Delta Marine Consultants

Technical design note

To	IMDC
From	Simon Berdenis van Berlekom
Telephone	+31 639704050
E-mail	simon.berdenis.van.berlekom@bam.com
Date	30 januari 2024
Reference	DMC-232011-M-00001-SBE
Page	1 of 43
Subject	**Pile design pontoon facility**

1. Ausfertigung
In statischer Hinsicht geprüft
Prüfnummer 2023 D122
Hamburg, den 23.02.2024
Dr.-Ing. Rainer Grzeschkowitz
Dr.-Ing. Olaf Drude (SFI)
Dipl.-Ing. (FH) Karsten Holste
Prüfingenieure für Bautechnik
gem. Prüfverordnung PVO vom 14.02.2006
Veritaskai 8 • 21079 Hamburg
Tel.: +49 40 790001-0 (Fax: -44) • www.wk-consult.com

Karsten Holste

Digital unterschrieben von Karsten Holste
Datum: 2024.03.12 09:28:49 +01'00'

Subject Pile design pontoon facility

1. INTRODUCTION

The Wilhelmshaven FSRU requires an emergency evacuation berth, which needs to be accessible for evacuation of personnel when the FSRU is at the berth. This structure consists of a pontoon supported by 12 piles, which is to be accessed from MD6 by a gangway.

The purpose of the pontoon is access during normal operations/conditions and for emergencies (both access for e.g. fire brigade and departure of FSRU crew)

The objective of this technical design note is to design the supporting piles of the pontoon facility.

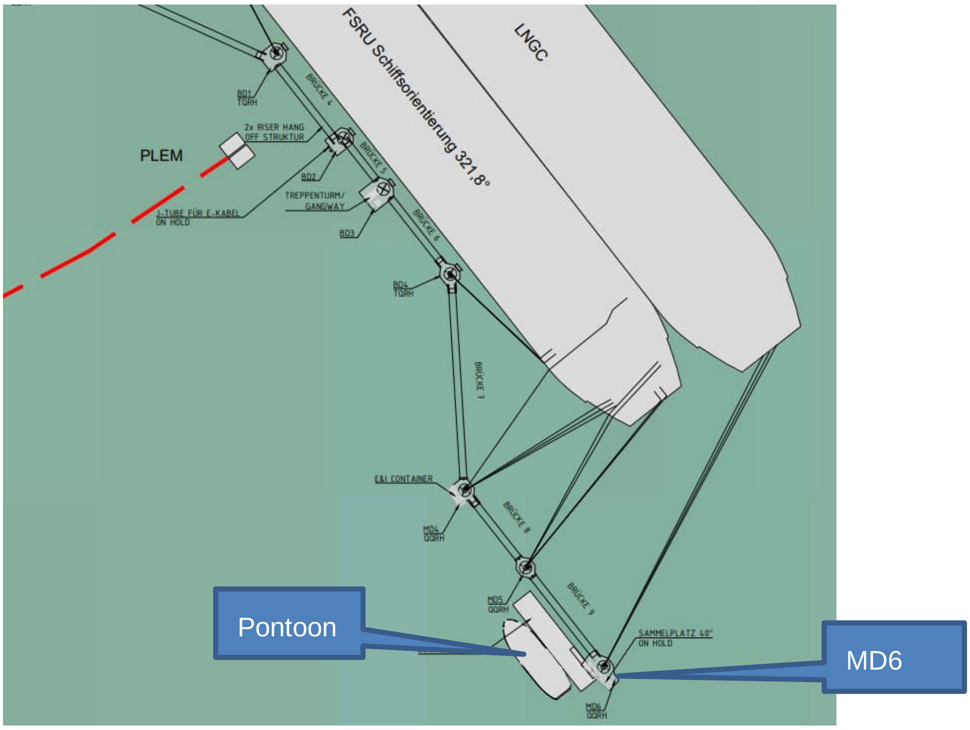


Figure 1 South East part of the FSRU mooring facility with

Die Ergebnisse der ANSYS-AQWA-Modellierung des Gesamtsystems sind als richtig vorausgesetzt worden (siehe ab Seite 16).

Es ist Stellung zu nehmen zu der Frage, wie sich die auf die Ponton-Pfähle wirkende Belastung verändert, wenn FSRU und/ oder LNG-Tank-Schiff am Terminal anliegen.

Es ist durch konstruktive Maßnahmen sicherzustellen, dass die an den Dalbenzangen ("Brackets") wirkenden Kräfte, insbesondere aus Wind und Wellen, gleichmäßig verteilt abgetragen werden.

gesehen

Subject Pile design pontoon facility

2. LITERATURE

[1] BOD Wilhelmshaven Floating pontoon
[2] DMC-230704-M-00019-MP Pontoon motions analysis
[3] EN-1990 Eurocode 0: Basis of structural design. DS-EN 1990, December 2011.
[4] EN-1991 Eurocode 1: Actions on structures. DS-EN 1991, December 2011.
[5] EN-1993-1-1 Eurocode 3: Design of steel structures – Part 1-1: General rules and rules for buildings DS-EN 1993, December 2016.
[6] EN-1993-5 Eurocode 3: Design of steel structures – Part 5: Piling, Februari 2008
[7] EN-1993-1-6 Eurocode 3: Design of steel structures – Part 1-6: General – Strength and Stability of Shell Structures
[8] EN-1993-1-9 Eurocode 3: Design of steel structures - Part 1-9: Fatigue
[9] EN-1997-1 Eurocode 7: Geotechnical design – Part 1 : General rules
[10] NEN 9997-1 Dutch Norm: Geotechnical design of structures – Part 1: General rules
[11] NEN-EN 10025-2 Hot rolled products of structural steels - Part 2: Technical delivery conditions for non-alloy structural steels
[12] NEN-EN 10025-3 Hot rolled products of structural steels - Part 3: Technical delivery conditions for normalized/normalized rolled weldable fine grain structural steels
[13] DNV-OS-C101, Design of offshore steel structures general (LRFD method), October 2008.
[14] DNV-RP-C203 Fatigue design of offshore steel structures, Amended version September 2021.
[15] DNV-OS-C401 Rules and standards for offshoure units, July 2023.
[16] Geotechnischer Bericht. Teilprojekt: Bootsanleger (Pontoon) Nr.23A012.00.00 Rev.0.0 12 Dezember 2023, Anlage 3 Rechnerische Bodenprofile fur erdstatische Berechnungen.
[17] Ergebnisse der in Aug/Sept 2023 ausgefuhrten Drucksondierungen CPT und der Bohrung BH1 (Bohrprofil, Sondierdiagramme, CPT-ASCII-Data), LANKELMA, bereitgestellt durch IMDC, 12.09.2023
[18] DIN 4085:2017-8 Baugrund – Berechnung des Erddrucks
[19] Empfehlungen des Arbeitsausschusses “Ufereinfassungen“ Hafen und Wasserstraßen (EAU 2022) 12.2.5.2 Ansatz nach Blum
[20] Vergleichsberechnungen zur Dalbenbemessung nach Blum und mit der p-y-Methode, Christina Rudolph e.a. Fachthemen DOI: 10.1002/gete.201100006

gesehen

[21] SBRCURnet Publication C206 Flexible Dolphins Committee 1720
[22] SBRCURnet Publication 211E Quay walls Second edition
[23] Accelerated Low Water Corrosion Report of Working Group 44 of the Maritime Navigation commission PIANC
[24] Shore Protection Manual DEPARTMENT OF THE ARMY Waterways Experiment Station, US Corps of Engineers.
[25] Einspanningsverhaltnisse bei Bohlwerken H.Blum Berlin 1931

gesehen

Subject Pile design pontoon facility

3. ABBREVIATIONS

Abbreviation	Definition
EEB	Emergency Evacuation Berth
SKN*	SeeKartenNull (Chart datum)
ALS	Accidental Limit State
ULS	Ultimate Limit State
SLS	Serviceability Limit State
FSL	Fatigue Limit State
DFF	Design Fatigue Factors
BD	Basic Design
BoD	Basis of Design
CC2	Consequence class 2 (according to Eurocode)
CC3	Consequence class 3 (according to Eurocode)
CPT	Cone Penetration Test
DA	Design Approach
DD	Detailed Design
FEM	Finite Element Method
MBL	Mean Breaking Load
SWL	Safe Working Load
UC	Unity Check
SBL	Sea Bed Level
COG	Centre of Gravity
HW	High Water
LW	Low Water
TDN	Technical Design Note

*In the open North Sea, the SKN is based on the lowest astronomical tide (LAT). In rivers influenced by tides, the SKN is determined separately.

4. SYMBOLS

γ_E	partial factor for the effect of an action
γ_F	partial factor for an action
γ_G	partial factor for a permanent action
$\gamma_{R;e}$	partial factor for earth resistance
γ_Q	partial factor for a variable action
d	depth [m]
w	witdh [m]
l	length [m]
D	pile diameter [mm]
t	pile wall thickness [mm]

5. DESCRIPTION OF THE STRUCTURE

Figure 2 shows a plan view of the access berth. The berth consists of the following elements:

- Floating pontoon with general dimensions l x w x d = 67.00m x 18.00m x 4.53m and minimum 2.23 m freeboard.
- 12 supporting piles D x t = 1500x50mm with L=44m
- Gangway of about 40m from mooring dolphin MD6 landing on the pontoon

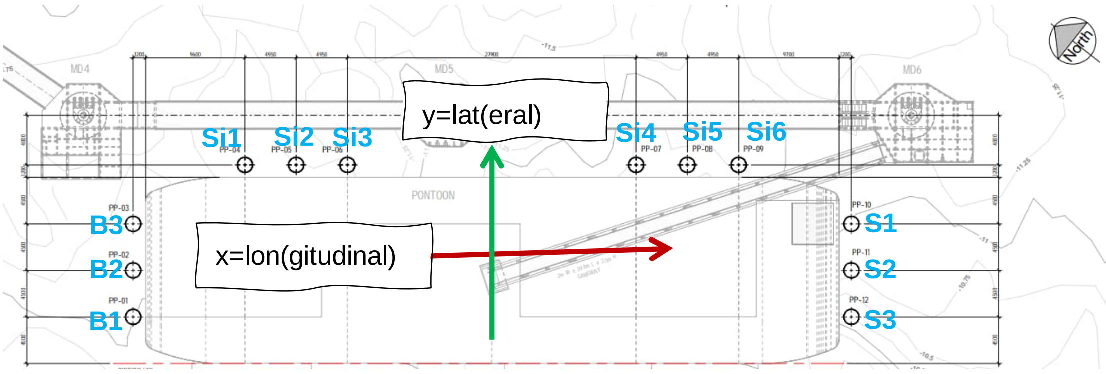


Figure 2 Schematical plan view of access berth, B=bow, S=stern, Si=Side

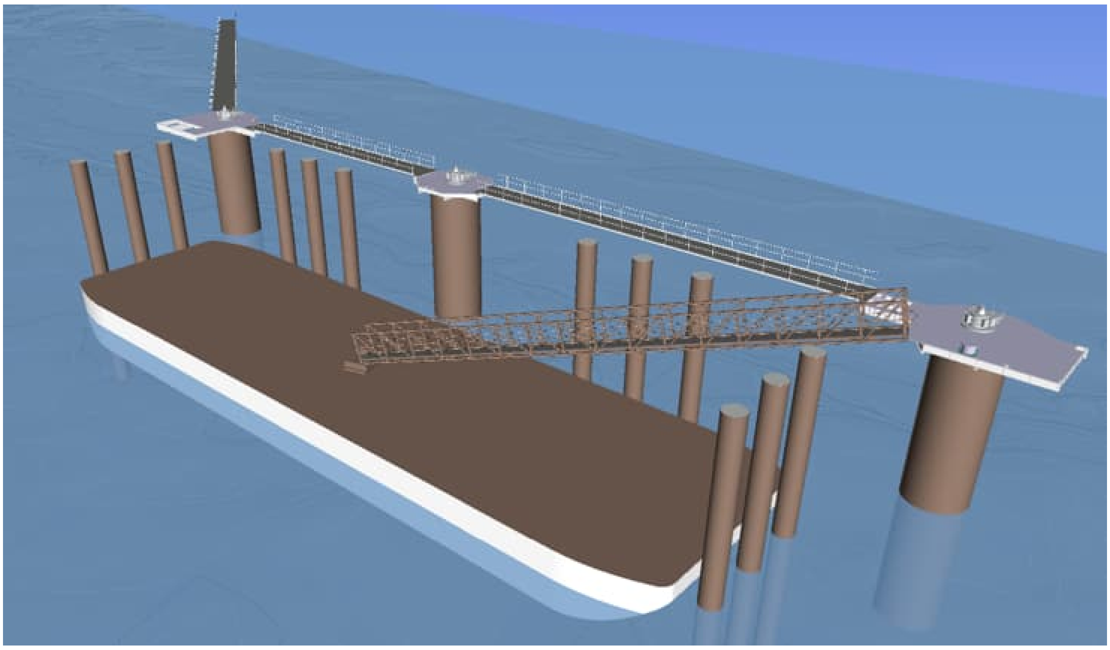


Figure 3 3D impression of pontoon facility

Subject Pile design pontoon facility

6. GENERAL STARTING POINTS

6.1 Design live

In accordance with [1] the design live is 10 years.

6.2 Impact levels

The pontoon transfers mooring forces at approximately deck level. The following impact levels have been considered:

1/R	High impact (maximum translations)	Low impact (maximum force)
1/10	6.40+2.20=8.60m	-0.71+2.20=1,49m
1/100	7.16+2.20=9.36m	-1.03+2.20=1.17m

Table 1 Extreme water levels for 1/10 and 1/100 year conditions (see also [1])

6.3 Codes

The pile design will be in accordance with [19] EAU in conjunction with the Eurocodes with the German NA. See also [1]. Where applicable other codes has been used such as DNV.

6.4 Consequence class

EAU does not explicitly mention the consequence class but provides safety factors and material factors to be used in the design which are in line with Consequence class 2. See also [1].

6.5 Design approach

2.4.7.3.4.3 Design Approach 2

(1)P It shall be verified that a limit state of rupture or excessive deformation will not occur with the following combination of sets of partial factors:

Combination: *A1* "+" *M1* "+" *R2*

NOTE 1 In this approach, partial factors are applied to actions or to the effects of actions and to ground resistances.

NOTE 2 If this approach is used for slope and overall stability analyses the resulting effect of the actions on the failure surface is multiplied by γ_E and the shear resistance along the failure surface is divided by $\gamma_{R;e}$.

geprüft

Subject Pile design pontoon facility

Tab. 12.1 Teilsicherheitsbeiwerte für den Nachweis der Grenztragfähigkeit von Dalben.

	Einwirkungen	**Widerstände Boden**	**Stahl**
	γ_Q	$\gamma_{R,e}$	γ_M
Lasten aus Anlegemanövern	1,00	1,00	1,00
Vertäukräfte (Trossenzug) und Anlehnkräfte	1,20	1,15	1,10
Kräfte aus Wellen, Wind und Strömung	1,20	1,15	1,10
Eislasten (siehe auch Abschn. 4.12)	1,00	1,10	1,10

Figure 4 Table 12.1 of document [19]

6.6 Steel

Designation		Minimum yield strength R_{eH} [a] MPa Nominal thickness mm								
Steel name	Steel number	≤ 16	> 16 ≤ 40	> 40 ≤ 63	> 63 ≤ 80	> 80 ≤ 100	> 100 ≤ 150	> 150 ≤ 200	> 200 ≤ 250	> 250 ≤ 400
S235JR	1.0038									
S235J0	1.0114	235	225	215	215	215	195	185	175	165
S235J2	1.0117									
S275JR	1.0044									
S275J0	1.0143	275	265	255	245	235	225	215	205	195
S275J2	1.0145									
S355JR	1.0045									
S355J0	1.0553	355	345	335	325	315	295	285	275	265
S355J2	1.0577									
S355K2	1.0596									
S460JR [b]	1.0507									
S460J0 [b]	1.0538	460	440	420	400	390	390	-	-	-
S460J2 [b]	1.0552									
S460K2 [b]	1.0581									
S500J0 [b]	1.0502	500	480	460	450	450	450	-	-	-

Figure 5 Table 6 Mechanical properties from document [11]

Designation		Minimum yield strength R_{eH} [a] MPa Nominal thickness mm							
Steel name	Steel number	≤ 16	>16 ≤ 40	>40 ≤ 63	> 63 ≤ 80	> 80 ≤ 100	> 100 ≤ 150	> 150 ≤ 200	> 200 ≤ 250
S275N	1.0490	275	265	255	245	235	225	215	205
S275NL	1.0491								
S355N	1.0545	355	345	335	325	315	295	285	275
S355NL	1.0546								
S420N	1.8902	420	400	390	370	360	340	330	320
S420NL	1.8912								
S460N	1.8901	460	440	430	410	400	380	370	370
S460NL	1.8903								

Figure 6 Table 4 Mechanical properties from document [12]

6.7 Corrosion

Corrosion allowance accordance with [1].

7. ACTIONS

7.1 Waves, wind and current

The sea state at various return periods has been analysed with the software Ansys Aqwa. Ansys Aqwa determines forces on the hull of the submerged part of the pontoon using linear wave theory. See for more information document [2].

The direct wave and current forces on the piles however are not included in this model. For the determination of the wave and current forces on the piles a method has been chosen that is a simplification of the Morison equation. See Figure 7 and Figure 8.

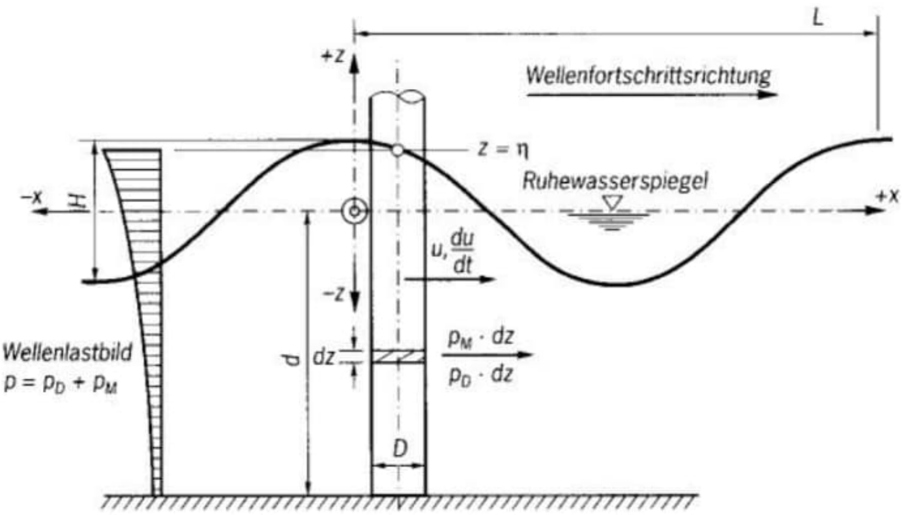


Figure 7 Wave action on a slender structure

$$F_{max} = F_I + F_D = C_I K_I H \rho g \frac{\pi D^2}{4} + C_D K_D H^2 \frac{1}{2} \rho g D$$

$$M_{max} = F_I d S_I + F_D d S_D$$

where:

C_I	[-]	=	inertia coefficient ≈ 2,0
C_D	[-]	=	drag coefficient (for small flow velocities C_D ≈ 1,2 , see Section 20.3)
K_I	[-]	=	correction for extent of inertia force
K_D	[-]	=	correction for extent of drag force
S_I	[-]	=	correction for position of resultant inertia force
S_D	[-]	=	correction for position of resultant drag force
H	[m]	=	wave height
D	[m]	=	diameter pile
d	[m]	=	depth

Figure 8 Linear wave theory

The “Shore Protection Manual” (CERC 1984) gives graphs with the maximum values of the coefficients CD, Ki, KD, Si, SD. These graphs are also included in the following sections. The values of the coefficients depend on the wave period, the phase, the water depth and the applicable wave theory for the determination of the velocity of the water particles.

geprüft

Subject Pile design pontoon facility

The graphs show various different curves. (See document [24].) These depend on the ratio: H/Hb in which Hb is the wave height when braking. A conventional estimate of the breaking wave height is to take 1/7 of the wave length for shallow water. In this report conservatively H/Hb=1 has been chosen which yields the highest values for Ki, KD, Si and SD.

The following input has been considered for the calculation:

In Abstimmung mit dem Aufsteller korrigierte Werte (siehe Email IMDC vom 19.02.2024):

General input					
g	Gravitation		9.81	[m/s2]	
T	Temperature		2.00	[°C]	
	Kinematic viscosity		1.67E-06	[m2/s]	
γw	Density of sea water		1030	[kg/m3]	
S	Safety factor for simplified calculation		1.10	[-]	
Environmental input					
umax	Current velocity		1.88	[m/s]	
Tp	Wave period		5.20	[s]	
Hs	Significant wave height (average height of the 1/3 highest waves)		3.78	[m]	
Hb	Wave height when breaking (individual waves)	1/7*L =	~~5.6~~	[m]	6,0
L0	Wave length		42.2	[m]	
d/L0	Relative water depth	[-] (intermediate water depth)	~~0.24~~	[-]	0,4
L	Wave length according to linear wave theory, exact to 3 decimals		~~39.0~~	[m]	41,7
Geometric input					
d	Water depth		~~10.0~~	[m]	16,93
D	Diameter of the pile		~~1.400~~	[m]	1,5
HA	Height of area facing flow		8.0	[m]	

Figure 9 General input simplified Morison calculation

The following factors have been determined from the tables from the rock manual:

Factors			
CI	inertia coefficient	2.00	[-]
Re	Renolds number	1.6E+06	[-]
d/gT2	Horizontal axis in graph Shore Protection Manual	3.77E-02	
H/Hb	Ratio of design wave/breaking wave	7/10	[-]
TABLES ROCK MANUAL			
CD	Drag coefficient	0.70	[-]
KI	Correction for extend of inertia force	0.45	[-]
KD	Correction for extent of drag force	0.50	[-]
SI	Correctionf for position of resultant inertia force	0.82	[-]
SD	Correctionf for position of resultant drag force	1.10	[-]

Figure 10 Factors rock manual

geprüft

Subject Pile design pontoon facility

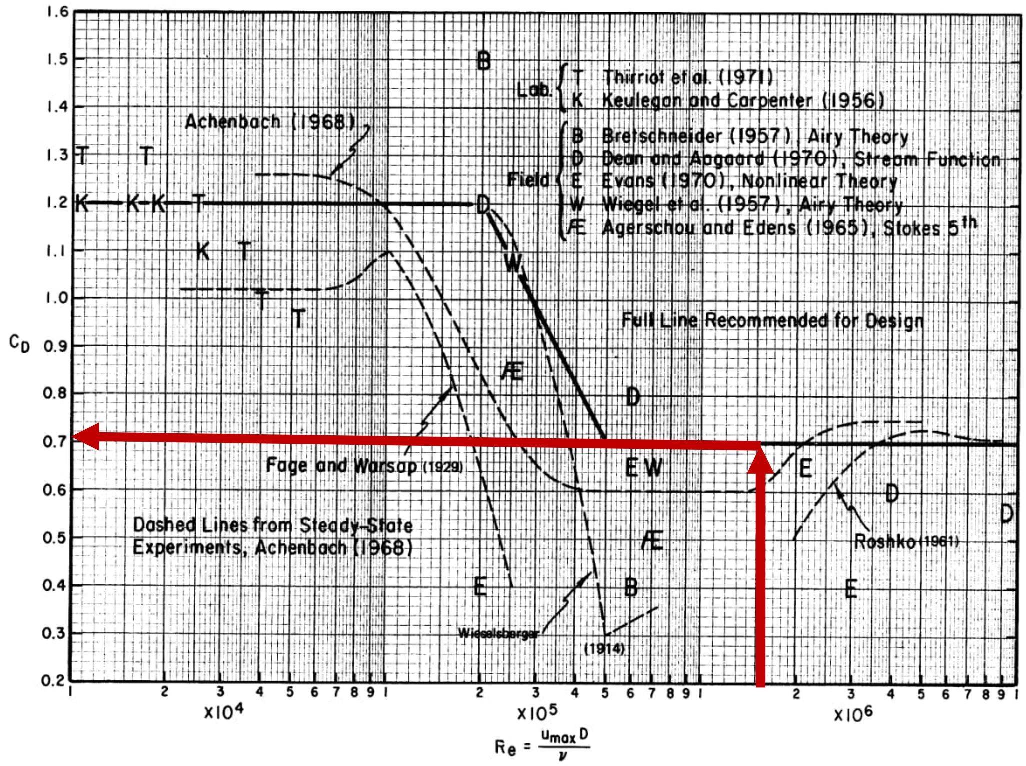


Figure 11 Determining drag coefficient

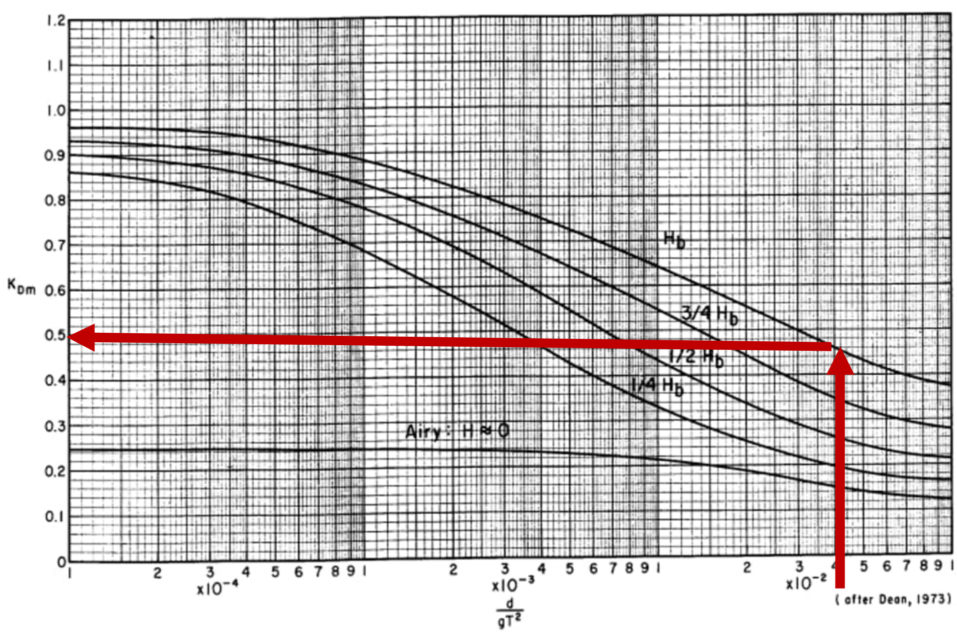


Figure 12 Determining KD factor (correction for the extend of the drag force)

Subject Pile design pontoon facility

Based on this he drag forces due to current and inertia can be determined. To the levers of these forces the vertical distance between the sea bed level and the point of fixity has been added for the calculation of the maximum moment. This distance has been taken at 0.22*t0=0.22*10.59=2.33m.

In Abstimmung mit dem Aufsteller korrigierte Werte (siehe Email IMDC vom 19.02.2024):

FI	Drag force due to inertia	~~52.9~~	[kN]	60,7
FD	Drag force due to current	~~35.4~~	[kN]	37,9
Mmax	Maximum moment ~~@ -13.11 [mSKN]~~ @ -17,0 mSKN	2178	[kNm]	

Figure 13 Forces and maximum moment pile under wave and current action

In the pile calculation he force at low impact of 1167kN (see section 8.4.1) has been increased until an extra maximum bending moment of 2178 is developed. The maximum bending moment thereby increases with 10%. For the fatigue limit state the same amount of increase has been assumed.

It should be noted that there are conservatisms in the calculation. They are deliberate for the purpose of not underestimating the loading.

- Shielding effects of the pontoon are ignored.
- An overall (self-invented) factor of 1.10 on the maximum moment has been used.
- It is argued that the current does not swiftly alter direction like the piles swaying under waves. Assuming that it does is conservative for the fatigue limit state.

geprüft

Subject Pile design pontoon facility

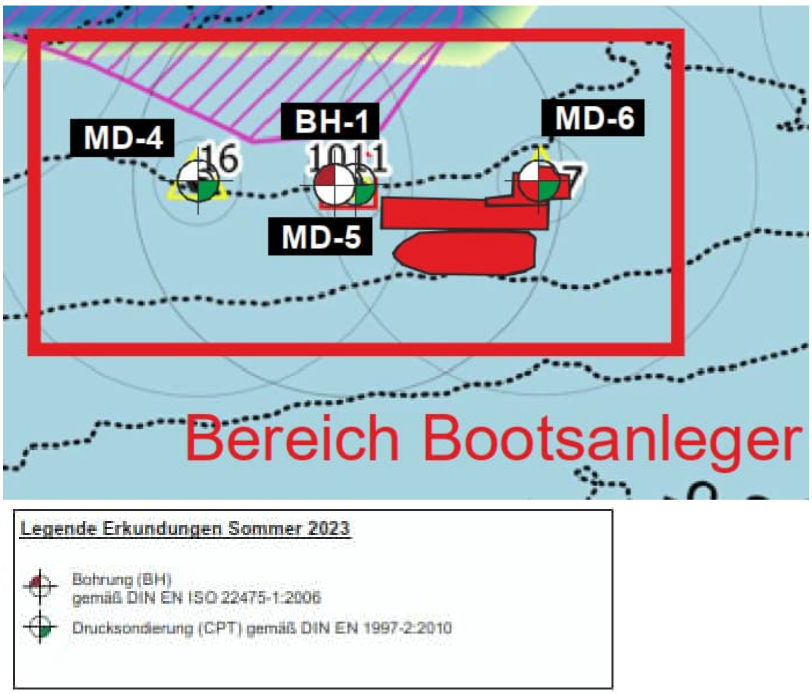
7.2 Geotechnical actions

The soil profile and soil parameters are taken from Anlage 3 Rechnerische Bodenprofile für erdstatische Berechnungen from document [16].

In this appendix there are two profiles given.

1. FSRU - Ponton-Dalben ohne Kolkschutzschüttung (pontoon dolphins w/o armor layer), Rechnerische Wassertiefe (DSL): -10.80 [m SKN]. It is assumed this profile stems from BH-1.
2. FSRU - Ponton-Dalben mit Kolkschutzschüttung (pontoon dolphins w/ armor layer), Rechnerische Wassertiefe (DSL): -9.80 [m SKN]. It is assumed this profile stems from MD-6.

The first profile is used for maximum flexibility at high impact with adding a scour of approximately 1xD meter. The second profile is used for maximum force at low impact with scour protection (Kolkschutzschüttung).



8. MODELLING

8.1 Determination of pile reaction forces

The pontoon is restrained by 12 piles. In the Ansys Aqwa model each pile is represented by 4 press only supports. Two in the direction perpendicular to the ponton in opposite direction en two in the direction parallel to the pontoon in opposite direction. This means that each pile is able to withstand sway and surge and that the whole pile system is able to withstand yaw. Heave, pitch and roll do not significantly impact the pile design. (See for those motions Appendix B.)

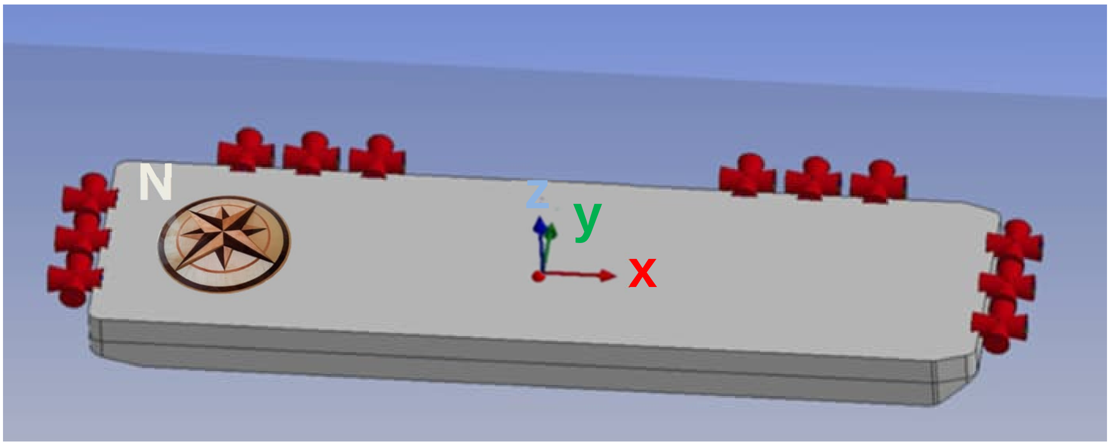


Figure 14 pile support model 12x4 press only restraints

The directions of the waves that yield the highest reaction forces has been found via checking 12 directions with an individual pile stiffness of 9224 kN/m for both extreme low and high water. In Figure 15 for 4 directions and for low impact, the forces have been shown, including the force dominant direction of 51.1°N. The figure is meant to be illustrative. Note that the directions are given relative to the North and also that wind and wind wave direction may differ somewhat. The positive longitudinal x-axis of the pontoon points to 141.75°North. The positive longitudinal axis in Ansys Aqwa is defined as 0°. Therefore e.g. 51.1°N = 360 - (180-141.75 + 51.1) = 321.75 - 51.1 = 270.65°AQWA.

Subject Pile design pontoon facility

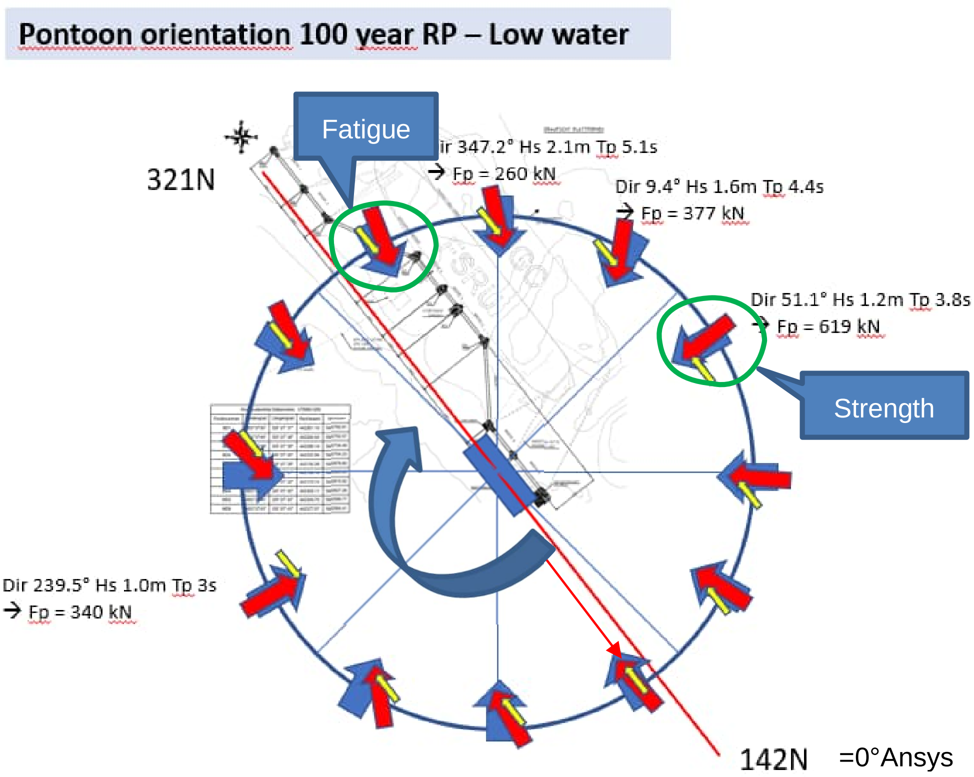


Figure 15 Pile reaction forces for 12 wind directions ELW at 9224kN/m.

For the fatigue limit state the dominant wave direction is 340N [DEG]. This has been explained in document [1] revision 2.

Table 2 shows the dominant wind and wave directions for STR/GEO and Fatigue limit state.

1/R	LS	DirN [DEG]	DirAnsys [DEG]
		Wind / Wave	Wind / Wave
1/10	Fatigue	330 / 340	-8 / -18
1/100	STR/GEO	60 / 51	262 / 271

Table 2 Dominant wind and wave directions

For the purpose of finding the reaction forces in the STR/GEO limit state (1/100 year conditions) and the stress range in the fatigue limit state (1/10 year conditions) force-displacement curves have been made for pile stiffness at extreme low and extreme high water. These curves are made with an inhouse calculation file hereafter named “DMC BLUM sheet”. See Figure 20, Figure 21, Figure 28 and Figure 29.

Subject Pile design pontoon facility

Table 3 shows the input parameters for Ansys Aqwa in the considered limit states. The numeric values are adopted from document [1].

- Hs is significant wave height (average of 1/3 highest waves)
- Tp is wave peak period
- gamma factor for the wave spectrum

1/R	LS	Hs [m]	Tp [s]	Gamma [-]
1/10	Fatigue	1.70	4.9	2.1
1/100	STR/GEO	1.20	3.8	3.3

Table 3 Input Ansys for 1/10 and 1/100 year conditions (see also [2])

With the input from Table 3 and the stiffness ranges from high to low impact for both limit states, Ansys Aqwa runs have been made. Thereafter peak values in the response of the system at certain stiffnesses were investigated. The result is a diagram with on the horizontal axis pile stiffness in kN/m and on the vertical axis reaction force (STR/GEO) in kN or reaction force range (Fatigue) in kN. See Figure 22 and Figure 32.

8.2 Determination of pile internal forces

The pile has been designed with method Blum in accordance with documents [19] en [20]. In its simplest form Blum assumes full passive mobilisation of one soil type for a sheet pile wall as shown in Figure 16f. The original method was formulated for sheet pile walls that are supported at the top. Horizontal equilibrium is assured by a theoretical horizontal force at the toe of the wall. The Blum method is a force driven method. From the static equilibrium the internal forces shear and bending moment are derived. In the DMC Blum sheet this has been done numerically by considering the equilibrium of pile slices with Δh=1cm from top to bottom.

Seiten 19 - 40: Durch Vergleichsrechnung geprüft.

geprüft

Subject Pile design pontoon facility

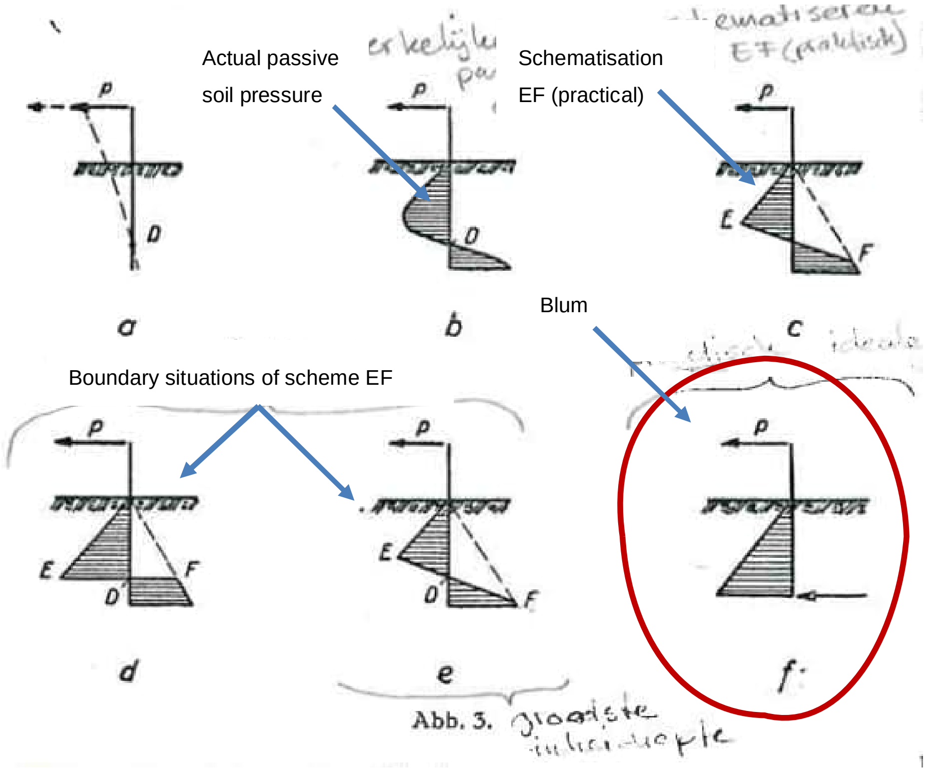


Figure 16 Blum schematisation from document [25]

The Blum method was adapted for flexible dolphins by introducing multiple soil layers and factors (formbeiwerte) that take into account the spatial effect of soil pressure on circular shaped sections. At the location where the bending moment becomes zero (M=0) a force is assumed that makes horizontal equilibrium. That force, with symbol C, is called Ersatzkraft (german for replacement force). The total pile depth is $t_0+\Delta t$ where t_0 is the distance between the top of the soil to the M=0 level in meter and Δt is the required depth under level M=0. In document [19] Δt is calculated as shown in Figure 18. In accordance with [19] and [21] the friction angle (or friction between the pile and the soil) has been set to 2/3 times the internal friction angle of the soil.

Subject Pile design pontoon facility

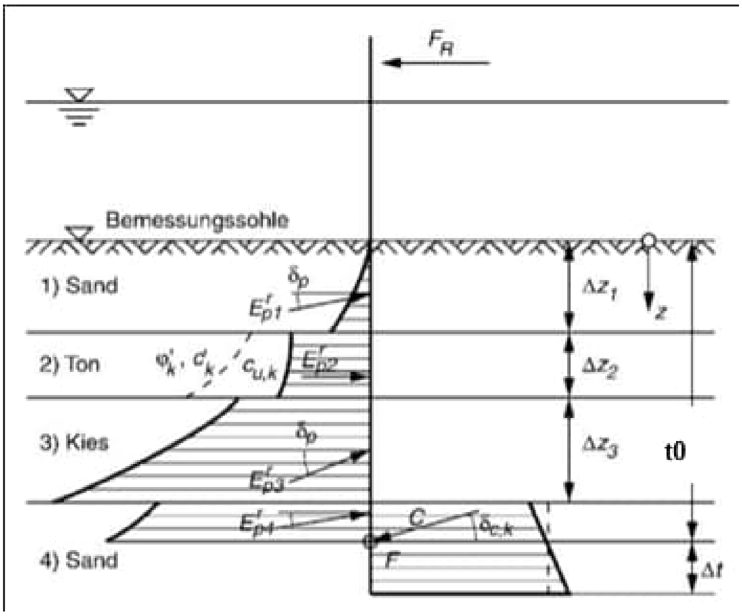


Figure 17 Ersatzkraft C at M=0 for mono pile in multiple soil layers from document [19]

$$\Delta t = \frac{1}{2} \cdot C_{\mathrm{h,k,Blum}} \cdot \gamma_{\mathrm{Q}} \cdot \frac{\gamma_{\mathrm{R,e}}}{e^{\mathrm{r}}_{\mathrm{ph,k}}}$$

mit

$e^{\mathrm{r}}_{\mathrm{ph,k}}$	Ordinate des charakteristischen räumlichen Erddrucks in Höhe der ErsatzkrafC.

Figure 18 Determination of Δt in accordance with document [19]

8.3 Pile depth

The depth Δt as discussed in the previous section is determined by replacing the ersatzkraft C by a line load which represents the soil pressure. A recommendation of the committee Flexible Dolphins is to determine Δt in a slightly different way than the EAU does. They do this to avoid underestimating the required toe depth, in case of softer layers under t_0. An intuitive method is used to determine Δt, refer to Figure 19 below.

In the ULS Δt is to be taken such that the resultant of the medium green soil pressure area (trapezium) on the LHS of Figure 19 equals half of the ersatzkraft. However, Δt should never become smaller than 0.2 times t_0.

An additional SLS check has been added by the Flexible Dolphin committee. The purpose of this check is to ascertain whether under frequent actions the soil at the toe does not plastically deforms with ongoing pile deformations as a consequence. Therefore this check corresponds with normal operational conditions.

In the SLS Δt is to be taken such that the resultant of the dark green soil pressure area (trapezium) on the RHS of Figure 19 times a β factor equals half of the ersatzkraft. Empirically has been found that for β=0.33 the results are satisfactory.

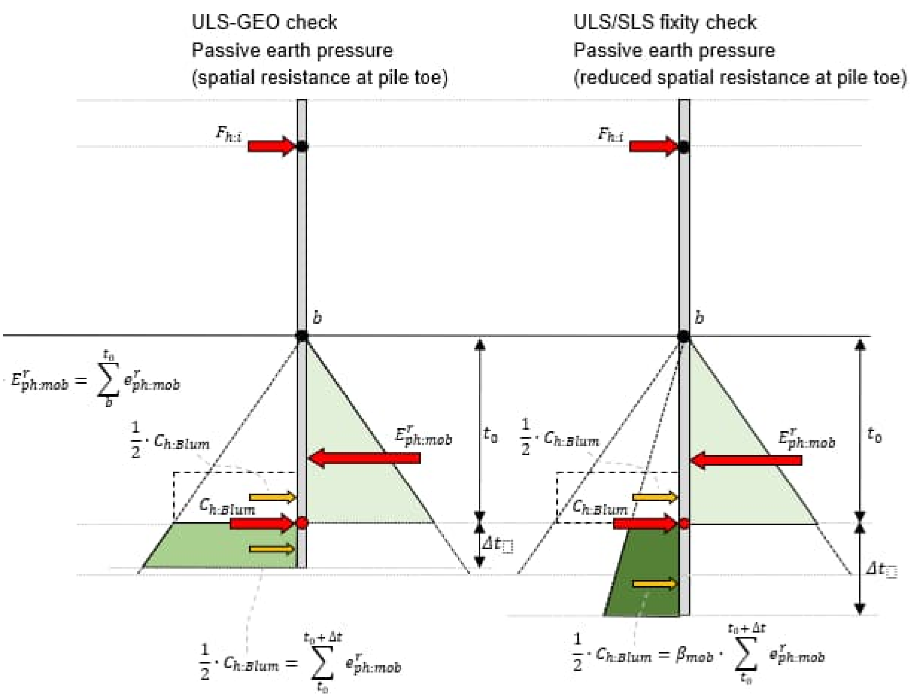


Figure 19 Pile depth in accordance with practice committee Flexible Dolphins

8.4 Pile design checks

8.4.1 STR/GEO limit state section check

The resistance of pile sections class 1, 2 and 3 in ULS limit state (STR/GEO) has been determined with document [5] section 6.2.

DMC checks resistance of pile sections class 4 in ULS limit state (STR/GEO) with:

- document [7] Euro Code 1993-1-6 section 8.5.2, 8.5.3 and Annex D (always for clay and zone1)
- document [21] section 3.8 Method Gresnigt (always for sand and zones 2 &3)

However, in this case class 4 piles are not part of the design and thus this check is not required.

*1) Gemäß Geotechn. Bericht, Kap. 6.5 wird ein Nachweis ohne Kolkschutz empfohlen.
-> Aufsteller-Erläuterung siehe gesonderte Technical Note bzw. in folgender Revision.

The piles have been checked for the following STR/GEO (1/100 condition) limit state:

- Design approach 2 with partial safety factors from table 12.1 of EAU (document [19]). See also Figure 4.
- Determination of internal forces with method Blum assuming full passive mobilization of the soil.
- Spatial passive soil resistance determined with DIN 4085 (document [18]).
- Required toe depth determined with conservatively considering the horizontal equilibrium beneath the level where the bending moment is zero. See also section Pile depth.
- Including wave and current forces on the piles themselves.
- No corrosion considered (higher stiffness equals higher forces).
- Highest impact level at +7.16+2.20=+9.36 mSKN with low seabed at -12.30 mSKN (including scour) for maximum deformation. *1)
- Lowest impact level at -1.03+2.20=+1.17 mSKN with high seabed at -9.80 mSKN (including scour protection) for maximum internal pile forces.
- Dominant wave direction for maximum pile forces is 51°N.

Figure 20 shows the determination with the DMC BLUM sheet of the pile stiffness at low impact. Figure 21 shows. the determination with the DMC BLUM sheet of the pile stiffness at high impact.

High Impact	9.36	[mSKN]
Freeboard	2.20	[m]
Return period	100	[years]
Diameter	1500	[mm]
Wand dikte	50	[mm]
Kolkschuttung	OHNE	
Seabed	Low	
Soil	LowChar	

	Frep	w		k
	[kN]	[mm]	[m]	[kN/m]
1	125	61	0.061	2042
2	250	137	0.137	1829
3	375	221	0.221	1701
4	500	311	0.311	1608
5	625	407	0.407	1535
6	750	508	0.508	1475
7	875	614	0.614	1424
8	1000	725	0.725	1380
9	1125	839	0.839	1341
10	1250	957	0.957	1306
11	1375	1079	1.079	1274
12	1500	1204	1.204	1246
				1513

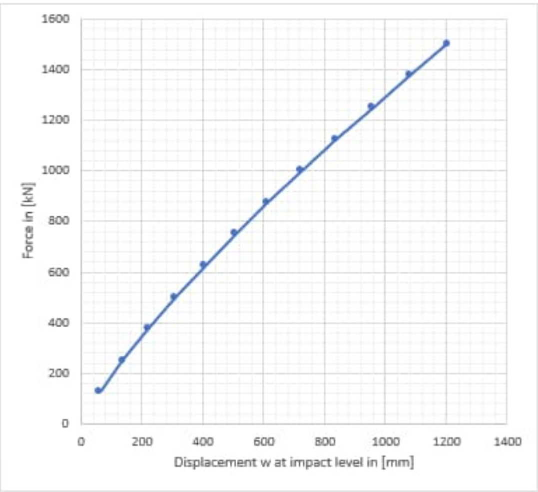
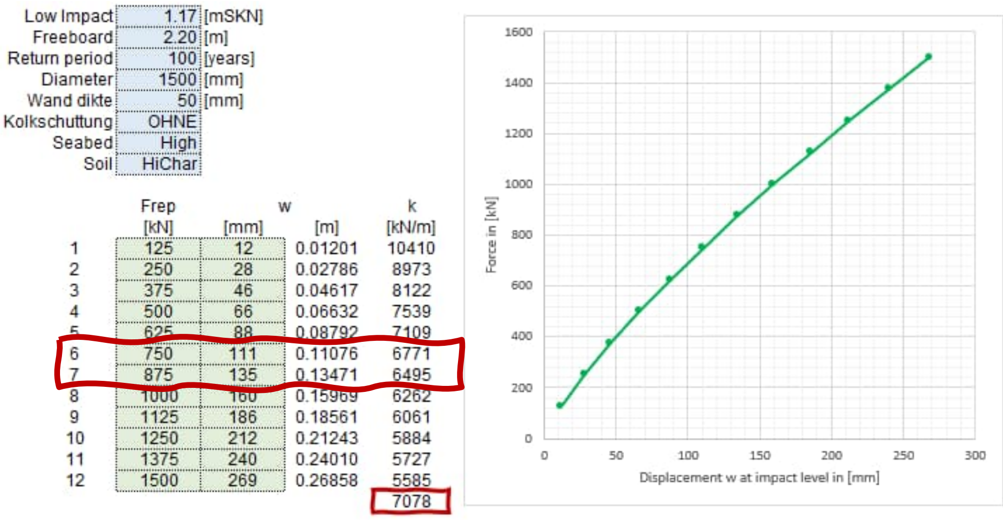


Figure 20 Pile stiffness at high impact

geprüft

Subject Pile design pontoon facility



Low Impact	1.17 [mSKN]
Freeboard	2.20 [m]
Return period	100 [years]
Diameter	1500 [mm]
Wand dikte	50 [mm]
Kolkschuttung	OHNE
Seabed	High
Soil	HiChar

	Frep	w		k
	[kN]	[mm]	[m]	[kN/m]
1	125	12	0.01201	10410
2	250	28	0.02786	8973
3	375	46	0.04617	8122
4	500	66	0.06632	7539
5	625	88	0.08792	7109
6	750	111	0.11076	6771
7	875	135	0.13471	6495
8	1000	160	0.15969	6262
9	1125	186	0.18561	6061
10	1250	212	0.21243	5884
11	1375	240	0.24010	5727
12	1500	269	0.26858	5585
				7078

Figure 21 Pile stiffness at low impact

In the Ansys Aqwa model all the piles are given the same stiffness in all directions. For instance 3000 kN/m. Then a run is made which results in reaction forces of the piles. The maximum force of all piles is selected which in this case is 1126 kN. This run is represented in Figure 22 with a single dot. Then more runs are made with different stiffness to establish the shape of the system response. Stiffnesses of interest are the ones that are at the extremes (low and high impact) and the ones that yield the highest reaction forces.

Ergebnisse der ANSYS-AQWA-Modellierung: Als richtig vorausgesetzt.

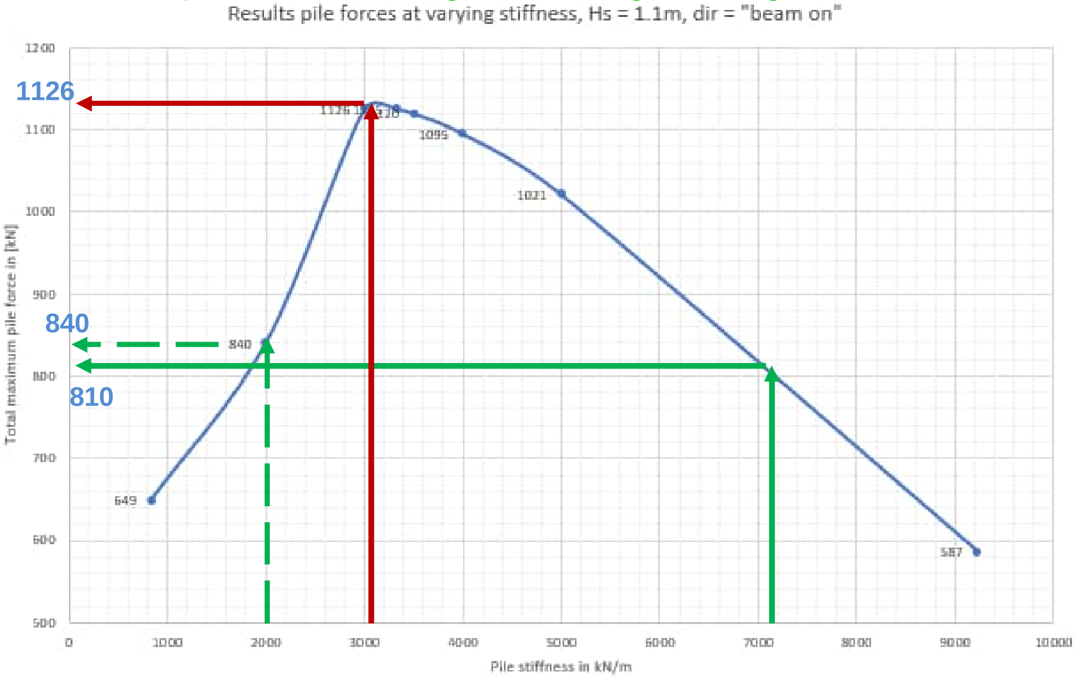


Figure 22 Pile reaction force as a function of pile stiffness in STR/GEO limit state

geprüft

Subject Pile design pontoon facility

The pile stiffness at low impact when scour protection is considered is about 20000 kN/m. From Figure 22 it may be observed that no run has been made for this stiffness. From the trend however it is expected that the pile reaction force at 200000 kN/m is lower then about 800, the force at a low impact level without scour protection. For a higher stiffness (with scour protection) the fixity point will be higher and thus the bending moment lower for low impact level of 1.17 mSKN and a force of 800 kN. Therefore the stiffness range from 9000 kN/m to 20000 kN/m does not yield governing bending moments and is omitted from the analysis.

Then the DMC Blum sheet is used to find the maximum bending moment in the piles. First the model is calibrated to the stiffness input (Figure 20 and Figure 21) with the Blum sheet. With partial factors equal to 1.0 a force of 840 kN at level 9.36 (high impact) is entered. The results are given in Figure 23. The DMC Blum sheet results in a stiffness of 1437 kN/m which corresponds to Figure 20. The same has been done for level 1.17 [mSKN] (low impact). See figure Figure 24.

	Levels			Loads			User choices	
Pile top	12.00	[mSKN]	Force	840	[kN]	Corrosion	No	
Impact high	9.36	[mSKN]	UDL	0.0	[kN/m2]	Impact	High	
Impact low	10.86	[mSKN]	E	246	[kNm]	Seabed	Low	
Water line	7.16	[mSKN]				Soil	LoChar	
Seabed high	-10.80	[mSKN]		Factors toe depth		PSFs Soil	Char	
Seabed nom	-10.80	[mSKN]	Passive β_{mob}	1.00	STR/GEO	Soil type (class 4)	Sand moderately/dense	
Seabed low	-12.30	[mSKN]	Distribution C_h	0.50	[-]	Kolkschutzschutting	Ohne	
Soil profile bottom	-70.00	[mSKN]						
				Calculation summary		Displacement@TOP	661.0	[mm]
Seabed	-12.30	[mSKN]	t0	10.18	[m]	Displacement@IP	585	[mm]
Impact	9.36	[mSKN]	Δt	1.76	[m]	Stiffness@IP	1437	[kN/m]
Level Mmax (V=0)	-16.69	[mSKN]	Toe depth	-24.24	[mSKN]	Energy absorption	246	[kNm]
Mmax	20873	[kNm]	0.2*t0	2.04	[m]	Applied toe depth	-32.00	[mSKN]
Level Ersatzkraft (M=0)	-22.48	[mSKN]	t_{fix}	2.04	[m]	Total pile length	44.00	[m]
Ersatzkraft	9044	[kN]	Toe depth	-24.52	[mSKN]	Total pile weight	78.7	[t]

Figure 23 Output DMC Blum sheet at high impact

	Levels			Loads			User choices	
Pile top	12.00	[mSKN]	Force	810	[kN]	Corrosion	No	
Impact high	9.36	[mSKN]	UDL	0.0	[kN/m2]	Impact	Low	
Impact low	1.17	[mSKN]	E	128	[kNm]	Seabed	High	
Water line	-1.03	[mSKN]				Soil	HiChar	
Seabed high	-10.80	[mSKN]		Factors toe depth		PSFs Soil	Char	
Seabed nom	-10.80	[mSKN]	Passive β_{mob}	1.00	STR/GEO	Soil type (class 4)	Sand moderately/dense	
Seabed low	-12.30	[mSKN]	Distribution C_h	0.50	[-]	Kolkschutzschutting	Ohne	
Soil profile bottom	-70.00	[mSKN]						
				Calculation summary		Displacement@TOP	232.0	[mm]
Seabed	-10.80	[mSKN]	t0	7.35	[m]	Displacement@IP	122	[mm]
Impact	1.17	[mSKN]	Δt	0.98	[m]	Stiffness@IP	6640	[kN/m]
Level Mmax (V=0)	-13.98	[mSKN]	Toe depth	-19.13	[mSKN]	Energy absorption	49	[kNm]
Mmax	11617	[kNm]	0.2*t0	1.47	[m]	Applied toe depth	-32.00	[mSKN]
Level Ersatzkraft (M=0)	-18.15	[mSKN]	t_{fix}	1.47	[m]	Total pile length	44.00	[m]
Ersatzkraft	8408	[kN]	Toe depth	-19.62	[mSKN]	Total pile weight	78.7	[t]

Figure 24 Output DMC Blum sheet at low impact

geprüft

High impact yields the highest bending moments for low characteristic values of the soil and a low seabed. Low impact yields the highest bending moments for high characteristic values of the soil and a high seabed. However, higher bending moments may occur at levels in between which the response curve of Figure 22 seems to indicate. At a stiffness of 3000 kN/m, the reaction force is equal to 1126 kN. The Blum sheet has been used to find the corresponding impact level for combinations of high and low seabed and high and low characteristic values of the soil. Figure 25 shows the impact level for low seabed and low characteristic soil parameters for a pile stiffness of 3000 kN/m.

	Levels			Loads			User choices	
Pile top	12.00	[mSKN]	Force	1126	[kN]	Corrosion	No	
Impact high	2.75	[mSKN]	UDL	0.0	[kN/m2]	Impact	High	
Impact low	1.17	[mSKN]	E	128	[kNm]	Seabed	Low	
Water line	7.16	[mSKN]				Soil	LoChar	
Seabed high	-10.80	[mSKN]		**Factors toe depth**		PSFs Soil	Char	
Seabed nom	-10.80	[mSKN]	Passive β_{mob}	1.00	STR/GEO	Soil type (class 4)	Sand moderately/dense	
Seabed low	-12.30	[mSKN]	Distribution C_h	0.50	[-]	Kolkschutzschutting	Ohne	
Soil profile bottom	-70.00	[mSKN]						
				Calculation summary		Displacement@TOP	596.3	[mm]
Seabed	-12.30	[mSKN]	t0	10.36	[m]	Displacement@IP	375	[mm]
Impact	2.75	[mSKN]	Δt	1.75	[m]	Stiffness@IP	3000	[kN/m]
Level Mmax (V=0)	-17.06	[mSKN]	Toe depth	-24.41	[mSKN]	Energy absorption	211	[kNm]
Mmax	20948	[kNm]	0.2*t0	2.07	[m]	Applied toe depth	-32.00	[mSKN]
Level Ersatzkraft (M=0)	-22.66	[mSKN]	t_{fix}	2.07	[m]	Total pile length	44.00	[m]
Ersatzkraft	9238	[kN]	Toe depth	-24.73	[mSKN]	Total pile weight	78.7	[t]

Figure 25 Output DMC Blum sheet at stiffness of 3000 kN/m, Seabed=Low and Soil=LowChar

The bending moment to be checked in the STR/GEO limit state had been checked for the following situations:

1. High Impact (9.36 mSKN), Low Seabed and LoChar Soil with a force of 840+126=966kN.
2. Level of impact corresponding with a stiffness of 3000 kN/m for combinations of Seabed and Soil with a force of 1126+169=1295kN.
3. Low Impact (1.17 mSKN), High Seabed and HiChar Soil with a force of 810+122=932kN.

For the calibrations done in the figures Figure 23, Figure 24 and Figure 25 no partial safety factors were used. For the STR/GEO limit state a partial factor on the variable action of 1.20 is used and a partial factor on the earth resistance of 1.15 is used. See also Figure 4.

Further the bending moment has been increased by 10% to account for waves and current exerted directly on the piles. See also section 7.1.

geprüft

Subject Pile design pontoon facility

W_pl=	1.05E+08	[mm3]	fy=	410	[N/mm2]	γ_{M0}=	1.10	UC
CMB	Stiffness	F	Impact	Seabed	Soil	Fw+c	Moment	Section check
[-]	[kN/m]	[kN]	[mSKN]	[-]	[-]	[kN]	[kNm]	(STR/GEO
1	1437	840	9.36	Low	LoChar	126	29476	0.75
2	3000	1126	2.75	Low	LoChar	169	29850	0.76
3	3000	1126	5.69	High	HiChar	169	30558	0.78
4	3000	1126	4.63	Low	HiChar	169	30880	0.79
5	3000	1126	3.94	High	LoChar	169	29792	0.76
6	6640	810	1.17	High	HiChar	122	16518	0.42

Table 4 STR/GEO checks at low and high impact and for highest reaction force at stiffness of 3000 kN/m.

CMB2 is now used as an example of the procedure that was followed to find the maximum STR/GEO bending moments shown in Table 4 here above.

1. A force of F=1126kN and partial factors equal to unity were used to find at what level the pile has a stiffness of 3000kN/m. See Figure 25.
2. A force of F=1126+169=1295kN and partial factors of 1.20 on the variable action and 1.15 on the soil pressure were used at a level of 2.75 mSKN and the resulting maximum bending moment is read from the DMC BLUM sheet.

8.4.1 SLS pile fixity and ULS pile stability check

The ULS check for the pile stability is performed for the 1/100 wave conditions (annual probability of exceedance of 1/00 and a probability of 9.6%) with partial factors of 1.20 on the variable action and 1.15 on the soil pressure.

The governing combination is CMB1. The required toe depth is -26.27 mSKN. See figure Figure 26.

	Levels			Loads			User choices	
Pile top	12.00	[mSKN]	Force	966	[kN]	Corrosion	No	
Impact high	9.36	[mSKN]	UDL	0.0	[kN/m2]	Impact	High	
Impact low	1.17	[mSKN]	E	128	[kNm]	Seabed	Low	
Water line	7.16	[mSKN]				Soil	LoChar	
Seabed high	-10.80	[mSKN]		Factors toe depth		PSFs Soil	Char	
Seabed nom	-10.80	[mSKN]	Passive β_{mob}	1.00	STR/GEO	Soil type (class 4)	Sand moderately/dense	
Seabed low	-12.30	[mSKN]	Distribution C_h	0.50	[-]	Kolkschutzschutting	Ohne	
Soil profile bottom	-70.00	[mSKN]						
				Calculation summary		Displacement@TOP	1016.7	[mm]
Seabed	-12.30	[mSKN]	t0	11.64	[m]	Displacement@IP	903	[mm]
Impact	9.36	[mSKN]	Δt	1.76	[m]	Stiffness@IP	1070	[kN/m]
Level Mmax (V=0)	-17.30	[mSKN]	Toe depth	-25.70	[mSKN]	Energy absorption	436	[kNm]
Mmax	29476	[kNm]	0.2*t0	2.33	[m]	Applied toe depth	-32.00	[mSKN]
.evel Ersatzkraft (M=0)	-23.94	[mSKN]	t_{fix}	2.33	[m]	Total pile length	44.00	[m]
Ersatzkraft	11550	[kN]	Toe depth	-26.27	mSKN]	Total pile weight	78.7	[t]

Figure 26 ULS pile stability check

geprüft

Subject Pile design pontoon facility

The SLS check for the pile fixity is performed for the 1/10 wave conditions (annual probability of exceedance of 1/10 thus a probability of 65.1%) with no partial factors. The reason for this is that the SLS pile fixity check is to be associated with normal conditions. See also section 8.3.

The maximum reaction force on the piles for the wave conditions with a return period of 10 years is 241kN. See also Figure 30. However, these forces were determined for head on wave direction. Therefore, as a conservative approach, the maximum reaction force is now taken equal to an arbitrary 500kN. The minimum required pile depth now becomes -27.95 SKN. See Figure 27.

The applied toe depth is -32 mSKN leaving a comfortable UC of (+12.00 - -27.95)/ (+12.00 - -32.00) = 0.90.

	Levels			**Loads**			**User choices**	
Pile top	12.00	[mSKN]	Force	500	[kN]	Corrosion	No	
Impact high	9.36	[mSKN]	UDL	0.0	[kN/m2]	Impact	High	
Impact low	1.17	[mSKN]	E	128	[kNm]	Seabed	Low	
Water line	7.16	[mSKN]				Soil	LoChar	
Seabed high	-10.80	[mSKN]		**Factors toe depth**		PSFs Soil	Char	
Seabed nom	-10.80	[mSKN]	Passive β_{mob}	0.33	SLS	Soil type (class 4)	Sand moderately/dense	
Seabed low	-12.30	[mSKN]	Distribution C_h	0.50	[-]	Kolkschutzschutting	Ohne	
Soil profile bottom	-70.00	[mSKN]						
				Calculation summary		Displacement@TOP	353.4	[mm]
Seabed	-12.30	[mSKN]	t0	8.77	[m]	Displacement@IP	311	[mm]
Impact	9.36	[mSKN]	Δt	1.51	[m]	Stiffness@IP	**1607**	[kN/m]
Level Mmax (V=0)	-15.93	[mSKN]	Toe depth	-22.58	[mSKN]	Energy absorption	78	[kNm]
Mmax	12107	[kNm]	0.2*t0	1.75	[m]	Applied toe depth	-32.00	[mSKN]
evel Ersatzkraft (M=0)	-21.07	[mSKN]	t_{fix}	6.88	[m]	Total pile length	44.00	[m]
Ersatzkraft	6154	[kN]	Toe depth	-27.95	[mSKN]	Total pile weight	78.7	[t]

Figure 27 SLS pile fixity check

8.4.2 Fatigue limit state

For the purpose of finding the governing forces in the fatigue limit state the same approach as for the STR/GEO limit state was used. The response of the Ansys Aqwa model (pile reaction force in kN as a function of pile stiffness in kN/m) was crafted with model runs with defining stiffnesses. Those defining stiffnesses are the stiffnesses at low and high impact and the stiffness of 5000kN/m that yield the highest reaction force of 241 kN.

Figure 28 shows the force-displacement diagram made with the DMC Blum sheet from which the pile stiffness at high impact (used in the Ansys Aqwa model) has been derived.

Figure 29 shows the force-displacement diagram made with the DMC Blum sheet from which the pile stiffness at low impact (used in the Ansys Aqwa model) has been derived.

geprüft

Subject Pile design pontoon facility

HW	6.40	[mSKN]
Freeboard	2.20	[m]
Return period	10	[years]
Diameter	1500	[mm]
Wand dikte	50	[mm]
Kolkschuttung	OHNE	
Seabed	Low	
Soil	LowChar	

	Frep	w		k
	[kN]	[mm]	[m]	[kN/m]
1	25	9	0.00895	2793
2	50	20	0.01956	2556
3	75	31	0.03106	2415
4	100	43	0.04319	2315
5	125	56	0.05583	2239
6	150	69	0.06893	2176
7	175	82	0.08244	2123
8	**200**	**96**	**0.09631**	**2077**
9	225	111	0.11053	2036
10	250	125	0.12508	1999
11	275	140	0.13993	1965
				2245

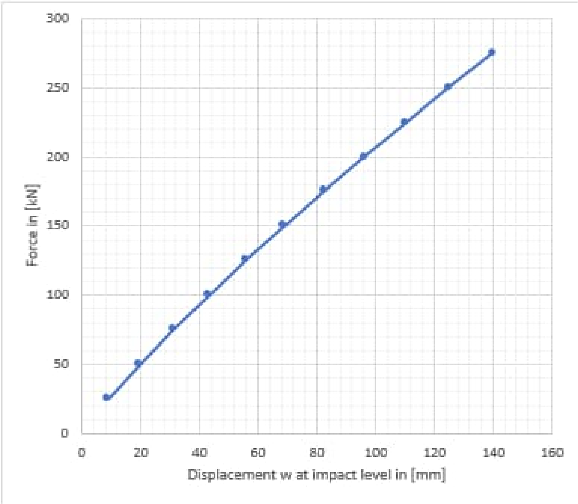


Figure 28 Pile stiffness at high impact

LW	-0.71	[mSKN]
Freeboard	2.20	[m]
Return period	10	[years]
Diameter	1500	[mm]
Wand dikte	50	[mm]
Kolkschuttung	MIT	
Seabed	High	
Soil	HiChar	

	Frep	w		k
	[kN]	[mm]	[m]	[kN/m]
1	25	1	0.00120	20833
2	50	2	0.00244	20492
3	75	4	0.00370	20270
4	100	5	0.00497	20121
5	125	6	0.00626	19968
6	150	8	0.00756	19841
7	175	9	0.00887	19729
8	200	10	0.01018	19646
9	225	12	0.01151	19548
10	250	13	0.01284	19470
11	275	14	0.01418	19394
				19938

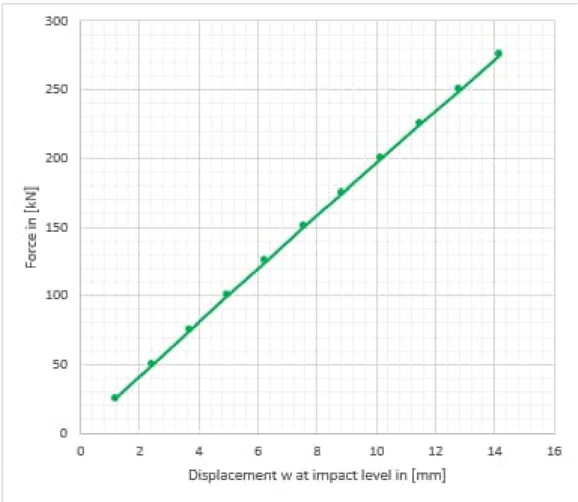


Figure 29 Pile stiffness at low impact

Gemäß Baugrundgutachten wird die p-y-Methode für hor. Pfahl-Design empfohlen.
-> Aufsteller-Erklärung siehe gesonderte Technical Note bzw. folgende Revision.

- Berücksichtigung der Auswirkungen der Nutzung der p-y-Methode auf die Verformungen bei Bemessung der Dalbenschlösser.

geprüft

Subject Pile design pontoon facility

Ergebnisse der ANSYS-AQWA-Modellierung: Als richtig vorausgesetzt.

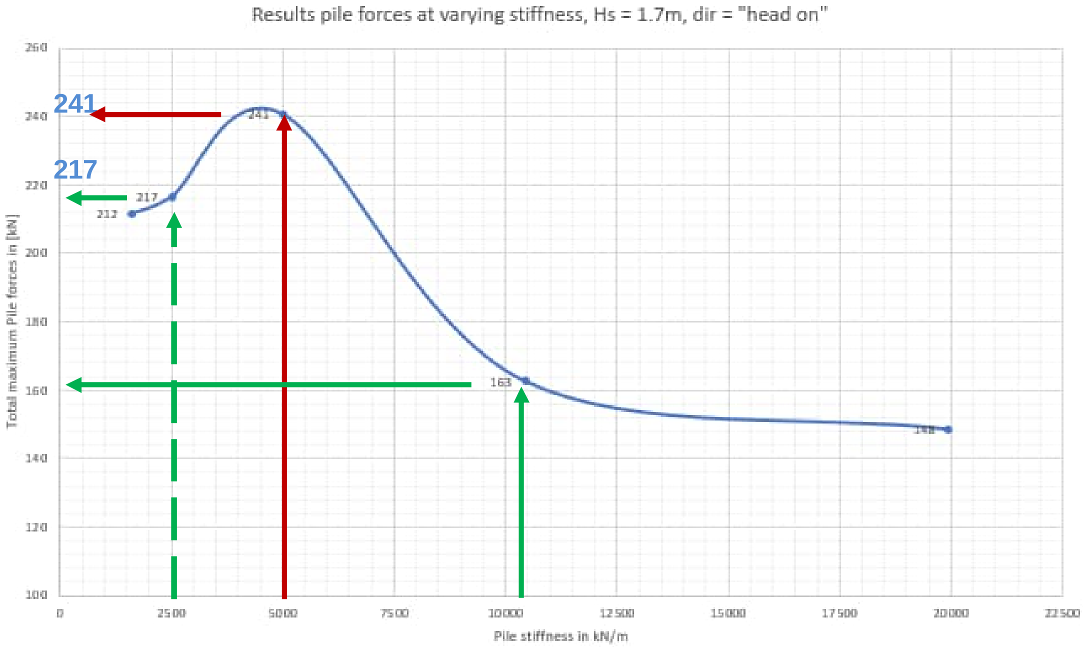
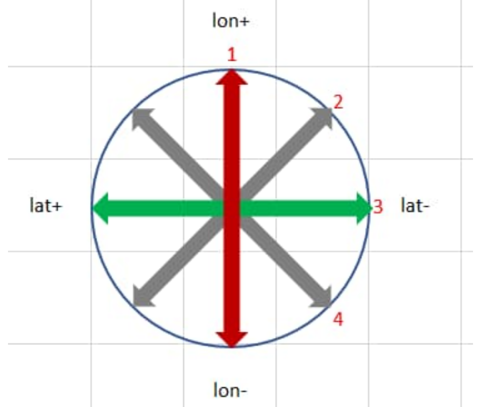


Figure 30 Pile reaction force as a function of pile stiffness in fatigue limit state

The piles sway back and forth whereby the stress changes from maximum positive to maximum negative (or the other way around). In the fatigue limit state reaction force range is considered (instead of maximum reaction force) resulting in a stress range which is to be checked to the maximum allowable stress range determined in document [1].

The result of the Ansys Aqwa model runs is per pile, reaction forces in longitudinal direction (the pontoon x-axis) and lateral direction (the pontoon y-axis). In both direction the maximum positive and the maximum negative are given. The maximum force range is determined with the formulas given in Figure 31.



Maximum of:

1. lon+ + lon-
2. $\sqrt{(lon+^2+lat-^2)} + \sqrt{(lon-^2+lat+^2)}$
3. lat+ + lat-
4. $\sqrt{(lon-^2+lat-^2)} + \sqrt{(lon+^2+lat+^2)}$

lon+ and lon- are the reaction forces in the longitudinal direction of the pontoon (x-direction) lat+ and lat- are the reaction forces in the lateral direction of the pontoon (y-direction)

Figure 31 Determining maximum force range for fatigue

geprüft

Subject Pile design pontoon facility

Forces in kN		19938	10434	5000	2500	1599
Side	Si1	168	215	342	313	308
	Si2	160	206	325	297	292
	Si3	154	197	311	284	278
	Si4	156	189	307	284	277
	Si5	161	194	319	298	294
	Si6	167	202	335	315	313
Stern	S1	194	247	408	363	356
	S2	198	253	424	370	360
	S3	202	260	440	378	365
Bow	B1	191	231	400	368	371
	B2	198	239	416	376	375
	B3	209	247	433	384	380
	MAX	209	260	440	384	380

Table 5 Maximum reaction force ranges per pile in kN for several stiffness runs

Table 5 shows the reaction forces in kN for the 12 piles (for pile numbering see Figure 2) for pile stiffnesses 19938, 10434, 5000, 2500 and 1599 kN/m. The maximum force range occurs in pile S3 for a pile stiffness of 5000 kN/m.

Figure 32 shows plots for both the maximum reaction forces (Figure 30) and the maximum reaction force ranges (Table 5). It has been noted that the used method shown in Figure 31 yields a lower reaction force range then 2 times the maximum reaction force.

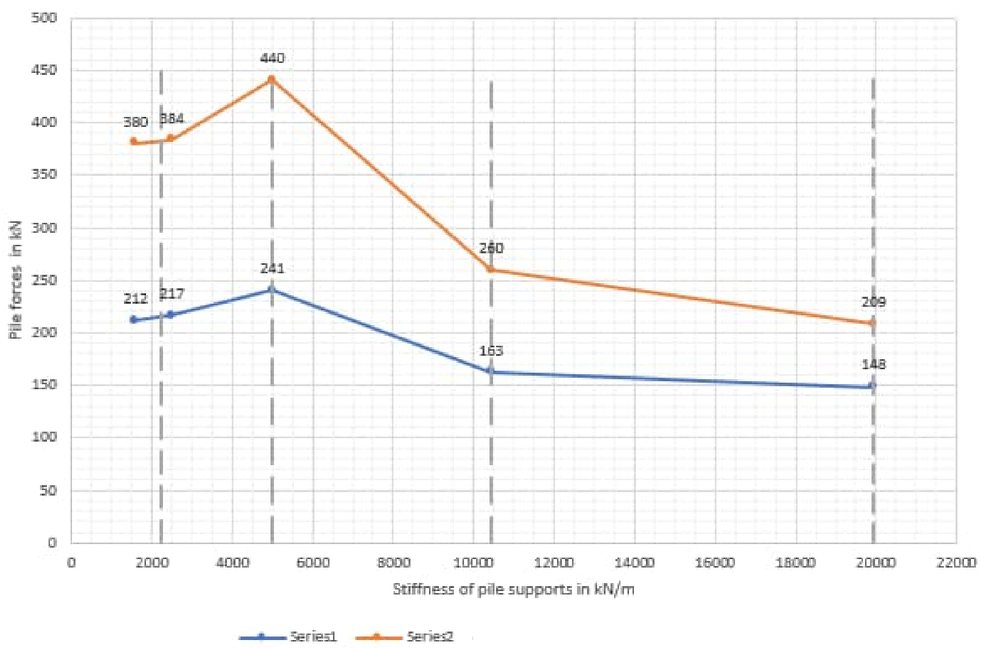


Figure 32 Pile forces as a function of stiffness of pile supports

geprüft

Subject Pile design pontoon facility

For the extreme levels and at the peak ranges the BLUM sheet is used to find maximum moments in the pile whereby also combinations of soil parameters (Low, Average, High) and sea bed level (High, Low) are considered. The combinations and the resulting stress ranges are shown in Table 6.

W_el=	7.99E+07	[mm3]								
CMB	Stiffness	ΔF	Impact	Seabed	Soil	chutzschi	F	Fw+c	Moment	Stress
[-]	[kN/m]	[kN]	[mSKN]	[-]	[-]	[-]	[kN]	[kN]	[kNm]	[N/mm2]
1	2055	384	8.60	Low	LoChar	Ohne	192	23	4832	121
2	5001	440	2.02	Low	LoChar	Ohne	220	26	3990	100
3	4999	440	2.18	Low	LoChar	Mit	220	26	3911	98
4	5002	440	4.44	High	HiChar	Ohne	220	26	4107	103
5	5000	440	8.24	High	HiChar	Mit	220	26	4696	118
6	10250	260	4.19	High	HiChar	Mit	130	16	2194	55
7	18511	209	1.49	High	HiChar	Mit	105	13	1454	36

Table 6 resulting stresses at peak, high and low impact stiffnesses for fatigue limit state (1/10 year conditions)

8.4.3 Ice action

Ice action has been considered in accordance with sections 4.11.2 and 4.11.3.1 of document [19]. See document [1] for the appreciation of the parameters involved in the calculation of the ice force. Since the pile has also been evaluated for higher forces and impact levels it is concluded that ice action is not governing.

Porosity	292	[‰]
Salinity (Tab.4.13)	12	[‰]
Temperature at the bottom of the ice sheet	-2.0	[°C]
Temperature at the top of the sheet ice	1.0	[°C]
Average ice temperature	1.5	[°C]
Ice pressure resistance	0.926	[MN/m2]
Specific strain speed	0.001	[s-1]
Average line load	0.122	[MN/m]
Contact coefficient	0.33	[-]
Thickness of the ice (Tab.4.14)	0.40	[m]
Length of pontoon	67	[m]
Number of piles resisting eisdruck	12	[m]
Kraft per Dalbe	683	[kN]

Table 7 Numeric values of parameters involved in calculation of ice action

geprüft

9. RESULTS

Important notices	⚠
• Strict specifications for fabrication and installation apply • Non verticality of the piles and other imperfections will result in additional force on the piles since this will result in an unequal load distribution.	

9.1 Results STR/GEO and fatigue limit state

It has been concluded that for the STR/GEO limit state section check, combination 4 is governing. Figure 33 shows the unity checks for wall thicknesses from 30mm up to 60mm for diameter 1500mm and for 2 steel qualities S355 and S460 in combination 4. From and above t=40mm, those qualities have reduced yield. Please note that both wall thickness as steel quality impacts the section class and thus the unity check.

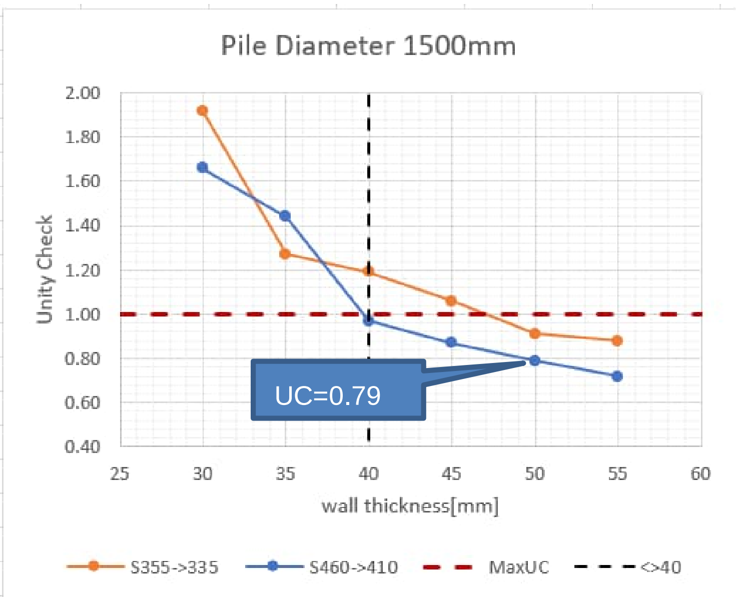


Figure 33 Unity checks for D=1500 and S355/460

The allowable fatigue stress range has been determined in document [1]. It is equal to 130 N/mm2. In Table 6 it may be observed that the maximum occurring stress range is equal to 121 N/mm2. Therefore the fatigue check is satisfactory.

In section 8.4.1 it was concluded that the pile fixity in the SLS and the pile stability in the ULS are satisfactory. Or simply put: the checks show that the piles are deep enough.

geprüft

Subject Pile design pontoon facility

At this stage where robustness is the driving factor for design, DMC deems the following option feasible when considering the STR/GEO limit state (ULS), serviceability limit state (SLS) and fatigue.

Part	Length [m]	D-t [mm]	Steel quality
Top	16	D1500-50	S355
Middle	22	D1500-50	S460
Bottom	6	D1500-50	S355

9.2 Additional analyses

The following additional analyses have been performed:

1. Influence of lower low characteristic value of the internal friction angle of layer 5
2. STR/GEO section check at low water grade S355 with pitting corrosion
3. Fatigue limit state at low water check by comparing maximum bending moment in the soil to the bending moment at low water
4. Influence of axial force due to pile weight and downward friction between the pile and the pontoon
5. Grouping effect

9.2.1 Internal friction angle ϕ = 32.5° (LoChar) for layer 5

The friction angle goes beyond 35 degrees in the lower values and therefore the DMC BLUM sheet has been altered by entering an internal friction angle of 32.5° for layer 5. Note that the pile depth is -32.0 m SKN so layers L6 and L7 are of no consequence for the pile design. The impact of using a value of 32 ° for layer 5 instead of 37.40 is insignificant.

Soil profile							
ID	Name	Layer-thickness	Level		LoChar	Expected	HiChar
			Top	Bottom	Friction angel		
		[m]	[mSKN]	[mSKN]	[DEG]		
1	L1	2.63	-12.30	-13.43	25.00	25.00	30.00
2	L2	3.07	-13.43	-16.50	25.00	27.50	30.00
3	L3	6.78	-16.50	-23.28	31.00	35.40	37.00
4	L4	6.52	-23.28	-29.80	32.50	34.60	37.00
5	L5	7.98	-29.80	-37.78	37.40	38.00	42.00
6	L6	2.80	-37.78	-40.58	34.20	35.00	37.10
7	L7	10.60	-40.58	-51.18	37.60	38.40	39.90

Table 8 Soil profile (low seabed) with characteristic values for the internal friction angle

9.2.2 STR/GEO limit state check at low water

The focus of the STR/GEO limit state check has been on the maximum bending moment. This maximum bending moment occurs in the ground where corrosion rates are low. Corrosion is much more severe around the low water line. Document [23] states that rates in excess of 1mm per year have been recorded.

geprüft

Subject Pile design pontoon facility

2.2.4 Low Water Zone (0.5 m Below MLWS to LAT)

Corrosion in this zone is relatively severe due to differential aeration at the uppermost point of continuous steel immersion, where electrolyte is permanent and oxygen levels peak. Corrosion rates of 0.08 to 0.17 mm/side/year are typical, but they can become very severe (concentrated) due to MIC by SRB and/or metal-reducing bacteria (MRB). With ALWC, typical corrosion rates of 0.5 mm/side/year can be expected, and rates in excess of 1 mm/side/year have been reported.

Table 9 Abstract from document [23]

Also, at low water, the pile has a grade of S355. A check with the DMC BLUM sheet has been performed with a low water corrosion of 10*1.0=10mm corrosion and a yield of 335 N/mm2 and in the governing combination 4. The unity check for the top section of 16 meter long with S355 is equal to 0.53 and thus satisfactory.

9.2.3 Fatigue limit state check at low water

The SN curve that has been used in the fatigue analyses is curve F from document [14] with a stress concentration factor of 1.27. See document [1]. This table is appropriate for steel elements in seawater with cathodic protection. The low water zone is alternating wet and dry and typically where severe corrosion occurs. A higher stress concentration for this zone applies.

2.4.5 S-N curves in seawater with cathodic protection

S-N curves for seawater environment with cathodic protection are given in Table 2-2 and Figure 2-8. The T curve is shown in Figure 2-9. For shape of S-N curves see also comment in 2.4.4.

Table 2-2 S-N curves in seawater with cathodic protection

S-N curve	*$N \le 10^6$ cycles*		*$N > 10^6$ cycles*	*Fatigue limit at 10^7 cycles*)*	*Thickness exponent k*	*Stress concentration in the S-N detail as derived by the hot spot method*
	m_1	$\log \bar{a}_1$	$\log \bar{a}_2$ $m_2 = 5.0$			
B1	4.0	14.917	17.146	106.97	0	
B2	4.0	14.685	16.856	93.59	0	
C	3.0	12.192	16.320	73.10	0.15	
C1	3.0	12.049	16.081	65.50	0.15	
C2	3.0	11.901	15.835	58.48	0.15	
D	3.0	11.764	15.606	52.63	0.20	1.00
E	3.0	11.610	15.350	46.78	0.20	1.13
F	3.0	11.455	15.091	41.52	0.25	1.27
F1	3.0	11.299	14.832	36.84	0.25	1.43
F3	3.0	11.146	14.576	32.75	0.25	1.61
G	3.0	10.998	14.330	29.24	0.25	1.80
W1	3.0	10.861	14.101	26.32	0.25	2.00
W2	3.0	10.707	13.845	23.39	0.25	2.25
W3	3.0	10.570	13.617	21.05	0.25	2.50
T	3.0	11.764	15.606	52.63	0.25 for SCF ≤ 10.0 0.30 for SCF >10.0	1.00

*) see also 2.11

Figure 34 Table 2.4.5 of document

S-N curves in alternating air and seawater without cathodic protection are not given in document [14]. Therefore the following approach has been followed. The governing stress range at the location of the maximum bending moment equals 121 N/mm2. See also section 8.4.1 and Table 6. When the occurring stress at low water is lower than 121/2.0 ≈ 60 N/mm2 the fatigue at low water will not be governing. This is engineering judgement.

The maximum bending moment in the governing fatigue limit state combination 1 is equal to 4832 kNm. The bending moment at low water -0.71 mSKN (1/10 year conditions) is equal to 2000 kNm. The stress at low water level is approximately 2000/4832*121 ≈ 50 N/mm2.

9.2.4 Second order effect due to axial force

The pile weighs 78.7 [t]. The axial force on the section of the maximum bending moment is equal to: (12 - -16.46)/(12 - -32) * 78.7 * 9.81 * 1.35 = 674 kN.

The maximum reaction force on the pile is 1295 kN (including current and waves directly on the pile). With a friction equal to 0.40 the downward friction force on the pile, adding to the compression, is equal to 518 kN.

The axial pressure in the governing section due to weight and friction is equal to (500+518)*10^3 / 227294 = 5.3 N/mm2. In the governing STR/GEO limit state combination the bending moment is equal to 30883 kNm. The pile is just yielding. (UC section class 3 is equal to 1.04.) Therefore the extra compression of 5.3 N/mm2 can be ignored.

The second order effect of the pile is the axial force times the length of the pile times the inclination (which is conservatively assumed to be 1/50) and therefore ΔM = 1192*44/50 = 1049 kNm. The DMC BLUM sheet indicates that an extra force of 1336-1295 = 41 kN has to be applied to the pile to achieve that ΔM.

In conclusion the bending moment in the governing combination 4 increases due to second order effect from 30880 kNm to 31929kNm. Figure 35 shows that the increased bending moment is still at the start of the plastic branch and thus can be resisted by the section. It can therefore be concluded that the second order effect has little impact.

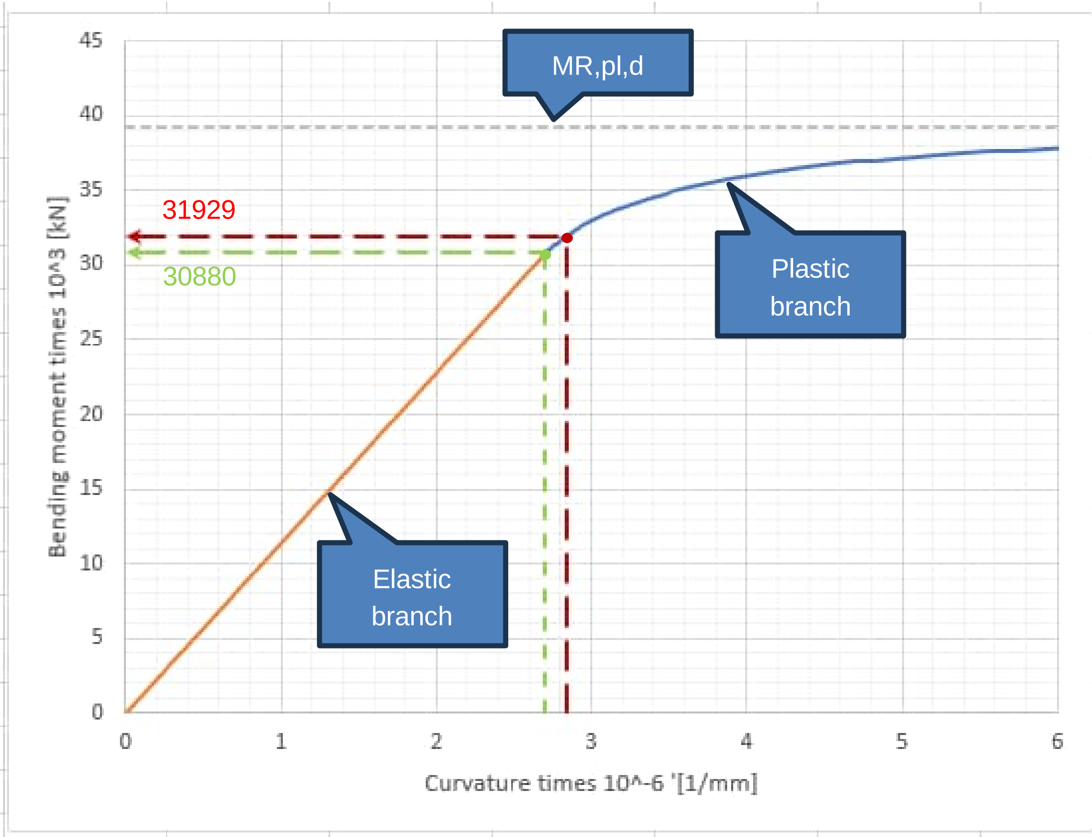


Figure 35 Increase in bending moment due to second order effect axial force

9.2.5 Grouping effect

In this document a straight forward grouping effect evaluation has not been done. Instead, the sensitivity of the decrease in ground pressure on the pile has been investigated.

Blum assumes full passive mobilization of the soil. However, when a spatial soil wedge develops (see Figure 38) due to a reaction force on the pile, soil wedges from other close by piles may overlap thereby decreasing the ability to resist the force.

The DMC BLUM sheet has the possibility to decrease the fully mobilized soil pressure on the pile by means of the factor γR,e. (See Table 4.) In the STR/GEO limit state for instance the factor is equal to 1.15. It has been investigated to what extend this factor can be increased before the section check or the fixity check fails.

Table 10 shows the input for the sensitivity analyses.

Subject Pile design pontoon facility

Limit State	STR/GEO	SLS
Check	Section	Pile fixity
Return period [year]	100	10
Force [kN]	1200	500
Impact level EHW [mSKN]	9.36	8.60
Seabed	Low	Low
Soil parameters	Low Char	Low Char
Scour	YES	YES

Table 10 Input for soil pressure reduction (sensitivity analyses)

For the STR/GEO section check (ULS) it holds that UC=1.00 when γR,e=3.0 (161% increase) the pile stability is sufficient.

For the SLS pile fixity check it holds that the pile is just deep enough when γR,e=3.1 (210% increase).

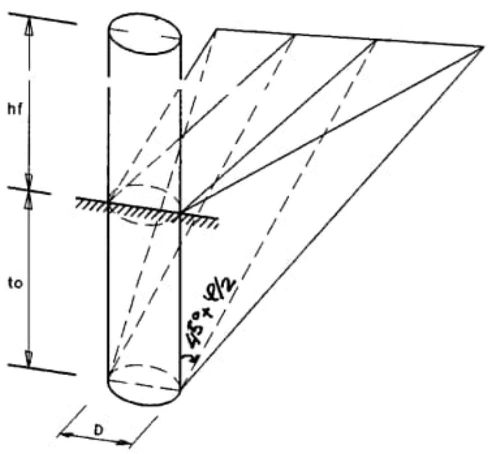
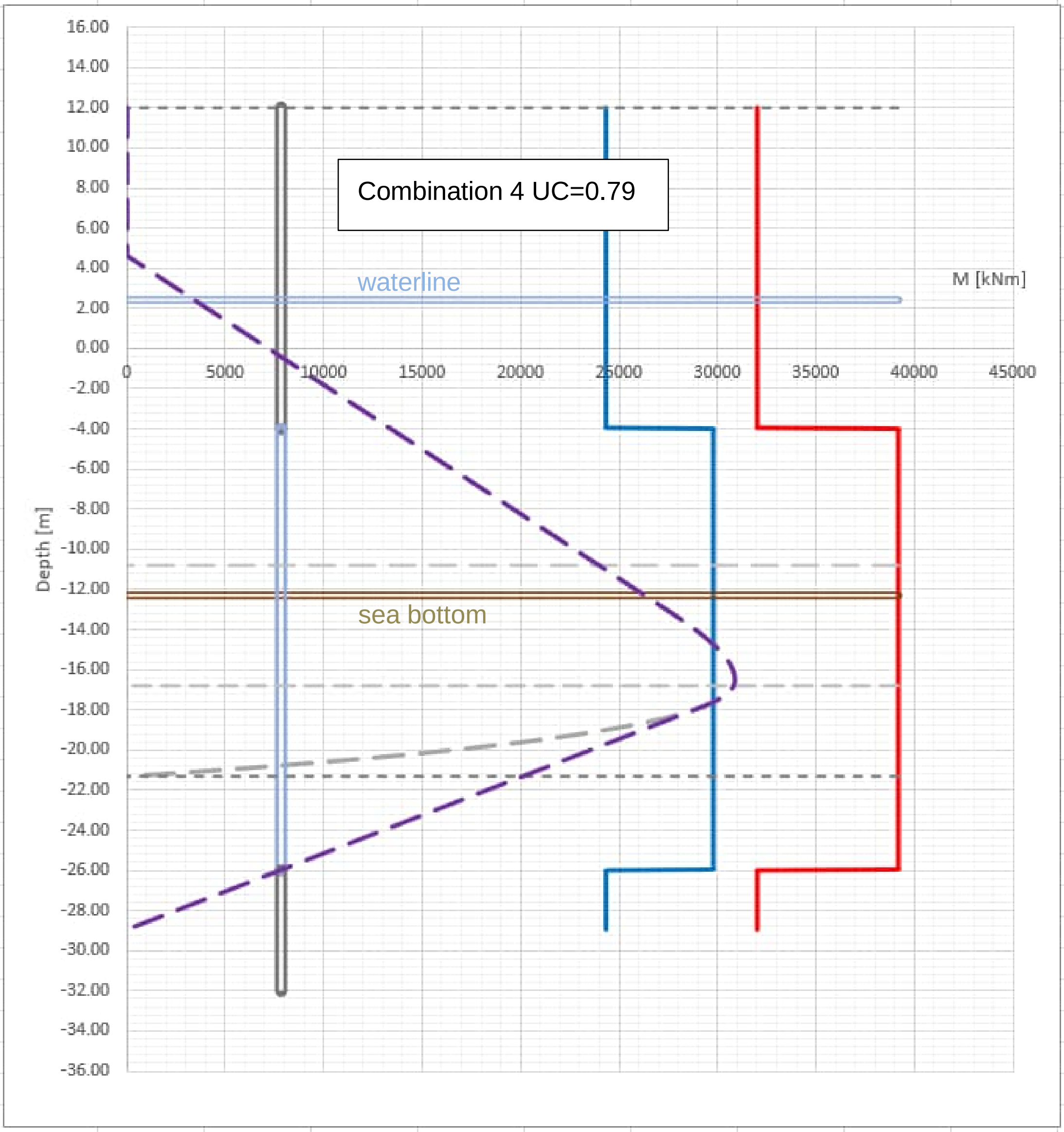


Figure 36 3D impression of mobilised soil wedge

Subject Pile design pontoon facility

Appendix A DMC BLUM sheet output

STR/GEO bending moment line (purple dotted) with capacity lines. Blue is yield and red plastic resistance.



Appendix B Pontoon motions

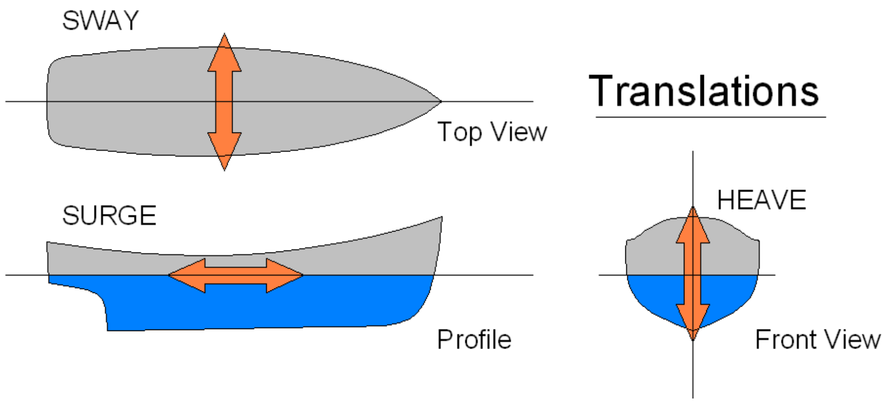


Figure 37 Translations sway, surge and heave

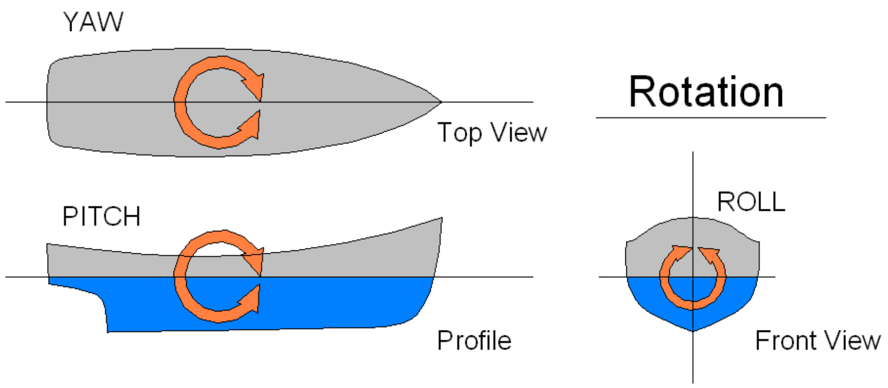


Figure 38 Rotations: yaw, pitch and roll

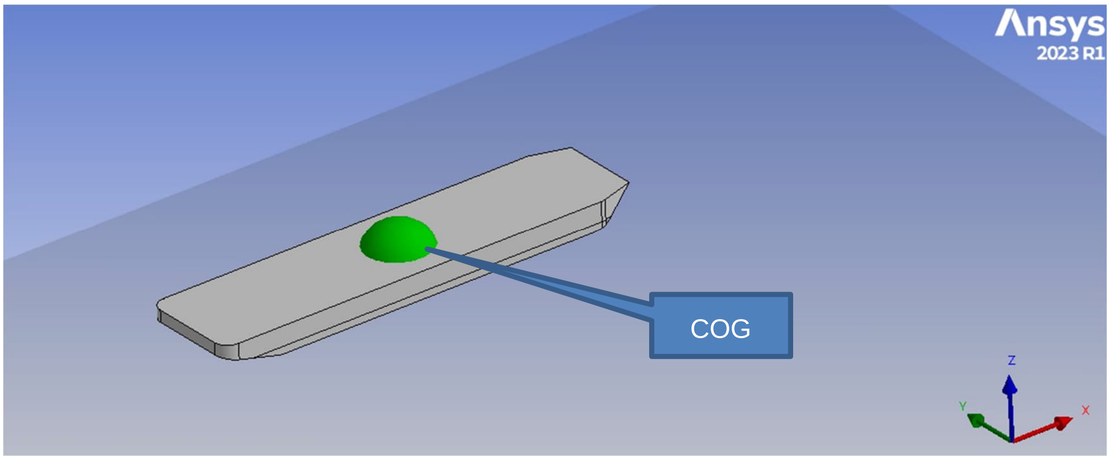


Figure 39 Directions Ansys Aqwa model, translations of COG is model output

Subject Pile design pontoon facility

Table 11 Ships motions related to axis system used in Ansys Aqwa

Motion	Term	Unit
x-direction	Surge	[mm]
y direction	Sway	[mm]
z direction	Heave	[mm]
rotation around the x-axis	Roll	[degree]
rotation around the y-axis	Pitch	[degree]
rotation around the z-axis	Yaw	[degree]

Subject Pile design pontoon facility

Appendix C Geotechnical parameters

ACP
GEOTECHNIK

Geotechnischer Bericht

Teilprojekt: **Bootsanleger (Pontoon)**

Nr. 23A012.00.00 Rev.0.0

Datum 12. Dezember 2023

Beratung | Planung | Gutachten
Objekt- und Tragwerksplanung für Baugrubensicherungen
Baugrund- und Gründungsgutachten
Offshore-Geotechnik
Spezialtiefbau
Böschungen und Stützmauern
Deiche und Dämme
Altlastengutachten
Grundbaudynamik
Grundwasserströmung
Numerische Untersuchungen von Boden-Bauwerks-Wechselwirkungen

Vereidigte Sachverständige
Gerichtsgutachten
Privatgutachten

Bauvorhaben
LNG Terminal Wilhelmshaven (FSRU)
Teilprojekt: Ponton/Bootsanleger (Pontoon)

Auftraggeber
ENGIE Deutschland AG / Tree Energy Solutions mit IMDC

erstellt durch:

Dipl.-Ing. Michael Müller
ACP Geotechnik GmbH

Dr.-Ing. Malek Hasan
ACP Geotechnik GmbH

ACP Geotechnik GmbH

Büro Berlin
Darwinstraße 15
10589 Berlin
Tel.: +49 (0)30 349906-70
Fax: +49 (0)30 349906-99
E-Mail: berlin@acp-geotechnik.de
web: www.acp-geotechnik.de

Büro Hannover
Georgstr. 38
30159 Hannover
Tel.: +49 (0)511 70034930
Fax: +49 (0)511 70034937
E-Mail: hannover@acp-geotechnik.de
web: www.acp-geotechnik.de

Geschäftsführer
Prof. Dr.-Ing. Martin Achmus
Dipl.-Ing. Michael Müller
Dr.-Ing. Thomas Schrepfer

Registergericht
AG Charlottenburg HRB 101773 B

Bankverbindung
LBB LandesBank Berlin
IBAN: DE44 1005 0000 0190 0536 90
SWIFT-BIC: BELADEBEXXX

gesehen

Subject Pile design pontoon facility

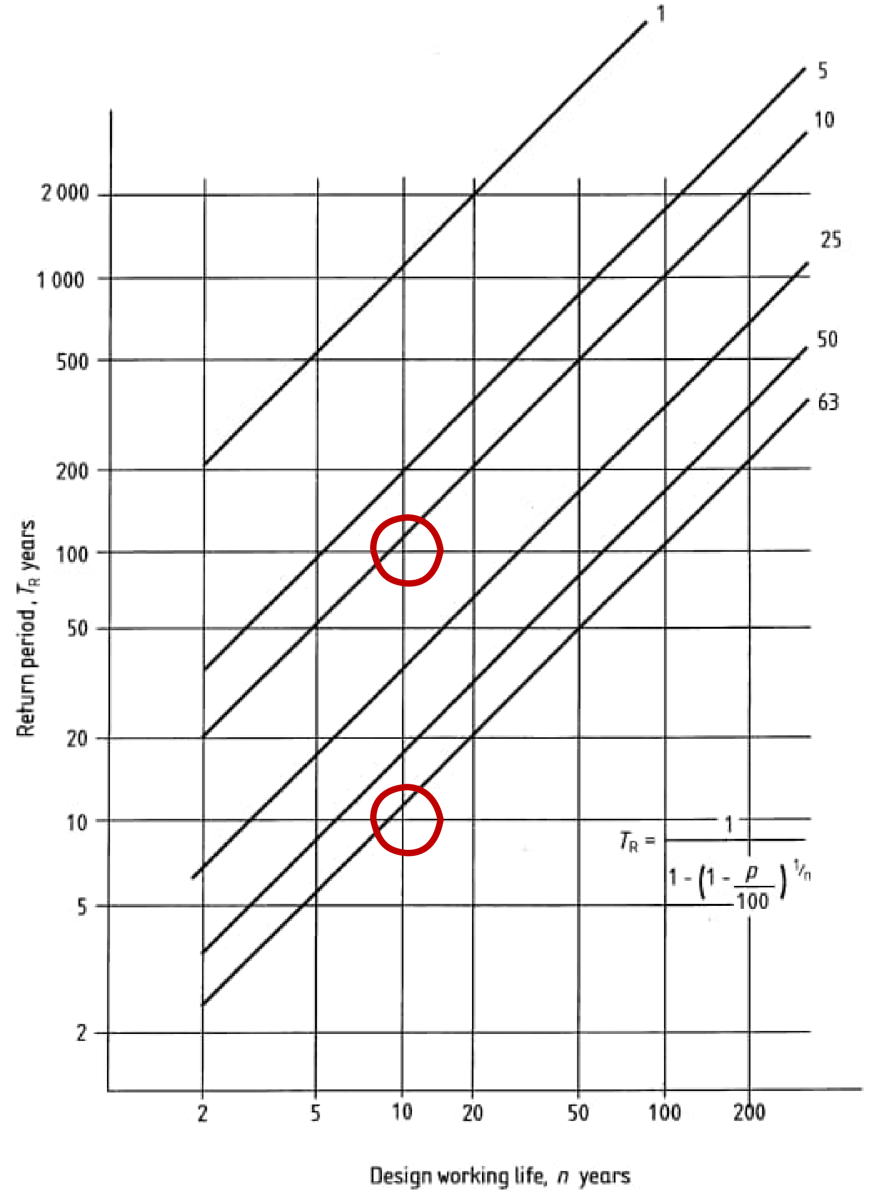
Ohne Kolkschutzschuttung	Depth under SBL		Δh	γ'	Saturated density		φ	Friction angel		δ	Delta		c	Effective cohesion	
	TOP	BOT			[kN/m3]			[DEG]			[DEG]			[kN/m2]	
-10.80 [mSKN]	[m]	[m]	[m]	LoChar	Expected	HiChar	LoChar	Expected	HiChar	LoChar	Expected	HiChar	LoChar	Expected	HiChar
L1: Sand/Schluff (locker)	0.00	2.63	2.63	16.55	17.55	18.55	25.00	25.00	30.00	16.67	16.67	20.00	0.00	0.00	0.00
L2: Sand/Schluff (locker)	2.63	5.70	3.07	16.55	17.55	18.55	25.00	27.50	30.00	16.67	18.33	20.00	1.00	2.00	5.00
L3:Sand (mitteldicht)	5.70	12.48	6.78	17.05	18.55	20.05	31.00	35.40	37.00	20.67	23.60	24.67	0.00	0.00	0.00
L4:Sand (mitteldicht)	12.48	19.00	6.52	19.05	19.55	21.05	32.50	34.60	37.00	21.67	23.07	24.67	0.00	0.00	0.00
L5:Sand (dicht)	19.00	26.98	7.98	20.05	20.55	21.05	37.40	38.00	42.00	24.93	25.33	28.00	0.00	0.00	0.00
L6:Sand (mitteldicht)	26.98	29.78	2.80	19.55	20.55	21.05	34.20	35.00	37.10	22.80	23.33	24.73	0.00	0.00	0.00
L7:Sand (dicht)	29.78	40.38	10.60	21.05	21.55	22.05	37.60	38.40	39.90	25.07	25.60	26.60	0.00	0.00	0.00
Bottom soil profile: -51,18 [mSKN]	40.38														

Mit Kolkschutzschuttung	Depth under SBL		Δh	γ'	Saturated density		φ	Friction angel		δ	Delta		c	Effective cohesion	
	TOP	BOT			[kN/m3]			[DEG]			[DEG]			[kN/m2]	
-9.80 [mSKN]	[m]	[m]	[m]	LoChar	Expected	HiChar	LoChar	Expected	HiChar	LoChar	Expected	HiChar	LoChar	Expected	HiChar
L0: Steinschuttung	0.00	1.00	1.00	18.05	19.05	20.05	20.00	37.50	55.00	13.33	25.00	36.67	0.00	0.00	0.00
L1: Sand/Schluff (locker)	1.00	3.63	2.63	16.55	17.55	18.55	25.00	25.00	30.00	16.67	16.67	20.00	1.00	2.00	5.00
L2: Sand/Schluff (locker)	3.63	6.70	3.07	16.55	17.55	18.55	25.00	27.50	30.00	16.67	18.33	20.00	0.00	0.00	0.00
L3:Sand (mitteldicht)	6.70	13.48	6.78	17.05	18.55	20.05	31.00	35.40	37.00	20.67	23.60	24.67	0.00	0.00	0.00
L4:Sand (mitteldicht)	13.48	20.00	6.52	19.05	19.55	21.05	32.30	34.60	38.00	21.53	23.07	25.33	0.00	0.00	0.00
L5:Sand (dicht)	20.00	27.98	7.98	20.05	20.55	21.05	37.40	38.00	42.00	24.93	25.33	28.00	0.00	0.00	0.00
L6:Sand (mitteldicht)	27.98	30.78	2.80	19.55	20.55	21.05	34.20	35.00	37.10	22.80	23.33	24.73	0.00	0.00	0.00
Bottom soil profile: -40,58 [mSKN]	30.78														

gesehen

Subject Pile design pontoon facility

Appendix C Relationship return period, probability and design life.



NOTE T_R is the return period of a particular extreme wave condition in years. p is the probability of a particular extreme wave condition occurring during design working life n years.

Figure 4 — Relationship between design working life, return period and probability of wave heights exceeding the normal average

gesehen