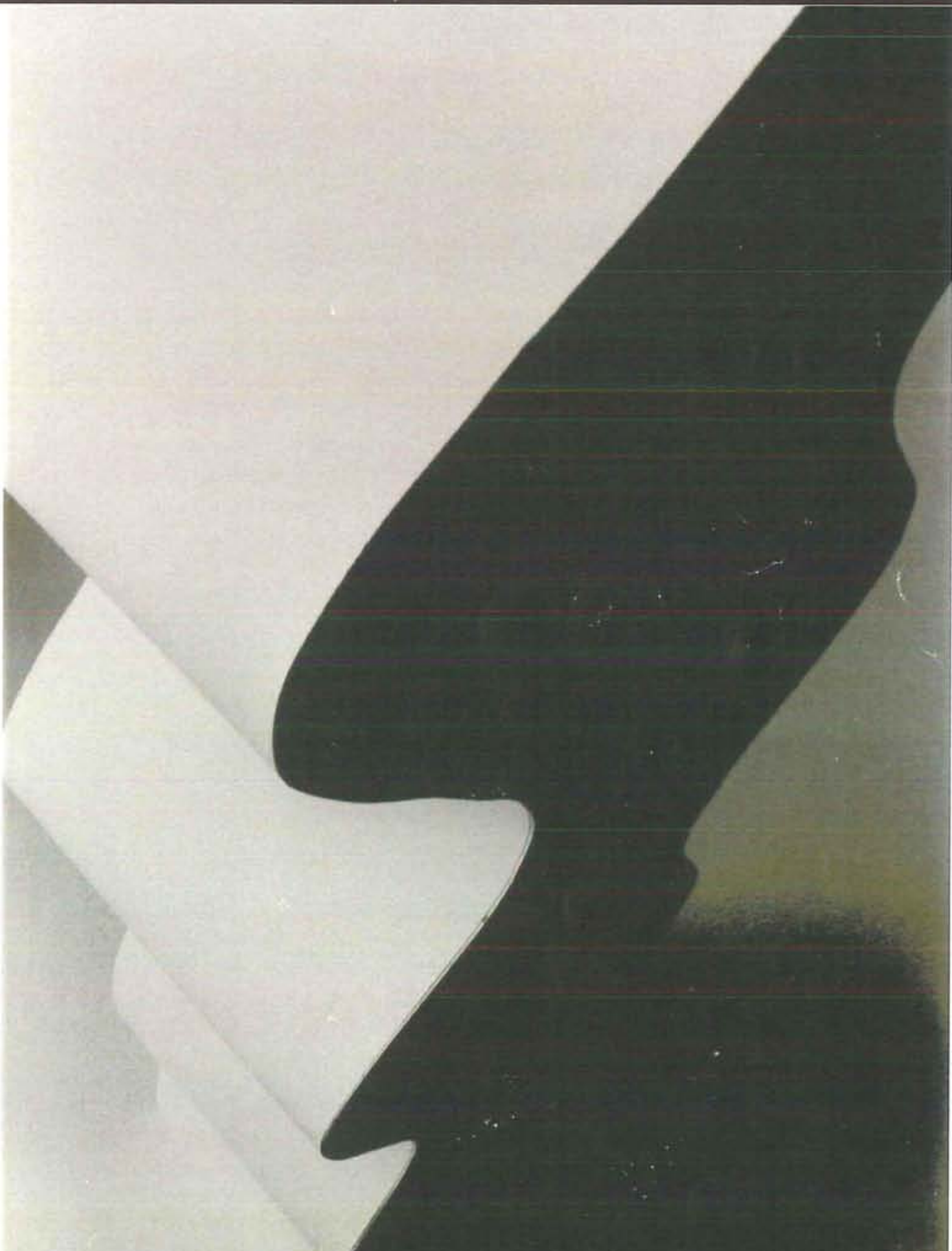
The curved high rise building

case study v.2



J. Thuis

The curved high rise building

Case study

Preface

After an extensive literature investigation followed by structural investigation part I and II, this final report discusses a case study based on the knowledge gained during the last months writing the thesis. This case study forms the last subject from the list of topics that were to be discussed according to the initial workplan.

During the writing of the thesis I had a lot of help from a number of people. First of all I would like to thank the committee, in particular Ir. Raven, for their technical support. And secondly but not less important, for the personal support I have to thank my parents and Ilse.

Committee:
Prof. dipl- ing. J.N.J.A. Vamberský, *chairman*
Prof. ir. L.A.G. Wagemans
Dr. ir. J. Paul MBA
Ir. W.J. Raven

August 2003
Jeroen Thuis

Symbols

A	*Amplitude*
b	*Building width*
d	*Wall width*
EA	*Yielding stiffness*
EI	*Bending stiffness*
γ	*Shearing angle*
F	*Force*
GA	*Shear stiffness*
GFA	*Gross floor area*
h	*Building height*
I	*Moment of inertia*
k	*Foundation rotation stiffness*
l	*Story height*
M	*Bending moment*
N	*Normal force*
n	*n-value 2^{nd} order effect*
η	*Amplitude factor*
q_g	*Gravity load*
q_w	*Wind load*
q_b	*Buckling load*
σ	*Compression/Tension*
u	*Deflection*
V	*Shear force*
w	*Deflection*
φ	*Rotation*
z	*Lever arm*

Table of Contents

1 Case study

The case study will be the final result of the thesis and therefore has to meet the final objective as stated in the initial workplan:

"The structural design of an alternative S-shaped skyscraper"

During the whole process of literature and structural investigation this objective has been defined more precisely and the objective of designing more than an S-shaped building was added: an extreme, uncommon curved building. Even more, some extra (secondary) objectives were stated as well. The secondary objectives that were found during the structural investigation process are:

1. No extra bending moment at $x = 0$
2. Smallest possible initial top deflection
3. No tension in the core

In the previous report "Structural investigation part II", the secondary objectives as stated above were met. This case study will provide the answer to the project objective: the realization of an extreme, uncommon and curved shaped building. Secondary objective 1, 2 and 3 will be strived after but are not a primary goal. As the results of the structural investigation are based on assumed dead loads and building measurements, the appliance of real measurements and materials may change the outcome as found in the structural investigation. During the case study design process, the secondary objectives will be checked in practice and the possible distortions will be clarified.

From the structural investigation it is clear that the most impressive building shape can be created by applying the reverse damped structure, combined with multiple curves. Using this

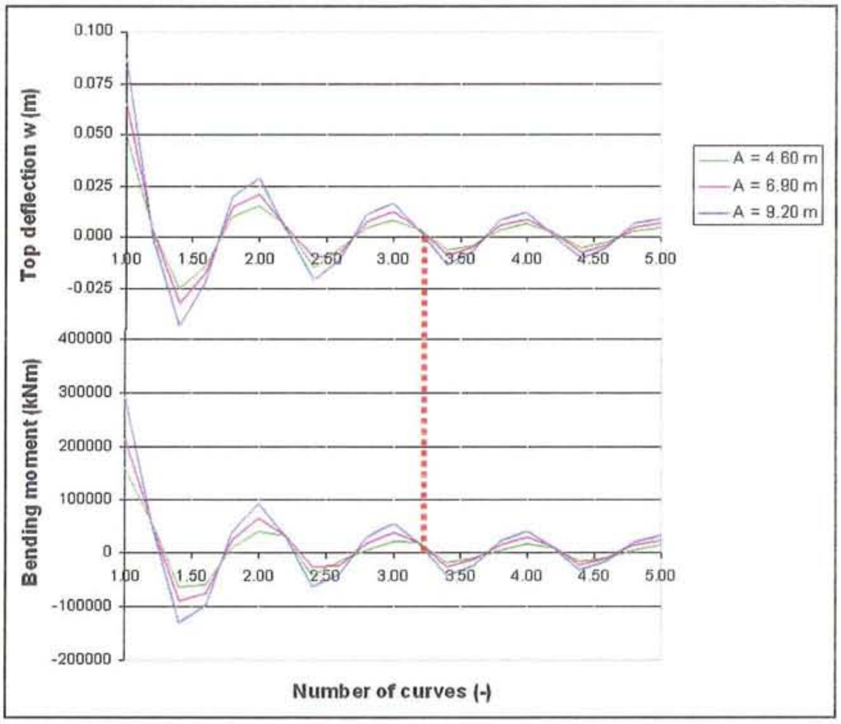


figure 1.1 Lateral moment at x=0 and top deflection

system as a starting point, the number of curves can be optimized in order to satisfy the other three (secondary) objectives stated above. In the structural investigation figure 1.1 showed the optimal combinations of number of curves, top deflection and lateral moment at ground level. The best building behaviour is achieved with 1.22, 1.72, 2.22, 2.72, etc curves. At these points, the initial top deflection and the lateral moment at ground level are minimized.
Which of these configurations results in the most extreme curved shape is arbitrary[1]. If the applied number of curves is too large, the resulting building shape tends to become ridicule, whereas a small number of curves fails to create the searched for extreme shape. An acceptable shape is achieved by applying 3 to 4 curves. With these number of curves the uncommon curved building is created and the secondary objectives can be met as well; resulting in a minor lateral moment at ground level, no tension in the core and a minor top deflection. For the case study, the number of curves will be set to 3.22 (see figure 1.1), a figure that lies at an optimum regarding the building behaviour.

The resulting building shape with 3.22 curves is presented in the drawing on the right. The reverse damped shape is clearly visible and creates the uncommon shape mentioned in the project objective statement.

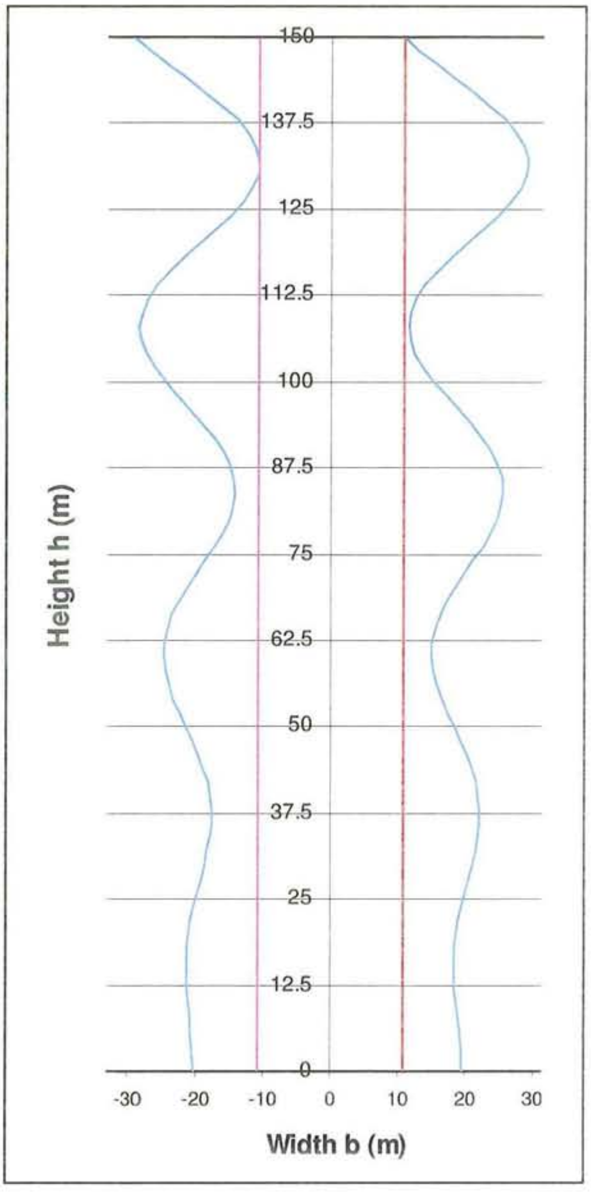


figure 1.2 Case study building shape, 3.22 curves

[1] For the optimal building shapes diagrams, see Appendix F

1.1 General building parameters

The list below shows the general building parameters for the case study building. The figures below are the starting point and will be used during the whole design process.

Shape (mathematical): $$y = A\sin\left(-\frac{2\pi}{h/v}\cdot(x+\beta)\right)\cdot\left(C\frac{x}{h}+B\right)$$

Amplitude	A:	9.2 m
Height	h:	150 m
Phase shifting	β:	0
Number of curves	v:	3.22
Factor	C:	1.20
Factor	B:	0

Height-to-width ratio core:	7
Width core:	21.5 m
Building width:	40 m
Story height:	4 m
Number of stories:	37
Initial office length:	7.25 m
Hall width:	2 m

Wind load:	2 kN/m^2
Variable floor load:	2.5 kN/m^2

The width of both core and building are rounded to practical measurements. According to the data above, the floor plan grid is presented in figure 1.3. This is the initial grid at ground level, the starting point. Due to the curved shape, the office length on the left and right side of the building will differ from story to story. The fixed dimensions are printed in black, the variable dimensions in red.

figure 1.3 Initial grid at ground level

1.2 Functional design

1.2.1 Floor plan

Although it is not the goal of this thesis it is still important to give an impression of the functional design of the building. The grid, as presented in the previous paragraph, is used to design a functional layout. The space around the central core is reserved for office area since the building will be used as an office building only. A problem with the curved shape is that no single floor layout will be the same (see figure 1.4), an exclusive layout has to be made for every story.
Because of this constantly shifting floor plan, it will be very difficult to employ structural members inside the building. On one level a column may be perfectly placed at for example the corner of an office, a story below it may well pierce right through the office.
The typical functional layout that is presented in figure 1.5 is designed in a way that no vertical structural members, apart from the central core, are employed inside the building. All vertical members will be located at the outside perimeter of the building. By doing so, the building will have no restrictions in flexible working space. Walls can be moved and offices can be adapted to the customer's demands.

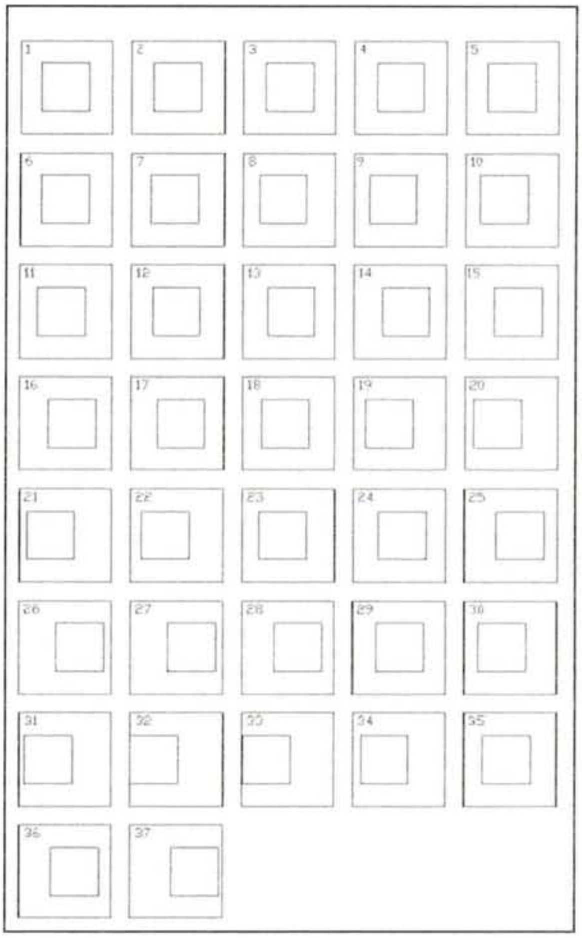


figure 1.4 Floor plan shifting

1.2.2 Elevators

In the literature investigation the possibility of an inclined elevator system was analyzed and found to be possible but with a large number of design problems occurring. An inclined elevator system is no longer necessary with the straight vertical core that will be applied to the building. The central core is a perfect location for vertical transport in the building. Because the building is 150 m high and has 37 stories, it is logical to divide the building into two zones regarding vertical transport. There is no need for shuttle elevators that start at ground level and only stop at the skylobbies. Those systems are usually applied to skyscrapers with more than 50 stories. The transportation scheme for the case study building is drawn in figure 1.6. There will be two elevator groups, one serving level 1-19 and elevator group 2 is serving story

19-37. The elevators for zone II start at ground level and then only stop at respectively zone II.

Now the elevator configuration is known, a proper elevator plan can be designed[2]:

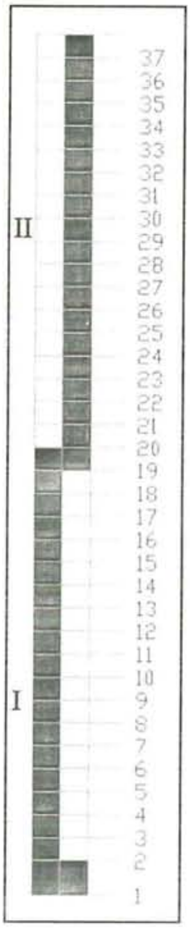


figure 1.6 Elevator configuration

Office area:	$40 \cdot 40 - (21.5+4)^2 = 950$ m^2/floor
Number of floors/zone:	18
Number of persons:	$\dfrac{Office\_area}{m^2 p.p.} = \dfrac{950}{8} = 119$ persons/floor
	$119 \cdot 13 = 1428$ persons/zone
Travel quality class 2:	$t_{wait} \leq 80\,\text{s}$
Elevator velocity:	1.6 m/s

Using a Schindler design program, the design diagrams and NEN 5081 (see appendix E), the optimal elevator plan would be to employ 2 groups elevators at every zone. Each group will consist of 2 elevators with a capacity of 1200 kg.

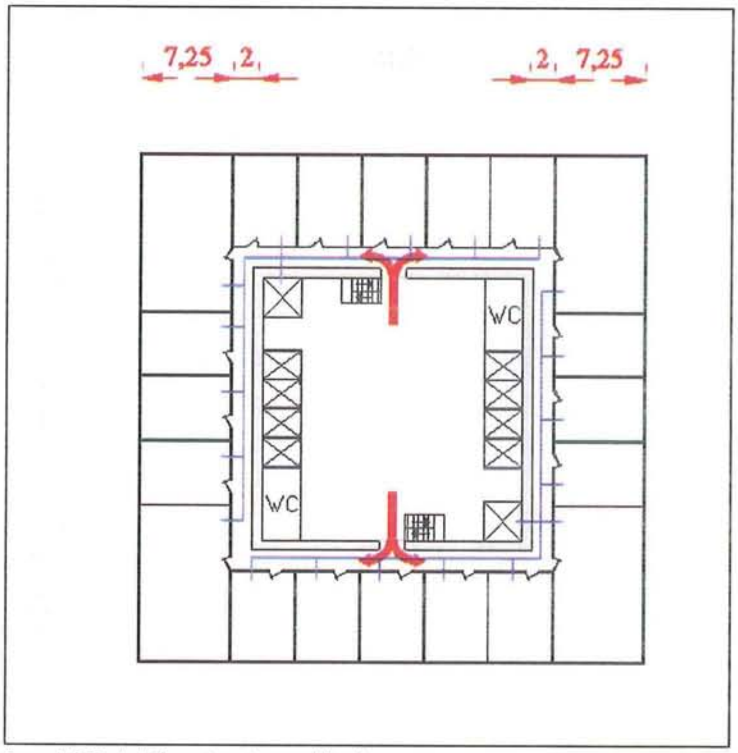


figure 1.5 Typical floor plan at ground level

In figure 1.5 a possible floor layout is drawn. All 8 elevators start at ground level, four for each zone.

The space inside the core that is left over can be used for toilets and vertical shafts for ducts and staircases. Other functions, like storage and meeting rooms can be located inside the core as well, as they do not need direct daylight. The drawing is only an example of a possible floor layout. A more detailed analysis of the functional behaviour of the building can be found in chapter 3.

[2] Elevator plan according to NEN5080 and 5081

2 Internal force flow

A quick recapitulation[3] of the structural investigation tells that the internal force flow can be modellised as presented in figure 2.1.

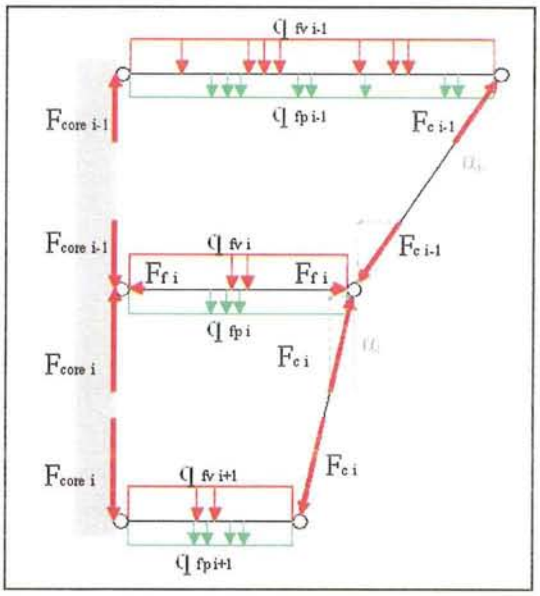


figure 2.1 Internal force flow

In the remainder of this paragraph the structural system will be divided into four components:

1. Floors
   - Permanent vertical load at floor level i: $\gamma_g \cdot q_{fpi}$
   - Variable vertical load at floor level i: $\gamma_q \cdot q_{fvi}$
   - Peak load at floor level i: $F_{ciV} = F_{ci-1V} + \frac{1}{2} \cdot (\gamma_g \cdot q_{fpi} + \gamma_q \cdot q_{fvi}) \cdot l_i$

2. Beams
   - Hor. comp. in inclined column i: $F_{ciH} = \tan(\alpha_i) \cdot F_{ciV}$
   - Horizontal force in beam i: $F_{fi} = F_{ciH} - F_{ci-1H}$

3. Inclined columns
   - Vert. comp. inclined column i: $F_{ciV} = F_{ci-1V} + \frac{1}{2} \cdot (\gamma_g \cdot q_{fpi} + \gamma_q \cdot q_{fvi}) \cdot l_i$
   - Normal force in inclined columns i: $F_{ci} = \dfrac{F_{ciV}}{\cos(\alpha_i)}$

[3] For a more detailed overview of the internal force flow see chapter 2.2 of Structural investigation part II

4. Core

- Vertical load at point i: $F_{core\cdot i1} = F_{core\cdot i-1} + \frac{1}{2} \cdot \left(\gamma_g \cdot q_{fpi} + \gamma_q \cdot q_{fvi}\right) \cdot l_i$
- Perm. + variable load core at point i: $F_{core\cdot i2} = q_{core} \cdot (h - x)$
- Total vertical force in core at point i: $F_{core\cdot i} = F_{core\cdot i1} + F_{core\cdot i2}$
- Horizontal force at point i: $F_{fi\_total} = F_{fi\_left} + F_{fi\_right}$

2.1 Floors

As mentioned before, the story height of the building will be 4 m. This distance should provide enough space for the construction elements and still leave room for the functions in the building.
With a story height of 4 m, the building will contain 37 stories. At ground level the story height is 6 m to complete the total of 150 m.

Due to the curved shape of the building lateral forces in the floors will occur. Whether this force is a compressive or a tensile force depends on the position of the floor. The horizontal components of the normal forces in the inclined columns have to be led through the floors to the central core as shown in figure 2.1 and figure 2.2. In figure 2.2 the building is stretched sideways in order to improve the readableness. The real shape can be found in figure 1.2.

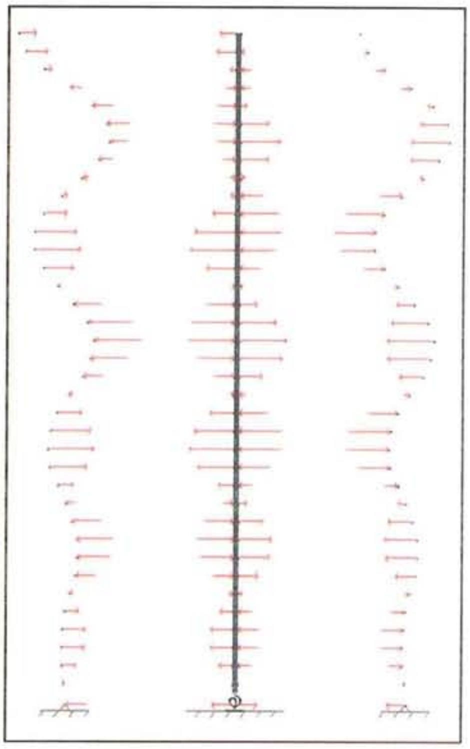


figure 2.2 Lateral force components

The floors will have to bear this extra compressive or tensile force, besides the force initiated by the wind load: $F_{wind} = p_{wind} \cdot b \cdot h_{story} = 2 \times 40 \times 4 = 320\,\text{kN}$ (force initiated by the wind, per floor).

As mentioned before the extra lateral force F_{fi} at floor level i is calculated by: $F_{fi} = F_{ciH} - F_{ci-1H}$. This means that the resulting horizontal force is calculated by subtracting the horizontal component of the normal force in inclined column i and the horizontal component of the normal force in column i-1 (see figure 2.1).

In the diagrams of figure 2.3 the lateral forces (excl. F_{wind}) for every floor level are drawn[4]. It is clear that the extra lateral forces in the floors due to the curved shape of the building are significantly larger than the force caused by the wind load. The lateral force in the floors can grow up to 7324 kN whereas the wind causes a constant force of only 320 kN. This is a factor that has to be reckoned with during the design of the floors and the beams (see chapter 2.2). The floors or/and beams will have to bear this extra normal force.

[4] A list of the exact lateral forces in the floors can be found in appendix G 1

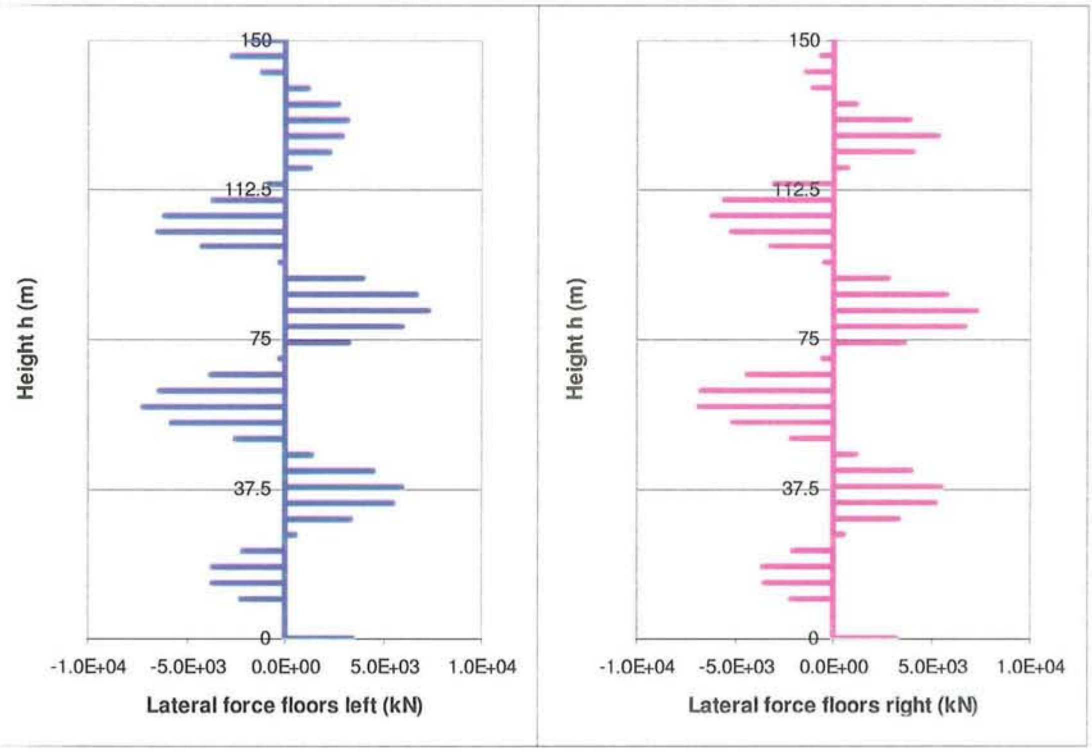


figure 2.3 Lateral forces on the left (↑) and right (↑) side of the building due to normal forces in the inclined façade columns

Although the floors and beams will have to bear the extra lateral force as mentioned above, the normative force will be caused by the perpendicular floor load on the floors and beams. The amplitude of the shape-function in set on 9.2 m as noted in paragraph 1.1. This is the amplitude, which means that the total floor length can grow to 2 x 9.2 = 18.4 m.

Because of the reverse damped shape, the floor length will not reach to this length in the lower parts of the building, but in the upper part of the building on the other hand the floor length will measure up to the maximum of 18.4 m. In the table on the next page, the span of the floors at every floor level is listed.

If the rule of thumb $h_b \approx \frac{1}{25} \cdot span$ is used for the construction height of the beams, problems will occur with spans larger than about 12.5 m. A span of 12.5 m will result in a construction height of the beam of approximately 500 mm.

Since the story height of the building is set on 4 m, this is an acceptable construction height. But, when the spans grow larger (in red in table 2.1), the construction height of the beams may very well become a problem.

A solution to decrease the large span (and the construction height) can be adding an extra row of columns inside the building, for instance halfway of the beam.

That is, in case of a normal orthogonal building. In the curved building, there is no room for vertical columns inside the building without piercing through the façade at various levels (see figure 2.4). By placing extra vertical columns in the building, the initial original concept is lost and the extreme idea fades away.

The solution for the large spans has to be found inside the building in order to stay with the original concept.

height x	floor span left	floor span right
m	m	m
0	9.67	8.72
6	9.98	8.40
10	10.38	8.00
14	10.66	7.72
18	10.59	7.79
22	10.06	8.33
26	9.13	9.25
30	8.07	10.32
34	7.23	11.15
38	6.99	11.40
42	7.56	10.83
46	8.92	9.47
50	10.77	7.62
54	12.60	5.78
58	13.85	4.54
62	14.02	4.37
66	12.92	5.47
70	10.72	7.67
74	7.94	10.45
78	5.35	13.03
82	3.74	14.65
86	3.68	14.71
90	5.34	13.05
94	8.40	9.99
98	12.09	6.29
102	15.41	2.98
106	17.36	1.03
110	17.28	1.11
114	15.03	3.36
118	11.10	7.29
122	6.48	11.90
126	2.46	15.93
130	0.20	18.18
134	0.47	17.92
138	3.34	15.05
142	8.16	10.23
146	13.68	4.71
150	18.39	0.00

table 2.1 Floor span left and right

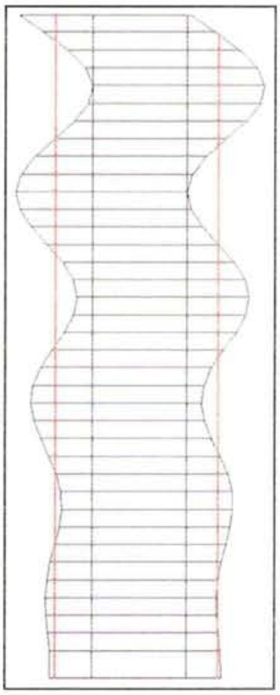


figure 2.4 Piercing columns

To see whether the rule of thumb of $h_b \approx \frac{1}{25} \cdot span$ provides adequate measurements for the construction height of the beams, a calculation will be made below. From this rule of thumb it is clear that the construction height may become a problem. In order to start with well prepared preconditions, the following measures can be taken:

1. Small construction height of the floors.
   By using a floor with a relatively small construction height, the height of the supporting beam can be larger, more space is available.

2. Light weight floor.
   A lighter floor will result in a smaller bending moment in the supporting beam, so the construction height can be lowered.

3. Relatively small centre-to-centre distance of the supporting beams.
   Due to the smaller c.t.c. distance the floor load will be spread on more beams, resulting in a smaller bending moment and thus a smaller minimum construction height.

The starting point is the columns and beams grid on ground level in figure 2.5. With this grid

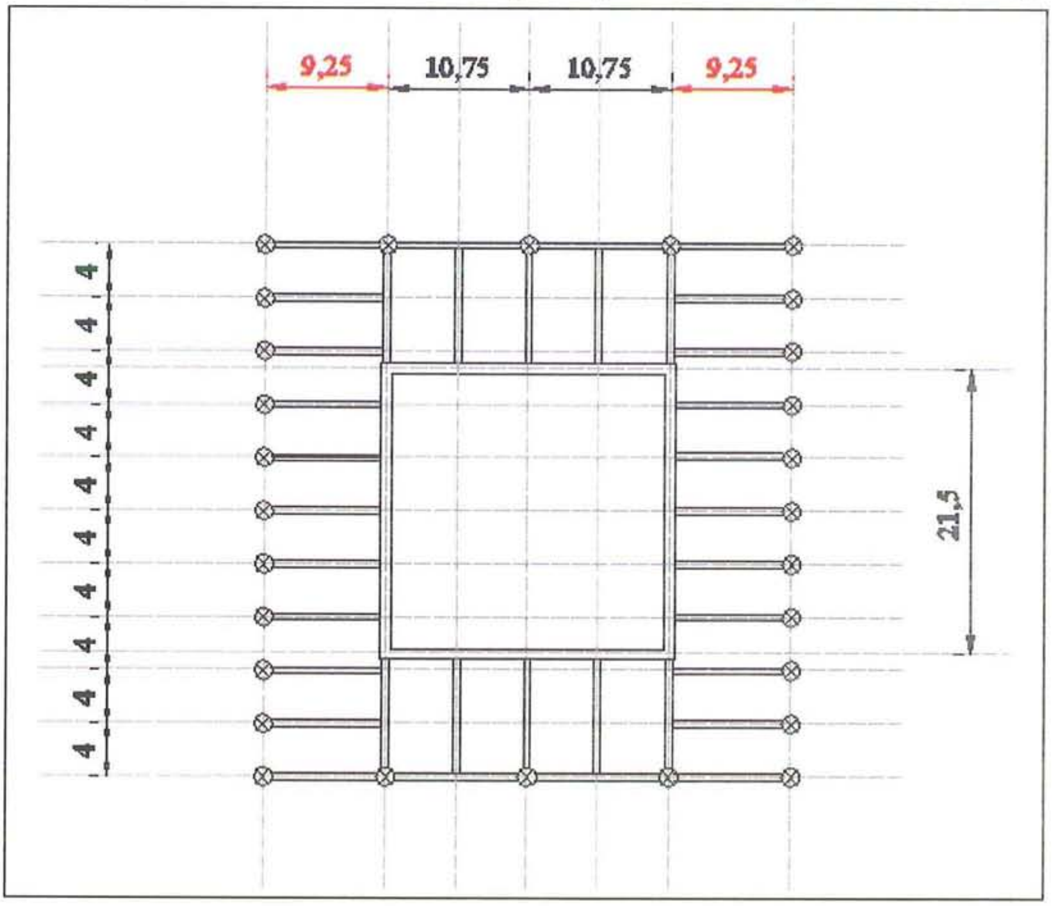


figure 2.5 Beam and column grid at ground

the centre-to-centre distance of the large span beams is set on 4 m. This is still a reasonable distance and it can be combined with the positioning of the inclined façade columns. An extra criterion that will influence the choice of the right floor system is the fact that every floor at every floor level has a different width (as shown in table 2.1):

4. A floor system that can be easily adjusted to the typical measurements of every floor level

2.1.1 Floor types

In the concrete and steel industry, numerous floor types are available. In search for a light floor system with a small construction height, some floor types are compared in the table below.

	Construction height (mm)	Weight (kN/m^2)	Flexibility
Concrete hollow-core slab A150	180	2.64	-
Concrete-steel floor Comflor 70	120	1.88	+
Bubbledeck floor	230	3.70	-

table 2.2 floor types

Concrete hollow-core slab A150

The concrete hollow-core slab A150 floor is a pretensioned concrete floor that is able to realize a span of more than 8 m. This span is not necessary with the current configuration, but may be useful if the c.t.c.-distance is enlarged at a later stage.

Due to the hollow cores in the concrete slabs, a weight reduction of 30-40% can be achieved compared to in situ concrete slabs. The construction height, including compression layer will be 180 mm.

The width of a hollow-slab unit is standard set on 1200 mm. Because of the altering floor position and the straight core, the floor will have to be completed by a slab with a unique width, or by in situ poured concrete at every floor level. This inflexibility will increase the cost of the building, either because of the unique hollow slabs that have to be constructed for every floor level or by the extra man-hours completing the floor with in situ poured concrete.

Concrete-steel floor Comflor 70

The concrete-steel floor is constructed by attaching a corrugated steel plate to the supporting beams. Once the steel plate floor is realized, a concrete layer is poured on top of it. The combination of the two materials will result in a light weight, stiff floor with a relatively small construction height.

The flexibility of the floor regarding the constantly changing floor length of the building is rather positive. The corrugated steel plates can be cut relatively easy, so the unique measurements of the floors should not lead to a significant increase in cost.

Bubbledeck floor

Similar to the concrete hollow-core slab, relatively large spans can be realized with the bubbledeck system. The advantage of this system is the ability to bear the load in two directions instead of one. The bubbledeck floor consists of a concrete slab with plastic spheres attached to it. Once the slab is tackled to its position, a concrete compression layer will be poured on top of it. Bubbles can be left out at places where more concrete is needed, for instance to increase the shear capacity.

If one compares the floor types in table 2.2 the concrete-steel floor seems to have the best properties for the construction of the floors in the curved building. It combines a relatively small construction height with a low weight and flexibility, the combination that was searched for.
In figure 2.6 the concrete-steel system is shown. The corrugated steel plate is fixed to the steel beam by tacks or by tap bolts in the bottom flange of the plate. As soon as the steel plates are fixed to the beams, the floor can be used as a safe work floor. The steel floors can be regarded as discs during the construction of the building and in combination with the substructure the steel plates will improve the stiffness of the building under construction.

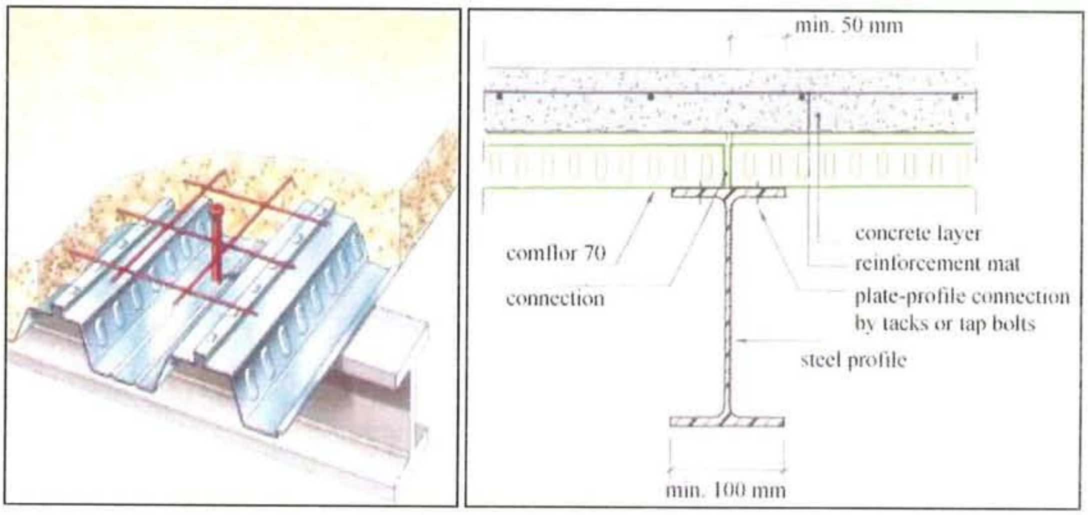


figure 2.6 Comflor 70 floor principle (l), section (r)

As mentioned before, the ever changing floor length is a problem because one has to construct a floor with a unique length for every floor level. This is a problem that cannot be avoided by choosing a certain floor type, but the concrete-steel floor has the advantage that it can be cut quite easily. The peace that needs to be cut of can be placed at the opposite side of the core and will not be lost.

Lowered ceilings and ducts can be hung on the 15 mm high swallow tail shaped ribs on the steel plates, as shown in figure 2.7. The ducts, cables and supporting beams can be kept out of sight by applying this suspended ceiling.

figure 2.7 connectors for suspended ceilings / ducts

2.1.2 *Floor stiffness*

The stiffness of the floors has to be secured in order to prevent the floors from deforming on a large scale. The wind load on a single floor will cause the floor plates to shift as indicated with the red line in figure 2.8.
This deformation of the floors is caused by a bending moment M_d:

$$M_d = \frac{1}{2} \cdot q_w \cdot l_f^2 = 0.5 \cdot 8 \cdot 18.39^2 = 1353\,\text{kNm}$$

In order to make the floor act like a stiff disc, this moment can be absorbed by applying tension rods from the core to the outside perimeter of the building. This is drawn in the down most picture of figure 2.8.

The bending moment is split into:

$$F_t = \frac{M_d}{18.39} = 74 \text{ kN}$$

The force in the tension rod will be:

$$F_{r1} = \frac{F_t}{\cos(\alpha)} = \frac{74}{0.87} = 85 \text{ kN}$$

$$F_{r2} = 42 \text{ kN}$$

The minimum needed section surface of the rods will have to be:

$$A_{\min} = \frac{F_r}{f_s} = \frac{85 \cdot 10^3}{235} = 362 \text{ mm}^2$$

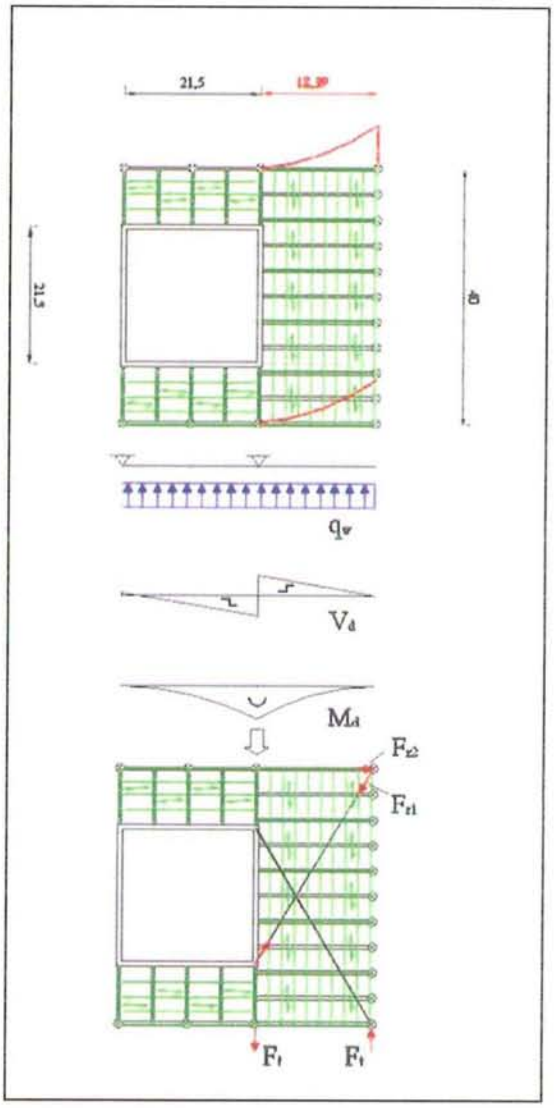


figure 2.8 Floor diaphragm-behaviour

The tension rod could be designed as a 50 x 75 mm steel strip, connected to the beams in the layout as projected in figure 2.8↓.

2.2 Beams

First the beams with the largest span will be calculated (I). The normative floor level lies at the 33rd floor, at 130 m. At this location the span of the beams from the central core to the inclined façade columns is 18.39 m. According to the rule of thumb $h_b \approx \frac{1}{25} \cdot span$, a beam with a height of 735 mm should be chosen as an indicative dimension.

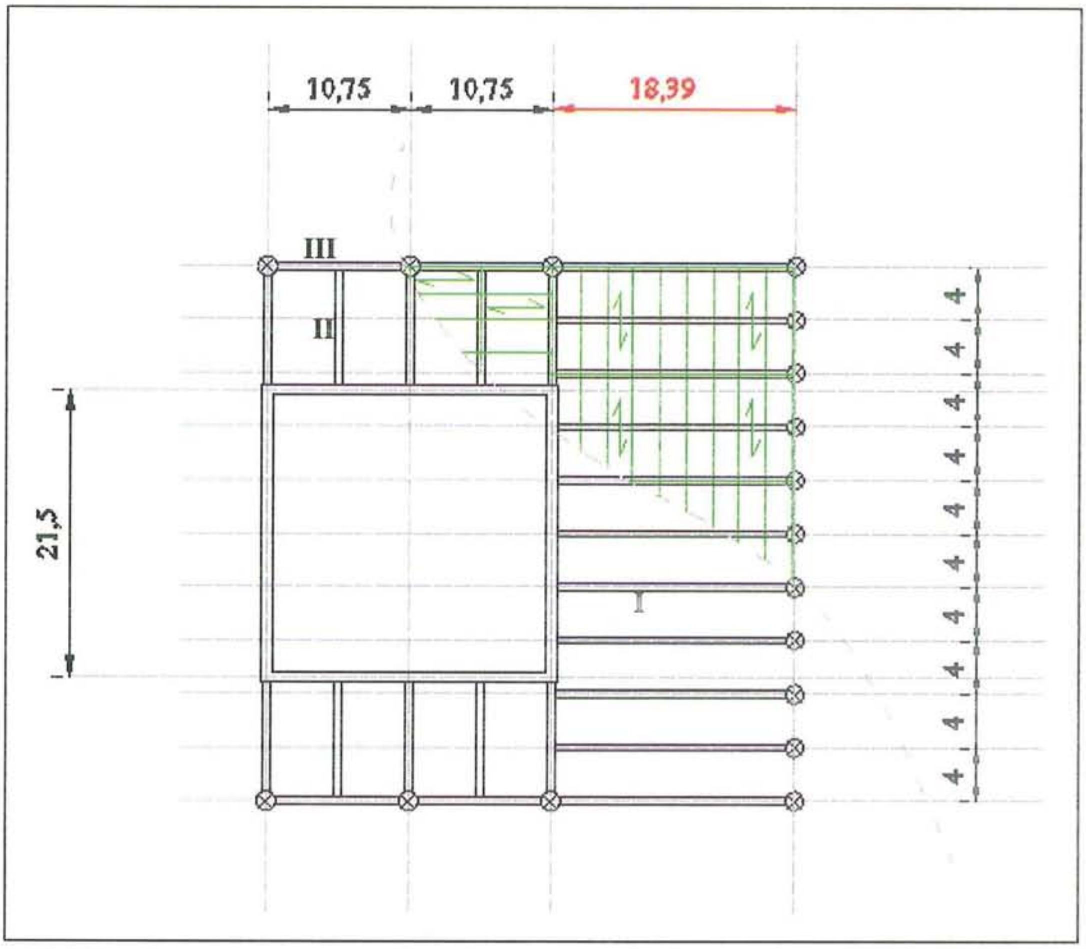
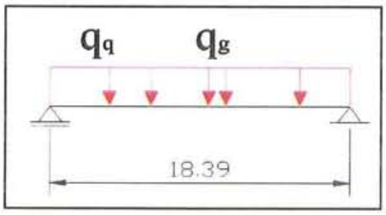


figure 2.9 Floor plan, level 33, 130 m



Because of the light floors and the small c.t.c. distance of the beams, the rule of thumb may provide a construction height that is too large. The construction height that is used to start the calculation with is $h_b \approx \frac{1}{30} \cdot span$. This results in a construction height of 600 mm, for instance an HE600A profile.

Span:		18.39	m
Floor span:		4	m
Floor gravity load:	$p_{g,floor,rep} =$	1.88	kN/m^2
	$q_{g,floor,rep} =$	1.88 x 4 = 7.52	kN/m
Variable load	$p_{q,rep} =$	2.5	kN/m^2
	$q_{q,rep} =$	2.5 x 4 = 10	kN/m
Beam:	$q_{g,HE600A,rep} =$	1.78	kN/m
	$W_{y;d} =$	4790 10^3	mm^3
	$f_{y;d} =$	235	N/mm^2

$q_d = \gamma_g \cdot q_g + \gamma_q \cdot q_q$ = 1.2 x (7.52 + 1.78) + 1.5 x 10 = 26.16 kN/m

Strength Requirement (u.g.t): $\dfrac{M_{y;s;d}}{M_{y;u;d}} \leq 1$

$M_{y;s;d} = \dfrac{1}{8} \cdot q_d \cdot l_{beam}^2$ = 0.125 x 26.16 x 18.39^2 = 1106 kNm

$M_{y;u;d} = f_{y;d} \cdot W_{y;d}$ = 235 x 4790 10^3 = 1126 kNm

$\dfrac{M_{y;s;d}}{M_{y;u;d}} = \dfrac{1106}{1126} = 0.98 \leq 1$, the HE600A profile is sufficient to bear the load.

The $\dfrac{M_{y;s;d}}{M_{y;u;d}}$ ratio of 0.98 still leaves some room for improvement. The construction height of 600 mm could be reduced by using a smaller profile, with thicker flanges. A HE450M profile results is the following outcome:

Beam:	$q_{g,HE450M,rep} =$	2.63	kN/m
	$I =$	131484 10^4	mm^4
	$W =$	5500 10^3	mm^3
	$f_{y;d} =$	235	N/mm^2

q_d = 1.2 x (7.52 + 2.63) + 1.5 x 10 = 27.36 kN/m

$$M_{y;s;d} = \frac{1}{8} \cdot q_d \cdot l_{beam}^2 = 0.125 \text{ x } 27.18 \text{ x } 18.39^2 = 1149 \text{ kNm}$$

$$M_{y;u;d} = f_{y;d} \cdot W_{y;d} = 235 \text{ x } 5500\ 10^3 = 1293 \text{ kNm}$$

$$\frac{M_{y;s;d}}{M_{y;u;d}} = \frac{1149}{1293} = 0.89 \leq 1$$

The HE450M profile results in a (close to) optimal construction height of the beams at level 33. A HE500B steel profile will result in a ratio of 1.10, and thus be insufficient to bear the load.

Sagging	Requirements (b.g.t.):	$u_2 \leq 0.003 \cdot l_{rep} = 0.003 \cdot 18.39 = 55.2$ mm $u_{end} \leq 0.004 \cdot l_{rep} = 0.004 \cdot 18.39 = 73.6$ mm

Where $u_2 =$ Sagging due to variable load

$u_i =$ Initial sagging due to permanent load

$u_c =$ Camber

$u_{end} = u_i + u_2 - u_c$

$$u_i = \frac{5}{384} \cdot \frac{(q_{g,floor} + q_{g,HE450B}) \cdot l_{rep}^4}{EI} = \frac{5}{384} \cdot \frac{(7.52 + 2.63) \cdot (18.39 \cdot 10^3)^4}{2.1 \cdot 10^5 \cdot 131484 \cdot 10^4} = 54.7 \text{ mm}$$

$$u_2 = \frac{5}{384} \cdot \frac{q_q \cdot l_{rep}^4}{EI} = \frac{5}{384} \cdot \frac{10 \cdot (18.39 \cdot 10^3)^4}{2.1 \cdot 10^5 \cdot 131484 \cdot 10^4} = 53.9 \text{ mm} \leq 55.2 \text{ mm}$$

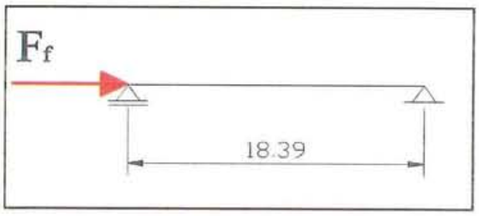
$$u_{end} = u_i + u_2 = 54.7 + 53.9 = 108.6 \text{ mm} \geq 73.6 \text{ mm}$$

The sagging of the beam will be too large. The requirement of 73.6 mm is exceeded by 35 mm. This has to be corrected by introducing a camber in the beam of : $u_c = 35$ mm.

Buckling

$$\text{Requirement: } \frac{F_{c;s;d}}{\omega_{buc} \cdot F_{c;u;d}} \leq 1$$

In the beginning of chapter 2.1, the extra lateral forces at every floor level were mentioned. These lateral forces will have to be led through the beams to the central core. If one states that in the most extreme case the beams have to bear the total normal force and the concrete-steel floor does not prevent the beam from buckling, the buckle length of the beam is 18.39 m (maximum length at level 33).



Buckle length l_{buc}:	18.39	m
I:	131484 10^4	mm^4
Section surface HE450M A:	33540	mm^2
Curve (table 23 NEN6770):	C	-
$f_{y;d}$:	235	N/mm^2

$$N_{c;s;d} = \frac{3918}{10} = 391.8 \text{ kN (10 beams)}$$ [5]

$$N_{c;u;d} = A \cdot f_{y;d} = 33540 \cdot 235 = 7882 \text{ kN}$$

$$F_E = \frac{\pi^2 EI}{l_{buc}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 131484 \cdot 10^4}{18390^2} = 8058 \text{ kN}$$

$$\lambda_{rel} = \sqrt{\frac{N_{c;u;d}}{F_E}} = \sqrt{\frac{7882}{8058}} = 0.99 \quad \rightarrow \quad \omega_{buc} = 0.59$$ [6]

$$\frac{N_{c;s;d}}{\omega_{buc} \cdot N_{c;u;d}} = \frac{391.8}{0.59 \cdot 8058} = 0.08 \leq 1$$

So, even under extreme circumstances there is no danger in buckling of the beams due to the lateral loads from the inclined columns. In reality, the beams are fixed to the concrete-steel floors by tacks or by tap bolts and thereby even more resistant against buckling.

The combination of sagging and buckling remains within the set margins, roughly one can say that: $\frac{M_{y;s;d}}{M_{y;u;d}} + \frac{N_{c;s;d}}{\omega_{buc} \cdot N_{c;u;d}} = 0.89 + 0.08 = 0.97 \leq 1$

[5] For an overview of the lateral forces see appendix G 1
[6] Calculation according to NEN6770

The calculations above were for the normative beam at level 33. The HE450M steel profile is a heavy profile, but it only has to used at a few floor levels. The same calculation as above has been made for the beams at the remaining floor levels. The results of these calculations can be found in appendix G 3-4.
At some levels the minimum beam dimensions shrink to a HE200A or even smaller profile because of very small story lengths. Although it is not necessary from a mechanical point of view, the minimum applied beam type will be HE300A. This because of construction purposes. The columns that the beams will be connected to have a width of 300 mm, in order to design a proper connection it would be convenient if the beams have the same width.

The beams that remain the same size irrespective of the floor level are indicated as II and III in figure 2.9. For beam type II the HE340A profile is sufficient to bear the load, whereas for beam type III the HE360A profile will be applied[7].

[7] For a calculation of beams type II and III see appendix G5-6

2.3 Columns

2.3.1 Inclined façade columns

The story height in the building is set on 4 m, with the exception of the 1st floor which will be 6 m high. Because the reverse damped shape of the building is not repetitive, no single inclined column in the façade of the building will have the same length or angle. This implies that the buckle length of these components will differ from column to column as well.
The angle of the different columns is calculated with the derivative of the shape-function:

$$\frac{dy}{dx} = -\frac{2\pi}{h/_{v}} \cdot A\cos\left(-\frac{2\pi}{h/_{v}} \cdot (x+\beta)\right) \cdot \left(C\frac{x}{h} + B\right) + A\sin\left(-\frac{2\pi}{h/_{v}} \cdot (x+\beta)\right) \cdot \frac{C}{h}$$

The resulting angles and lengths of the inclined façade columns are presented in the table below.

height	angle	Inclined column length		height	angle	Inclined column length
m	°	m		m	°	m
0	0.0	6.0		78	29.0	4.6
6	-5.3	4.0		82	12.7	4.1
10	-5.5	4.0		86	-11.5	4.1
14	-2.0	4.0		90	-31.5	4.7
18	4.2	4.0		94	-41.5	5.3
22	10.7	4.1		98	-42.6	5.4
26	14.7	4.1		102	-34.8	4.9
30	14.3	4.1		106	-14.1	4.1
34	8.4	4.0		110	16.7	4.2
38	-2.1	4.0		114	38.8	5.1
42	-13.9	4.1		118	48.2	6.0
46	-22.7	4.3		122	48.6	6.0
50	-25.9	4.4		126	39.6	5.2
54	-22.2	4.3		130	15.0	4.1
58	-10.8	4.1		134	-22.0	4.3
62	6.6	4.0		138	-45.1	5.7
66	23.2	4.4		142	-53.5	6.7
70	33.0	4.8		146	-53.3	6.7
74	35.2	4.9		150		

table 2.3 inclined columns, angle and length

The total normal force in the inclined columns is initiated by the vertical load on the floors (see figure 2.1). This vertical force at level i is calculated by: $F_{ciV} = F_{ci-1V} + \frac{1}{2}\cdot\left(q_{fvi} + q_{fci}\right)\cdot l_i$

The total normal force of the columns in the façade at level i is then: $F_{ci} = \frac{F_{ci1V}}{\cos(\alpha_i)}$. In the diagram below, the total normal force in the inclined columns on the left and the right side of

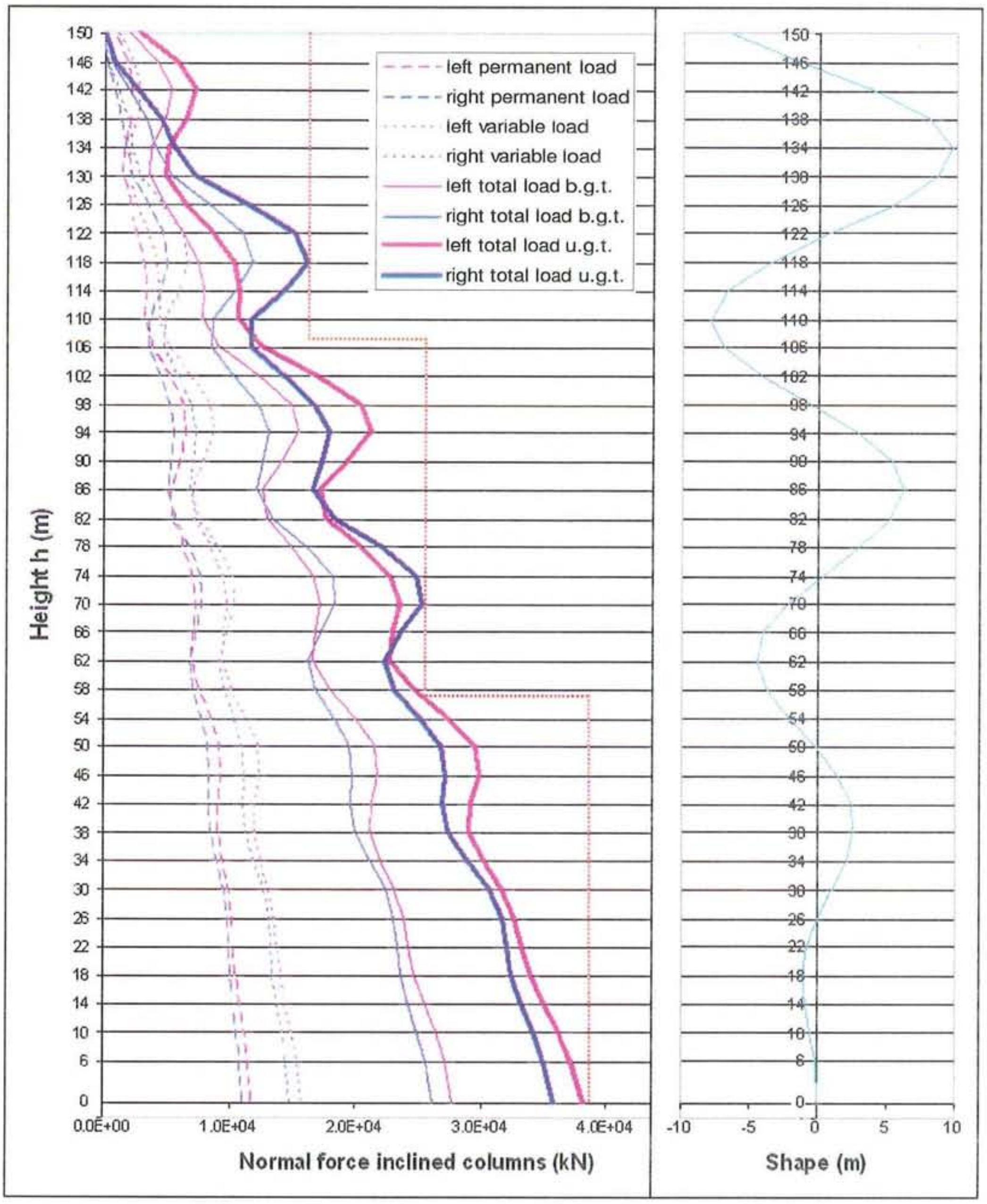
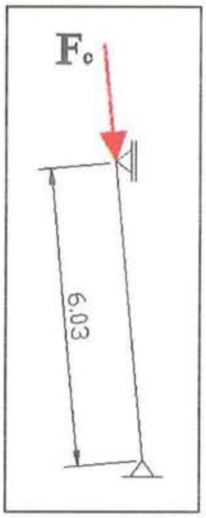


figure 2.10 Normal forces in inclined columns (l), building shape (r)

the building is drawn. Due to the curved shape of the building, the normal force in the façade columns increases (when the angle becomes larger) and decreases (when the angle becomes smaller). From the diagram it is clear that the normal forces in the façade columns on the left and right side are equal when the columns are positioned vertically.



Strength

$$\boxed{\text{Requirement (u.g.t): } \frac{N_{c;s;d}}{N_{c;u;d}} \leq 1}$$

$$N_{c;s;d} = \gamma_g \cdot F_g + \gamma \cdot F_q = 1.2 \cdot 11922 + 1.5 \cdot 15853 = 38086 \text{ kN}$$

This normal load is spread over 10 columns, per columns a load of 3808.6 kN.

$$A_{\min} = \frac{N_{c;s;d}}{f_{y;d}} = \frac{3808.6}{235} = 16207 \text{ mm}^2$$

A HE450A profile matches best with the minimal surface (17800 mm^2), but has to be checked on the buckling requirement.

Buckling

$$\boxed{\text{Requirement: } \frac{N_{c;s;d}}{\omega_{buc} \cdot N_{c;u;d}} \leq 1}$$

Buckle length l_{buc}:	6.0	m
I:	63722 10^4	mm^4
Section surface HE600B A:	17800	mm^2
Curve (table 23 NEN6770):	C	-
$f_{y;d}$:	235	N/mm^2

$$N_{c;s;d} = 3808.6 \text{ kN (10 columns)}$$

$$N_{c;u;d} = A \cdot f_{y;d} = 17800 \cdot 235 = 4183 \text{ kN}$$

$$F_E = \frac{\pi^2 EI}{l_{buc}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 63722 \cdot 10^4}{6000^2} = 36322 \text{ kN}$$

$$\lambda_{rel} = \sqrt{\frac{N_{c;u;d}}{F_E}} = \sqrt{\frac{4183}{36322}} = 0.34 \quad \rightarrow \quad \omega_{buc} = 0.94$$

$$\frac{N_{c;s;d}}{\omega_{buc} \cdot N_{c;u;d}} = \frac{3808.6}{0.94 \cdot 4183} = 0.97 \leq 1$$

By employing a HE450A profile as the inclined façade columns there is no danger in buckling of the column. Even more so because the investigated column is located at the first floor with a story height and buckle length of 6 m, whereas the rest of the columns is located at stories with a height of 4 m and thus a much smaller buckling length.
Obviously, the normal force in the columns will decrease at every floor level when moving upwards. Therefore, not all columns need to have the same profile. In figure 2.10 one can see that the building is divided into three areas that will be calculated with the same normal force (dashed red line).

The minimum steel profiles that are needed for the middle and upper section of the building are respectively a HE340A and a HE280A.[8]

2.3.2 *Straight columns*

The columns on the non-curved sides of the building are pointed out in red in figure 2.11. The calculation of these columns follows the same procedure as the inclined columns in the previous chapter. Therefore the only the resulting profiles are mentioned here[9].

Like with the inclined columns, the building is split into three areas where the columns have the same measurements.

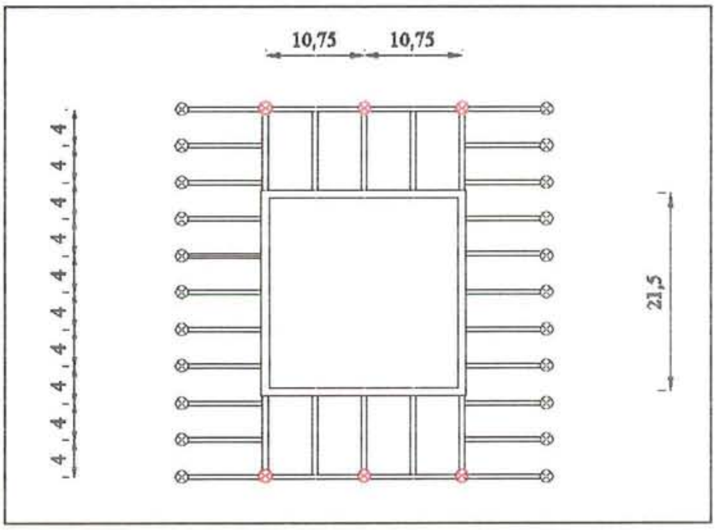


figure 2.11 Straight facade columns

The applied profiles will be:

- HD 400x818 (0-50 m)
- HE500A (50-100 m)
- HE300A (100-150 m)

[8] For a full calculation of the profiles see appendix G 7-8
[9] For a full calculation of the profiles see appendix G 9-11

2.3.3 *Column base plates*

Both the inclined and straight columns have to bear relatively large normal forces at ground level and therefore have relatively large measurements. In order to pass the forces to the foundation the base plates of the columns must be able to spread the load.

Inclined column base plate

The forces in the inclined columns are all centric normal forces as drawn in figure 2.12. Therefore one can assume that a continuous spread compression on the joint mortar will occur. To achieve such a spread compression the base plate should be high enough to ensure the needed stiffness of the plate. This can be done by welding steel plates perpendicular to the flanges of the columns as well, but this is a relatively expensive solution.
For the inclined columns a relatively high steel base plate will be employed to ensure a continuous spread load on the joint mortar.
The maximum normal force in the columns, at ground level is $N_{s;d} = 3808.6\,\text{kN}$ (see paragraph 2.3.1).

$N_{s;d}$:	3808.6	kN
Column section A_{HE450A}:	17800	mm^2
$h_l = h - 2\cdot(t_{flange} + r)$:	344	mm
Base plate width b:	500	mm
$f_{y;d}$:	235	N/mm^2

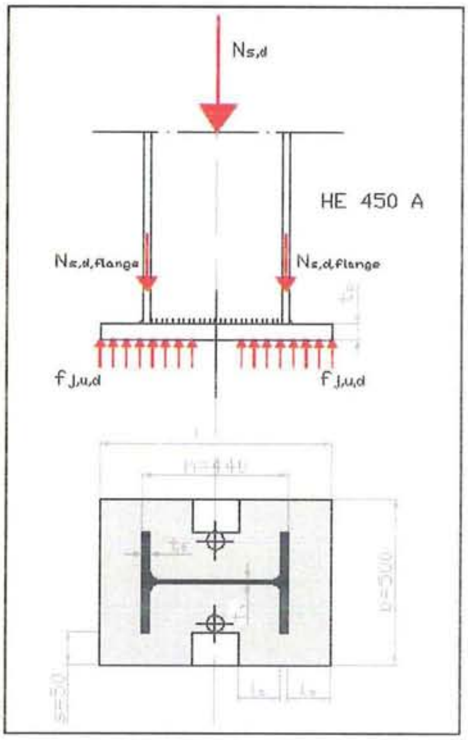


figure 2.12 Column base plate

The width of the steel base plate is set on 500 mm, in the calculations below the minimum length and height of the plate are calculated.

Flange section: $A_{flange} = \frac{1}{2}\cdot(A_{HE450A} - h_l \cdot t_w) = \frac{1}{2}(17800 - 344\cdot 11.5) = 6922\,\text{mm}^2$

Compr. in profile: $\sigma_{s;d} = \frac{N_{s;d}}{A_{HE450A}} = \frac{3808.6\cdot 10^3}{17800} = 214\,\text{N/mm}^2$

Flange normal force: $N_{s;d;flange} = A_{flange}\cdot\sigma_{s;d} = 6922\cdot 214 = 1481\ \text{kN}$

Joint mortar: $f_{j;u;d} = 0.67\cdot k_b\cdot f'_{b;d} = 0.67\cdot 1\cdot 15 = 10.05\,\text{N/mm}^2$

The needed length l_s is then: $l_s = \frac{1}{2}\left(\frac{N_{s;d;flange}}{f_{j;u;d} \cdot b} - t_{flange}\right) = \frac{1}{2}\left(\frac{1481 \cdot 10^3}{10.05 \cdot 500} - 21\right) = 137 \text{ mm}$

Base plate, minimum height: $t_{plate} = l_s \cdot \sqrt{\frac{3 \cdot f_{j;u;d}}{f_{y;d}}} = 137 \cdot \sqrt{\frac{3 \cdot 10.05}{235}} = 49 \text{ mm}$

This implicates a plate length of: $l_{plate} = h + 2 \cdot l_s = 440 + 2 \cdot 137 = 714 \text{ mm}$

Now the assumption of the continuous spread load has to be checked:

Requirement: $s \geq 1.5 \cdot t_{plate}$

$s = 100 \geq 1.5 \cdot 49 = 73.5 \text{ mm}$, the assumption was correct. A steel base plate of 715 x 500 x 50 mm will be sufficient to pass the normal forces from the inclined columns on to the foundation.

Straight column base plate

The base plates needed for the straight columns on the non-curved side of the building will have to bear a significantly larger load than the inclined façade columns, 6160 kN. The calculation of the base plate follows the same procedure as the shown above. Therefore, only the results are presented here, the calculation can be found in appendix G 12.

Minimum measurements steel base plate: 630 x 550 x 21 mm

2.4 Core

The central core has two primary structural functions:

1. To provide stability to the building, by absorbing the lateral loads

$$\text{Requirement: } w_{top} \leq \frac{1}{500} \cdot h = 0.3\text{ m};\ w_{story} \leq \frac{1}{300} \cdot h_{story} = 0.013\text{ m}$$
$$\varphi \leq \frac{1}{500} = 0.002 \text{ rad}$$

2. To bear the vertical load coming from the floors

$$\text{Requirement: } \sigma_b^{'} \leq f_b^{'}$$

From the literature investigation we know that the lateral force action becomes an important design factor for buildings higher than about 30 stories. Therefore the core will be designed based first on the resistance for horizontal loads and later on be checked on the minimal needed strength.
A normal, orthogonal high rise structure will be designed to withstand the lateral wind load, as this is the only lateral force the building will be subjected to. For the curved building, the core has to bear the extra horizontal loads coming from the floors as well.
This abnormal loading of the core will be analyzed first. When the behaviour of the core subjected to the extra lateral forces is clear, the wind load behaviour will be discussed as well.

2.4.1 Extra lateral loads

The normal forces at floor level can be calculated with (see figure 2.1 and chapter 2.2):

- $F_{fi\_left} = F_{ciH\_left} - F_{ci-1H\_left}$ (lateral force i, left side)
- $F_{fi\_right} = F_{ciH\_right} - F_{ci-1H\_right}$ (lateral force i, right side)

The lateral force on the central core at level i is then:

- $F_{fi\_total} = F_{fi\_left} + F_{fi\_right}$

The total lateral forces have a pattern similar to the scheme presented in figure 2.13 . The forces from the left and the right side (see figure 2.2) are added up, resulting in a force switching from left right.

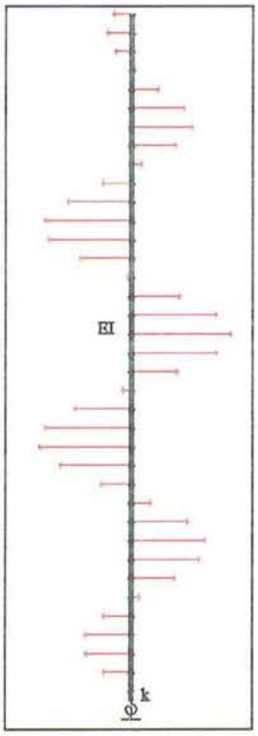


figure 2.13 Lateral forced on core

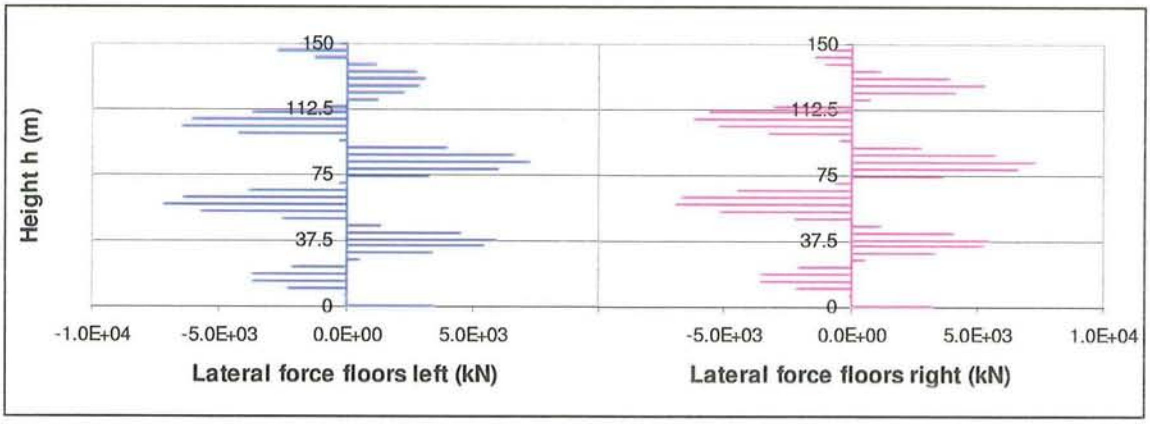


figure 2.14 Lateral forces on every floor level, left and right

The lateral forces coming from the inclined columns have been calculated in chapter 2.1 and shown above as a reminder. One can see that the force on the left ant right side of the building are pointing in the same direction at each level, and thus add up to a larger force at that particular level (according to the formulas on p. 29).

The resulting lateral forces (F_{fi_total}), working on the central core are presented in the diagram below.

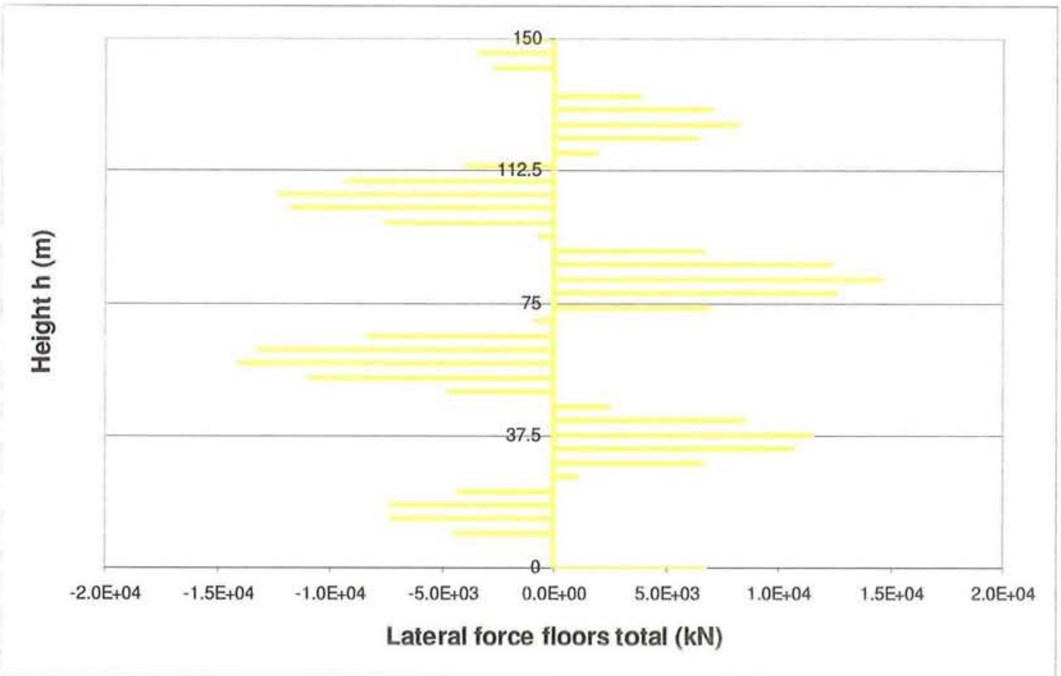


figure 2.15 Total lateral forces on central core (excl. wind load)

The lateral forces that are calculated above are independent of the wind load on the building and are solely caused by the vertical loads on the floors. This means that these lateral loads will be present, under all circumstances and thereby cause an initial deformation to the building.
The lateral forces cause a bending moment to the central core, according to the manner as presented in the Structural investigation part II. The lateral bending moment can be described by: $M_{lateral,i} = \sum_{i=1} F_{i\_total} \cdot (x_{Fi} - x_{Mi})$
The resulting bending moment on the core is presented in figure 2.16. One can see that the

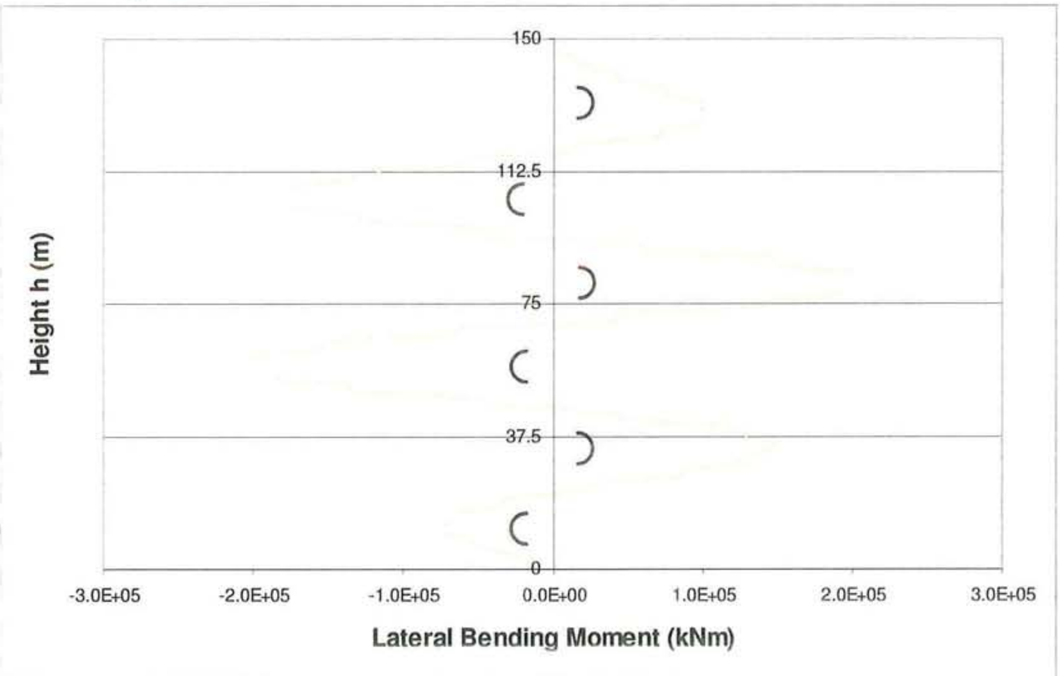


figure 2.16 Lateral bending moment on central core (excl. wind moment)

resulting bending moment can grow to a maximum of about 200000 kNm. Compared to the maximum bending moment due to the wind load of $M_{wind,x=0} = \frac{1}{2} \cdot q_{wind} \cdot h^2 = 0.5 \cdot 80 \cdot 150^2 = 900000$ kNm, the maximum lateral bending moment is should not cause extreme design problems. The extra bending moment at ground level is reduced to 14012 kNm, only 1.6% of the total wind moment at this point. This is a value fairly close to the optimum of 0 kNm as stated in chapter 1, the foundation will not have to bear an extra bending moment and can be designed with the wind moment as the normative load.

2.4.2 *Initial deformation*

The independent lateral forces on the core, described in the last paragraph, will cause an initial deformation as was concluded in the Structural investigation part II. The magnitude of the initial deformation of case study structure will be calculated in this paragraph.
As we know, the total deflection is built up by the deflection caused by the stiffness of the building and by the deflection due to the rotation stiffness of the foundation:

$$w_{top} = w_{building} + w_{foundation} = \frac{qh^4}{8 \cdot EI} + \frac{qh^3}{k} \leq \frac{1}{500} \cdot h$$

If one states that the both the building and the foundation contribute half of the total deflection, this equation can be split into:

$$w_{building} = \frac{qh^4}{8 \cdot EI} \leq \frac{1}{1000} \cdot h = 0.15 \text{ m}$$

$$w_{foundation} = \frac{qh^3}{k} \leq \frac{1}{1000} \cdot h = 0.15 \text{ m}$$

Minimum moment of inertia: $I_{\min} = 1406.25 \text{ m}^4$
Minimum foundation rotation stiffness: $k_{\min} = 9\ 10^8$ kNm/rad

This minimum moment of inertia is already multiplied by a camber-factor of 0.8 in order to take the cambers and holes in the core into account and to build in a certain margin for the core to be able to withstand the extra deformation caused by the lateral loads at floor level.
In the previous chapters the grid of the building is drawn. With a moment of inertia of 1406.25 m^4 and a centre-to-centre distance of 21.5 m of the core walls, the minimum wall width has to be 215 mm.
The core structure will not be a closed tube, but more like two U-shaped cores with door openings between them. The two cores will be connected to each other by lintels at every floor level (see chapter 2.4.9). During further calculations the wall width will be set on 250 mm to take this connection into account and create some margin. In chapter 2.4.9, the lintels will be designed and the resulting moment of inertia will be checked.
The initial deformation can now be calculated by calculating the bending $\kappa = \frac{M}{EI}$

$$\kappa = \frac{M}{EI} = \frac{d\varphi}{dx} \quad \rightarrow \quad \Delta\varphi_i = \kappa_i \cdot \Delta x = \frac{M_i}{EI} \cdot \Delta x$$

Deflection at point i: $w_x = \sum_{i=1} \varphi_i \cdot x_i$

The total rotation at point i: $\varphi_i = \sum_{i=1} \Delta\varphi_i$

The deformation pattern according to the formulas above is drawn in figure 2.17[10]. From this diagram it is clear that the initial top deflection of the core is minimal, only 1.24 mm (0.41%

[10] A table with the deflection at floor level can be found in appendix G 13

of the maximum allowed top deflection of 300 mm. This very small deflection is achieved by choosing the right combination of the number of curves (3.22) and the reverse damped shape, as mentioned in chapter1. As predicted in the Structural investigation part II, due to switching of the bending moment from left to right (figure 2.16), the deformation has no chance to grow to a significant value. When the deflection is directed to one side, the switching of the bending moment will cause the core to deform in the opposite direction before it becomes too large. If the building was not curved but just inclined, that is with the bending moment on only one side of the building, the "tail-effect" would cause a much larger deflection to the building.
One can conclude that the initial top deflection of the building is reduced to (close to) zero.

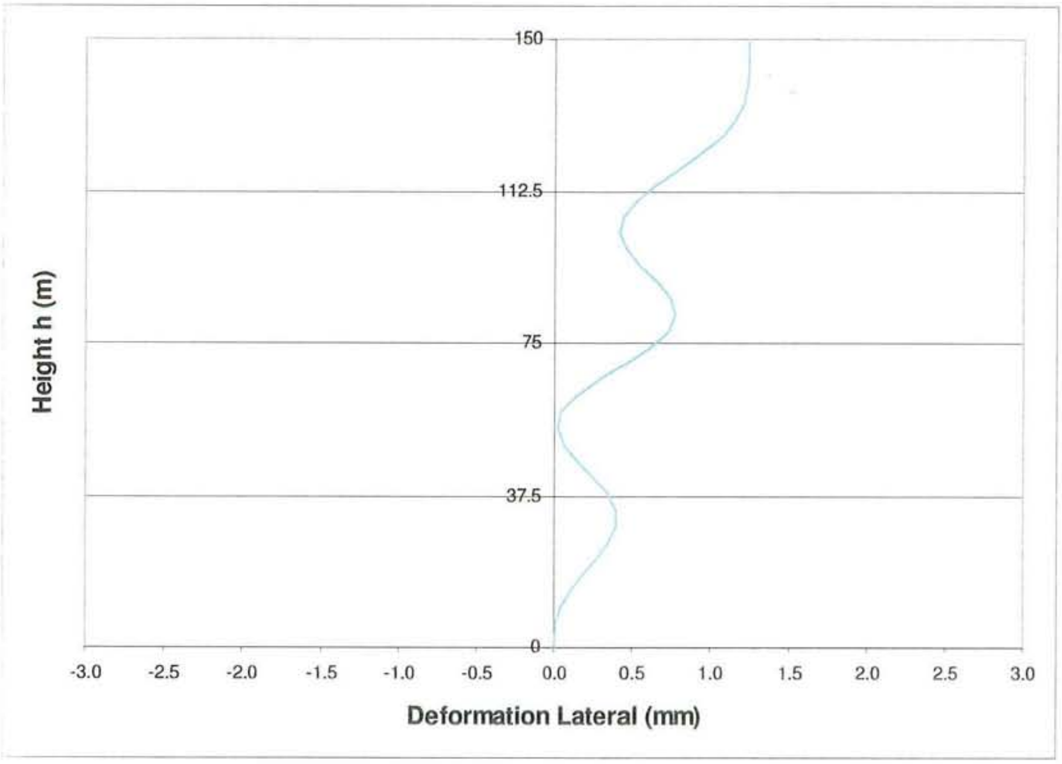


figure 2.17 Initial deformation central core due to lateral loads (excl. wind load)

2.4.3 Total deformation

In the previous paragraph the initial deformation of the central core due to lateral forces at every story level are defined. From the data above we know that the initial deflection of the core is very small. In this paragraph the total deformation of the building will be analyzed, thus including the deflection due to the wind load.

In figure 2.18 the bending moment caused by the wind load is drawn. The maximum bending moment at ground level:

$$M_{wind;\max} = \frac{1}{2} \cdot q_w \cdot h^2 = 0.5 \cdot 8 \cdot 150^2 = 900000 \text{ kNm}$$

The top deflection of the building (due to wind load) can be calculated with:

$$w_{top;wind} = \frac{q_w h^4}{8 \cdot EI} = \frac{80 \cdot 150^4}{8 \cdot 3 \cdot 10^7 \cdot 1406.25} = 120 \text{ mm}$$

The deflection caused by the rotation of the foundation can be calculated with:

$$w_{top;foundation} = \frac{Mh}{k} = \frac{(M_{wind} + M_{initial}) \cdot h}{k} = \frac{(900000 + 14012) \cdot 150}{9 \cdot 10^8} = 152 \text{ mm}$$

In paragraph 2.4.2 the initial deflection was calculated:

$w_{top;initial} = 1.24 \text{ mm}$

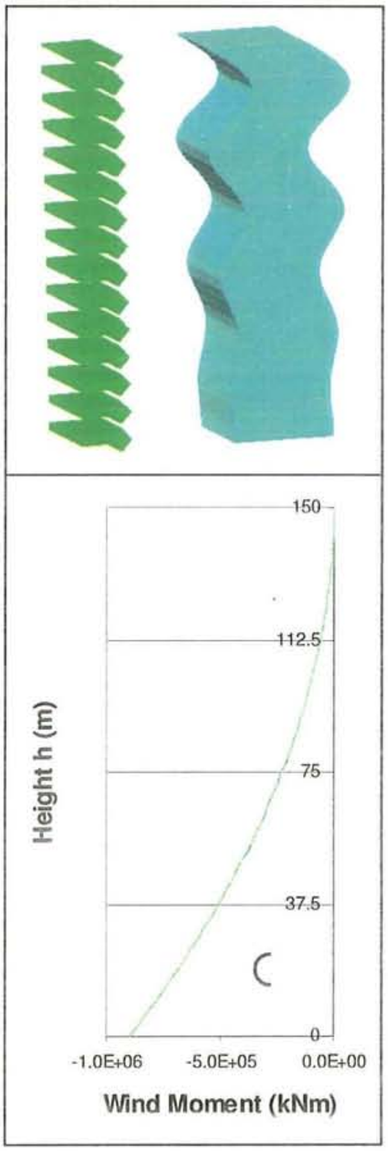


figure 2.18 Wind induced moment

The total 1st order deflection of the building will be:

$$w_{top} = w_{top;wind} + w_{top;foundation} + w_{top;initial} = 120 + 152 + 1.24 = 273.24 \text{ mm}$$

In figure 2.19 the deformation of the central core is projected. Clear is the small impact of the initial deflection compared to the contribution of the deflection caused by the wind load and by the rotation of the foundation. The second requirement of a maximum deflection of 0.013m per story is met easily.

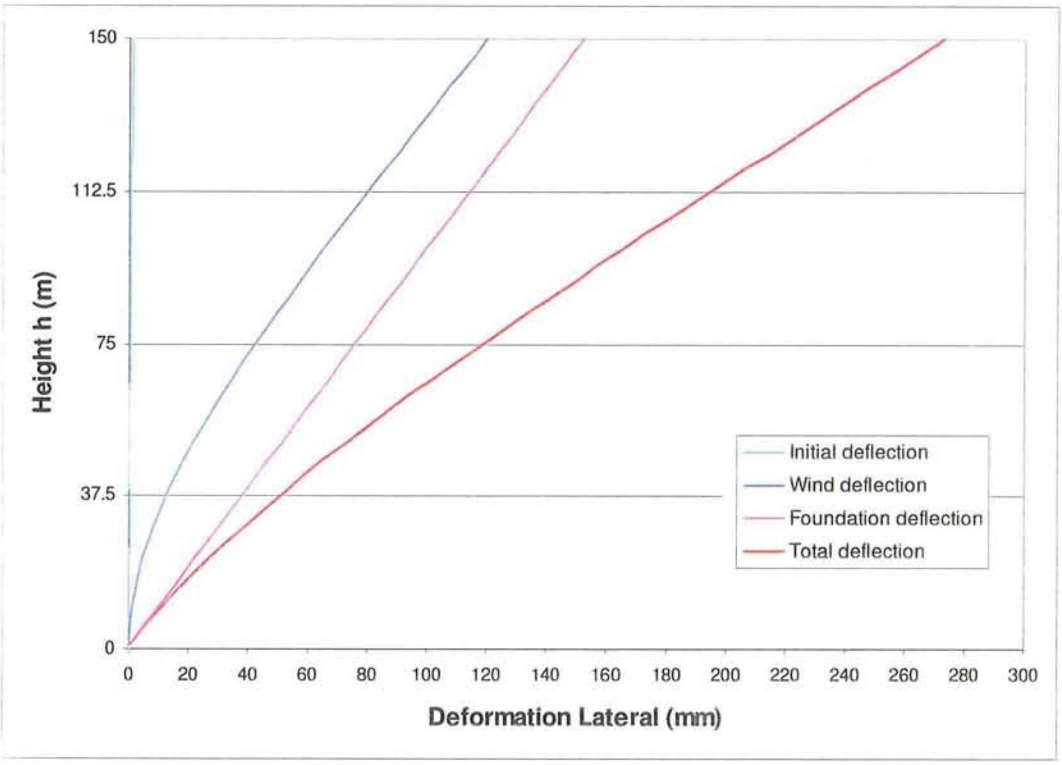


figure 2.19 1st order core deformation

2.4.4 2^{nd} order deformation

The 1^{st} order top deflection is well within range of the maximum deflection of 300 mm. Question is whether the building still behaves within the set boundaries, when the 2^{nd} order deflection is reviewed. When the 2^{nd} order deformation is analyzed the gravity load of the building is activated.

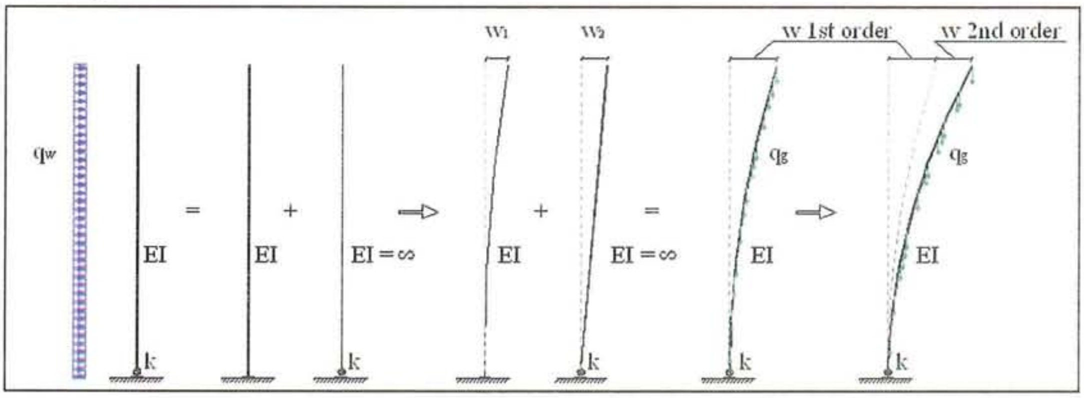


figure 2.20 Deformation scheme of central core

In figure 2.20 the central core is modellised as a column with bending stiffness EI and a connection to its foundation with a rotation stiffness k. To analyze the behaviour, the structure can be split into a column with bending stiffness EI and an infinite rotation stiffness k, and a column with an infinite bending stiffness EI and a rotation stiffness k. This system has been used to calculate the first order deformation of the building.

When the structure has reached its first order deformation, the gravity load of the building is activated and will cause the core to deflect further. This 2nd order deflection has to be controlled in order to secure the stability of the structure.

The gravity load of the building that is supported by the central core:

Surface per floor: $30.75 \times 30.75 = 945.6\ \text{m}^2$

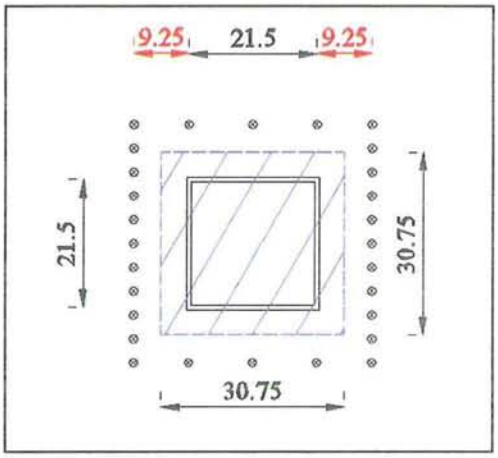


figure 2.21 Floor area on core

Floor weight: $F_{g;floor} = A_{floor} \cdot p_g \cdot (\#\ floors) = 945.6 \cdot 1.88 \cdot 37 = 65776\ \text{kN}$

Variable weight: $F_q = A_{floor} \cdot p_q \cdot (\#\ floors) = 945.6 \cdot 2.5 \cdot 37 = 87468\ \text{kN}$

Core weight: $F_{g;core} = A_{core} \cdot \rho_{concrete} \cdot h = 21.5 \cdot 24 \cdot 150 = 77400\,\text{kN}$

Total vertical load on core:

$F_d = \gamma_g \cdot (F_{g;floor} + F_{g;core}) + \gamma_q \cdot F_q = 1.2 \cdot (65776 + 77400) + 1.5 \cdot 87468 = 303013.2\ \text{kN}$

Buckle load of central core; stiffness EI, rotation stiffness foundation k=∞:

Buckle length[11]: $l_{buc;1} = 1.12 \cdot h = 1.12 \cdot 150 = 168\ \text{m}$

Stiffness EI: $E = 36000\ \text{N/mm}^2$ (B55 quality); $I = 1406.25\ \text{m}^4$

Buckle load: $F_{buc;1} = \dfrac{\pi^2 \cdot EI}{l_{buc}^2} = \dfrac{\pi^2 \cdot 3.6 \cdot 10^7 \cdot 1406.25}{168^2} = 17702973\ \text{kN}$

Reciprocal multiplication factor n_1: $n_1 = \dfrac{F_{buc}}{F_d} = \dfrac{17702973}{303013.2} = 58.42$

Buckle load of central core; stiffness EI=∞, rotation stiffness foundation k:

Buckle length: $l_{buc;2} = 0.5 \cdot h = 0.5 \cdot 150 = 75\ \text{m}$

Rotation stiffness foundation k: $k = 9 \cdot 10^8\ \text{kNm/rad}$

[11] Buckle length is set on 1.12 x h, according to Cement en beton; praktische betonberekeningen [23]

Buckle load: $F_{buc;2} = \frac{k}{l_{buc;2}} = \frac{9 \cdot 10^8}{75} = 12 \cdot 10^6 \text{kN}$

Reciprocal multiplication factor n_2: $n_2 = \frac{F_{buc;2}}{F_d} = \frac{12 \cdot 10^6}{249013.2} = 48.2$

2^{nd} order multiplication factor central core; stiffness EI and rotation stiffness foundation k:

$$\frac{1}{n} = \frac{1}{n_1} + \frac{1}{n_2} = \frac{1}{58.42} + \frac{1}{48.2} = 0.0379$$

$$n = 26.41$$

The n-value lies well above the stability requirement of 10. If the n-value would lie in short range to the minimum of 10, extra stability measures should be considered. In this case, with a value of 28.72 there is no danger for instability of the system due to 2^{nd} order deformation.

The actual 2^{nd} order multiplication factor is now: $\frac{n}{n-1} = 1.039$

The total deflection of the building, including the 2^{nd} order deflection will be:

$w_{total} = w_{1st\_order} \cdot \frac{n}{n-1} = 273.24 \cdot 1.039 = 283.90\,\text{mm}$. This deflection lies well within the maximum range of 300 mm.

The 2^{nd} order bending moment at ground level is:

$$M_{2nd\_order} = M_{1st\_order} \cdot \frac{n}{n-1} = 914012 \cdot 1.039 = 949658 \text{ kNm}$$

Perpendicular deformation

In the direction perpendicular to the curve, there is no initial deformation of the core. The lateral loads at floor level do not exist in this direction. Therefore, the core only has to bear the wind load as a horizontal load.
Since the core is square shaped, the moment of inertia will be the same in this direction: $I_{\min} = 1406.25 \text{ m}^4$

For the foundation, the assumption is made that it will have to provide the same rotation stiffness in both directions. This means a rotation stiffness in the direction perpendicular to the curve of: $k_{\min} = 9 \; 10^8$ kNm/rad.

The wind will set pressure on the side of the building and the core will have to resist this loading. The side surface of the building has a non-orthogonal shape but it covers the same amount of square meters as the curved sides. This, because of the constant building width; the floors have a different position at every level, but the total surface does not change.

The bending moment on the building side perpendicular to the curved side is presented in figure 2.22, $M_{\max} = \frac{1}{2} \cdot q_w \cdot h^2 = 0.5 \cdot 8 \cdot 150^2 = 900000 \text{ kNm}$.

The top deflection by building stiffness will be:

$$w_{top;wind} = \frac{q_w h^4}{8 \cdot EI} = \frac{80 \cdot 150^4}{8 \cdot 3 \cdot 10^7 \cdot 1406.25} = 120 \text{ mm}$$

Deflection due to rotation stiffness of foundation:

$$w_{top;foundation} = \frac{Mh}{k} = \frac{(900000) \cdot 150}{9 \cdot 10^8} = 150 \text{ mm}$$

$$w_{top} = 120 + 150 = 270 \text{ mm}$$

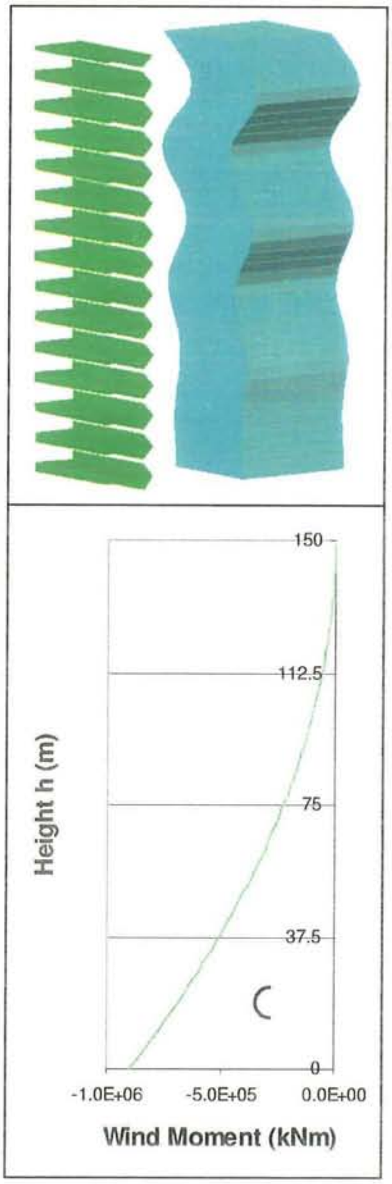


figure 2.22 Bending moment on non-curved side if building

This is the total first order deflection of the building in the direction perpendicular to the curved sides. The multiplication factor of this side of the core is considered to be the same as in the other direction (1.039), so the 2nd order top deflection will be:

$$w_{2nd\_order} = 1.039 \cdot 270 = 280.53 \text{ mm.}$$

Like in the perpendicular direction, the 2nd order top deflection stays well within range of the maximum top deflection of 300 mm.

2.4.5 E-modulus check

In the calculations above, the E-modulus was set on 36000 N/mm^2 for B55 concrete quality. Whether this assumption was right has to be checked. No tensile stresses are allowed in the core otherwise the concrete will crack and the E-modulus will decrease to significantly lower value. The stresses in the core are built up by the gravity load of the building that is supported by the core and by the bending moment on the central core caused by the lateral loads at floor level and the wind.

Floor weight: $$q_{g;floor} = \frac{A_{floor} \cdot p_g \cdot (\# floors)}{h} = \frac{945.6 \cdot 1.88 \cdot 37}{150} = 438.5 \text{ kN/m}$$

Core weight: $$q_{g;core} = \frac{A_{core} \cdot \rho_{concrete} \cdot h}{h} = \frac{21.5 \cdot 24 \cdot 150}{150} = 516 \text{ kN/m}$$

The variable load must not be taken into account for this calculation because it will have a positive effect on the compressive stresses in the core. A case with only permanent and no variable loads is normative. That is true when the structure is a normal building where the variable loads are directed vertically only. With the curved structure, where the vertical loads cause horizontal forces and thereby cause an extra bending moment, the variable load case has to be checked as well because of the tensile stresses that it may generate.

Variable floor load: $$q_q = \frac{A_{floor} \cdot p_q \cdot (\# floors)}{h} = \frac{945.6 \cdot 2.5 \cdot 37}{150} = 583.1 \text{ kN/m}$$

Total load on core: $$q_{d1;core} = \gamma_g \cdot (q_{g;floor} + q_{g;core}) = 1145.4 \text{ kN/m (excl. variable load)}$$

$$q_{d2;core} = \gamma_g \cdot (q_{g;floor} + q_{g;core}) + \gamma_q \cdot q_q = 2020.05 \text{ kN/m (incl. var. load)}$$

The bending moments caused by the lateral forces at floor level and by the wind were presented in paragraph 2.4.1 and 2.4.3. In figure 2.23 the total bending moments are displayed, with the separate bending moments still visible as the dashed lines.

Total bending moment: $$M_{tot} = M_{lateral} + M_{wind} = \sum_{i=1} F_{i\_total} \cdot (x_{Fi} - x_{Mi}) + \frac{1}{2} q_w \cdot (h - x)^2$$

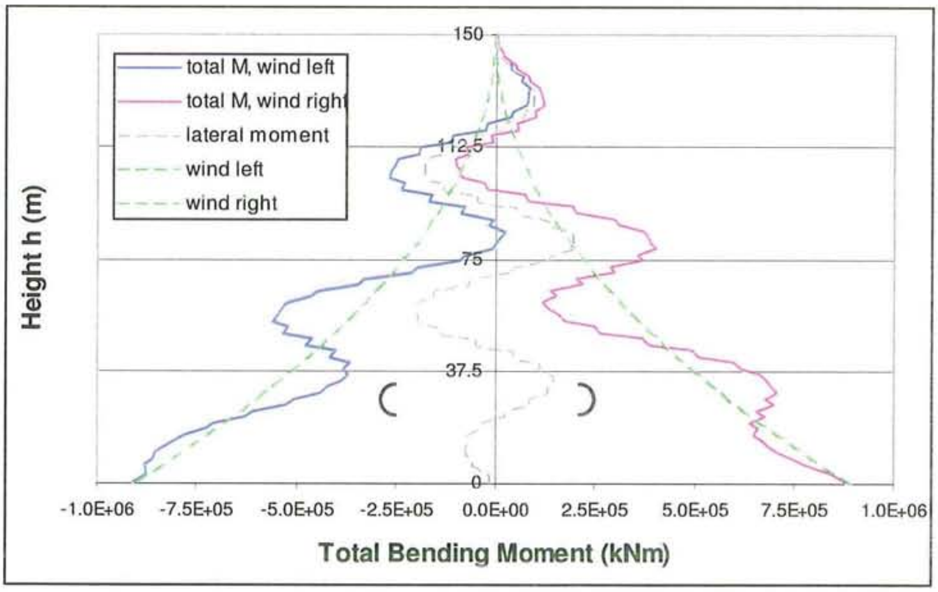


figure 2.23 Bending moments on central core

The stresses in the core can now be calculated by:

$$\sigma_{tot} = \sigma_N + \frac{n}{n-1} \cdot \sigma_M = \frac{F_d}{A} + \frac{n}{n-1} \cdot \frac{M_{tot}}{W}$$

Where:

$$F_d = q_d \cdot x$$

$$A = 21.5 \text{ m}^2$$

$$W = \frac{I}{\frac{1}{2} \cdot b_c} = \frac{1406.25}{0.5 \cdot 21.5} = 130.8 \text{ m}^3$$

At ground level, the minimum stresses in the central core will be:

Wind from left: $\sigma_{x=0} = \dfrac{-171810}{21.5} + 1.039 \cdot \dfrac{914012}{130.8} = -0.73 \text{ N/mm}^2$

Wind from right: $\sigma_{x=0} = \dfrac{-171810}{21.5} + 1.039 \cdot \dfrac{885988}{130.8} = -0.95 \text{ N/mm}^2$

On the next page in figure 2.24, the stresses in the central core are drawn.

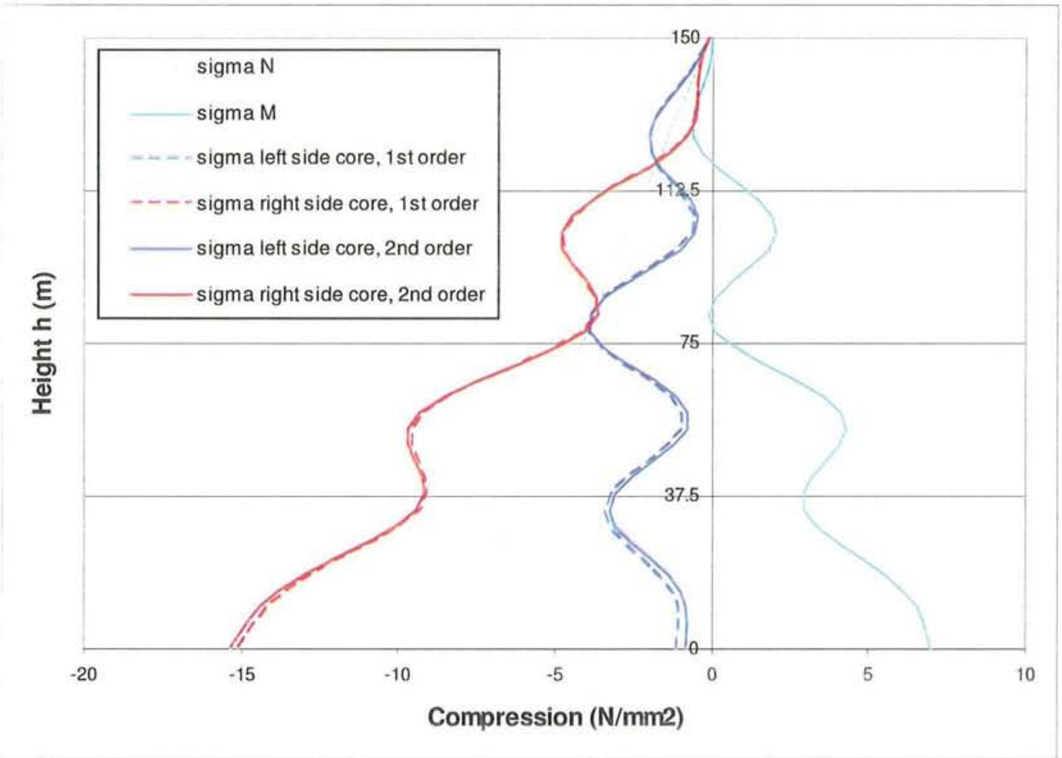


figure 2.24 Stresses on left and right side of the central core, with wind from the left side of the building

As one can see in the figure above, there will be no tensile stressed in the central core of the building. The bending moment alone causes tensile stresses (light bleu), but the vertical loading of the core compensates that by building up enough compression on the core. With wind from the left side of the building, the left side of the core is most likely to receive the lowest compressive stresses. This is the case in the largest part of the central core but, due to the curved shape, it is the right side of the upper part of the building that has the least compression.

The 2nd order multiplication does not cause any problems regarding the compression in the central core. The stresses come slightly closer to zero, but there is still enough compression in the core.

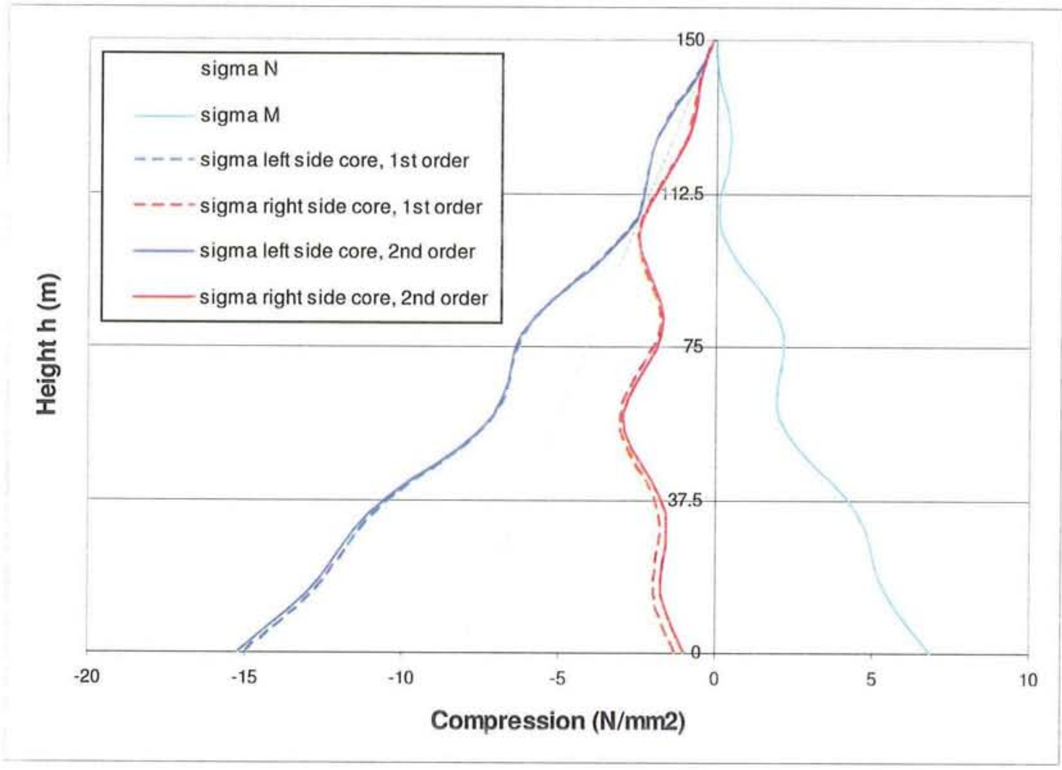


figure 2.25 Stresses on left and right side of the central core, with wind from the right side of the building

The stresses in the central core with wind from the right side of the building are closest to zero on the right side of the core (for the largest part). There is no danger for tensile stresses on either side of the core. Therefore, the assumption that the concrete core is not cracked and the E-modulus will be 36000 N/mm^2 can be made because the gravity load of the building is large enough to ensure compression in the core.

Before one can draw this conclusion definitively, the load case including the variable load has to be checked. If the variable floor load is added to the total load on the building there will be no tensile stresses in the central core either. The extra variable load does increase the bending moment and thereby the stresses created by the bending moment, but this is compensated by the extra vertical loading on the core (σ_N increases as well)[12].

The assumption of a non-cracked concrete core can now be justified. There is no need for additional reinforcement, only a minimum practical reinforcement is required.

[12] For the exact stress pattern of the core, including the variable load is referred to appendix G 14

2.4.6 *Vertical loading*

Besides the function of a stabilizing element, the core will have to be able to bare the vertical loads coming from the floors. In the previous paragraph the normal load on the core was calculated for the E-modulus check. The maximum compression that could occur at ground level in the central core is 21.4 N/mm^2. This is the maximum compression including a variable load on the floors, the diagram can be found in appendix G 14.

The requirement is stated as: $\sigma_b^{'} \leq f_b^{'}$

In this case: $\sigma_b^{'} = 21.4 \leq f_b^{'} = 33$ N/mm^2

The compressive strength is sufficient to bear the maximum compressive stresses in the central core.

2.4.7 *Shear force*

The shear force in the building is built up by the lateral forces from the floor and the wind load.

The shear force due to lateral forces from the floors can be calculated by:

$V_{i;lateral} = \sum F_{fi\_total}$

The shear force due to the wind load can be calculated by:

$V_{i;wind} = q_{wind} \cdot (h - x_i)$

The total shear force is then: $V_{i;total} = V_{i;lateral} + V_{i;wind}$

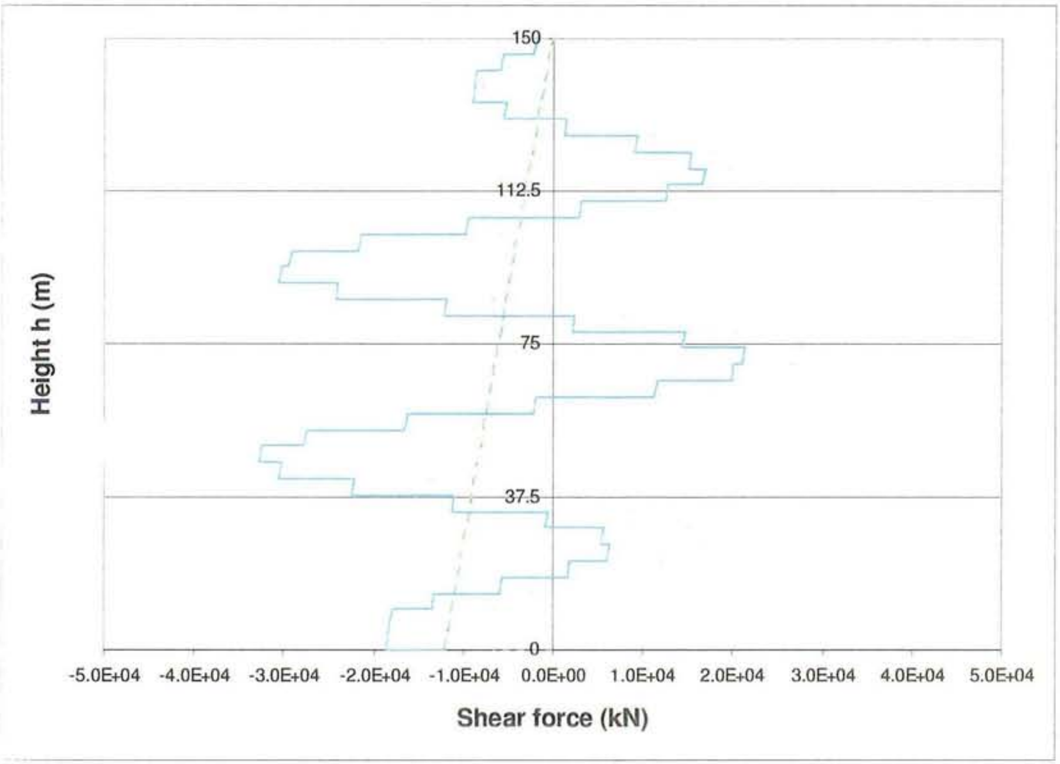


figure 2.26 Shear force in central core

The shear forces on the central core will result in shear stresses in the core. The requirement for shear stresses is:

$$\boxed{\text{Requirement (u.g.t): } \tau_d \leq \tau_u}$$

In this case $\tau_d = \dfrac{V_d}{b \cdot d} = \dfrac{V_d}{2 \cdot 250 \cdot 21800}$, where V_d changes from floor to floor (see figure 2.26). The resulting shear stresses are presented in figure 2.27.

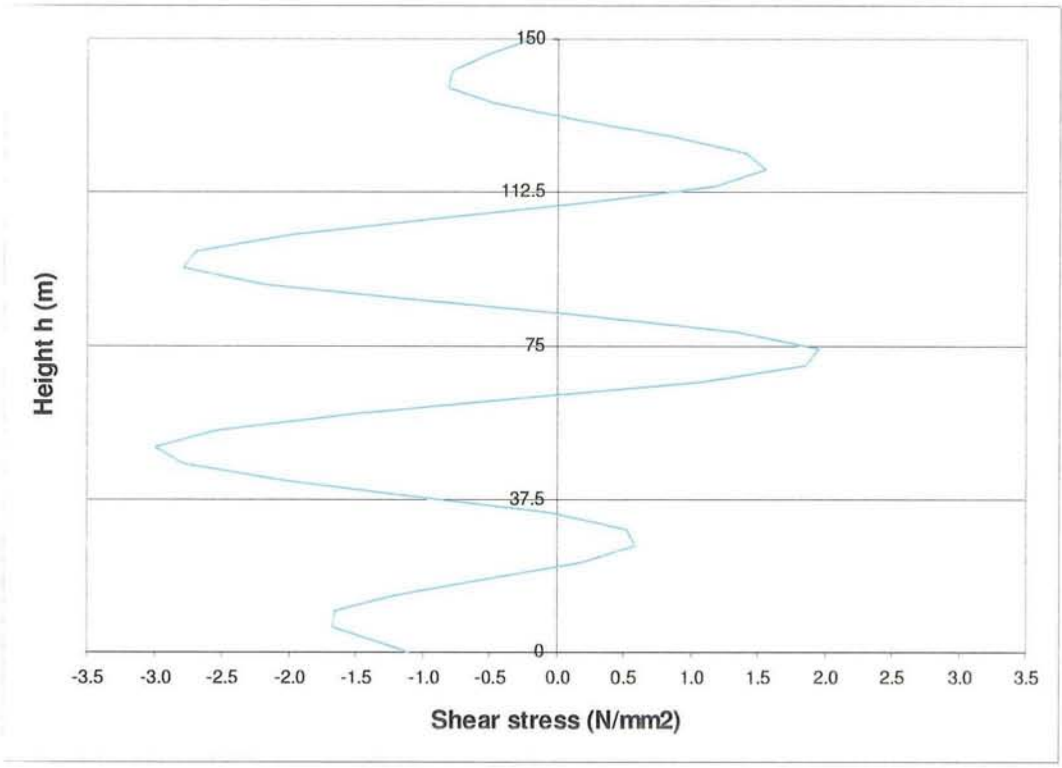


figure 2.27 Maximum shear stress pattern in central core

The maximum allowed shear stress is to be calculated with:

$\tau_u = \tau_1 \not=> \tau_2$, for a construction without shearing-reinforcement

$$\tau_2 = 0.2 \cdot f_b' \cdot k_n \cdot k_\theta = 0.2 \cdot 33 \cdot 1.43 \cdot 1 = 9.44 \text{ N/mm}^2$$

$$\tau_1 = \frac{I}{dS} \cdot \sqrt{f_b'^2 + f_b \cdot \sigma_{bmd}'}$$

$I = 1406.25\,\text{m}^4$
$d = 250\,\text{mm}$
$S = 21750 \cdot 250 \cdot 21500 \cdot 0.5 = 5.85 \cdot 10^{10}\ \text{mm}^4$
$f_b' = 1.9\,\text{N/mm}^2$ (B55 concrete)
σ_{bmd}' changes from floor to floor

The minimum allowed shear stress τ_1 will be 42.4 N/mm^2 near the top of the core. For a full view of the τ_1 pattern is referred to appendix G 16. In this case $\tau_1 \geq \tau_2$, for the maximum allowed shear stress one should refer to $\tau_2 = 9.44$ N/mm^2. The maximum shear stress that may occur in the central core will be 2.99 N/mm^2 at 50 m. This shear stress lies well within the margins of the maximum allowed shear stress of 9.44 N/mm^2. Therefore, there is no need for extra measures regarding shear stresses reduction.

2.4.8 Torsion

From the previous paragraphs it is clear that the maximum deflection of the building will stay within the margins of the maximum allowed deformation.
The deflection of the building (in both directions) is calculated with the full wind load on the side of the building. Apart from the deflection that the wind load may cause, the curved shape has to be checked on torsion as well.
If the wind blows on the side of the building as in figure 2.28, the middle part of the load will be led to the core directly. The remaining part of the wind load will cause torsion in the central core because of the difference in façade surface on the left and right side at every floor level.

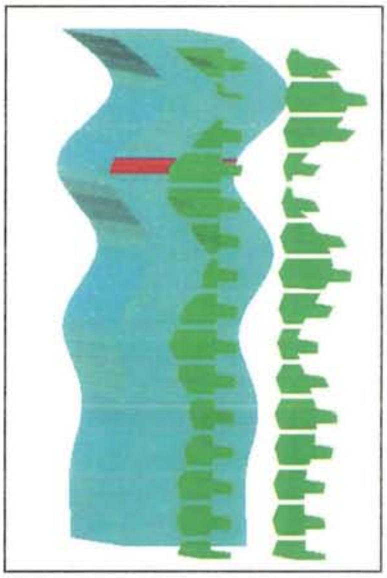


figure 2.28 Wind load that causes torsion in core

The torsion moment on the core is not constant but will differ from level to level. M_t can be calculated with:
Torsion moment at level i: $M_{t;i} = p_w \cdot (A_{f;i;left} \cdot a_{i;left} - A_{f;i;right} \cdot a_{i;right})$

Where $A_{f;i;left}$ = Façade surface on the left side of the building

In figure 2.29 the red story of figure 2.28 is enlarged in order to clarify the formula above.

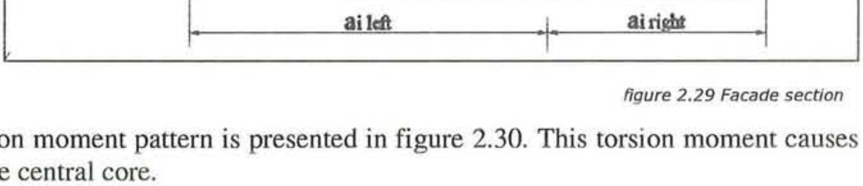


figure 2.29 Facade section

The resulting torsion moment pattern is presented in figure 2.30. This torsion moment causes shear stresses in the central core.

For shear stresses in the core the requirement is: $\boxed{\text{Requirement (u.g.t): } \tau_d \leq \tau_u}$

Where $\tau_u = \tau_1 = 0.3 f_b + 0.15 \cdot \sigma'_{bmd}$. For a core under compression and with no torsion reinforcement.

$$\tau_d = \frac{M_t \cdot \frac{1}{2} d}{I_t}$$

where $I_t = \sum \frac{1}{6} \cdot b_c \cdot d^3 = 2 \cdot \frac{1}{6} \cdot 21750 \cdot 250^3 + 2 \cdot \frac{1}{6} \cdot 21250 \cdot 250^3 = 2.24 \cdot 10^{11}$ mm^4

figure 2.30 Torsion moment

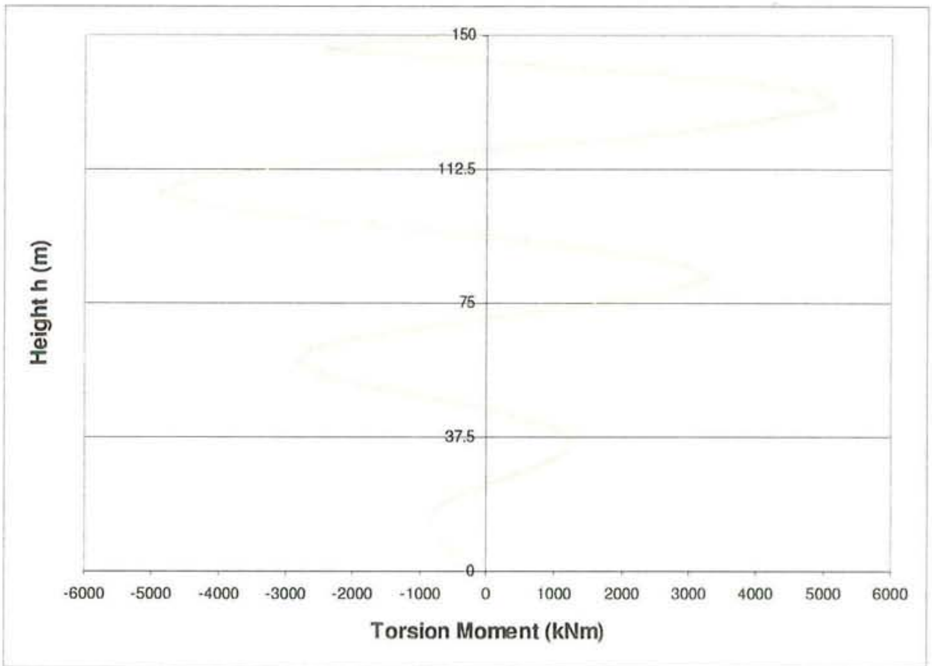
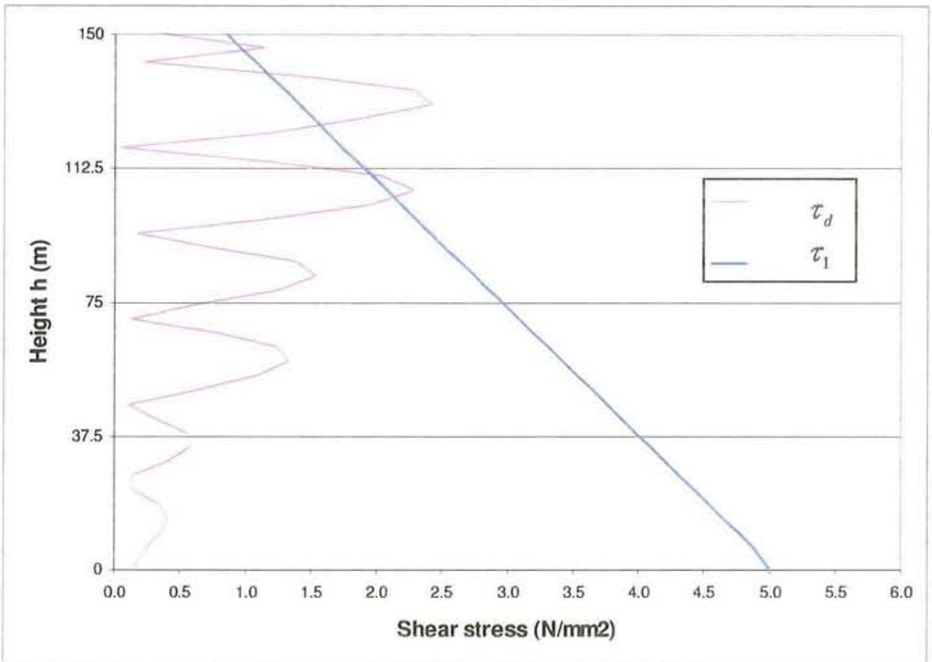


figure 2.31 Shear stress in core (absolute value)



In the diagram above, the absolute value of the shear stresses in the core due to torsion are drawn (red). The blue line resembles the maximum allowed shear stress τ_1. One can see that the shear stresses due to torsion in the core stay within the maximum allowed range, except for two peak shear stresses at 106 and 128 m.
At these locations one can improve the shear stress capacity of the core either by:

- Enlarging the wall width of the core to 300 mm [13]
- Applying reinforcement

If reinforcement is to be calculated for 106 m and upwards, the shear stress $\tau_s = \tau_1 - \tau_d$ that has to be bared by the reinforcement is drawn in figure 2.32. For the calculation the shear stress reinforcement, τ_s is considered to have a constant value of 1 N/mm^2.

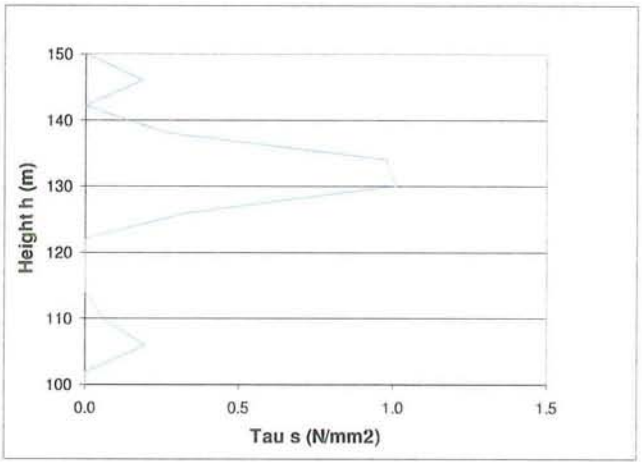


figure 2.32 Shear stress, to be bared by reinforcement

According to NEN6720, the torsion reinforcement can be calculated with:

$$\frac{A_s}{s} = \frac{(\tau_s - \tau_n) \cdot W_t}{2 \cdot A_e \cdot f_s \cdot \cot g\theta}$$

Where: $\tau_s = 1.0\,\text{N/mm}^2$

$$\tau_n = 0.15 \cdot \sigma'_{bmd} = 0.15 \cdot 4.3 = 0.645\,\text{N/mm}^2$$

$$W_t = \frac{M_t}{\tau_d} = \frac{5472.8 \cdot 10^6}{2.5} = 2.07 \cdot 10^9\,\text{mm}^3$$

$$A_e = 21690^2 - 21250^2 = 18893600\,\text{mm}^2$$

$$f_s = 435\,\text{N/mm}^2$$

$$\cot g\theta = 2.33$$

$$s = \text{stirrup distance}$$

This results in: $\frac{A_s}{s} = 0.019$ mm^2/mm

[13] See appendix G 15 for the shear stresses with a core wall width of 300 mm

Vortex shedding

Apart from the wind causing torsion to the central core, the danger of vortex shedding is present. Although the building is only 150 m high and not a super tall structure, the crosswind forces that cause the vortex shedding could be initiated by the asymmetry of the building. On the other hand, the irregular shape of the building may have a positive effect in distorting the shedding of the vortices. One has to make sure that the frequency of the shedding, in any direction, is not in close range to the natural frequency of the building. This would mean that, by a low damping, the building would pulsate as if its stiffness was close to zero. The determination of the natural frequency has to be done by wind tunnel testing, in particular with a complex shaped design like the curved building.

2.4.9 Perforated core

As mentioned before, the core is not a closed tube inside the building. In the functional design floor plan it is clear that there is a hallway planned around the central core at every floor level. The hallway connects the offices located at the outside perimeter of the building to the elevators and stairs in the central core. The cambers that have to be made in the concrete core can only be located on the sides of the flat (non curved) façades. Would the cambers be located on the sides of the curved façades, there may not be any space left for a hallway at certain levels (for instance level 27 and 34).

By locating the cambers on the same side at every level it is questionable if the construction still behaves like a single core, or that its behaviour resembles that of two separate U-shape cores.

If one wants the two U-shaped cores to work together, lintels will have to connect them. The largest bending moment will emerge at ground level (see figure 2.23). At this level, the lintel will have to pass the largest force.

The first lintel between 1st and 2nd floor is not normative because the story height of the first floor is 6 m, whereas the other stories are 4 m high. The normative lintel is located between the 2nd and 3rd floor (the red zone in figure 2.34). The camber size is set on $h \cdot w = 2.2 \cdot 3$ m.

The red zone is enlarged in figure 2.33 and the bending moments at floor levels are indicated on the right.

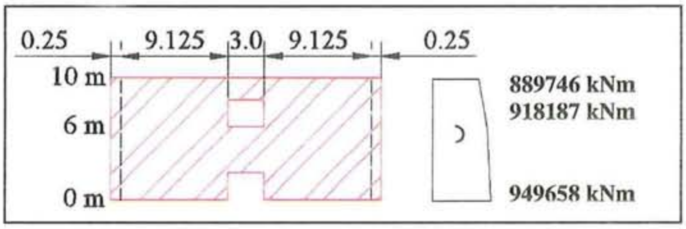


figure 2.33 Normative lintel

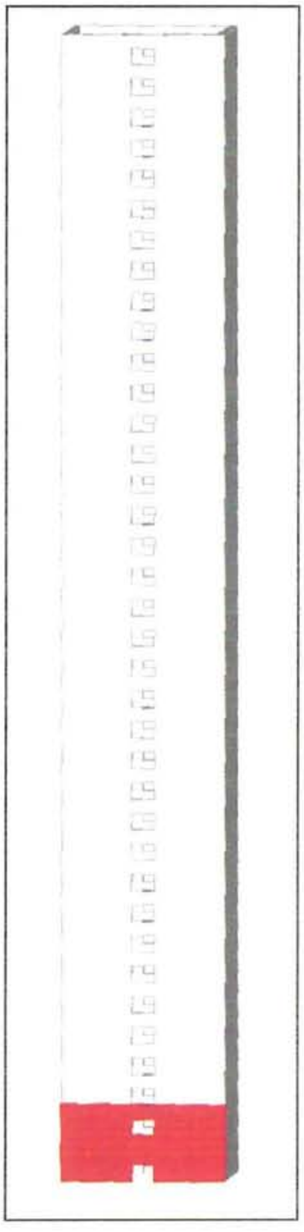


figure 2.34 Central core

$$\Delta M = M_{x=6} - M_{x=10} = 883722 - 856348 = 27374 \text{ kNm}$$

$$\Delta M_{2nd\_order} = \frac{n}{n-1} \cdot \Delta M = 1.039 \cdot 27374 = 28442 \text{ kNm}$$

As a result of this bending moment tensile and compressive stresses will emerge in the walls of the core on the left and the right of the camber as presented in figure 2.35.

$$\Delta\sigma_1 = \frac{\Delta M \cdot 0.5 \cdot b_1}{I} = \frac{28442 \cdot 0.5 \cdot 21.75}{1406.25} = 0.220 \text{ N/mm}^2$$

$$\Delta\sigma_2 = \frac{28442 \cdot 0.5 \cdot 21.25}{1406.25} = 0.215 \text{ N/mm}^2$$

$$\Delta\sigma_3 = \frac{28442 \cdot 0.5 \cdot 3}{1406.25} = 0.030 \text{ N/mm}^2$$

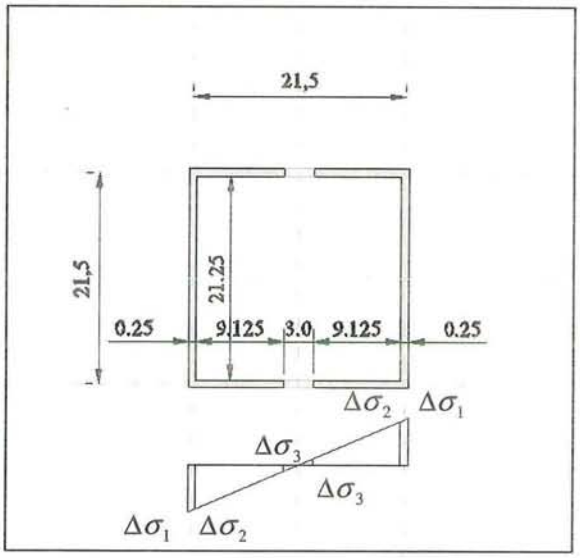


figure 2.35 Tensile and compressive stresses in core wall

The total force is then:

$$\sum F = 21750 \cdot 250 \cdot \left(\frac{0.220 + 0.215}{2}\right) + 2 \cdot 9125 \cdot 250 \cdot \left(\frac{0.215 + 0.030}{2}\right) = 1741.6 \text{ kN}$$

This implicates a force of $F = \frac{1741.6}{2} = 870.8$ kN at each side of the core.

A shear stress of $\tau_d = \frac{870.8}{250 \cdot 1800} = 1.94$ N/mm^2 < τ_2[14]

The resulting bending moment that has to be bared by the lintel is now: $M_{d;l\text{int}el} = F \cdot a = 870.8 \cdot 1.500 = 1306.2$ kNm

The minimal reinforcement in the lintel has to be:

$$A_{s;\min} = \frac{M_{d;l\text{int}el}}{z \cdot h_{l\text{int}el} \cdot f_s} = \frac{1306.2}{0.9 \cdot 1800 \cdot 435} = 1854 \text{ mm}^2$$

This can be achieved by applying 6Ø20 (1885 mm^2)

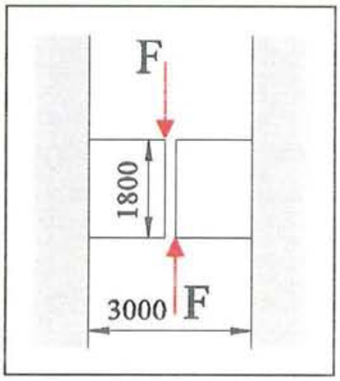


figure 2.36 Lintel scheme

[14] For the calculation of τ_2 see chapter 2.4.7

2.5 Foundation

The foundation of the central core will have to provide enough rotation stiffness in order to keep the top deflection of the building due to rotation of the foundation within the boundary of 150 mm. In paragraph 2.4.2 the minimum needed rotation stiffness of the foundation is determined to be $k_{\min} = 9\ 10^8$ kNm/rad.

The total vertical load on the foundation coming from the core:

Floor weight: $F_{g;floor} = A_{floor} \cdot p_g \cdot (\#\,floors) = 945.6 \cdot 1.88 \cdot 37 = 65776$ kN

Variable weight: $F_q = A_{floor} \cdot p_q \cdot (\#\,floors) = 945.6 \cdot 2.5 \cdot 37 = 87468$ kN

Core weight: $F_{g;core} = A_{core} \cdot \rho_{concrete} \cdot h = 21.5 \cdot 24 \cdot 150 = 77400\,\text{kN}$

Total vertical load: $F_{v;total} = \gamma_g \cdot (F_{g;floor} + F_{g;core}) + \gamma_q \cdot F_q + = 303013.2\,\text{kN}$

Apart from the vertical load, the bending moment will have to be bared by the foundation as well. This bending moment is calculated in paragraph 2.4.3:

$$M_{x=0} = \frac{n}{n-1} \cdot (M_{wind} + M_{initial}) = 1.039 \cdot (900000 + 14012) = 949658\,\text{kNm}$$

The foundation of the central core will consist of a stiff concrete plate under which the piles are to be placed. The assumption is made that a concrete plate with a height of 2.5 m should provide enough stiffness to bear the load[15]. This foundation plate will be supported by concrete precast piles of 500 x 500 mm.

Minimum c.t.c. distance piles: $e_{\min} = 4.5 \cdot d_{pile} = 4.5 \cdot 0.5 = 2.25$ m

Number of piles: $n_{piles} = \frac{A_{plate}}{A_{pile,eq}} = \frac{24^2}{2.25^2} = 114$

If 10.6 x 10.6 piles are employed to the foundation the available space is utilized to its full extend, 10 x 10 piles are placed in this case.

Total vertical load on pile: $F_{pile,total} = F_N + F_M$

Force on pile by vertical load core: $F_N = \frac{F_{v;total}}{n} = \frac{303013.2}{100} = 3030\,\text{kN}$

Force on pile by bending moment: $F_M = \sigma_M \cdot A_{pile}$

[15] Reference projects in [fund5330]

where: $\sigma_M = \pm \dfrac{M_{x=0} \cdot z}{I_{pilegroup}}$

$$I_{pilegroup} = rows \cdot \sum A_{pile;i} \cdot a_i^2 = 10 \cdot 0.5^2 \cdot 2 \cdot (1.25^2 + 3.75^2 + 6.25^2 + 8.75^2 + 11.25^2) = 1289 \text{ m}^4$$

$$\sigma_M = \pm \frac{949658 \cdot 11.25}{1289} = \pm 8.29 \text{ N/mm}^2$$

$$F_M = \pm 8.29 \cdot 0.5^2 = \pm 2073 \text{kN}$$

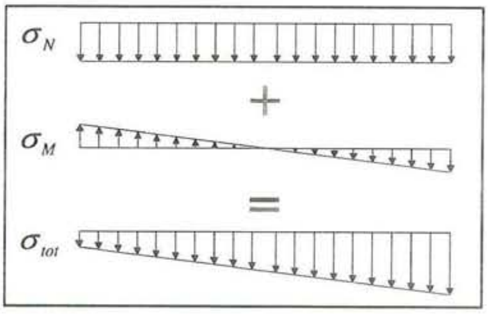


figure 2.37 Pile stresses

The normal forces in the piles generated by the gravity load and by the bending moment will result in compression in the piles at all times. No tension piles are required, only bearing piles.
The minimum and maximum forces in the piles (at the outside perimeter) are:

$$F_{pile,total,1} = -3030 + 2073 = -957 \text{ kN}$$
$$F_{pile,total,2} = -3030 - 2073 = -5103 \text{ kN}$$

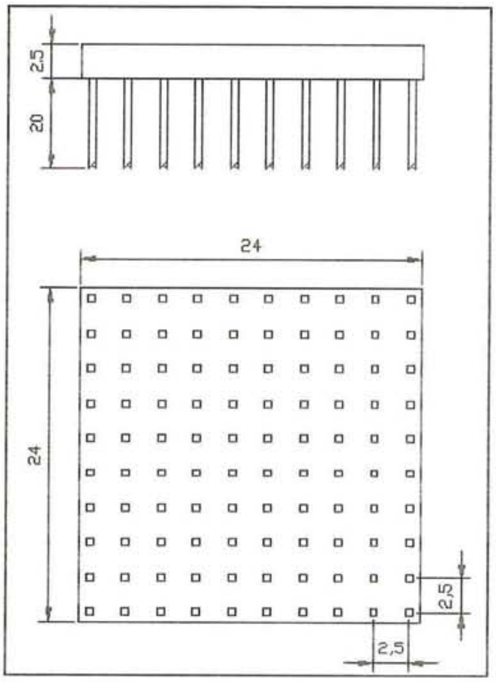


figure 2.38 Pile layout

Foundation rotation stiffness

The stiffness of pile i can be calculated with: $k_i = \frac{EA}{l_{eff}}$, with $l_{eff} = 1.5 \cdot l_{pile}$

Assumption location constructive soil layer: -20 m
E-modulus pile: 28.5 10^6 kN/m^2 (B25)
Section surface pile: 0.25 m^2

$$k_i = \frac{28.5 \cdot 10^6 \cdot 0.25}{1.5 \cdot 20} = 237500\,\text{kN/m}$$

The total stiffness of the foundation k_{total} consists of the sum of square distances from pile to centre point times the k_i:

$$k_{tot} = \sum a_i^2 \cdot k_i = 10 \cdot 2 \cdot (1.25^2 + 3.75^2 + 6.25^2 + 8.75^2 + 11.25^2) \cdot 237500 = 1.2 \cdot 10^9\,\text{kNm/rad}$$

$\boxed{\text{Requirement: } k_{tot} \geq k_{\min}}$

$$k_{tot} = 1.2 \cdot 10^9 \geq k_{\min} = 9 \cdot 10^8 \text{ kNm/rad}$$

The rotation stiffness of the foundation meets the required figure. The top deflection caused by the rotation of the foundation will therefore not exceed the maximum of $\frac{1}{1000} \cdot h = 150\,\text{mm}$.

2.6 Design overview

2.6.1 3D images

The resulting shape of the structure is shown in the picture to the right. In this drawing all structural members can be noticed except for the steel-concrete floor plates.

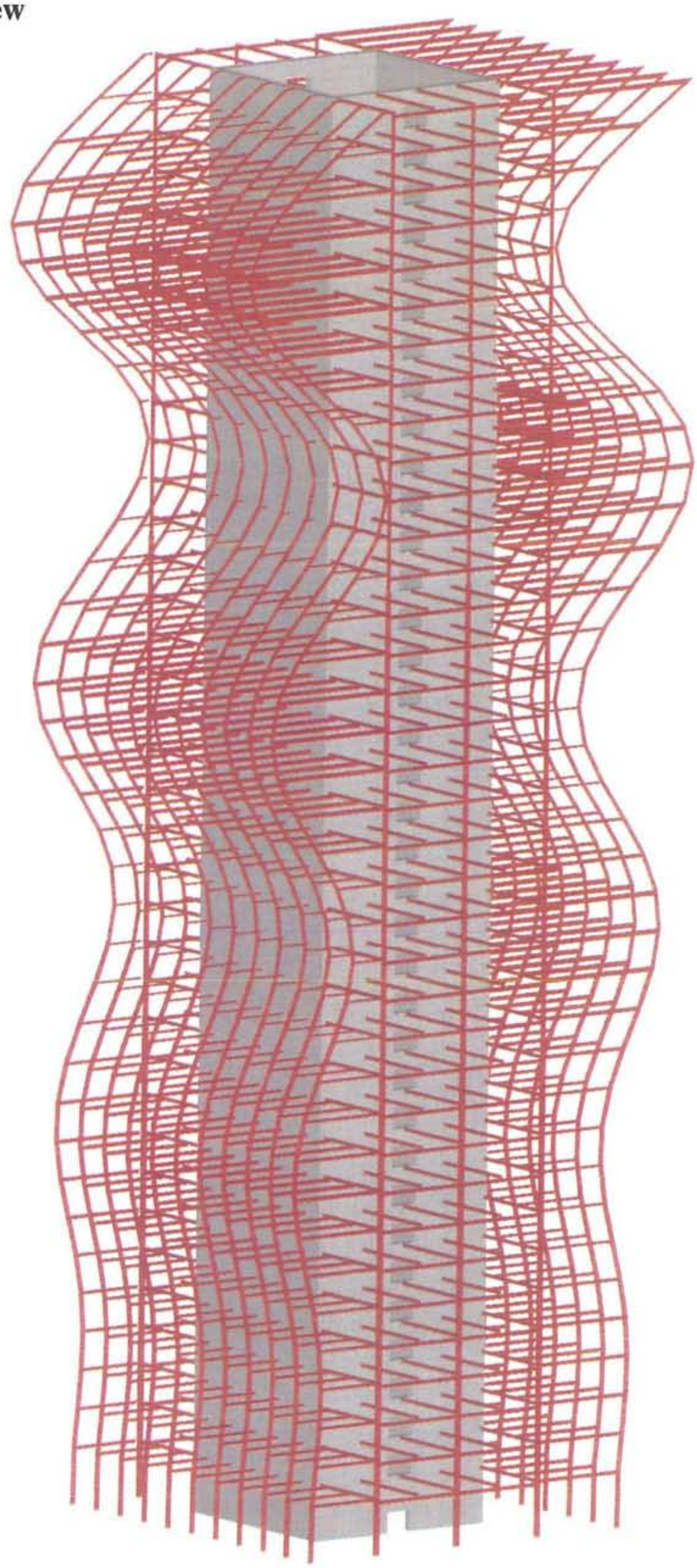


figure 2.39 3D structural drawing

In figure 2.40 the same structural drawing is presented, now including the steel-concrete floor plates.

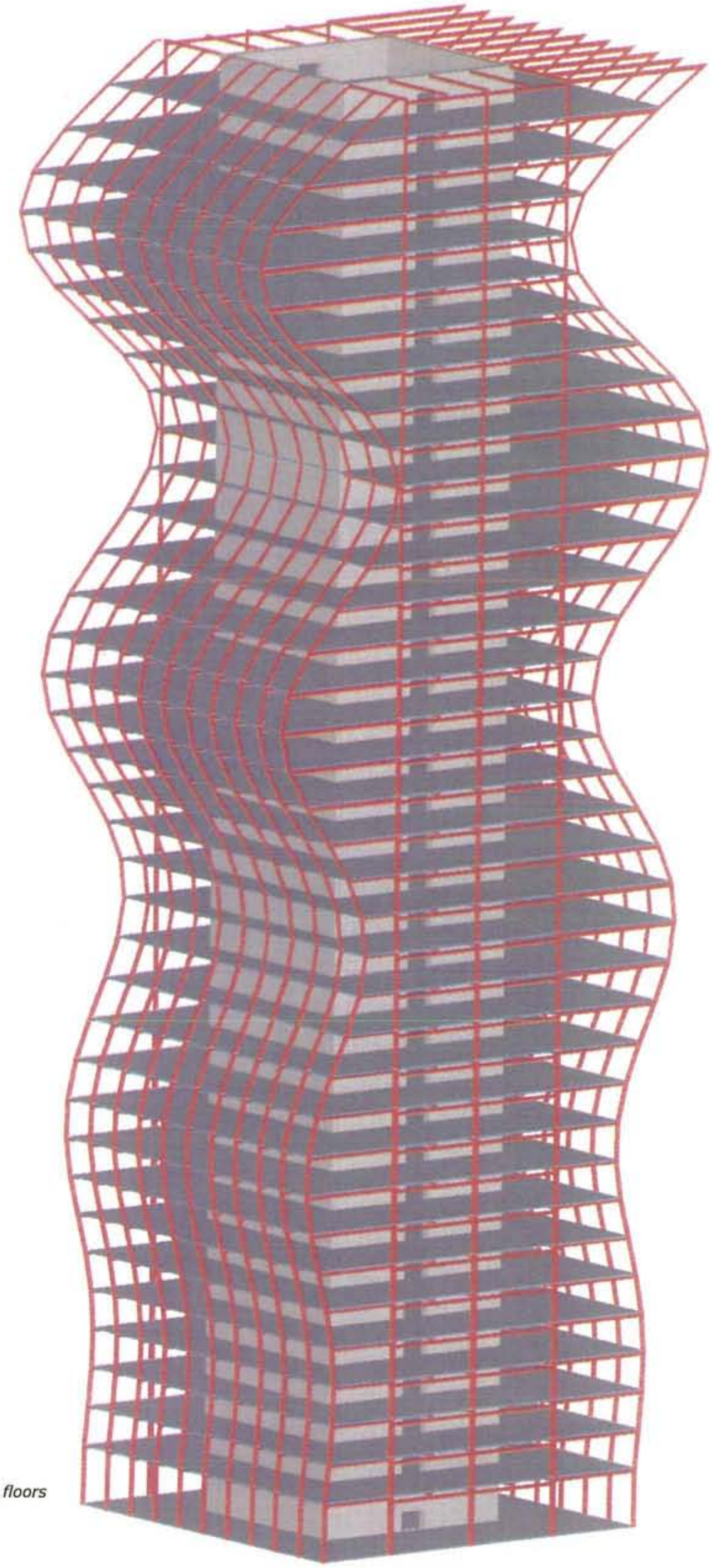


figure 2.40 3D structural drawing, incl. floors

2.6.2 *Typical details*

In this chapter some typical details that have a lot of repetitiveness are highlighted. The details that will be analyzed are:

D1: Inclined column – beam – inclined column
D2: Straight column – beams II and IIII
D3: Beam – core

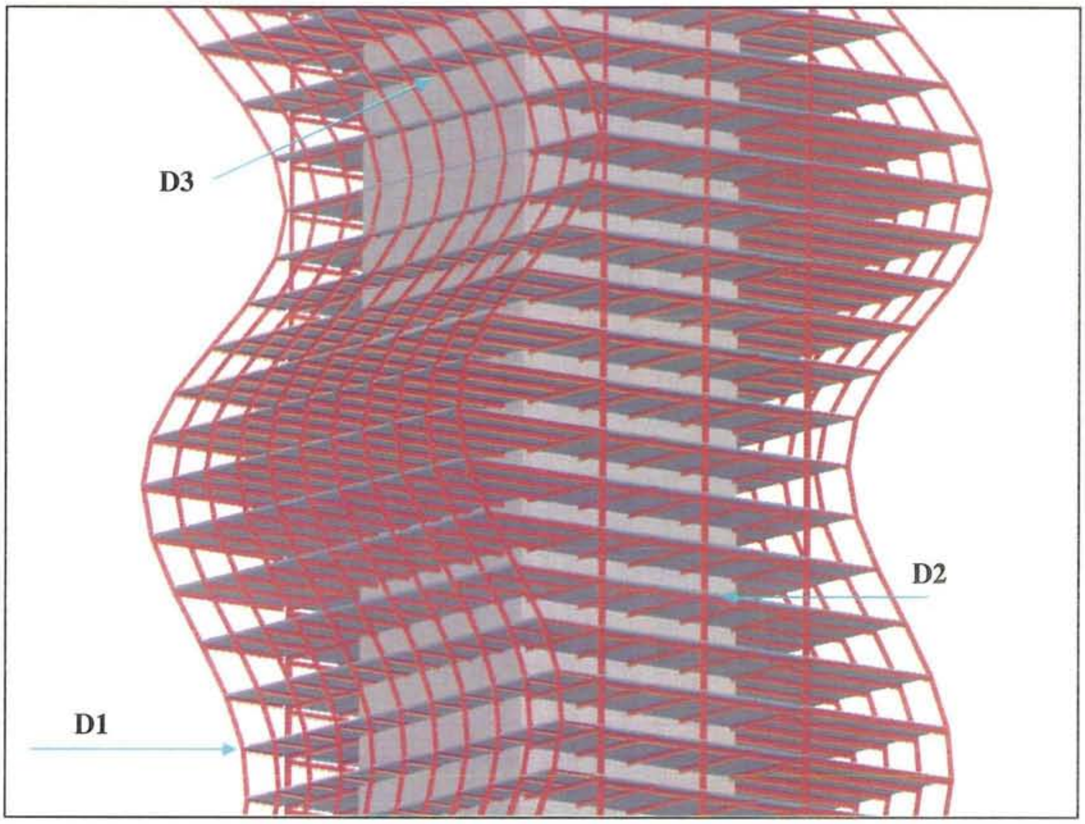


figure 2.41 Detail location

The location of the of the analyzed details are shown in figure 2.41.

Inclined column - beam - inclined column

The connections between the inclined façade columns and the beams from façade to core will have to be designed separately for every floor level. Because of the reverse damped shape of the building, the angle of each inclined column will be unique. Of course inclined columns have the same angle on both sides of the building and are made of the same profile, but the floor beams will differ from side to side because of the different story lengths on one level. This makes every connection unique, even connections on the same level.

The connections are designed to act like hinges and the inclined columns only have to bear a centric normal force. Because of the difference in angle of two connected columns a lateral component will occur, as shown in chapter 2.2. This lateral component has to be passed to the connected beam.
The joint at a level of 82 m high will be used as an example for the typical calculation of the joint. At this level, the largest lateral force will occur: $F_{fi;left;total} = 7297$ kN. This force is spread over 10 beams, thus a single beam is loaded with 729.7 kN.
The typical connection of the inclined columns and the floor beams is presented in figure 2.42. A base plate will be welded to both ends of the inclined column so that the columns can be bolted to the beam. The beam is reinforced by welded plates in order to secure a proper connection of the column flanges. Once the beam is bolted to the lower column, the next column can be placed, etc. Base plates and beam plates are coloured orange to make them more visible.

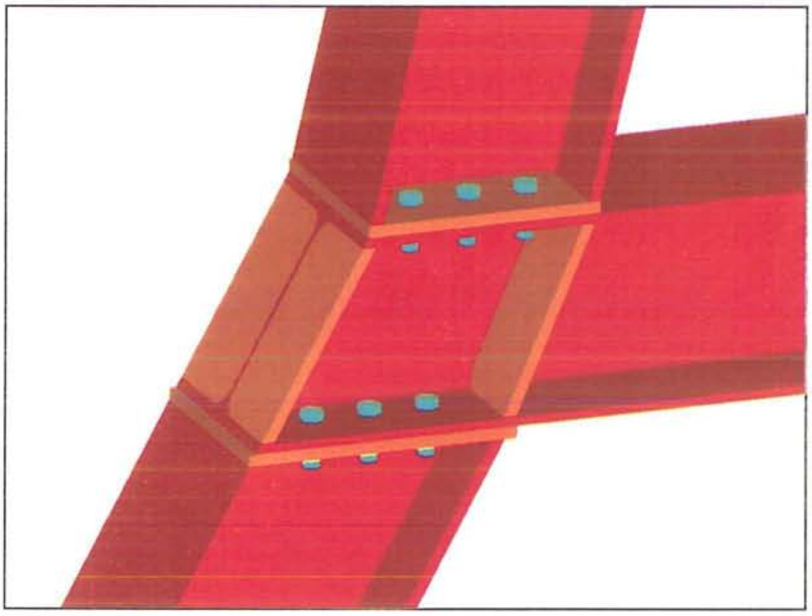


figure 2.42 3D Inclined column-beam joint at 82 m

The bolts in the joint will bear the lateral force component as a shear force, as modellised in figure 2.43. The maximum shear force capacity is dependent on the number and type of bolts that is applied. For the joint at 82 m, which has to bear the largest lateral force, six M24 bolts will be sufficient to withstand the load.

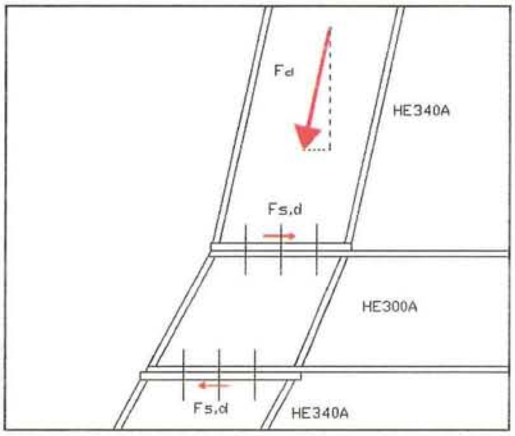


figure 2.43 82 m joint

Section surface M24: $A_{s;b} = 353\,\text{mm}^2$

Shear force on section: $F_{s;d} = F_{f;82m} = 729.7\,\text{kN}$

Maximum shear force M24: $F_{v;u;d} = \dfrac{0.6 \cdot \alpha_{red;2} f_{t;b;b} \cdot A_{b;s}}{\gamma_m} = \dfrac{0.6 \cdot 1 \cdot 800 \cdot 353 \cdot 10^{-3}}{1.25} = 135.5\,\text{kN}$

Number of bolts: $n = \dfrac{F_{s;d}}{F_{v;u;d}} = \dfrac{729.7}{135.5} = 5.38 \rightarrow$ 6 bolts M24

The minimum distances s_1 and e_1 will not be violated:

$e_1 \geq 1.2 \cdot d_{h,nom} = 1.2 \cdot (24+2) = 32.2$ mm

$s_1 \geq 2.2 \cdot d_{h,nom} = 2.2 \cdot (24+2) = 57.2$ mm

$e_1 + s_1 + s_1 + e_1 = 178.8\,\text{mm}$, whereas the inside height of the HE340A profile is 297 mm.

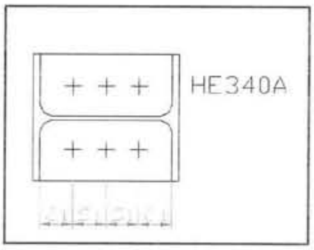


figure 2.44 Bolt location

Straight column – beams II and IIII

The beams of type III will show no difference in profile or length in the building. The beams will support the same floor surface irrespective of the floor level they are located (see paragraph 2.2).
A typical connection a beam to a straight column can be found at a height of 6 m. A connection needs to be made between a HD 400x818 column, two HE360A beam and a HE340A beam. All connections will be hinged connections, there is no need for any moment-fixed joints.

The concrete steel plate floors will be placed on top of the HE340A girders (beams II), whereas the HE360A profiles (beams III) will support the HE340A girders. The HE360A profiles will be placed in the perimeter of the building and therefore will not affect the internal height of the office. The typical connection is drawn below.

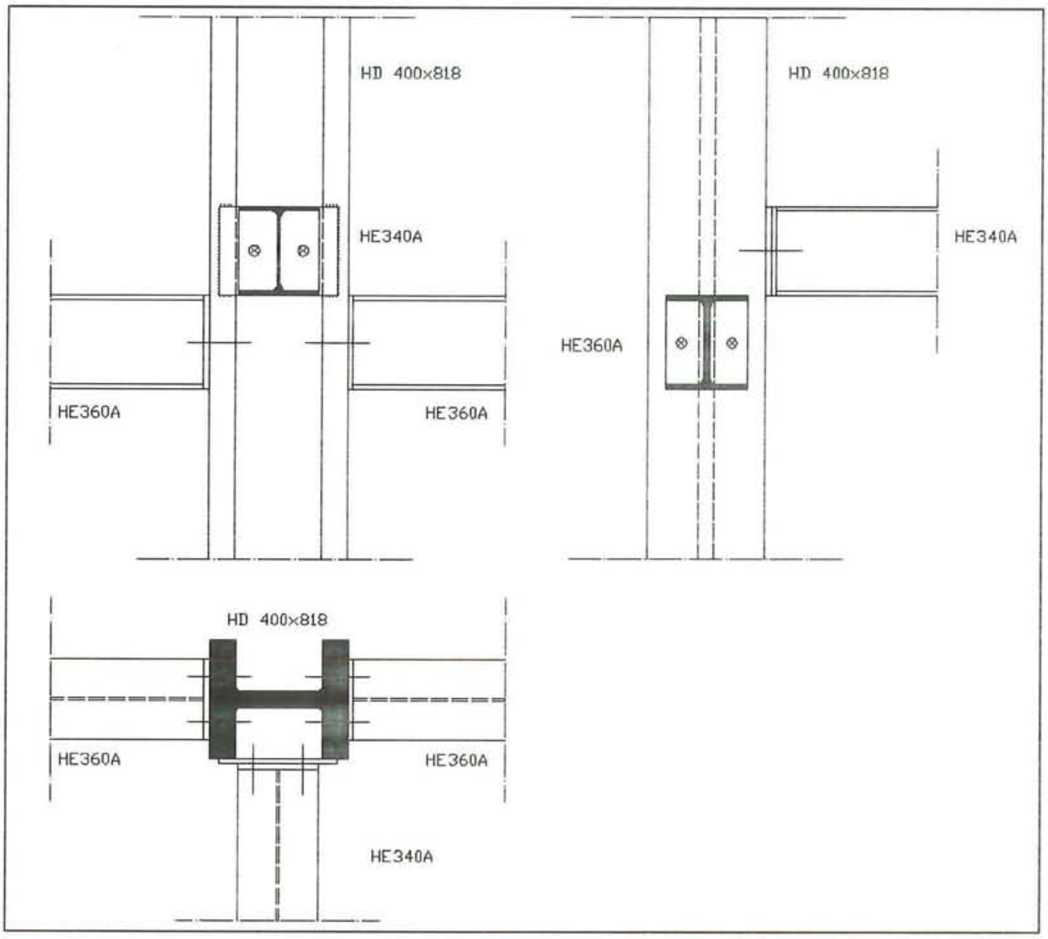


figure 2.45 Connection straight column- beam II and III

The connections will be made by bolts as drawn above. Since the connections are all hinged, the bolts will be subjected to shear forces. The bolts in the joint must be able to bear the vertical shear generated by the floor loads. The maximum shear force capacity is dependent on the number and type of bolts that is applied. For the connections at the two non-curved sides of the building, two M20 bolts will be sufficient to connect the HE340A (beam II) to the straight column and two M16 bolts will do for the connection of the HE360A to the straight columns. For the connection of the HE340A girder a steel plate will be welded to the column in order to make a bolted connection possible.

HE340A (beam II) connection to column:

Section surface M20: $A_{s;b} = 245\ \text{mm}^2$

Shear force on section: $F_{s;d} = \frac{1}{2} \cdot q_d \cdot l_{HE340A} = \frac{1}{2} \cdot 33.55 \cdot 9.25 = 155\ \text{kN}$

Maximum shear force M24: $F_{v;u;d} = \frac{0.6 \cdot \alpha_{red;2} f_{t;b;b} \cdot A_{b;s}}{\gamma_m} = \frac{0.6 \cdot 1 \cdot 800 \cdot 245 \cdot 10^{-3}}{1.25} = 94\ \text{kN}$

Number of bolts: $n = \frac{F_{s;d}}{F_{v;u;d}} = \frac{155}{94} = 1.65 \rightarrow 2\ \text{bolts M20}$

HE360A (beam III) connection to column:

Section surface M16: $A_{s;b} = 157\ \text{mm}^2$

Shear force on section:

$$F_{s;d} = \frac{1}{2} \cdot \frac{1}{2} \cdot q_d \cdot l_{HE340A} + \frac{1}{2} \cdot q_{HE360A} \cdot l_{HE360A} = \frac{1}{2} \cdot 155 + \frac{1}{2} \cdot 1.12 \cdot 10.75 = 83.6\ \text{kN}$$

Maximum shear force M24: $F_{v;u;d} = \frac{0.6 \cdot \alpha_{red;2} f_{t;b;b} \cdot A_{b;s}}{\gamma_m} = \frac{0.6 \cdot 1 \cdot 800 \cdot 157 \cdot 10^{-3}}{1.25} = 60\ \text{kN}$

Number of bolts: $n = \frac{F_{s;d}}{F_{v;u;d}} = \frac{83.6}{60} = 1.39 \rightarrow 2\ \text{bolts M16}$

In figure 2.46 a 3D model of the joint is presented. Again, the plates on the beams and the plate welded to the columns are coloured orange.

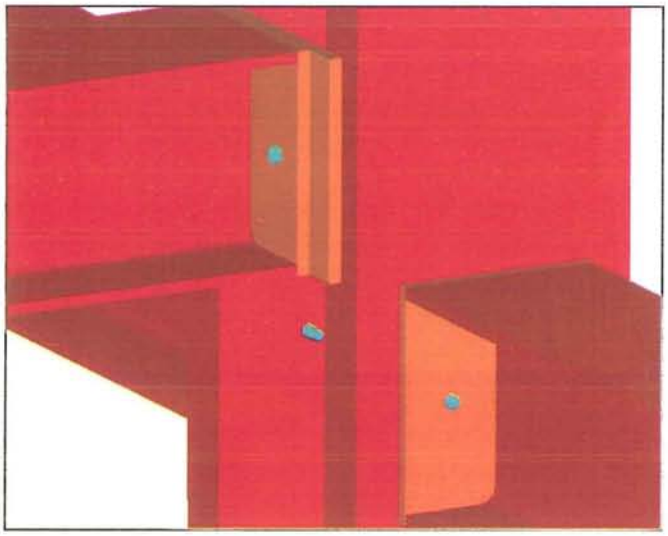


figure 2.46 3D joint

Beams – Core

As mentioned before, the central core provides the stability for the building. The steel framework with its curved shape is "folded" around the concrete core. A connection between the framework and the core is made by the beams and girders that are supported by the core on one side and by the steel construction in the façade of the building on the other side. The connection to the perimeter columns is analyzed in the paragraphs above, but the connection to the central core is still unknown.

Like all other joints in the steel framework, the connection of the beam and core is designed to be hinged. The type of connection between the beams and the core is dependent on the way the building will be constructed. The central concrete core will be constructed by applying sliding formwork. This is a relatively easy and fast way to erect the core.
A possible solution may be to place anchors in the concrete core. When the concrete is hardened the anchors can be bended into the right position so the girders can be attached to the core. A disadvantage of this solution is the low precision of the bended anchors. Large tolerances will be needed and the girders will not have the same horizontal position at every location.
A better solution is to let the girders stick into the core wall. This connection of the beams to the core can be achieved by leaving cambers open in the core and then hoist the girders into place. The remaining gap can be filled with epoxy in order to close the core and secure the stiffness of the system.

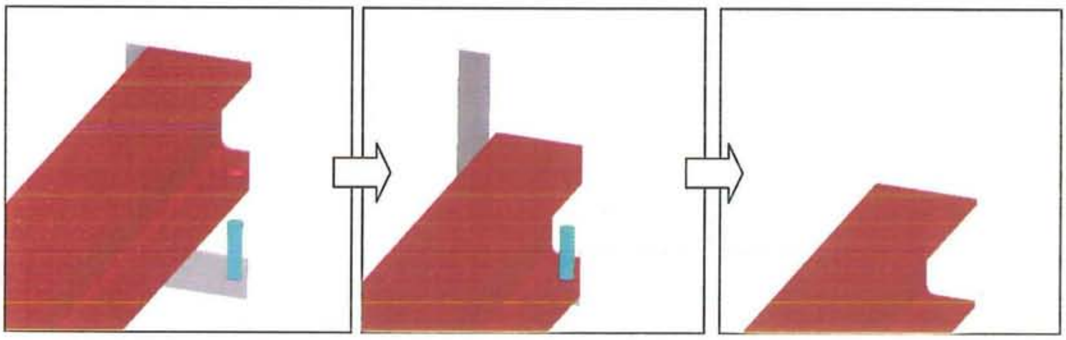


figure 2.47 Connection core-girder

3 Functional analysis

The structural investigation and the case study proof the combination of the straight central core and the inclined façade columns to result in an excellent building behaviour. That is, an excellent building behaviour from a structural point of view.
This applied primary structural system will have serious consequences for the functional behaviour of the building. The most obvious negative side effects are:

1. The loss of floor space
2. No repetitive floor plans

In this added chapter the negative side effects will be looked upon more closely. Practical solutions and recommendations will be made to provide a starting point for further examination.

3.1.1 Loss of floor space

The combination of the straight central core and the inclined façade of the building will have serious consequences for the floor plans. The available room in the "armpits" of the curves may not be sufficient to allocate offices or even other secondary functions like closets and toilets.
The minimum functional measurements for an office are stated in the "bouwbesluit" as:

- $Area \geq 10\ \text{m}^2$
- $Width \geq 1.8\ \text{m}$
- $Height \geq 2.4\ \text{m}$

The total floor area of the building, excluding the area inside the core is $40^2 - 21.75^2 = 1127$ m^2/floor. Which makes the total area $1127 \cdot 37 = 41697\ \text{m}^2$.

By projecting the minimum measurements on the case study building, the resulting usable office space can be found. In the drawing on the next page the vertical dashed lines represent the minimum with of the hallway and the minimum office width (1.8 m). The horizontal dashed lines represent the minimum office height (2.4 m). The lost office space is coloured red and the usable space is coloured green. The red triangles in the drawing resemble unusable (floor)space that cannot be avoided due to the inclined façade.

The usable (green) floor area covers 40020 m^2, that is 96% of the total available floor space, only 4% is considered to be lost office space.
This figure assumes a minor effect of the structural system on the usable floor space in the building. In a strict way this is true, the usable floor space is not reduced significantly by the structural system and the achieved ratio is very well acceptable. But, the minimum figure of 1.8 m office width as stated in the "bouwbesluit" is unlikely to be accepted by the future occupier of the building. A width of 1.8 m is simply to small for an occupant to find the building interesting. An office width of 3.5 to 4 m is more likely to be the minimum for a future occupant. If a minimum width of 3.5 m is applied to the building, the usable floor space shrinks to 38594 m^2, that is 93 % of the total available floor space. The ratio is still relatively

high, because of the large applied amplitude of 9.2 m. Due to this amplitude, the usable floor space is only reduced in the "armpits" of the building (see figure 3.1). At these spots the available floor space was already relatively low, so the reduction at these levels does not influence the available usable floor space too much. An overview of the available and usable floor space is listed in appendix H.

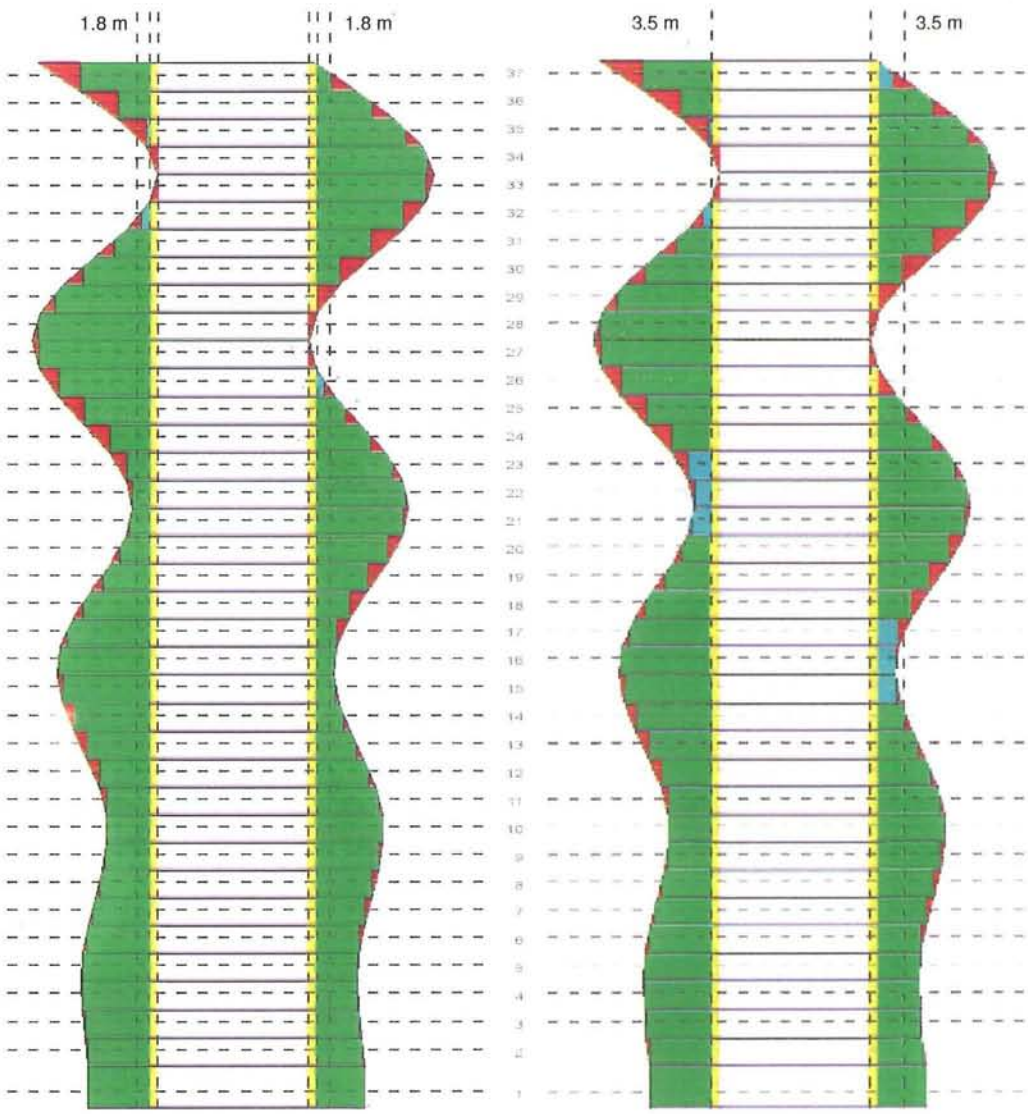


figure 3.1 Usable floor space

The loss of floor space is caused by the inclined façade and by the straight central core inside the building. The curved shape will always result in floor space that is not as useful as with a straight façade. There is no solution to change this effect other than to straighten the façade, which results in a building that will not meet the goal of creating a curved building. This means that the red triangles in figure 3.1 cannot be avoided if one wants to keep the shape of the building. These areas cannot be used in a way that people are able to work, because of the inclined façade. But they do not necessarily need to be useless. A storage function can very well be assigned to these areas and they can be used for cables and ducts as well. Like the red areas, the blue cannot be used as office space. The difference is that the blue areas are unusable because of the closed vertical core. The resulting floor lengths are too small to locate an office at these floors. These floors could be utilized if a more open core was employed to the building (see paragraph 3.1.3).

If a minimum office width of 3.5 m is applied to the building, the total loss of floor space will be 3103 m^2 (7%). The loss of floor space that cannot be avoided, caused by the inclined façade only, independent from the closed core, is 1877 m^2 (4%). Whereas the loss of floor space caused by the closed core is 1226 m^2, 3% of the total available floor space.
Only 3% of the total available floor area can be considered to be avoidable lost floor area. A percentage of 4% cannot be used for office purposes because of the inclined facades, but this cannot be avoided. This unusable floor space could be used for storage, ducts or cable purposes.

3.1.2 Repetitive floor plans

In the previous paragraph the usable floor space was determined to be 93 % of the total available floor space. This figure may be found acceptable, even more if the unavoidable floor space is kept in mind. The reduction of floor space due to the closed core is only 3%.
Still, with the relatively high usable floor space ratio, a possibly even larger problem remains unsolved: the non-repetitive floor plans.

Since no single floor plan will have the same measurements, all floor plans of the building need a unique design. The key of this problem lies in the straight central closed core of the building. Due to the closed core the offices have to be located at the outside perimeter and cannot be (partially) shifted inside the core, resulting in an altering floor plan from floor to floor as shown in figure 3.2.

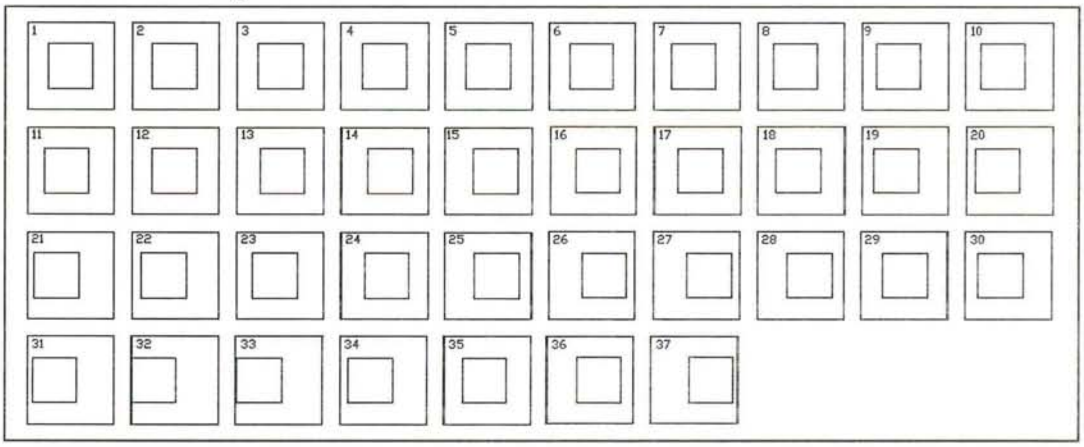


figure 3.2 Altering floor plan

Repetition in floor plans will be very hard to realize, in fact a specific layout has to be designed for (almost) every story. A floor plan that has to be re-designed 37 times will be the consequence of the straight and closed concrete core. The negative side effects are:

- High costs for design and construction of the interior offices
- High costs for the non-repetitive HVAC design and construction
- A lost feeling of the occupant because of the constantly changing office locations.

A repetitive floor plan could change these negative effects of the current design into cost-saving properties. A total design of a floor plan layout will only have to be made once and can be employed to all stories of the building.

Since the curved façade of the building cannot be altered without losing the curved shape of the building, the solution has to be found in altering the closed concrete core. By creating a more transparent core a more repetitive floor plan should be possible. Restrictions for the repetitive floor plan will no longer come from the closed core, but from the shafts for vertical transport (elevators, staircases and ducts). The floor space reserved for vertical transport is much smaller that the area enclosed by the core. In the drawings below, a number of possible floor layout configurations is drawn.

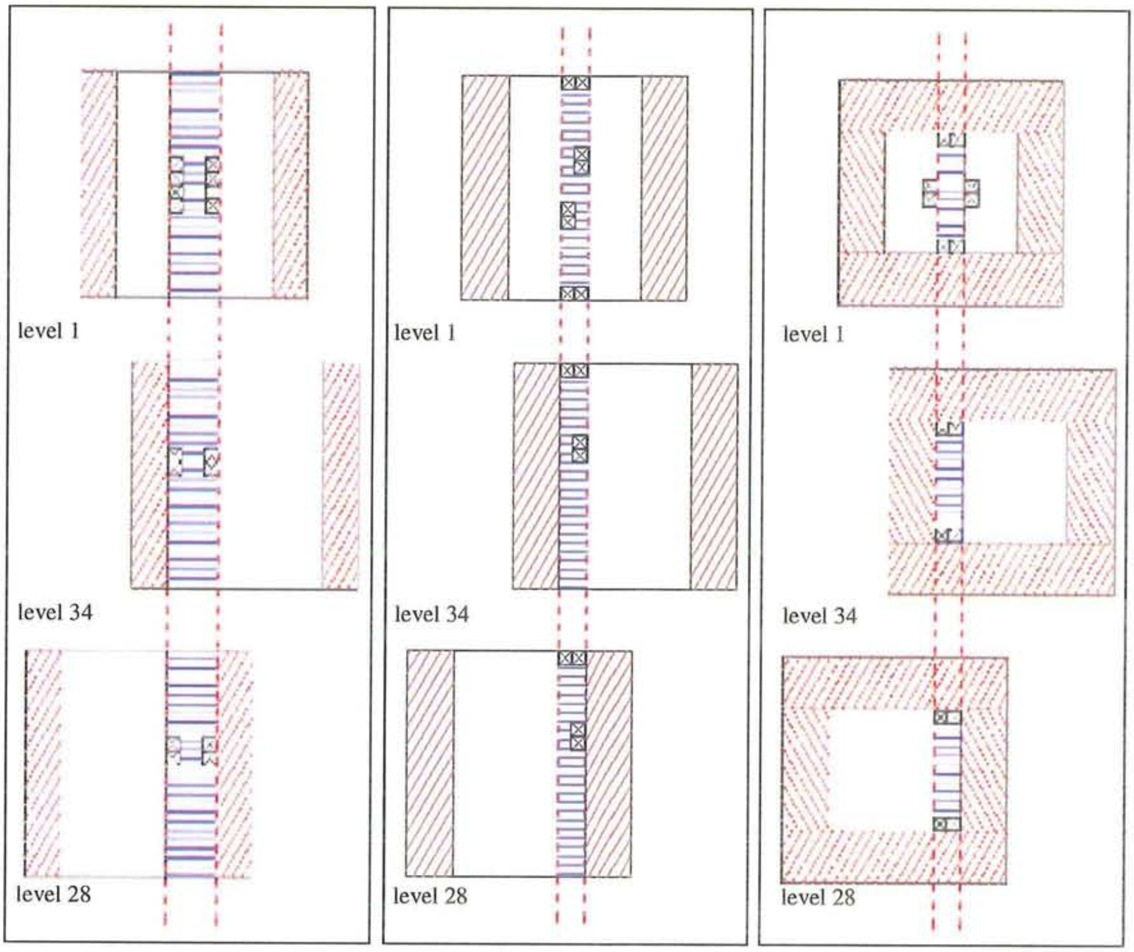


figure 3.3 floor plan configurations (a,b,c)

The dashed red line marks the "vertical transport zone" where vertical shafts and ducts do not interfere with the floor plan. The floor plan switches from left to right, with the most extreme position at level 28 and 34 (see above).
The vertical transport zone determines the maximum size of the repetitive floor plan on the left and right side of the building (diagonally hatched). By changing the width of this zone, the size of the repetitive offices will change as well. Elevators and vertical ducts that have to be located in the vertical transport zone will determine the minimum size of the transport zone. A problem that comes with the allocation of the elevators is that space for a hallway to reach the elevators has to be reserved as well. In figure 3.3 and figure 3.4 a number of elevator plans and the resulting maximum repetitive zones are drawn.
Options b and d result in the largest repetitive floor plans. Disadvantage of these options is that the elevators are not placed in groups, but stretched to the outside perimeter of the building. Consequence of this elevator plan is that people have to walk from one side of the building to the other in order to reach an available elevator. Therefore it is preferable that the elevators are placed close to each other.

The best layout may be floor plan e. It combines large repetitive zones on all four sides of the building with a relatively small vertical transport zone. The elevators are concentrated in the middle of the building and standard offices can be placed at the best location: the outside perimeter of the building.
In the vertical transport zone, there is enough space left to employ ducts and toilet groups. A possible floor layout, type e, is drawn in figure 3.5 on the next page.

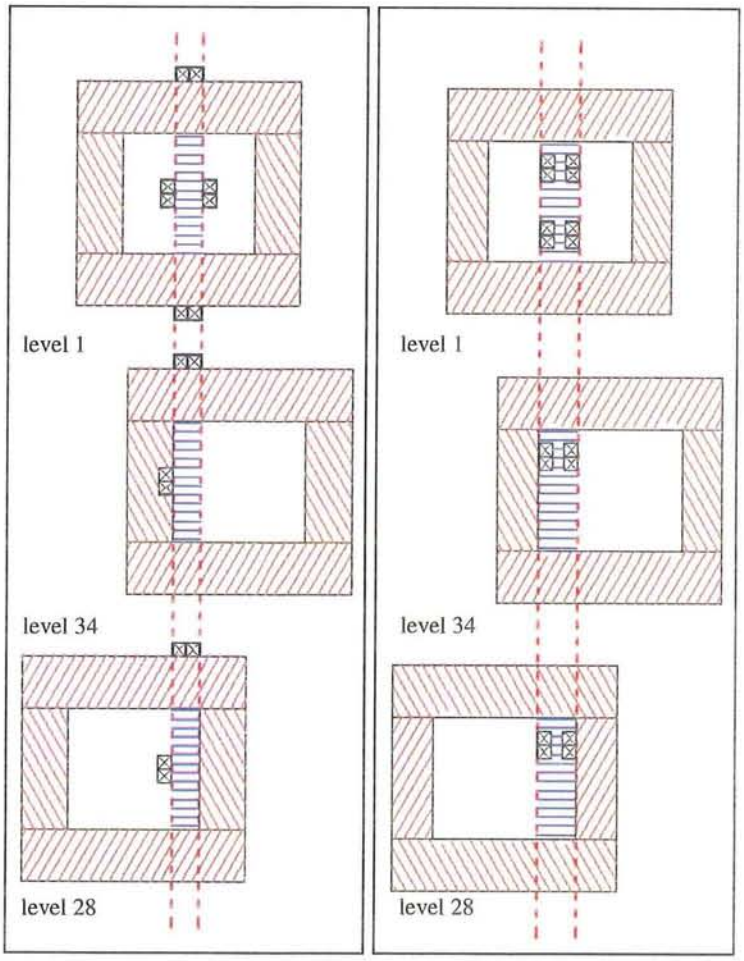


figure 3.4 floor plan configurations (d,e)

In the floor layout on the right, the offices on the outside perimeter of the building have a standard length of 7.2 m.
The vertical transport zone is enlarged and shows the eight elevators that connect the floors vertically. Apart from the elevators, two staircases and two vertical shafts for HVAC purposes can be noticed as well. The central part of the vertical zone is reserved for a toilet-block.
The offices can be located at the most valuable location; the outside perimeter of the building. Due to the shifting of the floors from left to right non repetitive floor space cannot be avoided, but it has been reduced to minimal proportions. The non-repetitive space in the centre of the building will be used as conference rooms, storage, fitness, lunch rooms, etc.
At level 28 and 34, the offices "touch" the vertical transport zone. This means that there is no room for a hallway to reach the offices like on the other levels. A direct entrance from the elevator lobby to the offices is a possibility, but it is not preferable. A better solution may be to connect the interior of these offices or by removing some of the interior walls. By doing so, an openplan office can be created and the offices can be reached via the repetitive hallways.

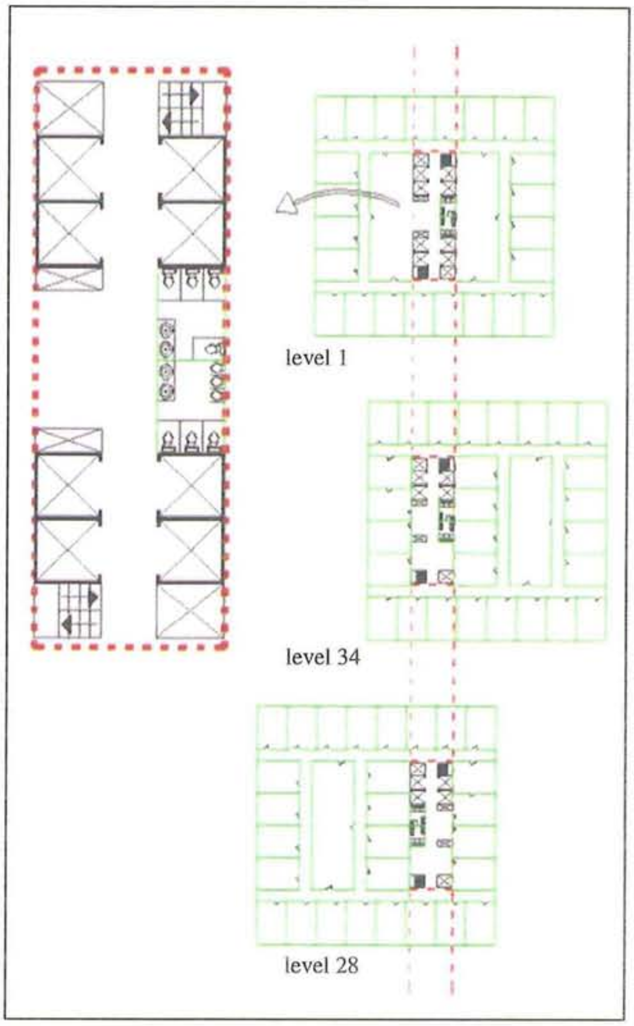


figure 3.5 Floor layout

3.1.3 *Structural alternatives for better functional behaviour*

The repetitive floor plan as described above cannot be employed to the building with a closed core as its primary structural system. The closed core will limit the repetitiveness of the floor layouts too much. If one wants to achieve the repetitive floor layout, the primary structural system will have to be an open and transparent construction. In this paragraph a number of open structural alternatives for the primary structural system of the curved building will be mentioned.

Braced frame

The braced frame system is a possible solution for a more open construction. With a combination of two frames the wind load and lateral load from the floors can be distributed between the two systems. The deflection pattern of the rigid frame structure is that of a shear construction. Each floor with its own deflection, depending on the magnitude of the shear at that particular floor level. The braced frame acts like a cantilever beam, regarding horizontal deflections. At the bottom the construction is very stiff, but deflections of every story cumulate, resulting in a relatively large horizontal drift at the top of the building. The connection of the two different systems provides a more overall stiff structure. The connection by floors enforces compatibility of deformation at the floor levels. The (shear mode) frame tends to pull back the top and push forward the lower part of the (cantilever) braced frame.
As an open construction, the braced frame is suitable for the curved building. What could cause functional difficulties are the diagonal members of the bracing. Door and hallways can be blocked at certain levels. The bracing can be placed at the most appropriate position so it will not block any hallways. A possible block of an office entrance can be solved by simply moving the door of the interior, non-constructive wall. The bracing can be constructed in many

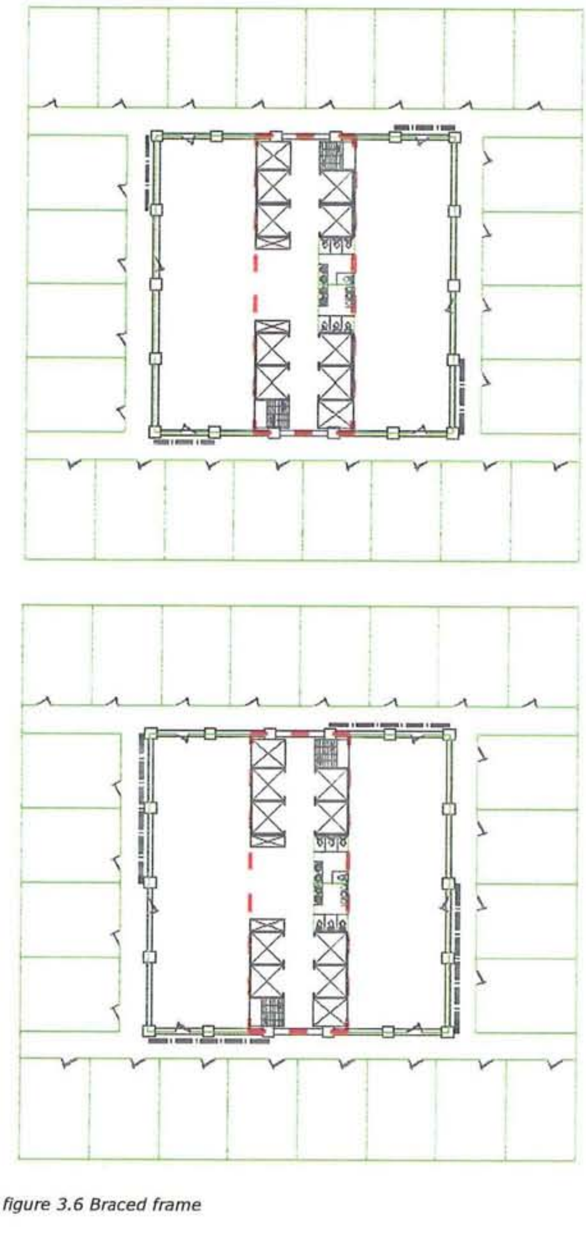


figure 3.6 Braced frame

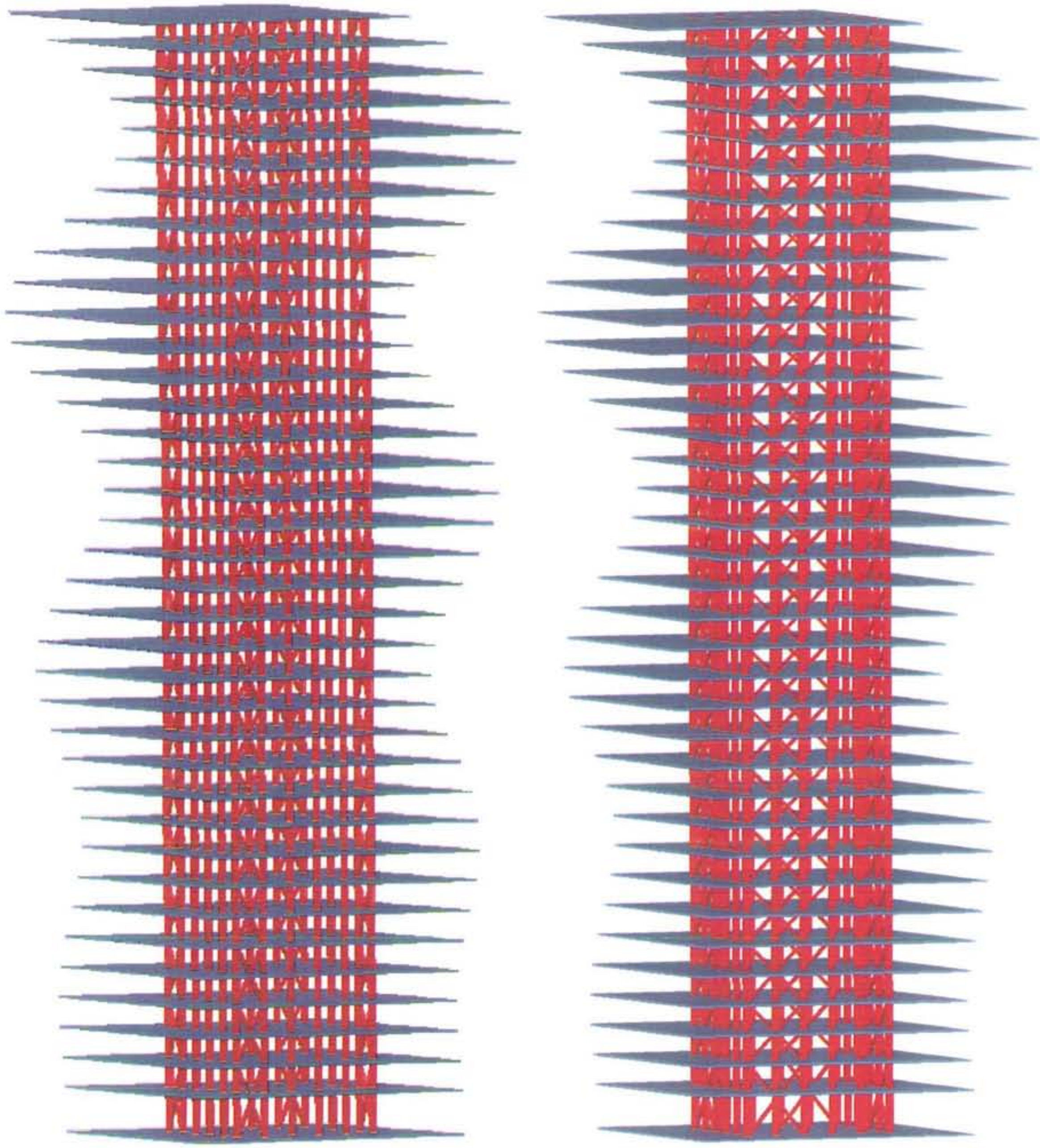


figure 3.7 3D model braced frame

different shapes and configurations. In the layouts on the previous page and in figure 3.7 two possible types of a braced frame are drawn. In the 3D-models only the primary structural system and the floors are drawn to improve the readableness. One should investigate and analyze the minimum measurements of the structural members to come to an exact structural design, but the open idea is clearly visible.

Megastructure

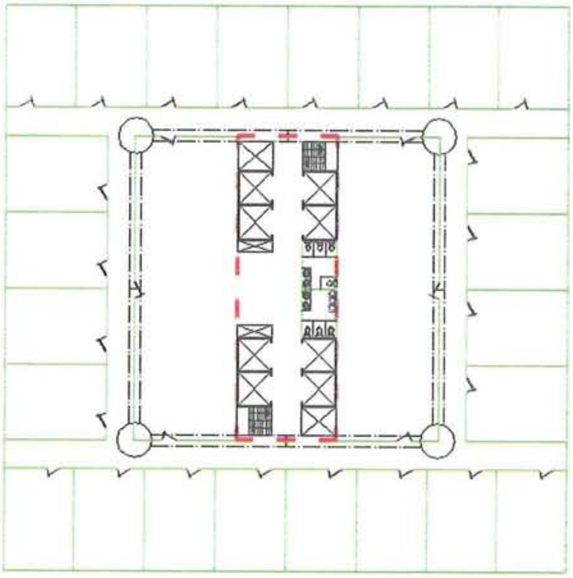


figure 3.8 Megastructure

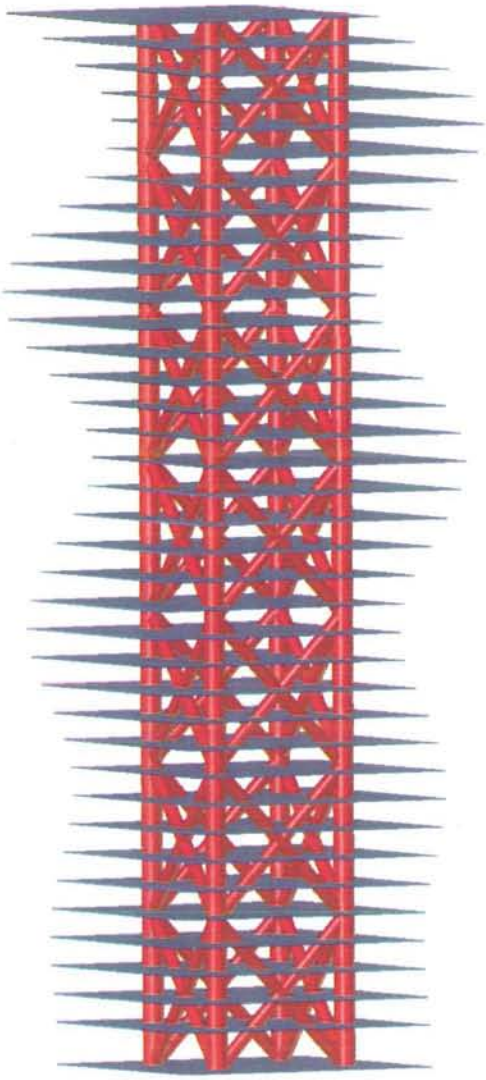


figure 3.9 3D model megastructure

Besides the braced frame system, the megastructure is well worth investigating more closely. By applying four massive vertical members at the corners as in figure 3.8, most of the primary construction is concentrated in the corners of the square. Large diagonal members can connect the four vertical columns so the total system will be able to resist the lateral loading from wind and floors and provide sufficient stiffness to the building. Without knowing the exact measurements of a megastructure as the primary structural system for the curved building, the drawing on the right gives an impression of the open effect of the system. Like with the braced frame, at some points the diagonal members will block an office entrance or a hallway, but the open character of the structure is obvious. The floor plan as drawn above can shift from left to right and will not be restricted as it would be in case of a closed concrete core.

The straight sides of the curved building

With applying a braced frame or a megastructure to the curved building, the major problem of non-repetitive floor plans is reduced to a relatively small problem of occasionally blocked doors and hallways. To reduce this problem even more, the straight and vertical sides of the building can be used as well. These two sides of the building are not curved and only need holes for windows instead of doors and hallways. By shifting the construction to the sides diagonals from either a braced frame or a megastructure will no longer interfere with the functional behaviour of the building.

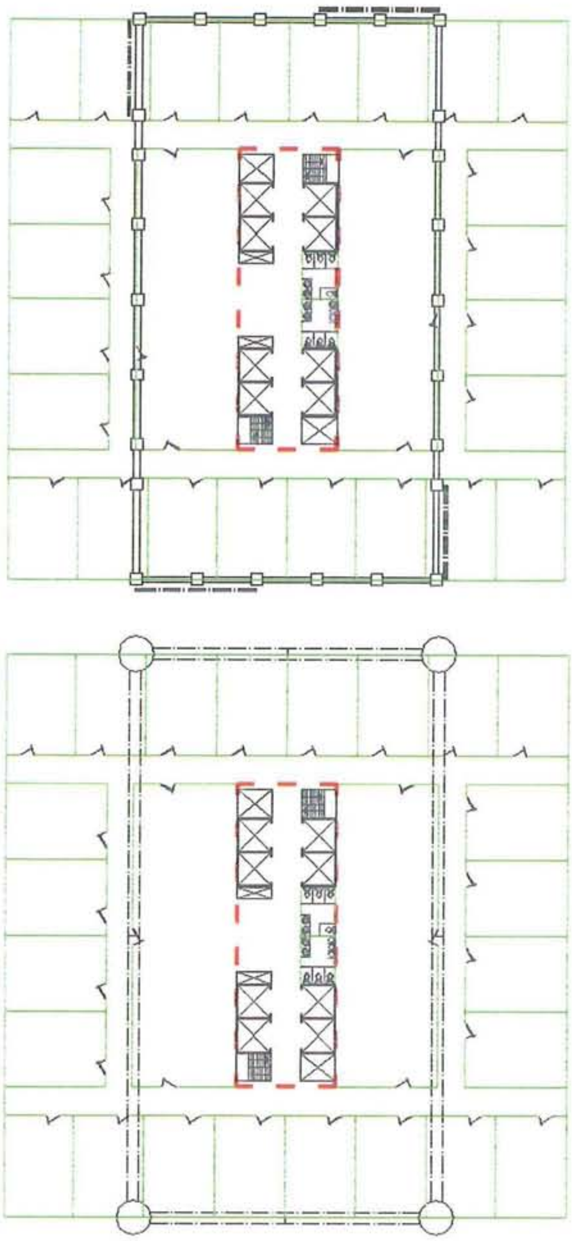


figure 3.10 Primary structure shifted to facade line

In the drawings on the next page, a 3D model of the braced frame construction and the megastructure is presented. The difference with the closed core is clearly visible. The offices located on the left and right side of the building can shift inside the open core and do not have to alter in shape. The number of hallways that will be blocked is reduced to a minimum and the vertical zone is still in place.

Now that the structure is shifted to the outside perimeter of the building, but still within the curved façade, the megastructure or the braced frame do not need to be open and transparent at the two straight sides of the building. Part of the structure that is located on the straight and vertical sides of the building can be a wall or perforated wall as well. The mass of the construction does not necessarily have to be concentrated at the corners of the structure. By spreading this mass over the non-curved façade creating a diaphragm the stiffness of the building should be ensured. An example of this kind of system is drawn in figure 3.12.

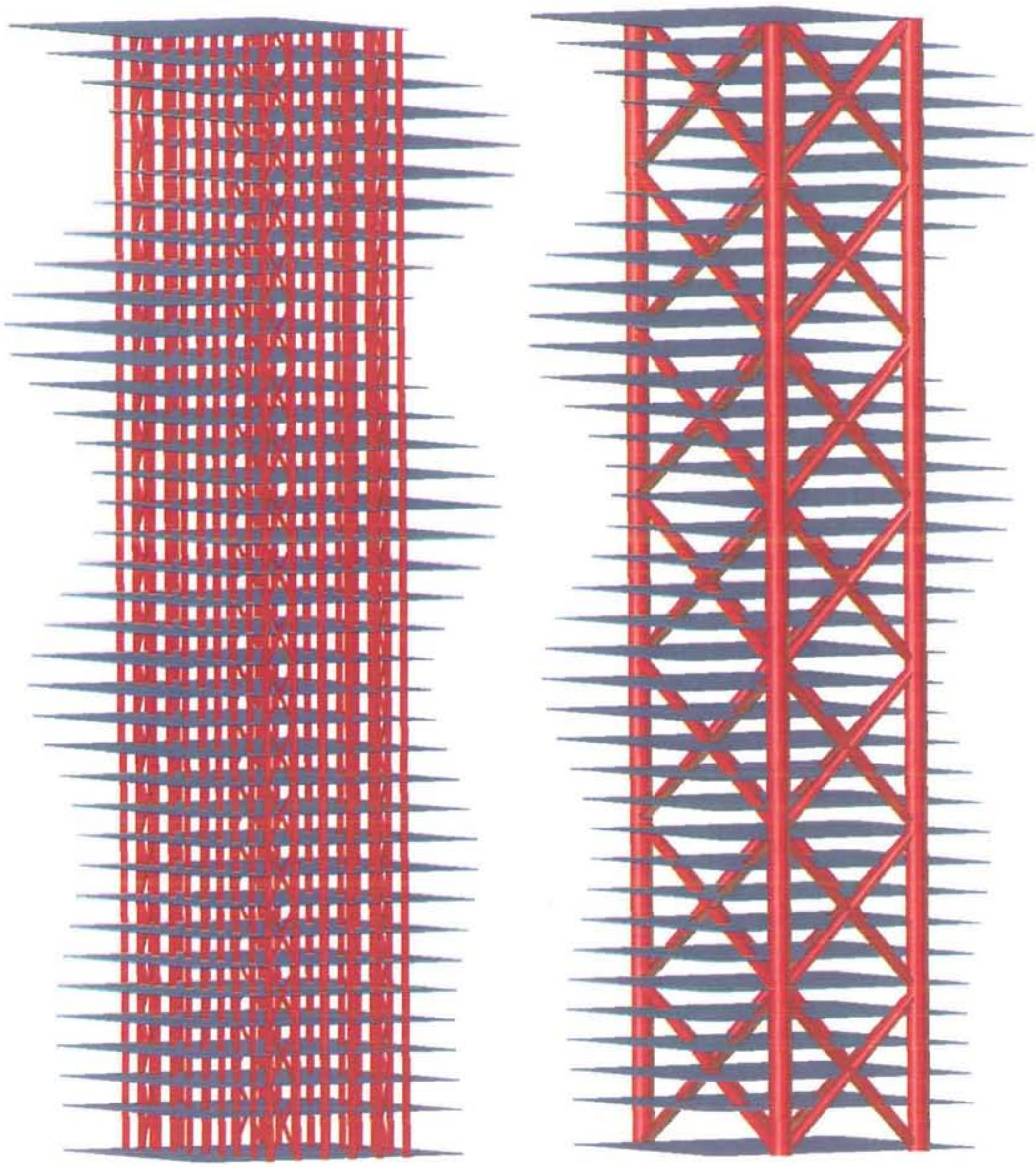


figure 3.11 3D models

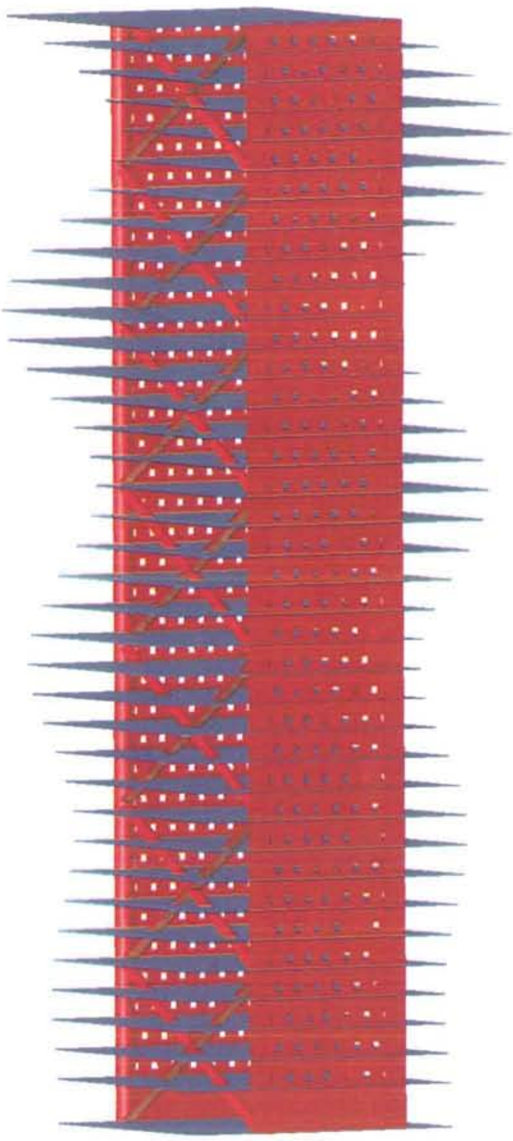


figure 3.12 3D model closed wall

The megastructure is the best solution from a functional point of view. The structural members will not (or hardly) interfere with doors and hallways of the proposed floor layout. On the few levels that a structural member does interfere with the functional layout of the floors, a practical solution will have to be realised for that particular floor. In practice this will mean a door has to be shifted in order to make an entrance to the office possible. When part of the megastructure is shifted to the vertical façade line the interference with the functional layout is minimized. Whether the structural system in the straight façade should be a megaframe or a perforated wall is up to the architect's opinion. The megastructure will result in a more impressive building, more in line with the extreme shape of the structure. Whereas

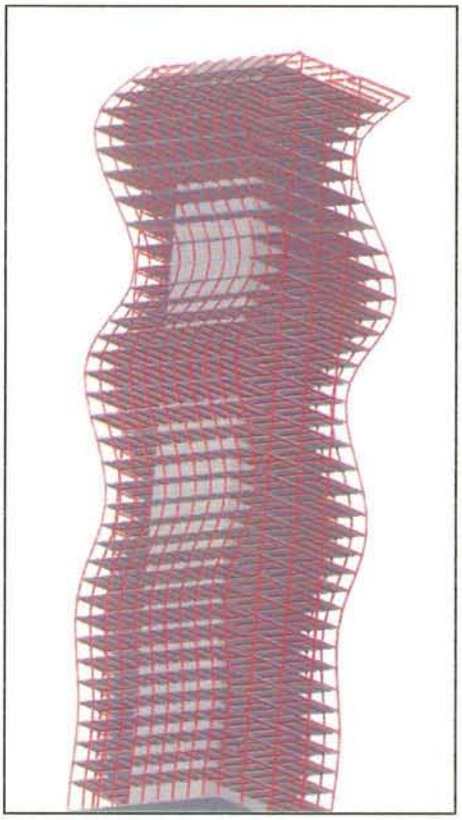
the perforated wall will make the building more solid and massive. The accent of the building may be shifted from the unique shape to the straight and massive perforated wall.
Although the structural performance of the alternatives mentioned above is not checked in detail, the global properties of the systems appear to be suitable for the curved building at first sight. The proposed alternatives mentioned above can form a starting point for further structural and functional investigation of a curved high rise building in order to achieve a better integration of structural and functional behaviour.

4 Conclusions

The initial primary objective of this final part of the thesis, as stated in the workplan, was to produce:

"The structural design of an alternative S-shaped skyscraper"

With the previous reports[16] the search for an alternative design began with the search for a true S-shape. The resulting possible structures were S-shaped, but during the investigation it became clear that perhaps a more extreme shape could be realized. A more extreme shape with even better structural behaviour.



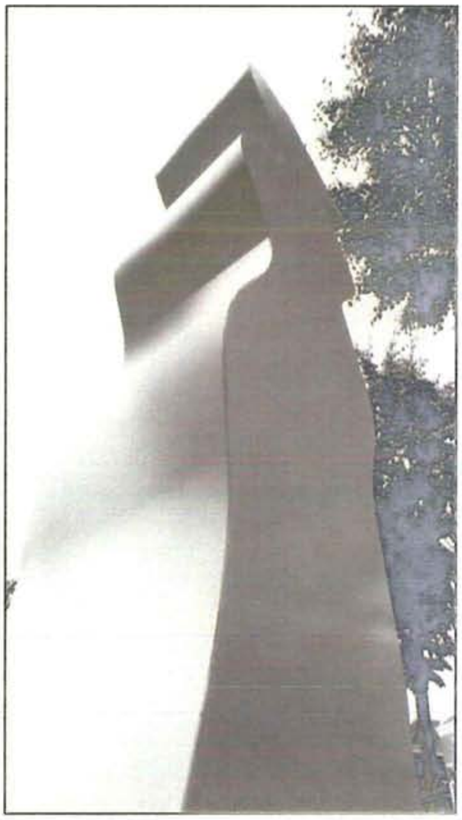


figure 4.1 Final design structure (l) and shape (r)

The best combination of structural behaviour and extreme building shape proves to be a reverse damped curved building. An increasing number of curves tends to improve the structural behaviour of the building. For the case study 3.22 curves are applied to the building to secure the ability of the building to be able to meet its functions as an office building and not to create a ridicule look.

During the whole process of literature and structural investigation the primary objective has been defined more precisely by adding some extra secondary objectives as listed in the

[16] Literature investigation, Structural investigation part I, Structural investigation part II

previous report, structural investigation part II. Those secondary objectives were the realization of a building with:

1. Extreme, uncommon building shape
2. No extra bending moment at $x = 0$
3. Smallest possible initial top deflection
4. No tension in the core

Now, after finishing the structural design of the building, the question is whether these objectives were met.
The first objective, the realization of an extreme and uncommon building shape, cannot be denied. The shape of the building is truly uncommon considering the wider spreading curve at a higher level and the large number of curves. Due to the chosen shape, the building seems to be pulled to the earth like a spring and just kept erected by support of the air.
As mentioned before, the building configuration was chosen in a way that not only the shape of the building would meet the objective, but also in a way that the building behaviour would be excellent.
The foundation of the building has to resist the bending moment caused by the wind. Due to the shape of the building, an extra bending moment could be generated leading to a heavier and more expensive construction. Therefore one should strive after a minimal extra bending moment at ground level, see objective 2. The resulting extra bending moment at ground level caused by the shape of the building is only 1.6% (14012 kNm) of the moment caused by the wind load. Although the figure is not zero, the additional bending moment is too small to cause a true difference in the foundation design.
Consequences of the curved shape of the building are the lateral load components coming from the inclined columns causing bending moments in the central core. The resulting initial deformation of the core should not become too large in order to leave some space for deformation by the wind and rotation of the foundation. Furthermore a large initial deflection would result in a larger the 2^{nd} order effect, decreasing the overall stability of the building. The initial top deflection of the case study design is 1.24 mm, a figure that is too small (0.4% of the allowed total deflection) to have a significant effect on the total deflection of the building.

Because the central core has to deal with compressive stresses only, the concrete can be considered to be non-cracked and the initial E-modulus can be maintained. The bending moment caused by the extra lateral load components does jeopardise the total compression of the core, but the normal force in the core itself is large enough to ensure compression throughout the whole core.

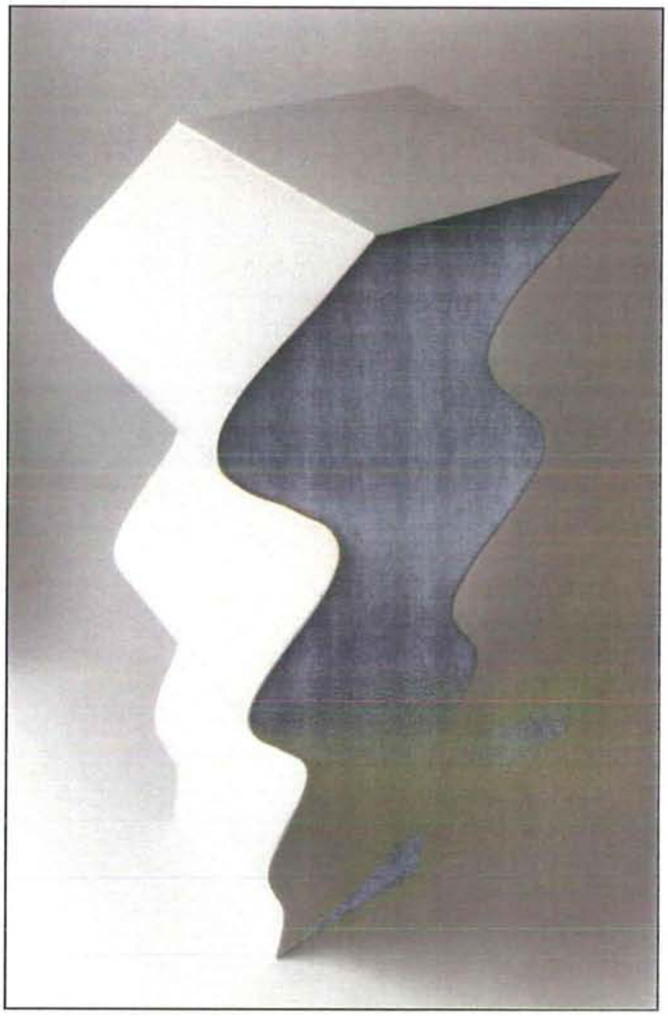
Results of applied structural system:

1. ***Reverse damped, multiple curves, extreme shape***
2. $M_{x=0} = 14012$ ***kNm (1.6% of wind moment)***
3. $w_{topi} = 1.24$ ***mm (0.4% of maximum deflection)***
4. $\sigma_{\max} = -0.8$ ***N/mm²***

The combination of a straight concrete core and inclined columns in the façade of the building proves to have excellent structural behaviour. On the other hand, the functional behaviour of the building is very poor. The usable floor area is reduced by the inclined façade where people are unable to stand or walk. However, this loss of floor space cannot be avoided other then by straightening the façade. The loss of floor space by the inclined façade must be accepted and is a result of the chosen shape of the building. Besides the inclined façade, the usable floor space is limited by the closed central core. Due to the closed core it is impossible to create a repetitive floor layout. The solution to this problem should be found in a more open core construction. A braced frame or even more a megastructure is likely to provide enough stiffness to the building and more functional flexibility. By applying one of these systems, the repetitive floor layout is no longer limited by a closed core, but just by a vertical transport zone, where elevators, staircases and ducts are located. The interference of the structural system with the functional layout can be reduced even more if the construction is stretched to the non-curved sides of the building. With this configuration the interference is minimized whereas the repetitiveness of the functional floor layout is maximized.

The final conclusion is that an S-shaped building, or even more extreme: a structure with a reverse damped shape and multiple curves, is very well possible from a structural point of view.
Of course, the resulting shape has some disadvantages. The major disadvantage of the curved building that there is hardly any repetition of structural members. Most of the structural members have unique measurements and therefore will be more expensive. As for the functional behaviour of the building, some more research needs to be done on repetitive floor plans and maximum usable floor space. The alternatives presented in this report can be used as a starting point.
But, looking from a structural point of view, the advantages of a central core and inclined columns as the structural system for the curved building are obvious. The multiple curves have a stabilizing and balancing effect on the structure, there is hardly any extra moment on the foundation, the initial top deflection is kept very small and there will be no tensile stresses in the core.

The final result is a truly curved building, more extreme than one had expected at the start of the thesis.



Appendix A: Literature

1 Structural analysis of tall buildings, *Bungale S. Tatanath,* New York 1988

2 Design of modern highrise reinforced concrete structures, *H. Aoyama,* London 2001

3 Wind-induced pressure fluctuations on building facades, *Chris Geurts,* Eindhoven 1997

4 Blast effects on buildings, *G.C. Mays & P.D. Smith,* London 1995

5 Structural dynamics for structural engineers, *Gary C. Hart & Kevin Wong,* New York 2000

6 Draagconstructies III B Verdiepingbouw, *Prof. ir. A.J. Hogeslag e.a,* Delft 1992

7 Form follows finance, *Carol Willis,* New York 1995

8 Twist & build, *Karel Vollers,* Delft 2001

9 Late-Twentieth-Century skyscrapers, *Piera Scuri,* New York 1990

10 Skyscrapers, *David Bennett,* New York 1995

11 Belastingen en vervormingen TGB 1990, *Nederlands Normalistatie Instituut,* Delft 2001

12 Staalconstructies TGB1990, *Nederlands Normalistatie Instituut,* Delft 1997

13 Voorschriften beton TGB 1990, *Nederlands Normalistatie Instituut,*Delft 1995

14 Highrise building structures, *Wolfgang Schueller,* London 1995

15 The vertical building, *Wolfgang Schueller,* London 1990

16 Dynamisch gedrag van hoge gebouwen, *CUR rapport 97-5,* Gouda 1997

17 Mechanical and electrical systems in buildings, *Tao and Janis,* Prentice-Hall 2001

18 The vertical transportation handbook, *Strakosch,* New York 1998

19 Mechanica van constructies: elastostatica van slanke structuren, *A.L. Bouma,* Delft 1989

20 Jellema Liften en roltrappen, bouwtechniek, Leiden 1996

21 Jellema uitvoeren, bouwproces, Leiden 1996

22 Stabiliteit voor ontwerpers, *D.Dicke,* Delft 1991

23 Cement en beton; praktische betonberekeningen, *Vis en Sagel*, Amsterdam 1978

24 Foundation design & construction, *M.J. Tomlinson*, Harlow 1986

25 Funderingstechnieken, *A.F. van Tol*, Delft 1993

26 Funderingstechniek en ondergronds bouwen, *A.F. van Tol*, Delft 1999

27 NEN 5080, *Nederlands Normalistatie Instituut,* Delft 1998

28 NEN 5081, *Nederlands Normalistatie Instituut,* Delft 1998

Appendix B: Internet

1 www.greatbuildings.com

2 www.hoogbouw.nl

3 www.hoogbouw.pagina.nl

4 www.skyscrapers.com

5 www.skyscraperphotos.com

6 www.twistscraper.com

7 www.arup.com

8 www.som.com

9 www.kpf.com

10 www.elevator-world.com

11 www.otis.com

12 www.schindler.com

Appendix C: Address file

Committee:

Chairman
name: Prof. dipl-ing. J.N.J.A. Vamberský
address: TU Delft, faculteit Civiele Techniek en Geowetenschappen
Stevinweg 1, kamer 4.09
2628 CN Delft
telephone: +31 (0)15 2783174
e-mail: j.n.j.a.vambersky@citg.tudelft.nl

Member
name: Prof. ir. L.A.G. Wagemans
address: TU Delft, faculteit Civiele Techniek en Geowetenschappen
Stevinweg 1, kamer 3.13
2628 CN Delft
telephone: +31 (0)15 2781573
e-mail: l.a.g.wagemans@citg.tudelft.nl

Member
name: Dr. ir. J. Paul MBA
address: Arup b.v.
Y-tech gebouw
Van Diëmenstraat 194
1013 CP Amsterdam
telephone: +31 (0)20 3058500
e-mail: joop.paul@arup.com

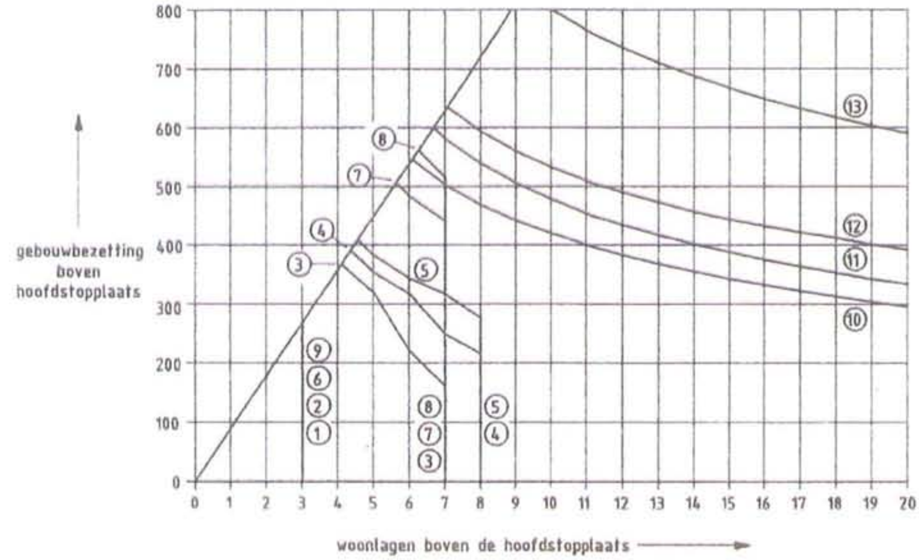
Member
name: Ir. W.J. Raven
address: TU Delft, faculteit Civiele Techniek en Geowetenschappen
Stevinweg 1, kamer 4.21
2628 CN Delft
telephone: +31 (0)15 2782764
e-mail: w.j.raven@citg.tudelft.nl

Student:

name: J.H.G. Thuis
address: Westerstraat 6
2613 RH Delft
telephone: +31 (0)15 2840070
mobile phone: +31 (0)6 2451 9836
e-mail: j.h.g.thuis@student.tudelft.nl
jthuis@hotmail.com

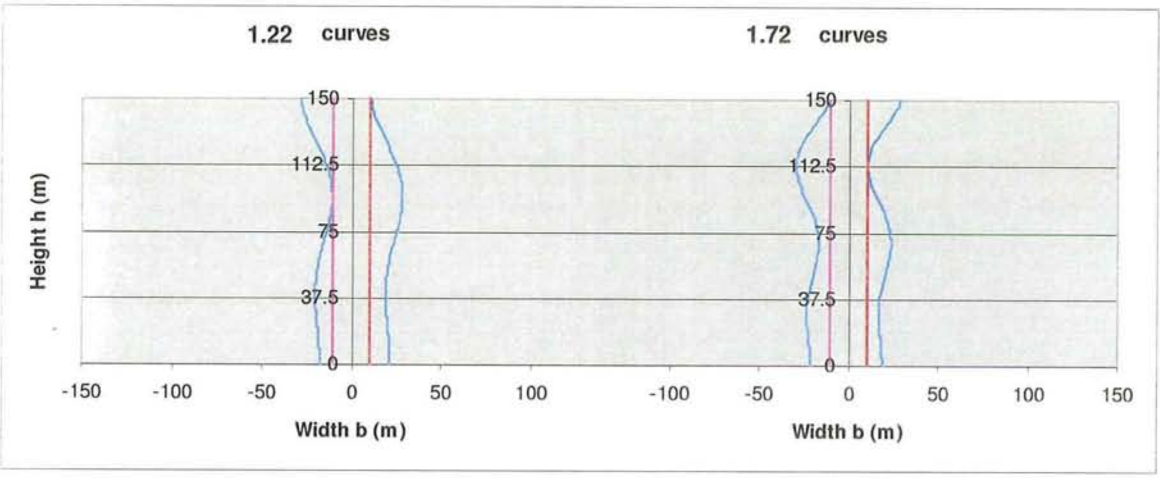
Appendix D: Timetable

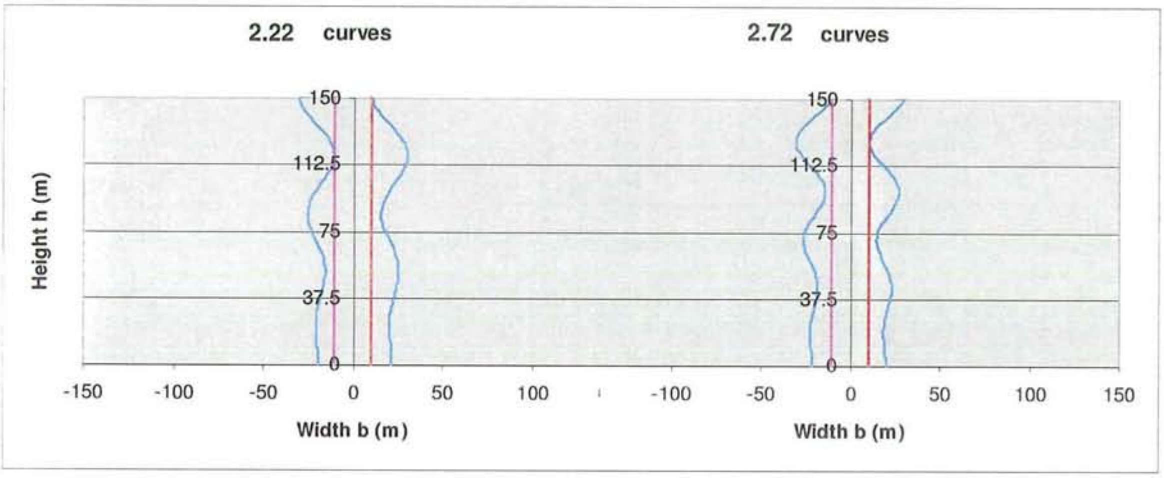
Appendix E: Elevator planning

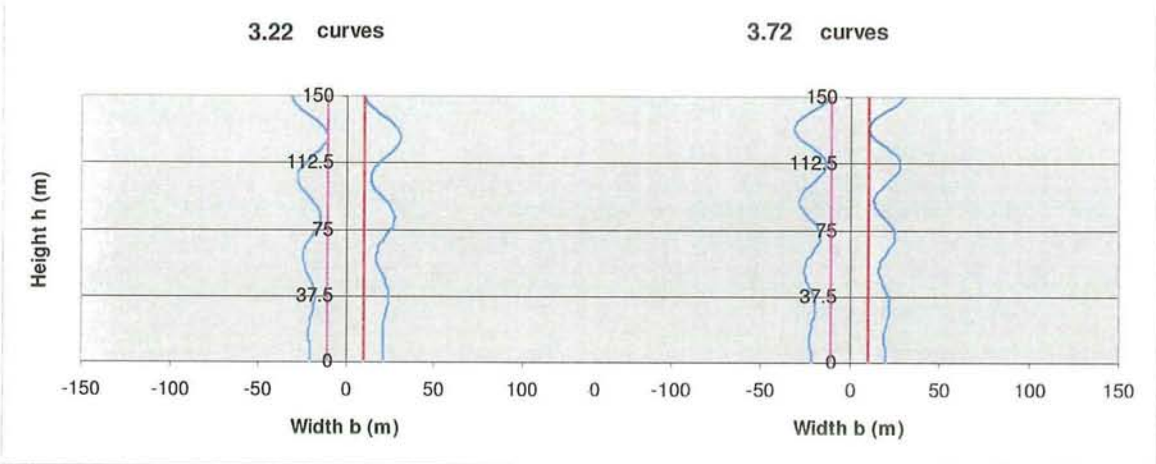


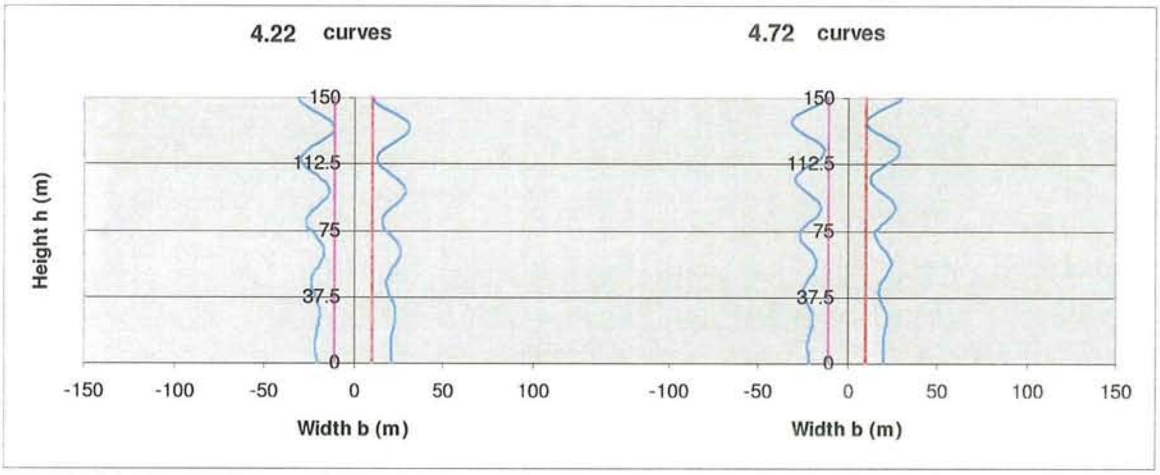
Lijn nr.	Aantal liften	Nominale last kg	Snelheid m/s	L_p	Wachttijd s	P_5	B_m			Klasse 1		
							kl.4	kl.3	kl.2	wachttijd s	P_5	B_m
10	2	1000	1,00	1	32,0	9,4	125	94	75	34,2	8,8	44
		+		2	44,0	13,6	182	136	109	47,2	12,7	64
		630		3	52,7	17,1	228	171	137	57,3	15,7	78
				4	60,4	19,9	265	199	159	66,3	18,1	n.v.t.
				5	67,0	22,4	298	224	179			
				6	72,9	24,7	329	247	197			
				7	78,3	26,8	358	268	215			
				8	83,1	28,9	385	289	n.v.t.			
11	2	1000	1,00	1	32,0	9,4	125	94	75	34,2	8,8	44
				2	44,0	13,6	182	136	109	47,2	12,7	64
				3	52,7	17,1	228	171	137	57,3	15,7	78
				4	60,4	19,9	265	199	159	66,3	18,1	n.v.t.
				5	67,0	22,4	298	224	179			
				6	72,9	24,7	329	247	197			
				7	78,3	26,8	358	268	215			
				8	83,1	28,9	385	289	n.v.t.			
				9	87,6	30,8	411	308	n.v.t.			
				10	91,6	32,7	436	327	n.v.t.			
12	2	1000	1,60	1	25,7	11,7	156	117	93	27,9	10,8	54
				2	35,8	16,8	223	168	134	39,0	15,4	77
				3	43,7	20,6	275	206	165	48,3	18,6	93
				4	50,8	23,6	315	236	189	56,8	21,1	106
				5	57,2	26,2	350	262	210	64,5	23,3	n.v.t.
				6	62,9	28,6	381	286	229			
				7	68,2	30,8	411	308	246			
				8	72,9	32,9	439	329	263			
				9	77,3	34,9	466	349	279			
				10	81,4	36,9	492	369	n.v.t.			
13	3	1000	1,60	1	17,1	17,5	233	175	140	18,6	16,1	81
				2	23,9	25,1	335	251	201	26,0	23,1	115
				3	29,1	30,9	412	309	247	32,2	28,0	140
				4	33,9	35,4	472	354	283	37,9	31,7	158
				5	38,1	39,3	524	393	315	43,0	34,9	175
				6	42,0	42,9	572	429	343	47,6	37,8	189
				7	45,4	46,2	616	462	370	51,9	40,5	202
				8	48,6	49,4	658	494	395	55,8	43,0	215
				9	51,5	52,4	698	524	419	59,4	45,5	227
				10	54,2	55,3	737	553	442	62,8	47,8	n.v.t.

Appendix F: Optimal building shapes








4.22 curves
4.72 curves
Height h (m)
150
112.5
75
37.5
0
150
112.5
75
37.5
0
-150
-100
-50
0
50
100
-100
-50
0
50
100
150
Width b (m)
Width b (m)

Appendix G: Force flow

1 Force flow in floors and inclined columns

height x	Lateral force floor left	Lateral force floor right	Inclined N force column left	Inclined N force column right
m	kN	kN	kN	kN
0	3459	3258	38086	35830
6	-14	-3	37225	35059
10	-2249	-2118	36172	34174
14	-3697	-3532	34929	33191
18	-3687	-3578	33873	32443
22	-2119	-2084	33241	32091
26	523	517	32669	31691
30	3373	3306	31603	30614
34	5456	5220	30101	28891
38	5960	5503	29030	27417
42	4505	4004	29126	26985
46	1362	1173	29786	27157
50	-2508	-2181	29486	26726
54	-5753	-5150	27423	25101
58	-7209	-6884	24494	23040
62	-6428	-6726	22740	22294
66	-3812	-4453	22958	23585
70	-271	-572	23550	25177
74	3244	3643	22763	24826
78	5978	6629	20324	21949
82	7297	7324	17636	18260
86	6664	5725	17149	16592
90	3931	2748	19261	17253
94	-226	-475	21151	17780
98	-4264	-3229	20340	16677
102	-6484	-5229	16667	14131
106	-6087	-6182	12429	11639
110	-3699	-5562	10665	11669
114	-845	-3060	10771	14202
118	1223	706	10198	16057
122	2287	4078	8504	15017
126	2838	5313	6414	11264
130	3134	3918	4844	7237
134	2737	1132	5024	5466
138	1138	-1041	6525	4492
142	-1204	-1470	7159	2659
146	-2700	-669	5678	834
150	-1854	0	2687	0

2 *Angle and length of inclined columns*

height	angle	inclined column length
m	º	m
0	0.0	6.0
6	-5.3	4.0
10	-5.5	4.0
14	-2.0	4.0
18	4.2	4.0
22	10.7	4.1
26	14.7	4.1
30	14.3	4.1
34	8.4	4.0
38	-2.1	4.0
42	-13.9	4.1
46	-22.7	4.3
50	-25.9	4.4
54	-22.2	4.3
58	-10.8	4.1
62	6.6	4.0
66	23.2	4.4
70	33.0	4.8
74	35.2	4.9
78	29.0	4.6
82	12.7	4.1
86	-11.5	4.1
90	-31.5	4.7
94	-41.5	5.3
98	-42.6	5.4
102	-34.8	4.9
106	-14.1	4.1
110	16.7	4.2
114	38.8	5.1
118	48.2	6.0
122	48.6	6.0
126	39.6	5.2
130	15.0	4.1
134	-22.0	4.3
138	-45.1	5.7
142	-53.5	6.7
146	-53.3	6.7
150		

3 *Beams I left side of building*

height x	floor level	floor length	M_d	W_{min}	Minimal profile	Applied profile	u_{tot}	u_{max}	u_{camber}
m	-	m	kNm	10^3 mm^3	HE	HE	mm	mm	mm
0	1	9.67	316	1344	320A	320A	43.72	39	5
6	2	9.98	337	1433	340A	340A	41.30	40	1
10	3	10.38	364	1550	340A	340A	48.31	42	7
14	4	10.66	384	1634	360A	360A	45.15	43	2
18	5	10.59	379	1613	360A	360A	43.98	42	2
22	6	10.06	342	1455	340A	340A	42.59	40	2
26	7	9.13	282	1199	300A	300A	43.49	37	7
30	8	8.07	220	936	280A	300A	35.13	32	3
34	9	7.23	177	752	260A	300A	29.53	29	1
38	10	6.99	165	702	260A	300A	25.75	28	0
42	11	7.56	193	821	280A	300A	27.05	30	0
46	12	8.92	269	1143	300A	300A	39.49	36	4
50	13	10.77	392	1667	360A	360A	46.96	43	4
54	14	12.60	537	2284	400A	400A	65.19	50	15
58	15	13.85	648	2756	450A	450A	67.65	55	12
62	16	14.02	664	2824	450A	450A	71.05	56	15
66	17	12.92	564	2398	450A	450A	51.24	52	0
70	18	10.72	388	1651	360A	360A	46.05	43	3
74	19	7.94	213	906	280A	300A	32.96	32	1
78	20	5.35	97	412	220A	300A	16.96	21	0
82	21	3.74	47	201	160A	300A	12.94	15	0
86	22	3.68	46	195	160A	300A	12.11	15	0
90	23	5.34	96	410	220A	300A	16.79	21	0
94	24	8.40	238	1014	300A	300A	31.08	34	0
98	25	12.09	494	2102	400A	400A	55.24	48	7
102	26	15.41	802	3414	450B	450B	84.17	62	23
106	27	17.36	1018	4333	450M	450M	86.32	69	17
110	28	17.28	1008	4291	450M	450M	84.66	69	16
114	29	15.03	763	3246	450B	450B	76.11	60	16
118	30	11.10	416	1770	360A	360A	52.94	44	9
122	31	6.48	142	604	240A	300A	25.60	26	0
126	32	2.46	20	87	120A	300A	6.65	10	0
130	33	0.20	0	1	100A	300A	0.00	1	0
134	34	0.47	1	3	100A	300A	0.02	2	0
138	35	3.34	38	160	160A	300A	8.21	13	0
142	36	8.16	225	956	280A	300A	36.70	33	4
146	37	13.68	632	2689	450A	450A	64.44	55	10
150	38	18.39	1142	4860	450M	450M	108.60	74	35

4 *Beams I right side of building*

height x	floor level	floor length	M_d	W_{min}	Minimal profile	Applied profile	u_{tot}	u_{max}	u_{camber}
m	-	m	kNm	10^3 mm^3	HE	HE	mm	mm	mm
0	1	9	256.7	1092	300A	300A	36	35	1
6	2	8	238.5	1015	300A	300A	31	34	0
10	3	8	216.4	921	280A	300A	34	32	2
14	4	8	201.5	858	280A	300A	30	31	0
18	5	8	205.2	873	280A	300A	25	31	0
22	6	8	234.2	997	300A	300A	30	33	0
26	7	9	289.2	1231	320A	320A	37	37	0
30	8	10	359.7	1531	340A	340A	47	41	6
34	9	11	420.3	1789	360A	360A	54	45	9
38	10	11	438.9	1868	400A	400A	44	46	0
42	11	11	396.1	1686	360A	360A	48	43	5
46	12	9	303.0	1289	320A	320A	40	38	2
50	13	8	196.1	834	280A	300A	28	30	0
54	14	6	112.9	481	220A	300A	23	23	0
58	15	5	69.7	296	200A	300A	13	18	0
62	16	4	64.5	275	200A	300A	11	17	0
66	17	5	101.1	430	220A	300A	18	22	0
70	18	8	198.8	846	280A	300A	29	31	0
74	19	10	368.6	1569	340A	340A	50	42	8
78	20	13	573.8	2442	450A	450A	53	52	1
82	21	15	724.6	3083	450B	450B	69	59	10
86	22	15	730.7	3109	450B	450B	70	59	11
90	23	13	575.0	2447	450A	450A	53	52	1
94	24	10	337.0	1434	340A	340A	41	40	1
98	25	6	133.8	569	240A	300A	23	25	0
102	26	3	29.9	127	140A	300A	8	12	0
106	27	1	3.6	15	100A	300A	0	4	0
110	28	1	4.2	18	100A	300A	0	4	0
114	29	3	38.1	162	160A	300A	8	13	0
118	30	7	179.6	764	260A	300A	31	29	1
122	31	12	478.5	2036	400A	400A	52	48	4
126	32	16	856.8	3646	450M	450M	61	64	0
130	33	18	1116.7	4752	450M	450M	104	73	31
134	34	18	1084.3	4614	450M	450M	98	72	26
138	35	15	764.9	3255	450B	450B	77	60	16
142	36	10	353.5	1504	340A	340A	46	41	5
146	37	5	74.9	319	200A	300A	15	19	0
150	38	0	0.0	0			0	0	0

5 Beams II

Span:		9.25	m
Floor span:		5.375	m
Floor gravity load:	$p_{g,floor,rep} =$	1.88	kN/m^2
	$q_{g,floor,rep} =$	1.88 x 5.375 = 10.105	kN/m
Variable load	$p_{q,rep} =$	2.5	kN/m^2
	$q_{q,rep} =$	2.5 x 5.375 = 13.44	kN/m
Beam:	$q_{g,HE340A,rep} =$	1.05	kN/m
	$W_{y;d} =$	$1680\ 10^3$	mm^3
	$f_{y;d} =$	235	N/mm^2

$q_d = \gamma_g \cdot q_g + \gamma_q \cdot q_q$ = 1.2 x (10.105 + 1.05) + 1.5 x 13.44 = 33.55 kN/m

Strength Requirement (u.g.t): $\dfrac{M_{y;s;d}}{M_{y;u;d}} \leq 1$

$M_{y;s;d} = \dfrac{1}{8} \cdot q_d \cdot l_{beam}^2 = 0.125 \text{ x } 33.55 \text{ x } 9.25^2 = 359 \text{ kNm}$

$M_{y;u;d} = f_{y;d} \cdot W_{y;d} = 235 \text{ x } 1680\ 10^3 = 395 \text{ kNm}$

$\dfrac{M_{y;s;d}}{M_{y;u;d}} = \dfrac{359}{395} = 0.91 \leq 1$, the HE340A profile is sufficient to bare@@ the load (A HE320A profile proves to be insufficient with a ratio of 1.02).

6 Beams III

Span:		10.75	m
Load from beam II:	$q_{d,beamII} =$	33.55	kN/m
Beam:	$q_{g,HE360A,rep} =$	1.12	kN/m
	$W_{y;d} =$	$1890\ 10^3$	mm^3
	$f_{y;d} =$	235	N/mm^2

Strength

$$\boxed{\text{Requirement (u.g.t): } \frac{M_{y;s;d}}{M_{y;u;d}} \leq 1}$$

$$M_{y;s;d} = \frac{1}{4} \cdot F_d \cdot l + \frac{1}{8} \cdot q_{d;HE360A} \cdot l_{beam}^2 = 436 \text{ kNm}$$

$$M_{y;u;d} = f_{y;d} \cdot W_{y;d} = 235 \text{ x } 1890\ 10^3 = 444 \text{ kNm}$$

$\frac{M_{y;s;d}}{M_{y;u;d}} = \frac{436}{444} = 0.98 \leq 1$, the HE360A profile is sufficient to bare@@ the load

7 *Inclined façade column, middle area*

Strength

$N_{c;s;d} = 25177$ kN

This normal load is spread over 10 columns, per columns a load of 2517.7 kN.

$$A_{\min} = \frac{N_{c;s;d}}{f_{y;d}} = \frac{2517.7}{235} = 10714\,\text{mm}^2$$

A HE300A profile matches best with the minimal surface (11250 mm^2), but has to be checked on the buckling requirement.

Buckling

$$\boxed{\text{Requirement: } \frac{N_{c;s;d}}{\omega_{buc} \cdot N_{c;u;d}} \leq 1}$$

Maximum buckle length l_{buc}:	5.44	m
I:	18263 10^4	mm^4
Section surface HE300A A:	11250	mm^2
Curve (table 23 NEN6770):	C	-
$f_{y;d}$:	235	N/mm^2

$N_{c;s;d} = 2517.7$ kN (10 columns)

$N_{c;u;d} = A \cdot f_{y;d} = 11250 \cdot 235 = 2644$ kN

$$F_E = \frac{\pi^2 EI}{l_{buc}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 18263 \cdot 10^4}{5440^2} = 12791 \text{ kN}$$

$$\lambda_{rel} = \sqrt{\frac{N_{c;u;d}}{F_E}} = \sqrt{\frac{2644}{12791}} = 0.45 \quad \rightarrow \quad \omega_{buc} = 0.85$$

$$\frac{N_{c;s;d}}{\omega_{buc} \cdot N_{c;u;d}} = \frac{2517.7}{0.85 \cdot 2644} = 1.12 \geq 1 \rightarrow \text{ A HE300A will not meet the buckling requirement.}$$

If one tests a HE340A profile, the results are:

$N_{c;u;d} = 3137$ kN

$F_E = 19395\,\text{kN}$

$\lambda_{rel} = 0.40 \rightarrow \qquad \omega_{buc} = 0.87$

$$\frac{N_{c;s;d}}{\omega_{buc} \cdot N_{c;u;d}} = \frac{2517.7}{0.87 \cdot 3137} = 0.92 \leq 1$$

The HE340A is a proper profile for the middle part of the structure.

8 *Inclined façade column, upper area*

Strength

$N_{c;s;d} = 16057 \text{ kN}$

This normal load is spread over 10 columns, per columns a load of 1605.7 kN.

$$A_{\min} = \frac{N_{c;s;d}}{f_{y;d}} = \frac{1605.7}{235} = 6833\,\text{mm}^2$$

A HE240A profile matches best with the minimal surface (7680 mm^2), but has to be checked on the buckling requirement.

Buckling

$$\boxed{\text{Requirement: } \frac{N_{c;s;d}}{\omega_{buc} \cdot N_{c;u;d}} \leq 1}$$

Maximum buckle length l_{buc}:	6.7	m
I:	7763 10^4	mm^4
Section surface HE300A A:	7680	mm^2
Curve (table 23 NEN6770):	C	-
$f_{y;d}$:	235	N/mm^2

$N_{c;s;d} = 1605.7 \text{ kN (10 columns)}$

$N_{c;u;d} = A \cdot f_{y;d} = 7680 \cdot 235 = 1805 \text{ kN}$

$$F_E = \frac{\pi^2 EI}{l_{buc}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 7763 \cdot 10^4}{6700^2} = 3584 \text{ kN}$$

$$\lambda_{rel} = \sqrt{\frac{N_{c;u;d}}{F_E}} = \sqrt{\frac{1805}{3584}} = 0.71 \qquad \rightarrow \qquad \omega_{buc} = 0.72$$

$$\frac{N_{c;s;d}}{\omega_{buc} \cdot N_{c;u;d}} = \frac{1605.7}{0.72 \cdot 1805} = 1.24 \geq 1$$ A HE240A will not meet the buckling requirement.

If one tests a HE280A profile, the results are:

$$N_{c;u;d} = 2287 \text{ kN}$$

$$F_E = 6313 \text{ kN}$$

$$\lambda_{rel} = 0.60 \rightarrow \quad \omega_{buc} = 0.78$$

$$\frac{N_{c;s;d}}{\omega_{buc} \cdot N_{c;u;d}} = \frac{1605.7}{0.78 \cdot 2287} = 0.90 \leq 1$$

The HE280A is a proper profile for the upper part of the structure.

9 *Straight façade columns, sides of building, lower area (0-50m)*

A_{floor}:	57.8	m^2
p_g:	1.88	kN/m^2
q_g:	2.5	kN/m^2
$q_{g;HE340A}$	1.05	kN/m
$q_{g;HE360A}$	1.12	kN/m

$$N_{c;s;d} = 1.2 \cdot (1.88 \cdot 57.8 + 1.05 \cdot 10.75 + 1.12 \cdot 10.75) + 1.5 \cdot 2.5 = 6160 \text{ kN}$$

$$A_{\min} = \frac{N_{c;s;d}}{f_{y;d}} = \frac{6160}{235} = 26213 \,\text{mm}^2$$

A HE650B or a HD 400 x 818 profile should be sufficient to bare the load, but have to be checked on the buckling requirement. The HD profile will be applies because of its more convenient measurements (514 x 437 mm).

Buckling

Requirement: $\dfrac{N_{c;s;d}}{\omega_{buc} \cdot N_{c;u;d}} \leq 1$

Maximum buckle length l_{buc}:	6	m
I:	392200 10^4	mm^4
Section surface HD 400x818 A:	28330	mm^2
Curve (table 23 NEN6770):	C	-
$f_{y;d}$:	235	N/mm^2

$$N_{c;s;d} = 6160 \text{ kN}$$
$$N_{c;u;d} = A \cdot f_{y;d} = 28330 \cdot 235 = 6658 \text{ kN}$$

$$F_E = \frac{\pi^2 EI}{l_{buc}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 392200 \cdot 10^4}{6000^2} = 225800 \text{ kN}$$

$$\lambda_{rel} = \sqrt{\frac{N_{c;u;d}}{F_E}} = \sqrt{\frac{6658}{225800}} = 0.17 \quad \rightarrow \quad \omega_{buc} = 1.00$$

$\dfrac{N_{c;s;d}}{\omega_{buc} \cdot N_{c;u;d}} = \dfrac{6160}{1.00 \cdot 6658} = 0.92 \geq 1$ A HD 400x818 will meet the buckling requirement.

10 Straight façade columns, sides of building, middle area (50-100m)

$N_{c;s;d} = 4215 \text{ kN}$

$$A_{\min} = \frac{N_{c;s;d}}{f_{y;d}} = \frac{4215}{235} = 17936 \, \text{mm}^2$$

A HE500A profile should be ok (19750 mm^2)

$N_{c;u;d} = A \cdot f_{y;d} = 19750 \cdot 235 = 4641 \text{ kN}$

$$F_E = \frac{\pi^2 EI}{l_{buc}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 111932 \cdot 10^4}{4000^2} = 144995 \text{ kN}$$

$$\lambda_{rel} = \sqrt{\frac{N_{c;u;d}}{F_E}} = \sqrt{\frac{4641}{144995}} = 0.18 \quad \rightarrow \quad \omega_{buc} = 1.00$$

$$\frac{N_{c;s;d}}{\omega_{buc} \cdot N_{c;u;d}} = \frac{4215}{1.00 \cdot 4641} = 0.91 \geq 1$$ A HE500A will meet the buckling requirement.

11 Straight façade columns, sides of building, upper area (100-150m)

$N_{c;s;d} = 2107 \text{ kN}$

$$A_{\min} = \frac{N_{c;s;d}}{f_{y;d}} = \frac{2107}{235} = 8966 \, \text{mm}^2$$

A HE280A profile should be ok (9730 mm^2)

$N_{c;u;d} = A \cdot f_{y;d} = 9730 \cdot 235 = 2287 \text{ kN}$

$$F_E = \frac{\pi^2 EI}{l_{buc}^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^5 \cdot 13673 \cdot 10^4}{4000^2} = 17712 \text{ kN}$$

$$\lambda_{rel} = \sqrt{\frac{N_{c;u;d}}{F_E}} = \sqrt{\frac{2287}{11712}} = 0.44 \quad \rightarrow \quad \omega_{buc} = 0.88$$

$$\frac{N_{c;s;d}}{\omega_{buc} \cdot N_{c;u;d}} = \frac{2107}{0.88 \cdot 2287} = 1.05 \geq 1$$ A HE280A will meet not the buckling requirement. A HE300A should be sufficient.

12 Base plate, straight columns

$N_{s;d}$:	6160	kN
Column section $A_{HD400x818}$:	28330	mm^2
$h_l = h - 2 \cdot (t_{flange} + r)$:	290	mm
Base plate width b:	550	mm
$f_{y;d}$:	235	N/mm^2

Flange section: $A_{flange} = \frac{1}{2} \cdot (A_{HD400x818} - h_l \cdot t_w) = \frac{1}{2}(28330 - 290 \cdot 60.5) = 5393\,\text{mm}^2$

Compr. in profile: $\sigma_{s;d} = \frac{N_{s;d}}{A_{HD400x818}} = \frac{6160 \cdot 10^3}{28330} = 217\,\text{N/mm}^2$

Flange normal force: $N_{s;d;flange} = A_{flange} \cdot \sigma_{s;d} = 5393 \cdot 217 = 1172$ kN

Joint mortar: $f_{j;u;d} = 0.67 \cdot k_b \cdot f'_{b;d} = 0.67 \cdot 1 \cdot 15 = 10.05\,\text{N/mm}^2$

The needed length l_s is then: $l_s = \frac{1}{2}\left(\frac{N_{s;d;flange}}{f_{j;u;d} \cdot b} - t_{flange}\right) = \frac{1}{2}\left(\frac{1172 \cdot 10^3}{10.05 \cdot 550} - 97\right) = 57.5\,\text{mm}$

Base plate, minimum height: $t_{plate} = l_s \cdot \sqrt{\frac{3 \cdot f_{j;u;d}}{f_{y;d}}} = 37 \cdot \sqrt{\frac{3 \cdot 10.05}{235}} = 49\,\text{mm}$

This implicates a plate length of: $l_{plate} = h + 2 \cdot l_s = 514 + 2 \cdot 57.5 = 629\,\text{mm}$

Now the assumption of the continuous spread load has to be checked:

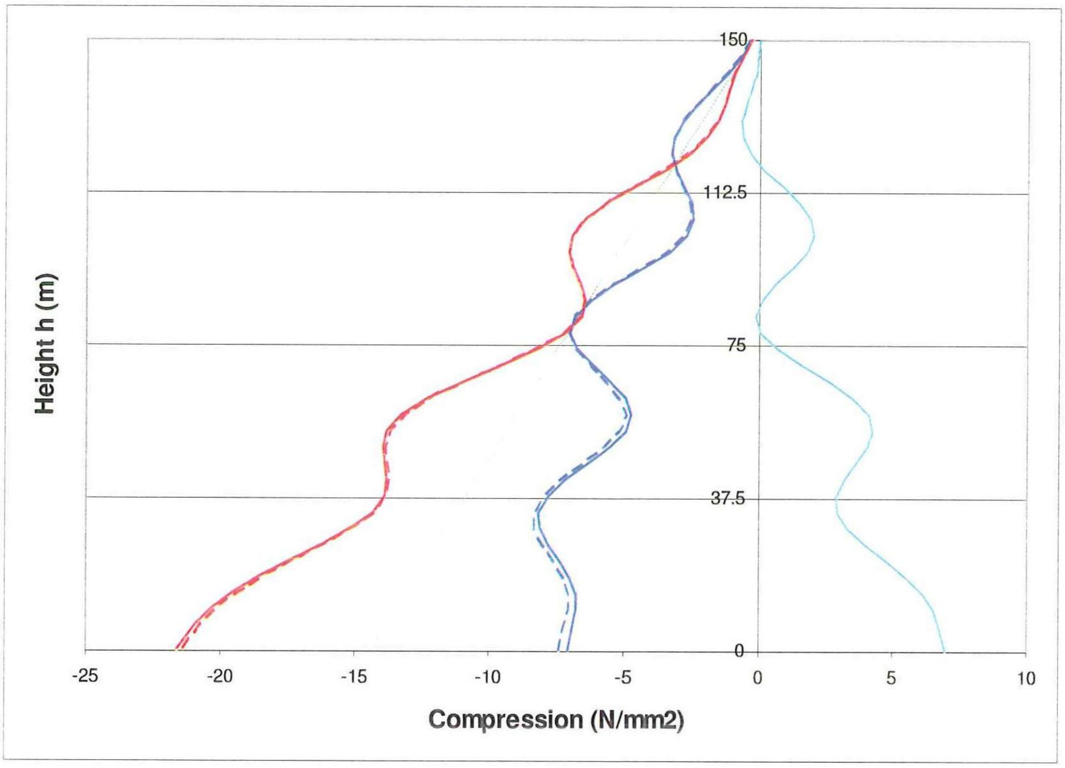
Requirement: $s \geq 1.5 \cdot t_{plate}$

$s = 56.5 \geq 1.5 \cdot 20.6 = 30.9\,\text{mm}$, the assumption was correct. A steel base plate of 630 x 550 x 21 mm will be sufficient to pass the normal forces from the inclined columns on to the foundation.

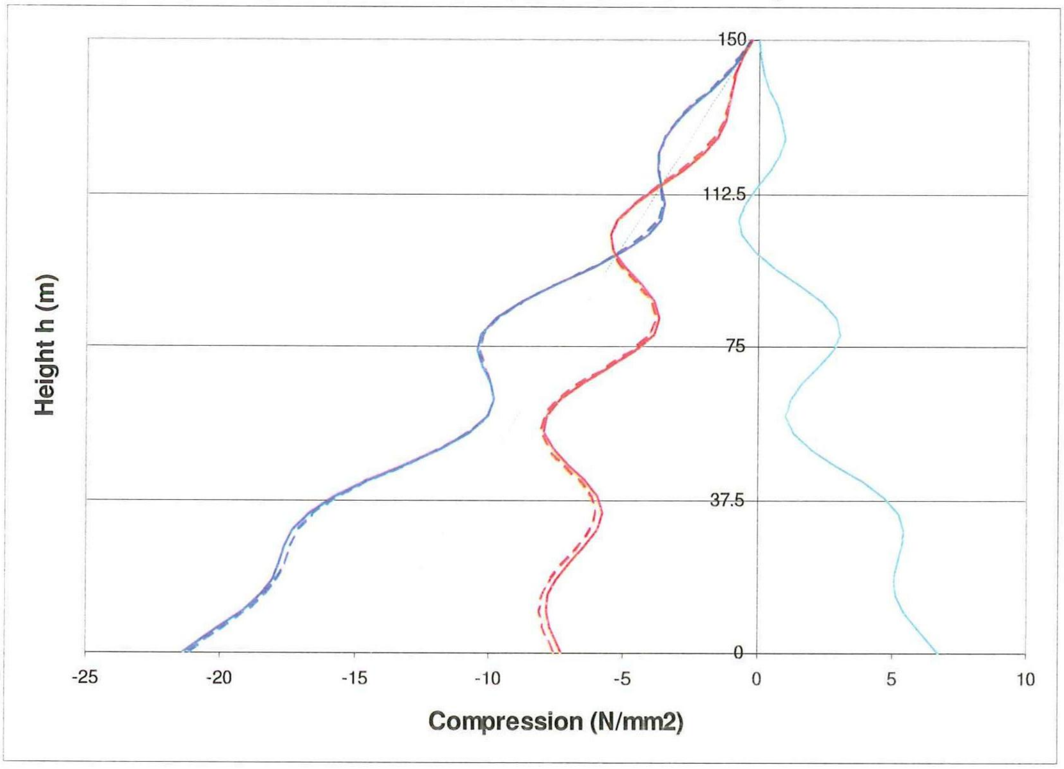
13 Initial deflection central core

height x	deflection
m	mm
0	0.00
6	0.01
10	0.05
14	0.10
18	0.18
22	0.27
26	0.35
30	0.40
34	0.40
38	0.35
42	0.26
46	0.15
50	0.06
54	0.02
58	0.04
62	0.14
66	0.29
70	0.46
74	0.62
78	0.73
82	0.77
86	0.74
90	0.66
94	0.55
98	0.47
102	0.42
106	0.44
110	0.52
114	0.65
118	0.80
122	0.95
126	1.07
130	1.16
134	1.21
138	1.23
142	1.24
146	1.24
150	1.24

14 Stresses in central core, including variable load

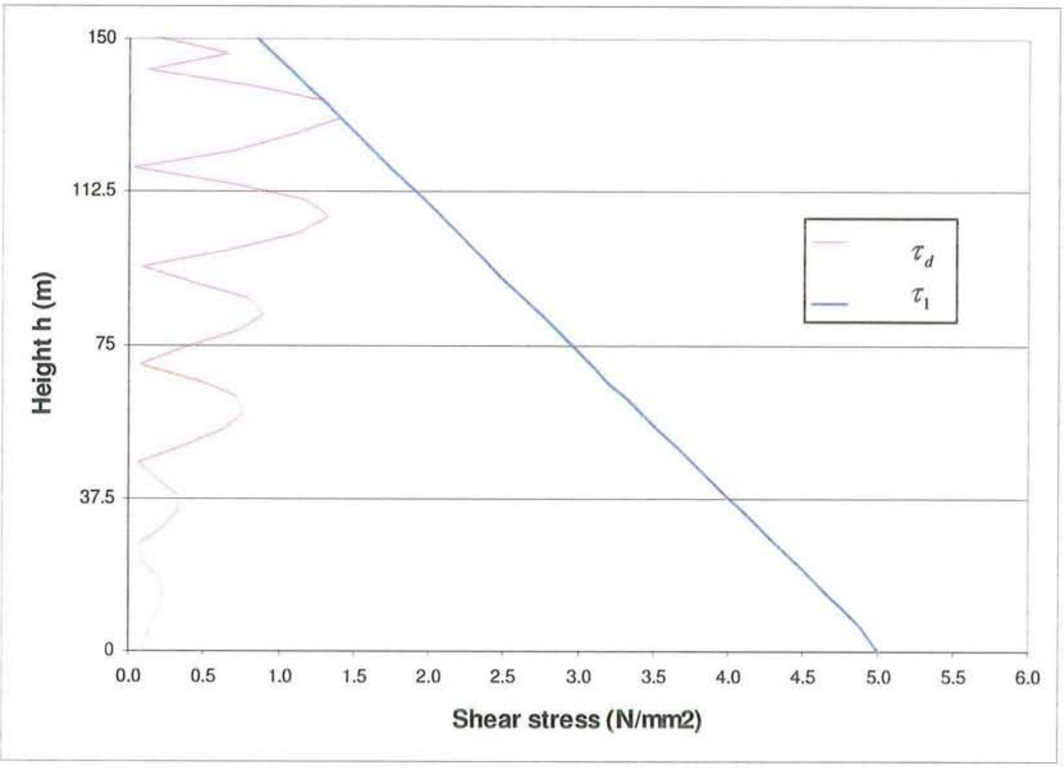


Stresses on left and right side of the central core, with wind from the left side of the building



Stresses on left and right side of the central core, with wind from the right side of the building

15 Shear stresses in central core, core wall width = 300 mm



16 Maximum shear stress τ_1

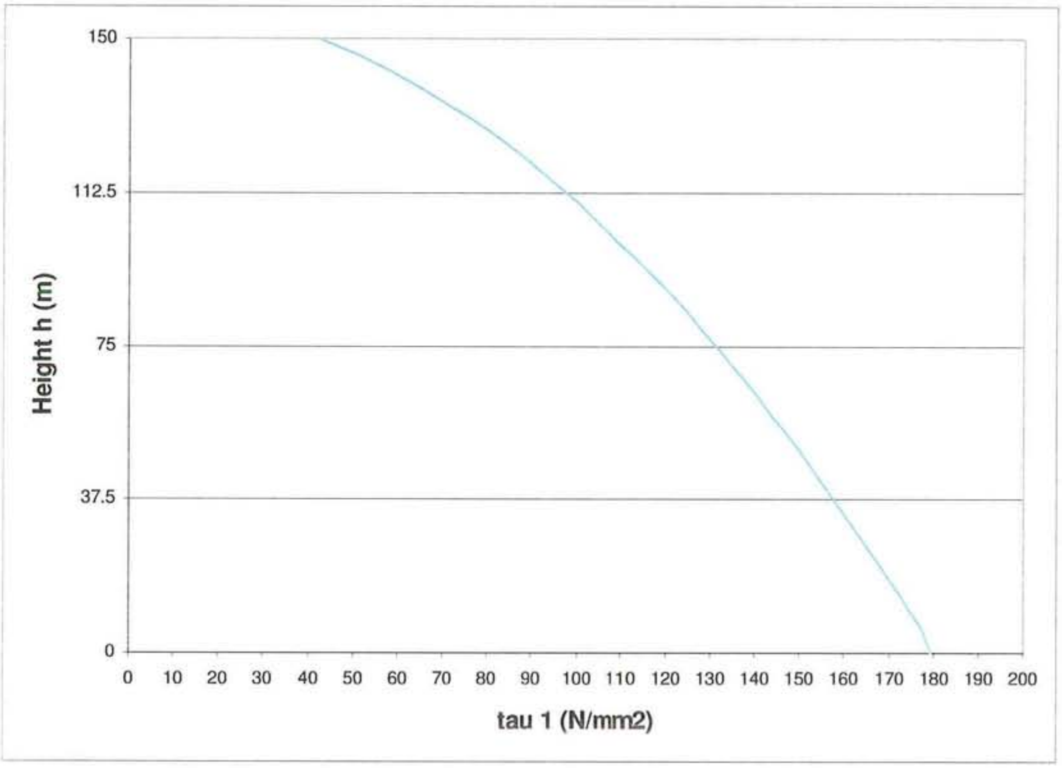
$$\tau_1 = \frac{I}{dS} \cdot \sqrt{f_b^{'2} + f_b^{'} \cdot \sigma_{bmd}^{'}}$$

$I = 1406.25\ \mathrm{m}^4$

$d = 250\ \mathrm{mm}$

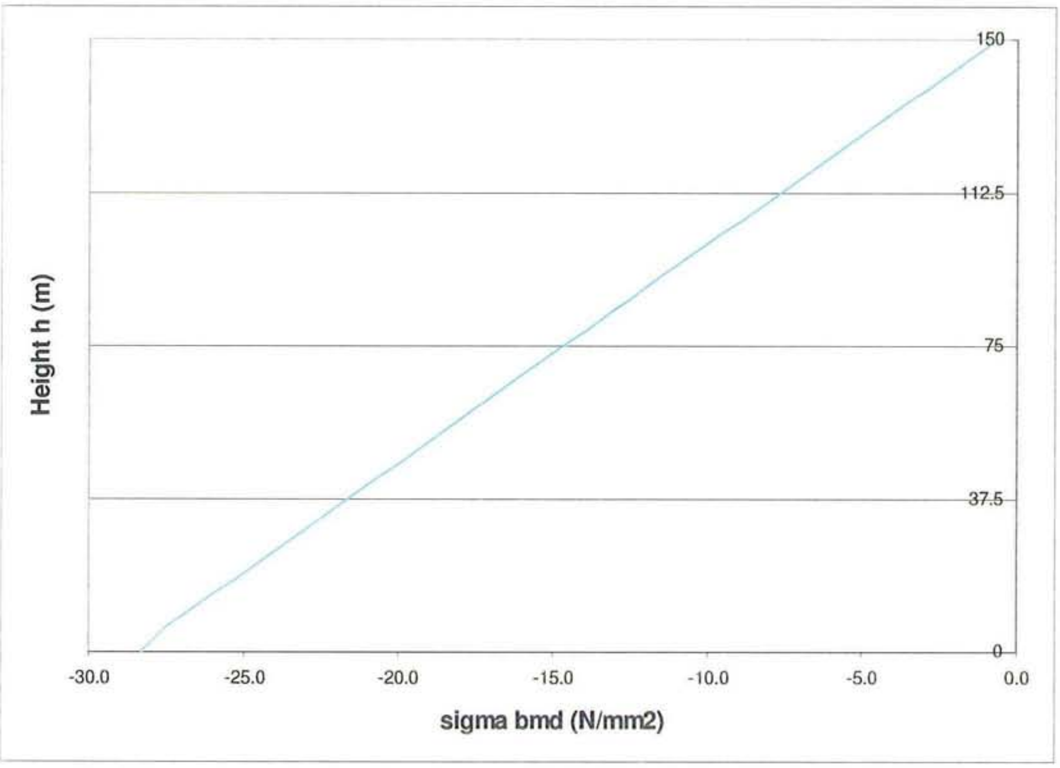
$S = 21750 \cdot 250 \cdot 21500 \cdot 0.5 = 5.85 \cdot 10^{10}\ \ \mathrm{mm}^4$

$f_b^{'} = 1.9\ \mathrm{N/mm}^2$ (B55 concrete)



17 Mean concrete compression

$$\sigma'_{bmd} = \frac{N'_d}{A}$$



Appendix H: Functional

Usable office space (min=1.8m)

Story	Height	usable floor length left	usable floor length right	total usable floor length	Floor surface
1	0	-9.66928	8.72	18.39	1126.809
2	6	-9.98344	8.40	18.39	1126.809
3	10	-10.3826	8.00	18.39	1126.809
4	14	-10.6626	7.72	18.39	1126.809
5	18	-10.5932	7.79	18.39	1126.809
6	22	-10.0605	8.33	18.39	1126.809
7	26	-9.13399	9.25	18.39	1126.809
8	30	-8.0681	10.32	18.39	1126.809
9	34	-7.23216	11.15	18.39	1126.809
10	38	-6.9886	11.40	18.39	1126.809
11	42	-7.55778	10.10	17.66	1097.649
12	46	-8.91642	8.39	17.31	1083.594
13	50	-10.7683	6.48	17.25	1081.269
14	54	-12.605	4.93	17.53	1092.736
15	58	-13.8455	4.29	18.14	1116.756
16	62	-13.47	4.37	17.84	1104.963
17	66	-11.66	5.47	17.13	1076.547
18	70	-9.03	7.67	16.70	1059.389
19	74	-6.27	10.45	16.72	1059.993
20	78	-4.18	13.03	17.21	1079.884
21	82	-3.46	14.65	18.11	1115.575
22	86	-3.6794	13.91	17.59	1094.914
23	90	-5.33943	11.34	16.68	1058.515
24	94	-8.39844	7.79	16.19	1038.875
25	98	-12.0935	4.19	16.28	1042.679
26	102	-15.4103	1.59	17.00	1071.349
27	106	-17.3611	0.00	17.36	1085.782
28	110	-16.13	0.00	16.13	1036.538
29	114	-12.78	3.36	16.14	1036.923
30	118	-8.29	7.29	15.58	1014.583
31	122	-6.45	11.90	18.35	1125.415
32	126	-1.2	15.93	17.13	1076.385
33	130	0	18.18	18.18	1118.614
34	134	0	16.48	16.48	1050.538
35	138	-1.2	12.32	13.52	932.1375
36	142	-8.15677	6.91	15.07	994.0081
37	146	-13.6779	1.69	15.37	1006.054
		Total usable floor area:			40020

Usable office space (min=3.5m)

Story	Height	usable floor length left	usable floor length right	total usable floor length	Floor surface
1	0	-9.66928	8.72	18.39	1126.809
2	6	-9.98344	8.40	18.39	1126.809
3	10	-10.3826	8.00	18.39	1126.809
4	14	-10.6626	7.72	18.39	1126.809
5	18	-10.5932	7.79	18.39	1126.809
6	22	-10.0605	8.33	18.39	1126.809
7	26	-9.13399	9.25	18.39	1126.809
8	30	-8.0681	10.32	18.39	1126.809
9	34	-7.23216	11.15	18.39	1126.809
10	38	-6.9886	11.40	18.39	1126.809
11	42	-7.55778	10.10	17.66	1097.649
12	46	-8.91642	8.39	17.31	1083.594
13	50	-10.7683	6.48	17.25	1081.269
14	54	-12.605	4.93	17.53	1092.736
15	58	-13.8455	0.00	13.85	945.1556
16	62	-13.47	0.00	13.47	930.1375
17	66	-11.66	0.00	11.66	857.7375
18	70	-9.03	7.67	16.70	1059.389
19	74	-6.27	10.45	16.72	1059.993
20	78	-4.18	13.03	17.21	1079.884
21	82	0	14.65	14.65	977.1747
22	86	0	13.91	13.91	947.7375
23	90	0	11.34	11.34	844.9375
24	94	-8.39844	7.79	16.19	1038.875
25	98	-12.0935	4.19	16.28	1042.679
26	102	-15.4103	0.00	15.41	1007.749
27	106	-17.3611	0.00	17.36	1085.782
28	110	-16.13	0.00	16.13	1036.538
29	114	-12.78	0.00	12.78	902.5375
30	118	-8.29	7.29	15.58	1014.583
31	122	-6.45	11.90	18.35	1125.415
32	126	0	15.93	15.93	1028.385
33	130	0	18.18	18.18	1118.614
34	134	0	16.48	16.48	1050.538
35	138	0	12.32	12.32	884.1375
36	142	-8.15677	6.91	15.07	994.0081
37	146	-13.6779	0.00	13.68	938.4538

Total usable floor area: 38594