

Earthquake resistant design Proton facility Groningen

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seismisch advies

concept



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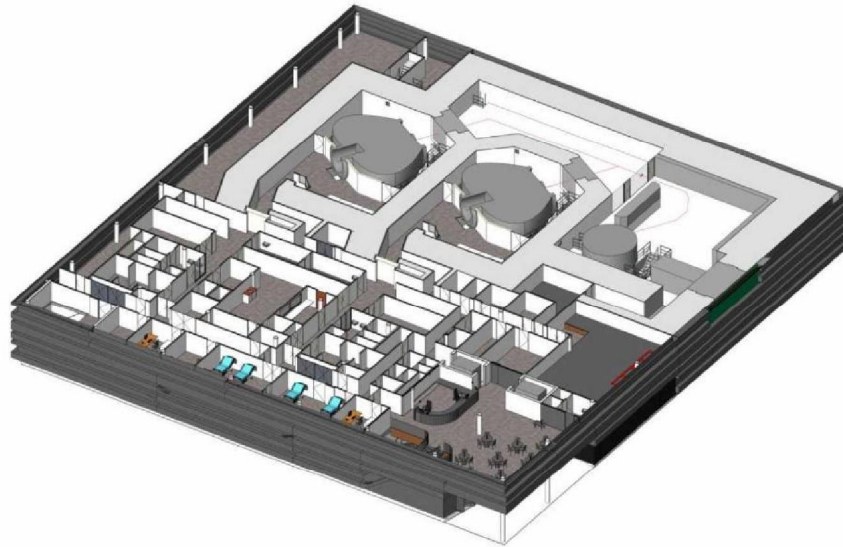
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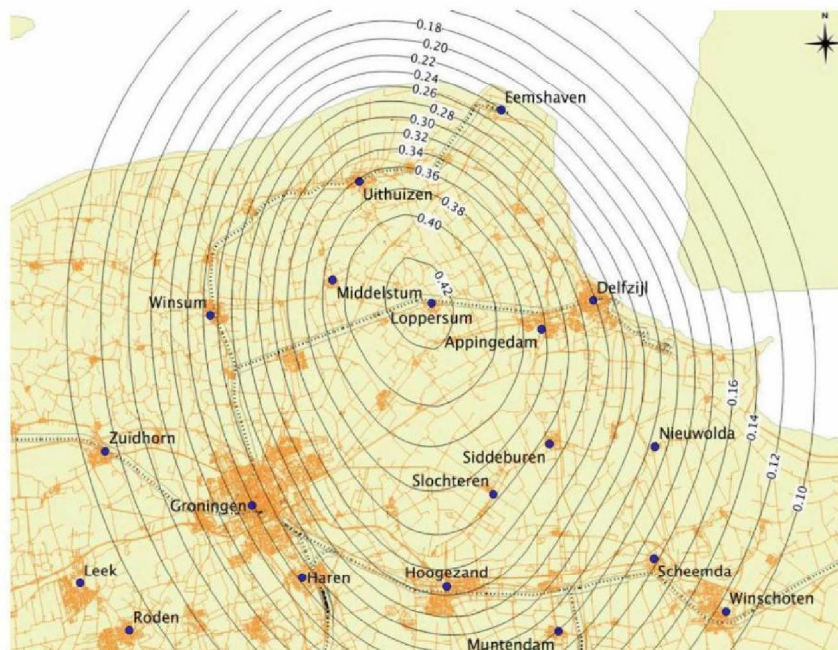
1. Introduction

1.1. General introduction

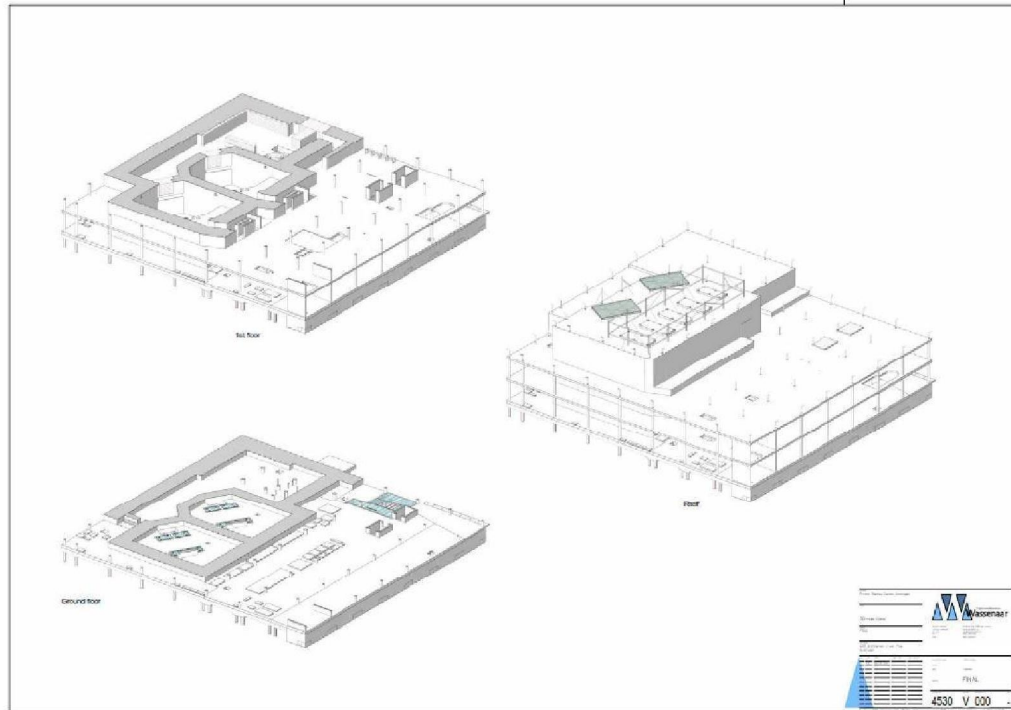
The UMCG is preparing the construction of a proton therapy facility. Patients will receive proton radiation therapy in this new building. This is a relatively new treatment and the general form and dimensions of the building are fully harmonised with the very specific requirements of the treatment process. Typical are the thick concrete walls surrounding the radiation areas.



The construction site is located on the north side of the UMCG site (corner of the Vrydemalaan and the SS Rosensteinlaan in Groningen). The interim advice on earthquake resistant design induced by gas extraction classifies the site to be prone to earthquakes (05-15-2014), all calculations are to be based on a PGA of 0.24g.



The UMCG/PTCG has commissioned that the design of the proton therapy to be earthquake resistant.

Building description

In the building of approximately 50 x 50 m² one can identify two building parts: The treatment building (left) and the clinic building (right).

Treatment building

The treatment building is where the patients will undergo proton therapy. To shield radiation to other spaces (inside as well as outside the building), the floors and roofs are designed as thick, massive concrete structures (2 to 4 meters thick).

The main dimensions are approximately 47 x 29 m x 15m (l x w x h). Approximately 11,000 m³ of concrete will be applied in the treatment building. That is 60 % of the outer volume of the building, this leads to a building that is massive and behaves as a rigid volume.

There are no underground floors in the building. The ground floor is 1.0 m thick, on ground level and is founded on a pile foundation.

Patients will be treated on the first floor. The cyclotron (where the proton beam starts) and the beam line (which transports the proton beam) are therefore situated on the first floor. The beam line transports the proton beam from the cyclotron to the gantry (where the patient is treated).

The gantry is a rotating structure with a diameter of approx. 10 m. The treatment of patients will take place in the centre of the gantry.

Parts of the equipment will be installed using hatches in the roof (gantry equipment) and the walls (cyclotron and beam line equipment). These openings will afterwards be filled with removable concrete shielding elements.

On the roof, in the open air, 4 chillers and an emergency power supply will be installed. The treatment and clinic building will be covered with nylon netting, tensioned with steel cables.

Clinic building

The clinic building is located in front of the treatment building and is two storeys high. The clinic- and the treatment building are joined to each other.

The ground floor consists of a insulated hollow core slab with cast-in-place topping over a crawl space. The first floor and the roof have wide slab flooring.



Measures have been taken (3.5m high retaining wall) around an existing pond (de Wetering) in order to lead the water in the future alongside the building (along axis 13).

Considering the underground the complex is entirely founded on piles.

1.3.

Objective

Based on the Plan of Action of Sept. 26 2014, the UMCG/PTCG commissioned on October 8 Wassenaar Engineers to make earthquake resistant designs of the new proton facility. In the past 6 weeks ABT Wassenaar seismic advice has cooperated intensively with Fugro to make an earthquake resistant structural design. This research is not yet fully completed, but advanced enough to report.

This should provide insight into:

- The applied principles
- The seismic behaviour of the exceptional structure.
- The various examined variants of the foundation, with advantages and disadvantages.
- Most viable solution direction(s)
- Other structural aspects
- Results architectural seismic scan.

Purpose of the report is to inform the UMCG about the work carried out so far and the current state of affairs regarding the earthquake-resistant design.

In addition, based on this report, a review should be carried out by the NAM. Upon approval by the NAM of the key principles and the proposed approach. Further development and implementation will lead to the final seismic design. To which a separate plan of action will be constructed.

Because the start of the construction is scheduled for April 1st 2015, all parties should act with the utmost diligence.

1.4.

Timeframe

When writing the Plan of action, one assumed that the activities mentioned would take about two months to complete. It was also indicated that there was a high level of uncertainty regarding the outcome and the course of the investigation. Because of that, one has defined three separate phases: analysis, evaluation and elaboration.

Start of the investigation: Oct 8-2014

Analysis phase approx. 4 weeks (Okt-08/Nov-05)

During this phase the interaction between the soil, the foundation and the building was especially studied. At this stage it became clear that a displacement difference between the piles and the building of about 75 mm is to be designed for. These are a considerable deformation for foundation piles. In this phase it remained uncertain whether these deformation were to be designed to responsibly. By applying increasingly sophisticated calculation methods one tries to answer the latter positively.

Evaluation phase approx. 2 weeks (Nov-06/Nov-19)

In this phase, it is concluded that the applied analysis methods are reasonably optimal. A solution using foundation piles seemed within reach, but none of the variants that have been calculated shows a 100% suitability. It was decided not to submit these results to the NAM for review yet.

Based on the obtained understanding of the seismic behaviour of the foundation, it was decided to investigate two types of piles with a potentially more favourable behaviour (Tubex, with high strength steel and a drilling pile round 2000 mm).

Elaboration phase approx. 4 weeks (Nov 20/Dec 19)

The variants with Tubex piles and bored piles are analysed and lead to viable results. It was decided to report the results and submit this to the client and the NAM for review. In the meantime investigations are going on (see chapter 7).

Review client / NAM approx. 2 weeks (Dec 5/Dec 19).

During this review the work of Wassenaar Engineers will not come to a standstill. They will continuously work on the remaining questions of the elaboration phase. The objective is, that by the end of the year, agreement will be reached on the necessary adjustments in the structural design to make it earthquake-resistant.

Technical Designphase 4 weeks (Jan 5/Feb 2)

The chosen principle solution will be worked out. A Plan of Action for this will still be delivered.

Other important data:

Mobilisation phase starts 2nd Feb 2015

Definitive piling plan: 2nd March 2015

Breaking ground: 20th April 2015

First pile: April 27th 2015.



2. Behaviour of the structure

2.1. Interaction Treatment building and Clinic building

The facility consists half of a heavy concrete structure for the treatment building, the other half of a relatively light structure for the clinic building. The latter has a "standard" construction with concrete columns and wide slab floors. Both components are connected as this has to do with the treatment process. A patient in the clinic building is carefully positioned, and cannot be transported in a bed to the treatment building over a joint. The positioning could be disturbed too much.

The clinic building is being stabilised by the treatment building. This means that during an earthquake the floors of the clinic building are constrained to the movement of the treatment building. The treatment building shows rigid behaviour, therefore no interstorey drift will occur in the clinic building. As a disadvantage this means a significant load on the diaphragm behaviour of the floors of the clinic building. In addition, eccentric loads will occur on the treatment building by the clinic building. The effects mentioned above will be further examined.

The behaviour of the treatment building dominates that of the clinic building, all attention initially focuses on the treatment building, as this governs the global behaviour.

2.2. Behaviour Treatment Building

During an earthquake a structure can, in principle, deform in 3 ways:

1. Bending of the structure on an infinitely rigid foundation

Typically 70% of the total deformation is caused by bending of the structure. As rigid behaviour is expected and verified, this bending deformations can be put on 0 %.



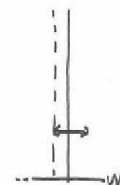
2. Rocking of the structure due to rotation of the foundation

Typically 30% of the total deformation originates in rocking of the pile foundation. The length and width of the treatment building result in a high rotation rigidity of the foundation. To this result only 10 % of the deformation appeared to be caused by rocking of the foundation.

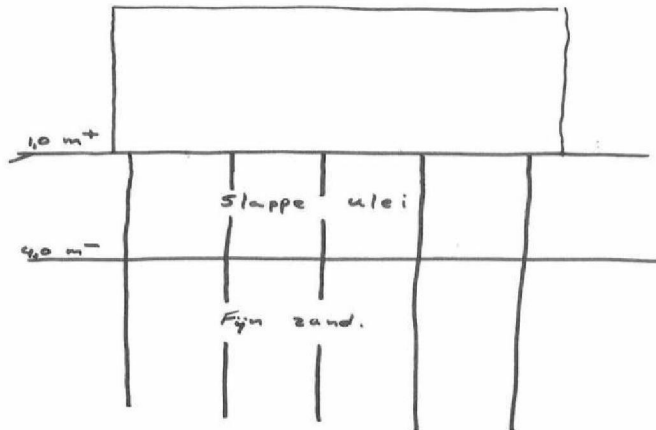


3. Horizontal translation of the foundation.

Typically the horizontal translation of a building is not taken into consideration. However, in case of the proton facility 90% of the total deformation is caused by horizontal translation. This is partly caused by the absence of a basement, in combination with the soil conditions.



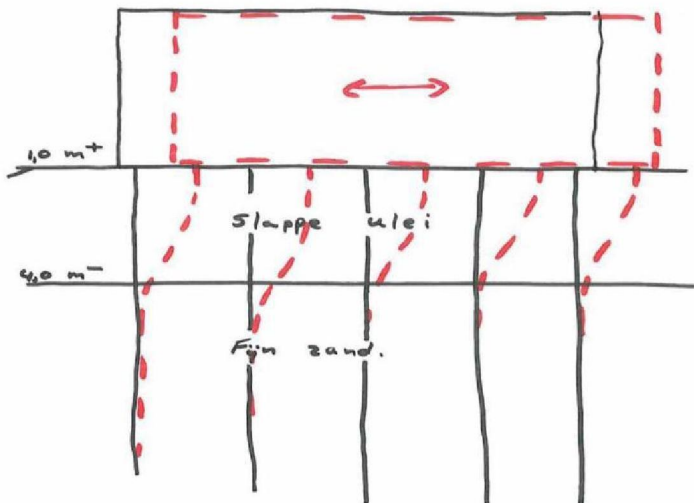
The following typical soil profile was found after field survey:



For an accurate classification, see the reports of Fugro

The top of the sand layer tapers between 2,8 m- en 5,0 m-NAP. The sand layer is not homogeneously packed and intersected with clay lenses. An optimal pile tip level is 14,0 m-NAP or 20,0 m-NAP with a higher load bearing capacity.

In case of an earthquake the primary horizontal motion will show the behaviour found below:



The piles will absorb the horizontal deformation by bending. The upper part of the piles are clamped in the heavy concrete structure of the treatment building. This leading to an underground soft-storey mechanism.

2.3.

Conclusion from behaviour structure

The description of the behaviour of the structure shows that the pile foundation to the primary seismic element. To accomplish a proper assessment of the structural consequences, the interaction between soil and pile rigidity should be adequately described. The deployment of a geotechnical engineer (in this case Fugro) in this case, is of great importance.

For a good description of the deformation behaviour of a horizontally loaded pile in the ground under influence of short-term loads, as is the case with earthquakes, we refer to the separate report of Fugro. The non-linear behaviour of the soil, the short-term character of the load and the influence of group effects are all to be taken into account.



3. Assumptions

3.1. *Peak Ground Acceleration (PGA)*

Based on the location and figure 1 of the Interim Advice (May 2014) the following applies:

$$PGA = 0.24.g$$

3.2. *Consequence class*

De consequence class has been determined on CC3

Explanation: this is a unique building that

- facilitates a type of treatment of cancer patients which is only available on two locations in the Netherlands.
- is particularly difficult to replace.
- will suffer structural damage in the foundation in case of an earthquake. Foundations cannot be inspected after an earthquake.

3.3. *Importance factor*

As importance factor γ_i the value belonging to CC3 is used, this according the NPR.

$$\gamma_i = 1.8.$$

Officially the Interim Advice is still valid. It mentions a value of 2.48, but because the NPR is expected to be published shortly, it is considered justified to use this lower value.

3.4. *Dynamic amplification factor (DAF)*

The DAF is determined on the basis of the model response chart from the Interim Advice.

3.5. *Behaviour factor q*

The behaviour factor q is not taken into account. The heavy concrete structure is considered to deform elastically. The piles are to preferably remain elastic (see NEN-EN 1998-5 5.4.2-7).

4. Liquefaction

4.1. General introduction

At approx. 4.0 m- NAP the top of a fine-grained and variably packed sand layer is found. This can be sensitive to liquefaction during an earthquake. Fugro carried out several calculations which indeed demonstrate this sensitivity to liquefaction. A more detailed descriptions is found in the report of Fugro .

Currently laboratory tests are being carried out to more sharply define the calculated results.

4.2. Influence on structure

If Liquefaction occurs, this has a number of consequences for the structure:

- A decrease of the load bearing capacity of the piles
- An increase in the buckling length of the piles
- Settlements of the ground level.
- An increase of negative skin friction.

Once more is known about the degree of liquefaction, the above mentioned effects will be assessed. When assessing the different pile types, the susceptibility to liquefaction will also be taken into account.



5. Calculation methodology

The earthquake response has been investigated using the following analyses. The order of analysis denotes the timeline in which these have been performed, as well as the complexity of such analyses.

5.1 *Modal Response analysis:*

Using linear ground and linear structural behaviour, the initial sensitivity of the structure to earthquakes has been investigated. From this analysis the first fundamental period was found and also the bandwidth in which rigidity variance has been investigated. Here it was found that the expected static variance of stiffness was less of a concern. This is mainly caused by the factor that the structure will remain within the constant displacement spectra region, where the deformation demand remains constant.

5.2 *Non-linear static pushover analysis:*

After the initial investigation the non-linear behaviour of the soil was investigated. The geotechnical engineer performed different analyses regarding group-effects. All of these were performed with elastic pile stiffness. From these analyses (as expected) the stiffness of the elastic pile is of great influence, moreover the results found from these analyses were that the piles failed to obtain the desired displacement capacity. To investigate this to further detail, the elastic pile was exchanged for non-linear pile behaviour. Using the non-linear soil and pile behaviour, and pushover analysis we were able to sufficiently determine whether certain solutions were feasible.

5.3 *Time-history analyses.*

The majority of feasible solutions based on the pushover lead to a specific displacement demand. It was investigated, whether the displacement demand could be reduced using time-history analyses. For these analyses the recorded acceleration data is used, these are scaled in amplitude and in time to account for normative peak ground acceleration and for moment magnitude. An elastic modal-response curve was constructed for a single event using 4 different accelerations. It was found that no significant reduction of the displacement demand could be obtained. It should also be noted that more ground motion data is needed to further investigate such a reduction.

6. Examined variants for the foundation

6.1. *General*

For the foundation, multiple pile variants were examined. For a detailed description see the annex, which describes the forces that occur from the seismic behaviour of the structure. First of all it is examined whether the pile is capable of absorbing occurring loads and moments. If this is the case, the following aspects will be evaluated:

- Does the pile remain elastic?
- What is the risk of spalling of the concrete?
- Sensitivity to a higher or lower level of the sand layer.
- Sensitivity to liquefaction as a result of the buckling length.
- Sensitivity to liquefaction as a result of decrease in load bearing capacity.
- Sensitivity in the execution.
- Consequences for the mobilisation time.
- Consequences for the execution time.
- Consequences for the costs.

As an alternative to a pile foundation the possibility of soil improvement was also examined. Further details can be found in §6.13 & §6.14. The possibility of base isolation was also examined, see §6.12.

6.2. *Kinematic effect*

In addition to a load from the dynamic behaviour of the superstructure, the piles are also loaded lengthwise by the kinematic effect (see NEN-EN 1998-5 art. 5.4.2): the pile is loaded by soil that passes in a wave-like motion. This effect is currently further investigated by Fugro. The forces of the kinematic effect should be added to the other forces and moments.

6.3. *Preliminary Design of 227 Terr-Econ piles (ø460-560) with grout injection*

We started with the calculations of the pile plan from the preliminary design, in which earthquakes were not yet taken into account.

Fugro has defined an horizontal spring stiffness, taking group effects into account. On this basis Wassenaar ABT Seismisch Advies defined the fundamental frequency and the base shear force.

From these calculation a deformation difference of 75 mm occurs between the sand layer and the bottom of the concrete structure. This deformation demand has to be either absorbed (plastic hinging) or followed (elastic) by the pile. The latter will lead to significant lateral forces and bending moments (in particular at the top of the pile). A first elastic evaluation found the moment to be too large, in such a manner that this was not designable to.

Therefore elasto-plastic analyses have been performed. The moment is thereby reduced, however, in combination with a normal force of ca. 1000 kN the pile still fails. The normal force is too large. Therefore it is decided to look upon larger diameter pile and to double the number of piles, which in turn decrease the normal force in the pile shaft.

6.4. *Design with 454 Terr-Econpiles (round 560-660) with grout injection.*

Twice as many piles lead to a lower fundamental period, but the deformation demand remains 75 mm. The base shear force for the total building doubles. The bending moments and lateral forces per pile remain the same, however the normal forces per pile halves. Due to the lower normal force the shaft has more deformation capacity to absorb the imposed deformations

It is questionable whether a double amount of piles could all be placed under the structure, this as the piles are to be placed in a grid of 1.75 x 1.75 m². The pile system is a soil displacing pile system, which causes a compaction of the underground due to the instalment process, resulting that piles may not be brought to the required depth.

Using the elasto-plastic calculation is also become clear that, up to a deformation of 35 mm, the reinforcement does not yield. After that, plastic hinging occurs on the pile head. A push-over analysis shows that all lateral deformations of the pile head from 35 to 75 mm are absorbed in the plastic hinge that is created at the bottom of the floor. Using a more accurate method, it was shown that the pile head ultimately fails at a deformation of 45mm.



- 6.5. *Design with 227 Tube screw piles (buisschroefpaal), ø800*
This variant on an auger pile also had a fundamental period larger than 0.7s. This does not reduce the deformation demand and it also requires a high rotation capacity of the pile head.
- This pile head is not suitable for this. The reinforcement is, as a result of the production process, placed deep inside the pile with a high cover of ca. 200 mm. This is a big risk due to spalling of concrete after an earthquake. Because the deformation demand stays the same, but the diameter is larger than that of other pile types, this pile will always respond less favourably.
- Previous studies show that a deformation demand of 75 mm should always be designed to. This seemed to be impossible for the pile types that were considered. This led to an approach to search for a foundation type with a lower deformation demand. This can be achieved by a lower fundamental period ($T_1 < 0,7s$). This means a more rigid foundation. Therefore a foundation with bored piles round 2000 mm are considered.
- 6.6. *80 Bored pile (with bentonite as support fluid) ø 2000 mm.*
This variant utilises the maximum amount of large diameter bored piles that can be installed underneath the entire treatment building, in a grid of 4 x 4 m². In order to sufficiently clamp the upper part of the piles, the ground floor will be thickened from 1.0 to 2.0 m.
- A thicker floor however has (apart from the costs) two disadvantages. The mass of the building further increases and the distance between the bottom of the foundation and the top of the sand layer decreases.
- The fundamental period decreases to 0.44 s which reduces the deformation demand to 38 mm. The dynamic amplification factor (DAF) increases to 2.8. As expected, the base shear force, shear force and bending moment in the pile, increase considerably. In the pile a large amount of reinforcement is needed (2 x 40 round 40). Which also means considerable reinforcement in the concrete floor of 2.0 m is needed.
- The pile however remains elastic. The reinforcement lies, with a cover of 70 mm, relatively far to the outside. The risk of spalling of the concrete is then greatly reduced. In addition, when spalling should take place, the pile will still have an enormous spare capacity.
- The outward bending of the pile does not start at the top of the sand layer, but at a deeper level in the sand layer compared to the other pile types. Therefore the bored pile is less sensitive to variations of the top level of the sand layer. In addition the bored pile will also be insensitive or less sensitive to a larger buckling length, as a result of liquefaction. This solution is therefore considered to be a robust one.
- 6.7. *227 Terr-econpalen (560/660) with a hinge at the top*
If the problem is due to the clamping of the pile in the foundation, making a hinge at the top, might offer a solution. One could consider to cover the pile head with a soft material and a central steel axle (round 130 mm). A complicated reinforcement cage shall be needed, around the steel axle, to responsively transfer the loads into the pile.
- Calculations show that the deformation demand is elastically solvable. The buckling length however becomes a problem. This becomes twice that of a pile that is clamped at the top. If liquefaction occurs in the sand, the buckling length can increase even further and the system will become unstable.
- 6.8. *227 Tubex-piles (screwed tube pile) round 406 x 16 (high quality steel S460).*
From the above mentioned study it is clear the concrete piles with reinforcement have insufficient rotation capacity (with the exception of large drilling piles). We have therefore looked at a variant with screwed tube piles (Tubex). The steel tube is filled with concrete and stays behind. This type combines relatively low bending rigidity with high yield failure. This is especially true when high yielding steel is used (S460).
- Because of the lower rigidity, the fundamental period remains larger than 0,7s. This again leads to a deformation demand of 75 mm. Using S460 steel allows this to be imposed with virtually any yielding.

An important issue is the loadtransfer from the pile to the foundation. Large bending moments are found using this system. In addition the sensitivity to corrosion should be taken into consideration. The pile is partly above ground water table. One should think of additional wall thickness that can be used as a mean to account for oxidation or one can apply cathodic protection.

This system consists of relatively slim piles, which are sensitive to buckling lengths and /or liquefaction.

Because of the relative slenderness, the vibrations are transmitted to the superstructure in a more damped manor. Which can be seen as a form of base-isolation.

6.9. *Bored piles with a large diameter and a length of 14 m*

In case a hinge at the top is favourable, this might be the case for a pile that is clamped at the top, but is free to rotate in the sand layer. Therefore we did a study of a variant with drilling piles with a length of 14 m.

The calculations show that the bending moment stays approximately the same compared to a much longer pile. However this system has both beneficial as non-beneficial effects which cancel each other.

Because a shorter pile is more sensitive to buckling length and liquefaction, bored piles to 20m-NAP are preferred.

6.10. *Tubex pile with gap around it over the upper 5 m.*

Normally the bending moment at the top of the pile are much bigger than in the sand. This among other things, due to the support pressure in the clay layer. If this pressure is removed, the clamping moment is expected to decrease.

In practice this would be possible by placing a tubed auger pile in a hollow casing into the clay layer and then to place a Tubex pile in the inside of the cavity. A complex method of which it remains questionable whether the hollow space will not become contaminated in time.

From calculations the effect of the support pressures appears to be limited. Therefore concluding that this variant is also not a feasible solution.

6.11. *Cross piles*

Cross piles can be seen as a variant on bored piles. With a diaphragm wall technique a cross-shaped pile is installed. This variant is still being examined by the general contractor Visser and Smit.

6.12. *Base isolation*

Worldwide a lot of experience exists with base isolation. A superstructure is placed on sliding blocks. During an earthquake the building remains in its place, while the foundation has the possibility to shift underneath the building.

We have presented this solution to our advisor Aurecon in New-Zealand. They pointed out that base isolation is only applicable, when the slide blocks can be placed on a rigid surface. If the soil is not rigid horizontally, the dynamic behaviour of the structure becomes uncertain and base isolation can prove to be quite unfavourable. Because of the 5 m thick soft layer of clay, a pile foundation will always be relatively weak in horizontal direction.

Base isolation in combination with pile foundation and the shear mass of the structure is therefore not an option.

6.13. *Soil improvement by means of soil mix*

Another option is to strengthen the soft clay layer by using soil-mix techniques. The clay is mixed with grout, which results in a sort of sandstone. The building can then get a shallow foundation. Research should show what strength can be achieved.

The material must in any case be as strong, that it can absorb the foundation load (approx. 300 kN/m²). And in addition must also be able to absorb horizontal force. If the dynamic behaviour is such that the DAF is 2.8, a horizontal force of $1.8 * 0.24 * 2.8 = 1.20$ g has to be absorbed.



The downside of this variant is, that if liquefaction occurs the entire foundation of the facility loses its load bearing capacity. Another disadvantage is, that the building is founded on the top of the sand package. These layers are more susceptible to settling. This variant is still under investigation.

- 6.14. *Soil improvement by means of excavating and reapplying of sand*
Because the layer of clay is approx. 5 m thick, it is possible to excavate the clay in strips, with a crane and at the same time supply it with sand. Downside of this is, that one should operate below groundwater table. Thus the sand cannot be compacted sufficiently. Mainly because of the requirement to work vibration-free on a hospital site, therefore the possibilities for compacting are limited. It will also lead to a lot of heavy transport in an urban area.

- 6.15. *Summary*
From all variants examined the fundamental period appears to be above 0.7 s in almost all cases. Therefore the majority of solution there is a 75 mm deformation requirement. This provides a maximum moment at the top of the pile.

The Terr-Econpile and the large tube screw pile are incapable of absorbing this pile top moment.

A Tubex-pile ($\varnothing 406-16$) with a high quality of steel (S460) is capable of absorbing the pile top moment and stays almost entirely elastic. It is a relatively slender pile, prone to buckling and therefore to liquefaction. The Tubex pile will fit in well into the current design with Terr-Econpiles. The Tubex-pile will transfer the vibrations in a damped way to the superstructure.

Special attention is needed for the mobilisation time (especially delivery of the tubes), execution time (drilling the piles), and difficult connection of the pile to the floor.

Another alternative is the bored pile ($\varnothing 2000$ mm). This one is robust and less susceptible to buckling and liquefaction. The total base shear force at the bored pile however is very large compared to the Tubex-pile. To absorb these loads, the ground floor needs to become 2.0 m thick and heavily reinforced. The vibrations of the earthquake are more directly transferred to the superstructure.

Special attention is needed for the mobilisation time (in Holland only a few cranes are available to produce these piles), execution time (drilling the piles with bentonite)

Base isolation is not feasible on a weak pile foundation, but can possibly be applied after soil improvement.

The possibilities of soil improvement are still being examined. Points of attention are: the strength, susceptibility to settling and to liquefaction.

7. Ongoing research

7.1. *Liquefaction*

The report elaborates on liquefaction. So far their calculations show that several sand layers are prone to liquefaction. These results for these analyses can be narrowed further if the distribution of the fine fraction is better known. So far the effects of liquefaction is derived solely from the CPT. The distribution of the fine fraction in the sand is currently being determined in a laboratory. Depending on the results the susceptibility to liquefaction be better estimated.

Depending on the susceptibility to liquefaction the:

- load bearing capacity of the pile during an earthquake needs to be adjusted.
- settling of the soil on ground level needs to be examined.
- negative skin friction after an earthquake needs to be reevaluated.

7.2. *Kinematic effect*

During an earthquake the vibrations move in a wave-like motion through the ground. These waves also pass the pile shaft. This can cause transfer force and moment. At present Fugro is still to quantify this effect.

7.3. *Damping*

The modal response spectrum, as mentioned in the interim advice is based on a damping of 5%. A damping can also be derived from the behaviour of the pile in the ground. Which is more than 5%. The modal response spectrum can be adjusted to this. One can only calculate with $q = 1.0$ in that case. This is especially important for the deformation demand. In this way the maximum deformation demand of 75 mm can be reduced. This method is known as displacement based design and it being suggested by our advisor Aurecon. The applicability of using damping in the case of soil-pile interaction needs to be further examined.

7.4. *Soil improvement*

At present the general contractor Visser & Smit is investigating the possibilities of soil improvement that has been mentioned.

7.5. *Eccentricity Clinic building*

The influence of the eccentricity of the floors of the clinic building on the entire system, turned out to be limited, compared to the mass of the treatment building. When the pile system is chosen, the foundation of the clinic building needs to be adjusted to this.

This will, among other things, mean that the hollow core slab on the ground floor needs to be poured with a much thicker pressure layer. Another possibility is to pour the floor on sand. Instead of a crawl space a cable duct can be applied.

7.6. *Retaining wall*

Along axis 13 one needs to take the construction of a pond into account. Therefore a retaining wall is designed on axis 13. This retaining wall can be loaded horizontally by soil during an earthquake. This could lead to collapse of the retaining wall. At this moment the retaining wall is piled and has a bearing function. The possibility of positioning the piles behind the retaining wall, so that it has no longer a bearing function, is currently being investigated.

7.7. *Architectural scan*

A quick scan is currently being performed regarding issues that are architecturally related, or related to building services. The report hereof will be sent separately. This will be a starting point for AAS (architect), v.d. Plas (building technique consultant) and Royal Haskoning (building services consultant), in order to design, for everyone in their own field of expertise, to obtain an earthquake resistant design.

8. Conclusion

From all variants examined the fundamental period appears to be above 0.7 s in almost all cases. Therefore the majority of solution there is a 75 mm deformation requirement. This provides a maximum moment at the top of the pile.



After studying many types of piles, two piles remain, of which the shaft stays almost completely elastic during an earthquake. This is important, as the condition of the piles after an earthquake cannot be inspected.

The two pile types are:

- Tubex piles with steel quality S460. (Attention should be paid to the connection to the ground floor and to corrosion).
- Bored piles round 2000 mm with a 2.0 m thick groundfloor.

When more is known about the susceptibility of liquefaction of the sand layers, both types will need to be reassessed, and a definitive choice will be made.

When liquefaction is not a problem the Tubex pile is favourable. The diameter of the tube, wall thickness and steel quality has to be optimised.

In case liquefaction is a problem the bored pile is, due to its robustness favourable.

The advantages and disadvantages of soil improvement will be studied further in order to make a proper decision.

9. **Draftversion NPR 9998 dated 1st December 2014**

Since the first of December 2014 a new draftversion of the NPR 9998 is known by some of our colleagues, because he is a member of TC13.

In this version the importance factor changed from 1.8 to 1.7. The values of the model response chart (fig 3.2) also change.

Both changes have an impact on the values mentioned in this report. For instance: the maximum deformation demand changes from 75mm to 30mm.

Coming weeks will be used to investigate the impact of these new values. The calculation methods which will be used for this investigation are mentioned in this report and will not change. The different types of foundation mentioned above will all be checked again.

When there is an agreement about assumptions, and method described in this report. An update of the conclusions can be reviewed in a short period of time.

10. Annex: Summary of calculation results



Variant study of solution for the seismic design of the foundation piles

Assumptions

Soil profile

In the report of FUGRO (5008-0324-011.R02) the following table (table 9) has been used in the determination of the main effects due to Soil-Structure interaction.

Top of layer [m t.o.v. NAP]	Soil type [-]	$\gamma_{\text{usat}}/\gamma_{\text{sat}}$ [-]	c_u [kPa]	J	$\epsilon_{50\%}$
0.25	Stiff Clay	17,0/17,0	50	0,25	0,01
-2.0	Soft Clay	16,0/16,0	25	0,25	0,02
			φ	qc	K0
			[°]	[kN/m ²]	[-]
-4.25	Sand	19,0 / 21,0	30,0	8000	0,5
-6.5	Sand	19,0 / 21,0	30,0	3000	0,5
-7.2	Sand	19,0 / 21,0	30,0	14000	0,5
-9.3	Sand	19,0 / 21,0	30,0	6000	0,5
-11.0	Sand	19,0 / 21,0	30,0	12000	0,5

Seismic loads

The building is categorized in consequence class 3. According to the NPR9998 the governing importance factor then becomes $\gamma_i = 1,8$. The peak ground acceleration is taken according to the map as $\text{PGA} = 0,24g$.

Global mass of the building:

Total 290000 kN

Therapy centre \approx 240000 kN

Concrete & Reinforcement

The modelled concrete quality is C28/35 ($f_{\text{cyl}} = 28$ Mpa). The reinforcement has taken as quality B500C. Spalling and confinement has not been modelled. After the ultimate force the section has failed. Moreover the upperbound strength and short-term effect have been used.

Group effects

The influence of the global group effect is derived from the data obtained from Fugro

The stiffness used as a reference is based on a Terr-econ pile 460/560 $E=15000\text{N/mm}^2$

Kh single 4992 kN/m¹ Table 12 5008-0324-011-DPG03

The group stiffness is based on table 19

Kh [kN/m ¹]	Number of piles	Kh pp [kN/m ¹]	Kh red	Diff
785000	227	3458	69%	-31%

A double number of pile lead to a reduction of 25% of the single pile stiffness therefore the following yields

1177500	454	2594	52%	-48%
---------	-----	------	-----	------

First investigations of the fundamental period

Global mass of proton therapy centre including the clinical building

$m = 24000$ metric ton, with a centre of mass @ 5,9m above surface level

Soil stiffness parameters have been provided by the geotechnical engineer, the stiffness is based on:

- 227 terr-econ piles shaft $\varnothing 460$ point $\varnothing 560$ length $\approx 29\text{m}^1$ individual axial stiffness 200MN/m^1 (extra information : <http://www.terracon.nl/wp-content/uploads/2014/02/Terr-econ-Eng-2014-def1.pdf>) $E= 15000$ Mpa
- Total horizontal stiffness of piles including soil, group effect etc. $\sum Kh = 785$ MN/m¹

It is assumed and has been verified that the fundamental period is primarily governed by the soil stiffness.

Therefore the fundamental period may be calculated as: $T = 2\pi\sqrt{(24.000.000/785.000.000)} = \mathbf{1,10s}$

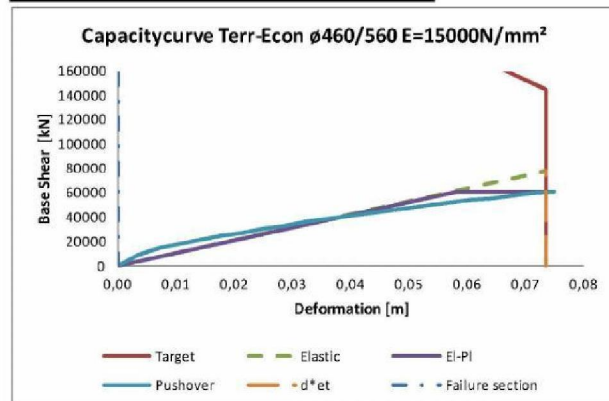
Variant 1: Current design, 227 Terr-Econ pile ϕ 460/560mm, elastic concrete

Pile data

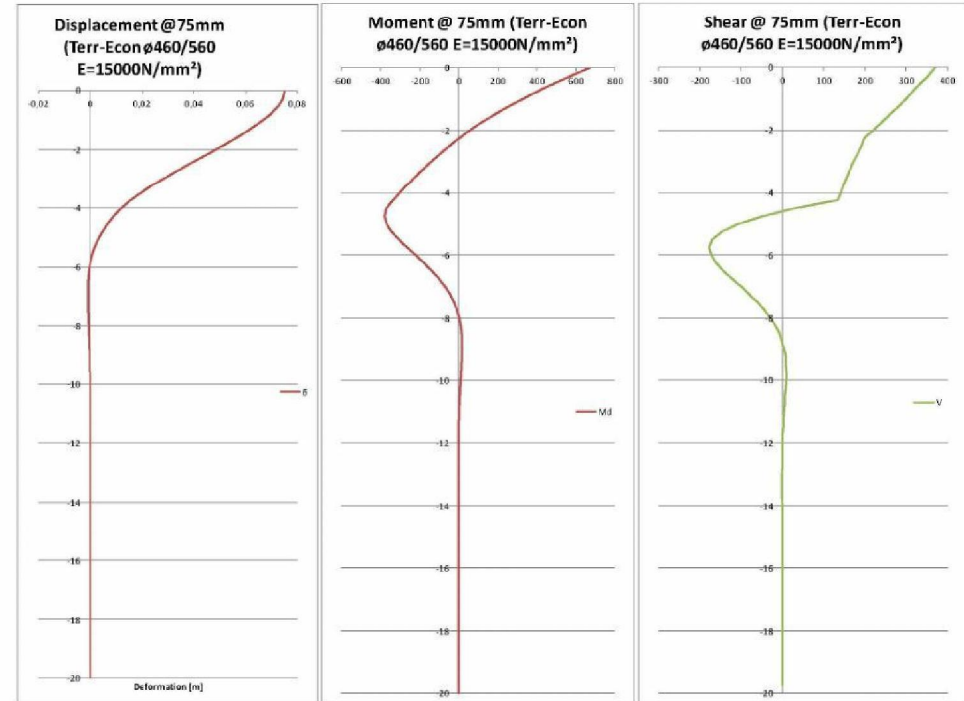
Pile type: Terr-econ paal rond ϕ 460/560 (top 4m ϕ 460mm)
 Amount: 227 (under treatment centre)
 Pile Force 1000 kN
 Pile property Elastic pile $E= 15000 \text{ N/mm}^2$,

Summary of results

Capacitycurve Terr-Econ ϕ 460/560 $E=15000\text{N/mm}^2$	
Fy*	387 kN
grp. Eff. Red.	-30,00%
Fy* (red)	271 kN
A	227
Fbase*	61526 kN
Em*/Fy*	0,046 m
dy*	0,058 m
M	240000 kN
m*	24465 Ton
T*	0,958 s
Sde	0,074 m
Sd(T)	0,323 g
q	1,26
Failure of tube @	mm



Pile moment & 75mm = 660 kNm, with a normal force of 1000kN, which yields 7% reinforcement. This results in a non-feasible design.



Conclusion

The fundamental period is greater than 0,7s. Which will due to the response spectrum result in constant displacement demand. The moment demand is also far greater than solvable with reinforcement.

Possible solutions:

The current approach is based on a constant E-value, however concrete behaves non-linearly. It is therefore appropriate to account for such behaviour. Moreover the normal force in the pile contribute to the capacity and stiffness of the section. Therefore more piles should firstly lead to a stiffer response and a higher bearing capacity for a single pile.

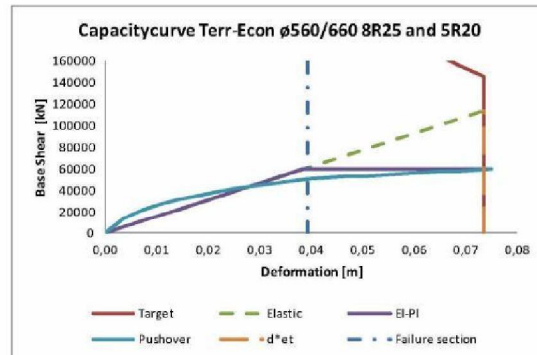
Variant 2: Current design 227 Piles Terr-Econ ø560/660, Elastic plastic pile

Pile data

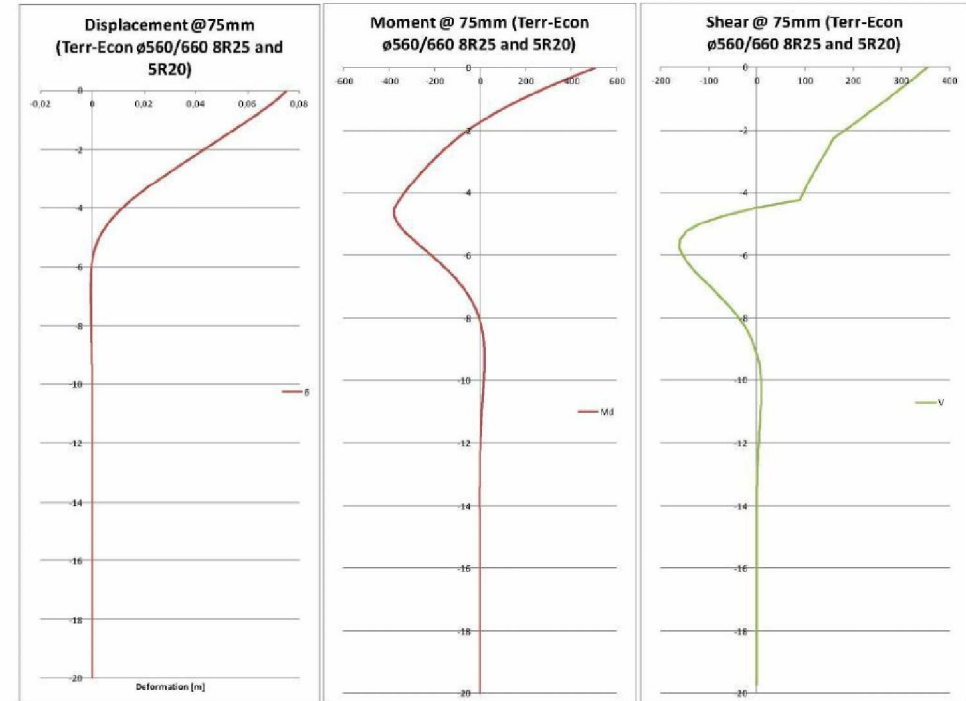
Pile type Terr-econ paal round ø560/660 no grout-injection
 Amount 227 (under treatment centre)
 Pile Force 1000 kN
 Pile property Elastic plastic, head reinforcement 8r25 first 4m, remainder 5r20. (MNk see appendix for appropriate diagram)

Summary of results

Capacitycurve Terr-Econ ø560/660 8R25 and 5R20	
Fy*	373 kN
grp. Eff. Red.	-30,00%
Fy* (red)	261 kN
A	227
Fbase*	59307 kN
Em*/Fy*	0,056 m
dy*	0,038 m
M	240000 kN
m*	24465 Ton
T*	0,789 s
Sde	0,074 m
Sd(T)	0,476 g
q	1,93
Failure of section @	0,039 m



Failure of section is defined by spalling of the cover.



Conclusion

- The fundamental period is slightly lower than the current non-seismic design.
- The pile fails @39mm, which is lower than the demand.

Possible solutions

- Double the amount of piles

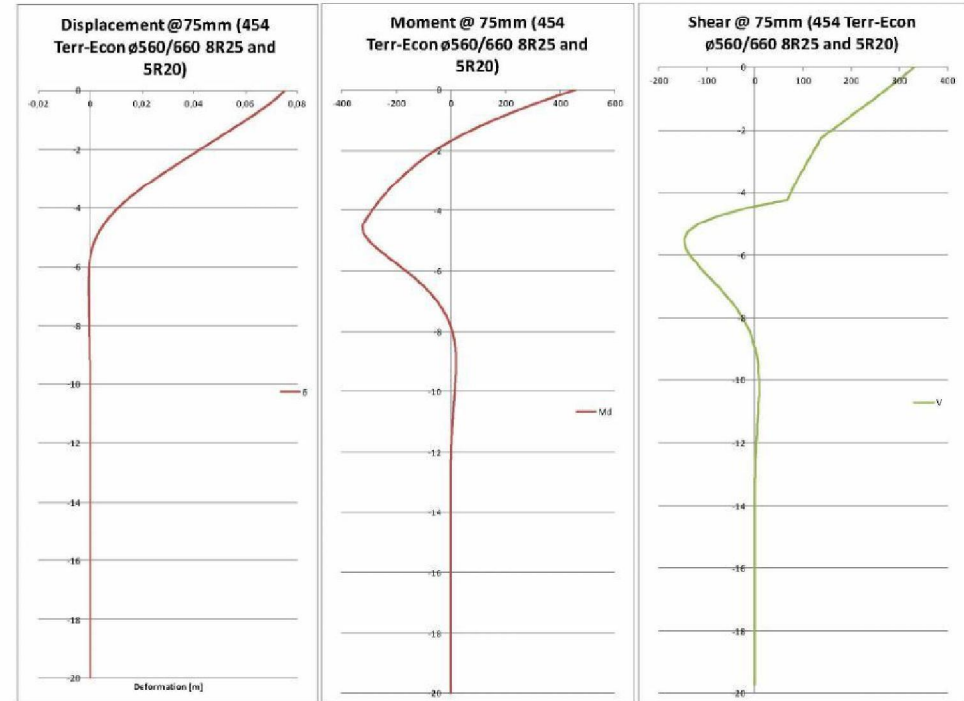
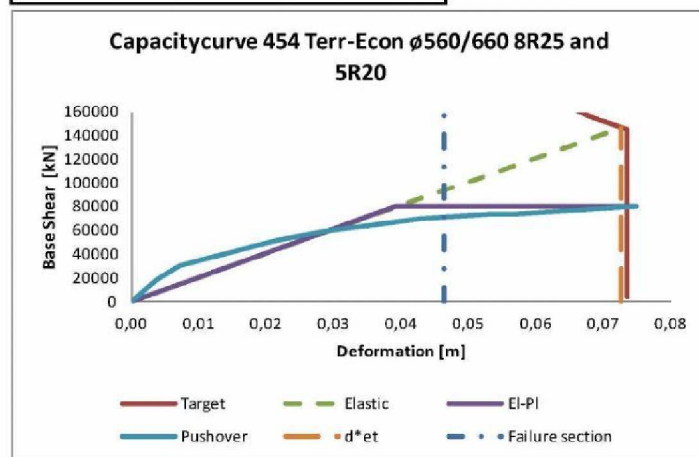
Variant 3: Current design with double amount of pile. 454 Piles Terr-Econ ϕ 560/660, Elastic plastic pile

Pile Data

Pile type Terr-econ paal round 560/660 no grout-injection
 Amount 454 (under treatment centre)
 Pile Force 500 kN
 Pile property Elastic plastic, head reinforcement 8r25 first 4m, remainder 5r20. (MNk see appendix for appropriate diagram)

Summary of results

Capacitycurve 454 Terr-Econ ϕ 560/660 8R25 and 5R20	
Fy*	350 kN
grp. Eff. Red.	-50,00%
Fy* (red)	175 kN
A	454
Fbase*	79358 kN
Em*/Fy*	0,055 m
dy*	0,039 m
M	240000 kN
m*	24465 Ton
T*	0,691 s
Sde	0,073 m
Sd(T)	0,613 g
q	1,85
Failure of section @	0,046 m



Conclusion

- The fundamental period is slightly lower than the current non-seismic design. It is however smaller than 0,7s. This slightly lowers the displacement demand.
- The pile fails @46mm, which is lower than the demand.

Possible solutions

- Increase the diameter of the piles, this to accomplish more moment capacity in the section.

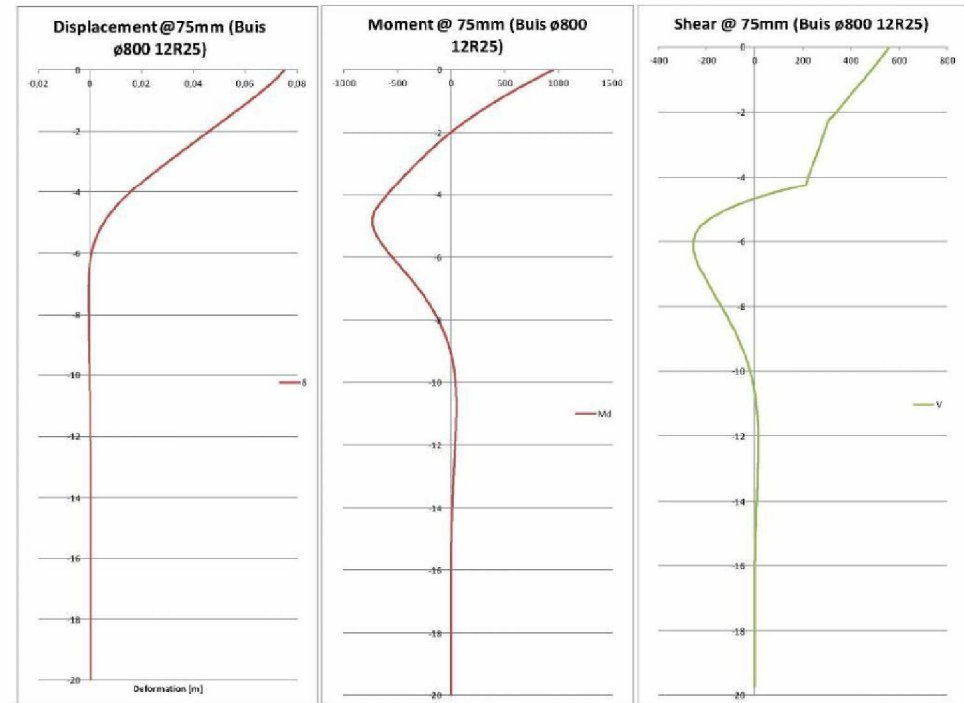
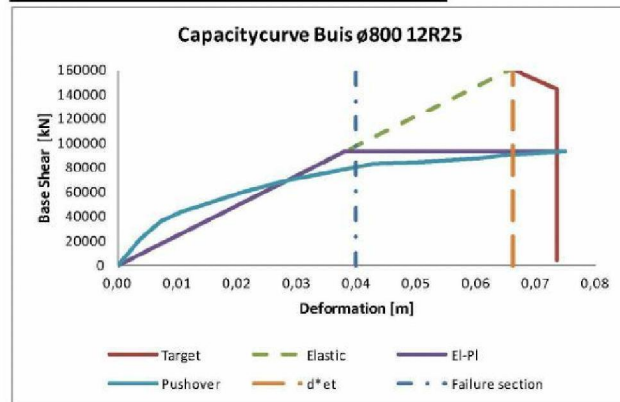
Variant 4: Current design with 227 Tube screw-pile (buisschroefpaal) ø800mm

Pile Data

Pile type: Tube screw-pile (buisschroefpaal) ø800 (top tot tile),
 Amount: 227 (under treatment centre)
 Pile force: 1000kN
 Pile property: Elastic plastic, reinforcement 12r25 (cover 200mm!)(MNk see appendix for appropriate diagram)

Summary of results

Capacitycurve Buis ø800 12R25	
Fy*	585 kN
grp. Eff. Red.	-30,00%
Fy* (red)	409 kN
A	227
Fbase*	92950 kN
Em*/Fy*	0,056 m
dy*	0,038 m
M	240000 kN
m*	24465 Ton
T*	0,629 s
Sde	0,066 m
Sd(T)	0,673 g
q	1,74
Failure of section @	0,040 m



Conclusion

- The fundamental period is slightly lower than the current non-seismic design. This slightly lowers the displacement demand.
- The pile fails @40mm, which is lower than the demand.

Possible solutions

- Change the approach to an upperbound solution with very large stiff piles.

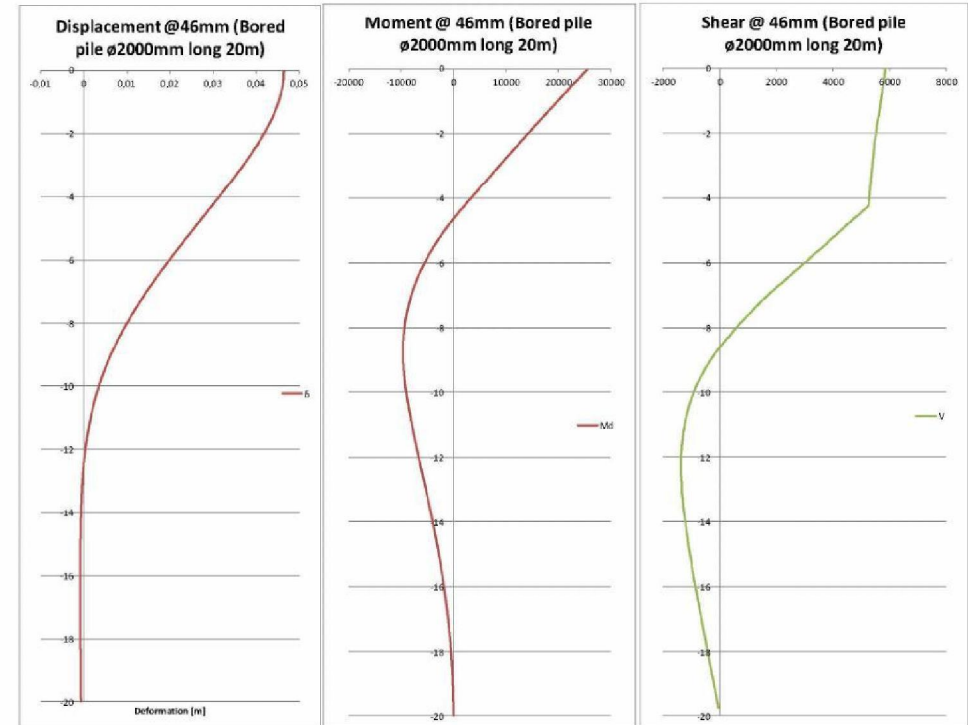
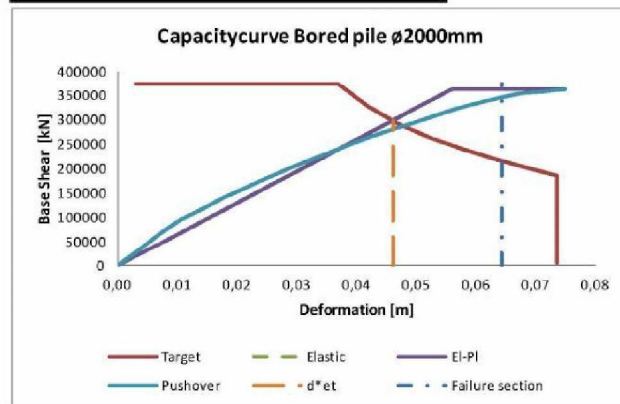
Variant 5: Redesign with large Bored piles ϕ 2000mm length 20m

Pile Data

Paaltype: Bored pile ϕ 2000mm
 Amount: 80 (under treatment building)
 Pile force: 3900kN (due to additional mass of bunker, 1m floor +80 piles ϕ 2m)
 Pile property: Elastic plastic, reinforcement 2x40r40 top 8m, 40r40 remained (MNk see appendix for appropriate diagram)

Summary of results

Capacitycurve Bored pile ϕ 2000mm	
Fy*	7564 kN
grp. Eff. Red.	-40,00%
Fy* (red)	4539 kN
A	80
Fbase*	363083 kN
Em*/Fy*	0,047 m
dy*	0,056 m
M	310000 kN
m*	31600 Ton
T*	0,439 s
Sde	0,046 m
Sd(T)	0,964 g
q	1,34
Failure of section @	0,064 m



Conclusion:

- The fundamental period is significantly lower than the current non-seismic design. And the capacity is higher than the displacement demand.
- The increase in stiffness, comes at a cost. The forces in the structure increases and the pile head moment need to be transferred to the structure. Therefore the thickness of the ground floor has been altered to 2m.
- The pile fails @64mm, which is lower than the demand.

Concerns

- The forces arising from this approach as significant.
- The large diameter bored pile need to be anchored to the ground floor.
- This solution introduces large eccentricities on plan level.

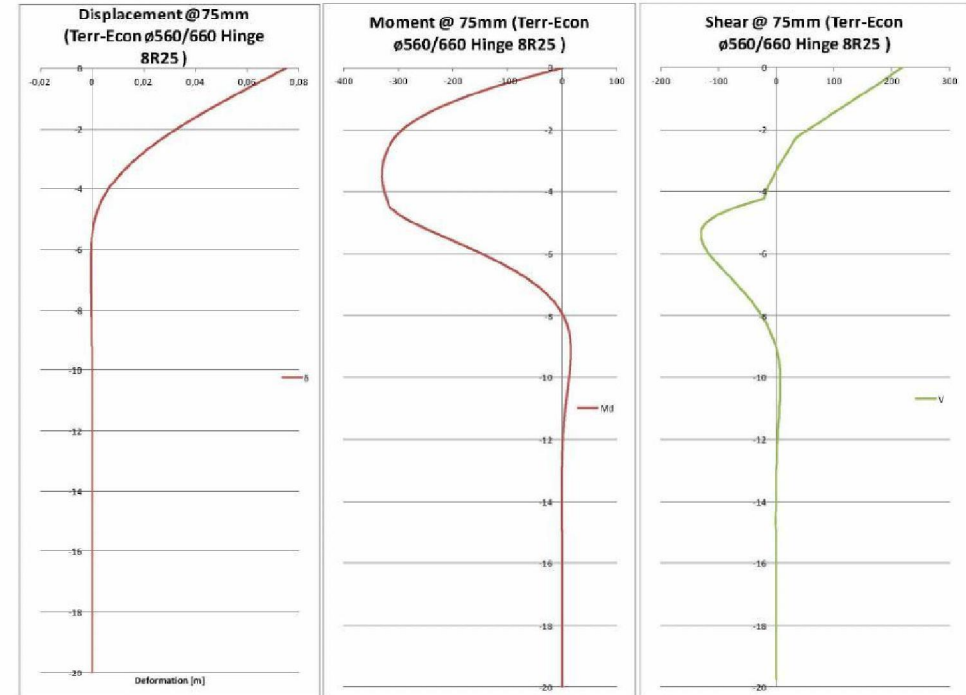
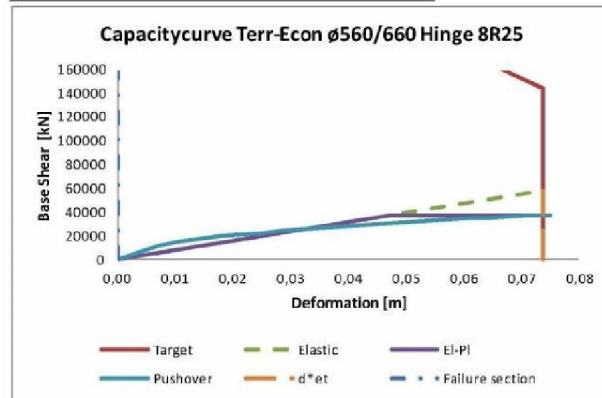
Variant 6: 227 Terr-econ pile with hinge at top of pile

Pile data

Pile type Terr-econ paal rond 560/660 no grout-injection
 Amount 227 (under treatment centre)
 Pile Force 1000 kN
 Pile property Elastic plastic, full length reinforcement 8r25 (MNk see appendix for appropriate diagram)

Summary of results

Capacitycurve Terr-Econ ø560/660 Hinge 8R25	
Fy*	237 kN
grp. Eff. Red.	-30,00%
Fy* (red)	166 kN
A	227
Fbase*	37680 kN
Em*/Fy*	0,051 m
dy*	0,047 m
M	240000 kN
m*	24465 Ton
T*	1,100 s
Sde	0,074 m
Sd(T)	0,245 g
q	1,56
Failure of tube @	mm



Conclusion:

The pile is able to follow the movement of the superstructure. This occurs without yielding of the pile. However there are certain concern on applying such a design.

Concerns:

- Detailing of a hinge with limited momentcapacity, and maximum shear capacity
- Only a single plastic hinge mechanism is possible, which is less safe
- P-δ effect in piles due to soil as a load and as a support
- Buckling length is always greater than 2L, where $L \approx 5m$
- The sand layer @ -4,25m is possibly susceptible to liquefaction, which will cause a larger buckling length to increase and yield an unsafe design.

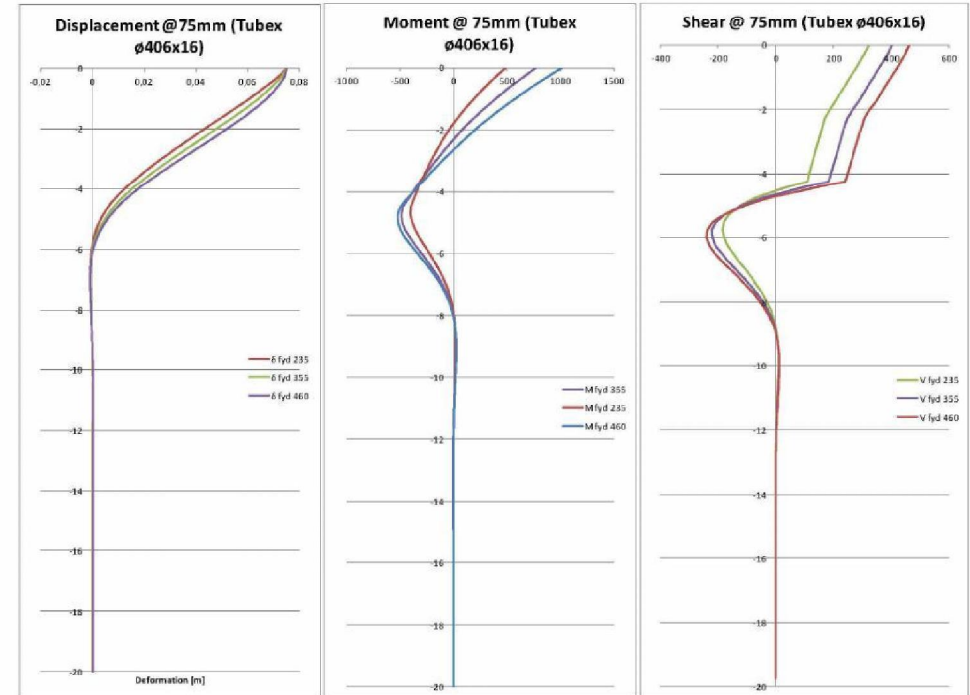
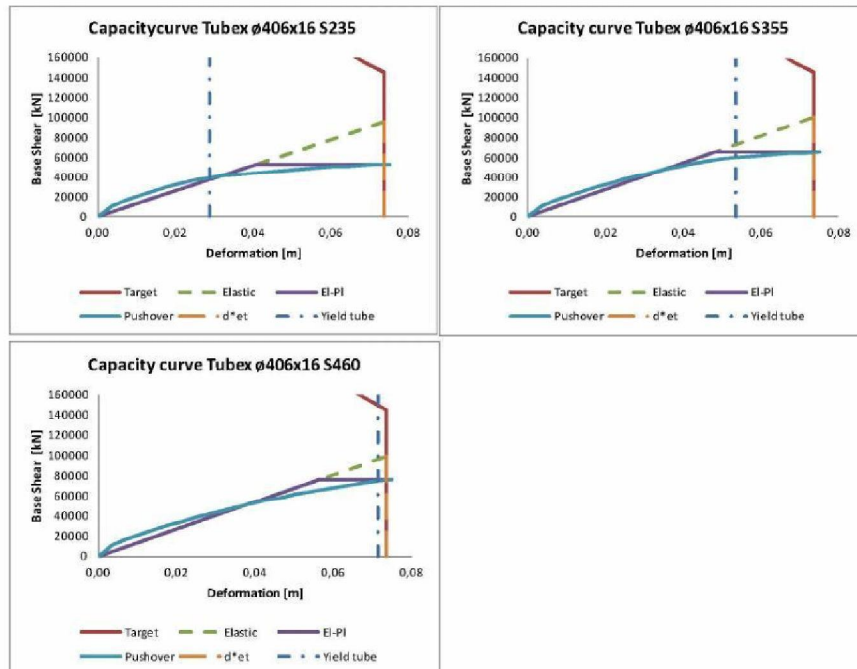
Variant 7: 227 Tubex pile (screwed tube pile) $\phi 406 \times 16$

Pile data

Piletype: Tubex pile with bearing external steel pile 406x16, unreinforced concrete core
 Amount: 227 (under treatment centre)
 Pile force: 1000kN
 E-modulus: 210000N/mm², yield strength 235,355 and 460 N/mm² (MNk see appendix for diagram)

Summary of results

Capacity curve Tubex $\phi 406 \times 16$ S235		Capacity curve Tubex $\phi 406 \times 16$ S355		Capacity curve Tubex $\phi 406 \times 16$ S460	
Fy*	335 kN	Fy*	417 kN	Fy*	478 kN
grp. Eff. Red.	-30,00%	grp. Eff. Red.	-30,00%	grp. Eff. Red.	-30,00%
Fy* (red)	235 kN	Fy* (red)	292 kN	Fy* (red)	335 kN
A	227	A	227	A	227
Fbase*	53411 kN	Fbase*	66271 kN	Fbase*	75959 kN
Em*/Fy*	0,055 m	Em*/Fy*	0,051 m	Em*/Fy*	0,047 m
dy*	0,041 m	dy*	0,048 m	dy*	0,057 m
M	240000 kN	M	240000 kN	M	240000 kN
m*	24465 Ton	m*	24465 Ton	m*	24465 Ton
T*	0,861 s	T*	0,839 s	T*	0,848 s
Sde	0,074 m	Sde	0,074 m	Sde	0,074 m
Sd(T)	0,400 g	Sd(T)	0,421 g	Sd(T)	0,412 g
q	1,80	q	1,52	q	1,30
Yield of tube	0,029 mm	Yield of tube	0,054 mm	Yield of tube	0,071 mm



Conclusion

Using high yielding steel as piles allows for a feasible seismic design. In all 3 cases the natural period is more or less constant. Using the steel grade as a variable it is possible to obtain a fit-for-purpose design.

Concerns

- Embedding the pile head in the foundation demands special attention to the detailing.
- Corrosion of piles during lifecycle

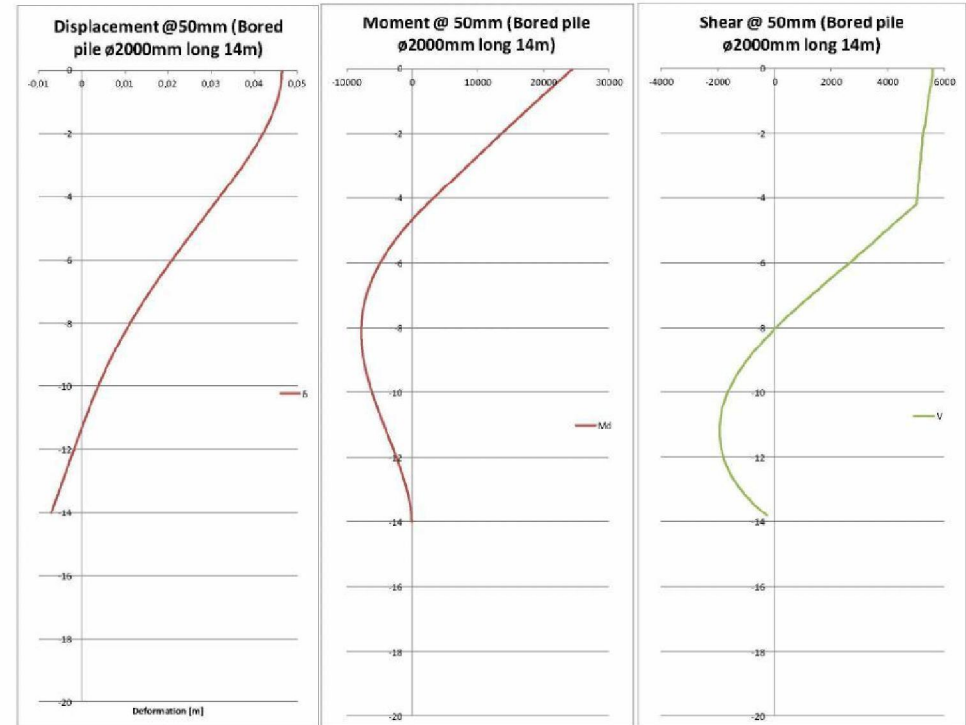
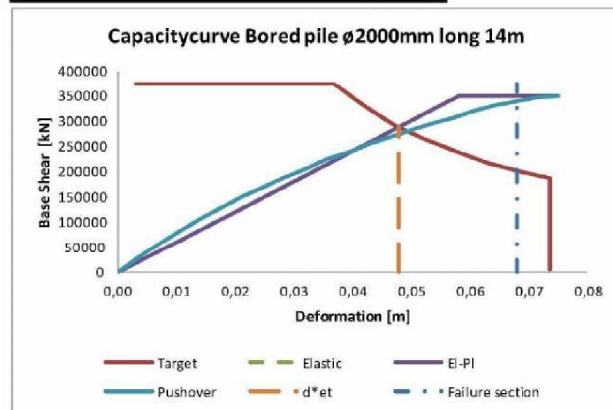
Variant 8: Redesign with large Bored piles ϕ 2000mm length 14m

Pile Data

Paatype: Bored pile ϕ 2000mm
 Amount: 80 (under treatment building)
 Pile force 3900kN (due to additional mass of bunker, 1m floor +80 piles ϕ 2m)
 Pile property Elastic plastic, reinforcement 2x40r40 top 8m, 40r40 remained (MNk see appendix for appropriate diagram)

Summary of results

Capacitycurve Bored pile ϕ 2000mm long 14m	
Fy*	7331 kN
grp. Eff. Red.	-40,00%
Fy* (red)	4399 kN
A	80
Fbase*	351893 kN
Em*/Fy*	0,046 m
dy*	0,058 m
M	310000 kN
m*	31600 Ton
T*	0,454 s
Sde	0,048 m
Sd(T)	0,932 g
q	1,29
Failure of section @	0,068 m



Conclusion:

- The fundamental period is slightly higher than the longer version. And the capacity is higher than the displacement demand.
- The increase in stiffness, comes at a cost. The forces in the structure increases and the pile head moment need to be transferred to the structure. Therefore the thickness of the groundfloor has been altered to 2m.
- The pile fails @68mm, which is lower than the demand.

Concerns

- The forces arising from this approach as significant.
- The large diameter bored pile need to be anchored to the ground floor.
- This solution introduces large eccentricities on plan level.
- Quantification of the groupeffect, more group effect reduces the global stiffness and increased the demand.

Variant 9: 227 Tubex pile (screwed tube pile) $\phi 406 \times 16$, with gap

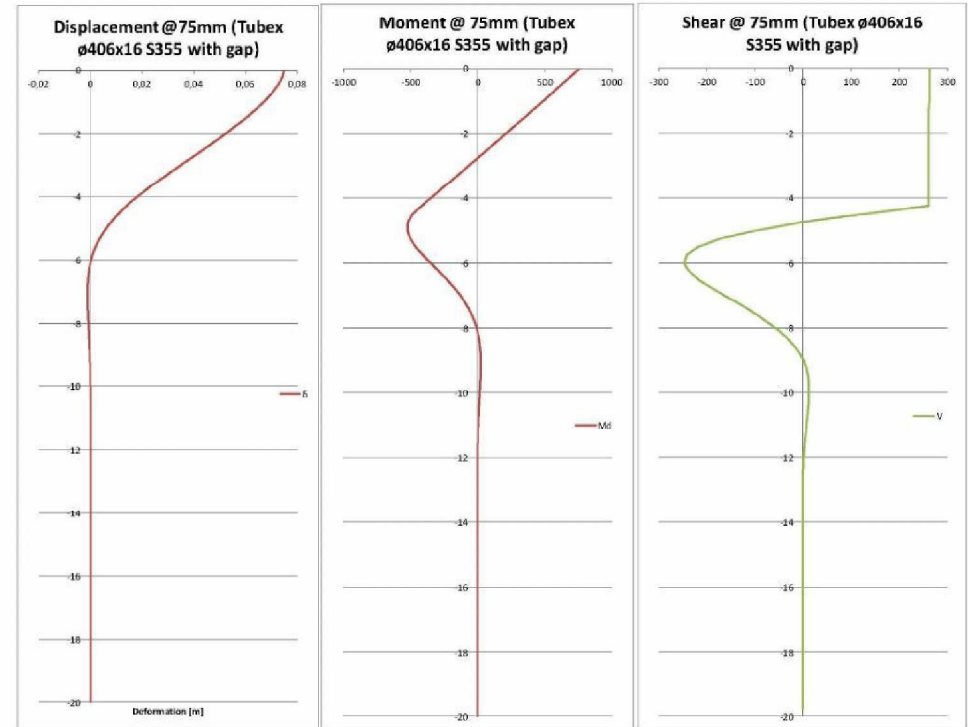
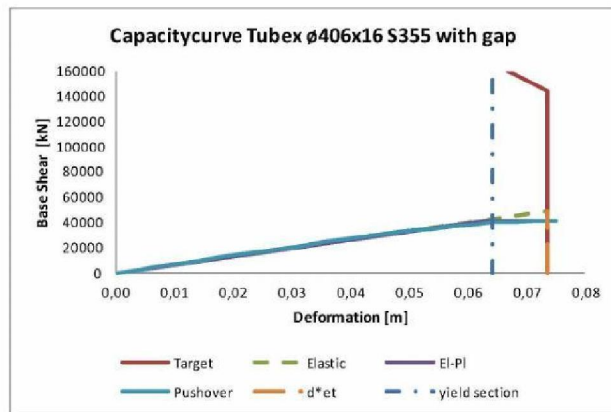
Pile data

Piletype: Tubex pile with bearing external steel pile 406x16, unreinforced concrete core
 Amount: 227 (under treatment centre)
 Pile force: 1000kN
 E-modulus 210000N/mm², yield strength 235,355 and 460 N/mm² (MNk see appendix for diagram)

The lateral springs of the top 4,25m will be omitted from the analyses. This to investigate the possibility of creating a filled gap the decrease the seismic demand.

Summary of results

Capacitycurve Tubex $\phi 406 \times 16$ S355 with gap	
Fy*	263 kN
grp. Eff. Red.	-30,00%
Fy* (red)	184 kN
A	227
Fbase*	41786 kN
Em*/Fy*	0,044 m
dy*	0,063 m
M	240000 kN
m*	24465 Ton
T*	1,205 s
Sde	0,074 m
Sd(T)	0,204 g
q	1,19
Yield of section @	0,064 m



Conclusion

Creating a medium between the pile and clay will reduce the shearforce and increase the displacement at which the section will yield.

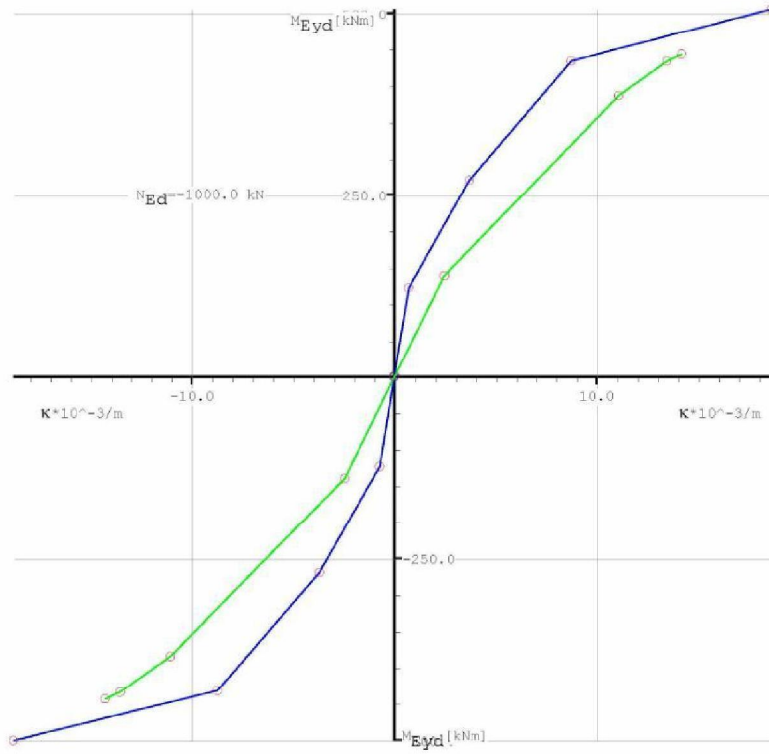
Concerns

This solution comes with the number of drawbacks with regard to the feasibility of such a system.

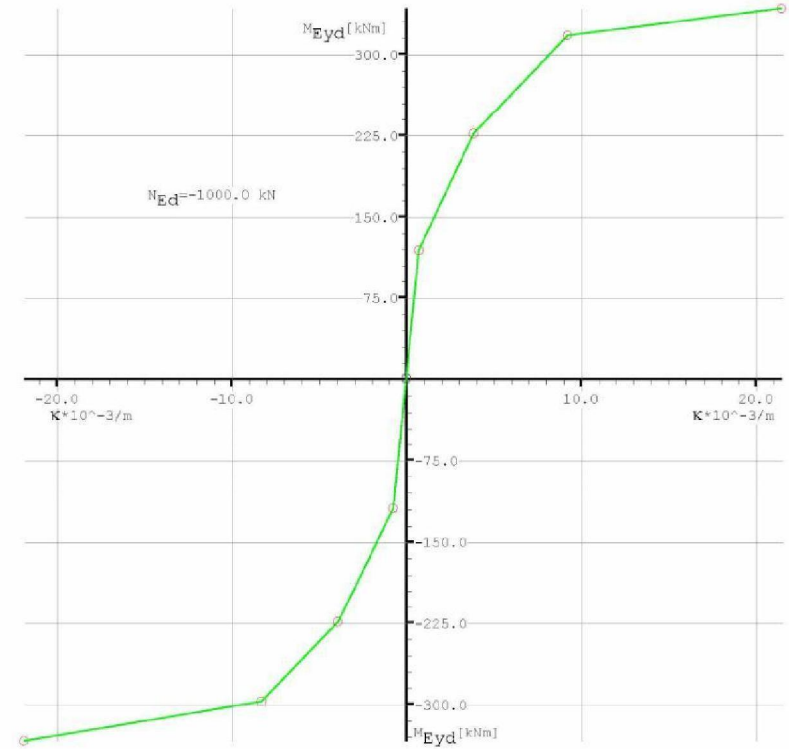
Moment-curvature relations

Round 560mm 8r25 Fd = 1000kN

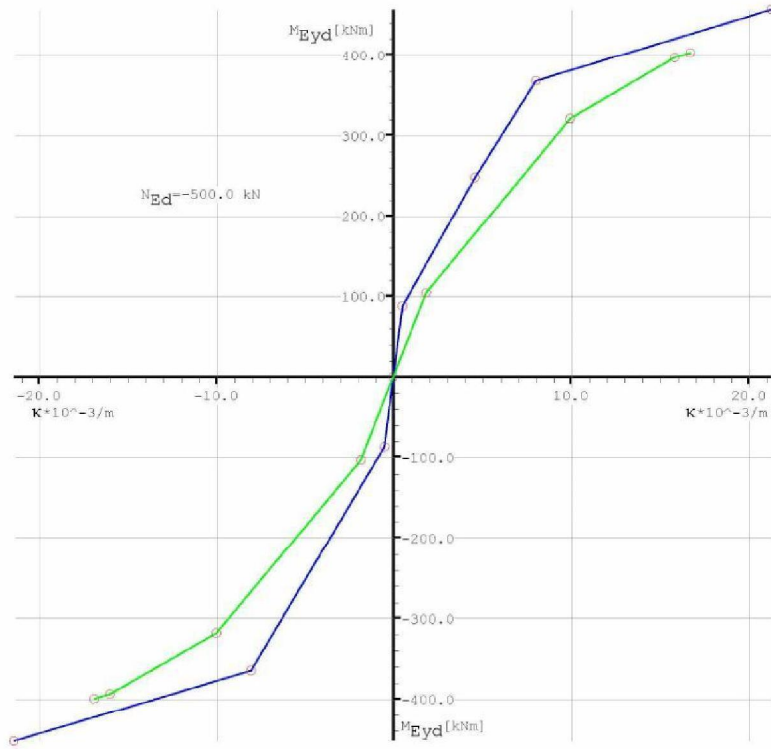
The blue line denotes the used curve



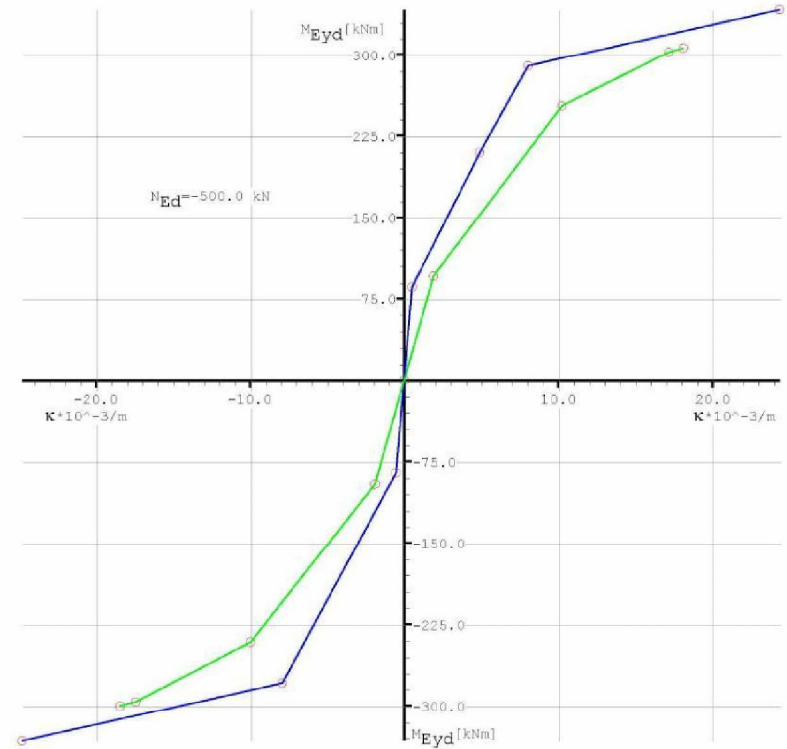
Round 560mm 5r20 Fd = 1000kN



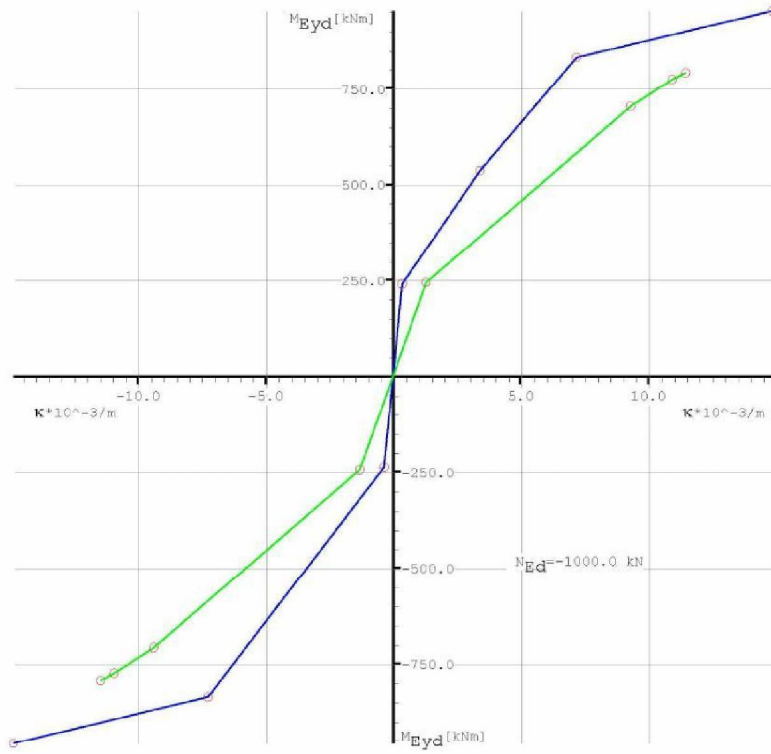
Round 560mm 8r25 Fd = 500kN
 The blue line denotes the used curve



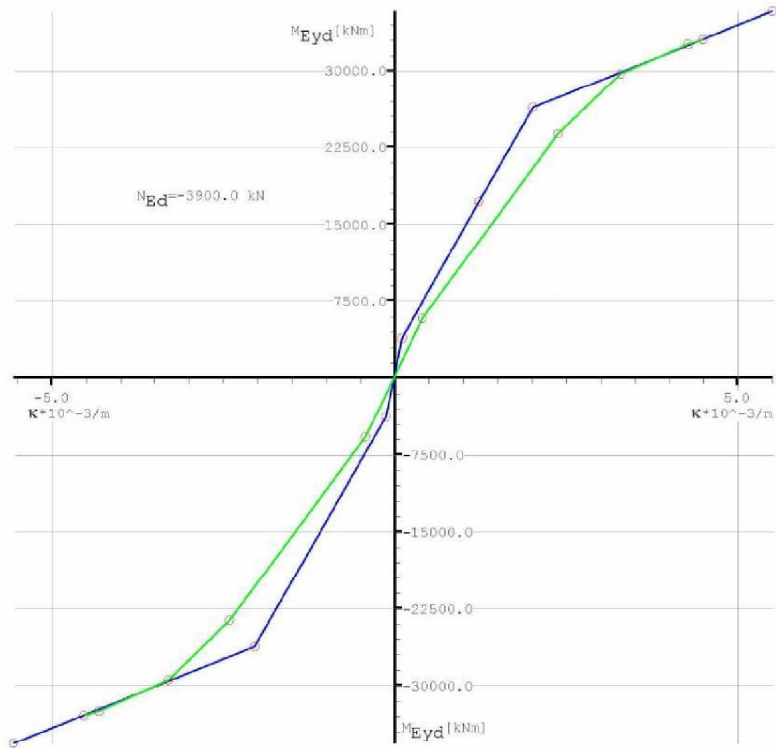
Round 560mm 5r20 Fd = 500kN
 The blue line denotes the used curve



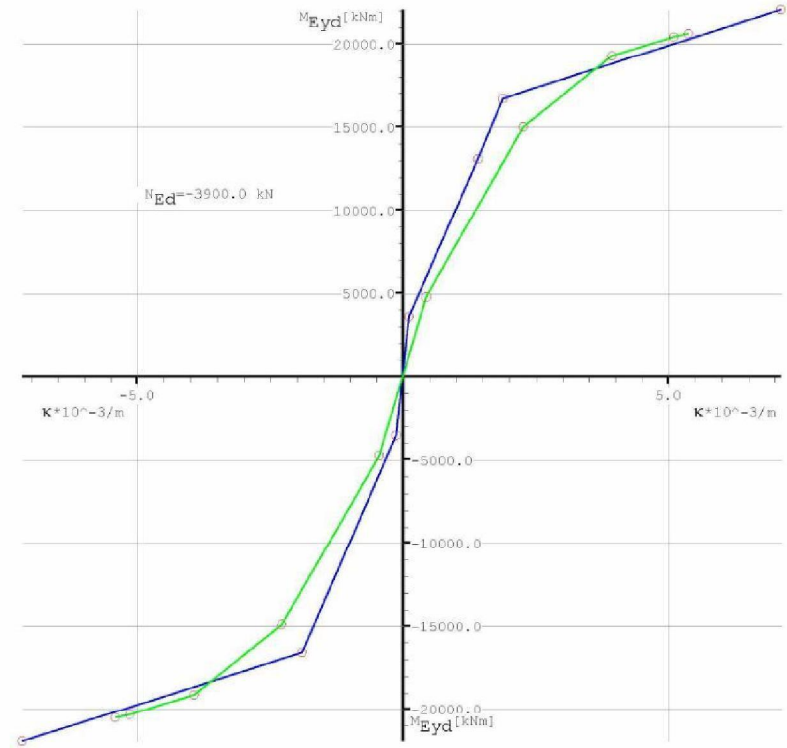
Round 800mm 12r25 Fd = 1000kN
The blue line denotes the used curve



Round 2000mm 2x40r40 Fd = 3900kN
 The blue line denotes the used curve



Round 2000mm 40r40 Fd = 3900kN
 The blue line denotes the used curve



Steel pile

The steel moment curvature relation is determined using the following relations. The contribution of concrete has not **yet** been taken into account.

$$M_{el} = W_{y,el} * (f_{yd} - F/A)$$

$$M_{pl} = W_{y,pl} * f_{yd} * (1 - N_d/N_{pl}) \text{ (linear interaction)}$$

$$\kappa_{el} = M_{el} / EI$$

$$\kappa_{pl} = \kappa_{el} * 2 \text{ (50\% of the section has yielded)}$$

