Black Rock Harbour Saba

Reference Design

Public entity Saba

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Project Leader	ir. A. van den Berg
Project Director	ing. R.W. Bos
Author(s)	ir. I.J.M. Schrijver dr.ir. E.L. Jorissen
Checked by	ir. D.J. Jaspers Focks
Approved by	ir. A. van den Berg
Paraph	

Address Witteveen+Bos Raadgevende ingenieurs B.V.
Leeuwenbrug 8
P.O. Box 233
7400 AE Deventer
The Netherlands
+31 570 69 79 11
www.witteveenbos.com
CoC 38020751

TABLE OF CONTENTS

APPENDICES

1

INTRODUCTION

1.1 Project background and description

The Public Entity of Saba has plans to develop an overall hurricane and seismic-resistant harbour on the southern side of Saba. At an earlier stage, the Fort Bay Harbour (figure 1-1) was considered to be developed into a future proof sea port [ref. 1]. However, it was found that the hurricane wave conditions at this location are extremely severe, which increases the probability that damage would continue to occur during future hurricanes [ref. 2]. This was the reason, amongst other considerations, that a new site was selected for the development of the harbour. This site is referred to as the Black Rocks area (figure 1-1), where hurricane wave conditions are less severe due to shallower depths and a lower gradient of the sea bed slope [ref. 3].

Geotechnical investigations were carried out at Black Rocks by Geotron, resulting in a factual report [ref. 4]. The follow-up interpretative report from Witteveen+Bos translates the results of the geotechnical investigations into geotechnical design parameters [ref. 5]. Based on these results, a first qualitative estimate of the susceptibility of the subsoil for liquefaction and drivability in the soils were additionally provided. Presently, Witteveen+Bos is contracted to develop a reference design for the new harbour lay-out at Black Rocks. The detailed design will be prepared by the selected Contractor.

Figure 1-1 Project locations on the southern side of Saba: current harbour at Fort Bay and future harbour at Black Rocks

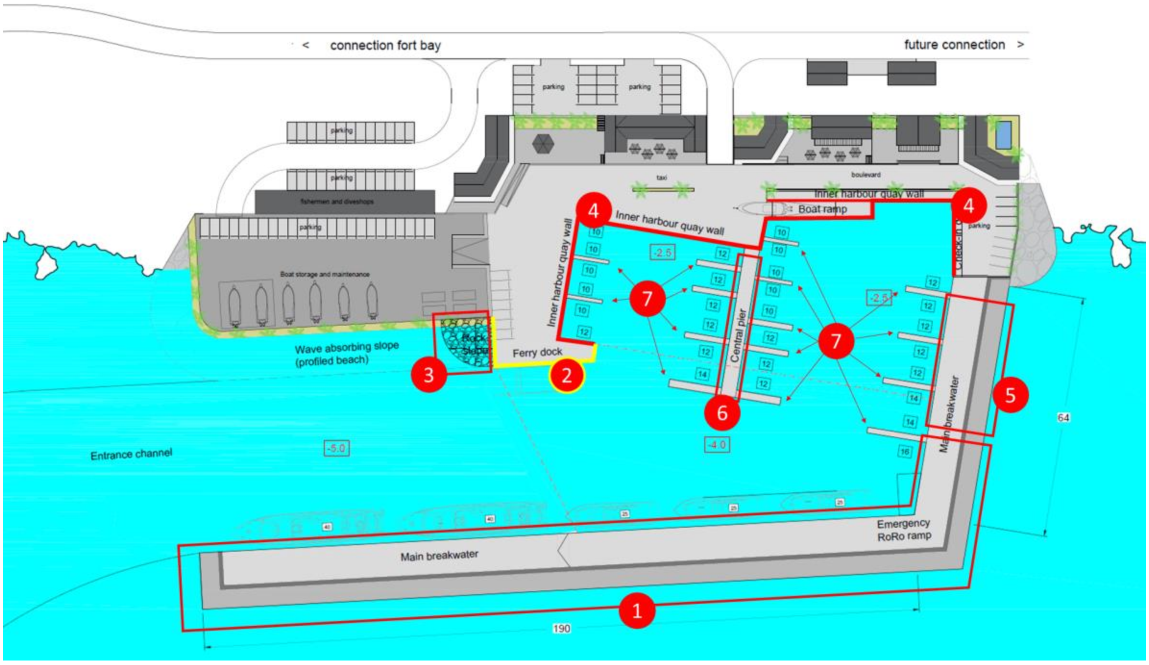
1.2 Design report

1.2.1 Objective

The objective of this report is to provide a reference design for the main structures of the new harbour at Black Rocks, which is divided into 6 sections presented in figure 1-2.

1 Main Breakwater.
2 Ferry quay wall.
3 Rock slope.
4 Inner harbour wall.
5 Main breakwater transition to coast.
6 Central pier
7 7 Finger piers.

Figure 1-2 Lay-out of the new harbour at Black Rock harbour with locations of the different sections



1.2.2 Scope

Based on the geotechnical investigations previously carried out at the Black Rock harbour location by Geotron [ref. 4] and translated into geotechnical design parameters by Witteveen+Bos [ref. 5], a reference design will be provided of the 7 sections of the new harbour lay-out. The reference design will include geotechnical calculations and verifications, as well as recommendations for the definitive design based on the geotechnical reliability of the different structures as well as preliminary cost estimations. Some elements of the design are free for the Contractor to design. This applies, amongst others, to the inner harbour quay walls and solutions to generate sufficient passive resistance for the main breakwater caissons

1.2.3 Outline

Chapter 2 begins with a summary of the geotechnical starting points and a design description of the future Black Rock harbour. Chapter 3 presents the results of the geotechnical calculations for the different sections. Chapter 4 summarizes the major outcomes of this design report. Finally, Chapter 5 provides the references used in this report.

2

STARTING POINTS AND DESIGN DESCRIPTION

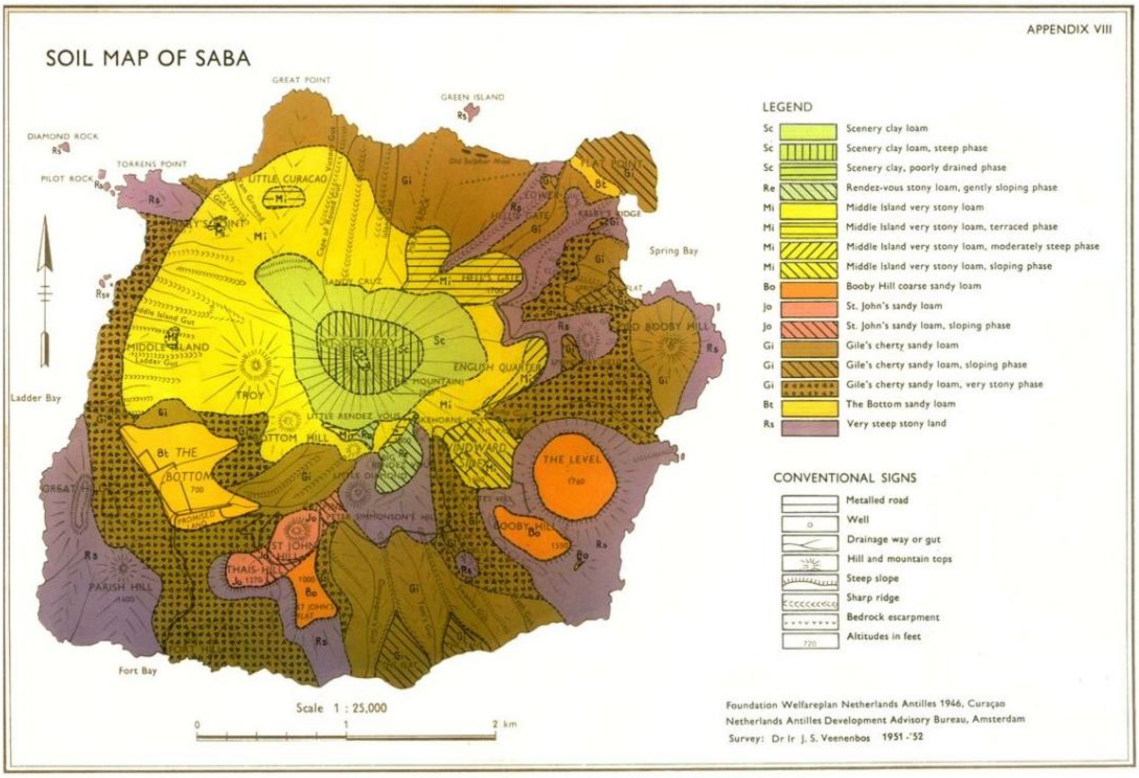
2.1 Geotechnical starting points

2.1.1 Geological settings

The island of Saba is part of the Caribbean volcanic arc that also includes the islands of St. Eustatius, Montserrat, Guadeloupe, Martinique and Grenada. The land stratigraphy of Saba consists entirely of volcanic deposits, which can be subdivided in old volcanic pipes (lava domes), lava flows and pyroclastic deposits. The domes are made of highly resistant rocks and can be recognised by the steep conical hills on the island.

The site of Black Rock harbour is located in a relatively flat area formed by a small alluvial fan as shown in figure 2-1. Sediments deposited in this area have a fluviatile and scree (slope debris) origin [ref. 7], i.e. volcanic boulders, cobbles, gravels and sands, which are prograding into coral limestone beds located further offshore [ref. 4].

Figure 2-1 Soil map of Saba with the localisation of the new Black Rock harbour [ref. 7]



2.1.2 Soil survey and geotechnical parameters

Soil investigations were carried out by Geotron at the site of Black Rock harbour [ref. 4]. The survey consisted of 17 boreholes with locations given in figure 2-2 and figure 2-3 and transect sections in appendix I, in situ DPT's at several boreholes and lab testing (such as sieving and UCS-testing). The interpretative report from Witteveen+Bos [ref. 5] translated the results of the geotechnical investigations for four main geotechnical materials:

- Andesite boulders: Medium to very large size boulders (size larger than 630 mm) consisting of reddish brown to grey andesite (red colour in the geotechnical profiles).
- Gravel and cobbles: Medium to small size boulders (size of 200 to 630 mm), cobbles and gravels consisting of reddish brown to grey andesite (green colour in the geotechnical profiles).
- Sand medium dense to dense: Dark (reddish) brown and grey dense sand with small gravels and/or white coral particles (yellow colour in the geotechnical profiles).
- Coral limestone: Greyish and off-white cobbles and pebbles consisting of coral limestone (blue colour in the geotechnical profiles).

Figure 2-2 Locations of the boreholes plotted on Google Earh (boreholes B01-B06b are located onshore and boreholes B7-B17 are located offshore)

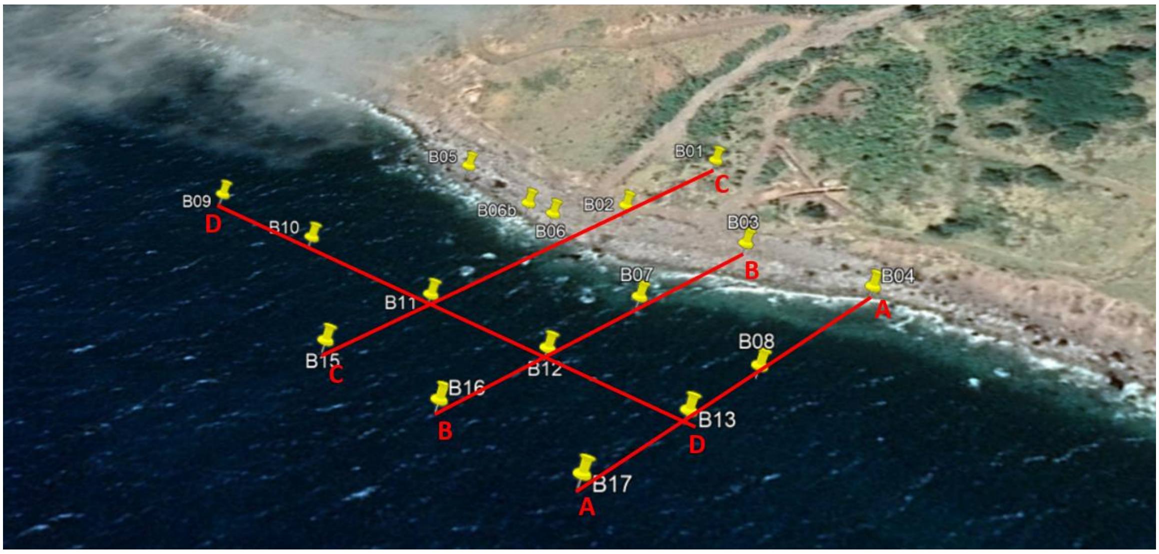
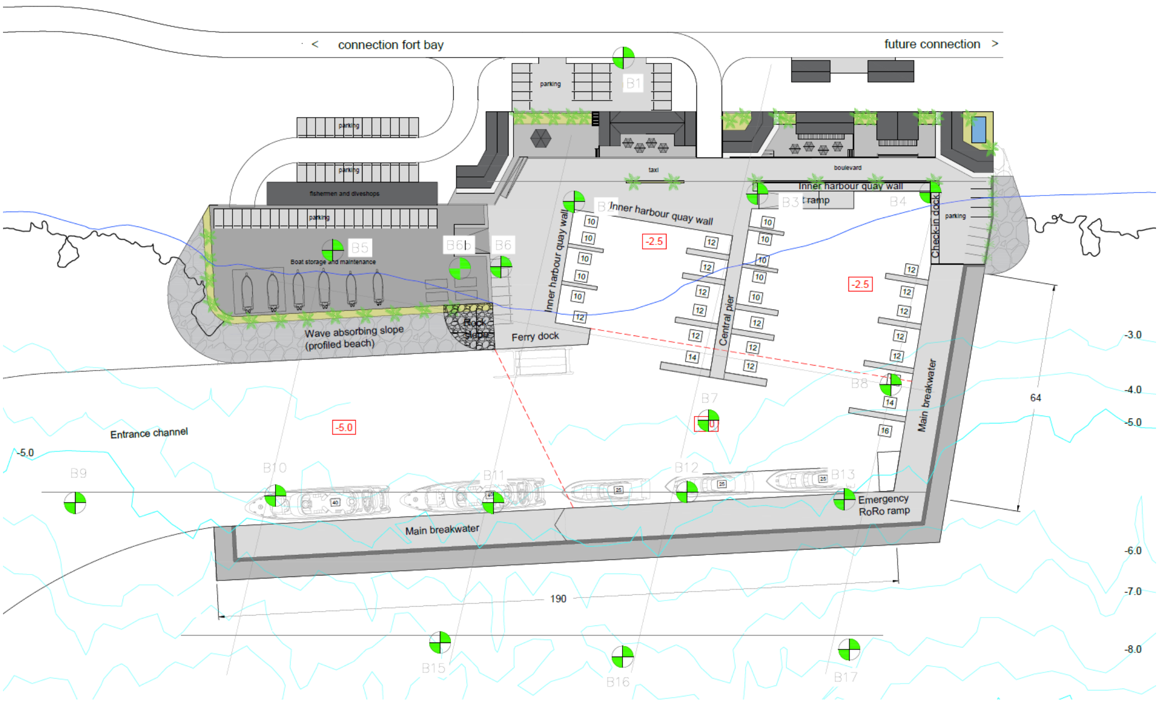


Figure 2-3 Locations of the boreholes plotted on the harbour layout. Existing depth contours and coastline are also shown



Geotechnical design parameters for these main materials, as well as the parameters for the supplemented materials are presented in table 2.1. For the design of the main breakwater in Plaxis some additional parameters are required. These Plaxis parameters are presented in table 2.2.

Table 2.1 Geotechnical design parameters

Soil type	γ/γ_{sat} [kN/m³]	c' [kPa]	φ' [°]	δ [°]	Unconfined Compression Strength (UCS)* [MPa]	Indirect Tensile Strength (BTS)** [MPa]
Andesite boulders	18/21	0	40,0	20,0	36,0	5,4
Gravel and cobbles	18/20	0	37,5	18,8	-	-
Sand dense	18/20	0	35,0	23,3	-	-
Sand medium dense***	18/20	0	33,5	22,3	-	-
Coral limestone	-	-	-	-	14,6	3,8
Supplemented material	18/21	0	37,5	25,0	-	-

- γ/γ_{sat} = Natural / saturated volumetric weight of the material
- c' = Effective cohesion
- φ' = Effective friction angle
- δ = Wall friction angle

*) Derived from unconfined compression tests.

**) Derived from Brazilian tensile strength tests.

***) According to the performed liquefaction analysis, there is a significant risk of liquefaction during the occurrence of earthquakes. Therefore, measures need to be taken to avoid liquefaction. The effect of these measures (e.g., application of stone columns or soil compaction) are taken into account by increasing the angle of internal friction of the medium dense sand layers to 33,5°.

Table 2.2 Additional Plaxis parameters for the design of the main breakwater

Soil type	ψ	$E_{50;ref}$ [MPa]	$E_{oed;ref}$ [MPa]	$E_{ur;ref}$ [MPa]	R_{inter}
Andesite boulders	-	-	-	-	-
Gravel and cobbles	1	42	42	126	0,8
Sand dense	1	39	39	117	0,8
Sand medium dense	1	33	33	99	0,8
Coral limestone	-	-	-	-	-
Supplemented material	2	40	40	120	0,8

2.1.3 Loads

Surface loads

All structures shall be designed to sustain a characteristic uniform surface load on the entire quay between 5 to 20 kN/m^2 depending on the construction. During earthquake conditions, a combination factor of 0.3 may be applied on the surface loads since no permanent loads will be present at the quays. Surface loads associated to each structure element are given in table 2.3.

Bollard and cleat loads

All structures shall be designed to sustain concentrated horizontal loads from bollards and cleats. Bollard and cleat loads associated to each structure element are given in table 2.3. As the loads often take effect slightly above the deck (at top of bollard/cleat), an additional moment of 20 kNm/m is added in the calculations. During earthquake conditions, these loads do not have to be taken into consideration since it is unlikely that storm conditions (maximum) bollard forces coincide with the occurrence of an earthquake.

Table 2.3 Surface and bollard and cleat loads applied on the different structure elements

Sections	Surface loads [kN/m^2]	Surface loads * 0,3 [kN/m^2]	Bollards and cleats [-]	Bollard and cleat loads [kN/m]
1: Main breakwater				
- mega yachts	20,0	6,0	20 T bollards each 15 m with a 5 T cleat in between	40,00*
- big yachts	20,0	6,0	10 T bollards each 15 m with a 5 T cleat in between	20,00*
2: Ferry quay wall	20,0	6,0	20 T bollards each 15 m with a 5 T cleat in between	26,67
3: Rock slope	15,0	4,5	-	-
4: Inner harbour wall	15,0	4,5	2 * 5 T bollards each 5 m	20,00
5: Cofferdam extension	20,0	6,0	10 T bollards each 15 m with a 5 T cleat in between	13,33
6: Central pier	10,0	3,0	2 * 5 T bollards each 5 m	20,00
7: Finger piers	5,0	3,0	2,5 T cleats each 6 m	4,17

*) For the main breakwater it is assumed that the bollard loads are evenly distributed over a width of 5,0 m.

Wave loads

Wave loads on the main breakwater (section 1) and the transition to the coast (section 5) during hurricane conditions

The main breakwater is primarily subjected to the wave loads acting outside the harbour. The wave loads during hurricane conditions are normative for the design of the breakwater. The main breakwater is designed for hurricane 5 water level and wave conditions. The loads on the vertical breakwater were determined by physical model tests in the wave flume of the department of Civil Engineering (Afdeling Weg- en Waterbouwkunde) at Ghent University and were reported in [ref. 14].

Force measurement tests were conducted on a vertical wall breakwater with a crest level of 5 m MSL and a recurved wall breakwater (vertical wall + parapet) with a crest level of 3,9 m MSL. In line with the recommendations by Goda [ref. 15] the 1/250 highest impact force is used for the design.

The design force for the design conditions (hurricane 5, offshore conditions Hs = 11.9 m, Tp = 13.15; water level = 1.3 m +MSL) was based on multiple samples resulting in a $F_{1/250}$ of 3785 kN/m as reported in figure 67 [ref. 14]. Based on the recommendations by UGent, the following distributions were proposed (based on discussions with UGent and PES) for the vertical pressure distribution:

- For the main breakwater as a cofferdam including a parapet and a retaining height of 9,9 m (bed level -6.0m MSL, parapet level +3.9m MSL).
  F_{hor} (z) = $80e^{0.265z}$ resulting in a total horizontal force of 3859 kN/m.
- For the main breakwater as a vertical caisson with a retaining height of 11,0 m (bed level -6.0m MSL, crest level +5.0m MSL).
  F_{hor} (z) = $80e^{0.245z}$ resulting in a total horizontal force of 4508 kN/m.
- z = is the vertical distance from seabed up, in meters, F_{hor} is the horizontal pressure in kN/m^2.

The applied wave force distribution is assumed to be linear in the Plaxis model (triangular shaped, with its maximum at the top side). The characteristic wave forces acting on the seaside wall of the main breakwater (section 1) are presented in table 2.4. The value for the maximum load at the top of the wall is determined such that the energy of the wave applied is equal to the maximum wave energy found from the tests. For the caisson structure the characteristic wave force acting on the crest of the structure, which is designed at an angle in order to reduce the wave impact, has been translated to a wave force perpendicular to the crest in accordance with the equations VI-5-169 from the Coastal Engineering Manual [ref. 17].

As a result of the decreasing water depth towards the coast and a larger angle of incidence, the wave loads acting on the breakwater transition to the coast (section 5) are smaller than the wave loads acting on the main breakwater. A representative section at section 5 is taken at a bed level -5 m MSL were the wave loads acting on the breakwater are 35% smaller. The characteristic wave forces acting on the seaside wall of the main breakwater transition are presented in table 2.5.

Table 2.4 Horizontal wave forces on seaside wall of the main breakwater (section 1, -6.0m MSL bed level)

Alternative	Retaining height [m]	Wave peak	Max. horizontal force [kN/m]	Linear load top [kN/m/m]	Linear load bottom [kN/m/m]
1. Cofferdam (crest +3.9m MSL)	9.9	Highest peak	3,859	780	0
2. Caisson (crest +5.0m MSL)	11.0	Highest peak	4,508	820*	0

*) In the design calculation the force at the crest has been translated to a wave force perpendicular to the crest.

Table 2.5 Horizontal wave forces on seaside wall of the main breakwater transition to the coast (section 5, -5.0m MSL bed level)

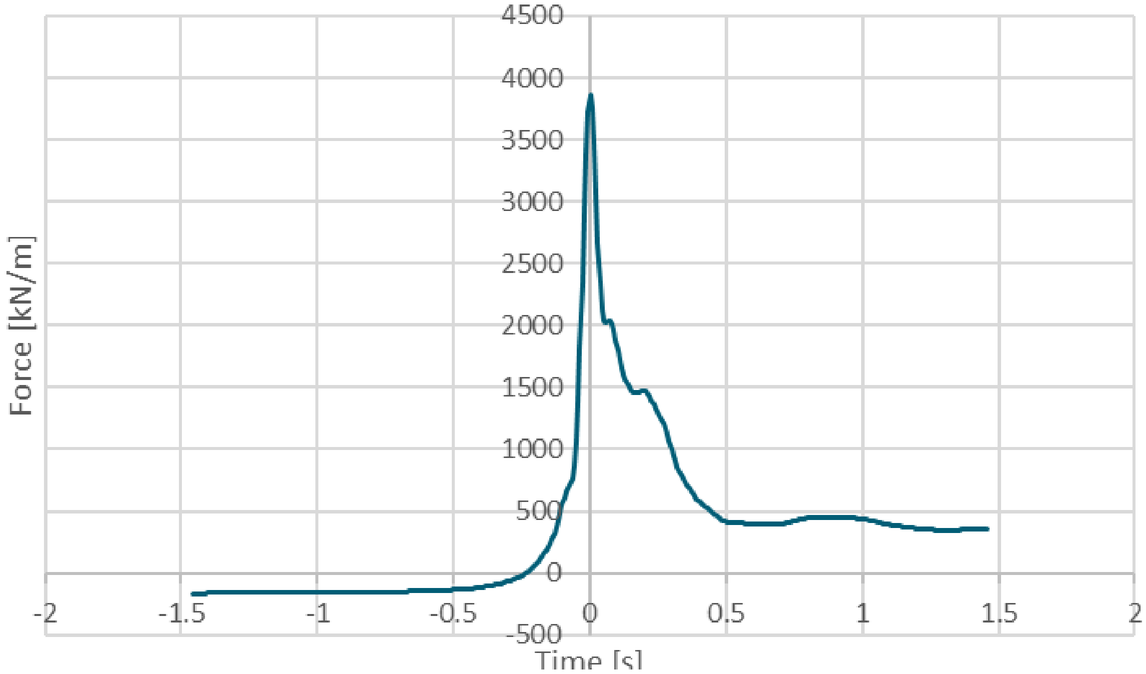
Alternative		Retaining height [m]	Wave peak	Max. horizontal force [kN/m]	Linear load top [kN/m/m]	Linear load bottom [kN/m/m]
1	Cofferdam (crest +3.9m MSL)	8.9	Highest peak	2,508	507	0
2	Caisson (crest +5.0m MSL)	10.0	Highest peak	2,930	533*	0

*) In the design calculation the force at the crest has been translated to a wave force perpendicular to the crest.

The cofferdam design includes a parapet at the seaside of the structure. During the peak of the wave this parapet is additionally subjected to a characteristic vertical wave load. For the main breakwater (section 1) this vertical wave load equals 1020 kN/m. For the main breakwater transition to the coast (section 5) a reduction of this wave load of 35 % considered, resulting in a vertical wave load on the parapet of 663 kN/m.

The time-series of measured impact forces were recorded with 1.000 Hz sampling frequency in all tests. The average of the time-series as given in below figure was suggested by UGent to be applied in the present study.

Figure 2-4 Average time-series of force measurements



Wave loads on the other structures (sections 2, 4, 6 and 7)

Sections 2, 4, 6 and 7 shall be designed to sustain horizontal (and vertical) wave loads occurring inside the harbour during normal and hurricane conditions. Section 6 (Central pier) and section 7 (Finger piers) are particularly sensitive to wave loads since these structures are not supported by soil. On the other hand, applying these loads on section 2 (Ferry quay wall) and section 4 (Inner harbour wall) is not required, as these loads play in favour of the stability of these structures.

2.1.4 Design approach and partial factors

The reliability class for the structures is Reliability Class 2 (RC2) in accordance with the Eurocode. The partial factors for geotechnical parameters for overall stability and design of the retaining walls and breakwater structures, as well as the partial factors for loads are presented in table 2.6.

Table 2.6 Partial factors for geotechnical parameters and loads

Partial factors		Symbol	Factor			
			For overall stability	For retaining wall	For sheet pile wall	For loads
Geotechnical parameters	Angle of internal friction	$\gamma_{\varphi'}$	1.2	1.25	1.175	-
	Effective cohesion	$\gamma_{c'}$	1.5	1.5	1.25	-
	Undrained shear stress	γ_{cu}	1.5	1.5	1.6	-
	Prism compressive strength	γ_{qu}	1.5	-	1.6	-
	Volumetric weight	γ_{γ}	1.1	1.1	1.0	-
Loads	Variable load	γ_Q	-	-	1.1	1.5
	Permanent load	γ_Q	-	-	1.0	1.35

2.1.5 Seismic aspects

Seismic loads

All permanent structures shall be designed to resist seismic events, in accordance with Eurocode 1998. All permanent structures are considered as Eurocode importance class II structures (consequence class CC2), with an importance class factor of γ_I =1.0 for the Ultimate Limit State (near collapse) verifications. According to PIANC [ref. 6], seismic L2 loads correspond to the Ultimate Limit State with a probability of exceedance of 10 % during the lifespan of the structure. Based on a Design Life of 50 years, seismic L2 loads with a Mean Return Period (MRP) of 475 years shall be taken into account for all permanent structures. This is equivalent to a peak acceleration of the bedrock a_{gR} of 0.275 g. The design ground acceleration a_g corresponding to the factor between the importance class factor and the peak acceleration of the bedrock and is therefore equal to 0.275 g.

Seismic soil pressure coefficients for seismic L2 loads are applied on the active and passive sides of retaining walls and breakwaters. These are calculated based on an in-house spreadsheet in accordance with the Mononobe-Okabe formulation for cohesionless soils following Eurocode 8 and the Japanese Technical Standard. Results for the different sections are presented in appendix V.

Seismic soil pressure coefficients for seismic L2 loads for the Inner harbour wall and the Central pier are applied on the structure as horizontal and vertical loads. These are calculated according to Eurocode EN 1998-5 based on the formula:

$$F_h = 0{,}5\alpha . S . W$$
$$F_v = \pm 0{,}5 . F_h$$

where:

α is the ratio of the design ground acceleration a_g to the acceleration of gravity g;
S are the soil factor equals to 1.15 based on the BoD [ref. 1]
W are the weight of the sliding mass equals to the factor between the gravity g and the mass m.

Liquefaction

Liquefaction constitutes an issue as sand layers were encountered at the project location. The amount of detailed soil investigation is limited, which makes it impossible to accurately determine the extent of the thickness and locations of liquefiable layers. Based on the available in-situ DPT-tests though, the resistance to liquefaction was determined (appendix II) in accordance with the method of Boulanger and Idriss [ref. 16]. The results show that in particular the less dense sand layers with a relatively low cone resistance (derived from the DPT-tests) are susceptible to liquefaction. For detailed design it is necessary to carry out additional soil investigations to establish the resistance to liquefaction of the subsoil in more detail in order to properly design any required mitigating measures against liquefaction.

In the design it is assumed that all layers will be resistant to liquefaction, since Eurocode 1998 does not allow the occurrence of liquefaction. As soon as a soil layer is found to be susceptible to liquefaction it needs to be improved. Therefore, no effect of liquefaction is taken into consideration in the reference design as it is assumed that soil improvement will be realised to prevent its occurrence. In the reference design it is assumed that the soil shall be improved within the first 5.0 m of the substrate under sections 1 and 5 (Main breakwater) and section 6 (Central pier). Any fill material behind the sheet pile walls and around the anchors present in section 1, 2 (Ferry quay wall) and section 4 (Inner harbour wall) shall also be improved.

2.1.6 Water levels

Water levels are defined based on long term measurements of water levels in Fort Bay Harbour [ref 18] and are presented in table 2.7 and are given relative to Saba Datum (SD). Saba datum is historically assumed to be equal to Mean Sea Level but based on the detailed measurements this appears to be approx. 8cm below MSL. Chart Datum (CD) is used for the representative low water, in order to take the wave trough into consideration.

According to the NEN9997-1 and conform RC2, water levels shall be adjusted for design calculations of retaining walls and breakwaters by:
- 0.05 m increase on the active side.
- 0.25 m decrease on the passive side.

Table 2.7 Water levels during frequent and extreme level conditions at the project location (nb. the values in the table may vary slightly (few cm's) as a result of ongoing updates of the tide harmonic assessments)

Conditions		Water levels [m +SD]
Frequent water level conditions	Chart Datum (CD)	-0.30
	Mean Low Water Spring (MLWS)	-0.09
	Mean Sea Level (MSL)	+0.08
	Mean High Water Spring (MHWS)	+0.24
	Highest Astronomical Tide (HAT)	+0.33
Extreme water level conditions	Design high water level	+1.30
	Design low water level	+0.30

2.2 Harbour design description

The new harbour lay-out at Black Rock harbour is divided into sections as presented in figure 1-2. Their principal characteristics are defined in table 2.8. A more complete description of their design is further explained in the following individual paragraphs.

Table 2.8 Principal characteristics of the different sections for the new Black Rock harbour

	Sections	Bed level [m +msl]	Design bed level for calculations [m +msl]	Margin [m]	Crest level [m +msl]	Retaining height [m]	Width [m]
1	1: Main breakwater (inside)						
	· mega yachts	- 5.0	-5.5	0.5	+1.5	7.0	t.b.d.
	· big yachts	-4.0	-4.5	0.5	+1.2	5.7	t.b.d.
2	Ferry quay wall	-5.0	-6.0*	1.0	+1.2*	8.1*	-
3	Rock slope	-	-	-	-	-	-
4	Inner harbour wall	-2.5	-3.0	0.5	+1.2**	4.5**	-
5	Main breakwater transition	variable	variable	variable	+1.5	variable	t.b.d.
6	Central pier	.-2.5 to -4.0	.-3.0 to -4.5	0.5	+1.2	.4.2 to 5.7	4.0
7	Finger piers	-2.5	-3.0	0.5	+1.0	4.0	2.0

* In the initial design, the Ferry Quay wall was extended to the west as a cargo (RoRo) quay, with a bed level of -5.0 m MSL and a quay level of +2.1 m MSL. The bed level in front of the ferry wall is currently limited to -5.0 m MSL. The design calculations are based on the original retaining height of 8.1 m.

** In the initial design, the inner harbour wall was designed at +1.5m quay level. This is recently reduced to +1.2 m. The design calculations are based on the original retaining height of 4.5 m. The inner harbour quay wall in the east basin now also includes a lower walkway at +0.75m level.

2.2.1 Section 1: Main Breakwater

Section 1 is located on the southern side of the new harbour and is intended to berth large transient cruisers and mega yachts. This section is part of the main breakwater for which two alternative structures have been considered, each build parallel to the east-west oriented coastline. For the first alternative the main breakwater is designed as a sand-filled cofferdam, the second alternative is based on a breakwater consisting of caissons. The main dimensions of the main breakwater are presented in table 2.9.

Table 2.9 Main breakwater cross section dimensions

Dimension	Alternative 1: Cofferdam	Alternative 2: Caisson
Topside elevation deck	MSL +1.5 m (west) / MSL +1.2 m (east)	MSL +1.5 m (west) / MSL +1.2 m (east)
Topside elevation seaside wall	MSL +3.9 m	MSL +5.0 m
Seabed elevations	MSL -6.0 m (seaside) MSL -5.5 m (harbour side, design)	MSL -6.0 m (seaside); MSL -5.5 m (harbour side, design)
Retaining height	9.9 m (seaside); 7.0 m (harbour side)	11.0 m (seaside); 7.0 m (harbour side)
Width structure	15.0 m	15.0 m. with a base width of 18.0 m
Front facing	Vertical wall with parapet	Vertical below MSL + 0.0 m, slope 1:1 above MSL + 0.0 m

Soil profile

At the location of the main breakwater (section 1) the soil profile consists of Andesite boulders or coral limestone on the seabed overlying layers of medium dense to dense sand and gravels and cobbles. The underlying layers possibly contain boulders as well. The soil profile applied for the geotechnical calculations of the main breakwater is based on the nearest normative borehole B13 for this section and the other boreholes in the corresponding transect section A (see figure 2-2, figure 2-3 and appendix I). It is assumed that the overlying layer of boulders will be removed before the main breakwater will be constructed and that provisions will be made to break down rocks at larger depths when these are encountered during pile driving (if a cofferdam design is chosen). The applied soil profile for the geotechnical calculations is given in table 2.10.

Table 2.10 Soil profile near the main breakwater based on transect section A (B17, B13 and B08)

Top of soil layer [m +MSL]				Lithology	Colour in geotechnical profile
x = -45 m	x = 0 m	x = 15 m	x = 50 m		
-10.0	-6.0	-5.5	-5.5	Sand with (coral) gravels, pebbles and cobbles, brownish to grey colour	Yellow
-12.0	-8.0	-8.0	-6.0	Gravel with pebbles and boulders, multi coloured	Green, Red
-12.0	-9.0	-9.0	-7.5	Sand with gravels, pebbles and cobbles, brownish to greyish colour	Yellow

Loads

Normal conditions consider the horizontal bollard loads (40 kN/m) and associated moments (20 kNm/m), as well as the vertical surface loads (20 kN/m^2). During earthquake conditions, a combination factor of 0.3 is applied on the surface load since no permanent loads will be present at the breakwater, resulting in a surface load of 6 kN/m^2. During these conditions, bollard loads are not taken into consideration since it is unlikely that storm conditions (maximum bollard forces) coincide with the occurrence of an earthquake. Hurricane conditions only consider the wave loads since the harbour will be empty during hurricanes.

Wave load application

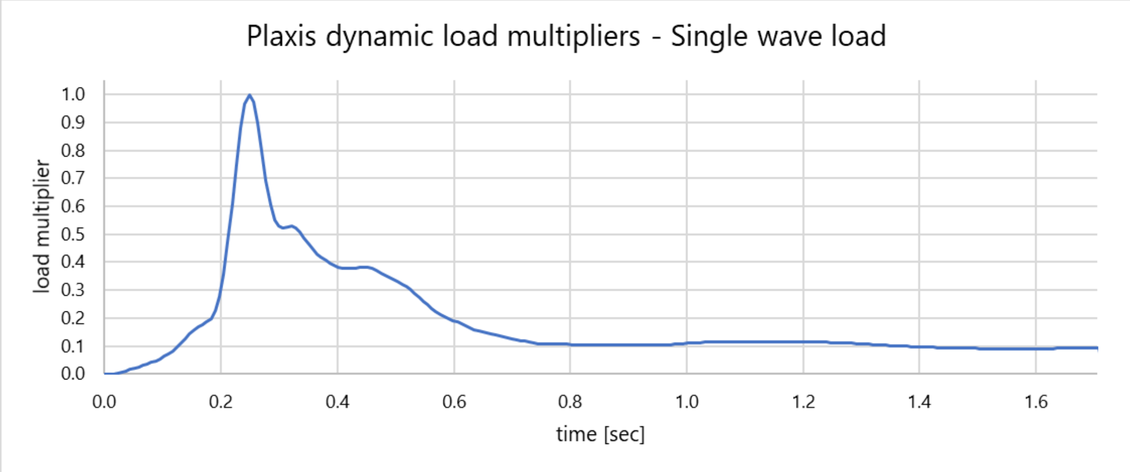
Static wave load

The static wave load application was performed as a first step using the highest peak wave presented in table 2.4. Results are not provided as the calculations resulted in unrealistic breakwater dimensions. After these results the analyses continued with modelling of the dynamic wave loads instead of a static wave load.

Dynamic single wave load

The value of the dynamic wave load is applied using load multipliers (factors) that vary with time. The wave load data has a timestep of 0.00728 sec. The load multiplier per step is determined based on the fraction of the largest load being modelled in the timeseries. The dynamic single wave load calculation includes the highest peak wave load. The load multiplier for this calculation is shown in figure 2-5.

Figure 2-5 Dynamic single wave multipliers



Dynamic multiple wave load
Three consecutives' waves were applied to the structure to determine how the compounding displacement and bending moments after several wave impacts affect the structure at the end of the storm. For this it is assumed that the duration of each wave equals 5 seconds as after this duration of time steady residual displacements are found and the accuracy of the results will not be sacrificed.

Calculation method
The geotechnical calculations for the main breakwater are performed using the software Plaxis 2D (version 20.04.00.790, build 790). For the calculations regarding the seismic conditions, the pseudo-static model conditions are used in combination with a horizontal acceleration equal to the equivalent seismic coefficient k_e determined in appendix V. For the calculations regarding the hurricane conditions a dynamic calculation in Plaxis 2D is used given the short time duration of the peaks of the waves and the repetitive loading. A static calculation is considered too conservative since this will have to be modelled by continuous wave forces equal to the peak load. The Hardening Soil model with small-strain stiffness is used to model the different soil layers. Overall stability is confirmed implicitly, and structural forces and displacements are used to verify the strength and stability of the structure.

2.2.2 Section 2: Ferry quay wall

Section 2 is located on the western side of the new harbour. Initially this was intended as a RoRo quay, with a depth of -5.0 m MSL and a quay level of +2.1 m MSL, resulting in a retaining height for design of 8.1 m (including 1.0 m margin). The RoRo quay is no longer part of the design, but a smaller Ferry quay wall remains at the transition between the western dock and the revetment. This Ferry quay wall has a lower elevation (+1.2 m) and has a dredging depth of -4.0m MSL. However, in view of potential future upgrades, it is decided by the Client to maintain a design bed level of -5.0 m MSL, allowing future dredging to that level. In this design report, the quay wall is designed as a sheetpile wall based on the original retaining height of 8.1 m. Additional stability is provided to the retaining wall by anchors fixed to a rear anchor-wall. The dimensions and properties of the sheet-pile, anchor and anchor-wall required to verify the geotechnical stability are determined in paragraph 3.2.

Soil profile
The soil profile applied for the geotechnical calculations of the Ferry quay wall is based on the nearest normative borehole B06 for this section (see appendix I) and is given in table 2.11.

Table 2.11 Soil profile near the Ferry quay wall based on borehole B06

Top layer [m +MSL]	Bottom layer [m +MSL]	Lithology	Colour in geotechnical profile
+1.5	+1.0	boulders, reddish colour	red
+1.0	-1.0	sand with boulders, cobbles and gravels, reddish to greyish colour	green
-1.0	-2.5	boulders, reddish colour	red
-2.5	-3.0	sand with gravels	yellow

Seismic soil pressure coefficients

Seismic soil pressure coefficients for seismic L2 loads need to be applied on the active and passive sides. They are calculated for the nearest normative borehole B06 using an in-house spreadsheet, which results are presented in table 2.12. Detailed calculations can be found in appendix V.

Table 2.12 Seismic soil pressure coefficients for B06, according to seismic L2 loads

Soil type-]	Top layer [m +MSL]	Bottom layer [m +MSL]	Coefficient on active side	Coefficient on passive side	Coefficient on neutral side
Sand supplemented	+2.5	+1.5	0.25	7.62	0.39
Boulders	+1.5	+1.0	0.25	7.62	0.39
Sand with boulders cobbles and gravels	+1.0	+0.07*	0.27	6.14	0.43
Sand with boulders, cobbles and gravels	+0.07*	-1.0	0.29	6.04	0.43
Boulders	-1.0	-2.5	0.26	7.30	0.39
Sand with gravels	-2.5	-3.0	0.33	4.84	0.45

*) Transition level corresponding to MSL.

Loads

Normal conditions consider the horizontal bollard loads (26,67 kN/m^2) and associated moments (20 kNm/m), as well as the vertical surface loads (20 kN/m^2). Earthquake conditions consider the seismic loads, as well as 30 % of the vertical surface loads. No wave loads are included for this section, as these loads play in favour of the stability of the structure.

Calculation method

Pseudo-static geotechnical calculations of the Ferry quay wall are performed using the software D-Sheet Piling (version 19.2, build 3.26114).

2.2.3 Section 3: Rock slope

Section 3 is the transition from the sheet pile wall in section 4 to the adjacent coast with a rock slope. The bed level at this location is taken at -5,5 m MSL and the crest level at +3,3 m MSL. The design is based on stability of the rock at a 1:2 (v:h) slope in relation to governing wave loads. No geotechnical verifications were performed in the current design stage, as it is expected that the structure will be stable with the current geometry. Geotechnical verification to be performed in the next design stage.

2.2.4 Section 4: Inner harbour wall

Section 4 is located along sides of the basins. The quay wall users are small fishing boats, marine park vessels, local recreational boaters, charter dive vessels and transient boaters. The inner harbour quay wall is designed for a retaining height of 4.5 m.

Various outline designs have been considered for the inner harbour quay wall, ranging from gravity-based walls (L-walls, block walls) to (anchored) sheetpile walls. For various reasons an anchored sheetpile wall has been selected as reference design[1], but the balance may shift in favour of gravity-based walls depending on the Contractor construction method and preferences. Alternatives are allowed with the anticipated Design & Build (or Enigneer & Construct) contract, provided that the design starting points as formulated in this Design Document are maintained.

Soil profile

The soil profile applied for the geotechnical calculations of the inner harbour wall is based on the nearest normative borehole B03 for this section (see appendix I) and is given in table 2.13.

Table 2.13 Soil profile in the inner harbour wall based on borehole B03

Top layer [m +MSL]	Bottom layer [m +MSL]	Lithology	Colour in geotechnical profile
+2.0	+1.5	Boulders, reddish colour	red
+1.5	+1.0	Sand with boulders, cobbles and gravels, reddish to greyish colour	green
+1.0	+0.5	Boulders, reddish colour	red
+0.5	-0.5	Sand with boulders, cobbles and gravels, reddish to greyish colour	green
-0.5	-1.0	Boulders, reddish colour	red
-1.0	-2.0	Sand with boulders, cobbles and gravels, reddish to greyish colour	green
-2.0	-5.0	Sand with gravels	yellow

Seismic soil pressure coefficients

Seismic soil pressure coefficients for seismic L2 loads need to be applied on the active and passive sides. They are calculated for the nearest normative borehole B03 using an in-house spreadsheet, which results are presented in table 2.14. Detailed calculations can be found in appendix V.

[1] Arguments in favour of a sheetpile wall are related to considerations about the large size of prefab L-walls and the fact that piling is also required for the finger piers. On the other hand, with an in-situ (dry) construction method insitu cast L-walls or concrete block walls may be more logical. The risk of obstruction (rocks etc) during pile driving is also absent with gravity walls.

Table 2.14 Seismic soil pressure coefficients for B03, according to seismic L2 loads

Soil type	Top layer [m +MSL]	Bottom layer [m +MSL]	Coefficient on active side	Coefficient on passive side	Coefficient on neutral side
Boulders	+2.0	+1.5	0.25	7.62	0.39
Sand with boulders. cobbles and gravels	+1.5	+1.0	0.27	6.14	0.43
Boulders	+1.0	+0.5	0.25	7.62	0.39
Sand with boulders. cobbles and gravels	+0.5	+0.07*	0.27	6.14	0.43
Sand with boulders. cobbles and gravels	+0.07*	-0.5	0.28	6.07	0.43
Boulders	-0.5	-1.0	0.25	7.41	0.39
Sand with boulders. cobbles and gravels	-1.0	-2.0	0.29	5.86	0.43
Sand with gravels	-2.0	-5.0	0.34	4.80	0.45

*) Transition level corresponding to MSL.

Loads

Normal conditions consider the horizontal bollard loads (20 kN/m) and associated moments (20 kNm/m). as well as the vertical surface loads (15 kN/m^2). Earthquake conditions consider the seismic loads. as well as 30 % of the vertical surface loads. No wave loads are included for this section, as these loads play in favour of the stability of the structure.

Calculation method

Pseudo-static geotechnical calculations of the inner harbour designed with a sheet pile are performed using the software D-Sheet Piling (version 19.2. build 3.26114) similar to section 2.

2.2.5 Section 5: Main breakwater transition to coast

Section 5 corresponds to the north-eastern section of the breakwater between the main caisson (section 1) and the onshore structures, see figure 1-2. For the breakwater transition two alternative structures have been considered, similar to the ones considered for the main breakwater (section 1). For the first alternative the breakwater transition is designed as a sand-filled cofferdam, the second alternative is a structure consisting of caissons.

The retaining height of the breakwater transition is variable as the structure is built perpendicular to the coastline. The dimensions for the breakwater transition with the largest retaining height (which is used in the design calculations) is presented in table 2.15. A smaller retaining height can be used in shallower water and at the location where the inner harbour seabed reduces towards -2.5 m MSL.

Table 2.15 Breakwater transition cross section dimensions for design calculations

Dimension	Alternative 1: Cofferdam	Alternative 2: Caisson
Topside elevation deck	MSL +1.5 m	MSL +1.5 m
Topside elevation seaside wall	MSL +3.9 m	MSL +5.0 m
Seabed elevations	MSL -5.0 m (seaside) MSL -4.5 m (harbour side)	MSL -5.0 m (seaside) MSL -4.5 m (harbour side)
Retaining height	8.9 m (seaside); 6.0 m (harbour side)	10.0 m (seaside); 6.0 m (harbour side)
Width structure	15.0 m	15.0 m

Soil profile

The soil profile applied for the geotechnical calculations of the breakwater transition is based on the nearest normative borehole B13 for this section and is therefore comparable to the soil profile applied for the main breakwater from section 1. For section 5 it is also assumed that the overlying layer of boulders will be removed before the breakwater extension will be constructed and the provisions will be made to break down rocks at larger depths when these are encountered during pile driving (if a cofferdam design is chosen). The applied soil profile for the geotechnical calculations is given in table 2.16

Table 2.16 Soil profile near the breakwater extension based on borehole B13

Top of soil layer [m +MSL]		Lithology [-]	Colour in geotechnical profile [-]
x = 0 m	x = 15 m		
-5.00	-4.50	Sand with (coral) gravels. pebbles and cobbles. brownish to grey colour	Yellow
-8.00	-8.00	Gravel with pebbles and boulders. multi coloured	Green. Red
-9.00	-9.00	Sand with gravels. pebbles and cobbles. brownish to greyish colour	Yellow

Loads

Normal conditions consider the horizontal bollard loads (40 kN/m) and associated moments (20 kNm/m). as well as the vertical surface loads (20 kN/m^2). During earthquake conditions, a combination factor of 0.3 is applied on the surface loads since no permanent loads will be present at the breakwater extension, resulting in a surface load of 6 kN/m^2. During these conditions, bollard loads are not taken into consideration since it is unlikely that storm conditions (maximum bollards forces) coincide with the occurrence of an earthquake. Hurricane conditions only consider the wave loads since the harbour will be empty during hurricanes.

Wave load application

From the performed calculations for the main breakwater from section 1 it was already concluded that the application of a static wave load results in unrealistic breakwater dimensions. Therefore, only dynamic wave loads have been considered for the design calculations for the breakwater extension. The used load multipliers are similar to ones used for the calculations for the main breakwater from section 1.

Calculation method

The geotechnical calculations for the main breakwater are performed using the software Plaxis 2D (version 20.04.00.790. build 790). For the calculations regarding the seismic conditions, the pseudo-static model conditions are used in combination with a horizontal acceleration equal to the equivalent seismic coefficient k_e determined in appendix V. For the calculations regarding the hurricane conditions a dynamic calculation in Plaxis 2D is used given the short time duration of the peaks of the waves and the repetitive loading. A static calculation is considered too conservative since this will have to be modelled by continuous wave forces equal to the peak load. The Hardening Soil model with small-strain stiffness is used to model the different soil layers. Overall stability is confirmed implicitly, and structural forces and displacements are used to verify the strength and stability of the structure.

2.2.6 Section 6: Central pier

Section 6 located in between the two harbour basins and provides access to the finger piers location on both sides. The anticipated users on the finger piers are fishing boats, marine park vessels, local recreational boaters, charter dive vessels and transient boaters. The Central pier is designed as a gravity-based block-dam with a height between 4.5m (north part) and 5.7 m (south part) and a width of 4.0 m.

Soil profile
The soil profile applied for the geotechnical calculations of the Rock central pier is based on the nearest normative borehole B03 for this section (see appendix I), already given in table 2.13.

Seismic soil pressure coefficients
Seismic soil pressure coefficients for seismic L2 loads need to be applied on the structure as additional horizontal and vertical loads. In the case of the Central pier, they are mostly dependant on the weight of the concrete pier. The coefficient is calculated according to Eurocode EN 1998-5 based on the formula:

$$F_h = 0.5\alpha.S.W = 0.5 * 0.275/9.81 * 1.15 * \left[9.81 * \left(24 * \frac{(1*4*4.5)}{2} + 14 * \frac{(1*4*16.3)}{2}\right)\right] = 53.19\ kN/\text{m}$$
$$F_v = \pm 0.5.F_h = \pm 0.5 * 53.19 = 26.60\ kN/\text{m}$$

Loads
Normal conditions consider bollard loads (20 kN/m) and associated moments (20 kNm/m), as well as the vertical surface loads (10 kN/m^2) applied along the entire 4.0 m width of the pier. The self-weight of the concrete pier above and under the water surface is also applied along the entire 4.0 m width of the pier.

Hurricane conditions only consider the horizontal wave loads acting at the crest level of the structure (48 kN/m^2) since the harbour will be empty during hurricanes, plus the mass of the concrete pier above and under the water surface applied along the entire 4.0 m width of the pier.

Earthquake conditions consider the horizontal seismic loads (53.19 kN/m), as well as the vertical seismic loads (26.60 kN/m), the mass of the concrete pier above and under the water surface applied along the entire 4.0 m width of the pier, plus 30% of the vertical surface loads (10 kN/m^2) applied along the entire 4.0 m width of the pier.

Calculation method
Pseudo-static geotechnical calculations of the Central pier are performed using an in-house spreadsheet calculating shallow foundations in accordance with Eurocode 7 (EN 1997-1 and NEN 9997-1). The spreadsheet verifies the vertical bearing capacity, the horizontal shear resistance and the tilting stability off the structure. The structure is designed with a foundation level situated 0.5 m under the surface level at -3.5 m MSL to prevent shearing between the foundation level and the structure.

2.2.7 Section 7: Finger piers

Section 7 refers to the finger piers situated in both harbour basins. These finger piers are designed to berth small fishing boats, marine park vessels, local recreational boaters, charter dive vessels and transient boaters. The length of the deck of the finger piers varies from 10 m till 16.0 m. The width differs from 2.0 m of the widest finger pier, 1.0 m of the smallest finger pier and there is also of finger pier with a deck wide 1.5 m. One typical section is designed for a finger pier width of 2m and a length of 14m. The structure is designed with 5 foundation piles per pier, with a cantilever height of 3.6 m between a pile head at +0.6 m MSL and a bed level at -3.0 m MSL.

Soil profile
The soil profile applied for the geotechnical calculations of the Finger piers is based on the nearest normative borehole B03 for this section (see appendix I), already given in table 2.13.

Seismic soil pressure coefficients
Seismic soil pressure coefficients for seismic L2 loads need to be applied on the structure as additional horizontal and vertical loads. In the case of the Finger piers, they are mostly dependant on the mass of the piles equals to 107.5 kN. The coefficient is calculated according to Eurocode EN 1998-5 based on the formula:

$$F_h = 0.5\alpha.S.W = 0.5 * 0.275/9.81 * 1.15 * (9.81 * 107.5) = 17.0\ kN/\mathrm{m}$$
$$F_v = \pm 0.5.F_h = \pm 0.5 * 20.65 = 8.5\ kN/\mathrm{m}$$

Loads
Section 7 shall be designed to sustain both horizontal and vertical wave loads applied on the finger piers. The pile reaction to horizontal wave loads, as well as the downwards and upwards pile reaction to vertical wave loads were calculated in a separate memo presented in appendix VI. Resulting wave loads are given in table 2.17. Normal conditions shall also consider a vertical surface load (5 kN/m^2) applied along the entire 2.0 m width of the pier.

Earthquake conditions include horizontal seismic loads (17.0 kN/m), as well as vertical seismic loads (8.5 kN/m). However, these loads are not normative for the design of the structure in comparison to the wave loads and are consequently not taken into consideration in the calculations.

Table 2.17 Wave loads acting on the piles

	Horizontal wave loads [kN/pile]	Vertical wave loads - pressure [kN/pile]	Vertical wave loads -Tension [kN/pile]
ULS	31.5	364.5	-122.6
SLS	-	253.7	-38.7

Calculation method
Pseudo-static geotechnical calculations of the Finger piers are performed using the software D-Sheet Piling model single pile (version 19.2. build 3.26114) to determine the resistance of the piles to horizontal loads and using the software D-Foundation (version 19.1 build 1.23780) to determine the bearing and tension capacity of the piles.

3

CALCULATION RESULTS

3.1 Section 1: Main Breakwater

For the main breakwater of section 1, two alternative structures haven been considered. The first alternative is a sand-filled cofferdam, for the second alternative the main breakwater is designed as a gravity-based caisson structure. The geotechnical calculation results for these two alternatives show that the hurricane conditions are normative for the breakwater design. For each of the alternatives, the results of the different Plaxis models (static and dynamic) will be considered first, followed by a verification of the breakwater design. The results of the geotechnical calculations for section 1 are presented in appendix VII.

3.1.1 Alternative 1: Cofferdam

Plaxis results

Static Plaxis model

The static load calculation results in soil body collapse after one wave load, demonstrating that the static approach is overly conservative, because it neglects any time-dependent effects such as inertia of the structure.

Dynamic Plaxis model

The results for a wave load consisting of a single wave show that the cofferdam is stable during the dynamic calculation. However, residual displacements and bending moments permanently deform the soil and combination walls after the peak load. figure 3-1 presents the horizontal displacements in the cofferdam at the end of a single wave load for a 15 m wide cofferdam. figure 3-2 shows the horizontal displacements at the top of the combination walls (front wall and rear wall) during the wave load. The results show that the wall displacements approach a stable equilibrium after the peak load occurs and only a smaller wave load is present. The residual displacements after a single wave are approximately 0.45 m. which is rather large. However, figure 3-1 shows that the deformations at the tip of the combination wall is close to 0, meaning that the walls are sufficiently fixed to be able to withstand the single wave load.

Figure 3-1 Total horizontal displacements at the end of a single wave (for 15 m wide cofferdam, tubular pile diameter of 1422 mm)

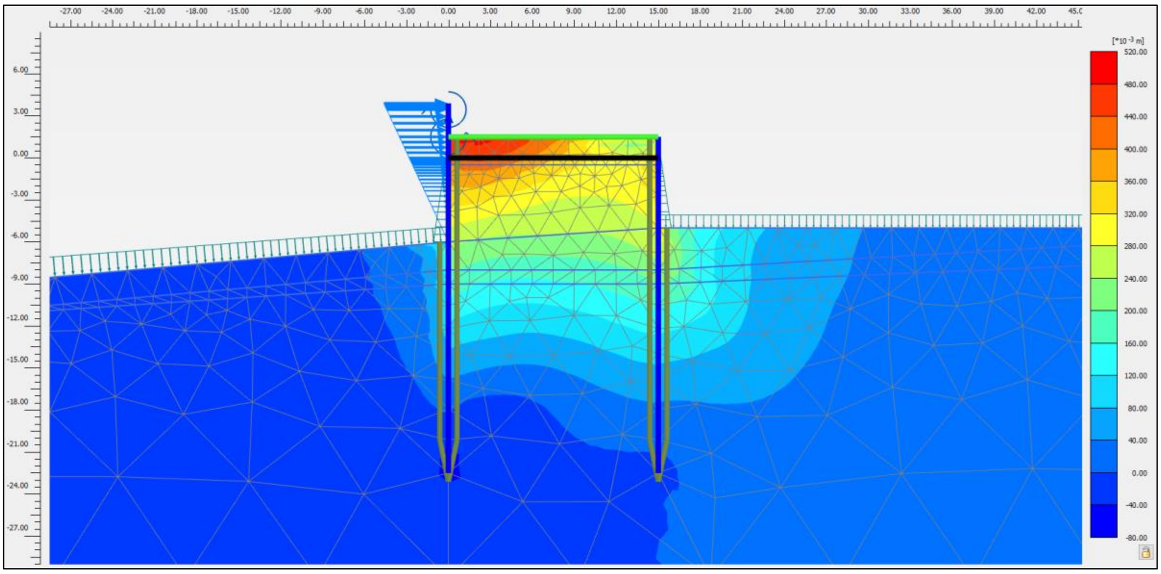
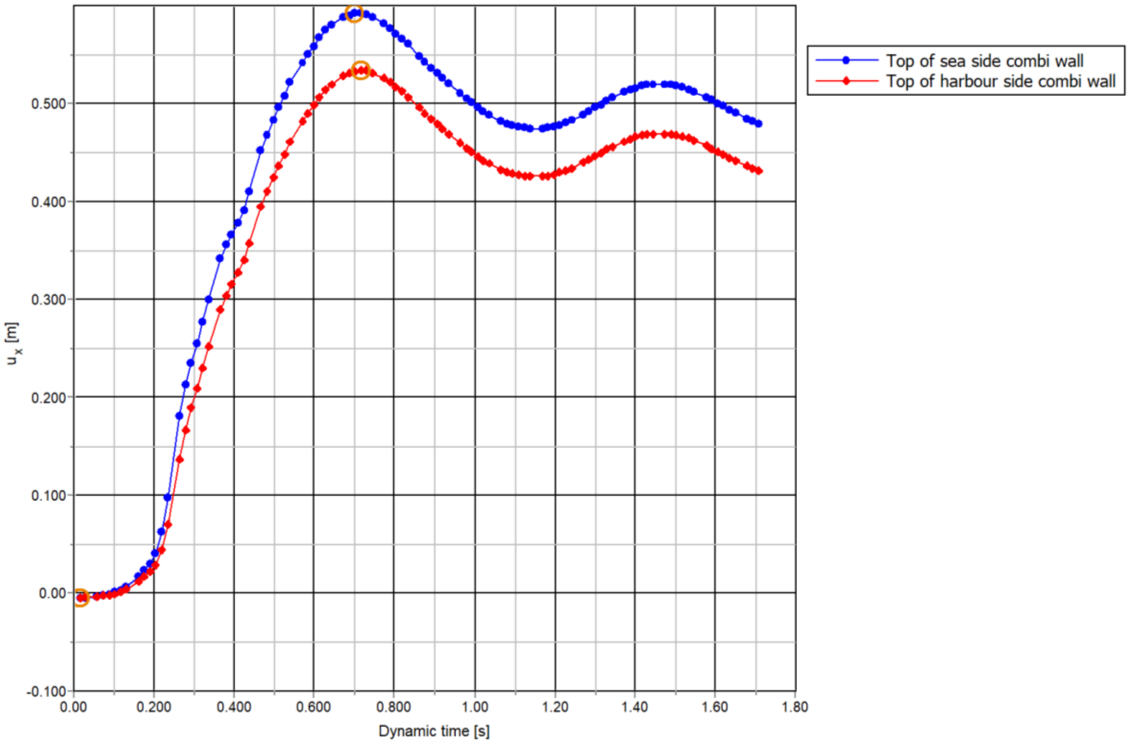


Figure 3-2 Total horizontal displacements at top of sea side combination wall (MSL +3.9 m) and harbour side combination wall (MSL +1.5 m) for a single wave (for 15 m wide cofferdam, tubular pile diameter of 1422 mm)

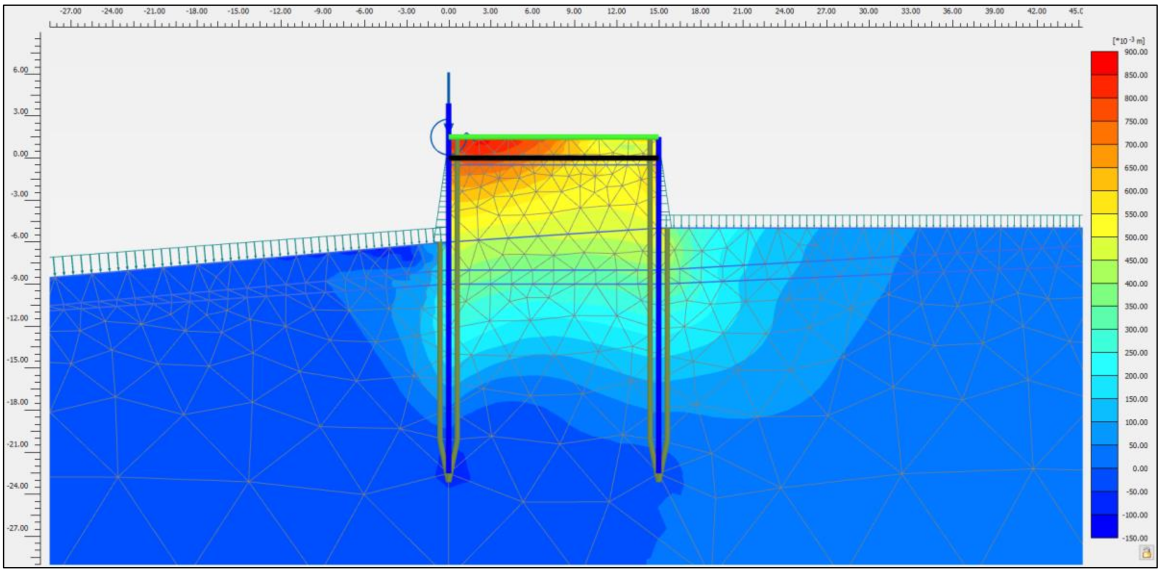


Multiple waves

The results for a wave load consisting of multiple (3) waves show that the residual displacements after each wave load do not decrease over time even after multiple peak wave loads. figure 3-3 presents the horizontal displacements in the cofferdam at the end of the multiple wave loads. It shows that the combined walls are sufficiently fixed to be able to withstand multiple wave loads.

It appears that the soil resistance is exceeded during each individual wave peak load resulting in a stepwise push-over of the structure. There is some plastic soil deformation each time a wave hits the structure and therefore no steady state is reached. However, in Plaxis only the landside-directed components of the wave load are modelled. The impact of the trough following each wave peak is neglected. In reality the wave trough will in fact cause a displacement in opposite direction compared to the modelled wave load. Furthermore, the flexibility of the structure is not included, whereas in practice the impact of each wave peak load is dampened as a result of a deflection of the structure. Finally, it is known that Plaxis can cope only to a limited extent with repetitive loading in one direction. Therefore, the results of the displacements for the multiple wave case are considered to be less certain. Because the structure remains stable also under these conservative assumptions this does indicate that the design solution is stable and safe. If in detailed design it is found that displacements remain an issue, it can be decided to stiffen the structure by applying a different and stiffer fill material (coarser grading) and/or unreinforced concrete layers to stiffen the behaviour of the entire cofferdam.

Figure 3-3 Total horizontal displacements at the end of three consecutive waves (for 15 m wide cofferdam, tubular pile diameter of 1422 mm)



Cofferdam design

In order to ensure the overall stability, a cofferdam with a width of 15 m is required. The combined walls of the cofferdam each include tubular piles with a diameter / wall thickness of 1422 / 25 mm (X70) with three intermediate sheet piles of the type PU28. The combined walls are connected by tie rods with a diameter of 4 inch at MSL. The top side deck is connected to both the seaside and harbour side combination wall by expansion joints. As a fill material for the cofferdam, a gravel material is considered. However, in case it is desirable to decrease the deformations of the structure during hurricane conditions, the stiffness of the cofferdam can be increased by applying a coarser grading of the fill or for example adding layers of unreinforced concrete to the cofferdam fill.

table 3.1 includes the verification of the combination walls according to CUR211 for a single wave loading. The design moments shown are the maximum bending moments found at the level of the topside of the deck and along the combination wall lengths at the seaside and harbour side. In the verification a corrosion of 1.25 mm in 50 years has been taken into account. The results show sufficient margin regarding the structural capacity of the combination walls.

Table 3.2 presents the anchor verification for a 4'' tie rod (f_y = 500 MPa) and a corrosion of 2.5 mm in 50 years.

Table 3.1 Verification of combination wall according to CUR211

	Single wave	Unit
$M_{E;d;level\ of\ deck\ (MSL\ +1.5\ m)}$	3,995	kNm/m
$M_{E;d;field}$	4,857	kNm/m
unity check	0.83	-

Table 3.2 Verification of tie rod (4'' tie rod)

	Single wave	Unit
$R_{t;d}$	1,818	kN/anchor
F_{anchor}	1,097	kN/anchor
unity check	0.60	-

Scour protection

A scour protection might be necessary in front of the cofferdam to prevent excessive scour hole development. The scour protection is designed for the cofferdam alternative in this design stage.

3.1.2 Alternative 2: Caisson

Plaxis results

Static Plaxis model

The static load calculation results in soil body collapse after one wave load, demonstrating that the static approach is overly conservative, because it neglects any time-dependent effects such as inertia of the structure.

Dynamic Plaxis model

The results for a wave load consisting of a single wave show that the caisson is stable during the dynamic calculations. However, residual displacements permanently deform the soil after the peak load. Figure 3-4 presents the displacements of the caisson at the end of a single wave for a 15 m wide caisson and an 18 m wide caisson. Figure 3-5 shows the horizontal displacements at the top of the caisson (harbour side wall) during the wave load. The results show that the displacements for both caissons approach a stable equilibrium after the peak load occurs and only a smaller wave load is present. However, the calculated displacements for an 18 m wide caisson are much smaller. The residual horizontal displacements after a single wave for a 15 m wide caisson are approximately 0.29 m. whereas these residual displacements are approximately 0.14 m for an 18 m wide caisson.

Figure 3-4 Total displacements at the end of a single wave for a 15 m wide caisson (left) and an 18 m wide caisson (right)

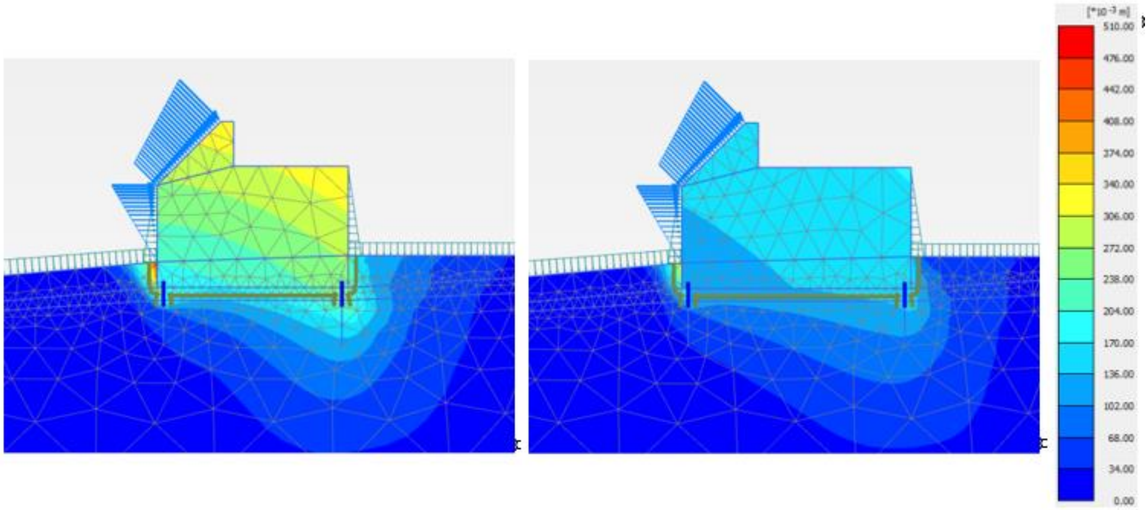
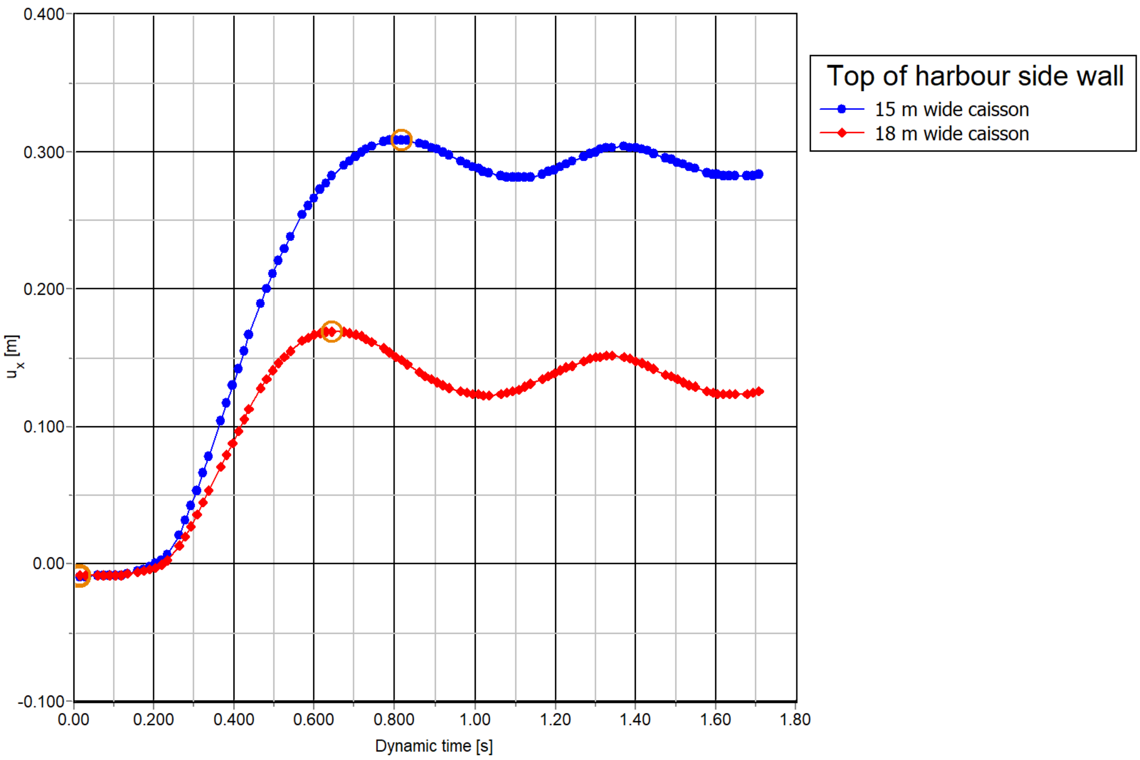


Figure 3-5 Total horizontal displacements at top of harbour side wall (MSL +1.5 m) for a 15 m wide caisson and an 18 m wide caisson for a single wave

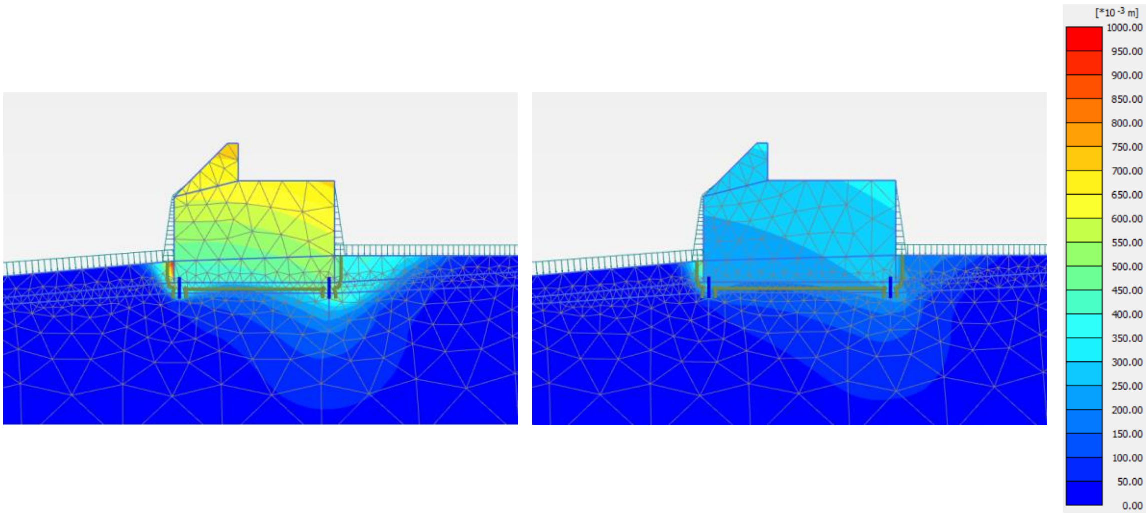


Multiple waves

The results of a wave load consisting of multiple (3) waves show that the residual displacements after each wave load do not decrease over time even after multiple wave loads. figure 3-6 presents the displacements of the caisson at the end of the multiple waves loads for a 15 m wide caisson and an 18 m wide caisson. It shows that both caissons also remain stable after multiple wave loads. However, for a smaller caisson, much larger displacements are calculated.

It appears that the soil resistance is exceeded during each individual wave peak load resulting in a stepwise push-over of the structure. There is some plastic deformation of the soil each time a wave hits the structure and therefore no steady state is reached. However, in Plaxis only the landside-directed components of the wave load are modelled. The impact of the trough following each wave peak is neglected. In reality the wave trough will in fact cause a displacement in the opposite direction compared to the modelled wave load. Furthermore, it is known that Plaxis can cope only with to a limited extent with repetitive loading in one direction. Therefore, the results for the displacements for the multiple wave case are considered less certain. Because the structure remains stable under these conservative assumptions this does indicate that the design solution is stable and safe. If in detailed design, it is found that the displacements remain an issue it can be decided to widen the entire caisson or to widen only the base of the structure.

Figure 3-6 Total displacements at the end of three consecutive waves for a 15 m wide caisson (left) and an 18 m wide caisson (right)



Caisson design

From the geotechnical calculation results it can be concluded that the overall stability is ensured for both a 15 m wide caisson and an 18 m wide caisson. However, based on the calculation results much larger displacements are expected for a 15 m wide caisson which are considered too large for the breakwater. These large displacements can to some extent be associated to tilting of the structure. By increasing the effective width at the foundation level of the caisson the structure becomes less prone to tilting, resulting in smaller displacements. In order to allow for a 15 m wide caisson, the base of the caisson therefore needs to be wider than the top part. Based on the calculated displacements for the 18 m wide caisson, a caisson with a width of 15 m and a widened base of 18 m is considered stable and safe. In case it is desirable to increase the stability of the caisson during hurricane conditions, the base of the structure can be widened further, which also results in the development of more resistance against sliding of the structure. However, it should be noted that widening of the base requires extra reinforcement of the base and adds weight to the structure, which makes transport more difficult. The current base extension (1.5 m at each side) is considered to be realistic.

In order to develop sufficient (passive) resistance against horizontal sliding of the structure, the caisson should be embedded in the seabed. To limit the embedding to a maximum of MSL -8.0 m, the caisson needs to be equipped with skid walls under the outer walls of the caisson. Since the submerged weight of the caisson needs to overcome the friction along the skid walls in order to press the walls into the seabed, the applicable length of the skid walls is limited. For 1.5 m long skid walls of the sheet pile wall type AZ26-700, the submerged weight of the caisson is considered large enough to properly install the skid walls into the seabed by deadweight of the structure only (no driving).

Alternative solutions that provide the same additional resistance as the skid wals may be considered by the Contractor (for example: tubular piles driven through the caisson, high-density caisson filling, ground anchors, etc.), as long as sufficient passive resistance is mobilized.

In case it is desirable to further decrease the horizontal sliding of the caisson, the layer under the caisson can be injected with grout in order to increase the skin friction along bottom of the structure.

Scour development and scour protection

The development of scour during design conditions is estimated in appendix IV. It is concluded that scour will not undermine the main caisson structure. A scour protection is therefore not included in the caisson alternative. Development of scour in front of vertical structures is uncertain and might be more extensive than calculated. It might be considered to protect the bed level in front of the caisson to mitigate scour development risks.

3.2 Section 2: Ferry quay wall

Geotechnical calculations for section 2 are presented in table 3.3. table 3.4 and appendix VIII. To ensure the strength and stability of the Ferry quay wall. The sheet pile wall must be a type AZ26-700 (S240) with the tip of the sheet pile wall located at -11.5 m MSL. The maximum displacements of the sheet pile wall occur during earthquake conditions (figure 3-7). Whereas the maximum bending moments and shear forces occur during normal conditions when bollards and cleats loads are applied.

The retaining wall must be supported by anchors of 20 m long with an angle of -1 degree and a tierod level located at +0.5 m MSL. The maximum anchor forces are recorded during normal conditions. An in-house spreadsheet allows to determine the design values of the anchors when taking into account a corrosion of 2 mm radial. The resulting withworth DIN thread of the anchors must be 2.50 inches (59.57 cm) and the centre-to-centre distance of the anchors behind the sheet pile wall must be 2.8 m.

Anchors must be attached to an anchor wall of type AZ26-700 (S240). Selected based on driveability. With a tip located at -2.35 m MSL. Concrete slabs will be placed on top of the anchor wall. Whose top level should therefore be located at least 15 cm below ground level at +1.95 m MSL. The maximum bending moments and shear forces of the anchor wall occur during normal conditions. Whereas the maximum displacements occur during earthquake conditions. However. The structural capacity of the sheet pile wall is higher than required. Because of the estimated driving resistance. If a thinner (and less stiff) sheet pile wall will be used the risk of damage during installation is much higher. Since the soil investigation identified the presence of gravel and cobbles.

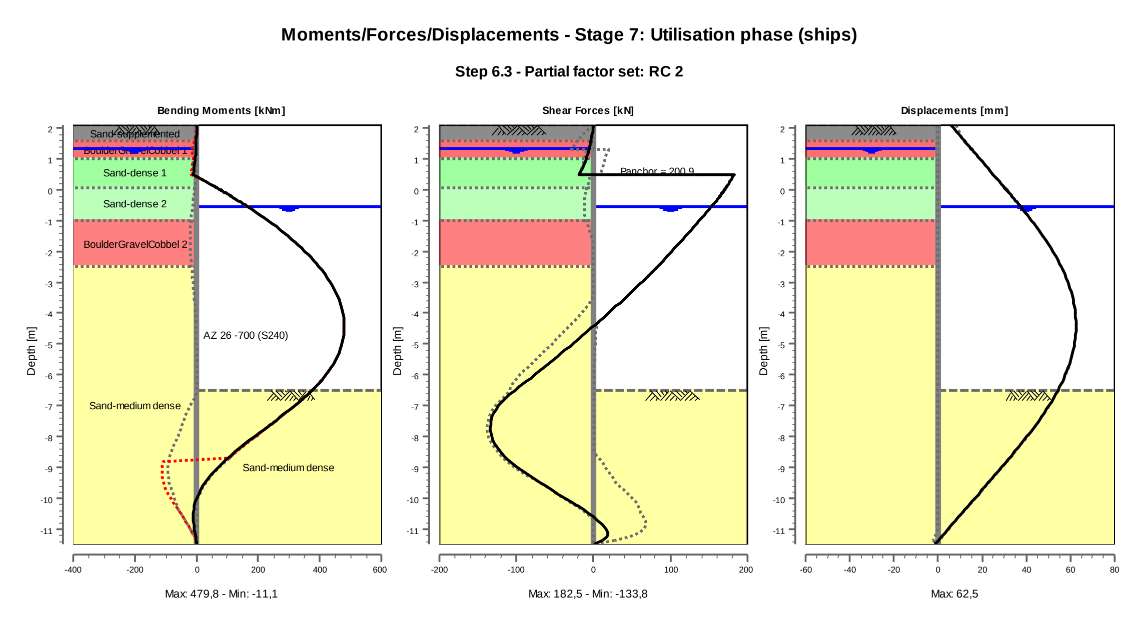
Table 3.3 Geotechnical calculations for the design of the sheet pile wall and the anchor wall in section 2

Construction	Maximum displacement	Maximum moment [kN/m]	Maximum shear force [kN]
Sheet pile wall	36.3	479.76	182.5
Anchor wall	1.6	114.00	161.18

Table 3.4 Geotechnical calculations for the design of the anchor in section 2

Construction	Maximum anchor force [kN]	Actual anchor force CUR (F_{max}*1.5) [kN]	Characteristic Kranz anchor strength [kN]	Check
Anchor	200.88	201.32	814.68	ok

Figure 3-7 Bending moments. Shear forces and displacements along the sheet pile wall of section 2 during normal conditions



The approximate degree of damage to anchored sheet pile walls during earthquakes is calculated using an in-house spreadsheet according to Gazetas et al. 1990. Detailed results are presented in appendix VIII. The degree of damage plot in zone 1. Corresponding to negligible damage to the wall itself (maximum 10 cm permanent displacement at top of the sheet pile) and possible noticeable damage to the related structures. The selected anchor length and anchor wall height is therefore considered to be sufficient.

3.3 Section 3: Rock slope

The design calculations for the rock slope are provided in appendix III. The main dimensions are given in table 3.5. Local rock may be suitable for use as an underlayer, or even as toplayer. This however depends on the rock density and other intrinsic properties.

Table 3.5 Main dimensions and characteristics rock slope

Wave damping slope	
Toe level	MSL - 5.0 m
Crest level	MSL + 3.3 m
Slope	1:2
Rubble mound	3 - 6 T
Sub layer	1 - 3 T

3.4 Section 4: Inner harbour wall

Geotechnical calculations for the installation of a sheet pile wall in section 4 are performed similarly as in section 2 and are presented in table 3.6. table 3.7 and appendix IX. To ensure the strength and stability of the inner quay, the sheet pile wall must be a type AZ26-700 (S240) with the tip of the sheet pile wall located at -8.5 m MSL. The maximum bending moments and the shear forces are recorded during earthquake conditions (figure 3-8), whereas the maximum displacements are recorded during normal conditions when bollards and cleats loads are applied.

The retaining wall must be supported by anchors of 15 m long with an angle of -2 degrees and a tierod level located at +0.5 m MSL. The maximum anchor forces are recorded during normal conditions. An in-house spreadsheet allows to determine the design values of the anchors when taking into account a corrosion of 2 mm radial. The resulting withworth DIN thread of the anchors must be 2.50 inches (59.57 cm) and the centre-to-centre distance of the anchors behind the sheet pile wall must be 2.8 m.

Although the retaining height is smaller in section 4 compared to section 2 allowing for a shorter sheet pile wall. the soil profile near section 4 contains finer-grained sediments and the water level is closer to the surface (smaller passive resistance), requiring a longer anchor wall. The anchor wall must be of type AZ26-700 (S240), selected based on driveability, with the tip of the anchor wall located at -3.50 m MSL. Concrete slabs will be placed on top of the anchor wall, whose top level should therefore be located 15 cm below ground level at +1.35 m MSL. The maximum bending moments and the shear forces are recorded during normal conditions, whereas the maximum displacements are recorded during earthquake conditions. However, the structural capacity of the sheet pile wall is higher than required, because of the estimated driving resistance. If a thinner (and less stiff) sheet pile wall will be used the risk of damage during installation is much higher, since the soil investigation identified the presence of gravel and cobbles.

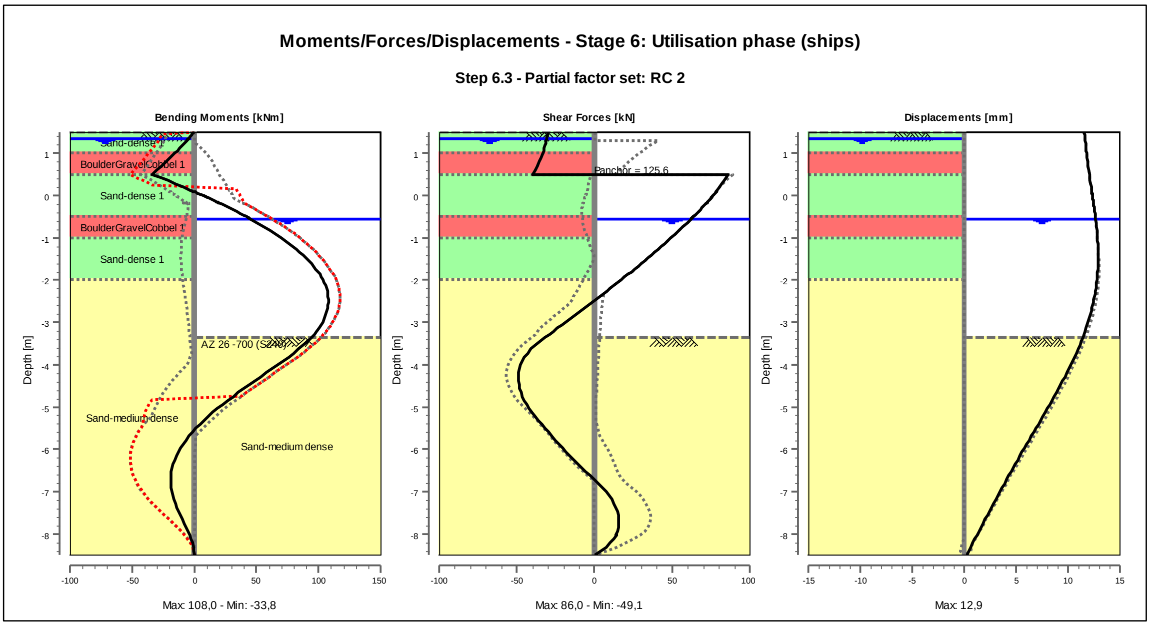
Table 3.6 Geotechnical calculations for the design of the sheet pile wall and the anchor wall in section 4

Construction	**Maximum displacement** [mm]	**Maximum moment [kN/m]**	**Maximum shear force [kN]**
Sheet pile wall	9.0	117.23	89.2
Anchor wall	2.3	57.43	124.24

Table 3.7 Geotechnical calculations for the design of the anchor in section 4

Construction	**Maximum anchor force** [kN]	**Actual anchor force CUR (F_{max}*1.5)** [kN]	**Characteristic Kranz anchor strength [kN]**	**Check**
Anchor	125.64	188.46	659.79	ok

Figure 3-8 Bending moments, shear forces and displacements along the sheet pile wall of section 4 during normal conditions



The approximate degree of damage to anchored sheet pile walls during earthquakes is calculated using an in-house spreadsheet according to Gazetas et al. 1990. Detailed results are presented in appendix IX. The degree of damage plot in zone 1 corresponding to negligible damage to the wall itself (maximum 10 cm permanent displacement at top of the sheet pile) and possible noticeable damage to the related structures. The selected anchor length and anchor wall height is therefore considered to be sufficient.

Note: although an anchored sheetpile wall has been selected as reference design, it may be preferred to design and built this quay wall as a gravity-based wall (L-wall or block wall). This depends on the Contractor construction method and preferences. Alternatives are allowed with the anticipated Design & Build (or Enigneer & Construct) contract, provided that the design starting points as formulated in this Design Document are maintained.

3.5 Section 5: Main breakwater transition to coast

For the main breakwater transition to the coast. two alternative constructions have been considered. similar to the alternatives considered for the main breakwater from section 1. The first alternative entails a sand-filled cofferdam. as a second alternative a gravity-based caisson structure is considered. The geotechnical calculation results for these two alternatives shows that the hurricane conditions are normative for the considered cross section for the breakwater transition. Closer to the shore, the operational conditions will become more normative as a result of the decreasing water depth and wave loads.

3.5.1 Alternative 1: Cofferdam

Plaxis results

The results for a wave load consisting of a single wave show that the cofferdam is stable during the dynamic calculations. However, residual displacements and bending moments permanently deform the soil and combination walls after the peak load. figure 3-9 presents the horizontal displacements in the cofferdam at the end of a single wave load for a 15 m wide cofferdam. figure 3-10 shows the horizontal displacements at the top of the combination walls (front wall and rear wall) during the wave load. The results show that the wall displacements approach a stable equilibrium after the peak load occurs and only a smaller wave load is present. The residual displacements after a single wave are approximately 0.17 m. However, figure 3-9 shows that the deformations at the tip of the combination walls is close to 0, meaning that the walls are sufficiently fixed to be able to withstand the single wave load.

Figure 3-9 Total horizontal displacements at the end of a single wave (for 15 m wide cofferdam, tubular pile diameter of 1422 mm)

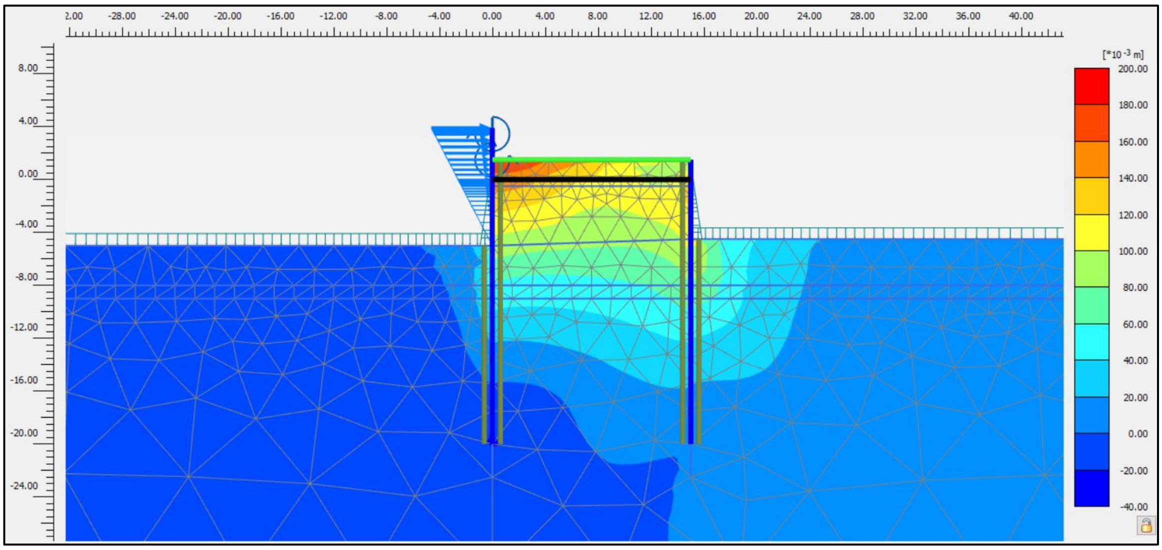
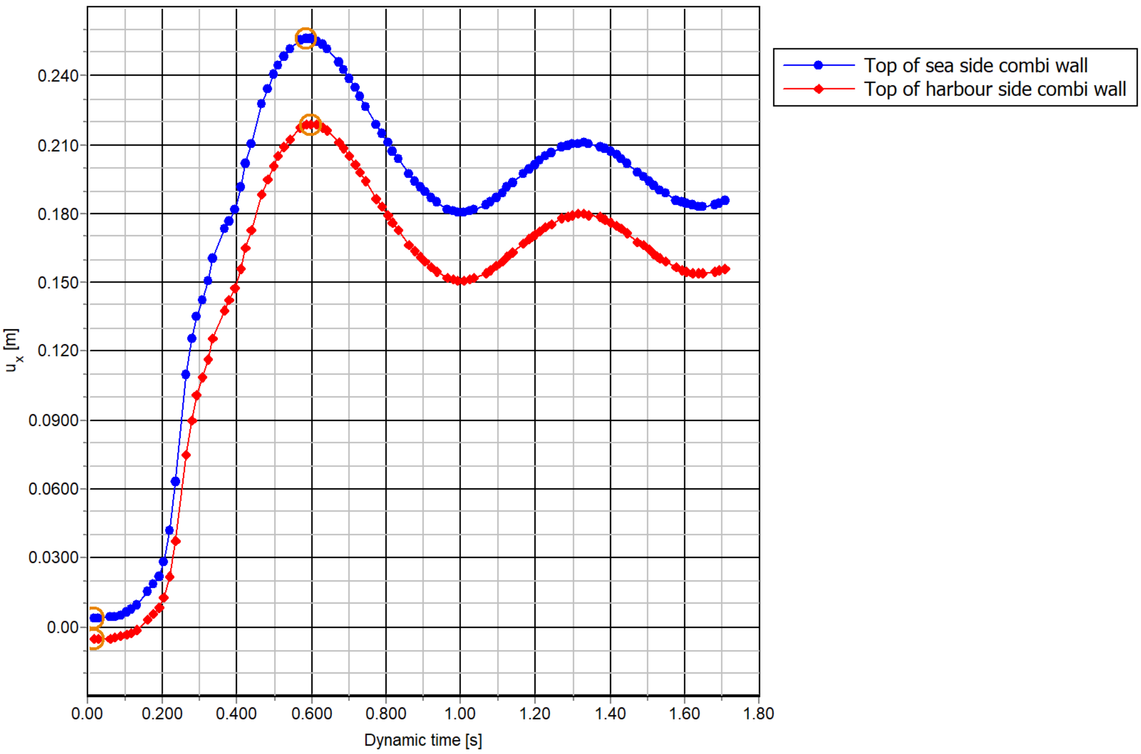


Figure 3-10 Total horizontal displacements at top of seaside combination wall (MSL +3.9 m) and harbour side combination wall (MSL +1.5 m) for a single wave (for 15 m wide cofferdam, tubular pile diameter of 1422 mm)



Cofferdam design

In order to ensure the overall stability. a cofferdam with a width of 15 m is required. The combined walls of the cofferdam each include tubular piles with a diameter / wall thickness of 1422 / 25 mm (X70) with three intermediate sheet piles of the type PU28. The combined walls are connected by tie rods with a diameter of

4 inch at MSL. The top side deck is connected to both the seaside and harbour side combination wall by expansion joints. As a fill material for the cofferdam, a gravel material is considered. However, in case it is desirable to decrease the deformations of the structure during hurricane conditions. the stiffness of the cofferdam can be increased by applying a coarser grading of the fill or for example adding layers of unreinforced concrete to the cofferdam fill.

table 3.8 includes the verification of the combination walls according to CUR211 for a single wave loading. The design moments shown are the maximum bending moments found at the level of the topside of the deck and along the combination wall lengths at the seaside and harbour side. In the verification a corrosion of 1.25 mm in 50 years has been taken into account. The results show sufficient margin regarding the structural capacity of the combination walls.

table 3.9 presents the anchor verification for a 4'' tie rod (f_y = 500 MPa) and a corrosion of 2.5 mm in 50 years.

Table 3.8 Verification of combination wall according to CUR211

	Single wave	Unit
$M_{E;d;level\ of\ deck\ (MSL\ +1.5\ m)}$	2,516	kNm/m
$M_{E;d;field}$	2,731	kNm/m
unity check	0.46	-

Table 3.9 Verification of tie rod (4'' tie rod)

	Single wave	Unit
$R_{t;d}$	1,818	kN/anchor
F_{anchor}	696	kN/anchor
unity check	0.38	-

3.5.2 Alternative 2: Caisson

Plaxis results

The results for a wave load consisting of a single wave show that the caisson is stable during the dynamic calculations. However, residual displacements permanently deform the soil after the peak load. Figure 3-11 presents the displacements of the caisson at the end of a single wave. Figure 3-12 shows the horizontal displacements at the top of the caisson (harbour side wall) during the wave load. The results show that the displacements approach a stable equilibrium after the peak load occurs and only a smaller wave load is present. The residual displacements after a single wave are approximately 0.09 m.

Figure 3-11 Total displacements at the end of a single wave for a 15 m wide caisson

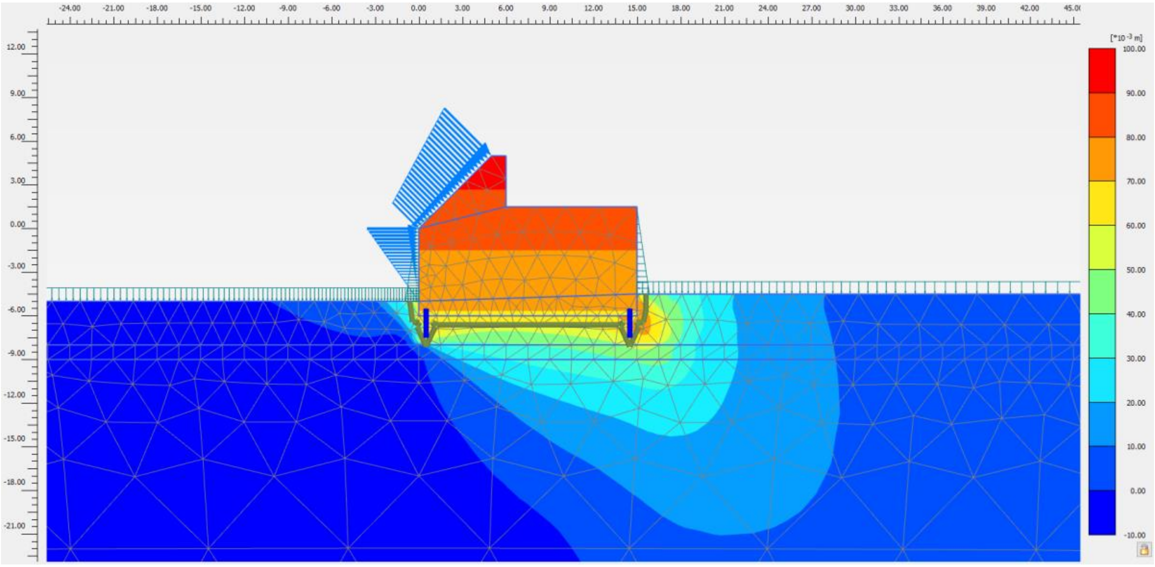
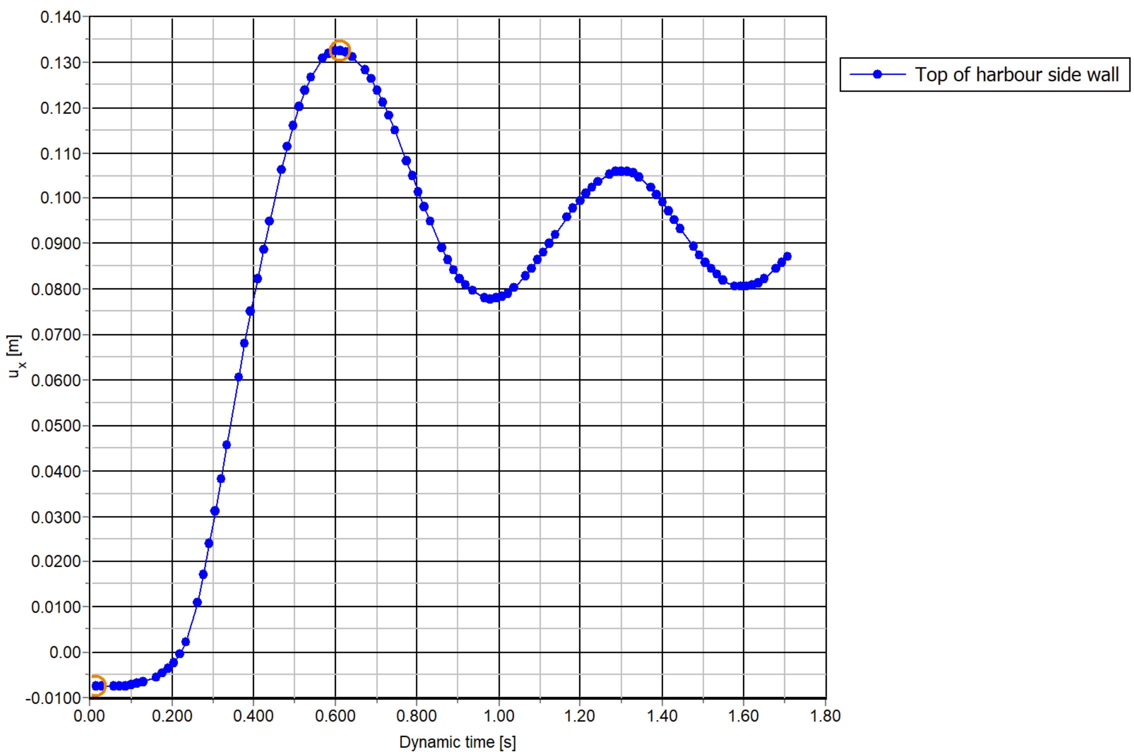


Figure 3-12 Total horizontal displacements at top of harbour side wall (MSL +1.5 m) for a 15 m wide caisson for a single wave



Caisson design

From the geotechnical calculations it can be concluded that the overall stability is ensured for a 15 m wide caisson. In case it is desirable to increase the stability of the caisson during hurricane conditions, the base of the structure can be widened, which also results in the development of more resistance against sliding of the structure. However, it should be noted that widening of the base requires extra reinforcement of the base and adds weight to the structure, which makes transport more difficult.

In order to develop sufficient (passive) resistance against horizontal sliding of the structure, the caisson needs be embedded in the seabed. To limit the embedding to a maximum of MSL -6.0 m, the caisson is equipped with skid walls under the outer walls of the caisson. Since the submerged weight of the caisson needs to overcome the friction along the skid walls in order to press the walls into the seabed, the applicable length of the skid walls is limited. For 1.5 m long skid walls of the sheet pile wall type AZ26-700, the submerged weight of the caisson is considered large enough to properly install the skid walls into the seabed by deadweight of the structure only (no driving).

Alternative solutions that provide the same additional resistance as the skid walls may be considered by the Contractor (for example: tubular piles driven through the caisson, high-density caisson filling, ground anchors, etc.), as long as sufficient passive resistance is mobilized.

In case it is desirable to further decrease the horizontal sliding of the caisson, the layer under the caisson can be injected with grout in order to increase the skin friction along the bottom of the structure.

3.6 Section 6: Central pier

Geotechnical calculations for section 6 include verifications of the vertical bearing capacity, the horizontal shear resistance and the tilting stability of the Central pier. The verifications are realised for the most critical situation, which corresponds to the most offshore transection of the construction where the retaining height reaches 5.7 m. Verifications performed under normal conditions are presented in table 3.10, under hurricane conditions in table 3.11 and under earthquake conditions in table 3.12. Detailed calculations can be found in appendix X. All verifications do meet the requirements to ensure the stability of the Central pier.

Table 3.10 Geotechnical verifications for the design of the Central pier under normal conditions in section 6

Verification	Situation	Vertical load	Load [kN/m]	Resistance [kN]	Unity check	Check
Vertical bearing capacity	Drained	unfavourable	376.4	927.1	0.41	ok
		favourable	336.4	903.7	0.37	ok
Horizontal shear resistance	Drained	unfavourable	20.0	131.5	0.15	ok
		favourable	20.0	117.5	0.17	ok
Tilting stability	Width direction	unfavourable	0.15	1.33	0.11	ok
		favourable	0.16	1.33	0.12	ok

Table 3.11 Geotechnical verifications for the design of the Central pier under hurricane conditions in section 6

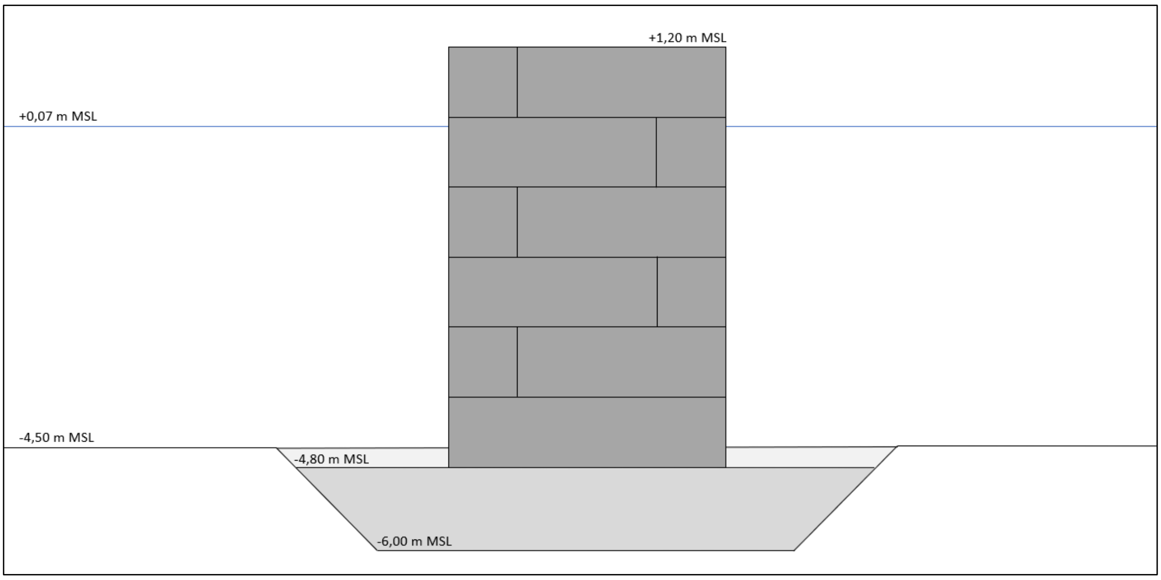
Verification	Situation	Vertical load	Load [kN/m]	Resistance [kN]	Unity check	Check
Vertical bearing capacity	Drained	unfavourable	363.0	726.2	0.50	ok
		favourable	336.4	698.4	0.48	ok
Horizontal shear resistance	Drained	unfavourable	53.2	126.8	0.42	ok
		favourable	53.2	117.5	0.45	ok
Tilting stability	Width direction	unfavourable	0.10	1.33	0.08	ok
		favourable	0.10	1.33	0.08	ok

Table 3.12 Geotechnical verifications for the design of the Central pier under earthquake conditions in section 6

Verification	Situation	Vertical load	Load [kN/m]	Resistance [kN]	Unity check	Check
Vertical bearing capacity	Drained	unfavourable	375.0	737.6	0.51	ok
		favourable	363.0	726.1	0.50	ok
Horizontal shear resistance	Drained	unfavourable	53.2	131.0	0.41	ok
		favourable	53.2	126.8	0.42	ok
Tilting stability	Width direction	unfavourable	0.10	1.33	0.08	ok
		favourable	0.10	1.33	0.08	ok

For the construction of the block dam, the weight of the blocks will be the limiting factor for the load capacity of the small mobile crane during construction. To allow for a 4 m wide central pier, two rows of blocks are considered with a length of 3.0 m. a width of 1.0 m and a height of 1.0 m resulting in a weight of 7.5 tonnes (figure 3-13). Assuming the maximal lifting capacity of the mobile crane is 8 tonnes, the distance between the crane and the quay shall not be more than approximatively 8 m.

Figure 3-13 Design of the Central pier (not on scale)



3.7 Section 7: Finger piers

To ensure the strength and stability of the finger piers against horizontal wave loads, the 5 foundation piles consist of open-ended steel pipe piles with a diameter of 762 mm. a thickness of 12 mm and with the tip of the piles located at -7.5 m MSL. Geotechnical calculations of the maximum bending moment, shear force and resulting stress of these piles are given in table 3.13. The overall buckling verification met the requirements as the unity check is equal to 0.11 for zone 1 EC. The piles have a relatively low u.c. on structural capacity since their size is selected based on tension bearing capacity (diameter) and driveability (wall thickness). Detailed calculations are presented in appendix XI.

The bearing and tension capacity of the piles against vertical wave loads was verified. The calculations met the requirements since the bearing capacity of about 750 kN remains far below the downwards pile reaction to vertical wave loads of 365 kN (figure 3-14). Tension capacity of the piles was the most critical, but calculations met the requirements as the tension’s capacity of about 128 kN remains lower than the upwards pile reaction to vertical wave loads of 122.6 kN (figure 3-15). Details of the calculations are presented in appendix XI.

Table 3.13 Geotechnical verifications for the design of the Finger piers in section 7

Construction	Maximum bending moment [kN/m]	Maximum shear force [kN]	Maximum resulting stress [kN]
Pile 762/12	143	67	108

Figure 3-14 Bearing capacity of the piles in section 7

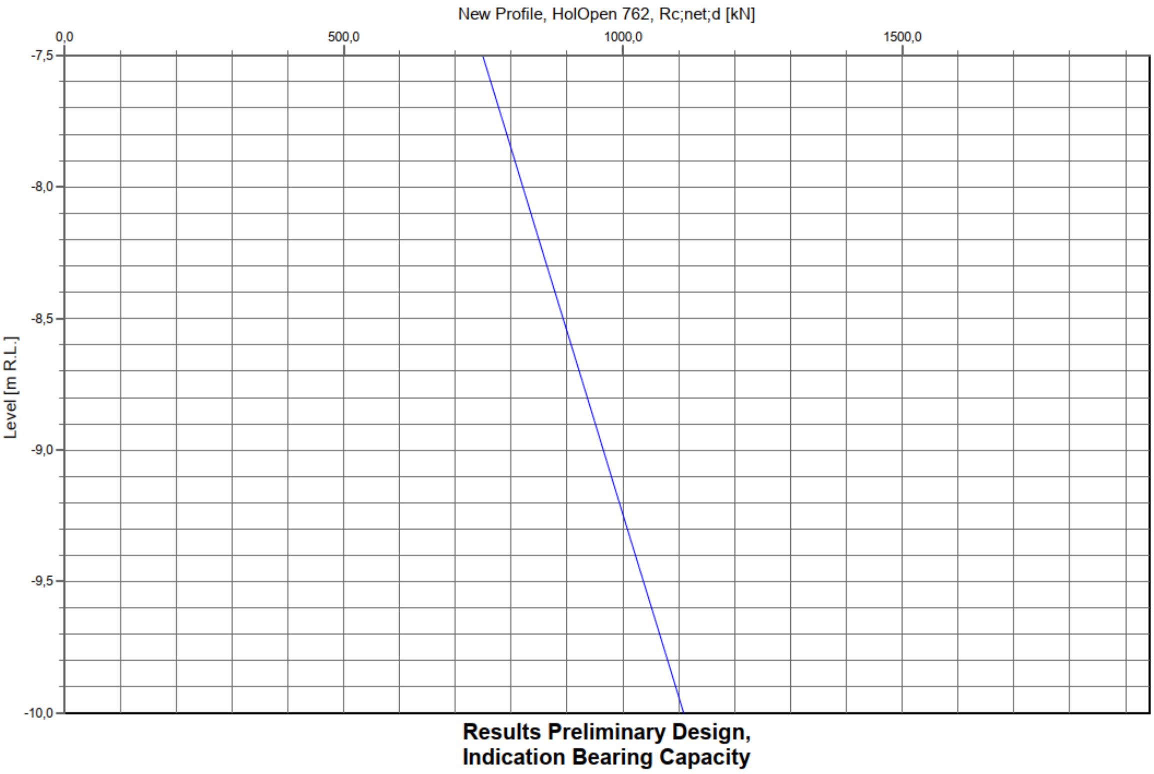
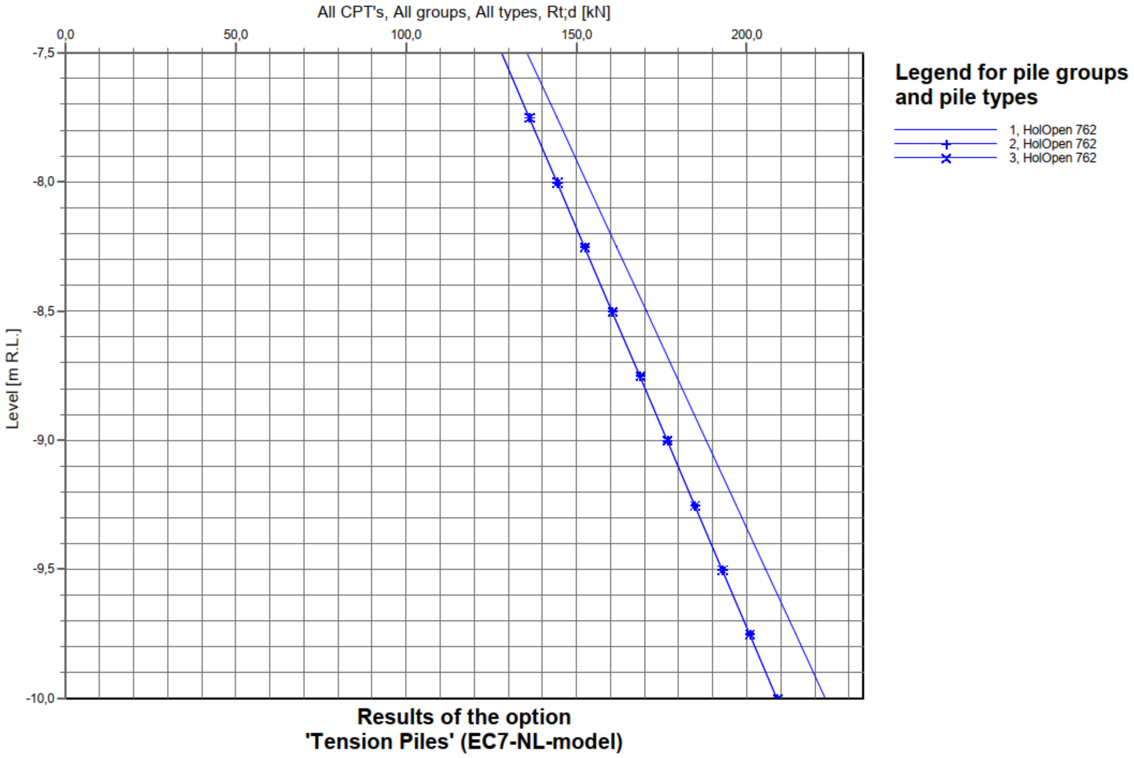


Figure 3-15 Tension capacity of the piles in section 7



4

CONCLUDING REMARKS

Based on the design calculations made for the new harbour lay-out at Black Rocks. the following recommendations are made:

- The site of Black Rock harbour is located in a small alluvial fan feed by slope debris. Attention must be drawn to the probability that (rock) debris flows and landslides might occur along the steep hills during important rainstorm or earthquake, which might cause damage to the harbour facilities.
- The risk and effects of liquefaction should be investigated in more detail since it may constitute an issue due to numerous sand layers encountered at the project location. In this design report, liquefaction wasn't taken into consideration as it was considered that soil improvement would be realised to prevent liquefaction.
- For section 1 two different alternatives have been considered:
  - Alternative 1: cofferdam:
    - The obtained unity checks for the combination walls and the tie rod show that the reference design for the cofferdam meets the requirements regarding the structural capacity for a single peak wave load.
    - For the multiple (3) wave load model the structure is stable as well, but the displacements were high and increasing with each wave load. However, some conservative assumptions were used in the calculation such as neglecting the opposite effect of a wave trough. The behaviour of the structure subjected to repetitive wave loading needs to be considered further in the next design stage. If displacements are still found to be too large, it can be considered to stiffen the cofferdam by applying stiffer fill material (coarse grading) or by applying layer(s) of unreinforced concrete within the fill.
    - The behaviour of the cofferdam structure under fatigue also needs to be taken into consideration in the detailed design.
    - The executed soil survey reveals the presence of medium and large size boulders and cobbles in the top layers of the soil and at greater depth. The large size boulders in the top layers can be removed in order to increase the driveability of steel piles. However, the boulders and larger cobbles found at greater depth increase the risks of not being able to drive steel piles into the soil with normal pile driving equipment (impact or vibration piles). Provisions will have to be made to be able to remove or break down rocks at larger depths when these are encountered during pile driving.
  - Alternative 2: caisson:
    - The performed calculation reveal that the structure is stable for both a 15 m wide caisson and an 18 m wide caisson for a single peak wave load. However, for a 15 m wide caisson the calculated displacements are considered too large. In order to allow for a 15 m wide caisson, the base of the caisson needs to be widened to 18 m.
    - For the multiple (3) wave load model the structure is stable as well, but the displacements were high and increasing with each wave load. However, some conservative assumptions were used in the calculation such as neglecting the opposite effect of a wave trough. The behaviour of the structure subjected to repetitive wave loading needs to be considered further in the next design stage. If displacements are still found too large, it can be considered to further widen the base of the structure. However, widening of the base requires extra reinforcement of the base and adds weight to the structure, which makes transport more difficult.
    - In order to decrease the horizontal sliding of the structure, the caisson needs to be equipped with skid walls, what makes construction of the caisson and transport more difficult. Furthermore,

the skid walls need to be installed into the seabed by deadweight of the structure only (no driving). Alternative solutions that provide the same additional resistance as the skid walls may be considered by the Contractor (for example: tubular piles driven through the caisson, high-density caisson filling, ground anchors, etc.) , as long as sufficient passive resistance is mobilized.
    - Regarding the transportation of the caisson, the floatability of the structure needs to be taken into consideration in the next design stage.
- For section 2 (Ferry quay wal) and section 4 (Inner harbour wall) anchored sheetpile walls were selected. Instead of an anchored sheetpile wall, alternative retaining walls may be considered by the Contractor (for example: L-shaped walls, blocks walls, etc).
- In the design calculations of section 2 (Ferry quay wall) and section 4 (Inner harbour wall), heavy sheet pile walls AZ26-700 (S240) were applied because of the estimated driving resistance. The structural capacity of the sheet pile wall is therefore higher than required. However, if thinner (and less stiff) sheet pile walls would be used, the risk of damage during installation is much higher, since the soil investigation identified the presence of gravel and cobbles.

5

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Appendices

APPENDIX: SOIL INVESTIGATIONS

Soil profiles across the new Black Rock harbour are realized according to [ref. 4] and [ref. 5].

<u>Legend</u>
The following four geotechnical units are presented:

RED Medium to very large size boulders consisting of reddish brown to grey andesite (red)
GREEN Medium to small size boulders, cobbles and gravels consisting of reddish brown to grey andesite
YELLOW Dark (reddish) brown and grey sand with small gravels and/or white coral particles
BLUE Greyish and off-white cobbles and pebbles consisting of coral limestone

Figure 5-1 Soil profile along transect A-A

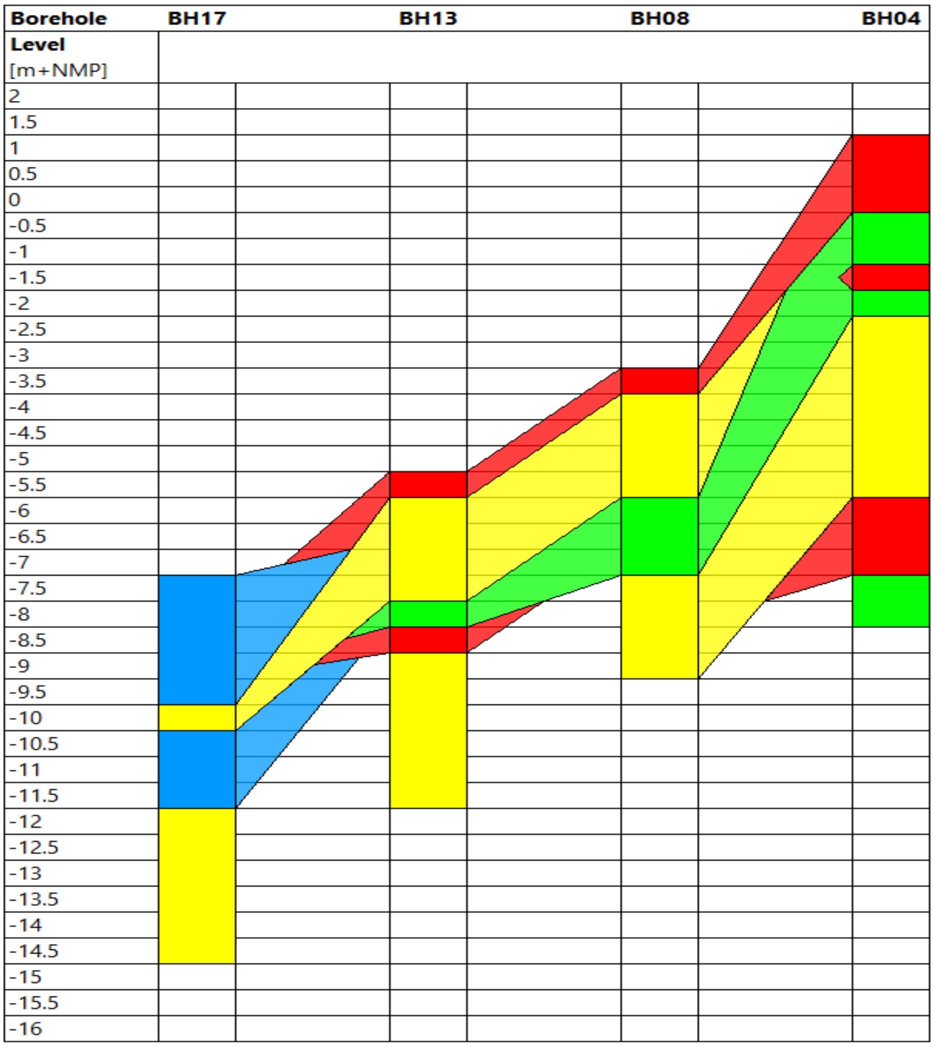


Figure 5-2 Soil profile along transect B-B

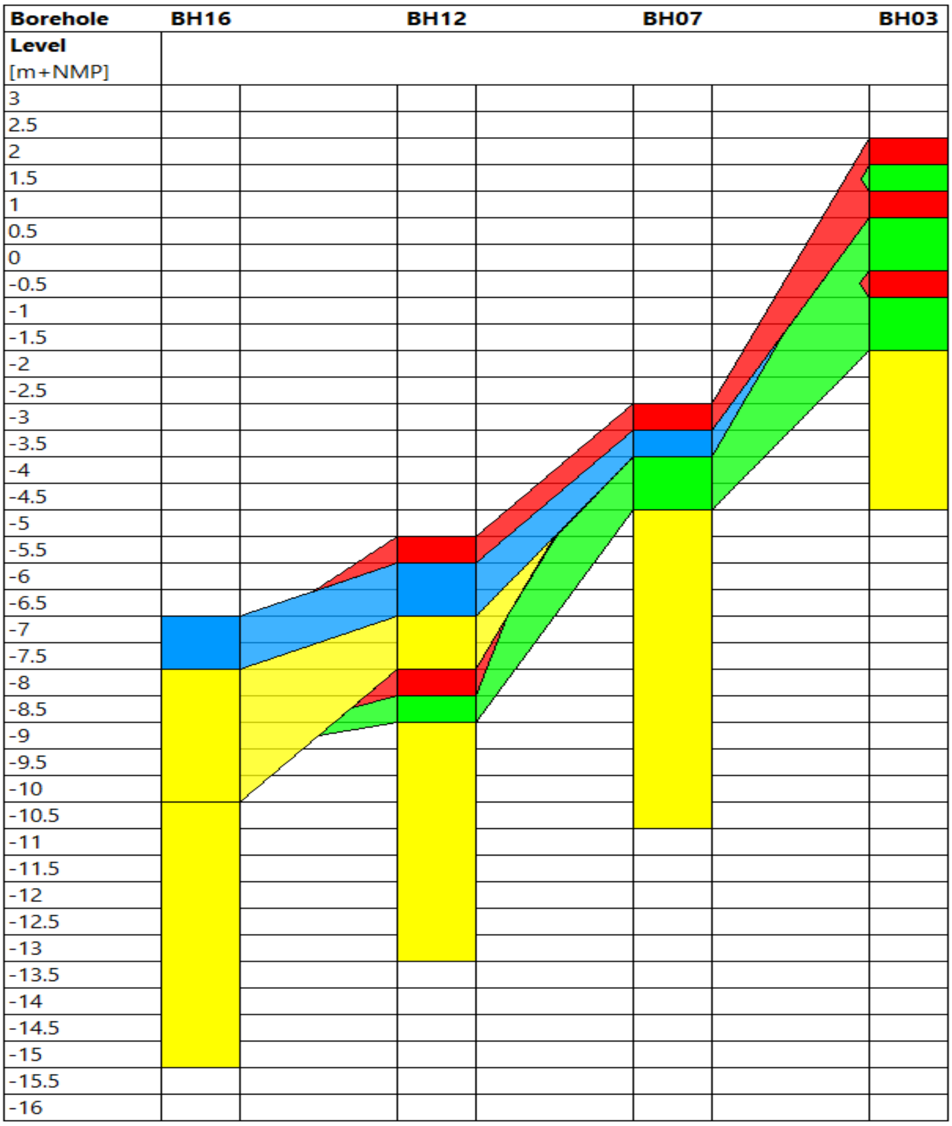


Figure 5-3 Soil profile along transect C-C

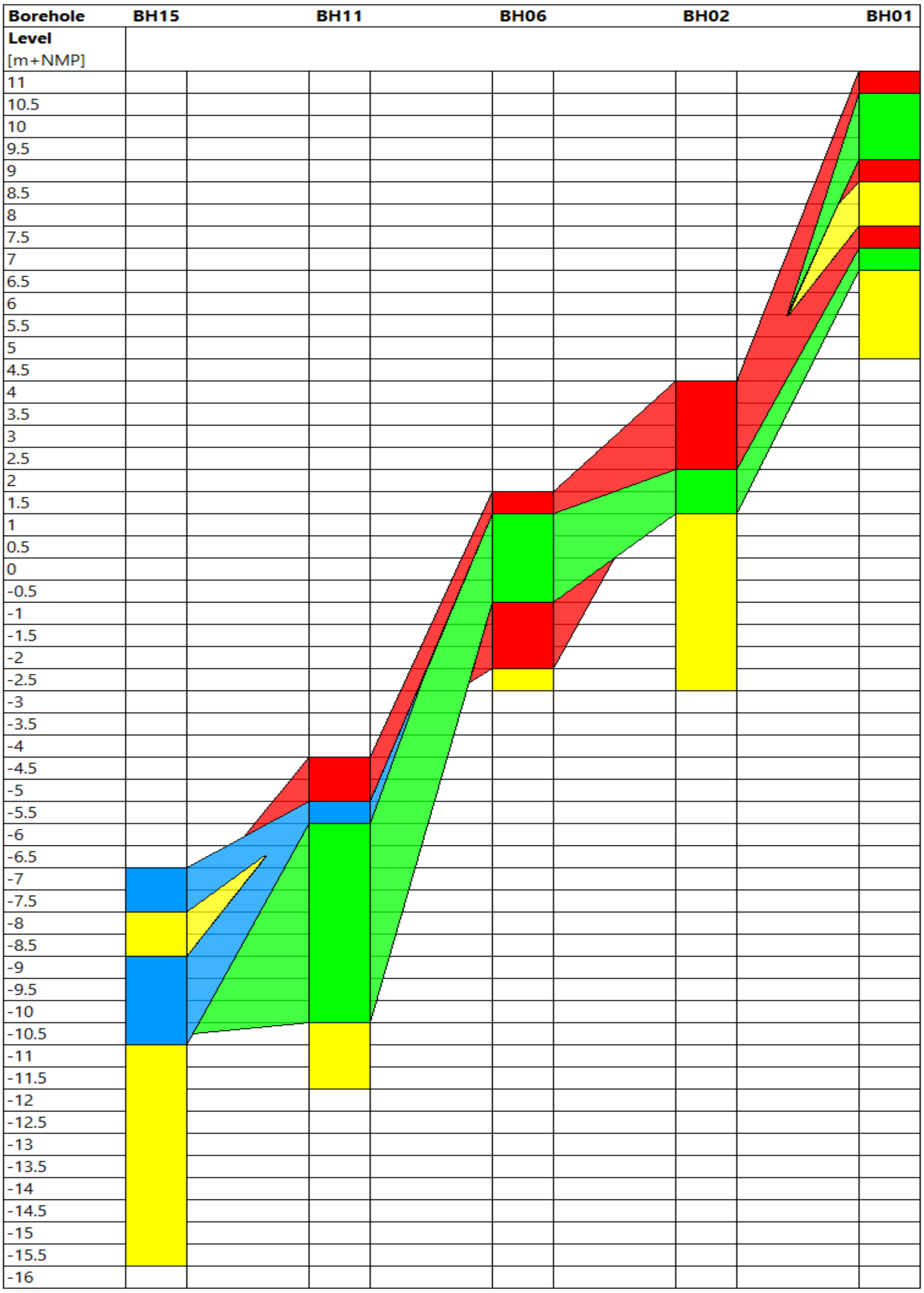
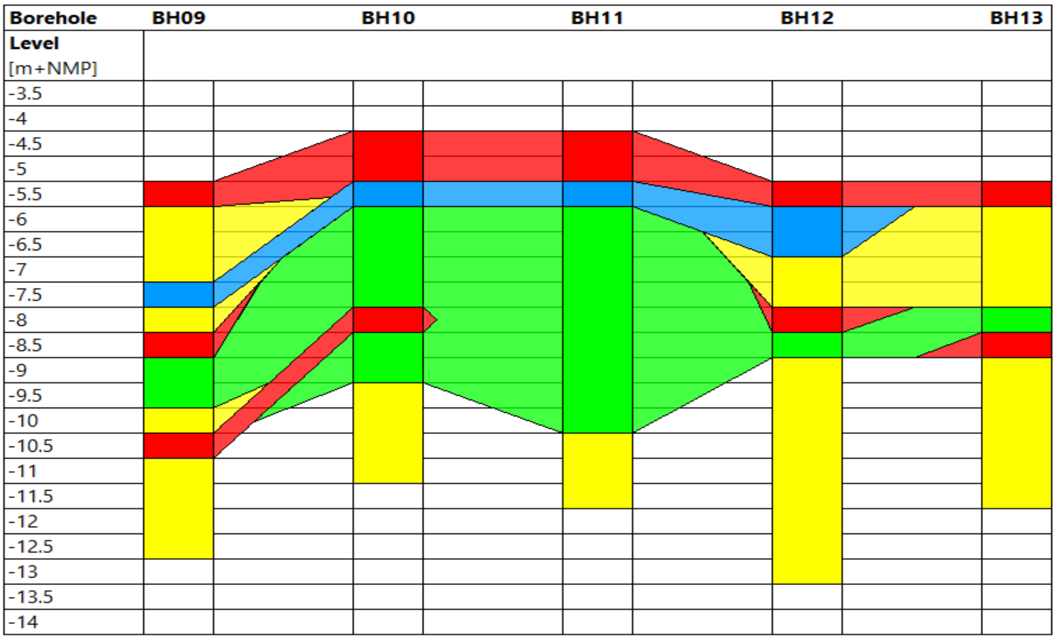


Figure 5-4 Soil profile along transect D-D



II

APPENDIX: LIQUEFACTION ASSESSMENT

LIQUEFACTION ASSESSMENT

*In accordance with the method of Boulanger and Idriss (2014)**

reference level	**SD**	
phreatic level	**0.07**	m+SD
γ_{soil} top layer	**20**	kN/m^3
γ_{soil} fill material	**20**	kN/m^3
γ_{water}	**10.10**	kN/m^3
atmospheric pressure	**0.1**	MPa
a_{max}	**0.275**	g
Mw	**7.5**	-

Borehole	**Surface level borehole**	**Depth DPL**			**Average depth DPL**	**Ref. depth DPL (average)**	**Static cone resistance q_c**	**Surface level design**	**σ_w**	**σ_{v1}**	**σ_{v1}'**	**σ_{v2}**	**σ_{v2}'**	**r_d**	**CSR**	**C_N**	**q_{c1N}**	**K_σ**	**MSF**	**CRR**	**FS**
[-]	[m+SD]	[m]			[m]	[m+SD]	[MPa]	[m+SD]	[kPa]	[kPa]	[kPa]	[kPa]	[kPa]	[-]	[-]	[-]	[-]	[-]	[-]	[-]	[-]
BH1	10.70	1.50	-	1.70	1.60	9.10	10	**1.57**	0.0	32.0	32.0	--	--	1.00	--	1.70	--	--	1.00	--	--
BH1	10.70	5.50	-	5.70	5.60	5.10	15	**1.57**	0.0	112.0	112.0	--	--	1.00	--	0.94	--	--	1.00	--	--
BH2	4.00	7.00	-	7.30	7.15	-3.15	-	**1.57**	32.5	143.0	110.5	94.4	61.9	0.96	0.26	0.95	--	--	1.00	--	--
BH3	2.10	7.10	-	7.60	7.35	-5.25	9	**1.57**	53.8	147.0	93.2	136.4	82.6	0.94	0.28	1.04	93.2	1.02	1.00	0.13	0.48
BH6	1.60	4.50	-	4.70	4.60	-3.00	13	**3.07**	31.0	92.0	61.0	121.4	90.4	0.95	0.23	1.28	166.5	1.02	1.00	0.37	1.68
BH6b	1.50	4.50	-	6.60	5.55	-4.05	13	**3.07**	41.6	111.0	69.4	142.4	100.8	0.94	0.24	1.20	156.1	1.00	1.00	0.30	1.28
BH11	-4.60	3.00	-	3.10	3.05	-7.65	23	**1.57**	78.0	108.2	30.2	184.4	106.4	0.91	0.28	1.70	391.0	1.02	1.00	> 1.00	> 1.00
BH13	-5.30	1.50	-	1.70	1.60	-6.90	7	**1.57**	70.4	86.3	15.8	169.4	99.0	0.92	0.28	1.70	119.0	1.00	1.00	0.18	0.63
BH15	-6.80	1.50	-	1.60	1.55	-8.35	7	**1.57**	85.1	100.4	15.3	198.4	113.3	0.90	0.28	1.70	119.0	0.98	1.00	0.18	0.62
BH16	-7.00	1.50	-	1.60	1.55	-8.55	5	**1.57**	87.1	102.4	15.3	202.4	115.3	0.89	0.28	1.70	85.0	0.99	1.00	0.12	0.42

**) Boulanger, R.W. & Idriss, I.M. (2014). CPT and SPT based liquefaction triggering procedures. April 2014, Report No. UCD/CGM-14/01.*

III

APPENDIX: DESIGN ROCK SLOPE

2.1 Slope stabiltiy - Van Gent

Project name:	Saba Black Rocks
Project code:	118421
Date:	29-6-2021 10:25

Intern data

Reference document:	CIRIA C683, "The rock manual (2nd edition), 2007, box 5.16, p. 578
Prepared by:	Peter v. To 16-8-2010
Checked by:	Bert van de 29-6-2021
Modification list:	

Name	Date	Modification

Equations

$$\frac{H_{sc}}{\Delta D_{n50}} = 1.75\sqrt{\cot\alpha}\left(1+\frac{D_{n;50-core}}{D_{n50}}\right)^{2/3}\left(\frac{S_d}{\sqrt{N}}\right)^{0.2}$$

Where:

$D_{n50-core}$ Median nominal stone size core (in case of wide grading, use NLL)

Restrictions:

Symbol	Value	Unity	Description
h	< $3*H_{s-toe}$	[m]	Water depth at toe, else use Van der Meer for deep water

Figures and graphs

Data

Input parameter	**Sd**	Damage level
Output parameter	Dn50	Nominal stone diameter

Symbol	Value	Unity	Description	Remark
H_s	**3,47**	[m]	Significant wave height	,55 * waterdepth for irregular waves
N	**3000**	[-]	Number of waves	Maximum number for this formulae
ρ_{water}	**1025**	[kg/m³]	Density of water	
ρ_{rock}	**2650**	[kg/m³]	Density of rock	
Δ	1,59	[-]	Relative density	Available densities vary. Sensitivity calculations at bottom
S_d	**3**	[-]	Damage level	Start of damage (2) - Intermediate damage (4-6)
$D_{n50-core}$	**0,65**	[-]	Median nominal stone size core (in case of wide grading, use NLL)	0.3-1.0T is assumed. Also a granular core/filter is assumed, no geotextile.
$\cot(\alpha)$	**2**	[-]	Slope of the breakwater, given as 1:m	Assumption

Result

Symbol	Value	Unity	Description
S_d	3,00	[m]	Nominal stone diameter
D_{n50}	1,18	[m]	Nominal stone diameter
Grading	3000-6000 kg		Associated grading

Data / Result (Table)

Input parameter	**Sd**	Damage level
Output parameter	Dn50	Nominal stone diameter

Symbol	Value	Unity	Description				
ρ_{water}	**1025**	**1025**	**1025**	**1025**	**1025**	**1025**	**1025**	**1025**
ρ_{rock}	**2650**	**2600**	**2550**	**2500**	**2500**	**2650**	**2500**	**2500**
Δ	1,59	1,54	1,49	1,44	1,44	1,59	1,44	1,44
H_s	3,465	3,465	3,465	3,465	3,465	3,465	3,465	3,465
N	3000	3000	3000	3000	3000	3000	3000	3000
Sd	3	4	4	5	3	2	2	3
$D_{n50-core}$	0,65	0,65	0,65	0,65	0,65	0,65	0,65	0,93
$\cot(\alpha)$	2	2	2	2	2	2	2	2
Dn50	1,18	1,14	1,19	1,17	1,34	1,31	1,48	1,18
Grading	3000-6000 kg	3000-6000 kg	3000-6000 kg	3000-6000 kg	6000-10000 k	6000-10000 k	Out of range	3000-6000 kg

IV

APPENDIX: SCOUR DEVELOPMENT CAISSON BREAKWATER

V

APPENDIX: SEISMIC SOIL PRESSURE COEFFICIENTS

SPREADSHEET
SEISMIC SOIL PRESSURE ON RETAINING WALLS
version 0.30 **[not validated]**

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 2 (RoRo quay wall)**
Consultant	**L.Jorissen**
Date	**25-5-2021**

TITLE

Calculation of:
- seismic soil pressure coefficients in accordance with Mononobe-Okabe for cohesionless soils;
- seismic soil pressure in accordance with Japanese Technical Standard;
- equivalent strength parameters for pseudostatic calculations of retaining walls.

STARTING POINTS

general

language		=	**EN**	
reference level		=	**CD**	
unit weight water	γ_w	=	**10**	kN/m^3

geometry

type retaining structure		=	**gravity**		
			active	passive	
inclination angle of back of the wall	ψ	=	**0.0**	**0.0**	°
		=	0.00	0.00	rad
inclination angle of backfill surface	β	=	**0.0**	**0.0**	°
		=	0.00	0.00	rad
surface level		=	**2.50**	**2.50**	m+CD
water level	gwl	=	**0.07**	**0.07**	m+CD
surcharge load	ω	=	**6**	**6**	kN/m^2

seismic

			level 1	level 2	
peak ground acceleration (surface level)	PGA	=	**0.175**	**0.275**	g
equivalent seismic coefficient	k_e	=	0.175	0.217	g
factor for average along structure height		=	**0.500**	**0.500**	-
horizontal seismic coefficient	k_h	=	0.088	0.108	g
vertical seismic coefficient	k_v	=	0.029	0.036	g

partial factors	$\gamma_{\varphi'}$ [-]	$\gamma_{c'}$ [-]	γ_{su} [-]	$\gamma_{\delta;a}$ [-]	$\gamma_{\delta;p}$ [-]
cohesionless soil	**1.00**	n.a.	n.a.	**1.00**	**1.00**
cohesive soil	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

soil layers and characteristic geotechnical properties

# [-]	soil type [-]	top [m+CD]	bottom [m+CD]	h [m]	γ_{unsat} [kN/m^3]	γ_{sat} [kN/m^3]	φ'_k [°]	c'_k [kN/m^2]	$s_{u;k}$ [kN/m^2]	$\delta_{a;k}$ [°]	$\delta_{p;k}$ [°]
1	**Sand supplemented**	2.50	**1.58**	0.92	**18.0**	**21.0**	**37.50**	**0.00**	**0.00**	25.00	18.75
2	**Bolder-gravel-cobbel**	1.58	**1.00**	0.58	**18.0**	**20.0**	**37.50**	**0.00**	**0.00**	25.00	18.75
3	**Sand dense**	1.00	**0.07**	0.93	**18.0**	**20.0**	**35.00**	**0.00**	**0.00**	23.33	17.50
4	**Sand dense**	0.07	**-1.00**	1.07	**18.0**	**20.0**	**35.00**	**0.00**	**0.00**	23.33	17.50
5	**Bolder-gravel-cobbel**	-1.00	**-2.50**	1.50	**18.0**	**20.0**	**37.50**	**0.00**	**0.00**	25.00	18.75
6	**Sand medium dense**	-2.50	**-20.00**	17.50	**18.0**	**20.0**	**33.50**	**0.00**	**0.00**	22.33	16.75
7											
8											
9											
10											
11											
12											

detailed calculation options

choose layer for detailed calculation [1 - 6]	=	**6**
soil type chosen layer	=	cohesionless

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 2 (RoRo quay wall)**
Consultant	**L.Jorissen**
Date	**25-5-2021**

CALCULATIONS

apparent seismic coefficients and dynamic angles for layer 6

	active					passive				
# [-]	soil state	g_{tot} [kN/m^3]	g_{eff} [kN/m^3]	h · g_{tot} [kN/m^3]	h · g_{eff} [kN/m^2]	soil state [-]	g_{tot} [kN/m^3]	g_{eff} [kN/m^3]	h · g_{tot} [kN/m^2]	h · g_{eff} [kN/m^2]
1	dry	18.0	18.0	17	17	dry	18.0	18.0	17	17
2	dry	18.0	18.0	10	10	dry	18.0	18.0	10	10
3	dry	18.0	18.0	17	17	dry	18.0	18.0	17	17
4	saturated	20.0	10.0	21	11	saturated	20.0	10.0	21	11
5	saturated	20.0	10.0	30	15	saturated	20.0	10.0	30	15
6	saturated	20.0	10.0	350	175	saturated	20.0	10.0	350	175
7										
8										
9										
10										
11										
12										

			active	passive	
factor for apperent seismic coefficient		=	1.69	1.69	-

			level 1			level 2		
			active	passive		active	passive	
apparent horizontal seismic coefficient	k'_h	=	0.148	0.148	g	0.184	0.184	g
apparent vertical seismic coefficient	k'_v	=	0.049	0.049	g	0.061	0.061	g
dynamic angle (+)	θ^+	=	8.0	8.0	°	9.8	9.8	°
		=	0.14	0.14	rad	0.17	0.17	rad
dynamic angle (−)	θ^-	=	8.9	8.9	°	11.1	11.1	°
		=	0.15	0.15	rad	0.19	0.19	rad

COHESIONLESS SOIL - MONONOBE-OKABE

geotechnical properties for layer 6

characteristic angle of internal friction	j'_k	=	33.50	°
partial factor angle of internal friction	$g_{j'}$	=	1.00	-
design value angle of internal friction	j'_d	=	33.50	°
		=	0.58	rad

			active	passive	
characteristic angle of wall friction	d'_k	=	22.33	16.75	°
partial factor angle of wall friction	$g_{d'}$	=	1.00	1.00	-
design value angle of wall friction	d'_d	=	22.33	16.75	°
		=	0.390	0.292	rad

earth pressure coefficients for layer 6

			level 1			level 2		
			active	passive		active	passive	
total earth pressure coefficient (+)	K^+	=	0.354	5.764	-	0.380	5.595	-
total earth pressure coefficient (−)	K^-	=	0.366	5.686	-	0.400	5.474	-
decisive total earth pressure coefficient	**K**	=	**0.366**	**5.686**	-	**0.400**	**5.474**	-
horizontal earth pressure coefficient (+)	K_h^+	=	0.305	5.233	-	0.322	5.004	-
horizontal earth pressure coefficient (−)	K_h^-	=	0.313	5.127	-	0.334	4.841	-
decisive horizontal earth pressure coefficient	$\mathbf{K_h}$	=	**0.313**	**5.127**	-	**0.334**	**4.841**	-
vertical earth pressure coefficient (+)	K_v^+	=	0.179	2.417	-	0.202	2.503	-
vertical earth pressure coefficient (−)	K_v^-	=	0.189	2.458	-	0.220	2.555	-
decisive vertical earth pressure coefficient	$\mathbf{K_v}$	=	**0.189**	**2.417**	-	**0.220**	**2.503**	-

equivalent strength parameters

			level 1			level 2		
			active	passive		active	passive	
equivalent angle of internal friction	j'_{eq}	=	27.30	30.90	°	25.80	30.10	°
equivalent effective cohesion	c'_{eq}	=	0.00	0.00	kN/m^2	0.00	0.00	kN/m^2

SPREADSHEET
SEISMIC SOIL PRESSURE ON RETAINING WALLS
version 0.30 **[not validated]**

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 2 (RoRo quay wall)**
Consultant	**L.Jorissen**
Date	**25-5-2021**

SUMMARY

				active				passive			
#	soil type	top	bottom	$k_{h;a;1}$	$K_{h;a;1}$	$j'_{eq;a;1}$	$c'_{eq;a;1}$	$k_{h;p;1}$	$K_{h;p;1}$	$j'_{eq;p;1}$	$c'_{eq;p;1}$
[-]	[-]	[m+CD]	[m+CD]	[g]	[-]	[°]	[kN/m²]	[g]	[-]	[°]	[kN/m²]
1	Sand supplemented	2.50	1.58	0.088	0.24	33.80	0.00	0.088	7.84	36.30	0.00
2	Bolder-gravel-cobbel	1.58	1.00	0.088	0.24	33.80	0.00	0.088	7.84	36.30	0.00
3	Sand dense	1.00	0.07	0.088	0.26	31.30	0.00	0.088	6.31	33.60	0.00
4	Sand dense	0.07	-1.00	0.096	0.27	31.00	0.00	0.096	6.24	33.50	0.00
5	Bolder-gravel-cobbel	-1.00	-2.50	0.111	0.25	32.80	0.00	0.111	7.59	35.90	0.00
6	Sand medium dense	-2.50	-20.00	0.148	0.31	27.30	0.00	0.148	5.13	30.90	0.00
7											
8											
9											
10											
11											
12											

Table 1: Seismic soil pressure coefficients and equivalent strength parameters per layer (Level 1)

				active				passive			
#	soil type	top	bottom	$k_{h;a;2}$	$K_{h;a;2}$	$j'_{eq;a;2}$	$c'_{eq;a;2}$	$k_{h;p;2}$	$K_{h;p;2}$	$j'_{eq;p;2}$	$c'_{eq;p;2}$
[-]	[-]	[m+0.175]	[m+0.175]	[g]	[-]	[°]	[kN/m²]	[g]	[-]	[°]	[kN/m²]
1	Sand supplemented	2.50	1.58	0.108	0.246	32.90	0.00	0.108	7.616	35.90	0.00
2	Bolder-gravel-cobbel	1.58	1.00	0.108	0.246	32.90	0.00	0.108	7.616	35.90	0.00
3	Sand dense	1.00	0.07	0.108	0.273	30.50	0.00	0.108	6.136	33.30	0.00
4	Sand dense	0.07	-1.00	0.119	0.279	30.00	0.00	0.119	6.045	33.10	0.00
5	Bolder-gravel-cobbel	-1.00	-2.50	0.137	0.260	31.60	0.00	0.137	7.304	35.40	0.00
6	Sand medium dense	-2.50	-20.00	0.184	0.334	25.80	0.00	0.184	4.841	30.10	0.00
7											
8											
9											
10											
11											
12											

Table 2: Seismic soil pressure coefficients and equivalent strength parameters per layer (Level 2)

* At active side in cohesive soils, $K_{h;a}$ is calculated as the ratio of Dp_a/Ds'_v. $K_{h;a}$ is always ≥ 1, therefore $j'_{eq;a}$ is taken as 0°.
There is a shift of active earth pressure p_a due to $c'_{eq;a}$ with a value of $K_{h;a;c} \cdot c'_{eq;a}$, where $K_{h;p;c}$ = -2.

** At passive side in cohesive soils, strength consists of only the undrained shear strength s_u. Friction angle j' is 0, and therefore $K_{h;p}$ is per definition 1.
There is a shift of passive earth pressure p_p due to s_u with a value of $K_{h;p;c} \cdot s_u$, where $K_{h;p;c}$ = -2.

SPREADSHEET
SEISMIC SOIL PRESSURE ON RETAINING WALLS
version 0.30 **[not validated]**

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 4 (Inner harbour wall)**
Consultant	**L.Jorissen**
Date	**27-5-2021**

TITLE

Calculation of:
- seismic soil pressure coefficients in accordance with Mononobe-Okabe for cohesionless soils;
- seismic soil pressure in accordance with Japanese Technical Standard;
- equivalent strength parameters for pseudostatic calculations of retaining walls.

STARTING POINTS

general

language		=	**EN**	
reference level		=	**CD**	
unit weight water	γ_w	=	**10**	kN/m^3

geometry

type retaining structure		=	**gravity**		
			active	passive	
inclination angle of back of the wall	ψ	=	**0.0**	**0.0**	°
		=	0.00	0.00	rad
inclination angle of backfill surface	β	=	**0.0**	**0.0**	°
		=	0.00	0.00	rad
surface level		=	**2.00**	**2.00**	m+CD
water level	gwl	=	**0.07**	**0.07**	m+CD
surcharge load	ω	=	**3**	**3**	kN/m^2

seismic

			level 1	level 2	
peak ground acceleration (surface level)	PGA	=	**0.175**	**0.275**	g
equivalent seismic coefficient	k_e	=	0.175	0.217	g
factor for average along structure height		=	**0.500**	**0.500**	-
horizontal seismic coefficient	k_h	=	0.088	0.108	g
vertical seismic coefficient	k_v	=	0.029	0.036	g

partial factors	$\gamma_{\varphi'}$ [-]	$\gamma_{c'}$ [-]	γ_{su} [-]	$\gamma_{\delta;a}$ [-]	$\gamma_{\delta;p}$ [-]
cohesionless soil	**1.00**	n.a.	n.a.	**1.00**	**1.00**
cohesive soil	**1.00**	**1.00**	**1.00**	**1.00**	**1.00**

soil layers and characteristic geotechnical properties

# [-]	soil type [-]	top [m+CD]	bottom [m+CD]	h [m]	γ_{unsat} [kN/m^3]	γ_{sat} [kN/m^3]	φ'_k [°]	c'_k [kN/m^2]	$s_{u;k}$ [kN/m^2]	$\delta_{a;k}$ [°]	$\delta_{p;k}$ [°]
1	**Bolder-gravel-cobbel**	2.00	**1.50**	0.50	**18.0**	**20.0**	**37.50**	**0.00**	**0.00**	25.00	18.75
2	**Sand dense**	1.50	**1.00**	0.50	**18.0**	**20.0**	**35.00**	**0.00**	**0.00**	23.33	17.50
3	**Bolder-gravel-cobbel**	1.00	**0.50**	0.50	**18.0**	**20.0**	**37.50**	**0.00**	**0.00**	25.00	18.75
4	**Sand dense**	0.50	**0.07**	0.43	**18.0**	**20.0**	**35.00**	**0.00**	**0.00**	23.33	17.50
5	**Sand dense**	0.07	**-0.50**	0.57	**18.0**	**20.0**	**35.00**	**0.00**	**0.00**	23.33	17.50
6	**Bolder-gravel-cobbel**	-0.50	**-1.00**	0.50	**18.0**	**20.0**	**37.50**	**0.00**	**0.00**	25.00	18.75
7	**Sand dense**	-1.00	**-2.00**	1.00	**18.0**	**20.0**	**35.00**	**0.00**	**0.00**	23.33	17.50
8	**Sand medium dense**	-2.00	**-20.00**	18.00	**18.0**	**20.0**	**33.50**	**0.00**	**0.00**	22.33	16.75
9											
10											
11											
12											

detailed calculation options

choose layer for detailed calculation [1 - 8]	=	**6**
soil type chosen layer	=	cohesionless

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 4 (Inner harbour wall)**
Consultant	**L.Jorissen**
Date	**27-5-2021**

CALCULATIONS

apparent seismic coefficients and dynamic angles for layer 6

#	active					passive				
	soil state	g_{tot}	g_{eff}	$h \cdot g_{tot}$	$h \cdot g_{eff}$	soil state	g_{tot}	g_{eff}	$h \cdot g_{tot}$	$h \cdot g_{eff}$
[-]		[kN/m^3]	[kN/m^3]	[kN/m^3]	[kN/m^2]	[-]	[kN/m^3]	[kN/m^3]	[kN/m^2]	[kN/m^2]
1	dry	18.0	18.0	9	9	dry	18.0	18.0	9	9
2	dry	18.0	18.0	9	9	dry	18.0	18.0	9	9
3	dry	18.0	18.0	9	9	dry	18.0	18.0	9	9
4	dry	18.0	18.0	8	8	dry	18.0	18.0	8	8
5	saturated	20.0	10.0	11	6	saturated	20.0	10.0	11	6
6	saturated	20.0	10.0	10	5	saturated	20.0	10.0	10	5
7										
8										
9										
10										
11										
12										

			active	passive	
factor for apperent seismic coefficient		=	1.18	1.18	-

			level 1 active	level 1 passive		level 2 active	level 2 passive	
apparent horizontal seismic coefficient	k'_h	=	0.103	0.103	g	0.128	0.128	g
apparent vertical seismic coefficient	k'_v	=	0.034	0.034	g	0.043	0.043	g
dynamic angle (+)	θ^+	=	5.7	5.7	°	7.0	7.0	°
		=	0.10	0.10	rad	0.12	0.12	rad
dynamic angle (−)	θ^-	=	6.1	6.1	°	7.6	7.6	°
		=	0.11	0.11	rad	0.13	0.13	rad

COHESIONLESS SOIL - MONONOBE-OKABE

geotechnical properties for layer 6

characteristic angle of internal friction	j'_k	=	37.50	°
partial factor angle of internal friction	$g_{j'}$	=	1.00	-
design value angle of internal friction	j'_d	=	37.50	°
		=	0.65	rad

			active	passive	
characteristic angle of wall friction	d'_k	=	25.00	18.75	°
partial factor angle of wall friction	$g_{d'}$	=	1.00	1.00	-
design value angle of wall friction	d'_d	=	25.00	18.75	°
		=	0.436	0.327	rad

earth pressure coefficients for layer 6

			level 1 active	level 1 passive		level 2 active	level 2 passive	
total earth pressure coefficient (+)	K^+	=	0.280	8.504	-	0.295	8.344	-
total earth pressure coefficient (−)	K^-	=	0.284	8.454	-	0.303	8.268	-
decisive total earth pressure coefficient	**K**	=	**0.284**	**8.454**	-	**0.303**	**8.268**	-
horizontal earth pressure coefficient (+)	K_h^+	=	0.240	7.742	-	0.250	7.517	-
horizontal earth pressure coefficient (−)	K_h^-	=	0.243	7.672	-	0.255	7.409	-
decisive horizontal earth pressure coefficient	$\mathbf{K_h}$	=	**0.243**	**7.672**	-	**0.255**	**7.409**	-
vertical earth pressure coefficient (+)	K_v^+	=	0.143	3.519	-	0.156	3.623	-
vertical earth pressure coefficient (−)	K_v^-	=	0.147	3.552	-	0.163	3.670	-
decisive vertical earth pressure coefficient	$\mathbf{K_v}$	=	**0.147**	**3.519**	-	**0.163**	**3.623**	-

equivalent strength parameters

			level 1 active	level 1 passive		level 2 active	level 2 passive	
equivalent angle of internal friction	j'_{eq}	=	33.10	36.00	°	32.10	35.60	°
equivalent effective cohesion	c'_{eq}	=	0.00	0.00	kN/m^2	0.00	0.00	kN/m^2

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 4 (Inner harbour wall)**
Consultant	**L.Jorissen**
Date	**27-5-2021**

SUMMARY

#	soil type	top	bottom	active				passive			
				$k_{h;a;1}$	$K_{h;a;1}$	$j'_{eq;a;1}$	$c'_{eq;a;1}$	$k_{h;p;1}$	$K_{h;p;1}$	$j'_{eq;p;1}$	$c'_{eq;p;1}$
[-]	[-]	[m+CD]	[m+CD]	[g]	[-]	[°]	[kN/m^2]	[g]	[-]	[°]	[kN/m^2]
1	Bolder-gravel-cobbel	2.00	1.50	0.088	0.236	33.80	0.00	0.088	7.836	36.30	0.00
2	Sand dense	1.50	1.00	0.088	0.263	31.30	0.00	0.088	6.314	33.60	0.00
3	Bolder-gravel-cobbel	1.00	0.50	0.088	0.236	33.80	0.00	0.088	7.836	36.30	0.00
4	Sand dense	0.50	0.07	0.088	0.263	31.30	0.00	0.088	6.314	33.60	0.00
5	Sand dense	0.07	-0.50	0.094	0.266	31.10	0.00	0.094	6.262	33.50	0.00
6	Bolder-gravel-cobbel	-0.50	-1.00	0.103	0.243	33.10	0.00	0.103	7.672	36.00	0.00
7	Sand dense	-1.00	-2.00	0.113	0.276	30.30	0.00	0.113	6.094	33.20	0.00
8	Sand medium dense	-2.00	-20.00	0.153	0.315	27.10	0.00	0.153	5.092	30.80	0.00
9											
10											
11											
12											

Table 1: Seismic soil pressure coefficients and equivalent strength parameters per layer (Level 1)

#	soil type	top	bottom	active				passive			
				$k_{h;a;2}$	$K_{h;a;2}$	$j'_{eq;a;2}$	$c'_{eq;a;2}$	$k_{h;p;2}$	$K_{h;p;2}$	$j'_{eq;p;2}$	$c'_{eq;p;2}$
[-]	[-]	[m+0.175]	[m+0.175]	[g]	[-]	[°]	[kN/m^2]	[g]	[-]	[°]	[kN/m^2]
1	Bolder-gravel-cobbel	2.00	1.50	0.108	0.246	32.90	0.00	0.108	7.616	35.90	0.00
2	Sand dense	1.50	1.00	0.108	0.273	30.50	0.00	0.108	6.136	33.30	0.00
3	Bolder-gravel-cobbel	1.00	0.50	0.108	0.246	32.90	0.00	0.108	7.616	35.90	0.00
4	Sand dense	0.50	0.07	0.108	0.273	30.50	0.00	0.108	6.136	33.30	0.00
5	Sand dense	0.07	-0.50	0.116	0.277	30.10	0.00	0.116	6.070	33.10	0.00
6	Bolder-gravel-cobbel	-0.50	-1.00	0.128	0.255	32.10	0.00	0.128	7.409	35.60	0.00
7	Sand dense	-1.00	-2.00	0.140	0.290	29.10	0.00	0.140	5.858	32.70	0.00
8	Sand medium dense	-2.00	-20.00	0.189	0.337	25.50	0.00	0.189	4.796	29.90	0.00
9											
10											
11											
12											

Table 2: Seismic soil pressure coefficients and equivalent strength parameters per layer (Level 2)

* At active side in cohesive soils, $K_{h;a}$ is calculated as the ratio of Dp_a/Ds'_v. $K_{h;a}$ is always ≥ 1, therefore $j'_{eq;a}$ is taken as 0°.
There is a shift of active earth pressure p_a due to $c'_{eq;a}$ with a value of $K_{h;a;c} \cdot c'_{eq;a}$, where $K_{h;p;c}$ = -2.

** At passive side in cohesive soils, strength consists of only the undrained shear strength s_u. Friction angle j' is 0, and therefore $K_{h;p}$ is per definition 1.
There is a shift of passive earth pressure p_p due to s_u with a value of $K_{h;p;c} \cdot s_u$, where $K_{h;p;c}$ = -2.

VI

APPENDIX: STARTING POINTS FOR THE DESIGN OF SECTION 7: FINGER PIERS

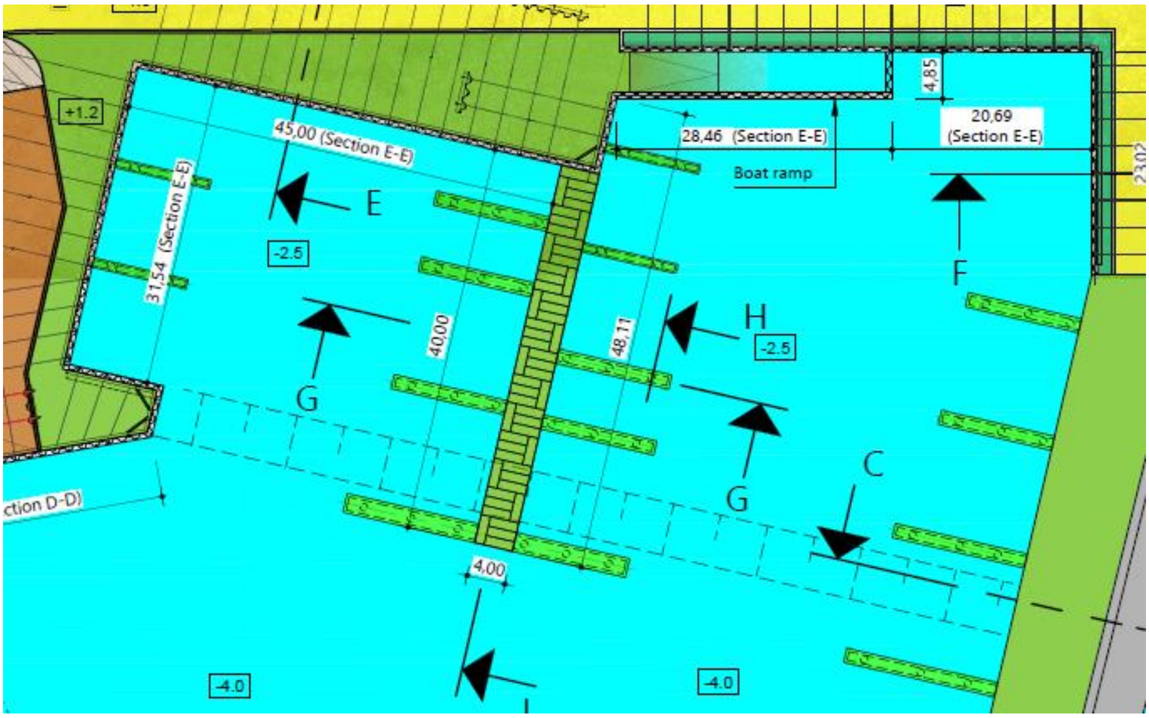
MEMORANDUM

Subject	Fingerpier Saba
Project	Saba
Client	
Project code	
Status	Draft version 01
Date	22 June 2021
Reference	118421/21-010.191
Author(s)	ing. D. Everling MSEng
Checked by	ir. B. Goeijenbier
Approved by	ir. A. van den Berg
Initials	
Appendices	I SCIA report II Structural checks

1 INTRODUCTION

Part of the Black Rock Marina design are several finger piers. In the picture below a plan view is shown with the finger piers on both sides of the central pier of the jetty. In this note the design of the finger piers will be reported as part of the reference design of the project.

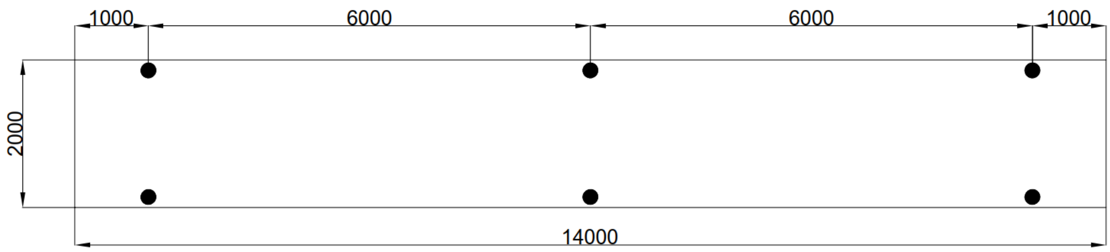
Figure 1.1 Top view jetty with finger piers



2 GEOMETRY FINGER PIER

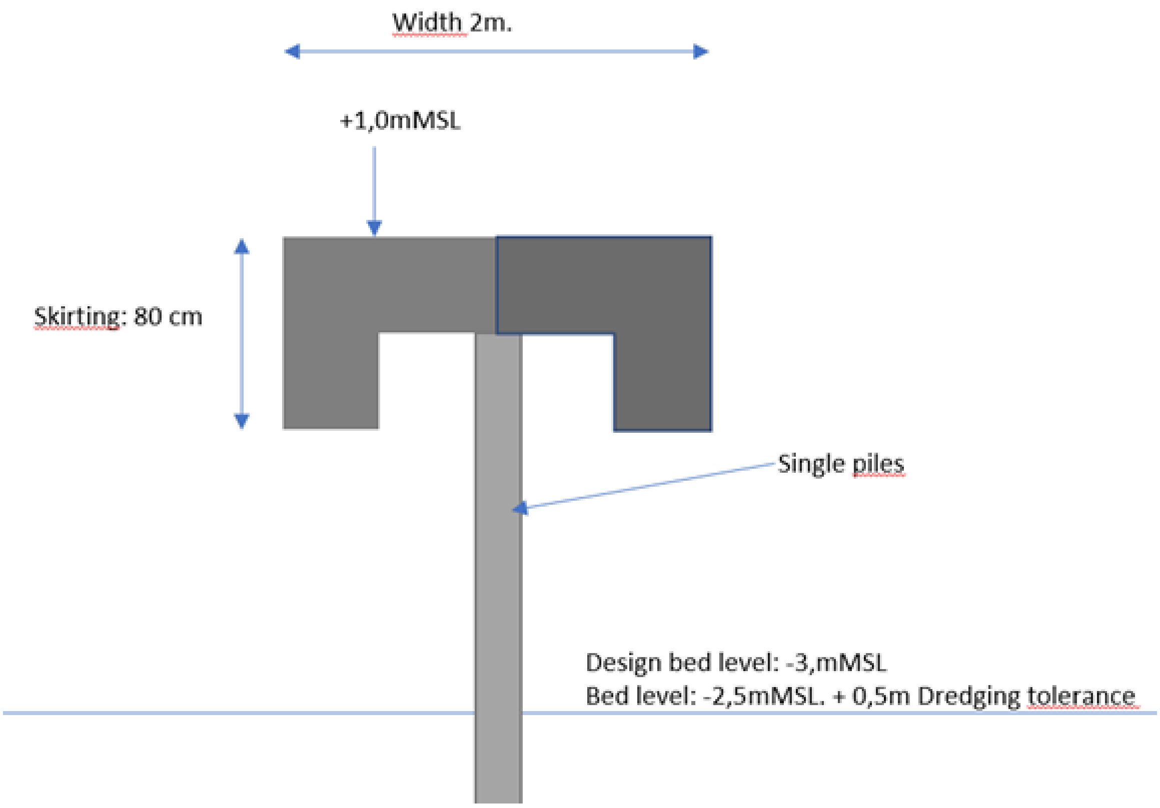
The length of the deck of the finger pier varies from 10 m till 16.0 m. The width differs from 2.0 m of the widest finger pier. 1.0 m of the smallest finger pier and there is also of finger pier with a deck wide 1.5 m. One typical section is designed in this memorandum for a finger pier width of 2m and a length of 14m. The deck is only accessible for pedestrians. On top of the deck 6 cleats are applied with a mooring load of 2.5 ton (25 kN) on each one. The distance between the cleats is 6.0 m on both sides of the pier deck. In the figure below a top view of the deck with the cleats is shown.

Figure 2.1 Top view deck finger pier with 6 cleats



The top side of the deck is at a level of +1.0 MSL. To avoid clashes between the concrete deck and the ships hull caused by swell. the skirts on both sides of the deck have a height of 0.80 m. The design sea bed level is -3.0 MSL including 0.50 m dredging tolerance.

Figure 2.2 Typical section



3 MATERIAL PROPERTIES

The deck of the finger pier is made of concrete. As safety approach concrete class C30/37 will be the starting point of this design and the reinforcement is class B500B. Based on NEN-EN1992-1-1 tabel 4.3N with a design life time of 100 years and environmental class XS3. structure class S6 will be applied. With the information in table 4.4N in NEN-EN1992-1-1 the minimum required concrete cover will be (50+5 =) 55 mm included Δc_{dev} of 5.0 mm. The applied concrete cover is 55 mm so the factor k_x is 1.0. The concrete plug inside the steel tube is enclosed with steel. so environmental class X0 is required. The applied concrete cover in the tube is 50 mm with a k_x-factor of 1.0 as safety approach.

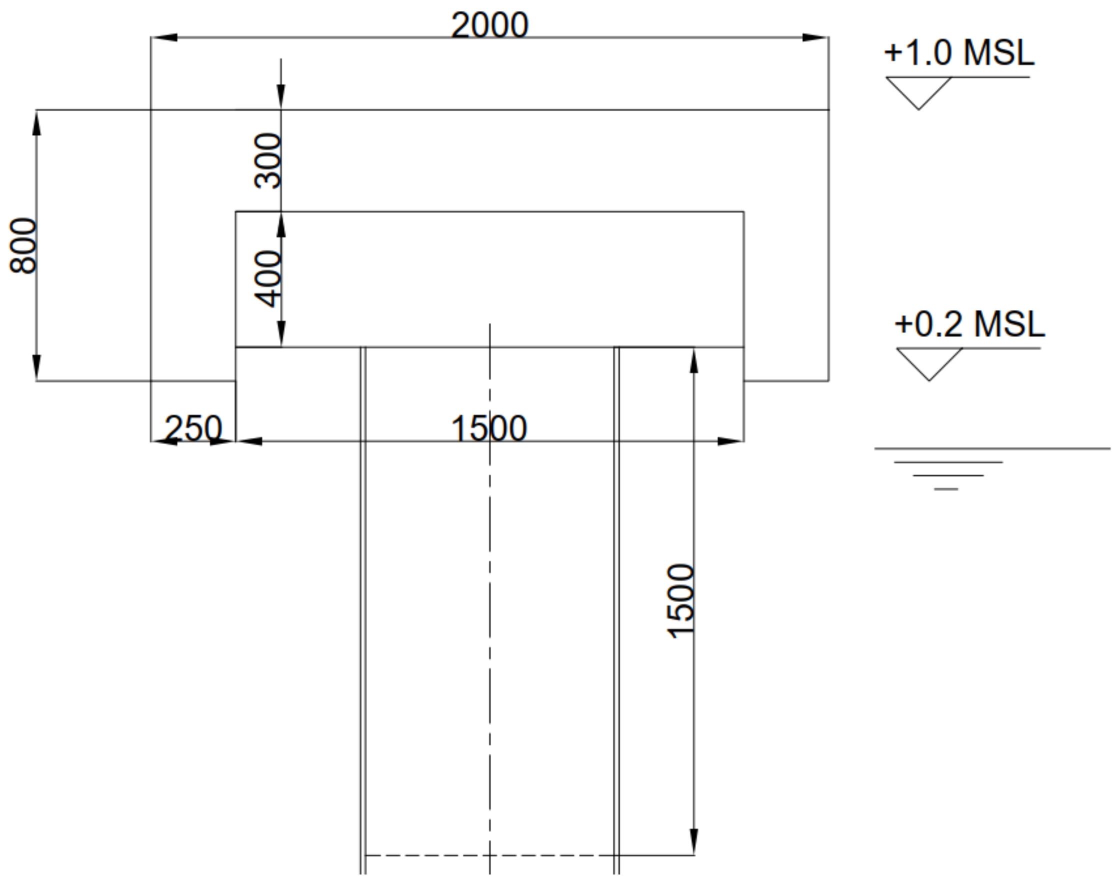
4 LOADS

Beside the self weight of the structure the wave loads are the primary loads in the ULS combinations. The loads from pedestrians have a minor effect and the mooring loads are only relevant in horizontal direction. In view of the self weight of the structure. the load in case of earth quakes is not governing against the other horizontal loads.

Self weight

The concrete decks are 0.3 m thick. both skirts are 0.5 m high and 0.25 m width. The line load from self weight of the deck with the maximum width of 2.0 m is (0.85*25 =) 21.25 kN/m. The concrete slab on top of the pile to support the deck is 1.3 m long. 1.5 m width and 0.4 m thick. the self weight is 19.5 kN. In the connection between pile head and deck a concrete plug long 1.5 m. diameter 0.70 m will be applied. the self weight of the plug is 14.5 kN/pile.

Figure 4.1 Typical section pile head - deck



Variable loads

In accordance with NEN-EN1991-2 art. 5.3.2.1 the loads from pedestrians will be 5.0 kN/m^2 on the whole deck. On top of the deck 6 pcs cleats will be applied with each a load of 25 kN. This load will be spreaded along the whole deck so a line load is taken into account of ((6*25) / 14 =) 10.7 kN/m.

Wave loads

The vertical wave load of 25.36 kN/m^2 will act upwards. The horizontal wave load of 7 kN/m acts on the side of the deck with the skirts.

The wave loads are determined following the method in ref. [McConnell. Kirsty_ Allsop. William_ Cruickshank. Ian-Piers. Jetties and Related Structures Exposed to Waves - Guidelines for Hydraulic Loadings-ICE Publishing (2004)]. Equation 5.19 and equation 5.21 of this guideline are applied. Input and output parameters are given in appendix

Load combinations

The partial factors of the load combinations will be derived from NEN-EN1990-0. The (extreme) wave loads and the other variable loads from pedestrians and cleats will not act together in the same combinations. Because of the extreme wave conditions. the wave load is not part of the SLS-combinations for the crack width control of the concrete. In accordance with CUR211 the ψ-factors for the mooring loads will be applied. As conservative approach in the load combinations all 6 cleats are activated together with a mooring load of 25 kN each. but multiplied with the ψ-factor of 0.70.

Table 4.1 Load combinations

load case	type	ULS		SLS
		formula 6.10a	formula 6.10b	formula 6.15
self weight	G	1.35	1.20	1.00
pedestrains	Q	1.50	1.50	1.00
mooring loads	Q	1.50	1.50	1.00
wave loads	Q	1.50	1.50	1.00*1

Table 4.2 load combinations

load case		ULS			SLS		
		variable loads	waves downwards	waves upwards	variable	waves*2	permanent
1	self weight	1.35	1.35	0.90	1.0	1.0	1.0
2	additional self weight	1.35	1.35	0.90	1.0	1.0	1.0
3	pedestrains	1.50	-	-	1.0	-	-
4	cleats	1.05*1	-	-	0.30	-	-
5	wave loads	-	1.50	1.50	-	1.0	-

*1) ψ_0-factor 0.70 and ψ_1 0.30 according to CUR211. Handboek Kademuren. 2013.

*2) the extreme wave conditions are not part of the SLS-combinations for crack width control.

4.2 Construction stages

The finger piers with a deck width of 1.0 m and 1.5 m are located landwards. In the construction stage these decks are located on the land side and after finishing the decks the harbour will be excavated. The decks from the land side will be construct with in-situ concrete poured on a temporary concrete floor on the ground level and connected to the piles.

The decks with a width of 2.0 m will be build above the water line. In view of the feasibility prefabricated concrete will be used for the deck of these finger piers. The construction stages are pointed out in the summary below:

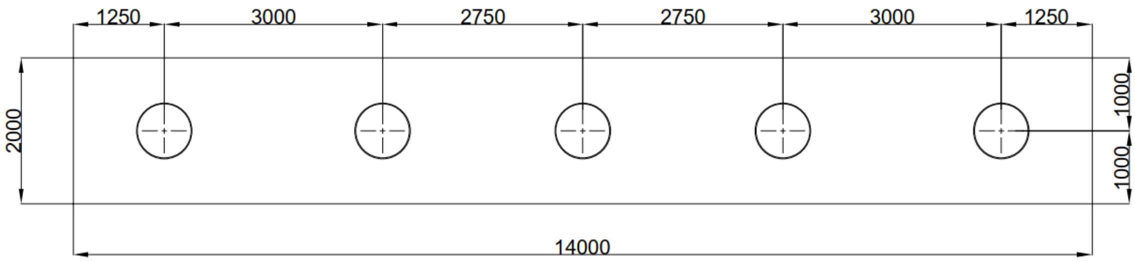
1 The piles are driven in the sea bottom.
2 Prefabricated concrete slabs will placed on top of the piles.
3 The slab will connect to the pile head with a temporary connection.
   - The space between the outside of the pile and the gap in the concrete. will be filled with a low-shrink casting mortar. so the connection is fixed against rotation.
   - On top of the pile head a welded connection with an inserted steel plate in the concrete slab will be made. so the connection is fixed against translation.
4 Inside the pile a reinforced concrete plug will be installed.
5 Prefabricated concrete deck elements will be placed on top of the pile heads.
6 Above the pile heads the joints will be connected with reinforcement and cast in placed concrete.

5 RESULTS AND STRUCTURAL CHECKS

5.1 Pile reactions

Below the deck 5 vertical piles will be applied. The c.t.c. distances of the piles is shown in the picture below.

Figure 5.1 Top view deck with piles



To determine the pile reactions a FEM-model in SCIA-engineer is made. The model consist of a slab of cracked concrete (E=10000 MPa) wide 2.0 m and thick 0.3 m. The piles with diameter 762 mm and wall thickness 14.3 mm are placed from the slab to the point of fixation in the bottom corresponding the geotechnical calculations. The self weight of the skirts. pile plugs and concrete blocks on top of the pile are part of the load case “added self weight”. The SCIA report with pile reactions is part of appendix I. the reactions are summarized in the table below.

Figure 5.2 View of SCIA-model

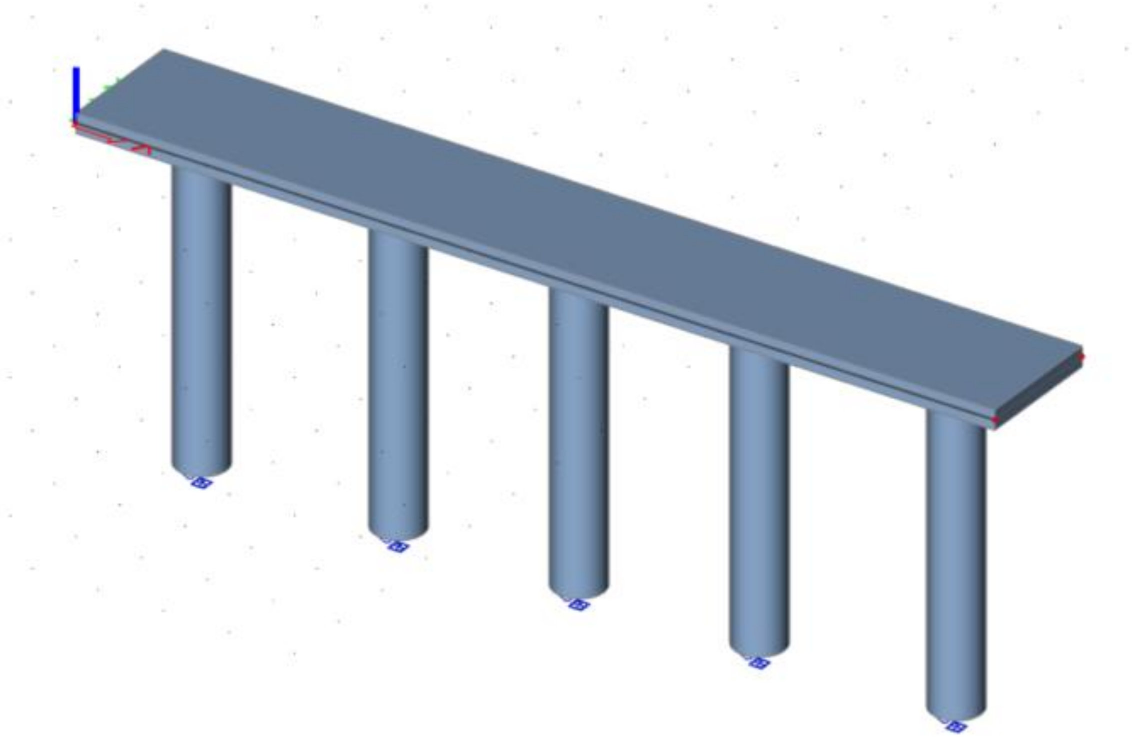


Table 5.1 Maximum pile reactions

	ULS	SLS	SLS_SW
	[kN/pile]	[kN/pile]	[kN/pile]
pressure	364.44	253.71	107.5
tension	-122.57	-38.71	-

5.2 Forces in the deck

The deck spans between the piles. the maximum span length is 3.0 m. The maximum shear force is half of the maximum ULS pile reaction (365 kN/pile). so 183 kN/2m. The shear force capacity of the concrete section thick 300 mm without stirrups is 156 kN/m. see appendix II for the calculation. With a unity check of 0.59 the result is o.k.

The bending moment will be calculated as a two-side supported beam with a span of 3.0 m without favourable effect of adjacent spans:

M_{SLS} = $1/8*(21.3+(2*5))*3^2$ = 36 kNm

M_{ULS} = $1/8*(1.35*21.3+1.50*(2*25.36))*3^2$ = 118 kNm

Without the favourable effect of the reinforced skirts. reinforcement Ø16-100 in the deck will be sufficient. see appendix II. The unity check in ULS is 0.33 and the crack width is 0.08 mm. so these results are o.k.

5.3 Forces in the pile head

The slab loads from the deck will be transferred to the pile head by the slab on top of the pile. Based on a maximum eccentricity of 0.60 m and a maximum shear force of 183 kN/2m the concrete checks are performed. The shear force resistance of a section thick 400 mm with stirrups Ø12-150 (4 sections) is 365 kN. with an effective width of 0.762 m equal to the diameter of the pile. The unity check is (183 / 365 =) 0.50 and is sufficient.

The bending moment in the slab is (183*0.6 =) 110 kNm (ULS) and (127*0.6 =) 77 kNm (SLS). Verification of the section is part of appendix II. The concrete slab is interrupted by the pile head. so as effective section width. the 1.50 m nominal width is reduced to 0.5 m. As can be seen in the calculation sheet. reinforcement Ø25-100 is sufficient. The unity check in ULS is 0.41 and the crack width is 0.19 mm. so these results are o.k.

During construction a deck element long 2.75 m and a self weight of 21.25 kN/m will be placed on top of the pile head with an eccentricity of 0.6 m. The (SLS) bending moment in the pile head is (1/2*2.75*21.25*0.60 =) 18 kNm. This relative low bending moment is not governing for the pile. the temporary connection is to be designed in more detail in the following design stages.

APPENDIX: SCIA REPORT

SCIA_report.pdf

II

APPENDIX: STRUCTURAL CHECKS

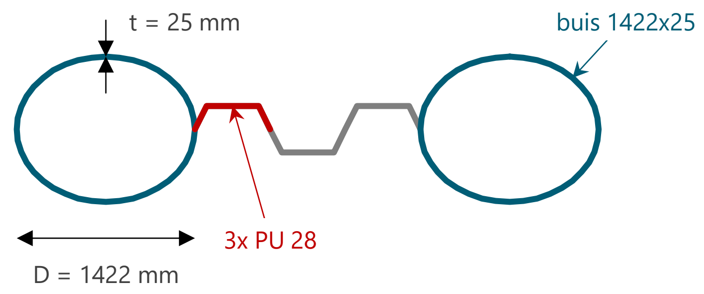
DRSN toetsen.pdf

VII

APPENDIX: GEOTECHNICAL CALCULATIONS FOR SECTION 1: MAIN BREAKWATER AND SECTION 5: MAIN BREAKWATER TRANSITION TO COAST

Project	**Black Rock Harbour Saba**
Projectcode	**118421**
Subject	**Section 1 - Cofferdam: Verification combination wall field**
Consultant	**I. Schrijver**
Date	**24-6-2021**

COMBINED WALL PROPERTIES



Geometry

Tubular pile - Outer diameter	D_o	=	**1422**	[mm]
Tubular pile - Wall thickness	t	=	**25**	[mm]
Tubular pile - Fabrication class	Class	=	**B**	[High]
Sheet pile type		=	**PU 28**	[-]
Number of intermediate sheet piles	#	=	**3**	[-]

Pile properties

			initial	corroded	
Outer diameter	D_o	=	1422.0	1419.5	[mm]
Inner diameter	D_i	=	1372.0	1372.0	[mm]
Average diameter	D_{av}	=	1397.0	1395.8	[mm]
Wall thickness water side	t_w	=	25.0	23.8	[mm]
Wall thickness land side	t_l	=	25.0	23.8	[mm]
Radius land side	r	=	0.699	0.698	[m]
Area cross section	A	=	1.097E+05	1.041E+05	[mm^2]
Weight per meter pile	w	=	861.3	817.5	[kg/m]
Elastic section modulus	W_{el}	=	3.766E+07	3.574E+07	[mm^3]
Plastic section modulus	W_{pl}	=	4.880E+07	4.627E+07	[mm^3]
Moment of inertia	I	=	2.677E+10	2.537E+10	[mm^4]
Bending stiffness pile	EI_{pile}	=	5.623E+06	5.327E+06	[kNm^2]
Bending stiffness plate	EI_{ring}	=	273	234	[kNm^2/m]

Corrosion

Water side	c_w	=	**1.3**	[mm]
Land side	c_l	=	**1.3**	[mm]
Inner side	c_b	=	**0.0**	[mm]

Steel properties

E-modulus	E	=	**2.10E+08**	[kN/m^2]
Unit weight	γ	=	**7850**	[kg/m^3]
Steel quality		=	**X70**	[-]
Yield stress	f_y	=	483	[N/mm^2]
Yield strain	ε_y	=	0.0023	[-]
Material factor	γ_{M0}	=	**1.10**	[-]

Uncorroded properties combined wall

Systemwidth	$x_{c.t.c.}$	=	3.297	[m]
Sectional area	A	=	4.508E+04	[mm^2/m]
Moment of inertia	I	=	8.473E+09	[mm^4/m]
Axial stiffness	EA	=	9.466E+06	[kN/m]
Bending stiffness	EI	=	1.779E+06	[kNm^2/m]
Weight per m^2 wall	w	=	354	[kg/m^2]
			3.47129474	

Pile resistance

			initial	corroded	
Normal force resistance	$N_{pl;Rd}$	=	48177	45727	[kN]
Elastic bending moment resistance	$M_{el;Rd}$	=	16535	15694	[kNm]
Plastic bending moment resistance	$M_{pl;Rd}$	=	21426	20318	[kNm]

Sectional class

			initial	corroded	
Limit parameter	D/t	=	56	59	[-]
Limit parameter	$D/t\varepsilon^2$	=	115	121	[-]
Sectional class EN1993-1-1 Table 5.2	Class	=	4	4	[-]

ACTIONS

Input values

Design value bending moment	$M_{max;d}$	=	**4857**	[kNm/m]
Design value normal force top side elements	N_{top}	=	**0**	[kN/m]
Excentricity normal force top side elements	e_{top}	=	**0.0**	[mm]
Design value anchor force	F_A	=	**0**	[kN/m]
Angle of anchorage with horizontal	α	=	**0**	[°]
Wall displacement top	$u_{wall;top}$	=	**355.9**	[mm]
Wall displacement at level of verification	$u_{wall;field}$	=	**30.9**	[mm]
Normal force from active wedge	N_{wedge}	=	**0**	[kN/m]
Effective active stress at level of verification	σ'_{active}	=	**0.00**	[kN/m^2]
Effective passive stress at level of verification	$\sigma'_{passive}$	=	**151.06**	[kN/m^2]
Shell factor at level of verification	S	=	**2.5**	[-]
Water head difference	Δh_w	=	**0.0**	[m]
Additional water head difference	$\Delta h_{w;add}$	=	**0.3**	[m]

Output values for verification of tubular pile

Normal force from top side elements	N_{top}	=	0	[kN/pile]
Normal force from anchor force	N_{anchor}	=	0	[kN/pile]
Normal force from active wedge	N_{wedge}	=	0	[kN/pile]
Total normal force	N_{Ed}	=	**0**	[kN/pile]
Bending moment (calculation)	M_{calc}	=	16013	[kNm/pile]
Bending moment 2nd order top elements	$M_{2nd;top}$	=	0	[kNm/pile]
Bending moment 2nd order anchor force	$M_{2nd;anchor}$	=	0	[kNm/pile]
Total bending moment	M_{Ed}	=	**16013**	[kNm/pile]
Water pressure difference	Δu	=	3.0	[kN/m^2]
Pressure active side	q_{active}	=	**3.0**	[kN/m^2/m]
Pressure passive side	$q_{passive}$	=	**350.2**	[kN/m^2/m]

CROSS SECTIONAL VERIFICATION OF TUBULAR PILE

Out of roundness / ovalization

			empty piles			sand-filled piles		
			initial	corroded		initial	corroded	
Maximum out of roundness	$U_{r;max}$	=	0.010	0.010	[-]	0.010	0.010	[-]
Ovalization due to initial out of roundness	a_1	=	3.49	3.49	[mm]	3.493	3.493	[mm]
Ovalization due to cross tensile forces	a_2	=	0.00	0.00	[mm]	0.000	0.000	[mm]
Ovalization due to outside soil pressure	a_3	=	12.81	14.89	[mm]	12.813	14.891	[mm]
Meridonial membrane stress	σ_y	=	417.9	440.7	[N/mm^2]	417.889	440.671	[N/mm^2]
Curvature	κ	=	0.0028	0.0030	[1/m]	0.003	0.003	[1/m]
Ovalization as second order effect of tube bending	a_4	=	2.16	2.65	[mm]	2.158	2.652	[mm]
Compression modulus of sand	E_{sand}	=	0	0	[MPa]	**10**	10	[MPa]
Stiffness sand	k_{sand}	=	0	0	[kN/m^3]	14316	14329	[kN/m^3]
Stiffness steel	k_{steel}	=	13784	11860	[kN/m^3]	13784	11860	[kN/m^3]
Total ovalization	a	=	18.46	21.04	[mm]	10.84	11.44	[mm]

Bending moment as a function of the plasticity rate

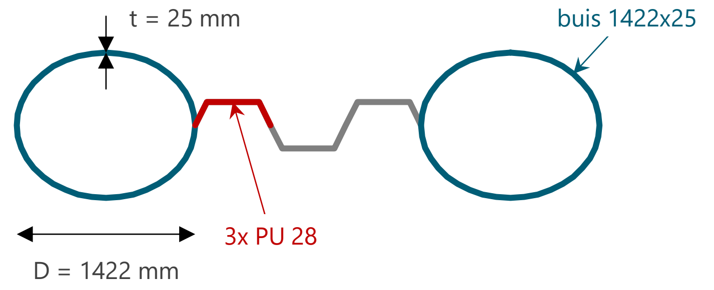
			initial	corroded		initial	corroded	
Radius of curvature	r'	=	0.759	0.767	[m]	0.733	0.734	[m]
Critical strain	ε_{cr}	=	0.0057	0.0052	[-]	0.0082	0.0073	[-]
Relative strain	μ	=	2.495	2.278	[-]	3.544	3.187	[-]
Angle of plasticity rate	θ	=	0.412	0.455	[rad]	0.286	0.319	[rad]
Bending moment as function of plasticity rate	M_R	=	20837	19645	[kNm]	21138	19979	[kNm]

Reduced resistance for effects of ovalisation

			initial	corroded		initial	corroded	
Bending moment in point A	M_A	=	-21.54	-21.50	[kNm/m]	-21.54	-21.50	[kNm/m]
Bending moment in point B	M_B	=	27.95	27.90	[kNm/m]	27.95	27.90	[kNm/m]
Tube wall bending moment	$m_{eff;Ed}$	=	24.74	24.70	[kNm/m]	12.14	11.19	[kNm/m]
Tube wall ultimate bending moment	$m_{pl;Rd}$	=	68.61	61.92	[kNm/m]	68.61	61.92	[kNm/m]
Reduction factor for ovalizing bending stresses	g	=	0.943	0.936	[-]	0.973	0.973	[-]
Reduction factor for ovalizing bending deformation	β_q	=	0.982	0.980	[-]	0.990	0.989	[-]
Reduction factor deformation capacity	β_s	=	0.937	0.910	[-]	1.000	1.000	[-]
Reduced bending moment resistance	M_{Rd}	=	18086	16397	[kNm]	20363	19224	[kNm]
Reduced normal force resistance	N_{Rd}	=	45435	42816	[kN]	46896	44484	[kN]
Unity check		=	**0.89**	**0.98**	[-]	**0.79**	**0.83**	[-]

Project	**Black Rock Harbour Saba**
Projectcode	**118421**
Subject	**Section 1 - Cofferdam: Verification combination wall top**
Consultant	**I. Schrijver**
Date	**24-6-2021**

COMBINED WALL PROPERTIES



Geometry

Tubular pile - Outer diameter	D_o	=	**1422**	[mm]
Tubular pile - Wall thickness	t	=	**25**	[mm]
Tubular pile - Fabrication class	Class	=	**B**	[High]
Sheet pile type		=	**PU 28**	[-]
Number of intermediate sheet piles	#	=	**3**	[-]

Pile properties

			initial	corroded	
Outer diameter	D_o	=	1422.0	1419.5	[mm]
Inner diameter	D_i	=	1372.0	1372.0	[mm]
Average diameter	D_{av}	=	1397.0	1395.8	[mm]
Wall thickness water side	t_w	=	25.0	23.8	[mm]
Wall thickness land side	t_l	=	25.0	23.8	[mm]
Radius land side	r	=	0.699	0.698	[m]
Area cross section	A	=	1.097E+05	1.041E+05	[mm^2]
Weight per meter pile	w	=	861.3	817.5	[kg/m]
Elastic section modulus	W_{el}	=	3.766E+07	3.574E+07	[mm^3]
Plastic section modulus	W_{pl}	=	4.880E+07	4.627E+07	[mm^3]
Moment of inertia	I	=	2.677E+10	2.537E+10	[mm^4]
Bending stiffness pile	EI_{pile}	=	5.623E+06	5.327E+06	[kNm^2]
Bending stiffness plate	EI_{ring}	=	273	234	[kNm^2/m]

Pile resistance

			initial	corroded	
Normal force resistance	$N_{pl;Rd}$	=	48177	45727	[kN]
Elastic bending moment resistance	$M_{el;Rd}$	=	16535	15694	[kNm]
Plastic bending moment resistance	$M_{pl;Rd}$	=	21426	20318	[kNm]

Sectional class

			initial	corroded	
Limit parameter	D/t	=	56	59	[-]
Limit parameter	D/tε^2	=	115	121	[-]
Sectional class EN1993-1-1 Table 5.2	Class	=	4	4	[-]

Corrosion

Water side	c_w	=	**1.25**	[mm]
Land side	c_l	=	**1.25**	[mm]
Inner side	c_b	=	**0.0**	[mm]

Steel properties

E-modulus	E	=	**2.10E+08**	[kN/m^2]
Unit weight	γ	=	**7850**	[kg/m^3]
Steel quality		=	**X70**	[-]
Yield stress	f_y	=	483	[N/mm^2]
Yield strain	ε_y	=	0.0023	[-]
Material factor	γ_{M0}	=	**1.10**	[-]

Uncorroded properties combined wall

Systemwidth	$x_{c.t.c.}$	=	3.297	[m]
Sectional area	A	=	4.508E+04	[mm^2/m]
Moment of inertia	I	=	8.473E+09	[mm^4/m]
Axial stiffness	EA	=	9.466E+06	[kN/m]
Bending stiffness	EI	=	1.779E+06	[kNm^2/m]
Weight per m^2 wall	w	=	354	[kg/m^2]
			3.47129474	

ACTIONS

Input values

Design value bending moment	$M_{max;d}$	=	**3995**	[kNm/m]
Design value normal force top side elements	N_{top}	=	**0**	[kN/m]
Excentricity normal force top side elements	e_{top}	=	**0.0**	[mm]
Design value anchor force	F_A	=	**0**	[kN/m]
Angle of anchorage with horizontal	α	=	**0**	[°]
Wall displacement top	$u_{wall;top}$	=	**424.3**	[mm]
Wall displacement at level of verification	$u_{wall;field}$	=	**424.3**	[mm]
Normal force from active wedge	N_{wedge}	=	**0**	[kN/m]
Effective active stress at level of verification	σ'_{active}	=	**0.00**	[kN/m^2]
Effective passive stress at level of verification	$\sigma'_{passive}$	=	**0.00**	[kN/m^2]
Shell factor at level of verification	S	=	**2.5**	[-]
Water head difference	Δh_w	=	**0.0**	[m]
Additional water head difference	$\Delta h_{w;add}$	=	**0.3**	[m]

Output values for verification of tubular pile

Normal force from top side elements	N_{top}	=	0	[kN/pile]
Normal force from anchor force	N_{anchor}	=	0	[kN/pile]
Normal force from active wedge	N_{wedge}	=	0	[kN/pile]
Total normal force	N_{Ed}	=	**0**	[kN/pile]
Bending moment (calculation)	M_{calc}	=	13170	[kNm/pile]
Bending moment 2nd order top elements	$M_{2nd;top}$	=	0	[kNm/pile]
Bending moment 2nd order anchor force	$M_{2nd;anchor}$	=	0	[kNm/pile]
Total bending moment	M_{Ed}	=	**13170**	[kNm/pile]
Water pressure difference	Δu	=	3.0	[kN/m^2]
Pressure active side	q_{active}	=	**3.0**	[kN/m^2/m]
Pressure passive side	$q_{passive}$	=	**0.0**	[kN/m^2/m]

CROSS SECTIONAL VERIFICATION OF TUBULAR PILE

Out of roundness / ovalization

			empty piles			sand-filled piles		
			initial	corroded		initial	corroded	
Maximum out of roundness	$U_{r;max}$	=	0.010	0.010	[-]	0.010	0.010	[-]
Ovalization due to initial out of roundness	a_1	=	3.49	3.49	[mm]	3.493	3.493	[mm]
Ovalization due to cross tensile forces	a_2	=	0.00	0.00	[mm]	0.000	0.000	[mm]
Ovalization due to outside soil pressure	a_3	=	0.11	0.13	[mm]	0.109	0.126	[mm]
Meridonial membrane stress	σ_y	=	343.7	362.4	[N/mm^2]	343.685	362.422	[N/mm^2]
Curvature	κ	=	0.0023	0.0025	[1/m]	0.002	0.002	[1/m]
Ovalization as second order effect of tube bending	a_4	=	1.46	1.79	[mm]	1.460	1.794	[mm]
Compression modulus of sand	E_{sand}	=	0	0	[MPa]	**10**	10	[MPa]
Stiffness sand	k_{sand}	=	0	0	[kN/m^3]	14316	14329	[kN/m^3]
Stiffness steel	k_{steel}	=	13784	11860	[kN/m^3]	13784	11860	[kN/m^3]
Total ovalization	a	=	5.06	5.41	[mm]	4.26	4.36	[mm]

Bending moment as a function of the plasticity rate

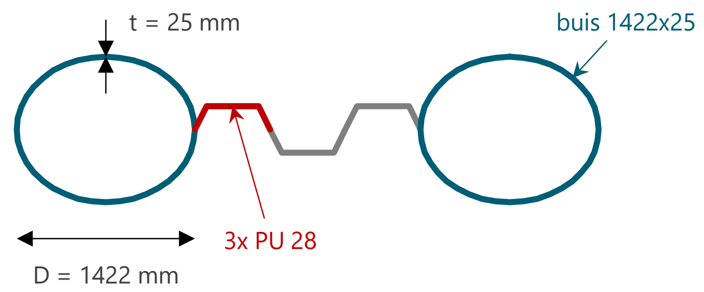
			initial	corroded		initial	corroded	
Radius of curvature	r'	=	0.714	0.714	[m]	0.712	0.711	[m]
Critical strain	ε_{cr}	=	0.0063	0.0058	[-]	0.0086	0.0078	[-]
Relative strain	μ	=	2.719	2.526	[-]	3.757	3.394	[-]
Angle of plasticity rate	θ	=	0.377	0.407	[rad]	0.269	0.299	[rad]
Bending moment as function of plasticity rate	M_R	=	20932	19774	[kNm]	21170	20020	[kNm]

Reduced resistance for effects of ovalisation

			initial	corroded		initial	corroded	
Bending moment in point A	M_A	=	-0.18	-0.18	[kNm/m]	-0.18	-0.18	[kNm/m]
Bending moment in point B	M_B	=	0.24	0.24	[kNm/m]	0.24	0.24	[kNm/m]
Tube wall bending moment	$m_{eff;Ed}$	=	0.21	0.21	[kNm/m]	0.10	0.10	[kNm/m]
Tube wall ultimate bending moment	$m_{pl;Rd}$	=	68.61	61.92	[kNm/m]	68.61	61.92	[kNm/m]
Reduction factor for ovalizing bending stresses	g	=	1.000	1.000	[-]	1.000	1.000	[-]
Reduction factor for ovalizing bending deformation	β_q	=	0.995	0.995	[-]	0.996	0.996	[-]
Reduction factor deformation capacity	β_s	=	0.965	0.941	[-]	1.000	1.000	[-]
Reduced bending moment resistance	M_{Rd}	=	20090	18497	[kNm]	21079	19932	[kNm]
Reduced normal force resistance	N_{Rd}	=	48156	45705	[kN]	48167	45717	[kN]
Unity check		=	**0.66**	**0.71**	[-]	**0.62**	**0.66**	[-]

Project	**Black Rock Harbour Saba**
Projectcode	**118421**
Subject	**Section 5 - Cofferdam transition to coast: Verification of combination wall field**
Consultant	**I. Schrijver**
Date	**24-6-2021**

COMBINED WALL PROPERTIES



Geometry

Tubular pile - Outer diameter	D_o	=	**1422**	[mm]
Tubular pile - Wall thickness	t	=	**25**	[mm]
Tubular pile - Fabrication class	Class	=	**B**	[High]
Sheet pile type		=	**PU 28**	[-]
Number of intermediate sheet piles	#	=	**3**	[-]

Pile properties

			initial	corroded	
Outer diameter	D_o	=	1422.0	1419.5	[mm]
Inner diameter	D_i	=	1372.0	1372.0	[mm]
Average diameter	D_{av}	=	1397.0	1395.8	[mm]
Wall thickness water side	t_w	=	25.0	23.8	[mm]
Wall thickness land side	t_l	=	25.0	23.8	[mm]
Radius land side	r	=	0.699	0.698	[m]
Area cross section	A	=	1.097E+05	1.041E+05	[mm^2]
Weight per meter pile	w	=	861.3	817.5	[kg/m]
Elastic section modulus	W_{el}	=	3.766E+07	3.574E+07	[mm^3]
Plastic section modulus	W_{pl}	=	4.880E+07	4.627E+07	[mm^3]
Moment of inertia	I	=	2.677E+10	2.537E+10	[mm^4]
Bending stiffness pile	EI_{pile}	=	5.623E+06	5.327E+06	[kNm^2]
Bending stiffness plate	EI_{ring}	=	273	234	[kNm^2/m]

Corrosion

Water side	c_w	=	**1.3**	[mm]
Land side	c_l	=	**1.3**	[mm]
Inner side	c_b	=	**0.0**	[mm]

Steel properties

E-modulus	E	=	**2.10E+08**	[kN/m^2]
Unit weight	γ	=	**7850**	[kg/m^3]
Steel quality		=	**X70**	[-]
Yield stress	f_y	=	483	[N/mm^2]
Yield strain	ε_y	=	0.0023	[-]
Material factor	γ_{M0}	=	**1.10**	[-]

Pile resistance

			initial	corroded	
Normal force resistance	$N_{pl;Rd}$	=	48177	45727	[kN]
Elastic bending moment resistance	$M_{el;Rd}$	=	16535	15694	[kNm]
Plastic bending moment resistance	$M_{pl;Rd}$	=	21426	20318	[kNm]

Uncorroded properties combined wall

Systemwidth	$x_{c.t.c.}$	=	3.297	[m]
Sectional area	A	=	4.508E+04	[mm^2/m]
Moment of inertia	I	=	8.473E+09	[mm^4/m]
Axial stiffness	EA	=	9.466E+06	[kN/m]
Bending stiffness	EI	=	1.779E+06	[kNm^2/m]
Weight per m^2 wall	w	=	354	[kg/m^2]
			3.47129474	

Sectional class

			initial	corroded	
Limit parameter	D/t	=	56	59	[-]
Limit parameter	D/tε^2	=	115	121	[-]
Sectional class EN1993-1-1 Table 5.2	Class	=	4	4	[-]

ACTIONS

Input values

Design value bending moment	$M_{max;d}$	=	**2731**	[kNm/m]
Design value normal force top side elements	N_{top}	=	**0**	[kN/m]
Excentricity normal force top side elements	e_{top}	=	**0.0**	[mm]
Design value anchor force	F_A	=	**0**	[kN/m]
Angle of anchorage with horizontal	α	=	**0**	[°]
Wall displacement top	$u_{wall;top}$	=	**159.8**	[mm]
Wall displacement at level of verification	$u_{wall;field}$	=	**10.8**	[mm]
Normal force from active wedge	N_{wedge}	=	**0**	[kN/m]
Effective active stress at level of verification	σ'_{active}	=	**0.00**	[kN/m^2]
Effective passive stress at level of verification	$\sigma'_{passive}$	=	**106.52**	[kN/m^2]
Shell factor at level of verification	S	=	**2.5**	[-]
Water head difference	Δh_w	=	**0.0**	[m]
Additional water head difference	$\Delta h_{w;add}$	=	**0.3**	[m]

Output values for verification of tubular pile

Normal force from top side elements	N_{top}	=	0	[kN/pile]
Normal force from anchor force	N_{anchor}	=	0	[kN/pile]
Normal force from active wedge	N_{wedge}	=	0	[kN/pile]
Total normal force	N_{Ed}	=	**0**	[kN/pile]
Bending moment (calculation)	M_{calc}	=	9004	[kNm/pile]
Bending moment 2nd order top elements	$M_{2nd;top}$	=	0	[kNm/pile]
Bending moment 2nd order anchor force	$M_{2nd;anchor}$	=	0	[kNm/pile]
Total bending moment	M_{Ed}	=	**9004**	[kNm/pile]
Water pressure difference	Δu	=	3.0	[kN/m^2]
Pressure active side	q_{active}	=	**3.0**	[kN/m^2/m]
Pressure passive side	$q_{passive}$	=	**247.0**	[kN/m^2/m]

CROSS SECTIONAL VERIFICATION OF TUBULAR PILE

			empty piles			sand-filled piles		
Out of roundness / ovalization			initial	corroded		initial	corroded	
Maximum out of roundness	$U_{r;max}$	=	0.010	0.010	[-]	0.010	0.010	[-]
Ovalization due to initial out of roundness	a_1	=	3.49	3.49	[mm]	3.493	3.493	[mm]
Ovalization due to cross tensile forces	a_2	=	0.00	0.00	[mm]	0.000	0.000	[mm]
Ovalization due to outside soil pressure	a_3	=	9.07	10.54	[mm]	9.068	10.538	[mm]
Meridonial membrane stress	σ_y	=	235.0	247.8	[N/mm^2]	234.973	247.783	[N/mm^2]
Curvature	κ	=	0.0016	0.0017	[1/m]	0.002	0.002	[1/m]
Ovalization as second order effect of tube bending	a_4	=	0.68	0.84	[mm]	0.682	0.838	[mm]
Compression modulus of sand	E_{sand}	=	0	0	[MPa]	**10**	10	[MPa]
Stiffness sand	k_{sand}	=	0	0	[kN/m^3]	14316	14329	[kN/m^3]
Stiffness steel	k_{steel}	=	13784	11860	[kN/m^3]	13784	11860	[kN/m^3]
Total ovalization	a	=	13.24	14.87	[mm]	8.28	8.64	[mm]
Bending moment as a function of the plasticity rate			initial	corroded		initial	corroded	
Radius of curvature	r'	=	0.741	0.746	[m]	0.724	0.725	[m]
Critical strain	ε_{cr}	=	0.0059	0.0055	[-]	0.0083	0.0075	[-]
Relative strain	μ	=	2.582	2.376	[-]	3.626	3.268	[-]
Angle of plasticity rate	θ	=	0.398	0.434	[rad]	0.279	0.311	[rad]
Bending moment as function of plasticity rate	M_R	=	20877	19701	[kNm]	21151	19996	[kNm]
Reduced resistance for effects of ovalisation			initial	corroded		initial	corroded	
Bending moment in point A	M_A	=	-15.25	-15.22	[kNm/m]	-15.25	-15.22	[kNm/m]
Bending moment in point B	M_B	=	19.74	19.71	[kNm/m]	19.74	19.71	[kNm/m]
Tube wall bending moment	$m_{eff;Ed}$	=	17.49	17.46	[kNm/m]	8.58	7.91	[kNm/m]
Tube wall ultimate bending moment	$m_{pl;Rd}$	=	68.61	61.92	[kNm/m]	68.61	61.92	[kNm/m]
Reduction factor for ovalizing bending stresses	g	=	0.961	0.956	[-]	0.981	0.981	[-]
Reduction factor for ovalizing bending deformation	β_q	=	0.987	0.986	[-]	0.992	0.992	[-]
Reduction factor deformation capacity	β_s	=	0.948	0.922	[-]	1.000	1.000	[-]
Reduced bending moment resistance	M_{Rd}	=	18773	17126	[kNm]	20594	19455	[kNm]
Reduced normal force resistance	N_{Rd}	=	46293	43736	[kN]	47282	44860	[kN]
Unity check		=	**0.48**	**0.53**	[-]	**0.44**	**0.46**	[-]

VIII

APPENDIX: GEOTECHNICAL CALCULATIONS FOR SECTION 2: FERRY QUAY WALL

Report for D-Sheet Piling 19.2

Design of Diaphragm and Sheet Pile Walls
Developed by Deltares

Company:	Witteveen+Bos
Date of report:	6/8/2021
Time of report:	5:07:43 PM
Report with version:	19.2.3.26114
Date of calculation:	6/8/2021
Time of calculation:	5:06:34 PM
Calculated with version:	19.2.3.26114
File name:	D:\..\Saba_Black Rock harbour_Retaining wall_Coeficients L2
Project identification:	Saba Black Rock Harbour Section 2 (RoRo quay wall)

Verification according to National Annex of Eurocode 7 in the Netherlands (NEN 9997-1:2016)

1 Summary

1.1 Overview per Stage and Test

Stage nr.	Verification type	Displace-ment [mm]	Moment [kNm]	Shear force [kN]	Mob. perc. moment [%]	Mob. perc. resistance [%]	Vertical balance
1	EC7(NL)-Step 6.1		-0,22	-0,26	0,0	11,2	Upwards
1	EC7(NL)-Step 6.2		-0,12	-0,19	0,0	11,2	Upwards
1	EC7(NL)-Step 6.3		0,47	0,50	0,0	11,4	Upwards
1	EC7(NL)-Step 6.4		0,24	0,42	0,0	11,4	Upwards
1	EC7(NL)-Step 6.5	0,0	0,00	0,00	0,0	7,4	Sufficient
1	EC7(NL)-Step 6.5 * 1,200		0,00	0,00			
2	EC7(NL)-Step 6.1		-14,09	-9,41	0,0	14,5	Upwards
2	EC7(NL)-Step 6.2		-13,05	-9,34	0,0	14,5	Upwards
2	EC7(NL)-Step 6.3		-15,88	-10,45	0,0	15,1	Upwards
2	EC7(NL)-Step 6.4		-14,70	-10,33	0,0	15,1	Upwards
2	EC7(NL)-Step 6.5	0,6	-7,75	6,15	0,0	9,3	Not sufficient
2	EC7(NL)-Step 6.5 * 1,200		-9,30	7,39			
3	EC7(NL)-Step 6.1		-14,10	-9,45	13,5	14,2	Upwards
3	EC7(NL)-Step 6.2		-13,03	-9,38	13,5	14,2	Upwards
3	EC7(NL)-Step 6.3		-15,90	-10,47	13,7	14,6	Upwards
3	EC7(NL)-Step 6.4		-14,66	-10,38	13,7	14,7	Upwards
3	EC7(NL)-Step 6.5	0,6	-7,75	6,15	8,9	9,3	Not sufficient
3	EC7(NL)-Step 6.5 * 1,200		-9,30	7,39			
4	EC7(NL)-Step 6.1		-19,08	-11,16	14,2	15,2	Sufficient
4	EC7(NL)-Step 6.2		-18,63	11,84	14,2	15,3	Sufficient
4	EC7(NL)-Step 6.3		-20,65	-12,00	14,4	15,6	Upwards
4	EC7(NL)-Step 6.4		-19,95	12,30	14,4	15,6	Upwards
4	EC7(NL)-Step 6.5	1,0	-12,19	8,65	9,4	10,0	Not sufficient
4	EC7(NL)-Step 6.5 * 1,200		-14,63	10,38			
5	EC7(NL)-Step 6.1		192,24	-74,99	44,8	48,1	Not sufficient
5	EC7(NL)-Step 6.2		168,42	-80,27	48,4	52,2	Not sufficient
5	EC7(NL)-Step 6.3		221,13	84,30	47,7	51,0	Not sufficient
5	EC7(NL)-Step 6.4		196,09	-87,51	52,0	55,7	Not sufficient
5	EC7(NL)-Step 6.5	10,6	101,77	-58,35	24,8	27,4	Not sufficient
5	EC7(NL)-Step 6.5 * 1,200		122,13	-70,02			
6	EC7(NL)-Step 6.1		264,98	105,18	52,4	55,7	Not sufficient
6	EC7(NL)-Step 6.2		238,87	99,53	57,6	61,2	Not sufficient
6	EC7(NL)-Step 6.3		296,83	116,43	55,0	58,3	Not sufficient
6	EC7(NL)-Step 6.4		270,82	110,87	60,7	64,3	Not sufficient
6	EC7(NL)-Step 6.5	13,8	140,46	-72,56	28,8	31,9	Not sufficient
6	EC7(NL)-Step 6.5 * 1,200		168,55	-87,07			
7	EC7(NL)-Step 6.1		443,75	171,21	65,9	68,9	Upwards
7	EC7(NL)-Step 6.2		422,63	166,59	72,2	75,2	Upwards
7	EC7(NL)-Step 6.3		**479,76**	**182,50**	68,4	71,3	Upwards
7	EC7(NL)-Step 6.4		459,52	178,32	**74,7**	**77,4**	Upwards
7	EC7(NL)-Step 6.5	29,1	255,75	120,58	39,5	43,5	Upwards
7	EC7(NL)-Step 6.5 * 1,200		306,90	144,69			
8	EC7(NL)-Step 6.1		442,20	170,16	60,4	63,2	Sufficient
8	EC7(NL)-Step 6.2		421,35	166,19	66,3	69,0	Upwards
8	EC7(NL)-Step 6.3		478,00	181,45	62,9	65,5	Upwards
8	EC7(NL)-Step 6.4		458,20	177,90	68,5	71,1	Upwards
8	EC7(NL)-Step 6.5	**36,3**	309,05	131,95	52,5	56,7	Upwards
8	EC7(NL)-Step 6.5 * 1,200		370,86	158,34			
Max		**36,3**	**479,76**	**182,50**	**74,7**	**77,4**	Not sufficient

1.2 Anchors and Struts

Stage nr.	Verification type	Anchor/strut Ankerscherm	
		Force [kN]	State
3	EC7(NL)-Step 6.1	-	
3	EC7(NL)-Step 6.2	-	
3	EC7(NL)-Step 6.3	-	
3	EC7(NL)-Step 6.4	-	
3	EC7(NL)-Step 6.5 x 1,200	-	
4	EC7(NL)-Step 6.1	2,91	Elastic
4	EC7(NL)-Step 6.2	2,12	Elastic
4	EC7(NL)-Step 6.3	2,60	Elastic
4	EC7(NL)-Step 6.4	1,91	Elastic
4	EC7(NL)-Step 6.5 x 1,200	2,49	Elastic
5	EC7(NL)-Step 6.1	80,00	Elastic
5	EC7(NL)-Step 6.2	74,71	Elastic
5	EC7(NL)-Step 6.3	90,64	Elastic
5	EC7(NL)-Step 6.4	85,12	Elastic
5	EC7(NL)-Step 6.5 x 1,200	62,83	Elastic
6	EC7(NL)-Step 6.1	120,87	Elastic
6	EC7(NL)-Step 6.2	115,22	Elastic
6	EC7(NL)-Step 6.3	132,12	Elastic
6	EC7(NL)-Step 6.4	126,56	Elastic
6	EC7(NL)-Step 6.5 x 1,200	68,12	Elastic
7	EC7(NL)-Step 6.1	189,64	Elastic
7	EC7(NL)-Step 6.2	184,27	Elastic
7	EC7(NL)-Step 6.3	**200,88**	Elastic
7	EC7(NL)-Step 6.4	196,65	Elastic
7	EC7(NL)-Step 6.5 x 1,200	162,42	Elastic
8	EC7(NL)-Step 6.1	188,46	Elastic
8	EC7(NL)-Step 6.2	183,85	Elastic
8	EC7(NL)-Step 6.3	199,68	Elastic
8	EC7(NL)-Step 6.4	196,22	Elastic
8	EC7(NL)-Step 6.5 x 1,200	179,49	Elastic
Max		**200,88**	

Due to multiplication of the representative value a force bigger than yield or buckling force may be present.

1.3 Overall Stability per Stage

Stage name	Stability factor [-]
Initial	2397,78
Excavation for the anchor	9,17
Install anchor	9,17
Replenish sand	7,13
Excavate seaside	2,00
Utilisation phase	1,82
Utilisation phase (ships)	1,60
Utilisation phase (earthquake)	1,70

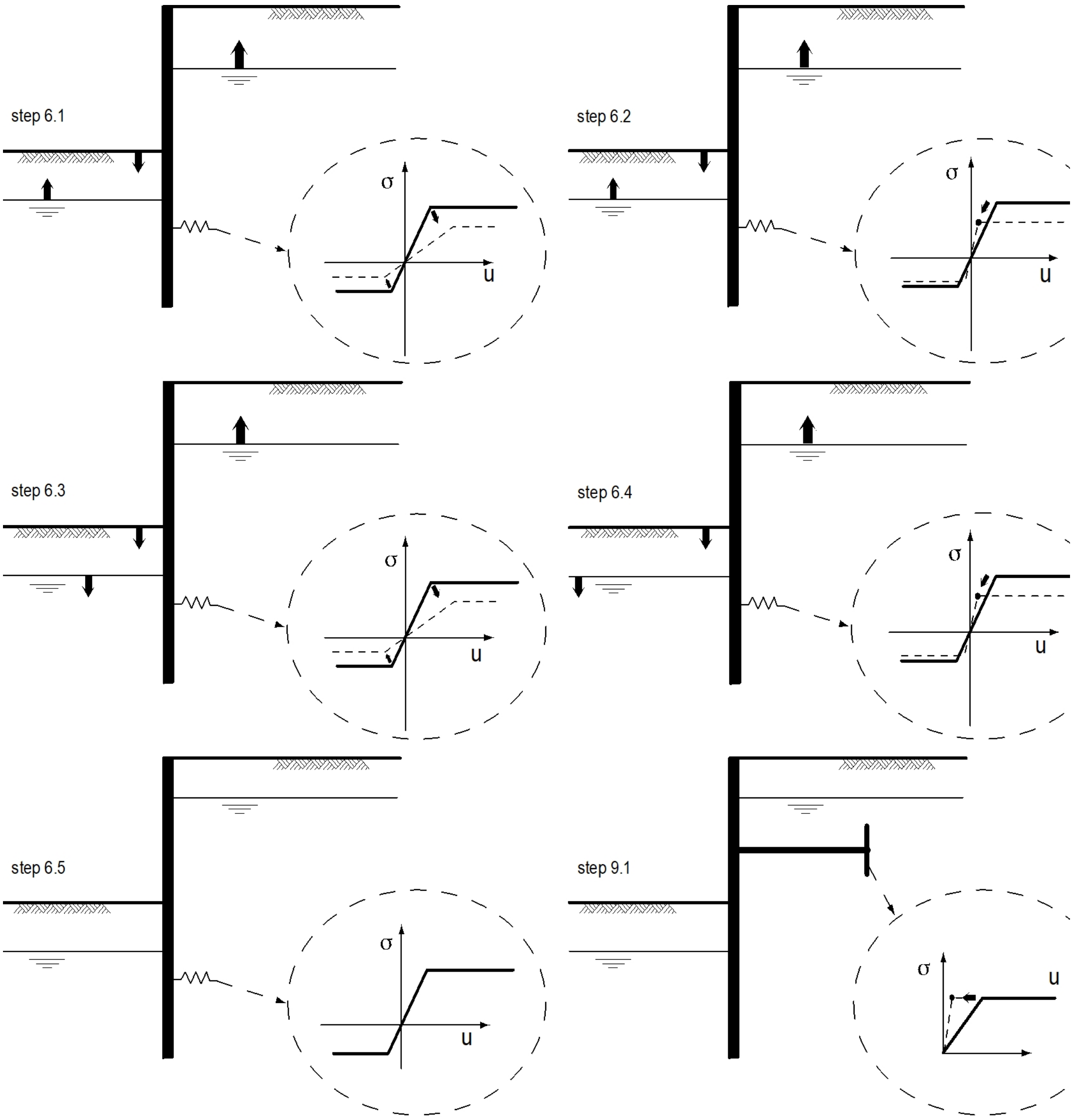
1.4 Warnings

Lambda
In a verification the Ka, Ko and Kp are recalculated with reduced phi and delta's. This was impossible with the next material(s) while they have a manual given Ka, Ko and Kp.
Sand-supplemented (f)
BoulderGravelCobbel 1 (f)
Sand-dense 1 (f)
Sand-dense 2 (f)
BoulderGravelCobbel 2 (f)

Sand-medium dense (f)

* Vertical balance: The resultant vertical friction force is directed upward in stage 7, 8, 1, 2, 3, 4, 7, 8, 1, 2, 3, 7, 1, 2, 3, 4, 7, 8, 1, 2, 3, 7 & 8
because the friction force on the passive side exceeds that on the active side.
This might be prevented by reducing the friction angle Delta on the passive side.

1.5 CUR Verification Steps



End of Report

Report for D-Sheet Piling 19.2

Design of Diaphragm and Sheet Pile Walls
Developed by Deltares

Company:	Witteveen+Bos
Date of report:	6/8/2021
Time of report:	5:11:03 PM
Report with version:	19.2.3.26114
Date of calculation:	6/8/2021
Time of calculation:	5:10:39 PM
Calculated with version:	19.2.3.26114
File name:	D:\..\Saba_Black Rock harbour_Retaining wall_Coeficients_L2_ankerscherm
Project identification:	Saba Black Rock Harbour
	Section 2 (RoRo quay wall)

Verification according to National Annex of Eurocode 7 in the Netherlands (NEN 9997-1:2016)

1 Summary

1.1 Overview per Stage and Test

Stage nr.	Verification type	Displace-ment [mm]	Moment [kNm]	Shear force [kN]	Mob. perc. moment [%]	Mob. perc. resistance [%]	Vertical balance
1	EC7(NL)-Step 6.1		-0,21	-0,30	**0,0**	8,4	Upwards
1	EC7(NL)-Step 6.2		-0,16	-0,25	**0,0**	8,4	Upwards
1	EC7(NL)-Step 6.3		0,38	0,47	**0,0**	8,5	Upwards
1	EC7(NL)-Step 6.4		0,27	0,38	**0,0**	8,5	Upwards
1	EC7(NL)-Step 6.5	0,0	0,00	0,00	**0,0**	4,9	Sufficient
1	EC7(NL)-Step 6.5 * 1,200		0,00	0,00			
2	EC7(NL)-Step 6.1		109,04	-160,37	**0,0**	49,7	Upwards
2	EC7(NL)-Step 6.2		108,46	**-161,18**	**0,0**	**49,8**	Upwards
2	EC7(NL)-Step 6.3		113,23	-154,94	**0,0**	45,5	Upwards
2	EC7(NL)-Step 6.4		111,54	-156,42	**0,0**	45,7	Upwards
2	EC7(NL)-Step 6.5	1,5	87,91	104,46	**0,0**	21,4	Upwards
2	EC7(NL)-Step 6.5 * 1,200		105,49	125,35			
3	EC7(NL)-Step 6.1		109,63	-159,85	**0,0**	42,9	Upwards
3	EC7(NL)-Step 6.2		109,05	-160,66	**0,0**	43,0	Upwards
3	EC7(NL)-Step 6.3		**114,00**	-154,28	**0,0**	39,3	Upwards
3	EC7(NL)-Step 6.4		112,30	-155,77	**0,0**	39,5	Upwards
3	EC7(NL)-Step 6.5	**1,6**	86,83	104,58	**0,0**	27,5	Upwards
3	EC7(NL)-Step 6.5 * 1,200		104,20	125,49			
Max		**1,6**	**114,00**	**-161,18**	**0,0**	**49,8**	Sufficient

1.2 Overall Stability per Stage

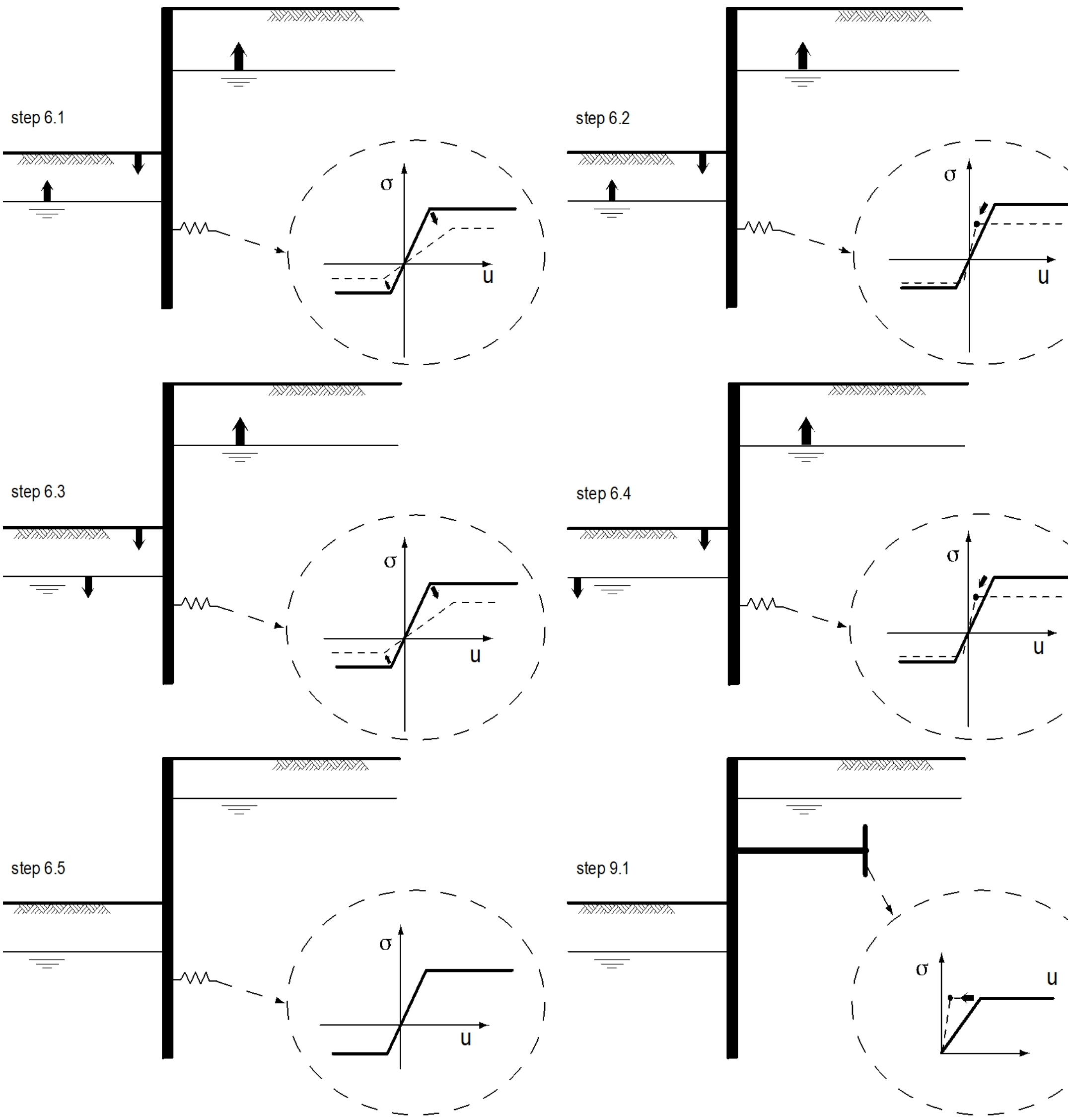
Stage name	Stability factor [-]
Initial	10000,00
Anchor utilisation phase (normal)	2,87
Anchor utilisation phase (earthquake)	2,90

1.3 Warnings

Lambda
In a verification the Ka, Ko and Kp are recalculated with reduced phi and delta's. This was impossible with the next material(s) while they have a manual given Ka, Ko and Kp.
Sand-supplemented (f)
BoulderGravelCobbel 1 (f)
Sand-dense 1 (f)
Sand-dense 2 (f)
BoulderGravelCobbel 2 (f)
Sand-medium dense (f)

* Vertical balance: The resultant vertical friction force is directed upward in stage 2, 3, 1, 2, 3, 1, 2, 3, 1, 2, 3, 1, 2 & 3
because the friction force on the passive side exceeds that on the active side.
This might be prevented by reducing the friction angle Delta on the passive side.

1.4 CUR Verification Steps



End of Report

Witteveen+Bos

SPREADSHEET
Anchor design values

version 0.10 **[not validated]**

Project	**Saba Black Rock harbour**
Projectcode	**118421**
Subject	**Section 2 (RoRo quay wall)**
Consultant	**L.Jorissen**
Date	**08-06-2021**

TITLE
Calculation of anchor design values.

STARTING POINTS
general
language = **EN**

design parameters

horizontal length behind sheet pile wall	L_{hor}	=	**2.8**	m
maximum force from anchors (ULS)	F_{max}	=	**200.88**	kN/m
yield characteristic thread	$F_{yt;char}$	=	**355**	kN
tensile characteristic thread	$F_{tt;char}$	=	**510**	kN
factor kt	kt	=	0.9	
factor gm;0	$g_{m;0}$	=	1.0	
factor gm;2	$g_{m;2}$	=	1.1	
factor gm;ser	$g_{m;ser}$	=	1.25	

anchor catalogue

Withworth DIN thread		Tensile stress area thread	Tensile stress area thread	Gross cross-section area shaft	Gross cross-section area shaft	Yield characteristic thread	Tensile characteristic thread	Yield characteristic shaft	Tensile characteristic shaft	Tensile thread	Tensile shaft	Design force (ULS)	Design force (SLS)	Tie rod failure	Design force (per m wall)	Design area (per m wall)	Design area model 2.8 ctc spacing
D		Ds	As	Dg	Ag	Fyt;char	Ftt;char	Fyg;char	Ftg;char	Ftt;Rd	Ftg;Rd	Ft;Rd	Ft;ser	fa	Ft;Rd		
[inch]	[mm]	[mm]	[mm^2]	[mm]	[mm^2]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN/m]		
2.50	63.50	57.57	2603	59	2734	924	1328	971	1394	956	971	956	840	1195	341	976	930
3.00	76.20	69.43	3786	71	3959	1344	1931	1406	2019	1390	1406	1390	1222	1738	496	1414	1352
3.50	88.90	81.60	5230	83	5411	1857	2667	1921	2759	1921	1921	1921	1688	2401	686	1932	1868
4.00	101.60	93.76	6904	96	7238	2451	3521	2570	3691	2535	2570	2535	2228	3169	905	2585	2466
4.50	114.30	106.05	8834	108	9161	3136	4505	3252	4672	3244	3252	3244	2851	4055	1158	3272	3155

corrosion catalogue 1 mm radial

![Witteveen+Bos logo]

SPREADSHEET
Anchor design values

version 0.10 **[not validated]**

Project	**Saba Black Rock harbour**
Projectcode	**118421**
Subject	**Section 2 (RoRo quay wall)**
Consultant	**L.Jorissen**
Date	**08-06-2021**

Withworth DIN thread		Tensile stress area thread	Tensile stress area thread	Gross cross-section area shaft	Gross cross-section area shaft	Yield characteristic thread	Tensile characteristic thread	Yield characteristic shaft	Tensile characteristic shaft	Tensile thread	Tensile shaft	Design force (ULS)	Design force (SLS)	Tie rod failure	Design force (per m wall)	Design area (per m wall)	Design area model 2.8 ctc spacing
D		Ds	As	Dg	Ag	Fyt;char	Ftt;char	Fyg;char	Ftg;char	Ftt;Rd	Ftg;Rd	Ft;Rd	Ft;ser	fa	Ft;Rd		
[inch]	[mm]	[mm]	[mm^2]	[mm]	[mm^2]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN/m]		
2.50	61.50	55.57241	2426	57.00	2552	861	1237	906	1301	891	906	891	783	1113	318	911	866
3.00	74.20	67.4258062	3571	69.00	3739	1268	1821	1327	1907	1311	1327	1311	1152	1639	468	1335	1275
3.50	86.90	79.6041656	4977	81.00	5153	1767	2538	1829	2628	1828	1829	1828	1606	2284	653	1840	1777
4.00	99.60	91.7576822	6613	94.00	6940	2347	3372	2464	3539	2428	2464	2428	2134	3035	867	2478	2362
4.50	112.30	104.053111	8504	106.00	8825	3019	4337	3133	4501	3123	3133	3123	2744	3903	1115	3152	3037

corrosion catalogue 2 mm radial

Withworth DIN thread		Tensile stress area thread	Tensile stress area thread	Gross cross-section area shaft	Gross cross-section area shaft	Yield characteristic thread	Tensile characteristic thread	Yield characteristic shaft	Tensile characteristic shaft	Tensile thread	Tensile shaft	Design force (ULS)	Design force (SLS)	Tie rod failure	Design force (per m wall)	Design area (per m wall)	Design area model 2.8 ctc spacing
D		Ds	As	Dg	Ag	Fyt;char	Ftt;char	Fyg;char	Ftg;char	Ftt;Rd	Ftg;Rd	Ft;Rd	Ft;ser	fa	Ft;Rd		
[inch]	[mm]	[mm]	[mm^2]	[mm]	[mm^2]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN/m]		
2.50	59.50	53.57	2254	55.00	2376	800	1150	843	1212	828	843	828	727	1035	296	849	805
3.00	72.20	65.43	3362	67.00	3526	1193	1715	1252	1798	1234	1252	1234	1085	1543	441	1259	1201
3.50	84.90	77.60	4730	79.00	4902	1679	2412	1740	2500	1737	1740	1737	1526	2171	620	1751	1689
4.00	97.60	89.76	6328	92.00	6648	2246	3227	2360	3390	2323	2360	2323	2042	2904	830	2374	2260
4.50	110.30	102.05	8180	104.00	8495	2904	4172	3016	4332	3004	3016	3004	2640	3755	1073	3034	2921

SUMMARY

new anchor cross section A_{design} = **8.05E-04** m^2/m

SPREADSHEET
Gazetas et al. (1990)
Degree of damage
versie 0.10 **[ongevalideerd]**

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 2 (RoRo quay wall)**
Consultatnt	**L.Jorissen**
Date	**8-6-2021**

Title

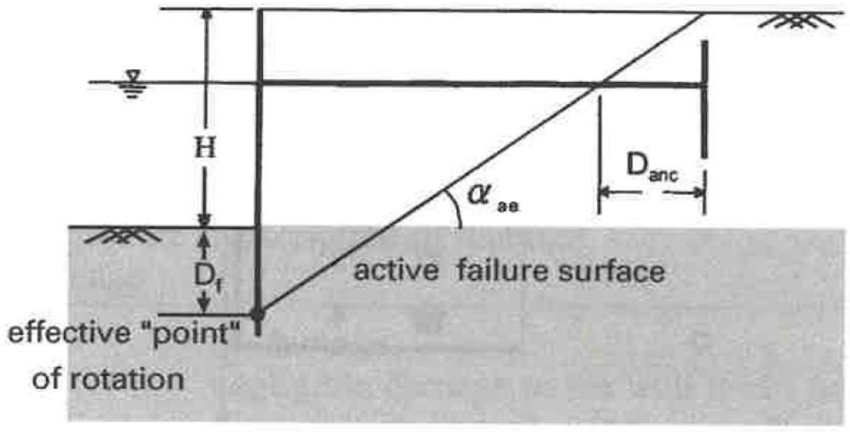
Calculation of the approximate degree of damage to anchored sheet pile walls during earthquakes according Gazetas et al. 1990

Seismic loads

Horizontal seismic coefficient	k_h	=	**0.275**	-
Vertical seismic coefficient	k_v	=	**0**	-
Equivalent seismic coefficient	k_e	=	0.22	-

Geometry

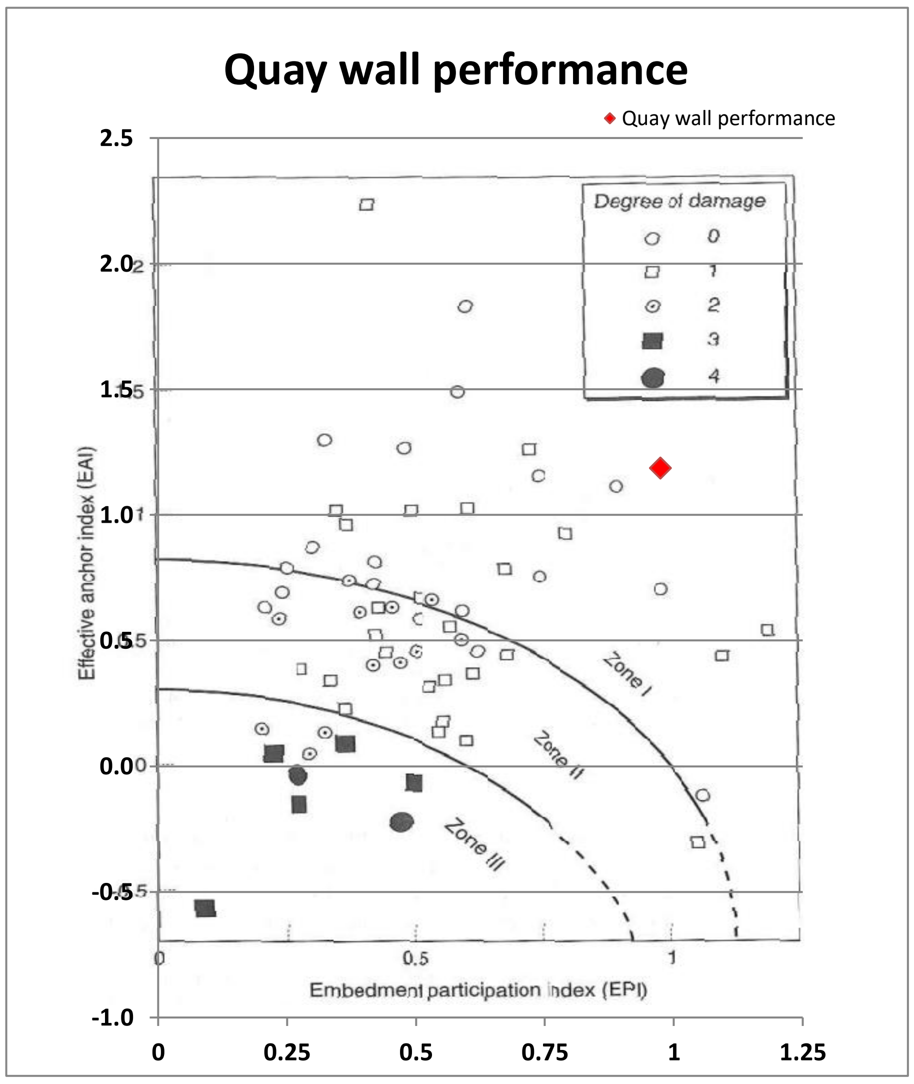
Retaining height	H	=	**8.1**	m
Water depth	H_{sub}	=	**7.3**	m
Imbedded length	D_{emb}	=	**5.5**	m
Anchor wall distance	L_{anchor}	=	**20**	m
Surface level	$z_{surface}$	=	**2.1**	m CD
Tierod level	z_{tierod}	=	**0.5**	m CD
Top level anhor wall	$z_{top\ anchor\ wall}$	=	**1.5**	m CD
Length anchor wall	L_{aw}	=	**5**	m
Inclination surface	β	=	**0**	°
Inclination sheet pile	θ	=	**0**	°



Soil

			active	passive	
Unit weigth	γ	=	**18**	**18**	kN/m^3
Saturated unit weight	γ_{sat}	=	**20**	**20**	kN/m^3
Internal friction angle	φ'	=	**35**	**35**	°
Friction anlge soil-wall	δ'	=	**23**	**23**	°

Unit weight water	γ_w	=	**10**	kN/m^3



CALCULATIONS

Depth effective point of rotation	D_f	=	3.0	m
Active wedge height	h_{aw}	=	11.1	m
Effective point of rotation level	z_{Df}	=	-9.0	m CD
Angle active failure plane	α_{ae}	=	42.4	°
Active wedge height to tierod level	$z_{aw\text{-}tierod}$	=	9.5	m
Active failure length to tierod level	$L_{aw\text{-}tierod}$	=	10.4	m
Length active wedge anchor wall	D_{anc}	=	9.6	m

Layer	γ_b [kN/m^3]	γ_e [kN/m^3]	$\gamma_{e\text{-}sat}$ [kN/m^3]	Ψ [°]	K_{ae} [-]	K_{pe} [-]
Soil active side	10.0	10.9	19.8	21.44	0.61	
Soil passive side	10.0	10.9	19.8	21.44		6.47

Effective anchor index	EAI	=	1.187	-
Embedment participation index	EPI	=	0.974	-

Degree of damage zone	=	**1**
Qualitative description damage	=	negligible damage to the wall itself, notaceable damage to the related structures (i.e. concrete apron)
Quantitive description damage	=	10 cm permanent displacement at top of sheet pile

IX

APPENDIX: GEOTECHNICAL VERIFICATIONS FOR SECTION 4: INNER HARBOUR WALL

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 4 (L-wall)**
Consultant	**L.Jorissen**
Date	**25-5-2021**

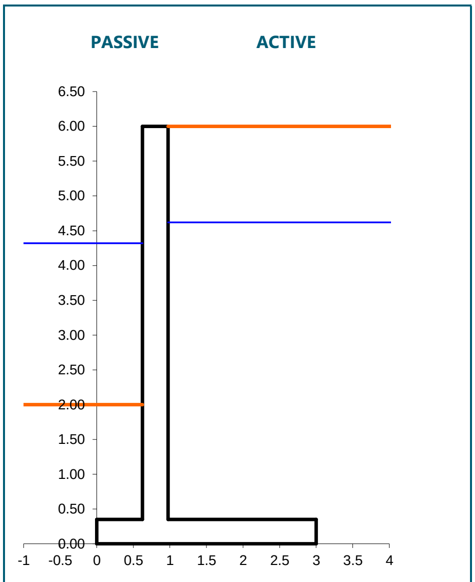
TITLE

Calculation of gravity wall in accordance with Eurocode 7 (EN 1997-1) with
Dutch National Annex (NEN-EN 1997-1/NB) and Dutch complementary rules for application of Eurocode 7 (NEN 9997-1).

STARTING POINTS

general

language		=	**EN**	
reference level		=	**NAP**	
reliability class		=	**RC2**	
unit weight water	$\gamma_{w;d}$	=	**10.0**	kN/m³

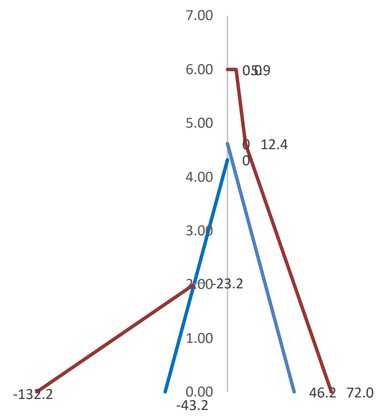


geometry

Prefab / in-situ			**prefab**	
Top level gravity wall	z_{top}	=	**1.50**	m+NAP
Bottom level gravity wall	z_{bottom}	=	**-4.50**	m+NAP
Height gravity wall	H	=	6.00	m
Retaining height	h	=	4.00	m
surface level (passive)	z_p	=	**-2.00**	m+NAP
Excavation allowance	ΔH_p	=	**0.50**	m+NAP
surface level (passive design)+tollerance	z_p	=	-2.50	m+NAP
(Ground)waterlevel (passive)	gws_p	=	**0.07**	m+NAP
(Ground)waterlevel (active)	gws_a	=	**0.07**	m+NAP
Thickness wall	t_w	=	**0.350**	m
Thickness base	t_b	=	**0.350**	m
Width heel	Bt	=	**2.025**	m
Width heel	Bh	=	**0.625**	m
Total width	B	=	3.00	m
Slope backfill	β_a	=	**0.00**	°
Slope toe	β_p	=	**0.00**	°

Waterlevel additions

Groundwaterlevel (active)	Δa_a	=	**0.25**	m
Groundwaterlevel (passive)	Δa_p	=	**0.05**	m



Material properties L-wall and fill

Weight density fill	$\gamma_{k;fill}$	=	**18.0**	kN/m³
Saturated weight density fill	$\gamma_{k;fill;sat}$	=	**21.0**	kN/m³
Angle of internal friction fill material	$\varphi_{a;k}$	=	**37.5**	°
Weight density concrete	$\gamma_{k;concrete}$	=	**24.0**	kN/m³

External forces

Surcharge load variable	$F_{k;sur}$	=	**15.00**	kN/m²
Partial factor variable load	$\gamma_{Q;var}$	=	**1.50**	-
External horizontal force variable	$F_{h;d}$		**40.00**	kN/m²
Elevation horizontal force	$z_{F;h;k}$	=	**1.50**	m+NAP

Horizontal soil pressure

Passive earh pressure mobilisation	$R_{Kp;k}$	=	**100%**

Forces on gravity wall

Horizontal Forces	Fh;d [kN/m]	arm [m]	M [kNm/m]
Active water pressure d	106.7	1.54	164.4
Active ground pressure d	100.9	2.44	246.2
Passive water pressure d	-93.3	1.44	-134.4
Passive ground pressure d	-89.0	0.67	-59.3
External horizontal force d	60.0	6.0	360.0
Σ	85.3		576.9

Vertical Forces	Fv;d;min [kN/m]	arm [m]
Concrete	-66.1	-0.5
Fill material active	-210.5	0.5
Free water + fill Passive	-34.2	-1.2
Variable Surcharge	0.0	0.5
Waterpressure uplift	134.1	0.02
Σ	-176.7	

loads horizontal

design horizontal load	H_d	=	85.3	kN/m

total moment

total moment, width direction	$M_{d;B}$	=	547.31	kNm/m

loads vertical

design vertical load (unfavourab =	251.9	kN/m
design vertical load (favourable) =	176.7	kN/m
chosen design vertical load	*251.858*	*kN/m*

normative vertical load (unfavourable/favourable)

show intermediate calculations f **unfavourable**

#DIV/0!

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 4 (L-wall)**
Consultant	**L.Jorissen**
Date	**25-5-2021**

soil profile

surface level	$z_{surface}$	=	-2.50	m+NAP	effective foundation area			
foundation level	$z_{foundation}$	=	-4.50	m+NAP	effective width at foundation lev	=	0.00	m
phreatic level	$z_{phreatic}$	=	-0.18	m+NAP	effective foundation area	=	0.00	m²/m
					>> WATCH OUT: effective foundation area = 0.00m²/m			

soil layers between surface level and influence level

layer nr.	soil type	top [m+NAP]	bottom [m+NAP]	thickness [m]	φ'_k [°]	c'_k [kPa]	$c_{u;k}$ [kPa]	γ_k [kN/m³]	$\gamma_{sat;k}$ [kN/m³]	γ'_d [kN/m³]	X	$H \cdot X \cdot c'_d$	$H \cdot X \cdot \varphi'_d$	$H \cdot X \cdot \gamma'_d$
1	**sand**	-2.50	**-4.50**	2.00	**33.5**	**0.0**	**0.0**	**18.0**	**20.0**	8.18	0.00	#DIV/0!	#DIV/0!	#DIV/0!
2	**sand**	-4.50	**-12.00**	7.50	**33.5**	**0.0**	**0.0**	**18.0**	**20.0**	8.18	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
3														
4														
5														
6														
7														
8														
9														
10														
	partial safety factors (material)				**1.25**	**1.50**	**1.50**	**1.10**			Σ	#DIV/0!	#DIV/0!	#DIV/0!

#DIV/0!

CALCULATIONS

dimensions slip plane

friction angle below foundation level	φ'_k	=	#DIV/0!	°
influence width slip plane	a_e	=	#DIV/0!	m
influence depth slip plane	z_e	=	#DIV/0!	m
influence level	d_e	=	#DIV/0!	m+NAP

calculation cases drained/undrained conditions

drained conditions according to §6.5.2.2(h)	#DIV/0!
#DIV/0!	#DIV/0!
undrained conditions according to §6.5.2.2(	n.a.
--	--

drained condition

effective vertical stress at foundation level	$\sigma'_{v;z;d}$	=	16.36	kPa
effective equivalent cohesion	$c'_{equ;d}$	=	#DIV/0!	kPa
effective equivalent angle of internal friction	$\varphi'_{equ;d}$	=	#DIV/0!	°
effective equivalent weight density	$\gamma'_{equ;d}$	=	#DIV/0!	kN/m³
bearing factor - cohesion	N_c	=	#DIV/0!	-
bearing factor - surcharge	N_q	=	#DIV/0!	-
bearing factor - effective unit weight	$N_{\gamma'}$	=	#DIV/0!	-
shape factor - cohesion	s_c	=	#DIV/0!	-
shape factor - surcharge	s_q	=	1.00	-
shape factor - effectieve unit weight	$s_{\gamma'}$	=	1.00	-
reduction factor load inclination - cohesion	i_c	=	#DIV/0!	-
reduction factor load inclination - surcharge	i_q	=	#DIV/0!	-
reduction factor load inclination - eff. unit weight	$i_{\gamma'}$	=	#DIV/0!	-
correction factor ground inclination - cohesion	λ_c	=	#DIV/0!	-
correction factor ground inclination - surcharge	λ_q	=	1.00	-
correction factor ground inclination - eff. unit weight	$\lambda_{\gamma'}$	=	1.00	-
maximum foundation pressure	$\sigma'_{max;d}$	=	#DIV/0!	kPa

<u>verification vertical bearing capacity</u>

vertical load (unfavourable)	=	251.9	kN/m
vertical bearing capacity drainec	=	#DIV/0!	kN/m
unity check ($V_d/V_{R;dr;d}$) ≤ 1,0	=	#DIV/0!	-

<u>verification horizontal shear resistance</u>

horizontal load	=	85.3	kN/m
δ / φ'	=	0.67	-
interface friction angle (with φ'k	=	18.6	°
friction coefficient	=	0.34	-
shear resistance drained	=	84.8	kN/m
unity check ($H_d/H_{R;dr;d}$) ≤ 1,0	=	1.01	-

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 4 (L-wall)**
Consultant	**L.Jorissen**
Date	**25-5-2021**

tilting stability

eccentricity width direction	e_B	=	1.50	m
⅓ width foundation	⅓ · B	=	1.00	m
unity check (e_B / (⅓ · B)) ≤ 1,0		=	1.50	-

SUMMARY

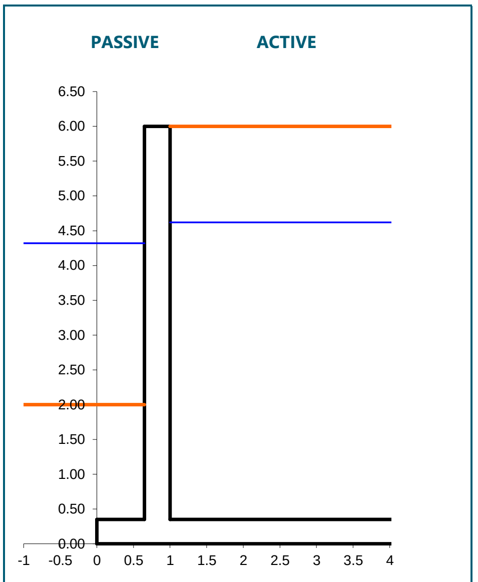
verification	situation	vertical load	load	resistance	nity check	check
verification vertical bearing capacity	drained	unfavourable	251.9	#DIV/0!	#DIV/0!	#DIV/0!
		favourable	176.7	#DIV/0!	#DIV/0!	#DIV/0!
	drained (punch)	unfavourable	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
		favourable	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
	undrained	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
verification horizontal shear resistance	drained	unfavourable	85.3	84.8	1.01	**not ok**
		favourable	85.3	59.5	1.43	**not ok**
	drained (punch)	unfavourable	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
		favourable	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
	undrained	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
verification tilting stability		unfavourable	1.50	1.00	1.50	**not ok**
		favourable	1.50	1.00	1.50	**not ok**

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 4 (L-wall)**
Consultant	**L.Jorissen**
Date	**25-5-2021**

TITLE

Calculation of gravity wall in accordance with Eurocode 7 (EN 1997-1) with
Dutch National Annex (NEN-EN 1997-1/NB) and Dutch complementary rules for application of Eurocode 7 (NEN 9997-1).

STARTING POINTS



general

language		=	**EN**	
reference level		=	**NAP**	
reliability class		=	**RC2**	
unit weight water	$\gamma_{w;d}$	=	**10.0**	kN/m³

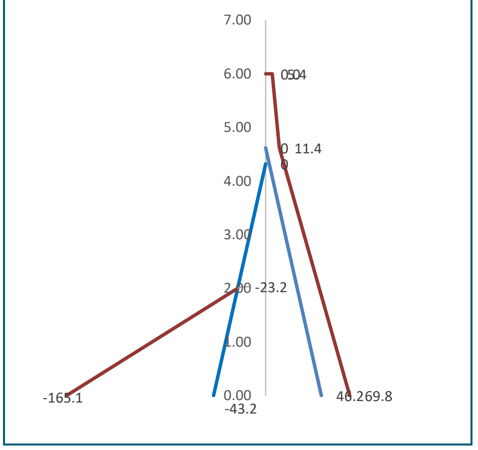
geometry

Prefab / in-situ			**in-situ**	
Top level gravity wall	z_{top}	=	**1.50**	m+NAP
Bottom level gravity wall	z_{bottom}	=	**-4.50**	m+NAP
Height gravity wall	H	=	6.00	m
Retaining height	h	=	4.00	m
surface level (passive)	z_p	=	**-2.00**	m+NAP
Excavation allowance	ΔH_p	=	**0.50**	m+NAP
surface level (passive design)+tollerance	z_p	=	-2.50	m+NAP
(Ground)waterlevel (passive)	gws_p	=	**0.07**	m+NAP
(Ground)waterlevel (active)	gws_a	=	**0.07**	m+NAP
Thickness wall	t_w	=	**0.350**	m
Thickness base	t_b	=	**0.350**	m
Width heel	Bt	=	**4.000**	m
Width heel	Bh	=	**0.650**	m
Total width	B	=	5.00	m
Slope backfill	β_a	=	**0.00**	°
Slope toe	β_p	=	**0.00**	°

Check dimensions, maximum width of gravity wall is 3,0m

Waterlevel additions

Groundwaterlevel (active)	Δa_a	=	**0.25**	m
Groundwaterlevel (passive)	Δa_p	=	**0.05**	m



Material properties L-wall and fill

Weight density fill	$\gamma_{k;fill}$	=	**18.0**	kN/m³
Saturated weight density fill	$\gamma_{k;fill;sat}$	=	**21.0**	kN/m³
Angle of internal friction fill material	$\varphi_{a;k}$	=	**37.5**	°
Weight density concrete	$\gamma_{k;concrete}$	=	**24.0**	kN/m³

External forces

Surcharge load variable	$F_{k;sur}$	=	**15.00**	kN/m²
Partial factor variable load	$\gamma_{Q;var}$	=	**1.50**	-
External horizontal force variable	$F_{h;d}$		**40.00**	kN/m²
Elevation horizontal force	$z_{F;h;k}$	=	**1.50**	m+NAP

Horizontal soil pressure

Passive earh pressure mobilisation	$R_{Kp;k}$	=	**100%**

Forces on gravity wall

Horizontal Forces	Fh;d [kN/m]	arm [m]	M [kNm/m]
Active water pressure d	106.7	1.54	164.4
Active ground pressure d	92.2	2.44	225.0
Passive water pressure d	-93.3	1.44	-134.4
Passive ground pressure d	-121.9	0.67	-81.3
External horizontal force d	60.0	6.0	360.0
Σ	43.7		533.8

Vertical Forces	Fv;d;min [kN/m]	arm [m]
Concrete	-81.3	-0.9
Fill material active	-415.9	0.5
Free water + fill Passive	-35.6	-2.2
Variable Surcharge	0.0	0.5
Waterpressure uplift	223.5	0.03
Σ	-309.2	

loads horizontal

design horizontal load H_d = 43.7 kN/m

total moment

total moment, width direction $M_{d;B}$ = 481.69 kNm/m

loads vertical

design vertical load (unfavourab = 451.0 kN/m
design vertical load (favourable) = 309.2 kN/m
chosen design vertical load 451.003 kN/m

normative vertical load (unfavourable/favourable)
show intermediate calculations f **unfavourable**
vertical load favourable is normative (tilting stability, , u.c. = 0.93)

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
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soil profile

surface level	$z_{surface}$	=	-2.50	m+NAP	effective foundation area			
foundation level	$z_{foundation}$	=	-4.50	m+NAP	effective width at foundation lev	=	2.86	m
phreatic level	$z_{phreatic}$	=	-0.18	m+NAP	effective foundation area	=	2.86	m^2/m

soil layers between surface level and influence level

layer nr.	soil type	top [m+NAP]	bottom [m+NAP]	thickness [m]	φ'_k [°]	c'_k [kPa]	$c_{u;k}$ [kPa]	γ_k [kN/m³]	$\gamma_{sat;k}$ [kN/m³]	γ'_d [kN/m³]	X	$H \cdot X \cdot c'_d$	$H \cdot X \cdot \varphi'_d$	$H \cdot X \cdot \gamma'_d$
1	**sand**	-2.50	**-4.50**	2.00	**33.5**	**0.0**	**0.0**	**18.0**	**20.0**	8.18	0.00	0.00	0.00	0.00
2	**sand**	-4.50	**-12.00**	7.50	**33.5**	**0.0**	**0.0**	**18.0**	**20.0**	8.18	2.41	0.00	324.94	95.29
3														
4														
5														
6														
7														
8														
9														
10														
	partial safety factors (material)				**1.25**	**1.50**	**1.50**	**1.10**			Σ	0.00	324.94	95.29

CALCULATIONS

dimensions slip plane

friction angle below foundation level	φ'_k	=	33.5	°
influence width slip plane	a_e	=	14.14	m
influence depth slip plane	z_e	=	4.83	m
influence level	d_e	=	-9.33	m+NAP

calculation cases drained/undrained conditions

drained conditions according to §6.5.2.2(h)	case a
--	--
undrained conditions according to §6.5.2.2(	n.a.
--	--

drained condition

effective vertical stress at foundation level	$\sigma'_{v;z;d}$	=	16.36	kPa
effective equivalent cohesion	$c'_{equ;d}$	=	0.00	kPa
effective equivalent angle of internal friction	$\varphi'_{equ;d}$	=	27.9	°
effective equivalent weight density	$\gamma'_{equ;d}$	=	8.18	kN/m³
bearing factor - cohesion	N_c	=	25.61	-
bearing factor - surcharge	N_q	=	14.56	-
bearing factor - effective unit weight	$N_{\gamma'}$	=	14.36	-
shape factor - cohesion	s_c	=	1.00	-
shape factor - surcharge	s_q	=	1.00	-
shape factor - effectieve unit weight	$s_{\gamma'}$	=	1.00	-
reduction factor load inclination - cohesion	i_c	=	0.80	-
reduction factor load inclination - surcharge	i_q	=	0.81	-
reduction factor load inclination - eff. unit weight	$i_{\gamma'}$	=	0.74	-
correction factor ground inclination - cohesion	λ_c	=	1.00	-
correction factor ground inclination - surcharge	λ_q	=	1.00	-
correction factor ground inclination - eff. unit weight	$\lambda_{\gamma'}$	=	1.00	-
maximum foundation pressure	$\sigma'_{max;d}$	=	316.9	kPa

verification vertical bearing capacity

vertical load (unfavourable)	=	451.0	kN/m
vertical bearing capacity drainec	=	907.6	kN/m
unity check ($V_d/V_{R;dr;d}$) ≤ 1,0	=	0.50	-

verification horizontal shear resistance

horizontal load	=	43.7	kN/m
δ / φ'	=	1.00	-
interface friction angle (with φ'k	=	27.9	°
friction coefficient	=	0.53	-
shear resistance drained	=	238.8	kN/m
unity check ($H_d/H_{R;dr;d}$) ≤ 1,0	=	0.18	-

Project	**Saba Black Rock Harbour**
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Consultant	**L.Jorissen**
Date	**25-5-2021**

tilting stability

eccentricity width direction	e_B	=	1.07	m
⅓ width foundation	⅓ · B	=	1.67	m
unity check (e_B / (⅓ · B)) ≤ 1,0		=	0.64	-

SUMMARY

verification	situation	vertical load	load	resistance	nity check	check
verification vertical bearing capacity	drained	unfavourable	451.0	907.6	0.50	**ok**
		favourable	309.2	460.6	0.67	**ok**
	drained (punch)	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
	undrained	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
verification horizontal shear resistance	drained	unfavourable	43.7	238.8	0.18	**ok**
		favourable	43.7	163.7	0.27	**ok**
	drained (punch)	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
	undrained	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
verification tilting stability		unfavourable	1.07	1.67	0.64	**ok**
		favourable	1.56	1.67	0.93	**ok**

Report for D-Sheet Piling 19.2

Design of Diaphragm and Sheet Pile Walls
Developed by Deltares

Company:	Witteveen+Bos
Date of report:	6/8/2021
Time of report:	4:19:45 PM
Report with version:	19.2.3.26114
Date of calculation:	6/8/2021
Time of calculation:	4:19:36 PM
Calculated with version:	19.2.3.26114
File name:	D:\..\4-L-wall\Saba_Black Rock harbour_L-wall_Coeficients L2
Project identification:	Saba Black Rock Harbour Section 4 (L-wall)

Verification according to National Annex of Eurocode 7 in the Netherlands (NEN 9997-1:2016)

1 Summary

1.1 Overview per Stage and Test

Stage nr.	Verification type	Displace- ment [mm]	Moment [kNm]	Shear force [kN]	Mob. perc. moment [%]	Mob. perc. resistance [%]	Vertical balance
1	EC7(NL)-Step 6.1		-0,24	-0,22	0,0	11,3	Upwards
1	EC7(NL)-Step 6.2		-0,11	-0,17	0,0	11,3	Upwards
1	EC7(NL)-Step 6.3		0,47	0,38	0,0	11,4	Upwards
1	EC7(NL)-Step 6.4		0,22	0,29	0,0	11,4	Upwards
1	EC7(NL)-Step 6.5	0,0	0,00	0,00	0,0	7,4	Sufficient
1	EC7(NL)-Step 6.5 * 1,200		0,00	0,00			
2	EC7(NL)-Step 6.1		-9,70	-7,44	0,0	15,6	Upwards
2	EC7(NL)-Step 6.2		-9,15	-7,40	0,0	15,6	Upwards
2	EC7(NL)-Step 6.3		-10,65	-8,01	0,0	16,3	Upwards
2	EC7(NL)-Step 6.4		-10,02	-7,93	0,0	16,4	Upwards
2	EC7(NL)-Step 6.5	0,4	-5,72	-4,87	0,0	10,0	Sufficient
2	EC7(NL)-Step 6.5 * 1,200		-6,86	-5,84			
3	EC7(NL)-Step 6.1		-9,75	-7,48	14,3	15,4	Upwards
3	EC7(NL)-Step 6.2		-9,16	-7,42	14,3	15,4	Upwards
3	EC7(NL)-Step 6.3		-10,74	-8,05	14,7	15,9	Upwards
3	EC7(NL)-Step 6.4		-10,03	-7,95	14,7	15,9	Upwards
3	EC7(NL)-Step 6.5	0,4	-5,72	-4,87	9,4	10,0	Sufficient
3	EC7(NL)-Step 6.5 * 1,200		-6,86	-5,84			
4	EC7(NL)-Step 6.1		37,94	25,72	25,6	28,0	Not sufficient
4	EC7(NL)-Step 6.2		30,63	-26,55	26,2	29,0	Not sufficient
4	EC7(NL)-Step 6.3		47,09	31,21	27,5	30,2	Not sufficient
4	EC7(NL)-Step 6.4		38,24	-30,85	28,1	31,3	Not sufficient
4	EC7(NL)-Step 6.5	2,0	19,73	-17,44	15,4	16,9	Not sufficient
4	EC7(NL)-Step 6.5 * 1,200		23,68	-20,93			
5	EC7(NL)-Step 6.1		54,27	38,67	29,9	32,8	Not sufficient
5	EC7(NL)-Step 6.2		43,63	-34,91	30,7	34,2	Not sufficient
5	EC7(NL)-Step 6.3		64,53	44,56	31,8	35,0	Not sufficient
5	EC7(NL)-Step 6.4		52,26	39,86	32,8	36,6	Not sufficient
5	EC7(NL)-Step 6.5	2,4	43,37	40,84	17,8	19,8	Not sufficient
5	EC7(NL)-Step 6.5 * 1,200		52,04	49,00			
6	EC7(NL)-Step 6.1		95,22	79,98	39,8	43,6	Upwards
6	EC7(NL)-Step 6.2		74,35	72,67	41,6	46,4	Upwards
6	EC7(NL)-Step 6.3		107,96	86,00	41,8	45,8	Upwards
6	EC7(NL)-Step 6.4		85,82	78,32	**43,8**	**48,7**	Upwards
6	EC7(NL)-Step 6.5	**9,0**	-43,04	60,40	23,9	27,3	Upwards
6	EC7(NL)-Step 6.5 * 1,200		-51,65	72,48			
7	EC7(NL)-Step 6.1		104,64	83,23	36,4	40,1	Upwards
7	EC7(NL)-Step 6.2		79,83	73,93	37,9	42,4	Upwards
7	EC7(NL)-Step 6.3		**117,23**	**89,20**	38,4	42,1	Upwards
7	EC7(NL)-Step 6.4		91,26	79,52	40,0	44,7	Upwards
7	EC7(NL)-Step 6.5	8,0	66,21	58,58	31,9	36,2	Upwards
7	EC7(NL)-Step 6.5 * 1,200		79,45	70,30			
Max		**9,0**	**117,23**	**89,20**	**43,8**	**48,7**	Not sufficient

1.2 Anchors and Struts

Stage nr.	Verification type	Anchor/strut Ankerscherm	
		Force [kN]	State
3	EC7(NL)-Step 6.1	-	
3	EC7(NL)-Step 6.2	-	
3	EC7(NL)-Step 6.3	-	

Stage nr.	Verification type	Anchor/strut Ankerscherm	
		Force [kN]	State
3	EC7(NL)-Step 6.4	-	
3	EC7(NL)-Step 6.5 x 1,200	-	
4	EC7(NL)-Step 6.1	28,10	Elastic
4	EC7(NL)-Step 6.2	25,21	Elastic
4	EC7(NL)-Step 6.3	33,59	Elastic
4	EC7(NL)-Step 6.4	30,12	Elastic
4	EC7(NL)-Step 6.5 x 1,200	21,86	Elastic
5	EC7(NL)-Step 6.1	45,45	Elastic
5	EC7(NL)-Step 6.2	41,33	Elastic
5	EC7(NL)-Step 6.3	51,35	Elastic
5	EC7(NL)-Step 6.4	46,65	Elastic
5	EC7(NL)-Step 6.5 x 1,200	16,73	Elastic
6	EC7(NL)-Step 6.1	119,62	Elastic
6	EC7(NL)-Step 6.2	112,30	Elastic
6	EC7(NL)-Step 6.3	**125,64**	Elastic
6	EC7(NL)-Step 6.4	117,96	Elastic
6	EC7(NL)-Step 6.5 x 1,200	105,97	Elastic
7	EC7(NL)-Step 6.1	113,04	Elastic
7	EC7(NL)-Step 6.2	109,55	Elastic
7	EC7(NL)-Step 6.3	119,06	Elastic
7	EC7(NL)-Step 6.4	115,21	Elastic
7	EC7(NL)-Step 6.5 x 1,200	103,03	Elastic
Max		**125,64**	

Due to multiplication of the representative value a force bigger than yield or buckling force may be present.

1.3 Overall Stability per Stage

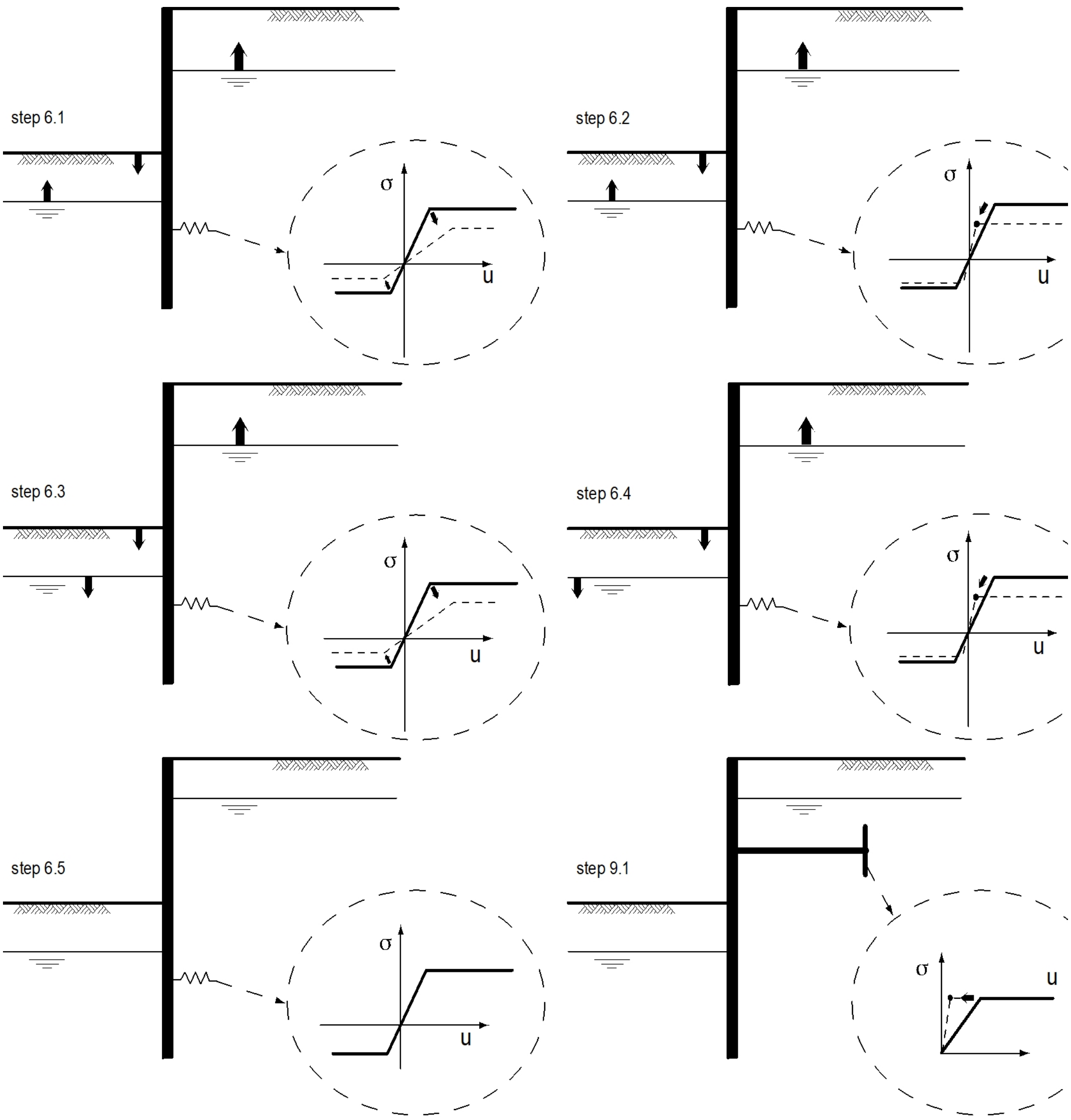
Stage name	Stability factor [-]
Initial	3573,87
Excavation for the anchor	7,39
Install anchor	7,39
Excavate seaside	2,80
Utilisation phase	2,46
Utilisation phase (ships)	2,03
Utilisation phase (earthquake)	2,21

1.4 Warnings

Lambda
In a verification the Ka, Ko and Kp are recalculated with reduced phi and delta's. This was impossible with the next material(s) while they have a manual given Ka, Ko and Kp.
BoulderGravelCobbel 1 (f)
Sand-dense 1 (f)
BoulderGravelCobbel 2 (f)
Sand-dense 2 (f)
Sand-dense 3 (f)
BoulderGravelCobbel 3 (f)
Sand-dense 4 (f)
Sand-medium dense (f)

* Vertical balance: The resultant vertical friction force is directed upward in stage 6, 7, 1, 2, 3, 6, 7, 1, 2, 3, 6, 7, 1, 2, 3, 6, 7, 1, 2, 3, 6 & 7
because the friction force on the passive side exceeds that on the active side.
This might be prevented by reducing the friction angle Delta on the passive side.

1.5 CUR Verification Steps



End of Report

Report for D-Sheet Piling 19.2

Design of Diaphragm and Sheet Pile Walls
Developed by Deltares

Company:	Witteveen+Bos
Date of report:	6/8/2021
Time of report:	4:21:35 PM
Report with version:	19.2.3.26114
Date of calculation:	6/8/2021
Time of calculation:	4:21:05 PM
Calculated with version:	19.2.3.26114
File name:	D:\..\Saba_Black Rock harbour_L-wall_Coeficients L2_ankerscherm
Project identification:	Saba Black Rock Harbour Section 4 (L-wall)

Verification according to National Annex of Eurocode 7 in the Netherlands (NEN 9997-1:2016)

1 Summary

1.1 Overview per Stage and Test

Stage nr.	Verification type	Displace-ment [mm]	Moment [kNm]	Shear force [kN]	Mob. perc. moment [%]	Mob. perc. resistance [%]	Vertical balance
1	EC7(NL)-Step 6.1		0,00	0,00	**0,0**	10,9	Sufficient
1	EC7(NL)-Step 6.2		0,00	0,00	**0,0**	10,9	Sufficient
1	EC7(NL)-Step 6.3		0,00	0,00	**0,0**	10,9	Sufficient
1	EC7(NL)-Step 6.4		0,00	0,00	**0,0**	10,9	Sufficient
1	EC7(NL)-Step 6.5	0,0	0,00	0,00	**0,0**	7,2	Sufficient
1	EC7(NL)-Step 6.5 * 1,200		0,00	0,00			
2	EC7(NL)-Step 6.1		**-57,43**	**-124,24**	**0,0**	**43,9**	Upwards
2	EC7(NL)-Step 6.2		-55,54	-124,05	**0,0**	43,6	Upwards
2	EC7(NL)-Step 6.3		**-57,43**	**-124,24**	**0,0**	**43,9**	Upwards
2	EC7(NL)-Step 6.4		-55,54	-124,05	**0,0**	43,6	Upwards
2	EC7(NL)-Step 6.5	2,2	34,38	-68,92	**0,0**	21,5	Upwards
2	EC7(NL)-Step 6.5 * 1,200		41,26	-82,70			
3	EC7(NL)-Step 6.1		-54,86	-122,96	**0,0**	36,1	Upwards
3	EC7(NL)-Step 6.2		-53,39	-122,97	**0,0**	36,0	Upwards
3	EC7(NL)-Step 6.3		-54,86	-122,96	**0,0**	36,1	Upwards
3	EC7(NL)-Step 6.4		-53,39	-122,97	**0,0**	36,0	Upwards
3	EC7(NL)-Step 6.5	**2,3**	34,43	-68,71	**0,0**	23,9	Upwards
3	EC7(NL)-Step 6.5 * 1,200		41,31	-82,45			
Max		**2,3**	**-57,43**	**-124,24**	**0,0**	**43,9**	Sufficient

1.2 Overall Stability per Stage

Stage name	Stability factor [-]
Initial	10000,00
Anchor utilisation phase (normal)	4,91
Anchor utilisation phase (earthquake)	4,91

1.3 Warnings

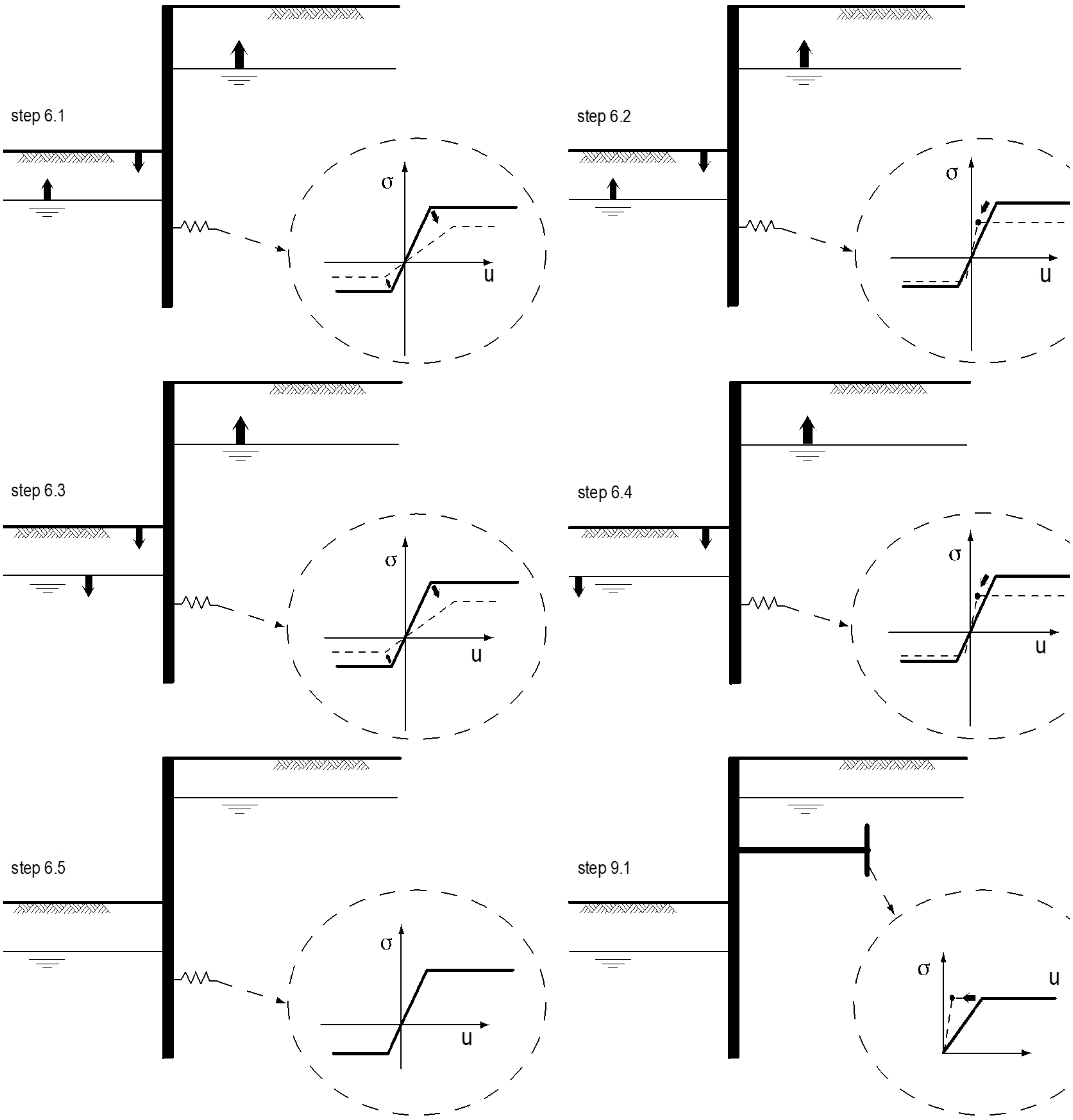
Lambda

In a verification the Ka, Ko and Kp are recalculated with reduced phi and delta's. This was impossible with the next material(s) while they have a manual given Ka, Ko and Kp.

BoulderGravelCobbel 1 (f)
Sand-dense 1 (f)
BoulderGravelCobbel 2 (f)
Sand-dense 2 (f)
Sand-dense 3 (f)
BoulderGravelCobbel 3 (f)
Sand-dense 4 (f)
Sand-medium dense (f)

* Vertical balance: The resultant vertical friction force is directed upward in stage 2, 3, 2, 3, 2, 3, 2, 3, 2 & 3 because the friction force on the passive side exceeds that on the active side. This might be prevented by reducing the friction angle Delta on the passive side.

1.4 CUR Verification Steps



End of Report

Project	**Saba Black Rock harbour**
Projectcode	**118421**
Subject	**Section 4 (Inner harbour wall)**
Consultant	**L.Jorissen**
Date	**08-06-2021**

TITLE

Calculation of anchor design values.

STARTING POINTS

general

language		=	**EN**	

design parameters

horizontal length behind sheet pile wall	L_{hor}	=	**2.8**	m
maximum force from anchors (ULS)	F_{max}	=	**125.64**	kN/m
yield characteristic thread	$F_{yt;char}$	=	**355**	kN
tensile characteristic thread	$F_{tt;char}$	=	**510**	kN
factor kt	kt	=	0.9	
factor gm;0	$g_{m;0}$	=	1.0	
factor gm;2	$g_{m;2}$	=	1.1	
factor gm;ser	$g_{m;ser}$	=	1.25	

anchor catalogue

Withworth DIN thread		Tensile stress area thread	Tensile stress area thread	Gross cross-section area shaft	Gross cross-section area shaft	Yield characteristic thread	Tensile characteristic thread	Yield characteristic shaft	Tensile characteristic shaft	Tensile thread	Tensile shaft	Design force (ULS)	Design force (SLS)	Tie rod failure	Design force (per m wall)	Design area (per m wall)	Design area model 2.8 ctc spacing
D		Ds	As	Dg	Ag	Fyt;char	Ftt;char	Fyg;char	Ftg;char	Ftt;Rd	Ftg;Rd	Ft;Rd	Ft;ser	fa	Ft;Rd		
[inch]	[mm]	[mm]	[mm²]	[mm]	[mm²]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN/m]		
2.50	63.50	57.57	2603	59	2734	924	1328	971	1394	956	971	956	840	1195	341	976	930
3.00	76.20	69.43	3786	71	3959	1344	1931	1406	2019	1390	1406	1390	1222	1738	496	1414	1352
3.50	88.90	81.60	5230	83	5411	1857	2667	1921	2759	1921	1921	1921	1688	2401	686	1932	1868
4.00	101.60	93.76	6904	96	7238	2451	3521	2570	3691	2535	2570	2535	2228	3169	905	2585	2466
4.50	114.30	106.05	8834	108	9161	3136	4505	3252	4672	3244	3252	3244	2851	4055	1158	3272	3155

corrosion catalogue 1 mm radial

Project	**Saba Black Rock harbour**
Projectcode	**118421**
Subject	**Section 4 (Inner harbour wall)**
Consultant	**L.Jorissen**
Date	**08-06-2021**

Withworth DIN thread		Tensile stress area thread	Tensile stress area thread	Gross cross-section area shaft	Gross cross-section area shaft	Yield characteristic thread	Tensile characteristic thread	Yield characteristic shaft	Tensile characteristic shaft	Tensile thread	Tensile shaft	Design force (ULS)	Design force (SLS)	Tie rod failure	Design force (per m wall)	Design area (per m wall)	Design area model 2.8 ctc spacing
D		Ds	As	Dg	Ag	Fyt;char	Ftt;char	Fyg;char	Ftg;char	Ftt;Rd	Ftg;Rd	Ft;Rd	Ft;ser	fa	Ft;Rd		
[inch]	[mm]	[mm]	[mm^2]	[mm]	[mm^2]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN/m]		
2.50	61.50	55.57241	2426	57.00	2552	861	1237	906	1301	891	906	891	783	1113	318	911	866
3.00	74.20	67.4258062	3571	69.00	3739	1268	1821	1327	1907	1311	1327	1311	1152	1639	468	1335	1275
3.50	86.90	79.6041656	4977	81.00	5153	1767	2538	1829	2628	1828	1829	1828	1606	2284	653	1840	1777
4.00	99.60	91.7576822	6613	94.00	6940	2347	3372	2464	3539	2428	2464	2428	2134	3035	867	2478	2362
4.50	112.30	104.053111	8504	106.00	8825	3019	4337	3133	4501	3123	3133	3123	2744	3903	1115	3152	3037

corrosion catalogue 2 mm radial

Withworth DIN thread		Tensile stress area thread	Tensile stress area thread	Gross cross-section area shaft	Gross cross-section area shaft	Yield characteristic thread	Tensile characteristic thread	Yield characteristic shaft	Tensile characteristic shaft	Tensile thread	Tensile shaft	Design force (ULS)	Design force (SLS)	Tie rod failure	Design force (per m wall)	Design area (per m wall)	Design area model 2.8 ctc spacing
D		Ds	As	Dg	Ag	Fyt;char	Ftt;char	Fyg;char	Ftg;char	Ftt;Rd	Ftg;Rd	Ft;Rd	Ft;ser	fa	Ft;Rd		
[inch]	[mm]	[mm]	[mm^2]	[mm]	[mm^2]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN]	[kN/m]		
2.50	59.50	53.57	2254	55.00	2376	800	1150	843	1212	828	843	828	727	1035	296	849	805
3.00	72.20	65.43	3362	67.00	3526	1193	1715	1252	1798	1234	1252	1234	1085	1543	441	1259	1201
3.50	84.90	77.60	4730	79.00	4902	1679	2412	1740	2500	1737	1740	1737	1526	2171	620	1751	1689
4.00	97.60	89.76	6328	92.00	6648	2246	3227	2360	3390	2323	2360	2323	2042	2904	830	2374	2260
4.50	110.30	102.05	8180	104.00	8495	2904	4172	3016	4332	3004	3016	3004	2640	3755	1073	3034	2921

SUMMARY

new anchor cross section A_{design} = **8.05E-04** m^2/m

SPREADSHEET
Gazetas et al. (1990)
Degree of damage
versie 0.10 **[ongevalideerd]**

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 4 (Inner harbour wall)**
Consultatnt	**L.Jorissen**
Date	**8-6-2021**

Title

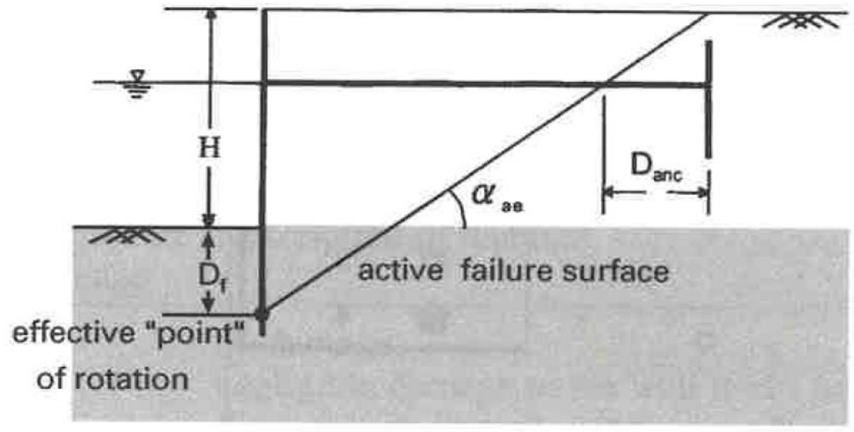
Calculation of the approximate degree of damage to anchored sheet pile walls during earthquakes according Gazetas et al. 1990

Seismic loads

Horizontal seismic coefficient	k_h	=	**0.275**	-
Vertical seismic coefficient	k_v	=	**0**	-
Equivalent seismic coefficient	k_e	=	0.22	-

Geometry

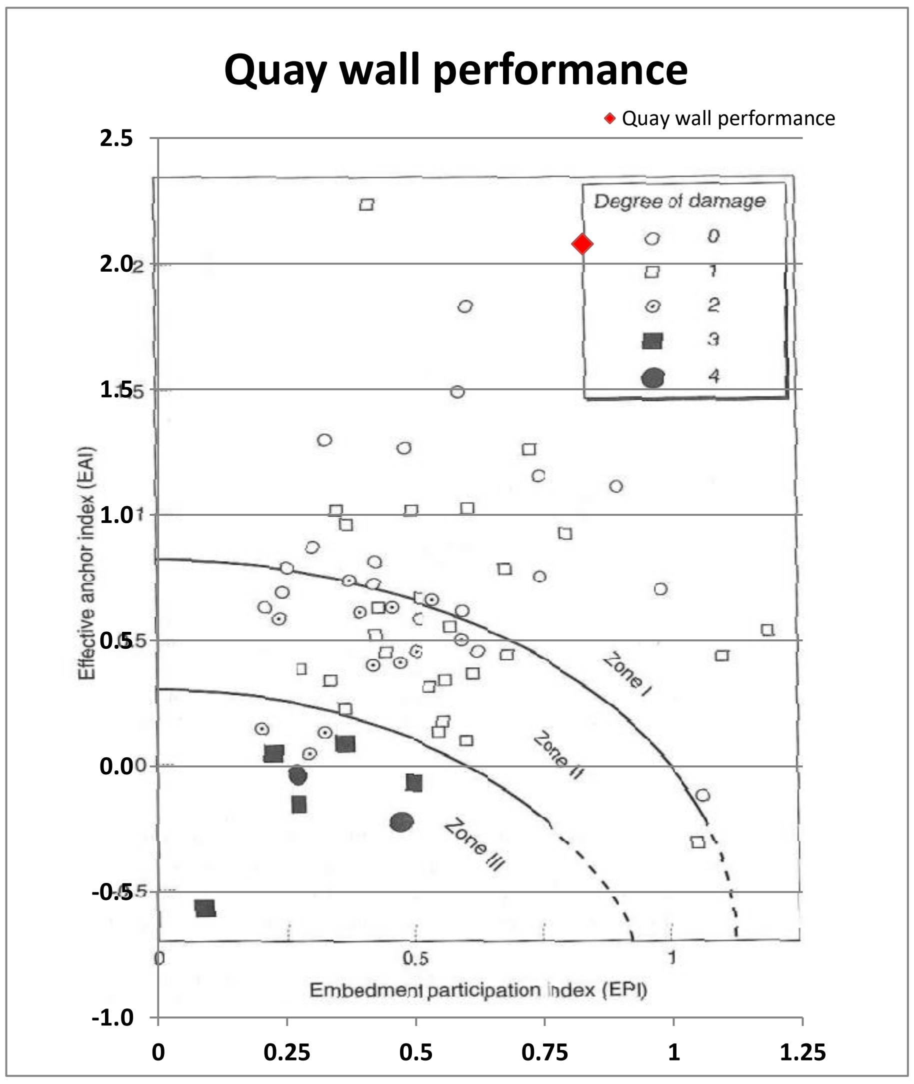
Retaining height	H	=	**4.5**	m
Water depth	H_{sub}	=	**4.5**	m
Imbedded length	D_{emb}	=	**6.5**	m
Anchor wall distance	L_{anchor}	=	**15**	m
Surface level	$z_{surface}$	=	**1.5**	m CD
Tierod level	z_{tierod}	=	**0.5**	m CD
Top level anhor wall	$z_{top\ anchor\ wall}$	=	**1**	m CD
Length anchor wall	L_{aw}	=	**9**	m
Inclination surface	β	=	**0**	°
Inclination sheet pile	θ	=	**0**	°



Soil

			active	passive	
Unit weigth	γ	=	**18**	**18**	kN/m³
Saturated unit weight	γ_{sat}	=	**20**	**20**	kN/m³
Internal friction angle	φ'	=	**35**	**35**	°
Friction anlge soil-wall	δ'	=	**23**	**23**	°

Unit weight water	γ_w	=	**10**	kN/m³



CALCULATIONS

Depth effective point of rotation	D_f	=	1.7	m
Active wedge height	h_{aw}	=	6.2	m
Effective point of rotation level	z_{Df}	=	-4.7	m CD
Angle active failure plane	α_{ae}	=	42.4	°
Active wedge height to tierod level	$z_{aw-tierod}$	=	5.2	m
Active failure length to tierod level	$L_{aw-tierod}$	=	5.6	m
Length active wedge anchor wall	D_{anc}	=	9.4	m

Layer	γ_b [kN/m³]	γ_e [kN/m³]	γ_{e-sat} [kN/m³]	Ψ [°]	K_{ae} [-]	K_{pe} [-]
Soil active side	10.0	10.0	20.0	23.44	0.68	
Soil passive side	10.0	10.0	20.0	23.44		6.08

Effective anchor index	EAI	=	2.078	-
Embedment participation index	EPI	=	0.823	-

Degree of damage zone	=	**1**	
Qualitative description damage	=	negligible damage to the wall itself, notaceable damage to the related structures (i.e. concrete apron)	
Quantitive description damage	=	10	cm permanent displacement at top of sheet pile

X

APPENDIX: GEOTECHNICAL VERIFICATIONS FOR SECTION 6: CENTRAL PIER

Witteveen+Bos

SPREADSHEET
SHALLOW FOUNDATIONS

version 1.60

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 6 (Central pier) during normal conditions**
Consultant	**L.Jorissen**
Date	**08-06-2021**

TITLE

Calculation of shallow foundations (strip foundation / plate foundation) in accordance with Eurocode 7 (EN 1997-1) with Dutch National Annex (NEN-EN 1997-1/NB) and Dutch complementary rules for application of Eurocode 7 (NEN 9097-1).

STARTING POINTS

geometry

type foundation			**strip foundation**	
prefab / in-situ			**prefab**	
width	B	=	**4.00**	m
--		=	--	
--		=	--	

general

language		=	**EN**	
reference level		=	**NAP**	
reliability class		=	**RC2**	
unit weight water	$\gamma_{w;d}$	=	**10**	kN/m^3

loads horizontal

design horizontal load	H_d	=	**20.0**	kN/m
eccentricity H	e_H	=	**0.00**	m
moment due to H	$M_{H;d}$	=	**20.0**	kNm/m
angle H with length direction	κ	=	**90**	°
moment due to H, width direction	$M_{H;d;B}$	=	20.0	kNm/m
--		=	--	

loads vertical

design vertical load (unfavourable)	$V_{d;max}$	=	**376.4**	kN/m
design vertical load (favourable)	$V_{d;min}$	=	**336.4**	kN/m
eccentricity V (width)	$e_{V;B}$	=	**0.10**	m
--		=	--	
moment due to V, width direction	$M_{V;d;B}$	=	37.6	kNm/m
--		=	--	

total moment

total moment, width direction	$M_{d;B}$	=	57.6	kNm/m
--		=	--	

normative vertical load (unfavourable/favourable)

show intermediate calculations for **unfavourable**

vertical load unfavourable is normative (vertical bearing capacity, drained, u.c. = 0.41)

soil profile

surface level	$z_{surface}$	=	**-4.50**	m+NAP
surface inclination (§6.5.2.2(p))	β	=	**0.00**	°
foundation level	$z_{foundation}$	=	**-4.80**	m+NAP
phreatic level	$z_{phreatic}$	=	**0.07**	m+NAP

effective foundation area

effective width at foundation level	B_{eff}	=	3.69	m
effective length at foundation level	L_{eff}	=	1.00	m/m
effective foundation area	A_{eff}	=	3.69	m²/m

soil layers between surface level and influence level

layer nr.	soil type	top [m+NAP]	bottom [m+NAP]	thickness [m]	φ'_k [°]	c'_k [kPa]	$c_{u;k}$ [kPa]	γ_k [kN/m^3]	$\gamma_{sat;k}$ [kN/m^3]	γ'_d [kN/m^3]	$\sigma'_{v;z;d}$ [kPa]	X	H·X	H·X·c'$_d$	H·X·φ'$_d$	H·X·γ'$_d$
1	**sand**	-4.50	**-4.80**	0.30	**33.5**	**0.0**	**0.0**	**18.0**	**20.0**	8.2	2.5	0.00	0.00	0.00	0.00	0.00
2	**sand**	-4.80	**-20.00**	15.20	**33.5**	**0.0**	**0.0**	**18.0**	**20.0**	8.2	126.8	3.21	20.57	0.00	593.94	168.27
3																
4																
5																
6																
7																
8																
9																
10																
	partial safety factors (material)				**1.20**	**1.50**	**1.50**	**1.10**				Σ	20.57	0.00	593.94	168.27

CALCULATIONS

dimensions slip plane

friction angle below foundation level	φ'_k	=	33.5	°
influence width slip plane	a_e	=	18.78	m
influence depth slip plane	z_e	=	6.41	m
influence level	d_e	=	-11.21	m+NAP

calculation cases drained/undrained conditions

drained conditions according to §6.5.2.2(h)	case a
--	--
undrained conditions according to §6.5.2.2(f)	n.a.
--	--

drained condition

effective vertical stress at foundation level	$\sigma'_{v;z;d}$	=	2.45	kPa
effective equivalent cohesion	$c'_{equ;d}$	=	0.00	kPa
effective equivalent angle of internal friction	$\varphi'_{equ;d}$	=	28.9	°
effective equivalent weight density	$\gamma'_{equ;d}$	=	8.18	kN/m^3
bearing factor - cohesion	N_c	=	27.60	-
bearing factor - surcharge	N_q	=	16.22	-
bearing factor - effective unit weight	$N_{\gamma'}$	=	16.79	-
shape factor - cohesion	s_c	=	1.00	-
shape factor - surcharge	s_q	=	1.00	-
shape factor - effectieve unit weight	$s_{\gamma'}$	=	1.00	-
reduction factor load inclination - cohesion	i_c	=	0.89	-
reduction factor load inclination - surcharge	i_q	=	0.89	-
reduction factor load inclination - eff. unit weight	$i_{\gamma'}$	=	0.85	-

<u>verification vertical bearing capacity</u>

vertical load (unfavourable)	V_d	=	376.4	kN/m
vertical bearing capacity drained	$V_{R;dr;d}$	=	927.1	kN/m
unity check ($V_d/V_{R;dr;d}$) ≤ 1,0		=	0.41	-

<u>verification horizontal shear resistance</u>

horizontal load	H_d	=	20.0	kN/m
δ / φ'	R_{int}	=	0.67	-
interface friction angle (with φ'k = 33.5°)	δ_d	=	19.3	°
friction coefficient	c_f	=	0.35	-
shear resistance drained	$H_{R;dr;d}$	=	131.5	kN/m
unity check ($H_d/H_{R;dr;d}$) ≤ 1,0		=	0.15	-

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
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Date	**08-06-2021**

correction factor ground inclination - cohesion	λ_c	=	1.00	-
correction factor ground inclination - surcharge	λ_q	=	1.00	-
correction factor ground inclination - eff. unit weight	$\lambda_{\gamma'}$	=	1.00	-
maximum foundation pressure	$\sigma'_{max;d}$	=	251.0	kPa

Project	**Saba Black Rock Harbour**
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Date	**08-06-2021**

tilting stability

width direction				length direction		
eccentricity width direction	e_B	=	0.15 m	n.a. → strip foundation	=	--
⅓ width foundation	⅓ · B	=	1.33 m	--	=	--
unity check (e_B / (⅓ · B)) ≤ 1,0		=	0.11 -	--	=	--

SUMMARY

verification	situation	vertical load	load	resistance	unity check	check
verification vertical bearing capacity	drained	unfavourable	376.4	927.1	0.41	**ok**
		favourable	336.4	903.7	0.37	**ok**
	drained (punch)	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
	undrained	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
	undrained (squeezing)	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
verification horizontal shear resistance	drained	unfavourable	20.0	131.5	0.15	**ok**
		favourable	20.0	117.5	0.17	**ok**
	drained (punch)	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
	undrained	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
verification tilting stability	width direction	unfavourable	0.15	1.33	0.11	**ok**
		favourable	0.16	1.33	0.12	**ok**
	length direction	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.

WITTEVEEN + BOS

SPREADSHEET
SHALLOW FOUNDATIONS

version 1.60

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 6 (Central pier) during hurricane conditions**
Consultant	**L.Jorissen**
Date	**08-06-2021**

TITLE

Calculation of shallow foundations (strip foundation / plate foundation) in accordance with Eurocode 7 (EN 1997-1) with Dutch National Annex (NEN-EN 1997-1/NB) and Dutch complementary rules for application of Eurocode 7 (NEN 9097-1).

STARTING POINTS

geometry

type foundation			**strip foundation**	
prefab / in-situ			**prefab**	
width	B	=	**4.00**	m
--		=	--	
--		=	--	

general

language		=	**EN**	
reference level		=	**NAP**	
reliability class		=	**RC2**	
unit weight water	$\gamma_{w;d}$	=	**10**	kN/m³

loads horizontal

design horizontal load	H_d	=	**53.2**	kN/m
eccentricity H	e_H	=	**0.00**	m
moment due to H	$M_{H;d}$	=	**0.0**	kNm/m
angle H with length direction	κ	=	**90**	°
moment due to H, width direction	$M_{H;d;B}$	=	0.0	kNm/m
--		=	--	

loads vertical

design vertical load (unfavourable)	$V_{d;max}$	=	**363.0**	kN/m
design vertical load (favourable)	$V_{d;min}$	=	**336.4**	kN/m
eccentricity V (width)	$e_{V;B}$	=	**0.10**	m
--		=	--	
moment due to V, width direction	$M_{V;d;B}$	=	36.3	kNm/m
--		=	--	

total moment

total moment, width direction	$M_{d;B}$	=	36.3	kNm/m
--		=	--	

normative vertical load (unfavourable/favourable)

show intermediate calculations for **unfavourable**

vertical load unfavourable is normative (vertical bearing capacity, drained, u.c. = 0.50)

soil profile

surface level	$z_{surface}$	=	**-4.50**	m+NAP
surface inclination (§6.5.2.2(p))	β	=	**0.00**	°
foundation level	$z_{foundation}$	=	**-4.80**	m+NAP
phreatic level	$z_{phreatic}$	=	**0.07**	m+NAP

effective foundation area

effective width at foundation level	B_{eff}	=	3.80	m
effective length at foundation level	L_{eff}	=	1.00	m/m
effective foundation area	A_{eff}	=	3.80	m²/m

soil layers between surface level and influence level

layer nr.	soil type	top [m+NAP]	bottom [m+NAP]	thickness [m]	φ'_k [°]	c'_k [kPa]	$c_{u;k}$ [kPa]	γ_k [kN/m³]	$\gamma_{sat;k}$ [kN/m³]	γ'_d [kN/m³]	$\sigma'_{v;z;d}$ [kPa]	X	H·X	H·X·c'd	H·X·φ'd	H·X·γ'd
1	**sand**	-4.50	**-4.80**	0.30	**33.5**	**0.0**	**0.0**	**18.0**	**20.0**	8.2	2.5	0.00	0.00	0.00	0.00	0.00
2	**sand**	-4.80	**-20.00**	15.20	**33.5**	**0.0**	**0.0**	**18.0**	**20.0**	8.2	126.8	3.09	19.12	0.00	552.16	156.43
3																
4																
5																
6																
7																
8																
9																
10																
	partial safety factors (material)				**1.20**	**1.50**	**1.50**	**1.10**				Σ	19.12	0.00	552.16	156.43

CALCULATIONS

dimensions slip plane

friction angle below foundation level	φ'_k	=	33.5	°
influence width slip plane	a_e	=	18.12	m
influence depth slip plane	z_e	=	6.18	m
influence level	d_e	=	-10.98	m+NAP

calculation cases drained/undrained conditions

drained conditions according to §6.5.2.2(h)	case a
--	--
undrained conditions according to §6.5.2.2(f)	n.a.
--	--

drained condition

effective vertical stress at foundation level	$\sigma'_{v;z;d}$	=	2.45	kPa
effective equivalent cohesion	$c'_{equ;d}$	=	0.00	kPa
effective equivalent angle of internal friction	$\varphi'_{equ;d}$	=	28.9	°
effective equivalent weight density	$\gamma'_{equ;d}$	=	8.18	kN/m³
bearing factor - cohesion	N_c	=	27.60	-
bearing factor - surcharge	N_q	=	16.22	-
bearing factor - effective unit weight	$N_{\gamma'}$	=	16.79	-
shape factor - cohesion	s_c	=	1.00	-
shape factor - surcharge	s_q	=	1.00	-
shape factor - effectieve unit weight	$s_{\gamma'}$	=	1.00	-
reduction factor load inclination - cohesion	i_c	=	0.70	-
reduction factor load inclination - surcharge	i_q	=	0.72	-
reduction factor load inclination - eff. unit weight	$i_{\gamma'}$	=	0.62	-

<u>verification vertical bearing capacity</u>

vertical load (unfavourable)	V_d	=	363.0	kN/m
vertical bearing capacity drained	$V_{R;dr;d}$	=	726.2	kN/m
unity check ($V_d/V_{R;dr;d}$) ≤ 1,0		=	0.50	-

<u>verification horizontal shear resistance</u>

horizontal load	H_d	=	53.2	kN/m
δ / φ'	R_{int}	=	0.67	-
interface friction angle (with φ'k = 33.5°)	δ_d	=	19.3	°
friction coefficient	c_f	=	0.35	-
shear resistance drained	$H_{R;dr;d}$	=	126.8	kN/m
unity check ($H_d/H_{R;dr;d}$) ≤ 1,0		=	0.42	-

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 6 (Central pier) during hurricane conditions**
Consultant	**L.Jorissen**
Date	**08-06-2021**

correction factor ground inclination - cohesion	λ_c	=	1.00	-
correction factor ground inclination - surcharge	λ_q	=	1.00	-
correction factor ground inclination - eff. unit weight	$\lambda_{\gamma'}$	=	1.00	-
maximum foundation pressure	$\sigma'_{max;d}$	=	191.1	kPa

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 6 (Central pier) during hurricane conditions**
Consultant	**L.Jorissen**
Date	**08-06-2021**

tilting stability

width direction				length direction		
eccentricity width direction	e_B	=	0.10 m	n.a. → strip foundation	=	--
⅓ width foundation	⅓ · B	=	1.33 m	--	=	--
unity check (e_B / (⅓ · B)) ≤ 1,0		=	0.08 -	--	=	--

SUMMARY

verification	situation	vertical load	load	resistance	unity check	check
verification vertical bearing capacity	drained	unfavourable	363.0	726.2	0.50	**ok**
		favourable	336.4	698.4	0.48	**ok**
	drained (punch)	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
	undrained	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
	undrained (squeezing)	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
verification horizontal shear resistance	drained	unfavourable	53.2	126.8	0.42	**ok**
		favourable	53.2	117.5	0.45	**ok**
	drained (punch)	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
	undrained	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
verification tilting stability	width direction	unfavourable	0.10	1.33	0.08	**ok**
		favourable	0.10	1.33	0.08	**ok**
	length direction	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.

SPREADSHEET
SHALLOW FOUNDATIONS

version 1.60

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 6 (Central pier) during Earthquake conditions**
Consultant	**L.Jorissen**
Date	**08-06-2021**

TITLE

Calculation of shallow foundations (strip foundation / plate foundation) in accordance with Eurocode 7 (EN 1997-1) with Dutch National Annex (NEN-EN 1997-1/NB) and Dutch complementary rules for application of Eurocode 7 (NEN 9097-1).

STARTING POINTS

geometry

type foundation			**strip foundation**	
prefab / in-situ			**prefab**	
width	B	=	**4.00**	m
--		=	--	
--		=	--	

general

language		=	**EN**	
reference level		=	**NAP**	
reliability class		=	**RC2**	
unit weight water	$\gamma_{w;d}$	=	**10**	kN/m^3

loads horizontal

design horizontal load	H_d	=	**53.2**	kN/m
eccentricity H	e_H	=	**0.00**	m
moment due to H	$M_{H;d}$	=	**0.0**	kNm/m
angle H with length direction	κ	=	**90**	°
moment due to H, width direction	$M_{H;d;B}$	=	0.0	kNm/m
--		=	--	

loads vertical

design vertical load (unfavourable)	$V_{d;max}$	=	**375.0**	kN/m
design vertical load (favourable)	$V_{d;min}$	=	**363.0**	kN/m
eccentricity V (width)	$e_{V;B}$	=	**0.10**	m
--		=	--	
moment due to V, width direction	$M_{V;d;B}$	=	37.5	kNm/m
--		=	--	

total moment

total moment, width direction	$M_{d;B}$	=	37.5	kNm/m
--		=	--	

normative vertical load (unfavourable/favourable)

show intermediate calculations for **unfavourable**

vertical load unfavourable is normative (vertical bearing capacity, drained, u.c. = 0.51)

soil profile

surface level	$z_{surface}$	=	**-4.50**	m+NAP
surface inclination (§6.5.2.2(p))	β	=	**0.00**	°
foundation level	$z_{foundation}$	=	**-4.80**	m+NAP
phreatic level	$z_{phreatic}$	=	**0.07**	m+NAP

effective foundation area

effective width at foundation level	B_{eff}	=	3.80	m
effective length at foundation level	L_{eff}	=	1.00	m/m
effective foundation area	A_{eff}	=	3.80	m²/m

soil layers between surface level and influence level

layer nr.	soil type	top [m+NAP]	bottom [m+NAP]	thickness [m]	φ'_k [°]	c'_k [kPa]	$c_{u;k}$ [kPa]	γ_k [kN/m^3]	$\gamma_{sat;k}$ [kN/m^3]	γ'_d [kN/m^3]	$\sigma'_{v;z;d}$ [kPa]	X	H·X	$H \cdot X \cdot c'_d$	$H \cdot X \cdot \varphi'_d$	$H \cdot X \cdot \gamma'_d$
1	**sand**	-4.50	**-4.80**	0.30	**33.5**	**0.0**	**0.0**	**18.0**	**20.0**	8.2	2.5	0.00	0.00	0.00	0.00	0.00
2	**sand**	-4.80	**-20.00**	15.20	**33.5**	**0.0**	**0.0**	**18.0**	**20.0**	8.2	126.8	3.10	19.25	0.00	555.87	157.48
3																
4																
5																
6																
7																
8																
9																
10																
	partial safety factors (material)				**1.20**	**1.50**	**1.50**	**1.10**				Σ	19.25	0.00	555.87	157.48

CALCULATIONS

dimensions slip plane

friction angle below foundation level	φ'_k	=	33.5	°
influence width slip plane	a_e	=	18.18	m
influence depth slip plane	z_e	=	6.20	m
influence level	d_e	=	-11.00	m+NAP

calculation cases drained/undrained conditions

drained conditions according to §6.5.2.2(h)	case a
--	--
undrained conditions according to §6.5.2.2(f)	n.a.
--	--

drained condition

effective vertical stress at foundation level	$\sigma'_{v;z;d}$	=	2.45	kPa
effective equivalent cohesion	$c'_{equ;d}$	=	0.00	kPa
effective equivalent angle of internal friction	$\varphi'_{equ;d}$	=	28.9	°
effective equivalent weight density	$\gamma'_{equ;d}$	=	8.18	kN/m^3
bearing factor - cohesion	N_c	=	27.60	-
bearing factor - surcharge	N_q	=	16.22	-
bearing factor - effective unit weight	N_{γ}	=	16.79	-
shape factor - cohesion	s_c	=	1.00	-
shape factor - surcharge	s_q	=	1.00	-
shape factor - effectieve unit weight	s_{γ}	=	1.00	-
reduction factor load inclination - cohesion	i_c	=	0.71	-
reduction factor load inclination - surcharge	i_q	=	0.73	-
reduction factor load inclination - eff. unit weight	i_{γ}	=	0.63	-

<u>verification vertical bearing capacity</u>

vertical load (unfavourable)	V_d	=	375.0	kN/m
vertical bearing capacity drained	$V_{R;dr;d}$	=	737.6	kN/m
unity check ($V_d/V_{R;dr;d}$) ≤ 1,0		=	0.51	-

<u>verification horizontal shear resistance</u>

horizontal load	H_d	=	53.2	kN/m
δ / φ'	R_{int}	=	0.67	-
interface friction angle (with φ'k = 33.5°)	δ_d	=	19.3	°
friction coefficient	c_f	=	0.35	-
shear resistance drained	$H_{R;dr;d}$	=	131.0	kN/m
unity check ($H_d/H_{R;dr;d}$) ≤ 1,0		=	0.41	-

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 6 (Central pier) during Earthquake conditions**
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Date	**08-06-2021**

correction factor ground inclination - cohesion	λ_c	=	1.00	-
correction factor ground inclination - surcharge	λ_q	=	1.00	-
correction factor ground inclination - eff. unit weight	$\lambda_{\gamma'}$	=	1.00	-
maximum foundation pressure	$\sigma'_{max;d}$	=	194.1	kPa

Project	**Saba Black Rock Harbour**
Projectcode	**118421**
Subject	**Section 6 (Central pier) during Earthquake conditions**
Consultant	**L.Jorissen**
Date	**08-06-2021**

tilting stability

width direction				length direction		
eccentricity width direction	e_B	=	0.10 m	n.a. → strip foundation	=	--
⅓ width foundation	⅓ · B	=	1.33 m	--	=	--
unity check (e_B / (⅓ · B)) ≤ 1,0		=	0.08 -	--	=	--

SUMMARY

verification	situation	vertical load	load	resistance	unity check	check
verification vertical bearing capacity	drained	unfavourable	375.0	737.6	0.51	**ok**
		favourable	363.0	726.1	0.50	**ok**
	drained (punch)	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
	undrained	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
	undrained (squeezing)	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
verification horizontal shear resistance	drained	unfavourable	53.2	131.0	0.41	**ok**
		favourable	53.2	126.8	0.42	**ok**
	drained (punch)	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
	undrained	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.
verification tilting stability	width direction	unfavourable	0.10	1.33	0.08	**ok**
		favourable	0.10	1.33	0.08	**ok**
	length direction	unfavourable	n.a.	n.a.	n.a.	n.a.
		favourable	n.a.	n.a.	n.a.	n.a.

XI

APPENDIX: GEOTECHNICAL VERIFICATIONS FOR SECTION 7: FINGER PIERS

Report for D-Sheet Piling 19.2

Design of Diaphragm and Sheet Pile Walls
Developed by Deltares

Company:	Witteveen+Bos
Date of report:	22-6-2021
Time of report:	14:35:52
Report with version:	19.2.3.26114
Date of calculation:	22-6-2021
Time of calculation:	14:35:20
Calculated with version:	19.2.3.26114
File name:	D:\..\Saba_Black Rock harbour_Paalberekening finger piers
Project identification:	Saba Black Rock Harbour Section 7 (Finger piers)

1 Table of Contents

2 Summary

2.1 Overview of Maxima

Displacement [mm]	Moment [kNm]	Shear force [kN]	Mob. perc. moment [%]	Mob. perc. resistance [%]
-20,5	142,84	-67,02	0,0	17,7

3 Input Data

3.1 General Input Data

Model	Single pile; Pile loaded by forces
Unit weight of water	9,81 kN/m³
Elastic calculation	Yes

3.2 Pile Properties

Length	8,10 m
Level top side	0,60 m
Number of sections	1

3.2.1 General Properties

Section name	From [m]	To [m]	Material type	Diameter [m]
762/12	-7,50	0,60	User defined	0,76

3.2.2 Stiffness EI (elastic behaviour)

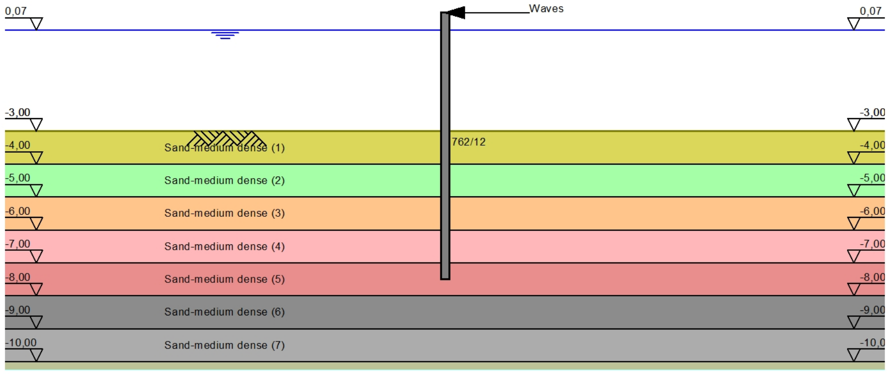
Section name	Elastic stiffness EI [kNm²]	Red. factor on EI [-]	Corrected elas. stiffness EI [kNm²]	Note to reduction factor
762/12	4,1760E+05	1,00	4,1760E+05	

3.2.3 Maximum Allowable Moments

Section name	Mr;char;el [kNm]	Modification factor [-]	Material factor [-]	Red. factor allow. moment [-]	Mr;d;el [kNm]
762/12	10000,00	0,50	1,00	1,00	5000,00

3.3 Outline

Outline



3.4 Horizontal Forces

Name	Level [m]	Load [kN]
Waves	0,60	-32,00

3.5 Water Level

Water level: 0,07 [m]

3.6 Surface

Surface level: -3,00 [m]

3.7 Soil Material Properties

Layer name	Level [m]	Unit weight		Cohesion [kN/m²]	Friction angle phi [°]	Brinch Hansen used
		Unsat [kN/m³]	Sat. [kN/m³]			
Sand-medium d...	-2,00	18,00	20,00	0,00	27,90	Yes
Sand-medium d...	-3,00	18,00	20,00	0,00	27,90	Yes
Sand-medium d...	-4,00	18,00	20,00	0,00	27,90	Yes
Sand-medium d...	-5,00	18,00	20,00	0,00	27,90	Yes
Sand-medium d...	-6,00	18,00	20,00	0,00	27,90	Yes
Sand-medium d...	-7,00	18,00	20,00	0,00	27,90	Yes
Sand-medium d...	-8,00	18,00	20,00	0,00	27,90	Yes
Sand-medium d...	-9,00	18,00	20,00	0,00	27,90	Yes
Sand-medium d...	-10,00	18,00	20,00	0,00	27,90	Yes
Sand-medium d...	-11,00	18,00	20,00	0,00	27,90	Yes
Sand-medium d...	-12,00	18,00	20,00	0,00	27,90	Yes
Sand-medium d...	-13,00	18,00	20,00	0,00	27,90	Yes
Sand-medium d...	-14,00	18,00	20,00	0,00	27,90	Yes
Sand-medium d...	-15,00	18,00	20,00	0,00	27,90	Yes

Layer name	Level [m]	Earth pressure coefficients			Additional pore pressure	
		Active [-]	Neutral [-]	Passive [-]	Top [kN/m²]	Bottom [kN/m²]
Sand-medium d...	-2,00	0,00	0,00	4,06	0,00	0,00
Sand-medium d...	-3,00	0,00	0,00	4,78	0,00	0,00
Sand-medium d...	-4,00	0,00	0,00	5,94	0,00	0,00
Sand-medium d...	-5,00	0,00	0,00	6,83	0,00	0,00
Sand-medium d...	-6,00	0,00	0,00	7,54	0,00	0,00
Sand-medium d...	-7,00	0,00	0,00	8,12	0,00	0,00
Sand-medium d...	-8,00	0,00	0,00	8,60	0,00	0,00
Sand-medium d...	-9,00	0,00	0,00	9,00	0,00	0,00
Sand-medium d...	-10,00	0,00	0,00	9,34	0,00	0,00
Sand-medium d...	-11,00	0,00	0,00	9,64	0,00	0,00
Sand-medium d...	-12,00	0,00	0,00	9,90	0,00	0,00
Sand-medium d...	-13,00	0,00	0,00	10,13	0,00	0,00
Sand-medium d...	-14,00	0,00	0,00	10,33	0,00	0,00
Sand-medium d...	-15,00	0,00	0,00	20,54	0,00	0,00

3.8 Soil Material Properties Calculated Using Brinch Hansen

Layer name	Level [m]	Fictive cohesion [kN/m²]
Sand-medium d...	-2,00	0,00
Sand-medium d...	-3,00	0,00
Sand-medium d...	-4,00	0,00
Sand-medium d...	-5,00	0,00
Sand-medium d...	-6,00	0,00
Sand-medium d...	-7,00	0,00
Sand-medium d...	-8,00	0,00
Sand-medium d...	-9,00	0,00

Layer name	Level [m]	Fictive cohesion [kN/m²]
Sand-medium d...	-10,00	0,00
Sand-medium d...	-11,00	0,00
Sand-medium d...	-12,00	0,00
Sand-medium d...	-13,00	0,00
Sand-medium d...	-14,00	0,00
Sand-medium d...	-15,00	0,00

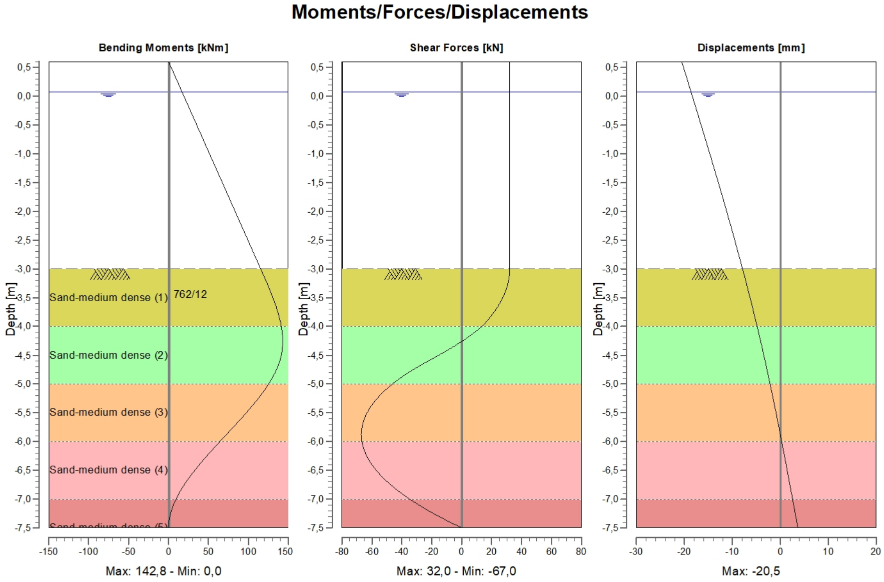
3.9 Modulus of Subgrade Reaction

Layer name	Level	Ménard used	E-Mod Ménard	Soil type Ménard	Branch 1	
	[m]		[kN/m²]		Top [kN/m³]	Bottom [kN/m³]
Sand-medium d...	-2,00	Yes	7000,00	Sand	29552,26	29552,26
Sand-medium d...	-3,00	Yes	7000,00	Sand	29552,26	29552,26
Sand-medium d...	-4,00	Yes	7000,00	Sand	29552,26	29552,26
Sand-medium d...	-5,00	Yes	7000,00	Sand	29552,26	29552,26
Sand-medium d...	-6,00	Yes	7000,00	Sand	29552,26	29552,26
Sand-medium d...	-7,00	Yes	7000,00	Sand	29552,26	29552,26
Sand-medium d...	-8,00	Yes	7000,00	Sand	29552,26	29552,26
Sand-medium d...	-9,00	Yes	7000,00	Sand	29552,26	29552,26
Sand-medium d...	-10,00	Yes	7000,00	Sand	29552,26	29552,26
Sand-medium d...	-11,00	Yes	7000,00	Sand	29552,26	29552,26
Sand-medium d...	-12,00	Yes	7000,00	Sand	29552,26	29552,26
Sand-medium d...	-13,00	Yes	7000,00	Sand	29552,26	29552,26
Sand-medium d...	-14,00	Yes	7000,00	Sand	29552,26	29552,26
Sand-medium d...	-15,00	Yes	7000,00	Sand	29552,26	29552,26

4 Calculation Results

Number of iterations: 5

4.1 Charts of Moments, Forces and Displacements

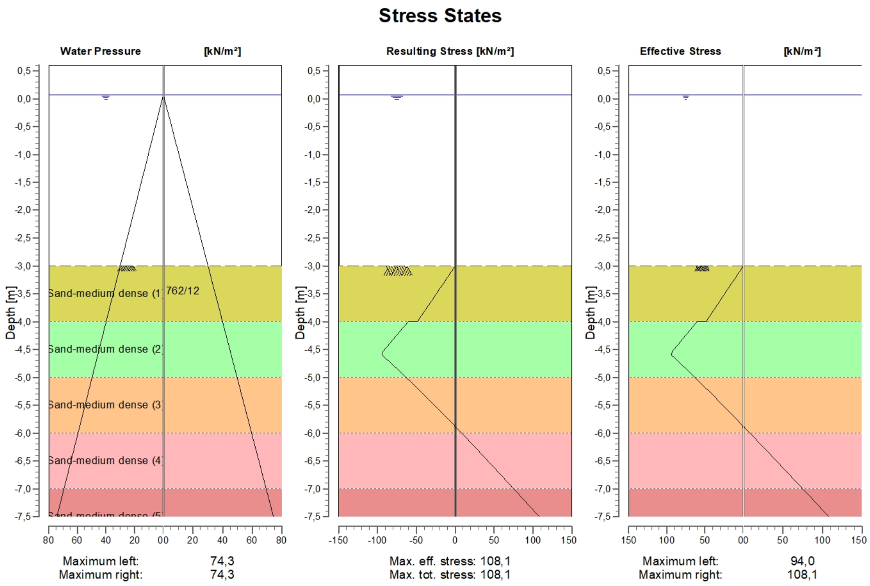


4.2 Moments, Forces and Displacements

Segment number	Level [m]	Moment [kNm]	Shear force [kN]	Displacement [mm]
1	0,60	0,00	32,00	**-20,5**
1	0,34	8,48	32,00	-19,6
2	0,34	8,48	32,00	-19,6
2	0,07	16,96	32,00	-18,6
3	0,07	16,96	32,00	-18,6
3	-0,28	28,00	32,00	-17,3
4	-0,28	28,00	32,00	-17,3
4	-0,62	39,04	32,00	-16,1
5	-0,62	39,04	32,00	-16,1
5	-0,96	50,08	32,00	-14,8
6	-0,96	50,08	32,00	-14,8
6	-1,31	61,12	32,00	-13,6
7	-1,31	61,12	32,00	-13,6
7	-1,66	72,16	32,00	-12,4
8	-1,66	72,16	32,00	-12,4
8	-2,00	83,20	32,00	-11,2
9	-2,00	83,20	32,00	-11,2
9	-2,33	93,87	32,00	-10,1
10	-2,33	93,87	32,00	-10,1
10	-2,67	104,53	32,00	-9,0
11	-2,67	104,53	32,00	-9,0
11	-3,00	115,20	32,00	-7,9
12	-3,00	115,20	32,00	-7,9
12	-3,33	125,64	29,94	-6,8
13	-3,33	125,64	29,94	-6,8
13	-3,67	134,70	23,77	-5,8

Segment number	Level [m]	Moment [kNm]	Shear force [kN]	Displacement [mm]
14	-3,67	134,70	23,77	-5,8
14	-4,00	141,03	13,48	-4,8
15	-4,00	141,03	13,48	-4,8
15	-4,33	**142,68**	-4,42	-3,9
16	-4,33	**142,68**	-4,43	-3,9
16	-4,67	137,53	-26,98	-3,0
17	-4,67	137,53	-27,01	-3,0
17	-5,00	125,14	-46,23	-2,1
18	-5,00	125,14	-46,23	-2,1
18	-5,33	107,41	-59,09	-1,3
19	-5,33	107,41	-59,09	-1,3
19	-5,67	86,42	-65,83	-0,5
20	-5,67	86,42	-65,83	-0,5
20	-6,00	64,18	**-66,65**	0,3
21	-6,00	64,18	**-66,65**	0,3
21	-6,33	42,63	-61,68	1,0
22	-6,33	42,63	-61,68	1,0
22	-6,67	23,69	-51,05	1,8
23	-6,67	23,69	-51,05	1,8
23	-7,00	9,22	-34,81	2,5
24	-7,00	9,22	-34,81	2,5
24	-7,25	2,44	-18,97	3,1
25	-7,25	2,44	-18,97	3,1
25	-7,50	0,00	0,00	3,7
Max		**142,68**	**-66,65**	**-20,5**
Max, minor nodes incl.		142,84	-67,02	-20,5

4.3 Charts of Stresses



4.4 Stresses

Node number	Level [m]	Left Effective Stress [kN/m²]	Left Water stress [kN/m²]	Left Stat*	Left Mob** [%]	Right Effective Stress [kN/m²]	Right Water stress [kN/m²]	Right Stat*	Right Mob** [%]
1	0,60	0,00	0,00	-		0,00	0,00	-	

Node number	Level [m]	Left				Right			
		Effective Stress [kN/m²]	Water stress [kN/m²]	Stat*	Mob** [%]	Effective Stress [kN/m²]	Water stress [kN/m²]	Stat*	Mob** [%]
1	0,34	0,00	0,00	-		0,00	0,00	-	
2	0,34	0,00	0,00	-		0,00	0,00	-	
2	0,07	0,00	0,00	-		0,00	0,00	-	
3	0,07	0,00	0,00	-		0,00	0,00	-	
3	-0,28	0,00	3,38	-		0,00	3,38	-	
4	-0,28	0,00	3,38	-		0,00	3,38	-	
4	-0,62	0,00	6,77	-		0,00	6,77	-	
5	-0,62	0,00	6,77	-		0,00	6,77	-	
5	-0,96	0,00	10,15	-		0,00	10,15	-	
6	-0,96	0,00	10,15	-		0,00	10,15	-	
6	-1,31	0,00	13,54	-		0,00	13,54	-	
7	-1,31	0,00	13,54	-		0,00	13,54	-	
7	-1,66	0,00	16,92	-		0,00	16,92	-	
8	-1,66	0,00	16,92	-		0,00	16,92	-	
8	-2,00	0,00	20,31	-		0,00	20,31	-	
9	-2,00	0,00	20,31	-		0,00	20,31	-	
9	-2,33	0,00	23,58	-		0,00	23,58	-	
10	-2,33	0,00	23,58	-		0,00	23,58	-	
10	-2,67	0,00	26,85	-		0,00	26,85	-	
11	-2,67	0,00	26,85	-		0,00	26,85	-	
11	-3,00	0,00	30,12	-		0,00	30,12	-	
12	-3,00	0,00	30,12	P		0,00	30,12	A	
12	-3,33	16,25	33,39	P		0,00	33,39	A	
13	-3,33	16,25	33,39	P		0,00	33,39	A	
13	-3,67	32,49	36,66	P		0,00	36,66	A	
14	-3,67	32,49	36,66	P		0,00	36,66	A	
14	-4,00	48,74	39,93	P		0,00	39,93	A	
15	-4,00	60,55	39,93	P		0,00	39,93	A	
15	-4,33	80,73	43,20	P		0,00	43,20	A	
16	-4,33	80,73	43,20	P		0,00	43,20	A	
16	-4,67	88,73	46,47	-	88	0,00	46,47	A	
17	-4,67	88,73	46,47	-	88	0,00	46,47	A	
17	-5,00	63,15	49,74	-	52	0,00	49,74	A	
18	-5,00	63,15	49,74	-	45	0,00	49,74	A	
18	-5,33	38,55	53,01	-	24	0,00	53,01	A	
19	-5,33	38,55	53,01	-	24	0,00	53,01	A	
19	-5,67	14,80	56,28	-	8	0,00	56,28	A	
20	-5,67	14,80	56,28	-	8	0,00	56,28	A	
20	-6,00	0,00	59,55	A		8,28	59,55	-	4
21	-6,00	0,00	59,55	A		8,28	59,55	-	4
21	-6,33	0,00	62,82	A		30,85	62,82	-	12
22	-6,33	0,00	62,82	A		30,85	62,82	-	12
22	-6,67	0,00	66,09	A		53,08	66,09	-	19
23	-6,67	0,00	66,09	A		53,08	66,09	-	19
23	-7,00	0,00	69,36	A		75,12	69,36	-	24
24	-7,00	0,00	69,36	A		75,12	69,36	-	23
24	-7,25	0,00	71,81	A		91,60	71,81	-	26
25	-7,25	0,00	71,81	A		91,60	71,81	-	26
25	-7,50	0,00	74,26	A		108,07	74,26	-	29

Stat* Status (A=active, P=passive, Number is branche, 0 is unloading)
Mob** Percentage passive mobilized

End of Report

Witteveen+Bos

SPREADSHEET

VERIFICATION FLEXIBLE DOLPHINS

version 1.50

Project	**Saba**
Projectcode	**118421**
Subject	**Finger piers**
Consultant	**JORL**
Date	**22-6-2021**

This calculationsheet is based on the Ontwerpmethodiek Flexible Dolphins versie 1.0

ZONE DEFINITION

Pile diameter	**762**	mm
PPN	**-7**	[m. NAP]
Design depth	**-3**	[m. NAP]
Top level	**0.6**	[m. NAP]

Zone	Top level [m]	Bottom level [m]	Section length [m]
Zone 1	0.60	-3.00	3.60
Zone 2a	-3.00	-4.80	1.80
Zone 2b	-4.80	-6.50	1.70
Zone 3	-6.50	-7.00	0.50

PILE PROPERTIES

Zone	Outer diameter [mm]	Wall thickness [mm]	Steel quality	Corrosion inner side [mm]	Corrosion outer side [mm]	Soil in the pile	Fabrication Class
Zone 1	**762**	**12**	**S355**	**0.0**	**1.0**	**None**	**C**
Zone 2a	**762**	**12**	**S355**	**0.0**	**1.0**	**Dense Sand (qc > 7 Mpa)**	**C**
Zone 2b	**762**	**12**	**S355**	**0.0**	**1.0**	**Dense Sand (qc > 7 Mpa)**	**C**
Zone 3	**762**	**12**	**S355**	**0.0**	**1.0**	**Dense Sand (qc > 7 Mpa)**	**C**

LOADS

Zone	Calculations	M_{ed}	V_{ed}	σ_p'	N	e_{top}
Zone 1	Zone 1 EC	**143.0**	**67**	**108**	**132.5**	**0**
Zone 2a	Zone 2a CUR211 sand 5MPa	**143.0**	**67**	**108**	**132.5**	**0**
Zone 2b	Zone 2b CUR211 sand 5MPa	**143.0**	**67**	**108**	**132.5**	**0**
Zone 3	Zone 3 CUR211 sand 10 Mpa	**145.0**	**67**	**108**	**132.5**	**0**

UNITY CHECKS

Zone	**initial**	**corroded**
Zone 1	**0.10**	**0.11**
Zone 2a	**0.07**	**0.08**
Zone 2b	**0.07**	**0.08**
Zone 3	**0.07**	**0.08**

Report for D-Foundations 19.1

Design and Verification according to Eurocode 7 of Bearing/Tension Piles and Shallow Foundations
Developed by Deltares

Company:	Witteveen+Bos
Date of report:	23-6-2021
Time of report:	11:37:46
Report with version:	19.1.1.23780
Date of calculation:	22-6-2021
Time of calculation:	13:22:52
Calculated with version:	19.1.1.23780
File name:	D:\..\7-Finger piers (=piles)\Section 7-Finger piers-downwards
Project identification:	Black Rock Harbour Saba Section 7: Finger piers D-Foundations Section 7-Finger piers-downwards

1 Table of Contents

2 Input Data

2.1 General Input Data

Model	Bearing Piles (EC7-NL)

2.2 General Report Data

Geotechnical consultant :	
Design engineer superstructure :	
Principal :	
Title 1 :	Black Rock Harbour Saba
Title 2 :	Section 7: Finger piers
Title 3 :	D-Foundations Section 7-Finger piers-downwards
Number of project :	-
Location of project :	

2.3 Application Area Model Bearing Piles

The verifications performed by the model BEARING PILES of D-FOUNDATIONS concern pile foundations on which axial static or quasi-static loads cause pressures in the piles. The calculations of pile forces and pile displacements are based on Cone Penetration Tests. Possible rise of (tension-)piles and horizontal displacements of piles and/or pile groups are not taken into account.

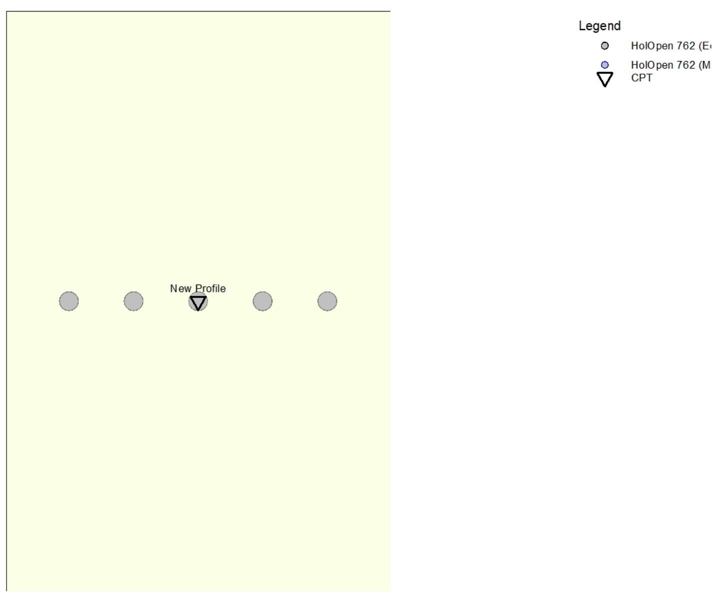
2.4 Superstructure

Rigidity of the superstructure :	Non-Rigid

2.5 General CPT Data

Number of CPT's :	1
Timing of CPT's :	CPT - Excavation - Install

2.5.1 View of CPT's in Foundation Plan



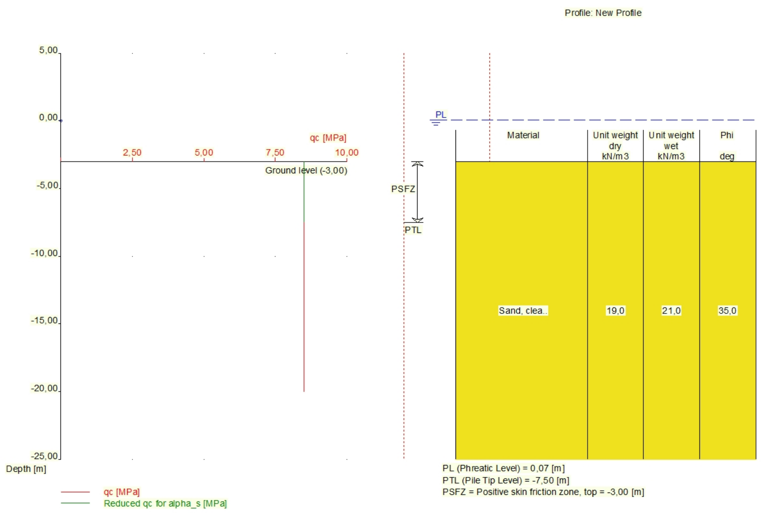
Name CPT	Pile tip level [m R.L.]	Top of pos. friction zone [m R.L.]	Bottom of neg. friction zone [m R.L.]	X-coor-dinate [m]	Y-coor-dinate [m]
New Profile	-7,50	-3,00	-3,00	0,00	0,00

2.6 Soil Data

Number of soil profiles (= number of CPT's) : 1

2.6.1 Soil Profile New Profile

Belonging to CPT New Profile
Surface level in [m. reference level] : -3,00
Phreatic level in [m. reference level] : 0,07
Pile tip level in [m. reference level] : -7,50
Top of positive skin friction zone in [m. reference level] : -3,00
Bottom of negative skin friction zone in [m. reference level] : -3,00
OCR-value foundation layer : 1,00
Expected groundlevel settlement in [m] : 0,00
Number of layers in profile : 1



Number layer	Top layer [m R.L.]	Gamma [kN/m3]	Gamma;sat [kN/m3]	Phi [deg]	Soil Type	Median (Sand/Gravel) [mm]
1	-3,000	19,00	21,00	35,00	Sand	0,200

2.7 Pile Types

2.7.1 Pile type : HolOpen 762

Pile type : Open-ended steel pipe pile

Materialtype for pile : Steel
Slip layer : None
Pile shape : Round open-ended hollow pile

beta (Shape factor) according to figure 7.i, NEN 9997-1:2016.
s (user defined : factor for the influence of the
shape of the crosssection of the pile base) : 1,00

Pile dimensions :
Diameter at tip [m] : 0,762
Wall thickness [mm] : 12,0

2.8 Foundation Plan

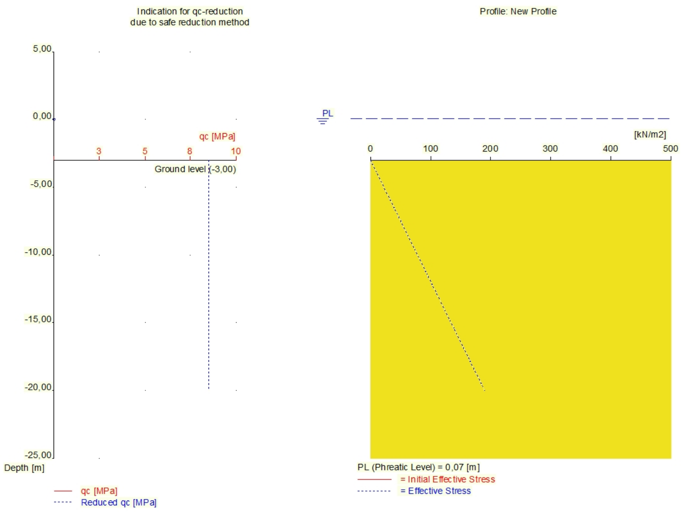
Number of piles : 5
Number of collaborating piles* : 1
* : 0 = not defined, 1 = non rigid superstructure, >1 = rigid superstructure

2.8.1 View of Foundation Plan

Pile nr/name	X-coor-dinate [m]	Y-coor-dinate [m]	Fc;d (EQU/STR/GEO) [kN]	Fc;d (SLS) [kN]	P0 [kN/m2]	Pile head level [m R.L.]
1: 1	-5,00	0,00	365,00	255,00	5,00	0,60
2: 2	-2,50	0,00	365,00	255,00	5,00	0,60
3: 3	0,00	0,00	365,00	255,00	5,00	0,60
4: 4	2,50	0,00	365,00	255,00	5,00	0,60
5: 5	5,00	0,00	365,00	255,00	5,00	0,60

2.9 Excavation Data

Excavation level in [m. reference level] : -3,00
Reduction model : Safe (NEN)



2.10 Overruled Parameters

All parameters according to standard.

2.11 Model Options

Use pilegroup for negative skin friction (standard)
Do not create intermediate results file
Use reduction for continuous flight auger piles (standard)
Use the influence of excavations (standard).

2.12 Model Options

Selected pile types :
- HolOpen 762

Selected profiles :
- New Profile

Trajectory

-begin [m] :	-7,50
-end [m] :	-10,00
-interval [m] :	0,25

3 Bearing Piles (EC7-NL): Results Preliminary Design, Indication Bearing Capacity

3.1 Errors and Warnings

Pile Type HolOpen 762: Warning : The factor s (NEN 9997-1:2016 art. 7.6.2.3(h)) is user defined. Evidence to support this from the NEN deviating value has to be provided.

3.2 Remarks

When checking the survey and testing of soil according to NEN 9997-1:2016 art. 3.2.3 section (e), the program uses the provided CPT test level. It does NOT take into account possible different pile tip levels. When different pile tip levels are used in this calculation, the user itself must check for possibly required additional survey and testing of soil.

Note : The calculations performed are based on a single pile for limit state EQU/STR/GEO (= ultimate limit state). Due to the nature of preliminary design, a single pile is always assumed. A possible pileplan is disregarded when using the preliminary design option. Hence a non rigid superstructure is assumed and pile group effects are not considered.

3.3 Calculation Parameters

3.3.1 Pile Factors

gamma;b (NEN 9997-1:2016, table A.6 A.7 A.8, Limit State EQU/STR/GEO) :	1,20
gamma;b (NEN 9997-1:2016, table A.6 A.7 A.8, the Serviceability Limit State) :	1,00
gamma;s (NEN 9997-1:2016, table A.6 A.7 A.8, Limit State EQU/STR/GEO) :	1,20
gamma;s (NEN 9997-1:2016, table A.6 A.7 A.8, the Serviceability Limit State) :	1,00
xi3 (NEN 9997-1:2016, table A.10a, for N = 1) :	1,39
xi4 (NEN 9997-1:2016, table A.10a, for N = 1) :	1,39

3.3.2 Pile type : HolOpen 762

Pile type :	Open-ended steel pipe pile
Materialtype for pile :	Steel
Slip layer :	None
Pile shape :	Round open-ended hollow pile
beta (Shape factor: figure 7.i, NEN 9997-1:2016 art. 7.6.2.3(g) : Pile tip) :	1,00
s (user defined : factor for the influence of the shape of the crosssection of the pile base) :	1,00
Pile dimensions :	
Diameter at tip [m] :	0,762
Wall thickness [mm] :	12,0

Number/Name CPT	Alpha_s Sand/ Gravel	Alpha_s Clay/Loam Peat	Alpha_p
0:New Prof..	0,0060	--	0,7000

3.4 Results Bearing Forces for Pile type : HolOpen 762

Number/Name CPT	Level [m R.L.]	Rb;cal;max [kN]	Rs;cal;max [kN]	Rc;cal;max [kN]	Rc;d [kN]	F;nsf;k [kN]	Fnsf;d [kN]	Rc;net;d [kN]
0:New Prof..	-7.50	168	1081	1249	749	0	0	749

Number/Name CPT	Level [m R.L.]	Rb;cal;max [kN]	Rs;cal;max [kN]	Rc;cal;max [kN]	Rc;d [kN]	F;nsf;k [kN]	Fnsf;d [kN]	Rc;net;d [kN]
0:New Prof..	-7.75	168	1142	1310	785	0	0	785
0:New Prof..	-8.00	168	1202	1370	821	0	0	821
0:New Prof..	-8.25	168	1262	1430	857	0	0	857
0:New Prof..	-8.50	168	1322	1490	893	0	0	893
0:New Prof..	-8.75	168	1382	1550	929	0	0	929
0:New Prof..	-9.00	168	1442	1610	965	0	0	965
0:New Prof..	-9.25	168	1502	1670	1001	0	0	1001
0:New Prof..	-9.50	168	1562	1730	1037	0	0	1037
0:New Prof..	-9.75	168	1622	1790	1073	0	0	1073
0:New Prof..	-10.00	168	1682	1850	1109	0	0	1109

* Rc;net;d = Rc;d - Fnsf;d

3.5 Summary Net Bearing Capacity in kN

Number/Name CPT	Groundlevel [m R.L.]	Level [m R.L.]	HolOpen 762 Rc;net;d [kN]
0:New Prof..	-3,00	-7,50	749,00
0:New Prof..	-3,00	-7,75	785,00
0:New Prof..	-3,00	-8,00	821,00
0:New Prof..	-3,00	-8,25	857,00
0:New Prof..	-3,00	-8,50	893,00
0:New Prof..	-3,00	-8,75	929,00
0:New Prof..	-3,00	-9,00	965,00
0:New Prof..	-3,00	-9,25	1001,00
0:New Prof..	-3,00	-9,50	1037,00
0:New Prof..	-3,00	-9,75	1073,00
0:New Prof..	-3,00	-10,00	1109,00

* Rc;net;d = Rc;d - Fnsf;d

End of Report

Report for D-Foundations 19.1

Design and Verification according to Eurocode 7 of Bearing/Tension Piles and Shallow Foundations
Developed by Deltares

Company:	Witteveen+Bos
Date of report:	23-6-2021
Time of report:	11:38:00
Report with version:	19.1.1.23780
Date of calculation:	22-6-2021
Time of calculation:	13:03:22
Calculated with version:	19.1.1.23780
File name:	D:\..\7-Finger piers (=piles)\Section 7-Finger piers-upwards
Project identification:	Black Rock Harbour Saba Section 7: Finger piers D-Foundations Section 7-Finger piers-upwards

1 Table of Contents

2 Input Data

2.1 General Input Data

Model	Tension Piles (EC7-NL)

2.2 General Report Data

Geotechnical consultant :	
Design engineer superstructure :	
Principal :	
Title 1 :	Black Rock Harbour Saba
Title 2 :	Section 7: Finger piers
Title 3 :	D-Foundations Section 7-Finger piers-upwards
Number of project :	-
Location of project :	

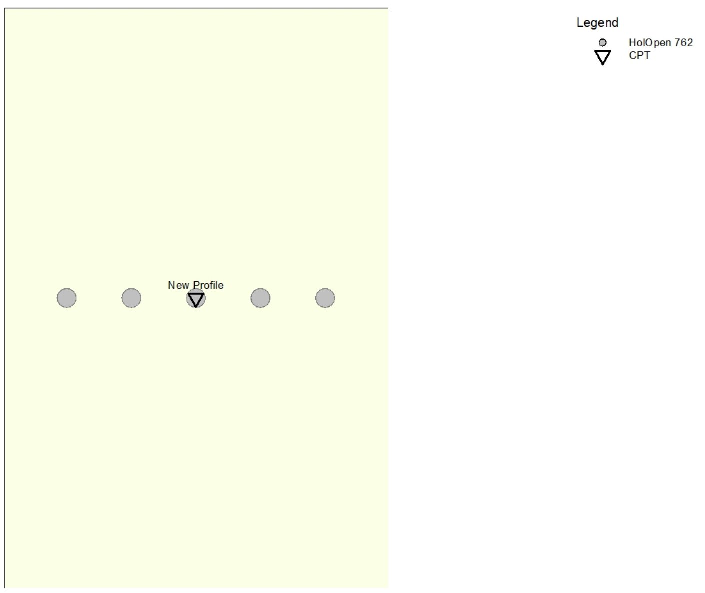
2.3 Application Area Model Tension Piles (EC7-NL)

The design and verifications performed by the TENSION PILES (EC7-NL) model of D-FOUNDATIONS concern pile foundations on which axial static or quasi-static loads cause tensile forces in the piles. Pilegroup effects are taken into account. Calculation of pile forces is based on Cone Penetration Tests. Pile capacities are based on the NEN 9997-1:2016, chapter 7 and where pile/safety factors are concerned, on Dutch Standards NEN 9997-1:2016. Horizontal displacements of piles are not taken into account. Vertical displacements of piles are not calculated. Design of Tension piles based on NEN 9997-1:2016 is limited to piles with lengths between 7 and 50 m and a minimum Length over (equivalent) diameter ratio of 13.5.

2.4 General CPT Data

Number of CPT's :	1
Timing of CPT's :	CPT - Excavation - Install

2.4.1 View of CPT's in Foundation Plan



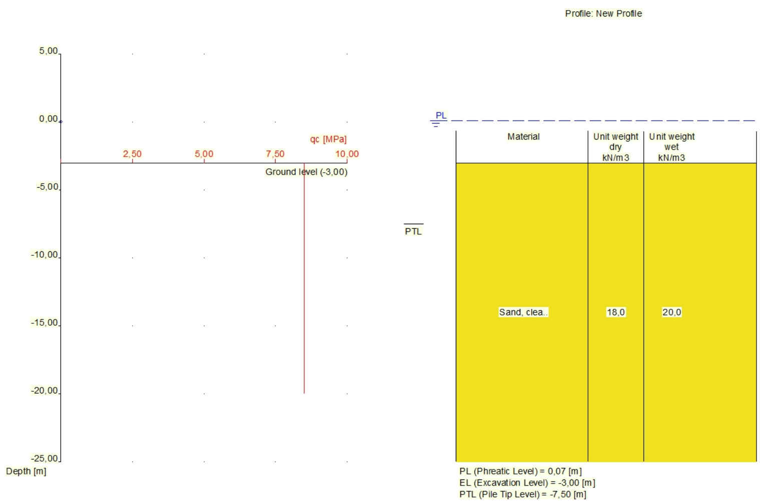
Name CPT	X-coor- dinate [m]	Y-coor- dinate [m]
New Profile	0,00	0,00

2.5 Soil Data

Number of soil profiles (= number of CPT's) : 1

2.5.1 Soil Profile New Profile

Belonging to CPT	New Profile
Surface level in [m. reference level] :	-3,00
Phreatic level in [m. reference level] :	0,07
Top of tension zone [m. reference level]:	0,00
Pile tip level in [m. reference level] :	-7,50
Number of layers in profile :	1



Number layer	Top layer [m R.L.]	Soil Type	Gamma [kN/m3]	Gamma sat [kN/m3]	Min. Void Ratio [%]	Max. Void Ratio [%]	Median [mm]	Max. Cone resistance [kPa]	Use Max. Cone resistance
1	-3,000	Sand	18,00	20,00	0,40	0,80	0,200	12/15	Standa...

Number layer	Top layer [m R.L.]	Soil Type	Phi [deg]	Addit. PP at top [kN/m2]	Addit. PP at bottom [kN/m2]	OCR value [-]	Use Tension
1	-3,000	Sand	32,50	0,00	0,00	1,000	True

2.6 Pile Types

Note : if alpha;t is not user defined, the next rules apply :
- alpha;t according to table 7.g and table 7.h of NEN 9997-1:2016
- for clay: alpha;t depends on the CPT-value and relative depth
- for peat: alpha;t = 0
- for sand/gravel: alpha;t also depends on the median

Number of pile types : 1

2.6.1 Pile type : HolOpen 762

Pile type for shaft friction factor (alpha;t) sand/gravel : Open-ended steel pipe pile

Pile type for shaft friction factor (alpha;t) clay : According to standard
Materialtype for pile : Steel
Pile shape : Round open-ended hollow pile

Pile dimensions :
Diameter at tip [m] : 0,762
Wall thickness [mm] : 12,0
Note : this pile type is regarded as a low vibration pile. Reduction for pile installation after excavation according to NEN 9997-1:2016.

2.7 Foundation Plan

Number of piles : 5
Number of collaborating piles* : 1
* : 0 = not defined, 1 = non rigid superstructure, >1 = rigid superstructure

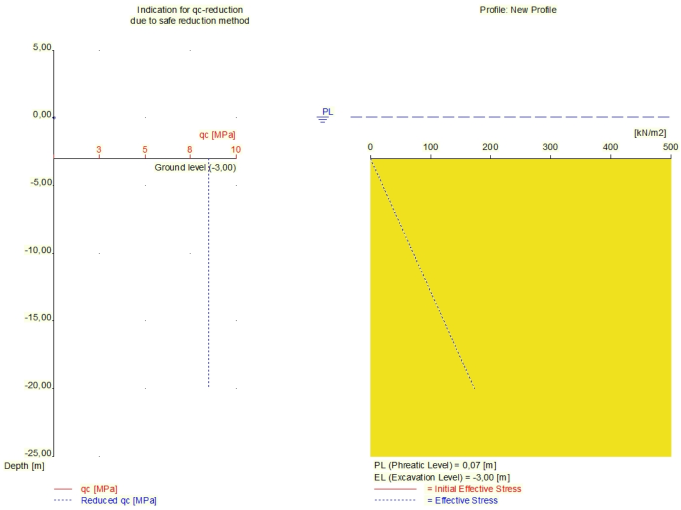
2.7.1 View of Foundation Plan

Pile nr./code	X-coor-dinate [m]	Y-coor-dinate [m]	Maximum load [kN]	Minimum load [kN]	Pile head level [m R.L.]	Use alternat. loads	Factor Gamma;var
1: 1	-5,00	0,00	365,00	-125,00	0,60	True	1,34
2: 2	-2,50	0,00	365,00	-125,00	0,60	True	1,34
3: 3	0,00	0,00	365,00	-125,00	0,60	True	1,34
4: 4	2,50	0,00	365,00	-125,00	0,60	True	1,34
5: 5	5,00	0,00	365,00	-125,00	0,60	True	1,34

Note regarding the loads: tension forces are positive, compressive forces are negative

2.8 Excavation Data

Excavation level in [m. reference level] :	-3,00
Reduction model :	Safe (NEN)



2.9 Optional Parameters

Unit weight water [kN/m3] :	9,81
Surcharge [kN/m2] :	0,00

2.10 Overruled Parameters

All parameters according to standard.

2.11 Model Options

Suppress compaction
If compaction is used, according to NEN 9997-1:2016 CPT's should be made after installation to verify this assumption
Use the influence of excavations (standard).
Suppress excess pore pressure

2.12 Model Options

Selected pile types :
-HolOpen 762

Selected profiles :
-New Profile
Trajectory

-begin [m] :	-7,50
-end [m] :	-10,00
-interval [m] :	0,25

3 Tension Piles (EC7-NL): Indication Bearing Capacity

3.1 Errors and Warnings

At pile type HolOpen 762 :
Due to the use of a low vibrating pile type, the reduction of CPT-value due to the excavation is done using NEN 9997-1:2016.

At pile type HolOpen 762 :
The length over (equivalent) diameter ratio of the pile is less then the required minimum value of 13.5.

When reviewing the following results, warnings listed above should be considered.

3.2 Remarks

When calculating the max. mobilized soil weight, the topangle is used according to NEN 9997-1:2016.

3.3 Calculation Parameters

3.3.1 Pile Factors

xi3 (NEN 9997-1:2016, table A.10a, for N = 1) :	1,39
xi4 (NEN 9997-1:2016, table A.10a, for N = 1) :	1,39
For factor gamma;var values, see FOUNDATION PLAN table	
Factor gamma;st according to NEN 9997-1:2016 A.3.3.2 [-]	1,350
Factor gamma;gamma according to NEN 9997-1:2016 table A.2 [-]	
Above excavation level	1,0
Below excavation level	1,1

3.3.2 Pile type : HolOpen 762

Pile type for shaft friction factor (alpha;t) sand/gravel :	Open-ended steel pipe pile
Pile type for shaft friction factor (alpha;t) clay :	According to standard
Materialtype for pile :	Steel
Pile shape :	Round open-ended hollow pile
Pile dimensions :	
Diameter at tip [m] :	0,762
Wall thickness [mm] :	12,0

Note : this pile type is regarded as a low vibration pile. Reduction for pile installation after excavation according to NEN 9997-1:2016.

3.4 Results for all CPT's

3.4.1 Results for pile type : HolOpen 762

3.4.1.1 Pile group 1

Number of piles belonging to this pile group : 2
Names of piles belonging to this pile group
1
5

Level [m R.L.]	Rt;d min [kN]	Rt;d avg [kN]	Rt;d [kN]	Ksi used [-]
-7,50	135,41	135,41	135,41	Ksi4
-7,75	144,20	144,20	144,20	Ksi4
-8,00	152,98	152,98	152,98	Ksi4

Level [m R.L.]	Rt;d min [kN]	Rt;d avg [kN]	Rt;d [kN]	Ksi used [-]
-8,25	161,75	161,75	161,75	Ksi4
-8,50	170,53	170,53	170,53	Ksi4
-8,75	179,29	179,29	179,29	Ksi4
-9,00	188,06	188,06	188,06	Ksi4
-9,25	196,82	196,82	196,82	Ksi4
-9,50	205,58	205,58	205,58	Ksi4
-9,75	214,34	214,34	214,34	Ksi4
-10,00	223,10	223,10	223,10	Ksi4

Rt;d min: [(Rs;cal)min / Ksi4] / Gamma_s;t
Rt;d avg: [(Rs;cal)avg / Ksi3] / Gamma_s;t

3.4.1.2 Pile group 2

Number of piles belonging to this pile group : 2
Names of piles belonging to this pile group
2
4

Level [m R.L.]	Rt;d min [kN]	Rt;d avg [kN]	Rt;d [kN]	Ksi used [-]
-7,50	127,98	127,98	127,98	Ksi4
-7,75	136,10	136,10	136,10	Ksi4
-8,00	144,21	144,21	144,21	Ksi4
-8,25	152,31	152,31	152,31	Ksi4
-8,50	160,41	160,41	160,41	Ksi4
-8,75	168,50	168,50	168,50	Ksi4
-9,00	176,58	176,58	176,58	Ksi4
-9,25	184,66	184,66	184,66	Ksi4
-9,50	192,73	192,73	192,73	Ksi4
-9,75	200,80	200,80	200,80	Ksi4
-10,00	208,86	208,86	208,86	Ksi4

Rt;d min: [(Rs;cal)min / Ksi4] / Gamma_s;t
Rt;d avg: [(Rs;cal)avg / Ksi3] / Gamma_s;t

3.4.1.3 Pile group 3

Number of piles belonging to this pile group : 1
Names of piles belonging to this pile group
3

Level [m R.L.]	Rt;d min [kN]	Rt;d avg [kN]	Rt;d [kN]	Ksi used [-]
-7,50	127,98	127,98	127,98	Ksi4
-7,75	136,10	136,10	136,10	Ksi4
-8,00	144,21	144,21	144,21	Ksi4
-8,25	152,31	152,31	152,31	Ksi4
-8,50	160,41	160,41	160,41	Ksi4
-8,75	168,50	168,50	168,50	Ksi4
-9,00	176,58	176,58	176,58	Ksi4
-9,25	184,66	184,66	184,66	Ksi4
-9,50	192,73	192,73	192,73	Ksi4
-9,75	200,80	200,80	200,80	Ksi4
-10,00	208,86	208,86	208,86	Ksi4

Rt;d min: [(Rs;cal)min / Ksi4] / Gamma_s;t
Rt;d avg: [(Rs;cal)avg / Ksi3] / Gamma_s;t

3.5 INDICATIVE: Results using Ksi3

3.5.1 Results for pile type : HolOpen 762

3.5.1.1 Pile group 1

Number of piles belonging to this pile group : 2
Names of piles belonging to this pile group
1
5

Number/Name CPT	Level [m R.L.]	Rt;d Indicative [kN]	Max. mobilized soil weight [kN]	Pile weight [kN]	Tension from cohesive layers [%]
0:New Prof..	-7,50	135,41	332,82	29,19	0,00
0:New Prof..	-7,75	144,20	380,93	30,52	0,00
0:New Prof..	-8,00	152,98	433,48	31,85	0,00
0:New Prof..	-8,25	161,75	490,68	33,18	0,00
0:New Prof..	-8,50	170,53	552,74	34,51	0,00
0:New Prof..	-8,75	179,29	613,96	35,84	0,00
0:New Prof..	-9,00	188,06	677,60	37,17	0,00
0:New Prof..	-9,25	196,82	745,50	38,50	0,00
0:New Prof..	-9,50	205,58	817,81	39,83	0,00
0:New Prof..	-9,75	214,34	894,70	41,16	0,00
0:New Prof..	-10,00	223,10	976,33	42,49	0,00

Number/Name CPT	Alpha t aver. overall	Alpha t aver. sand/gravel	Alpha t aver. clay/peat/loam
0:New Prof..	0,0040	0,0040	0,0000

3.5.1.2 Pile group 2

Number of piles belonging to this pile group : 2
Names of piles belonging to this pile group
2
4

Number/Name CPT	Level [m R.L.]	Rt;d Indicative [kN]	Max. mobilized soil weight [kN]	Pile weight [kN]	Tension from cohesive layers [%]
0:New Prof..	-7,50	127,98	282,19	29,19	0,00
0:New Prof..	-7,75	136,10	318,13	30,52	0,00
0:New Prof..	-8,00	144,21	357,03	31,85	0,00
0:New Prof..	-8,25	152,31	399,03	33,18	0,00
0:New Prof..	-8,50	160,41	444,27	34,51	0,00
0:New Prof..	-8,75	168,50	481,08	35,84	0,00
0:New Prof..	-9,00	176,58	515,71	37,17	0,00
0:New Prof..	-9,25	184,66	551,54	38,50	0,00
0:New Prof..	-9,50	192,73	588,65	39,83	0,00
0:New Prof..	-9,75	200,80	627,06	41,16	0,00
0:New Prof..	-10,00	208,86	666,82	42,49	0,00

Number/Name CPT	Alpha t aver. overall	Alpha t aver. sand/gravel	Alpha t aver. clay/peat/loam
0:New Prof..	0,0040	0,0040	0,0000

3.5.1.3 Pile group 3

Number of piles belonging to this pile group : 1
Names of piles belonging to this pile group
3

Number/Name CPT	Level [m R.L.]	Rt;d Indicative [kN]	Max. mobilized soil weight [kN]	Pile weight [kN]	Tension from cohesive layers [%]
0:New Prof..	-7,50	127,98	282,19	29,19	0,00
0:New Prof..	-7,75	136,10	318,13	30,52	0,00

Number/Name CPT	Level [m R.L.]	Rt;d Indicative [kN]	Max. mobilized soil weight [kN]	Pile weight [kN]	Tension from cohesive layers [%]
0:New Prof..	-8,00	144,21	357,03	31,85	0,00
0:New Prof..	-8,25	152,31	399,03	33,18	0,00
0:New Prof..	-8,50	160,41	444,27	34,51	0,00
0:New Prof..	-8,75	168,50	481,08	35,84	0,00
0:New Prof..	-9,00	176,58	515,71	37,17	0,00
0:New Prof..	-9,25	184,66	551,54	38,50	0,00
0:New Prof..	-9,50	192,73	588,65	39,83	0,00
0:New Prof..	-9,75	200,80	627,06	41,16	0,00
0:New Prof..	-10,00	208,86	666,82	42,49	0,00

Number/Name CPT	Alpha t aver. overall	Alpha t aver. sand/gravel	Alpha t aver. clay/peat/loam
0:New Prof..	0,0040	0,0040	0,0000

End of Report