MASTER

Trussed façade constructions
a study of the opportunities of a structural system

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Trussed Façade Constructions:
A Study of the Opportunities of a Structural System

Appendices of the Master's thesis

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June, 2008

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Preface

These appendices form, together with the main report, my Master's thesis about trussed façade constructions. This part contains the chapters that deal with the design consequences that spring from the structural design considered in the main report. Furthermore, it provides the necessary information – in the form of the appendices – to back up and substantiate the arguments and choices that are made.

Rick Roelofs
June 2008

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1 Design Variant

1.1 Purpose of the design variant

The purpose of conceiving a structural design variant with a trussed façade construction is to explore the possibilities of its implementation in a real design, thereby analysing the advantages and the disadvantages. The structural design variant is based on an actual design that will be discussed hereafter.

1.2 Preferences for the design choice

The preferences for the design choice stem predominantly from the reference projects that have been analysed previously. The analyses made clear that for an advantageous implementation a number of conditions need to be fulfilled or approximated, which are listed below.

Firstly, a trussed façade construction has the inherent quality that horizontal loads, as well as vertical loads, are transferred to the foundations by means of normal forces. Therefore, the sway of the construction will be small as well. This implies that horizontal loads should be dominant in the design and that sway should be normative for the design, implying a high-rise building[1].

Secondly, a trussed façade construction provides the possibility to conceive the floor plan without stability elements, resulting in more possibilities to arrange the lay-out. This feature is best manifested with a wide and deep building that requires a great flexibility with open floor spaces, such as offices.

Thirdly, to engineer a viable trussed façade construction, the plane of the construction needs to be vertically continuous and fluent. This implies that setbacks are not desirable: they subvert the functioning of a trussed façade construction.

Fourthly, the triangular structure of trussed façade constructions should be disrupted minimally or not at all because this weakens the construction substantially. Therefore apertures in the façade should coincide with openings in the construction, and the apertures should not be too large either.

Lastly, it would be more convenient and interesting to select a design which is topical, making it more accessible since all the parties involved are currently working on it.

1.2.1 Design choice

The design that has been selected is the project New Orleans. It was chosen because it largely satisfied the mentioned preferences. The prime condition, that it is a high-rise building, is satisfied since the design has a height of 158 metres. Additionally, the tower measures 30 by 30 metres, thereby being deep and wide enough to require flexibility. Moreover, the design is topical since the construction started in September 2007.

However, the (architectural) design of New Orleans also entails disadvantages. The irregular setbacks of the five upper storeys are difficult to engineer with a trussed façade construction. Besides, the building has numerous loggias which have to be accessible, meaning that apertures are unavoidable. Similarly, an underground car park is located in the basement of the building, requiring apertures as well. Additionally, it is a residential building where sound insulation is an important issue which is often resolved with stony materials, subverting the concept the trussed façade constructions.

All together, the advantages outweigh the disadvantages of the design and several disadvantages can be circumvented. Another important, more pragmatic reason, was that chances were remote that a more

[1] Derived from the definition of a high-rise building in a structural context (Hoenderkamp, 2005)

suitable building in the Netherlands could be found. For these reasons New Orleans has been selected to design a trussed façade construction for.

Figure 1.1: Overview of the master plan at the 'Kop van Zuid' in Rotterdam. DHV, 2007

Figure 1.2: Artist's impression of New Orleans. HMADP Architects, 2007

1.2.2 Design description

New Orleans is conceived by the Portuguese architecture office Alvaro Siza. The executive architect is HMADP architects and DHV is responsible for the structural design of the project. The project is located at the Kop van Zuid in Rotterdam and is part of a master plan including multiple high-rise projects, as Figure 1.1 illustrates.
New Orleans consists of a high-rise building, the residential tower, and a low-rise building called the 'Arthouse'. In this context, the 'Arthouse' is not of particular interest because of its minor height and is not taken into account; in the actual design, the low-rise and the high-rise part are structurally separated as well. Henceforth, when referring to New Orleans, only the high-rise building is meant.

1.2.3 Functional design

New Orleans is a residential building with a height of 158 metres and measures approximately 30 by 30 metres. It comprises a total of 46 storeys: the main floor and the first storey will house office space combined with a lobby and bicycle garaging, the third storey will be an installations floor, and from the fourth storey upwards apartments will be located. Additionally, the two underground storeys will serve as a car park. The current design has a core which comprises three lifts, two emergency stairways that form a double helix, and service ducts.

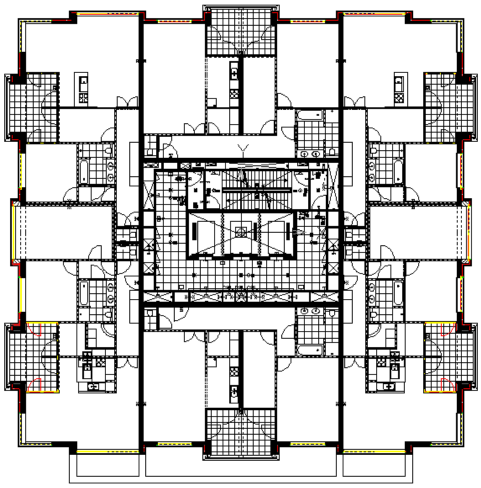


Figure 1.3: Standard architectural floor plan **Structural design.** *HMADP Architects, 2007*

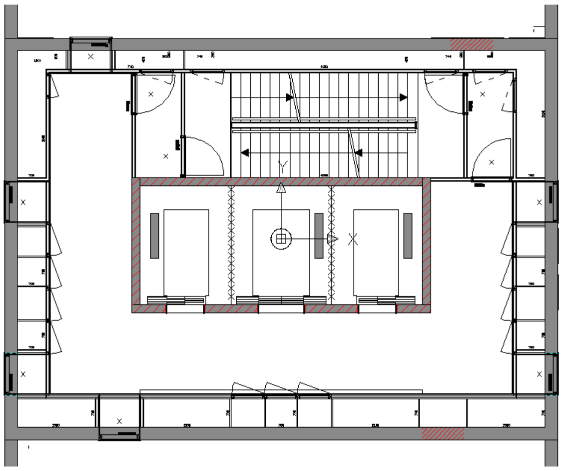


Figure 1.4: Functional core. HMADP Architects, 2007

1.2.4 Structural design

The construction of New Orleans is almost completely made of in-situ concrete with 300 mm thick walls. The stability of the tower is guaranteed by a concrete core with extended 'flanges' that are monolithically connected. For bending around the vertical axis (in Figure 1.5), the walls parallel with the flanges, together with the floors, function as rigid frames to contribute to the stiffness: the core alone does not suffice in this direction. For bending around the horizontal axis, the core and the flanges alone guarantee the stability.

At the base of the building, the core is reduced to a rectangle and the wall thickness is increased to 600 mm. The flanges and perimeter walls are replaced by 14 columns – that take over the load bearing function – to allow for an open floor plan at the two lowest storeys, which is shown in Figure 1.6.

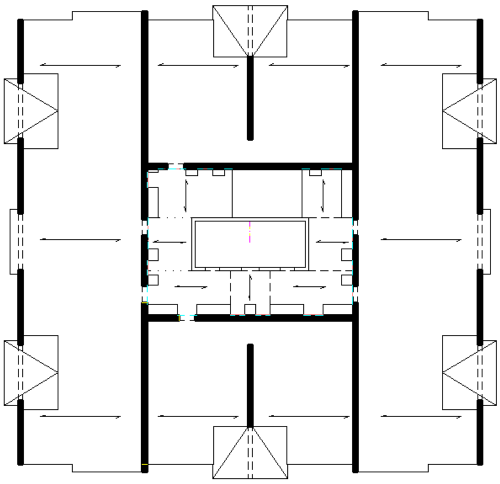


Figure 1.5: Structural floor plan of the higher storeys. DHV, 2007

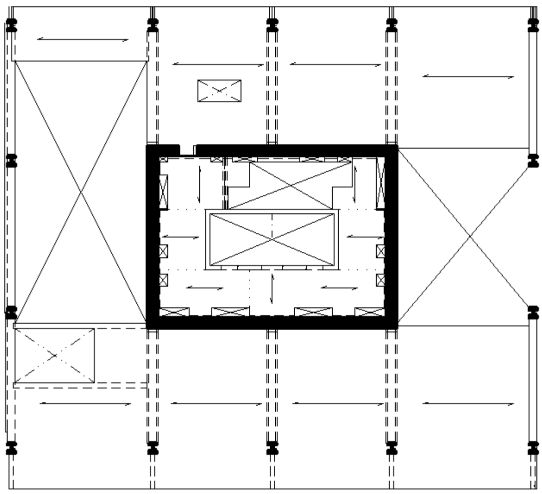


Figure 1.6: Structural floor plan of the 1st storey. DHV, 2007

1.2.5 Architectural design

The current architectural design is dominated by external features and the lay-out of the floor plans. Besides, the design is charactarised by an austere and almost sculptural look both internally and externally.
Spatially, the tower, together with the horizontal volume which is called the 'Arthouse', constitute one unity. To achieve this unity, both volumes have a number of external features in common which are carried on from one to another to smooth the transition. Concerning the tower, the main feature is the protruding volume in the lower part of the south façade, which can be seen in the façade in Figure 1.2.
In addition, the tower is characterised by 3 protruding bays per façade, forming vertical, discontinuous strips on the façade. Several of these bays are actually loggias; Others are simply parts of the building that stick out from the rest, to complete the architectural composition.
Furthermore, from the 40th storey until the 45th and top storey, the horizontal section is gradually reduced until the tower reaches its roof, illustrated by Figure 1.7 and Figure 1.8. This reduction takes place with setbacks on several places per storey, but always symmetrically, giving the tower a less abrupt ending.
Concerning the lay-out, the architecture is marked by the arrangement of the apartments: all of them have a central entrance hall, with a generous width of 1.5 m, which opens onto the sitting room with an open kitchen at one side, and onto the bedrooms at the other side. As a result, the bedrooms and the sitting room remain functionally separated, which can be seen as an architectural merit.

Figure 1.7: Characteristic 'crown' of the tower. HMADP architects, 2007

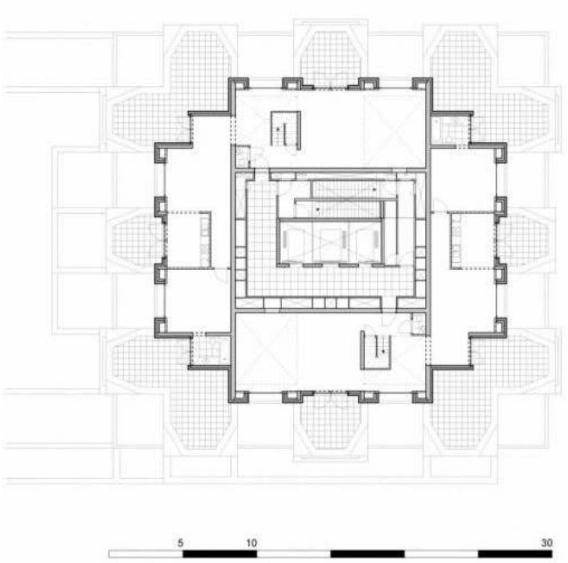


Figure 1.8: Floor plan of the penultimate storey. HMADP architects, 2007

1.2.6 Interpretation of equivalence

To make a fair comparison between the design variant and the original design, both have to be equivalent. Therefore, certain features of the design have to stay the same, otherwise the comparison is distorted.
The structural design variant needs to meet the same structural requirements as the original construction according to the Dutch design codes or stricter in some cases. Regarding the non-structural functions of the original construction (e.g. sound insulation which is inextricably connected to certain structural members); these functions need to be taken over by other building elements to guarantee an equivalent outcome.
The architectural design, however, might be compromised on specific points. This is justified because this particular architectural design is of subordinate importance in relation to the entire study. In other words,

designing an architecturally accurate variant for New Orleans is not the main objective of the thesis: the objective is to study the possibilities of trussed façade constructions. Practically, this means that structural issues take priority over architectural issues where they conflict and when no other convincing solution is possible.

In contrast, the functional design is completely adopted. That is to say, that the number of lifts, the stairwells, the service ducts and facilities alike are copied as well as their size and location.

2 Structural Nodes

2.1 Functions of Structural Nodes

The structural nodes discussed hereafter concern the nodes of the trussed façade construction that simultaneously constitute the connection with the interior construction. Due to the separation these nodes play a vital role and their careful design enables a viable total design. For a better understanding their functions are considered below.

2.1.1 Stability Connections

To guarantee the stability of the building, the interior construction needs to be connected to the trussed tube structure which functions like a stability cage. The connection between the two constructions is made every three storeys: this way the connections coincide with the nodes of the tube structure – the tube structure members span three storeys, shown in Figure 2.2 –, ensuring that the introduced forces only render normal forces in the trussed tube structure.

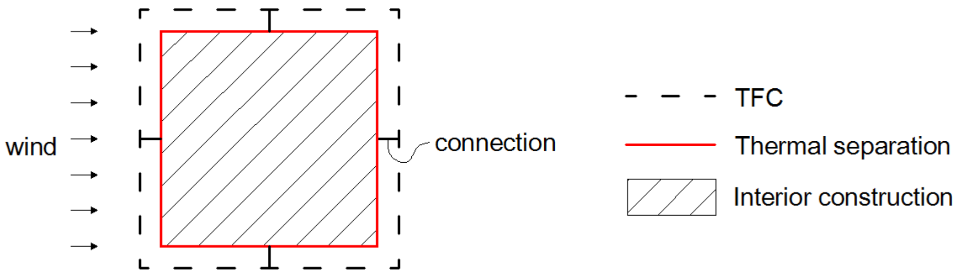


Figure 2.1: Connections for stability between TFC and interior construction.

The connections, illustrated in Figure 2.1, are made in such a way that wind loads perpendicular to the façade are transferred to the most rigid parts of the tube structure in this direction, namely the "webs". Subsequently, the "webs" spread the forces further to the "flanges" utilizing the entire tube structure. For wind loads on the perpendicular façade this same principle works identically since the connections are present at all four sides.

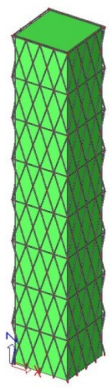


Figure 2.2: Tube structure with nodes every three storeys.

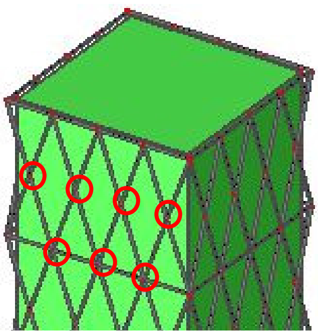


Figure 2.3: Connections as buckling supports.

2.1.2 Buckling Supports

In addition to stabilizing the interior construction, the connections play another vital role: they prevent the trussed tube structure from buckling by supporting it every three storeys. As a consequence, the governing buckling length of the tube structure is reduced to the length of one diagonal member. This also means that more connections need to be made than strictly necessary for the stability: every odd storey four connections per façade are made and every even storey three connections. The nodes at the corner of the tube structure are stable by themselves.

2.1.3 TFC Member Connections

The connection between the trussed tube structure members is another aspect in the node design. Since the loads only seize at the nodes of the trussed tube structure, merely normal forces are present in the members, as illustrated in Figure 2.4. Therefore, the nodes have to transfer both tensile and compressive forces without instability problems for the latter load.

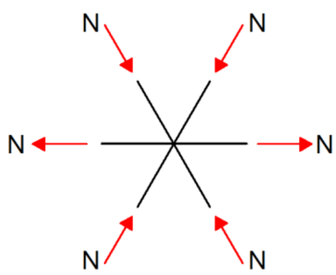


Figure 2.4: Only normal forces in the TFC nodes.

2.1.4 Thermal Separation

Since the trussed tube structure is located outside, certain temperature differences will occur that are not present in the interior construction. In other words: The tube structure wants to expand and shrink according to the temperature while the interior construction – having a stable temperature – does not undergo these changes. The connections between the interior construction and the tube structure are also located at the exterior, exposing them to the weather conditions.

2.2 Thermal Behaviour

2.2.1 Temperature Loads

The trussed tube structure is exposed to temperature loads, meaning that the temperature range will either lead to expansion and shrinking or stress accumulation. To calculate the magnitude the prescribed total temperature range according to the Dutch design codes, shown below, would amount to 75˚C which is rather substantial. However, in reality it is not very likely that one entire member will reach these extreme temperatures: the extremes will only occur on the surfaces that are directly exposed to the sun. The shady side of the member will not reach the extreme temperature also because the sections have quite a large specific heat capacity.

	Standard [˚C]	extreme [˚C]
Summer – outdoors	17	50[1)]
Winter – outdoors	4	-25

[1)] very light color (e.g. weight, lightgrey, yellow, cream), NEN 6702

2.2.2 Expansion versus Prevention

In theory, two extreme approaches are conceivable regarding the temperature loads: either the expansion can take place unhindered meaning that the expansion and shrinking will take place according to the temperature loads. Or the expansion can be completely prevented, leading to a stress accumulation in the structural members. The first approach would lead to a maximal difference (extreme winter, extreme summer) in height at the top of the building of

$$\Delta l = \Delta T \cdot \alpha \cdot l = 75 \cdot 12 \cdot 10^{-6} \cdot 158 = 0.142\ m$$

Or the latter approach could lead to a stress differential in the members of

$$\sigma = \epsilon \cdot E = \frac{0.142}{158} \cdot 2.1 \cdot 10^{5} = 189\ N/mm^{2}$$

In practice, these extremes are not likely to happen, yet a more balanced effect will occur. Yet, it is clear that a total vertical prevention of the expansion is not possible since a stress differential in the above-mentioned order is impossible to hinder.

2.2.3 Trussed Tube Structure Behaviour

To study the behaviour of the trussed tube structure under temperature loads more closely, models have been built with the computer programme ESA PT to examine the occurring stresses. Arguing that the standard (reference) temperature is approximately 15˚C and that the range is approximately 50˚C, a temperature load of ΔT=25˚C has been applied. The full study can be found in Appendix 4.

Based on the outcome of the study the following design choices are made: vertically, the trussed tube structure is not connected with the interior construction, meaning that the tube structure is freely allowed to shorten and lengthen over the height according to the temperature, illustrated in Figure 2.5. Horizontally, the tube structure is connected to guarantee the stability; however, only in the middle of the façade to allow for a horizontal expansion. Yet the horizontal expansion is relatively minor in proportion to the vertical expansion, so that it can be resolved with tolerances.

Regarding the connections for stability in the middle of the façade; at the odd storeys, no centre nodes are present. Therefore, the stability connections at the odd storey are located at either side of the middle of the façade, as Figure 2.6 illustrates: this only leads to a marginal stress accumulation because no direct horizontal member is present between the nodes.

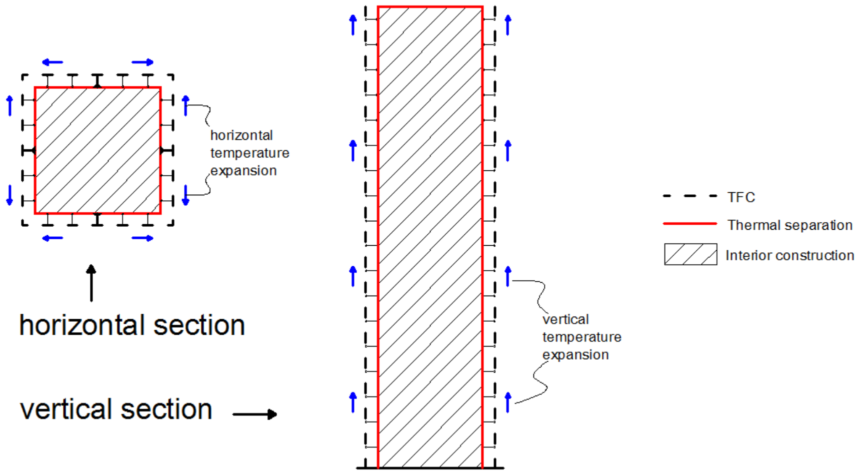


Figure 2.5: Horizontal & vertical allowed expansion of the tube structure.

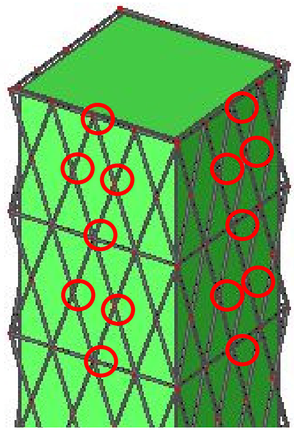


Figure 2.6: Stability connections – horizontal displacement restricted – in the middle per façade.

2.3 Principle of Structural Nodes

2.3.1 Degrees of Freedom

Based on the previous considerations, the degrees of freedom of the structural node are established, as shown in Figure 2.7 and Figure 2.8. In the plane of the façade (XZ-plane) the horizontal force for the stability has to be transferred which amounts to approximately 690 kN. Perpendicular to the façade, a resultant force is present due to buckling prevention. Lastly, a vertical displacement should be allowed for with a maximum range of 140 mm at the top of the building.

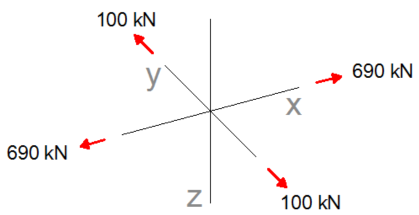


Figure 2.7: Forces to be transferred by main structural node.

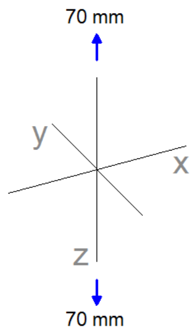


Figure 2.8: Degrees of freedom of main structural node.

However, the plane of the façade (construction) and the plane of the tube structure do not coincide. They are separated by a distance (d). Ideally, the transfer of forces is represented as a moment-resisting connection at the interior construction and a hinge at the tube structure, illustrated in Figure 2.9: this way the moment due to the force couple (with distance d) is entirely resisted by the interior construction, and no bending moments are introduced in the members of the tube structure. Regarding the unhindered vertical displacement of the tube structure, the connection also needs to be moment-resisting in the vertical direction, as Figure 2.10 shows.

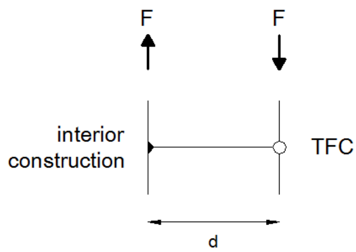


Figure 2.9: Horizontal elevation: transfer of forces.

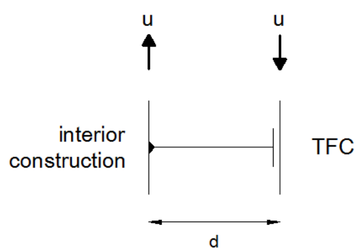


Figure 2.10: Vertical elevation: free displacement.

2.3.2 Maintenance

Besides the structural requirements, practical aspects also play a key role, especially because the connections are exposed to weather conditions. Practically this means that the connections need to be designed for low-maintenance. For this reason, the connection consists of closed sections, giving corrosion and pollution as little chance as possible. Additionally, the design detail needs to be hard-wearing in order to guarantee the same lifespan as for the building, thereby preventing costly and difficult replacements.
To guarantee the functioning of the vertically sliding connection, nylon is used as an intermediate: nylon has a low sliding resistance on steel – even after being exposed to water –, it is self-lubricant, and hard-wearing.

2.3.3 Structural Node Design

The principle design of the structural node is shown in Figure 2.11. No further calculations have been performed.

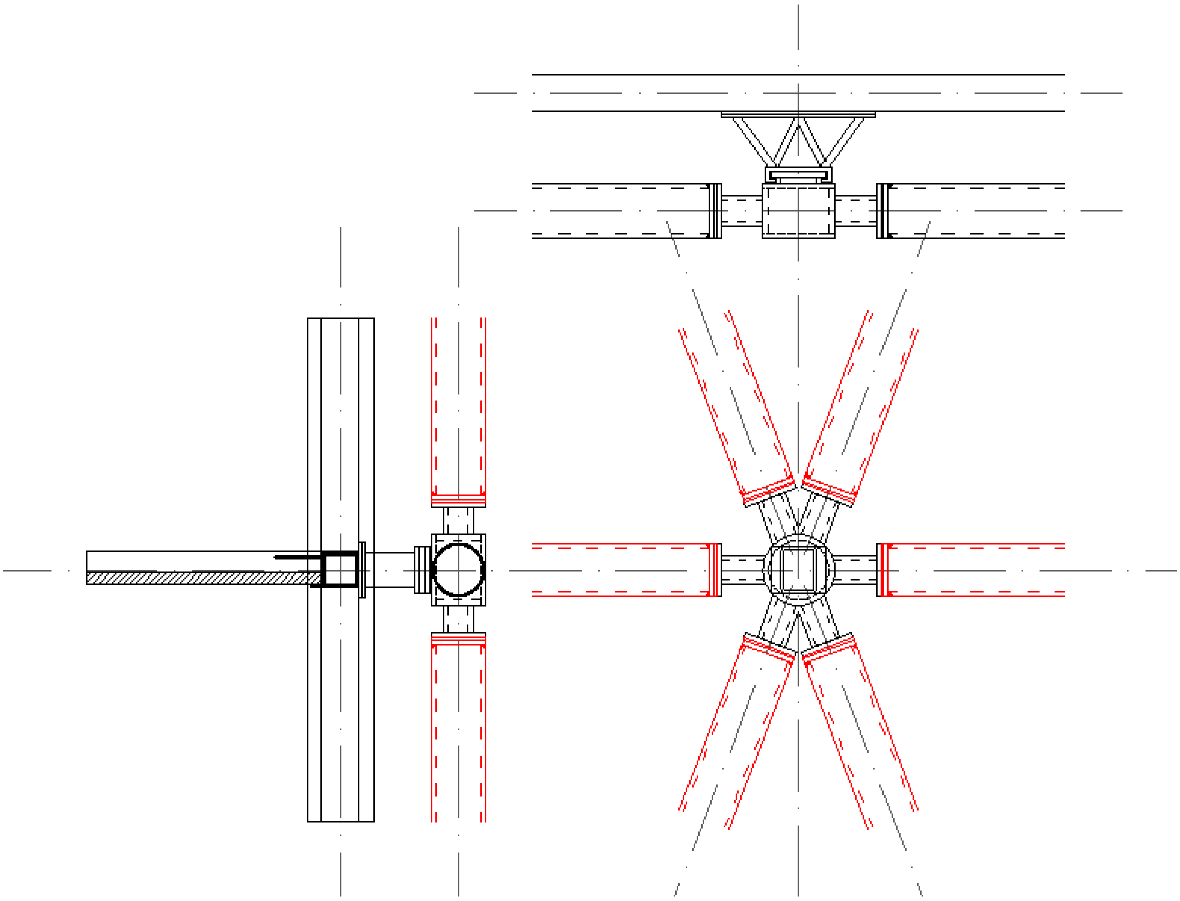


Figure 2.11: Elevations of the structural node.

3 Substructure & Foundation

3.1 Substructure

3.1.1 Function

The function of the substructure is to balance the forces from the trussed tube structure (upward and downward) and the façade columns (downward), making sure no – or minor – tensile forces occur in the foundation piles. However, to do so the structure needs to overcome the eccentricities between the tube structure and the façade columns, depicted in Figure 3.1; the centre-to-centre distance from the tube structure to the façade is exactly 1 m. Furthermore, the base of the tube structure and the façade columns are not aligned on the same axes for all locations: these eccentricities amount to maximally 4.75 m.
Because of these substantial eccentricities, a wall structure is conceived on the grid of the tube structure with perpendicular compact walls located at the position of the façade columns, as Figure 3.2 illustrates. The wall is purposely located directly under the tube structure – i.e. not under the façade columns – because the range of the forces (from tension to compression) in the tube structure is substantial and volatile due to the wind: this way, the wall is able to spread the occurring stresses. The façade columns, on the other hand, always exert a compressive force in the same order of magnitude.
To introduce the large forces from the façade columns and the tube structure, steel sections are cast into the concrete substructure: the height of the two basement storeys (nearly 6 m.) is sufficient to introduce the concentrated load in the pile foundation, thereby preventing stress peaks.

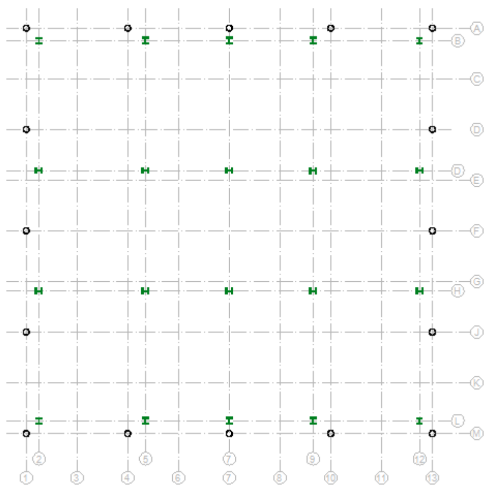


Figure 3.1: Position of tube structure members & façade columns.

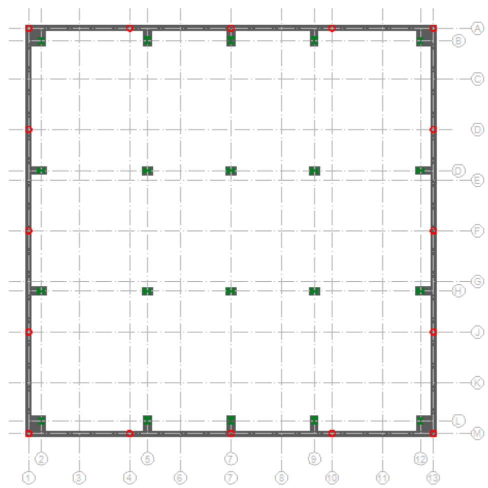


Figure 3.2: Design of the substructure in plan: a continuous wall under the trussed tube structure.

3.1.2 Underground Car Park

The current plan comprises a two-storey basement with a car park, shown in Figure 3.3, that extends underneath the entire complex: evidently, this car park needs to be implemented in the design variant as well. The prime adaptation is that openings need to be made in the walls of the substructure to allow for cars to pass through. Yet, the location of the openings is subject for discussion: either the openings can be made between axis D and E – and at the same locations at the other side of the plan – meaning that the driving lanes, and thereby the car park layout, need to be altered. Or the openings can be made at the same location as for the current design, with the consequence that the forces from the tube structure seize upon the lintel

above the openings. However, the storey height of approximately 3 m. and the height of the opening (minimal 2.2 m.) leads to a fairly minor height of the lintel. Another solution would be to alter the geometry of the trussed tube structure at the base, so the diagonals do not come out above the original location of the driving lanes.
All together, the second option – openings at the same location as the current design – has drastic consequences for the structural system. Besides, the car park layout is considered to be of subordinate importance. Therefore, the choice is made to make openings in the walls of the substructure between axis D and E and axis J and L, shown in Figure 3.4. As a consequence, some parking spaces are lost due to a less efficient layout which is accepted in this case.

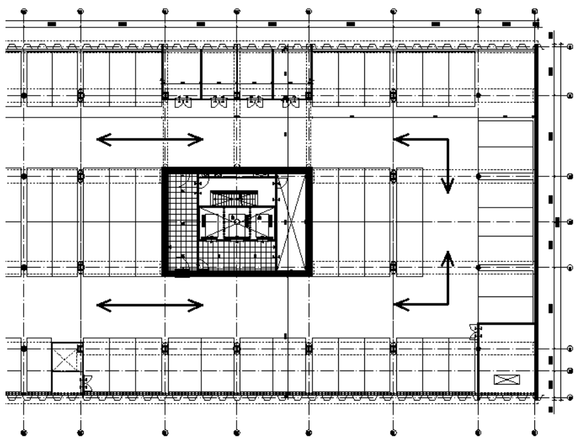


Figure 3.3: Car park underneath the current design: driving lane indicated with arrows.

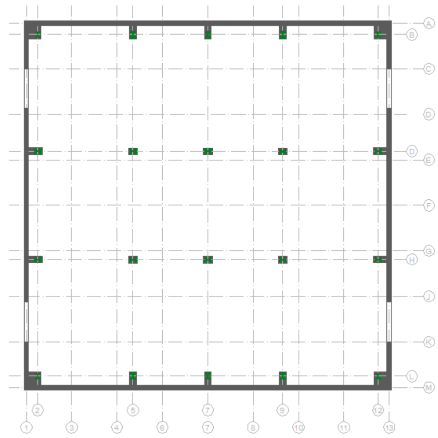


Figure 3.4: Location of the openings in substructure in the design variant.

4 Dimensioning of the Structural Design

4.1 Global Dimensioning

Now the geometry of the construction is known, a global dimensioning is performed to obtain preliminary dimensions of the sections and to serve as a reference for the dimensioning with the computer programme ESA PT.

4.1.1 Structural Diagram

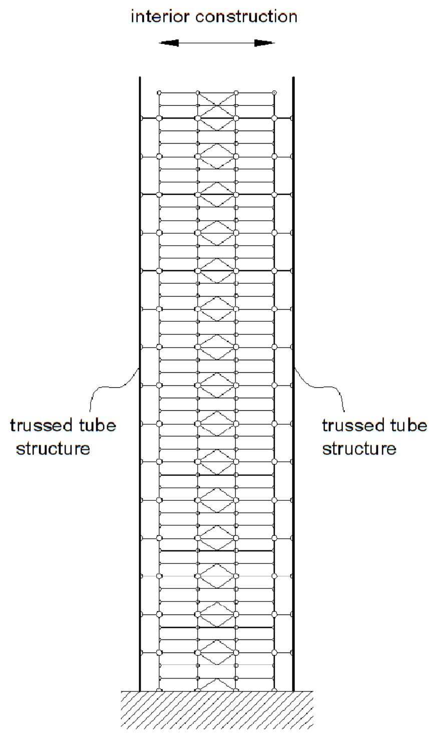


Figure 4.1: Structural diagram (cross section) of the design variant.

The structural diagram of the design variant consists of the interior construction that is stabilised by the trussed façade construction at all four sides, every 3 storeys. The interior construction is made up of vertical columns and horizontal beams that all have pinned connections. Therefore, a secondary windbracing – located in the core – is necessary to distribute the wind load from two intermediate floors to every third floor that is linked with the tube structure.

4.1.2 Section Choices

For the trussed tube structure circular hollow sections (CHS) are adopted for three main reasons. Firstly, circular hollow sections do not have a minor axis, making them ideal for normal forces since they are not supported between the main structural nodes. Secondly, hollow sections require less maintenance: no (sharp) edges or angles are present; they have a smaller exterior surface cross-section ratio. Thirdly, circular sections look more slender than rectangular sections with the same cross-sectional area.

The adopted sections for the columns of the interior construction are all H-sections, ranging from HD to HEM sections. The choice for these heavy sections (with a large section area) is made to limit the dimensions of the columns in the floor plan.

4.1.3 Preliminary Sections

An overview of the chosen preliminary sections is given below with an indicative floor plan in Figure 4.2. The calculations on which the choice of sections is based can be found in appendix 6.

Façade columns		
Side	HD 400x677	(base column)
Corner	HD 400x421	(base column)
Core columns	HD 400x900	(base column)
TFC		
diagonals	CHS 457x40	(entire height)
horizontals	CHS 457x10	(entire height)
Core windbracings	IPE 160	(entire height)
Integrated floor beams	THQ 320x80-290x35-500x20	(entire height)
Rim joists	IPE 400	(entire height)

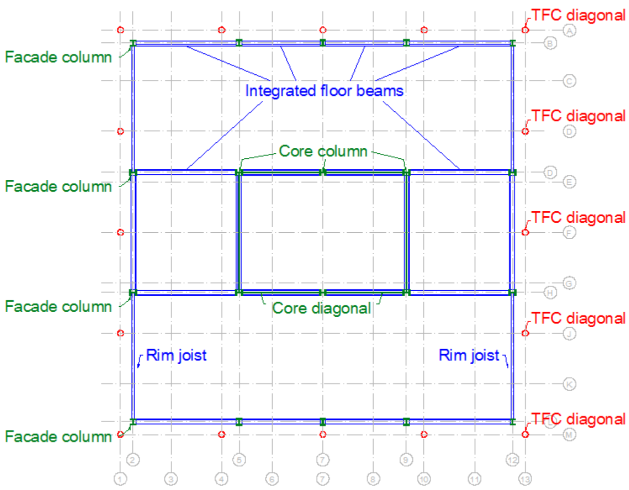


Figure 4.2: Indication of the members in the floor plan.

4.2 Dimensioning

4.2.1 Loads

An overview of all considered loads for the dimensioning is given below.

Own weight;	dead load of the steel construction (calculated by ESA PT)
Façade;	dead load of the facade
Floors dead loads;	dead load of the floors

Floors live loads;	live load of the floors
Wind NW;	wind from the north west
Wind NE;	wind from the north east
Wind N;	wind from the north

Load combinations

The load combinations for the ultimate limit state and the serviceability limit state are summarized in Table 4.1 and Table 4.2, respectively. All load cases that are extreme and unfavourable are indicated.

= unfavourable dead load

= extreme live load case

Regarding the ultimate limit state (ULS)

	Dead loads			Live loads			
Load combination	own weight	Façade	Floors DL	Floors LL	Wind NW	Wind NE	Wind N
Fundamental	1.35	1.35	1.35				
Floors extreme	1.2	1.2	1.2	1.5			
Wind NW	0.9	0.9	0.9	1.5·Ψ	1.5		
Wind NW	1.2	1.2	1.2	1.5·Ψ	1.5		
Wind NE	0.9	0.9	0.9	1.5·Ψ		1.5	
Wind NE	1.2	1.2	1.2	1.5·Ψ		1.5	
Wind N	0.9	0.9	0.9	1.5·Ψ			1.5
Wind N	1.2	1.2	1.2	1.5·Ψ			1.5
Alternate load path	1.0	1.0	1.0	1.0·Ψ	0.2		
Alternate load path	1.0	1.0	1.0	1.0·Ψ		0.2	
Alternate load path	1.0	1.0	1.0	1.0·Ψ			0.2

Table 4.1: Load factors ultimate limit state.

	Dead loads			Live loads			
Load combination	own weight	Façade	Floors DL	Floors LL	Wind NW	Wind NE	Wind N
Floors extreme	1.0	1.0	1.0	1.0			
Wind NW	1.0	1.0	1.0	1.0·Ψ	1.0		
Wind NE	1.0	1.0	1.0	1.0·Ψ		1.0	
Wind N	1.0	1.0	1.0	1.0·Ψ			1.0

Table 4.2: Load factors serviceability limit state.

4.2.2 Assumptions

The following assumptions have been made as regards the structural model in ESA PT;

- The buckling length of all interior columns equals the storey height: the buckling length has been set manually to guarantee the accuracy.
- The buckling length over the major and minor axis is the same: buckling supports of non-structural building elements (e.g. façades, fire walls) are neglected.
- The connection of the integrated floor beams, rim joists and the concrete floor slabs to the columns are hinged: they do not introduce any bending moments in the columns.
- The section of the integrated floor beams and the rim joists are the same at every storey. They have been calculated with an excel-sheet and the sections have been adopted in the structural model (no further verification is done with ESA PT).

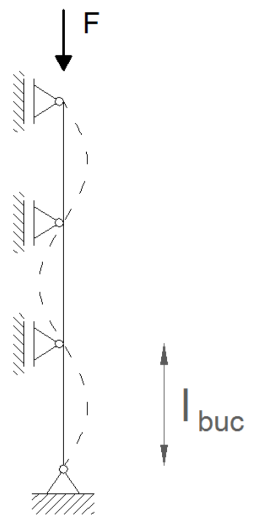


Figure 4.3: the buckling length equals the storey height

- The precast floor slabs have been modeled by a concrete slab that spreads the loads in two directions instead of one, which alters the load distribution: yet, the difference in distribution has been accepted since it is marginally for this specific floor plan.
- All floor slabs are of concrete grade C53/65, with E = 38,500 N/mm^2.
- The own weight function of the structural model is used to calculate the weight of the steel structural members; the dead weight of the floors slabs was already calculated by hand, so the density of the concrete floor slabs is set at 0 kN/m^3 so ESA PT does not calculate the additional load of the concrete floor slabs.
- All construction materials have a linear elastic behaviour.
- The substructure is of concrete grade C28/35, with a reduced E = 10,000 N/mm^2, to take cracking into account.
- The substructure is anchored by rigid supports: the foundation rotation is calculated by hand.
- The façade elements are supported by the floor below.

4.2.3 Verifications

Ultimate Limit State

The checks that have been performed by ESA PT to dimension the construction are taken from the NEN 6770 (for steel) and NEN 6720 (for concrete). The structural members have been checked automatically by ESA PT. Regarding the verifications the fire safety has not been checked in particular: This will be considered separately later. The structural members that have been checked concern the façade columns (side & corner), the core columns, the trussed tube structure (diagonals & horizontals), and the diagonal windbracings in the core.

Serviceability Limit State

The verifications concerning the serviceability limit state have been performed in conformity with NEN 6702. The main verifications that have been performed concern the total horizontal deflection of the building at the top and the interstorey deflection. The vertical deflection of the precast wing floors is assumed to meet the requirements.

4.2.4 Definite Dimensions of Sections

The steel sections of the columns have been altered over the height to account for the decreasing forces higher up the building. For practical purposes, the sections have only been altered four times; obviously, more material could be saved if the section of the columns were to be reconsidered six or eight times. A table with an overview per member is given below.

Façade columns	basement	storey 1-12	storey 13-24	storey 25-36	storey 37-46
Side	HD 400x1086[1)2)]	HD 400x1086[1)]	HD 400x634	HD 400x382	HEM 300
Corner	HD 400x592[2)]	HD 400x592	HD 400x382	HEM 300	HEM 240
Core columns	HD 400x1086[1)2)]	HD 400x1086[1)]	HD 400x1086	HD 400x634	HD 400x382
TFC					
diagonals	CHS 457x40	(entire height)			
horizontals	CHS 457x10	(entire height)			
Core windbracings	IPE 160	(entire height)			
Integrated floor beams	THQ 320x80-290x35-500x20	(entire height)			
Rim joists	IPE 400	(entire height)			

Table 4.3: Definite dimensions of sections per structural member.

1) S460

2) embedded in concrete

5 Fire safety

5.1 Requirements

The minimum fire resistance for the main load bearing construction amounts to 120 minutes since it concerns a residential building taller than 70 meters (SBR, 2005). Reaching a fire resistance of 120 minutes with a steel construction requires additional measures. However these measures tend to be labour-intensive and one of the major cost aspects for steel construction. Therefore it is necessary to find a sound approach to engineer an economically feasible construction.

5.2 Fire Safety Philosophy

The adopted fire safety philosophy is twofold: the interior construction – which comprises floor beams, core columns and façade columns – is integrated into the walls and floors, which functions as a primary protection for the construction; the trussed tube structure is located outside, at a certain distance from the façade, where the temperature will not mount as high as inside.

5.3 Interior construction

To meet the fire resistance for the interior construction, the structural members are integrated into the walls and floors. However, many sections are too large for complete integration (façade columns) or the flanges simply need to protrude at one side (THQ beams). Therefore additional fire-resistant coverings needs to be applied to reach the requirements of 120 minutes. Promatect boards are used for the covering since they give an austere appearance.

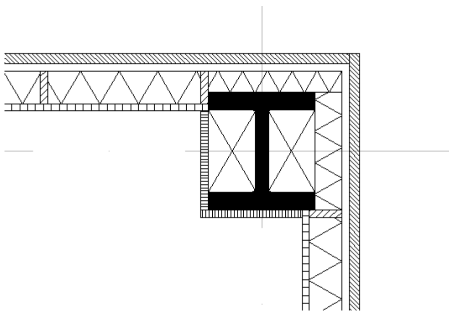


Figure 5.1: Façade columns; partially integrated, partially covered with fire-resistant board.

5.4 Trussed tube structure

The trussed tube structure is located at the exterior of the façade where the temperature will not reach the same values as in the interior: this means that the reduction of the strength for the tube structure is not as significant as usual. Besides, an overcapacity in the sections is present – the dimensioning is based on the stiffness – and the trussed tube structure has a statically indeterminate triangular structure, meaning that ample robustness is present. Given these facts, a qualitative analysis is made of the fire safety of the tube structure.

5.5 Fire Compartmentation

To have a closer look at the fire resistance of the tube structure, the fire compartmentation of the building needs to be studied more closely: this way it can be determined which parts of the tube structure are likely to be exposed to fire.

All floors in the building are part of the main load bearing construction (for diaphragm action) and therefore have a fire resistance of 120 minutes: accordingly, this means that every storey is considered as a separate fire compartment. Furthermore the storeys are divided into separate fire cells with a minimum fire transfer time of 60 minutes, as Figure 5.2 illustrates for an apartment floor. This means that the fire will take considerable time to spread over the storey. Hence the tube structure will also be affected less as more fire cells are present. An inventory of the fire cells is made in Appendix 7. The conclusion is that the first storey – with one fire cell – is normative and the entire tube structure can be exposed to fire at the same time.

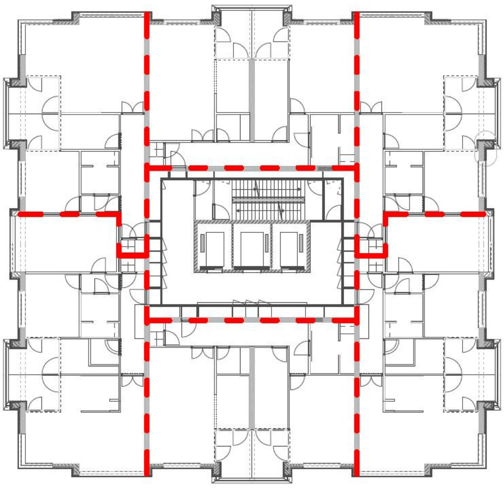


Figure 5.2: Fire compartmentation apartment floor current design: 6 fire cells excluding the core.

5.6 Strength Reduction

The strength reduction of steel is correlated with the temperature: when it rises above 400˚C the strength declines abruptly, as Figure 5.3 illustrates. Therefore, it is important to consider the maximum steel temperature of the tube structure and to know the overcapacity of the sections: in case of fire only 20% wind of the maximum wind load needs to be taken into account (NEN 6702).

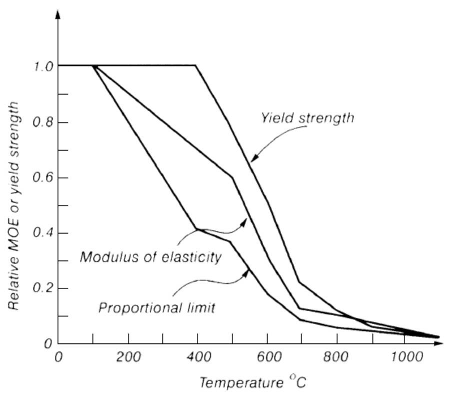


Figure 5.3: Decrease of yield strength and elasticity at high temperatures.

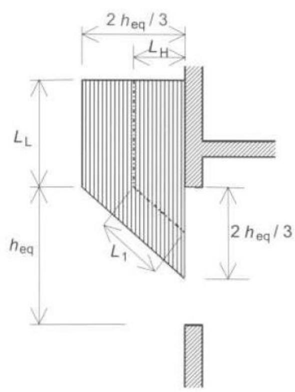


Figure 5.4: The temperatures drops further along the flame path: from approximately 1050 to 550˚C.

The overcapacity in the tube structure members is such that in case of fire only 25% (on average) of the capacity is used. This corresponds with a critical temperature of approximately 700˚C. The temperature in the interior will be around 1050˚C – for a fully developed fire. Yet, the temperature will decrease outside the façade openings, exposing the construction to a lower temperature: the flame tips have a temperature of approximately 550˚C (NEN-EN 1991-1-2, 2007). Furthermore, the construction will also give off a lot of heat through convection and radiation because of the relatively low outside temperature and increased air speeds that arise. Besides, the construction is not likely to be exposed to fire from all sides (Jakobs, 1999). Therefore, it is plausible that the tube structure can meet the fire safety requirements without additional measures.

6 Comparison

6.1 Equivalence of Comparison

Now that the sections and dimensions of the design variant have been established, the comparison with the current structural design can be made. However, before the comparison is made careful consideration needs to be given to the equivalence of both constructions: The concrete construction fulfills certain roles (e.g. fire protection & façade elements) that are taken over by other materials in the steel construction variant. Therefore, the weight and costs of certain building materials have been taken into account: This involves the infilling of the core for fire retardant and sound insulation measures. Yet, the infilling of the façade is not included; for the current variant prefabricated wall elements are used that are not part of the construction, whereas the façade of the design variant consists of steel columns – both solutions are considered to be equivalent.

It is also important to note that only the superstructure is considered; the basement and the foundation differ for both design. Yet, they are not included since their design is not elaborated and therefore little reliable data is present.

6.2 Construction weight

The construction weight is calculated in Appendix 8; the summary is given below.

	current design	design variant	difference
Floors [tons] (based on equal floor area of 30,875 m^2)	23,156	18,427	-20%
Walls [tons] (internal + façade)	10,197		-52%
TFC [tons]		2,395	
Interior construction [tons]		2,452	
Total [tons]	33,353	23,274	-30%

Table 6.1: Comparison construction weight.

From the overview of the construction weight, the conclusion can be drawn that the design variant is significantly lighter. Partially, the weight reduction is due to the lighter concrete floors in the design variant. Besides, the profit is earned by the vertical construction elements – i.e. the walls for the current design and the interior construction & TFC for the design variant.

However, the outcome is not directly surprising since steel constructions are known for lightweight buildings in comparison with concrete. Therefore, the outcome needs to be seen in perspective to make a fair judgement.

6.3 Construction costs

The calculation of the construction costs for the current design and the design variant are based on the amount of construction material for the superstructure alone: the substructure and the foundation is not taken into account. The used prices include the direct costs (construction material, labour, contractors, machinery & equipment) and the indirect costs (general costs, construction site costs, insurances, profit, risk), specifically for a multi-storey building. More specific data can be found in Appendix 8, the outcome of the constructions cost calculations is given below.

	Construction costs
Current design	15.6 million
Design variant	18,2 million

Table 6.2: Comparison of construction costs.

It follows from the construction costs that the design variant is more expensive than the current design. However, the comparison tells more about the cost difference for an in-situ construction and a steel frame with concrete floor construction; the specific impact of the contribution from the trussed tube structure cannot be obtained from this cost comparison.

6.4 Flexibility

When the flexibility of both designs is compared, the most significant gain regards the flanges of the core and the intermediate walls that are omitted, as Figure 6.1 and Figure 6.2 illustrate. This leads to enhanced flexibility for the layout of the floor plans, both in the design phase and during the lifespan of the building. Regarding the core design, more flexibility is acquired in two ways: first the dimensions of the core are no longer linked with the moment of inertia – and thereby the stiffness – of the construction. Second, additional openings can be made in the core walls – as long as they do not interfere with the diagonals – throughout the lifespan of the building.

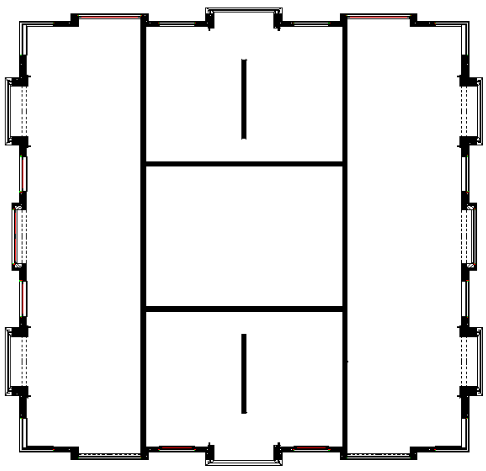


Figure 6.1: Structural floor plan current design.

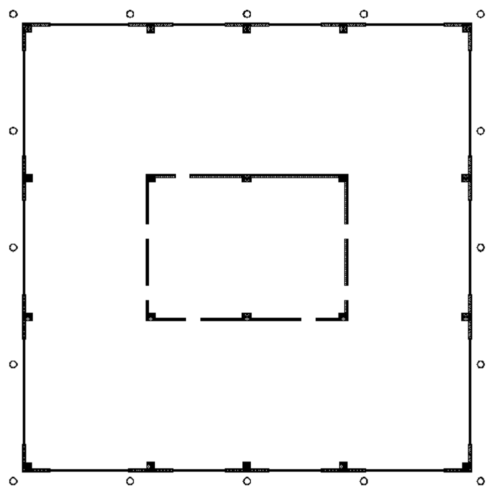


Figure 6.2: Structural floor plan design variant.

6.5 Summary

The design variant seems to perform better regarding the construction weight and the flexibility. However, the construction costs give the impression to be significantly higher, making the design variant a lot less attractive. In this respect it is important to note that the difference in construction weight and the construction costs can be primarily attributed to the difference between concrete constructions and steel frame construction: the comparison is too global to pronounce upon the costs of trussed tube structure in particular.

7 Appendix 1: Design Variant

7.1 Starting Points Current Design

The content of the document “Uitgangspunten ten Behoeve van Berekeningen V2090” is adopted below. The irrelevant parts are omitted from the original text.

INHOUD BLAD

Inleiding

Dit rapport geeft de uitgangspunten voor de gewichts- en stabiliteitsberekening en de constructieberekeningen van het project New Orleans, woontoren en ‘Arthouse’, op de Kop van Zuid in Rotterdam.

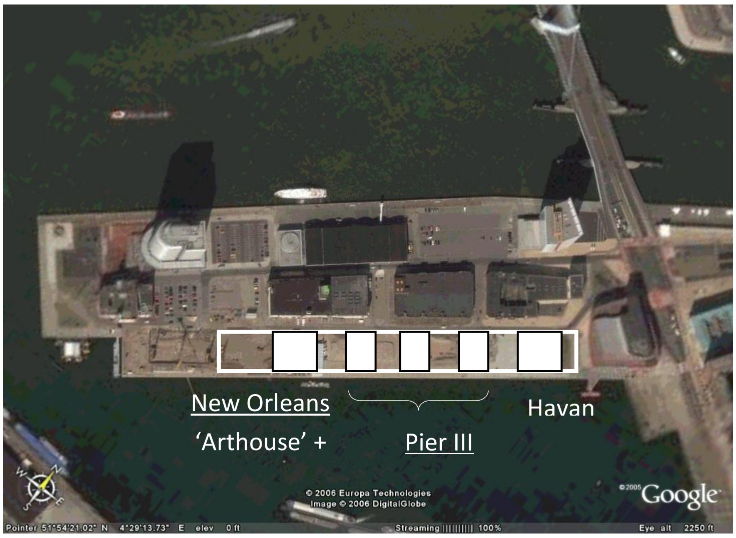
Het architectonisch ontwerp van de woontoren en het ‘Arthouse’ is van de architect Alvaro Siza, in opdracht van Vesteda. DHV BV is verantwoordelijk voor het constructief ontwerp. De woontoren heeft een hoogte van circa 155 meter boven maaiveld

Project New Orleans op Kop van Zuid

Op de Kop van Zuid te Rotterdam wordt aan de Otto Reuchlinweg een 2-laagse verdiepte parkeergarage van ca. 300 meter lang gebouwd met daarop een vijftal woontorens.

Twee torens van ca. 155 meter (New Orleans en Havanna) en drie torens van ca. 70 meter (Pier III).

Het project New Orleans bestaat uit een woontoren en een plintgebouw. De woontoren is ca. 155 meter hoog en heeft een bebouwd oppervlak van ca. 30 meter bij 30 meter. Het plintgebouw wordt 'Arthouse' genoemd. Het 'Arthouse' is ca. 18 meter hoog en heeft een bebouwd oppervlak van ca. 30 meter bij 68 meter. Het 'Arthouse' staat aan de voet van de woontoren. Beide gebouwen staan op de verdiepte 2-laagse parkeergarage.



Constructieprincipe

De constructie van het gehele project is in zes delen onder te verdelen;

- de bouwput,
- de fundering,
- de parkeergarage,
- de woontoren,
- het 'Arthouse' en
- dilataties en/of tijdelijke voegen.

De fundering

De woontoren wordt gefundeerd op een grote funderingspoer met een zware betonnen ring eromheen. De poer en de ring worden gekoppeld door een relatief slap vloertje. Onder de toren komen circa 336 funderingspalen, in een driehoekstramien geplaatst. Onder de kolommen is de h.o.h. afstand van de palen 1,5 meter. Onder de stabiliteitskern is de h.o.h. afstand van de palen 1,4 meter.
Er worden prefab betonnen funderingspalen toegepast met een doorsnede van vierkant 450x450 mm^2 en een inheidiepte van circa 25 m -NAP.

De resultaten van het grondonderzoek en de uitgangspunten voor de fundering worden beschreven in de rapporten van Geomet (AA08350-4 en AA08350-7).

Bij het heien zal worden gelet op verdichting en heivolgorde. De verwachtte heiverdichting zal circa 6,3% bedragen. De heivolgorde is van belang i.v.m. het in tact blijven van de bestaande kademuurconstructie, e.e.a. conform eisen van het Havenbedrijf.

Er worden schoorpalen toegepast om de horizontale windbelasting af te dragen.

De palen zullen, afgezien van een lichte ontgraving en mogelijke zandaanvulling, vanaf maaiveld worden geheid met een oplanger.

Ten behoeve van het opvangen van verschilzettingen zal de funderingspoer van de woontoren, gedurende de constructiefase van de ruwbouw, volledig losgehouden worden van de rest van de gebouwdelen (t.w. parkeergarage, 'Arthouse'). Door middel van een tijdelijk dilatatie profiel wordt voorkomen dat water de parkeergarage in kan stromen. Zodra de constructie van het gebouw op hoogte is en de directe vervormingen van de ondergrond zijn opgetreden zal de kernpoer definitief aan de garage vloer worden vastgestort.

Het 'Arthouse' en het plein worden gefundeerd op meerpaalspoeren met daaronder prefab palen. De –2 vloer is vrijdragend verbonden met dezelfde poeren.

In verband met de hoge investeringskosten is de fundering niet zo uitgevoerd dat hij zijn standzekerheid behoudt bij het weghalen van de kademuur. Derhalve worden speciale eisen gesteld bij een eventuele sloop van de kademuur.

De parkeergarage

Onder het project wordt een tweelaagse parkeergarage aangelegd. Deze parkeergarage loopt onder het 'Arthouse' de toren en een deel van het plein. Hij heeft een lengte van ongeveer 100 meter en een breedte van ca 33 meter. Er wordt in vier stroken recht geparkeerd in twee rijbanen met eenrichtingsverkeer.

De –1 vloer en de begane grond vloer wordt met uitzondering van de vloeren onder de toren uitgevoerd en de begane grond vloer tussen as A en B en F en H in kanaalplaten met een druklaag op prefab balken. De prefab balken worden als Gerberliggers uitgevoerd en lopen door over de kolommen. De kolommen zijn rond en worden uitgevoerd in prefab beton. In het gebied onder de toren worden breedplaatvloeren gemaakt op prefab balken. Er wordt hier voor breedplaatvloeren gekozen omdat hier de vloeren ook als kniksteun dienen voor de zware stalen HD kolommen waardoor er aanzienlijke horizontale krachten in de vloeren naar de kern worden afgedragen.

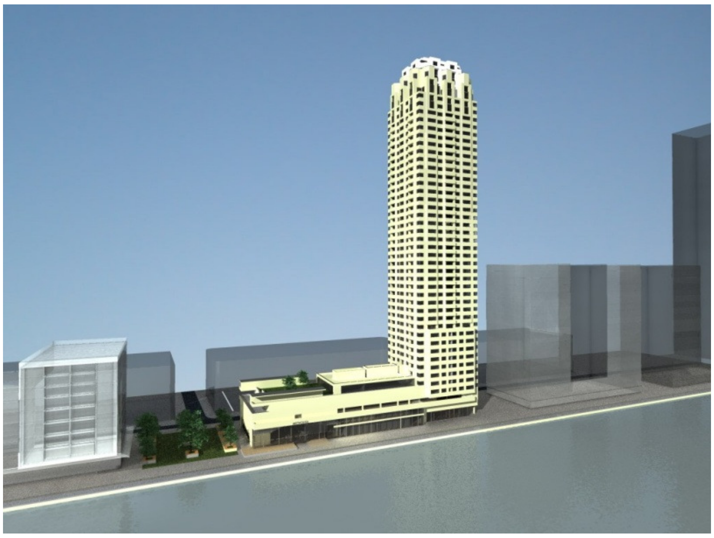
De begane grond vloer tussen as A en B wordt in breedplaat uitgevoerd in verband met de hoge verkeersbelasting VK45. De begane grond vloer tussen as F en H dient als overgangsconstructie voor de prefab kolommen en wordt in het werk gestort.

Op beide parkeerlagen is in de betonwand op as 01 een mogelijkheid voorzien om een sparing voor een vluchtdeur naar Montevideo te maken.

De woontoren

De opbouw
De toren wordt opgetrokken in beton. De constructie is in drie delen onderverdeeld;

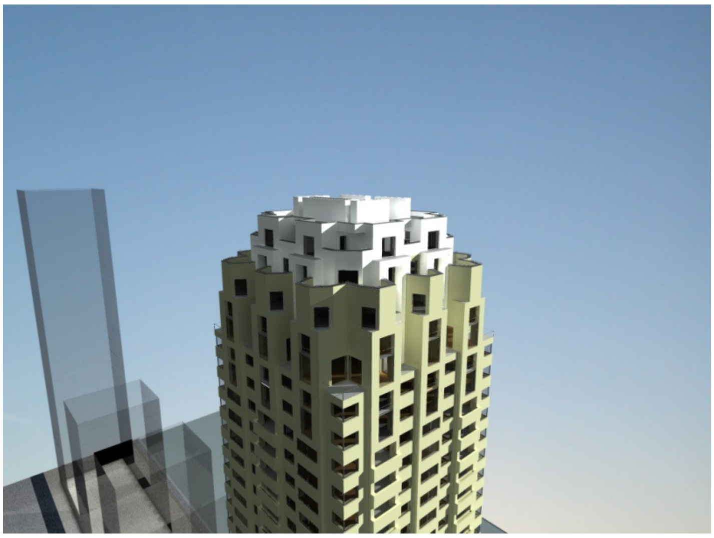
- de voet, (-1^e, b.g., 1^e, 2^e en 3^e verdieping),
- de schacht (4^e t/m 40^e verdieping) en
- de top (41^e t/m de 45^e verdieping).
-



De voet bestaat uit een rechthoekige betonnen kern met een wanddikte van 600 mm en veertien zware stalen kolommen (HD-profielen). Op deze stabiliteitskern en kolommen rust de schacht.
Op de begane grond en de 1^e verdiepingsvloer worden de entree, de fietsenberging een aantal commerciële ruimtes ondergebracht. De 2^e verdiepingsvloer is de technische ruimte en vanaf de derde verdieping worden de appartementen gerealiseerd.
De 1^e; 2^e en 3^e verdiepingsvloer worden gerealiseerd in breedplaten. Hierbij worden de vloeren op de 1^e en de 2^e verdieping op prefab balken gelegd, en op de 3^e verdieping op de wanden. De vloeren worden gerekend als kniksteun voor de zware stalen kolommen. Met uitzondering van de kolommen op as C11; E11; C15 en E15 op de eerste verdieping. De kolommen worden uitgerekend op een aanrijdbelasting op begane grond niveau.
Op de 2^e en de 3^e verdieping worden, daar waar de stalen kolommen aansluiten op de betonwanden, stalen overgangsconstructies gerealiseerd om de grote krachten van de bovenliggende betonwanden in de kolommen leiden.

Vanaf de 3^e tot de 41^e verdiepingsvloer wordt de toren uitgevoerd als een in het werk gestorte constructie. Dit deel van de woontoren wordt de schacht genoemd. De vloeren, wanden van de stabiliteitskern en de overige wanden zijn 300 mm dik. De uitvoering van de schacht vindt plaats volgens het gietbouw-principe, met de mogelijkheid gebruik te maken van een viertal tunnelkistsystemen.

In de stabiliteitskern worden breedplaatvloeren van 200 mm dik toegepast. De liftkern wordt opgetrokken uit dragende prefab elementen met een dikte van 200 mm.



De top wordt ook als een in het werk gestorte constructie uitgevoerd. Het gebruik van een tunnelkistsysteem is hier niet mogelijk door de grote variatie in de plattegronden. De sprongen in de gevels worden gecreëerd door betonnen wandelementen dusdanig te stapelen en te laten verspringen t.o.v. elkaar dat er een trapsgewijs verloop ontstaat. De wandelementen vormen samen met de vloeren een soort verborgen boogconstructie. Door de symmetrie van de toren is deze boogconstructie in evenwicht. De horizontale krachten die in de vloeren ontstaan maken evenwicht met elkaar.
Tussen stramien 12 en 14 worden op de 41e verdiepingsvloer wandelementen gestapeld om een deel van de bovenliggende vloeren op te vangen. Hierbij ontstaat een sprong in de dragende lijnen. De 41e verdiepingsvloer wordt tussen deze stramienen 12 en 14 dikker uitgevoerd.
De belastingen op het dak, t.g.v. o.a. de gevelreinigingsinstallatie, worden opgevangen door een betonnen balk die boven het dak uitsteekt. Deze balk draagt de belastingen af naar de ‘boogconstructie’ en de wandelementen.

De stabiliteit
De draagconstructie van de woontoren is een monoliete betonconstructie. De wanden en de vloeren worden in het werk gestort.
In hoofdzaak wordt de stabiliteit verzorgd door een H-vormige stabiliteitskern. Deze stabiliteitskern bestaat uit de rechthoekige betonnen kern (9,5 m x 13,2 m) en de, aan beide zijden van de kern uitkragende, betonnen wanden op stramien 12 en stramien 14.
Door raamwerking tussen de wanden en de vloeren wordt extra stijfheid verkregen.

Op de 2e en de 3e verdieping staan de betonwanden op kolommen. De wanden werken hier als verborgen ‘outrigger’.
De belastingen worden via de kern en de kolommen naar de –2 vloer afgedragen waar ze via een funderingspoer van de 2,5 meter dik naar de palen worden afgedragen. Voor het afdragen van de horizontale belastingen naar de ondergrond worden schoorpalen gebruikt.

De bouwfasering
Zettingen in de Laag van Kedichem veroorzaken ongelijkmatige zakkingen van de hoogbouw. De zwaarbelaste kern zakt meer dan de buitenste ring van kolommen. De kern is via de zeer stijve 'outrigger' wanden constructief gekoppeld aan de kolommen. De ongelijkmatige zakkingen veroorzaken daardoor tijdens de bouwfase en een deel van de gebruiksfase een verschuiving van belastingen van de kern naar de kolommen. De krachten in de kolommen onder de 'outrigger' wanden mogen niet te groot te worden. Om deze belastingverschuiving enigszins onder controle te houden mogen de vulplaten tussen de kolommen en de overgangsconstructies pas worden aangebracht als de bovenliggende betonwand reeds meerdere verdiepingen hoog is. Een en ander is afhankelijk van de bouwfasering van de aannemer en zal in de werkfase moeten worden afgestemd.
Het betreft hier de kolommen op as B12 en B14 en F12 en F14.

Dilataties en/of tijdelijke voegen
Concept voor de eindfase:
Uitgangspunt voor dit ontwerp is dat het gebouw ongedilateerd wordt opgeleverd. In de eindfase zijn er geen grote constructieve (beton)oppervlaktes die direct door de zon worden aangestraald. In de bouwfase zal de aannemer een aantal aanvullende voorzieningen moeten treffen om schade door temperatuursbelastingen en krimp te voorkomen.

Concept voor de bouwfase:
Ten behoeve van het opvangen van verschilzettingen zal de kernpoer gedurende de constructiefase volledig losgehouden worden van de rest van de gebouwdelen. Door middel van een tijdelijk dilatatie profiel wordt voorkomen dat water de parkeergarage in kan stromen. Zodra de constructie van het gebouw op hoogte is en de directe vervormingen van de ondergrond zijn opgetreden zal de kernpoer definitief aan de garage vloer worden gestort.

De hoogbouw zal hoger worden aangelegd dan het 'Arthouse' en het plein. Door de grotere elastische zakking en de grotere zakking van de laag van Kedichem onder de hoogbouw zal getracht worden om zo veel mogelijk het uiteindelijke peil van de verschillende gebouwdelen gelijk te krijgen. De definitieve verticale maatvoering zal in overleg met de aannemer in de werkfase bepaald moeten worden.

Uitgangspunten

Voorschriften
De volgende voorschriften zijn voor het gebouw van kracht:

Algemeen : TGB 1990, NEN 6700, NEN 6702
Beton : TGB 1990, NEN 6720 (VBC 1995)

Geotechniek : TGB 1990, NEN 6740 t/m NEN 6744
Staal : TGB 1990, NEN 6770

Bijbehorende tekeningen
Bouwkundige tekeningen Alvaro Siza Architectos / ADP.
Constructieve tekeningen DHV.

Gegevens derden

Rapporten Geomet*

AA08350-1	Rapport betreffende grondonderzoek New Orleans	definitief	16-12-2003
AA08350-2	Rapport betreffende bouwput en bemaling New Orleans	definitief	16-02-2004
AA08350-4	Rapport betreffende bemaling parkeergarage New Orleans	definitief	07-07-2006
AA08350-5	Rapport betreffende grondonderzoek nieuwbouw New Orleans	definitief	26-06-2006
AA08350-6	Rapport betreffende funderings nieuwbouw New Orleans	definitief	25-07-2006
AA08350-7	Rapport betreffende bouwput parkeergarage New Orleans	definitief	26-07-2006
AA08350-8	Briefrapport Plaxis-berekening laagbouw incl. beoordeling invloed op kademuur	definitief	13-12-2006

Rapporten Peutz**

WG 4448-3	Bepaling representatieve winddrukken op omhullende alsmede krachten en momenten op hoofddraagconstructie d.m.v. windtunnelonderzoek	definitief	03-05-2006

Uitgangspunten berekening

Algemeen

Woongebouw	: veiligheidsklasse 3	
Referentieperiode	: 50 jaar	
Uiterste grenstoestand	: γ_g = 0,9/1,2/1,35	γ_q = 1,5
Bruikbaarheidsgrenstoestand	: γ_g = 1,0	γ_q = 1,0
Windgebied	: II, onbebouwd	
Milieuklassen	: zie blad 17	

Beton

IHWG fundering	C28/35
IHWG -2 vloer	C20/25
IHWG waterdichte kelderwanden	C28/35
IHWG druklagen op kanaal- / breedplaten	C28/35
IHWG hellingbanen + balken parkeergarage	C28/35
IHWG vloeren toren verdieping 4 t/m 41	C53/65
IHWG wanden toren verdieping 4 tm 41	C53/65
IHWG vloeren toren boven 41e verdieping	C28/35
IHWG wanden toren boven 41e verderdieping	C28/35

T.p.v. overgangsconstructies 2e / 3e verdieping Hoge Sterkte beton B105

* Geomet is een geotechnisch adviesbureau, die in eigen beheer bodemonderzoek uitvoert d.m.v. elektrische sonderingen.

** Peutz bestaat uit een groep van onafhankelijke bureaus van raadgevend ingenieurs op het gebied van akoestiek, bouwfysica, duurzaam bouwen, lawaaibeheersing, trillingstechniek, milieutechnologie, (brand-) veiligheid en arbeidsomstandigheden.

IHWG wanden 'Arthouse' 3e verdieping	C53/65
IHWG wanden 'Arthouse' as 6 bgg tot 3e verd.	C53/65
IHWG wanden 'Arthouse' overigen	C28/35
IHWG vloer 'Arthouse' begane grond as F-G	C53/65
Prefab beton kolommen as E/7-8 bgg	Hoge Sterkte beton B105
Prefab betonkolommen parkeergarage	C53/65 en Hoge sterkte beton B105
Prefab beton kolommen overigen	C53/65
Prefab beton balken	C53/65
Prefab beton wanden	C53/65
Betonstaal	
Betonstaal (staven)	FeB 500 – HWL
Betontaal (netten)	FeB 500 – HKN
Staal	
Kolommen woontoren	S460 warmgewalst (Histar 460 klasse 1)
Overgangsconstructie woontoren	S460 warmgewalst
Stalen liggers en vakwerken 'Arthouse'	S355 warmgewalst
Bouten en moeren	minimaal 8.8
Ankers	minimaal 4.6

Belastingen

Permanente en veranderlijke vloerbelastingen

Toren

(belastingen in kN/m^2)

Verdiepingsvloeren toren (t.p.v. woningen), tussen assen 11-12 en 14-15	PB	VB
IHWG betonvloer d = 300 mm	7,5	
zwevende dekvloer (60 mm)	1,2	
lichte scheidingswanden	1,2	
VB personen e.d. (ψ = 0,4)		1,75
	9,9	1,75

Verdiepingsvloeren toren (t.p.v. woningen), tussen assen 12-13 en 13-14	PB	VB
IHWG betonvloer d = 300 mm	7,5	
zwevende dekvloer (60 mm)	1,2	
lichte scheidingswanden	1,0	
VB personen e.d. (ψ = 0,4)		1,75
	9,7	1,75

Balkons / Loggia’s	PB	VB
IHWG betonvloer d = 300 mm	7,5	
Isolatie + tegels + plafond	1,3	
VB personen e.d. (ψ = 0,5)		2,50
	8,8	2,50

Verdiepingsvloeren toren (t.p.v. kern)	PB	VB
IHWG betonvloer d = 200	5,0	
zwevende dekvloer (60 mm)	1,2	
VB personen e.d. (ψ = 0,25)		3,00
	6,2	3,00

Technische laag (t.p.v. kern; exclusief prefab balken)	PB	VB
Breedplaat vloer d = 300 mm	7,5	
zwevende dekvloer (60 mm)	1,2	
VB personen e.d. (ψ = 1,0)		5,00
NB. Hier is geen rekening gehouden met evt. betonnen platen t.b.v. machines	8,7	5,00

Eerste verdiepingsvloer (winkel + fietsenstalling; exclusief prefab balken)	PB	VB
Breedplaatvloer op prefab balken d = 300	7,5	
Zwevende dekvloer (60 mm)	1,2	
plafond + leidingen	0,5	
VB personen e.d. (ψ = 0,25)		5,00
	9,2	5,00

BGG vloer toren (exclusief prefab balken)	PB	VB
Breedplaat vloer op prefab balken d = 400 mm	10,0	
Schuimbeton d = 370 (12 kN/m^2)	4,44	
Deklaag d = 50	1,00	
Leidingen	0,5	
VB personen e.d. (ψ = 0,25)		10,0
	15,9	10,0

BGG vloer toren verhoogde deel (exclusief prefab balken)	PB	VB
Breedplaat vloer op prefab balken d = 300 mm	7,5	
Deklaag d = 100	2,0	
Leidingen	0,5	
VB personen e.d. (ψ = 0,25)		10,0
	10,0	10,0

-1 vloer t.p.v. toren (exclusief balkbodems)	PB	VB
IHWG d = 260 mm	6,5	
VB auto’s e.d. (ψ = 0,7)		2,0
	6,5	2,0

-2 vloer t.p.v. toren	PB	VB
IHWG d = 2500 mm	62,5	
VB auto's e.d. (ψ = 0,7)		2,0
	62,5	2,0

Gevelbekleding toren	PB	VB
Natuursteen gevel (4cm)	1,1	

Windbelasting

De locatie van het project New Orleans (woontoren en 'Arthouse') is de Otto Reuchlinweg op de Kop van Zuid te Rotterdam.

Windgebied II; onbebouwd

Woontoren

- op 155 meter is de extreme waarde van de stuwdruk p_w = 1,88 kN/m^2.
- op 30 meter is de extreme waarde van de stuwdruk p_w = 1,26 kN/m^2.

Factor voor afmetingen van het gebouw

Woontoren

- Hoogte woontoren h = 155 meter.
- Breedte woontoren b = ca. 30 m.
- C_{dim} = 0,87

Windvormfactoren, C_{index}

Woontoren

C_{pe} = 0,8 voor druk.

C_{pe} = 0,4 voor zuiging.

$C_{pe,loc}$ = n.v.t. voor stabiliteitsberekening.

C_{pi} = n.v.t. voor stabiliteitsberekening.

C_f = 0,04.

C_t = n.v.t.

Dynamische vergrotingsfactor

Woontoren

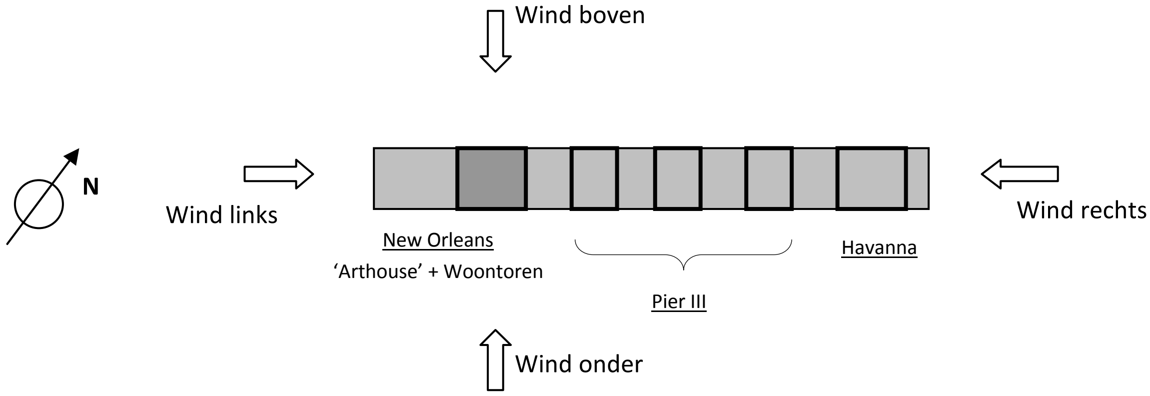
ϕ_1 = 1,40

(Voor berekening van deze waarde zie rapport V2090-1.2 gewichts- en stabiliteitsberekening)

Windtunnelonderzoek

Woontoren

Voor het totale project op de Kop van Zuid (New Orleans, Pier III, Havanna) is windtunnelonderzoek uitgevoerd. Zie Peutz rapport WG 4448-3 'Bepaling representatieve winddrukken op omhullende alsmede krachten en momenten op hoofddraagconstructie d.m.v. windtunnelonderzoek', d.d. 03-05-2006.
Op basis van de resultaten wordt er een reductiefactor van 0,9 op de windbelasting toegepast uit de richting noordwest en zuidoost (wind van 'boven' en wind van 'onder'). Geen reductie op wind uit de richting zuidwest en noordoost (wind van 'links' en wind van 'rechts'). Zo ook voor overige windrichtingen.
Zie rapport V2090-1.2 'gewichts- en stabiliteitsberekening' voor de onderbouwing van de reductiefactor.



Bijzondere belastingen

Brand:
Alle betonnen constructieonderdelen (kolommen, lateien, wanden en vloeren) zijn zo gedimensioneerd dat de bij het constructieonderdeel behorende brandwerendheid verkregen wordt zonder het verlies aan draagkracht.
De stalen kolommen worden 120 minuten brandwerend bekleed.

Gasexplosie:
De constructieonderdelen worden niet specifiek gedimensioneerd op een explosiebelasting. De explosiebestendigheid wordt gewaarborgd door de uitvoering van de gevelpuien en de bevestiging van de gevelpuien aan het skelet. Door de opbouw en de manier van bevestiging van de gevelpuien, kan deze gezien worden als ontlastopening in geval van een explosie.

Botsing door voertuigen:
De volgende constructieonderdelen zullen worden gedimensioneerd op het belastingsgeval aanrijdbelasting, conform NEN 6702 artikel 8.5.

- Alle kolommen in de parkeerkelder. De parkeergarage is ingericht voor het parkeren van voertuigen tot 2500 kg.
- Alle 5 kolommen van de woontoren op as B op begane grond niveau.

Tweede Draagweg

Woontoren
Het bezwijken van een onderdeel van de bouwconstructie van de woontoren mag niet tot onevenredig grote schade leiden. Na bezwijken van een onderdeel, door welke oorzaak dan ook, moet de schade beperkt blijven tot de aangrenzende ruimten of aangrenzende dragende onderdelen van de beschouwde constructie.
De betonconstructie van de schacht en de top zijn zeer robuust met wanden en vloeren van 300 mm dik. De desbetreffende onderdelen hebben daardoor al een grote weerstand tegen bijzondere belastingen.
Bovendien heeft de betonconstructie dusdanige samenhang dat bij bezwijken van een onderdeel een tweede draagweg gevonden wordt.
Bij de beton- en staalconstructie van de voet ligt e.e.a. wat kritischer. De twaalf stalen kolommen rond de betonnen kern zijn zwaar belast. Een viertal kolommen ondersteunen zelfs de 'outrigger' wanden en spelen daardoor een belangrijke rol in de stabiliteit van de toren.

Ondanks dat de stalen kolommen worden berekend op bijzondere belastingen (NEN2702) en er speciale aandacht gevraagd wordt in de kwaliteitsborging (NEN6700) van het ontwerp- en uitvoeringsproces, wordt de toren gecontroleerd cq. berekend op een tweede draagweg voor het geval dat een willekeurige stalen kolom bezwijkt, e.e.a. conform de eis van de Dienst Stedebouw en Volkshuisvesting van de Gemeente Rotterdam.

Brandwerendheidseisen

Functie:	Woongebouw
B.k. hoogste vloer:	70 meter +
Eis hoofddraagconstructie:	120 minuten

Ter plaatse van de omloop op de tweede verdieping tussen de assen A-B en G-H ontstaat bij brand een gewijzigd constructief schema voor de afdracht van de belasting uit de kanaalplaten.
De kanaalplaten liggen op geïntegreerde liggers HE260B. In de gebruikssituatie overspannen de liggers op 2 steunpunten tussen de prefab kolommen vierkant 450 en de IPE270 kolommen in de gevel. Deze IPE270's worden niet brandwerend beschermd. In de situatie brand functioneren de HE260B liggers daarom als uitkragende liggers vanuit de prefab vierkant 450 kolommen.

Functie:	Parkeergarage (niet direct gelegen onder de woontoren)
Eis hoofddraagconstructie:	90 minuten

Functie:	Parkeergarage (direct gelegen onder de woontoren)
Eis hoofddraagconstructie:	120 minuten

WBDBO-eisen volgens opgave architect.

Trillingen

Het gebouw dient aan een aantal eisen met betrekking tot trillingen te voldoen. Trillingen mogen de doelmatigheid van een constructieonderdeel het gebouw niet belemmeren en geen schade veroorzaken. Hinderlijke trillingen moeten worden voorkomen.

De eerste eigenfrequentie van de vloer van het podium van het multi-theater ('Arthouse') en de verhoogde vloer van de fitnessruimte mag niet lager zijn dan 5 Hz. De overige vloeren niet lager dan 3 Hz.

Voor de versnellingen in de top ten gevolgen van de trillingen door het belastingsgeval wind wordt verwezen naar NEN 6702 artikel 10.5.3.

Grondwaterpotentialen

Geomet heeft geadviseerd in de toe te passen grondwaterpotentialen:

Hoogste grondwaterstand:	1.70m + NAP = 1.95m - peil
Laagste Grondwaterstand:	5.00m - NAP = 8.65m - peil

8 Appendix 2: Structural Design Aspects

8.1 Weight Calculation Core – Concrete versus Steel

<u>Core in concrete:</u>
The data are as follows;

Width =	9.5 m
Length =	13.2 m
Wall thickness =	250 mm
Height =	155 m
$\gamma_{concrete}$ =	25 kN/m^3

NB: The wall thickness of the concrete core is not based on structural requirements, but on construction aspects; afterwards a verification will be done to see whether the chosen dimensions sufficed.

Concrete surface

$$A = (13.45 \cdot 9.75) - (12.95 \cdot 9.25) = 11.35\ m^2$$

Volume of the concrete

$$V = 11.35 \cdot 155 = 1759\ m^3$$

Dead load

$$G_{concrete\ core} = 1759 \cdot 25 = 43981\ kN$$

Verification compression strength concrete
Dead load concrete core

$$1.2 \cdot 43981 = 52778\ kN$$

Dead load floor + live load floors (including safety factor, taken from the weight calculations)

$$238437\ kN$$

Total load on concrete core

$$52778 + 238437 = 291215\ kN$$

$$\sigma = \frac{291215 \cdot 10^3}{11 \cdot 10^6} = 25.7\ N/mm^2$$

Grade of concrete C35/45 would suffice

Core in steel, data:
The data are as follows;

Width = 9.5 m
Length = 13.2 m
Height = 155 m

The load on the core (including safety factor) amounts to 238437 kN and six columns are present in the core.

Estimation own weight columns:

$$F_{column} = \frac{238437}{6} = 39740\ kN$$

Selected section: HD 400 x 990 (S355) (N_{pl} = 44816 kN / G = 990 kg/m)
The minimum section at the top of the building is based on a practical dimension, namely: HEB 300 (M = 119 kg/m)

The weight of the columns has a linear trajectory - analogous with the normal forces – so the average can be calculated as follows an average

$$G_{average} = \frac{990 + 119}{2} = 555\ kg/m \cong 5.55\ kN/m$$

Own weight columns

$$5.55 \cdot 155 \cdot 6 = 5162\ kN$$

Total load on the steel core

$$5162 + 238437 = 243599\ kN$$

The weight of the steel columns is marginal in addition to the dead and live load of the floors. Therefore a reiteration is not useful. The outcome of the comparison is in Table 8.1.

	Own weight (kN)
Concrete core	43981
Steel core	5162

Table 8.1: Comparison construction weight of concrete and steel core construction.

8.2 Core participation in stiffness

The contribution of a potential concrete core – in addition to the TFC – in the total stiffness can be simply determined. The deformation of the building is inversely proportional with the stiffness of the construction. In this stadium of preliminary design, only the bending stiffness is considered and the shear stiffness is ignored.

The following data has been used;

Core dimensions =	11 x 11 m (a square core is envisaged for practical reasons, no openings present)
Wall thickness =	250 mm (the minimum to meet the acoustic requirements ≥ 725 kg/m^2)
Allowed sway =	155mm (the 1st order sway of is set at half the value of u_{max} = 310 mm)
E =	10,000 N/mm^2 (cracked)
Wind load =	q = 30·2.34 = 70.2 kN/m^1
Building height =	155 m

The moment of inertia equals:

$$I_{core} = \frac{1}{12}(11.25^4 - 10.75^4) = 222\ m^4$$

And the bending stiffness becomes:

$$EI_{core} = 10.0 \cdot 10^6 \cdot 222 = 2.22 \cdot 10^9\ kNm^2$$

The corresponding deflection is:

$$u = \frac{ql^4}{8EI} = \frac{70.2 \cdot 155^4}{8 \cdot 4.27 \cdot 10^9} = 2.282m = 2282\ mm$$

The total deflection can be found by reciprocally summing the deflections of the core and the TFC:

$$\frac{1}{u_{max}} = \frac{1}{u_{core}} + \frac{1}{u_{TFC}}$$

$$\frac{1}{155} = \frac{1}{2282} + \frac{1}{u_{TFC}}$$

Meaning that the deflection of the TFC would be:

u_{TFC} = 166 mm

The contribution of the concrete core as regards deformation can then be determined:

$$\frac{1206^{-1}}{155^{-1}} * 100\% = 6.8\%$$

The contribution of the concrete core is 6.8%, signifying the relatively minor impact of the concrete core to the horizontal deformation of the building. If the stiffness of the building needs to be augmented even more, it is more efficient to increase the sections of the TFC than to increase the dimensions of the concrete core; simultaneously, this leads to a further reduction of the participation of the concrete core in the stiffness.

9 Appendix 3: Geometry of TFC

9.1 Derivation of stiffness formulae

Below, the discrete models of the three different geometries have been translated into continuous models with a bending stiffness (EI) and a racking shear stiffness (GA). With these two characteristics, the stiffness of the discrete model can be approximated which enables;

- An optimisation of the efficiency via a more ideal distribution of the construction material over the different members
- A better understanding of the functioning of the construction
- An easy change of parameters (geometry, material) and assessment of the effect on the deflection

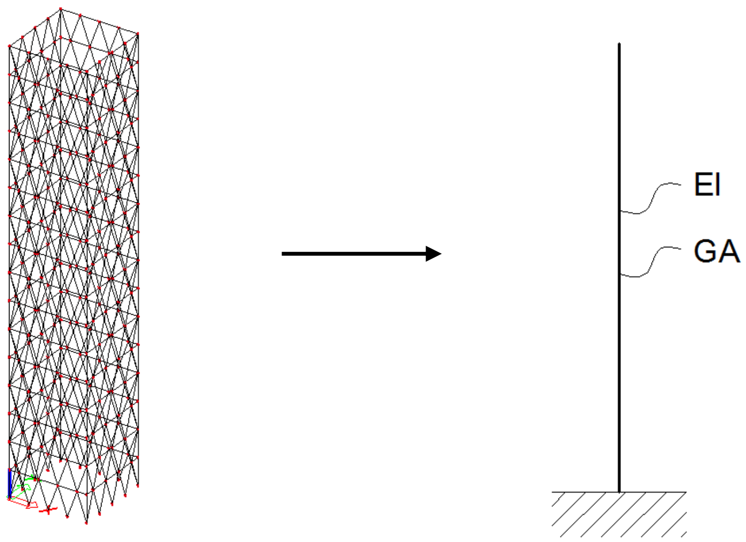


Figure 9.1: Discrete model versus continuous model

These formulae are derived for all three geometries hereafter referred to as;

- “vertical columns”
- “diagonal columns”
- “diagonal & vertical columns”

TFC variant: "Vertical columns"

The racking shear stiffness and the bending stiffness are first determined for the 2-dimensional trussed frame and then for the 3-dimensional trussed tube structure, as illustrated in Figure 9.2 and Figure 9.3, respectively.

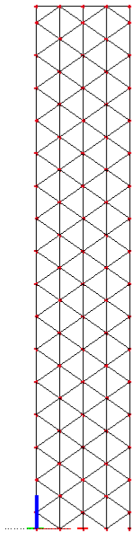


Figure 9.2: 2-dimensional variant "vertical columns"

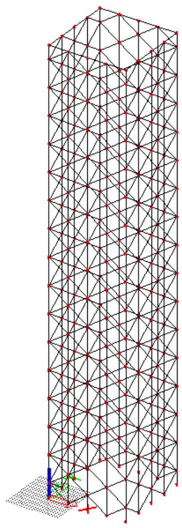


Figure 9.3: 3-dimensional variant "vertical columns"

<u>Racking shear stiffness</u>

The force F is spread evenly over the 4 diagonals in one horizontal section of the TFC. Due to force F the members undergo a shortening or a lengthening as illustrated in Figure 9.4.

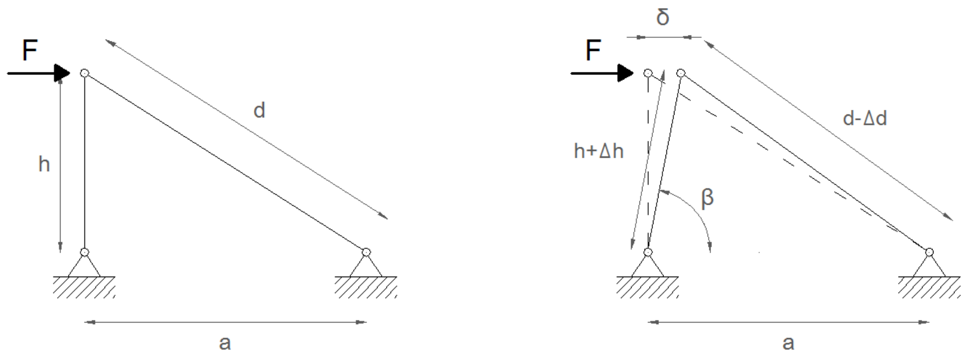


Figure 9.4: Section of a quarter-side of the variant "vertical columns"

With the help of the cosinus-rule, the horizontal deflection (δ) due to a force (F) can be calculated:

$$\cos(\beta)\, 2ah = a^2 + h^2 - d^2$$

$$\frac{\delta}{(h+\Delta h)} 2a(h+\Delta h) = a^2 + (h+\Delta h)^2 - (d-\Delta d)^2$$

$$2\delta a = a^2 + h^2 + 2h\Delta h + \Delta h^2 - d^2 + 2d\Delta d - \Delta d^2$$

Given that $a^2 + h^2 - d^2 = 0$ (Pythagoras' theorem), and that $\Delta d^2 \ll \Delta d$, the equation can be simplified:

$$2\delta a = 2h\Delta h + 2d\Delta d$$

$$\delta_{discrete} = \frac{h\Delta h}{a} + \frac{d\Delta d}{a}$$

This is the discrete function for the horizontal deflection (δ) as the sum of the bending deflection (lengthening of the vertical) and the shear deflection (shortening of the diagonal), respectively. The continuous function for horizontal deflection, with one concentrated load at the top is given by:

$$\delta_{continuous} = \frac{Fh^3}{3EI} + \frac{Fh}{GA}$$

Analogous, the continuous function for the horizontal deflection (δ) is the sum of the bending deflection and the shear deflection. Subsequently, the parameter for shear deflection from the discrete function can be equated with the parameter from the continuous function:

$$\frac{d\Delta d}{a} = \frac{Fh}{GA}$$

$$GA = \frac{Fha}{d\Delta d}$$

With:

$$\Delta d = F\frac{d}{a}\frac{d}{EA_d}$$

The formula can be written as:

$$GA = \frac{a^2hEA_d}{d^3}$$

However, this formula only gives the racking shear stiffness for one diagonal, whereas the 2-dimensional variant has 4 diagonals. Therefore, the racking shear stiffness needs to be multiplied by four, to give the correct formula for the trussed façade frame:

$$GA = \frac{4a^2hEA_d}{d^3}$$

In the case of the trussed tube structure, the expression for the racking shear stiffness needs to be multiplied by two since shear occurs in the two web planes of the tube. The formula for the tube structure is:

$$GA = 2 \cdot \frac{4a^2hEA_d}{d^3}$$

<u>Bending stiffness</u>

The gross bending stiffness can be determined with:

$$EI_y = \mathrm{E}I_{own;y} + \mathrm{E}A_v y^2$$

Here, $\mathrm{EI_{own;y}}$ does not contribute to the bending stiffness since the connections are modeled as pinned nodes. The other parameter $\Sigma \mathrm{E}A_v y^2$ represents the bending stiffness from the axial stiffness of the vertical members. Therefore the formula for the bending stiffness becomes:

$$EI_y = \mathrm{E}A_v y^2$$

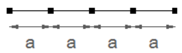


Figure 9.5: Plan of 2-dimensional geometry "vertical columns"

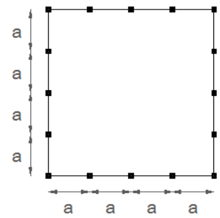


Figure 9.6: Plan of 3-dimensional geometry "vertical columns"

In the case of the 2-dimensional construction, with 5 columns, the formula is:

$$EI_y = \mathrm{E}A_v(2 \cdot a^2 + 2 \cdot 2a^2) = 10a^2 EA_v$$

In the case of the 3-dimensional construction, with 16 columns arranged in an orthogonal grid, the formula can be written as:

$$EI_y = \mathrm{E}A_v(4 \cdot a^2 + 10 \cdot 2a^2) = 44a^2 EA_v$$

The term 'E' in the abovementioned 2 formulae represent the Young's modulus of the construction material.

TFC variant: "diagonal columns"

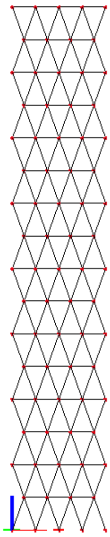


Figure 9.7: 2-dimensional variant "diagonal columns"

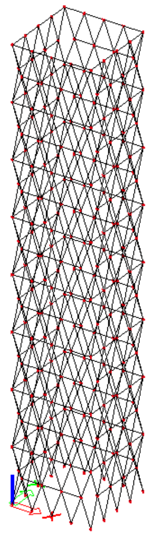


Figure 9.8: 3-dimensional variant "diagonal columns"

<u>Racking shear stiffness</u>

To determine the racking shear stiffness, a force F is spread over the diagonals in one horizontal section of the TFC. The assumption is made that the force is spread more or less evenly over the 4 diagonal.

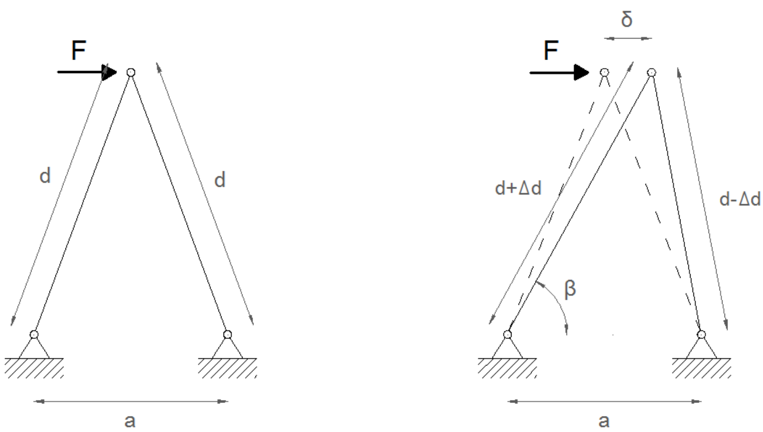


Figure 9.9: Section of a quarter-side of the variant "diagonal columns"

Due to force F the diagonal members undergo a shortening or a lengthening. With the help of the cosinus-rule, the horizontal deflection (δ) can be calculated:

$$\cos(\beta)\,2ac = a^2 + c^2 - b^2$$

$$\frac{\delta + 1/2a}{(d + \Delta d)} 2a(d + \Delta d) = a^2 + (d + \Delta d)^2 - (d - \Delta d)^2$$

$$\left(\delta + \frac{1}{2}a\right) 2a = a^2 + d^2 + 2d\Delta d + \Delta d^2 - d^2 + 2d\Delta d - \Delta d^2$$

$$\left(\delta + \frac{1}{2}a\right)2a = a^2 + 4d\Delta d$$

$$\left(\delta + \frac{1}{2}a\right) = \frac{1}{2}a + \frac{2d\Delta d}{a}$$

$$\delta_{discrete} = \frac{2d\Delta d}{a}$$

Note that this discrete function only holds an equation for shear deflection since bending deflection does not occur in the section considered here. By equating the discrete with the continuous function for horizontal deflection – with one concentrated load at the top – the racking shear stiffness can be determined:

$$\delta_{continuous} = \frac{Fh}{GA}$$

$$\frac{2d\Delta d}{a} = \frac{Fh}{GA}$$

$$GA = \frac{Fha}{2d\Delta d}$$

With:

$$\Delta d = F\frac{d}{a}\frac{2d}{EA_d}$$

The formula becomes:

$$GA = \frac{a^2hEA_d}{2d^3}$$

Since four triangles are present per horizontal section the racking shear stiffness needs to be multiplied by four to find the total racking shear stiffness:

$$GA = \frac{2a^2hEA_d}{d^3}$$

In the case of the trussed tube structure, the expression for the racking shear stiffness needs to be multiplied with two since shear occurs in the two web planes of the tube. The formula for the tube structure is:

$$GA = 2 \cdot \frac{2a^2hEA_d}{d^3}$$

Bending stiffness

The bending stiffness comprises Young's modulus, dependent on the construction material, and the moment of inertia, which can be calculated with Steiner's theorem:

$$I_y = \Sigma A_{equ} y^2$$

However an equivalent section (A_{equ}) needs to be derived for the moment of inertia. This equivalent section is found by deriving equivalent formulae for the shortening of the diagonals and lengthening of the horizontal, and then combining them. The deformation of such a rhomboid can be seen in Figure 9.10.

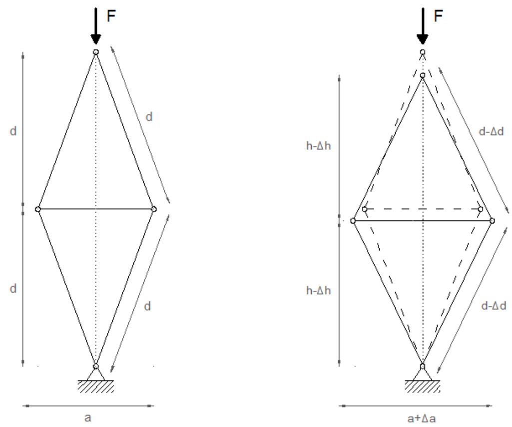


Figure 9.10: Deformation of an 'equivalent section'

Shortening of the diagonals

Figure 9.11 represents the schematization of the shortening of the diagonal.

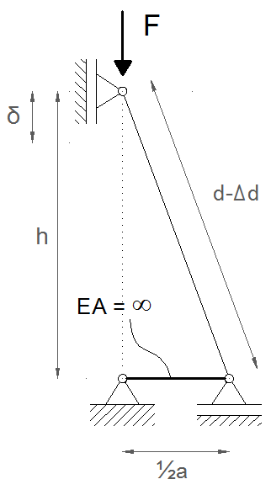


Figure 9.11: Part of the equivalent section: shortening of the diagonal

The force in the diagonal can be expressed as:

$$F_d = F \cdot \frac{d}{h}$$

And the shortening of the diagonal member can subsequently be written as:

$$\Delta d = F\frac{d}{h}\cdot\frac{d}{EA_d} = \frac{Fd^2}{hEA_d}$$

Pythagoras' theorem gives:

$$(h-\delta)^2 = (d-\Delta d)^2 - (\tfrac{1}{2}a)^2$$

$$h^2 - 2h\delta + \delta^2 = d^2 - 2d\Delta d + \Delta d^2 - (\tfrac{1}{2}a)^2$$

Given that $(\frac{1}{2}a)^2 + h^2 - d^2 = 0$ (Pythagoras' theorem), that $\Delta d^2 \ll \Delta d$, and that $\delta^2 \ll \delta$ the equation can be simplified:

$$2h\delta = 2d\Delta d$$

$$\delta = \frac{d\Delta d}{h}$$

After substitution of Δd the formula becomes:

$$\delta = \frac{d}{h}\cdot\frac{Fd^2}{hEA_d} = \frac{Fd^3}{h^2EA_d}$$

The displacement δ can be equated with the formula for the shortening of a member due to a normal force. However the abovementioned formula only takes one diagonal into account; since there are 2 per rhomboid (equivalent section), the formula needs to be multiplied by 2 as well :

$$\delta = \frac{Fd^3}{2h^2EA_d} = \frac{Fh}{EA_{equ;1}}$$

$$\frac{d^3}{2h^2A_d} = \frac{h}{A_{equ;1}}$$

$$A_{equ;1} = \frac{2h^3}{d^3}\cdot A_d$$

<u>Lengthening of the horizontal</u>
Figure 9.12 represents the schematization of the lengthening of the horizontal.

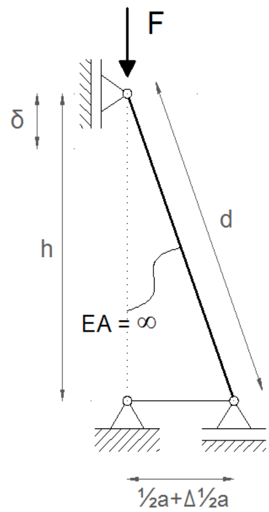


Figure 9.12: Part of the equivalent section: lengthening of the horizontal

The force in the horizontal can be expressed as:

$$F_a = F \cdot \frac{\frac{1}{2}a}{h}$$

And the lengthening of the horizontal member can subsequently be written as:

$$\Delta\tfrac{1}{2}a = F\frac{\frac{1}{2}a}{h} \cdot \frac{\frac{1}{2}a}{EA_h} = \frac{F(\frac{1}{2}a)^2}{hEA_h}$$

Pythagoras' theorem gives:

$$(h-\delta)^2 + (\tfrac{1}{2}a + \Delta\tfrac{1}{2}a)^2 = (d)^2$$

$$h^2 - 2h\delta + \delta^2 = d^2 - (\tfrac{1}{2}a)^2 - \tfrac{1}{2}a\Delta a - (\Delta\tfrac{1}{2}a)^2$$

Given that $(\frac{1}{2}a)^2 + h^2 - d^2 = 0$ (Pythagoras' theorem), that $(\Delta\frac{1}{2}a)^2 \ll \Delta\frac{1}{2}a$, and that $\delta^2 \ll \delta$ the equation can be simplified:

$$2h\delta = \tfrac{1}{2}a\Delta a$$

$$\delta = \frac{\frac{1}{4}a\Delta a}{h}$$

After substitution of Δd the formula becomes:

$$\delta = \frac{\frac{1}{2}a}{h} \cdot \frac{F(\frac{1}{2}a)^2}{hEA_h} = \frac{F(\frac{1}{2}a)^3}{h^2EA_h}$$

The displacement δ can be equated with the formula for the shortening of a member due to a normal force:

$$\delta = \frac{F(\frac{1}{2}a)^3}{h^2 EA_h} = \frac{Fh}{EA_{equ;2}}$$

$$\frac{(\frac{1}{2}a)^3}{h^2 A_h} = \frac{h}{A_{equ;2}}$$

$$A_{equ;2} = \frac{h^3}{(\frac{1}{2}a)^3} \cdot A_h$$

Now that $A_{equ;1}$ and $A_{equ;2}$ are known, both can be combined into one formula: A_{equ} the is the inverse sum of the formulae for $A_{equ;1}$ and $A_{equ;2}$:

$$\frac{1}{A_{equ}} = \frac{1}{A_{equ;1}} + \frac{1}{A_{equ;2}}$$

$$\frac{1}{A_{equ}} = \frac{d^3}{2 \cdot h^3 \cdot A_d} + \frac{(\frac{1}{2}a)^3}{h^3 \cdot A_h}$$

$$A_{equ} = \left(\frac{d^3}{2 \cdot h^3 \cdot A_d} + \frac{(\frac{1}{2}a)^3}{h^3 \cdot A_h}\right)^{-1}$$

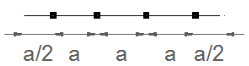


Figure 9.13: Plan of 2-dimensional geometry "diagonal columns"

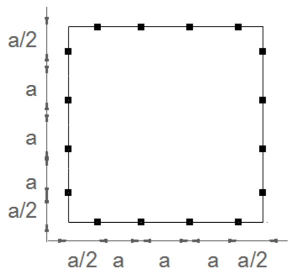


Figure 9.14: Plan of 3-dimensional geometry "diagonal columns"

Subsequently, the bending stiffness can be calculated by multiplying the moment of inertia by Young's modulus:

$$EI_y = \mathrm{E}A_{equ}(2 \cdot (\tfrac{1}{2}a)^2 + 2 \cdot (\tfrac{3}{2}a)^2) = 5a^2 EA_{equ}$$

In the case of the 3-dimensional construction, with 16 columns arranged in an orthogonal grid, the formula can be written as:

$$EI_y = \mathrm{E}A_{equ}(4 \cdot (\tfrac{1}{2}a)^2 + 4 \cdot (\tfrac{3}{2}a)^2 + 8 \cdot (2a)^2) = 42a^2 EA_{equ}$$

TFC variant: “diagonal & vertical columns”

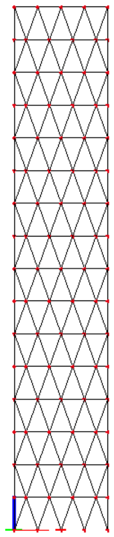


Figure 9.15: 2-dimensional variant “diagonal & vertical columns”

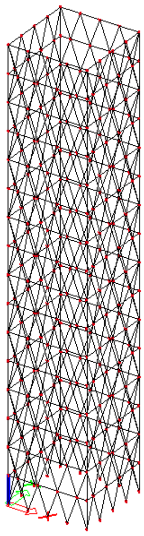


Figure 9.16: 3-dimensional variant “diagonal & vertical columns”

Racking shear stiffness

The racking shear stiffness (GA) of this variant is equal to the racking shear stiffness of the TFC variant “diagonal columns”: the vertical columns at the corners do not affect it. The formula for the trussed façade frame is:

$$GA = \frac{2a^2hEA_d}{d^3}$$

And the formula for the trussed tube structure is:

$$GA = 2 \cdot \frac{2a^2hEA_d}{d^3}$$

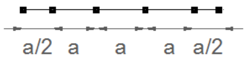
Bending stiffness

The bending stiffness comprises Young’s modulus, dependent on the construction material, and the moment of inertia, which can be calculated with Steiner’s theorem:

$$I_y = \Sigma A_{equ}y^2 + \Sigma A_d y^2$$

Where the equivalent section (A_{equ}) is the same as for the geometry variant “diagonal columns”:

$$A_{equ} = \left(\frac{d^3}{2 \cdot h^3 \cdot A_d} + \frac{(\frac{1}{2}a)^3}{h^3 \cdot A_h}\right)^{-1}$$



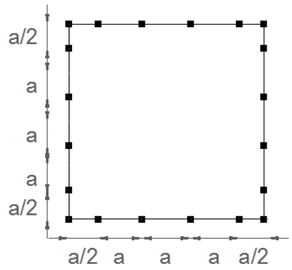


Figure 9.17: Plan of 2-dimensional geometry "diagonal & vertical columns"

Figure 9.18: Plan of 3-dimensional geometry "diagonal & vertical columns"

In the case of the 2-dimensional construction, with 4 equivalent plus 2 vertical columns, the formula is:

$$EI_y = \mathrm{E}A_{equ}(2 \cdot (\tfrac{1}{2}a)^2 + 2 \cdot (\tfrac{3}{2}a)^2) + \mathrm{E}A_v(2 \cdot (2a)^2) = \mathrm{E}a^2(5A_{equ} + 8A_v)$$

In the case of the 3-dimensional construction, with 16 equivalent plus 4 vertical columns arranged in an orthogonal grid, the formula can be written as:

$$EI_y = \mathrm{E}A_{equ}(4 \cdot (\tfrac{1}{2}a)^2 + 4 \cdot (\tfrac{3}{2}a)^2 + 8 \cdot (2a)^2) + \mathrm{E}A_v(4 \cdot (2a)^2) = \mathrm{E}a^2(42A_{equ} + 16A_v)$$

Summary

The formulae for all three geometry variants are summarized in Table 9.1.

“vertical columns”	2-D	3-D
Racking shear stiffness	$GA = \frac{4a^2hEA_d}{d^3}$	$GA = 2 \cdot \frac{4a^2hEA_d}{d^3}$
Bending stiffness	$EI = 10a^2\mathrm{E}A_v$	$EI = 44a^2EA_v$
“diagonal columns”		
Racking shear stiffness	$GA = \frac{2a^2hEA_d}{d^3}$	$GA = 2 \cdot \frac{2a^2hEA_d}{d^3}$
Bending stiffness	$EI = 5a^2\mathrm{E}A_{equ}$	$EI = 42a^2EA_{equ}$
“diagonal & vertical columns”		
Racking shear stiffness	$GA = \frac{2a^2hEA_d}{d^3}$	$GA = 2 \cdot \frac{2a^2hEA_d}{d^3}$
Bending stiffness	$EI = Ea^2(5A_{equ} + 8A_v)$	$EI = Ea^2(42A_{equ} + 16A_v)$

Table 9.1: Overview of the stiffness formulae per geometry variant.

With

$$A_{equ} = \left(\frac{d^3}{2 \cdot h^3 \cdot A_d} + \frac{(\frac{1}{2}a)^3}{h^3 \cdot A_h}\right)^{-1}$$

9.2 Load – deformation formulae

To calculate the shear deflection and bending deflection by hand, formulae need to be derived as well. The starting points are that 16 concentrated loads are present with the same value and evenly distributed along the height.

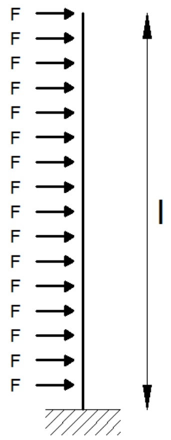


Figure 9.19: Load pattern on construction

<u>Bending deflection</u>
The bending deflection formula for a cantilevered rod with 16 concentrated loads can be derived with help of two existing formulae for a rod with one concentrated load at the top.

The deflection at the top:

$$y = \frac{Fl^3}{3EI}$$

The rotation at the top:

$$\varphi = \frac{Fl^2}{2EI}$$

These 2 formulae need to be combined several times to derive the formula for a rod with 16 concentrated loads based on a cumulative principle:

$$y_i = \Sigma(y_i + \varphi_i l_i)$$

$$y_{bending} = \frac{F}{EI}\left(\frac{{}^1\!/_{16}\,l^3}{3} + \frac{{}^1\!/_{16}\,l^2}{2}\cdot\tfrac{15}{16}l + \frac{{}^2\!/_{16}\,l^3}{3} + \frac{{}^2\!/_{16}\,l^2}{2}\cdot\tfrac{14}{16}l + \frac{{}^3\!/_{16}\,l^3}{3} + \frac{{}^3\!/_{16}\,l^2}{2}\cdot\tfrac{13}{16}l\right.$$

$$\left. + \;\ldots\ldots\ldots + \frac{{}^{15}\!/_{16}\,l^3}{3} + \frac{{}^{15}\!/_{16}\,l^2}{2}\cdot\tfrac{1}{16}\right)$$

$$y_{bending} = 2\frac{65}{384}\frac{Fl^3}{EI}$$

Shear deflection

The shear deflection formula for a cantilevered rod with 16 concentrated loads can be derived with help of the existing formula for a rod with one concentrated load at the top:

$$y = \frac{Fl}{GA}$$

$$y_i = \Sigma y_i$$

$$y_{shear} = \frac{1}{GA}\left({}^{1}/_{16}\, l \cdot F + {}^{1}/_{16}\, l \cdot 2F + {}^{1}/_{16}\, l \cdot 3F + \ \dots\dots\dots + {}^{1}/_{16}\, l \cdot 16F\right)$$

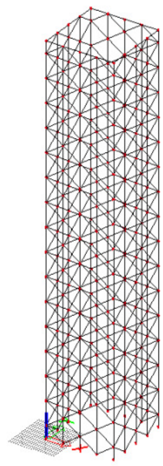
$$y_{shear} = 8\frac{1}{2}\frac{Fl}{GA}$$

NB: The bending deflection and shear deflection formulae for a cantilevered rod under a uniformly distributed load (q) could have been used as well, and can be found in the literature easily. However, the formulae from the literature would yield a different outcome and therefore different deviations, that are not present with the derived formulae for the deflection. Hence, the derived formulae guarantee a more accurate result.

9.3 Differences hand & ESA PT calculations

Now that the formulae for the EI and GA of the geometries are known, the hand results can be compared with the results of the discrete model computed by ESA PT. The deflection is split up into the bending deflection and the shear deflection to analyse the differences more exactly. All geometries are discussed at which the 2-dimensional construction and 3-dimensional construction are treated, respectively.
To increase the reliability of the comparison, the sections of the structural members have been varied. However, the maximum difference between the sections has been limited to a factor 4. In other words, the section of the largest member is 4 times larger than the section of the smallest member. The factor 4 is based on the fact that it is not likely that the sections will differ more, due to the connections that need to be in proportion. The two sections that have been adopted are;

CHS 457.0/40.0	$A = 5.24 \cdot 10^{-2}\ m^2$	O
CHS 273.0/16.0	$A = 1.29 \cdot 10^{-2}\ m^2$	o



Hand calculations
For a better understanding, one example calculation is given below for the 3-dimensional TFC variant “vertical columns”

h =	5.25 m
a =	7.5 m
d =	9.15 m
F =	2000 kN
l =	168 m
E =	$2.1 \cdot 10^{8}\ kN/m^2$
$A_{verticals}$ =	$5.24 \cdot 10^{-2}\ m^2$
$A_{diagonals}$ =	$5.24 \cdot 10^{-2}\ m^2$

$$EI = 44a^2EA_v = 44 \cdot 7.5^2 \cdot 2.1 \cdot 10^8 \cdot 5.24 \cdot 10^{-2} = 2.72 \cdot 10^{10}\ kNm^2$$

$$y_{EI} = 2\frac{65}{384}\frac{Fl^3}{EI} = 2\frac{65}{384} \cdot \frac{2000 \cdot 168^3}{2.72 \cdot 10^{10}} = 755 \cdot 10^{-3}\ m$$

$$GA = 2 \cdot \frac{4a^2hEA_d}{d^3} = 2 \cdot \frac{4 \cdot 7.5^2 \cdot 5.25 \cdot 2.1 \cdot 10^8 \cdot 5.24 \cdot 10^{-2}}{9.15^3} = 3.39 \cdot 10^7\ kN$$

$$y_{GA} = 8\frac{1}{2}\frac{Fl}{GA} = 8\frac{1}{2} \cdot \frac{2000 \cdot 168}{3.39 \cdot 10^7} = 84.3 \cdot 10^{-3}\ m$$

$$y_{total} = 755 \cdot 10^{-3} + 84.3 \cdot 10^{-3} = 840 \cdot 10^{-3}\ m = 840\ mm$$

ESA PT models
The ESA PT models have been built so they resemble the hand models as closely as possible: naturally, the geometries match those of the hand calculations exactly. And all connections are hinged, to eliminate bending and shear in the members.

To distinguish the shear deflection and bending deflection, two models have been built: one that is only supported at the base to compute the total deflection, as illustrated in Figure 9.20; and another identical one, where the nodes can only undergo a horizontal movement to compute the shear deflection, as illustrated in Figure 9.21. The bending deflection is obtained by subtracting the shear deflection from the total deflection.

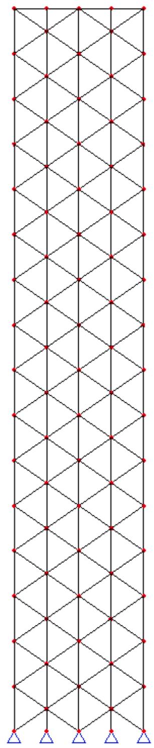


Figure 9.20: ESA model for total deflection

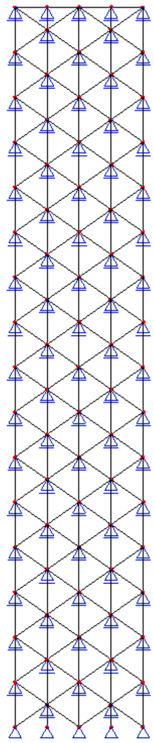


Figure 9.21: ESA model for shear deflection

TFC variant "vertical columns"

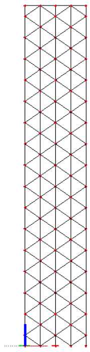


Figure 9.22: 2-dimensional variant

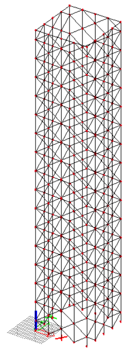


Figure 9.23: 3-dimensional variant

A (m^2)		δ	Hand (mm)	SCIA ESA PT (mm)	difference
A_v = 5.24e-2	O	$\delta_{bending}$	1662	1645	1.0%
A_d = 5.24e-2	O	δ_{shear}	84.30	84.4	-0.1%
		δ_{total}	1746	1729	1.0%
A_v = 1.29e-2	o	$\delta_{bending}$	6750	6683	1.0%
A_d = 1.29e-2	o	δ_{shear}	342.4	342.9	-0.1%
		δ_{total}	7093	7026	0.9%
A_v = 1.29e-2	o	$\delta_{bending}$	6750	6591	2.4%
A_d = 5.24e-2	O	δ_{shear}	84.30	84.5	-0.2%
		δ_{total}	6834	6676	2.3%
A_v = 5.24e-2	O	$\delta_{bending}$	1662	1661	0.0%
A_d = 1.29e-2	o	δ_{shear}	342.4	342.6	-0.1%
		δ_{total}	2004	2004	0.0%

Table 9.2: Differences 2-dimensional geometry variant "vertical columns".

$A\ (m^2)$		δ	Hand (mm)	SCIA ESA PT (mm)	difference
$A_v = 5.24e\text{-}2$	O	$\delta_{bending}$	755	748	1.0%
$A_d = 5.24e\text{-}2$	O	δ_{shear}	84.30	84.4	-0.1%
		δ_{total}	840	832.4	0.9%
$A_v = 1.29e\text{-}2$	o	$\delta_{bending}$	3068	3039	1.0%
$A_d = 1.29e\text{-}2$	o	δ_{shear}	342.4	342.9	-0.1%
		δ_{total}	3411	3381.9	0.8%
$A_v = 1.29e\text{-}2$	o	$\delta_{bending}$	3068	2930	4.5%
$A_d = 5.24e\text{-}2$	O	δ_{shear}	84.30	84.5	-0.2%
		δ_{total}	3153	3014.5	4.4%
$A_v = 5.24e\text{-}2$	O	$\delta_{bending}$	755	794	-5.1%
$A_d = 1.29e\text{-}2$	o	δ_{shear}	342.4	342.6	-0.1%
		δ_{total}	1098	1136.6	-3.5%

Table 9.3: Differences 3-dimensional geometry variant "vertical columns".

TFC variant "vertical columns"

Regarding the 2-dimensional variant; in case 1 and 2 where the verticals and diagonals have equal sections, the deflection only differs marginally where the computer calculation yields a higher value. Yet, this difference can be understood from the results from the 3rd and 4th comparison where the influence of the diagonals is becomes clear: In case the diagonals have a large section and the verticals a small section, the hand calculation gives a greater deflection which can be primarily attributed to the bending deflection. In other words, the bending stiffness calculated via the mechanics formula is smaller. This result can be ascribed to the influence of the diagonals that has been discarded for the bending stiffness: when the verticals shorten, the diagonals have to shorten as well and will provide a certain resistance, depending on their tensile rigidity, as illustrated in Figure 9.24.

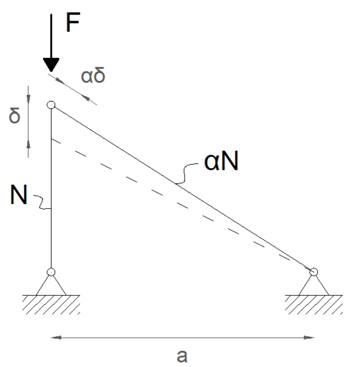


Figure 9.24: Contribution of the diagonal due to its shortening

This means that if the diagonals have relatively large sections compared to the verticals, the tensile rigidity and thus the bending stiffness of the 2-dimensional construction increases. In case the diagonals have a small section and the verticals a large section, the hand and computer calculation give nearly the same answer. This

can be explained by the inversing the abovementioned phenomenon: since the section of the diagonals is relatively small, their influence on the tensile rigidity and thus the bending stiffness can be neglected.
Looking at the comparison for the 3-dimensional construction, the differences for the 1st and 2nd case are also small. In the 3rd and 4th case, a similar behaviour is found as for the 2-dimensional construction: if the diagonals have a larger section than the verticals, their contribution in the bending stiffness becomes more apparent. However, the difference in percentages is larger for the 3-dimensional than for the 2-dimensional construction, 2.4% and 4.5% in the 3rd case, respectively. This prolific augmentation can be explained in view of the stress distribution in a tube construction compared to that in a trussed frame: in a tube construction, the tensile rigidity plays a relatively more important role due to the two "flanges" that are activated. Therefore the influence of the diagonals increases as well.

Summary: the approximation for the shear deflection of the geometry is nearly exact in all cases. However, the bending deflection varies when calculated by hand and computer. These differences are correlated with the influence of the diagonals on the bending stiffness which is neglected in the mechanics formulae.

TFC variant “diagonal columns”

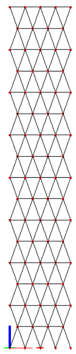


Figure 9.25: 2-dimensional variant

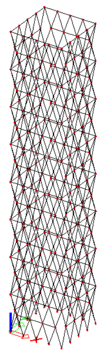


Figure 9.26: 3-dimensional variant

A (m^2)		δ	Hand (mm)	SCIA ESA PT (mm)	difference
A_h = 5.24e-2	O	$\delta_{bending}$	2141	2102	1.8%
A_d = 5.24e-2	O	δ_{shear}	152.3	153	-0.5%
		δ_{total}	2293	2255	1.7%
A_h = 1.29e-2	o	$\delta_{bending}$	8695	8539	1.8%
A_d = 1.29e-2	o	δ_{shear}	618.5	619.3	-0.1%
		δ_{total}	9314	9158	1.7%
A_h = 1.29e-2	o	$\delta_{bending}$	2604	2393	8.1%
A_d = 5.24e-2	O	δ_{shear}	152.3	153	-0.5%
		δ_{total}	2756	2546	7.6%
A_h = 5.24e-2	O	$\delta_{bending}$	8232	8211	0.3%
A_d = 1.29e-2	o	δ_{shear}	618.5	618.7	0.0%
		δ_{total}	8850	8830	0.2%

Table 9.4: Differences 2-dimensional geometry variant “diagonal columns”.

A (m^2)		δ	Hand (mm)	SCIA ESA PT (mm)	difference
A_h = 5.24e-2	O	$\delta_{bending}$	510	516.3	-1.3%
A_d = 5.24e-2	O	δ_{shear}	152.3	152.5	-0.2%
		δ_{total}	662	668.8	-1.0%
A_h = 1.29e-2	O	$\delta_{bending}$	2070	2099	-1.4%
A_d = 1.29e-2	O	δ_{shear}	618.5	619.3	-0.1%
		δ_{total}	2689	2718	-1.1%
A_h = 1.29e-2	O	$\delta_{bending}$	620	569.9	8.1%
A_d = 5.24e-2	O	δ_{shear}	152.3	153	-0.5%
		δ_{total}	772	722.9	6.4%
A_h = 5.24e-2	O	$\delta_{bending}$	1960	2031	-3.6%
A_d = 1.29e-2	O	δ_{shear}	618.5	618.7	0.0%
		δ_{total}	2578	2650	-2.8%

Table 9.5: Differences 3-dimensional geometry variant "diagonal columns".

Regarding the 2-dimensional variant; in the 1st and 2nd case the differences are relatively small. Yet in the 3rd case the difference is substantial, which can be subscribed to the difference in bending deflection: the bending deflection calculated by ESA PT is smaller, meaning a larger bending stiffness. This effect can be explained by the fact that horizontal displacements at the base of the computer model are restricted. Indirectly, this is translated into an increased bending stiffness of the bottom part; the bottom part is the most important to limit the deflection at the top, hence the relatively large effect of 8.1%. This phenomenon is apparent in the 3rd case where the horizontals have a small section and the diagonals a large section. In the 4th case, where the sections are inversed, the differences are marginal: the horizontal displacements – and thus of the deforming of the rhomboids – are restricted by the relatively large sections of the horizontals.

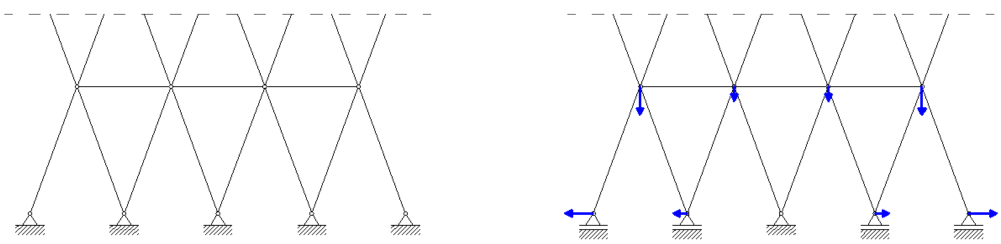


Figure 9.27: Restricted versus free movement at the base

Overall, the deflection calculated by computer is smaller for the 2-dimensional construction in all 4 cases due to a larger bending stiffness. The larger bending stiffness can be explained by the fact that the base of computer model has a larger width, whereas the hand calculation is based on the width between the lines of action, as illustrated in Figure 9.28 and Figure 9.29. If the model was infinitely tall, the results from the hand and the computer calculations would converge.

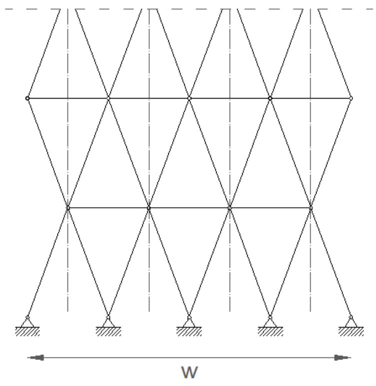


Figure 9.28: Effective width at the base of the computer model

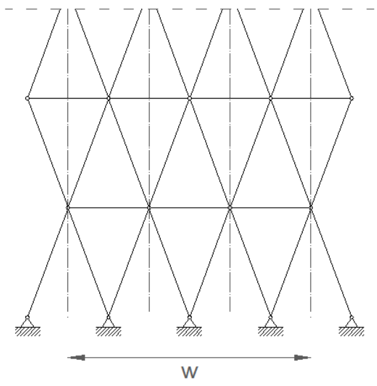


Figure 9.29: Effective width at the base of the hand calculations

Looking at the comparison for the 3-dimensional construction, the differences for the 1st and 2nd case are also small. Similarly, in the 3rd case the effect of the restricted movement for the computer model also results in a smaller deflection at the top. The effect of the greater width of the base and hence a greater bending stiffness is marginalized due to the floor plan with a square perimeter.

Additionally, the conclusion that can be drawn from the comparison is that the horizontal members every sixth storey – where the construction is at the smallest – do not actually contribute to the shear or bending stiffness: they have not been included in the hand calculation, though they are present in the ESA models. To verify their contribution, the secondary horizontal in the ESA models have been left out and the deflections have been computed again.

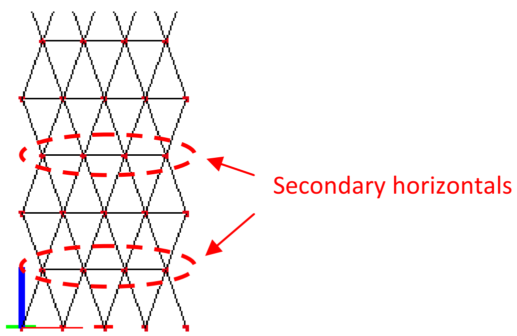


Figure 9.30: Omitted secondary horizontals in geometry variant "diagonal columns"

A (m^2)		δ	Hand (mm)	SCIA ESA PT (mm) (without sec. hor.)	difference	SCIA ESA PT (mm)
A_h = 5.24e-2	O	$\delta_{bending}$	2141	2107	1.6%	2102
A_d = 5.24e-2	O	δ_{shear}	152.3	152.5	-0.2%	153
		δ_{total}	2293	2260	1.5%	2255

A (m^2)		δ	Hand (mm)	SCIA ESA PT (mm) (without sec. hor.)	difference	SCIA ESA PT (mm)
A_h = 5.24e-2	O	$\delta_{bending}$	510	520.8	-2.2%	516.3
A_d = 5.24e-2	O	δ_{shear}	152.3	152.5	-0.2%	152.5
		δ_{total}	662	673.3	-1.7%	668.8

The computer results calculated with the new models without secondary horizontals are compared with the original hand calculations and the differences are shown. This has been done for the 2-dimensional and the 3-dimensional construction without any variation in the sections. The right column shows the initial results from the ESA models with the secondary horizontals still present. The comparison proves that the secondary horizontals do not contribute to the rigidity and can be left out.

Summary: the approximation for the shear deflection of the geometry is nearly exact in all cases. However, the bending deflection varies when calculated by hand and computer. These differences are correlated with the influence of the restricted movements and a different effective width at the base of the model on the bending stiffness. Furthermore, the comparison showed that the secondary horizontals can be omitted.

TFC variant "diagonal & vertical columns"

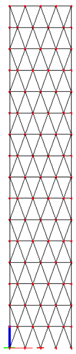


Figure 9.31: 2-dimensional variant

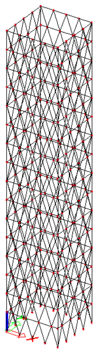


Figure 9.32: 3-dimensional variant

A (m^2)		δ	Hand (mm)	SCIA ESA PT (mm)	difference
A_h = 5.24e-2	O	$\delta_{bending}$	1054	1039	1.4%
A_d = 5.24e-2	O	δ_{shear}	152.3	153	-0.5%
A_v = 5.24e-2	O	δ_{total}	1206	1192	1.2%
A_h = 1.29e-2	o	$\delta_{bending}$	4282	4223	1.4%
A_d = 1.29e-2	o	δ_{shear}	618.5	619.3	-0.1%
A_v = 1.29e-2	o	δ_{total}	4901	4842	1.2%
A_h = 5.24e-2	O	$\delta_{bending}$	1707	1679	1.7%
A_d = 5.24e-2	O	δ_{shear}	152.3	152.5	-0.2%
A_v = 1.29e-2	o	δ_{total}	1860	1832	1.5%
A_h = 1.29e-2	o	$\delta_{bending}$	1156	1104	4.5%
A_d = 5.24e-2	O	δ_{shear}	152.3	153	-0.5%
A_v = 5.24e-2	O	δ_{total}	1308	1257	3.9%
A_h = 5.24e-2	O	$\delta_{bending}$	1659	1645	0.8%
A_d = 1.29e-2	o	δ_{shear}	618.5	618.7	0.0%
A_v = 5.24e-2	O	δ_{total}	2277	2264	0.6%
A_h = 5.24e-2	O	$\delta_{bending}$	4167	4142	0.6%
A_d = 1.29e-2	o	δ_{shear}	618.5	618.7	0.0%
A_v = 1.29e-2	o	δ_{total}	4785	4761	0.5%

A_h = 1.29e-2	o	$\delta_{bending}$	1990	1858	6.6%
A_d = 5.24e-2	O	δ_{shear}	152.3	153	-0.5%
A_v = 1.29e-2	o	δ_{total}	2142	2011	6.1%
A_h = 1.29e-2	o	$\delta_{bending}$	1677	1658	1.1%
A_d = 1.29e-2	o	δ_{shear}	618.5	619.3	-0.1%
A_v = 5.24e-2	O	δ_{total}	2295	2277	0.8%

Table 9.6: Differences 2-dimensional geometry variant "diagonal & vertical columns".

A (m^2)		δ	Hand (mm)	SCIA ESA PT (mm)	difference
A_h = 5.24e-2	O	$\delta_{bending}$	409	412.5	-0.8%
A_d = 5.24e-2	O	δ_{shear}	152.3	152.5	-0.2%
A_v = 5.24e-2	O	δ_{total}	562	565	-0.6%
A_h = 1.29e-2	o	$\delta_{bending}$	1662	1677	-0.9%
A_d = 1.29e-2	o	δ_{shear}	618.5	619.3	-0.1%
A_v = 1.29e-2	o	δ_{total}	2281	2296	-0.7%
A_h = 5.24e-2	O	$\delta_{bending}$	481	468.5	2.5%
A_d = 5.24e-2	O	δ_{shear}	152.3	152.5	-0.2%
A_v = 1.29e-2	o	δ_{total}	633	621	1.9%
A_h = 1.29e-2	o	$\delta_{bending}$	477	445.8	6.6%
A_d = 5.24e-2	O	δ_{shear}	152.3	153	-0.5%
A_v = 5.24e-2	O	δ_{total}	630	598.8	4.9%
A_h = 5.24e-2	O	$\delta_{bending}$	1008	1021	-1.2%
A_d = 1.29e-2	o	δ_{shear}	618.5	618.7	0.0%
A_v = 5.24e-2	O	δ_{total}	1627	1640	-0.8%
A_h = 5.24e-2	O	$\delta_{bending}$	1590	1633	-2.7%
A_d = 1.29e-2	o	δ_{shear}	618.5	618.7	0.0%
A_v = 1.29e-2	o	δ_{total}	2209	2252	-1.9%
A_h = 1.29e-2	o	$\delta_{bending}$	578	533.4	7.6%
A_d = 5.24e-2	O	δ_{shear}	152.3	153	-0.5%
A_v = 1.29e-2	o	δ_{total}	730	686.4	6.0%
A_h = 1.29e-2	o	$\delta_{bending}$	1037	1039	-0.2%
A_d = 1.29e-2	o	δ_{shear}	618.5	619.3	-0.1%
A_v = 5.24e-2	O	δ_{total}	1655	1658	-0.2%

Table 9.7: Differences 3-dimensional geometry variant "diagonal & vertical columns".

Regarding the 2-dimensional variant; in all cases the differences are marginal except for the 4th and 7th case. In both cases, the horizontals have small sections and the diagonals have large sections – while the vertical section is large and small respectively. The explanation can be found in the same effect as for the geometry variant “diagonal columns”, where the movements at the base of the computer model are restricted yielding a smaller deflection. However the difference is less significant for this geometry variant since the vertical columns – which are not affected by the restricted movement – also contribute in the bending stiffness.
Regarding the 3-dimensional variant; the differences are marginal except for the 4th and 7th case – just as for the 2-dimensional variant. The differences can be explained in the same way as for the 2-dimensional construction.

Additionally, the conclusion that can be drawn from the comparison is that the horizontal members every sixth storey – where the construction is at the smallest – do not actually contribute to the shear or bending stiffness: they have not been included in the hand calculation, though they are present in the ESA models. To verify their contribution, the secondary horizontal in the ESA models have been left out and the deflections have been computed again.

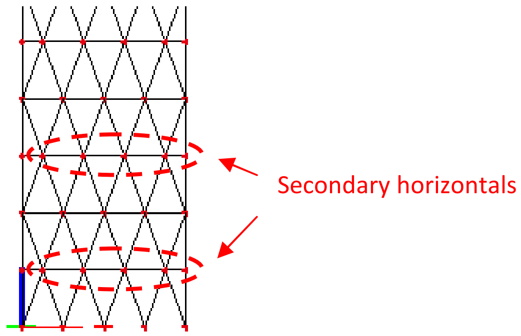


Figure 9.33: Omitted secondary horizontals in geometry variant “diagonal & vertical columns”

A (m^2)		δ	Hand (mm)	SCIA ESA PT (mm) (without sec. hor.)	difference	SCIA ESA PT (mm)
A_h = 5.24e-2	O	$\delta_{bending}$	1054	1040	1.3%	1039
A_d = 5.24e-2	O	δ_{shear}	152.3	152.5	-0.2%	153
A_v = 5.24e-2	O	δ_{total}	1206	1193	1.2%	1192

A (m^2)		δ	Hand (mm)	SCIA ESA PT (mm) (without sec. hor.)	difference	SCIA ESA PT (mm)
A_h = 5.24e-2	O	$\delta_{bending}$	409	415.3	-1.5%	412.5
A_d = 5.24e-2	O	δ_{shear}	152.3	152.5	-0.2%	152.5
A_v = 5.24e-2	O	δ_{total}	562	567.8	-1.1%	565

The computer results calculated with the new models without secondary horizontals are compared with the original hand calculations and the differences are shown. This has been done for the 2-dimensional and the 3-dimensional construction without any variation in the sections. The right column shows the initial results from the ESA models with the secondary horizontals still present. The comparison proves that the secondary horizontals do not contribute to the rigidity and can be left out.

Summary: the approximation for the shear deflection of the geometry is nearly exact in all cases. However, the bending deflection varies when calculated by hand and computer. These differences are correlated with the influence of the restricted movements and a different effective width at the base of the model on the bending stiffness. Furthermore, the comparison showed that the secondary horizontals can be omitted.

General conclusion

The hand calculations, based on the mechanics formulae, give a good approximation of the reality. The calculated shear deflection is an accurate approach while the bending deflection still shows some deviations. However, the order of these deviations is minor and, more important, can be estimated based on the comparison with the computer results for different sections. Therefore, the mechanics formulae seem reliable and a good base for the further design calculations.

9.4 Strength of Geometry Variants

The strength verifications have been performed below: first the member forces are calculated with ESA PT, then the capacity of the sections – based on the deflection – is calculated and lastly the unity checks are carried out. This same order is used for all three geometry variants.

“Vertical columns”

The forces in the vertical columns are due to the bending moment on the construction: the sections in the flanges take up the largest force because they are furthest from the centre. It needs to be remarked that there is no ideal linear distribution of the forces over the sections in the flanges: a phenomenon occurs that is analogous to shear lag, yet due to normal forces: the forces in the flanges are higher towards the corners and lower in the middle.

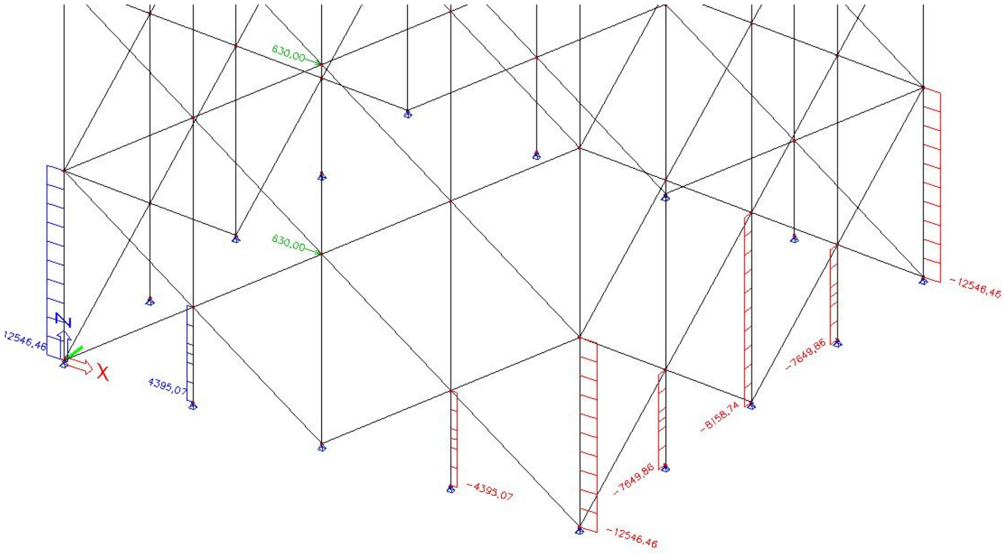


Figure 9.34: Forces in the vertical members.

The forces in the web-diagonals are mainly due to shear in the trussed tube structure. The forces in the diagonal members in the flanges are significantly smaller and due to the bending moment.

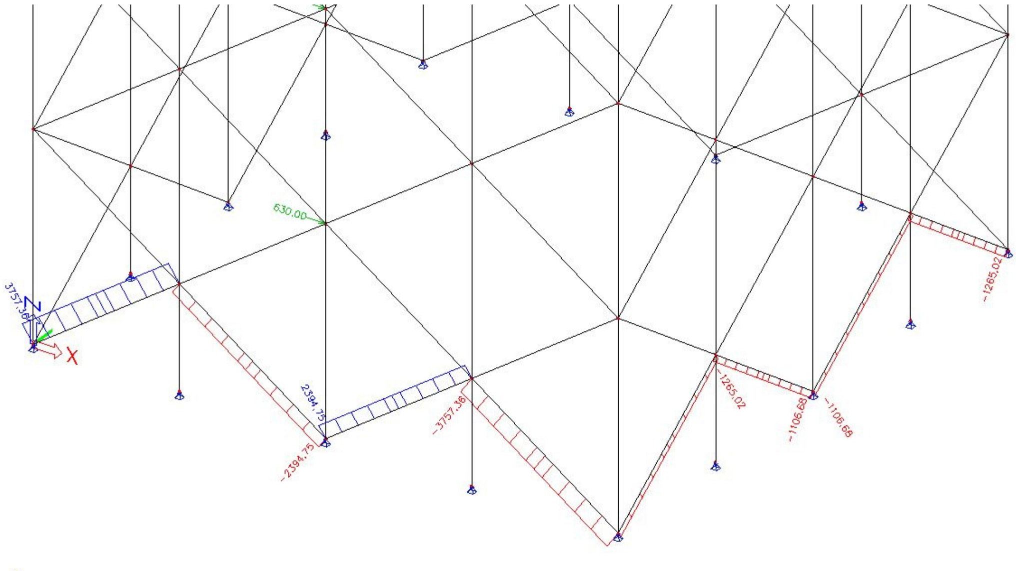


Figure 9.35: Forces in the diagonal members.

The maximal forces in the structural members are summarized in the table below

	F_{max} [kN]
Verticals	12546
Diagonals	3757

Table 9.8: Maximum forces in members of geometry "vertical columns".

Capacity verticals

A calculation is made for the compressive strength of a vertical member. The data are as follows:

Vertical		Diagonal	
section =	CHS 508x40 (S355)	section =	CHS 508x16 (S355)
l_{buc} =	10500 mm	l_{buc} =	9155 mm
I =	$1.662 \cdot 10^9$ mm^4	I =	$7.491 \cdot 10^8$ mm^4
E =	$2.1 \cdot 10^5$ N/mm^2	E =	$2.1 \cdot 10^5$ N/mm^2
N_{pl} =	20874 kN	N_{pl} =	8769 kN
curve =	a	curve =	A

Regarding the vertical members:

$$F_e = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 1.662 \cdot 10^9 \cdot 10^{-3}}{(10500)^2} = 31244\ kN$$

$$\lambda = \sqrt{\frac{20874}{31244}} = 0.82$$

$$\omega_{buc} = 0.78$$

$$N_{compressive;d} = 0.78 \cdot 20874 = 16282\ kN$$

Regarding the diagonal members

$$F_e = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 7.491 \cdot 10^8 \cdot 10^{-3}}{(9155)^2} = 18524\ kN$$

$$\lambda = \sqrt{\frac{8769}{18524}} = 0.69$$

$$\omega_{buc} = 0.86$$

$$N_{compressive;d} = 0.86 \cdot 8769 = 7541\ kN$$

Unity checks

Vertical members: $\frac{12546}{16282} = 0.77$

Diagonal members: $\frac{3757}{7541} = 0.50$

“Diagonal columns”

The forces in the diagonal columns are cause by the bending moment on the construction: the sections in the flanges take up the largest force because they are furthest from the centre. It needs to be remarked that there is no ideal linear distribution of the forces over the sections in the flanges: a phenomenon occurs that is analogous to shear lag, yet due to normal forces: the forces in the flanges are higher towards the corners and lower in the middle.
The diagonal columns located in the webs of the tube structure sustain additional forces due to shear. Therefore, the maximum force occurs in the diagonals in the webs, closest to the flange.

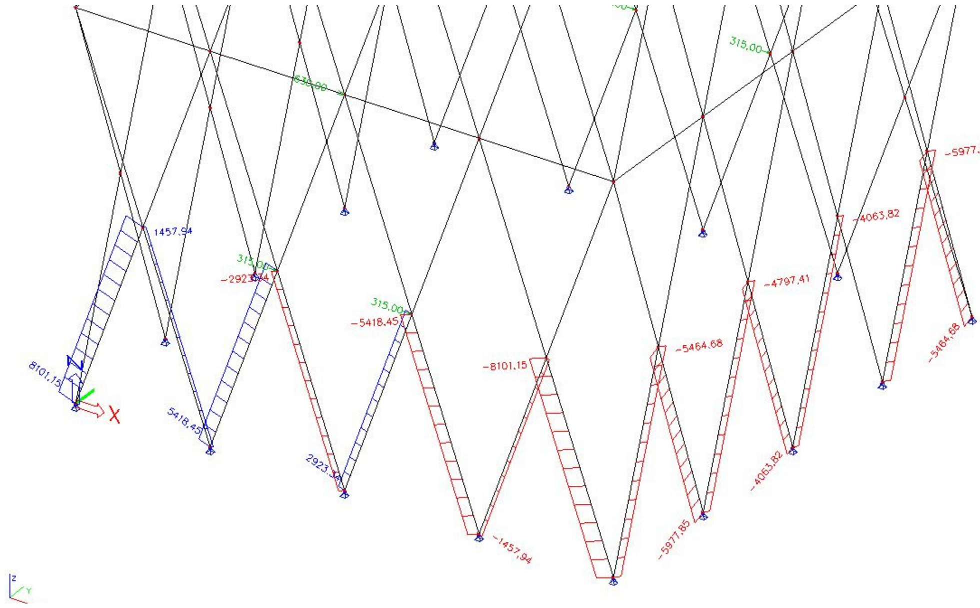


Figure 9.36: Forces in the diagonal members.

The forces in the horizontal members occur due to the lateral thrust of the diagonals and are at their largest in the webs.

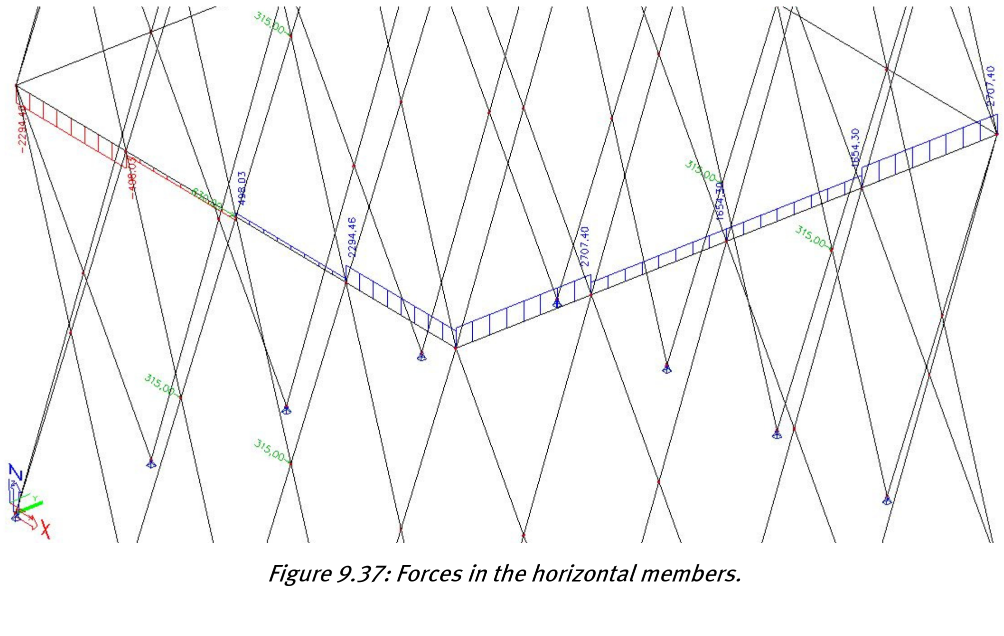


Figure 9.37: Forces in the horizontal members.

The maximal forces in the structural members are shown in REF and REF and are summarized in the table below

	F_{max} [kN]
Diagonals	8101
Horizontals	2707

Table 9.9: Maximum forces in members of geometry "diagonal columns".

Capacity verticals
A calculation is made for the compressive strength of a vertical member. The data are as follows:

Diagonal		Horizontal	
section =	CHS 457x40 (S355)	section =	CHS 457x10 (S355)
l_{buc} =	11150 mm	l_{buc} =	7500 mm
I =	$1.149 \cdot 10^9$ mm^4	I =	$3.509 \cdot 10^8$ mm^4
E =	$2.1 \cdot 10^5$ N/mm^2	E =	$2.1 \cdot 10^5$ N/mm^2
N_{pl} =	18602 kN	N_{pl} =	4970 kN
curve =	a	curve =	a

Regarding the diagonal members:

$$F_e = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 1.149 \cdot 10^9 \cdot 10^{-3}}{(11150)^2} = 19155\ kN$$

$$\lambda = \sqrt{\frac{18602}{19155}} = 0.99$$

$$\omega_{buc} = 0.67$$

$$N_{compressive;d} = 0.67 \cdot 18602 = 12463\ kN$$

Regarding the horizontal members

$$F_e = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 3.509 \cdot 10^8 \cdot 10^{-3}}{(7500)^2} = 12929\ kN$$

$$\lambda = \sqrt{\frac{4970}{12929}} = 0.62$$

$$\omega_{buc} = 0.88$$

$$N_{compressive;d} = 0.88 \cdot 4970 = 4374\ kN$$

Unity checks

Diagonal members: $\frac{8101}{12463} = 0.65$

Horizontal members: $\frac{2707}{4374} = 0.62$

“Diagonal & vertical columns”

The forces in the diagonal columns are cause by the bending moment on the construction: the sections in the flanges take up the largest force because they are furthest from the centre. It needs to be remarked that there is no ideal linear distribution of the forces over the sections in the flanges: a phenomenon occurs that is analogous to shear lag, yet due to normal forces: the forces in the flanges are higher towards the corners and lower in the middle.
The diagonal columns located in the webs of the tube structure sustain additional forces due to shear. Therefore, the maximum force occurs in the diagonals in the webs, closest to the flange.

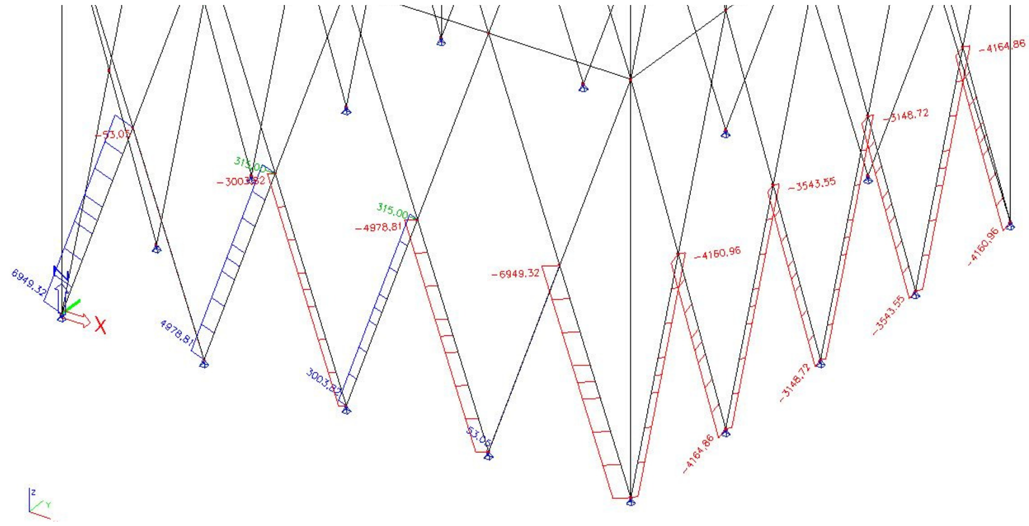


Figure 9.38: Forces in the diagonal members.

The forces in the vertical columns are merely caused by the bending moment on the construction.

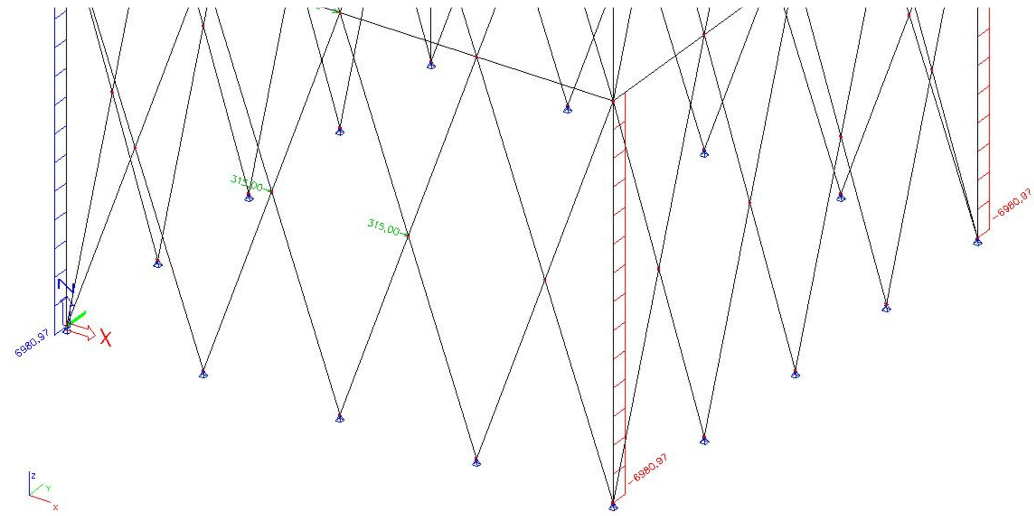


Figure 9.39: Forces in the vertical members.

The forces in the horizontal members occur due to the lateral thrust of the diagonals and are at their largest in the webs.

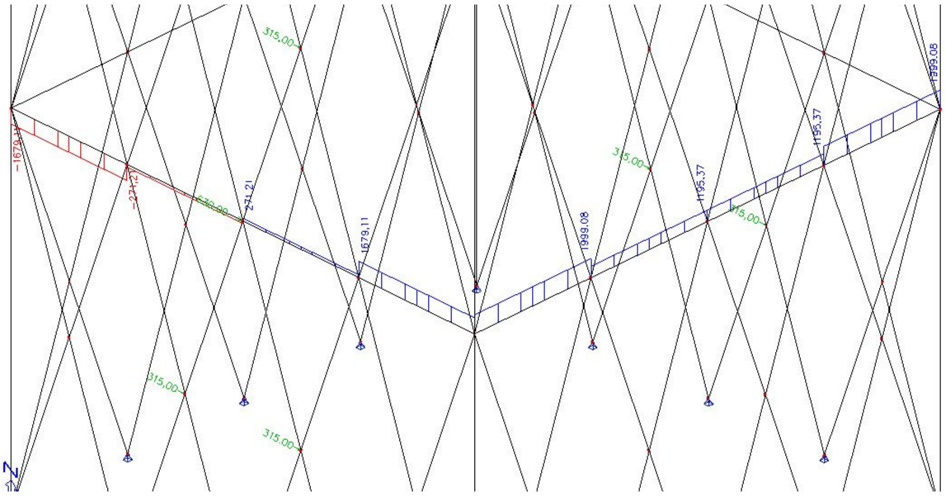


Figure 9.40: Forces in the horizontal members.

The maximal forces in the structural members are shown in REF and REF and are summarized in the table below

	F_{max} [kN]
Diagonals	6949
Verticals	6981
Horizontals	1999

Table 9.10: Maximum forces in members of geometry "diagonal & vertical columns".

Capacity verticals

A calculation is made for the compressive strength of a vertical member. The data are as follows:

Diagonal		Vertical		Horizontal	
section =	CHS 457x40 (S355)	section =	CHS 508x40 (S355)	section =	CHS 457x10 (S355
l_{buc} =	11150 mm	l_{buc} =	10500 mm	l_{buc} =	7500 mm
I =	$1.149 \cdot 10^9$ mm^4	I =	$1.662 \cdot 10^9$ mm^4	I =	$3.509 \cdot 10^8$ mm^4
E =	$2.1 \cdot 10^5$ N/mm^2	E =	$2.1 \cdot 10^5$ N/mm^2	E =	$2.1 \cdot 10^5$ N/mm^2
N_{pl} =	18602 kN	N_{pl} =	20874 kN	N_{pl} =	4970 kN
curve =	a	curve =	a	curve =	a

Regarding the diagonal members:

$$F_e = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 1.149 \cdot 10^9 \cdot 10^{-3}}{(11150)^2} = 19155\ kN$$

$$\lambda = \sqrt{\frac{18602}{19155}} = 0.99$$

$$\omega_{buc} = 0.67$$

$$N_{compressive;d} = 0.67 \cdot 18602 = 12463 \; kN$$

Regarding the vertical members

$$F_e = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 1.662 \cdot 10^9 \cdot 10^{-3}}{(10500)^2} = 31244 \; kN$$

$$\lambda = \sqrt{\frac{20874}{31244}} = 0.82$$

$$\omega_{buc} = 0.78$$

$$N_{compressive;d} = 0.78 \cdot 20874 = 16282 \; kN$$

Regarding the horizontal members

$$F_e = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 3.509 \cdot 10^8 \cdot 10^{-3}}{(7500)^2} = 12929 \; kN$$

$$\lambda = \sqrt{\frac{4970}{12929}} = 0.62$$

$$\omega_{buc} = 0.88$$

$$N_{compressive;d} = 0.88 \cdot 4970 = 4374 \; kN$$

Unity checks

Diagonal members: $\frac{6949}{12463} = 0.56$

Vertical members: $\frac{6981}{16282} = 0.43$

Horizontal members: $\frac{1999}{4374} = 0.46$

9.5 Weight Calculation Optimised Geometries

The weight calculations of the optimised geometries are performed and given below.

	Section	weight [kg/m]	length [m]	amount	weight [kg]
verticals	CHS 508x50	564	9.9	256	1429402
diagonals	CHS 508x16	194	9.41	512	934676
total					2364078

Table 9.11: Construction weight geometry "vertical columns".

	Section	weight [kg/m]	length [m]	amount	weight [kg]
horizontals	CHS 457x10	110	8	128	112640
diagonals	CHS 457x40	411	10.68	512	2247414
total					2360054

Table 9.12: Construction weight geometry "diagonal columns".

	section	weight [kg/m]	length [m]	amount	weight [kg]
verticals	CHS 508x40	462	9.9	64	292723
horizontals	CHS 457x10	110	8	128	112640
diagonals	CHS 457x40	411	10.68	512	2247414
total					2652777

Table 9.13: Construction weight geometry "diagonal & vertical columns".

10 Appendix 4: Structural Nodes

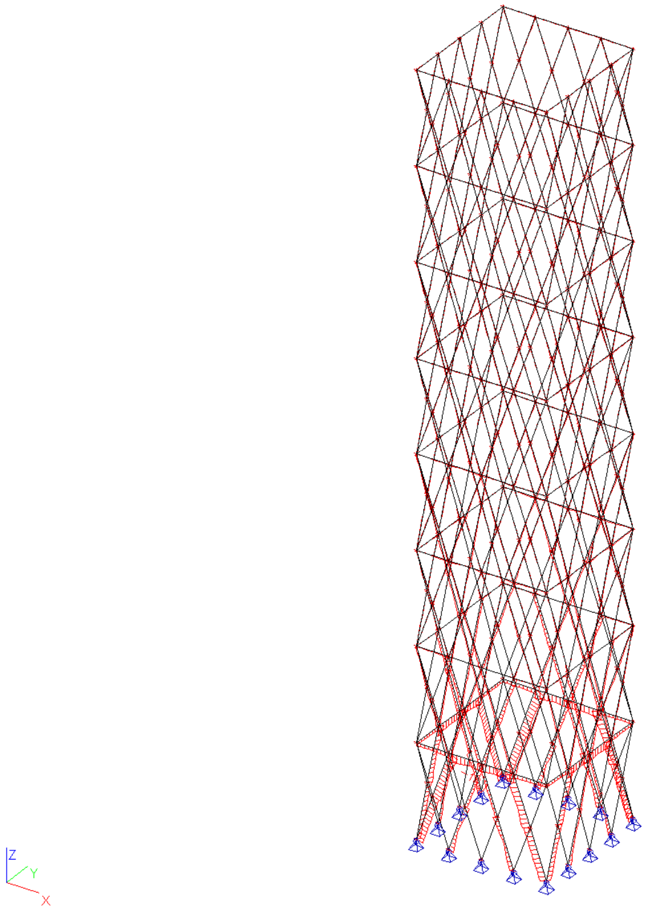
10.1 Behaviour under temperature loads

The following models are built to study the TFC behaviour under temperature loads;

- Temperature load on 4 façades: all 4 sides of the TFC are exposed to a temperature load of 25˚C – the model is only supported at the base with hinges.
- Temperature load on 1 façade: only 1 side of the TFC is exposed to a temperature load of 25˚C – the model is only supported at the base with hinges.
- Temperature load on 2 façades: only 2 sides of the TFC are exposed to a temperature load of 25˚C – the model is only supported at the base with hinges.
- Rigid connections, 3 directions: the entire TFC is exposed to a temperature load of 25˚C – the displacement of all nodes is prevented in all 3 directions.
- Rigid connections, 2 directions: the entire TFC is exposed to a temperature load of 25˚C – the displacement of all nodes is prevented in 2 directions (vertically expansion possible).
- Rigid connections partially, 2 directions: the entire TFC is exposed to a temperature load of 25˚C – the displacement of the nodes in the middle of the facades is prevented in 2 directions (vertically expansion possible).

Temperature load on 4 facades

1.Image



2.Thermal load on beam

Name	Member	Load case	Delta	Pos x_1	Pos x_2	Coor	Orig	Distribution
Temp1	all	temp	25	0,000	1,000	Rela	From start	Constant

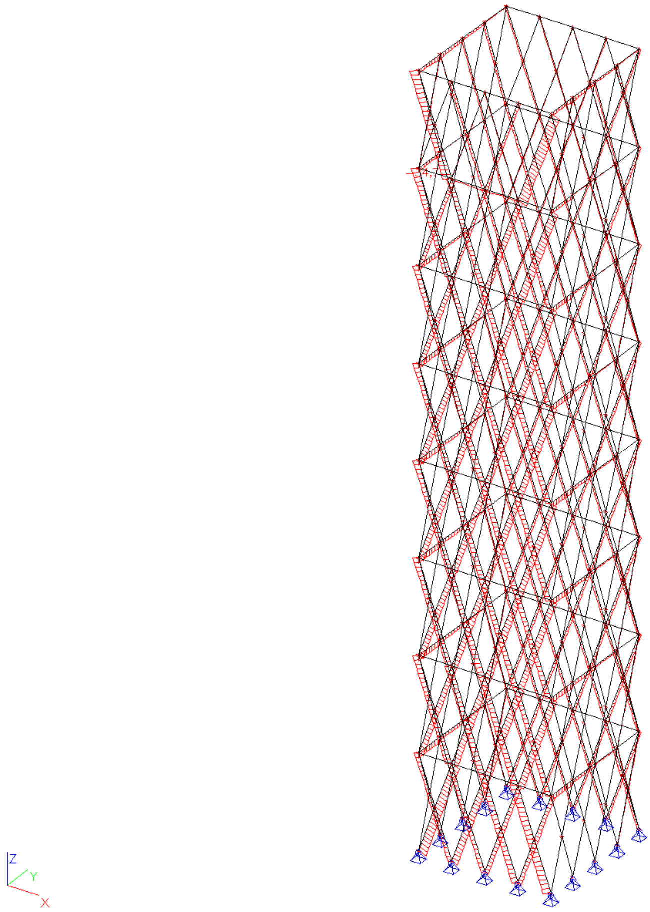
3.Stress

Linear calculation, Extreme : Global
Selection : All
Load cases : temp

Member	Case	dx [m]	Normal - [MPa]	Normal + [MPa]	Shear [MPa]	von Mises [MPa]	Fatigue [MPa]	Kappa [1]
S25	temp	10,512	**-10,1**	0,0	0,1	**10,1**	0,0	0,00
S14	temp	0,876	**0,0**	7,2	0,1	7,2	0,0	0,00
S13	temp	0,000	-7,5	**0,0**	0,0	7,5	0,0	0,00
S16	temp	0,000	0,0	**8,0**	0,1	8,0	0,0	0,00
S174	temp	0,000	-0,1	0,2	**0,0**	0,2	0,0	0,00
S56	temp	0,000	-0,1	4,0	**0,1**	4,0	0,0	0,00
S4	temp	5,256	0,0	0,0	0,0	**0,0**	0,0	0,00
S1	temp	0,000	-0,1	0,1	0,0	0,1	**0,0**	**0,00**

Temperature load on 1 facade

1.Image



2.Thermal load on beam

Name	Member	Load case	Delta	Pos x_1	Pos x_2	Coor	Orig	Distribution
Temp1	1 façade	temp	25	0,000	1,000	Rela	From start	Constant

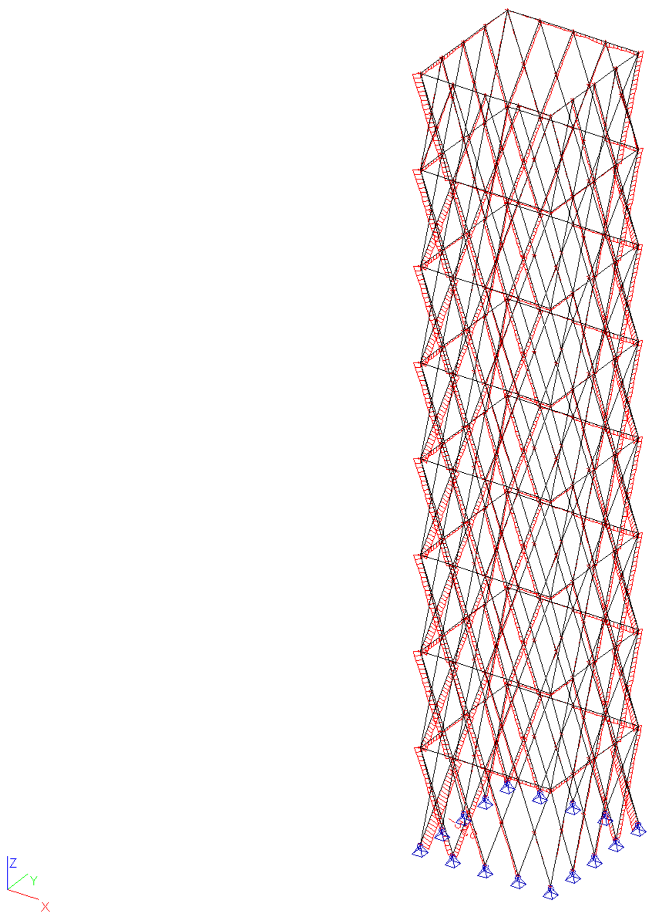
3.Stress

Linear calculation, Extreme : Global
Selection : All
Load cases : temp

Member	Case	dx [m]	Normal - [MPa]	Normal + [MPa]	Shear [MPa]	von Mises [MPa]	Fatigue [MPa]	Kappa [1]
S175	temp	10,512	**-34,7**	0,0	0,2	34,7	0,0	0,00
S2	temp	0,000	**0,0**	11,2	0,2	11,2	0,0	0,00
S1	temp	0,000	-34,3	**0,0**	0,3	34,3	**0,0**	**0,00**
S537	temp	0,000	0,0	**35,4**	0,2	**35,4**	0,0	0,00
S326	temp	0,000	-5,0	0,0	**0,0**	5,0	0,0	0,00
S380	temp	0,000	-17,8	0,0	**1,0**	17,8	0,0	0,00
S188	temp	6,132	-0,1	0,0	0,1	**0,1**	0,0	0,00

Temperature load on 2 facades

1.Image



2.Thermal load on beam

Name	Member	Load case	Delta	Pos x_1	Pos x_2	Coor	Orig	Distribution
Temp1	2 facades	temp	25	0,000	1,000	Rela	From start	Constant

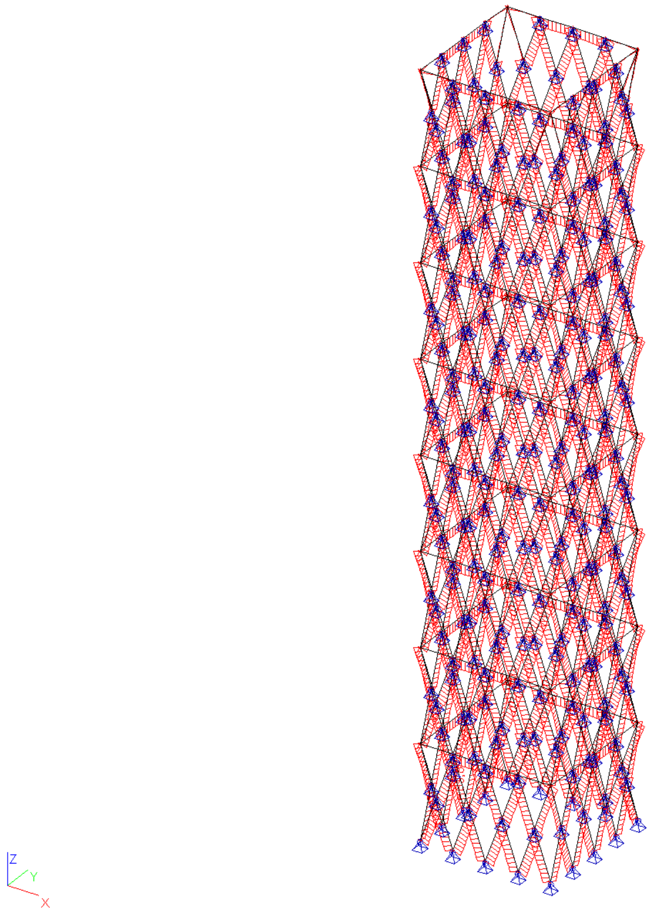
3.Stress

Linear calculation, Extreme : Global
Selection : All
Load cases : temp

Member	Case	dx [m]	Normal - [MPa]	Normal + [MPa]	Shear [MPa]	von Mises [MPa]	Fatigue [MPa]	Kappa [1]
S13	temp	10,512	**-32,0**	0,0	0,2	**32,0**	0,0	0,00
S3	temp	5,256	**0,0**	2,2	0,1	2,2	0,0	0,00
S1	temp	0,000	-28,1	**0,0**	0,2	28,1	**0,0**	**0,00**
S192	temp	0,000	0,0	**28,9**	0,3	28,9	0,0	0,00
S119	temp	0,000	-11,3	0,0	**0,0**	11,3	0,0	0,00
S9	temp	0,000	0,0	12,0	**0,9**	12,0	0,0	0,00
S8	temp	4,380	0,0	0,1	0,1	**0,2**	0,0	0,00

Rigid connections, 3 directions

1.Image



2.Thermal load on beam

Name	Member	Load case	Delta	Pos x_1	Pos x_2	Coor	Orig	Distribution
Temp1	all	temp	25	0,000	1,000	Rela	From start	Constant

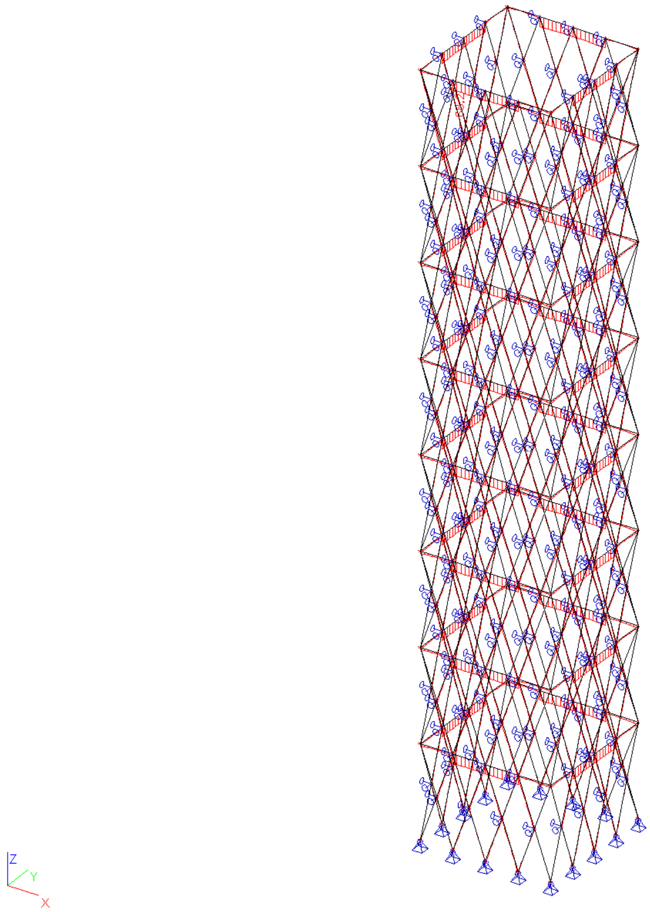
3.Stress

Linear calculation, Extreme : Global
Selection : All
Load cases : temp

Member	Case	dx [m]	Normal - [MPa]	Normal + [MPa]	Shear [MPa]	von Mises [MPa]	Fatigue [MPa]	Kappa [1]
S33	temp	0,000	**-69,8**	0,0	0,3	**69,8**	0,0	0,00
S32	temp	0,000	**0,0**	**41,7**	0,7	41,7	0,0	0,00
S2	temp	0,000	-66,3	**0,0**	0,2	66,3	0,0	0,00
S16	temp	0,000	-63,1	0,0	**0,0**	63,1	0,0	0,00
S9	temp	0,000	-14,4	13,8	**0,8**	14,4	0,0	0,00
S1	temp	5,256	-0,9	0,4	0,6	**1,3**	0,0	0,00
S1	temp	0,000	-7,9	7,4	0,6	7,9	**0,0**	**0,00**

Rigid connections, 2 directions

1.Image



2.Thermal load on beam

Name	Member	Load case	Delta	Pos x_1	Pos x_2	Coor	Orig	Distribution
Temp1	all	temp	25	0,000	1,000	Rela	From start	Constant

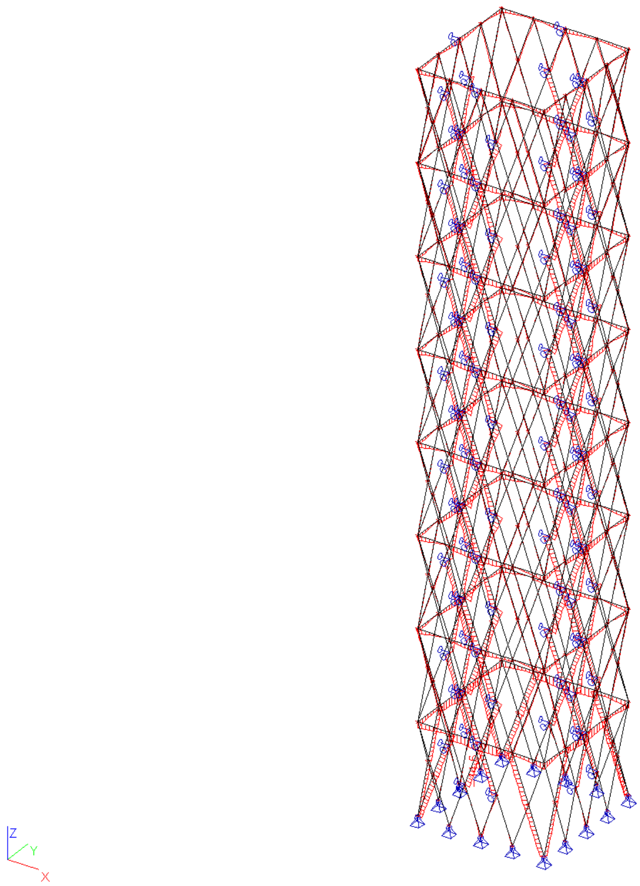
3.Stress

Linear calculation, Extreme : Global
Selection : All
Load cases : temp

Member	Case	dx [m]	Normal - [MPa]	Normal + [MPa]	Shear [MPa]	von Mises [MPa]	Fatigue [MPa]	Kappa [1]
S10	temp	0,000	**-68,2**	0,0	0,3	**68,2**	0,0	0,00
S2	temp	1,752	**0,0**	4,9	0,1	4,9	0,0	0,00
S3	temp	2,628	-4,9	**0,0**	0,3	4,9	0,0	0,00
S105	temp	0,000	0,0	**14,2**	0,4	14,2	0,0	0,00
S25	temp	0,000	-6,8	0,0	**0,1**	6,8	0,0	0,00
S9	temp	0,000	-9,6	9,1	**0,8**	9,7	0,0	0,00
S4	temp	5,256	-0,2	0,2	0,1	**0,3**	0,0	0,00
S1	temp	0,000	-1,9	2,0	0,5	2,1	**0,0**	**0,00**

Rigid connections partially, 2 directions

1.Image



2.Thermal load on beam

Name	Member	Load case	Delta	Pos x_1	Pos x_2	Coor	Orig	Distribution
Temp1	all	temp	25	0,000	1,000	Rela	From start	Constant

3.Stress

Linear calculation, Extreme : Global
Selection : All
Load cases : temp

Member	Case	dx [m]	Normal - [MPa]	Normal + [MPa]	Shear [MPa]	von Mises [MPa]	Fatigue [MPa]	Kappa [1]
S33	temp	7,071	**-14,6**	0,0	0,3	**14,6**	0,0	0,00
S3	temp	2,628	**0,0**	1,9	0,3	2,0	0,0	0,00
S2	temp	2,628	-1,6	**0,0**	0,3	1,7	0,0	0,00
S61	temp	10,512	0,0	**10,8**	0,3	10,8	0,0	0,00
S15	temp	0,000	-1,0	3,6	**0,1**	3,6	0,0	0,00
S177	temp	0,000	-9,3	0,0	**0,5**	9,3	0,0	0,00
S175	temp	6,132	-0,2	0,0	0,2	**0,5**	0,0	0,00
S1	temp	0,000	-1,5	1,4	0,4	1,6	**0,0**	**0,00**

The occurring stresses are summarised in the table below.

	tensile stress [N/mm^2]	compressive stress [N/mm^2]
1) Only supports at the base	+8	-10
2) Only supports at the base: sun on 1 side	+35	-35
3) Only supports at the base: sun on 2 sides	+29	-32
4) All nodes restricted	+42	-70
5) Vertically free support, middle of façades	+11	-15
6) Vertically free support, entire façades	+14	-68

Table 10.1: Summary of the stress in the trussed tube structure.

Consequently, the following conclusions can be drawn;
If the tube structures warms up evenly and no connections above ground level are present (case 1), the occurring stresses in the trussed structure are marginal – and only present at the base – and do not cause any problems.
In case one or two sides of the tube structure are warmed up and no connections above ground level are present (case 2 & 3), the occurring stresses in the trussed structure are not negligible, yet still acceptable.
If the movement of all nodes is restricted in all three directions (case 4), the stresses that occur are substantial – also taken into account that the temperature differential considered here is only 25˚C. This means that the full restriction of the nodes is not viable: it will inevitably lead to deformations.
In case the movement of the nodes is restricted in 2 directions (vertical movement unhindered) over the entire width of the four façade (case 6), the occurring stresses amount to 68 N/mm^2 which is not viable. However when these same connections are only made in the middle of the façade – allowing for a horizontal expansion as well – the occurring stresses remain minor.
The overall conclusion is that the vertical expansion should be allowed for: the displacements at the top of the building are such that it cannot be resolved in any other way. Concerning horizontal displacements, these should also be allowed for by fixing the structure in the middle, permitting displacements towards the corners.

11 Appendix 5: Substructure & Foundation

11.1 Calculation Pile loads

The loads on the columns on the inner ring can simply be calculated: it is the result of the dead and live floor loads and can be calculated by dividing the columns loads by the amount of piles per column

$$F_{pile} = \frac{45000}{14} \approx 3200\ kN$$

11.2 Calculation Foundation Rotation

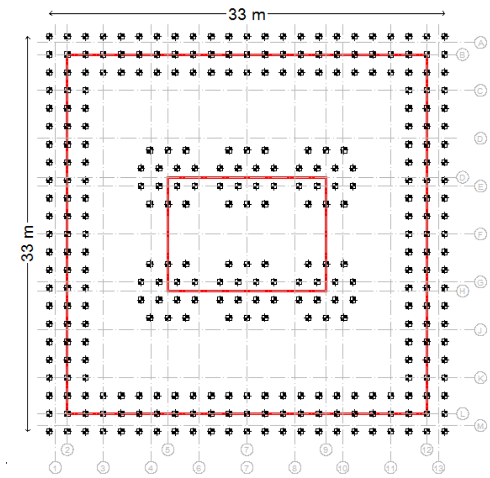


Figure 11.1: Pile plan design variant.

The following data are used to calculate the rotation of the foundation

p_{rep} = 2.54 kN/m^2
e = 16.5 m (distance centre to edge of pile plan)
A_{pile} = 0.2025 m^2 (square, 0.45 m)
l_{pile} = 20 m (adopted from current design)
E_{pile} = 8.395·10^6 kN/m^2 (adopted from current design, based on Geomet research)
h = 158 m

Calculation of the bending moment due to wind

$$q_{rep} = 2.54 \cdot 30 = 76.2\ kN/m$$

$$M_{rep} = \frac{1}{2} q l^2 = \frac{1}{2} \cdot 76.2 \cdot 158^2 = 951{,}128\ kNm^2$$

$$M_d = 1.0 \cdot 951{,}128 = 951{,}128\ kNm^2$$

The moment of inertia of the pile plan equals

$$I_{pile\ plan} = A_{pile}(12 \cdot 1.5^2 + 12 \cdot 3.0^2 + 12 \cdot 4.5^2 + \ \dots + 46 \cdot 13.5^2 + 46 \cdot 15.0^2 + 46 \cdot 16.5^2)$$
$$= 36765 A_{pile}$$

The maximal pile load (SLS) can be calculated with

$$F_{pile} = A_{pile} \cdot \frac{M \cdot e}{I_{pile\ plan}} = \frac{951{,}128 \cdot 16.5}{36765} = 427\ kN$$

Accordingly the shortening of the pile can be given with

$$\Delta l_{pile} = \frac{427 \cdot 20}{8.395 \cdot 10^6 \cdot 0.2025} = 5.02 \cdot 10^{-3}\ m$$

Meaning that the rotation of the foundation due to wind equals

$$\varphi = \frac{\Delta l}{e} = \frac{5.02 \cdot 10^{-3}}{16.5} = 3.04 \cdot 10^{-4}\ m$$

The rotation stiffness of the foundation can be expressed as

$$C = \frac{M}{\varphi} = \frac{951{,}128}{3.04 \cdot 10^{-4}} = 3.13 \cdot 10^9\ kNm/rad$$

And lastly the deflection due to the foundation rotation is

$$y_{foundation} = \varphi \cdot h = 3.04 \cdot 10^{-4} \cdot 158 = 48.1 \cdot 10^{-3}\ m$$

Initially, 25% of the total deflection – amounting to approximately 80 mm – was reserved for the foundation rotation. This means that the deflection due to foundation rotation is at the safe side.

12 Appendix 6: Dimensioning of the Structural Design

12.1 Starting points calculations

Construction principle
The constructions of New Orleans consists of 5 parts, namely;

- Building pit
- Foundation
- Underground car park
- Highrise apartment building

NB: the low-rise building called the 'Arthouse' is left out since it is does not affect the construction of the highrise and is thus not elaborated structurally.

Building pit
A building pit is needed to create an underground car park. All structural starting points for the drainage of the building pit and the pit itself are described in the reports from Geomet (AA08350-4 and AA08350-7).
The building pit consists of a permanent sheetpile wall parallel to the quay and a temporary sheetpile wall on axis 17, perpendicular to the quay. The basement of Montevideo (adjacent building at the southwest) was designed in such a way that future sheetpile walls can be joined, which needs to be executed in such a way. At the location of the high-rise building, 2 bracing frames are needed. When the lowest basement floor (-2) is cast and hardened, the frames can be removed.

The outline of the building order of the building pit is as follows;

- Vibrate the sheetpile walls into the ground
- 1^{st} excavation
- Pile driving
- Construction lift pit highrise
- Placing bracing frames
- 2^{nd} excavation

Foundation
Raking piles will be used to cope with horizontal loads due to wind.

To absorb the differential settlements of the highrise in relation to the lowrise, the footing of the tower will not be connected with the adjoining foundation during the construction of the structural work; Through a temporary dilatation profile, water leaks to the car park will be prevented. When the structural work of the highrise has reached its peak and direct settlements have taken place, the footing of the highrise will be connected permanently to the adjoining foundation.

Underground car park
A two-story underground car park will be built underneath the project. The car park, measuring 100 m by 33 m, is located under the whole length of the building and the adjoining square. It holds two one-way lanes providing access to 4 parking strips with places for straight parking.

Starting points

Building regulations
The prevailing building regulations are as followed;
General: TGB 1990, NEN 6700, NEN 6702

Concrete:	TGB 1990, NEN 6720
Geotechnics:	TGB 1990, NEN 6740 until NEN 6744
Steel:	TGB 1990, NEN 6770

Corresponding drawings

The structural and architectural drawings are enclosed elsewhere in this appendix.

All original structural and architectural drawings, from DHV and Alvaro Siza Architectos/ADP respectively, deemed necessary to establish the initial image are enclosed as well.

Starting points calculations

General

Residential building:	safety class 3	
Reference period:	50 years	
Ultimate limit state:	$\gamma_g = 0.9/1.2/1.35$	$\gamma_q = 1.5$
Serviceability limit state:	$\gamma_g = 1.0$	$\gamma_q = 1.0$

Concrete (substructure)	C35/45
Reinforcement steel	FeB 500

Steel	S355 (= standard, HD 400x1086 in S460)

Loads

Dead & live floor loads

Apartment floors	Dead load	Live load
Precast wing floor (280 mm)	3.61	
Compression layer in covering chases	2.25	
Floating screed (60 mm)	1.2	
Lightweight partition walls	1.2	
Live load people, etc. (Ψ = 0.4)		1.75
	8.26	1.75

NB: the weight of the integrated steel floor beams is assumed to be accounted for in the weight of the concrete floor slabs.

Loggias	Dead load	Live load
Precast wing floor (280 mm)	3.61	
Compression layer in covering chases	2.25	
Insulation + tiles + ceiling	1.3	
Live load people, etc. (Ψ = 0.5)		2.50
	7.16	2.50

Core floors	Dead load	Live load
Precast concrete plank slab (spancon 80)	1.92	
Compression layer (120 mm)	3.0	
Floating screed (60 mm)	1.2	
Live load people, etc. (Ψ = 0.4)		3.00
	6.12	3.00

Installation floor	Dead load	Live load
Precast concrete plank slab (300 mm)	7.5	
Floating screed (60 mm)	1.2	
Live load people, etc. (Ψ = 1.0)		5.00
	8.7	5.00

First storey floor (shop)	Dead load	Live load
Precast concrete plank slab (300 mm)	7.5	
Floating screed (60 mm)	1.2	
Conduits + ceiling	0.5	
Live load people, etc. (Ψ = 0.25)		5.00
	9.2	5.00

Ground floor	Dead load	Live load
Precast concrete plank slab (300 mm)	7.5	
Foam concrete (370 mm) (12 kN/m^2)	4.4	
Screed (100 mm)	2.0	
Conduits	0.5	
Live load people, etc. (Ψ = 0.25)		10.00
	14.4	10.00

Basement floors	Dead load	Live load
In-situ cast concrete (260 mm)	6.5	
Live load cars, etc. (Ψ = 0.4)		2.00
	6.5	2.00

Sandwich façade panels + stone cladding	Dead load
	2.0

Wind load

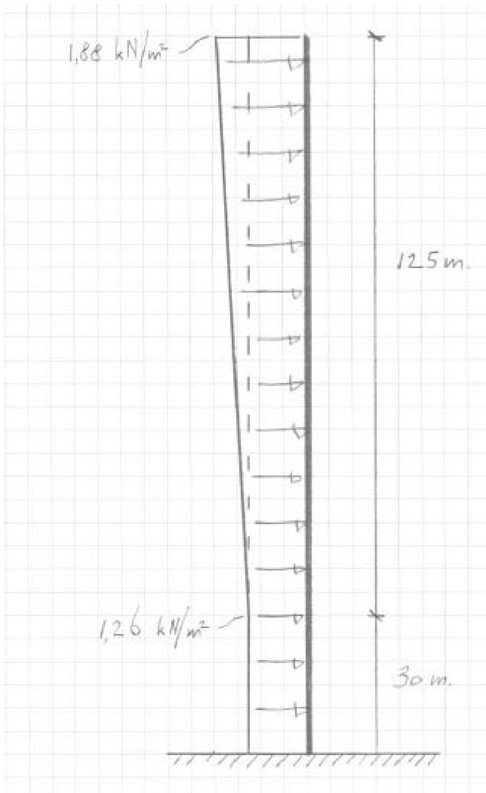
The project New Orleans is located at the Otto Reuchlinweg in Rotterdam

$P_{rep} = C_{dim} C_{index} C_{eq} \phi_1 p_w$

Wind area II; not built-on

The wind pressure at 155 meter has an extreme value:
$p_w = 1.88$ kN/m^2.

The wind pressure at 30 meter has an extreme value:
$p_w = 1.26$ kN/m^2.



Factor reckoning with building dimensions

Height of the tower = 155 meter.

Width of the tower = 30 meter.

C_{dim} = 0.87

Factor reckoning with building shape

C_{pe} = 0.8 for pressure

C_{pe} = 0.4 for suction

$C_{pe,loc}$ = not applicable

C_{pi} = not applicable

C_f = 0.04 for friction

C_t = not applicable

Pressure equalization factor

$C_{eq} = 1.0$

Dynamic multiplication factor

Φ_1 = to be determined later

Wind tunnel research

For the project New Orleans wind tunnel research has been conducted. See also: Peutz report WG 4448-3 'Bepaling representatieve winddrukken op omhullende alsmede krachten en momenten op hoofddraagconstructie d.m.v. windtunnelonderzoek', dated 03-05-2006.

Based on the results, a reduction factor of 0.9[1] can be applied for wind from the northwest and the southeast. No reduction on wind from the southwest and northeast can be applied.

Alternate load path

Local failure of any structural element may never cause disproportional damage: after failure of any local structural element, the damage needs to be limited to the adjoining elements and/or spaces. Therefore the failure of any of the columns at the base of the building (e.g. due to a collision) will be taken into account when calculating the structural design.

Fire safety requirements

Highrise building
Function: residential building
Level highest floor: 70 meter +
Requirement: 120 minutes

Underground car park
Function: parking (located directly underneath the highrise)
Requirement: 120 minutes

Fire spread requirements: 60 minutes per fire cell, 120 minutes per fire compartment.

Vibrations & horizontal accelerations

The structural design has to meet a number of requirements regarding vibrations. Vibrations should not hamper the functionalism of a structural element or the entire building, or cause any damage. Therefore, the first natural frequency of the floors should not be lower than 3 Hz.

For the calculations of the horizontal accelerations of the top of the building due to wind, the prevailing building regulation is NEN 6702 art. 10.5.3

Groundwater levels

Based on advice from Geomet, the following groundwater levels will be adopted;

Highest groundwater level: 1.70m + NAP (Amsterdam Ordnance Datum) = 1.95m – level datum
Lowest groundwater level: 5.00m - NAP (Amsterdam Ordnance Datum) = 8.65m – level datum

12.2 Global Dimensioning

Trussed Tube Structure

The global dimensioning of the trussed tube structure is based on the deflection of the top. The data used and the dimensions of the sections are found via the mechanics formulae and are as follows.

h =	9.99 m
a =	8.00 m
d =	10.76 m
F =	761 kN (equals p_{rep} = 2.54 kN/m^2)
l =	160 m
E =	$2.1 \cdot 10^8$ kN/m^2

The deflection can be calculated with the derived formula for the geometry

$$y_{total} = \frac{36}{697} \cdot \frac{Fl^3}{a^2 E \frac{h^3}{(\frac{1}{2}a)^3} A_h} + \frac{36}{697} \cdot \frac{Fl^3}{a^2 E \frac{2h^3}{d^3} A_d} + 2\frac{1}{8} \cdot \frac{Fld^3}{ha^2 E A_d}$$

This formula is plotted for different values of A_h and A_d.

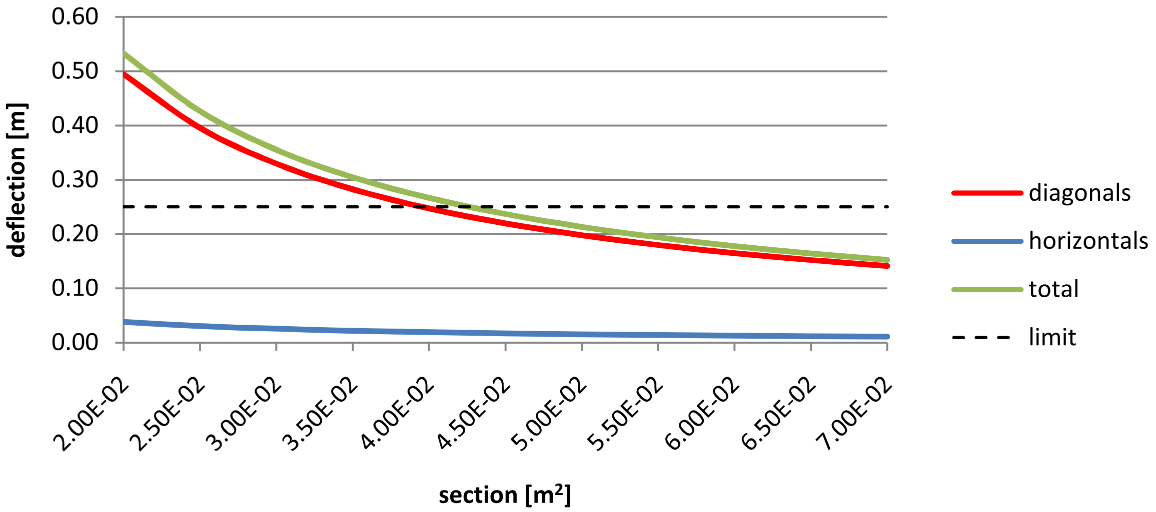


Figure 12.1: section-deflection graph for the trussed tube structure.

The deflection needs to be limited to approximately 250 mm – taking into account an additional deflection due to the foundation rotation – which can be reached with the following sections

$A_{diagonals}$ = $5.24 \cdot 10^{-2}$ m^2
$A_{horizontals}$ = $1.40 \cdot 10^{-2}$ m^2

These sections correspond with a deflection of 244 mm. And since the deflection has proven to be governing, these two sections are selected.

Façade columns

For the global dimensioning only a compressive force is considered and local bending moments are neglected. To account for instability due to buckling, a factor of 0.9 is considered reasonable to reduce the plastic compressive strength. In this case, it is assumed that the columns are not supported along their major or minor axes. In total, 4 different periphery columns are present which bear a different load area, as indicated in *Figure 12.2*.

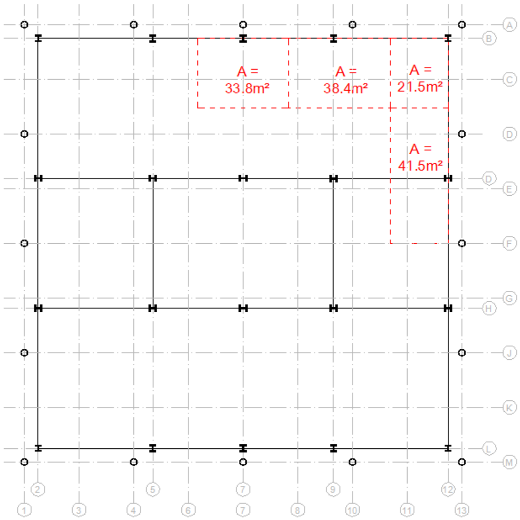


Figure 12.2: Floor plan with load areas of façade columns

Column type	Type 1	Type 2	Type 3	Type 4
Load area [m^2]	33.8	38.4	21.5	41.5

The maximum load is calculated based on the load combination 1.2 x dead load + 1.5 x live load floors.

Type 1	load	area	gamma	storeys	total
own weight	3	158	1.2		568.8
floors DL	8.5	33.8	1.2	47	16204
floors LL	1.83	33.8	1.5	47	4361
Façade	3.33	6.6	1.2	47	1240
					22373

Type 2	load	area	gamma	Storeys	total
own weight	3	158	1.2		568.8
floors DL	8.5	38.4	1.2	47	18409
floors LL	1.83	38.4	1.5	47	4954
façade	3.33	7.5	1.2	47	1409
					25341

Type 3	load	area	gamma	storey	total
own weight	3	158	1.2		568.8
floors DL	8.5	21.5	1.2	47	10307
floors LL	1.83	21.5	1.5	47	2774
façade	3.33	9.3	1.2	47	1747
					15396

Type 4	load	area	gamma	storeys	total
own weight	3	158	1.2		568.8
floors DL	8.5	41.5	1.2	47	19895
floors LL	1.83	41.5	1.5	47	5354
façade	3.33	9.9	1.2	47	1859
					27677

Only 2 different sections are adopted to prevent a too large variety of sections and to prevent confusion during construction. For column type 1, 2 and 4 a HD 400 x 677 in S355 ($N_{pl;d}$ = 30651 kN) would suffice. For the corner column, type 3, a HD 400 x 421 in S355 ($N_{pl;d}$ = 19067 kN) would suffice.

Façade column	Section
Type 1	HD 400 x 677
Type 2	HD 400 x 677
Type 3	HD 400 x 421
Type 4	HD 400 x 677

Core columns

For the global dimensioning only a compressive force is considered and bending moments are neglected. To account for instability due to buckling, a factor of 0.9 is considered reasonable to reduce the plastic compressive strength. The core columns are loaded by the core area of 125 m^2 and an additional area of the apartments of 302 m^2.

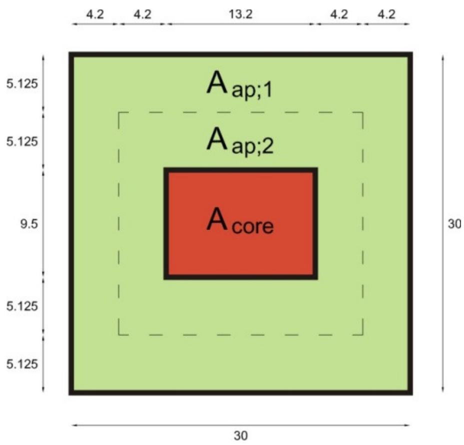


Figure 12.3: Distribution of floor loads to core and façade construction.

The total vertical load on the core construction follows from the weight calculations and is 57488 kN and 17135 kN, from the core and apartments respectively, adding up to a total of 228838 kN.
The load is more or less evenly distributed over the 6 core columns yielding a force per column of

$$F_{core\ column} = \frac{228838}{6} = 38140\ kN$$

An HD 400 x 990 in S355 ($N_{pl;d}$ = 44816 kN) would suffice.

Core diagonals
The core diagonals form a secondary windbracing – in addition to the trussed tube structure – with the aim to distribute the wind load from two intermediate floors to the floors that are linked with the tube structure, as illustrated in *Figure 12.4*.

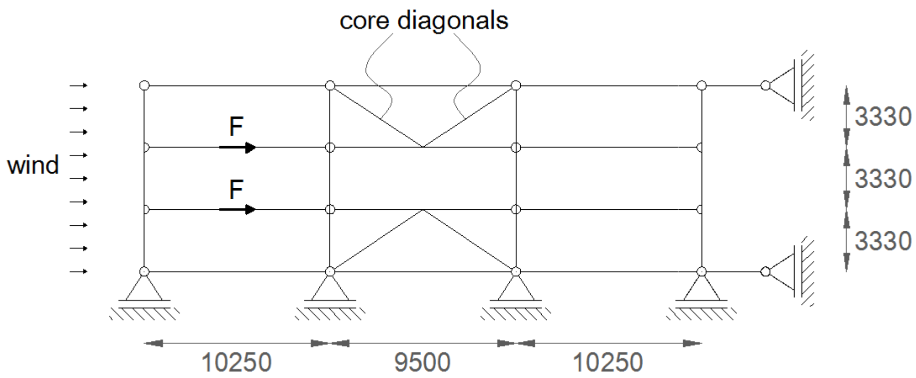


Figure 12.4: Structural diagram with core diagonals every three storeys

The wind load is distributed to the floors which transfer their forces (F) to every third floor that is connected to the tube structure. The maximum wind load at the top of the building amounts to 1.88 kN/m^2.

$$p_{rep} = 0.87 \cdot (0.8 + 0.4 + 0.08) \cdot 1.0 \cdot 1.39 \cdot 1.88 = 2.91\ kN/m^2$$

The force F in every floor is then

$$p_{rep} = 2.91 \cdot (30 \cdot 3.33) = 291\ kN$$

$$p_d = 1.5 \cdot 291 = 436\ kN$$

Between 2 storeys, 4 diagonals are present per direction, integrated into the walls of the core. The force in all 4 diagonals can be calculated with the length of the diagonals, d = 5801 mm

$$F = \frac{5801}{4750} \cdot 436 = 532\ kN$$

Per diagonal, this yields a force of

$$F_{diagonal} = \frac{532}{4} = 133\ kN$$

This force is relatively small. Therefore, a practical section is adopted, namely an IPE 160.

Integrated floor beams & rim joists

The integrated floor beams have been calculated with an excel-sheet – toetsing geïntegreerde ligger' – developed by the foundation 'Bouwen met Staal'. This excel-sheet is developed for hollow core floor slabs in combination with integrated floor beams, though precast wing floors are used in the design variant. However, the concept and the calculations are the same for wing floors as for hollow core floor slabs, making it pressumable that the results are accurate. The excel-sheet performs all the unity checks for the serviceability limite state and the ulitimate limit state, also in case of fire. The excel-sheet can be found in appendix A 8.
The integrated floor beams can be subdivided into 4 types, all with another load or span, illustrated in REF. From a uniformity point of view, only 2 different sections have been adopted, meaning that some beams have extra capacity. For the rim joists practical sections are selected since they do not carry any (significant) load.
Insert floor diagram

The results obtained here are the definite sections; naturally they are part of the structural computer model, though merely for the integrety and appearance of the model. All the determined sections are summarised below. Hoedliggers, no petliggers, but weight/section is a good estimation/approximation.

Integrated floor beam	Section
Type 1	THQ 265x6-290x35-500x20
Type 2	THQ 265x6-290x35-500x20
Type 3	THQ 265x6-290x35-500x20
Type 4	THQ 320x8-290x35-500x20
Rim joists	IPE 300

12.3 ESA PT models

Floor loads

The floor loads have been applied as prescribed in the 'starting point calculations' document. However to facilitate the application of the apartment and loggia floor loads, they have been averaged with respect to their respective floor area. The combined floor area of both is 775 m^2.

	Floor area [m^2]	Dead load [kN/m^2]	Live load [kN/m^2]
Apartments	697	8.26	1.75
Loggia	89	7.16	2.50

The mean averages can be calculated as follows

$$G_{apartment/loggia} = \frac{697}{775} \cdot 8.26 + \frac{78}{775} \cdot 7.16 = 8.15\ kN/m^2$$

$$Q_{apartment/loggia} = \frac{697}{775} \cdot 1.75 + \frac{78}{775} \cdot 2.50 = 1.83\ kN/m^2$$

Wind load

For calculations of the entire construction, the triangular wind load over the façade has been replaced by a uniformly distributed load to facilitate the calculations. The uniformly distributed load is calculated as follows.

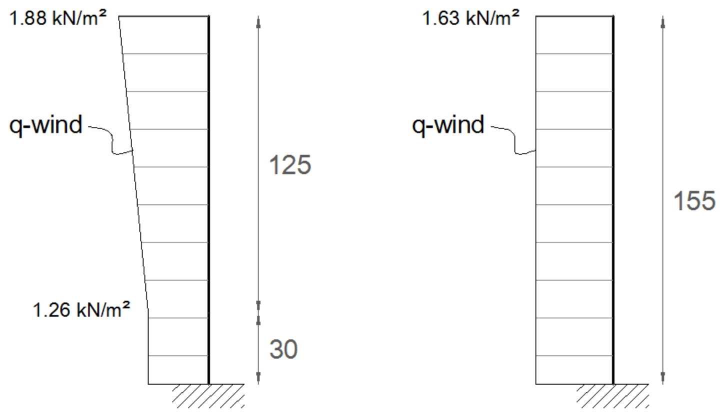


Figure 12.5: replacement of triangular load by evenly distributed load

$$M_{base} = \frac{1}{2} \cdot 1.26 \cdot 155^2 + \left(30 + \frac{2}{3} \cdot 125\right) \cdot \frac{1}{2} \cdot |1.88 - 1.26| \cdot 125 = 1.95 \cdot 10^4\ kNm/m$$

$$M_{base} = \frac{1}{2} p_{equ} l^2 = 1.95 \cdot 10^4\ kNm/m$$

$$p_{equ} = 1.63\ kN/m^2$$

$$p_{rep} = 0.87 \cdot (0.8 + 0.4 + 0.08) \cdot 1.0 \cdot 1.39 \cdot 1.63 = 2.54\ kN/m^2$$

Given the fact that the building is perfectly symmetrical, the design is calculated with wind loads from only the northwest and northeast. The wind load from the southwest differs slightly from the wind load from the northeast due to the presence of the Arthouse; in this case the governing wind load is that from the northeast. Additionally, wind tunnel research has shown that the wind load from the northwest can be reduced with a factor of 0.9.

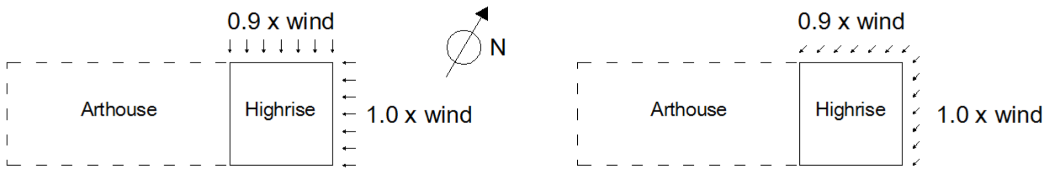


Figure 12.6: Considered main wind directions

The wind load is evenly transferred from the façade element to the both floors it is attached to. For a standard storey height of 3.33 m the wind load from the northeast on the floors is

$$p_{rep} = 2.54 \cdot 3.33 = 8.46\ kN/m^2$$

For wind load from the northwest, the load on the floors is

$$p_{rep} = 0.9 \cdot 2.54 \cdot 3.33 = 7.61\ kN/m^2$$

The wind load from the north, subdivided for the northwest and northeast side of the floor, respectively are

$$p_{rep} = 0.5 \cdot \sqrt{2} \cdot 8.46 = 5.99\ kN/m^2$$

$$p_{rep} = 0.5 \cdot \sqrt{2} \cdot 7.61 = 5.38\ kN/m^2$$

<u>Verifications</u>
Ultimate Limit State
The specific checks have been performed automatically by ESA PT and sections have been optimised where necessary.

Serviceability Limit State
The deflection at the top of the building is extreme load case: wind from the NorthEast):

$$u_{TFC} = 290\ mm$$

The deflection due to the foundation rotation needs to be added:

$$u_{rotation\ foundation} + u_{TFC} = 48 + 290 = 338\ mm$$

Subsequently, the amplification factor (1.07) needs to be taken into account for the second order effect:

$$1.07 \cdot 338 = 362\ mm$$

Finally the ULS check is performed:

$$362\ mm \leq \frac{161 \cdot 10^3}{500}$$

$$362\ mm \nless 322\ mm$$

It follows that the chosen sections do not meet the requirements regarding the SLS. This is due to the larger deflection of the tube structure: an extra rotation at the base of the tube structure occurs due to the basement concrete that has been modeled with an E = 10.000 N/mm^2 (cracked concrete), which has not been taken into account in the global dimensioning. Yet, the calculations are not revised, but it needs to be noted that larger sections are needed to meet the SLS requirements.

Interstorey deflection for the highest storeys.
The maximum deflection at 161 m (top of the building) equals:

$$u_{TFC} = 290\ mm$$

The maximum deflection at 151 m equals:

$$u_{TFC} = 271\ mm$$

The interstorey deflection can be calculated as follows:

$$u_{interstorey} = 290 - 271 = 19\ mm$$

Finally the ULS check is performed:

$$19\ mm \leq \frac{10 \cdot 10^3}{300}$$

$$19\ mm < 33\ mm$$

This means that the requirements regarding interstorey deflection are met.

12.4 2nd order effect

To estimate the influence of the 2nd order effect the critical load on the TFC is calculated for the combination of horizontal and vertical loads on the construction.

When the critical load for a trussed tube structure needs to be calculated, the bending stiffness (EI) and the spring constant (C) of the foundation needs to be taken into account. Additionally, the racking shear stiffness (GA) also needs to be considered since the shear deflection plays a major part in the total deflection of the construction. The three parameters can be combined into one formula for the critical load (Hoenderkamp, 2002).

$$\frac{1}{F_{cr}} = \frac{1}{F_{cr;b}} + \frac{1}{F_{cr;s}} + \frac{1}{F_{cr;f}}$$

In case the construction is loaded with a uniformly distributed horizontal load (wind) and a uniformly distributed vertical load (floor loads) the formula can be rewritten as

$$\frac{1}{F_{cr}} = \frac{l^2}{8EI} + \frac{1}{2GA} + \frac{l}{2C}$$

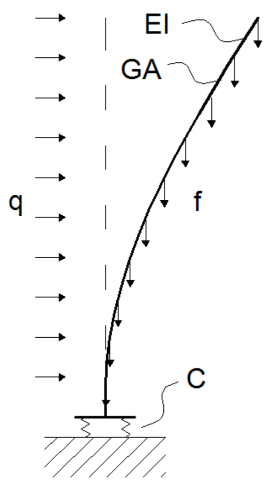


Figure 12.7: Structural diagram with the 2nd order effect-parameters

From the global dimensioning and the optimalization of the trussed tube structure, the necessary information is gathered so the parameters can be completed in the formulae

$$EI = 42a^2EA_{equ}$$

$$GA = 2 \cdot \frac{2a^2hEA_d}{d^3}$$

$$C = \frac{M}{\varphi}$$

With the geometry of the trussed tube structure variant “without corner columns” and the additional data the critical force can be calculated.

h =	10 m
a =	8 m
d =	10.77 m
E =	$2.1 \cdot 10^{8}$ kN/m^2
$A_{diagonal}$ =	$5.24 \cdot 10^{-2}$ m^2
$A_{horizontal}$ =	$5.24 \cdot 10^{-2}$ m^2

$$A_{equ} = \left(\frac{10.68^3}{2 \cdot 9.9^3 \cdot 5.24 \cdot 10^{-2}} + \frac{4^3}{9.9^3 \cdot 5.24 \cdot 10^{-2}}\right)^{-1} = 7.56 \cdot 10^{-2}\, m^2$$

$$EI = 42 \cdot 8^2 \cdot 2.1 \cdot 10^8 \cdot 7.86 \cdot 10^{-2} = 3.75 \cdot 10^{10}\, kNm^2$$

$$GA = 2 \cdot \frac{2 \cdot 8^2 \cdot 9.9 \cdot 2.1 \cdot 10^8 \cdot 5.24 \cdot 10^{-2}}{10.68^3} = 2.29 \cdot 10^7\, kN$$

$$C = \frac{1{,}373{,}029}{4.39 \cdot 10^{-4}} = 3.13 \cdot 10^9\, kNm/rad$$

(Note that the bending stiffness of the substructure is not taken into account.) Subsequently the critical load can be calculated

$$\frac{1}{F_{cr}} = \frac{158^2}{8 \cdot 3.75 \cdot 10^{10}} + \frac{1}{2 \cdot 2.29 \cdot 10^7} + \frac{158}{2 \cdot 3.13 \cdot 10^9}$$

$$F_{cr} = 7{,}675{,}362\, kN$$

Now F_{cr} is known, the multiplication factor can be calculated

$$n = \frac{F_{cr}}{F_v}$$

With

$$F_v = 505{,}901\, kN$$

Yields

$$n = \frac{7{,}675{,}362}{505{,}901} = 15.17$$

$$\frac{n}{n-1} = \frac{15.17}{14.17} = 1.07$$

Concluding, the forces and the loads need to be multiplied by an amplification factor of 1.07 to take the second order effect into account.

12.5 Dynamic multiplication factor Φ_1

The dynamic multiplication factor is calculated according to NEN 6702, appendix A.4. The data are as follows

h = 158 m
b = 30 m
D = 0.01 (steel construction)
a = 0.384 m/s^2 (weight uniformly distributed over the height)

The term δ is determined as follows: the resultant reactions due to the vertical loads are placed horizontally on the construction to determine the corresponding deflection. The loads have been calculated by ESA PT. To determine the accompanying live load of the floor loads, an average reduction factor Ψ = 0.4 is used.

Load cases	Resultant force in Z-direction [kN]
Own weight	69421
Floors dead load	358824
Floors live load	39183 (=97958 · 0.4)
Façade	19980
Total load	487408

Given the fact that the construction is identical in the two perpendicular main wind directions, only one value of δ is calculated, being 9.568 m.

$$f_e = \sqrt{\frac{0.384}{9.568}} = 0.200\ Hz$$

$$I = \frac{1}{ln\left(\frac{158}{0.2}\right)} = 0.150$$

$$B = \frac{1}{0.94 + 0.021 \cdot 158^{\frac{2}{3}} + 0.029 \cdot 30^{\frac{2}{3}}} = 0.545$$

$$E = \frac{0.0394 \cdot 0.200^{-\frac{2}{3}}}{0.01(1 + 0.1 \cdot 0.200 \cdot 158) \cdot (1 + 0.16 \cdot 0.200 \cdot 30)} = 1.41$$

$$\phi_1 = \frac{1 + 7 \cdot 0.150 \cdot \sqrt{0.545 + 1.41}}{1 + 7 \cdot 0.150 \cdot \sqrt{0.545}} = 1.39$$

12.6 Vibrations

The factor ϕ_2 is calculated according to NEN 6702, appendix A.5 and indicates whether the vibrations of the building cause nuisance. The used data are as follows.

h = 158 m
b = 30 m
D = 0.01 (steel construction)
a = 0.384 m/s^2 (weight uniformly distributed over the height)

$$f_e = \sqrt{\frac{0.384}{9.568}} = 0.200\ Hz$$

$$D = 0.01$$

$$\phi_2 = \sqrt{\frac{0.0344 \cdot 0.200^{-\frac{2}{3}}}{0.01(1 + 0.12 \cdot 0.200 \cdot 158) \cdot (1 + 0.20 \cdot 0.200 \cdot 30)}} = 0.98$$

$$\tilde{p}_{w;1} = 100ln\left(\frac{158}{0.2}\right) = 667$$

$$C_t = 0.8 + 0.4 + 0.08 = 1.28$$

The mass of the building including the accompanying live load of the floors is calculated in the previous section and equals 487408 kN, or 48,740,800 kg.

$$\rho_l = \frac{48{,}740{,}800}{158} = 308{,}486\ kg/m$$

a = 0.16 m/s^2 (curve 2, residential buildings)

$$1.6\left(\frac{0.98 \cdot 677 \cdot 1.28 \cdot 30}{308{,}486}\right) < 0.16$$

$$0.13 < 0.16$$

The calculations indicate that no nuisance due to vibrations is likely to occur.

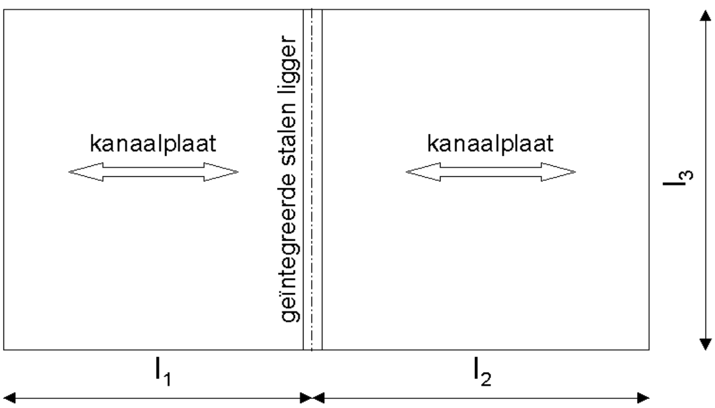
12.7 Excel sheet for integrated floor beam

The original excel sheet provided by Bouwen met Staal has been inserted below with the governing values and corresponding verifications for the normative integrated floor beam. The selected section () meets all ULS requirements, with exception of the fire requirements. However, the section will be wrapped with Promatect, to increase the fire resistance to 120 minutes.

bedrijf:	Bouwen met Staal
project:	Spreadsheet geïntegreerde ligger, versie 1.2
datum:	05-11-2007

Toetsing geïntegreerde stalen ligger

Invoer



l_1 =	4200	mm	l_3 =	9500	mm
l_2 =	4200	mm			

Algemeen

bedrijf:	Bouwen met Staal
project:	Spreadsheet geïntegreerde ligger, versie 1.2
datum:	05-11-2007

veiligheidsklasse:	3	
referentieperiode:	50	jaar
staalsoort:	S355	
eis bijkom. doorbuiging, a	0.002	$u_{bijk;toel} = a \cdot l_3$

=		
brandwerendheid:	120	minuten

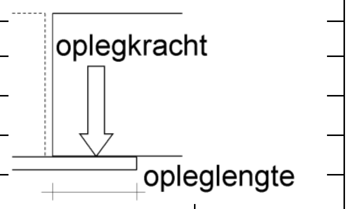
Belastingen

Permanente belasting:	eigen gewicht vloer	3.61	kN/m^2
	afwerkvloer	2.25	--
	lichte scheidingswanden	1.20	--
	overige	1.20	--
	$p_g =$	8.26	kN/m^2
Veranderlijke belasting:	$p_q =$	1.75	kN/m^2
	$\psi =$	0.4	

bedrijf:	Bouwen met Staal
project:	Spreadsheet geïntegreerde ligger, versie 1.2
datum:	05-11-2007

Profiel en opleglengte vloer

profiel:	THQ 265x6-290x35-500x20	
opleglengte vloer:	80	mm



Resultaten

Fundamentele belastingcombinatie	unity check
toetsing onderplaat:	**0.03**
toetsing onderflens:	**--**
toetsing momentcapaciteit doorsnede:	**0.63**
toetsing dwarskrachtcapaciteit:	**0.42**

Incidentele belastingcombinatie

spanningscontrole tijdens gebruik:	
- bovenflens	**0.56**
- onderplaat	**0.58**
- onderflens	**--**
bijkomende doorbuiging:	**0.55**
doorbuiging eindtoestand:	**1.64**

minimaal benodigde zeeg:				**25**	mm	
Bijzondere belastingcombinatie (brand)						
toetsing onderplaat:				**0.76**		
toetsing onderflens:				**--**		
toetsing momentcapaciteit doorsnede:				**5.58**		
toetsing dwarskrachtcapaciteit:				**1.03**		
Conclusie						
Het gekozen profiel voldoet tijdens kamertemperatuur, maar is NIET 120 minuten brandwerend.						

13 Appendix 7: Fire safety

13.1 Inventory Fire Compartmentation

Below an overview of the amount of fire cells is given per storey, excluding the core (which is a fire cell as well)

	Fire cells
Ground floor (business space)	4
1^{st} storey (business space)	2
2^{nd} storey (installation floor)	1
3^{rd} storey (apartments)	7
4^{th} storey (apartments)	5
5^{th} – 35^{th} storey (apartments)	6
36^{th} – 39^{th} storey (apartments)	5
40^{th} – 43^{rd} storey (apartments)	4
44^{th} – 45^{th} storey (apartments)	2

The most disadvantageous storey is the 1^{st} where businesses will be located and multiple façade openings are present (the installation floor is not normative with only 1 fire cell, since no façade openings are made, except for air inlets). Its gross floor area is still below a 1000 m^2 meaning that further compartmentation is not mandatory. If a fire starts on the 1^{st} storey, one half of the trussed tube structure will be exposed to fire for two hours and the other half for one hour.

14 Appendix 8: Comparison

14.1 Calculation of Construction Weight

Current Design

The weight of the current design is based on the amount of concrete (m^3)

Project	:	Woontoren New Orleans te Rotterdam
Onderdeel	:	Begroting ruwbouw deel 2: woontoren
Opdrachtgever	:	Vesteda Project BV
Architect	:	Alvaro Siza Portugal / ADP Architecten Amsterdam
Constructeur	:	DHV Bouw en Industrie BV
Peildatum	:	1 juli 2007

Hoofdstuk		**Omschrijving**	**Hoeveelheid**	**Eh**
21.00	*	**BETONWERK**	**1.00**	**PST**
21.50	+	**BETONWANDEN IHWG D=300MM TOREN**	**253.70**	**M2**
	.	Betonwand: principe systeemkist		
	.	Betonwanden str. 11 en 15		
	.	Betonwanden h=3030mm		
	.	Wanden per niveau:		
	,	P3 niveau 11.390+P	169.10	m2
	,	P4 niveau 14.720+P	84.60	m2
21.32.10-b		kimaansluiting	83.70	m1
21.32.10-b		wandkist	507.40	m2
21.32.10-b		kopschot br=300mm h=3030mm	6.00	st
21.32.10-b		sparingkist br=300mm	74.00	m1
21.40.10-a	.	wapening > zie totaal opgave		
21.50.10-h		beton C53/65 mkXC1 cgS3	58.90	m3
	-			
21.50	+	**BETONWANDEN IHWG D=300MM TOREN**	**9826.70**	**M2**
	.	Betonwanden: principe klimkist		
	.	Totaal 38 bouwlagen		
	.	Betonwanden str. 12, 13 en 14		
	.	Betonwanden h= 3330mm		
	.	Wanden per niveau:		
	,	P3 niveau 11.390+P	275.80	m2
	,	P4 niveau 14.720+P	275.80	m2
	,	P5 niveau 18.050+P	275.80	m2
	,	P6 niveau 21.380+P	275.80	m2
	,	P7t/mP10 niveau 24.710 t/m 34.700+P	1102.90	m2
	,	P11 niveau 38.030+P	258.50	m2

	,	P12t/mP35 niveau 41.360 t/m 117.950+P	6201.80	m2
	,	P36t/mP38 niveau 121.280 t/m 127.940+P	775.30	m2
	,	P39 niveau 131.270+P	192.50	m2
	,	P40 niveau 134.600+P	192.50	m2
21.32.10-b		kimaansluiting	2950.80	m1
21.32.10-b		wandkist	19653.40	m2
21.32.10-b		kopschot br=300mm h=3330mm	224.00	st
21.32.10-b		sparingkist br=300mm	1953.70	m1
21.40.10-a	.	wapening > zie totaal opgave		
21.50.10-h		beton C53/65 mkXC1 cgS3	2555.50	m3
	-			
21.50	**+**	**BETONWANDEN IHWG D=300MM TOREN**	**3265.50**	**M2**
	.	Betonwanden: principe klimkist		
	.	Totaal 38 bouwlagen		
	.	Betonwanden str. C en E		
	.	Betonwanden h= 3330mm		
	.	Wanden per niveau:		
	,	P3 niveau 11.390+P	86.00	m2
	,	P4 niveau 14.720+P	86.00	m2
	,	P5 niveau 18.050+P	86.00	m2
	,	P6 niveau 21.380+P	86.00	m2
	,	P7t/mP10 niveau 24.710 t/m 34.700+P	343.70	m2
	,	P11 niveau 38.030+P	86.00	m2
	,	P12t/mP35 niveau 41.360 t/m 117.950+P	2062.00	m2
	,	P36t/mP38 niveau 121.280 t/m 127.940+P	257.80	m2
	,	P39 niveau 131.270+P	86.00	m2
	,	P40 niveau 134.600+P	86.00	m2
21.32.10-b		kimaansluiting	980.40	m1
21.32.10-b		wandkist	6531.00	m2
21.32.10-b	.	kopschot br=300mm h=3330mm n.v.t.		
21.32.10-b		sparingkist	548.40	m1
21.40.10-a	.	wapening > zie totaal opgave		
21.50.10-h		beton C53/65 mkXC1 cgS3	911.00	m3
	-			
21.50	**+**	**BETONWANDEN IHWG D=300MM TOREN**	**706.00**	**M2**
	.	Betonwanden: principe klimkist		
	.	Totaal 5 bouwlagen		
	.	Betonwanden str. 12 en 14		
	.	Betonwanden h= 3330mm		
	.	Wanden per niveau:		
	,	P41 niveau 137.930+P	141.20	m2
	,	P42 niveau 141.260+P	141.20	m2
	,	P43 niveau 144.590+P	141.20	m2
	,	P44 niveau 147.920+P	141.20	m2
	,	P45 niveau 151.250+P	141.20	m2
21.32.10-b		kimaansluiting	212.00	m1
21.32.10-b		wandkist	1412.00	m2

21.32.10-b		kopschot br=300mm h=3330mm	20.00	st
21.32.10-b		sparingkist	204.00	m1
21.40.10-a	.	wapening > zie totaal opgave		
21.50.10-i		beton C28/35 mkXC1 cgS3	184.20	m3
	-			
21.50	**+**	**BETONWANDEN IHWG D=300MM TOREN**	**430.00**	**M2**
	.	Betonwanden: principe klimkist		
	.	Totaal 5 bouwlagen		
	.	Betonwanden str. C en E		
	.	Betonwanden h= 3330mm		
	.	Wanden per niveau:		
	,	P41 niveau 137.930+P	86.00	m2
	,	P42 niveau 141.260+P	86.00	m2
	,	P43 niveau 144.590+P	86.00	m2
	,	P44 niveau 147.920+P	86.00	m2
	,	P45 niveau 151.250+P	86.00	m2
21.32.10-b		kimaansluiting	129.00	m1
21.32.10-b		wandkist	860.00	m2
21.32.10-b	.	kopschot br=300mm h=3330mm n.v.t.		
21.32.10-b		sparingkist	47.70	m1
21.40.10-a	.	wapening > zie totaal opgave		
21.50.10-i		beton C28/35 mkXC1 cgS3	123.00	m3
	-			
21.50	**+**	**BETONWANDEN IHWG D=200MM TOREN**	**396.40**	**M2**
	.	Betonwanden: principe systeemkist		
	.	Totaal 2 bouwlagen		
	.	Betonwanden h=3030mm		
	.	Wanden per niveau:		
	,	P39 niveau 131.270+P	121.20	m2
	,	P40 niveau 134.600+P	275.20	m2
21.32.10-b		kimaansluiting	130.80	m1
21.32.10-b		wandkist	792.80	m2
21.32.10-b		kopschot br=200mm h=3030mm	48.00	st
21.32.10-b		sparingkist	48.50	m1
21.40.10-a	.	wapening > zie totaal opgave		
21.50.10-h		beton C53/65 mkXC1 cgS3	68.60	m3
	-			
21.50	**+**	**BETONWANDEN IHWG D=200MM TOREN**	**885.70**	**M2**
	.	Betonwanden: principe systeemkist		
	.	Totaal 5 bouwlagen		
	.	Betonwanden h=3030mm		
	.	Wanden per niveau:		
	,	P41 niveau 137.930+P	324.60	m2
	,	P42 niveau 141.260+P	306.60	m2
	,	P43 niveau 144.590+P	149.10	m2
	,	P44 niveau 147.920+P	44.80	m2
	,	P45 niveau 151.250+P	60.60	m2

21.32.10-b		kimaansluiting	289.20	m1
21.32.10-b		wandkist	1771.40	m2
21.32.10-b		kopschot br=200mm h=3030mm	118.00	st
21.32.10-b		sparingkist	107.10	m1
21.40.10-a	.	wapening > zie totaal opgave		
21.50.10-i		beton C28/35 mkXC1 cgS3	164.50	m3
	-			
21.50	**+**	**BETON PENANTEN IHWG TOREN**	**24.00**	**ST**
	.	Betonpenanten h=3030mm		
	,	P44 niveau 147.920+P afm:200x800mm	12.00	st
	,	P45 niveau 151.250+P afm:200x1000mm	12.00	st
21.32.10-b		penant bekisting	155.20	m2
21.40.10-a	.	wapening > zie totaal opgave		
21.50.10-i		beton C28/35 mkXC1 cgS3	13.10	m3
	-			
21.50	**+**	**IHWG VLOER KERN TOREN D=200MM**	**3879.90**	**M2**
	.	Ihwg vloer gerekend ipv een breedplaat		
	.	vloer!		
	.	Totaal 47 bouwlagen		
21.32.10-b		vloerbekisting	3879.90	m2
21.32.10-b		randkist h=200mm	1885.40	m1
	.	Vloer t.p.v. liftschacht tegen prefab		
	.	schachtwanden aanstorten		
21.40.10-a	.	wapening > zie totaal opgave		
21.50.10-h		beton C53/65 mkXC1 cgS3	606.90	m3
	.	Niveau P-1 t/m P41		
21.50.10-i		beton C28/35 mkXC1 cgS3	58.50	m3
	.	Niveau P42 t/m P45		
	.	wapening > zie totaal opgave		
25.32.30-b		L150x100x12mm	852.15	m1
25.32.30-b		aankoop HE200A l=2950mm	499.14	kg
25.32.30-b		aankoop HE200A l=3100mm	262.26	kg
25.32.30-b		aanbrengen HE200A	6.00	st
	-			
21.50	**+**	**BETONVLOEREN D=300MM TOREN**	**25897.00**	**M2**
	.	Totaal 38 bouwlagen		
	,	Betonvloer d=300mm	23026.00	m2
	,	Betonvloer loggia d=260-280mm	2871.00	m2
	.	Vloeren per niveau:		
	,	P4 niveau 14.720+P	721.30	m2
	,	P5 niveau 18.050+P	721.30	m2
	,	P6 niveau 21.380+P	721.30	m2
	,	P7t/mP10 niveau 24.710 t/m 34.700+P	2885.30	m2
	,	P11 niveau 38.030+P	721.30	m2
	,	P12t/mP35 niveau 41.360 t/m 117.950+P	16329.60	m2
	,	P36t/mP38 niveau 121.280 t/m 127.940+P	2041.20	m2
	,	P39 niveau 131.270+P	444.10	m2

	,	P40 niveau 134.600+P	681.00	m2
	,	P41 niveau 137.930+P	630.60	m2
21.32.10-b		vloerbekisting	26065.00	m2
21.32.10-b		randkist h=300mm	3556.70	m1
21.32.10-b		randkist h=20mm - overgang loggia	2477.90	m1
21.40.10-a	.	wapening > zie totaal opgave		
21.50.10-h		beton C53/65 mkXC1 cgS3	7682.97	m3
	,	loggia/balkonvloeren onder afschot	2871.00	m2
		stekkenbak t.b.v. klimkist	5958.60	m1
	-			
21.50	**+**	**BETONVLOER D=500MM TOREN**	**231.20**	**M2**
	.	Stalen ligger instorten: det. 41.01		
		aankoop HEM320 l=13.200mm	19404.00	kg
		instorten HEM320 l=13.200mm	6.00	st
		vulgaten voor beton	26.00	st
		hrsp. aanname hoh 300mm	180.00	st
	.			
	,	Betonvloer d=500mm	206.50	m2
	,	Betonvloer loggia d=460-480mm	24.70	m2
	.	Vloeren per niveau:		
	,	P39 niveau 131.270+P	231.20	m2
21.32.10-b		vloerbekisting	239.40	m2
21.32.10-b		randkist h=500mm	27.20	m1
21.32.10-b		randkist h=20mm - overgang loggia	18.40	m1
21.40.10-a	.	wapening > zie totaal opgave		
21.50.10-h		beton C53/65 mkXC1 cgS3	114.86	m3
	,	loggia/balkonvloeren onder afschot	24.70	m2
		stekkenbak t.b.v. klimkist	61.80	m1
	-			
21.50	**+**	**BETONVLOER UITKRAGENDE LOGGIAS**	**517.50**	**M2**
	.	Vloer loggias d=260-280mm		
	,	uitkragende loggia afm: 900x4100mm	138.00	st
	,	uitkragende loggia afm: 500x4100mm	4.00	st
21.32.10-b		vloerbekisting	517.50	m2
21.32.10-b		randkist h=260mm	834.60	m1
21.40.10-a	.	wapening > zie totaal opgave		
21.50.10-h		beton C53/65 mkXC1 cgS3	139.70	m3
	.			
	.	Stalen ligger instorten: det. A.13.03		
		aankoop UNP 260	22066.00	m1
		instorten UNP 260 l=4100mm	142.00	st
	-			
21.50	**+**	**BETONVLOER UITKRAGENDE ERKERS**	**166.10**	**M2**
	.	Vloer erker d=300mm		
	,	uitkragende erker afm: 500x4100mm	81.00	st
21.32.10-b		vloerbekisting	166.10	m2
21.32.10-b		randkist h=300mm	413.10	m1

21.40.10-a	.	wapening > zie totaal opgave		
21.50.10-h		beton C53/65 mkXC1 cgS3	49.90	m3
	-			
21.50	**+**	**BETONVLOEREN D=300MM TOREN**	**2055.00**	**M2**
	.	Totaal 5 bouwlagen		
	,	Betonvloer d=300mm	1823.80	m2
	,	Betonvloer loggia d=260-280mm	231.20	m2
	.	Vloeren per niveau:		
	,	P42 niveau 141.260+P	446.90	m2
	,	P43 niveau 144.590+P	537.40	m2
	,	P44 niveau 147.920+P	483.00	m2
	,	P45 niveau 151.250+P	198.20	m2
	,	P46 niveau 154.580+P	389.50	m2
21.32.10-b		vloerbekisting	2055.00	m2
21.32.10-b		randkist h=300mm	690.00	m1
21.32.10-b		randkist h=20mm - overgang loggia	65.20	m1
21.40.10-a	.	wapening > zie totaal opgave		
21.50.10-i		beton C28/35 mkXC1 cgS3	609.56	m3
	,	loggia/balkonvloeren onder afschot	231.20	m2
		stekkenbak t.b.v. klimkist	401.00	m1

The summary is given below

Total quantity (m^3)	m^2	m^3	kN	tons
structural walls		4,079	101,970	10,197
floors	30,875	9,262	231,560	23,156
total		13,341	333,530	33,353
specific gravity [kN/m3]	25			
weight [kN]	333,530			
weight [tons]	33,353			

Design Variant

Regarding the equivalence of the construction weight the following assumptions – for the construction variant – are made;

Façade: 1.0 kN/m^2, the assumption is that 40% of the façade consists of openings

Core: fire-resistance measures: 100 mm cellular concrete, 600 kg/m^3 (Xella)
Metalstud, 30 kg/m^3 (Gyroproc.be), storey height 3 m.

.

Interior Construction (steel construction)							weight [tons]
façade column (side)	basement	storey 1-12	storey 13-24	storey 25-36	storey 37-48		
section		HD 400x1086	HD 400x634	HD 400x382	HEM 300		
weight per length [kg/m]		1,086	634	382	238		
length [m]		41	40	40	40		
weight [kg]		45,004	25,335	15,265	9,510	95,114	
façade columns (corner)							
section		HD 400x592	HD 400x382	HEM 300	HEM 240		
weight per length [kg/m]		592	382	238	157		
length [m]		41	40	40	40		
weight [kg]		24,532	15,265	9,510	6,274	55,581	
core columns							
section	HD 400x1086	HD 400x1086	HD 400x1086	HD 400x634	HD 400x382		
weight per length [kg/m]	1,086	1,086	1,086	634	382		
length [m]	6	41	40	40	40		
weight [kg]	6,234	45,004	43,397	25,335	15,265	135,233	
core windbracings							
section	IPE 160						
weight per length [kg/m]	16						
length per 3 storeys [m]	106						
weight [kg]	1,667	16				26,670	
integrated floor beams							
section	400x8-300x35-400x20 (A=24900mm2)						
weight per length [kg/m]	195						
length per storey [m]	158						
weight [kg]	30,810	48				1,478,880	
rim joists							
section	IPE 400						
weight per length [kg/m]	66						
length per storey [m]	41						
weight [kg]	2,718	48				130,478	
fire-resisting core measures							530
total weight [kg]						1,921,957	

total weight [ton]				2,452
Interior Construction (concrete floors)				
total weight [kN]			257,825	
total weight [ton]				25,783
total weight in ratio [tons]				18,427
TFC				
diagonals	storey 1-3	storey 4-48		
section	CHS 457x40	CHS 457x40		
weight per length [kg/m]	411	411		
length per member [m]	12	11		
amount of member	32	480		
weight [kg]	159,797	2,122,733	2,282,530	
horizontals	storey 1-48			
section	CHS 457x10			
weight per length [kg/m]	110			
length per member [m]	8			
amount of member	128			
weight [kg]	112,640		112,640	
total weight [kg]			2,395,170	
total weight [ton]				2,395
Total weight entire construction				23,274

14.2 Estimation of Construction Costs

The prices that have been used to calculate the construction costs are given in Table 14.1. For the current design the in-situ concrete carcass price is calculated. For the design variant, the price is based on the steel interior construction with prefabricated concrete floors plus the costs for the trussed façade construction.

In-situ concrete (walls & floors 300 mm)	350 euro/m^2
Steel interior construction with concrete floors	200 euro/m^2
Trussed façade construction	5 euro/kg

Table 14.1: Prices construction material. Bouwen met Staal & v. Herwijnen, 2008.

The prices are multiplied with the amount of construction material to calculate the total costs which are given in Table 14.2 and Table 14.3.

	area [m^2]	price [euro/m^2]	price [euro]
Structural walls (300 mm)	13,596	350	4,758,600
Floors (300 mm)	30,875	350	10,806,250
Total			15,564,850

Table 14.2: Estimation construction costs current design.

	gross floor area [m^2]/weight [kg]	price [euro/m^2]	price [euro]
Interior construction	30,875	200	6,175,000
Trussed façade construction	2,395,170	5	11.975.850
Total			18.150.850

Table 14.3: Estimation construction costs design variant.