Subject : Recommendations for the foundation of 2 formaline storage tanks

at

ROTTERDAM

Commissioned by : Hexion Pernis Rotterdam

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1. INTRODUCTION

Hexion Pernis Rotterdam commissioned Mos Grondmechanica B.V. to carry out a soil investigation and based on the results, provide recommendations regarding the foundation of 2 formaline storage tanks (pile foundation) and the tank-pit slab (shallow foundation) at a plant location in Botlek, Rotterdam.

In Mos Grondmechanica report R1901679-01 (date of issue September 17th 2019) the results of the soil investigation have been presented.

Report R1901679-02 (date of issue September 23rd 2019) contains the initial foundation advise.

This report contains recommendations regarding the foundation of 2 formaline storage tanks (deep foundation) and the tank-pit slab (shallow foundation) with some comments of the structural engineer integrated.

Bilfinder Tebodin Netherlands B.V. is the structural engineer involved in this project.

At the request of the structural engineer, the foundation of the 2 formaline storage tanks is based on Fundex piles.

2. PROJECT INFORMATION

Hexion Pernis Rotterdam intends to build new formaline storage tanks at a plant location in Botlek. This facility consists mainly of two storage tanks, steel structure for supporting pipes and miscellaneous steel structures within a tank-pit structure. This report contains recommendations regarding the foundation of the two formaline storage tanks.

Provided relevant documents for the above mentioned project:

- Bilfinger Tebodin document ''Application specification for foundation advice'' regarding project: Formaline storage Bund. Order number: 53591.01, document number: 1315001, date 06 September 2019;
- 3D later scanning services drawings: Job no. 90001. Drawing nos.:
  - 308-CL-FA-Bund600-6000, Issue 03. Date drawn: 10.7.2019. Description: Formalin storage bund plan view Existing and Design;
  - 308-CL-FA-Bund600-6001, Issue 03. Date drawn: 10.7.2019. Description: Formalin storage bund plan view Existing tanks;
  - 308-CL-FA-Bund600-6002, Issue 04. Date drawn: 10.7.2019. Description: Formalin storage bund plan view Design.

From this document and additional information from the structural engineer, the following project information is derived:

- Elevation of the tank foundation slab is approximately = NAP +4.8 m. There will not be any significant excavations or site risings;
- Requested foundation type: Vibration free, screwed displacement piles, type Fundex or equivalent;
- Storage tanks dimensions and loads:
  - Foundation slab is octagonal, width approximately 10 m and thickness 0.75 m;
  - Tank diameter = 9.5 m;
  - Tank height = 10.0 m;
  - Tank weight empty = 500 kN;
  - Tank weight operationg = 9328 kN (empty weight + content).
- Maximum occurring pile reaction:

	ULS (design value):	SLS (Characteristic):
Compression	$F_{v;d}$ = 550 kN	$F_{v;k}$ = 350 kN;
Tension	$F_{t;d}$ = N/A;	$F_{t;k}$ = N/A;
Horizontal	$F_{h;d}$ = 20 kN	$F_{h;k}$ = 10 kN;

Based on the derived information the above mentioned structures have been classified into the Geotechnical Category 2.

3. SOIL INVESTIGATION

3.1 Executed soil investigation

On September 4th 2019 Mos Grondmechanica executed the CPT's numbers 1 to 6 to a depth of approximately ground level –35 m (maximum NAP -30.3 m). During the execution of the CPT's, the time, cone resistance (q_c), sleeve friction (f_s) and the angle (˚) of the penetration path with respect to the vertical (i) are measured per 20 mm penetration and saved. Using the measured cone resistance and sleeve friction, the friction ratio ($R_f = f_s / q_c * 100\%$) is calculated. The friction ratio (R_f) in combination with the cone resistance (q_c) provides insight in the soil types present.

The results of the above mentioned executed soil investigation are presented in Mos Grondmechanica report R1901679-01, date of issue September 17th 2019.

3.2 Geotechnical soil profile

The ground level at the CPT locations varies from NAP +4.78 m to NAP +4.62 m.

Based on the CPT data the following geotechnical profile is derived:

- From ground level to NAP -3.4 m down to NAP -5.1 a heterogeneous stratum is encountered consisting of mainly sand intersected by a clay layer of varying thickness. In the top of this layered stratum no cone resistance was measured, due to pre-drilling to detect cables and pipes. The pre-drilling is executed from ground level to approximately 2 m below surface. In the layers cone resistances (q_c) were registered ranging from 0.5 MPa to 2 MPa in clay and up to 3 MPa to 14 MPa in sand;
- From NAP -3.4 m down to NAP -5.1 m down to about NAP -10.5 m to NAP -12.5 m a layered and heterogeneous stratum is encountered consisting of sand and silty sand intersected at arbitrary depths by clay layers of varying thickness. In this stratum the cone resistance is registered ranging from 4 MPa to 16 MPa in sand and around 2 MPa in clay;
- From NAP -10.5 m to NAP -12.5 m to the maximum penetrated depth of NAP -30.5 m a sand layer is encountered, locally the sand is silty. In this layer the cone resistance is registered ranging from 3 MPa to 30 MPa, and higher, values of the cone resistance as low as 2 MPa to 3 MPa are caused by less compacted sand layers or thin clay layers.

The executed soil investigation gives no information on the ground water level.

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4. PILE FOUNDATION ADVICE ''SCREWED DISPLACEMENT PILES

4.1 Choice of the pile type

Given the project data and the structure and the composition of the subsoil, we can agree from a geotechnical point of view with a choice for a foundation on cast-in-place screw piles namely the Fundex pile or equivalent.

The calculations of the design values of the maximum pile resistance are executed for the Fundex piles or equivalent. These calculations are based on the Dutch geotechnical standard NEN 9997-1:2017 “Geotechnical design of structures – Part 1: General rules”.

4.2 Pile tip levels with maximum pile tip resistances and pile shaft frictions

In Table 4-1 for each cone penetration test for Fundex piles or equivalent the recommended pile tip level(s) for the needed pile resistance is given with the corresponding negative skin friction ($F_{nk;rep;i}$), the maximum pile tip resistance ($q_{b;max;i}$) and the maximum pile shaft friction ($q_{s;cal;maxi;i}$).

In the future settlements can occur in the compressible layers of the subsoil. These settlements cause negative skin friction along the foundation piles. For the calculation of the negative skin friction the groundwater level has been adopted at a level of NAP +0.50 m. The negative skin friction is calculated from ground level to a depth of NAP –3.4 m down to NAP –5.1 m.

The maximum pile shaft friction is calculated with the percentage method. For compression over a range of the upper side of the bearing sand layers at NAP –3.4 m down to NAP –5.1 m to the recommended pile tip level. Hereby for Fundex piles a pile dependant factor α_s = 0.009 is applied for compression.

The maximum pile tip resistances for Fundex piles are calculated with a pile class factor α_p = 0.63. For the other pile dependant factors apply β = s = 1.0.

Table 4-1 Recommended pile tip levels with maximum pile frictions and pile tip resistances

			Fundex piles or equivalent		
CPT No.	Ground level [NAP + m]	Pile tip level [NAP + m]	$F_{nk;rep;i}$ [kN/m]	$q_{s;cal;max;i}$ [kN/m] Compression	$q_{b;max;i}$ 1) [MPa]
1	4.75	-17.5	170	820	2.5
2	4.77	-17.5	150	830	2.4
3	4.62	-17.5*	130	830	2.4
4	4.72	-17.5	145	770	3.8
5	4.78	-17.5	200	785	3.9
6	4.74	-17.5	140	910	3.0

$F_{nk;rep;i}$ is the representative value of the negative skin friction at CPT i, per meter pile circumference;

$q_{s;cal;max;i}$ is the representative value of maximum pile shaft friction force at CPT i, per meter pile circumference;

$q_{b;max;i}$ is de maximum pile tip resistance at CPT i;

1) these values are valid for piles, Diameter Ø 380 mm / 450 mm;

2) these values refer to the maximum pile shaft friction for a single tensile pile, thus excluding any reductions due to group effects;

( ) the pile tip levels between brackets serve to make transitions in pile tip levels possible;

* strictly maintain this level and certainly do not install deeper, because of a quickly diminishing pile tip resistance or pile *compression* resistance !

4.3 Design values of net pile *compression* resistance

With the previously given values of the negative skin friction, maximum pile shaft friction and maximum pile tip resistance for precast concrete piles the design values for the net pile compression resistance and the net pile tensile resistance are calculated. Thereby, in accordance with NEN 9997-1, the following factors are applied; ξ = 1,30 (3 relevant CPT's; non-rigid structure), γ_t (= γ_b = $\gamma_{s;c}$) = 1.20 and $\gamma_{f;nk}$ = 1.00.

This gives the following design values for the net pile *compression* resistance (Table 4-2):

Table 4-2 Design values for the net pile compression resistance

Fundex piles or equivalent		
Pile dimensions [$Ø_S$ / $Ø_B$ mm/mm]	Pile tip level [NAP + m]	$R_{c;net;d}$ [kN]
380/ 450	-17.5	680
460/ 560	-17.5	885
540/ 660	-17.5	1075

$R_{c;net;d}$ is the design value of the net pile *compression* resistance, *after deduction of the negative skin friction.*

The mentioned design values of the net pile *compression* resistance ($R_{c;net;d}$) relate to the design values of the maximum pile *compression* resistance that can be derived from the soil by the pile at pile head level. The structural strength must be assessed by the structural engineer.

Calculation examples are included in annex A.

4.4 Pile head settlements and axial pile spring stiffnesses

The expected maximum pile head settlements of *compression* piles in the serviceability state (under the maximum characteristic pile *compression* loads) are approximately 5 to 10 mm. Dependent upon the soil profile the maximum differential settlements may reach up to approximately 5 mm, assuming practically equal pile compression loads.

The real occurring settlements and differential settlements of compression piles (and risings and differential risings in case of tensile piles) are among others dependent of the actual location, the actual applied pile dimension(s) and the actual pile loads occurring.

For long present (permanent) loads a characteristic vertical pile spring stiffness can be maintained in the size of:

- $k_{v;long;k}$ = 80 [MN/m] for soil displacement screw piles Ø 380 mm/ 450 mm
- $k_{v;long;k}$ = 101 [MN/m] for soil displacement screw piles Ø 450 mm/ 560 mm
- $k_{v;long;k}$ = 125 [MN/m] for soil displacement screw piles Ø 540 mm/ 660 mm

This value is determined for a relatively poorer load-bearing soil profile.

Because of uncertainties, including variations in the foundation soil profile, we recommend to take into account high and low values of the pile spring stiffnesses that can be determined from the said spring stiffnesses by multiplying by an uncertainty factor of 1.5, respectively, division by an uncertainty factor of 1.35.

The pile spring stiffnesses in the ultimate state are obtained by division of said spring stiffnesses by a factor of 1.3.

A calculation example of the characteristic load-settlement behaviour with the corresponding load-settlement diagram (for the serviceability state) is included in annex B.

4.5 The behaviour of laterally loaded piles

At pile head level the piles will not only vertically but also laterally (horizontally) be loaded. Due to lateral load(s) the pile will be subjected to bending and the pile (head) will displace.

For the calculation of laterally loaded piles no calculation method (proposed by Dutch geotechnical standard) is available yet. Therefore these calculations are carried out in the "traditional way" with characteristic (representative) values; note that this implies that the results of these calculations therefore also concern characteristic (representative) values.

For the behaviour of the laterally loaded piles characteristic (≈ representative) lateral pile loads at pile head level are given at the size of 10 kN.

The behaviour of laterally loaded piles is calculated with the single pile module of the D-SheetPiling program (version 18.1).

This is based on the following pile depending factors and assumptions:

- De bending stiffness $EI_{pile} = {}^{2}/_{3} \times E_{concrete}I_{concrete;uncracked}$
- E_{pile} = 25 GPa for Cast-in-place soil displacement screw piles.
- For the acting pile width is calculated with $d_{eq\ (shaft)}$.
- The pile head level is located at a level equal to (future) ground level NAP + 4.39 m.
- For the connection between the piles with the overhead structure it is assumed that the pile head translation is unhindered and the pile head rotation is prevented ("fixed") or unhindered ("hinged"); both situations are calculated.

- For the pre dug surface layer, sand with a cone resistance of 2.5 MPa is assumed.
- The calculations are carried out for single piles with a characteristic (≈ representative) horizontal loads at pile head level at the size of 10 kN/pile.

lculation results of the behaviour of single, laterally loaded Cast-in-place soil displacement screw piles are tabular summarized in:

- Table 4-3 for piles with a pile head that can translate unhindered and whose rotation is prevented ("fixed").
- Table 4-4 for piles with a pile head that can translate unhindered and whose rotation is unhindered ("hinged").

*Table 4-3 Calculation results of single laterally loaded Cast-in-place soil displacement screw piles with a pile head that can translate unhindered and whose rotation is prevented ("fixed") **

Cast-in-place soil displacement screw piles [$Ø_S$ / $Ø_B$ mm/mm]	Characteristic lateral pile head load [kN]	"fixed" pile head; translation is unhindered			
		$M_{pile\ head;max}$ * [kNm]	$M_{field;max}$ * [kNm]	Level of $M_{field;max}$ [NAP + m]	$u_{pile\ head;max}$ [mm]
380/ 450	10	9.9	2.3	1.8	0.9
460/ 560	10	11.3	2.6	1.5	0.7
540/ 660	10	13.0	2.9	1.2	0.5

* NOTE: the values shown her are characteristic values.

The allowed ability of the deformations and the structural strength (also taking into account the normal force) is to be assessed by the structural engineer separately.

*Table 4-4 Calculation results of single laterally loaded precast concrete piles with a pile head that can translate unhindered and whose rotation is unhindered ("hinged") **

Cast-in-place soil displacement screw piles [$Ø_S$ / $Ø_B$ mm/mm]	Characteristic lateral pile head load [kN]	"hinged" pile head; translation is unhindered			
		$M_{field;max}$ * [kNm]	Level of $M_{field;max}$ [NAP + m]	$\theta_{pile\ head}$ [10^{-3} rad]	$u_{max;pile\ head}$ [mm]
380/ 450	10	6.8	3.0	1.1	2.0
460/ 560	10	7.7	2.6	0.7	1.5
540/ 660	10	9.2	2.3	0.5	1.1

* NOTE: the values shown her are characteristic values.

The allow ability of the deformations and the structural strength (also taking into account the normal force) is to be assessed by the structural engineer separately.

Calculation examples are included in annex C.

The characteristic spring stiffnesses for the behaviour of piles under lateral pile head loads can be determined from $k_{u;Fh;k} = F_h / u_{max;pile\ head}$ and from $k_{\theta;Fh;k} = F_h / \theta_{pile\ head}$. The high and the low values of the spring stiffnesses can be determined by multiplication, respectively division of these characteristic spring stiffnesses by an uncertainty factor of 1.5.

Note that piles in a pile group will per pile show a less stiff lateral pile behaviour than single piles. The extent of this depends upon the pile group configuration (this concerns the number of piles in the pile group as well as the pile dimensions and the mutual centre to centre distance between the piles). The determination of the pile group effect is not included in the agreed scope of work. Therefore no pile group effect has been investigated for the behaviour of horizontally loaded piles.

4.6 Execution

For general directives for the execution of pile driving we refer to annex E (soil displacement screw piles).

Due to the fact that the intention is to install the piles narrower than 4 times D_{eq} the routing of the piles is essential. Piles can be closer than 4 times D_{eq} centre to centre however these cast in situ piles cannot be installed directly after on another (see annex E General execution directives of Vibration free and soil displacement screw piles).

5. SHALLOW FOUNDATION ADVICE CONCRETE REINFORCED FLOOR AND BUNDWALL

5.1 Selection foundation type

Around the piled tank foundation a reinforced concrete slab is created. Given the project data and the structure and expected composition of the underground a shallow foundation is well possible.

5.2 Concrete reinforced floor and incorporated bundwall

The concrete (reinforced) slab with a thickness of 0,25 m is positioned on several elevations. The total slab dimensions are approximately 26 x 18 m². The edge of the slab around the outside of the slab has a wall of 1,0 m. The foundation layers need to consist of non cohesive materials, due to predigging of the test locations the top soil is not exactly known. So it is relevant to perform tests to investigate the top soil by hand CPT's for example (see annex F).

The calculation of the maximum resistance (resistance force) of the foundation is based on the (Dutch) geotechnical standard NEN 9997-1: 2017. The calculation of the design values of the maximum vertical resistance of shallow foundations with a horizontal foundation surface is based on article 6.5.2.2 of NEN 9997-1: 2017. See annex D for the effective resistance for several values of the effective width.

The effective bearing resistance of the concrete slab of the given dimensions is in principle not normative. The failure circles cannot arise due to the large surface area and the reinforcement. The real bearing resistance depends on the stiffness of the concrete pavement (floor).

5.3 Modulus of subgrade reaction

For the calculation of (a) on a sand bed shallow founded concrete floor(s) or foundation slabs, provided well executed soil improvement(s), a static modulus of subgrade reaction of 7.500 kN/m^3 to 10.000 kN/m^3 can be adopted for the given loads.

5.4 Execution

For general guidelines for the execution of excavations and soil improvements for shallow foundations we refer to annex F.

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Rotterdam, June 8th 2020

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Annex A
Example of calculation vertically loaded Fundex piles

Berekening paaldrukweerstand Sond. 1

Conform NEN 9997-1 (α_p-factoren per 1-1-2017)

Fundexpaal

Opdracht: 1901679 — Printdatum: 18-9-2019

Project: Storage tank capacity te Rotterdam — Versie 6.0.1.0

Maaiveld hoogte:	NAP + 4,75 m	Omschrijving:
Grondwaterstand:	NAP + 0,50 m	
Putbodem:		
Afmetingen ontgraving:		
Terreinbelasting:	0 kN/m²	
Percentages schachtwrijving:	100,0 % vanaf NAP -4,06 m;	

Geotechnisch profiel						Terreinspanningen		F_{pos}	F_{neg}
laag nr.	ok. laag [NAP +m]	γ [kN/m³]	$q_{c,gem}$ [MPa]	$K_0 \tan\delta$	grondsoort	$\sigma_{v;z;i;gem}$ [kN/m²]	$\sigma_{v;z;i;ontgr}$ [kN/m²]	$q_{s;cal;max;i}$ [kN/m]	$F_{nk;rep}$ [kN/m]
1	2,74	17,5	2,0	0,30	Voorboren/ Klinker etc	17,59	17,59	0	11
2	2,19	19,8	7,2	0,30	Zand	40,61	40,61	0	17
3	2,09	18,0	1,3	0,27	Leem	46,94	46,94	0	18
4	0,50	19,4	5,3	0,29	Zand	63,22	63,22	0	48
5	-0,60	19,4	3,7	0,28	Zand	83,75	83,75	0	74
6	-1,00	16,5	1,1	0,25	Klei	90,18	90,18	0	83
7	-1,78	17,2	0,8	0,25	Leem	94,26	94,26	0	102
8	-1,94	18,1	1,6	0,25	Leem	97,70	97,70	0	106
9	-2,06	18,0	1,7	0,25	Leem	98,83	98,83	0	109
10	-2,94	19,1	3,6	0,28	Zand	103,29	103,29	0	134
11	-3,12	18,2	2,4	0,26	Leem	108,01	108,01	0	139
12	-3,50	19,2	3,8	0,28	Zand	110,49	110,49	0	151
13	-3,74	17,0	2,3	0,26	Klei	113,07	113,07	0	158
14	-3,92	17,7	2,0	0,25	Leem	114,60	114,60	0	163
15	-4,06	19,9	7,3	0,29	Zand	115,98	115,98	0	168
16	-5,71	19,9	7,9		Zand	124,84	124,84	118	
17	-5,87	17,6	2,6		Klei	133,62	133,62	122	
18	-6,01	18,9	3,2		Leem	134,85	134,85	126	
19	-7,54	19,9	8,6		Zand	143,01	143,01	241	
20	-7,62	19,0	3,3		Leem	150,92	150,92	244	
21	-7,78	19,6	6,1		Zand	152,05	152,05	253	
22	-7,92	18,0	3,5		Klei	153,38	153,38	257	
23	-8,04	18,8	3,5		Leem	154,47	154,47	260	
24	-8,20	17,4	2,6		Klei	155,59	155,59	264	
25	-8,88	20,0	8,0		Zand	159,57	159,57	314	
26	-9,02	18,4	4,5		Leem	163,55	163,55	319	
27	-9,38	19,9	7,4		Zand	165,93	165,93	343	
28	-9,75	17,5	3,0		Klei	169,11	169,11	353	
29	-9,87	18,7	3,4		Leem	171,02	171,02	357	
30	-9,99	20,0	8,2		Zand	172,14	172,14	366	
31	-10,31	18,8	4,2		Leem	174,14	174,14	377	
32	-10,43	16,7	2,1		Klei	175,94	175,94	380	
33	-10,81	18,6	3,6		Leem	177,97	177,97	392	
34	-10,93	17,5	2,3		Klei	180,05	180,05	395	
35	-11,07	19,9	6,5		Zand	181,19	181,19	403	
36	-11,42	18,9	4,6		Leem	183,45	183,45	417	
37	-11,62	19,9	8,2		Zand	186,00	186,00	432	
38	-11,80	19,0	5,4		Leem	187,80	187,80	441	
39	-12,36	19,9	6,7		Zand	191,39	191,39	474	
40	-12,50	19,0	3,6		Leem	194,80	194,80	479	
41	-13,47	20,0	10,1		Zand	200,26	200,26	566	
42	-13,61	19,1	5,8		Leem	205,73	205,73	573	

Berekening paaldrukweerstand Sond. 1

Conform NEN 9997-1 (α_p-factoren per 1-1-2017)

Fundexpaal

Opdracht: 1901679 — Printdatum: 18-9-2019
Project: Storage tank capacity te Rotterdam — Versie 6.0.1.0

Maaiveld hoogte: NAP + 4,75 m
Grondwaterstand: NAP + 0,50 m
Putbodem:
Afmetingen ontgraving:
Terreinbelasting: 0 kN/m^2
Percentages schachtwrijving: 100,0 % vanaf NAP -4,06 m;

Omschrijving:

Geotechnisch profiel						Terreinspanningen		F_{pos}	F_{neg}
laag nr.	ok. laag [NAP +m]	γ [kN/m^3]	$q_{c,gem}$ [MPa]	$K_o\tan\delta$	grondsoort	$\sigma_{v;z;i;gem}$ [kN/m^2]	$\sigma_{v;z;i;ontgr}$ [kN/m^2]	$q_{s;cal;max;i}$ [kN/m]	$F_{nk;rep}$ [kN/m]
43	-14,03	19,6	6,2		Zand	208,39	208,39	596	
44	-14,17	19,0	4,8		Leem	211,04	211,04	602	
45	-14,50	19,7	7,3		Zand	213,28	213,28	624	
46	-14,60	19,0	3,9		Leem	215,33	215,33	628	
47	-16,51	19,9	8,2		Zand	225,27	225,27	768	
48	-16,96	19,2	4,7		Leem	236,83	236,83	787	
49	-17,50	20,0	6,9		Zand	241,58	241,58	821	

Parameters	
α_s (in zand)	0,009
α_p	0,63
ξ_3	1,30
γ_t	1,20
$\gamma_{f,nk}$	1,00
OCR	1,0

$F_{nk,rep}$	168	kN/m
$q_{s;cal;max}$	821	kN/m

Rekenwaarde drukweerstand op een diepte van NAP - 17,50 m									
Buisdiam. [mm]	Puntdiam. [mm]	A_{punt} [mm^2]	O_s [mm]	β	$q_{b;max}$ [MPa]	$R_{b;cal;max}$ [kN]	$R_{s;cal;max}$ [kN]	$F_{nk;rep}$ [kN]	$R_{c;net;d}$ [kN]
380	450	159043	1194	1,00	2,48	394	980	200	681

Rekenvoorbeeld :

$q_{c;I;gem}$ = 5,16 MPa $q_{c;II;gem}$ = 4,09 MPa $q_{c;III;gem}$ = 3,24 MPa $q_{b;max}$ = 2,48 MPa

$R_{c;cal;max} = A_{punt}\, q_{b;max} + O_s\, q_{s;cal;max}$ = 394 + 980 = 1374 kN

$R_{c;d;net} = R_{c;cal;max} / (\xi_3 \cdot \gamma_t) - F_{nk;rep}\, \gamma_{f;nk}$ = 881 - 200 = 681 kN

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Annex B
Calculation examples characteristic load-settlement behaviour

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Order No.	1901679	Date 18-9-2019	ver 20170515

LOAD - SETTLEMENT BEHAVIOUR OF FOUNDATION PILES

Project: Hexion Botlek Rotterdam
Formaline tanks foundation slab
Fundex o.e.. 380/450 mm pile

PILE DATA

Pile type (S / A / B)		G	<= WRONG INPUT !	ξ	=	1,30	
Area (R / S / Re / W / O)		R	(round pile)	$\gamma_{f;nk}$	=	1,0	(state 1B)
Shaft diameter	=	380	[mm]	Pile length L	=	22,50	[m]
Pile tip diameter	=	450	[mm]	Length neg.sk. friction	=	9,00	[m]
				Length pos.sk. friction	=	13,50	[m]
Qb;max	=	2,50	[N/mm^2]	Young's modul: Epile	=	2,00E+07	[kN/m²]
Qs;cal;max	=	821	[kN/m]	Deq	=	450	[mm]
Fneg.sk;k	=	168	[kN/m]	Oshaft	=	1194	[mm]
Type of pile load		**C**	(static pile compression load)	Ashaft	=	113411	[mm^2]
				Abase	=	159043	[mm^2]

SERVICEABILITY STATE (2)

γ_t	=	1,00		Loads:			
				Fc;k	=	512	[kN]
Rb;max;k	=	306	[kN]	Fneg.sk;k	=	201	[kN]
Rs;max;k	=	754	[kN]	Fc;total;k	=	713	[kN]
Rc;k	=	1060	[kN]				
Rc;net;k	=	859	[kN]	Sb;k	=	4,02	[mm]
				Sel;k	=	4,92	[mm]
Rb;k	=	123	[kN]	S1;k	=	8,9	[mm]
Rs;k	=	589	[kN]	S2;k	=		[mm]
Rc;total;k	=	713	[kN]	Sk	=		[mm]

Vertical spring stiffness for long term load K1;k = 80 MN/m.

ULTIMATE LIMIT STATE (1B)

γ_t	=	1,20		Loads:			
				Fc;d	=	683	[kN]
Rb;max;d	=	255	[kN]	Fneg.sk;d	=	201	[kN]
Rs;max;d	=	628	[kN]	Fc;total;d	=	883	[kN]
Rc;d	=	883	[kN]				
Rc;net;d	=	683	[kN]	Sb;d	=	47,25	[mm]
				Sel;d	=	6,31	[mm]
Rb;d	=	255	[kN]	S1;d	=	53,6	[mm]
Rs;d	=	628	[kN]	S2;d	=		[mm]
Rc;total;d	=	883	[kN]	Sd	=		[mm]

Order No. : 1901679

Hexion Botlek Rotterdam

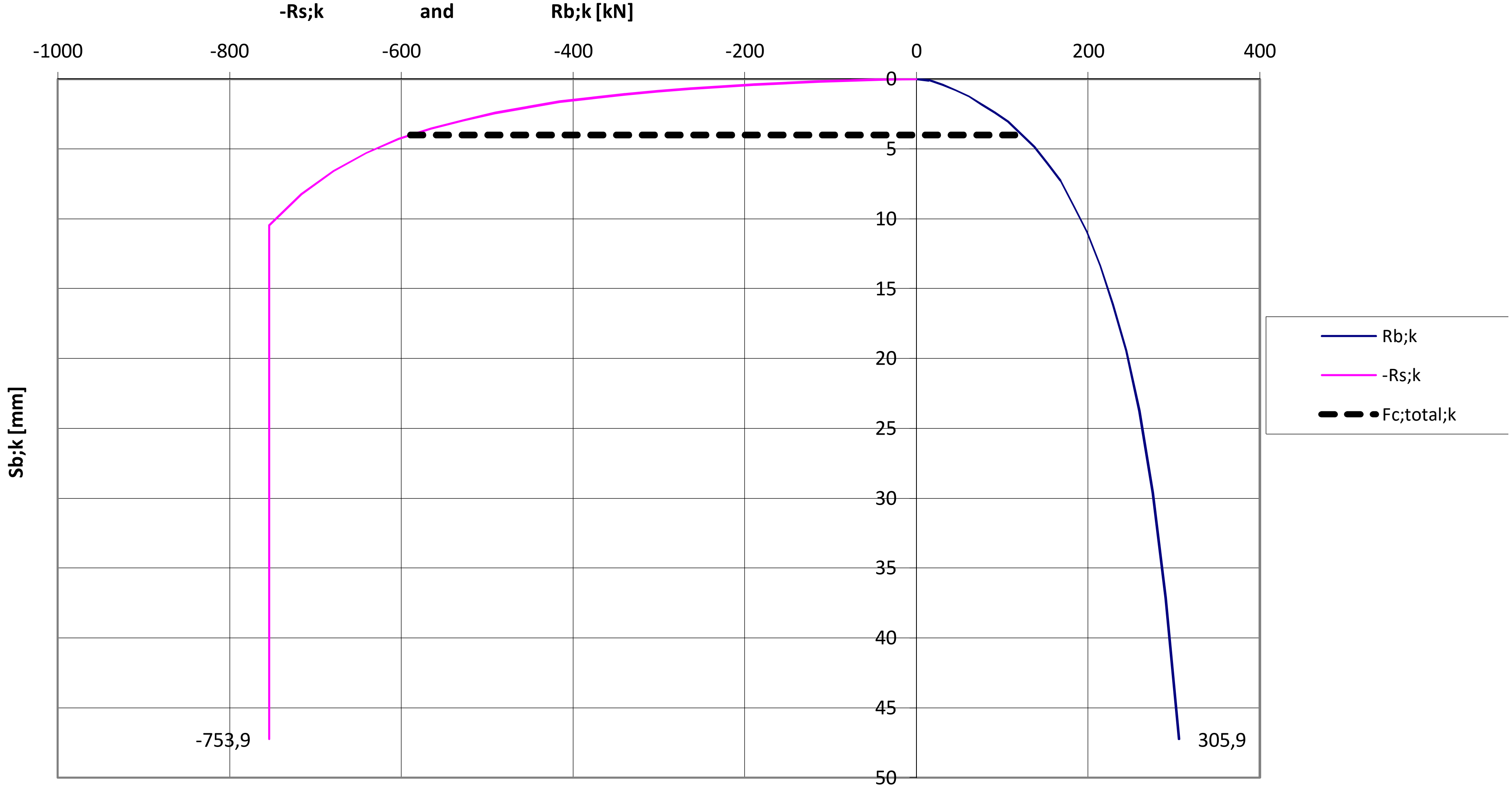
Formaline tanks foundation slab

Fundex o.e.. 380/450 mm pile

Load - setlement behaviour of foundation piles in the serviceability state (2)

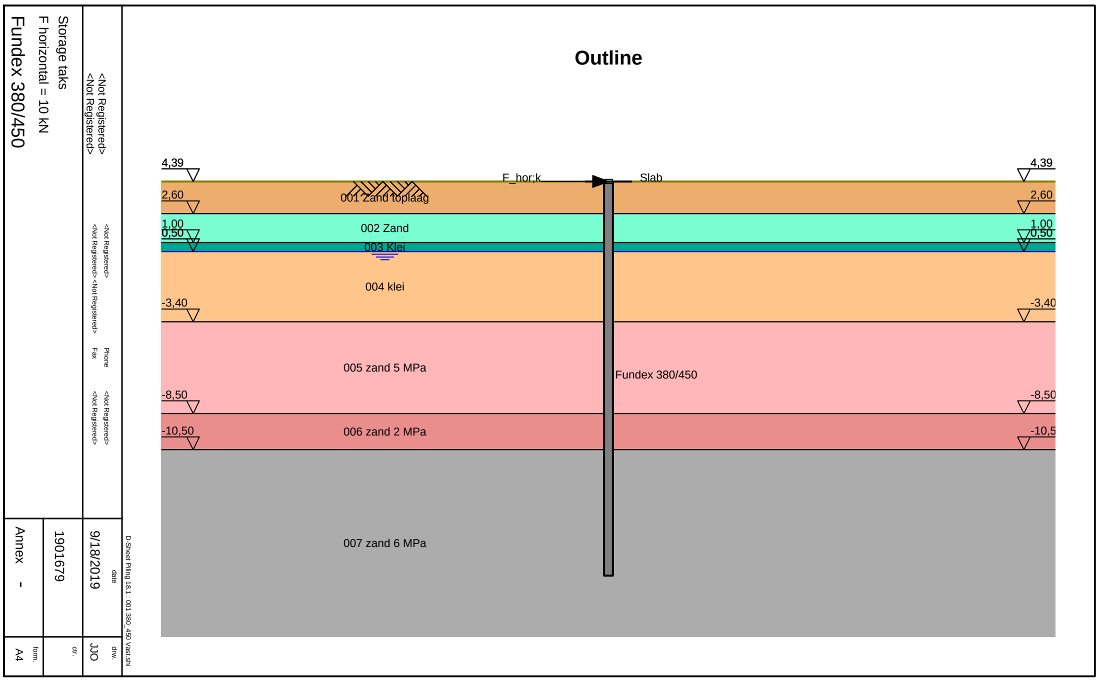
Fc;total;k	713	[kN]
Sb;k	4,02	[mm]

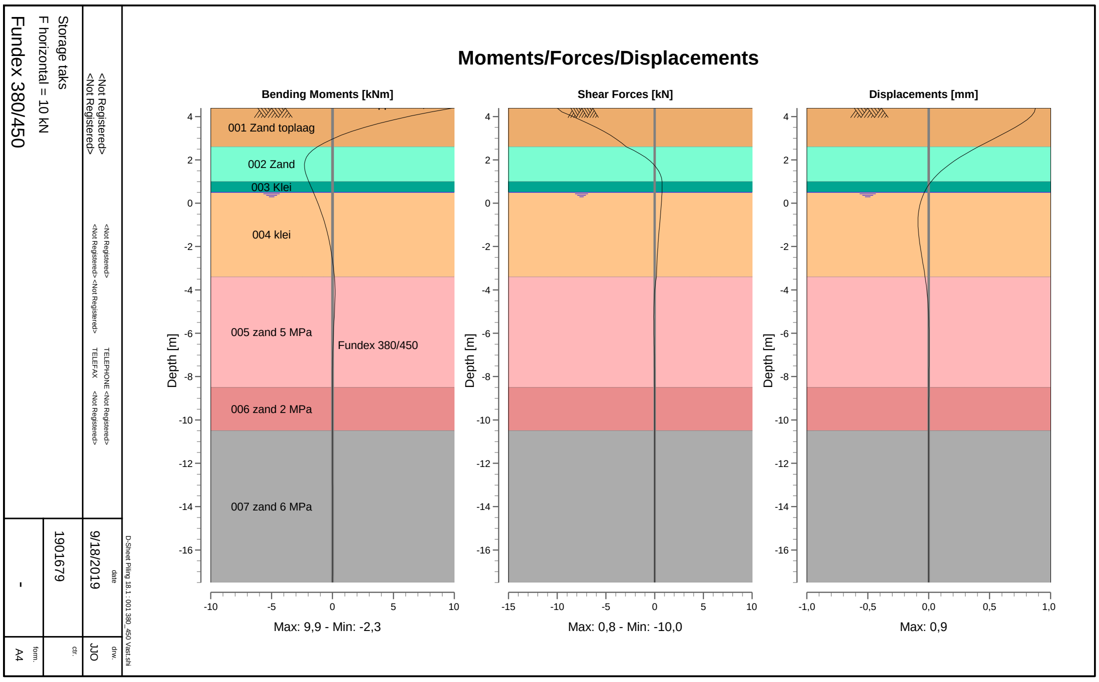
(static pile compression load)

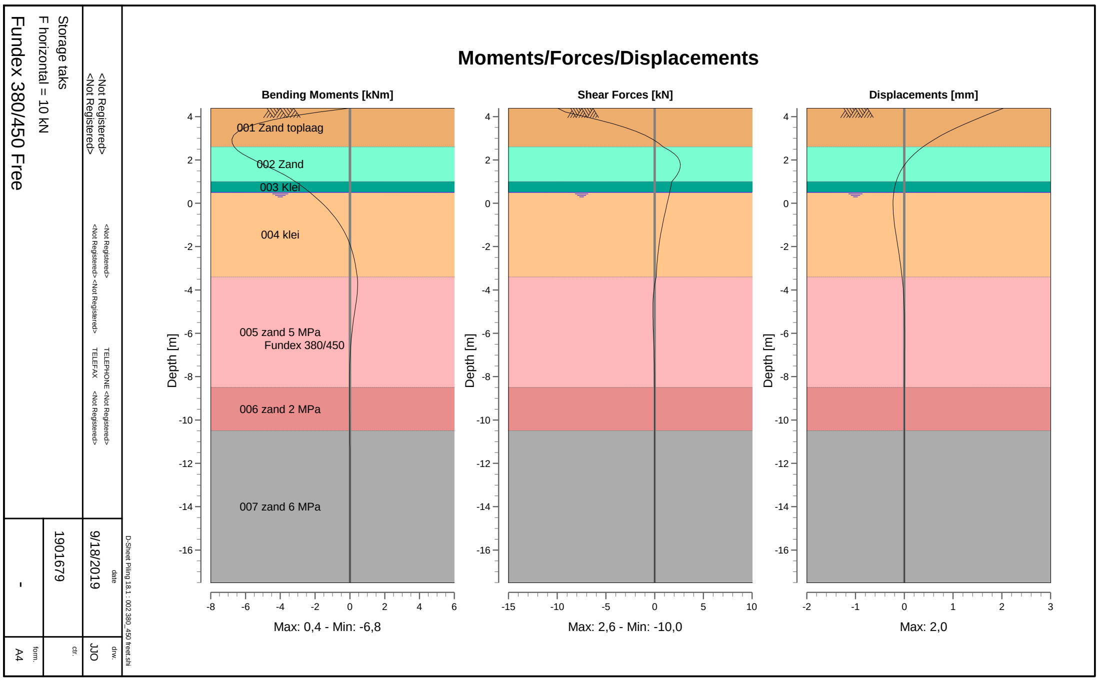


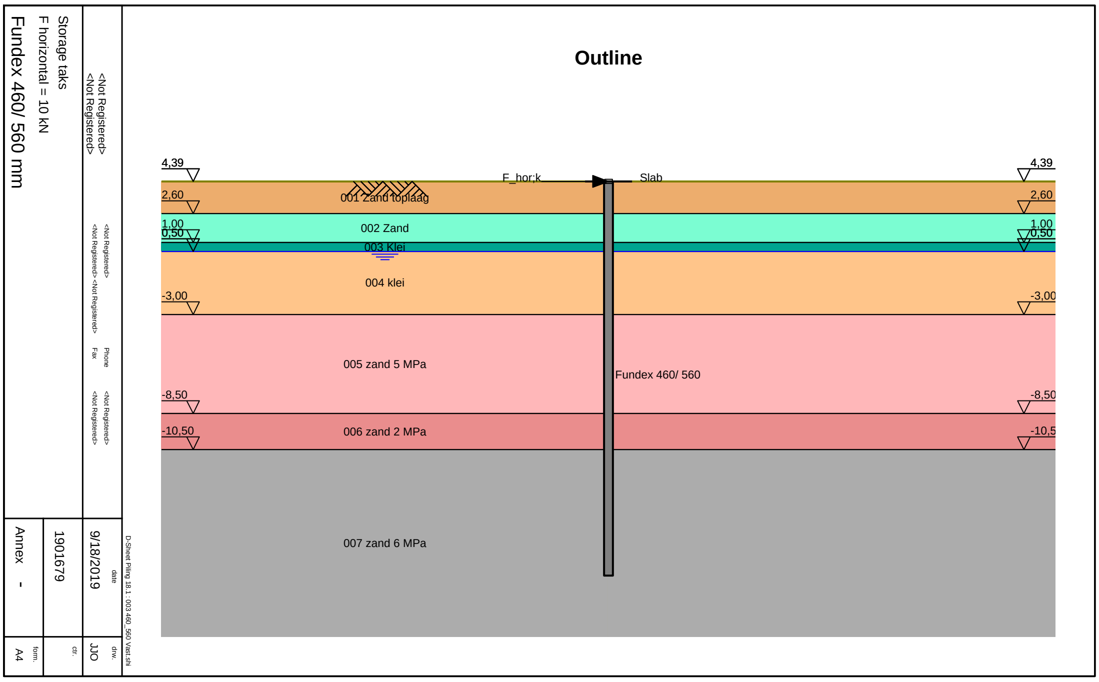
MOS GRONDMECHANICA

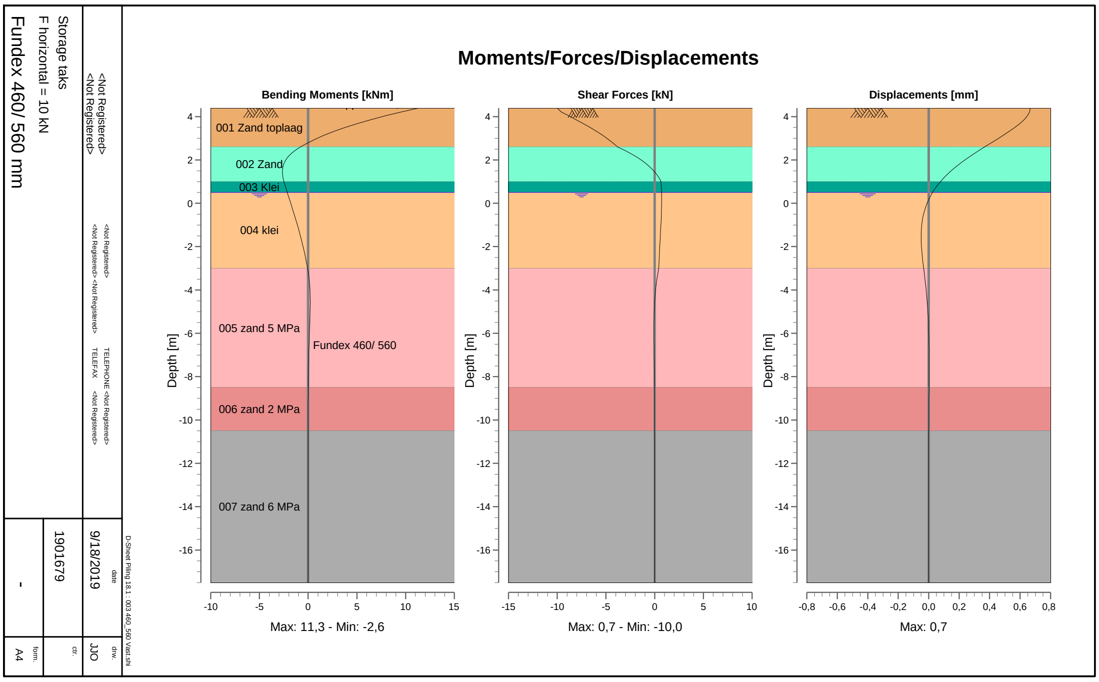
Annex C
Calculation examples horizontal pile load

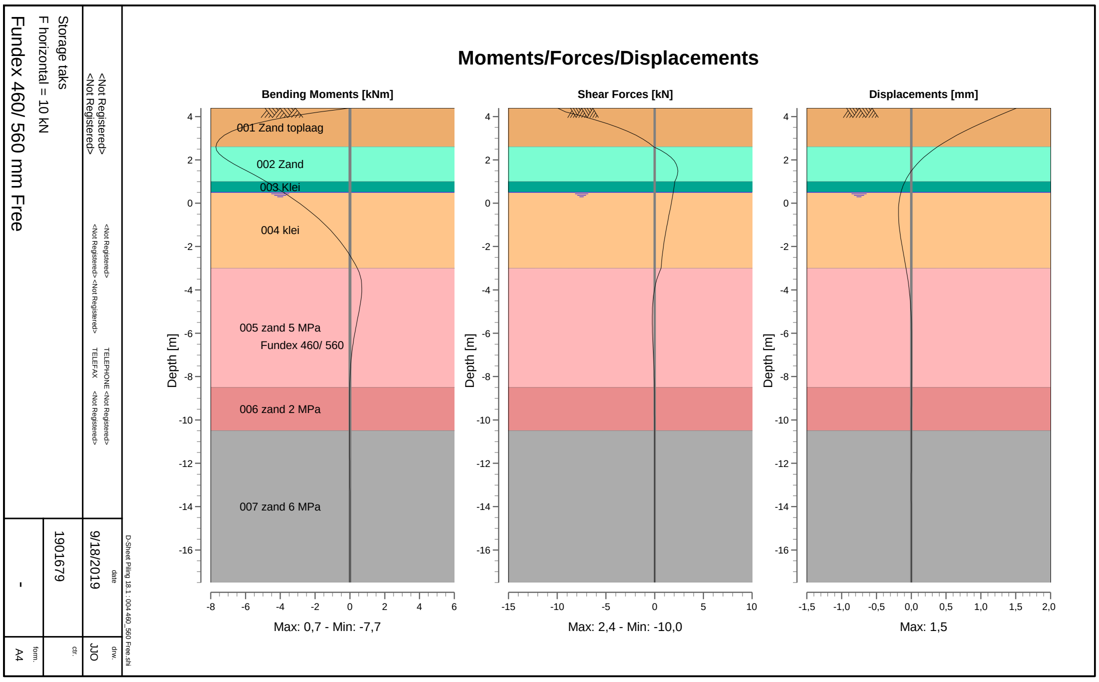

Outline
Fundex 380/450
F horizontal = 10 kN
Storage taks
Annex -
1901679
9/18/2019
JJO
A4
001 Zand toplaag
002 Zand
003 Klei
004 klei
005 zand 5 MPa
006 zand 2 MPa
007 zand 6 MPa
F_hor;k
Slab
Fundex 380/450
4,39
2,60
1,00
0,50
-3,40
-8,50
-10,50

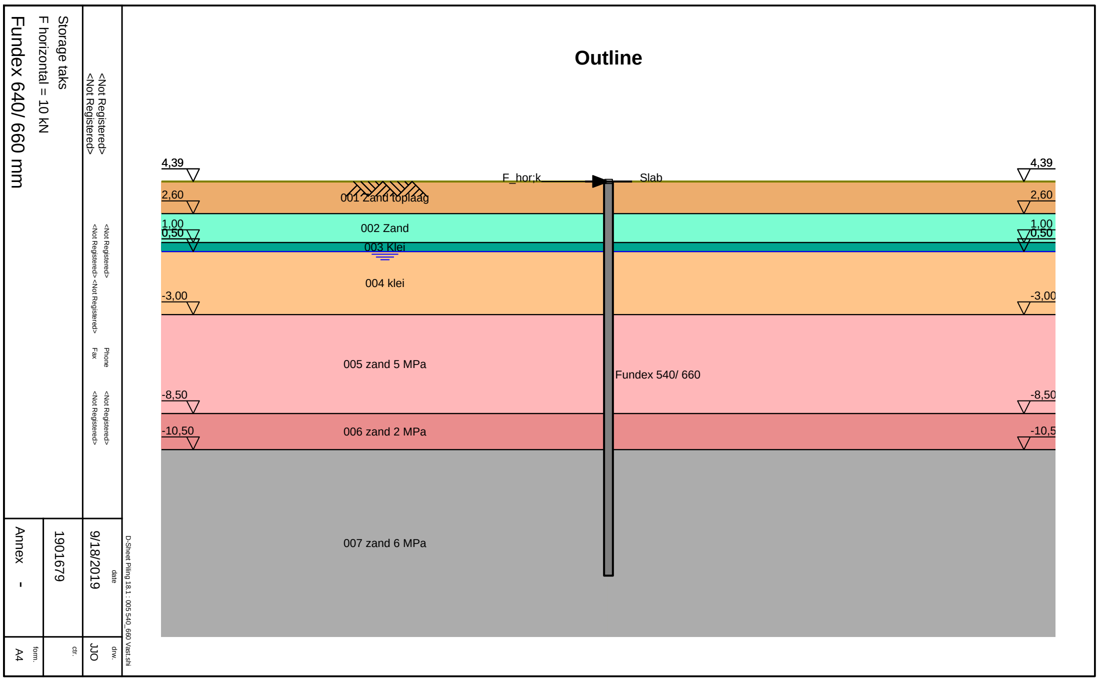

Moments/Forces/Displacements
Fundex 380/450
F horizontal = 10 kN
Storage taks
1901679
9/18/2019
JJO
A4
Bending Moments [kNm]
Shear Forces [kN]
Displacements [mm]
Depth [m]
001 Zand toplaag
002 Zand
003 Klei
004 klei
005 zand 5 MPa
006 zand 2 MPa
007 zand 6 MPa
Fundex 380/450
Max: 9,9 - Min: -2,3
Max: 0,8 - Min: -10,0
Max: 0,9

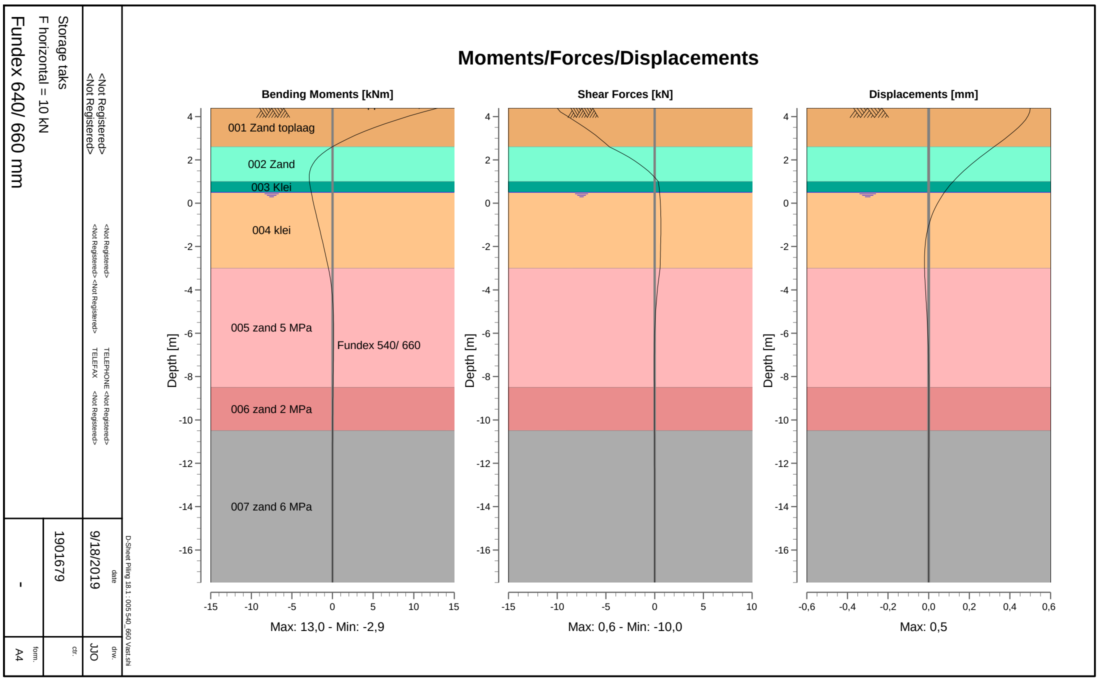

Moments/Forces/Displacements
Bending Moments [kNm]
Shear Forces [kN]
Displacements [mm]
Depth [m]
001 Zand toplaag
002 Zand
003 Klei
004 klei
005 zand 5 MPa
Fundex 380/450
006 zand 2 MPa
007 zand 6 MPa
Max: 0,4 - Min: -6,8
Max: 2,6 - Min: -10,0
Max: 2,0
Fundex 380/450 Free
F horizontal = 10 kN
Storage taks
1901679
9/18/2019
JJO
A4

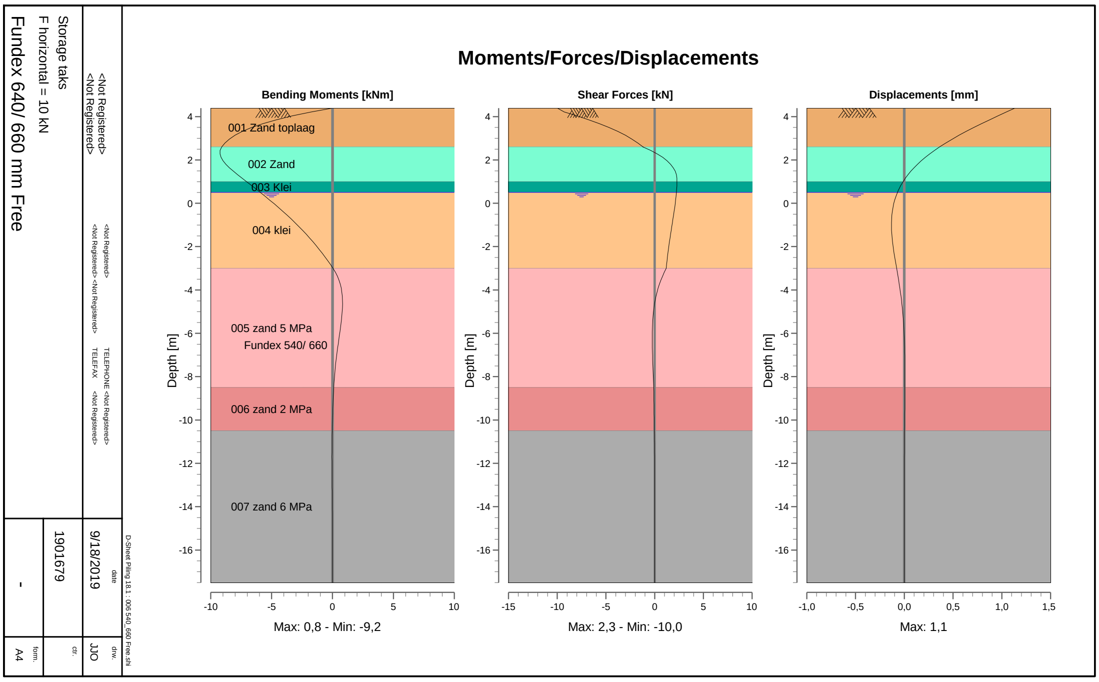

Outline
F_hor;k
Slab
4,39
2,60
1,00
0,50
-3,00
-8,50
-10,50
001 Zand toplaag
002 Zand
003 Klei
004 klei
005 zand 5 MPa
Fundex 460/ 560
006 zand 2 MPa
007 zand 6 MPa
Fundex 460/ 560 mm
F horizontal = 10 kN
Storage taks
Annex -
1901679
9/18/2019
JJO
A4


Moments/Forces/Displacements
Bending Moments [kNm]
Shear Forces [kN]
Displacements [mm]
Depth [m]
001 Zand toplaag
002 Zand
003 Klei
004 klei
005 zand 5 MPa
Fundex 460/ 560
006 zand 2 MPa
007 zand 6 MPa
Max: 11,3 - Min: -2,6
Max: 0,7 - Min: -10,0
Max: 0,7
Fundex 460/ 560 mm
F horizontal = 10 kN
Storage taks
1901679
9/18/2019
JJO
A4


Moments/Forces/Displacements
Bending Moments [kNm]
Shear Forces [kN]
Displacements [mm]
Depth [m]
001 Zand toplaag
002 Zand
003 Klei
004 klei
005 zand 5 MPa
Fundex 460/ 560
006 zand 2 MPa
007 zand 6 MPa
Max: 0,7 - Min: -7,7
Max: 2,4 - Min: -10,0
Max: 1,5
Fundex 460/ 560 mm Free
F horizontal = 10 kN
Storage taks
1901679
9/18/2019
JJO
A4


Outline
Fundex 640/ 660 mm
F horizontal = 10 kN
Storage taks
<Not Registered>
<Not Registered>
4,39
2,60
1,00
0,50
-3,00
-8,50
-10,50
001 Zand toplaag
002 Zand
003 Klei
004 klei
005 zand 5 MPa
006 zand 2 MPa
007 zand 6 MPa
F_hor;k
Slab
Fundex 540/ 660
Annex -
1901679
9/18/2019
JJO
A4


Moments/Forces/Displacements
Fundex 640/ 660 mm
F horizontal = 10 kN
Storage taks
Bending Moments [kNm]
Shear Forces [kN]
Displacements [mm]
Depth [m]
001 Zand toplaag
002 Zand
003 Klei
004 klei
005 zand 5 MPa
006 zand 2 MPa
007 zand 6 MPa
Fundex 540/ 660
Max: 13,0 - Min: -2,9
Max: 0,6 - Min: -10,0
Max: 0,5
1901679
9/18/2019
JJO
A4


Moments/Forces/Displacements
Bending Moments [kNm]
Shear Forces [kN]
Displacements [mm]
Depth [m]
001 Zand toplaag
002 Zand
003 Klei
004 klei
005 zand 5 MPa
Fundex 540/ 660
006 zand 2 MPa
007 zand 6 MPa
Max: 0,8 - Min: -9,2
Max: 2,3 - Min: -10,0
Max: 1,1
Fundex 640/ 660 mm Free
F horizontal = 10 kN
Storage taks
9/18/2019
1901679
JJO
A4

Annex D
Calculation examples shallow foundation

Mos Grondmechanica

Albert Plesmanweg 47 | 3088 GB Rotterdam | Tel. +31 (0)88 5130200

Order No. 1901679 | Date 8-6-2020 | ver 20180307

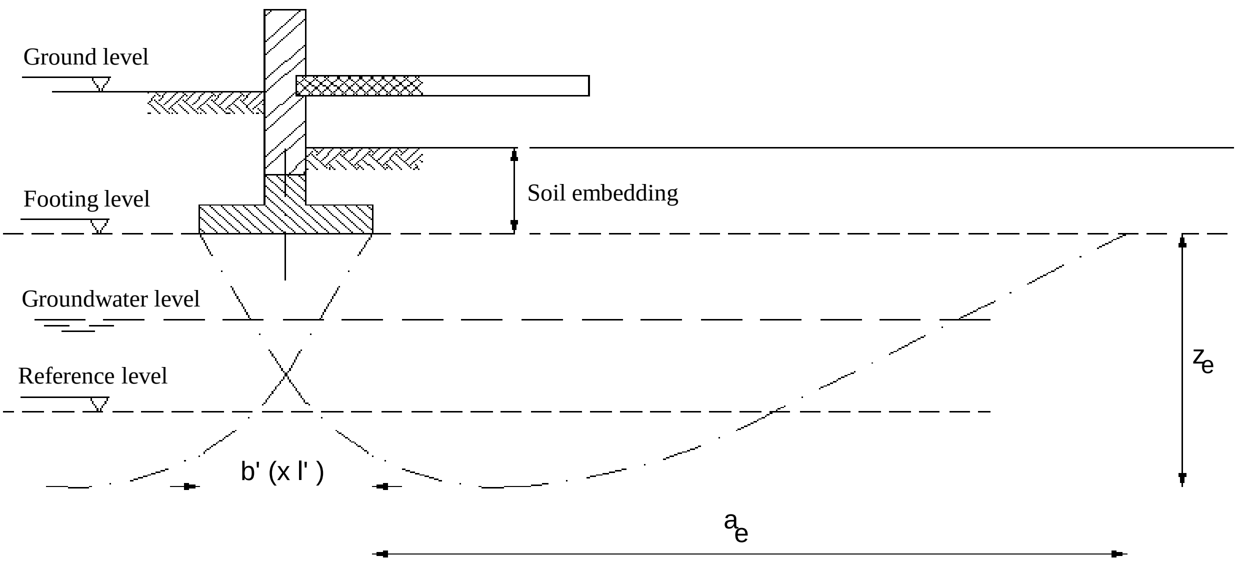
MAXIMUM BEARING RESISTANCE OF SHALLOW FOUNDATIONS

Ref. level	NAP		Partial material factors		Project:	Hexion
Ground level	[NAP + m]	4,80	$\gamma_{m;\gamma}$ =	1,10		
Footing level	[NAP + m]	4,35	$\gamma_{m;\varphi'}$ =	1,15		Formaline storage tanks
Gw. level	[NAP + m]	3,90	$\gamma_{m;c'}$ =	1,60		

CHARACTERISTIC VALUES OF THE SOIL PROPERTIES							DESIGN VALUES SOIL PROPERTIES			
Layer No.	top layer [NAP + m]	bottom layer [NAP + m]	γ_k [kN/m³]	$\gamma_{sat;k}$ [kN/m³]	φ'_k [°]	c'_k [kN/m²]	γ_d [kN/m³]	$\gamma_{sat;d}$ [kN/m³]	φ'_d [°]	c'_d [kN/m²]
GL / embedd.	4,80	4,35	17,0	19,0			15,45			
1	4,35	3,90	18,0	20,0	30,0	0,0	16,36		26,66	0,00
2	3,90	1,00	18,0	20,0	30,0	0,0		18,18	26,66	0,00
3	1,00		15,0	15,0	25,0	1,0		13,64	22,07	0,63
4										
5										
6										
7										
8										

DESIGN VALUES OF THE VERTICAL RESISTANCE ON A HORIZONTAL FOOTING AREA (Rv;d)

Effective footing area		embedding: 0,00 m			embedd.: 0,30 m			embedd.: 0,45 m			Influence area	
b' [m]	l' [m]	$\sigma'_{max;d}$ [kN/m²]	$R_{v;d}$		$\sigma'_{max;d}$ [kN/m²]	$R_{v;d}$		$\sigma'_{max;d}$ [kN/m²]	$R_{v;d}$		z_e [m]	a_e [m]
0,40	strip	37	15	[kN/m']	98	39	[kN/m']	129	52	[kN/m']	0,57	1,43
0,50	strip	44	22	[kN/m']	105	53	[kN/m']	136	68	[kN/m']	0,71	1,79
0,60	strip	50	30	[kN/m']	111	67	[kN/m']	142	85	[kN/m']	0,85	2,15
0,70	strip	56	39	[kN/m']	117	82	[kN/m']	148	104	[kN/m']	0,99	2,50
0,80	strip	61	49	[kN/m']	123	98	[kN/m']	154	123	[kN/m']	1,14	2,86
0,90	strip	66	60	[kN/m']	128	115	[kN/m']	159	143	[kN/m']	1,28	3,22
1,00	strip	72	72	[kN/m']	133	133	[kN/m']	164	164	[kN/m']	1,42	3,58
1,10	strip	77	84	[kN/m']	138	152	[kN/m']	169	186	[kN/m']	1,56	3,94
1,20	strip	82	98	[kN/m']	143	172	[kN/m']	174	209	[kN/m']	1,70	4,29
1,50	strip	97	145	[kN/m']	158	237	[kN/m']	189	283	[kN/m']	2,13	5,37
0,75	0,75	42	24	[kN]	128	72	[kN]	170	96	[kN]	1,06	2,68
1,00	1,00	52	52	[kN]	137	137	[kN]	180	180	[kN]	1,42	3,58
1,25	1,25	61	95	[kN]	146	228	[kN]	189	295	[kN]	1,77	4,47
1,50	1,50	70	157	[kN]	155	349	[kN]	198	445	[kN]	2,13	5,37



Annex E
General execution directives of Vibration free and soil displacement screw piles

GENERAL DIRECTIVES FOR THE CONSTRUCTION OF VIBRATION FREE AND SOIL DISPLACEMENT SCREW PILES

Vibration free and soil displacement screw piles are to be subdivided in:

– Cast-in-place screw piles (like the Fundex pile or the screw injection pile).
– Prefabricated screw piles (like the Tubex pile).

Before the start of the manufacturing of the piles the following items must be known:

– The piling plan with the pile sizes and the pile tip levels, including the cone penetration test locations and the proposed pile installation sequence.
– The ground level height at the location of each of the piles to be installed.
– The ground level height at the location of each cone penetration test (CPT).
– The soil investigation report and the corresponding foundation advice.

For the manufacturing of vibration free and soil displacement screw piles next items have to be taken care of:

– The "lost" or to be retrieved casing tube has to be straight.
– The diameter of the casing tube or cylindrical prefabricated pile has to be even over the full length.
– In case of pile screwing near adjacencies it is (mostly) preferable to start pile screwing at the smallest distance to the adjacencies and next follow a pile screwing sequence moving away from the adjacencies with an increasing distance.
– When a difference in pile tip level is prescribed it is (mostly) preferable to start the pile screwing work at the deepest pile tip level and next to proceed piling from the deepest to the highest pile tip level.
– As the foundation layer can vary between the CPT locations a check on this is advisable. This can be done by recording the oil pressure during pile screwing and to set out the obtained values of the maximum oil pressure per 0.25 m penetration against the pile penetration depth; the thus obtained diagram is called an oil-pressure diagram[1)]. For a well-chosen drilling engine (with a sufficient large drilling torque) and equal circumstances a clear correlation will mostly be seen between the oil pressure diagram and the cone resistance diagram.
– In order to be able to compare the oil pressure diagrams in a proper way it is advisable to screw the first pile at or near a CPT location ("calibrating"). At the first pile and all other piles in the neighbourhood of a CPT location an oil pressure diagram has to be made over the complete pile length.
– For the remaining piles it is sufficient to have an oil pressure diagram showing the transition to the bearing sand formation and which is continued to the pile tip level. The oil pressure diagrams have to be compared to the cone resistance diagrams of the belonging CPT('s), where a maximum deviation in adverse sense of $^1/_3$ is permitted.
– In case a permanent or temporary tube is used a check on the presence of water and soil in the tube has to take place after the recommended pile tip level has been reached. In case of rejection the tube has to be filled before extraction with concrete, grout or - on condition that there are no geohydrological objections - a mixture of sand and gravel. The pile tip level of a new (replacing) pile has to be at least as deep as the reached pile tip level of the rejected pile.

- The reinforcement has to be placed centrally.
- In case of a to be retrieved casing tube the extraction may only start once the concrete or grout has reached the pile tip level and is under pressure; the casing tube has to be extracted gradually.
- The concrete or grout consumption has to correspond to at least the theoretical contents of the pile.
- The piles can be manufactured immediately after each other, in case the mutual centre to centre distance is at least 4 times the pile tip diameter. A smaller distance is permitted, when the time between the making of the first and the second pile is thus long that the concrete or grout in the first made pile has stiffened sufficiently. For the mentioned time at least 20 hours have to be taken. In case a delaying additive is applied, the length of this period has to be lengthened if necessary.

Further it is referred to:

- NEN-EN 12699 "Execution of special geotechnical work - Displacement piles ".
- BRL 2356 (1992-06-01) "cast-in-place piles", with annex E (1992-08-01), for the manufacturing of vibration free, soil displacement screw piles.
- CUR Recommendation 114 (2009) "Supervision of the execution of pile foundations".

In cases of doubt with respect to the execution or other circumstances it is recommended to contact the geotechnical consultant.

Finally we draw to your attention that Mos Grondmechanica has at its disposal:

- Expert supervisors to guide all kind of soil- and foundation works.
- Adequate equipment and employees:
- To set out and or to measure pile fields.
- To test the integrity of piles (sonic check on eventual present serious defects).

Note:

[1] An oil pressure diagram is obtained by graphically setting out the maximum measured oil pressure per 0.25 m penetration to the corresponding penetration depth. Thereto the maximum oil pressure has to be measured and recorded (time after time) while the pile penetrates over a fixed distance of 0.25 m into the soil. The thus obtained oil pressure diagram is preferably plotted in the CPT diagram of the CPT at the smallest distance from the pile.

(August 1, 2017)

Annex F
General directives for the execution of shallow foundations

GENERAL DIRECTIVES FOR THE EXECUTION OF EXCAVATIONS AND SOIL IMPROVEMENTS FOR SHALLOW FOUNDATIONS

Before the start of the execution of excavations and soil improvements the following items should be known:

- The foundation plan with the sizes and construction depths of the foundation elements, including the locations where the cone penetration tests (CPT's) (and borings) have been executed.
- The ground level height at the location of each shallow foundation to be made.
- The ground level height at the location of each CPT (and boring).
- The soil investigation report and the corresponding foundation advice.

If the recommended excavation level lies below the footing level a soil improvement shall be applied. For each construction part the excavation works should start at the CPT location, where the deepest excavation level has been recommended. In that way the changes to shallower excavation levels can be determined during the works. These changes should be executed gradually or (in case of an abrupt change in excavation level) in terraces, related to the thickness of each of the layers of the soil improvement.

In general the excavations can be executed with slopes with an inclination of 1 : 1. In case of a soil profile where water can flow out of the slope, extra measures are necessary. Further it has been assumed that along the upper rim of the slope no heavy equipment will be placed nor heavy material will be stored, and that the groundwater level will remain or will be kept permanently at least 0.5 m below the (deepest) actual excavation level.

After the recommended excavation levels have been reached, in case of a shallow foundation on sand it should to be checked with a hand CPT apparatus whether at or just below this level compressible layers are still present. This check should be carried out intensively, especially between the CPT's (and borings). When such layers are encountered, these layers have to be removed and replaced by sand or another material that should be appropriate for this purpose. Then the bottom of the pit or trench should be compacted with a soil vibrator device. The material to be compacted shall have a moisture content approximately corresponding to the optimum of the Proctor test. The degree of compaction shall be checked, for instance with a hand CPT apparatus. The criterion is, that the cone resistance shall increase in depth to at least 2.5 MPa at 0.10 m and 5 MPa at 0.30 m depth. The degree of compaction can also be related to the maximum Proctor density derived from the (previously executed!) modified Proctor tests. In this case the density found in situ, has to be at least 98 %, with an average density of at least 100 %. After this the working floor can be poured or - in case of an excavation level below the footing level - the first layer of the soil improvement can be installed.

Sometimes it turns out (also after compaction) that the previous given compaction demands are not (at once) met. This can be caused by several reasons or by a combination of such reasons. For this it can be thought of among others an insufficient low ground water table, a too high moisture content, a less favourable grain size distribution and or the use of a too heavy compaction apparatus which is less suitable for the compaction of the upper surface layers.

In case of doubt in deliberation with the geotechnical consultant it shall be determined how to deal with this. The geotechnical consultant then generally first wants to know whether the material present in principle is suitable (control by means of hand borings, in case of doubt followed by the execution of sieve analysis and or the execution of an in situ suitability test) and the actual ground water table is low enough (observation well measurements).

The sand for the soil improvement shall be mineral, moderate coarse material and may at most contain 5 weight percents (of the grains) of grains smaller than 16 μm and at most 10 weight percents of grains smaller than 63 μm. The content of organic matter (loss on ignition) must be smaller than or equal to 3 weight percents. The soil improvement should be superimposed in layers with a thickness of maximum 0.3 m. Each layer should be compacted mechanically, in at least 4 runs that have to cross and overlap. For each layer the already formulated moisture requirement applies. In case the upper layer has been shaken loose by the use of a relatively heavy vibrator, the foundation level should be re-compacted with a light vibrating-plate compactor before the working floor of the foundation is poured. For the check of the degree of compaction the above mentioned criterion apply.

The width of the soil improvement at the bottom of the pit or trench shall be at least $B + 2d$ respectively $L + 2d$. At this B and L are respectively the width and the length of the footing and d the thickness of the soil improvement.

Sometimes a shallow foundation is applied on (stiff, over-consolidated) clay, loam or loess. In this case the last 0.1 m should be sliced thus carefully, that the clay or loam below the excavation level is not disturbed. Next to avoid sloughing of the sub grade by precipitation as soon as possible after excavation on the bottom of the excavation a protective layer with a thickness of at least 0.1 m shall be superimposed.

Extra attention shall be given to excavations next to, or close to an existing, adjoining property on a shallow foundation. This applies in special for excavations deeper than the construction depth of the existing foundation. Such excavations reduce the bearing force of the existing foundation and should therefore be avoided as much as possible. In case such excavations are necessary then it should be worked out whether special precautions have to be taken.

During the compaction of the soil layer the groundwater level should be at least 0.5 m below the excavation level. If this is not the case, a (pumped) drainage should be installed, suitable to lower the groundwater level to at least this level. This drainage should be realised before the required excavation depth is reached.

For the monitoring of the groundwater level it shall be considered to install one or more observation wells in advance.

In cases of doubt with respect to the execution of the works or other circumstances it is recommended to contact the geotechnical consultant.

Finally we draw to your attention that Mos Grondmechanica has to its disposal:

- Expert supervisors to guide of all kind of soil and foundation works.
- Good equipment and employees to check the realised compaction(s).
- Laboratory facilities to test the suitability of the material for the soil improvement.

(April 7, 2015)

MOS GRONDMECHANICA B.V.

Hieronder treft u de dienstverlening van Mos Grondmechanica b.v. aan. Voor specifieke diensten die niet direct in het overzicht terug zijn te vinden kunt u uiteraard vrijblijvend contact met ons opnemen.

MANAGEMENTSYSTEEM CERTIFICATIE DNV·GL ISO 9001 = VCA**

MOS GRONDMECHANICA

VELDWERK

Sonderen op land, water en in beperkte ruimte, elektrisch, waterspanning, dissipatie, seismisch, magnetisch, geleidbaarheid, Bolconus, T-bar en slagsonderen

Geotechnisch boren en (on)geroerde monstername

Peilbuizen en waterspanningsmeters plaatsen

X, Y en Z metingen en Lintvoegmetingen

Plaatdruk-, CBR- en CPM proeven

In situ doorlatenheidsproeven

LABORATORIUM

Classificatie proeven (o.a. vol. gewicht, KVD, PI)

Samendrukkingsproeven (Oedometer en CRS)

Triaxiaalproeven

DS en DSS-proeven

Doorlatenheidsproeven

Dichtheidsbepaling (Proctor en CBR)

Cementbentoniet onderzoek

GEOMONITORING

Deformatiemeting (inclino- en extensometing)

(Grond)waterspanningsmeting

Zettingsmonitoring

Trillingsmonitoring (SBR)

Akoestisch doormeten van palen (CUR 109)

Online meetgegevens via portal

MILIEU (MOS MILIEU B.V.)

Verkennend-, nader- en saneringsonderzoek

Partijkeuringen besluit bodemkwaliteit (Bbk)

Saneringsbegeleiding. Waterbodem onderzoek.

Vergunning aanvragen.

2nd Opinion / Contra-Expertise Bodemonderzoeken.

GEOTECHNISCH ADVIES

Paalfundering

Fundering op staal

Grondkerende constructies

Bouwputontwerp

Omgevingsbeïnvloeding (Plaxis)

Zettingsanalyse (bouwrijp maken, opslagtanks)

Taludstabiliteit

Tankbouwadvies

Trillingsprognose

Schade expertise

Review en 2nd Opinion

GEOHYDROLOGISCH ADVIES

Bemalingen (incl. retourbemalingen)

Vergunningsaanvragen

Pompproeven

Omgekeerde Osmose

Barrièrewerking

Drainage

Infiltratie hemelwater

BEMALINGEN (MOS GRONDWATERTECHNIEK)

Bronbemaling

Ondergrondse energie-opslag

Pomp- en leidingsystemen

Brandputten

OVERIG

Uitvoeringsbegeleiding

Meer weten?	Bezoek onze website www.mosgeo.com
Vragen?	Mail ons op info@mosgeo.com
Offerte aanvragen?	Mail ons op offerte@mosgeo.com

Mos Grondmechanica opereert structureel vanuit 5 vestigingen in Nederland en in Suriname. Via het zusterbedrijf Mosgeo b.v. worden wereldwijd projecten uitgevoerd, daar waar onze specifieke kennis en ervaring wordt gevraagd.

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