# Analysis of the effects of deterioration of a wooden pile foundation in masonry abutments


By

Jurre Knijff


A thesis presented for the degree of
Bachelor of Science


Civil Engineering
University of Twente


June 2019

# Colophon

| | |
|---|---|
| Title | Analysis of the effects of deterioration of a wooden pile foundation in masonry abutments |
| Date | June $28^{th}$ 2019 |
| Version | Final Version |


**Personal information**

| | |
|---|---|
| Author | Jurre Knijff |
| Student number | 1882228 |

**TNO Supervisor**

| | |
|---|---|
| Name | dr. A.T. Slobbe |
| Function | Medior Scientist Specialist |

**UT Supervisor**

| | |
|---|---|
| Name | dr. A. Hartmann |
| Function | Associate Professor |

# Preface

This thesis is an exploratory research into the behaviour of a masonry abutment under the influence of a deteriorating pile foundation. This report is the conclusion of a ten week internship at the Stuctural Reliability group within TNO and the finalisation of my bachelors at University of Twente.

This report could not be done without the help of a number people. I would like to thank both of my supervisors for their help and feedback during the thesis. I want to thank my daily supervisor, Arthur Slobbe, for his guidance and giving me the possibility of doing my bachelor thesis within TNO. I have learned a lot not only about doing research itself, but also about how to document the produced knowledge on paper. I will thankfully take all of his feedback into account during the later stages of doing research. I also want to thank the whole research group at TNO for the things i learned within and about TNO, doing research in civil engineering and the amazing atmosphere within the group. I want to thank Sebastiaan van Rossum and the team at Ingenieursbureau Amsterdam for their guidance and insights into working at a governmental organisation.

Last, I want to thank my friends and family for their support during the months of my bachelor thesis, without whom I would not have been able to complete this thesis.

Jurre Knijff
Delft, June 28$^{\text{th}}$

# Abstract

The aim of this thesis is to find out how a masonry abutment behaves under the circumstances of a deteriorating wooden pile foundation.

A case study was chosen, containing information about the current and possible deterioration of four piles under the abutment. These values were used as input for different scenarios within the analysis. The pile foundation was modelled using a soil structure interaction software, specifically made to convert piles and soil into spring stiffness values. The spring stiffness values were used together with the abutment in a finite element software, in which multiple deterioration scenarios were applied. Within the scenarios, either groups of piles were highly deteriorated or the whole foundation had a certain deterioration. Using these scenarios, a linear static analysis was executed to analyse the behaviour of the abutment.

The results of the analysis showed that the movement of the abutment in line with the bridge was about 22 millimetres towards the water without deterioration. In a high deterioration scenario, the abutment moved 0.5% more. Considering vertical movement, the abutment movers up to 12 millimetres downwards without deterioration and 10 % more with the high deterioration case scenario. A stress analysis was also done, which showed that the tensile stress in the abutment exceeded the limit of the masonry, even without deterioration.

To conclude, a difference between the scenarios could be found in the vertical displacement, but relatively little difference could be found in the movement in the direction of the bridge span. Moreover, the tensile stress in the masonry exceeded the maximum tensile stress of the material. Therefore, a non-linear analysis should be performed to give an insight into the abutment behaviour after crack forming.

# Contents

# 1 Introduction

## 1.1 Background

### 1.1.1 The Amsterdam situation

According to Gemeente Amsterdam (2019), about 250 bridges and 200 kilometres of quay wall contain a wooden pile foundation. Although only a very small number of bridges has currently been closed due to a wooden pile foundation being in a bad state (AT5, 2018), there are expectations that the state of the wooden pile foundations will worsen. The climate and changing phreatic level in the city of Amsterdam are could exacerbate the deterioration in the wooden piles (de Putter, 2013), creating the need to inspect the piles more often. However, since the piles are often below ground and cannot easily be inspected, the wooden foundation is one of the hardest parts of the bridges inspect. Therefore, Ingenieursbureau Amsterdam is looking into the possibilities of using indirect measurements to predict the structural state of the pile foundation. To achieve this, the process of structural health monitoring could be used. Within structural health monitoring, the structural behaviour of the bridge is monitored, after which the collected data is used to improve the assessment of the bridge.

### 1.1.2 Towards a probabilistic damage identification tool

TNO and Ingenieursbureau Amsterdam have the plan to work on the development of a SHM tool for bridges with wooden pile foundations via a MSc thesis work. The core idea is to update our knowledge of the (damage) state of the wooden pile foundation using data from easy-to-measure response of the structure, and to use this information in the assessment of the bridge. This so-called damage identification requires a mechanical model, such as a finite element model, needs to be made to use the measurements as damage parameters. However, both the model and the measurements have uncertainties, which need to be taken into account when doing any prediction. Therefore, both the model parameters and the monitoring data should be embedded into a probabilistic framework. Using this probabilistic framework in the finite element model, the likelihood of certain damage parameters occurring can be determined, based on the variability of input parameters.

## 1.2 Problem statement

The bachelor thesis is a start of the probabilistic damage identification took and is a extract of a masters thesis at TNO[1]. This bachelor thesis will not focus on the whole masters thesis, but will focus on a selection of objectives. The aim of this thesis is to examine masonry abutment behaviour under the circumstances of a deteriorating wooden pile foundation. Therefore, main research question is: What is the effect of a deteriorating wooden pile foundations on the structural response of a masonry abutment? To answer the main question, sub-questions have to answered:

[1] https://www.tno.nl/en/career/vacancies/internship-probalilistic-structural-health-monitoring-in-masonry-bridges-with-a-wooden-pile-foundation/vacid-a0s0x000013dzmnuaw/

- What information is currently available on bridge monitoring in Amsterdam?
- How can we model the masonry abutment and wooden pile foundation?
- What structural behaviour is relevant to monitor?

## 1.3 Approach

The research will consider a small portion of the 250 bridges which is considered to have a wooden pile foundation in Amsterdam. This subset is bounded by the fact that wooden pile foundation is present and the foundation does not contain a coupled floor. The coupled floor will be further explained in subsection 2.1, together with the reason why these bridges are excluded. The research will start with an archive research into the subset of bridges, focusing on the typical characteristics of the subset. Based on the desk research, a case study bridge is chosen, which will be modelled using a finite element software. The case study is chosen based on the available information on geometry and materials, but also on the amount of monitoring information available on the bridge. After learning the finite element software, the case study is modelled within the software. The soil and piles are modelled in a different software, transferring the soil structure interaction into springs. The case study is then analysed within the finite element software, while using the output of the soil structure interaction software. Based on the outcome of the finite element analysis, the main question is answered.

## 1.4 Report Structure

In the following sections the research will be described. Section 2 focuses on the bridges that have been analysed in the desk study. The section starts with a description of the structure of the bridge, based on the 11 bridges in the desk study. After the overall structure is described, the research dives deeper into the possible damage scenarios and how the bridges are currently monitored.

In section 3, a description of the case study 'Bridge 137' is given, starting with its geometry and components and ending with going deeper into the monitoring as currently done on the bridge.

Section 4 will describe the finite element model used in the analysis. The section will start with defining the units and continue with information in the geometry simplifications. After the geometry is given, the material models and parameters are given for the finite element model. Then, the materials used for the soil structure interaction are given, in which the spring values are described. The section will continue with describing the mesh and elements used in the model. the section will end with the loads that are applied in the model.

Section 5 will go into the results. The section starts with describing the different scenarios that were applied, based on the current and possible deterioration in the piles. The section will then describe the model checks, which are based on hand calculation. The section will continue with describing the results, which are analysed in the discussion of the section.

The report will end with a conclusion and a number of recommendations for future studies into the behaviour of the abutment.

# 2 Archive research into bridges with masonry abutments and wooden piles

The full archive research can be found in appendix A. This section gives a general description of the structure of the bridges, highlighting the relevant similarities and differences between the 11 bridges within the subset. After the similarities and differences are given, the possible failure scenarios are given. These are based on information from the desk study, available in the municipality of Amsterdam. The current monitoring within the municipality of Amsterdam will be described in the last part of this section.

## 2.1 Description of the bridges from a structural point of view

This subsection gives an overview of the bridges and starts with looking into the geometry of the bridges. After a geometry, the subsection gives a structural description, starting with the deck and working its way down through abutment to the foundation piles.

### General information

An overview of the bridge locations in Amsterdam can be seen in figure 1. Out of the 11 bridges that have been analysed in the research, eight are located in the ‘Centrum’ district. Bridges 184 and 137 are not located in the centre, but are respectively in the ‘West’ and ‘Zuid’ district of Amsterdam. Bridge 170 is the only bridge between districts and is built in between ’Centrum’ and ’West’.

Although most of the bridges have been widened and parts were replaced. However, the foundation of the bridges has never been replaced. The first construction year is given in figure 2. In this figure, it is very clear that there are two peaks. The first peak is around 1750 in which the bridges 8, 29, 30 and 41 were built. These bridges span the canals of the Grachtengordel, a large expansion of the Amsterdam centre area in the 18$^{\text{th}}$ century(Museum Amsterdam, n.d.). The second peak is around 1900, explained by the expansion of the city at the end of the 19$^{\text{th}}$ century(Museum Amsterdam, n.d.).

An insight into the length of the bridges is given in figure 3. The bridges 15, 110 and 295 are supported by abutments only and do not have extra support points. The length of these bridges is about 7 to 8 meters. The larger bridges have two abutments and are also supported by two pillars. The bridges as build in the first peak of the building year are very close together, being about 20 meter in total. Bridges 137, 170 and 315 are all longer, increasing in length in the same order.

### Deck

From the desk research a conclusion was that the four bridges of which the deck was described, only two of the bridge had the same deck type. The three deck types all exist out of steel and concrete, but the way in which the steel beams are placed is different. The three types are visible in figure 4. The first deck is a composite deck, placed in bridge 137. A composite deck is visualised in figure 4a, seen in the direction of the span. A composite decking consists out of waved

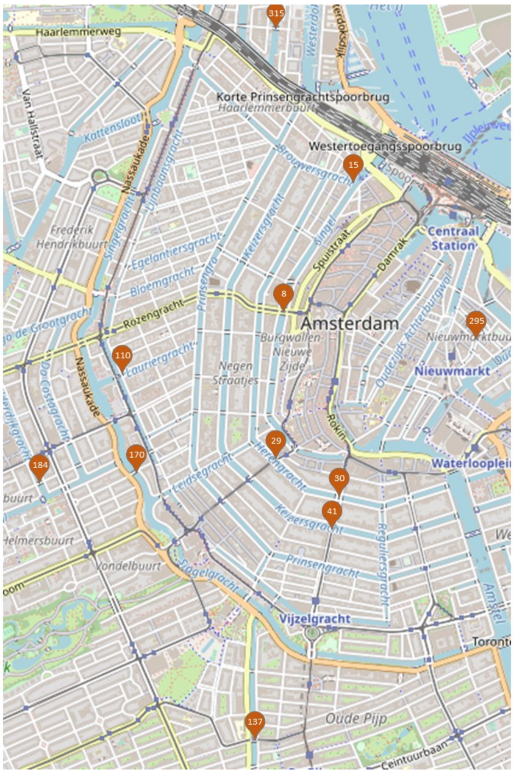



Figure 1: Map of the centre of Amsterdam showing the location of the analysed bridges (OpenStreetMap contibuters, n.d.)

steel plates which overlap on the edges. Onto the waved steel plate, a concrete floor is poured.

The second type is visible in figure 4b. This is a simple concrete deck, supported by steel beams. This type is used in bridge 8.

The third deck type is visible in figure 4c and is in shape similar the composite deck, except for the fact that the steel beams are placed in the concrete, instead of under. This type of deck is used in bridges 41 and 30.

### Masonry abutment and pillars

The abutment of the bridge transfers the load from the deck into the foundation. The corresponding geometry to the terms 'length', 'thickness' and 'height' is given in figure 5. When looking at the collection of bridges, the average abutment thickness is about 1.5 metres. The abutment of bridge 30 has the lowest thickness, being 1.21 metres thick. The thickest abutment is part of bridge 170, being 1,76 metres thick. The variation in pillar thickness is similar, with the steel pillars being around 50 centimetres thick, while the masonry pillars from bridges 8, 29 and 30 are all 1.43 metres thick.

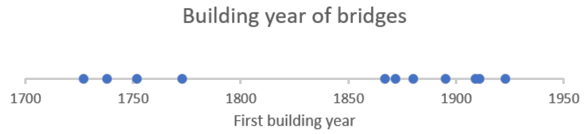



Figure 2: Two peaks of building years in the subset

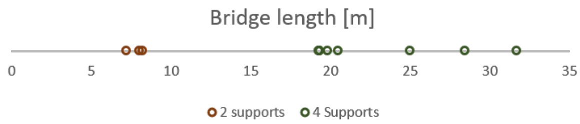
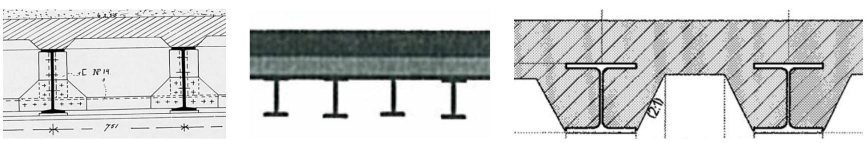



Figure 3: A large variation in length for bridges with 4 support points

### Wooden pile foundation

The bridges all have a similar foundation build up, which is visualised in figure 6. The foundation consists out of piles and on top of these piles a ground beam, which is placed in the direction of the span of the bridge. The number of piles under a ground beam differs, varying between 3 and 7 piles, even differing under the same abutment. On top of these ground beams, a wooden floor is placed. The flooring is horizontally secured by shear beams, which also functions as part of the flooring under the abutment. An abutment is built onto the flooring and shear beam. During the research, a coupled floor was found under one of the bridges. A coupled floor is visible in figure 7a, which shows that the ground beam continues throughout the spans, in opposition to figure 7b. Coupled floors were used in former sluices, of which most were later converted into bridges. The reason why bridges with a coupled floor are excluded from the research is that the bridge abutment foundation cannot be inspected and it is therefor impossible to either conclude or reject pile foundation failure. This is crucial for the validation of the model, since the model is supposed to predict pile failure. However, if the model proves itself in validation, the model could be used in bridges with a coupled floor without the need for validation before taking safety measure.

The bridges beams, flooring and piles are all pine wood. The foundation piles under the pillars are in two cases not pine, but oak or jarrah. The length of the piles ranges from 13 meters to 15 metres.



(a) Composite deck (Gemeente Amsterdam, 1909a)

(b) Concrete deck, supported by steel beams (Neijzing, 2011)

(c) Steel beams surrounded by concrete (Quansah, 2014)

Figure 4: Different types of deck used in four of the analysed bridges

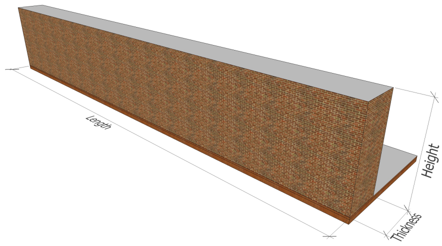



Figure 5: The defined height, thickness and length of the abutment

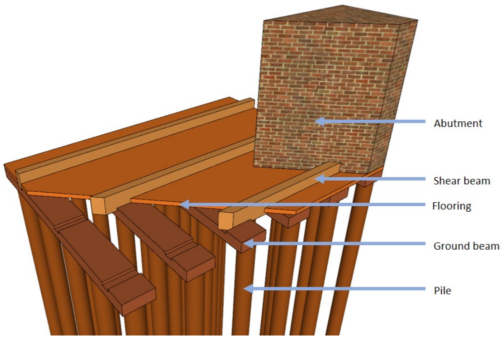


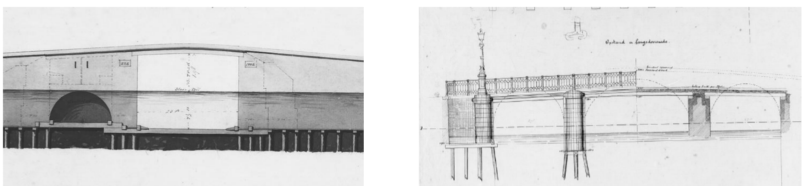

Figure 6: Typical build-up of the foundation used under the analysed bridges

**Soil conditions**

The soil conditions in Amsterdam are different in each of the bridge surrounding areas. However, a few similarities can be found when looking at the soil samples and cone penetration tests (CPT). In most cases, a thick layer of sand is present around 14 meters below ground level. Above this sand layer, it is mostly layers of clay, which some layers of peat. Sometimes the layer of peat is only a few dozen centimetres thick, while in some cases, such as bridge 41, the layer can be multiple meters thick.

## 2.2 Damage scenario's

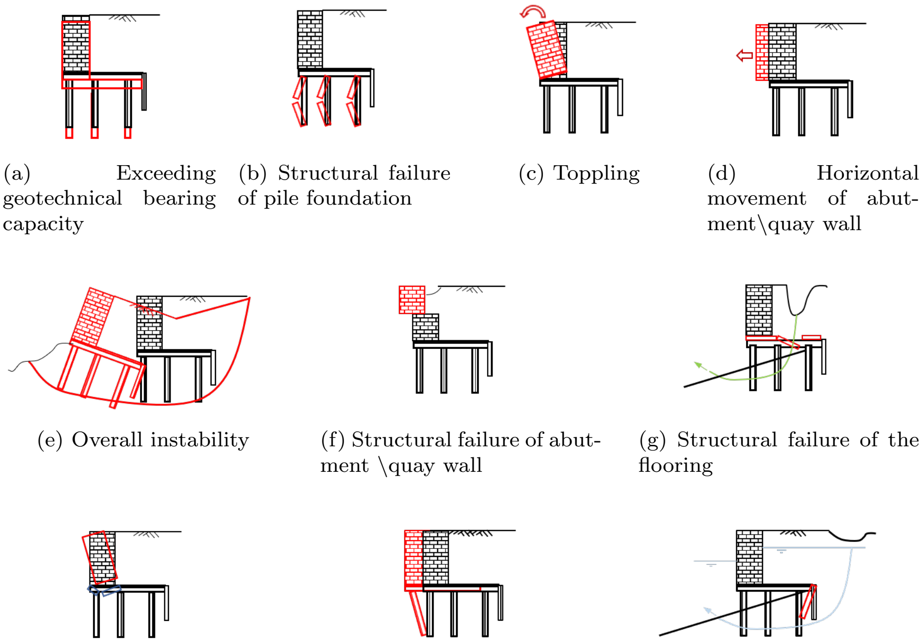
Over the past years, Ingenieursbureau Amsterdam (IBA) has done research into the possible failure scenarios in both quay walls and bridges. Within this thesis



(a) A bridge with a coupled floor (Gemeente Amsterdam, 1802)

(b) Bridge without a coupled floor (Gemeente Amsterdam, 1921)

Figure 7: Two types of wooden pile foundations



(a) Exceeding geotechnical bearing capacity
(b) Structural failure of pile foundation
(c) Toppling
(d) Horizontal movement of abutment\quay wall
(e) Overall instability
(f) Structural failure of abutment \quay wall
(g) Structural failure of the flooring
(h) Structural failure of ground beams
(i) Horizontal movement of overall structure
(j) Absence of a cut off wall

Figure 8: Failure mechanisms on quay walls (Neijzing, 2019)

research, the bridges are compared to quay walls, since the abutments in the research are structurally not different than the quay walls. The only exception is that the quay wall is not loaded with a bridge deck. Over the last years, IBA has listed ten damage scenarios that could occur in quay walls (Neijzing, 2019). The ten damage scenarios are described below. Each section starts with a explanation of how this occurs and an overview of how this influences the movement of the abutment and quay wall. Since this research is mainly focused on the pile foundation, the pile foundation failure will be dealt with more extensively. The ten failure mechanisms are visualised in figure 8.

### Exceeding the geotechnical bearing capacity

One of the failure mechanism that could occur under a quay wall in Amsterdam is exceeding the geotechnical limit, visible in figure 8a. This could be caused by three reasons. The first possibility is that the loading of the quay increased over time. This is a serious issue in Amsterdam, since a part of the bridges and quay walls in Amsterdam were build in times where horse carts were the determining load, while in these days, trams and trucks are still using the same structure. One of the ways to solve these issues are setting a weight limit in the bridge or strengthening the foundation.

Another way that this failure could occur is the loss of positive skin friction over time, possibly resulting into negative skin friction. This could occur due to the settlement of the ground around the pile being larger than the movement of the pile itself. Due to this settlement, the ground moves downwards, pulling the pile down with the settlement.

A third reason for exceeding the geotechnical bearing capacity is due to the tip of the pile. Due to deterioration of the pile tip, the bearing capacity of pile could decrease, such that the pile soil interaction fails and the pile lowers to such extend that the bridge and abutment could be damaged.

The results of this failure mechanism would be vertical movement of the abutment, and rotational movement when a portion of all piles fail.

**Structural failure of the pile foundation, flooring or ground beams**

The failure mechanics as shown in figures 8b, 8d and 8g are all dealt with in one *sub-subsection*, since all of the failure mechanisms are determined by the failure of timber elements in the abutment. The shear beam is however not considered here, but is considered later on.

The three failure mechanisms could occur due to decay. The most common decay types are bacterial decay and fungal decay. Clausen (2010) states that fungal decay could occur in different types, distinguishing brown, white and soft rot. According to Clausen (2010), brown rot removes the cellulose. The cellulose is mainly present in the $S_1$, $S_2$ and $S_3$ layer of the structure, as can be concluded from figure 9. Brown rot gives the wood, as the name suggests, a brown colour, might show cracks across the grain and collapses. This shows that brown rot can be very dangerous for bearing elements, such as the parts in the abutment. The white rot however removes both the cellulose and the lignin, which leads to a white colour, but does not crack across the grain and does not shrink (Clausen, 2010). Blanchette, Nilsson, Daniel, and Abad (1990) however, mentions that there are multiple types of white wood, either being selective or non-selective. This means that, depending on the type of white rot, the white rot can either attack the lignin or remove all cell wall components. The last kind of fungal decay that takes place is soft rot. Soft rot occurs on a thin layer of would and is mainly an influence on thin pieces of wood (Clausen, 2010). Although this does not seem to be of much importance to the piles in Amsterdam, it is also said by Clausen (2010) that the soft rot favours wet situations, unlike the brown and white rot. The Amsterdam piles are placed under water, which means that only soft rot could occur. However, due to changing water levels, the heads of the piles can be in dry conditions, giving the white and brown rot a chance to grow.

The other decay that takes place in a wooden foundation pile is bacterial decay. Bacterial decay has only been considered since the 1980s. Before that, pile deterioration was not expected under water and only the fungal was considered to harm the piles. Bacteria can be subdivided into tunnelling bacteria (TB) and erosion bacteria (EB). Klaassen (2008) mentions that the erosion bacteria mainly attack the S2 layer of timber, while the tunnelling bacteria attack all layers of the timber. The erosion bacterial decay leaves some of the layers relatively intact, since, according to Klaassen (2008), erosion bacteria do not attack the S3 layer and lamella. However, the tunnelling bacteria, does attack all layers.

Both of these decays can cause in the bearing capacity of the pile. This means that over time, the pile will be able to carry less weight. Therefor, the pile will not be able to structurally hold the pile and therefor collapses due to the force exerted onto the pile.

According to Neijzing (2019), the failure of a single pile could lead to small amount of damage, but is not dangerous for the surrounding area. However, when the number of piles that are structurally below the required values, deformation could occur, leading to deformation of the abutment and structural damage.

### Toppling of the wall

Toppling of the wall occurs when the ground behind the structure is too heavy for the structure to push back. This could be caused by a load, such as traffic, being applied on the quay wall or a large water pressure behind the wall. Toppling of the wall could also occur when the wall is pulled to hard, when for example ships are anchored to the wall. However, this does not easily occur in the case of bridges, since the bridge deck prevents the abutment form moving upwards.

### Horizontal displacement of the full wall

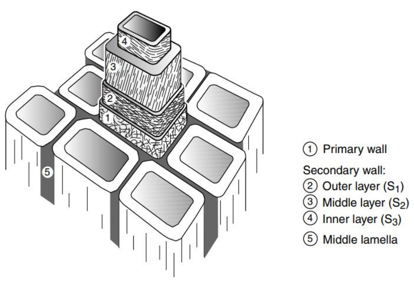
Another failure mechanism that could occur is when the wall moves sideways due to failure of the shear beams. Due to the absence of shear beams, the abutments is able to move over the flooring. This could happen due to the same reasons as mentioned in the former section into the pile failure. Since the shear beams are the most upper wooden part of the structure and therefor even more susceptible to deterioration by fungi, this could be one of the large issues .

### Exceeding the overall stability

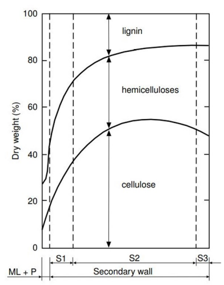
In the overall stability, the whole wall collapses, leading to failure in a large area around a quay wall or bridge abutment. The reason that this would occur is due to overloading of the whole structure, failure in certain parts of the geotechnical parts or due to large differences in phreatic level.

### Structural failure of the wall

Although this occurs very rarely (Sas, 2006), failure could occur in the masonry. The reason for this to occur is high shear and tension forces. High shear forces




(a) Schematic drawing of the layers in the cellular structure of wood (Persson, 2000)




(b) Chemical composition of the different layers (Panshin & de Zeeuw, 1970)

Figure 9: Overview of wood composition

could be caused by non uniform settlement of the structure, caused by the fact that the settlement of the original bridge is different compared to the settlement of extensions of the bridge. High tension forces could be caused by the abutment not being supported in middle parts, but this is not proven to take place in the bridges and quay walls in Amsterdam.

**Horizontal displacement of the structure**

Figure 8j describes the occurrence of the movement of the full structure in a horizontal way towards the water. According to Neijzing (2019), the failure could occur due to similar reasons as the stability of the full structure, adding the possibility of damaged pile foundation.

**The absence of a cut off wall**

The last scenario that IBA describes is the absence of a cut off wall, comparable to a grout curtain. The absence of a cut off wall could cause water to move from one side of the abutment or quay wall to the other side. When this occurs, potentially taking away soil, a water flow could occur, comparable to piping under a dike. When this water flow fully develops, the soil under the abutment or quay wall could fail, which causes overall stability failure. Another thing that could happen is that due to the absence of a shear wall, the water flow could flow to the other side and cause settlement on the dry side of the abutment or quay wall. This leads to structural damage in roads and roadside structures.

**Conclusion**

Most of the quay wall failures that are mentioned by Neijzing (2019) can be applied on the bridge abutments as well. However, the toppling of the abutment as shown in 8c is much less likely in the situation of the abutment. This is due to the extra weight of the bridge deck, which would likely prevent the bridge abutment from making upwards movement. The chance that this occurs in the case of a quay wall is higher, since nothing prevents this movement, except for the weight of the abutment. As mentioned, the masonry failure is also unlikely.

## 2.3 Current monitoring in Amsterdam

Within Amsterdam, two types of monitoring are done. The first one is a visual assessment of all above water parts of the bridge. The focus of this assessment is on Reliability, Availability, Maintainability, Safety, Health, Environment, Economics and Politics, therefore called RAMSHEEP. Although safety is one of the focus points, the assessment is not a structural assessment, but only a recommendation based on photographs. During the RAMSHEEP assessment, cracks and crack patterns within the abutment could however be analysed, giving reason to do a recalculation of the bridge. The output of the RAMSHEEP assessment is a report, giving a cost, priority scale and time indication for repairing the damage.

Another way of monitoring as done by Ingenieursbureau Amsterdam is a XYZ-measurement, using a theodolite. By doing these measurements, the X, Y and Z coordinates of points of the bridge can be compared to earlier measurements to determine the settlement and movement over time. The points are stationed on the bridge and in the surrounding area, which are represented as bolts in walls, doorposts or sewage covers. The deformation measurements are not taken every year, but are taken based on a earlier measurement or based on a recalculation of the bridge.

# 3 Case 'Bridge 137': Detailed description

Bridge 137 is chosen to be used as a case study, since the geometry has straight and perpendicular lines, making it easy to model the bridge in finite element software. Another reason to chose for bridge 137 is the availability of multiple studies into the stability of the bridge. Within the studies, tests have been done on the conditions of the bridge and foundation, which can be used as input for the bridge model. In the coming subsections, information will give insight into the buildup of the bridge and the material properties from desk research at Ingenieursbureau Amsterdam.

## 3.1 Description of bridge 137 from a structural point of view

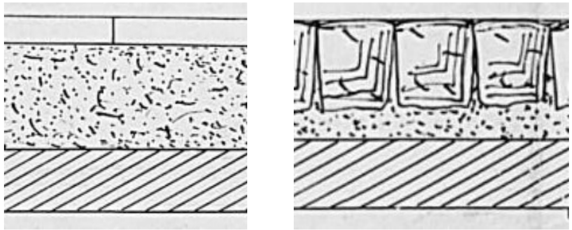
Bridge 137 is a three span bridge which is located in the Ruysdaelstraat, spanning the Boerenwetering. The bridge is made in 1909. The lane distribution is visible in figure 10a. The bridge has 3 lanes for trams and normal traffic, cycling lane on the north side of the bridge and sidewalks on both sides of the bridge. The bridge is 16.28 metres wide and 25 metres long. The bridge is supported by two abutments and two steel pillars, visible in figure 10b. The distance between the pillars is the same as the distance between the pillars and the abutments, which is 8 metres.

(a) Lane distribution of bridge 137 (b) Supports of bridge 137

Figure 10: Photographs of Bridge 137 (Korrel, n.d.)

#### Deck

The deck is a composite deck supported by steel beams, as shown in figure 4a. There are however a number of differences between the deck used under the sidewalk and the deck used under the road. The two deck compositions are visible in figure 11. The first type is present under both of the sidewalks and looks as following. The composite deck is supported by INP380 steel profiles. Onto this deck, a layer of sand is placed, onto which the tiles are placed. The side walk deck is the only part which contains composite decking. The decking under the traffic lane is structural concrete, also supported by INP380 profiles. Onto the structural concrete, filling concrete is placed, onto which the finishing layer on asphalt is placed. The asphalt layer is 0,12 metres thick, the filling concrete layer is 15 cm thick and the structural concrete is 10 cm thick. The



(a) Deck used under the sidewalk

(b) Deck under the traffic lanes

Figure 11: Two different types of deck used on bridge 137 (Gemeente Amsterdam, 1909a)

Table 1: Deck composition bridge 137

| Layer | Thickness [m] | Type |
|---|---|---|
| **Sidewalks** | | |
| Finishing layer | 0.05 | Tiles |
| Filling layer | 0.15 | Sand |
| Constructive layer | 0.10 | Structural Concrete |
| Steel profiles | - | INP380 |
| **Road** | | |
| Finishing layer | 0.12 | Asphalt |
| Filling layer | 0.15 | Filler concrete |
| Constructive layer | 0.10 | Structural Concrete |
| Steel profiles | - | INP380 |

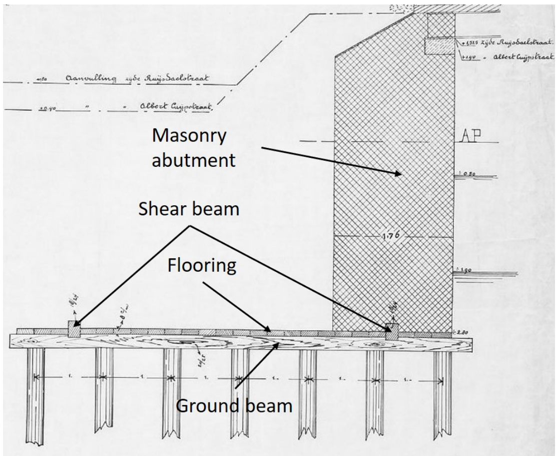
distances between the steel profiles is 729 on one side of the bridge, and changes into 751 mm halfway.

Within the recalculation as done by (Quansah, 2010), multiple strength tests were done. The strength of the concrete was tested, which concluded that the concrete was of type C45/55. A strength of reinforcement, having an $f_{yk}$ of 240 MPa and an $f_{yd}$ of 209 MPa. The masonry was not tested.

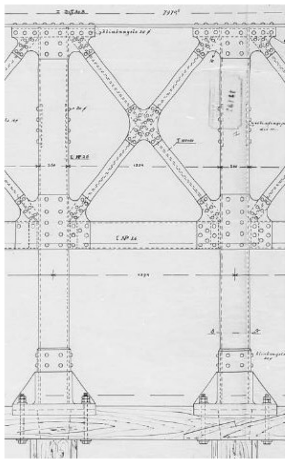
**Masonry abutment and pillar**

The abutments under the deck of bridge 137 are placed perpendicular to the span of the bridge. the thickness of the abutment is equal to 1.76 metres. On the side of the Albert Cuypstraat, the abutment has a height of 4.20 metres and a height of 4.325 metres at the side of the Ruysdaalstraat. The width of both of the abutments are 26.40 metres wide, which makes the abutment 10 meters wider than the middle of the deck. According to the (Gemeente Amsterdam, 1909b), the mortar between the bricks is 1 volumetric part Portland cement, 1/2 volumetric part chalk and 3 volumetric parts of sand.

A part of the pillars is visible in figure 12b. The pillars exist out of vertical steel columns, placed with 1334 mm in between each of the columns. The side columns are INP300, the collumns in the middle are INP260. Halfway from the top, another INP260 is placed to horizontally secure the columns. Above this INP260 beam, a truss structure is made by filling all of the rectangles above




(a) Cross section plane of the abutment



(b) A section plane of the pillar

Figure 12: Section planes of bridge 137 support points (Gemeente Amsterdam, 1909a)

the beam with a cross shaped metal structure. The structure is secured to the beams using metal plates and rivets . The vertical steel profiles have a length of 3.49 metres. The structural steel has been tested, which resulted in the fact that it is S235.

**Wooden pile foundation**

The original foundation drawings are visible in figure 13. Although only half of the abutment foundation is visible, the abutment is symmetric. This figure is applicable to both of the foundations, since the abutments are, except for their height, similar.

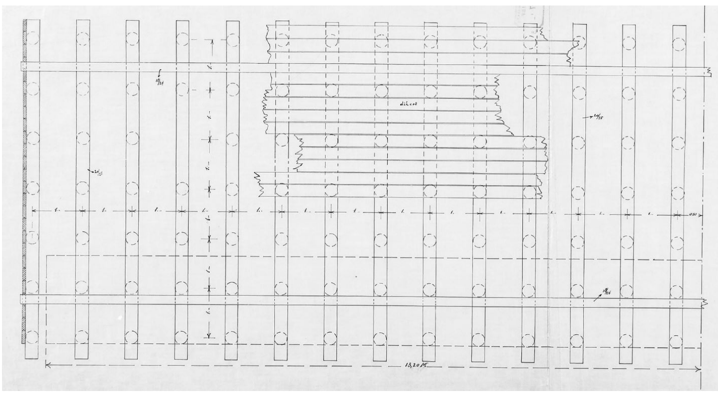


Figure 13: Half of the foundation of the abutment of bridge 137, with the masonry abutment on the lower side (Gemeente Amsterdam, 1909a)

Each of the foundations contains 196 pine wood piles. The piles are made of pine tree trunks, which causes the piles to be tapered. The top of the tree is drilled in first, meaning that the top and bottom diameter of the pile are switched compared to the top and bottom of the tree. the piles have a top diameter of around 27 centimetres and a bottom diameter of 12 centimetres, based on the building report from Gemeente Amsterdam (1909b). According to figure 13, the piles are placed in a grid with a spacing of 1 metre in both directions. The grid is 7 piles into the longitudinal direction of bridge, and 28 in the direction perpendicular to the span. Onto the 7 piles in each of the rows, a ground beam is placed. The ground beam is 20 centimetres high and 25 centimetres wide. Due to the both the piles and the beams being 25 centimetres wide, the pile fits under the ground beams. Onto the ground beams, a floor is placed. The floor is 8 centimetres thick and exists out of planks, placed to span the distance between the ground beams. This means that the grain direction of the planks is perpendicular to the grain direction of the ground beams. The floor carries both the abutment and the ground behind the abutment. Two slots are made in the ground beams, visible in 12a, in which a shear beam is placed, perpendicular to the ground beams. A distance between the shear beams is not given, but based on the drawings is estimated between 5 tot 5.5 meters. Each of the shear beams has a height of 24 centimetres and a width of 18 centimetres. The shear beams prevent the wood and masonry abutment from sliding over the ground beams. Onto the flooring and shear beams, the masonry abutment is placed. All timber used under the abutment is pine wood (Gemeente Amsterdam, 1909b).

Under the pillars, a single line of 11 piles is placed, all straight. The piles under the pillars are oak wood piles, which are 40 times 40 cm at the top and 15 times 15 at the bottom. These piles are 16 metres and in total. According to Gemeente Amsterdam (1909b), 22 piles were used, which matches the drawings.

### Soil conditions

The soil conditions are based on a soil sample, supported by multiple CPTs. Figure 14 shows the location of the samples and CPTs. Based on the known length of the bridge, it can be concluded that the soil information is extracted relatively close by. The numbers in the figure at the CPTs refers to the numbers used under the figures.

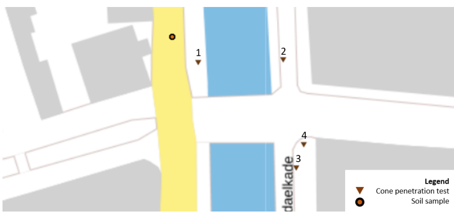



Figure 14: Locations of the soil samples and CPTs for bridge 137 (OpenStreetMap contibuters, n.d.)

The sample by DINOloket (n.d.) shows a lot of different layers, as can be seen in figure 15. The soil sample shows multiple different fine sand layers in the first few metres from ground level, ending with a peat layer around -6 metres. This layer of peat is approximately 0.7 metres thick and shows signs of clay. The peat layer is followed by different layers of clay, containing different gradations

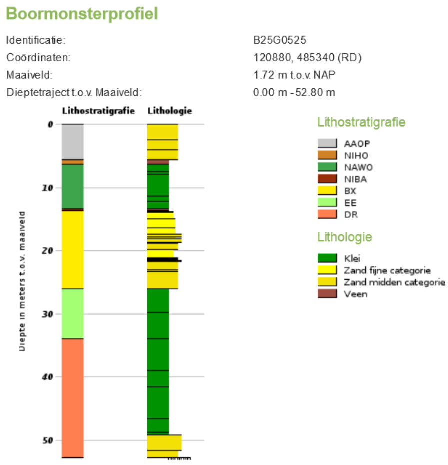
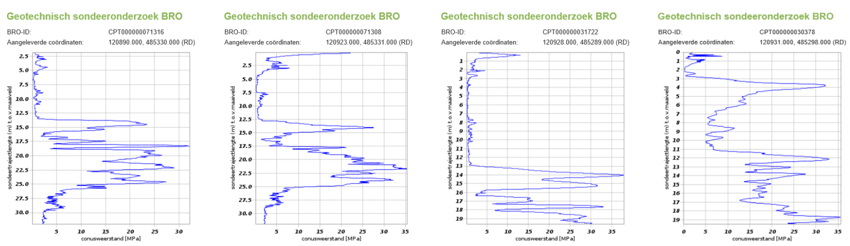



Figure 15: Soil sample from close proximity of bridge (DINOloket, n.d.)

of sand, ranging from showing large amount of sand in the middle layers and no signs of sand at the layers on the outer parts of the clay. The clay layer ends with a small layer of peat at 13 metres below ground level of 20 centimetres. The peat layer is followed by a thick layer of sand, starting at 14 metres below ground level and ending at 20 metres below ground level. The sand layer mainly contains fine sand, but also contains layers that are slightly more coarse. The thick layer of sand describes why the piles are 14 meters long. The sand layer ends at 26 meters and is than followed by multiple layers of clay, containing no to high amounts of sand. Looking further down, the clay layer is present until about 50 metres, which is followed by a sand layer. However, looking any deeper would not be of any interest, since the piles only go to 14 meters.

Looking at the cone penetration tests, the soil composition is similar to the sample, but there are a small amount of differences. The CPTs are enlarged in Appendix B, such that they are visible more easily. The sand layers are clearly visible around 14 meters, but do start around 12 metres below ground level in some of the CPTs. Moreover, the sand layers at the start of the sample can be seen in almost all CPTs, except for figure 16a. Figure 16d also shows a very high




(a) CPT 1 (DINOloket, 2012b) (b) CPT 2 (DINOloket, 2012a) (c) CPT 3 (DINOloket, 1990) (d) CPT 4 (DINOloket, 2002)

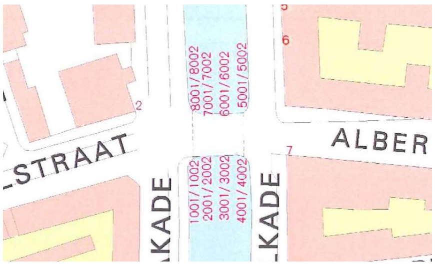
Figure 16: Cone penetration tests from surrounding area bridge 137

cone resistance around 4 metres, which can not be found in the other CPTs. The cone resistance in the sand layer is fluctuating a lot around the 14 metres, ranging from 22 MPa in figure 16a to 37 in figure 16c. The CPTs from figure 16a and 16b are very similar, due to the fact that they are taken at the same time, in 2012. The cone penetration tests as shown in figures 16c and 16d are different, but taken at the same side of the bridge. This might be explained by the fact that they were taken ten years apart, ten and 20 years earlier than the other two.

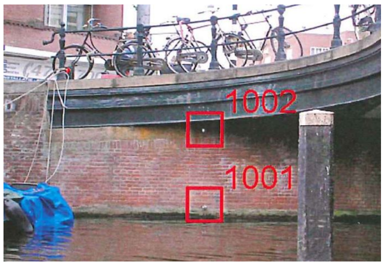
## 3.2 Inspection and monitoring data

### Deformations

Within the monitoring of bridge 137, two deformation measurements have been done. The first measurement in 2007 is used as an initial state measurement. The future measurements can then be compared to the initial state to see what changed in respect to the initial state. Within this measurement, 16 points were monitored, visible in figure 17a. The difference between for example 1001 and 1002 is visible in figure 17b. At all locations, the top point and a point just above water is measured, such that the points can be detected using a theodolite.




(a) All monitoring points on bridge 137 (Buishand, 2007)




(b) The difference between points 1001 and 1002 (Buishand, 2007)

Figure 17: On the abutment, 16 measurement points are located

In 2010, another XYZ measurement was done to see if indeed the bridge changed from the initial state. According to Buishand (2010), the abutment did move compared to the first repetition measurement. The bridge mostly moved in the vertical direction, with up to 13 millimetres of downwards movement. However, while looking into the measurement, there was noticed that the top of the abutment moved less (-9 mm) than the bottom (-12 mm). This means that the abutment has enlarged in vertical direction. However, this does not seem to be logical, since the forward movement does not suggest toppling. Therefore, the measurements were not used as input for a possible finite element scenario.

### Pile conditions

In 2009, Rikkers and Lodema tested four piles under the abutment. The tested piles are located next to each other, in the middle of the abutment on the side of the Ruysdaalstraat. The front piles at the water are tested, since these are easiest inspected. The results of the test are visible in table 2.

Table 2: Results of pile testing (Rikkers & Lodema, 2009)

| Pile number | 1 | 2 | 3 | 4 | Avg. |
|---|---|---|---|---|---|
| Current severe degradation [mm] | 15 | 35 | 10 | 0 | - |
| Relative remaining pile area [%] | 77 | 51 | 85 | 100 | 78 |
| Maximum degradation [mm] | 65 | 50 | 63 | 12 | - |
| Relative remaining pile area [%] | 23 | 36 | 25 | 82 | 42 |

The current severe degradation is measured as distance towards the centre. It was assumed that the pile deterioration is continuous over its circumference. The relative remaining pile area was calculated by dividing the remaining area by the original area, using equation 1.

$$\frac{A_{det}}{A_0} = \frac{\pi * (r_{det})^2}{\pi * r_0^2} = \frac{(r_0 - d_{det})^2 * \pi}{r_0^2 * \pi} = (1 - \frac{d_{det}}{r_0})^2 \tag{1}$$

In equation 1, $A_0$ is the original pile area, $A_{det}$ is the deteriorated area. The radius of the deteriorated area $r_{det}$ is calculated by subtracting the degradation distance $d_{det}$ from the original radius $r_0$. For $r_0$, a value of 125 mm was used. This is close to the value according to Gemeente Amsterdam (1909b), in which 135 millimetres was given, and also used by Rikkers and Lodema (2009) in the pile tests. Rikkers and Lodema (2009) concluded that the piles under the abutment were influenced by biological deterioration, but the centre area is still intact. This was tested by taking a sample from the piles, which were visually inspected and tested in a compression test. Rikkers and Lodema expected that the deterioration would continue for another 25 years, if looked at the taken samples. The piles were mainly deteriorated by erosion bacteria. From the results, it can be concluded that although the piles are closely together, the piles are not equally deteriorated. It also shows that the piles are not all possibly deteriorating towards one distance, but have a large variation within the possible deterioration.

In the same research, Rikkers and Lodema also concluded that the piles under the pillars, which are oak wood, are not deteriorated and still have a large bearing capacity.

# 4 Case 'bridge 137': Finite element model

This section explains the adopted modelling approach to simulate the structural behaviour of bridge 137. A finite element model is made by using the software DIANA FEA version 10.3. Details are given with respect to the units, geometry, material models and parameters, mesh properties and loading. In order to reduce the computation time, the wooden piles and soil layers are modelled in a simplified way, replacing them by spring elements. The combined spring stiffnesses are calculated by using the software DynaPile. Details of this calculation are provided in section 4.4.

## 4.1 Units

Within the finite element model, a defined set of units was chosen. Due to size of the abutment, the length is defined in meters. Since the abutment has a defined density in kg/m$^3$, mass was chosen to define kilograms. Although most forces and loads were defined in kN and MN, it was decided to use N as the unit for force.

## 4.2 Geometry

The modelling of the abutment matches figure 6 and the description in section 2.1. Since the abutment behaviour is expected to change in different directions in multiple directions and the loading conditions are not equal in all directions, the model was built in 3D.

### 4.2.1 Deck

Although the deck itself is not considered within the finite element model, the loading of the deck onto the abutment has been simplified. In the real situation, a steel beam transfers the forces from the deck into the abutment, loading the abutment with a distributed force. However, considering the height and width of the abutment, it was considered that modelling the distributed load from the beam as a point load with equal value, it would not be a large difference. The same thing was also done by (Quansah, 2010), making it easy to use the forces from the report.

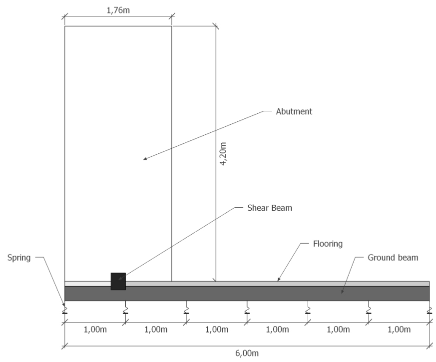
### 4.2.2 Abutment, flooring and beams

The abutment is modelled similar to the abutment as shown in figure 6. A side view of the abutment as modelled is visible in figure 18.

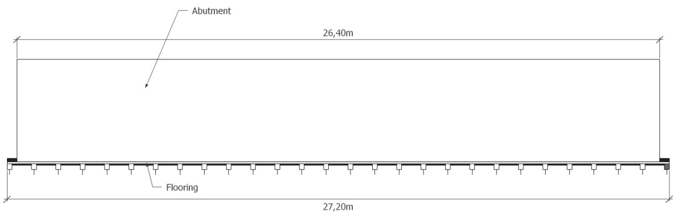
The model consists four components, namely: ground beam, shear beam, flooring and the masonry abutment.

The ground beam is visible in figure 19a and has a width of 0.20 metres, a height of 0.24 metres and a total length of 6 metres. The springs are modelled in the middle of the beam.

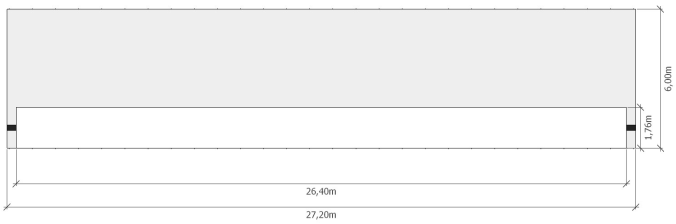
This a mistake in the model, since the height and the width should be the other way around, according to subsection 3.1. Since the area moment of inertia is higher in the model than in the reality, the mistake is expected to have a reducing effect on the output of the model. In total, 28 ground beams are used, all placed 1 metre apart, centre to centre.




(a) Side view




(b) Front view

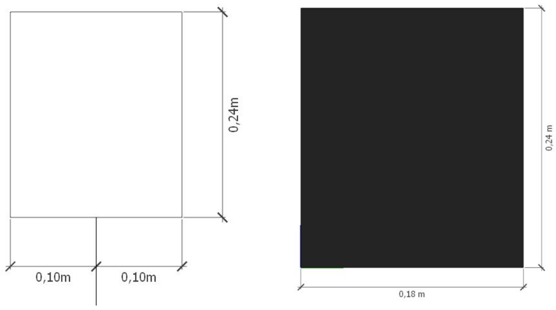



(c) Top view

Figure 18: The representation of the model as used in the finite element analysis

The second geometry is the shear beam, visible in figure 19b. The shear beam has a height of 0.24 metres and a width of 0.18 metres. Although two beams are visible in the drawings form Gemeente Amsterdam (1909a), only the shear beam under the abutment was modelled. The reason for this is that the shear beam at the landwards part of the abutment was not expected to have a large influence on the outcome of the model and leaving out the second shear beam reduces the computational time.

The third geometry is the flooring, which covers the ground beams. The flooring is, as mentioned in subsection 3.1 build with wooden planks. However, to simplify the modelling, the flooring as modelled as one big piece, instead of multiple planks. Although this leads to a possibility to have tension between the individual planks, the tension in the timber is not expected to have a large influence on the outcome of the model, which is mainly focused on the abutment. Due to the flooring being modelled as one large plate, the floor has an area of 6 times 27.2 meters.




(a) Ground beam as modelled (b) Shear beam as modelled

The fourth geometry is the abutment. This is modelled with a height of 4.2 metres, a length of 26.4 metres and a thickness of 1.76 metres, as visible in figure 18. The shear beam is modelled in a slot in the abutment.

### 4.2.3 Foundation and soil

The foundation and soil have been modelled using software that transfers the piles and pile soil interaction into springs. The soil is modelled after the sample as shown in figure 15. Within the model, it was taken into account that the piles are placed 4 metres deeper than the top of the abutment. Therefore, the top four metres or the abutment were left out of the soil structure interaction software. This assumption is subject to error, since the bridge might be higher than the surrounding area, meaning that a different soil composition needs to be taken into account. The layout of the pile foundation is modelled as described in section 3.1: A 7 by 28 grid of piles, with 1 metre spacing centre to centre.

## 4.3 Material models and parameters for masonry and wood

Within the finite element model, two materials are defined. The first one is the timber. According to Quansah (2010), the E modulus of the timber is equal to the E modulus of C18 quality timber. This timber quality is also assumed in the recalculations of bridges 30 and 41 (van den Broek, 2014; Quansah, 2014). Within the finite element model, the properties as mentioned in table 3 were used. However, these properties only apply to the shear beam and flooring, since the fibre is in a different direction for the ground beams. For the ground beams, the properties in the direction of x and y are switched.

The second material is masonry. The material properties as used in the finite element model are shown in table 4. The masonry parameters are collected from literature. Since the masonry types tested in literature do not exactly contain the same composition and bricks, interpolation was done between masonry types that were comparable in composition to the masonry used in bridge 137.

## 4.4 Material model and parameters for the wooden pile foundation and soil

For modelling the soil structure interaction, different parameters were needed than for the finite element. The model parameters for the piles are given in

Table 3: Input parameters of C18 as used in the finite element software

| Property | Value |
|---|---|
| **Linear material properties** | |
| Mass density | 380 kg/m$^3$ (Centrum Hout, 2014) |
| **Youngs modulus** | |
| $E_x$ | 9 MPa (Centrum Hout, 2014) |
| $E_y$ | 0.3 MPa (Centrum Hout, 2014) |
| $E_z$ | 0.3 MPa (Centrum Hout, 2014) |
| **Poisson’s ratio** | |
| $\nu_{xy}$ | 0.15 (Quansah, 2010) |
| $\nu_{yz}$ | 0.15 (Quansah, 2010) |
| $\nu_{xz}$ | 0.15 (Quansah, 2010) |
| **Shear modulus** | |
| $G_{xy}$ | 0.56 MPa (Centrum Hout, 2014) |
| $G_{yz}$ | 0.56 MPa (Centrum Hout, 2014) |
| $G_{xz}$ | 0.56 MPa (Centrum Hout, 2014) |

table 5.

The properties of the soil as used in the soil structure interaction software are shown in table 6. For sand and clay, the shear wave velocity was derived by the soil structure interaction software based on the Elastic modulus. Since the elastic modulus for peat was not found, the elastic modulus was derived from the shear wave velocity by the soil interaction software.

Within the soil structure interaction, different pile deterioration scenarios were taken into account. This was done by defining three different pile radii, either being not deteriorated at all, being as deteriorated as the three worst piles currently are or being as degraded as the worst three piles. The reason that three piles were chosen instead of all four is that three of the four are much more deteriorated than the other. Although taking these three values might therefore not be representative for the whole abutment, these values could occur within the abutment. The three pile diameters and other sectional properties are given in table 7. Since the pile is circular, the area moment of inertia in the x and y direction are equal.

Implementing the soil and piles into the SSI software lead to the pile stiffness results shown in table 8. Since the piles are expected to be double symmetric, $K_{xx}$ and $K_{yy}$ are equal (Roesset, Wang, Vasquez, & Fadaifard, 2016).

## 4.5 Element types and finite element mesh

The mesh of the geometry is visible in figure 19. In figure 19a, the abutment is visible, with the flooring modelled under it. The ground beams are also visible, with the yellow points being the springs. In top view, the large flooring is visible. In the side view, the shear beam is clearly visible as a different part than the masonry. Again, the springs are visible as yellow points. The mesh of the geometry was made using a desired element size within the settings of the finite element software mesh property settings. To be able to integrate as well as possible, it was desired to keep the element as much as cube shaped as possible. To achieve this, the elements needed to have approximately the same size. The

Table 4: Input parameters of masonry as used in the finite element software

| Property | Value |
|---|---|
| **Linear material properties** | |
| Young’s Modulus | 4000 MPa (Kaushik, Rai, & Jain, 2007) |
| Poisson’s ratio | 0.19 (Dhanasekar, Page, & Kleeman, n.d.) |
| Mass density | 1600 kg/m$^3$ (Walker, Kioy, & Jowsey, 2014) |
| **Total strain based crack model** | |
| Crack orientation | Rotating |
| **Tensile behaviour** | |
| Tensile curve | Hordijk |
| Tensile strength | 0.12 MPa (Pluijm, van der, 1997) |
| Mode I tensile fracture energy | 4.2 N/m (Pluijm, van der, 1997) |
| Crack bandwidth specification | Rots |
| **Compressive behaviour** | |
| Compression curve | Parabolic |
| Compressive strength | 7.1 MPa (Kaushik et al., 2007) |
| Compressive fracture energy | 10000 N/m (van Noort, 2012) |

Table 5: Pile properties as used in soil the structure interaction software

| Property | Value |
|---|---|
| Density | 380 kg/m$^3$ (Centrum Hout, 2014) |
| Poisson’s Ratio | 0.15 (Quansah, 2010) |
| Damping Ratio | 0.06 (Moore & Maguire, 2004) |
| Elastic Modulus | 9 GPa (Centrum Hout, 2014) |
| Length | 14 m |

size was determined by the flooring, which is 0.08 metres thick. However, to make the model more manageable, an desired element size of 0.10 meters. was used. This desired element size was used for all shapes.

Within the model, 2 different types of elements are used. all using quadratic interpolation between points. The element can be seen in figure 20. The most common type in the model is the CHX60. This is a cube shaped block, containing 20 nodes. The piles have been modelled as springs, which the finite element software meshed into N6SPR elements. The N6SPR elements are one-node directly integrated generic spring elements.

## 4.6 Loading

The loads on the abutment and are based on the loading conditions as used by Quansah (2010). The loading conditions onto the abutment exits out of water pressure, ground pressure and deck and vehicle loading. The deck loading is modelled as point loads, but the ground and water pressure is modelled as distributed loads. The distributed loads are visualised in figure 21. The values for the loads as used in the finite element software are displayed in table 9.

The first load is the weight of the soil and infrastructure above the flooring of the abutment. Quansah (2010) takes the load as 79.40 kN/m$^2$. Considering that the weight of sand is equal to 19 kN/m$^3$(NEN, 2018) and about 4 meters

Table 6: Soil properties as used in soil the structure interaction software

| Property | Value |
|---|---|
| **Sand (clayey)** | |
| Density | 19 kN/m$^3$ (NEN, 2018) |
| Poisson's Ratio | 0.35 (Bowles, 1968) |
| Damping Ratio | 0.01 (Bowles, 1968) |
| Elastic Modulus | 81 MPa (Bowles, 1968) |
| Shear wave velocity | 122 m/s |
| **Peat** | |
| Density | 13 kN/m$^3$ (NEN, 2018) |
| Poisson's Ratio | 0.35 (Huat, Kazemian, Prasad, & Barghchi, 2011) |
| Damping Ratio | 0.20 (Zainorabidin & Wijeyesekera, 2019) |
| Elastic Modulus | 7.1 MPa |
| Shear wave velocity | 45 m/s (Zainorabidin & Mad Said, 2015) |
| **Clay (sandy)** | |
| Density | 18 kN/m$^3$ (NEN, 2018) |
| Poisson's Ratio | 0.25 (Bowles, 1968) |
| Damping Ratio | 0.25 (Bowles, 1968) |
| Elastic Modulus | 125 MPa (Bowles, 1968) |
| Shear wave velocity | 167 m/s |

Table 7: Section properties as used in soil the structure interaction software

| Relative remaining area [-] | Diameter [m] | Area [m$^2$] | $I_x$,$I_y$ [m$^4$] |
|---|---|---|---|
| 100% | 0.195 | $2.99{*}10^-2$ | $70.98\ {*}10^{-6}$ |
| 70% | 0.180 | $2.09{*}10^-2$ | $34.78\ {*}10^{-6}$ |
| 30% | 0.107 | $0.89{*}10^-2$ | $6.39{*}10^{-6}$ |

of soil is on top of the flooring with traffic load, the value taken by Quansah (2010) is plausible.

The second load is the weight of the soil against the masonry abutment. This linearly increases from top to bottom. At the top, the value of the load is equal to 9.56 kN/m$^2$. Although the layer of sand above this point is small, the value is plausible due to the tram, which causes extra weight. The highest value of the load against the abutment is 79.40 kN and is equal to the vertical soil load. The next two loads are the vertical water loads. At the top, load three, the load is equal to 24.8 kN. Considering there is about 2.5 meters water above and the density of water is equal to 10 kN/m$^3$, the load is plausible. Load number 4 is similar, but since the lower part of the flooring is 8 centimetres lower, the load is slightly higher and equal to 25.6 kN/m$^3$. Loads five and 6, against the masonry part of the abutment, are the water loads against the abutment. At the beginning of the water line, these are zero and increase linearly to 24.8.

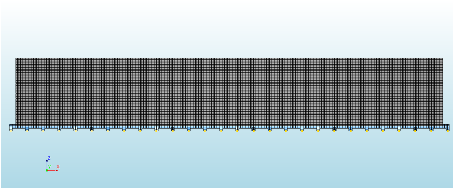
The point loads from the deck are visible in figure 22.

The values as used in the deck loading can be found in table 10.

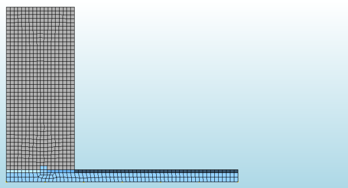
As visible in figure 22, the point loads are not symmetric. The reason for this is that from figure 10a there can be concluded that the tram does not drive on all three lanes, but on two side by side lanes at one side of the bridge. Therefore, the loads are much heavier on one side, compared to the other side. Since the

Table 8: Pile Stiffness components

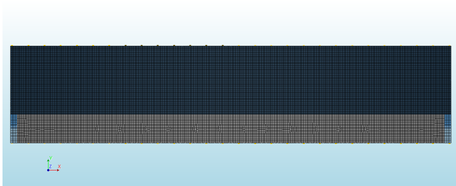
| Relative remaining area | $K_{zz}$ [MN/m] | $K_{xx}$,$K_{yy}$ [MN/m] |
|---|---|---|
| 100% | 104.8 | 40.33 |
| 70% | 86.65 | 33.89 |
| 30% | 56.34 | 22.78 |



(a) Front view of the mesh



(b) Side view of mesh



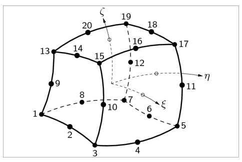
(c) Top view of the mesh

Figure 19: Mesh of the finite element model

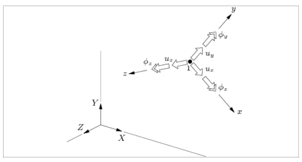
abutment at the Albert Cuypstraat was modelled, the forces where applied in this order. The value for each of the forces modelled as given in table 10. The forces are negative, since they are in downwards direction.

Although the loads seem to be high, the trams of the pubic transport company in Amsterdam (GVB) have a mass of 34 metric tons (GVB, n.d.), which means that the weight of a tram is equal to 333.5 kN. Since the abutment carries a large part of the tram and possibly two trams at the same time, the weight applied onto the abutment can be relatively high.

The loads as described above are limit state design values. A breaking load was not implemented within the current finite element model, since the breaking loading could be implemented into both directions. Therefore, the number of analysis needed to be doubled, which could not be implemented into the current time frame.

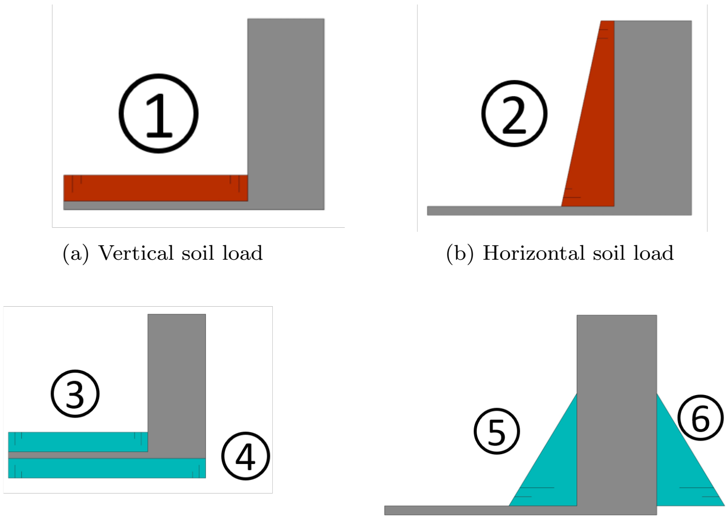


(a) CHX60



(b) N6SPR

Figure 20: Element types used in the the finite element software model (Roesset et al., 2016)




(a) Vertical soil load
(b) Horizontal soil load
(c) Vertical water pressure
(d) Horizontal water pressure

Figure 21: Distributed loads onto the abutment

Table 9: Load values for the loads on the abutment

| Load | Attached to: | Value (maximum) [kN/m$^2$] | Minimum Value [kN/m$^2$] |
|---|---|---|---|
| 1 | Flooring | 79.40 | - |
| 2 | Abutment | 79.40 | 9.56 |
| 3 | Flooring | 24.80 | - |
| 4 | Flooring | 25.60 | - |
| 5 | Abutment | 24.80 | 0 |
| 6 | Abutment | 24.80 | 0 |

# 5 Case ‘bridge 137’: Results

This section covers the results from the analysis that has been done. In the first subsection, the different scenarios are explained. The second subsection goes deeper into the hand calculations, which are used to check the output of the model. After the hadn calcualtions, the results are described. The results are discussed and compared to the hand calculation in the final section

## 5.1 Overview of analysis

Within the finite element analysis, a static linear analysis is done. This static analysis was done with multiple different scenarios, changing the pile deterioration values of different groups of piles. These groups of piles are given in figure 23.

The different scenarios are given in table 11. In the first three scenarios, the full pile field is considered to have equal deterioration. In scenario 4 to 7, multiple zones are expected to be deteriorated, but not all. The zones that are deteriorated for each scenario are given in table 11. The zones that are not mentioned in the table have undeteriorated area of 100%. For example, in scenario 5, zones four, five and six have an undeteriorated area of 30%, while zones one to three and seven to 12 are not deteriorated.

## 5.2 Model checks

Two small model checks were done to see whether the model behaves according to expectations. The model checks are simplified, such that a quick calculation could be done. The first check is the forces in the piles. The sum of the forces in the springs representing the piles should be equal to the total weight of the structure and the vertical loads. Table 12 shows a rough calculation of the own weight of the structure, using the density from table 3 and 4. The total weight can be compared to the reaction forces in case of only own weight. The calculation is a rough calculation, since the shear beam has been taken out, visible in figure 24.

The second check is done on the behaviour of the masonry abutment, under the influence of the horizontal forces. Since the water pressure cancels itself out, considering that it is equal on both sides, only the soil load was taken into account. The original situation is given in figure 25a, with the soil load as the brown trapezium. The original situation was transferred into a simplification shown in 25b. Since the distributed load resembles a triangle, the sum of the distributed load was applied at one third of the height. The total deflection can be calculated according to formula 2.

$$\delta_B = \delta_B + \phi_B * (h_A - h_B) \tag{2}$$

Using standard mechanics, $\delta_B$ and $\phi_B$ can be calculated according to equation 3 and 4.

$$\delta_B = \frac{1}{3}\frac{Fh_B^3}{EI} \tag{3}$$

$$\phi_B = \frac{1}{2}\frac{Fh_B^2}{EI} \tag{4}$$

Within equation 3 and 4, the force F is equal to the sum of the distributed load onto the abutment, equal to 3.87 MN. The E modulus is taken from table 4 and equal to 4 GPa. The area moment of inertia is calculated using the formula for a beam, visible in equation 5.

$$I = \frac{1}{12}h^3 * b \tag{5}$$

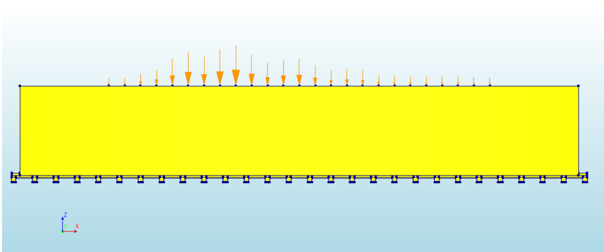


Figure 22: Orange arrows representing the loading from deck onto the abutment

Table 10: Vertical loading conditions on the abutment as used in the the finite element software model

| Number | x coordinate from centre point of the bridge [m] | Value [kN] |
|---|---|---|
| 1 | -9 | -40,78 |
| 2 | -8,25 | -40,78 |
| 3 | -7,5 | -55,9 |
| 4 | -6,75 | -77,28 |
| 5 | -6 | -133,48 |
| 6 | -5,25 | -169,56 |
| 7 | -4,5 | -144,22 |
| 8 | -3,75 | -178,23 |
| 9 | -3 | -198,59 |
| 10 | -2,25 | -152,95 |
| 11 | -1,5 | -112,22 |
| 12 | -0,75 | -127,25 |
| 13 | 0 | -135,52 |
| 14 | 0,75 | -100,77 |
| 15 | 1,5 | -76,61 |
| 16 | 2,25 | -81,29 |
| 17 | 3 | -75,11 |
| 18 | 3,75 | -53,81 |
| 19 | 4,5 | -47,27 |
| 20 | 5,25 | -45,17 |
| 21 | 6 | -45,74 |
| 22 | 6,75 | -46,49 |
| 23 | 7,5 | -45,98 |
| 24 | 8,25 | -42,45 |
| 25 | 9 | -42,45 |

Within equation 5, b is the width and h is the height of the 'beam'. Since the situation as shown in figure 25b bends towards the water, h is equal to the thickness of the abutment, 1.76 metres. The value for b is equal to the length of the abutment, 26.4 metres. This gives an area moment of inertia of 11.99 $m^4$. The values for $h_A$ and $h_B$ are the full height of 4.2 metres and $\frac{1}{3}$ of the full height, equal to 1.4 metres. Applying all values in equations 2, 3 and 4 gives a displacement of 0.35 millimetres.

## 5.3 Results

### 5.3.1 Reaction forces in the piles

First, there was checked whether the results of the hand calculation of the weight can be compared to the sum of the forces in the piles. The total weight of the structure was roughly calculated to be 3142 kN. The results form the analysis conclude that the sum of vertical forces in the piles for the own weight calculation is equal to 3137 kN.

When applying the loads as mentioned in subsection 4.6, the following results were found. First of all, the forces in the springs are the highest at the

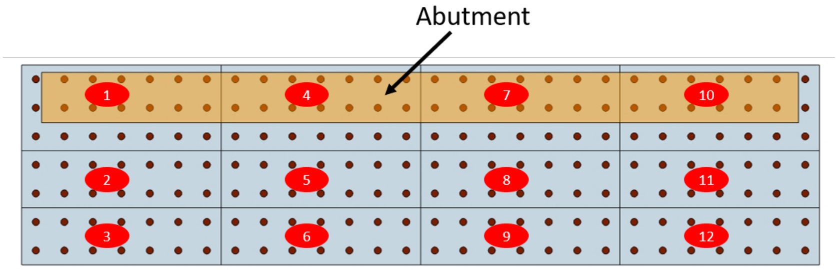



Figure 23: Pile zone division

Table 11: Model scenarios

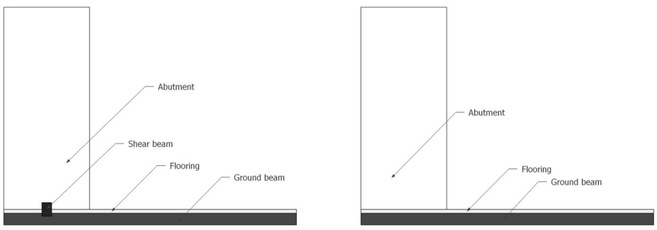
| Scenario number | Effected zones | undeteriorated area [%] |
|---|---|---|
| 1 | 1-12 | 100 |
| 2 | 1-12 | 70 |
| 3 | 1-12 | 30 |
| 4 | 1-3 | 30 |
| 5 | 4-6 | 30 |
| 6 | 7-9 | 30 |
| 7 | 10-12 | 30 |

location where the highest forces were applied at the top of the abutment, just as expected. Moreover, the forces in the piles correspond to the values as calculated by Quansah (2010). According to Quansah (2010), it is also stated that the determining piles are in the second row from the water inwards, which can also be concluded from the overall analysis. The highest forces are equal to 143 kN.

### 5.3.2 Horizontal and vertical displacement

The scenarios as given in table 11 are monitored on their vertical and horizontal displacement.

Although the abutment behaviour was monitored on multiple points over its width, no differences were found regarding the behaviour of the horizontal displacement towards the water. Based on the results as given in figure 26,

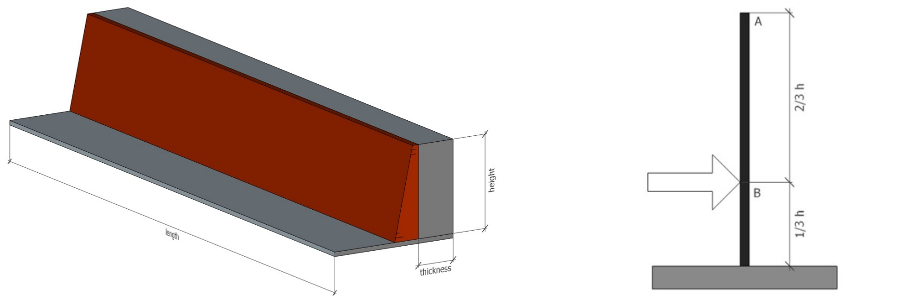



(a) Original situation (b) Simplified situation

Figure 24: Substituting the shear beam with the ground beam, abutment and flooring

Table 12: A rough calculation of the total weight of the structure

| Element | number | size per unit [m*m*m] | total volume [$m^3$] | weight [kN] |
|---|---|---|---|---|
| Abutment | 1 | 26.4 * 4.2 * 1.76 | 195.15 m3 | 3063 |
| Flooring | 1 | 27.2 * 6 * 0.08 | 13.06 | 49 |
| Ground beam | 28 | 0.2 * 0.24 * 6 | 8.06 | 30 |
| Total | | | | 3142 |




(a) Original situation

(b) Situation as used in the hand calculation

Figure 25: Simplifying the real situation for a hand calculation

the total displacement of the top is 22 millimetres towards the water. When comparing the different scenarios, a difference of 0.78 millimetre of top abutment was found between scenario 1 and 3.

For scenarios 4 to 7 the same analysis was done. Again, no difference was found over the width of the abutment. The displacement in the original stiffness situation is 22 millimetres, while the movement in case of the other scenarios is 0.1 millimetre more.

The abutment was checked for deformation due to the horizontal loading. Since the abutment has rotated, the deformation due to rotation needs to be subtracted form the total deformation to check the deformation due to loading. By using extrapolation, the slope between the first two points, which are not or very little deformed by the horizontal load, was extrapolated to the top of the abutment. Using this extrapolation, the deformation due to the horizontal load is equal to 0.32 millimetres.

Regarding the vertical behaviour, the abutment behaviour was monitored at multiple points over the height of the abutment. However, no difference was found between the different heights. From the results, there can be concluded that the abutment moves vertically without pile deterioration, which is equal to -10.5 millimetres at the sides and around -12 millimetres at the points where the applied loading is the highest. When comparing the different scenarios, the difference between scenario 1 and 3 is 1 millimetre at the sides, and 1.1 millimetre at the higher loading conditions. The fact that the displacement change between the scenarios is almost constant over the width can be explained by the fact that the loading conditions do not change, only the stiffness of the springs changes. Therefor, the vertical strain in the abutment does not change, which does not cause deformation of the abutment between the scenarios.

For scenarios 4 to 7 the same analysis was done. The difference between the

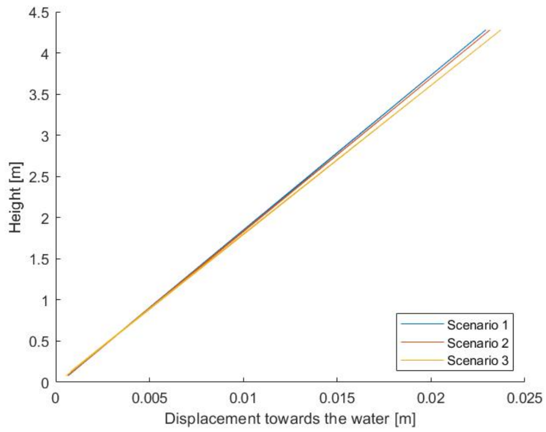



Figure 26: Movement of the abutment in the longitudinal direction in case of equal deterioration over the entire field

original situation (scenario 1) and 70% deterioration in section 7 to 9 (scenario 6) is equal to 0.8 millimetres. The difference of movement at the sides is roughly 8 millimetres, comparing scenarios 4 and 7 to the original situation.

### 5.3.3 Stresses in the masonry

To conclude whether a non linear analysis needs to be done, the stresses from the different scenario are analysed. If the stresses in the masonry are lower than the compressive stress or higher than the tensile stress, a non linear analysis should be done to check how the abutment behaviour when being subjected to its limits.

The tensile stress has been plotted in a contour plot, setting the boundaries between no tensile stress and the maximum tensile stress of 0.12 MPa. The values above these limits are removed from the figure, leaving a gap when the stress is higher. Only the stress in the x direction has been plotted, in the width of the abutment. For scenario 1, this leads to the stresses in the masonry as found in 11.

The large gap at the left bottom of the abutment shows how the stress in this part is outside the boundaries of the set limits. Since the surrounding area shows tensile stress, there can be concluded that this part experiences higher tensile stress than the boundaries. The large gap at the centre shows that this part is subject to compressive stress, since this is also outside of the boundaries of the contour plot.

All other scenarios show a similar output compared to figure 30, with slightly larger areas of high compressive stress. In figure 31, the tensile stress contour plot for scenario 6 is given, showing a slightly larger area surpassing the tensile stress limit.

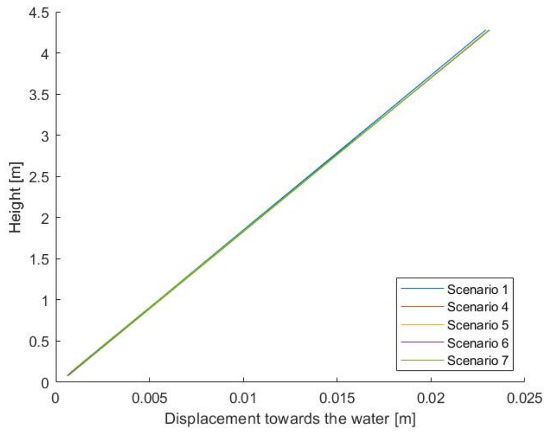



Figure 27: Movement of the abutment in the longitudinal direction in the case of section deterioration

## 5.4 Discussion

Considering the results from the finite element analysis, a number of things were found to be interesting. Although the hand calculation of the weight did not exactly match the total force from the piles, the numbers are very close. The difference between the values is most likely caused by the simplification of leaving out the shear beam and continuing the flooring, abutment and ground beams. Although the volume of the shear beam is relatively small, the difference between the mass density of wood and masonry is high, resulting in a larger difference.

Comparing the hand calculation of the horizontal movement of the abutment to the model value, the two are not equal, but close together. The reason for this might be that due to extrapolation through a small number of points over a large distance, there will be a slight difference in the model outcome and hand calculation. However, the magnitude and value are similar, concluding that the input value of the model and the hand calculation are the same.

When looking at forwards movement, little change could be seen between the different scenarios. Considering that the total movement of the abutment is 22 millimetres towards the water, the movement of other scenarios is up to 0.5% more.

Considering the vertical abutment behaviour, a larger difference was found. The maximum vertical movement in the original situation is equal to 12 millimetre, while the worst deterioration for the full foundation shows a vertical movement that is 11% more. The same can be concluded for the scenarios in which parts of the abutment were deteriorated, while all other parts were left intact. Regarding these percentages, there could be distinguished which of the pile sections is deteriorated, based on side and middle measurements.

Regarding the stress in the masonry, a few things were noticed. In the original situation, already a high portion within the abutment surpasses its maximum tensile stress. This means that a non linear analysis should be done to investigate how the model behaves after reaching the maximum tensile stress

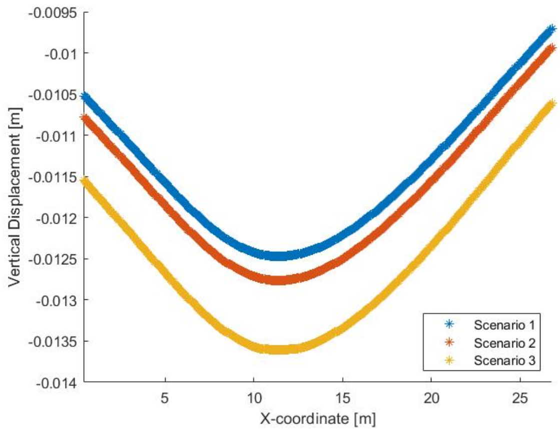



Figure 28: Vertical movement of the abutment in the case of equal deterioration over the entire field

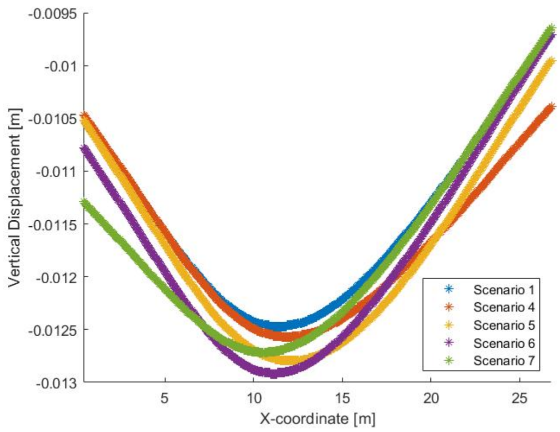



Figure 29: Vertical movement of the abutment in section wise deterioration

and whether crack measurements could be an indication for a deteriorating pile foundation.

Within the model, a number of simplifications have been done. First of all, the flooring was modelled as one piece. This means that the individual pieces of wood of which the floor exists, was able to resist tension, while in the real situation, the pieces of wood would move. Moreover, the interface between parts has not been modelled. This means that all intersecting faces between two parts are connected and cannot simply move apart from each other or shear.

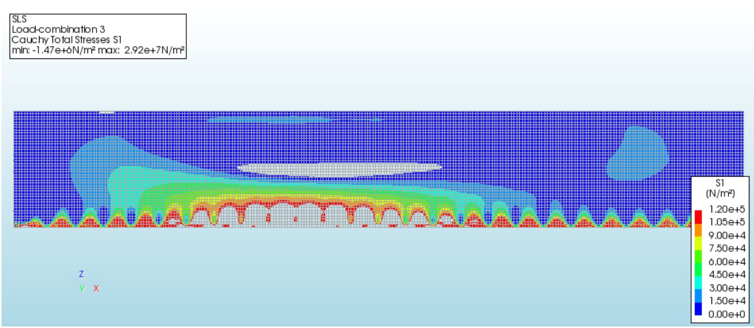



Figure 30: The tensile stress in the model surpasses the tensile stress according to material properties for scenario 1

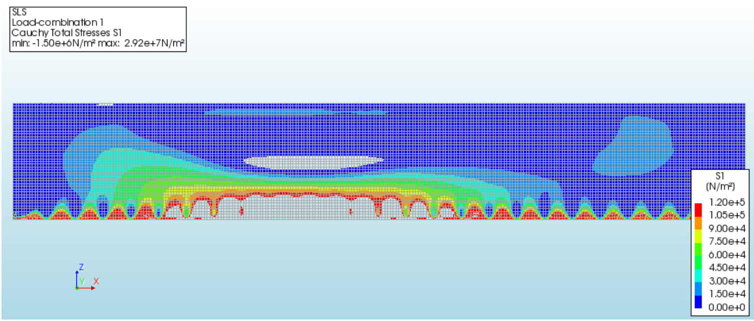



Figure 31: The area of the tensile stress that surpasses the limit is higher for scenario 6 than scenario 1

# 6 Conclusion and recommendations

Structural health monitoring can be used to predict structural behaviour of bridge elements using indirect measurements. This is convenient to the municipality of Amsterdam, since monitoring the structural behaviour of wooden pile foundation is very hard, considering the pile foundations can often not be inspected. However, to be able to use the structural health monitoring, a finite element model is needed to use damage parameters as input into the model to predict the structural state of the bridge elements. The aim of this work was to look into the effect of pile deterioration on the structural response of the abutment. This aim was broken down into three objectives.

The first objective looked into the current available information on bridge monitoring in Amsterdam. The archive research shows a lot of information about the bridges as build and can at the same time give information about the original masonry properties. Subsection 2.1 shows the variety in the bridges, but also shows how the building years of the bridges in the subset can be divided into a group of bridges with a building year of around 1750 and a building year from around 1900. From subsection 2.2, it was concluded that the pile degradation is not the only case of abutment failure. It was also concluded that deterioration can occur due to both fungi and bacteria. This is important, since the bacteria also deteriorate the pile while being under water. Currently, monitoring on these bridges is done in a small number of ways, of which deformation monitoring are the most descriptive for the behaviour of the bridge.

The second objective went into the possibility of modelling the abutment and foundation. To be able to make a less time consuming model, two types of software were used to model the behaviour. The first software was a soil structure interaction model, which transfers the piles and soil into spring stiffness values. Using this software, the different pile deterioration were implemented. The other software that was used is a software for doing a finite element analysis. In the finite element software, the abutment was modelled, using the spring stiffness values for boundary springs.

The third objective goes into the relevant structural behaviour to be monitored when looking into the possibility of applying structural health monitoring. Based on the results for the current linear analysis, a difference could be found in vertical movement between the different scenarios, both when lowering the pile stiffness over all piles as lowering the pile stiffness in certain sections. A variation in movement in direction towards the water could not be found in the analysis.

Therefore, the effect of a deteriorating pile foundation on the structural behaviour can be concluded in a number of points. Based on the current linear analysis, reducing the pile stiffness only causes a difference in the vertical movement of the masonry abutment. The horizontal behaviour of the abutment did change, but changes very little. However, a tensile stress above the limit could be found in a large part of the abutment, giving reasons to do a non linear finite element analysis.

This research does however have a number of limitations. First of all, only the pile deterioration has been considered. Although a research into pile deterioration is the goal of this work, it needs to be taken into account that movement of the abutment can be caused by much more factors than just this one.

Second of all, there is no validation of the model. Hand calculations were done regarding the vertical forces and horizontal movement, but the structural behaviour of the abutment has not been validated with real bridge movements.

From the stress contour plots in the masonry, there could be concluded that a non linear analysis should be done to predict the behaviour of the abutment. However, before being able to do an accurate linear analysis, more research into masonry and timber should be done. Currently, average values of, where possible, similar types of masonry were used as input of the masonry in the model. Although this currently gives a good output, for a future use within the municipality, testing the masonry as used would improve the models accuracy. This also gives better possibilities for using the masonry in a probabilistic framework. The same applies to the timber model, in which values were used, set by dutch norms. When building an improved model, the deterioration of individual sections could also be considered, since this analysis only deterioration in groups of sections was taken into account.

Regarding the possibilities of structural health monitoring, the possible output of the data of current monitoring is limited. Currently, the monitoring of the abutment takes place on 4 points above water, as mentioned in subsection 3.2. Measuring deflections only at four points is insufficient to draw conclusions from it regarding pile foundations. It is recommended to increase the number of measuring points along the abutment, such as monitoring the bed joints on their behaviour.

# References

AT5. (2018). Brug de clercqstraat gaat zeven maanden dicht door bacteriën. *AT5*. Retrieved from https://www.at5.nl/artikelen/177198/brug-de-clercqstraat-gaat-zeven-maanden-dicht-door-bacterien

Blanchette, R. A., Nilsson, T., Daniel, G., & Abad, A. (1990, 01). Biological degradation of wood. In (p. 141-174). doi: 10.1021/ba-1990-0225.ch006

Bowles, J. E. (1968). *Foundation analysis and design / joseph e. bowles* [Book]. McGraw-Hill New York.

Buishand, E. M. (2007). *Deformatiemeting brug 137* (Tech. Rep.). Gemeente Amsterdam.

Buishand, E. M. (2010). *Deformatiemeting brug 137* (Tech. Rep.). RPS BCC B.V.

Centrum Hout. (2014). *Sterktegegevens van gezaagd en gelamineerd hout.*

Clausen, C. A. (2010). *Wood handbook, chapter 14: Biodeterioration of wood* (Vol. 190; Tech. Rep.).

de Putter, P. (2013). Als een paal boven water. *Vastgoedrecht*, *5*.

Dhanasekar, M., Page, A., & Kleeman, P. (n.d.). *The behaviour of brick masonry under biaxial stress with particular reference to infilled frames* (Tech. Rep.). Department of Civil Engineering and Surveying, University of Newcastle, N.S.W. Australia.

DINOloket. (n.d.). *B25g0525.* (Coordinates: 120880, 485340 (RD))

DINOloket. (1990). *Cpt000000031722.* (Coordinates: 120928.000, 485289.000 (RD))

DINOloket. (2002). *Cpt000000030378.* (Coordinates: 120931.000, 485298.000 (RD))

DINOloket. (2012a). *Cpt000000071308.* (Coordinates: 120923.000, 485331.000 (RD))

DINOloket. (2012b). *Cpt000000071316.* (Coordinates: 120890.000, 485330.000 (RD))

Gemeente Amsterdam. (n.d.). *Duinwaterleiding.* Retrieved from https://archief.amsterdam/stukken/water/duinwaterleiding/index.html

Gemeente Amsterdam. (1802). *Archive object 14224.*

Gemeente Amsterdam. (1872). *Archiefstuk 12015-1.*

Gemeente Amsterdam. (1875). *Archive object 26026.*

Gemeente Amsterdam. (1882a). *Archive object 13030-3.*

Gemeente Amsterdam. (1882b). *Bestek en voorwaarden.*

Gemeente Amsterdam. (1895a). *Archive object 13008-1.*

Gemeente Amsterdam. (1895b). *Archive object 13183.*

Gemeente Amsterdam. (1898). *Plaat 2 diverse aanzichten.*

Gemeente Amsterdam. (1903). *Archive object 12002.*

Gemeente Amsterdam. (1907). *Archive object 11029.*

Gemeente Amsterdam. (1908). *Archive object 12010-4.*

Gemeente Amsterdam. (1909a). *Achive object 13137-1.*

Gemeente Amsterdam. (1909b). *Bestek brug 137.*

Gemeente Amsterdam. (1911). *Archive object 12131.*

Gemeente Amsterdam. (1913a). *Archive object 14295-2.*

Gemeente Amsterdam. (1913b). *Archive object 22460-1.*

Gemeente Amsterdam. (1921). *Archive object 23997.*
Gemeente Amsterdam. (1922a). *Archive object 13030: Bestek en voorwaarden.*
Gemeente Amsterdam. (1922b). *Archive object 24250.*
Gemeente Amsterdam. (1923a). *Archive object 244183.*
Gemeente Amsterdam. (1923b). *Archive object 24418: Bestek en voorwaarden.*
Gemeente Amsterdam. (1925a). *Archive object 24760-1.*
Gemeente Amsterdam. (1925b). *Archive object 24760-6.*
Gemeente Amsterdam. (1925c). *Archive object 24760:bestekstuk 9-1925.*
Gemeente Amsterdam. (1967). *Archive object 45204: Berekening brug 15.*
Gemeente Amsterdam. (2019). *Projectleider wetenschappelijk onderzoek.* Retrieved from `https://www.amsterdam.nl/bestuur-organisatie/werkenbij/externe/@1138810/19040090/`
GVB. (n.d.). *Tram in cijfers.* Retrieved from `https://over.gvb.nl/ov-in-amsterdam/feiten-en-cijfers/tram-in-cijfers/`
Huat, B., Kazemian, S., Prasad, A., & Barghchi, M. (2011, 02). A study of the compressibility behavior of peat stabilized by dmm: Lab model and fe analysis. *Scientific Research and Essays*, *6*, 196-204.
Kaushik, H. B., Rai, D. C., & Jain, S. K. (2007). Stress-strain characteristics of clay brick masonry under uniaxial compression. *Journal of Materials in Civil Engineering*, *19*(9), 728-739. doi: 10.1061/(ASCE)0899-1561(2007)19:9(728)
Klaassen, R. K. (2008). Bacterial decay in wooden foundation piles—patterns and causes: A study of historical pile foundations in the netherlands. *International Biodeterioration & Biodegradation*, *61*(1), 45 - 60. Retrieved from `http://www.sciencedirect.com/science/article/pii/S0964830507001072` doi: https://doi.org/10.1016/j.ibiod.2007.07.006
Korrel, P. (n.d.). *Diamantbrug, brug 137 in de ruysdaelstraat over de boerenwetering.*
Moore, J., & Maguire, D. (2004, 01). Natural sway frequencies and damping ratios of trees: Concepts, review and synthesis of previous studies. *Trees - Structure and Function*, *18*, 195-203. doi: 10.1007/s00468-003-0295-6
Museum Amsterdam. (n.d.). *Groeikaart amsterdam.* Retrieved from `https://hart.amsterdam/nl/page/238543`
Neijzing, L. (2011). *Herberekening brug 8, raadhuisstraat, op houten palen gefundeerde staalbeton brug* (Tech. Rep.). Gemeente Amsterdam.
Neijzing, L. (2019). *Toetskader amsterdamse kades: Versie 1* (Tech. Rep.). Ingenieursbureau Amsterdam.
NEN. (2018). *Nen-en 1997-1+c1+a1/nb+c1* (Tech. Rep.). Author.
OpenStreetMap contibuters. (n.d.). *Openstreetmap.* Retrieved from `https://www.openstreetmap.org/#map=14/52.3688/4.8802`
Panshin, A., & de Zeeuw, C. (1970). *Textbook of wood technology* (3rd ed., Vol. 1). McGraw-Hill.
Persson, K. (2000). *Micromechanical modelling of wood and fibre properties* (Doctoral dissertation). doi: 10.13140/RG.2.1.1389.7685
Pluijm, van der, R. (1997). Non-linear behaviour of masonry under tension. *Heron*, *42*(1), 25–54.
Quansah, A. (2010). *Herberekening brug 137, albert cuypstraat* (Tech. Rep.). Ingenieursbureau Amsterdam.

Quansah, A. (2014). *Constructieve beroordeling van brug 41* (Tech. Rep.). Ingenieursbureau Amsterdam.

Rikkers, M., & Lodema, M. (2009). *Diverse onderzoeken brug 137* (Tech. Rep.). NEBEST B.V.

Roesset, J., Wang, S. T., Vasquez, L., & Fadaifard, H. (2016). Dynapile 2016 – user's manual [Computer software manual].

Sas, F. (2006). *De houten paalfundering doorgezaagd.* (Tech. Rep.). F3O.

van den Broek, B. (2014). *Constructieve beoordeling van brug 30* (Tech. Rep.). Ingenieursbureau Gemeente Amsterdam.

van Noort, J. (2012). *Computational modelling of masonry structures* (Master Thesis). TU Delft.

Walker, P., Kioy, S., & Jowsey, A. (2014). An experimental comparison of hydrated lime and an admixture for masonry mortars. International Masonry Conference.

Zainorabidin, A., & Mad Said, M. J. (2015, 12). Determination of shear wave velocity using multi-channel analysis of surface wave method and shear modulus estimation of peat soil at western johore. *Procedia Engineering*, *125*, 345-350. doi: 10.1016/j.proeng.2015.11.073

Zainorabidin, A., & Wijeyesekera, D. (2019, 05). Shear modulus and damping properties of peat soils.

# Appendices

# A Analysed Bridges

For 11 bridges within the municipality of Amsterdam, a small archive study was done. The location of the 11 bridges within Amsterdam is visible in figure 32. The next 11 subsections are a description of what was found during these archive studies. The subsection starts with a bit of information into the bridge, such as location and building year. The subsection continues with more depth into the geometry of the bridge and a more detailed view of the bridge materials and build up. Each of the section ends with a summation of the monitoring data that has been found in the study.

## Bridge 8

Bridge 8 is located in the Raadhuisstraat and crosses the Singel. The first bridge on this location had been built in 1773, which has been highly altered in 1894. In 1901, the bridge was reinforced to support the tram. In 1925 the bridge has been changed for the last time, widening the bridge again. The bridge is available to both normal traffic and trams. The bridge has two lanes dedicated to pubic transport, two lanes for cars and other motorised traffic and two cycling lanes. the bridge also has two wide pedestrian pathways. When coming to the centre of Amsterdam by car, this is the one of the few route that leads to the centre (OpenStreetMap contibuters, n.d.). Therefore this bridge is of great importance for the city of Amsterdam, its citizens and tourists.

### Geometry and build up

The bridge is built on two abutments and two pillars. The spans of the bridge are 5.05 metres between the pillars and the abutments and 6.86 metres in between the pillars. With a pillar thickness of 1.43 metres, the total span between the two abutments is equal to 19,82 metres. The width of the bridge is 26 metres. The bridge has been built in three different times, which makes it difficult to determine the combined situation form the different drawings. The deck of the bridge is a concrete plate, supported by steel beams. The deck is visible in figure 33.

### Foundation and abutment

The first bridge is built in 1773, of which drawings are unavailable. More than a century later, in 1895, the bridge has been widened according to Gemeente Amsterdam (1895a), visible in figure. A wooden pile foundation was used again. This new wooden foundation exists out of the same pattern on both sides of the bridge. Under the abutments are 7 piles in a row, which are coupled to a ground beam. Under the pillars is a foundation of three piles, also under a ground beam. The beam under the abutment and the beam under the pillar are in half of the cases one single beam, under which another pile is placed in the middle of the span from abutment to pillar.

In 1925, the bridge has been widened again according to Gemeente Amsterdam (1925a), making the bridge available to house multiple lanes for public transport and cars. However, only foundations under the wings of the bridge and an extension of the pillars have been added in this widening, which does not

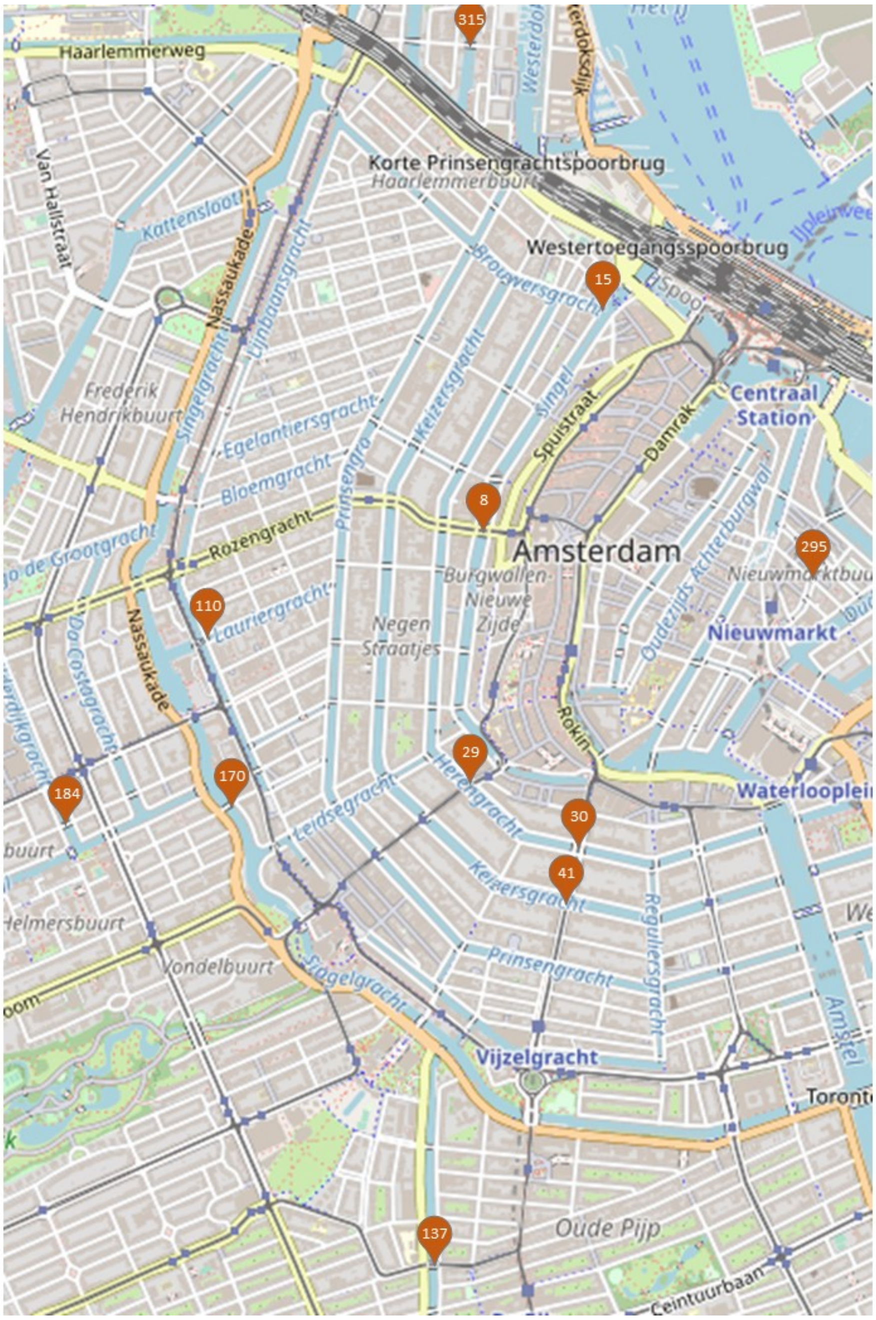



Figure 32: Location of the analysed bridges

Figure 33: The deck as used in bridge 8 (Neijzing, 2011)

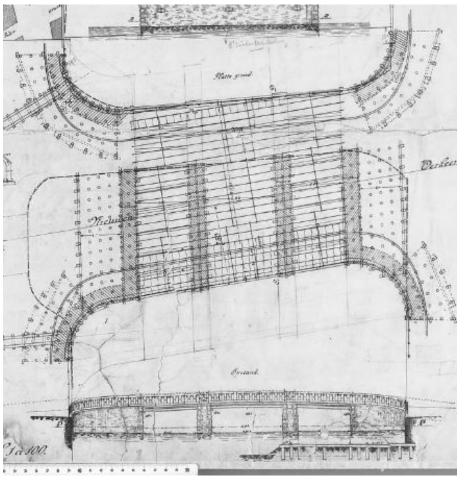


Figure 34: Foundation bridge 8 as built 1895 (Gemeente Amsterdam, 1895a)

change the abutment itself. The foundation under the wings varies from four pile towards the centre of the bridge towards 8 at the corner of the bridge. At the water side, the first pile has a slope of 1/10. The foundation that is placed under the extension of the pillar has 3 piles, just like the bridge widening from 1925. Two of the three of which the water side have a slope of 1/10 again. The ground beam under the abutment and the pillar do not seem to be connected, like in the old situation. The width of the abutment is 1.54 m. The width of the bridge comes to a total of 25.97 metres. According to the drawings, the widening at this stage was 6.40 m and 6.565 m(Gemeente Amsterdam, 1925b). The foundation under the new wings and parts of the pillar exists out of 14 metre, pinewood piles. In total, the widening required 220 piles under the foundation, and 63 under the pillars (Gemeente Amsterdam, 1925c).

The foundation is placed with the shear timber in the direction of the bridge, while the ground beams are perpendicular to this direction. Under the pillars, there are three foundation piles under the ground beams, while at the abutment, up to seven piles are placed under the beams. The two outer piles at the pillars are sloped with a ratio of 1/10. At the abutments, the same is done. However, at the abutments, the first two piles are placed under a ration of 1/10. The timber construction between the piles and the abutment of pillars, also contains a piece of shear timber.

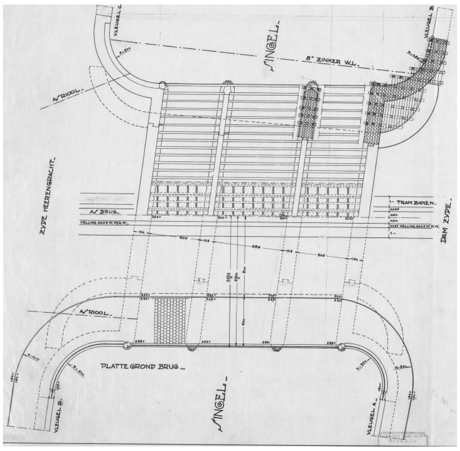



Figure 35: Foundation bridge 8 as built 1925 (Gemeente Amsterdam, 1925a)

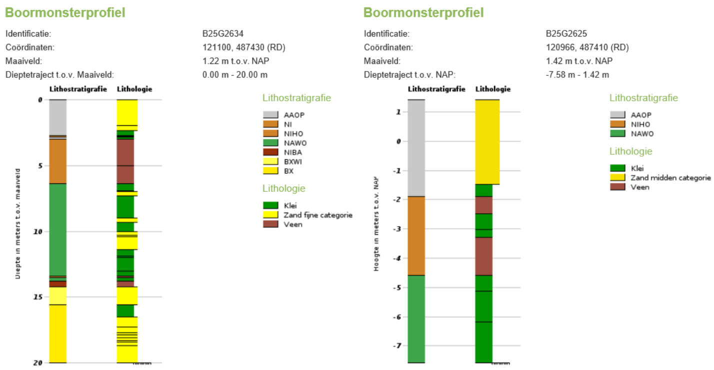
### Monitoring and testing

Over the last years measurements have thoroughly been done on this bridge, which might allow this bridge to be used as a case study for the model. Deformation measurements have been first done in 2006. Repetition measurements have been done in 2009 and 2011, which gives a detailed overview of bridge movements over three years. The bridge has also been recalculated in 2011, in which multiple tests have been done, both on deck and foundation. This recalculations also give some materialistic properties. However, these properties are not focused on the foundation, but on the concrete deck. The foundation was expected to hold 100 kilonewtons, but has not further been tested (Neijzing, 2011). The calculation report could still be used to make a force estimation on the bridge and to look at the loading that needs to be applied in the model. Although a lot of information is present on deformation, the bridge has not been proven to are in a state of pile foundation failure. There is however the expectancy that the bridge has a bad foundation.

The bridge deck consists out of a combination of steel profiles and concrete. The tram tracks are supported by a concrete deck

### Soil conditions

Around bridge two, two different soil profiles were looked at. The two are visible in figure 36. Since one of the two is taken up to 20 metres and one to 7 metres, only the first metres can be compared. The two are however very similar, except for the layer of peat. According to figure 36a, the layer of peat is from 3 metres below ground level to 7 meters below ground level. Figure 36b shows that the metres contain both clay and peat layers. The peat is followed by multiple layers of sand and clay. In 36a, a thick sand layer can be found from about 14 meters, which is a good explanation for the length of 14 meters for the pile foundation.




(a) Soil sample 1 (b) Soil sample 2

Figure 36: Soil conditions in the surrounding area of bridge 8

## Bridge 15

Bridge 15 is built over the Brouwersgracht and is part of the Singel. The first bridge has been build in 1880, which has been renewed in 1897. After 1897, the bridge has been strengthened in 1924. The bridge supports only normal traffic and only has room for one vehicle at a time.

### Geometry and build up

The bridge has a length of 8 metres and a width of 9 metres and is supported on 2 abutments(Gemeente Amsterdam, 1872).

### Detailed overview of foundation and abutment

The foundation under both of the abutments seems to be the same, based on the limited information on the bridge. Starting at the water side of the foundation, the foundation rows start with one sloped pile, of which the exact slope is undefined. Further from the water, the row has five more foundation piles, which all seem to not be sloped. The sloped pile and vertical piles are all connected to one ground beam. The masonry abutment is built onto the first two piles, on top of the wooden flooring. Three pieces of shear timber are used.The abutment is supported by ten rows of the described foundation piles, with two extra rows on both sides of the abutment to support the quay walls surrounding the bridge. The height of the abutment is 3.40 metres.(Gemeente Amsterdam, 1872)

### Monitoring and testing

Except for a visual inspection in 2016 (Antea), no monitoring has been done on bridge 15. A full calculation of the bridge deck was done (Gemeente Amsterdam, 1967), but there are no reports on other structural elements.

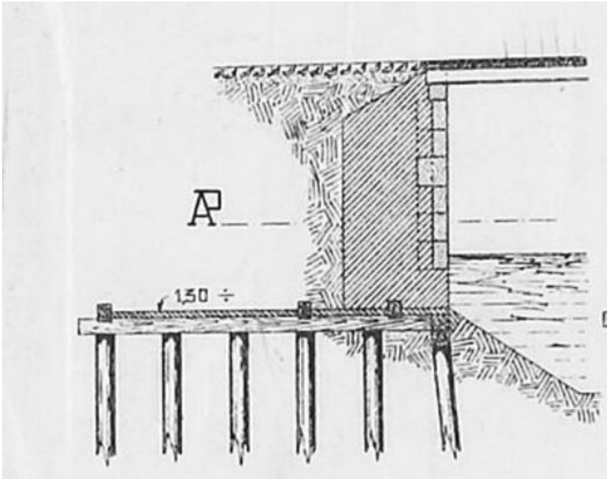
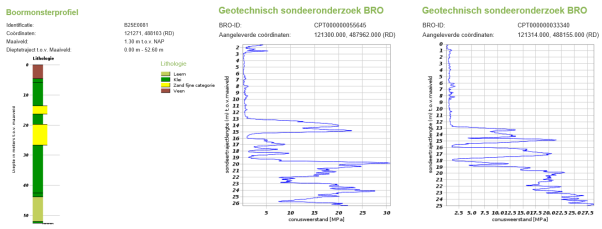



Figure 37: Foundation bridge 15 (Gemeente Amsterdam, 1872)




(a) Sample (b) CPT 1 (c) CPT 2

Figure 38: Soil conditions in the surrounding area of bridge 15

**Soil conditions**

The sample in figure 38 shows a peat leayer at the top of the sample and a bigger sand layer at approaximatly 15 meters below the ground level. The soil conditions from the surrounds of bridge 15 are mainly from the CPT. This clearly shows a large layer of peat at the top of the soil layers. The also shows the carrying layer of sand around 15 to 18 metres. This is even better visible in the cone penetration testings, visible in figures . This shows a high cone resistance ranging from 13 to 15 metres and again rising around 19 metres.

## Bridge 29

Bridge 29 is built in the Koningsplein and crosses the Herengracht. The first bridge on the location of what is currently bridge 29 has been built in 1752, which has been lowered and widened over the centuries. The bridge was lowered in 1875 (Gemeente Amsterdam, 1875) and widened in both 1907 and 1921. In 1903, the bridge was altered to allow tram traffic to use the bridge (Gemeente Amsterdam, 1903). The bridge houses normal traffic and tram traffic. There is only one lane available to normal traffic, which is why the bridge is a one way bridge for cars. The bridge has tram tracks in two direction and multiple

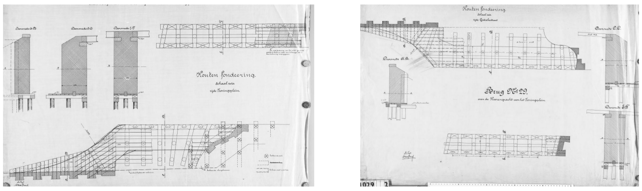


Figure 39: Foundation bridge 29 as built in 1907 (Gemeente Amsterdam, 1907)

locations for slow traffic, both on the sides and in the middle of the bridge.

**Geometry and build up**

The bridge is supported by two abutments and two pillars. The bridge from 1907 was 18 metres wide and the final bridge from 1921 is 25 metres wide. The distances between the pillars is 6.95 metres and the distance between pillar and abutments is 4,73 on the side of the Leidseplein and 4.75 on the side of the Koningsplein. (Gemeente Amsterdam, 1921)

**Detailed overview of foundation and abutment**

The earliest drawings found are form 1907, which is the first time the bridge had been widened. According to Gemeente Amsterdam (1907), two new wings were added on the southeastern part of the bridge, which widened the bridge to 18 metres. Under both of the new wings, a ground beam is placed at the front side of the new wing, in line with the abutment. Under this ground beam, multiple piles are placed. In between the piles that are placed under the ground beam perpendicular to the span, more ground beams are placed in line with the span. Under these ground beams, more foundation piles are placed. Under each of these beams, 2 piles are placed. More close to the original abutment, the ground beams are placed under a angle, such that they do not interfere with the original abutment. It appears that both under the new parts of the abutment and under the pillars, three piles have been placed, of which none are sloped. The piles are not placed in line with the span of the bridge but are under an even larger corner than the bridge and the perpendicular direction of the bridge. Under both of the pillars, the foundation is placed in a diamond like pattern, with the outside piles and inside piles alternating, both supported by a ground beam. IT appears that about twenty piles were used under the pillars. Both the piles under the pillars and under the abutments did not seem to be sloped. (Gemeente Amsterdam, 1907) The abutment thickness is 1.54 metres, the pillar thickness is 1.43.

The drawings form 1929 show the final widening, which are mainly based on the build up of the newly build wings. These wings are built on 14 metre, pine wood piles. In total 51 are used at the side of the Singel, while 69 are used at the side of the Leidsestraat (Gemeente Amsterdam, 1921). Under the wings, all ground beams have been placed in perpendicular direction to the bridge span. Furthermore, the whole new foundation makes no sense whatsoever.

Also in this foundation are two different water pipes. The reason that these are two different pipes, both for drinking water is, that two different companies formerly supplied the city of Amsterdam with drinking water. from the

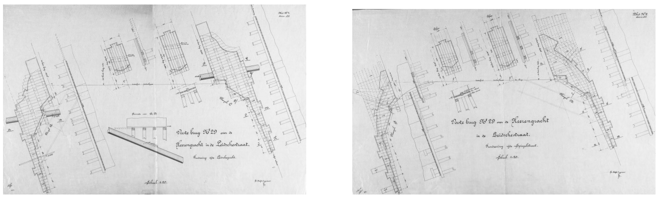
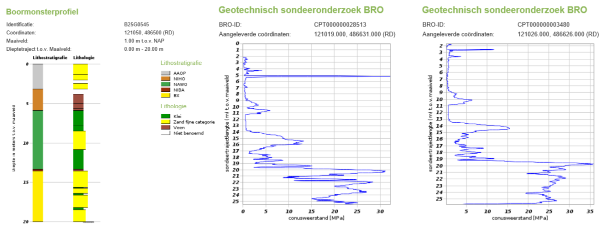


Figure 40: Foundation bridge 29 as built in 1921 (Gemeente Amsterdam, 1921)




(a) Sample (b) CPT 1 (c) CPT 2

Figure 41: Soil conditions in the surrounding area of bridge 29

eighteenth century, the water in the quays was of such bad condition, that other methods had to be found. The solution was to built water wells in the city, filled with rain water and water from the Vecht, a river in Utrecht. This changed in 1854, when dune water was imported by a private company and supplied throughout the city. However, since the water from the Vecht was still supplied, 2 separate water systems needed to be built to be able to supply the water.(Gemeente Amsterdam, n.d.)

### Monitoring and testing

No measurement of testing have been done on bridge 29, except for the visual inspections of Antea. Therefore this bridge would not be suitable as a test case.

### Soil conditions

According to the soil sample, visible in figure 41a, the first layers are unclear, but containing a thick peat layer from 4 to six meters. A lower sand layer is visible from 8 to 11 metres, but the layers form 13 metres downwards are thicker, containing more bearing capacity. Considering the cone penetration test, the sand layer can be distinguished around ten meters, but has a low bearing capacity. The layer form 14 to 15 meters has a much higher bearing capacity. The other CPT also shows a strange peak around 5 meters,, which appears to be a small sand layer, but uncertainty remains.

## Bridge 30

Bridge 30 is built over the Herengracht and is part of the Vijzelstraat. Bridge 30 was built in 1727 and widened over the past centuries. The bridge has room for normal traffic and trams. Four lanes are dedicated to normal traffic, while there are also two lanes assigned to public transport. There are no separate lanes for cyclists, but there are pedestrian paths on both sides of the bridge.

### Geometry and build up

The total length of the bridge is 22 metres long and has a width of 26 metres. The bridge is supported by two abutments and two pillars. The distance from pillar to pillar equals 6.98 metres and the distance form pillar to abutment 4.75 metres on both sides of the bridge. The deck exists out of steel beams which are surrounded by concrete. The deck is visible in figure 42.

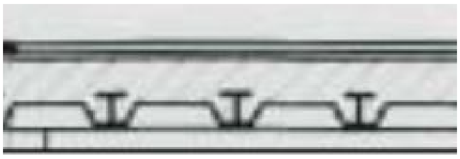


Figure 42: The deck as used in bridge 30 (van den Broek, 2014)

### Detailed overview of foundation and abutment

The first bridge from 1727 has been widened in 1882. In this widening, wings have been added to the structure form 1727. The new parts of the abutment are built on four row of foundation piles, with three piles closer to the middle of the bridge and 5 piles at the more outwards parts of the bridge. The piles most close to the water is sloped, but the slope of this pile is not given. The foundation under the pillars is built up out of three piles, of which the middle is placed vertical and the other two are sloped. According to Gemeente Amsterdam (1882b), the foundation form 1882 contains 72 piles under the pillars and 86 under the abutments. According to the same document, the piles are pinewood piles and are 14 metres long. The piles have a circumference of 85 centimetres at the top and 35 centimetres at the bottom.

The bridge has been widened again in 1921, with wing extensions on both sides of the bridge. The smallest extension is on the east side of the bridge, where the extension was limited to two row under the abutment of the bridge. The extension of the wings and abutment is build on existing foundations and therefor does not require additional foundation piles.

On the west side of the bridge, the extension was more complex. According to Gemeente Amsterdam (1921) (objects 23997-43 and 23997-47) 12 extra rows of piles were added under both abutments. The added rows exist out of three piles. The pile closest to the water is sloped, but a sloped remains unknown. Under each of the pillars, eleven rows of piles were added, of which the two closest to the water are sloped. According to the specifications, pine wood piles were used, which were 14 metres long. In total, 38 piles were used under each of the abutments. A circumference of the piles is not given. Since the eastern part of the extension did not require piles, the total is right. Under the pillars, 66 foundation piles are added. This corresponds to the fact that in the drawings, 11 rows of three piles were added to both of the pillars. The thickness of the

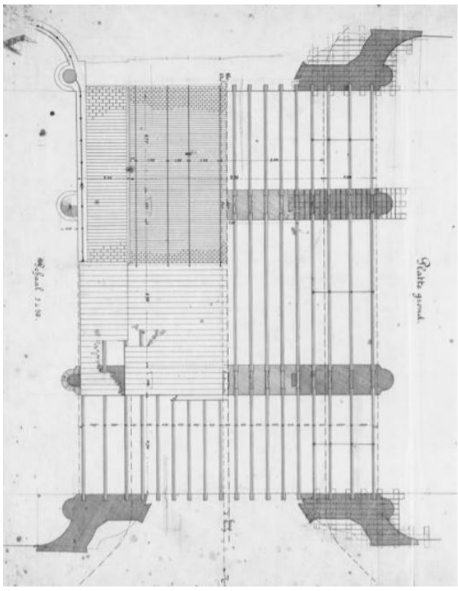


Figure 43: Foundation bridge 30 as built in 1882 (Gemeente Amsterdam, 1882a)

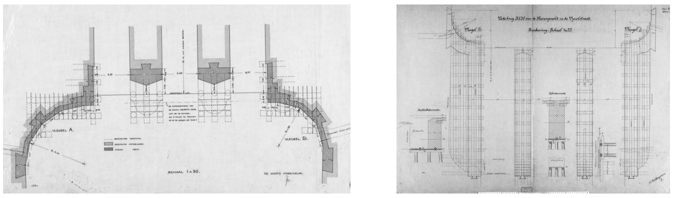


Figure 44: Foundation bridge 30 as built in 1921 (Gemeente Amsterdam, 1921)

abutment is equal to 1.21 metres, while the pillars have a thickness of 1.43 metres.

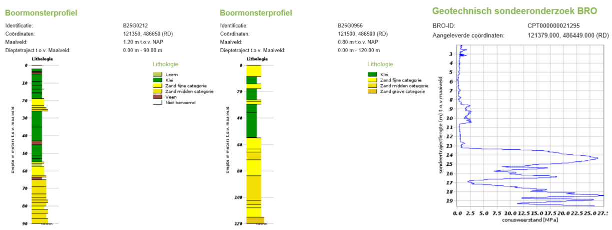
**Monitoring and testing**

On the bridge, deformation measurements are available from both 2010 and 2011, but the bridge has not been tested since. A structural assessment has been done on the bridge in 2014, in which all parts have been modelled. This concluded that the bridge piles where not safe, but no samples have been taken form the bridge which showed that this was in fact the case. The report could be used as a reference to the parameters for the model.

According to (van den Broek, 2014), the bridge deck exists out of steel and concrete. The deck is loaded with cars, trucks and trams. For the recalculation, the own weight, the traffic load, both in vertical and horizontal direction, and the wind load have been used. The assessment did however show that almost all of the parts meets the requirements, the pile foundation appears to not be strong enough.

Table 13: Material properties bridge 30 (van den Broek, 2014)

| Material | Quality | Specific weight (kN/m$^3$) |
|---|---|---|
| Concrete (Bridge deck) | K300 (=C20/25) | 25.0 |
| Reinforcement Steel | QR24 (=S235) | 78.5 |
| Structural Steel | QR24 (S235) | 78.5 |
| Finishing layers (tiles, asphalt, etc.) | *Not applicable* | 23,0 |
| Masonry | *Not available* | 20/30 (Dry/wet) |
| Timber | Pine wood (C18) | 10 |
| Soil | *Not applicable* | 18/20 (Dry/wet) |




(a) Sample 1 (b) Sample 2 (c) CPT

Figure 45: Soil conditions bridge 30

**Soil conditions**

The soil conditions from bridge are based on three soil conditions. However, tow of the three are very broad meetings, which shows limited information on the upper soil layers. The fact that these measurements are so deep might be explained by the fact that these bridges are above the Noord/Zuidlijn, which might required soil conditions form larger depth. From 13 to 17 metres, a large sand layer is visible in the CPT. This is also visible in figure in figure ..., but not visible in the figure. Both of the figures do however show a sandy layer from 60 metres downwards.

## Bridge 41

Bridge 41 is located in the Vijzelstraat and crosses the Keizersgracht. Considering the fact that this used to be the best route to travel from south the centre before the opening of the Noord/Zuidlijn, it is still a highly used road. When coming from the south towards the centre, the Vijzelstraat is the easiest way to enter the centre (OpenStreetMap contibuters, n.d.), which is why the Vijzelstraat is very much used. The first bridge on this location was built in 1738 and changed throughout the last centuries, widening and lowering the bridge over time. Considering that this bridge also lies in the Vijzelstraat, just like bridge 30, the bridge also has four lanes for normal traffic, two lanes dedicated to public transport and no assigned cycling lanes. There are also wide pedestrian paths on both sides of the bridge.

### Geometry and build up

The bridge has a length of 19 metres and a width of 26.25 metres. The bridge is supported by two abutments and two pillars. The distance form pillar to pillar is equal to 6.88 metres, the distance for pillar to abutment is equal to 4.72 metres on both sides. The thickness of the abutments is equal to 1.43 metres. The bridge has a similar deck to bridge 30, with steel beams surrounded by concrete. the deck is visble in figure

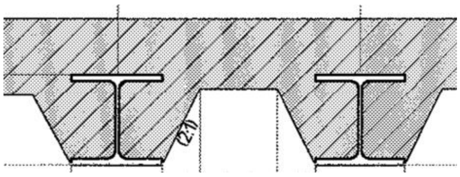


Figure 46: The deck as used in bridge 41 (Quansah, 2014)

### Detailed overview of foundation and abutment

The first bridge was built in 1738, but the first drawings are form the widening and lowering in 1882. According to Gemeente Amsterdam (1882a), the foundation that has been placed in 1882, the widening mainly includes 4 wings, which have been built on a foundation containing multiple rows of piles. The rows of piles range from having 3 piles more towards the centre of the bridge, while having five piles at the edge of the abutment. The piles are coupled by a ground beam, onto which flooring and multiple shear beams are placed. The thickness of the masonry abutment is 1.25 metres. The row closest to the water seems to be sloped, but the exact slope is undefined. Under the pillars, the rows of piles exist out of three piles, of with the outer ones are sloped. According to Gemeente Amsterdam (1882b), the same piles were used as for bridge 30, which are 15 metre piles with a top circumference of 85 centimetres and a bottom circumference of 35 centimetres. In total 72 piles are used under the pillars and 100 piles were used under the abutments. These numbers would be reasonable, when looking at the drawings.

The widening in 1922 involved the building of two new wings, built on the west side of the bridge. The north-west wing is supported by three foundation piles in the middle, where it partially overlaps with the wings built in 1882. After the one row of three piles, the number increases to four piles for two rows, and then five rows until the end of the abutment. At the south-west wing, something similar happened, starting with one row of 3 piles, and then increasing to four piles all the way to the end. Under the pillar, the old situation does not repeat in the new situation, but a change is made in the pattern. In 1882, three piles where placed in line with the bride, but in 1922, a diamond shaped pattern was used, such that the outside piles alternate with the inner pile. This is also why ground beams are used in multiple directions. The outside lines are connected by a ground beam over the full width of the bride, while the inner line of piles is connected to the outer lie ground beam with another ground beam. On top of the middle ground beam, flooring is placed with a piece of shear beam. The widening in 1922 widened the bridge from 14.30 metres to

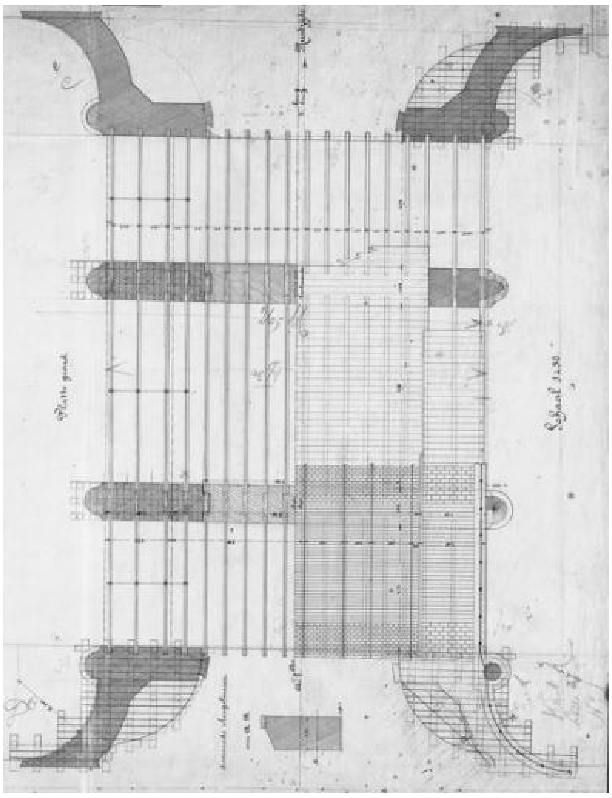


Figure 47: Foundation bridge 41 as built in 1882

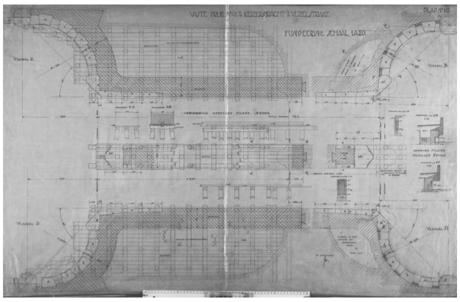


Figure 48: Foundation bridge 41 as built in 1922 (Gemeente Amsterdam, 1922b)

the final width of 26.25 metres. According to Gemeente Amsterdam (1922a), for the latest widening 53 piles were used under the north-west wing, 63 under the south west wing and 70 under the pillars. The piles are pinewood piles, 14 metres long and have a circumference of 80 centimetres at the top and 38 centimetres at the bottom.(Gemeente Amsterdam, 1922b)

### Monitoring and testing

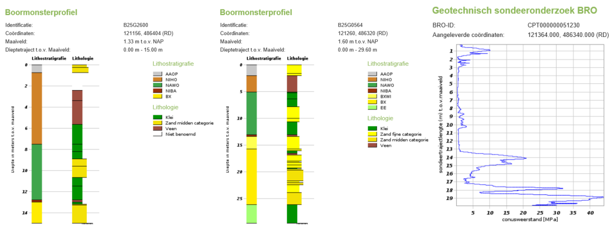
Deformation measurements have been done in multiple years on bridge 41. In 1973, 2002, 2009, and 2011 measurements have been done. These calculations could be used to create a scenario case. However, foundation problems are not proven, which means that the case could not be used to validate the model. According to the recalculation of bridge 41, the foundation does not function as well as it should, but this has not been tested with a pile boring (Quansah, 2014). However, the recalculation report does give some knowledge into the specific bridge properties.

Table 14: Material properties bridge 41 (Quansah, 2014)

| Material | Quality | Specific weight (kN/m$^3$) |
|---|---|---|
| Concrete (Bridge deck) | C30/37 | 25.0 |
| Reinforcement Steel | B500B | 78.5 |
| Structural Steel | S355 | 78.5 |
| Finishing layers (tiles, asphalt, etc.) | *Not applicable* | 23,0 |
| Masonry | *Not available* | 20/23 (Dry/wet) |
| Timber | Pine wood (C18) | 10 |
| Soil | *Not applicable* | 18/20 (Dry/wet) |

### Soil conditions

The soil conditions are based on a CPT and two samples. these samples show how a sand layer is present from about 13 below ground level. What can also be clearly distinguished form all three sources are the peat and clay layers above the sand layers, being susceptible to settlement.



(a) Sample 1 (b) Sample 2 (c) CPT

Figure 49: Soil conditions bridge 41

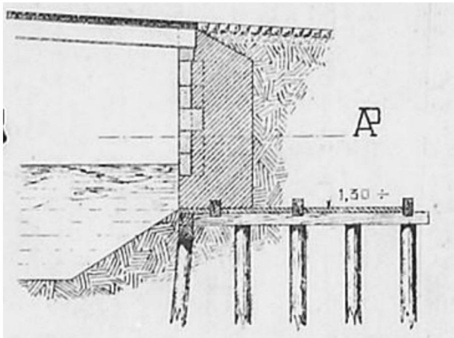



Figure 50: Foundation bridge 110 (Gemeente Amsterdam, 1872)

## Bridge 110

Bridge 110 crosses the Lauriersgracht and is part of the Lijnbaansgracht. Bridge 110 was built in 1897. Over the last 100 years, the bridge deck has been replaced multiple times. The bridge houses normal traffic and has room for one car at a time, but a car can use the sidewalk to pass.

### Geometry and build up

The bridge has a length of 8,2 metres and a width of 10 metres. The bridge is supported on two abutments. (Gemeente Amsterdam, 1872)

### Detailed overview of foundation and abutment

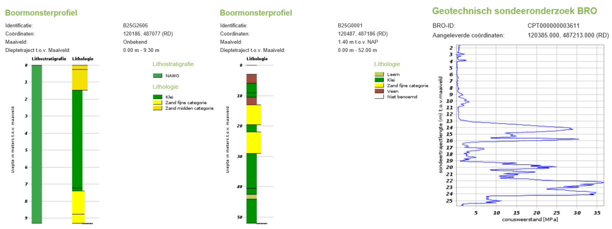
The bridge abutment and foundation are very similar to bridge 15, which can be explained by the fact that they are built in the same time frame. The foundation under both of the abutments seems to be the same, based on the limited information on the bridge. Starting at the water side of the foundation, the bridge has one sloped pile, followed by four non sloped piles. (Gemeente Amsterdam, 1872)

### Monitoring and testing

No measurement of testing have been done on bridge 29, except for the visual inspections of Antea. Therefore this bridge would not be suitable as a test case.

### Soil conditions

The soil figures from 110 are very different in ranges, which makes it harder to state things. The shorter sample shows two small sand layers, with a large clay layer in between the sand layers. The larger sample shows multiple peat and clay layers above 12 to 13 meters, while the sample continuous with a large sand layer to 20 meters. This also explains why the piles are 13 metres long, since this is a bearing sand layer. Much deeper, the sample also shows a layer of loam, which is not seen very often in the samples. The CPT shows a similar soil layer composition as the larger sample, containing a layer of sand at 14 to 16 meters, and again at 20 metres.




(a) Sample 1 (b) Sample 2 (c) CPT

Figure 51: Soil conditions bridge 110

## Bridge 137

Bridge 137 is a part of the Ruysdaelstraat and spans the Boerenwetering. The bridge is made in 1909. The bridge has 3 lanes, of which one is assigned form tram only, one is used both by tram and normal traffic and the final one can be used by normal traffic. The bridge has a cycling lane on one side, which is headed west and pedestrian paths on the sides of the bridge.

### Geometry and build up

The bridge is 16.28 metres wide and 25 metres long. The bridge is supported by two abutments and two metal pillars. The distance between the pillars is the same as the distance between the pillars and the abutments, which is 8 metres. The thickness of the pillars is equal to 50 centimetres. The deck in bridge 137 exists out of waved plates filled with concrete, supported by steel beams. The deck is visible in figure 52

### Detailed overview of foundation and abutment

The foundations under this bridge are very simple, according to Gemeente Amsterdam (1909a). Under the abutments, which are metal, are single straight lines of 11 piles, all straight. According to the drawing, these piles seem to be different than the ones used under the abutments. This can be concluded form the building documents, since the piles under the pillars are oak wood piles, which are 40 times 40 cm at the top and 15 times 15 at the bottom. These piles are 16 metres and in total, 22 are said to be used, which matches the drawings. Under

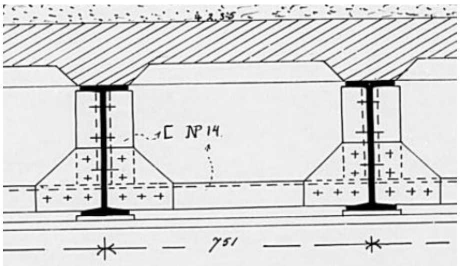


Figure 52: The deck as used in bridge 137 (Quansah, 2010)

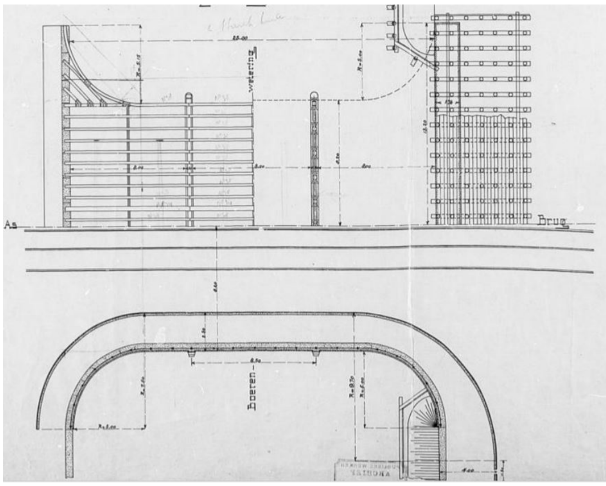


Figure 53: Foundation bridge 137 (Gemeente Amsterdam, 1909a)

the abutments, 28 rows of 7 piles are placed. Again, all of the piles used seem to be straight and not under a slope. The distance between the piles is defined in the drawings and is 1 metre, both in the direction of the ground beam as from ground beam to ground beam. On the ground beam, a flooring is placed, which is 8 centimetre thick. The abutment at the side of the Albert Cuypstraat is 4.9 metres high, while the abutment at the side of the Ruysdahlstraat is 5.02 metres high. According to Gemeente Amsterdam (1909b), the piles under the abutment are 14 metres log and have a circumference of 85 cm at the top and 38 centimetres at the bottom. Furthermore, the used piles are pinewood. 196 piles were used under both of the abutments, which suits the drawing. (Gemeente Amsterdam, 1909b)

**Monitoring and testing**

In 2009, Nebest has tested the piles, of which the conclusion was the piles under the abutment were influenced by biological deterioration which would continue for another 25 years such that 30% of the pile would be deteriorated(Rikkers & Lodema, 2009). In the same report however, Rikkers and Lodema also concluded that the piles under the pillars, which are oak wood, are not deteriorated and still have a large bearing capacity. Next to the test, two deformation measurements have been done, in 2007 and in 2010. Finally, a recalculation has been done on the bridge. In this recalculation, multiple things were tested. First of all, the strength of the concrete was tested, which concluded that the concrete was of type , which is of the type C45/B55, which appears to be the same as C45/55. A strength of reinforcement is also given, which is QR24. This type of reinforcement has an $f_{yk}$ of 240 MPa and and $f_{yd}$ of 209 MPa. The structural steel has also been tested, which resulted in the fact that it is S235. As stated earlier, the piles are both of oak and pine. The piles under the abutment have been partially tested, which resulted in finding out that the piles are partially deteriorated, which has taken into account during the recalculation. The middle area area of the piles of bridge 137 is however still stable. The strength of the remaining timber appears to be at least 11 MPa. The pile tips bearing capacity is however lower, which has been calculated based on the bearing capacity of the pile tip and the area of the pile tip. Since this is the lowest value of the stress and the bearing capacity, the 108 kN is used for the calculation whether the bridge is still safe. The oak piles under the pillars are however much stronger, which can be explained by the fact that the oak wood of the pile is hardly damaged. This gives the oak wood piles a remaining strength of 719 kilonewtons per pile.Therefor, the could help providing mechanical properties of the bridge (Quansah, 2010).

### Soil conditions

The soil conditions around bridge 137 are mainly based on the CPT's, since these contain the most precise information. The CPT's all show a sand layer around 14 metres and multiple smaller layers of sand further down. This is also shown in the sample, which shows sand layers of multiple categories.

## Bridge 170

Bridge 170 is part of the Bosboom Toussantstraat and lies over the Singelgracht. The bridge is called 'Koekjesbrug', but the reason for the name is debated over many years. Theories range from a police officer that was called Koek up to the cookie sellers that used this bridge to sell, since it was a main road to the centre for many. The bridge has first been rebuilt in 1896, but was replaced in 1911. The bridge has two lanes for normal traffic and two sidewalks for pedestrians.

### Geometry and build up

The bridge is supported on two abutments and two pillars, which are metal structures. The distance between the pillars is 10.80 metres the distance form pillar to abutment is 6.86 metres on both sides. the pillar thickness is 0.35 metre, which brings the total span of the bridge to 25 metres. the width of the bridge is 10 metres. Under the eastern bridge abutment, a former culvert is present, which allows water flow under the abutment. It was not found where this culvert leads to.

### Detailed overview of foundation and abutment

According to the drawings, the bridge from 1896 is completely renewed in 1911, which is why only the drawings of the bridge from 1911 are used in describing

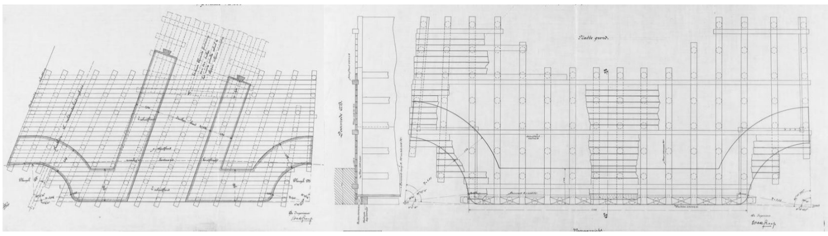
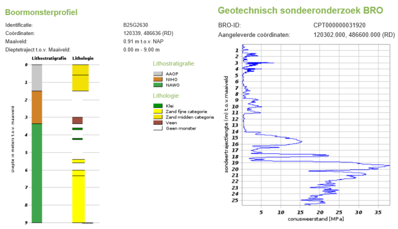


Figure 54: Foundation bridge 170




(a) Sample (b) CPT

Figure 55: Soil conditions bridge 170

the foundation.

On the west side, the bridge foundation starts with a row of piles at the side of the water. This row of piles is on the front of the abutment and is perpendicular to the bridge span. perpendicular to this row and in the direction of the bridge span, 16 rows of piles are placed, of which 12 are directly under the bridge abutment. Of the twelve rows, the middle 6 exist out of five piles, while the next row exists out of six piles and the final 2 outside rows exists out 7 piles. The thickness of the abutment is equal to 1.43 metres.

At the east side of the bridge, the abutment foundation rows are not placed perpendicular to the front of the abutment, but are placed in the direction of the bridge span.At the water side there is not a first row of piles, like on the other abutment. 17 rows of piles are placed under the abutment, of which ten are placed in the field of the bridge. The middle six rows exist out of 7 piles, while the piles on the outside exist out of 8 piles. The abutment thickness is equal to 1.76 metres. Three pieces of shear wood are used to fixate the flooring, which is 8 centimetres thick. (Gemeente Amsterdam, 1911)

### Monitoring and testing

Except for a RAMSHEEP assessment done by Antea, no inspection have been done on the bridge. Deformation measurements are also not done over the years.

### Soil conditions

Only two sources were available for this bridge. The sample is very short and even contains limited information in the sample. The sample contains mostly sand layers, but also contains a half metre layer of peat. small clay layers are

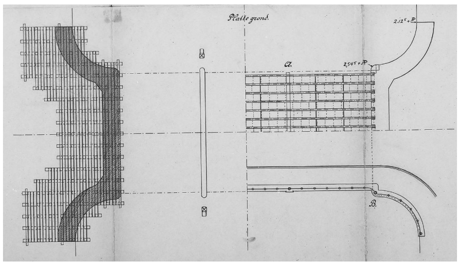


Figure 56: Foundation bridge 184 as built in 1895 (Gemeente Amsterdam, 1895b)

also present, but the sample is missing a large amount of information. The CPT shows a lot more information, indeed containing sand layers around 9 metres. The CPT also shows a large layer of sand from 14 to 16 meters. from 19 meter onwards, a thick layer of sand is present.

## Bridge 184

Bridge 184 lies in the Jacob van Lennepstraat and crosses the Bilderdijkgracht. The bridge was built in 1895. The bridge has room for 2 cars and allows normal traffic. Furthermore, the bridge has large sidewalks.

### Geometry and built up

The length of the bridge is around 22 metres and the width is 10.18 metres. The bridge is supported on 2 abutments and two pillars. the distance between the pillars is 6.79 metres, and the distance between the pillars and the abutments is 6.895 metres on both sides. (Gemeente Amsterdam, 1895b)

### Detailed overview of foundation and abutment

According to the drawings from 1895, 8 piles are placed under each of the pillars. The piles under the pillars look different than the piles under the abutment, which could suggest that the piles under the pillars are oak wood. The abutment is supported by 11 rows of piles, of which the five middle rows contain 6 piles and the outer ones contain extra piles, going up to 9 piles under a ground beam.(Gemeente Amsterdam, 1895b)

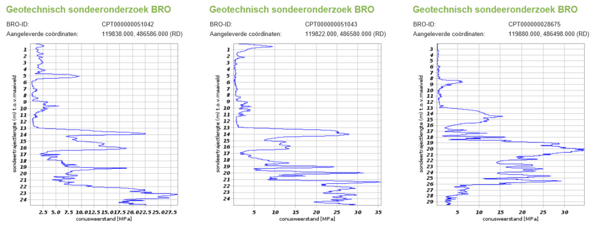
According to drawings from drawings from 1992, the wooden foundation under the pillars is replaced with a concrete foundation. It is to be expected that the wooden foundation stayed the same.

### Monitoring and testing

Except for a visual inspection of Antea in 2016, no other testing or monitoring has been done on bridge 184.

### Soil conditions

For the soil conditions in the surrounding area, only CPTs were found. These three CPTs however do show a similar soil condition pattern. Around 13 to 17




(a) CPT 1 (b) CPT 2 (c) CPT 3

Figure 57: Soil conditions bridge 184

metres below ground level, a sand layer is visible. Above the 13 metres, multiple sand layers can be distinguished, but these layers are at different heights in the different CPTs. Further down from the 13 metres, a large sand layer can be distinguished in figure 57. However, the other two CPTs show a different pattern, with a sudden decrease of cone resistance directly after the sand layer. This slowly increases in multiple steps, reaching a 27.5 MPa resistance around 22 meters below ground level.

## Bridge 295

The bridge was built over the Krombootsloot and lies in the Rechtboomsloot. The first bridge on the location of bridge 295 has been built in 1867, which has been renewed in 1913. The bridge has one lane, which can be used by normal traffic. No sidewalks are present on the bridge, which means that people have to walk on the road.

### Geometry and built up

The bridge is about 7 metres long and 4,75 metres wide (Gemeente Amsterdam, 1913b). The bridge is supported on two abutments.

### Detail of the foundation and abutment

According to Gemeente Amsterdam (1913a), the foundation under both of the abutments is similar, as can be seen in figure 58.The bridge abutment foundation is connected to the foundation of the quay wall, which makes the foundation more complex. The foundation under the abutments alternates with longer ground beams supported by 4 piles and shorter beams supported by 3 piles. The thickness of the abutment is equal to 1.03 metres on the top, but 1.30 at the bottom of the abutment, as is visible in figure.

### Monitoring and testing

Except for a RAMSHEEP assessment done by Antea, no inspection have been done on the bridge. Deformation measurements are also not done over the years.

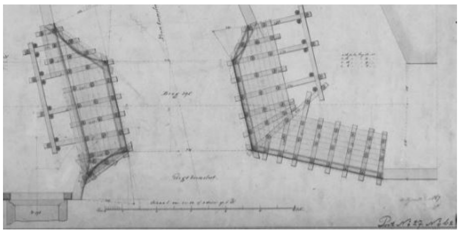


Figure 58: Foundation bridge 295 (Gemeente Amsterdam, 1913a)

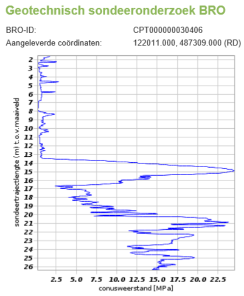



Figure 59: CPT from surrounding area bridge 295

### Soil conditions

For bridge 295, only one CPT is considered. This CPT however, very clearly shows little shaft resistance. This is however not necesary because of the very large cone resistance at the sand layer from 14 to 17 meters. the cone resistance reaches values of up to 25 MPa. Downwards from the sandlayer, more sand layers are present, however with much lower cone resistances.

## Bridge 315

Bridge 315 lies in the Galgenstraat and crosses the Bickersgracht. The first bridge has been built in 1881, which has been replaced by the current bridge in 1923. The bridge allows normal traffic, but is not able to let two cars pass on the road at the same time.

### Geometry and build up

Bridge 315 is supported on two abutments and two pillars, which are placed 12 metres apart. The distance from pillar to abutment is equal to 9.37 metres. In total, the bridge is 31.70 metres.

### Detailed overview of foundation and abutments

On the side of the Prinseneiland, the foundation exists out of a 9 by five grid (Gemeente Amsterdam, 1908). The ground beams are going over the 5 piles, which form a row of piles. Most beautiful foundation of all. On one side, at the Grote Bickerstraat, the bridge is composed in a 4 by twelve grid, with ground beams over the 4 piles (Gemeente Amsterdam, 1898).

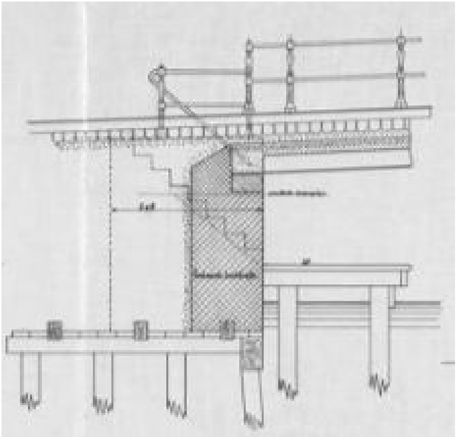
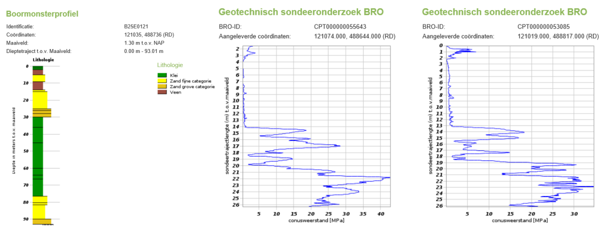


Figure 60: Foundation bridge 315 (Gemeente Amsterdam, 1923a)




(a) Sample (b) CPT 1 (c) CPT 2

Figure 61: Soil conditions bridge 315

The pillars are 48 centimetres thick and are both built on a more complex foundation. The piles under the pillars are vertically placed when considering from the side, put are all sloped when looking in the bridge plane. The slope of the piles is not defined, but increases when moving away from the centre. The piles under the pillars are of special tropical hard wood called jarrah. the piles that are used have a length of 17 metres and are 35 centimetres thick and 35 centimetres wide.(Gemeente Amsterdam, 1923b).

### Monitoring and testing

Except for a RAMSHEEP assessment done by Antea, no inspection have been done on the bridge. Deformation measurements are also not done over the years.

### Soil conditions

From the surrounding area of bridge 315, both a sample and two CPTs were available. The sample shows a layer of sand from 5 to 8 metres, surrounded by layers of peat. This layer of sand can however not be distinguished from both of the CPTs. The sample also shows a large layer of sand from 15 to 25 meters. This can also be seen in the CPTs, which shows a increase of the cone resistance from around 13 meters. Throughout the sand layer form the sample, the CPTs do show sudden decreasing in cone resistance, which could indicate small sand layers.

# B Soil conditions bridge 137

This appendix contains the enlarged versions of the cone penetration tests from bridge 137, which were shown smaller in figure 16.

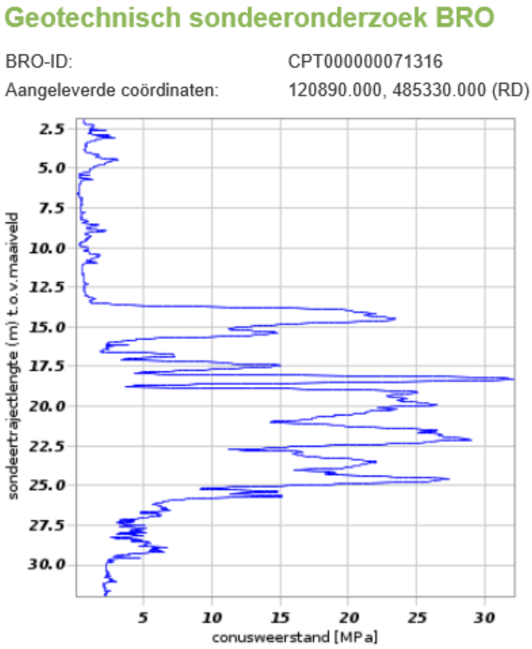



Figure 62: Cone penetration test 1 (DINOloket, 2012b)

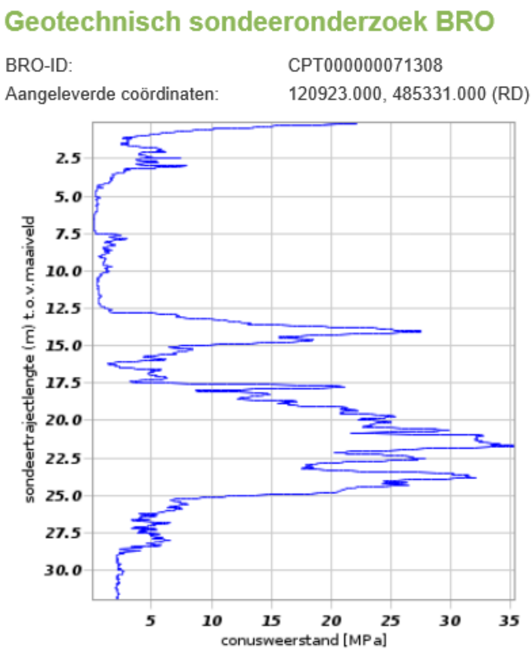



Figure 63: Cone penetration test 2 (DINOloket, 2012a)

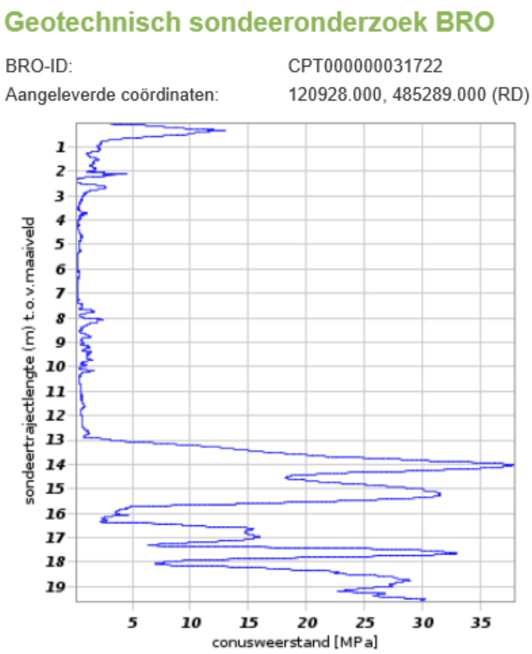



Figure 64: Cone penetration test 3 (DINOloket, 1990)

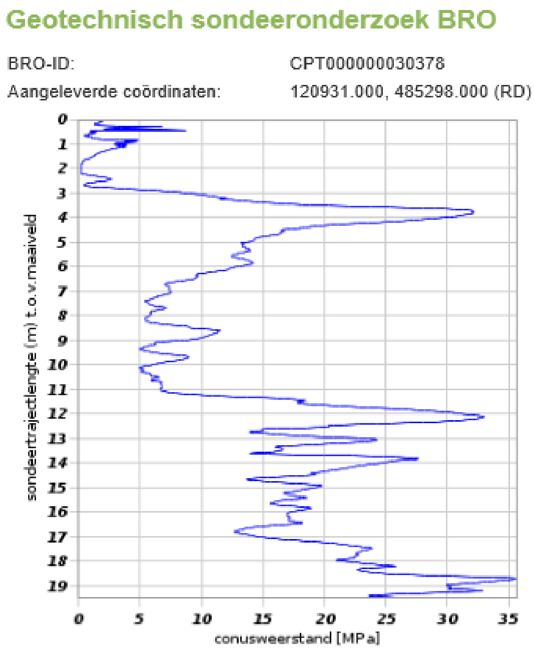



Figure 65: Cone penetration test 4 (DINOloket, 2002)