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Project References: Agreement n. P. O. 4512422949
Conceptual evaluation of the structural response to seismic action of the Groninger Forum

1. FINAL REPORT

EP201507215390

NAM.GFB_1_FINAL REPORT_R1

REVISION RECORD SHEET

REV.	DATE	PROVIDED BY	CHECKED BY	APPROVED BY	NOTE
REV. 0	13/06/2015	[redacted] 5.1.2e	[redacted] 5.1.2e	[redacted] 5.1.2e	Updated in accordance with NAM comments and conf. call with the designers
REV. 1	13/07/2015	[redacted] 5.1.2e	[redacted] 5.1.2e	[redacted] 5.1.2e	Updated revision based on ABT-bam comments

DESCRIPTION OF REVISION:
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For the Client Approval: ____

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Distributed to: NAM____

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CONTENTS

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1 INTRODUCTION

This document has been made on request of NAM.

The objective of this document is:

- To get an independent review of the solutions as proposed by BAM/abt for strengthening the Groninger Forum against seismicity.
- To propose an alternative "best" option to cope with the induced seismicity.

Besides the above mentioned 2 objectives, a review of the original design is presented according to the seismic effects.

After the preparation of the preliminary version of this report, a number of questions were posed to the designers and were discussed in a conference call. For the sake of time, the resulting evidence has not been introduced point by point in the report, but rather summarised and critically commented in the section "Executive Summary". A fundamental suggestion related to the implementation of a base isolation solution has risen and has been introduced and commented, but it has not been studied in detail nor analysed applying non-linear time history, because of time constraints.

The work has been developed by Studio Calvi s.r.l. based in Pavia, Italy and experienced in the design, engineering and application of Base Isolation.

The Client is: Nederlandse Aardolie Maatschappij B.V. – Schepersmaat 2, 9405 TA Assen, The Netherlands.

The contract reference is n. P. O. 4512422949.

The whole responsibility of the project is in charge to Studio Calvi S.r.l.

The personnel involved in the project are listed below:

Table 1 Subjects involved in the project

Subject	Role	Contact
[redacted]	Responsible	[redacted]@studiocalvi.eu
[redacted]	Coordinator	[redacted]@studiocalvi.eu
[redacted]	Wind and mass analysis	
[redacted]	General review	
[redacted]	Non-linear analysis	
[redacted]	Non-linear analysis	
[redacted]	BIM model analysis	

2 EXECUTIVE SUMMARY

The study here summarized aims to provide a critical review of the solutions proposed to upgrade the current design of the *Groninger Forum* building in order to make it compatible with a seismic action only approximately defined at this stage, as well as to indicate preliminarily the order of magnitude of the costs to be sustained. The definition of the costs is achieved by a parametric estimate related to the supply and installation of the isolation devices. Much more detailed information is given in the "*Groninger Forum Building Feasibility study of a base isolation solution at the first underground level*" document, separately delivered.

This study has been performed by Studio Calvi in two weeks; it is therefore possible that some crucial aspect has been neglected or not considered in its full relevance. Its preliminary nature has thus to be fully understood and recognized. It is therefore recommended to arrange follow-up discussions.

To pursue the stated objective, all available documents (listed in § 9) have been examined and used when appropriate, including architectural and structural models, reference codes, and alternative solutions envisaged by the designers of the building. Based on this, some viable proposals have been identified from a structural engineering point of view and limited non-linear time history analyses have been run on simplified though consistent models.

Following the preparation of a first draft of this document, a number of questions have been prepared and proposed to the designers, to clarify some aspects and verify some assumptions. The answers to the questions have been discussed in a conference call and carefully considered. This has been quite useful for a better focalization of the conclusions.

The main findings of this study are summarized as follows.

a) *Structural System*

Upon a first examination, the current conceptual design of this structure seems to be sound and well-suited to resist horizontal actions, with a central pendulum steel structure transmitting shear forces to two strong lateral concrete towers.

After a more careful consideration, however, it is noted, from a seismic engineering point of view, that:

- The steel structure is quite irregular and not properly braced; the floors are discontinuous, hanging like horizontal cantilevers, with consequent difficulties in transmitting horizontal forces (§ 4).
- The shear capacity of the concrete cores, calculated as their bending moment capacity divided by some "*equivalent building height*," i.e. by the height of the point of application of the resultant of the horizontal forces, exceeds 20% of the total weight of the building (§ 6.5).
- However, the concrete towers are also characterized by a very irregular distribution of openings, in several locations inhibiting the force path to the foundations (§ 4).
- Some very large openings, particularly at the ground level, diminish the shear capacity of the walls to levels lower than 50% of the capacity derived from the bending strength, estimated assuming minimum reinforcement percentages (§ 6.5).
- As a consequence, a brittle shear collapse mode is predicted, which runs contrary to any sound structural design concept, in seismic conditions.

In conclusion, even assuming that the structure respects all applicable codes of practice, the current design is not resilient, i.e. unlikely able to sustain unforeseen events like seismicity.

The clarifications provided by the designers have confirmed that:

I. *The weights and masses considered are approximately the same (differences lower than 15 %, Q: 1, 2, 3, 4).*

II. *The periods and modes of vibration obtained are approximately the same, in the range of 1.0 - 1.3 s (Q: 5).*

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III. *The participating masses in the first two modes are in the range of 65 – 70 % in each direction (Q: 5).*

IV. *While the designers are assuming that most of the remaining mass should be attributed to some not better identified "higher modes", we believe that most of it refers to two main torsional modes (figure 15 and 16).*

V. *The yield displacement (Δ_y) of a model globally equivalent to the structure has not been determined by the designers (therefore the use of a q factor equal to 1.5 is in principle questionable).*
   a. *We estimate Δ_y in the range of 70 mm at an equivalent height of 31.5 m or in the range of 100 mm at the top of the building. This corresponds approximately to 0.2 % of the height, quite low, but consistent with the presence of very wide walls.*
   b. *The maximum displacement demand at the top of the building found by the designers applying the assumed design ground motion is 64 mm.*
   c. *Consequently, the displacement demand is in the range of 50 % of the yield displacement capacity and the structure should be far from entering a non-linear range under the design ground motion, regardless of the reinforcement percentages and distribution. This applies even to the wider shear wall. (Q: 6, 15, 23, 24).*

VI. *The shear capacities estimated by the designers are in the range of 25 MN in both directions (it is noted a very low -unless in presence of local shear problems - contribution of the West concrete tower in the Y direction).*
   a. *The shear capacity is thus in the range of 8 – 9 % of the building weight.*
   b. *If an approximately resulting period of vibration is calculated from mass and stiffness (as resulting from shear and displacement) a value of T = 1.75 s is obtained, consistently with the declared assumption of a stiffness reduction of 50 %, i.e. a period elongation of 20.5. (Q: 7, 8, 9, 10).*

b) *Wind Shear Demand*

The total shear demand due to the design wind load is estimated to be about 3% of the "*seismic*" weight of the building (§ 5.4), the majority of which is likely carried by the concrete towers. According to what stated by the designer, the base shear capacity of the concrete towers is approximate 8% of the seismic weight.

VII. *While the shear demand due to wind action calculated by the designers (2600 kN in the EW direction, 6000 in the NS) is slightly lower than what we estimated, the shear capacity is essentially confirmed.*

c) *Seismic Action*

The fundamental periods of vibration for the building is 1.0-1.3 sec and applying spectral shapes consistent with the NEN-NPR 2015 code or with the basic indications of EC8 (§ 6.2 and § 6.3) the seismic action on the building is estimated to be around 20% of the acceleration of gravity. Due to the expected brittle horizontal response of part of the structure, no reduction due to nonlinear response is considered. According to the assumed spectra, the displacement demand at this period of vibration would be around 70 mm, while the maximum displacement demand at longer periods of vibration would be around 125 mm. As already stated, the displacement demand (that do not consider possible reductions due to additional energy dissipation) corresponds to an average inter-story drift in the range of 0.2 %. The reason why these drifts are not compatible with the structural response and capacity is the predicted shear deficiency with respect to the bending moment capacity, which would lead to a brittle failure.

The acceleration and displacement demand may vary as a consequence of the local hazard and amplification analysis in preparation, in case it should be adopted.

VIII. *The designers have considered a PGA = 0.5 g, a force reduction factor q = 1.5 and have obtained total base shear values in the range of 40 MN, i.e. about 15 % of the weight of the building.*

IX. *The designers have also obtained a maximum vertical force of about 140 MN when considering the vertical ground motion as the principal acting action. This roughly corresponds to 50 % of the weight of the building, and seems quite reasonable. The corresponding overturning moments, however, are very high (Q: 17, 18, 19, 20).*

d) *Review of Design Solutions proposed by abt-BAM*

1) Strengthening the present structure on a member by member basis is a technically viable and sound possibility, particularly if the aim is to eliminate any brittle collapse mode in the concrete cores and to make the steel structure capable of transmitting the story shear forces to the concrete towers (§ 7.1).
2) The weak story solution, though in principle plausible, looks more like an academic exercise than a real and practical design concept (§ 7.2).
3) The transformation of the concrete towers into steel towers appears like an unnecessary and exaggerated way of emphasizing what can be reached relatively easily with the first solution. A reduction of weights and masses could be pursued with a progressive tapering of the concrete walls, which is not apparently applied in the current design (§ 7.3).
   Some insertion of steel trusses and bracings into the concrete cores could be an effective measure to regularize and improve their response, as well as that of the underground structure. However, these kinds of interventions have to be regarded as part of solution 1.
4) A total base isolation of the building is viable, but may not completely eliminate the need of some member strengthening, due to conceptual design problems not necessarily connected to a seismic action (§ 7.4). The base isolation solution has also the big advantage of higher flexibility with regard to probable changes in the seismic ground motion definition. The problems connected to the vertical action and the possible induced tensile forces in the devices have to be evaluated and addressed.
   The disadvantages and required time to develop and implement the B.I. solution indicated by the designers appear to be exaggerated.

X. *The discussion with the designers provided some insight on the reasons why the BI solution had been dropped. While we still believe that most motivations (such as the required time of design and production, or the asserted problems with higher modes of vibration) may be questioned, the problems related to a potential need for some increasing of the ground floor level and to the interaction with the vertical ground motion have to be better explored.*
We believe that most problems with the vertical ground motion may be eliminated by a better definition of the ground motion itself and by the application of sound non-linear analyses.
On the contrary, the difficulties in arranging a base isolation layer at the ground floor, together with other practical problems related to the part of the structure already constructed, may suggest addressing the possibility of isolation at the first floor or at an underground floor.

e) *Alternative design solutions proposed by NAM/Studio Calvi*

Three solutions (see Figure 1, Figure 2 and Figure 3) have been considered and preliminarily studied by Studio Calvi, applying non-linear time history analyses (NLTH) to consistently simplified models. The solutions A and B (§ 8.1 and 8.2) are based on the same principles as solutions 1) and **4**) above,

while solution C (§ 8.4) is a hybrid (new) solution, considering the isolation of the steel structure connected to the concrete towers by means of other types of isolation devices.
The consideration of the design solutions and the exams of the results of the NLTH analyses (§ 8.4) allows us to draw the following conclusions:

A. This solution requires a shear capacity in the concrete towers in the range of 15 to 20% of the seismic weight (Figure 29). This can be obtained by local strengthening of the concrete towers (adding bracing, closing openings, etc.) to make their shear capacity consistent with their flexural capacity (Figure 30). The steel structure may still show relatively large and irregular displacement demand (Figure 31), and consequently some locally large inter-story drift (up to 0.5 %), which may in turn require upgrading of some non-structural elements (Figure 34).
This solution is technically viable, but will impact the architectural features of the building and may imply some local non-structural damage unless proper measures are taken.

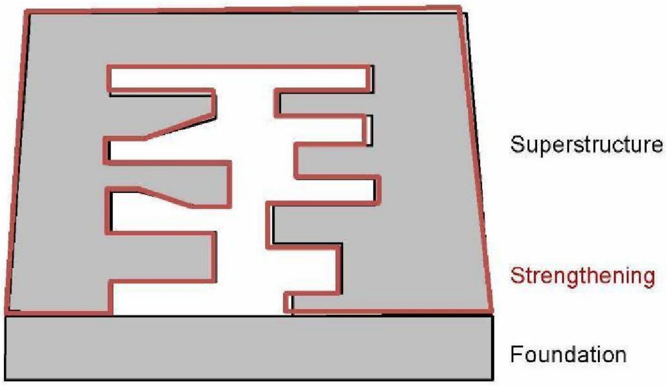
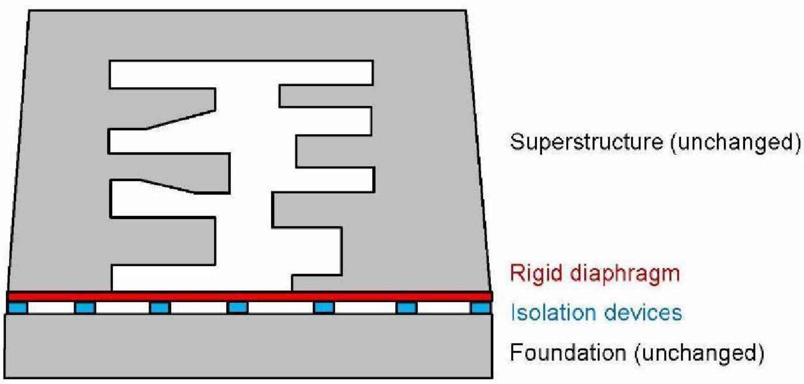


Figure 1 – Solution A: Strengthening

B. The base isolation solution, using low friction devices (see Figure 2), will limit the total shear demand to about 5% of the seismic weight of the building (Figure 29). This value is of the same order of magnitude as the wind load (5% against 3%, see item (b) above). The displacement demand (Figure 33) will be regular, and the inter-story drift limited to a maximum of 0.35 % (Figure 36), consequently any non-structural damage is unlikely.
This solution requires the construction of a stiff slab above the existing slab at ground level supported by about 100 low friction devices.(see figure 3). Based on the parametric analysis of previous projects, it is preliminarily estimated that the cost of this measure will be in the range of 1 to 2 million Euros (not including architectural and service modifications).



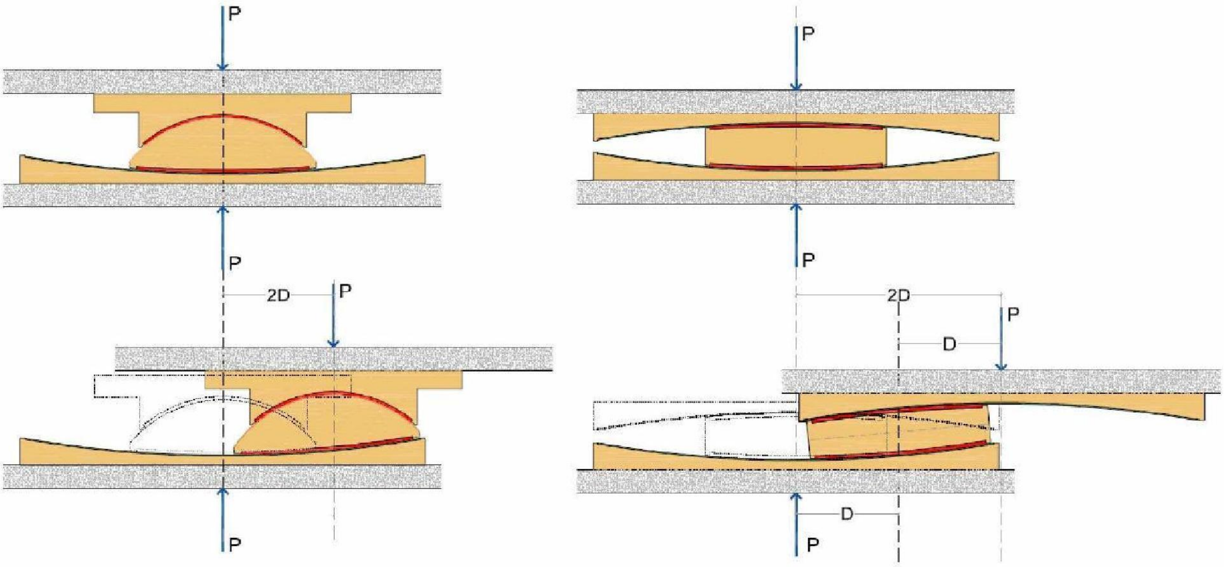


Figure 2 – Solution B: Base Isolation (a) and curved surface sliding device ((b), single sliding surface, left, double sliding surfaces, right)

C. The hybrid solution requires a shear capacity in the concrete towers in the range of 10 to 15% of their seismic weight (fig. 3). The concrete cores will still have to be locally strengthened, at least to a shear capacity commensurate with 80 % of the flexural capacity (Figure 30). The steel structure will have to be upgraded to be able to transmit some shear to the concrete cores and to the foundation, but the horizontal forces to be transmitted will be limited to about 1% of the weight (to the foundation) and 2 % of the weight (to the concrete towers). These values are consistent with the present demand resulting from wind (§ 5). The displacement demand (Figure 32) will be regular, and the inter-story drift limited to a maximum of 0.25 % (Figure 35). Any non-structural damage is unlikely.
This solution is also viable and will likely imply fewer and lighter interventions on the current design of members. In case of installations (e.g. elevators) present in the concrete towers which are present in both parking garage and building this solution may be attractive.

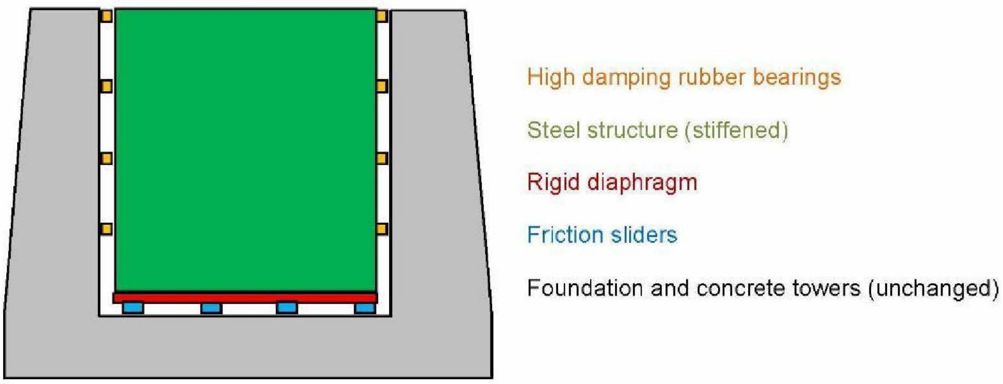


Figure 3 – Solution C: Hybrid solution

XI. As anticipated at point X, the discussion with the designers highlighted the difficulties of increasing the level of the ground floor for architectural reasons; they also illustrated a number of potential problems related to architectural issues. These problems, together with other practical problems related to the part of the structure already constructed, suggest to consider additional alternative *solutions:*

1. ***the insertion of an isolation layer at the first floor rather than at the ground floor***

This solution is attractive from a structural point of view, since it is very likely that the structure will not require any further strengthening measure from the second floor up.
Strengthening will be mainly required at the first level and demolition will be reduced, but architectural problems will not be solved.

2. ***the insertion of an isolation layer under the ground floor***

Structural intuition indicates that this solution may present a number of advantages:

- *from isolation level up the structure should not require any modification.*
- *the structure between the ground floor and the first floor can be strengthened without any demolition*
- *the isolation devices will be located at the top of columns and shear walls, without any need of horizontal bracing, under the current ceiling location*
- *cutting of columns and shear walls can be performed applying well known and tested techniques*
- *cutting of the external retaining wall will need some additional external protection against water, but this will not imply major works*
- *columns and shear walls may need some strengthening, but this can likely be contained in a few centimetres.*

We believe that this solution may result to be the most effective in term of timing, cost and viability, therefore it will be the object of a specific report. (Studio Calvi Report Feasibility BI at 1st underground level)

f) Conclusions

In summary, it is our opinion that:

- The Groningen Forum building may require some design revision even in the absence of any consideration of seismic action to make it more resilient.
- The consideration of seismic action will require additional interventions. Viable interventions are possible taking into account the current status of the project.
- The solution based on local strengthening is viable, but it should be pursued clearly separating the interventions intended to improve the structural layout from those inherent to the increased horizontal action.
- The base isolation of the building (solution B, fig. 2) using low friction devices is based on proven technology and is a technically viable and cost effective solution. Since it will likely reduce the global acting forces to values lower than those presently identified as capacities, any strengthening intervention on the structural elements should be attributed to a wish of improving the structural layout.
- The (new proposed) hybrid solution C (fig. 3) is considered to be an attractive alternative to solution B which may be further developed during follow-up discussions.
- The insertion of the isolation layer at the top, rather than at the bottom, of the ground floor, will solve both problems related to the level of the ground floor and problems related to the part of the structure already constructed. Note that the ground floor is that characterized by wider openings that impair the shear capacity of the buildings, where more strengthening interventions are thus needed. This conclusion has been made prior to the information with regard to functionality received during the workshop dated 18 June 2015.

- Finally, the insertion of an isolation layer at the first underground level may result in the best compromise to contain costs and reduce to a minimum the need of architectural modifications.

We believe that this solution may result to be the most effective in term of timing, cost and viability. For this reason it has been further evaluated and documented in a a specific report *(Studio Calvi Report Feasibility BI at 1st underground level)*

3 FOREWORD

3.1 OBJECTIVES

The objectives of the study presented in this report are:

- to perform a preliminary evaluation of the expected seismic performance of the *Groninger Forum*, with reference to the original design and configuration
- to examine and evaluate the upgraded design solutions proposed by the designers (abt –BAM)
- to suggest improvements for one or more of the solutions or to propose potential alternatives
- to provide preliminary evaluations of the potential cost of a *seismic upgrading*.

These ambitious objectives have been pursued in the context of a critical contribution, as requested by NAM, to the current evaluation of solutions, without the presumption of providing any definitive conclusion or absolute statements.

3.2 LIMITS AND DISCLAIMER

This study has been performed in two weeks; it is therefore possible that some crucial aspect has been neglected or not considered in its full relevance. Its preliminary nature has thus to be fully understood and recognized.

As pointed out in the previous section, the general objective is to provide a sound basis for discussion rather than conclusive remarks.

To perform the study, data, documents, and models provided to the team have been used extensively, even without the information required for detailed and in-depth checks. Therefore, although we have worked with due diligence and are convinced of having all the competences and the expertise required to prepare a useful and reliable document, at this stage all the results and conclusions presented should be considered as preliminary evaluations, to be discussed and revised before fully adopting them.

3.3 SCOPE

The methods and work sequence applied to this study can be summarized as follows, bearing in mind the time constraints that were faced:

- Examination of the provided BIM model
- Examination of the provided SAP model
- Examination of wind provisions likely to have been applied to the design
- Re-run of the model to assess the minimum strength required by the wind load
- Estimation of the likely strength of some potentially critical elements
- Analysis of the designer's proposed solutions
- Development of alternative solutions
- Preliminary evaluations of potential costs involved in some basic cases
- Development of simplified models to perform non-linear time history analyses
- Production of time history inputs consistent with the assumed seismic input
- Critical evaluation of the results and discussion of the potential conclusions

4 DESCRIPTION OF THE BUILDING

4.1 GEOMETRY AND STRUCTURAL ELEMENTS

The *Groninger Forum* is composed of 5 garage levels and 11 above-ground floors. The plan dimensions of the building above ground are about 79 m x 39 m and the maximum height is 45 m. The structural scheme is characterized by two lateral concrete cores (west and east; in this report these parts of the structures may be indicated as "core" or "tower" without any implication on the meaning) and a central steel system made of trusses and steel slabs. The structural systems are referred to as NW for the west concrete core, NC for the central steel structure and NE for the east concrete core.

The two lateral cores NW and NE have plan dimensions of about 12 x 48 m and 13 x 25 m respectively. The structural system is made of inclined RC walls (about 12° from vertical) with a depth of 50 cm for the interior walls and 40 cm for the three outer walls. The cores are characterized by high irregularity because of the inclination of walls, the large number of internal and external irregular openings, and the presence of discontinuous structural elements.

At ground level, transfer beams are employed in order to support several discontinuous bearing structures; this is another cause of irregularity for the whole building in terms of structural response under horizontal actions.

The central system NC is made of steel trusses. The three upper levels are continuous (11th, 10th and 9th) while the others consist of plates protruding from the concrete cores and which act as cantilevers considering the horizontal response of the structure. The vertical loads are carried by a system of steel columns supporting truss-beams. The central core is characterized by high vertical irregularity, some soft storeys, and possible out-of-plane bending of the cantilever system.

The above observations are illustrated in the following figures.

In Annex 1 the plan views of each level are summarized (for the concrete: red for the structural elements of the level, yellow for the structural elements below).

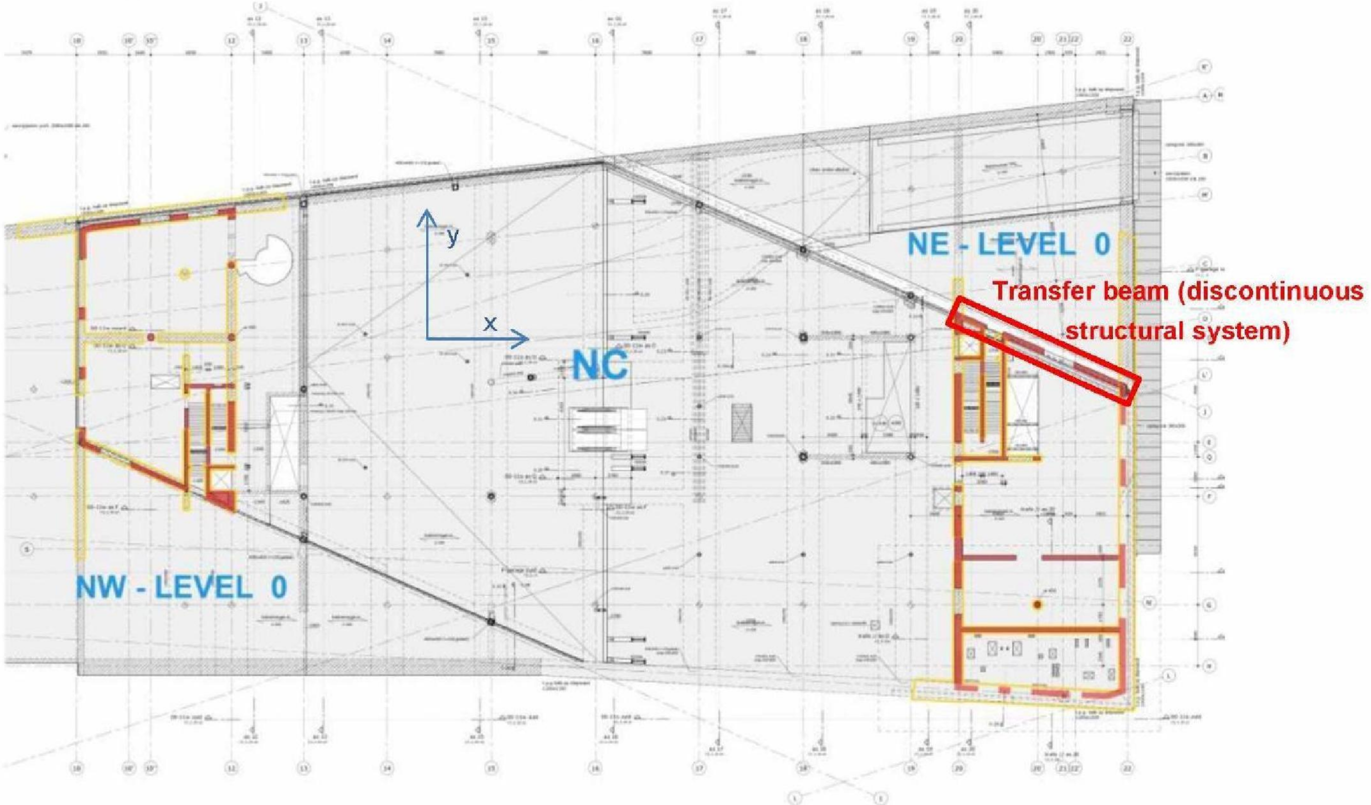


Figure 4 – Plan view of ground level (red for the structural elements of the level, yellow for the structural elements below)

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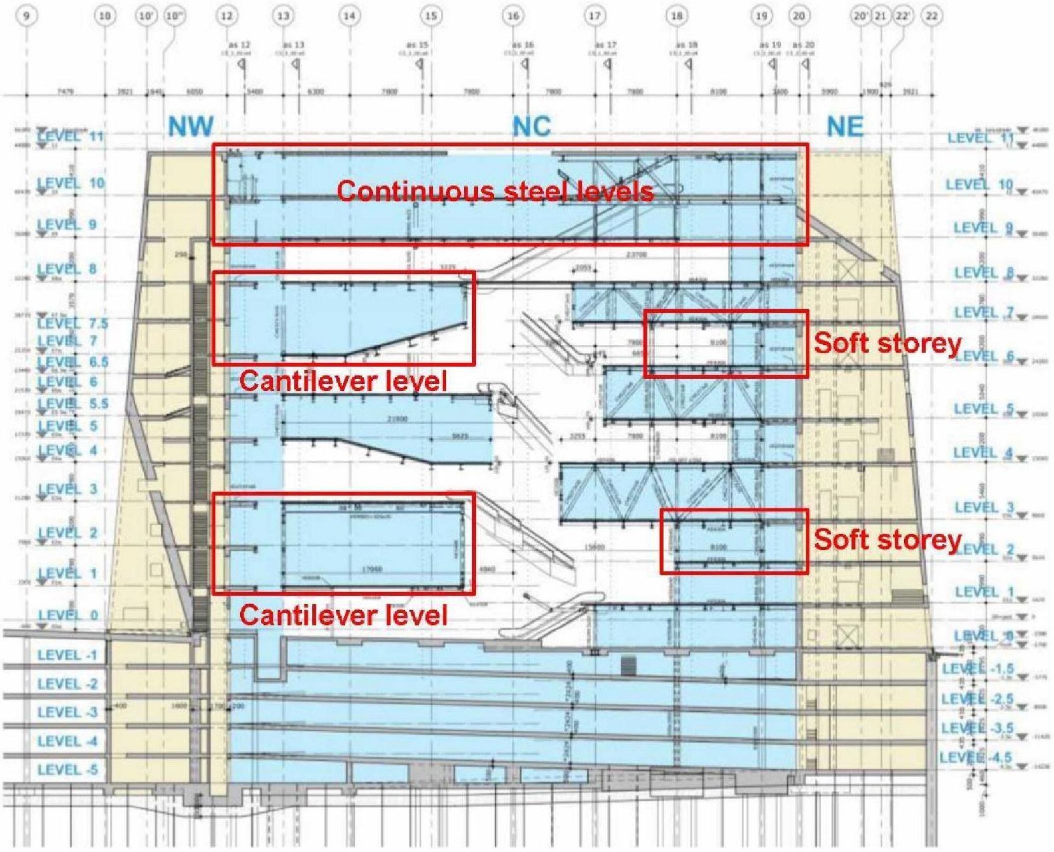

Figure 5 – A typical section of the building (yellow for concrete cores and blue for central steel system)

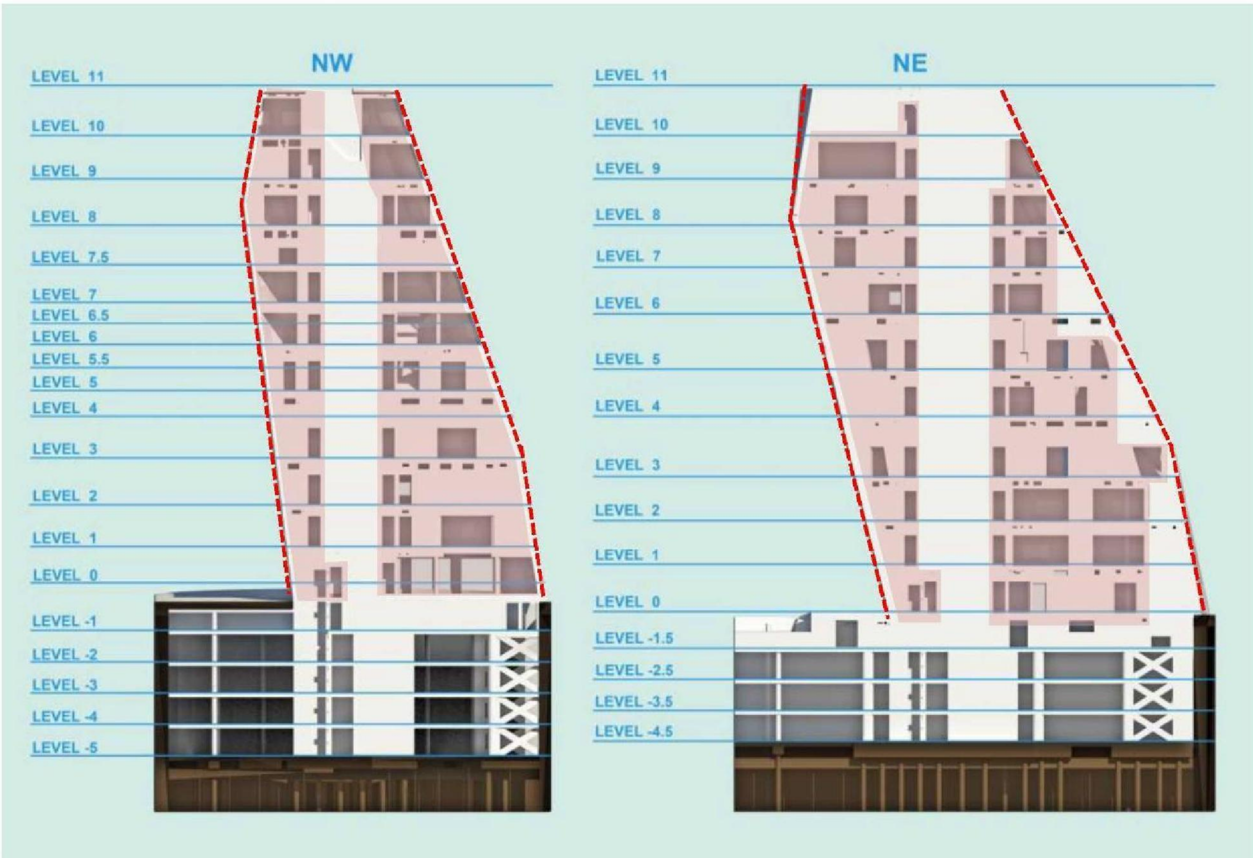

Figure 6 – Sections of NW and NE cores (dot lines for walls inclination and red filling for openings areas)

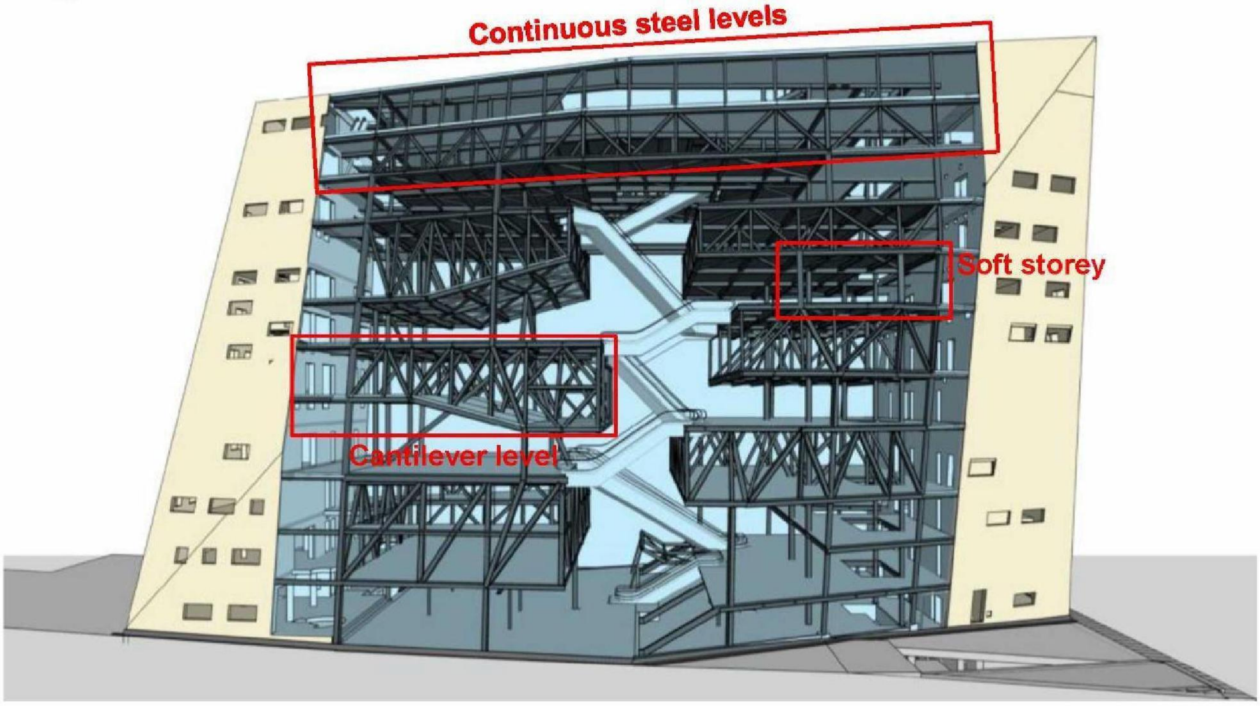

Figure 7 – A three-dimensional sketch of the building

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4.2 MASS DISTRIBUTION

In order to evaluate the masses and loads distribution, both the finite-element numerical model and the three-dimensional architectural model have been considered.

As before, the structural systems are referred to as NW for the west concrete core, NC for the central steel structure and NE for the east concrete core (see Figure 8).

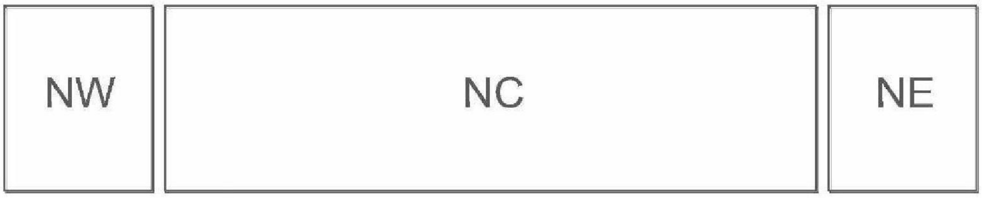


Figure 8 – Structural schematization

The gravitational mass is defined at each floor considering:

- Self-weight of slabs (1);
- Permanent loads on slabs (2);
- Imposed loads (3);
- Self-weight of outer walls (4);
- Self-weight of outer walls' covering (5);
- Self-weight of lifts' inner walls (6);
- Self-weight of steel structures, escalator and inner covering ((4) only NC).

Hereafter, self-weight and permanent loads are identified as "G" and the imposed loads as "Q".

4.2.1 Percentage mass distribution

The following table reports the estimated percentages of mass distribution for gravity (G+Q) and seismic (G+0.36Q) loads considering the three cores (west, central and east).

The seismic mass is defined in the Eurocode 8 as $\psi_{Ei} = \varphi \cdot \psi_{2i}$. Considering the category C for congregation areas, $\psi_{2i} = 0.6$ (according to EN1991-1 and Dutch National Annex) and $\varphi = 0.6$ (according to EN1998-1 and NPR9998).

Table 2 – Percentage mass distribution

GRAVITY LOAD			SEISMIC LOAD		
NW	NC	NE	NW	NC	NE
16.65%	58.85%	24.50%	18.12%	55.18%	26.70%

The floor masses are also computed as percentages of the total gravity weight (G+Q) at each floor, as shown in Table 3, Table 4 and Table 5.

Table 3 – Percentage mass distribution west core – Gravity load

"WEST CORE" MASS DISTRIBUTION - GRAVITY LOAD				
Storey height [mm]	Floor: "dead+perm" load/(G+Q) [%]	Floor: imposed load /(G+Q) [%]	Outer walls: dead load /(G+Q) [%]	Inner walls: dead load /(G+Q) [%]
44595	0.34%	0.14%	0.33%	0.05%
40395	0.43%	0.17%	0.65%	0.11%
36405	0.39%	0.15%	0.65%	0.11%
32205	0.39%	0.15%	0.62%	0.10%
28635	0.43%	0.16%	0.70%	0.11%
23385	0.42%	0.16%	0.73%	0.12%
19395	0.40%	0.16%	0.48%	0.08%
17295	0.39%	0.16%	0.35%	0.06%
14985	0.58%	0.24%	0.48%	0.08%
11205	0.65%	0.26%	0.63%	0.10%
7005	0.71%	0.29%	0.63%	0.10%
3225	0.80%	0.32%	0.95%	0.15%
Partial	**5.93%**	**2.33%**	**7.22%**	**1.17%**

In the west core the contribution of the outer wall and of the slab floor is significant.

Table 4 – Percentage mass distribution central core – Gravity load

"CENTRAL CORE" MASS DISTRIBUTION - GRAVITY LOAD			
Storey height [mm]	Towards core	Floor: "dead+perm" load/(G+Q) [%]	Floor: imposed load /(G+Q) [%]
44595	w-e	3.44%	1.39%
40395	w-e	4.56%	1.84%
36405	w-e	3.83%	2.00%
32205	w-e	3.40%	1.88%
28425	o	2.09%	1.03%
25275	w , var. (up)	1.77%	0.97%
24225	e	1.69%	0.94%
21495	w	2.06%	1.13%
19185	e	1.79%	0.98%
17295	w , var. (down)	2.08%	1.21%
15060	e	2.27%	1.21%
11205	w	2.03%	1.06%
9525	e	2.14%	1.25%
7000	w	0.99%	0.43%
5535	e	0.92%	0.45%
3245	w	1.84%	1.05%
1545	e	2.09%	1.05%
Partial		**39.00%**	**19.85%**

In the central core the contribution of the slab floor is significant.

Table 5 – Percentage mass distribution east core – Gravity load

"EAST CORE" MASS DISTRIBUTION - GRAVITY LOAD				
Storey height [mm]	Floor: "dead+perm" load/(G+Q) [%]	Floor: imposed load /(G+Q) [%]	Outer walls: dead load /(G+Q) [%]	Inner walls: dead load /(G+Q) [%]
44595	0.36%	0.22%	0.47%	0.10%
40395	0.63%	0.24%	0.91%	0.19%
36405	0.66%	0.25%	0.91%	0.19%
32205	0.67%	0.26%	0.89%	0.18%
28425	0.74%	0.29%	0.89%	0.18%
24225	0.82%	0.32%	1.03%	0.21%
19185	0.82%	0.32%	1.03%	0.21%
14985	0.82%	0.32%	1.07%	0.22%
9525	1.06%	0.43%	1.05%	0.22%
5535	0.64%	0.25%	0.89%	0.18%
1545	1.20%	0.47%	1.39%	0.29%
Partial	**8.42%**	**3.39%**	**10.50%**	**2.19%**

In the east core the contribution of the outer wall and of the slab floor is significant.

The floor masses are computed as percentage of the total seismic weight (G+0.36Q) at each floor, as shown in Table 6, Table 7 and Table 8.

Table 6 – Percentage mass distribution west core – Seismic load

"WEST CORE" MASS DISTRIBUTION - SEISMIC LOAD				
Storey level [mm]	Floor: "dead+perm" load/(G+0.36Q) [%]	Floor: imposed load /(G+0.36Q) [%]	Outer walls: dead load /(G+0.36Q) [%]	Inner walls: dead load /(G+0.36Q) [%]
44595	0.40%	0.06%	0.40%	0.06%
40395	0.52%	0.07%	0.78%	0.13%
36405	0.47%	0.06%	0.78%	0.13%
32205	0.46%	0.06%	0.74%	0.12%
28635	0.51%	0.07%	0.84%	0.14%
23385	0.50%	0.07%	0.88%	0.14%
19395	0.48%	0.07%	0.58%	0.09%
17295	0.47%	0.07%	0.42%	0.07%
14985	0.69%	0.10%	0.58%	0.09%
11205	0.78%	0.11%	0.76%	0.12%
7005	0.85%	0.12%	0.76%	0.12%
3225	0.96%	0.14%	1.14%	0.18%
Partial	**7.09%**	**1.00%**	**8.63%**	**1.40%**

Table 7 – Percentage mass distribution central core – Seismic load

"CENTRAL CORE" MASS DISTRIBUTION - SEISMIC LOAD			
Storey level [mm]	Towards core	Floor: "dead+perm" load/(G+0.36Q) [%]	Floor: imposed load /(G+0.36Q) [%]
44595	w-e	4.12%	0.60%
40395	w-e	5.45%	0.79%
36405	w-e	4.58%	0.86%
32205	w-e	4.06%	0.81%
28425	o	2.50%	0.44%
25275	w , var. (up)	2.12%	0.42%
24225	e	2.02%	0.40%
21495	w	2.47%	0.49%
19185	e	2.14%	0.42%
17295	w , var. (down)	2.49%	0.52%
15060	e	2.71%	0.52%
11205	w	2.43%	0.46%
9525	e	2.56%	0.54%
7000	w	1.19%	0.18%
5535	e	1.10%	0.20%
3245	w	2.20%	0.45%
1545	e	2.50%	0.45%
Partial		**46.63%**	**8.55%**

Table 8 – Percentage mass distribution east core – Seismic load

DISTRIBUTION MASSES IN "EAST CORE" - SEISMIC LOAD				
Storey level [mm]	Floor: "dead+perm" load/(G+0.36Q) [%]	Floor: imposed load /(G+0.36Q) [%]	Outer walls: dead load /(G+0.36Q) [%]	Inner walls: dead load /(G+0.36Q) [%]
44595	0.42%	0.09%	0.56%	0.12%
40395	0.75%	0.10%	1.09%	0.23%
36405	0.78%	0.11%	1.09%	0.23%
32205	0.80%	0.11%	1.06%	0.22%
28425	0.89%	0.13%	1.06%	0.22%
24225	0.98%	0.14%	1.23%	0.26%
19185	0.98%	0.14%	1.23%	0.26%
14985	0.99%	0.14%	1.28%	0.27%
9525	1.27%	0.18%	1.25%	0.26%
5535	0.77%	0.11%	1.06%	0.22%
1545	1.43%	0.20%	1.66%	0.35%
Partial	**10.07%**	**1.46%**	**12.56%**	**2.62%**

4.2.2 *Best estimate of total gravitational loads*

The computation of gravity and seismic loads has been carried out in some detail for each floor and core (NW, NC and NE). A uniform imposed load of q_a = 5.0 kN/m^2[(1)] has been considered at each floor; the weight of inner and outer walls has been lumped at each floor considering their relevant height.

1) <u>*West core (NW)*</u>

Considering the west RC core the estimate of the total weight is:

Self-weight and permanent loads G = 41,091.5 KN

Imposed loads Q = 6,685.0 KN

Table 9 – West Core (NW) total weight

htot=	45.495	m								
Storey level [mm]	Storey height [m]	RC slab area [mq]	(1) RC slab self-weight [KN]	(2) Permanent load [KN]	(3) Imposed load q=5KN/m2 [KN]	(4) Self-weight of outer RC walls [KN]	(5) Self-weight of outer covering q=1,25KN/m2 [KN]	(6) Self weigth of inner RC walls [KN]	(1)+(2)+(3)+(4)+(5)+(6) Total weigth [KN]	(1)+(2)+(3)+(4)+(5)+(6) Total distributed weight [KN/m2]
44595	4.2	81	641.5	251.1	405.0	956.5	77.4	154.8	2486.4	30.7
40395	3.99	95	798.0	294.5	475.0	1865.3	151.0	301.9	3885.7	40.9
36405	4.2	84	705.6	260.4	420.0	1865.3	151.0	301.9	3704.2	44.1
32205	3.57	84	705.6	260.4	420.0	1769.6	143.2	286.5	3585.3	42.7
28635	5.25	92	772.8	285.2	460.0	2008.8	162.6	325.2	4014.5	43.6
23385	3.99	90	756.0	279.0	450.0	2104.4	170.3	340.6	4100.4	45.6
19395	2.1	90	756.0	279.0	450.0	1387.0	112.3	224.5	3208.8	35.7
17295	2.31	90	756.0	279.0	450.0	1004.4	81.3	162.6	2733.3	30.4
14985	3.78	135	1134.0	418.5	675.0	1387.0	112.3	224.5	3951.3	29.3
11205	4.2	150	1260.0	465.0	750.0	1817.4	147.1	294.2	4733.8	31.6
7005	3.78	165	1386.0	511.5	825.0	1817.4	147.1	294.2	4981.3	30.2
3225	4.125	181	1520.4	561.1	905.0	2739.8	221.8	443.5	6391.6	35.3
-900										
Partial sum [KN]		1337.0	11191.9	4144.7	6685.0	20722.9	1677.5	3354.5		
								Total weight P=	47776.5	KN

2) <u>*Central core (NC)*</u>

Considering the central steel core the estimate of the total weight is:

Self-weight and permanent loads G= 111,921.4 KN

Imposed loads Q= 56,977.5 KN

Table 10 – Central Core (NC) total weight

htot, m=	46.395	m							
Storey level [mm]	Towards core	Storey height [m]	RC slab area [mq]	(1) RC slab self-weight [KN]	(2) Permanent load [KN]	(3) Imposed load q=5KN/m2 [KN]	(4) Steel structures, etc. [KN]	(1)+(2)+(3)+(4) Total weigth [KN]	(1)+(2)+(3)+(4) Total distributed weight [KN/m2]
44595	w-o	4.2	796.5	6308.3	2469.2	3982.5	1108.7	13868.7	17.4
40395	w-o	3.99	1058	7647.8	3279.8	5290.0	2162.0	18379.6	17.4
36405	w-o	4.2	1150	5260.8	3565.0	5750.0	2162.0	16737.8	14.6
32205	w-o	3.78	1077	4304.4	3338.7	5385.0	2106.6	15134.7	14.1
28425	o	3.15	592	2346.8	1835.2	2960.0	1829.4	8971.4	15.2
25275	w , var. (up)	1.05	556	2257.6	1723.6	2780.0	1108.7	7869.9	14.2
24225	o	2.73	537	2191.6	1664.7	2685.0	997.9	7539.2	14.0
21495	w	2.31	649	2581.2	2011.9	3245.0	1330.5	9168.6	14.1
19185	o	1.89	560	2286.0	1736.0	2800.0	1108.7	7930.7	14.2
17295	w , var. (down)	2.235	693	2742.8	2148.3	3465.0	1088.9	9445.0	13.6
15060	o	3.855	696	2736.8	2157.6	3480.0	1607.7	9982.1	14.3
11205	w	1.68	607	2483.2	1881.7	3035.0	1461.2	8861.1	14.6
9525	o	2.525	717	2804.0	2222.7	3585.0	1110.1	9721.8	13.6
7000	w	1.465	244	1040.8	756.4	1220.0	1053.3	4070.5	16.7
5535	o	2.29	261	835.2	809.1	1305.0	991.3	3940.6	15.1
3245	w	1.7	602	2352.8	1866.2	3010.0	1053.3	8282.3	13.8
1545	o	3.345	600	1920.0	1860.0	3000.0	2214.8	8994.8	15.0
-1800									
Partial sum [KN]			11395.5	52100.0	35326.1	56977.5	24495.3		
							Total weight P=	168898.9	KN

3) <u>*East core (NE)*</u>

Considering the east RC core the estimate of the total weight is:

Self-weight and permanent loads G= 60,586.3 KN

Imposed loads Q= 9,725.0 KN

PR.: 5.1.2e	CK.: 5.1.2e	AP.: 5.1.2e

Table 11 – East Core (NE) total weight

htot=	47.295	m								
Storey level [mm]	Storey height [m]	RC slab area [mq]	(1) RC slab self-weight [KN]	(2) Permanent load [KN]	(3) Imposed load q=5KN/m2 [KN]	(4) Self-weight of outer RC walls [KN]	(5) Self-weight of outer covering q=1,25KN/m2 [KN]	(6) Self weigth of inner RC walls [KN]	(1)+(2)+(3)+(4)+(5)+(6) Total weigth [KN]	(1)+(2)+(3)+(4)+(5)+(6) Total distributed weight [KN/m2]
44595	4.2	126	645.0	270.8	630.0	1338.1	103.2	279.4	3266.4	25.9
40395	3.99	140	1176.0	434.0	700.0	2609.2	201.3	544.8	5665.3	40.5
36405	4.2	146	1226.4	452.6	730.0	2609.2	201.3	544.8	5764.3	39.5
32205	3.78	150	1260.0	465.0	750.0	2542.3	196.1	530.8	5744.3	38.3
28425	4.2	168	1411.2	520.8	840.0	2542.3	196.1	530.8	6041.3	36.0
24225	5.04	185	1554.0	573.5	925.0	2943.7	227.1	614.6	6838.0	37.0
19185	4.2	185	1554.0	573.5	925.0	2943.7	227.1	614.6	6838.0	37.0
14985	5.46	185	1554.0	573.5	925.0	3077.5	237.4	642.6	7010.1	37.9
9525	3.99	245	2058.0	759.5	1225.0	3010.6	232.3	628.6	7914.0	32.3
5535	3.99	143	1201.2	443.3	715.0	2542.3	196.1	530.8	5628.8	39.4
1545	4.245	272	2284.8	843.2	1360.0	3976.0	306.8	830.1	9600.9	35.3
-2700										
Partial sum [KN]		1945.0	15924.6	5909.7	9725.0	30135.1	2325.0	6291.9		
								Total weight P=	70311.3	KN

1) *Total building*

Our best estimate of the total gravitational loads (NW + NC + NE) acting on the building is:

Self-weight and permanent loads	G =	213,599 KN	(1)
Imposed loads	Q =	73,387 KN	(2)
Total weight (1) + (2)	G+Q =	286,986 KN	

The best estimate of the total seismic loads (NW + NC + NE) acting on the building is computed considering the total self-weight and permanent loads and the 36% of the imposed loads.

Self-weight and permanent loads	G =	213,599 KN	(3)
Imposed loads	0.36 Q =	26420 KN	(4)
Total weight (3) + (4)	G+0,36 Q =	240,019 KN	

4.3 CONCEPTUAL AND STRUCTURAL DEFICIENCIES UNDER SEISMIC ACTION

Upon a first examination, the conceptual design of the current structure seems to be sound and well-suited to resist horizontal actions, with a central pendular steel structure transmitting shear forces to two strong lateral concrete towers.

After a more careful consideration, however, it is noted that:

- The steel structure is quite irregular and not properly braced; the floors are discontinuous, hanging like horizontal cantilevers, with consequent difficulties in transmitting horizontal forces (§ 4).
- The shear capacity of the concrete cores, calculated as their bending moment capacity divided by some "*equivalent building height*," i.e. by the height of the point of application of the resultant of the horizontal forces, exceeds 20% of the total weight of the building (§ 6.5).
- However, the concrete towers are also characterized by a very irregular distribution of openings, in some cases inhibiting any possible force path to the foundations (§ 4).
- Some very large openings, particularly at the ground level, diminish the shear capacity of the walls to levels that are lower than 50% of the capacity derived from the bending strength, estimated assuming minimum reinforcement percentages (§ 6.5).
- As a consequence, a brittle collapse mode is predicted, which runs contrary to any sound structural design concept, particularly when considering seismic loads.
- Torsional effects are relevant, as demonstrated by the participation mass in the first two torsional modes (20 – 30 %, fig. 15).

In conclusion, though while it is not questioned that the structure respects all applicable codes of practice, it could be defined as "*non-resilient,*" i.e. incapable of sustaining any unforeseen event like seismicity.

5 WIND ACTIONS

The wind is the major horizontal action considered in the original "non-seismic" design. However, a significant fraction of the horizontal shear action is due to the leaning towers and to the mass eccentricity, which induce bending moments and consequently horizontal forces. However, as a first approximation, the comprehension of the performance of the building under wind load represents the starting point for the improvements required due to the seismic effects.

The wind load has been estimated according to the Dutch National Code NEN 6702. This is consistent with the design choices of the original project and is essential for the evaluation of the loads for which the building was designed.

5.1 ESTIMATE OF TOTAL SHEAR

The wind load is computed as following:

$p_{rep} = C_{dim} * C_{index} * C_{eq} * \phi_1 * p_w$ [KN/m^2]

where:

C_{dim}	is the factor that takes into account the dimensions of structure
C_{index}	is the wind shape factor (trend of pressure on surfaces)
C_{eq}	is the pressure equalization factor
ϕ_1	is the amplification factor for dynamic effects
p_w	is the maximum dynamic pressure

The area parameters are:

Zone:	II
Environment:	Uncultivated
Height above the ground-line:	45m

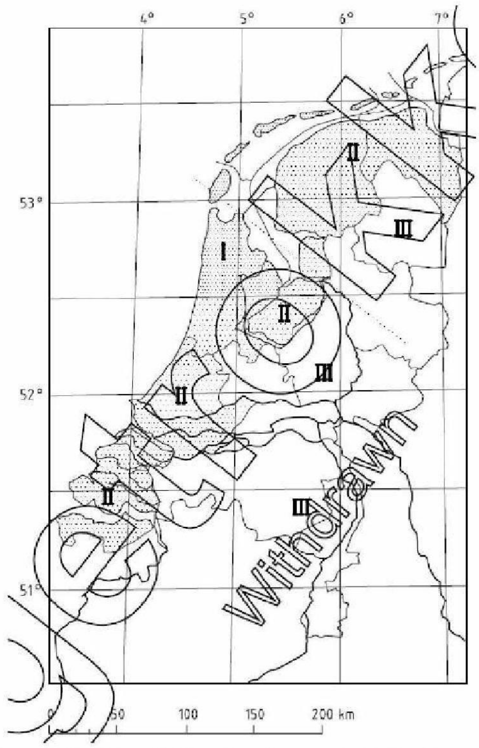


Figure 9 – The Netherland area distribution

The following tables were used for the evaluation of each parameter, in order to estimate the wind load acting on the building.

PR.: [redacted]	CK.: [redacted]	AP.: [redacted]

Table 12 – Wind-induced dynamic pressure as a function of the height

h m	*p*w kN/m²					
	Gebied I		Gebied II		Gebied III	
	onbebouwd	bebouwd	onbebouwd	bebouwd	onbebouwd	bebouwd
≤ 2	0,64	0,64	0,54	0,54	0,46	0,46
3	0,70	0,64	0,54	0,54	0,46	0,46
4	0,78	0,64	0,62	0,54	0,49	0,46
5	0,84	0,64	0,68	0,54	0,55	0,46
6	0,90	0,64	0,73	0,54	0,59	0,46
7	0,95	0,64	0,78	0,54	0,63	0,46
8	0,99	0,64	0,81	0,54	0,67	0,46
9	1,02	0,64	0,85	0,54	0,70	0,46
10	1,06	0,70	0,88	0,59	0,73	0,50
11	1,09	0,76	0,91	0,64	0,76	0,54
12	1,12	0,81	0,94	0,68	0,78	0,58
13	1,14	0,86	0,96	0,72	0,80	0,61
14	1,17	0,90	0,99	0,76	0,82	0,64
15	1,19	0,94	1,01	0,79	0,84	0,67
16	1,21	0,98	1,03	0,82	0,86	0,70
17	1,23	1,02	1,05	0,85	0,88	0,72
18	1,25	1,05	1,07	0,88	0,90	0,75
19	1,27	1,08	1,09	0,90	0,91	0,77
20	1,29	1,11	1,10	0,93	0,93	0,79
25	1,37	1,23	1,18	1,03	1,00	0,88
30	1,43	1,34	1,24	1,12	1,06	0,95
35	1,49	1,43	1,30	1,20	1,11	1,02
40	1,54	1,50	1,35	1,26	1,15	1,07
45	1,58	1,57	1,39	1,32	1,19	1,12
50	1,62	1,62	1,43	1,37	1,23	1,16
55	1,66	1,66	1,46	1,42	1,26	1,20
60	1,69	1,69	1,50	1,46	1,29	1,24
65	1,73	1,73	1,53	1,50	1,32	1,27
70	1,76	1,76	1,56	1,54	1,34	1,31
75	1,78	1,78	1,58	1,57	1,37	1,33
80	1,81	1,81	1,61	1,60	1,39	1,36
85	1,83	1,83	1,63	1,63	1,41	1,39
90	1,86	1,86	1,65	1,65	1,43	1,41
95	1,88	1,88	1,68	1,68	1,45	1,44
100	1,90	1,90	1,70	1,70	1,47	1,46
110	1,94	1,94	1,74	1,74	1,51	1,50
120	1,98	1,98	1,77	1,77	1,54	1,54
130	2,01	2,01	1,80	1,80	1,57	1,57
140	2,04	2,04	1,83	1,83	1,60	1,60
150	2,07	2,07	1,86	1,86	1,62	1,62

Table 13 – Evaluation of factor C_{dim} as a function of height "h" and width "b" of the building

h m	*b* m							
	1	10	20	30	40	50	75	100
2	1,00	0,96	0,94	0,92	0,90	0,89	0,86	0,84
3	1,00	0,96	0,94	0,92	0,91	0,89	0,87	0,85
4	0,99	0,96	0,94	0,92	0,91	0,89	0,87	0,85
5	0,99	0,96	0,94	0,92	0,91	0,89	0,87	0,85
6	0,99	0,96	0,94	0,92	0,91	0,90	0,87	0,85
7	0,99	0,95	0,93	0,92	0,91	0,90	0,87	0,85
8	0,98	0,95	0,93	0,92	0,91	0,90	0,87	0,86
9	0,98	0,95	0,93	0,92	0,91	0,89	0,87	0,86
10	0,98	0,95	0,93	0,92	0,91	0,89	0,87	0,86
11	0,98	0,95	0,93	0,92	0,90	0,89	0,87	0,86
12	0,98	0,95	0,93	0,92	0,90	0,89	0,87	0,86
13	0,97	0,95	0,93	0,92	0,90	0,89	0,87	0,86
14	0,97	0,95	0,93	0,91	0,90	0,89	0,87	0,86
15	0,97	0,95	0,93	0,91	0,90	0,89	0,87	0,86
16	0,97	0,94	0,93	0,91	0,90	0,89	0,87	0,86
17	0,97	0,94	0,93	0,91	0,90	0,89	0,87	0,86
18	0,97	0,94	0,93	0,91	0,90	0,89	0,87	0,86
19	0,97	0,94	0,92	0,91	0,90	0,89	0,87	0,86
20	0,97	0,94	0,92	0,91	0,90	0,89	0,87	0,86
25	0,96	0,94	0,92	0,91	0,90	0,89	0,87	0,86
30	0,96	0,93	0,92	0,91	0,90	0,89	0,87	0,86
35	0,95	0,93	0,91	0,90	0,89	0,89	0,87	0,85
40	0,95	0,93	0,91	0,90	0,89	0,88	0,87	0,85
45	0,94	0,92	0,91	0,90	0,89	0,88	0,87	0,85
50	0,94	0,92	0,91	0,90	0,89	0,88	0,86	0,85
55	0,94	0,92	0,90	0,89	0,89	0,88	0,86	0,85
60	0,93	0,92	0,90	0,89	0,88	0,88	0,86	0,85
65	0,93	0,91	0,90	0,89	0,88	0,88	0,86	0,85
70	0,93	0,91	0,90	0,89	0,88	0,87	0,86	0,85
75	0,93	0,91	0,90	0,89	0,88	0,87	0,86	0,85
80	0,92	0,91	0,89	0,89	0,88	0,87	0,86	0,85
85	0,92	0,90	0,89	0,88	0,88	0,87	0,86	0,85
90	0,92	0,90	0,89	0,88	0,88	0,87	0,86	0,84
95	0,92	0,90	0,89	0,88	0,87	0,87	0,85	0,84
100	0,91	0,90	0,89	0,88	0,87	0,87	0,85	0,84
110	0,91	0,90	0,89	0,88	0,87	0,86	0,85	0,84
120	0,91	0,89	0,88	0,87	0,87	0,86	0,85	0,84
130	0,90	0,89	0,88	0,87	0,87	0,86	0,85	0,84
140	0,90	0,89	0,88	0,87	0,86	0,86	0,85	0,84
150	0,90	0,88	0,87	0,87	0,86	0,86	0,85	0,84

Table 14 – Evaluation of factor shape factor C_{pe}

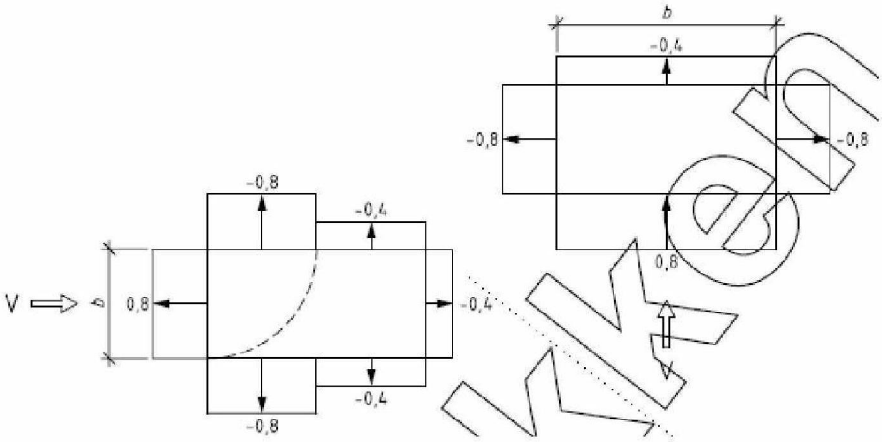
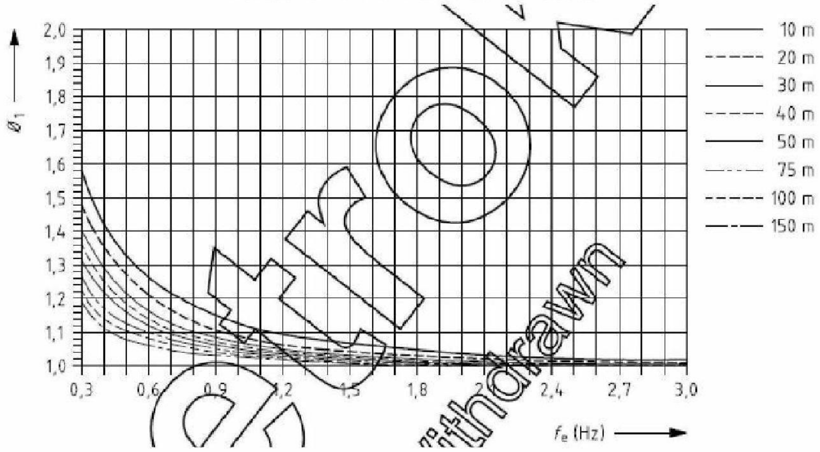


Table 15 – Evaluation of factor ϕ_1



The factors that determine the wind pressure are summarized below and in Table 16.

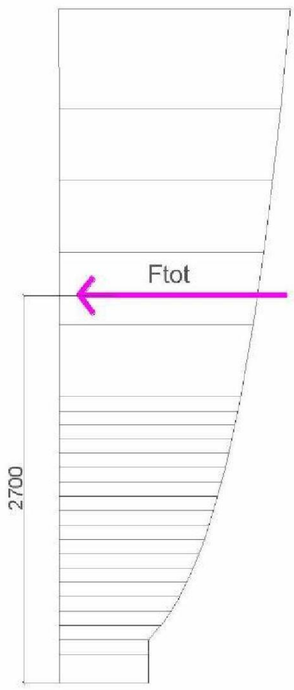
Width of the building	b= 79 m
Average building height	h = 47 m
Dimension factor	C_{dim}=0.87
Shape factor	C_e = 0.8+0.4 = 1.2 (windward + leeward)
Pressure equalization factor	C_{eq}=1
Dynamic factor	ϕ_1=1.1

Table 16 – Wind load distribution

z [m]	I(z)	vw(z)	pw(z)	Cdim	Φ1	Ceq	Ce (sop)	Ce (sot)	Ce (tot)
0									
2	0.434	13.240	0.543	0.870	1.100	1.000	0.800	0.40	1.2
3	0.369	15.571	0.543	0.870	1.100	1.000	0.800	0.40	1.2
4	0.334	17.225	0.619	0.870	1.100	1.000	0.800	0.40	1.2
5	0.311	18.509	0.680	0.870	1.100	1.000	0.800	0.40	1.2
6	0.294	19.557	0.731	0.870	1.100	1.000	0.800	0.40	1.2
7	0.281	20.443	0.775	0.870	1.100	1.000	0.800	0.40	1.2
8	0.271	21.211	0.815	0.870	1.100	1.000	0.800	0.40	1.2
9	0.263	21.888	0.850	0.870	1.100	1.000	0.800	0.40	1.2
10	0.256	22.494	0.882	0.870	1.100	1.000	0.800	0.40	1.2
11	0.250	23.042	0.911	0.870	1.100	1.000	0.800	0.40	1.2
12	0.244	23.542	0.939	0.870	1.100	1.000	0.800	0.40	1.2
13	0.240	24.003	0.964	0.870	1.100	1.000	0.800	0.40	1.2
14	0.235	24.429	0.988	0.870	1.100	1.000	0.800	0.40	1.2
15	0.232	24.826	1.010	0.870	1.100	1.000	0.800	0.40	1.2
16	0.228	25.197	1.031	0.870	1.100	1.000	0.800	0.40	1.2
17	0.225	25.545	1.050	0.870	1.100	1.000	0.800	0.40	1.2
18	0.222	25.874	1.069	0.870	1.100	1.000	0.800	0.40	1.2
19	0.220	26.185	1.087	0.870	1.100	1.000	0.800	0.40	1.2
20	0.217	26.480	1.104	0.870	1.100	1.000	0.800	0.40	1.2
25	0.207	27.763	1.180	0.870	1.100	1.000	0.800	0.40	1.2
30	0.200	28.811	1.244	0.870	1.100	1.000	0.800	0.40	1.2
35	0.194	29.698	1.298	0.870	1.100	1.000	0.800	0.40	1.2
40	0.189	30.465	1.346	0.870	1.100	1.000	0.800	0.40	1.2
47	0.183	31.393	1.406	0.870	1.100	1.000	0.800	0.40	1.2

The resulting wind shape is thus as follows, where the *z* coordinates are measured from the top of the building.

z [m]	Prep [KN/mq]
0	
2	0.624
3	0.624
4	0.711
5	0.781
6	0.840
7	0.891
8	0.936
9	0.976
10	1.013
11	1.047
12	1.078
13	1.107
14	1.134
15	1.160
16	1.184
17	1.206
18	1.228
19	1.249
20	1.268
25	1.355
30	1.428
35	1.491
40	1.546
47	1.614



The total wind load is estimated as F_v = 4647 KN and the height of application of the equivalent static force is H≈27m (total building height = 47 m).

At the ultimate limit state the amplified value is computed as $\mathbf{F_{vu}}$ = 1.5*4,647 = 6,970.5 KN ≈ **7,000 KN.**

5.2 PERFORMED ANALYSES

The analyses of the building under wind loads have been performed using the available SAP model. The concrete cores (west and east) are modelled as “shell” elements while the central steel system is defined by “frame” elements. The finite element model represents the above-ground structure and the base constraints are modelled by elastic springs (concentrated and distributed) in order to take into account the flexibility of the beams that support several bearing structures (discontinuous structural system). The elastic modulus of concrete is reduced in order to take into account cracking.

As before, the structural systems are referred to as NW for the west concrete core, NC for the central steel structure and NE for the east concrete core.

The wind pressure is applied in the direction perpendicular to the front of the building (b = 79 m with Ce = 1.2). The total load is applied to the three cores depending on the related exposed surfaces; wind actions are applied either as uniformly distributed or as a lumped force every 5 m.

The following figures describe the simplified scheme of application of loads.

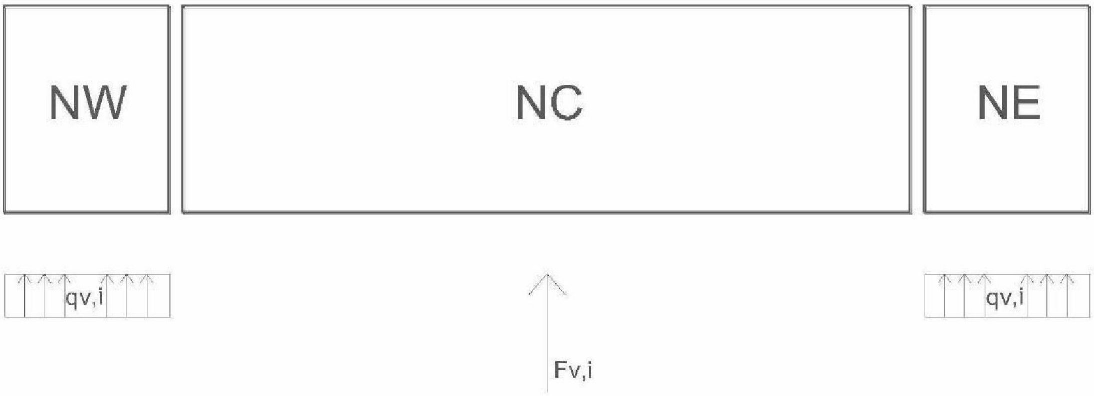


Figure 10 – Application of wind load: plan view

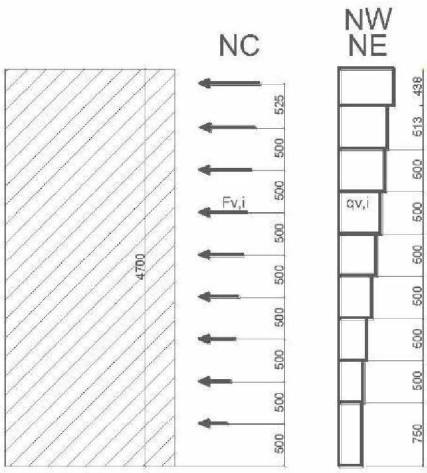


Figure 11 – Application of wind load: elevation view

5.3 TRANSFORMATION INTO “BEST ESTIMATE”

The basic purpose of the numerical analysis is to estimate the distribution of shear reactions between the three cores NW, NC and NE; the results are as follows:

NW = 46%	→	Fw = 7,000 * 0.46	= 3,220 KN
NC = 17 %	→	Fc = 7,000 * 0.17	= 1,190 KN
NE = 37 %	→	Fe = 7,000 * 0.37	= 2,590 KN

Considering the height of application of the equivalent total forces, the estimate of the reacting bending moment for each core is:

M_{NW} = 3220 kN * 27 m = 86940 KN m
M_{NC} = 1190 kN * 27 m = 32130 KN m
M_{NE} = 2590 kN * 27 m = 69930 KN m

5.4 TRANSFORMATION INTO % OF WEIGHT

Defining G and Q respectively the "self-weight + permanent load" and the "imposed load", the seismic mass is computed as G+0.36Q.

The wind loads for each cores can thus be re-calculated as a percentage of the total seismic weight of the building and the total weight of each core (see Table 17).

Table 17 – Wind actions expressed as percentages of total and element weights

				$\%_{E,total}$	$\%_{E,i}$
NW	Fw=	3220	KN	1.34%	7.40%
NC	Fc=	1190	KN	0.50%	0.90%
NE	Fe=	2590	KN	1.08%	4.04%
	TOTAL			2.92%	

5.5 RESERVE OF CAPACITY

In order to obtain a preliminary sense of potential critical issues, two load combinations have been considered, as follows:

According to Eurocode

a) 1.2*G+1.5*Q+(1.5*0.6)*Qv (gravity live load governing)
b) 1.2*G+(1.5*0.7)Q+1.5*Qv (wind load governing)

According to Dutch regulation

a) 1.2*G+1.5*Q (gravity live load governing)
b) 1.2*G+(1.5*0.25)Q+1.5*Qv (wind load governing)

in which Qv is the wind load calculated according the procedure described in chapter 5.

The performed analyses have allowed an estimate of the approximate distribution of the wind action (see the next section) between the resisting systems.

The following considerations apply to the concrete cores:

- The extensive openings at the ground level induce local stresses that exceed 20 MPa. This value should be carefully approached in terms of local ductility of the structure.
- The geometry of the ground floor openings is in general incompatible with the basic principles of any seismic design, regardless of demand and capacity evaluations.
- The irregularity of the opening distribution induces high local stresses, which require careful reinforcement detailing.
- The presence of highly loaded vertical elements supported on beams requires careful local analyses and may induce dynamic problems not necessarily made evident by the model.
- The shear walls in Y direction at block NE have a high work ratio (3,000kN demand).

6 SEISMIC ACTIONS

6.1 DEFINITION OF THE DESIGN GROUND MOTION

In absence of a reliable site hazard analysis, the following considerations and the following conclusions and decisions are based on the provisions of the following documents:

- NPR9998 "Assessment of buildings in case of erection, reconstruction and disapproval – Basic rules for seismic actions; Induced earthquake"
- EN1998-1 "Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings"

Both documents are based on a definition of acceleration spectra as specified by the following equations:

$$0 \le T < T_B \qquad S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot F_0 - 1)\right]$$

$$T_B \le T < T_C \qquad S_e(T) = a_g \cdot S \cdot \eta \cdot F_0$$

$$T_C \le T < T_D \qquad S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left(\frac{T_C}{T} \cdot\right)$$

$$T_D \le T \qquad S_e(T) = a_g \cdot S \cdot \eta \cdot F_0 \cdot \left(\frac{T_C T_D}{T^2}\right)$$

NPR and EC indicate to adopt the following parameters:

Table 18 – Elastic spectrum parameters for NPR9998 AND EC8

	NPR9998	EC8
S	1	1
η	$\sqrt{10/(5+\xi)} \ge 0.55$	$\sqrt{10/(5+\xi)} \ge 0.55$
F_0	3	2.5
T_B	0.10	0.15
T_C	0.22	0.40
T_D	0.45	2.00

Since in the NPR9998 the peak ground acceleration is given on surface level, in the case of the Eurocode spectrum the parameters related to soil A are considered in order to make a comparison.

6.2 ACCELERATION AND DISPLACEMENT SPECTRA FROM NPR9998 AND EC8

The Netherlands have not yet issued specific legislation on seismic actions and design. The NPR 9998 "Assessment of buildings in case of erection, reconstruction and disapproval – Basic rules for seismic actions; Induced earthquake" is being developed and a first version was distributed in February 2015 for peer review. It is expected that these rules will be the precursor to a Dutch National Annex for Eurocode 8.

The NPR9998 elastic spectrum is calculated adopting the designer's hypotheses as shown below:

- Importance factor: $\gamma_I = 1.6$
- Peak ground acceleration $a_{g,ref} = 0.24$ g

The resulting acceleration and displacement response spectra are defined in the following figures. The pale blue dashed line represents the NPR basic spectrum; the grey line represents the spectrum multiplied by the importance factor.

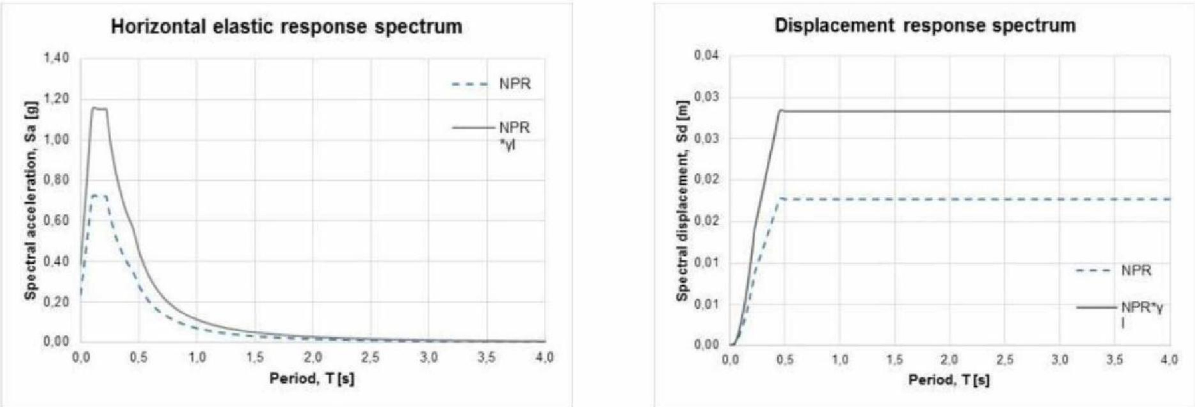


Figure 12 – Acceleration and displacement elastic spectra from NPR9998

Comparing the NPR spectrum with the corresponding Eurocode 8 spectrum (see Figure 13), it appears that:

- The spectral shape of NPR is rather narrow, being more similar to EC8 spectral shapes recommended for low seismicity zones (type 2 in EC8), but the peak ground acceleration (a_g = 0.24 g) is comparable to those characterizing high seismicity zones.
- The NPR spectrum shape is characterized by a very low value of the corner period T_D (T_D = 0.45 sec in NPR versus T_D = 2.0 sec in EC8), which is even significantly lower than that recommended period for type 2 spectra (T_D = 1.20 sec).
- The NPR spectrum adopts a high amplification factor (3.0, against 2.5 recommended by EC8), but considering the limited width of the plateau (Tc = 0.22 sec) compared with the fundamental periods of the building (around 1 sec), this high amplification is likely to have a limited (possibly negligible, depending on the relevance of higher modes) effect on the response.

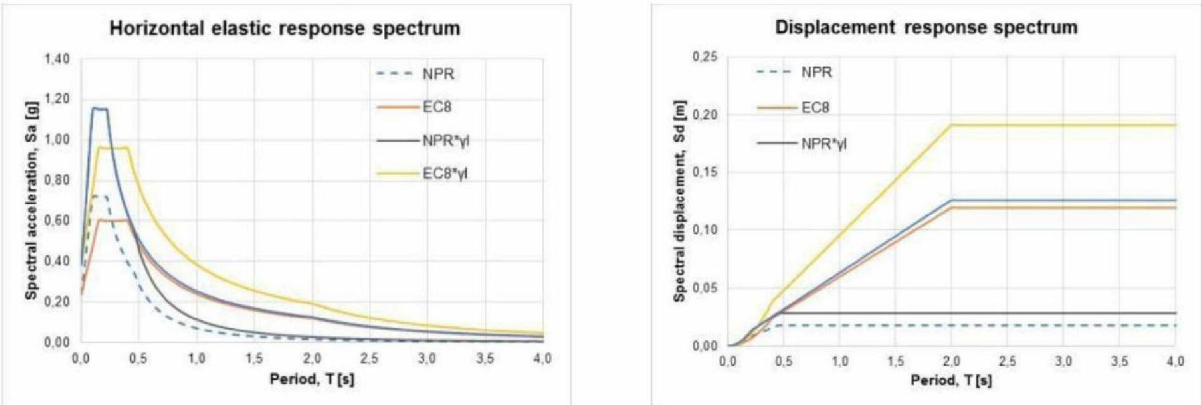


Figure 13 – Comparison between acceleration and displacement elastic spectra

In the absence of a specific local hazard analysis, it has been decided to modify the NPR spectral shape considering the corner period suggested by EC8 (T_D = 2.0 sec).

This results in a correction of the spectral ordinates only for period values larger than 0.45 sec, as shown in Figure 13, where it appears that in this period range the amplified (by the importance factor) NPR spectrum corresponds essentially to the basic (unamplified) EC8 spectrum.

It is also evident that the proposed correction affects the displacement demand to a much larger extent than the acceleration spectra ordinates.

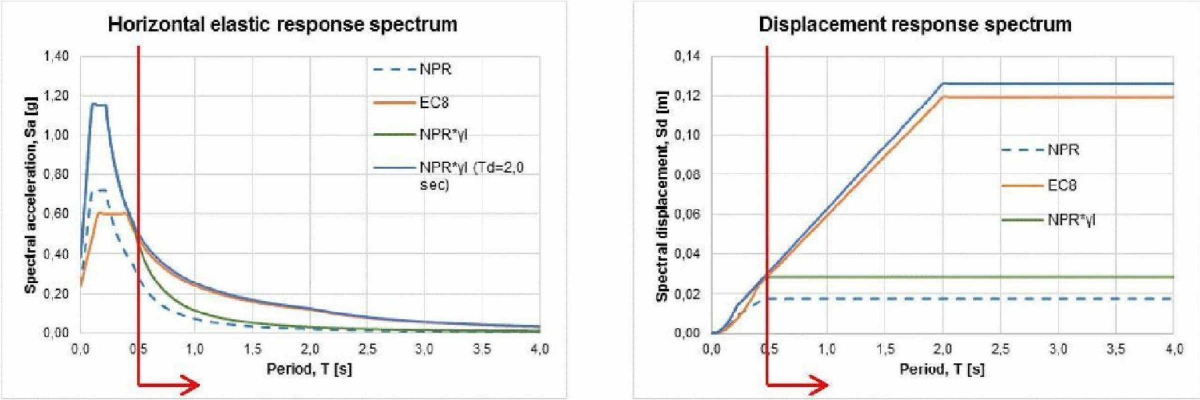


Figure 14 – Comparison between acceleration and displacement elastic spectra. Red line: EC8 basic; pale blue dashed line: NPR basic; green yellow line: importance factor amplified NPR; dark blue: adopted spectrum

6.3 EVALUATION OF THE PERIOD OF VIBRATION OF THE STRUCTURE

From an analysis of the SAP model provided, it appears that the properties of the elastic spring elements used to simulate the effect of the substructure (and possibly soil-structure interaction) govern the dynamic response, determining the values of the relevant periods of vibration. As a consequence, some of the following notes may be severely biased. The sensitivity of the model to the properties of spring elements representing complex phenomena, not necessarily fully understood and properly verified, represents a serious problem, at minimum requiring some sensitivity analyses with a discussion of the implications of lower and upper bound solutions.

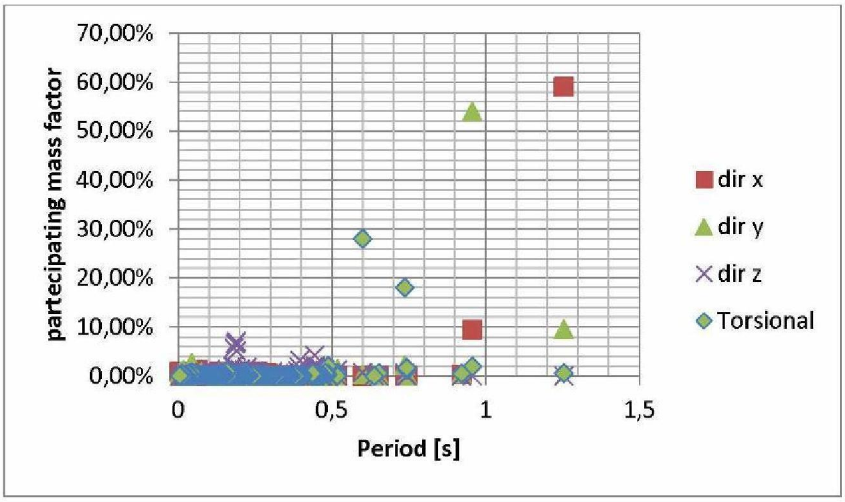


Figure 15 – Participating mass factor according modal analysis of the existing model considering an elastic equivalent stiffness

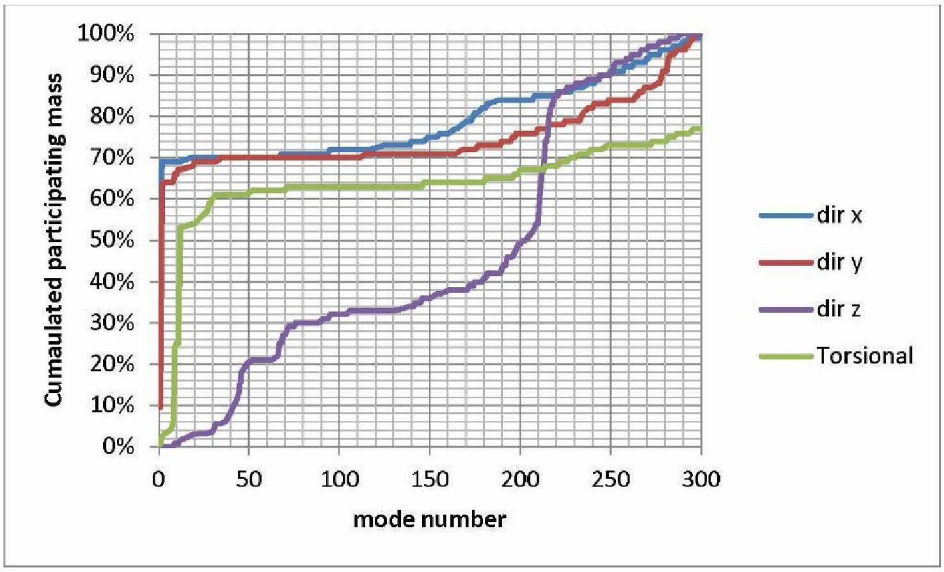


Figure 16 – Participating mass factor according modal analysis of the existing model considering an elastic equivalent stiffness

The model may not allow a proper estimate of the governing vertical modes, if any. The inclined shape of the concrete cores and the discontinuity of some vertical bearing elements may amplify the effects of the vertical excitation.

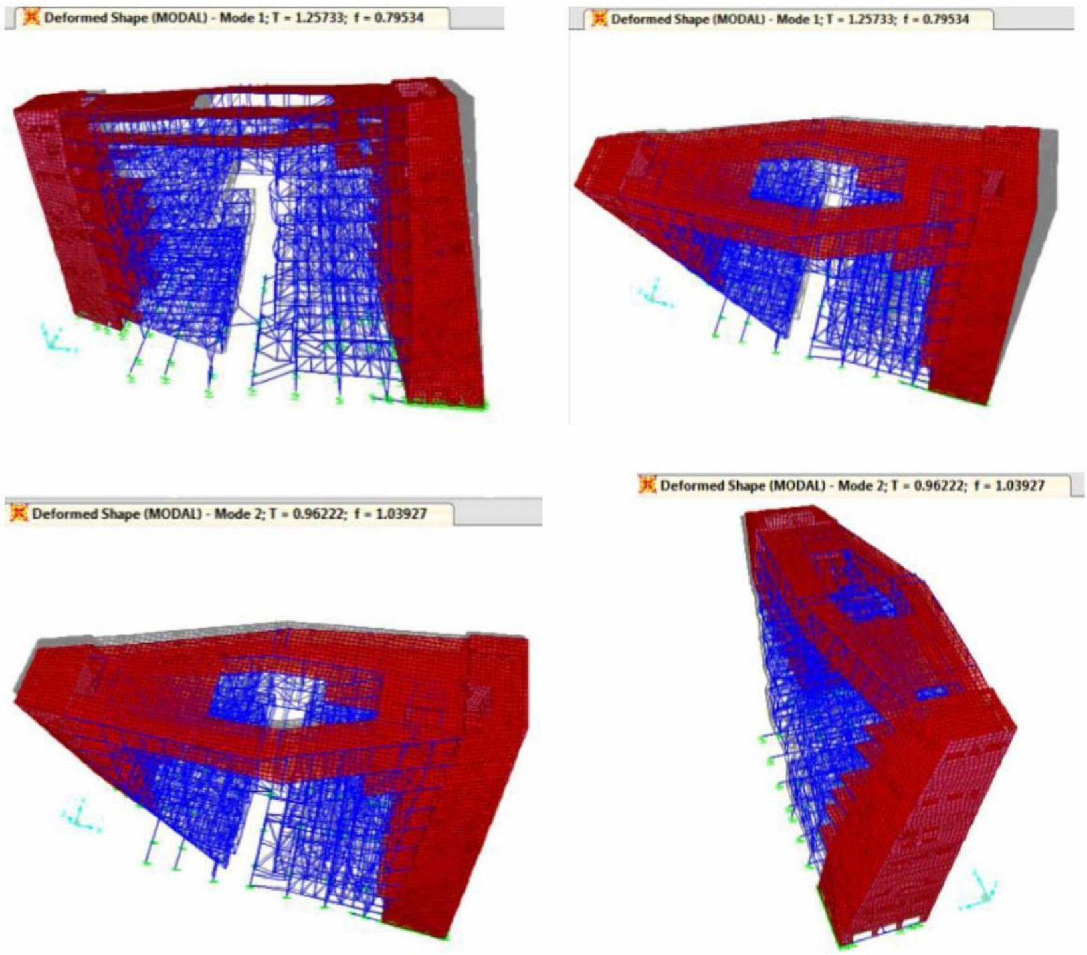


Figure 17 – deformed shape of the 1st mode and of the 2nd mode

6.4 EVALUATION OF THE YIELD DISPLACEMENT OF THE STRUCTURE

The yield curvature for a C-shape concrete wall has been estimated as recommended in Annex 1 of the DBD12 Model Code:

$$\phi_y = \frac{1.4\varepsilon_y}{l_w}$$

where:

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{500}{200000} = 0.0025\frac{m}{m}$$

Considering $l_1 = 30\,m$; $l_2 = 20\,m$ (these values are just samples to show some possible outcome, all walls depths have been actually considered):

$$\phi_{y,1} = \frac{1.4 \cdot 0.0025}{30} = 1.166E^{-4}\frac{1}{m}; \phi_{y,2} = \frac{1.4 \cdot 0.0025}{20} = 1.75E^{-4}\frac{1}{m}$$

The yield displacements at the equivalent centre of mass are calculated as:

$$\Delta_{y,i} = \frac{\phi_{y,i} \cdot H_{eq}}{3} = \frac{\phi_{y,i} \cdot 0,67 \cdot 47}{3}$$

$$\Delta_{y,1} = 0.0385\,m\,; \Delta_{y,2} = 0.0578\,m$$

PR.: 5.1.2e	CK.: 5.1.2e	AP.: 5.1.2e

Considering a total yielding shear of 0.2 g:

$T_{y,1} = 0.88\,s\ ;\ T_{y,2} = 1.08\,s$

Clearly, shorter walls will end up showing longer periods of vibration.

Torsional effects have not been considered in this context.

6.5 POTENTIAL DEMAND AND COMPARISON WITH CAPACITY

6.5.1 Shear associated with flexural capacity

One of the main principles of good practice in seismic design is that "capacity design" should be used to ensure that brittle failure modes are suppressed. In practice, this means that walls should be designed such that their shear strength is higher than the shear demand that would cause flexural yielding. A sense of the potential base shear capacity of the structure can therefore be obtained by estimating the flexural strength of the cores.

It is assumed here that the strength of the cores may be estimated by considering only the walls acting in-plane. The flexural strength of each of the walls may be calculated independently using nonlinear moment-curvature analysis; the program Response-2000 has been used for this purpose. From the design documents, the reinforcement in each wall has been approximated as 0.4% longitudinally. Yield strength of 500 MPa has been assumed. A sample output from Response-2000 is shown in Figure 18. The equivalent yield curvatures resulting from these section non-linear analyses are consistent with the approximate calculations discussed in the previous section.

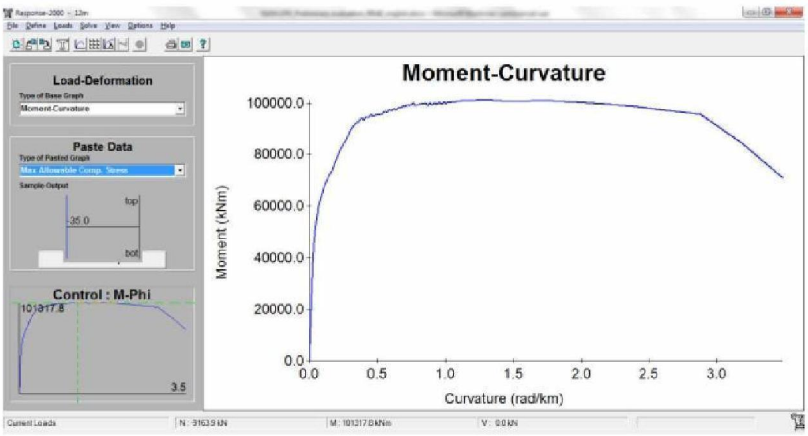


Figure 18 – Sample Response-2000 moment curvature analysis, for the case of 12 m deep wall

Knowing the flexural strength of each core, one may approximate the corresponding shear at the base of the core from statics. Although the actual distribution of shears will vary along the height, as a first approximation, it may be assumed that the effective location of loading is at two thirds of the height. Therefore, the shear corresponding to flexural yield is given by equilibrium as:

$$V_y = \frac{M_y}{0.67 \times H}$$

The resulting flexural and shear capacities for each of the cores (obtained as a combination of the results obtained for each wall) is given below.

Core	Direction	Moment Capacity [kNm]	Corresponding Shear Capacity [kN]	% of Seismic Weight [%]
NW	EW	202636	6435	2.7%
	NS	629294	19984	8.5%
NE	EW	202636	6435	2.7%
	NS	1087484	34534	14.7%

Considering some of the shear distributions obtained by the designers, an upper-limit condition could be considered:

$$V_y = \frac{M_y}{0.8 \times H}$$

Core	Direction	Moment Capacity [kNm]	Corresponding Shear Capacity [kN]	% of Seismic Weight [%]
NW	EW	202636	5389	2.26%
	NS	629294	16737	7.12%
NE	EW	202636	5389	2.26%
	NS	1087484	28922	12.31%

It can be seen that the structure has considerable potential to carry large base shears (23.1% of the seismic weight in the stronger direction), even if a minimum reinforced percentage has been assumed. These capacities can be largely increased with reasonable modifications of the flexural reinforcement.
However, this is contingent on the proper seismic shear design of the walls. If they are improperly detailed and have an actual shear capacity lower than this value, the available flexural capacity of the walls is "not fully exploited." Therefore, it is necessary to check the shear strength of the walls taking into account openings and other variations in geometry.

6.5.2 Shear capacity check

As described above, a sound application of capacity design principles, the walls should be designed to be able to form a plastic hinge before failing in shear. The most critical walls for shear will be those with large openings near ground level. In particular, the N-S walls of the west tower are very vulnerable to shear failure, since only a fraction of the total wall length continues all the way to the foundations.

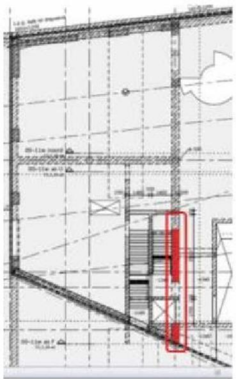


Figure 19 – Critical N-S wall in west core

This case will be thus considered here, as a typical case in which a reduced shear capacity may

PR.: 5.1.2e	CK.: 5.1.2e	AP.: 5.1.2e

jeopardize the response and the capacity of the element and of the whole system under seismic actions.

One method of analyzing the behaviour of a disturbed region such as a wall with openings is by a strut and tie model. The flow of forces from the confines of the area under consideration is considered explicitly through "struts" of concrete and "ties" of reinforcement. Such a model of the region illustrated in Figure 20 was used to assess the shear capacity of wall 4. Calculations were performed according to the CAN/CSA-A23.3-04 standard, with shear and flexural tension applied to the top nodes of the model.

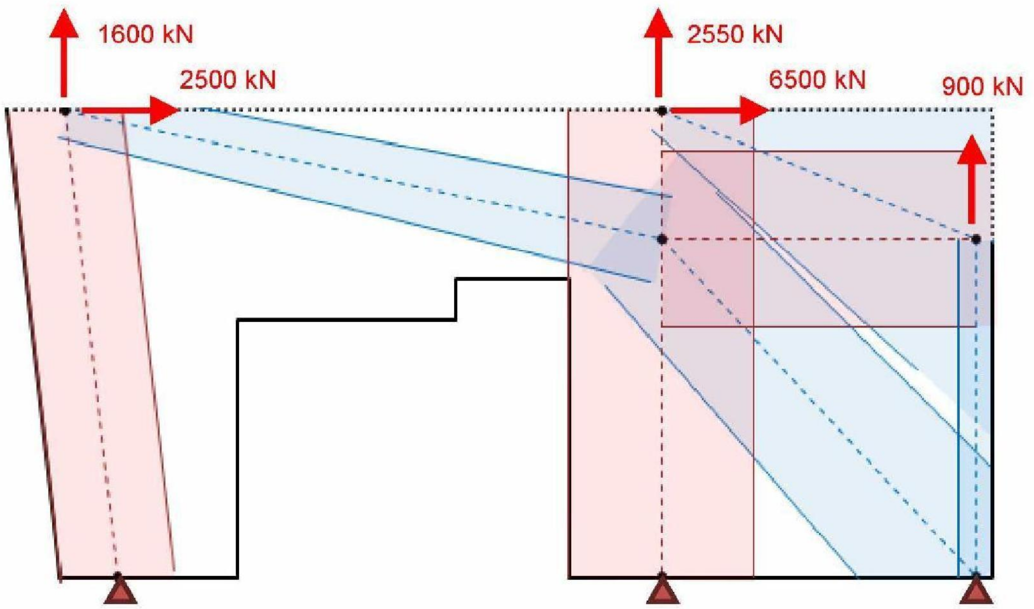


Figure 20 – Strut (blue) and tie (red) model of wall 4, showing maximum load pattern

Using the strut and tie methodology and assuming a concrete compressive strength of 50 MPa, a maximum shear capacity of 9,000 kN was attained for the critical wall in the west core. This is approximately 50% (from 45% to 54% depending on the height of application of shear) of the shear corresponding to the maximum flexural capacity of the core, confirming that it was not originally designed with seismic principles in mind, but as well that the wall response does not conform to sound structural design principles. The reinforcement ratios required to transmit the tension in the ties are also quite high: 4% vertically and 1% horizontally.

This shear verification indicates some potential problem in some element and point to the need of considering the possibility of some structural improvements regardless of the seismic action.

Since the core walls appear to be under-designed for shear with respect to the flexural capacity, it is recommended to consider the option of strengthening them regardless of the intervention chosen to upgrade the structure to seismic actions. Closing some of the openings could achieve this goal and ensure that brittle failure modes such as shear failure do not govern.

7 SOLUTIONS DISCUSSED BY THE DESIGNERS

This section has been developed on the basis of information provided to NAM by the designers of the building. No detailed information in English was available, and therefore no significant quantitative data can be analysed in details; however, in the following sections some considerations are provided on the proposed four solutions.

7.1 STRENGTHENING (SOLUTION 1)

This solution seems to be practical, although it is highly conditional on the possible variation of the seismic input which currently appears very inadequate with respect to contingent local hazard and amplification analysis.

The strengthening solution is a sort of remedial action for the irregularity of the building and for its inappropriateness for the horizontal response (i.e. closing of openings, increasing of shear and flexural resistance, etc.)

A final evaluation of this solution should consider some quantitative engineering parameters such as a detailed cost estimation and the safety factor of the solution.

Some favourable improvements of the original design could also be considered such as reduction of the masses, optimization of the reinforcement, and weakening of some parts of the core.

7.2 WEAK STOREY (SOLUTION 2)

A full application of this interesting approach requires that the expected behaviour of the structure under the seismic load should be very regular. For the Groninger Forum building it seems that the architectural restrains are not consistent with this request of regularity.

In the consultant's opinion the full solution is not applicable in this case.

Some weakening of the cores could be useful to increase the local ductility but this intervention should be considered as a possible part of 7.1.

7.3 STEEL CORES (SOLUTION 3)

The nature of the proposed solution is not entirely clear. It seems that it is possible to substitute the RC cores or part thereof by a steel truss system. This solution could be effective and could probably overcome design restraints imposed by the already built irregular underground structure. It is not easy to determine the cost of this solution with the available data. This solution, if partially applied, should be considered an application of 7.1.

7.4 BASE ISOLATION (SOLUTION 4)

This solution is probably the most flexible in terms of reduction and control of the seismic input. As discussed in the executive summary, even if the use of base isolation is considered, minor adjustments in the design of should be expected due to the general inappropriateness of the structure for horizontal response. The inconveniences pointed out by the designer regarding this solution (i.e. time required for production, selection of input ground motion, guidelines, tests, etc.) seem to be overemphasized with respect to our experience.

The designers' comment about the cost of this solution is unclear. In the qualitative presentation the designers define base isolation as an expensive solution, but no detailed evaluation of the costs is available.

The technology to be adopted for base isolation could be in principle either based on curved friction sliders type or rubber bearing. From the preliminary analysis provided in Chapter 8, however, curved friction sliders have been adopted, since it is felt that a better control of the maximum shear could be achieved and time and cost could be reduced. In the case of the alternative solution implying partial isolation (§ 8.3), a combination of lubricated flat sliders and high damping rubber bearing has been used.

8 ALTERNATIVE SOLUTIONS

Two of the potential solutions presented in Section 7 (strengthening and complete base isolation) as well as one additional alternative solution (partial isolation) have been identified (by us) as the most favourable and attractive candidates for further development. These three solutions are described below, followed by a numerical comparison of the behaviour of each, conducted through non-linear time history (NLTH) analysis.

8.1 FIXED BASE (SOLUTION A)

The baseline solution to which the other alternatives may be compared involves strengthening the structure in order to allow it to carry the seismic forces to the ground without significantly altering the load path. The majority of inertial forces on the central steel structure (NC), therefore, would be transmitted in the same manner as wind loads; namely, carried through the floor slabs back to the concrete towers, to be resisted through shear and torsion of the cores. Owing to the presence of soft storeys at ground level, very little of the lateral force would be able to be transmitted directly from the steel structure to the foundation.

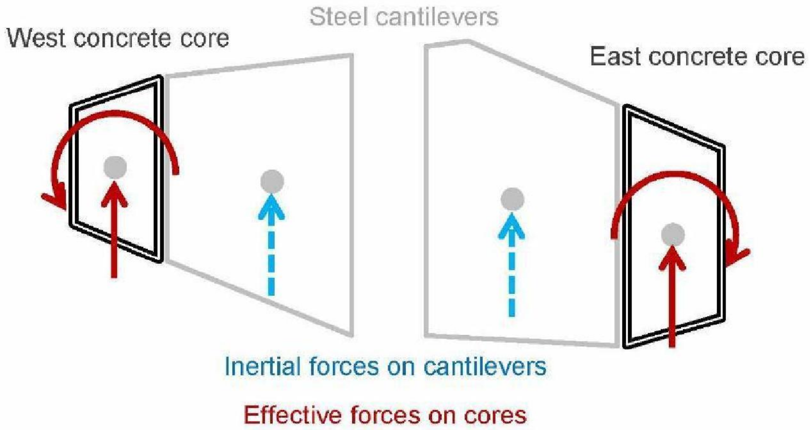


Figure 21 – Transmission of inertial forces from steel structure to concrete cores

This solution is conceptually simple, requiring no major structural reconfiguration, but will require a complete revision of the design, with extensive interventions on both the concrete cores and the steel structure.

As discussed and shown through simple calculation, the lateral loads due to seismic actions are expected to be considerably larger than the wind loads for which the lateral system was originally designed. In § 6.3 it was seen that the fundamental period of vibration of the structure is approximately 1 s. Using the acceleration spectrum presented in § 6.2, therefore, the expected lateral demand due to earthquake is on the order of 20% of the weight of the structure. This is significantly higher than the wind load, calculated in § 5.4 to be only 3% of the weight of the structure.

Again as discussed, a total shear force in the order of 20% of the weight could be easily sustained by the concrete towers alone, provided that the steel structure be made able to transmit its share of shear and that some existing weakening due to excessive and ill-distributed openings be closed or alternative provisions (local bracings) be adopted.

For this reason, a successful intervention of this nature would potentially require strengthening of every element in the load path. Floor diaphragms in the central steel section would need to be made rigid, so as to carry the lateral forces back to the cores. The cores themselves would need to be strengthened to resist the additional shear and torsion due to the lateral load. Finally, the transmission of these forces from the concrete towers to the foundations would need to be verified.

The details of the necessary strengthening cannot be discussed within the scope of this document, nor can a preliminary evaluation of the implied cost be made at this stage.

8.2 BASE ISOLATION (SOLUTION B)

As indicated in § 8.1, the baseline solution requires the structure to carry a total lateral force on the order of 20% of the building's weight; alternative solutions that reduces this ratio should be explored. Since the foundation of the structure has already been constructed, it may be necessary to reduce the total lateral load to a value limited by the strength of the existing foundations. For this purpose, base isolation is an efficient solution. Base isolation entails creating a very flexible isolation layer between the foundation and the ground storey of the structure, such that the structure is free to remain relatively still while the earth moves underneath it. The total lateral force is reduced through two means: an increase in the natural period of vibration of the system, and an increase in the effective damping of the system, both of which reduce the spectral acceleration. In addition, if the adopted isolation system shows a very clear equivalent yielding and a relatively flat second branch of the force – displacement curve, the transmitted shear results physically limited and the structure on one side and the foundations on the other are effectively capacity-protected.

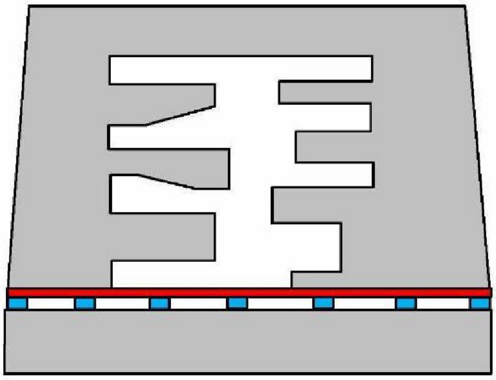


Figure 22 – Structural configuration with complete base isolation

At this preliminary stage, it is assumed that the displacement demand can be directly derived from the spectrum shown in Figure 14 and estimated as 125 mm, assuming that the isolated period will be well beyond the corner period value. This demand could be significantly reduced as a function of the provided equivalent damping, which could be in the range of 15% in the case of high damping rubber bearings and 30% in the case of curved friction sliders. This latter option is considered preferable for several reasons and would reduce the displacement demand to about 80 mm; however, a final decision will have to be made at a later stage in the case that base isolation is adopted.

For preliminary evaluation, a friction coefficient of about 4% is adopted (achievable with current devices), while a curvature inducing a force corresponding to 6% of the weight at 100 mm displacement will be considered (see Figure 23). Since the resulting actions will thus be of the same order as the wind loading (5% against 3% as a fraction of the weight), then if the foundation and superstructure were originally capable of sustaining the wind load, they should be able to carry this reduced seismic load as well without significant modifications. (Some improper detailing and inadequate distribution of stiffness will have to be adjusted regardless). An additional structural element that will greatly improve this solution is a diaphragm at ground level (be it a concrete slab or steel bracing) to connect the east and west towers (and the steel structure) together rigidly. Without such a connection, the towers could twist independently, possibly causing unexpected loading of the central steel structure.

In the case of complete isolation, further verification will be required to ensure that no uplift of the isolators will occur; the towers must be able to resist overturning without developing tension at their base. The inclined form of the concrete towers may also contribute to this overturning effect.

Based on a parametric application of previous experiences, it is preliminarily estimated that the cost of this measure should not exceed 1.5-2 million Euros (excluding technical services and adjustments of the

PR.: [redacted]	CK.: [redacted]	AP.: [redacted]

pipelines to allow the relative displacement at the isolation level). Further information will be provided in the "*Groninger Forum Building Feasibility study of a base isolation solution at the first underground level*" document, separately delivered. This cost does not include the interventions on the existing structure (which is questionable whether should be considered to be "*induced by seismic action*" rather than, at least partially, required by sound structural engineering principles) and all costs required by adjustments of services elements.

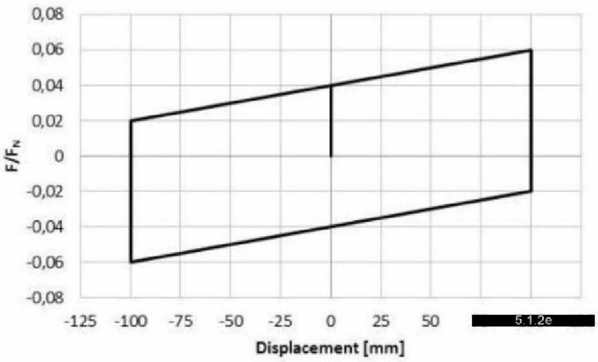


Figure 23 – Hysteresis loop for curved sliding devices

8.3 AN ALTERNATIVE SOLUTION (SOLUTION C)

Isolation of the entire superstructure is a significant departure from the original structural system of the building. Furthermore, it has been shown that the concrete tower could have a significant strength capacity, provided that some elements are modified and some detailing improved. Therefore, in lieu of a radical redesign such as full isolation, an intermediate intervention, referred to here as the "partial isolation solution," is offered for consideration.

More than 50% of the mass of the structure is located in the central steel structure (as calculated in § 4.2); therefore, significant reductions of base shear can be effected by isolating only this portion of the building. The concrete towers, on the other hand, may be built to be contiguous with the foundations, as originally designed. In this plan, points of isolation would need to be provided between the steel structure and the foundation, as well as between the steel structure and the adjacent towers. For the isolation layer at the base of the steel structure, a flat friction slider is an effective means of limiting the shear force while allowing the transmission of large gravity loads. Such flat friction sliders may limit the transmitted shear to about 1% of the weight (adopting lubricated sliding surfaces), thus being fully in line with the shear capacity required by the wind loading (see § 5.4).

On the other hand, at the interfaces between the upper steel storeys and those of the concrete cores, high damping rubber bearings could be used to transmit the vertical and horizontal loads, increasing the effective damping and reducing the relative displacement demand.

Since this solution proposes to isolate the steel structure from the concrete cores, only a minimal amount of lateral load will be transferred from the central section to the cores, perhaps of the same order of magnitude as the wind loading in the original design. This will help to reduce the burden on the concrete towers. At the same time, the central steel structure will now need to be able to transmit shear directly to its base, therefore the "soft storeys" must be eliminated and the entire steel structure stiffened to act more like a single rigid body. This solution will facilitate and reduce the interventions required by any installation or pipeline crossing the isolation level: any crossing inside the concrete towers (for example the elevators) will not require any intervention. Clearly, on the opposite, pipelines that will connect horizontally to concrete towers and to steel structures will have to be able to cope with the allowed relative movement. Implication on architectural and functional design could not be excluded.

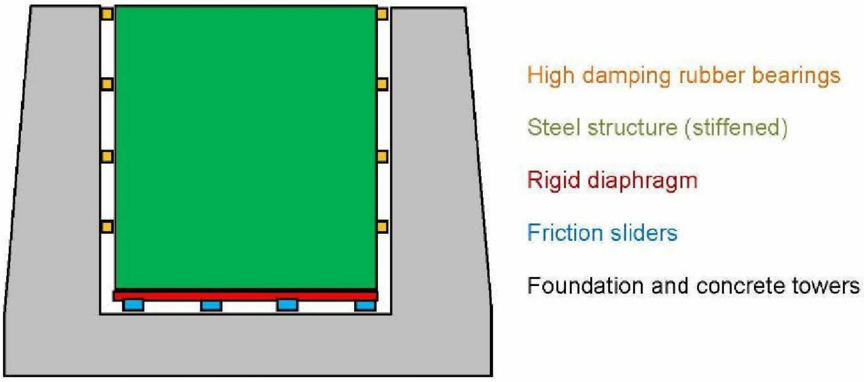


Figure 24 – Structural configuration with partial isolation

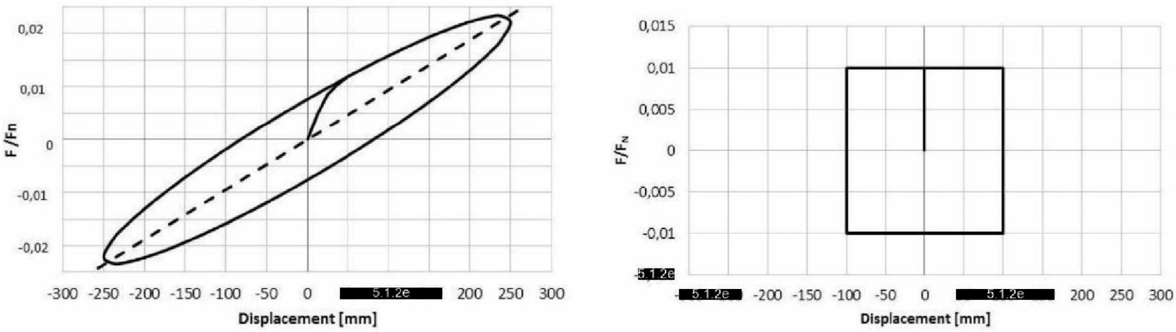


Figure 25 – Hysteresis loop for High Damping Rubber Bearing (left) and flat bearing (right). The displacement demand at different devices will not be the same, since at the location of the rubber bearings the relative displacement will result from the combination of the rigid movement of the steel structure and the cantilever response of the concrete towers

This case is intermediate between the two previous ones, implying more extensive interventions on the existing design than in the case of total isolation, but possibly less important that in the case of a global element by element strengthening. The base slab structure will be much lighter than in the fully isolated case and could possibly be made with steel braces.

It is impossible to provide a preliminary evaluation of the potential costs for this intervention, but clearly the figure given for the case of total isolation should be taken as a reference.

8.4 NUMERICAL MODELLING

The previous qualitative estimates have to be substantiated by some simplified non-linear time history analyses to verify the expected response and the values of relative displacements and shear forces at a preliminary level.

Simplified models of Groningen Forum were used to evaluate the three different design alternatives through analysis in the program SAP2000. A lumped plasticity approach was used and structural stiffness was modelled using frame elements with appropriate shear and flexural stiffness. The geometry of the structure was simplified, although the distribution of mass (translational and rotational) was consistent with the actual design in order to capture accurate torsional effects. 3D analyses were conducted using 7 ground motions with X and Y components, scaled to match the spectrum referred to in § 6.2.

The results of these analyses are relevant in providing guidance on horizontal displacements, drifts and accelerations demands. Clearly, a final design should be checked considering a full model and a 3D input that includes the vertical component.

In these simplified analyses, the walls in the cores have been modelled independently, and have been assigned stiffness only in strong-axis bending and shear. Thus, there is no effective coupling (tube action) between the walls. If desired, a degree of coupling could be added to the model by specifying nonzero axial stiffness of the walls. The geometry has been simplified to consider the walls as acting orthogonally; either E-W or N-S. Stiffness variation over the height of the structure has been ignored, but could be accounted for in more refined models since the elements are discretized by storey.

Each wall has an elastic-plastic rotational link at its base to allow for the potential formation of a plastic hinge. The flexural capacity of these hinges has been calculated using the moment-curvature analysis of the program Response-2000.

The steel structure takes the form of an inverted "U", with a bridge across the top two storeys. On the lower storeys, the gap between the west and east wings (forming the cantilevers) has been modelled explicitly. The storey stiffness is represented by a single frame element with the desired shear and torsional stiffness. It is assumed that the storeys deform solely in a "shear" manner; therefore, flexural behaviour of the equivalent frame is suppressed. Where no braces are present, the lateral stiffness of the frame is taken to be very small (almost zero). Rotational stiffness has been calculated by assuming the translational stiffness acts in a distributed manner (continuum analogy).

Floor plates for each section (concrete or steel) have been considered to be rigid bodies for purposes of calculating relative displacements at the interfaces between sections. Depending on the model in question, different types of link elements have been used to connect the different sections and the ground (for example: uniaxial springs with high damping, flat friction sliders, curved sliding isolators).

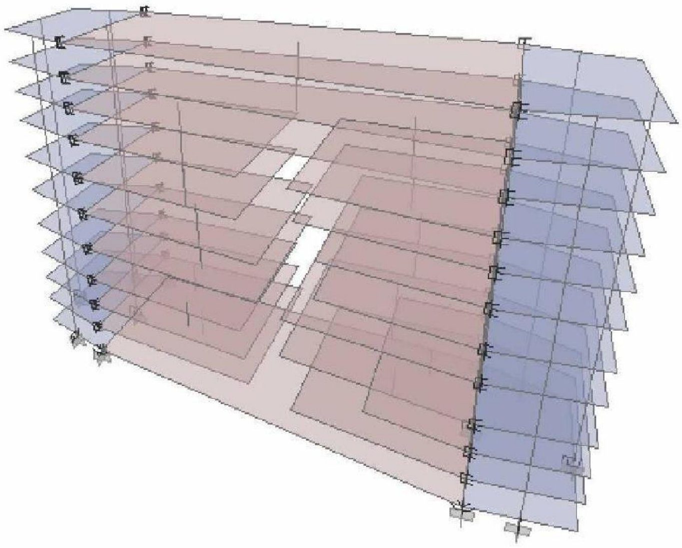


Figure 26 – Idealized SAP2000 model

The three models are distinguished as follows:

1) The first model is the baseline, corresponding to the "as-designed" condition of the structure. The steel section is modelled with the existing soft stories and expansion joints (where appropriate, including expansion joints) with the concrete cores.
2) The second model is the partial isolation solution, in which the steel structure is placed on a rigid base and flat sliders, and connected to the towers using viscoelastic dampers. For stability reasons, this solution requires the steel structure to be stiffened to act as a rigid body; thus the weak stories have been strengthened to match the stiffness of the other floors.
3) The third model represents a complete isolation of the concrete and steel structures using curved sliding isolators. Strengthening of the steel components has not been assumed in this case.

8.4.1 *Material properties*

The material properties used for the analysis are described below:

Concrete walls – plastic hinges

The flexural strength of the plastic hinges at the base of the walls has been calculated using Response-2000 assuming a reinforcing ratio of 0.4%, and a yield strength of 500 MPa. In order to capture some of the non-linearity in the moment-curvature response, a trilinear moment-rotation response has been used for each spring. The first branch is effectively rigid, after which some flexibility is allowed to simulate distributed non-linearity at the base of the wall. No shear non-linearity is considered in the hinge element, and openings (which have more of a detrimental effect on shear than flexural behaviour) were ignored; therefore an independent check of shear capacity is required.

Concrete walls – elastic stiffness

Above the base, it has been assumed that the walls have an elastic stiffness corresponding to the cracked modulus as determined by Response-2000.

Steel braces

Where braces are present, the translational stiffness has been calibrated to correspond to a drift of 0.0015 rad for a shear corresponding to 1% of the total weight of the steel section.

Rubber bearings

The high damping rubber bearing elements in Model 2 have been assumed to have a stiffness corresponding to a total lateral load of 2% of the weight of the steel section at a displacement of 250 mm. An equivalent viscous damping ratio of 17% is assumed.

Sliders

For the flat sliders at the base of Model 2, a friction coefficient of 1% has been assumed.

Curved sliding isolators

For the curved sliding isolators in Model 3, a friction coefficient of 4% has been assumed, with the total shear rising to 6% of the structure's weight at a displacement of 150 mm. This in turn corresponds to a radius of curvature of 7.5 m for the pendulum.

8.4.2 *Time history inputs*

A suite of seven ground motions was matched using the program REXEL to a spectrum based on the EC8 Type 1 (Site Class A) with a PGA of 0.24g. As discussed in § 6.2, this provides spectral acceleration values commensurate with those specified by NEN but allows for the possibility of a corner period of 2.0 s instead of 0.45 s, which seems (as discussed) more realistic and conservative in terms of displacement demand.

The time history analysis was conducted using a time step of 0.01 s, which was the same as the resolution of the ground motion records. The vertical component of the ground motion was not included, since the simplification of the model would have anyway eliminated any effects of vertical eccentricities. This simplification may induce errors, it would anyway provide significant insight in the expected response; this kind of simplification is common practice in a preliminary design or assessment phase.

The spectra of the scaled records are shown below:

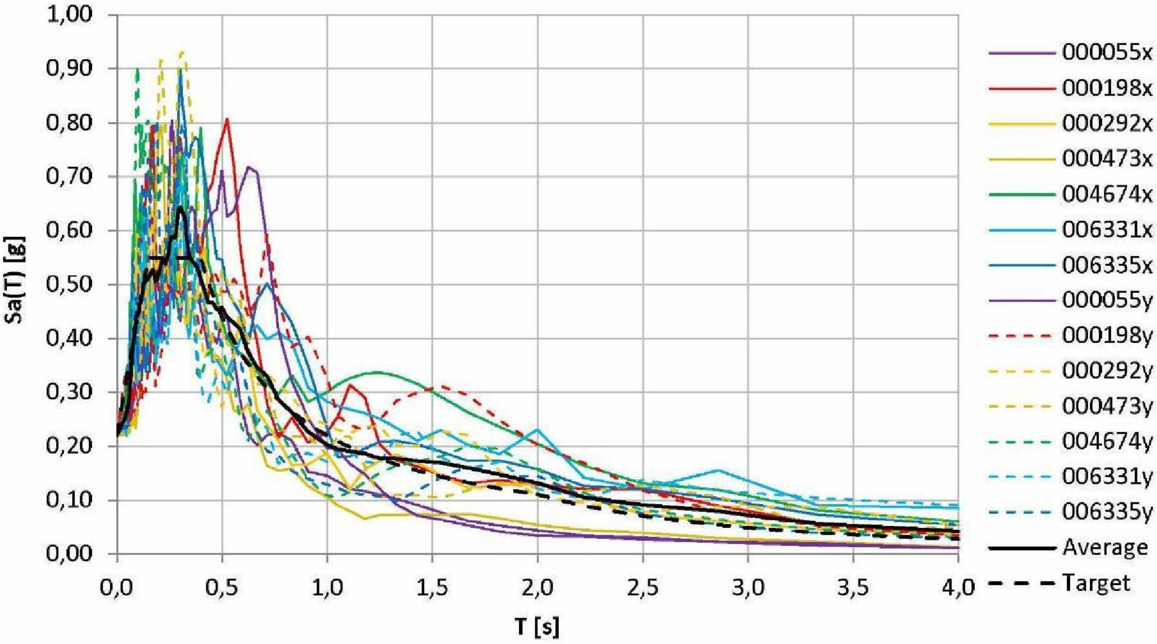


Figure 27 – Acceleration spectra of the 7 selected scaled ground motions to match EC8 spectrum

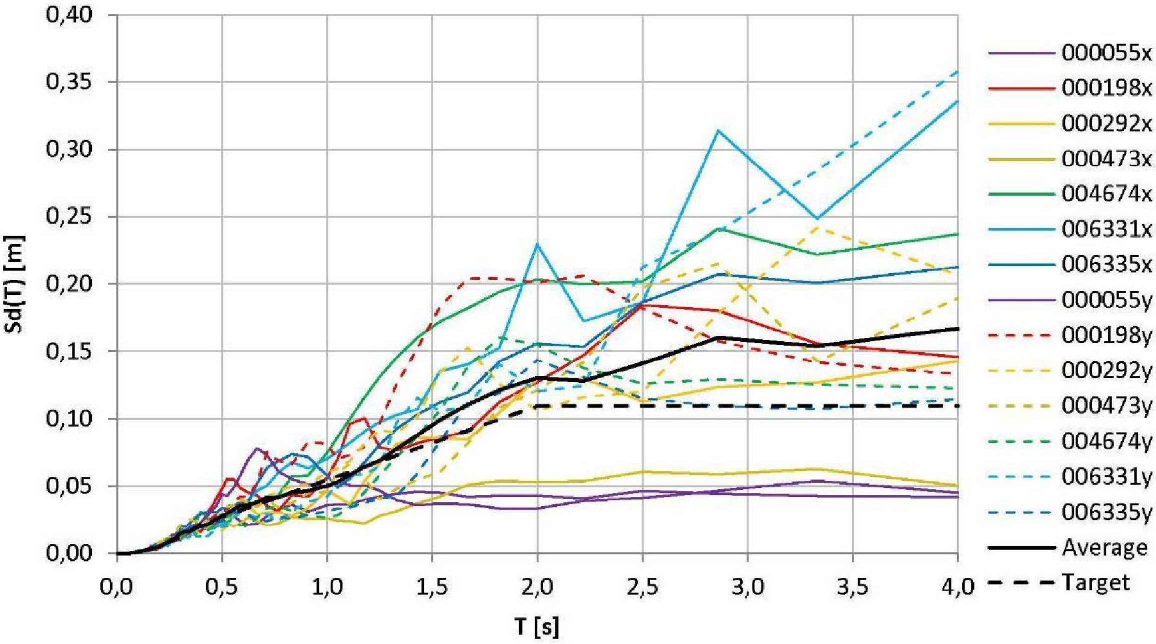


Figure 28 – Displacement spectra of the 7 selected scaled ground motions to match EC8 spectrum

8.4.3 Outputs

The three models have been compared on the basis of the following values:

1. Base shear
2. Flexural utilization of the walls
3. Centre of mass displacement
4. Storey drifts

For each independent time history analysis, the desired quantities were computed at each time step of the analysis. The maximum (in absolute value) was then recorded. These maxima were averaged across all seven records to obtain the desired behavioural measure.

Graphs comparing the results of the three models are shown below, followed by a brief discussion of trends in the results.

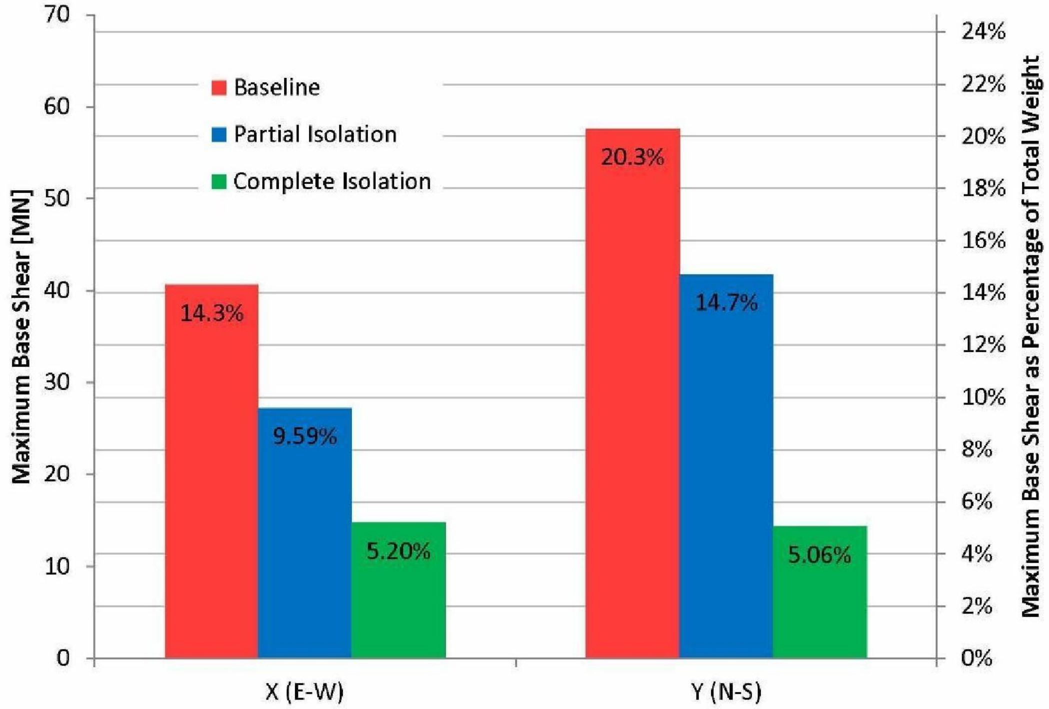


Figure 29 – Base shear demand for X and Y directions

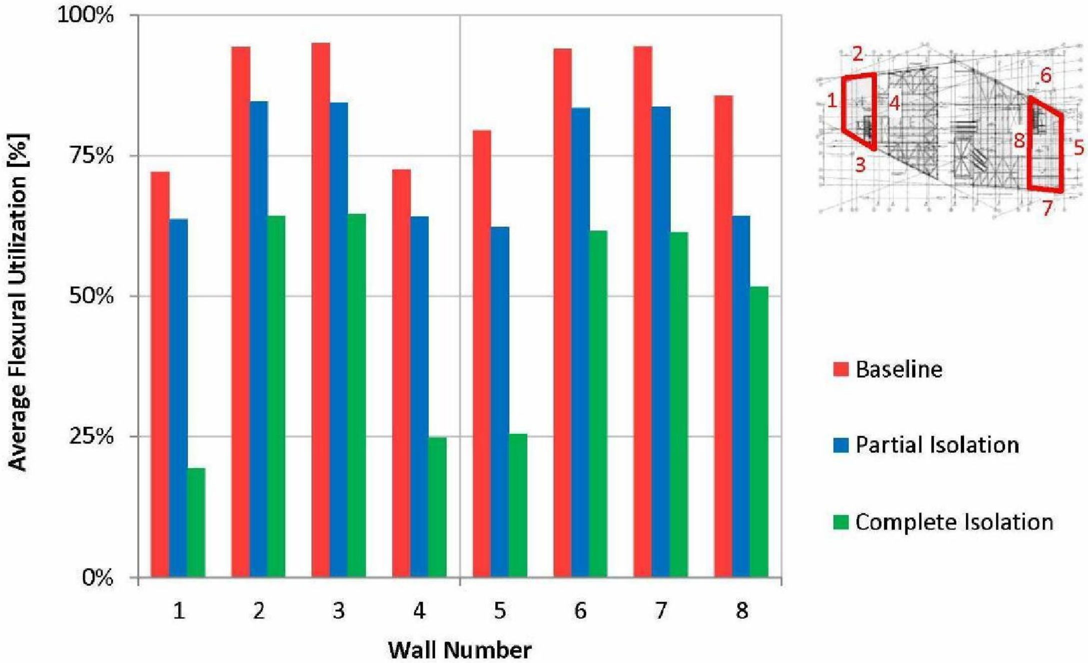


Figure 30 – Average percentage of the flexural utilization of the walls under seismic action

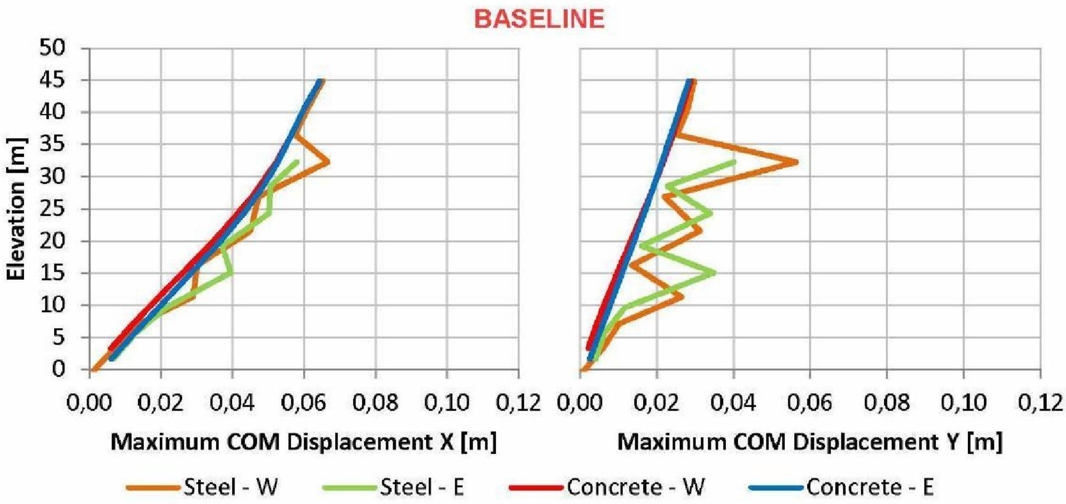


Figure 31 – Displacements of the centre of mass of each floor for baseline solution (X and Y directions)

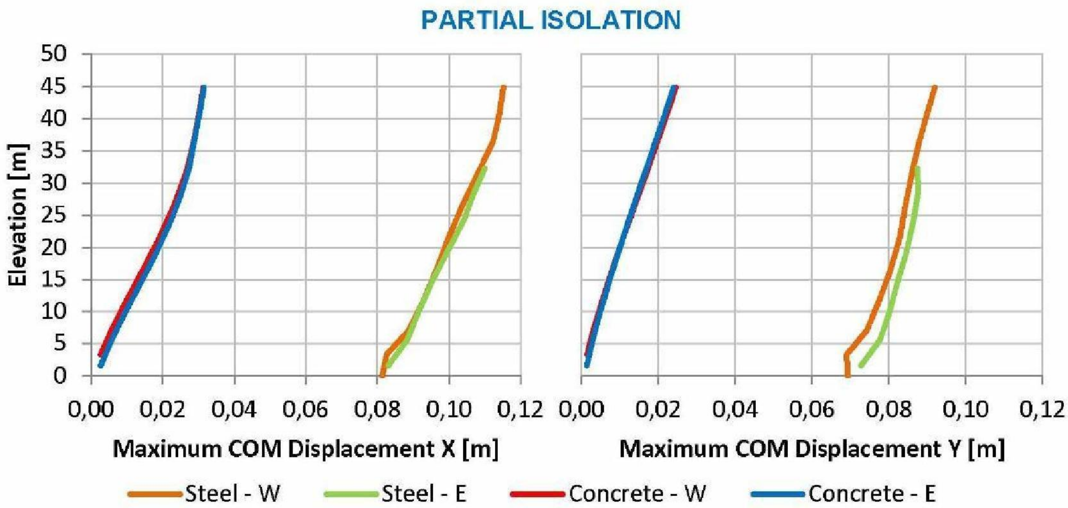


Figure 32 – Displacements of the centre of mass of each floor for partial isolation solution (X and Y directions)

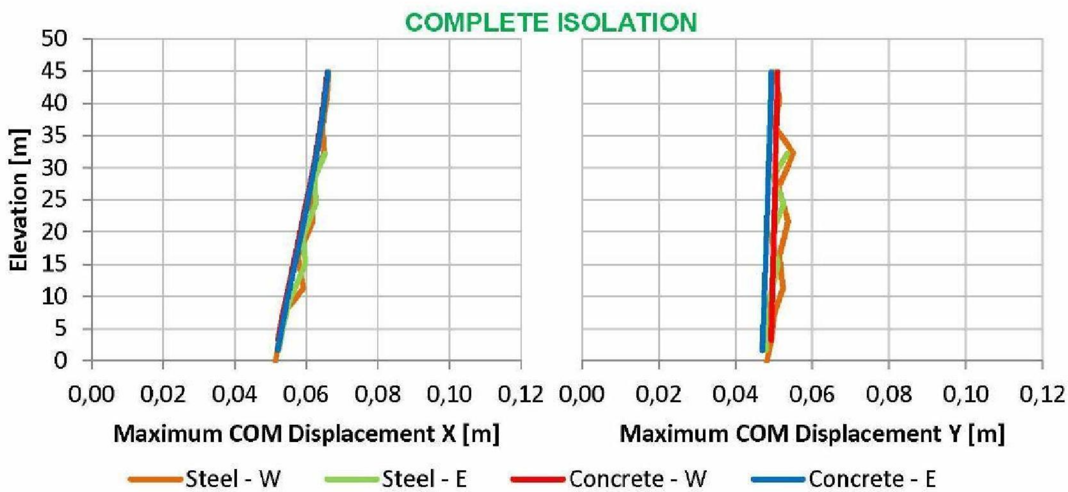


Figure 33 – Displacements of the centre of mass of each floor for complete isolation solution (X and Y directions)

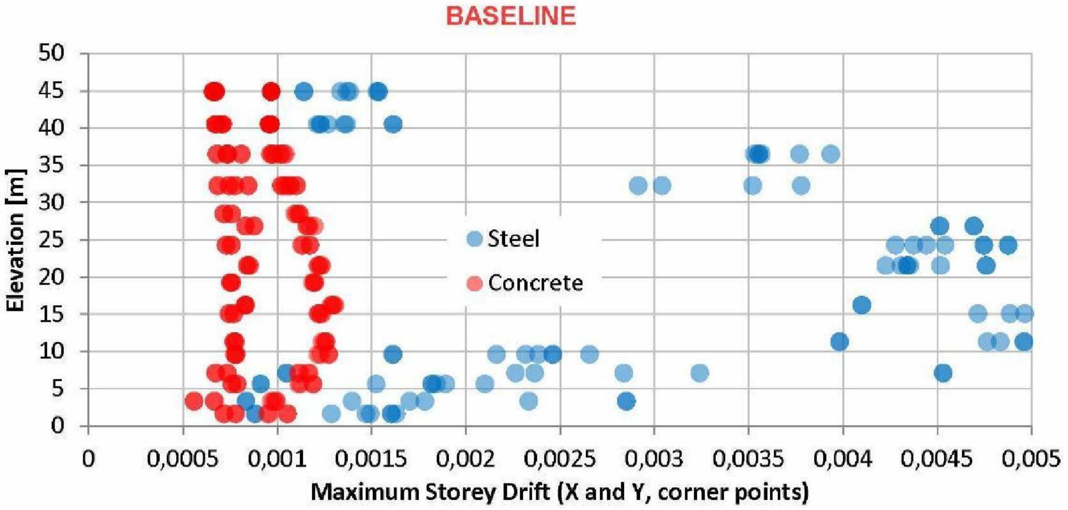


Figure 34 – Maximum inter-storey drifts for baseline solution for concrete cores (red dots) and for steel structure (blue dots)

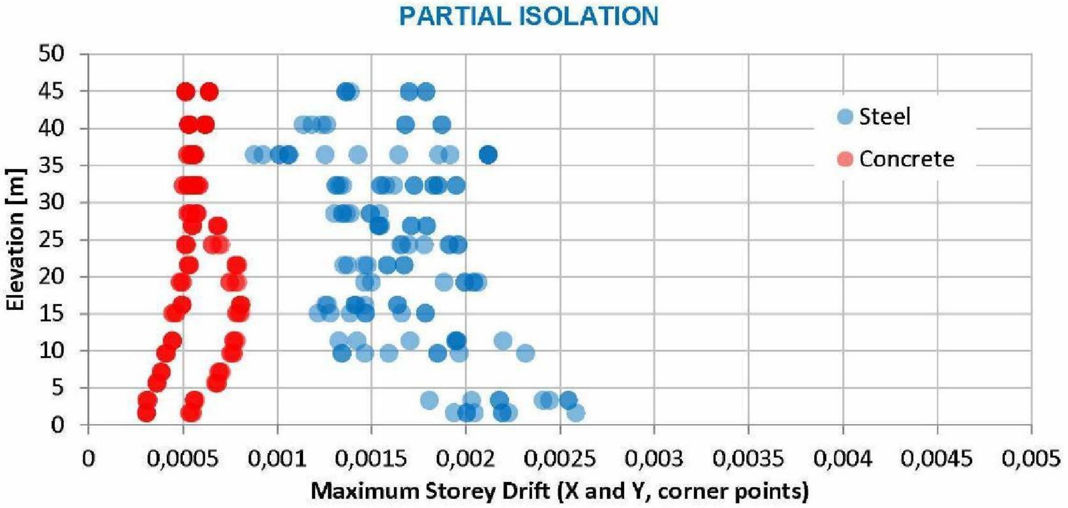


Figure 35 – Maximum inter-storey drifts for partial isolation solution for concrete cores (red dots) and for steel structure (blue dots)

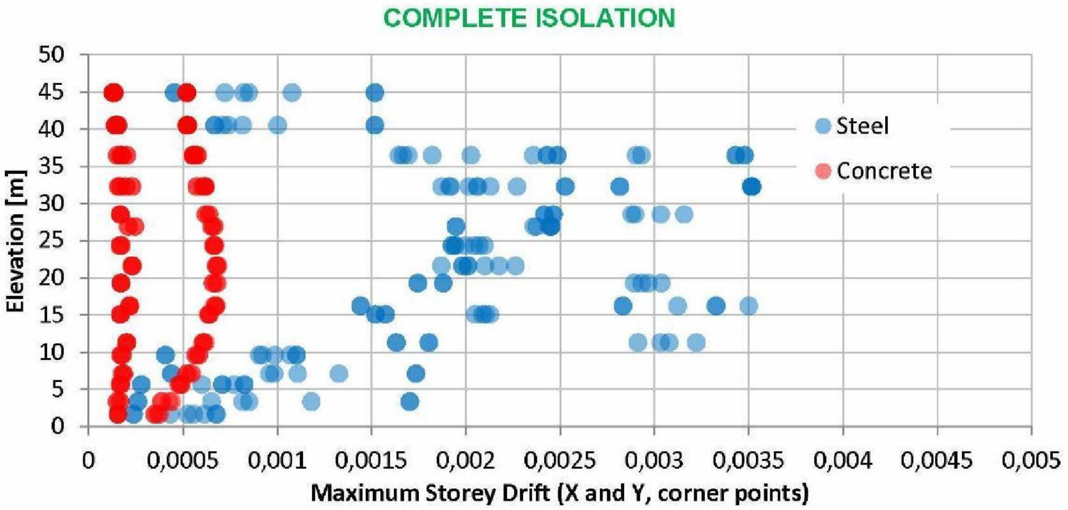


Figure 36 – Maximum inter-storey drifts for complete isolation solution for concrete cores (red dots) and for steel structure (blue dots)

The results show a clear behavioural trend as one passes from a monolithic structure to partial isolation to complete isolation. As the structural system is made more flexible, the total shear demand is reduced. The reduction in base shear is most notable in the case of complete isolation, in which case the maximum shear in both directions is approximately 5%. Accordingly, the demand on the walls also drops with isolation. Although plasticity was observed in the hinge elements for some of the time histories, when considering all seven earthquake records, the average peak utilization is less than 100% for each wall. (Once again, it should be noted that this is assuming a wall with a shear strength higher than the flexural strength everywhere).

Considering the storey displacements, the effects of base isolation become easily noticeable. In the case of partial isolation there is a relative drift between the steel and concrete structures. Allowing for out-of-phase behaviour of the two substructures, the maximum damper displacement is approximately 140 mm. In the baseline model (and to a lesser extent, the fully isolated model) the effect of the soft storeys and torsional sensitivity can be seen in the plots; maximum displacements of the steel structure are more scattered. This is once again reflected in the graphs of interstorey drift, where values up to 0.5% are encountered in the steel portion of the structure. This suggests that some strengthening of the steel substructure may be required regardless in order to satisfy serviceability requirements.

9 REFERENCES

The considerations, observations and conclusions reported in the previous chapter are based on the following references (legislation and available documents).

GUIDELINES

- Sullivan, T. J., Priestley, M. J. N. and Calvi, G. M., *A Model Code for the Displacement-Based Seismic Design of Structures*, IUSS Press, Pavia, 2012.
- Priestley, M. J. N., Calvi, G. M. and Kowalsky, M. J., *Displacement-Based Seismic Design of Structures*, IUSS Press, Pavia, 2007.

REGULATIONS

- NPR9998 “Assessment of buildings in case of erection, reconstruction and disapproval – Basic rules for seismic actions; Induced earthquake” (2015)
- EN1998-1 “Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings” (2005)
- NEN6702 (nl) “Technical principles for building structures - TGB 1990 - Loadings and deformations”, September 2007 (withdrawn)

AVAILABLE DOCUMENTS

- *Original structural and geotechnical drawings*

  Plans: from “C5_1_-6” to “C5_1_11”

  Perspective view: “C5_2_00-1” and “C5_2_00-2”

  Sections: “C5_3_00-1”, from “C5_3_00-o1” to “C5_3_00-o1”; from “C5_3_00-w1” to “C5_3_00-w6”; “C5_3_-5”

  Bridge trusses: “C5_3_09-1” and “C5_3_09-2”

  Details: “c5_4_00”, “c5_4_-5” and “C5_9_00o”

  Grid system: “C5_9_00

  Piles plan: “C5_11_-6”

  Excavation: “ct5_18_00”
- *Original design reports*

  Calculation: “berekening deel A – algemene gegevens en overzicht belastingen – 9 januari 2012”

  Geotechnical reports: “Notitie betrefende het uitgevoerde grondonderzoek – Concept – 9 januari 2012” and “Besteksontwerp bouwput en fundering - - 9 januari 2012”
- *Seismic investigation: structural reports*

  “Seismische uitgangspunten Hoofddraagconstructie Groninger Forum - 30 januari 2015”

  “Groninger Forum Seismic Upgrading – 13663 SA016 – Basis for Design – 16 February 2015 – Rev. 9 March 2015 – additions extracts from NPR valid for GFSU – Rev. 17 March 2015- p.11 γR”

  “Groninger Forum – Berekening deel V Veerstijfheden onderbouw – Code: 13663SA016 – 14 april 2015”

  “Groninger Forum Aardbevingsveilig – Code: 09615X – 1 juni 2015”
- *Geotechnical information*

  “Seismische CPT test Forum Groningen”

  “RAPPORTAGE – GEOTECHNISCH VELDWERK – betreffende – GRONINGER FORUM TE GRONINGEN – Opdrachtnummer: 5012-0254-041”

  “RAPPORTAGE – GEOTECHNISCH VELDWERK – betreffende – GRONINGER FORUM

TE GRONINGEN – Opdrachtnummer: 5012-0254-040"

- *Seismic action*

  "Seismic Design Loads for the Groningen Forum – version 1 – 10 June 2015" by Julian J Bommer, Stephen J Bourne, Helen Crowley, Ben Edwards, Pauline Kruiver, Steve Oates, Rui Pinho & Adrian Rodriguez-Marek

- *Three-dimensional models*

  Navisworks model: "09615_Groninger Forum_BKC.nwd"
  SAP2000 model: "Forum Building rev14_CONCEPT.sdb"

- *Presentation files*

  "Groninger Forum - preliminary conclusions structural upgrading" by abt (19 Januari 2015)
  "Groninger Forum – Aardbevingsdossier" by abt (7 April 2015)
  "Groninger Forum – Aardbevingsveilig" by abt (7 May 2015)
  "Meeting notes presentation design solutions Groninger Forum_7 May 2015

10 ANNEX 1