



LC2



LC3



FigureA6- 3: Beam deflection

APPENDIX 7. STABILITY ALTERNATIVES FOR THE NEW STRUCTURE

When a building is loaded with horizontal load, it tends to deflect and the structure can collapse when horizontal deflection is large.
To avoid this, stabilizing elements have to be provided .In this appendix the calculation will be made to see if concrete core can be sufficient to stabilize the whole structure , if not , other alternatives will be made in order to have the whole building fully stabilized.

The total horizontal force acting on the building per unit length is 54,4KN/m (see Appendix 2).
The NEN6702 [1] provides the requirements for maximum allowable horizontal deflection.

$u_{all} = \frac{1}{500}h$, where h, is the total height of the building, and u_{all} , the allowable horizontal deflection

therefore $u_{all} = \frac{1}{500}47200 = 94,4mm$

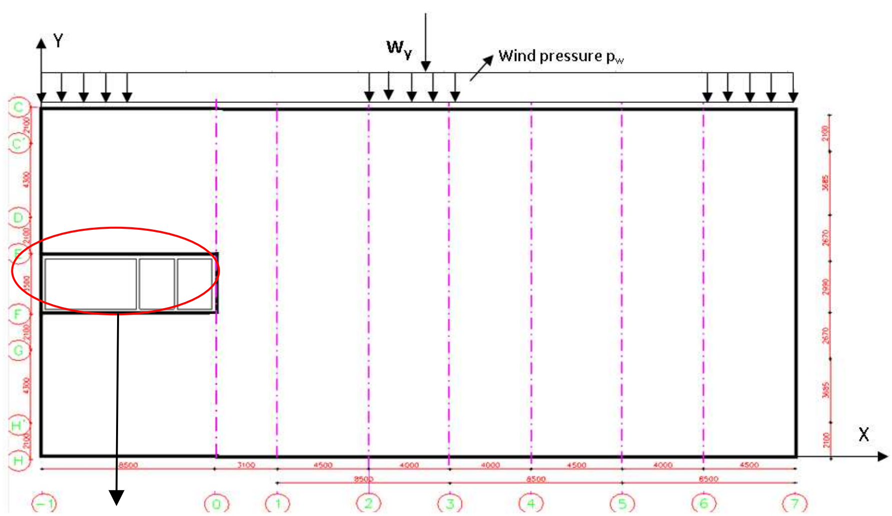
Furthermore, the additional calculation of the deflection due to the rotation of the foundation and deflection due to the second order effects can also be
Included. The rotation of the foundation makes an increase of $1/4$ the total deflection (1, 33 times) and the second order effect for an increase of 20% (1, 20 times).
Then the new allowable horizontal deflection is:

$$u_{all} = \frac{1}{1,2.1,33.500}h \Rightarrow \frac{1}{1,2.1,33.500}47200 = 59mm$$

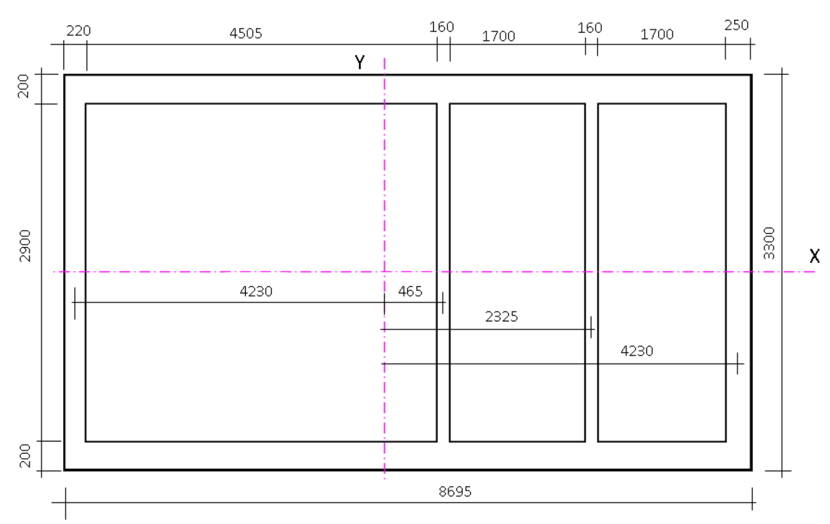
A7. 1 Concrete core working alone

The concrete core can be checked if it can restrain the lateral load alone. It will depend on its bending stiffness, that is why the moment of inertia of the core will be first calculated and then the horizontal deflection can be checked.

A7.1.1 Calculation of the concrete core moment of inertia



a. Floor plan



b. Concrete core.
FigureA7- 1: Floor plan showing concrete core as lateral load restraining structure

The total area : 8,695x 3,300= 28,7.10^6 mm^2
The area of voids: (4,505 x 2,900) + [2(1700 x 2900)] = 22, 9.106 mm^2
The net Area: 28,7.10^6 – 22,9.10^6 = **5,8. 10^6 mm^2**

- **Moment of inertia in X- axis (I)**

In X-axis, the moment of inertia will be the sum of the individual moment of inertia of transverse walls , plus the contribution of the longitudinal walls.

$$I = \frac{1}{12}220.3100^3 + \frac{1}{12}250.3100^3 + 2.\frac{1}{12}160.3100^3 + 2.8460.200.1550^2 = 10,1.10^{12}\,mm^4$$

- **Polar moment of inertia in X- axis (I_t)**

$I_t = I_x + I_y$, with $I_y = I_{yc} + Ad_y^2$

$$I_x = I_{xc} + Ad_x^2$$

With thin walled section, I_{xc} and I_{xc} are negligible; $I_t = Ad_x^2 + Ad_y^2$

$$I_t = \left[3100.220.4230^2 + 3100.160.465^2 + 3100.160.2325^2 + 3100.250.4230^2\right] +$$

$$2.8460.200.1550^2$$

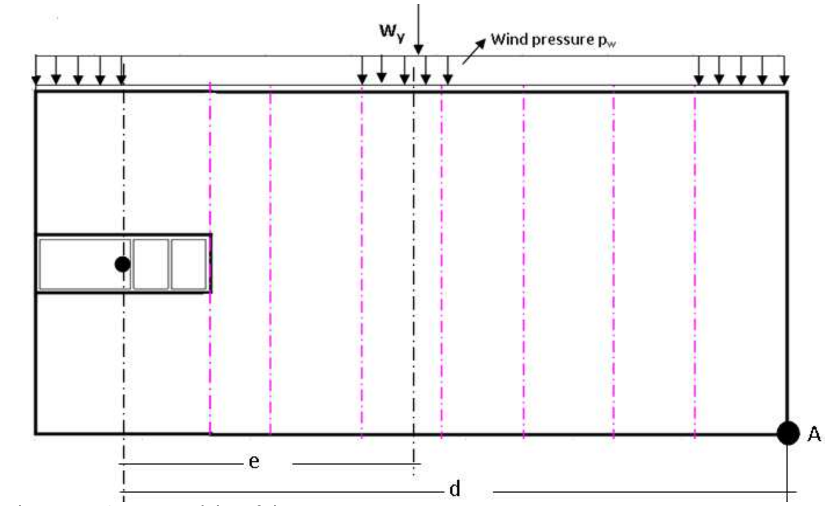
$$I_t = 37.10^{12}\,mm^4$$

A7.1.2 Main stabilizing structure derivation equations

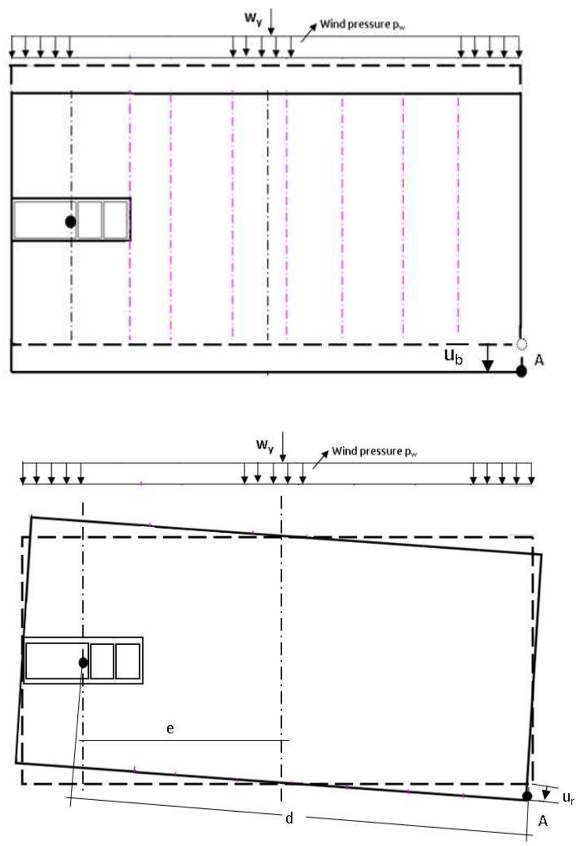
As the core is eccentrically located in the floor plan, the wind load causes two kind of movement considering the far away point (A) from the core:

u_r : movement due to rotation

u_b : movement due to bending



FigureA7- 2: Eccentricity of the core



FigureA7- 3: Possible movement (translation and rotation) of the building due to lateral load

- **Bending** u_b

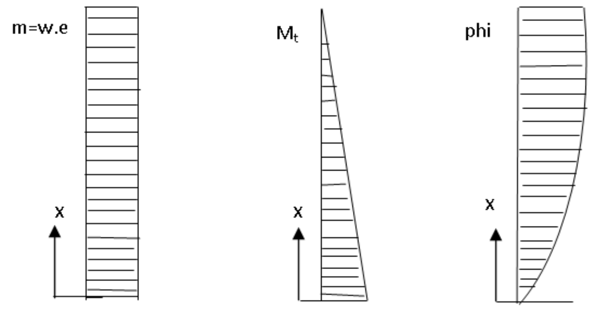
The bending displacement is given by the following relation:

$$u_b = \frac{wh^4}{8EI},$$

Where
w: wind load
h: building height
I : moment of inertia
E: Modulus of elasticity of concrete

- **Rotation** u_r

u_r , is determined by the angle "phi" and the distance (d) from the rotational center to the point A.
The equation for the angle "phi" is from the Mt-line



FigureA7- 4: m, Mt, and phi diagrams

The equation for M_t is: (1). $M_t = -mx + C_1$

φ is obtained by integrating the equation (1): (2). $GI_t\varphi = -\frac{1}{2}mx^2 + C_1x + C_2$

The integration constants C_1 and C_2 are calculated from the following boundary conditions:

(1). $x = h \rightarrow M_t = 0 \rightarrow -m.h + C_1 = 0$

$C_1 = m.h$

$M_t = m(h - x)$

(2). $x = 0 \rightarrow \varphi = 0 \rightarrow -\frac{1}{2}m0^2 + h.0 + C_2 = 0$

$C_2 = 0$

$GI_t\varphi = \frac{1}{2}mx(2h - x)$

$\varphi_{\max}$ is obtained at $x = h$

$GI_t\varphi_{\max} = \frac{1}{2}mh(2h - h)$

$GI_t\varphi_{\max} = \frac{1}{2}mh^2$

$\varphi_{\max} = \frac{1}{2}\frac{mh^2}{GI_t}$

u_r is $\varphi_{\max}$ multiplied by 'd'

$u_r = \varphi_{\max}.d \Rightarrow \frac{1}{2}.\frac{mh^2}{GI_t}.d$ Where: $m = w.e$

w = wind load
h = Height of the building
I_t = Torsion moment of inertia

$G = \frac{E}{2(1+v)}$ with $v = 0,2$; $G = \frac{5}{12}E$

By replacing the G-value in the equation of u_r , one obtains,

$$u_r = \frac{1}{2}\cdot\frac{mh^2}{\left(\frac{5}{12}\right)EI_t}.d = \frac{6}{5}\cdot\frac{(w.e)h^2}{EI_t}.d$$

$$u_r = \frac{6}{5}\cdot\frac{wh^2}{EI_t}.e.d$$

Finally the equation for the movement of point A is the sum of u_b and u_r

$$u_A = \frac{wh^4}{8EI} + \frac{6}{5}\frac{(w.e)h^2}{EI_t}.d$$

Where: d is distance from core center to the point A
e: is the eccentricity of core with respect to the center of rotation

A7.1.3 Check for the actual deflections

The deflection of the building edge (point A) can be calculated using the obtained formula. In precast structure, high Concrete grade is required. By using B45, its properties are shown in tableA7 –according to NEN-EN 2 [17]

TableA7- 1: Concrete B45 properties

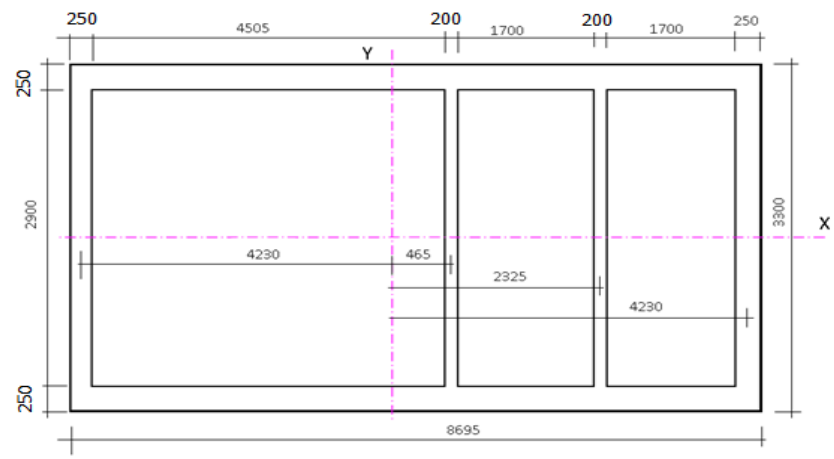
Concrete grade	B45
f_{ck} [N/mm^2]	45
f_{cm} [N/mm^2]	53
f_{ctm} [N/mm^2]	3,8
E_{cm} [kN/mm^2]	36

$$u_A = \frac{54,4.47200^4}{8.36000.10,1.10^{12}} + \frac{6}{5}\frac{(54,4.14320)47200^2}{36000.37.10^{12}}.32870$$

$$u_A = 93 + 51 = 144mm$$

A7.1.4 increasing the sizes of walls that make the core.

As the horizontal deflection is too big due to small lateral stiffness of the core, one can increase the sizes of some walls(see Figure A8-5) and thereafter recalculating the moment of inertia as well as the deflection.



FigureA7- 5: Increased core dimensions

- **The new moment of inertia**:

$$I = 2.\frac{1}{12}250.3100^3 + 2.\frac{1}{12}200.3100^3 + 2.8460.250.1550^2 = 12,4.10^{12}mm^4$$

$$A_{Core} = 8695.3500 - \left(4505.3000 + 2(1700.3000)\right) = 6717500mm^2$$

- **The new polar moment of inertia in X- axis (I_t)**

$$I_t = \left[3100.250.4230^2 + 3100.200.465^2 + 3100.200.2325^2 + 3100.250.4230^2\right] +$$

$$2.8460.250.1550^2$$

$$I_t = 41,4.10^{12}mm^4$$

$$u_A = \frac{wh^4}{8EI} + \frac{6}{5}\frac{(w.e)h^2}{EI_t}.d \Rightarrow \frac{54,4.47200^4}{8.36000.12,4.10^{12}} + \frac{6}{5}\frac{(54,4.14320)47200^2}{36000.41,4.10^{12}}.32870$$

$$= 122mm$$

Even if the core dimension are increased but yet the resulting deflection continues to be bigger than the required value. Other alternatives can be developed.

A7.2 Concrete core with the existing building

A7.2.1 Upper part

- **Forces in bracing structures**

The bracing structure is only the core, it is subjected to translation and rotation. Each component is calculated and summed up to get the total force acting on bracing structure.

$$F_{y,transl.;i} = \frac{I_{y,i}}{\sum_{i=1}^{n} I_{y,i}}.W_y \text{ and } F_{y,rotation,i} \frac{I_{y,i}.\overline{x}_i}{\sum_{i=1}^{n}\left(I_{y,i}.\overline{x}_i^{\,2} + I_{x,i}.\overline{y}_i^{\,2}\right)}.M_T$$

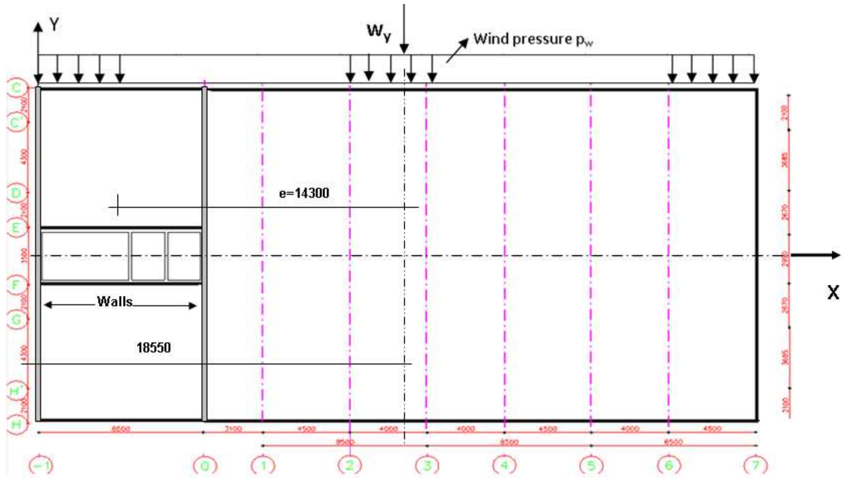
$I_{y,i}$ and $I_{x;i}$: moment of inertia of bracing structures i, in Y and X- direction

W_y : Total lateral forces action on floor level

$F_{y,transl.;i}$ and $F_{y,rotation.;i}$: Translation and rotation force acting on bracing structure i

$\overline{x}_i$ and $\overline{y}_i$: Distance from rotation center to structure I, in X and Y – direction.

M_T :Tortional moment $M_T = W_y.e$; where e is the eccentricity



- **Moment of inertia of concrete core**

Concrete core: $I_y = 12,4.10^{12}\,mm^4$ and $I_x = 76,8.10^{12}\,mm^4$

- **Moment of inertia of steel frame and old building**

$I_{old,1-7} = A_F.L_F{}^2 + A_{C'}.L_{C'}{}^2 \quad I_{old,1-7} = 510.210.1765^2 + 510.210.8150^2 = 7,45.10^{12}\,mm^4$

$$I_{y,\text{in-fill}} = \frac{210.10000^3}{12} = 1,75.10^{13}\,mm^4$$

$$I_{y,total} = 17,5 + 7,45 + 16 = 41.10^{12}\,mm^4$$

$I_{x,total} = 0$ is negligible for each frame

$$F_{y,transl.;i} = \frac{I_{y,i}}{\sum_{i=1}^{n} I_{y,i}}.W_y = 2018KN$$

$$F_{y,rotation,i}\ \frac{I_{y,i}.\overline{x}_i}{\sum_{i=1}^{n}\left(I_{y,i}.\overline{x}_i^{\,2} + I_{x,i}.\overline{y}_i^{\,2}\right)}.M_T\ ,\ F_{y,rotation,i}\ \frac{12,4.10^{12}.4,250}{\left(12,4.10^{12}.4,250^2 + 76,8.10^{12}.0^2\right)}.2018.14,300 = 6790KN$$

A7.2.2 Lower part

- **Location of the rotation center**

$$\overline{x}_c = \frac{\sum_{i=1}^{n} I_{y,i}.x_i}{\sum_{i=1}^{n} I_{y,i}} \qquad \overline{y}_c = \frac{\sum_{i=1}^{n} I_{x,i}.y_i}{\sum_{i=1}^{n} I_{x,i}} \Rightarrow \overline{y}_c = 0$$

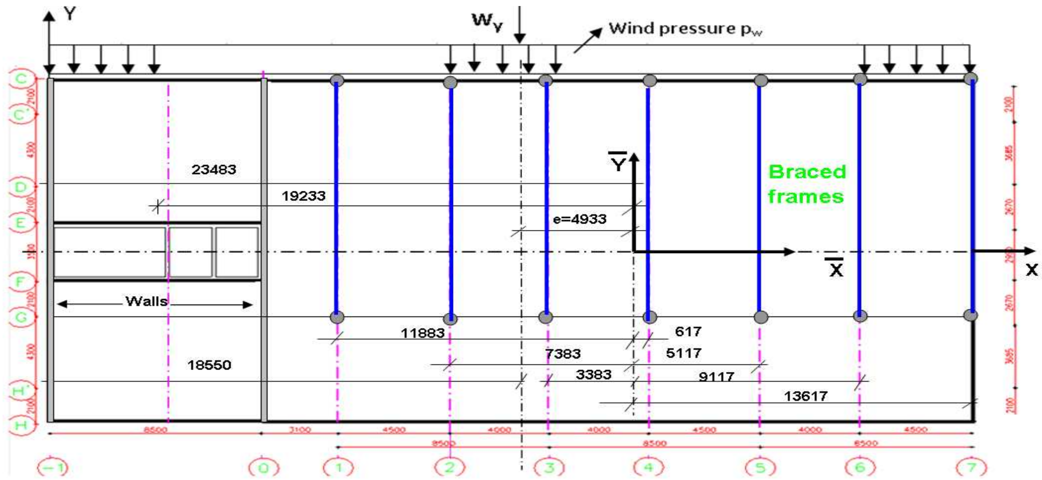


Figure A7-8: Location of rotation centre in plan(core and old building frames frame) in floor 0 to 7

- **Forces in bracing structures**

a. Translation

(i) In core : $F_{y,transl,core} = \dfrac{12,4.10^{12}}{(12,4+7(41)).10^{12}}.2018 = 84KN$

(ii) In steel frame and old building: $F_{y,transl,\,frame} = \dfrac{41.10^{12}}{(12,4+7(41)).10^{12}}.2018 = 276KN$ for each frame

b. Rotation

The rotation component is calculated using the formula stated above and the values are put in table A7-3. It has been obtained that the rotation force is 0 for all bracing members(frames).

Table A7-3: Rotation component in bracing member:

	Wy KN	e mm	MT KNmm	Iy mm^4	Ix mm^4	xi	xi^2	yi	yi^2	Iy.xi	$Iy.xi^2$	$Ix.yi^2$	Σ(Iyxi2+ Ixi.yi2)	Fy; rot=(I y.xi)/ Σ(Iyxi2+Ixi.yi2))MT
W1	2018	4933	9954794	4,10E+13	0,00E+00	11883	1E+08	3192,5	1E+07	4,87E+17	5,79E+21	0,00E+00		1,93E+02
W2	2018	4933	9954794	4,10E+13	0,00E+00	7383	5E+07	3192,5	1E+07	3,03E+17	2,23E+21	0,00E+00		1,20E+02
W3	2018	4933	9954794	4,10E+13	0,00E+00	3383	1E+07	3192,5	1E+07	1,39E+17	4,69E+20	0,00E+00		5,48E+01
W4	2018	4933	9954794	4,10E+13	0,00E+00	617	380689	3192,5	1E+07	2,53E+16	1,56E+19	0,00E+00		1,00E+01
W5	2018	4933	9954794	4,10E+13	0,00E+00	5117	3E+07	3192,5	1E+07	2,10E+17	1,07E+21	0,00E+00		8,29E+01
W6	2018	4933	9954794	4,10E+13	0,00E+00	9117	8E+07	3192,5	1E+07	3,74E+17	3,41E+21	0,00E+00		1,48E+02
W7	2018	4933	9954794	4,10E+13	0,00E+00	13617	2E+08	3192,5	1E+07	5,58E+17	7,60E+21	0,00E+00		2,21E+02
Core	2018	4933	9954794	1,24E+13	7,68E+13	19233	4E+08	0	0	2,38E+17	4,59E+21	0,00E+00		9,43E+01
											2,52E+22	0,00E+00	2,52E+22	

Table A7-4: Forces acting on bracing members

Lower

	Translation KN	Rotation KN	Total force KN	Height m	UDL N/m
W1	276	193	469	20.56	23
W2	276	120	396	20.56	19
W3	276	55	331	20.56	16
W4	276	10	286	20.56	14
W5	276	83	359	20.56	17
W6	276	148	424	20.56	21
W7	276	221	497	20.56	24
Core	84	94	178	20.56	9

A7.3 Concrete core with old building and one braced steel frame

As the concrete core is located at the end of building (eccentrically placed), to avoid moment of torsion in building a braced frame will be used to help concrete core. It will be placed in axis 5 where oblique struts will not hinder the movement inside the building.

All steel frames from axis 1 to 7 are connected to the old building up to 7th floor; therefore they help the core to carry the lateral loads. It is assumed that the walls of the old building to which the new building is connected are very stiff. Beyond 7th floor the core and braced steel frame stabilize the building by themselves.

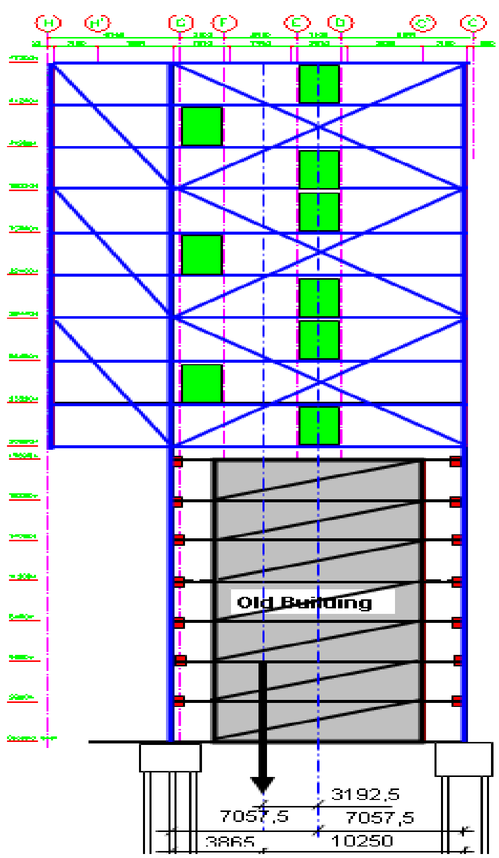
A7.3.1 Upper part

- **Moment of inertia of steel frame**

The moment of inertia is calculated by taking the columns with respect to the axis of symmetry. Only the part reaching the ground is considered because it's the one conveying the lateral load to the ground.

TableA7- 2: Components of composite supporting columns

		#	E[N/mm2]	A[mm2]	r[mm]	r2	I mm4	EI[Nmm2]
G	Steel	CHS457x16	2.10E+05	2.22E+04	3.87E+03	1.49E+07	3.32E+11	6.96E+16
	Concrete	B45	3.60E+04	1.39E+05	3.87E+03	1.49E+07	2.08E+12	7.47E+16
	R-bars	10Φ20	2.10E+05	3.14E+03	3.87E+03	1.49E+07	4.69E+10	9.86E+15
C	Steel	CHS406,4x8	2.10E+05	1.00E+04	1.03E+04	1.05E+08	1.05E+12	2.21E+17
	Concrete	B45	3.60E+04	1.17E+05	1.03E+04	1.05E+08	1.23E+13	4.43E+17
	R-bars	8Φ20	2.10E+05	2.51E+03	1.03E+04	1.05E+08	2.64E+11	5.54E+16
						Sum	1.61E+13	8.74E+17



FigureA7-6: Steel frame member contributing to the stability

The moment of inertia of steel frame along the old building is as follow:

$I = 16.10^{12} mm^4$

It is this moment of inertia which will be used, because it is given by those columns connected to the foundation.

- **Moment of inertia of concrete core**

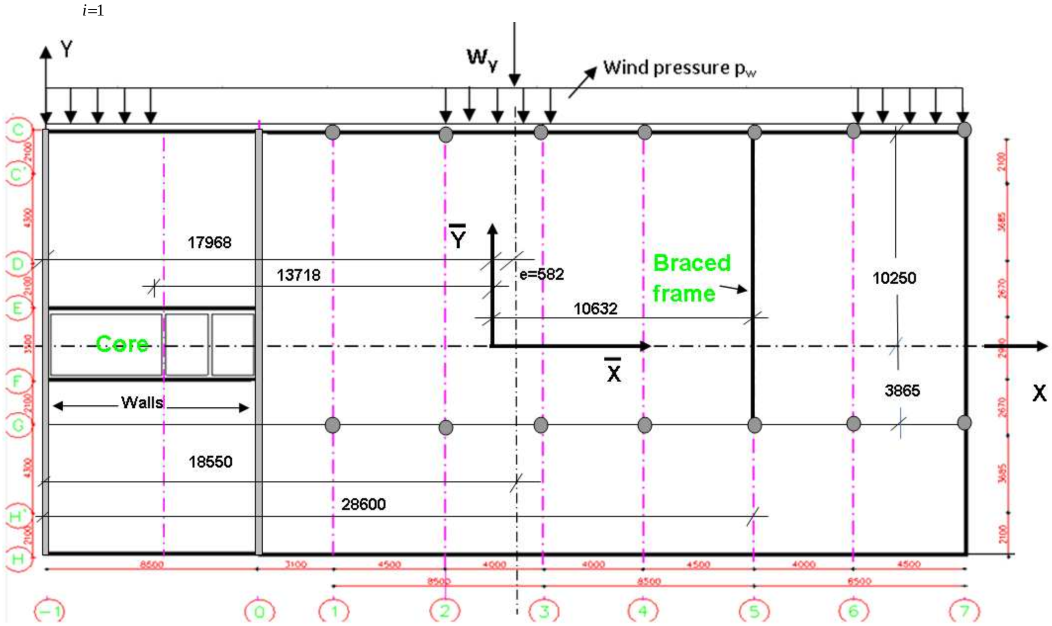
It is taken after increasing the wall sizes of core

$I_y = 12,4.10^{12} mm^4$ and $I_x = \left[2(8500.250).4250^2\right] = 76,8.10^{12} mm^4$

- **Location of the rotation center**

$$\overline{x}_c = \frac{\sum_{i=1}^{n} I_{y,i}.x_i}{\sum_{i=1}^{n} I_{y,i}} \Rightarrow \overline{x}_c = \frac{12,4.10^{12}.4250 + 16.10^{12}.28600}{(12,4+16).10^{12}} = \frac{5,103.10^{17}}{28,4.10^{12}} = 17968mm$$

$$\overline{y}_c = \frac{\sum_{i=1}^{n} I_{x,i}.y_i}{\sum_{i=1}^{n} I_{x,i}} \Rightarrow \overline{y}_c = \frac{76,8.10^{12}.0+0}{76,8.10^{12}} = 0mm$$



FigureA7-7: Location of rotation centre in plan(core and one frame) in floor 7 to 15

- **Forces in bracing structures**

(i) In core

$$F_{y,transl,core} = \frac{12,4.10^{12}}{(12,4+16).10^{12}}.2018 = 881KN$$

$$F_{y,rotation,core} = \frac{12,4.10^{12}.13718}{12,4.10^{12}.13718^2 + 16.10^{12}10632^2) + \left\{76,8.10^{12}.0^2 + 0\right\}}.2018.582 = 48KN$$

(ii) In steel frame

$$F_{y,transl,frame} = \frac{16.10^{12}}{(12,4+16).10^{12}}.2018 = 1137KN$$

$$F_{y,rotation,frame} = \frac{16.10^{12}.10632}{12,4.10^{12}.13718^2 + 16.10^{12}.10632^2)}.2018.582 = 48KN$$

A7.3.2 Lower part

- **Moment of inertia of concrete core**

Concrete core: $I_y = 12,4.10^{12}mm^4$ and $I_x = 76,8.10^{12}mm^4$

- **Moment of inertia of steel frame and old building**

$I_{old,1-7} = A_F.L_F{}^2 + A_{C'}.L_{C'}{}^2$ $\quad I_{old,1-7} = 510.210.1765^2 + 510.210.8150^2 = 7,45.10^{12}mm^4$

$$I_{y,\text{in-f}\,ill} = \frac{210.10000^3}{12} = 1,75.10^{13}mm^4$$

$$I_{y,total} = 17,5 + 7,45 + 16 = 41.10^{12}mm^4$$

$I_{x,total} = 0$ is negligible for each frame

- **Location of the rotation center**

$$\overline{x}_c = \frac{\sum_{i=1}^{n} I_{y,i}.x_i}{\sum_{i=1}^{n} I_{y,i}} \Rightarrow \overline{x}_c = \frac{12,4.10^{12}.4250 + 41.10^{12}(11600 + 16100 + 20100 + 24100 + 28600 + 32600 + 37100)}{\left(12,4 + 7(41)\right).10^{12}} =$$

$$\frac{7,0309.10^{18}}{299,4.10^{12}} = 23483mm$$

$$\overline{y}_c = \frac{\sum_{i=1}^{n} I_{x,i}.y_i}{\sum_{i=1}^{n} I_{x,i}} \Rightarrow \overline{y}_c = 0$$

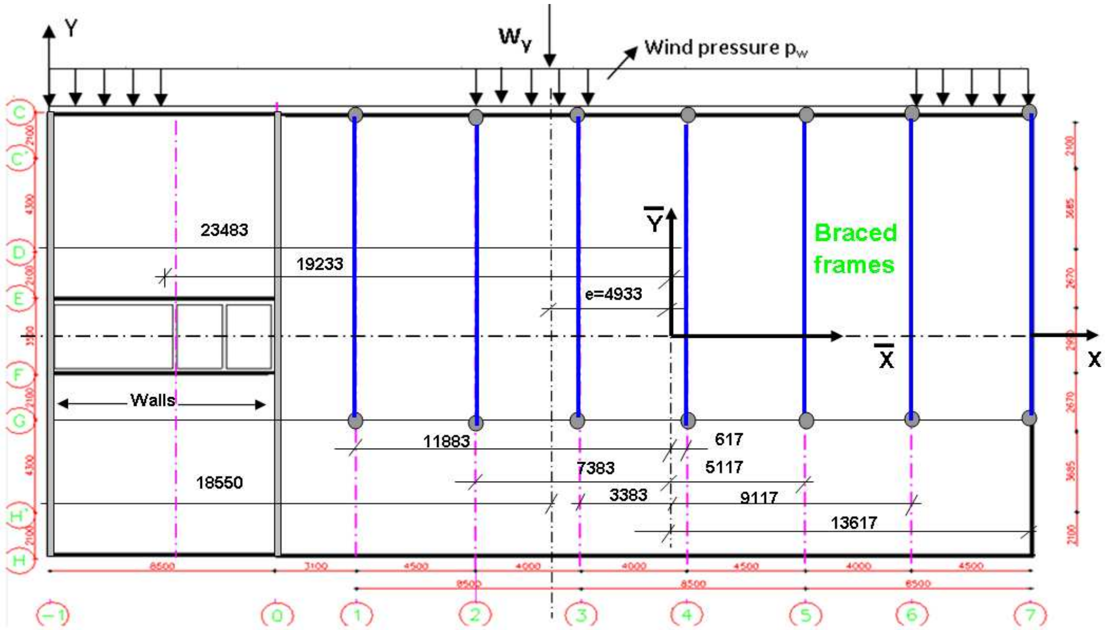


Figure A7-8: Location of rotation centre in plan (core and old building frames frame) in floor 0 to 7

- **Forces in bracing structures**

a. Translation

(i) **In core :** $F_{y,transl,core} = \dfrac{12,4.10^{12}}{(12,4+7(41)).10^{12}}.2018 = 84KN$

(ii) In steel frame and old building

$$F_{y,transl,frame} = \frac{41.10^{12}}{(12,4+7(41)).10^{12}}.2018 = 276KN \text{ for each frame}$$

a. Rotation

The rotation component is calculated using the formula stated above and the values are put in table A7-3. It has been obtained that the rotation force is 0 for all bracing members(frames).

Table A7-3: Rotation component in bracing member:

	Wy KN	e mm	MT KNmm	Iy mm^4	Ix mm^4	xi	xi ^{2}	yi	yi^2	Iy.xi	Iy.xi^2	Ix.yi^2	Σ(Iyxi2+ Ixi.yi2)	Fy; rot=(I y.xi)/ Σ(Iyxi2+Ixi.yi2))MT
W1	2018	4933	9954794	4,10E+13	0,00E+00	11883	1E+08	3192,5	1E+07	4,87E+17	5,79E+21	0,00E+00		1,93E+02
W2	2018	4933	9954794	4,10E+13	0,00E+00	7383	5E+07	3192,5	1E+07	3,03E+17	2,23E+21	0,00E+00		1,20E+02
W3	2018	4933	9954794	4,10E+13	0,00E+00	3383	1E+07	3192,5	1E+07	1,39E+17	4,69E+20	0,00E+00		5,48E+01
W4	2018	4933	9954794	4,10E+13	0,00E+00	617	380689	3192,5	1E+07	2,53E+16	1,56E+19	0,00E+00		1,00E+01
W5	2018	4933	9954794	4,10E+13	0,00E+00	5117	3E+07	3192,5	1E+07	2,10E+17	1,07E+21	0,00E+00		8,29E+01
W6	2018	4933	9954794	4,10E+13	0,00E+00	9117	8E+07	3192,5	1E+07	3,74E+17	3,41E+21	0,00E+00		1,48E+02
W7	2018	4933	9954794	4,10E+13	0,00E+00	13617	2E+08	3192,5	1E+07	5,58E+17	7,60E+21	0,00E+00		2,21E+02
Core	2018	4933	9954794	1,24E+13	7,68E+13	19233	4E+08	0	0	2,38E+17	4,59E+21	0,00E+00		9,43E+01
											2,52E+22	0,00E+00	2,52E+22	

Table A7-4: Forces acting on bracing members

Upper

	Translation KN	Rotation KN	Total force KN	Height m	UDL KN/m
W5	1137	48	1185	26,64	44
Core	881	48	929	26,64	35

Lower

	Translation KN	Rotation KN	Total force KN	Height m	UDL N/m
W1	276	193	469	20.56	23
W2	276	120	396	20.56	19
W3	276	55	331	20.56	16
W4	276	10	286	20.56	14
W5	276	83	359	20.56	17
W6	276	148	424	20.56	21
W7	276	221	497	20.56	24
Core	84	94	178	20.56	9

A7.4 Concrete core with old building and three braced steel frames

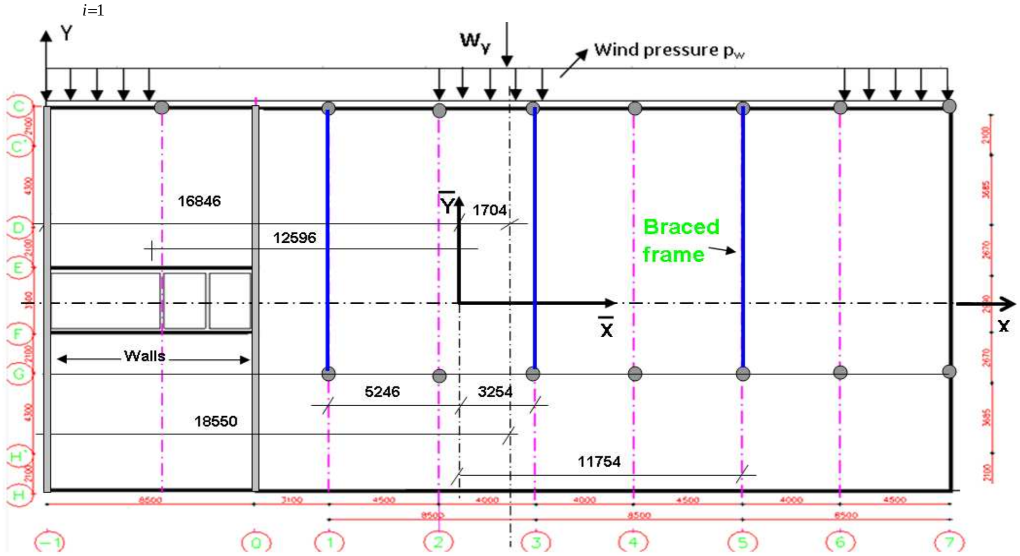
For 3 frames, there is change in the upper part only because the lower part is the same as the previous case

A7.4.1 Upper part

- **Location of rotation center**

$$\overline{x}_c = \frac{\sum_{i=1}^{n} I_{y,i}.x_i}{\sum_{i=1}^{n} I_{y,i}} \Rightarrow \overline{x}_c = \frac{12,4.10^{12}.4250 + 16.10^{12}(+11600 + 20100 + 28600)}{\left(12,4 + 3(16)\right).10^{12}} = \frac{1,0175.10^{18}}{60,4.10^{12}} = 16846mm$$

$$\overline{y}_c = \frac{\sum_{i=1}^{n} I_{x,i}.y_i}{\sum_{i=1}^{n} I_{x,i}} \Rightarrow \overline{y}_c = 0$$



FigureA7-9: Location of rotation centre in plan(core and three frames) in floor 7 to 15

- **Forces in bracing structures**

The bracing structures are subjected to translation and rotation. Each component is calculated and summed up to get the total force acting on bracing structure.

$$F_{y,transl.;i} = \frac{I_{y,i}}{\sum_{i=1}^{n} I_{y,i}}.W_y \quad \text{and} \quad F_{y,rotation,i} \frac{I_{y,i}.\overline{x}_i}{\sum_{i=1}^{n}\left(I_{y,i}.\overline{x}_i^{\,2} + I_{x,i}.\overline{y}_i^{\,2}\right)}.M_T$$

a. Translation

(i) In core

$$F_{y,transl,core} = \frac{12,4.10^{12}}{(12,4+3(16)).10^{12}}.2018 = 414KN$$

(ii) In steel frame

$$F_{y,transl,frame} = \frac{16.10^{12}}{(12,4+3(16)).10^{12}}.2018 = 535KN$$

a. Rotation

Table A7-5: Calculation of rotation forces

	Wy KN	e mm	MT KNmm	Iy mm^4	Ix mm^4	xi	xi^2	yi	yi^2	Iy.xi	$Iy.xi^2$	$Ix.yi^2$	Σ(Iyxi2+ Ixi.yi2)	Fy; rot=(I y.xi)/ Σ(Iyxi2+Ixi.yi2))MT
W1	2018	1704	3438672	1.60E+13	0.00E+00	5246	3E+07	3192.5	1E+07	8.39E+16	4.40E+20	0.00E+00		6.03E+01
W3	2018	1704	3438672	1.60E+13	0.00E+00	3254	1E+07	3192.5	1E+07	5.21E+16	1.69E+20	0.00E+00		3.74E+01
W5	2018	1704	3438672	1.60E+13	0.00E+00	11754	1E+08	3192.5	1E+07	1.88E+17	2.21E+21	0.00E+00		1.35E+02
Core	2018	1704	3438672	1.24E+13	7.68E+13	12596	2E+08	0	0	1.56E+17	1.97E+21	0.00E+00		1.12E+02
											4.79E+21	0.00E+00	4.79E+21	

Table A7-6: Total forces acting on bracing members

Upper

	Translation KN	Rotation KN	Total force KN	Height m	UDL N/m
W1	535	60	595	26,64	22
W3	535	37	572	26,64	21
W5	535	135	670	26,64	25
Core	414	112	526	26,64	20

A7.3.2 Lower part

The characteristics of the lower part is the same as what obtained in previous option (see A7.2.2)

	Translation KN	Rotation KN	Total force KN	Height m	UDL N/m
W1	276	193	469	20.56	23
W2	276	120	396	20.56	19
W3	276	55	331	20.56	16
W4	276	10	286	20.56	14
W5	276	83	359	20.56	17
W6	276	148	424	20.56	21
W7	276	221	497	20.56	24
Core	84	94	178	20.56	9

A7.5 Conclusion

To stabilize the whole building using only the concrete core will not be possible, the method results in excessive deflection due to translation and rotation since the core in eccentrically placed in plan.

The existing building can help the new structure to restrain the lateral loads when the two structures are would be laterally connected up to 7th floor. From 7th floor upwards, the core could act alone, but it has obtained that it would deflect more because it is not stiff enough in lateral direction to take all lateral loads acting on the whole building.

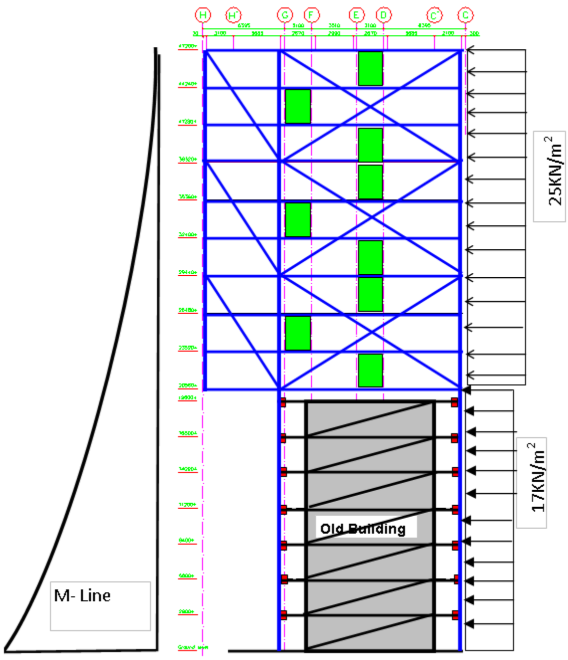
Furthermore minimize the deflection, a steel frame can be put to act in parallel with concrete core. It has been decided to put a braced frame in axis 5 due to non obstruction benefits. But the frame is taking too much load that is why 2 additional frames can be proposed in axis 1 and 3 to help 5.

The construction of cantilever near the core is very challenging, but if concrete walls would be constructed in axis -1 and 0, the lateral stiffness increases and floor up wards, it is easy to construct cantilevers by extending the wall at every floor level.

A8. DESIGN OF BRACINGS IN THE FINAL DESIGN

The bracing members are loaded mainly by the lateral force acting to the entire structure and a part of vertical load since braces prevent columns from horizontal deflection and buckling. For simplicity, it's suggested to use cross bracing in the upper part of the building because cross bracing are loaded in tension but they have to be checked for buckling since lateral load can reverse and loading the bracing member in compression. In addition to this, they also carry some parts of the vertical loads.

From Chapter 9 and Appendix 7, the best way of stabilizing the building is by using concrete walls in axes -1 and 0 plus 3- braced frame in axes 1, 3 and 5. The frame in axis 5 is governing; the horizontal loads acting on it are 25 and 17 KN/m respectively on upper and lower parts. The dead load and live load are resprectively 30,4 and 7,4 KN/m.



FigureA8- 1: Bracing pattern and lateral load in braced frame 5

Bracing members can be sized by using ultimate load acting on structure.

Table A8-1: Used load combination

PC / LC	1 SLS	2 SLS	3 ULS	4 ULS
Self weight	1,0	1,0	1,2	1,2
Variable load	1,0	1,0	1,5	1,5
Lateral load from East	1,0		1,5	
Lateral load from west		1,0		1,5

The ultimate shear force in foundation of the structure is as follow:

$$V_{Bottom} = \gamma.w_1.h_1 + \gamma.w_2.h_2 \Rightarrow 1,5.17.20,56 + 1,5.25.26,64 = 1523KN$$

- The lower part of the building is braced by the old building; the connectors between old and new building transmit lateral loads to the bracing structure (old building). These connector can be sized based on shear force to be transmitted to the old building(bracing system)
  The minimum area of steel required is

$$A = \frac{F}{\sigma} \Rightarrow \frac{1523.10^3}{235} = 6481mm^2,$$

Due to the presence of concrete beam supporting balconies of the old building, two channel profiles UPN220: 220x80 (2x3740mm^2) can be used and put on both sides of the beam. They are connected to the column's bracket by means of bolts.

- In upper structure from the 7th floor the horizontal force must be restrained by a cross bracing braces. By using cross bracing ,the maximum force in those bracing member is due to lateral load as well as some components of gravity load :

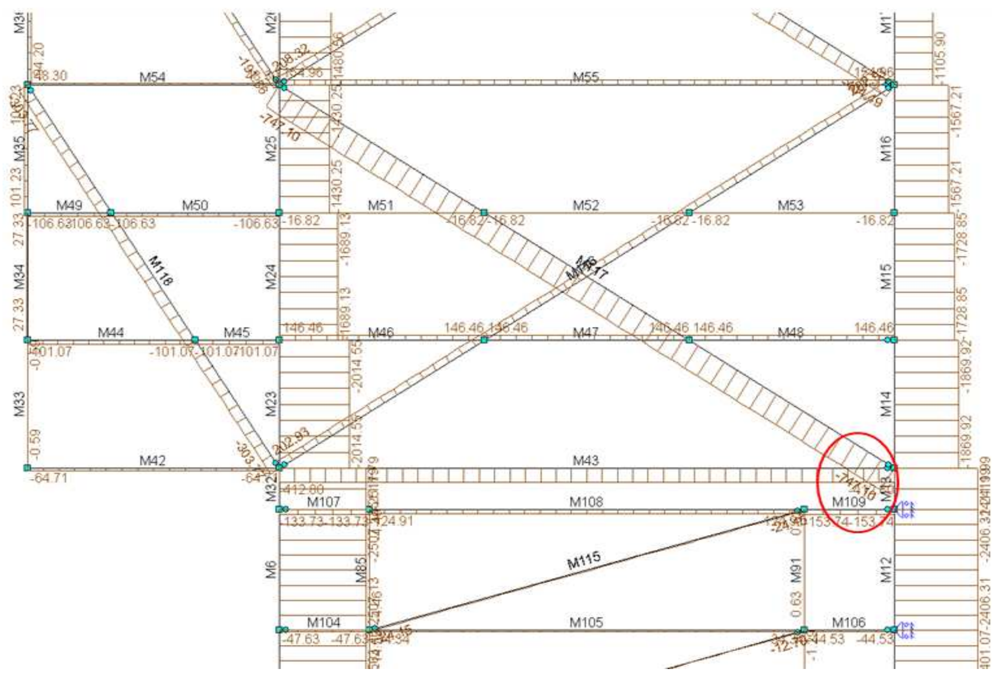
The ultimate shear force acting on 7th floor is: $V_{7th} = q_{lateral}.h_{7th} \Rightarrow 1,5.25.26,64 \approx 1000KN$

$$\theta = \tan - 1\left(\frac{8880}{14115}\right) = 32,2^o$$

The tensile force in bracing members:

$$T = \frac{V}{\cos 32,2^0} \Rightarrow \frac{1000}{0,846} = 1182KN$$

The maximum compressive force in bracing members is 747x2 which is given by the load combination 4.(See figure A8-2), therefore these members will be designed by summing the two forces: 1182 KN + 2(747) = 2676KN



FigureA8- 2: Calculation of axial force in cross bracing members

$A = \frac{T}{\sigma} \Rightarrow \frac{2676.10^3}{235} = 11387mm^2$, Use HE280B; $A = 13100mm^2$

Verification for stability:

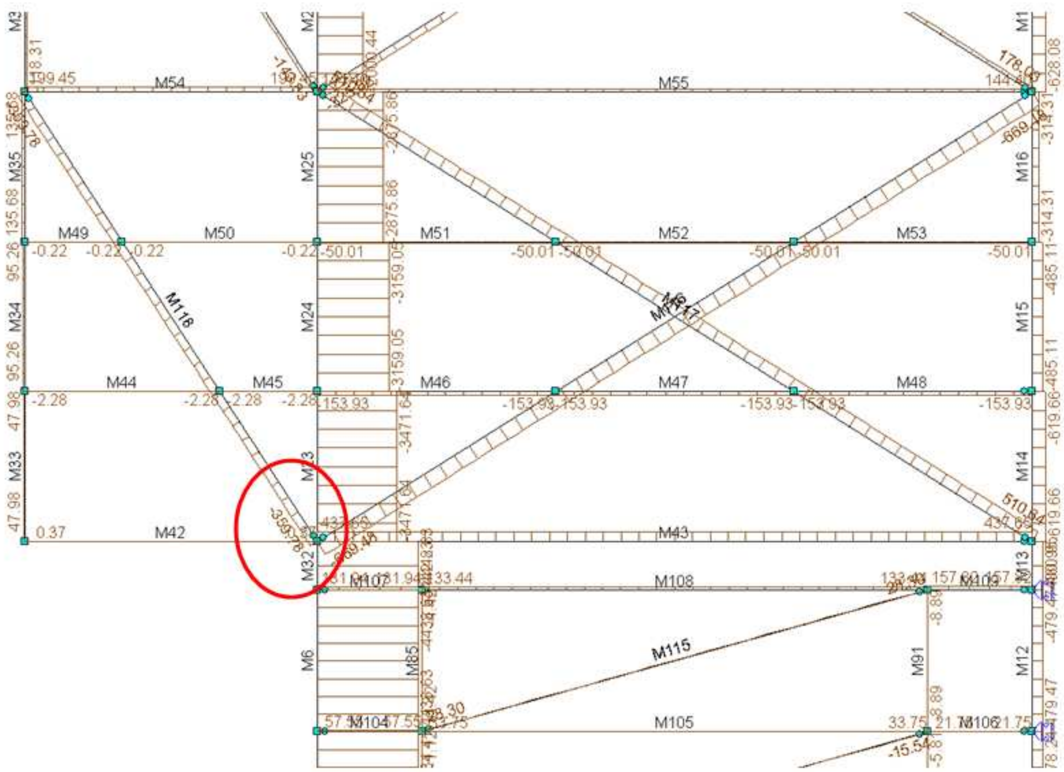
$$\lambda_{rel} = \frac{L_{buc}}{i.94} \Rightarrow \frac{16676}{121.94} = 1,5 \quad , \quad \omega_{buc} = 0,37$$

$\frac{F}{\omega_{buc}.A.f_y} < 1,0 \Rightarrow \frac{2676.10^3}{0,37.13100.235} = 2,3 > 1,0$, not OK, Many section failed this requirement and finally

HE400B was tried

$$A = 19800mm^2$$
$$\lambda_{rel} = \frac{L_{buc}}{i.94} \Rightarrow \frac{16676}{171.94} = 1,0 \quad , \quad \omega_{buc} = 0,6$$
$$\frac{F}{\omega_{buc}.A.f_y} < 1,0 \Rightarrow \frac{2676.10^3}{0,67.19800.235} = 0,85 < 1,0 \text{ OK}$$

The frame restraining lateral loads is likely to have more deflection on cantilevers, therefore 3-storey braces in cantilevers can help to convey facade and some beam load to the supporting column.
The maximum axial force in these bracings is given by load combination 3 (Figure A8-3) is 360.2= 720 KN



FigureA8- 3: Calculation of axial force in cantilever bracing

$$A = \frac{F}{\sigma} \Rightarrow \frac{720.10^3}{235} = 3064mm^2 \text{ , Use HE200 B}$$

Verification for stability:

$$\lambda_{rel} = \frac{L_{buc}}{i.94} \Rightarrow \frac{10600}{85,4.94} = 1,3 \quad , \quad \omega_{buc} = 0,47$$
$$\frac{F}{\omega_{buc}.A.f_y} < 1,0 \Rightarrow \frac{720.10^3}{0,47.7810.235} = 0,83 < 1,0 \text{ , OK}$$

A9.CALCULATION OF HORIZONTAL DEFLECTION OF BRACING STRUCTURES IN FINAL DESIGN

A9.1 Braced frame

Once the frame moment of inertia and frame loading are known, it is possible to calculate the horizontal deflection. The deflection has two components; the flexural and shear components.

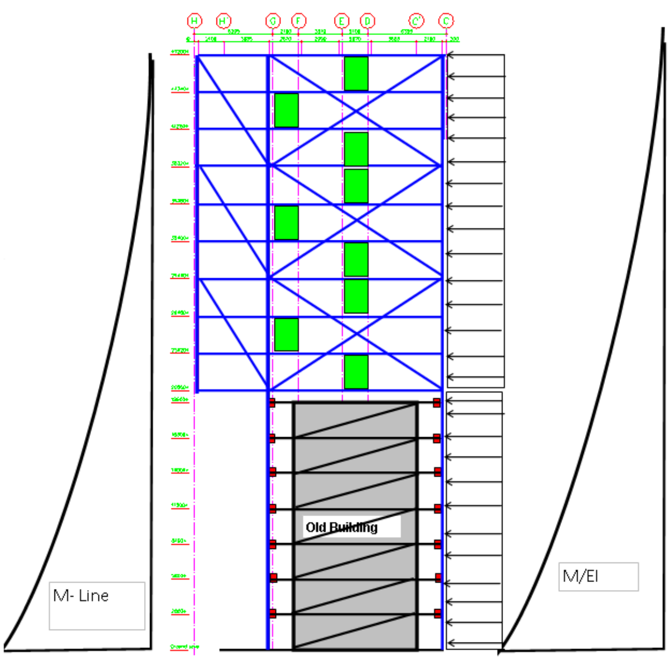
A9.1.1 First order deflection

- **Flexural component**

The procedure for obtaining the flexural component of drift is to first calculate for the structure the external moment diagram. Then to calculate for the different vertical regions of the bent, the second moment of area I of the column sectional areas about their common centroid

$I = \sum_{i=1}^{n} A_i . L_i^2$ Where Ai, is the sectional area of steel i, and Li is the distance from the common centroid to the steel i.

The storey deflection in story i, u_{if} due to flexure of the structure is then obtained from; $u_{if} = h_i . \theta_{if}$ in which h_i is the height of story i, and θ_{if} is the inclination of story i, which is equal to the area under the $\frac{M}{EI}$ curve between the base of the structure and the mid height of the story i.



FigureA9- 1: Bending moment due to lateral load

The total deflection at floor n, due to flexure is then given by the sum of the story drifts from the first to n[th] stories.

$$u_{f,tot} = \sum_{i=1}^{n} u_{if}$$

Calculation procedures:

6. Compute the moment of inertia of the column sectional area about their common centroid. The part contributing to the lateral load resistance is the one starting from the ground. It includes 2-support columns from the ground level to the 15th floor.

This values is put in table 12-3 column 7.

Compute the value of the external moment at each story and enter the value in column 6.

7. Determine for each story the value of hM/EI, and enter the results in column 9.
8. Determine for each story i, the accumulation of $u.\theta_{if}$ from the ground level up to and including story i, θ_{if} and record it in column10. Such accumulated values give the inclination of each story i due to flexure θ_{if} .
9. Record the product of h_i and θ_{if} in column 11. $h_i.\theta_{if}$ is the drift in story i, u_{if} , due to flexure.
10. At each level, the value of the lateral drift is required, evaluate the accumulation of the story drift , u_{if} ,from the ground level up and considered n.th floor to give the drift u_{total} due to flexure. Enter these in column 12.

Table9- 1: Evaluation of flexural component of deflection for new building

Story	Story heigh mm	Cum.height Ground -roof m	Cum. height roof-ground m		Lateral load KN/m	External moment Nm	Frame inertia x10^{12}mm^4	Mod. Elast E x10^{4}N/mm^2	Storey Inclination δθi=h*M/EI rads	Cumul.storey inclinastion θ_{if} Rads	storey drift mm	Cumul. Drift mm
Roof	2960	47200	0				16	21				
15	2960	44240	2960				16	21				
14	2960	41280	5920				16	21				
13	2960	38320	8880				16	21				
12	2960	35360	11840				16	21				
11	2960	32400	14800				16	21				
10	2960	29440	17760				16	21				
9	2960	26480	20720				16	21				
8	2960	23520	23680				16	21				
7	2960	20560	26640				16	21				
Roof 6	2800	19600	27600	0			41	22,5				
6	2800	16800	30400	2800			41	22,5				
5	2800	14000	33200	5600			41	22,5				
4	2800	11200	36000	8400			41	22,5				
3	2800	8400	38800	11200			41	22,5				
2	2800	5600	41600	14000			41	22,5				
1	2800	2800	44400	16800			41	22,5				
Ground floor		0	47200	19600			41	22,5				

$$u_{s,tot} = \sum_{i=1}^{n} u_{if}$$

Calculation data

The calculation data for steel braced frame are summarized in table A9-2.

TableA9- 2: Calculation data

	Upper part		Lower part	
L_{BR} (mm)		16676		10385
L_B (mm)		14115		10000
A_B (mm^2)	2xHE 700M	76600	250x210	52500
A_{BR} (mm^2)	HE400B	19800	650x336	218400
I_B (mm^4)	HE550	2,73E+09	250x210	2,73E+08
E_{BR} N/mm^2)		210000	210000+15000	225000
Frame inertia (mm^4)		1,60E+13	16e12 + 25 e12	4,10E+13

TableA9- 3: Flexural component of deflection when core and three braced frames are used for stability

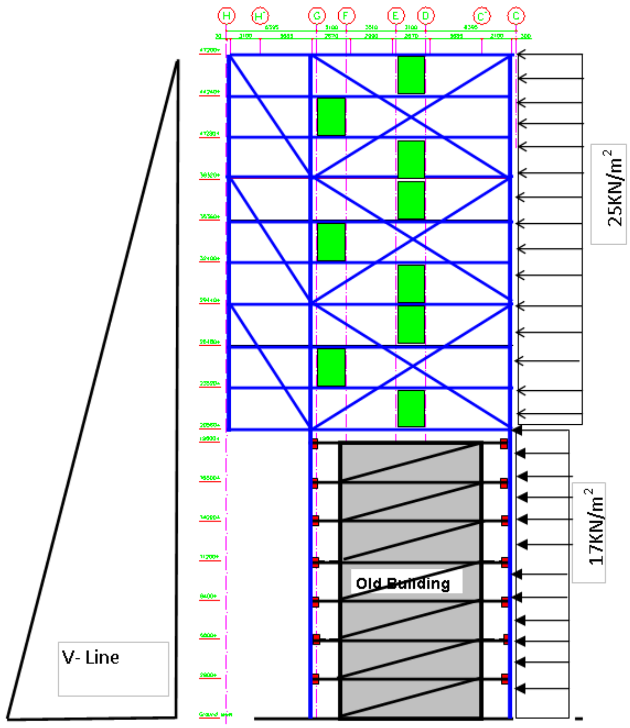
Story	Story heigh mm	Cum.height Ground -roof m	Cum. height roof-ground m		Lateral load KN/m	External moment Nm	Frame inertia $x10^{12}mm^4$	Mod. Elast E $x10^4 N/mm^2$	Storey Inclination $\delta\theta i=h*M/EI$ rads	Cumul.storey inclinastion θ_{if} Rads	storey drift mm	Cumul. Drift mm
Roof	2960	47200	0		25	0	16	21	0	6,14E-05	0,18	2,00
15	2960	44240	2960		25	1,1E+08	16	21	9,648E-08	6,14E-05	0,18	1,82
14	2960	41280	5920		25	4,4E+08	16	21	3,859E-07	6,13E-05	0,18	1,64
13	2960	38320	8880		25	9,9E+08	16	21	8,683E-07	6,09E-05	0,18	1,46
12	2960	35360	11840		25	1,8E+09	16	21	1,544E-06	6,01E-05	0,18	1,28
11	2960	32400	14800		25	2,7E+09	16	21	2,412E-06	5,85E-05	0,17	1,10
10	2960	29440	17760		25	3,9E+09	16	21	3,473E-06	5,61E-05	0,17	0,93
9	2960	26480	20720		25	5,4E+09	16	21	4,728E-06	5,27E-05	0,16	0,76
8	2960	23520	23680		25	7E+09	16	21	6,175E-06	4,79E-05	0,14	0,60
7	2960	20560	26640		25	8,9E+09	16	21	7,815E-06	4,17E-05	0,12	0,46
Roof 6	2800	19600	27600	0	17	9,5E+09	41	22,5	2,887E-06	3,39E-05	0,00	0,34
6	2800	16800	30400	2800	17	1,1E+10	41	22,5	3,473E-06	3,10E-05	0,09	0,34
5	2800	14000	33200	5600	17	1,4E+10	41	22,5	4,1E-06	2,76E-05	0,08	0,25
4	2800	11200	36000	8400	17	1,6E+10	41	22,5	4,767E-06	2,35E-05	0,07	0,17
3	2800	8400	38800	11200	17	1,8E+10	41	22,5	5,474E-06	1,87E-05	0,05	0,11
2	2800	5600	41600	14000	17	2,1E+10	41	22,5	6,222E-06	1,32E-05	0,04	0,06
1	2800	2800	44400	16800	17	2,3E+10	41	22,5	7,011E-06	7,01E-06	0,02	0,02
Ground floor		0	47200	19600	17	2,6E+10	41	22,5	0	0,00E+00	0,00	0,00

$$u_{f,tot} = \sum_{i=1}^{n} u_{f,i} = 2mm$$

- **Shear component**

The shear component of the story drift is story i, u_{is} is a function of the external shear and the properties of braces and girder in that story [6]. The shear component of the total drift at floor level n, $u_{s,total}$ is equal to the sum of the story shear component of the drift from the ground to nth stories, that is

$$u_{s,tot} = \sum_{i=1}^{n} u_{is}$$



FigureA9- 2: Steel frame and lateral loads

The top part has cross bracing, the lower part as it braces the new building, it is modelled as frame with single braced bracing.
The horizontal displacement for both cases is given by these relations:

$$u_{0-7} = \frac{V}{E}\left[\frac{L_{BR}^{\ 3}}{L_B^{\ 2}.A_{BR}} + \frac{L_B}{A_B}\right] \text{ (Equivalent single diagonal of in-fill) and } u_{7-15} = \frac{V}{2E}\left[\frac{L_{BR}^{\ 3}}{L_B^{\ 2}.A_{BR}}\right] \text{ (Cross bracing)}$$

in the upper section of the building

L_{BR} and A_{BR} are respectively is the length and the area of the bracing
L is the length of girder
E is modulus of elasticity.
V: The shear force
L_B and A_B are respectively is the length and the area of the upper beam

TableA9- 4: Evaluation of shear component of deflection for new building

Story	Lateral load KN/m	Cumul.height Ground -roof mm	Shear Q KN	L_{BR} mm	L_B mm	A_{BR} mm^2	A_B mm^2	I_B mm^4	E_{BR} N/mm^2	Storey drift(mm)	Cumulat. Drift mm
Roof		0		16676	14115	19800	76600	2,734E+09	210000		
15		2,96		16676	14115	19800	76600	2,734E+09	210000		
14		5,92		16676	14115	19800	76600	2,734E+09	210000		
13		8,88		16676	14115	19800	76600	2,734E+09	210000		
12		11,84		16676	14115	19800	76600	2,734E+09	210000	$\delta_i = \frac{Q}{2E}\left(\frac{d^3}{L^2 A_d}\right)$	
11		14,8		16676	14115	19800	76600	2,734E+09	210000		
10		17,76		16676	14115	19800	76600	2,734E+09	210000		
9		20,72		16676	14115	19800	76600	2,734E+09	210000		
8		23,68		16676	14115	19800	76600	2,734E+09	210000		
7		26,64		16676	14115	19800	76600	2,734E+09	210000		
Roof 6		0		10385	10000	218400	52500	2,73E+08	225000		
6		2,8		10385	10000	218400	52500	2,73E+08	225000	$\delta_i = \frac{Q}{E}\left(\frac{d^3}{L^2 A_d} + \frac{L}{A_g}\right)$	
5		5,6		10385	10000	218400	52500	2,73E+08	225000		
4		8,4		10385	10000	218400	52500	2,73E+08	225000		
3		11,2		10385	10000	218400	52500	2,73E+08	225000		
2		14		10385	10000	218400	52500	2,73E+08	225000		
1		16,8		10385	10000	218400	52500	2,73E+08	225000		
Ground floor	3,64	19,6	231,144	10385	10000	218400	52500	2,73E+08	15000		

$$u_{s,tot} = \sum_{i=1}^{n} u_{s,i} \;;$$

TableA9-5: Shear component of deflection when core and three braced frames are used for stability

Story	Lateral load KN/m	Cumul.height Ground -roof mm	Shear Q KN	L_{BR} mm	L_B mm	Ad mm^2	A_B mm^2	I_B mm^4	E_{BR} N/mm^2	Storey drift(mm)	Cumulat. Drift mm
Roof	25	0	0	16676	14115	19800	76600	2,734E+09	210000	0,00	20,51
15	25	2,96	74	16676	14115	19800	76600	2,734E+09	210000	0,21	20,51
14	25	5,92	148	16676	14115	19800	76600	2,734E+09	210000	0,41	20,30
13	25	8,88	222	16676	14115	19800	76600	2,734E+09	210000	0,62	19,89
12	25	11,84	296	16676	14115	19800	76600	2,734E+09	210000	$\delta_i = \frac{Q}{2E}\left(\frac{d^3}{L^2 A_d}\right)$ 0,83	19,27
11	25	14,8	370	16676	14115	19800	76600	2,734E+09	210000	1,04	18,44
10	25	17,76	444	16676	14115	19800	76600	2,734E+09	210000	1,24	17,40
9	25	20,72	518	16676	14115	19800	76600	2,734E+09	210000	1,45	16,16
8	25	23,68	592	16676	14115	19800	76600	2,734E+09	210000	1,66	14,71
7	25	26,64	666	16676	14115	19800	76600	2,734E+09	210000	1,86	13,05
Roof 6	17	27,6	1135,2	10385	10000	218400	52500	2,73E+08	225000	1,22	11,19
6	17	30,4	1182,8	10385	10000	218400	52500	2,73E+08	225000	1,27	9,97
5	17	33,2	1230,4	10385	10000	218400	52500	2,73E+08	225000	1,32	8,70
4	17	36	1278	10385	10000	218400	52500	2,73E+08	225000	$\delta_i = \frac{Q}{E}\left(\frac{d^3}{L^2 A_d} + \frac{L}{A_g}\right)$ 1,37	7,38
3	17	38,8	1325,6	10385	10000	218400	52500	2,73E+08	225000	1,42	6,00
2	17	41,6	1373,2	10385	10000	218400	52500	2,73E+08	225000	1,48	4,58
1	17	44,4	1420,8	10385	10000	218400	52500	2,73E+08	225000	1,53	3,10
Ground	17	47,2	1468,4	10385	10000	218400	52500	2,73E+08	225000	1,58	1,58

$$u_{s,tot} = \sum_{i=1}^{n} u_{s,i} \;; \; u_{s,tot} = 20,5mm$$

Having obtained the flexural and shear component of drift, the total drift at level n is given by;

$$u_{tot} = u_{f,tot} + u_{s,tot}$$

$$u_{tot} = u_{f,tot} + u_{s,tot} \Rightarrow u_{tot} = 2 + 20,5 = 22,5mm$$

A8.1.2 Second order effect

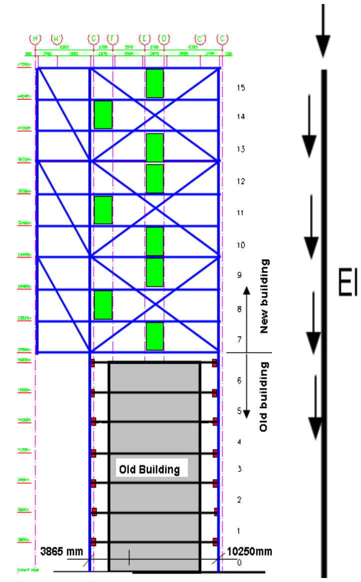
Buckling load of the frame

The braced frame is assumed to be partially restrained, therefore:

$$P_{E1;\,frame} = \frac{\pi^2 .EI}{L^2_{\ 0}} \quad L_o = 1{,}12L$$ for a building of more than 5 stories

The frame is made of two supporting composite columns and bracing.
The member properties are shown in the following table.

		#	E[N/mm2]	A[mm2]	r[mm]	r2	I mm4	EI[Nmm2]
G	Steel	CHS457x16	2.10E+05	2.22E+04	3.87E+03	1.49E+07	3.32E+11	6.96E+16
	Concrete	B45	3.60E+04	1.39E+05	3.87E+03	1.49E+07	2.08E+12	7.47E+16
	R-bars	10Φ20	3.14E+03	2.95E+03	3.87E+03	1.49E+07	4.40E+10	1.38E+14
C	Steel	CHS406,4x8	2.10E+05	1.00E+04	1.03E+04	1.05E+08	1.05E+12	2.21E+17
	Concrete	B45	3.60E+04	1.17E+05	1.03E+04	1.05E+08	1.23E+13	4.43E+17
	R-bars	8Φ20	2.10E+05	2.51E+03	1.03E+04	1.05E+08	2.64E+11	5.54E+16
						Sum	1.61E+13	8.64E+17



FigureA9-3: Braced frame and its simplified model

$$P_{E1;\,frame} = \frac{\pi^2 .86{,}4.10^{16}}{(1{,}12.47200)^2} = 30{,}5.10^8\,KN$$

$$P_{E2} = \frac{C}{0{,}5L}$$

$$\frac{1}{P_E} = \frac{1}{P_{E_1}} + \frac{1}{P_{E_2}}$$

$$P_{E2;core} = \frac{C}{0{,}5L}$$, where C is the foundation spring constant

Determination of foundation spring constant

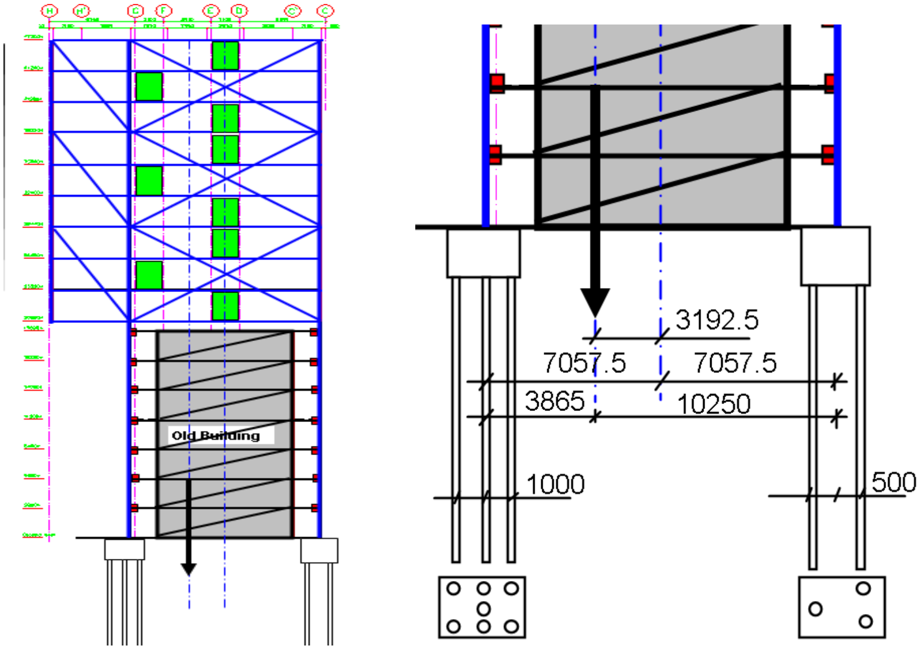
Data:

- Number of piles in one frame: 10 piles of 15 m long

$L_p = 15m$, $\phi = 380mm$

$$A_p = \frac{\pi.380^2}{4} = 113354mm^2$$

Material: Concrete B45



FigureA9-4: Braced frame and its foundation

- **Loads**:

Vertical loads :
$$DL_1 = 7{,}14KN/m^2.(4{,}25m.19{,}9m) = 604KN$$
$$DL_2 = 2KN/m.2.4{,}25 = 17KN$$
$$LL = 1{,}75KN/m^2.(4{,}25m.19{,}9m) = 148KN$$
$$\Sigma = 769KN.10\ floors = 7690KN$$

- Lateral moment in foundation

$$M_{s;d;foot,x} = \frac{w_1.h_1^2}{2} + w_2.h_2\left(H - \frac{h_2}{2}\right) \Rightarrow \frac{17.20{,}56^2}{2} + 25.26{,}64\left(47{,}2 - \frac{26{,}64}{2}\right) = 26157KNm$$

- Moment due to the eccentricity:

$$M_{s;d;ecc} = P.e \Rightarrow 7690.3{,}2 = 24608KNm$$

- Total moment at the base is $Mtotal = M_{s;d;foot,x} + M_{s;d;ecc} \Rightarrow 26157 + 24608 = 50765KNm$

- Pile moment of inertia: $I_p = \sum_{i=1}^{n} n.e^2_i$ where n = number of pile in a row , e_i = eccentricity of pile i, from the application point of the vertical load;

$$I_p = (2.8057{,}5^2 + 3.7057{,}5^2 + 2.6057{,}5^2 + 1.6557{,}5^2 + 2.7557{,}5^2) = 5{,}1.10^8 mm^4$$

$P_n = \left(\dfrac{M}{I_p}.e\right)/n$, with “n”, the number of piles with the same distance from centroidal axis

$$P_n = \left(\frac{50765.10^6}{5{,}1.10^8}.8057{,}5\right)/2 = 401KN$$

- Elastic deformation of a pile is as follow:

$$\Delta L = \frac{P_n.1{,}5.l_p}{E_p.A_p} \qquad \Delta L = \frac{401.10^3.1{,}5.15000}{36000.113354} = 2{,}2mm$$

Then the rotation angle $\varphi = \dfrac{\Delta L}{e} \Rightarrow \varphi = \dfrac{2{,}2}{8057{,}5}$ and $C = \dfrac{M}{\varphi} \Rightarrow C = \dfrac{56765.8057{,}5}{2{,}2} = 208.10^6 KNm/rad$

$$P_{E2} = \frac{C}{0,5L} \Rightarrow \frac{208.10^6}{0,5.47,2} = 8,8.10^6 \, KN$$

The buckling force of partly restrained stabilizing core P_E is determined as follows:

$$\frac{1}{P_E} = \frac{1}{P_{E_1}} + \frac{1}{P_{E_2}} \Rightarrow \frac{1}{P_E} = \frac{1}{30,83.10^8} + \frac{1}{8,8.10^6} \Rightarrow \frac{1}{P_E} = \frac{8,8.10^6 + 30,83.10^8}{2,7.10^{16}} =$$

$$\frac{1}{P_E} = \frac{3,0918.10^9}{2,7.10^{16}} \Rightarrow P_E = 8,73.10^6 \, KN$$

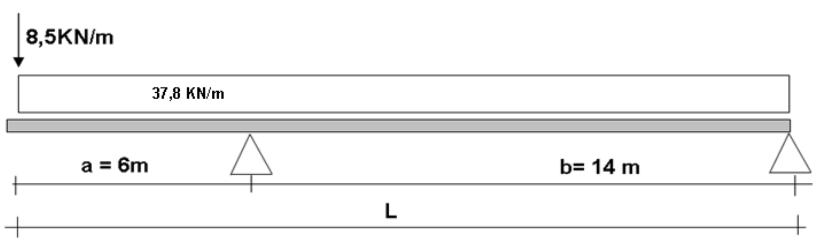
Magnification factor

$P_E = n\gamma P \Rightarrow n = \frac{P_E}{\gamma P} = \frac{8,73.10^6}{1,5.7690} = 757$ then, $\frac{n}{n-1} = \frac{757}{757-1} = 1,001$

The second degree effect is not large.

Deflection

- First order deflection is $u_{1st} \, 22,5mm$
- The second order deflection $u_{2nd} = \frac{1}{n-1} u_{1st} \Rightarrow \frac{1}{757-1} 22,5 = 0,03mm$
- With the second order effect, the resulting deflection $u_{tot} = 22,5 + 0,03 = 22,53mm$
- Foundation rotation $u_{rot} = \frac{M}{C} = \frac{50765}{208.10^6} \Rightarrow 0mm$
- Deflection due to moment loading in cantilevers: At each level , there will be moments from cantilevers. These moment magnify the horizontal deflection of the bracing members
  The considered loads: Dead load= 7,9KN/m and 8,5KN point load from façade;
  $$7,14.4,25 = 30,35KN/m$$
  Live load = $1,75.4,25 = 7,44KN/m$. Total distributed load is
  $$30,35 + 7,44 = 37,8KN/m$$



FigureA9-5: Girder model

The bending moment of cantilevers: $M_{cant} = \frac{37,8.6^2}{2} + 8,5.6 = 731KNm$

The deflection due to single moment loading $u_M = \frac{ML^2}{2EI}$

$$u_M = \frac{731}{2.210000.16.10^{12}} \left(20560^2 + 23520^2 + 26480^2 + 29440^2 + 32400^2 + 35360^2 + 38320^2 + 41280^2 + 44240^2 + 47200^2\right)$$

$$u_M = \frac{731.10^6}{2.210000.16.10^{12}} .1,22.10^{10} = 1,3mm$$

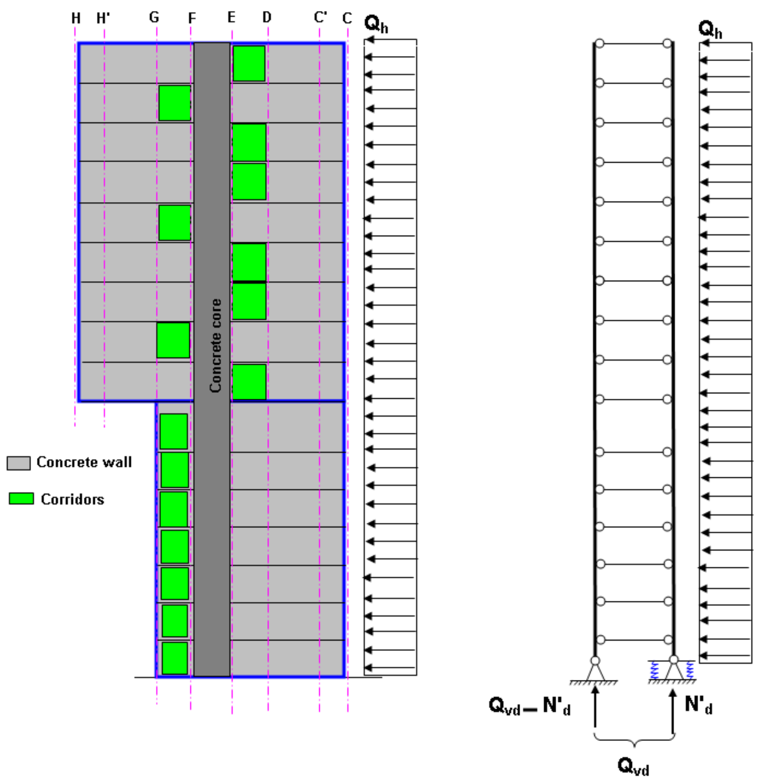
The final deflection; $u_{Total} = 22,53 + 0 + 1,3 = 23,83mm$

A8.2 Core

A8.2.1 First order deflection

In this building the concrete core is provided and constructed just next to the existing building; this section of the building starts from the ground.
This core is 8, 5 m long and 3, 5 m wide; it houses the stair case and elevators. Its main role is to restrain both horizontal load and a part of gravity load acting to the building. It is composed by an assembly of connected shear walls forming a box section with openings closed partially by floor slabs.

The core in this building plays a role of bracing element and is schematized as partly restrained vertical cantilever. The walls around it are schematized as column with pin jointed end. These walls have a large number of openings (corridors, windows and door) in such a way that they cannot be used to restrain lateral loads. They are only considered for vertical loads.



N'_d = Vertical load acting on core

Q_{vd} = Total vertical load of the building

Q_{hd} = Total horizontal load of the building

Figure A9-6: Wall with main openings for corridors in axis -1 and 0 and schematization

Calculation of core

A further schematization can be done (see figure A9-7). The core is represented as a partly restrained column subjected to wind loads, imperfections Q_{hd} and vertical loads made by core own weight and loads which are directly supported by the core N'_d. The rest of the vertical loads $Q_{vd} - N'_d$ are carried to the foundation by the two walls in vicinity of the core. These loads carried by walls can have influence on second degree moment and shear force of the core.

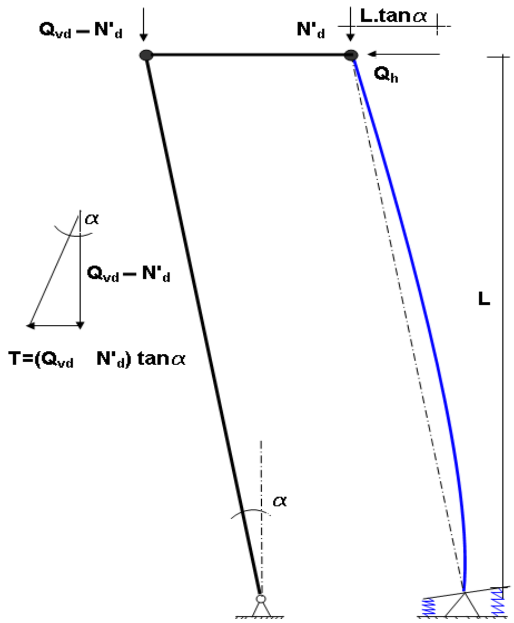


Figure A9-7: simplified model of the core and other supporting elements

The partly restrained core is subjected to horizontal force Q_{hd} and normal force N'_d . The walls(represented as one column hinged at the top and base are attached to the core by floor slab and are loaded with $Q_{vd} - N'_d$.

By horizontal force, the displacement of the whole system is $L\tan\alpha$.

By the angular rotation α , a tensile force $T = \left(Q_{vd} - N'_d\right)\tan\alpha$ occurs.

At the base of the restrained core the second degree moment $M_{core;2nd}$ is:

$$M_{core;2nd;1} = T.\alpha \Rightarrow L\left(Q_{vd} - N'_d\right)\tan\alpha$$

The normal force N'_d causes:

$$M_{core;2nd;2} = N'_d\, L\tan\alpha$$

Therefore the total second degree moment is $M_{core;2nd} = Q_{vd} L\tan\alpha$

Determination of the load carried by the core and walls.

The walls in axes -1 and 0 carry the main vertical load. As the core makes part of these walls, it obviously takes a portion of vertical load (FigureA9-8)

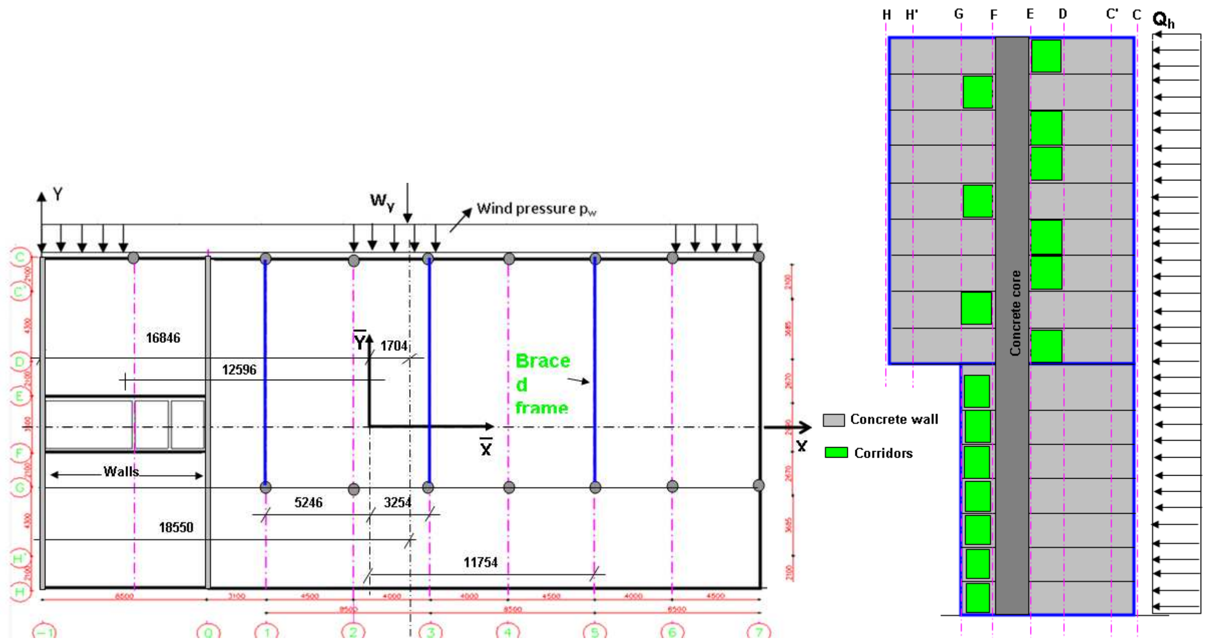


Figure A9-8: Floor plan 7 to 15 floor Section in axis -1 or 0

(i). wall -1:

- **Self weight of wall and cantilever**

Total wall area: $14{,}115.47{,}2 = 666m^2$

Openings: $\{(1{,}62.1{,}225)+(1{,}82.2{,}185)+(2{,}02.1{,}225)+(1{,}795.1{,}875)\}.16 = 189m^2$

Net area: $666-189 = 477m^2$

Design load from wall self weight: $(A_{net}.t_{wall}.\gamma_{concrete}) \Rightarrow (477.0{,}250.24) = 2862KN$

Cantilever:

Total wall area: $5{,}785.26{,}64 = 154m^2$

Openings: $\{(1{,}96.1{,}875)+(2{,}02.1{,}225)\}.9 = 55{,}3m^2$

Net area: $154-55{,}3 = 99m^2$

Design load from wall self weight: $(A_{net}.t_{wall}.\gamma_{concrete}) \Rightarrow (99.0{,}250.24) = 475KN$

- **Load from floors:**

$$F_{floors} = (q_{DL}+q_{VL})(A_{floors})$$
$$F_{floors} = (7{,}9+1{,}75)\{[16(4{,}25.14{,}115]\}$$
$$F_{floors} = 9262KN$$

Cantilever:

$$F_{floors} = (q_{DL}+q_{VL})(A_{floors})$$
$$F_{floors} = (7{,}9+1{,}75)[10(4{,}25.5{,}785]$$

$F_{floors} = 2373KN$

- **Load from staircase**

$F_{stair} = (q_{DL} + q_{VL})(A_{stair})$

$F_{floors} = (3,7+2)\left[16\left(\frac{4,74}{2}.3,1\right)\right]$

$F_{stair} = 670KN$, a half goes to wall -1

Total load: $F_{Total;-1} = 2862+475+9262+2373+335 = 15307KN$

The load supported by the core is: $F_{Core;-1} = \frac{F_{total-1}}{L_{total-1}}.L_{core} \Rightarrow \frac{15307KN}{14,115}.3,5 = 3796KN$

ii).Wall 0

- **Self weight of wall 0 and its cantilever:**

Total wall area: $14,115.47,2 = 666m^2$

Openings:

$\left\{9\left[\left(2,230.2,68\right)+2\left(1,100.1,280\right)\right]+9\left(1,795.1,100\right)+7\left(1,795.2,460\right)+\left(2,52(2,1+2+2,01\right)\right\}$

$79,13+17,8+31+15,4 = 143,3m^2$

Net area: $666-143,3 = 523m^2$

Design load from wall self weight:

$\left(A_{net}.t_{wall}.\gamma_{concrete}\right) \Rightarrow \left(545.0,250.24\right) = 3270KN$

Total wall area of cantilever: $5,785.26,64 = 154m^2$

Openings: $\left\{\left(2.1,28\right)+\left(1,1.2,46\right)\right\}.9 = 47,4m^2$

Net area: $154-47,4 = 107m^2$

Design load from cantilever wall self weight: $\left(A_{net}.t_{wall}.\gamma_{concrete}\right) \Rightarrow \left(107.0,250.24\right) = 513KN$

- **Load from floors**:

$F_{floors} = (q_{DL} + q_{VL})(A_{floors})$

$F_{floors} = (7,9+1,75)\left[16(5,8.14,115\right]$

$F_{floors} = 12640KN$

Cantilever floor:

$F_{floors} = (q_{DL} + q_{VL})(A_{floors})$

$F_{floors} = (7,9+1,75)\left[10(5,8.5,785\right]$

$F_{floors} = 3238KN$

Total load: $F_{Total;0} = 3270+513+12640+3238 = 19661KN$

The load supported by the core is: Total load: $F_{Core;-1} = \frac{F_{total-1}}{L_{total-1}}.L_{core} \Rightarrow \frac{19661KN}{14,115}.3,5 = 4875KN$

(iii) Small walls:

- **-1':** **Own weight**: $(A_{wall}.t_{wall}.\gamma_{concrete}) \Rightarrow (3,1.47,2.0,20.24) = 703KN$

  Load from staircase: $F_{stair} = \frac{670}{2} = 335KN$

  Total load on -1" is : $F_{wall-1'} = 703 + 335 = 1038KN$
- **-1'': Own weight**: $(A_{wall}.t_{wall}.\gamma_{concrete}) \Rightarrow (3,1.47,2.0,20.24) = 703KN$

(iv). Longitudinal walls:

As there are two, each will weigh
Own weight: $2(A_{wall}.t_{wall}.\gamma_{concrete}) \Rightarrow 2(8,5.47,2.0,20.24) = 3852KN$

Total load on core is: $F_{Core} = 3796 + 4875 + 1038 + 703 + 3852 = 14264KN$

- **First order deflection**

$$M_{s;d;foot,x} = \frac{w_1.h_1^2}{2} + w_2.h_2\left(H - \frac{h_2}{2}\right) \Rightarrow \frac{9.20,56^2}{2} + 20.26,64\left(47,2 - \frac{26,64}{2}\right)$$
$$= 1902 + 18051 = 19953KNm$$

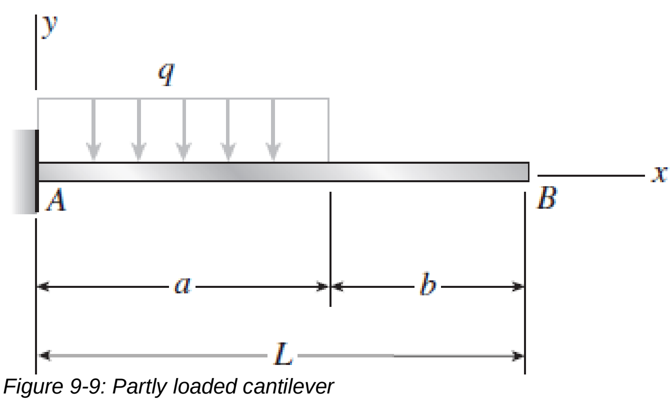


Figure 9-9: Partly loaded cantilever

$$EIv'' = M = -\frac{q}{2}(a-x)^2 = -\frac{q}{2}(a^2 - 2ax + x^2) \qquad (0 \le x \le a)$$

$$EIv' = -\frac{q}{2}\left(a^2x - ax^2 + \frac{x^3}{3}\right) + C_1 \qquad (0 \le x \le a)$$

B.C. 1 $v'(0) = 0 \quad \therefore C_1 = 0$

$EIv'' = M = 0 \qquad (a \le x \le L)$

$EIv' = C_2 \qquad (a \le x \le L)$

B.C. 2 $(v')_{\text{Left}} = (v')_{\text{Right}}$ at $x = a$

$$\therefore C_2 = -\frac{qa^3}{6}$$

$$EIv = -\frac{q}{2}\left(\frac{a^2x^2}{2} - \frac{ax^3}{3} + \frac{x^4}{12}\right) + C_3 \qquad (0 \le x \le a)$$

B.C. 3 $v(0) = 0 \quad \therefore C_3 = 0$

$$EIv = C_2 x + C_4 = -\frac{qa^3x}{6} + C_4 \qquad (a \le x \le L)$$

B.C. 4 $(v)_{\text{Left}} = (v)_{\text{Right}}$ at $x = a$

$$\therefore C_4 = \frac{qa^4}{24}$$

$$v = -\frac{qx^2}{24EI}(6a^2 - 4ax + x^2) \qquad (0 \le x \le a)$$

$$v = -\frac{qa^3}{24EI}(4x - a) \qquad (a \le x \le L)$$

$$\delta_B = -v(L) = \frac{qa^3}{24EI}(4L - a)$$

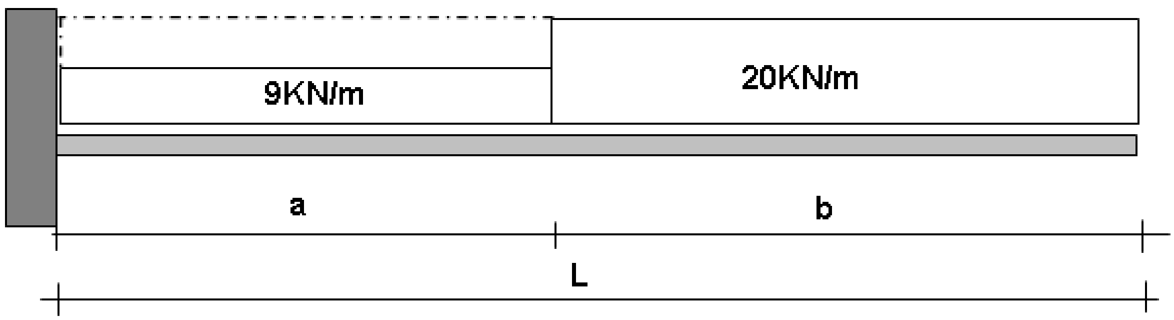


Figure 9-10: Core model as vertical cantilever

The deflection of core is calculated by considering that 20KN/m is applied on the total height and deducting the contribution of additional 11KN/m added to the part "a"

$$u_{core} = \frac{q_b L^4}{8EI} - \frac{q_a . a^3}{24EI}(4L - a) \Rightarrow \frac{20.47200^4}{8.36000.12,4.10^{12}} - \frac{11.20560^3}{24.36000.12,4.10^{12}}(4.47200 - 20560)$$

$$u_{core} = 27,8 - 1,5 = 26,3mm$$

- **Second order effect**

Second order moment

The angle of rotation: $Tan\theta = \frac{u}{L} \Rightarrow Tan\theta = \frac{26,3}{47200} = 5,6.10^{-4}$

The vertical load applied on walls without core contribution:

$$Q_{vd} - N_d' = (F_{Total;-1} - F_{core;-1}) + (F_{Total;0} - F_{core;0}) \Rightarrow (15307 - 3796) + (19661 - 4875) = 26297KN$$

$$Q_{vd} = N_{walls} + N_d' = 26297 + 14264 = 40561KN$$

Second degree moment is $M_{core;2nd} = Q_{vd} L \tan\alpha$; $M_{core;2nd} = 40561.47,2.5,6.10^{-4} = 1072KNm$

The total bending moment: $M_{core;1st} + M_{core;2nd} \Rightarrow 19953 + 1072 = 21025KNm$

Buckling load of core

The core is assumed to be partially restrained, therefore:

$$P_{E1;core} = \frac{\pi^2.EI}{L^2_0} \Rightarrow \frac{\pi^2.36000.12,4.10^{12}}{(1,12.47200)^2} = 15,8.10^8\,KN$$

$P_{E2;core} = \dfrac{C}{0,5L^2_0}$, where C is the foundation spring constant

Determination of foundation spring constant

Data: Number of piles in the area of core: 39 piles of 15 m long $L_p = 15m$, $\phi = 380mm$

$$A_p = \frac{\pi.380^2}{4} = 113354mm^2$$

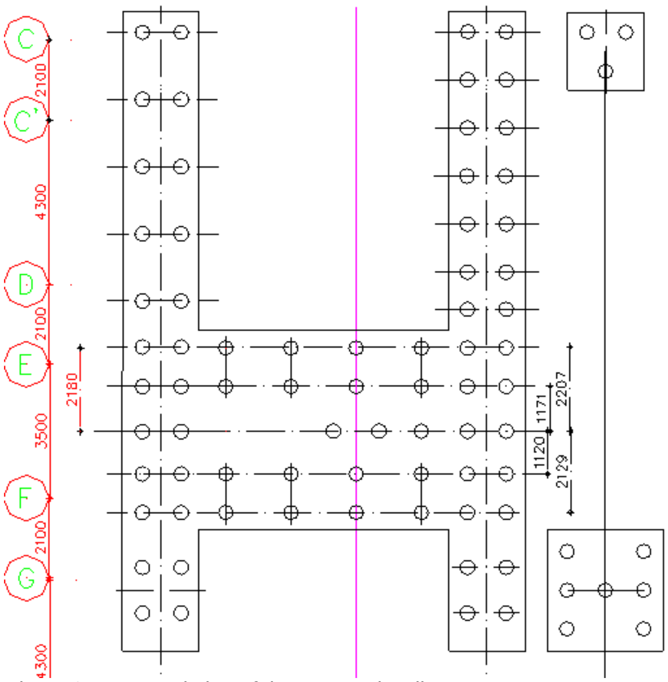


Figure 9-11: Foundation of the core and walls

Loads: Vertical loads is 14264 KN

Wind moment at the base: $21025KNm$

$P_n = \left(\dfrac{M}{I_p}.e\right)/n$, with "n", the number of piles with the same distance from centroidal axis

$$I_p = 8\left(1171^2 + 2207^2 + 1120^2 + 2129^2\right) = 96,2.10^6\,mm^4$$

$$P_n = \left(\frac{21025.10^6}{96,2.10^6}.2207\right)/8 = 60,3KN$$

Elastic deformation of a pile is as follow:
Pile material: Concrete B45

$$\Delta L = \frac{60,3.10^3.1,5.15000}{36000.113354} = 0,33mm$$

Then the rotation angle $\varphi = \dfrac{0,33}{2207}$ and $C = \dfrac{21025.2207}{0,33} = 140,6.10^6\,KNm/rad$

$$P_{E2} = \frac{C}{0,5L} \Rightarrow \frac{140,6.10^6}{0,5.47,2} = 5,95.10^6\,KN$$

The buckling force of partly restrained stabilizing core P_E is determined as follows:

$$\frac{1}{P_E} = \frac{1}{P_{E_1}} + \frac{1}{P_{E_2}} \Rightarrow \frac{1}{P_E} = \frac{1}{15,8.10^8} + \frac{1}{5,95.10^6} \Rightarrow \frac{1}{P_E} = \frac{5,95.10^6 + 15,8.10^8}{9,401.10^{15}} = \frac{1,586.10^9}{9,401.10^{15}}$$

$$\frac{1}{P_E} = \frac{1,586.10^9}{9,401.10^{15}} \Rightarrow P_E = 5,93.10^6\,KN$$

Magnification factor

$P_E = n\gamma P \Rightarrow n = \frac{P_E}{\gamma P} = \frac{5,93.10^6}{1,5.14125} = 280$ then, $\frac{n}{n-1} = \frac{280}{280-1} = 1,004$

The second degree effect is not large.

Deflection

- First order deflection is $u_{1st}\,26,3mm$
- The second order deflection $u_{2nd} = \frac{1}{n-1}u_{1st} \Rightarrow \frac{1}{280-1}26,3 = 0,1mm$
- With the second order effect, the resulting deflection $u_{tot} = 26,3 + 0,1 = 26,4mm$
- Foundation rotation $u_{rot} = \frac{qH^2}{2C} = \frac{19953}{140,6.10^6} \Rightarrow 0mm$
- Deflection due to moment loading in cantilevers: At each level , there will be moments from cantilevers. These moment magnify the horizontal deflection of the bracing members
  The considered loads: Dead load= $7,9.4,25 = 33,6KN/m$ and 8,5KN point load from façade
  Live load = $1,75.4,25 = 7,44KN/m$. Total distributed load is $33,6 + 7,44 = 41KN/m$

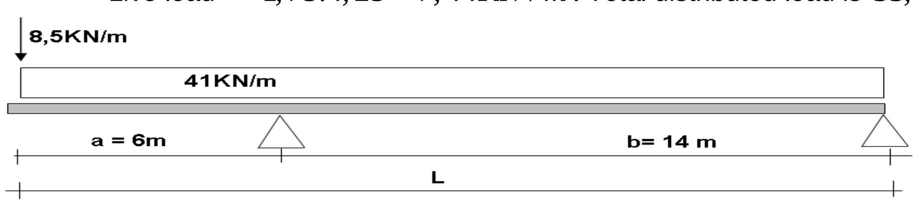


Figure 9-12: Girder model

The bending moment of cantilevers: $M_{cant} = \frac{41.6^2}{2} + 8,5.6 = 789KNm$

The deflection due to single moment loading $u_M = \frac{ML^2}{2EI}$

$$u_M = \frac{789}{2.36000.12,4.10^{12}}\left(20560^2 + 23520^2 + 26480^2 + 29440^2 + 32400^2 + 35360^2 + 38320^2 + 41280^2 + 44240^2 + 47200^2\right)$$

$$u_M = \frac{789}{2.36000.12,4.10^{12}}.1,22.10^{10} = 10,8mm$$

The final deflection; $u_{Total} = 26,3 + 0,1 + 0 + 10,8 = 37,1mm$

Stresses in core foot

$$\sigma_{core;\,foot} = \frac{P}{A_{core}} \pm \frac{M_{foot}}{W}, \qquad W = \frac{I_{core}}{1/2b} \Rightarrow W = \frac{12,4.10^{12}}{1650} = 75,2.10^{8}\,mm^{3}$$

From Appendix 7, $A_{core} = 6717500 mm^2$

$$\sigma_{core;\,foot} = -\frac{14264.10^{3}}{6717500} \pm \frac{21025.10^{6}}{75,2.10^{8}} \Rightarrow -2,1 \pm 2,8$$

$$\sigma_{core;\,foot} = 0,7 N/mm^2$$

$$\sigma_{core;\,foot} = -4,9 N/mm^2$$

With concrete grade B45 $f_{ctm} = 3,8 N/mm^2$ and $f_{cm} = 53 N/mm^2$; the core is then safe.

A10. CAPACITY OF OLD BUILDING

This section will emphasize on capacity of columns, foundation piles and lateral stiffness which can help in stabilizing the new building.

A10.1 The capacity of basic structural elements of the existing building.

The actual loads on structure are based on design made by "Construction Company A. van ECK N.V" in 1967.

Load on floors

Table10- 1: Loading of the existing building

(a) Loading on floors

	Type of load	Load[kg/m^2]	Total load [kg/m^2]
Roof	Own weight	240	400
	Finishing and isolation	50	
	Gravel	60	
	Snow	50	
6th floor	Own weight	240	550
	Finishing and isolation	60	
	Partition	50	
	Live load	200	
5th floor		550-20	530
4th floor		550-40	510
3rd floor		550-60	490
2nd floor		550-80	470
1st floor		550-100	450
Ground floor		550-120	430
q-total			3830

(b): Loading on balconies and corridors

	Type of load	Load[kg/m^2]	Total load
Roof	Own weight	270	510
	Light concrete	180	
	Finishing and isolation	50	
	Gravel	60	
	Snow	50	
6th floor	Own weight	270	470
	Live load	200	
5th floor		470-20	450
4th floor		470-40	430
3rd floor		470-60	410
2nd floor		470-80	390
1st floor		470-100	370
q-total			3030

(c) Loading on stair cases

	Type of load	Load[kg/m^2]	Total load [kg/m^2]
Roof			510
6th floor	Own weight	335	590
	Finishing and isolation	55	
	Live load	200	
5th floor		590-20	570
4th floor		590-40	550
3rd floor		590-60	530
2nd floor		590-80	510
1st floor		590-100	490
Ground floor	Own weight	240	380
	Finishing and isolation	60	
	Live load	80	
q-total			4130

A10.1.1 Foundation

The foundation consists of foundation beam and piles. The beams are made of reinforced concrete whose reinforcement are designed based on load coming from the above structure.
The load from the beams is transferred to the soil through the concrete piles-figure
After finding out the loading for each component using the load from table2-1a, b, c; the total loads on piles were calculated. Those loads on piles are summarized in tableA10-2.and were used in designing the capacity of the existing foundation. All piles used have a square cross section 340x340 mm, 15m long and a square point 450x450 mm with a maximum load of 70 ton (700KN).

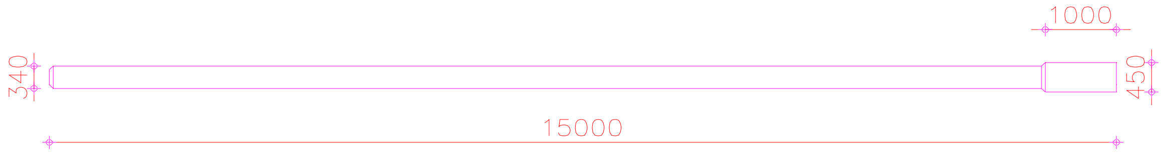


Figure A10- 1: Existing pile

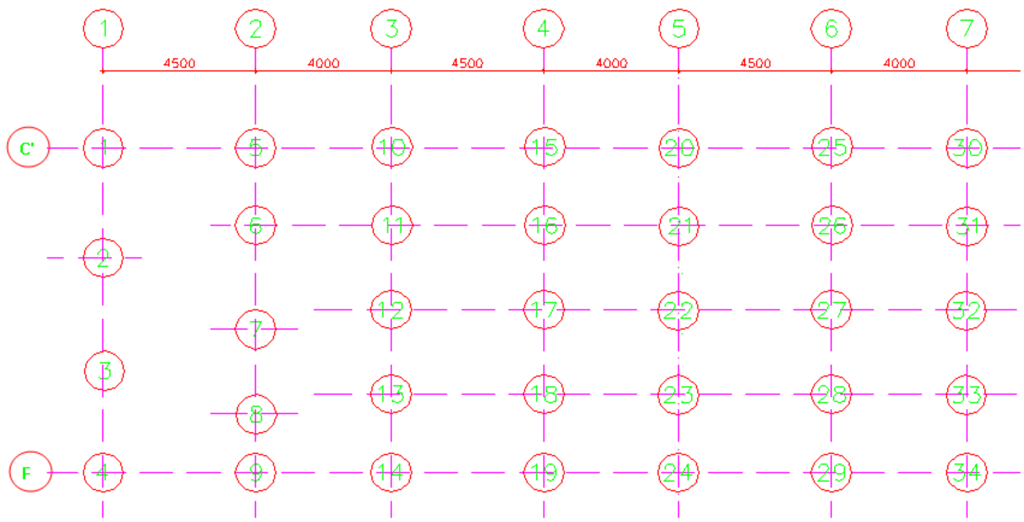


Figure A10- 2: Existing pile in plan

TableA10- 2 : Total loading on piles

# Pile	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Load[Kg]	66005	69170	68770	68600	61450	64120	48345	69625	64020	59880	60955	63080	69260	69260	69260	54600	58780
# Pile	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34
Load[Kg]	53355	59880	59880	60955	63080	69260	69260	69260	54600	58780	53355	59880	59880	60955	63080	69260	69260

The maximum capacity of existing piles is 1736 KN as it was calculated in Appendix 11. Table A10-2 shows that a heavily loaded pile receives approximately 700 KN; there is therefore unused capacity which can bear moderate loads.

A10.1.2 Columns

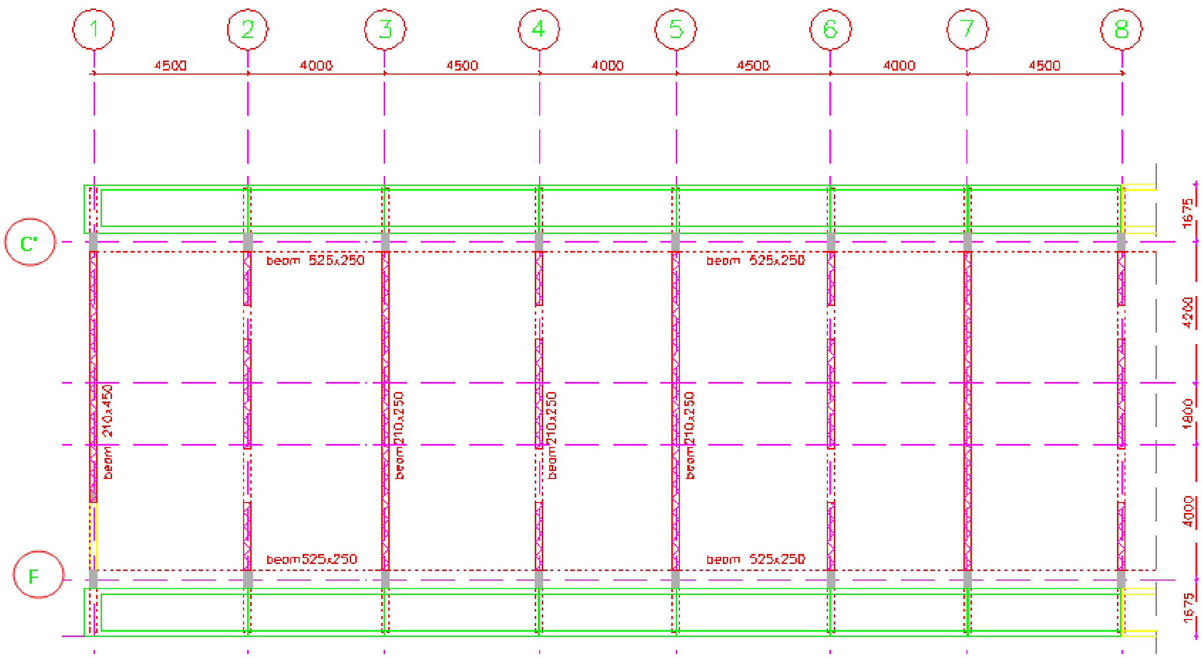


Figure A10- 3: Location of columns in old building

- **Applied axial forces from gravity load**

The gravity load forces in the columns come from the tributary areas. According to the calculation sheets from archives [20] , the loads used to design the columns of the existing structure are recorded in table A10-3. Axial forces in the columns and beams resulting from the horizontal loading should be estimated by simple static analysis of the analogous braced frame considering each infill as a diagonal strut. These two kind of axial forces are added to form the design force for each column.

Table A10-3 Load, moments and reinforcement of existing columns [20]

N_{max} in Kg
M_{max} in Kgm

Floor	Member	**Vertical load in Kg, moment in Kgm and reinforcement per column**																				
		Wall 1			Wall 2			Wall 3			Wall 4			Wall 5			Wall 6			Wall 7		
		N_{max}	M_{max}	R-bar	N_{max}	M_{max}	R-bar	N_{max}	M_{max}	R-bar	N_{max}	M_{max}	R-bar	N_{max}	M_{max}	R-bar	N_{max}	M_{max}	R-bar	N_{max}	M_{max}	R-bar
Under 6th floor	Col.C'	17060	1347	6ф10	19055	1048	6ф10	25580	1017	6ф10	25580	1017	6ф10	24030	1049	6ф10	25580	1017	6ф10	16510	1147	6ф10
	Col. F	17060	1347	6ф10	19055	1048	6ф10	27295	1017	6ф10	25580	1017	6ф10	24030	1049	6ф10	25580	1017	6ф10	18220	1147	6ф10
Under 5th floor	Col.C'	17060	1347	6ф10	19055	1048	6ф10	25580	1017	6ф10	25580	1017	6ф10	24030	1049	6ф10	25580	1017	6ф10	16510	1147	6ф10
	Col. F	17060	1347	6ф10	19055	1048	6ф10	27295	1017	6ф10	25580	1017	6ф10	24030	1049	6ф10	25580	1017	6ф10	18220	1147	6ф10
Under 4th floor	Col.C'	22600	1136	6ф10	25580	1017	6ф10	25580	1017	6ф10	25580	1017	6ф10	24030	1049	6ф10	25580	1017	6ф10	22150	1475	6ф12
	Col. F	27215	1556	6ф12	30180	1449	6ф14	27295	1017	6ф10	25580	1017	6ф10	24030	1049	6ф10	25580	1017	6ф10	24435	1475	6ф12
Under 3rd floor	Col.C'	28320	1346	6ф12	32090	1307	6ф12	32090	1307	6ф12	32090	1307	6ф12	30440	1348	6ф12	32090	1307	6ф12	27810	1806	6ф14
	Col. F	34460	1766	6ф16	38230	1629	6ф18	34375	1307	6ф12	32090	1307	6ф12	30440	1348	6ф12	32090	1307	6ф12	25525	1806	6ф14
Under 2nd floor	Col.C'	34050	1555	6ф14	38645	1598	6ф16	38645	1598	6ф16	38645	1598	6ф16	36650	1647	6ф16	38645	1598	6ф16	33465	2135	6ф18
	Col. F	41725	1975	6ф20	46320	1960	6ф20	41500	1598	6ф18	38645	1598	6ф16	36650	1647	6ф16	38645	1598	6ф16	36320	2135	6ф18
Load Under 1st floor	**Col.C'**	**39790**	**1765**	**6ф18**	**45170**	**1925**	**6ф20**	**45170**	**1925**	**6ф20**	**45170**	**1925**	**6ф20**	**42820**	**2033**	**6ф20**	**45170**	**1925**	**6ф20**	**39020**	**2465**	**6ф22**
	Col. F	**48990**	**2185**	**6ф22**	**54370**	**2250**	**6ф24**	**48600**	**1925**	**6ф22**	**45170**	**1925**	**6ф20**	**42820**	**2033**	**6ф20**	**45170**	**1925**	**6ф20**	**42450**	**2465**	**6ф22**
Total Moment	Col.C'		8496			7943			7881			7881			8175			7881			10175	
	Col. F		10176			9384			7881			7881			8175			7881			10175	
	Total moment		**18672**			**17327**			**15762**			**15762**			**16350**			**15762**			**20350**	

- **Applied force due to horizontal load**

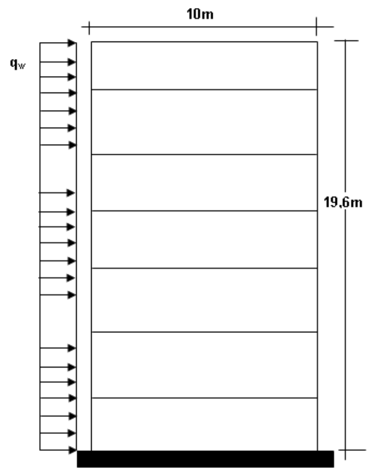


Figure A10- 4: Old building frame loaded horizontally

As the frame with in-fill masonry behaves as braced frame, the axial load in columns due to horizontal load can be calculated as follow:

$F_c = \pm \dfrac{M_H}{\frac{1}{2}b}$; $M_H = \frac{1}{2} q_w . H^2$; $q_w = q_{w,rep}.L \Rightarrow 1.28,6 = 28,6 KN/m$; as there are 7 frames, one takes $28,6/7 = 4KN/m$

$M_H = \frac{1}{2} q_w . H^2 \Rightarrow \frac{1}{2} 4.19,6^2 = 768 KNm$, therefore $F_c = \pm \dfrac{768}{\frac{1}{2}10} = \pm 154 KN$

Where F_c : axial force as a result of horizontal load
M_H: Moment due to horizontal load
b and L: the width and the length of building
H : height of building
$q_{w,rep}$: Representative wind pressure
q_w : wind load

TableA10-4: Axial load due to lateral moment and moment from vertical load

Lateral axis	Total moment due to vertical load [KNm]	Moment due to horizontal load [KNm]	Total moment [KNm]	Width [m]	Equivalent axial load [KN]
1	187	768	955	10	191
2	173	768	941	10	188
3	158	768	926	10	185
4	158	768	926	10	185
5	164	768	932	10	186
6	158	768	926	10	185
7	204	768	972	10	194

- **Axial load capacity of columns**

In a reinforced concrete column, both longitudinal steel and concrete assist in carrying the load. The links (stirrups) prevent the longitudinal bars from buckling.
According to Euro Code 2 [23], the ultimate load capacity of column is given by the following expression

$$N = \alpha f_{cd} A_c + A_{st} f_{yd}$$

$\alpha = 0,85$, $f_{cd} = \frac{f_{ck}}{1,5} \Rightarrow \frac{20}{1,5} = 13,3N/mm^2$ and $f_{yd} = \frac{f_{yk}}{1,15} \Rightarrow \frac{400}{1,15} = 348N/mm^2$

Where N is the ultimate load capacity

F_{cu}: ultimate strength of concrete (K300 or 20N/mm^2)
A_c: area of concrete ($A_{tot} - A_{st}$)
A_{st}: Total area of longitudinal reinforcement
f_y: Yield strength of steel(Q40 or 400N/mm^2)

The capacity for all columns on ground level can be calculated and compared to the applied load. The applied load is found in table A10-3 and A10-4 under first floor. The calculations are summarised in table A10-5:

TableA10- 6: Calculation of the capacity of the existing columns

Longitudinal Axis	Transversal Axis	f_{cd} N/mm^2	A_c mm^2	A_{sc} mm^2	f_{yd} N/mm^2	Ultimate capacity[KN]	Applied load load [KN]	Applied load Due to M_H KN	Applied load Total applied load[KN]	% of used capacity
C'	1	13.3	107100	1526	348	1741.8	397.9	191.0	588.9	34
	2	13.3	107100	1884	348	1866.4	451.7	188.0	639.7	34
	3	13.3	107100	1884	348	1866.4	451.7	185.0	636.7	34
	4	13.3	107100	1884	348	1866.4	451.7	185.0	636.7	34
	5	13.3	107100	1884	348	1866.4	428.2	186.0	614.2	33
	6	13.3	107100	1884	348	1866.4	451.7	185.0	636.7	34
	7	13.3	107100	2280	348	2004.2	390.2	194.0	584.2	29
F	1	13.3	107100	2280	348	2004.2	489.9	191.0	680.9	34
	2	13.3	107100	2713	348	2154.9	543.7	188.0	731.7	34
	3	13.3	107100	2280	348	2004.2	486.0	185.0	671.0	33
	4	13.3	107100	1884	348	1866.4	451.7	185.0	636.7	34
	5	13.3	107100	1884	348	1866.4	428.2	186.0	614.2	33
	6	13.3	107100	1884	348	1866.4	451.7	185.0	636.7	34
	7	13.3	107100	2280	348	2004.2	424.5	194.0	618.5	31

According to the results, most of the columns use less than 40% of their capacity. The capacity left can take moderate loads but not as much as 9 floors.

A10.2 Stability of the existing building

The frame of old building is filled with masonry work or in-fill required for space separation and for bracing means of the frame.

- **Calculation of old frame stability**

Properties of frame

The frame consists of seven parallel bents as shown in figure A10-3 and are located at 4, 5 and 4, 0 m centers. All bays are in-filled by 210m- thick walls made of concrete hollow blocks filled with concrete and reinforced by steel bars.

The horizontal wind pressure is 1KN/m2:

Columns; h=510mm, b= 210 mm,

$$I = \frac{b.h^3}{12} \Rightarrow I = \frac{210.510^3}{12} = 2,32.10^9 mm^4$$

Floor beams: h= 250 mm; b = 210 mm
Façade beam: h= 250mm; b = 510 mm
Concrete grade K300:

Concrete grade	K300	K225		Steel grade	QR 24	QR 40
f_{ck} [N/mm2]	20	15		f_{sk} [N/mm2]	240	400
f_{ctm} [N/mm2]	2,0	1,5		E_s [kN/mm2]	210	210
R_d [N/mm2]	3,6	2,7				

Properties of infill

The infill consists of concrete blocks filled with concrete both of K 225.
The allowable compressive stress: $f_m = 15N/mm^2$
The allowable coefficient of friction: $\mu = 0,2$
The allowable bond shear strength [NEN-EN998-2:2003, NEN-EN206] is
$f_{bs} = 0,15N/mm^2$
The modulus of elasticity [NEN-EN1996-1-1] for short term is $E = K_E.f_k$
$K_E = 1000$,.
Then $E = 1000.15 = 15000N/mm^2$

The infill diagonal:
$\theta = \tan^{-1}(2800/10000) = 15,6^0 \quad \cos\theta = 0,96$
The diagonal length is
$L_d = \sqrt{2800^2 + 10000^2} = 10385mm$
Wind shear at the base of one frame is as follow:

$Q_0 = \frac{q_w}{n}.h$ Where q_w is the wind load acting on old building section to be raised to 15 floors
Q_o: Shear force at the base of frame
h: height of building and n: number of lateral frames

$\Rightarrow Q_o = \frac{25,5}{7}.19,6 = 71,4KN$

Structure shear strength – Infill shear failure:

$$Q_s = \frac{f_{bs}Lt}{1,43 - \mu(0,8h/L - 0,2)}$$

$$Q_s = \frac{0,15.10000.210}{1,43 - 0,2(0,8(2800/10000) - 0,2)} = 221022N \Rightarrow 221KN$$

Structural shear strength – Infill Compressive failure:

$$Q_c = 2f_m\cos^2\theta\sqrt[4]{Iht^3}$$

$$Q_c = 2.15.0,96^2\sqrt[4]{2,32.10^9.2800.210^3} = 2435KN$$

The infill is just adequate to carry the external shear on the basis of the shear failure criterion(strength =221KN compared with load of 71,4KN) and more than adequate on the basis of compressive failure criterion

(strength = 2435KN). In addition to these calculations for the strength of the infill, the frame members should be checked to see that they are adequate to carry the bending moment and shear force due to lateral load.

Checking the deflection

A conservative estimate of the horizontal deflection of an in-filled frame is done by the calculating deflection of the equivalent pin jointed braced frame by replacing each infill by a diagonal strut(Figure A11-5) with a cross sectional area equal to the product of one-tenth of its diagonal length and its thickness [6]

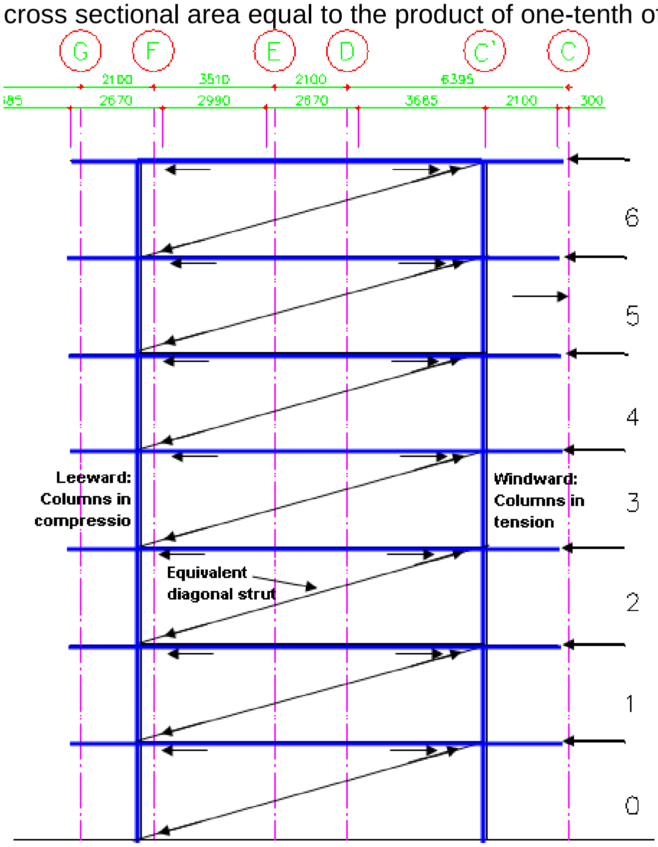


Figure A10-5: Old building modelled as braced frame

(i). Flexural component

The equivalent diagonal strut has this cross section:

$$A_d = \frac{1}{10} L_d . t \Rightarrow A_d = \frac{1}{10} 10385.210 = 218085 mm^2$$

An equivalent rectangular section of 650x336 mm can be used in a calculation which gives an equivalent area of $A_d = 218400 mm^2$

As the infill with frame behave as braced bent, the deflection for each transverse frame is calculated as follows

$$\delta = \frac{Q}{E}\left(\frac{d^3}{L^2 A_d} + \frac{L}{A_g} \right)$$ [6]

Calculation procedures:

Compute the moment of inertia of the column sectional area about their common centroid.

$$I_{old,1-7} = A_F . L_F{}^2 + A_{C'} . L_{C'}{}^2 \quad I_{old,1-7} = 510.210.1765^2 + 510.210.8150^2 = 7,45.10^{12} mm^4$$

$$I_{y,\text{in-fill}} = \frac{210.10000^3}{12} = 1,75.10^{13}\,mm^4$$

$$I_{y,total} = 17,5 + 7,45 = 25.10^{12}\,mm^4$$

These values are put in table A10-5 column 7.

Compute the value of the external moment at each story and enter the value in column 6.

Determine for each story the value of hM/EI, and enter the results in column 9.
Determine for each story i, the accumulation of $u.\theta_{if}$ from the ground level up to and including story i, θ_{if} and record it in column10. Such accumulated values give the inclination of each story i due to flexure θ_{if} .
Record the product of h_i and θ_{if} in column 11. $h_i\theta_{if}$ is the drift in story i, u_{if} , due to flexure.
At each level, the value of the lateral drift is required, evaluate the accumulation of the story drift , u_{if} ,from the ground level up and considered n[th] floor to give the drift u_{total} due to flexure. Enter these in column 12. Therefore one can see that the total drift due to flexure on top of the building is 0,16 mm

TableA10- 5: Evaluation of flexural component of drift

Story	Story heigh mm	Cum.height Ground -roof mm	Cum.height roof-ground mm	Lateral load KN/m	External moment Nmm	Frame inertia $x10^{12}mm^4$	Mod. Elast. E $x10^4N/mm^2$	Storey Inclination(Rads) δθi=h*M/EI	Inclinastion θ_{if} Rads	Storey drift mm	Cum. Drift mm
Roof	2800	19600	0	3,64	0	25	1,5	0,00	9,70E-06	0,03	0,16
6	2800	16800	2800	3,64	1,4E+07	25	1,5	0,00	9,70E-06	0,03	0,13
5	2800	14000	5600	3,64	5,7E+07	25	1,5	0,00	9,59E-06	0,03	0,10
4	2800	11200	8400	3,64	1,3E+08	25	1,5	0,00	9,16E-06	0,03	0,08
3	2800	8400	11200	3,64	2,3E+08	25	1,5	0,00	8,20E-06	0,02	0,05
2	2800	5600	14000	3,64	3,6E+08	25	1,5	0,00	6,50E-06	0,02	0,03
1	2800	2800	16800	3,64	5,1E+08	25	1,5	0,00	3,84E-06	0,01	0,01
Ground floor	0	2800	19600	3,64	7E+08	25	1,5	0,00	0,00E+00	0,00	0,00

The total drift on top of the building due to the flexural component is $\delta_{f;total} = \sum \delta_{f;i} \Rightarrow 0,16mm$

(ii).Shear component

Compute the value of Q_i for each story, and enter in column 4.

Compute the drift for each storey δ_i , and enter it in column 11

Make the summation of each storey drift from the first floor up to the roof in column12. The final drift due to shear is 4,6 mm (see table 10-6)

TableA10- 7 Evaluation of shear component of drift

Story	Lateral load KN/m	Acc.height Ground -roof mm	Shear Q KN	L_{BR} mm	L_B mm	Ad mm^2	A_B mm^2	I_B mm^4	E_{BR} N/mm^2	storey drift(mm) $\delta_i = \frac{Q}{E}\left(\frac{d^3}{L^2 A_d} + \frac{L}{A_g}\right)$	Accu. Drift mm
Roof	3,64	0	0	10385	10000	218400	52500	2,73E+08	15000	0,00	4,60
6	3,64	2,8	10,192	10385	10000	218400	52500	2,73E+08	15000	0,16	4,60
5	3,64	5,6	20,384	10385	10000	218400	52500	2,73E+08	15000	0,33	4,44
4	3,64	8,4	30,576	10385	10000	218400	52500	2,73E+08	15000	0,49	4,11
3	3,64	11,2	40,768	10385	10000	218400	52500	2,73E+08	15000	0,66	3,61
2	3,64	14	50,96	10385	10000	218400	52500	2,73E+08	15000	0,82	2,96
1	3,64	16,8	61,152	10385	10000	218400	52500	2,73E+08	15000	0,99	2,14
Ground floor	3,64	19,6	71,344	10385	10000	218400	52500	2,73E+08	15000	1,15	1,15

$$\delta_{s;total} = \sum \delta_{s;i} \Rightarrow 4,6mm$$

The total drift is the summation of both flexural and shear drifts:

$$\delta_{total} = \delta_{f;total} + \delta_{s;total} \Rightarrow 0,16 + 4,6 = 4,8mm$$

APPENDIX 11. FOUNDATION DESIGN

A11.1 Load on foundation

The load on foundation is basically made of three components:

- Self weight of all structural members
- Variable loads(Occupants and furniture)
- Lateral load (Wind and second order effect)

A11.1.1 Vertical load of foundation

Self weight

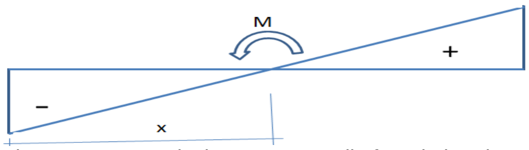
Self weight comes from structural members themselves and depends on their properties. One considers the weight from floor slabs and floor finishing, services, columns, beams and concrete core.
The part of the building which is put on top of the existing building has a load of 6,14 KN/m^2 for roof and 7,14 KN/m^2 for other floors (see Appendix2).

Variable load

The function of this building is accommodating people; the variable representative load of 1,75KN/m^2 was selected according to NEN 6702. [1]

A11.1.2 Lateral loads

The lateral used in this design is the wind load and the load coming from imperfection. According to Appendix 2, the design lateral load to the structure is 54,4KN/m.
The lateral load moment causes tension in windward side and compression in leeward side to the foundation-figure A11-1



FigureA11- 1: Vertical pressure on pile foundation due to lateral load

The load on pile due to lateral moment is determined by using the following formula [24].

$$P_{later.} = \frac{M.x}{I} \quad \text{With} \quad M = \frac{1}{2} q_{later.} h^2$$

X: distance from the building center to the edge pile and h, the height of the building

$$I = \sum_{1}^{i\max} n.A_i.x_i^2$$

A_i is the area of pile

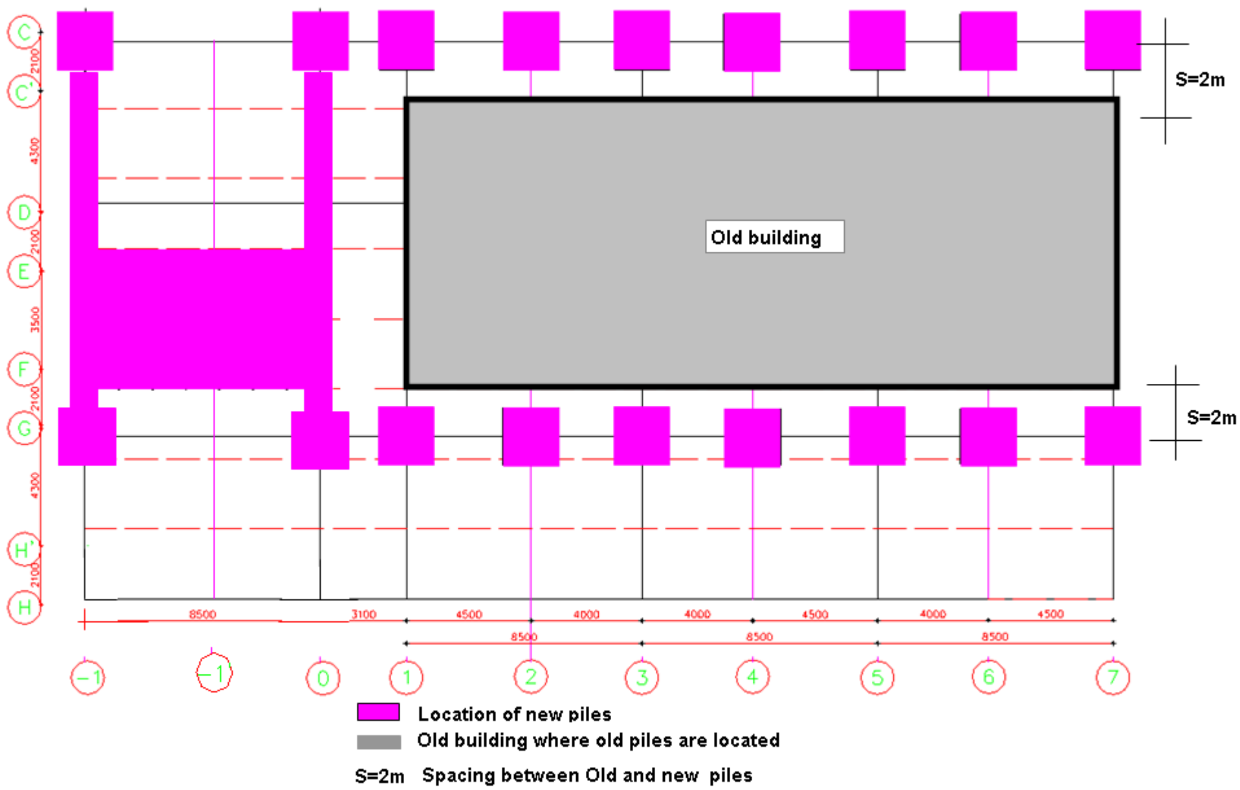
n: is the number of pile

X_i is distance from the building center to the ith pile

All loads stated above are combined as it is shown in appendix 10 and the critical combination will be used as design load on foundation.

The structure is analyzed by using matrix frame program, where by all load combination are considered to obtain member forces and support reaction.
The design loads for foundation are found in chapter 11; various load combinations were analyzed . Then, the design load for foundation is taken from the most taken as critical load combination.

The l load to the foundation shall be distributed among piles in the plan. The considered pile rows are located along both long sides of the building since the load of the new structure has to be carried to the sides of the old building. Figure11-2 shows where foundation pile will be located.



FigureA11- 2: Location of piles (pink areas)

A11.2 Calculation bearing capacity of new piles (Fundex piles)

The capacity of foundation pile is calculated by using the Dutch code NEN6743 [3] and is calculated in vicinity of CPT

The maximum point resistance is calculated according to the following formula:

$$p_{r;\max;point} = \frac{1}{2}\alpha_p \beta s \left(\frac{q_{c;I;mean} + q_{c;II;mean}}{2} + q_{c;III;mean} \right)$$, where,

$p_{r;\max;point}$: is the maximum point resistance

$q_{c;I;mean}$: Is the mean value of the cone resistance $q_{c;z;corr}$ in trajectory I, determined in accordance with NEN 6743:5.3.3.3, that runs from the pile point level to a level that is at 0.7 times and at most 4 times the equivalent diameter D_{eq} deeper.

$q_{c;II;mean}$: Is the mean value of the cone resistance $q_{c;z;corr}$ in trajectory II, determined in accordance with NEN 6743:5.3.3.3, that runs from the bottom of trajectory I to the pile point level.

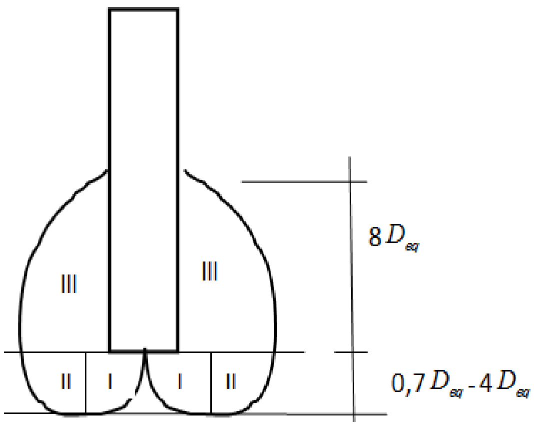
$q_{c;III;mean}$: Is the mean value of the cone resistance $q_{c;z;corr}$ in trajectory III, determined in accordance with NEN 6743:5.3.3.3, that runs from the pile point level to a level that is 8 times the equivalent diameter D_{eq} higher.

α_p : is the pile class factor, determined in accordance NEN 6743:5.3.3.1.1

β is the factor that takes into account the influence of shape of the pile base determined in accordance NEN 6743:5.3.3.1.2

s is the factor that takes into account the influence of shape of the cross-Section of the pile base determined in accordance NEN 6743:5.3.3.1.3.

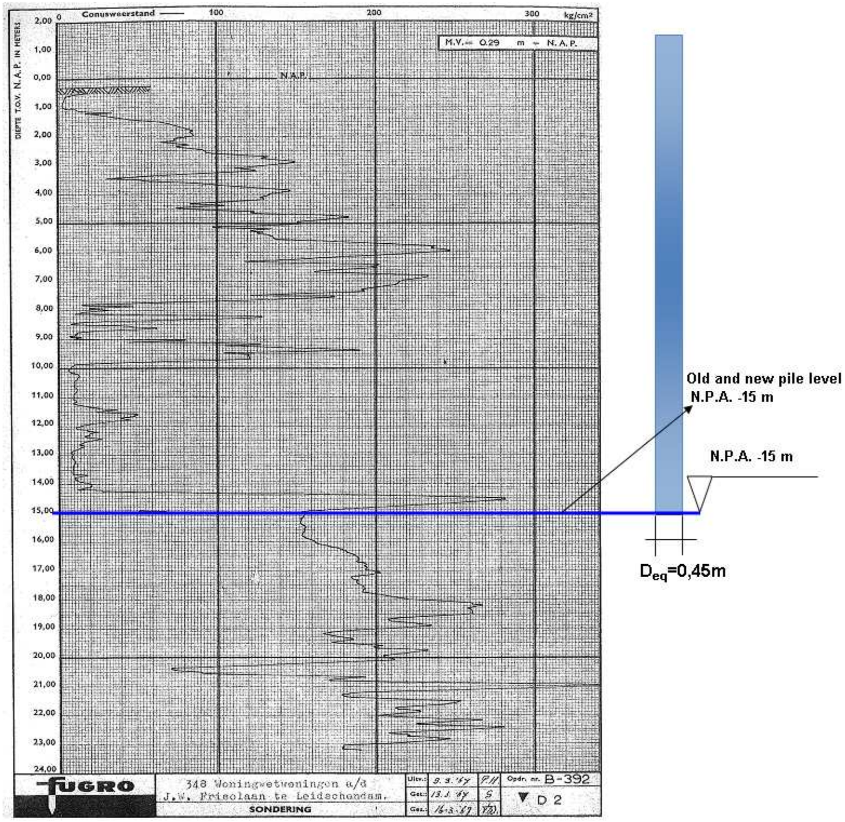
The figure 14-3, give the failure envelope of the foundation sole around the pile point and where the mean cone resistances are taken.



FigureA11- 3: Impact depth

(i). Foundation data

According to the boring profile, the pile tip will be put on sand layer with sufficient bearing capacity (figure 11-4). It will be put at 15 m deep and Fundex pile with 380mm diameter will be used due to their benefits toward the non disturbance of the existing building foundation.



FigureA11- 4: Boring Profile

$D_{eq} = D_{point} = 0,45m$ for shaft diameter of 380mm

$0,7D_{eq} = 0,7.0,45 = 0,315m$

$4D_{eq} = 4.0,45 = 1,8m$

$8D_{eq} = 8.0,45 = 3,6m$

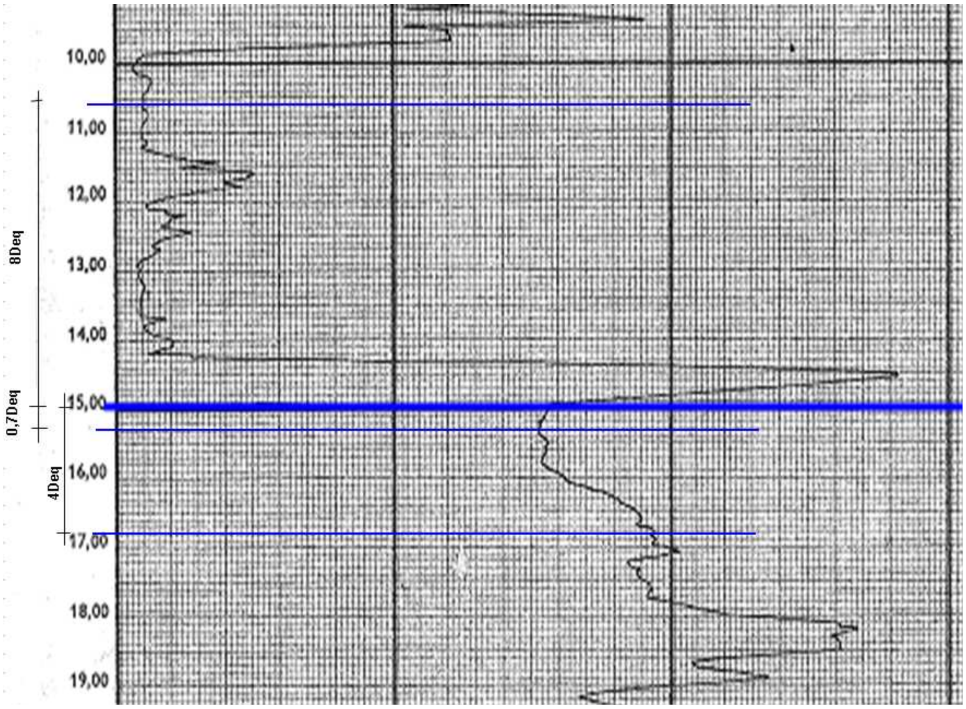
In determination of $q_{c;I;mean}$ and $q_{c;II;mean}$, the smallest values are taken from $0,7D_{eq}$ to $4D_{eq}$

The determination of the respective average values of $q_{c;I;mean}$, $q_{c;II;mean}$, $q_{c;III;mean}$ for the bottom range of I / II N.A.P -16,8 m($4D_{eq}$) is explained in figure 11-5.

$q_{c;I;mean} = 14,7MPa$

$q_{c;II;mean} = 14,7MPa$

$q_{c;III;mean} \simeq 4,5MPa$



FigureA11- 5: Determination of $p_{r;\max;point}$ with a depth range up to N.A.P-16,8m

(ii). Factors

The following factors depend on the geometry and the type of the pile [NEN6743: 5.4.2.2.2-4]
$\alpha_p = 0{,}9$; $\beta = 1$ and $s = 1$

$$p_{r,\max,point} = \frac{1}{2}.0{,}9.1.1\left(\frac{14{,}7+14{,}7}{2}+4{,}5\right) = 8{,}64MPa$$

(iii). Calculation of $F_{r;\,max;\,point}$

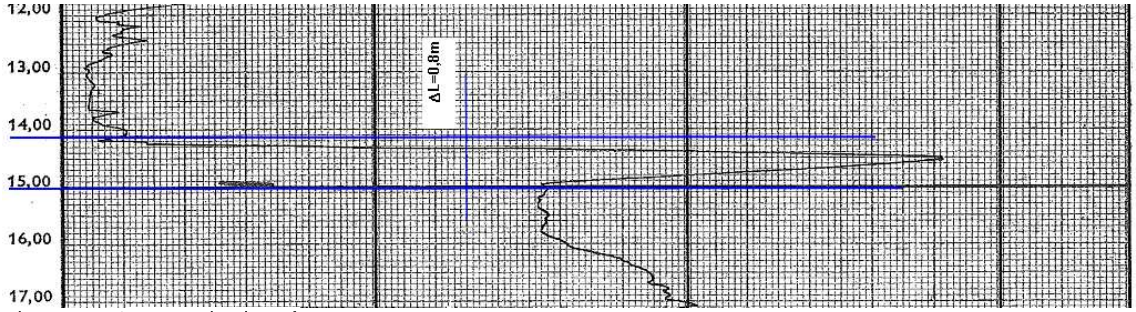
$$F_{r,\max,point} = p_{r,\max,point}.A_{point} = 8{,}64.10^6.\frac{\pi.0{,}45^2}{4} = 1373KN$$

(iv). Calculation of the maximum shaft resistance.

The pile capacity may increase due to some positive friction layers in the soil profile.

$$F_{r;\max;shaft} = Op.\Delta L.p_{r;\max;shaft}$$

$F_{r,\,max,\,shaft}$, is the maximum frictional force at sounding i, in KN
$p_{r,\,max,shaf}$, is the maximum shaft resistance in kN/m2;
Os; ΔL, is the average circumference of the portion of the pile shaft in the layer, where pile foot is placed in m;
ΔL is the length of the part of the pile, evolved in the shaft friction in m
ΔL is determined by using CPT, figure 11-4 shows that the sand layer from the pile foot NAP-15m up to NAP - 14,2m may be counted. The sand layers above the clay layer (between NAP-14,2 and NAP-10) are not included because of this clay layer.



FigureA11- 6: Determination of ***D*** *L*

The maximum unit shaft resistance is given by the following formula:

$p_{r;\max;shaft} = a_s.q_{;c;z;a}$

Where: a_s is the factor, that takes into account the influence of pile installation; it is equal to 0,009 for displacement pile cast -in-place.

$q_{;c;z;a}$ Is the average cone resistance over **D** L, in this case is $15Mpa$

Therefore, $p_{r,\max,shaft} = 0.009.15 = 135MPa$,

O_p= π.D= π.0, 38 = 1,2m
D L=0,8m

Then, the maximum skin friction resistance will be :

$F_{r,\max,shaft} = 1,2.0,8.0,135.10^3 = 130KN$

(v). The total capacity of the pile:

$F_{r,\max} = F_{r,\max,po\text{int}} + F_{r,\max,shaft}$

$F_{r,\max} = 1373 + 130 = 1503KN$

(vi). Safety factor

$$F_{r;\max;d} = \frac{\xi.F_{r;\max}}{\gamma_M}$$

ξ is a factor that depends on number of pile (M) and the number of cone penetration test CPT(N). In this project, six CPT were done and by assuming that the number of pile under columns is 4., therefore ξ = 0,87

γ_M is material factor = 1,20.

Finally $F_{r,\max,d} = \dfrac{0,87.1503}{1,2} = 1090KN$

A11.3 Capacity of piles in old building

The capacity of old piles is calculated in the same way as new pile, however some changes exist.

The pile dimensions are 450x450mm and 340x340mm for point and shaft respectively

$D_{eq} = 1{,}13.450 = 0{,}51m$ for shaft diameter of 380mm

$$0{,}7D_{eq} = 0{,}7.0{,}51 = 0{,}36m$$

$$4D_{eq} = 4.0{,}51 = 2{,}04m$$

$$8D_{eq} = 8.0{,}51 = 4{,}1m$$

Assumption: No negative friction

$\alpha_p = 1 \quad a_s = 0{,}014$

$$p_{r,\max,point} = \frac{1}{2}.1.0{,}9.1\left(\frac{14{,}7+14{,}7}{2} + 4{,}5\right) = 8{,}64MPa$$

$$F_{r,\max,point} = p_{r,\max,point}.A_{point} = 8{,}64.10^6.0{,}51^2 = 2247KN$$

$$p_{r,\max,shaft} = 0.009.15 = 135MPa$$

O_p= 4.0.34 = 1,36m
D L=0,8m

Then, the maximum skin friction resistance will be :

$$F_{r,\max,shaft} = 1{,}36.0{,}8.0{,}135.10^3 = 147KN$$

(v). The total capacity of the pile:

$$F_{r,\max} = F_{r,\max,point} + F_{r,\max,shaft}$$

$$F_{r,\max} = 2247 + 147 = 2394KN$$

$$F_{r;\max;d} = \frac{\xi.F_{r;\max}}{\gamma_M}$$

$\xi = 0{,}87$ and $\gamma_M = 1{,}2$, $F_{r,\max,d} = \frac{0{,}87.2394}{1{,}2} = 1736KN$

A11.4 Required number of pile for new foundation

(a). Columns supporting steel frames in axis 1 to 7

The stabilizing frames (axis 1, 3 and 5) were analyzed; the resulting vertical loads from all load combinations are listed in appendix 11. All foundations in this area should be designed according to the reaction force observed in stabilizing frames.

The maximum vertical load for supports is given by load combination 5.

- Total vertical load in axis **G** is 6658 KN

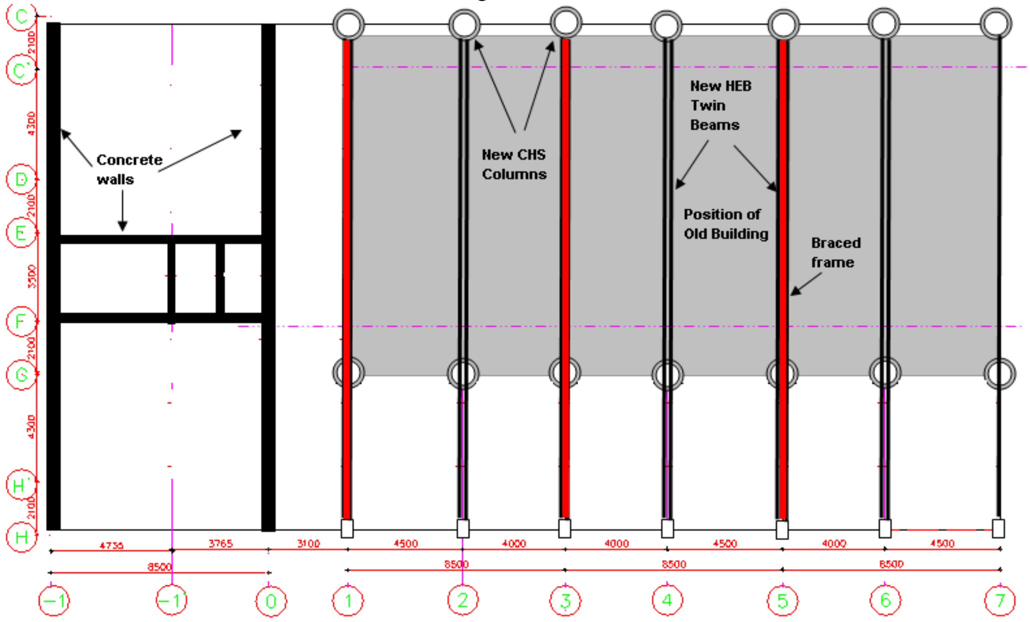
The number of piles is: $\frac{6658}{1090} \simeq 7$

- Total vertical load in axis **C** is 2749 KN

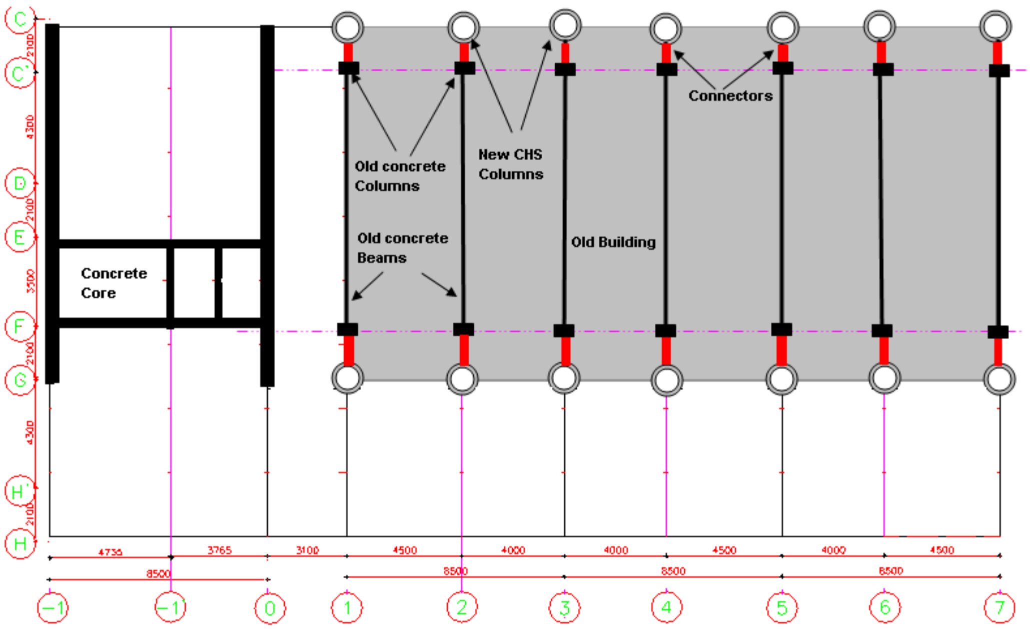
The number of piles is: $\frac{2749}{1090} \simeq 3$

(b). For concrete walls.

The concrete walls are the main stabilizing structures of the whole building. The reactions forces acting in the foot of the walls consist of walls own weight, live load, and load from horizontal force moment.

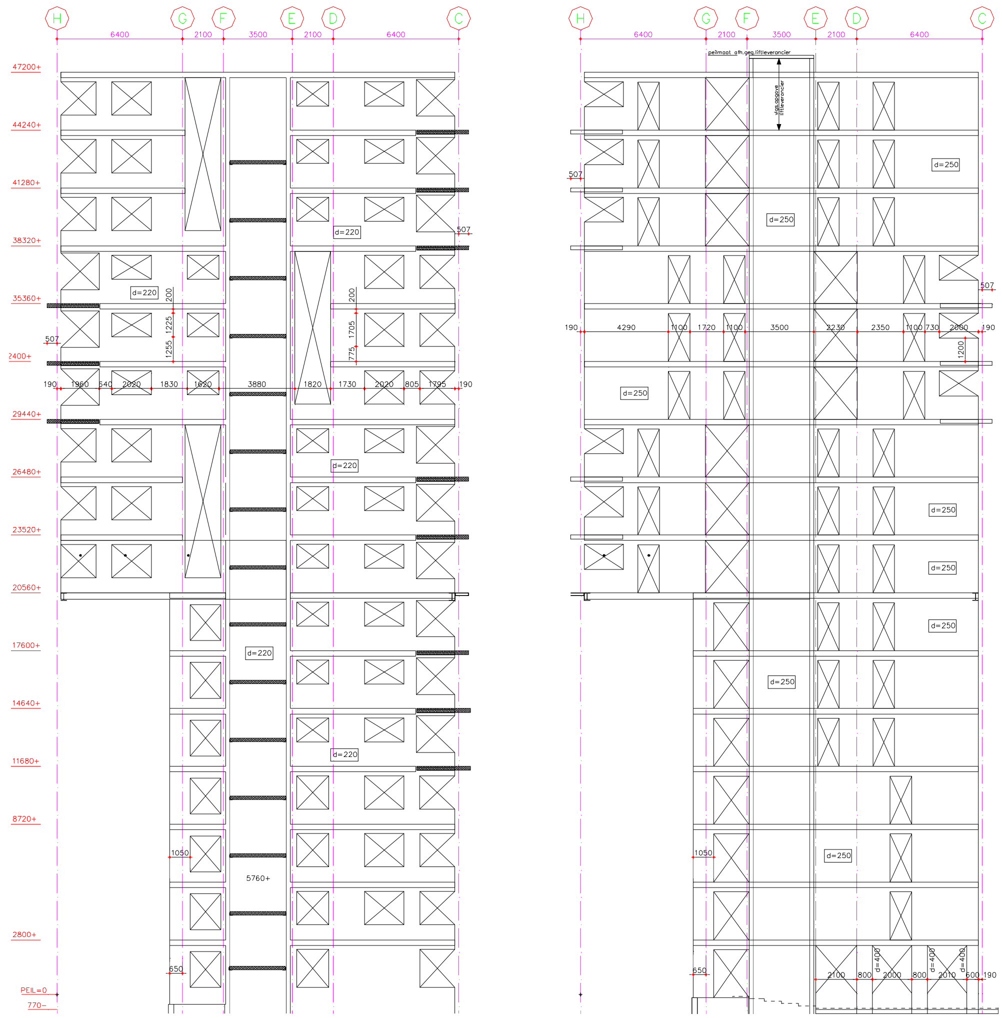


From 7th to 15th floor



From 1st to 6th floor

FigureA11- 7: Floor plans with concrete wall in axes -1 and 0



Section in axis -1 *Section in axis 0*

FigureA11- 8: section in axes -1 and 0

(i). wall -1:

- **Self weight of wall and cantilever**

Total wall area: $14,115.47,2 = 666m^2$

Openings: $\{(1,62.1,225)+(1,82.2,185)+(2,02.1,225)+(1,795.1,875)\}.16 = 189m^2$

Net area: $666-189 = 477m^2$

Design load from wall self weight: $1,2(A_{net}.t_{wall}.\gamma_{concrete}) \Rightarrow 1,2(477.0,250.24) = 3434KN$

Cantilever:

Total wall area: $5,785.26,64 = 154m^2$

Openings: $\{(1,96.1,875)+(2,02.1,225)\}.9 = 55,3m^2$

Net area: $154-55,3 = 99m^2$

Design load from wall self weight: $1,2(A_{net}.t_{wall}.\gamma_{concrete}) \Rightarrow 1,2(99.0,250.24) = 570KN$

- **Load from floors:**

$F_{floors} = (1,2q_{DL} + 1,5q_{VL})(A_{floors})$

$F_{floors} = (1,2.7,9 + 1,5.1,75)[16(4,25.14,115]$

$F_{floors} = 11619KN$

Cantilever:

$F_{floors} = (1,2q_{DL} + 1,5q_{VL})(A_{floors})$

$F_{floors} = (1,2.7,9 + 1,5.1,75)[9(4,25.5,785] = 2976KN$

Load from staircase:

$F_{stair} = (1,2q_{DL} + 1,5q_{VL})(A_{stair})$

$F_{floors} = (1,2.3,7 + 1,5.2)\left[16\left(\frac{4,74}{2}.3,1\right)\right]$

$F_{stair} = 7,44.118 = 878KN$, a half goes to the wall -1.

Total load is: $F_{wall-1} = F_{self} + F_{floors} + F_{stair} \Rightarrow 3434 + 570 + 11619 + 2976 + 439 = 19038KN$

Core contribution: $F_{core;-1} = \dfrac{19038}{14,115}.3,5 = 4721KN$

ii).Wall 0

- **Self weight of wall 0 and its cantilever:**

Total wall area: $14,115.47,2 = 666m^2$

Openings:

$\{9[(2,230.2,68) + 2(1,100.1,280)] + 9(1,795.1,100) + 7(1,795.2,460) + (2,52(2,1 + 2 + 2,01)\}$

$79,13 + 17,8 + 31 + 15,4 = 143,3m^2$

Net area: $666 - 143,3 = 523m^2$

Design load from wall self weight:

$1,2(A_{net}.t_{wall}.\gamma_{concrete}) \Rightarrow 1,2(523.0,250.24) = 3766KN$

Total wall area of cantilever: $5,785.26,64 = 154m^2$

Openings: $\{(2.1,28) + (1,1.2,46)\}.9 = 47,4m^2$

Net area: $154 - 47,4 = 107m^2$

Design load from cantilever wall self weight: $1,2(A_{net}.t_{wall}.\gamma_{concrete}) \Rightarrow 1,2(107.0,250.24) = 616KN$

- **Load from floors**:

$F_{floors} = (1,2q_{DL} + 1,5q_{VL})(A_{floors})$

$F_{floors} = (1,2.7,9 + 1,5.1,75)[16(5,8.14,115]$

$F_{floors} = 15856KN$

Cantilever floor:

$$F_{floors} = (1,2q_{DL} + 1,5q_{VL})(A_{floors})$$

$$F_{floors} = (1,2.7,9 + 1,5.1,75)\left[10(5,8.5,785\right] = 4062KN$$

Total load is: $F_{wall0} = F_{self} + F_{floors} + F_{stair} \Rightarrow 3766 + 616 + 15856 + 4062 = 24300KN$

$$F_{core;-0} = \frac{24300}{14,115}.3,5 = 6026KN$$

(iii). Small walls:

-1': Own weight: $1,2\left(A_{wall}.t_{wall}.\gamma_{concrete}\right) + stair \Rightarrow 1,2\left(3,1.47,2.0,20.24\right) + 439 = 1282KN$

-1'': Own weight: $1,2\left(A_{wall}.t_{wall}.\gamma_{concrete}\right) + stair \Rightarrow 1,2\left(3,1.47,2.0,20.24\right) = 843KN$:

(iv). Longitudinal wall:

- **Own weight**: $1,2\left(A_{wall}.t_{wall}.\gamma_{concrete}\right) \Rightarrow 1,2\left(8,5.47,2.0,2.24\right) = 2311KN$

(c). Core

Load from horizontal bending moment.

- **Transverse direction**

From the appendix 8, the final lateral load on wall is 20 and 9KN/m respectively on top and bottom. The design moments in wall foot are taken in Y direction.

$$M_{s;d;foot,x} = \frac{\gamma.w_1.h_1^{\ 2}}{2} + \gamma.w_2.h_2\left(H - \frac{h_2}{2}\right) \Rightarrow \frac{1,5.9.20,56^2}{2} + 1,5.20.26,64\left(47,2 - \frac{26,64}{2}\right)$$ and

$$= 2853 + 27077 = 29930KNm$$

The maximum load from bending moment

In Y- axis $F_{M,foot,Y} = \frac{1}{\frac{b}{2}} M_{foot} \Rightarrow \frac{1}{\frac{3,1}{2}} 29930 = 19310KN$, this force must be shared among 4 core's walls**.**

The moments of inertia of core walls -1 and 0 are $I_{-1and0} = \frac{H^3.b}{12} = \frac{3100^3.250}{12} = 62.10^{10} mm^4$

$$I_{-1'and-1"} = \frac{H^3.b}{12} = \frac{3100^3.200}{12} = 49,65.10^{10} mm^4$$

Then $F_{-1and0} = \frac{62.10^{10}}{\left(2.62.10^{10}\right) + \left(2.49,65.10^{10}\right)} 19310 = 5364KN$ and

$$F_{-1'and-1"} = \frac{49,65.10^{10}}{\left(2.62.10^{10}\right) + \left(2.49,65.10^{10}\right)} 19310 = 4291KN$$

- **Longitudinal direction**

$$M_{s;d;foot,y} = \frac{1}{2}\left(1,5.\frac{54,4}{2}.47,2^2\right) = 45448KNm$$

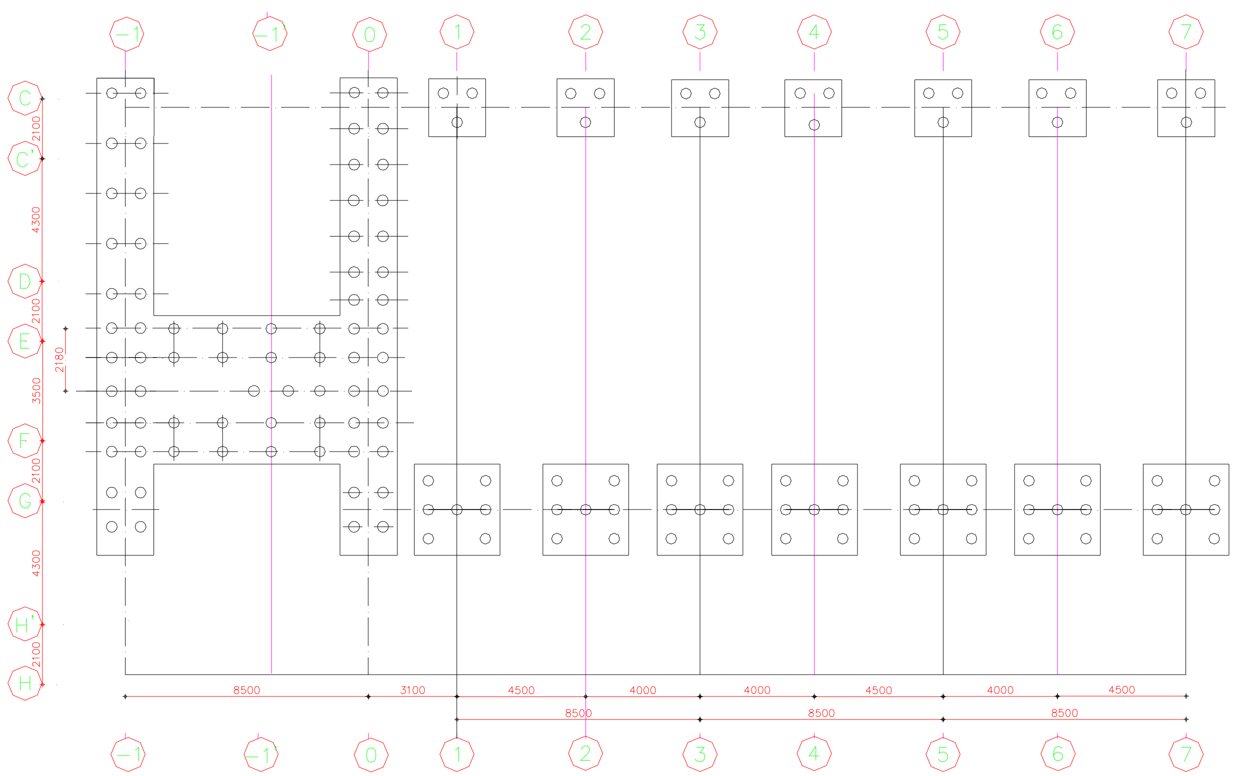
Vertical force brought by this moment:

$$F_{M,foot,X} = \frac{1}{1/2b} M_{foot} \Rightarrow \frac{1}{(1/2).8,5} 45448 = 10694KN$$

Once the loads of the structure and pile capacity are determined, then the number of piles can be estimates. In table A11-1, the loads associated to each structural member are presented as well as the required number of piles. In figure A11-9, one can see the layout of all piles.

TableA11- 1: Required number of piles

Structure		Self weight[KN]	Load from slab[KN]	Load from wind[KN]	Stair case [KN]	Total load[KN]	Pile capacity [KN]	Required number of piles
Wall -1	Wall	3434	11619		439	15492	1090	15
	Cantilever	570	2979			3549		4
	core			5364		5364		5
Wall 0	Wall	3766	15856			19622	1090	18
	Cantilever	616	4062			4678		4
	Core			5364		5364	1090	5
Small wall -1'		843		4291	439	5573	1090	6
Small wall -1''		843		4291		5134	1090	5
Longitudinal wall E		2311		5347		7658	1090	8
Longitudinal wall F		2311		5347		7658	1090	8
Steel frame axis G								7x7
Steel frame axis C								3x7
Total number of piles								**148**



FigureA11- 9: Layout of foundation piles on ground plan

For symmetry reason, axis 0 can have 28 piles instead of 27 pile and the total number of piles goes up to 149.