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Foundation advice

Biogas Installation in Leeuwarden

VN-78441-1 | 5th March 2021

Wiertsema & Partners

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Project number:	VN-78441-1
Client:	BioLNG ECL
	Ynduksjewei 4
	8914 CA Leeuwarden

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1 Introduction

At the request of BioLNG ECL in Leeuwarden, Raadgevend Ingenieursbureau Wiertsema and Partners B.V. issued a foundation (draft) advice for the construction of a Biogas Installation in Leeuwarden.

This preliminary advice is based upon exploratory CPTs. In accordance with NEN 9997+2C:2017, additional CPTs are required. These additional CPTs may demand a review of this advice.

This report discusses the bearing and tensile capacity at foundation level as well as the associated consolidation, for the purpose of the design stage.

We emphasise that this advice does not provide a detailed discussion of the implementation of the advised foundation type, which is the responsibility of the contractor.

1.1 Data

This advice was prepared using data from a soil survey, which was also performed by Wiertsema and Partners. This survey is presented in the following report:

Ref [1] 'Geotechnisch onderzoek' with project number VN-67926-1 versie 1, report number R52151, d.d. 8th September 2017)

1.2 Standards and guidelines

The following standards and guidelines apply to this report:

- NEN 9997+C2:2017, Geotechnisch ontwerp van Constructies – Deel 1: Algemene regels, november 2017.

1.3 Quality assurance

The foundation advice was prepared under our quality system NEN-EN-ISO-9001 and environmental management system NEN-EN-ISO-14001. Wiertsema & Partners has an HSE management system VCA**.

1.4 Report structure

The introduction in this first chapter is followed by the project description in chapter 2. Chapter 3 describes the soil structure. The foundation is detailed in chapter 4, whilst chapter 5 describes the calculation of the pile bearing and tensile capacities. The results are presented in chapter 6. The implementation is detailed in chapter 7, followed by the closing remarks in chapter 8.

2 Project outline

This projects concerns the construction of a new biogas installation in Leeuwarden. Figure 2.1 shows the biogas plant's location as part of the Leeuwarden energy campus. Figure 2.2 shows a technical drawing of the biogas installation's components. Attachment 1 presents a map of the surveyed sites.

Figure 2.1: biogas plant location within the Leeuwarden energy campus

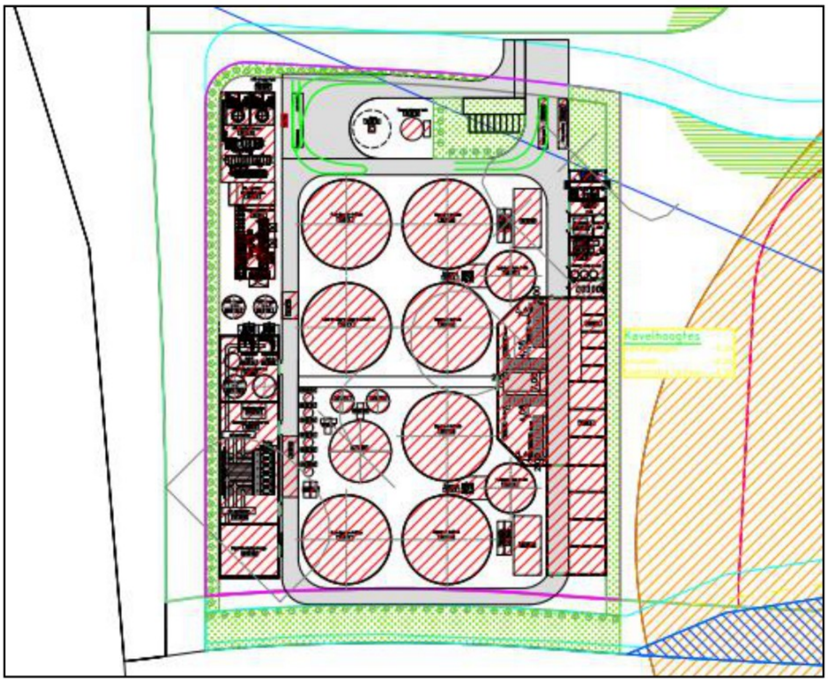

Figure 2.2: technical drawing of biogas installation. North side up.

3 Soil structure

3.1 Available survey data

The survey data available for this report was collected by Wiertsema & Partners. It consists of:

- 5 continuous CPTs with electrical conus and electrical data collection. De CPTs were performed following the NEN-EN-ISO 22476-1 class 3 standard and reached maximum depths of approximately NAP –17,15 m to -25,50 m (DKM001, DKM002, DKM005-DKM007);
- 3 manual drillings to 3,0 m below ground level (B001, B003, B004).

The collected data is georeferenced relative to NAP.

3.2 Ground level

Ground levels measured at the CPT tips and the manual drilling site vary from NAP +0,12 m to -0,41 m.

The elevation measurements were performed with the aim of recording the soil structure relative to NAP. The reported elevations are not suitable for any other purposes.

3.3 Soil structure

Based on the soil survey's results, the project location's soil structure is summarised in tables 3.1-3.5.

Table 3.1: summative table of soil structure at DKM001

Depth [m NAP]			Soil description
maaiveld	to	-4,50	Compressible clay, locally weakly organic and/or weakly silty
-4,50	to	-7,25	Sand, lightly to strongly clayey/silty
-7,25	to	-9,25	Compressible clay
-9,25	to	-10,50	Clay or loam, locally strongly sandy
-10,50	to	-11,00	Sand, weakly to moderately compacted
-11,00	to	-17,25	Lightly sandy clay/loam, locally strongly sandy (boulder clay)
-17,25	to	-17,50*	Sand, strongly to moderately compacted

**maximum reached depthTable 3.2: summative table of soil structure at DKM002*

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Table 3.1: summative table of soil structure at DKM002

Depth [m NAP]			Soil description
maaiveld	to	-3,50	Compressible clay, locally weakly organic and/or weakly silty
-3,50	to	-6,00	Sand, lightly to strongly clayey/silty
-6,00	to	-6,50	Peat or strongly organic clay
-6,50	to	-10,50	Sand, weakly to moderately compacted, locally cut by peat or strongly organic clay
-10,50	to	-16,75	Lightly sandy clay/loam (boulder clay)
-16,75	to	-17,25*	Sand, strongly to moderately compacted

**maximum reached depth*

Table 3.3: summative table of soil structure at DKM005

Depth [m NAP]			Soil description
maaiveld	to	-3,00	Compressible clay, locally weakly organic and/or weakly silty
-3,00	to	-6,00	Sand, lightly to strongly clayey/silty
-6,00	to	-6,75	Peat or strongly organic clay
-6,75	to	-11,00	Weakly to moderately compacted sand, locally cut by clay/loam layers
-11,00	to	-16,25	Strongly silty/clayey sand; loam, locally strongly sandy (boulder clay)
-16,25	to	-18,50*	Overconsolidated clay

**maximum reached depth*

Table 3.4: summative table of soil structure at DKM006

Depth [m NAP]			Soil description
maaiveld	to	-4,00	Compressible clay, locally weakly organic and/or weakly silty
-4,00	to	-6,00	Sand, lightly to strongly clayey/silty
-6,00	to	-10,50	Compressible clay
-11,50	to	-16,00	Clay/loam, locally strongly sandy (boulder clay)
-16,00	to	-20,25	Sand, strongly to moderately compacted
-20,25	to	-25,50*	Overconsolidated clay

**maximum reached depth*

Table 3.5: summative table of soil structure at DKM007

Depth [m NAP]			Soil description
maaiveld	to	-4,75	Compressible clay, locally weakly organic and/or weakly silty
-4,75	to	-6,25	Clay, lightly sandy
-6,25	to	-10,00	Weakly to moderately compacted sand, locally lightly clayey / silty
-10,00	to	-12,75	Interlayered sand and clay
-12,75	to	-17,25	Lightly to strongly sandy loam (boulder clay)
-17,25	to	-17,75*	Sand, strongly to moderately compacted

**maximum reached depth*

3.4 Ground water level

During the soil survey on 8th September 2017, ground water levels were measured in the manual drilling holes. These ranged from NAP -1,10 to -2,10 m.

These observations do not reflect longer-term ground water level fluctuations. The observed ground water level may have been perturbed by the drilling activities. Ground water levels may fluctuate in response to seasonal changes and changes in weather conditions. The observed ground water level only is an estimate, which should not be used for any purpose whatsoever without additional research.

In this report, a ground water level of NAP -1,10 m will be applied.

4 Foundation

4.1 General

Considering the observed soil structure and the expected load on the foundation, a pile foundation is advised.

In consultation with the client, a foundation on prefab concrete piles was investigated.

<u>Points of attention – prefab concrete piles</u>

- This preliminary advice is based upon exploratory CPTs. In accordance with NEN 9997+2C:2017, additional CPTs are required. These additional CPTs may demand a review of this advice.
- The driving of prefab concrete piles causes vibrations. Whilst compiling this advice, it was assumed that there are no vibration-sensitive objects and/or buildings in the near vicinity of the site. Hence, it was assumed that pile driving is feasible and permitted at this location. We did not assess whether this is correct or not. When uncertainties arise regarding this, we recommend to get in touch with us.

5 Pile foundation calculation

5.1 Geotechnical bearing capacity

The geotechnical bearing capacity of the sub-surface was assessed in accordance with Eurocode 7 (NEN-9997-1+C2). Considering the nature of the superstructure, both pressure- and tension-loaded piles were investigated. This only concerns axially loaded piles.

5.2 Starting points used

The following starting points were used for the calculation:

- The project falls under geotechnical category 2;
- The construction is considered to be non-rigid;
- The building will not have a basement;
- The pile head was assumed to be at NAP -0,50 m.
- Negative skin friction was applied down to a level of NAP -5,50 to -6,75 m;
- Positive skin friction was applied from a level of NAP -5,50 to -6,75 m;
- Assessing the ultimate limit state means that the following needs to be met:
  $F_{c;d} \leq R_{c;netto;d}$ for pression piles;
  $F_{t;d} \leq R_{t;d}$ for tension piles;
- The following pile factors were used in the calculation:

Pile type	Installation method	α_p	$\alpha_{s,\ sand}$	$\alpha_{s,\ clay}$	$\alpha_{s,\ loam}$	α_t	β
Prefab concrete pile	Driving	0,70	0,010	0,020	Cone friction number $\leq$0,025	0,007	1,0

- Correlation factors ξ_3 en ξ_4 = 1,39 en partial factor γ_b = 1,20, γ_s = 1,20;
- The serviceability limit state according to NEN 9997-1+C2 is adhered for assessing distortion. As the piles are only subjected to minimal distortion, these distortions are not decisive.

5.3 Extension of CPTs

In CPTs DKM001, DKM002 and DKM007, the CPTs were terminated at relatively shallow depths, as the cone was subjected to very high cone resistances, presumably due to the presence of a sand layer. Consequently, the CPTs are of insufficient length for the bearing capacity calculations according to NEN 9997: the 4D trajectory requires sufficient coverage of the layers underlying the pile tip to calculate the pile tip bearing capacity.

Hence, it was necessary to extend these CPTs. As a similar ca. 4,5 metre-thick layer was observed in adjacent CPT DKM006, these CPTs were manually extended appproximately 1.5-3 metres, down to NAP -20.00. A conservative cone resistance of 25 Mpa was used for the extension.

This assumption should be validated with the additional CPTs required for this project.

6 Results

Tables 5.1-2 present the maximum design value of the geotechnical bearing and tensile capacity for every CPT tip, pile (shaft) dimension and pile tip elevation.

The listed design values of the soil mechanical bearing capacity consist of pile tip and shaft resistance, with the negative skin friction deducted. The given tensile capacities are only applicable to single, quasi-static (variably pressure- and tension-loaded) piles. The listed carrying capacities only apply to vertically and axially, pressure- and tension-loaded piles.

Attachment 2 presents a detailed example calculation of the carrying capacity, the tensile capacity and the negative skin friction.

Tabel 5.1 overview net bearing capacity pression piles

All levels and elevations are relative to N.A.P.

CPT	ground level	pile tip level	$R_{c;net;d}$ [kN] 250x250D	290x290D	320x320D	350x350D
DKM001*	-0.10	-17.25	680.3**	853.4	995.0	1146.4
		-17.50	778.0	966.8	1120.1	1283.3
		-17.75	749.4	933.7	1083.5	1243.3
DKM002*	0.12	-17.00	631.3	794.6	928.4	1071.8
		-17.25	758.3	941.9	1090.9	1249.6
		-17.50	701.9	876.5	1018.7	1170.6
DKM005	0.10	-15.50	323.7	385.0	432.3	481.2
		-15.75	335.1	398.0	447.2	497.1
		-16.00	343.5	408.0	456.7	506.6
		-16.25	350.6	415.0	465.6	517.5
		-16.50	349.8	415.0	465.6	515.2
		-16.75	349.8	413.4	463.2	514.8
		-17.00	348.4	413.1	463.2	
		-17.25	348.2			
DKM006	-0.10	-16.25	385.3	488.7	572.7	664.2
		-16.50	479.5	608.6	714.8	829.0
		-16.75	530.0	668.4	781.8	903.4
		-17.00	547.1	683.1	792.4	911.6
		-17.25	675.2	847.3	987.9	1138.5
		-17.50	733.2	915.4	1064.6	1225.9
DKM007*	-0.41	-17.25	823.0	1020.2	1179.9	1349.8
		-17.50	858.1	1244.4	1435.0	1628.8
		-17.75	879.3	1085.5	1252.0	1428.7

**This CPT was extended. Please see 4.3 for explanation.*
***example calculation included in attachment 2.*

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Tabel 5.2 overview net tensile capacity tension piles

All levels and elevations are relative to N.A.P.

CPT	ground level	pile tip level	$R_{t;netto;d}$ [kN] 250x250T	290x290T	320x320T	350x350T
DKM001*	-0.10	-17.25	197.0	230.9	256.8	283.0
		-17.50	205.6	240.9	267.8	295.1
		-17.75	215.1	251.9	280.0	308.5
DKM002*	0.12	-17.00	141.3	166.3	185.4	204.9
		-17.25	150.9	177.4	197.7	218.3
		-17.50	160.4	188.4	209.9	231.8
DKM005	0.10	-15.50	184.5	216.1	240.2	264.7
		-15.75	192.9	225.9	251.1	276.6
		-16.00	198.6	232.6	258.5	284.7
		-16.25	203.6	238.4	264.9	291.7
		-16.50	203.6	238.4	264.9	291.8
		-16.75	203.8	238.7	265.2	292.2
		-17.00	203.9	238.9	265.6	
		-17.25	204.1			
DKM006	-0.10	-16.25	118.2	139.3	155.6	172.2
		-16.50	126.7	149.2	166.5	184.2
		-16.75	136.3	160.4	178.9	197.8
		-17.00	145.8	171.5	191.1	211.2
		-17.25	155.3	182.5	203.4	224.6
		-17.50	164.8	193.6	215.6	238.0
DKM007*	-0.41	-17.25	242.5	283.6	314.9	346.6
		-17.50	252.9	295.8	328.4	361.4
		-17.75	262.5	306.9	340.6	374.8

**This CPT was extended. Please see 4.3 for explanation.*

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7 Implementation

7.1 Prefab concrete piles

7.1.1 Impact block

When driving the prefab concrete piles, we recommend using a hydraulic block with a weight of 60 to 70 kN. A different block with a comparable energy transmission would suffice as well.

7.1.2 Driving method

The first pile should be driven at the site of a CPT. When driving, the number of impacts needed to drive down the pile a set distance of 25 centimetres should be recorded along the whole length of the pile (“kalenderen” in Dutch). This impact record serves as a guideline for the driving levels of the piles to be driven in in between the CPT sites. For all piles, the “kalender” values should be recorded along the last two metres, as well as the attained pile tip elevation and the applied driving energy.

When multiple pile tip elevations are used, we recommend to commence the pile driving at the CPT site with the deepest desired pile tip elevation. Subsequently, one would drive piles in the order of their desired pile tip elevation, from deeper to shallower.

7.1.3 Pile driving-related vibrations

Driving piles causes vibrations in the subsurface. Apart from the soil structure, the risk of damaging effects due to these vibrations depends on the driving method, the architectural condition of nearby buildings and/or objects as well as their proximity to the driving site.

By recording vibration levels during the pile driving activities, the risk of damaging nearby buildings can be assessed (as well as the degree of disturbance for individuals in adjacent buildings and the risk of damaging equipment). If desired, such measurements can be performed by Wiertsema & Partners.

8 Closing remark

If changes occur in the described construction plan or in the starting points used for this advice over the course of this project, we request you to get in touch with us. In this case, we can assess the implications of these changes on this report.

Attachment 1

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LEGENDA

DKM Diepsondering met plaatselijke wrijving

B Handboring

B Handboring met peilbuis

Situatietekening	Datum : 03.04.17	Gew: 04.04.17/AE
Realisatie Energiecampus Leeuwarden	Getekend: J	Gew: 25.04.17 JB
	Schaal : 1:1000	Gew:
	Formaat : A0	Gew:
Wiertsema & Partners RAADGEVEND INGENIEURS / Blad : 1-1	Opdracht: VN-67926-1	AKKOORD UITV

0 m 10 m 50 m

Attachment 2

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DETAIL BER. DRAAGVERMOGEN 250x250D; DKM001; N.A.P.-17.25

Uitgangspunten

- gehanteerde sondering : DKM001
- gehanteerde paal : 250x250
- paalpuntniveau : N.A.P.-17.25 m
- traject positieve kleef : N.A.P. -6.50 m tot: N.A.P.-17.25 m

Maximale draagkracht van de paalpunt

De maximale puntweerstand volgens art. 7.6.2.3 (e) bedraagt :

$$q_{b;max} = 0.5 * \alpha_p * \beta * s * ((q_{c;I;gem} + q_{c;II;gem})/2 + q_{c;III;gem})$$
$$= 10.977 \text{ MPa}$$

waarin :		in dit geval :
$q_{c;I;gem}$	= de gemiddelde waarde van de conusweerstanden over traject I	= 31.75 MPa
$q_{c;II;gem}$	= de gemiddelde waarde van de conusweerstanden over traject II	= 25.00 MPa
$q_{c;III;gem}$	= de gemiddelde waarde van de conusweerstanden over traject III	= 2.99 MPa
α_p	= paalklassefactor	= 0.70 -
β	= factor voor de paalvoetvorm	= 1.00 -
φ	= hoek van de inwendige wrijving	= 27.5 -
r	= verhouding b/a	= 1.00 -
s	= factor voor de vorm van de voet	= 1.00 -

Voor een uitgebreide beschrijving van het bepalen van de gemiddelde conusweerstanden in de gebieden I, II en III wordt verwezen naar art. 7.6.2.3 (e) in de norm.

De maximale draagkracht van de paalpunt volgens art. 7.6.2.3 (c) bedraagt:

$$R_{b;cal;max;i} = A_b * q_{b;max;i}$$
$$= 686 \text{ kN}$$

waarin :		in dit geval :
A_b	= oppervlak van de paalvoet	= 0.0625 m^2

Maximale paalschachtwrijving

De maximale paalschachtwrijving volgens art. 7.6.2.3 (i) bedraagt:

$$q_{s;max;z} = \alpha_s * q_{c;z;a}$$

De maximale schachtwrijvingskracht volgens art. 7.6.2.3 (c) bedraagt:

$$R_{s;cal;max;i} = O_{s;\Delta l;gem} * \Sigma\, q_{s;max;z;i} * d_z$$
$$= 536 \text{ kN}$$

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Per laag

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: N.A.P.

Nr	Laag	Nivo [m]	$O_{s;gem}$ [m¹]	α_s	Perc. [%]	$q_{c;z;a}$ [MPa]	$q_{s;max}$ [MPa]	d_z [m]	$R_{c;cal}$ [kN]
--	----	-6.50	--	--	--	--	--	--	--
1	Zand - Zwak siltig - Kleiig	-7.25	1.00	0.0100	100	1.51	0.015	0.75	11.3
2	Klei - Schoon - Slap	-9.25	1.00	0.0200	100	0.49	0.010	2.00	19.8
3	Klei - Zwak zandig - Slap	-10.50	1.00	0.0267	100	2.54	0.068	1.25	84.8
4	Zand - Schoon - Matig	-11.00	1.00	0.0100	100	11.94	0.119	0.50	59.7
5	Leem - Sterk zandig	-17.25	1.00	0.0250	100	2.30	0.057	6.25	358.6
totaal			1.00	0.0209		2.38	0.050	10.75	535.8

Maximale draagkracht

De maximale draagkracht van de paal volgens art. 7.6.2.3 (c) bedraagt:

$R_{c;cal;i} = R_{b;cal;max;i} + R_{s;cal;max;i}$
$= 1222$ kN (= 686 + 536)

De karakteristieke waarde van de maximale draagkracht van de paal volgens art. 7.6.2.3 (b) bedraagt:

$R_{c;k} = R_{c;cal} / \xi_{3\ (n=1)}$
$= 879$ kN

waarin : in dit geval :

$\xi_{3\ (n=1)}$ = factor volgens art. A.3.3.3 bij 1 sondering = 1.39 -

Voor de rekenwaarde van de maximale draagkracht van de paal kan volgens art. 2.4.7.3.3 worden aangehouden :

$R_{c;d} = R_{c;k} / \gamma_R$
$= 733$ kN

waarin : in dit geval :

γ_R = partiële weerstandsfactor volgens art. A.3.3.2 tabel A.6, A.7 of A.8 = 1.20 -

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DETAIL BER. NEGATIEVE KLEEF 250x250D; DKM001; N.A.P.-17.25

Uitgangspunten

- gehanteerde sondering : DKM001
- gehanteerde paal : 250x250
- paalpuntniveau : N.A.P.-17.25 m
- paalkopniveau : N.A.P. -0.50 m
- traject negatieve kleef : N.A.P. -0.10 m tot : N.A.P. -6.50 m
- $p_{sur;k}$: 6.80 kN/m^2

Berekening negatieve kleef

De karakteristieke waarde van de maximale negatieve kleefbelasting v.e. alleenstaande paal volgens art. 7.3.2.2 (d) bedraagt:

$F_{nk;k}$ = $O_{s;gem} * \sum d_j * K_{0;j;k} * \tan \delta_{j;k} * (\sigma'_{v;j-1;k} + \sigma'_{v;j;k})/2.0$
= -52.3 kN

waarin :

$O_{s;gem}$ = omtrek van de dwarsdoorsnede van de paalschacht
d_j = de dikte van de grondlaag i
$K_{0;j;k}$ = de karakteristieke waarde van de neutrale gronddrukfactor in laag i
$\delta_{j;k}$ = de karakteristieke waarde van de wrijvingshoek
$\sigma'_{v;j;k}$ = de karakteristieke waarde van de effectieve verticale spanning onder in laag j

Per laag

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: N.A.P.

Nr	Laag	Nivo [m]	Hoogte [m]	$O_{s;gem}$ [m^1]	$K_{0;j} * \tan(\delta_i)$	$\sigma'_{v;j;k}$ [kN/m^2]
--	----	-0.50	--	--	--	6.80
1	Klei - Schoon - Slap	-1.10	0.60	1.00	0.25	17.00
2	Klei - Schoon - Slap	-4.50	3.40	1.00	0.25	40.80
3	Zand - Zwak siltig - Kleiig	-6.50	2.00	1.00	0.25	62.80

Rekenwaarde

De rekenwaarde van de maximale negatieve kleefbelasting van een alleenstaande paal bedraagt :

$F_{nk;d}$ = $F_{nk;k} * \gamma_{f;nk}$ = -52.3 kN

waarin : in dit geval :

$\gamma_{f;nk}$ = belastingfactor voor de negatieve kleef (art. 7.3.2.2 (b)) 1.0 -

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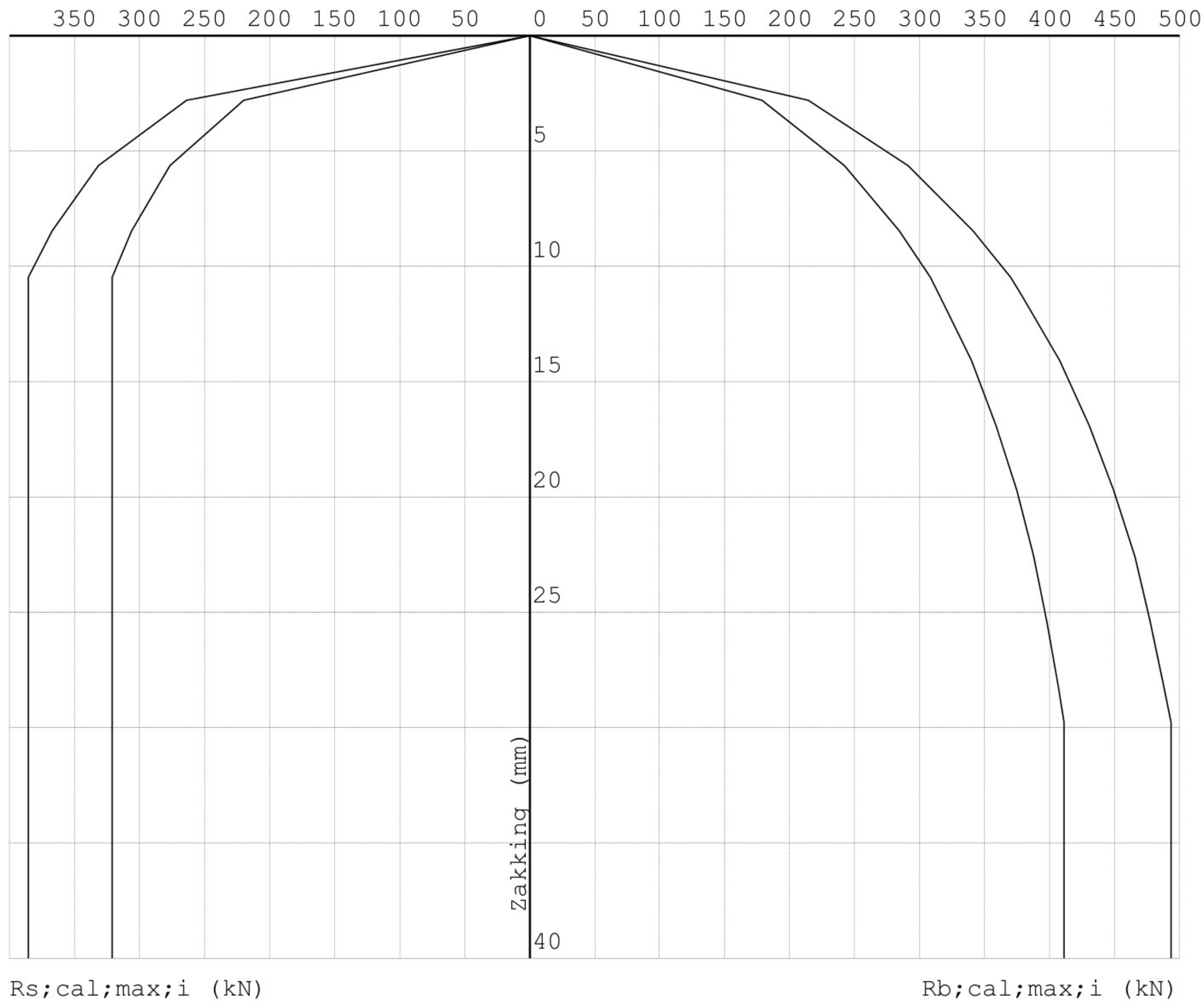
LAST_ZAKKINGSDIAGRAM 250x250D

Uitgangspunten

- gehanteerde sondering : DKM001
- gehanteerde paal : 250x250
- paalpuntniveau : N.A.P.-17.25 m

Last-zakkingsdiagram grenstoestand 1B en 2



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Toelichting grondslagen

In dit document kunt u secties vinden die onleesbaar zijn gemaakt. Deze informatie is achterwege gelaten op basis van de Wet open overheid (Woo). De letter die hierbij is vermeld correspondeert met de bijbehorende grondslag in onderstaand overzicht.

J Art. 5.1 lid 2 sub e

Het belang van de openbaarmaking van deze informatie weegt niet op tegen het belang van de eerbiediging van de persoonlijke levenssfeer van betrokkenen