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Symposium:
"Pile design and displacements"
May 27th, 2016
The Hague

Franki Foundations Belgium

A case study of large screw pile groups behavior

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Presented by ir Maurice Bottiau

Content of the presentation

- Introduction to the geotechnical issues of oil tanks and the case study in Ostend
- Specific soil conditions at the site
- The original tank foundation design
- The specific problems during hydrotest
- The new foundation concept
- The instrumented single pile load test
- The tank settlement behavior (monitoring & analysis of the time settlement behavior)
- Conclusions

INTRODUCTION

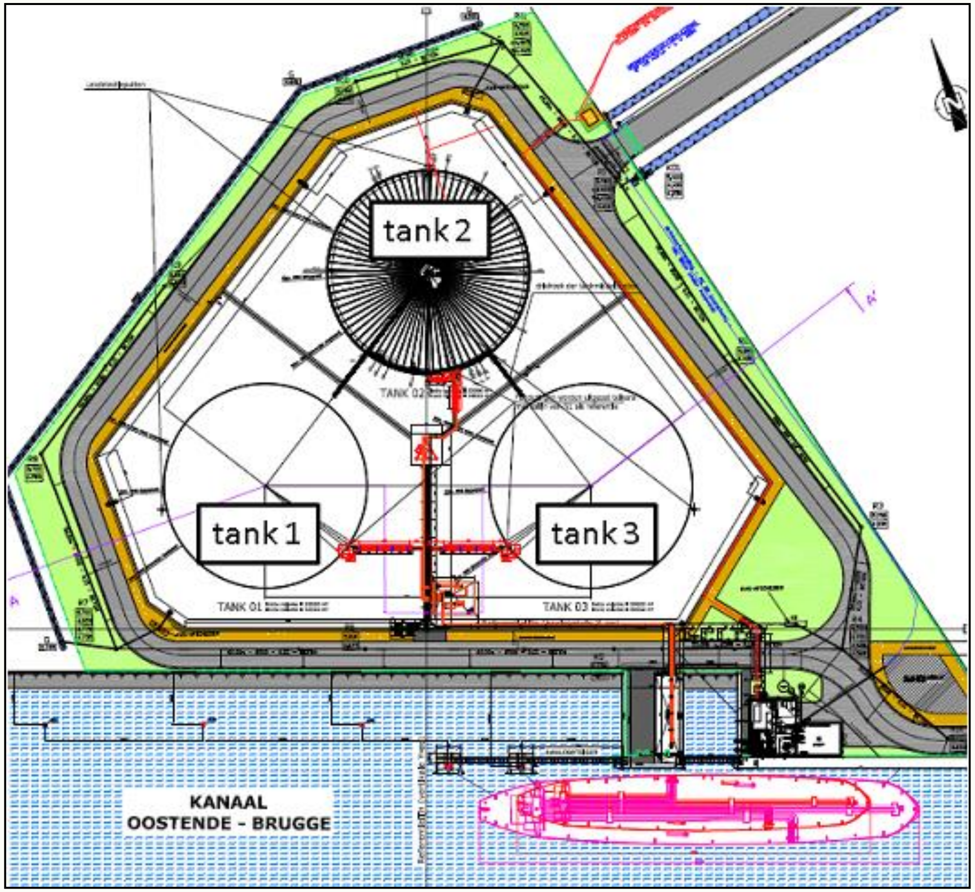
Introduction - Geotechnical issues with oil tanks

- Fuel tanks are often to be erected on sites with very poor soil conditions. In harbor area, the soil typically consists of soft alluvial clayey-, silty clayey- or even peaty-layers.

- In order to solve the foundation engineering problems in such soil conditions, three main solutions are possible:

1. **starting from a deep foundation concept**
2. **Pre-consolidating (improving) the soft layers with temporary overburden or with vacuum consolidation principles, combined with vertical drains to accelerate and improve the consolidation process**
3. **establishing the tank directly on a well designed artificial embankment on the soft layers; installing hydraulic vessels to adjust continuously for the differential and absolute settlements**

Introduction - The oil tanks' foundation case study



3 oil tanks (on very soft deposits), each of 33000 m^3 : steel structures of D=48 m and H=19 m, positioned in a triangular shape at a center to center inter-distance of about 65 m

Fuel tanks under construction (Ostend, autumn 2012)

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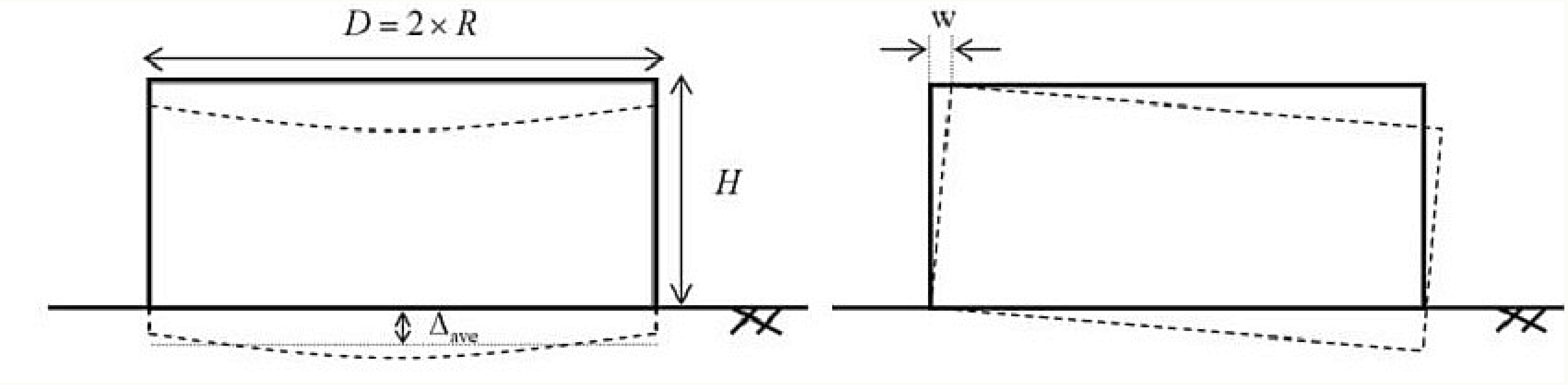
Introduction – Allowable settlements for steel tanks

Reference	Total average settlement Δ_{ave} (mm)	Differential settlement (mm)	Tilt w
API 653 (1995)	-	0.031R	-
Klepikov (1989)	180 (large tank) 110 (small tank)	0.004D (large tank) 0.008D (small tank)	0.004H (visible) 0.007H (ultimate)
USACE (1990)	-	0.008R	-



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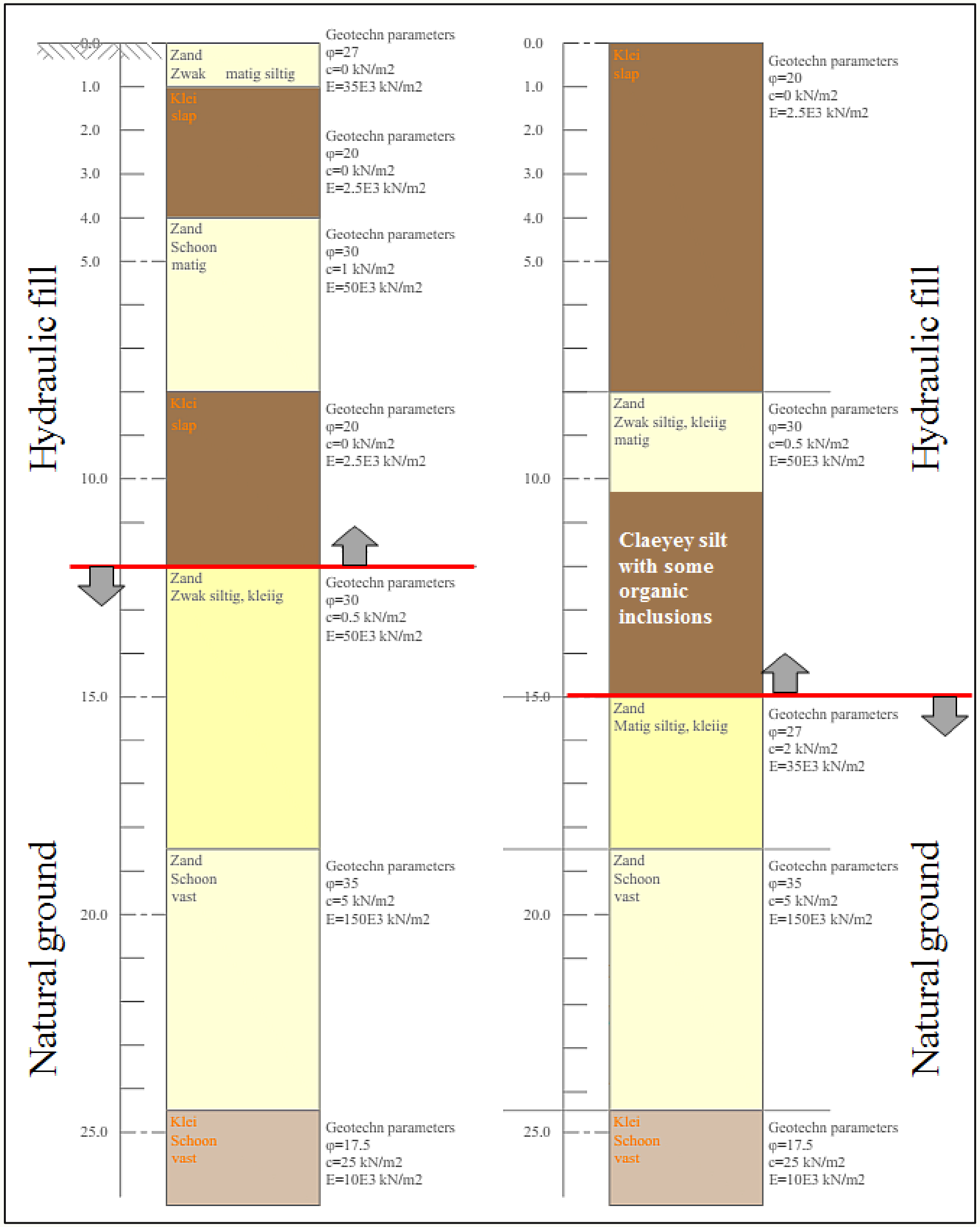
SOIL CONDITIONS AT THE SITE

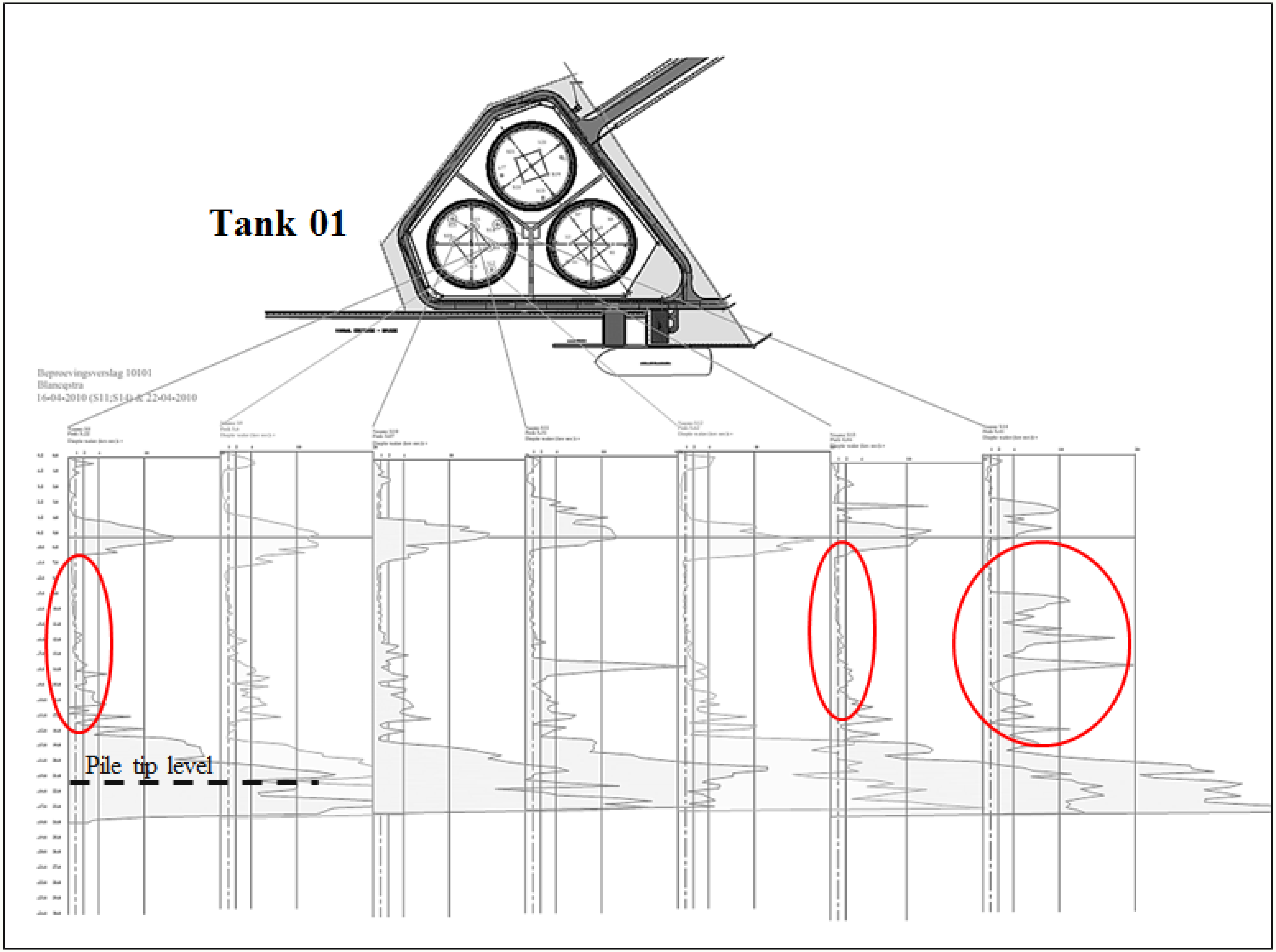
Soil conditions at the site



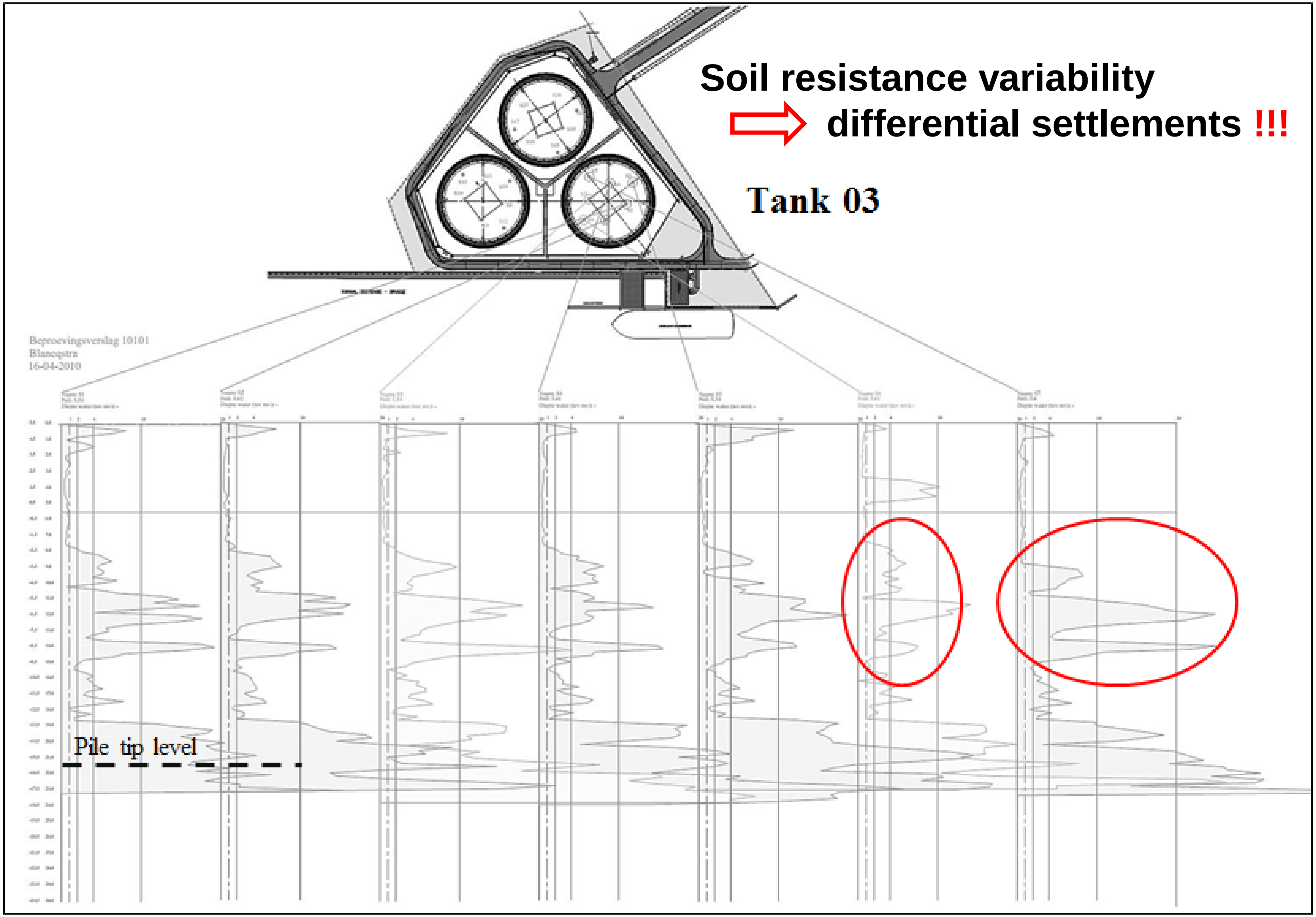
General pattern of the soil layering from large diameter borings and undisturbed sampling.

Until depths of about 15-17m; large differences, even at less than 50m interdistance, actually resulting from the ‘man made’ deposited layers over the last 25 years in this harbour/polder conditions

Pattern of soil resistance variability



Pattern of soil resistance variability



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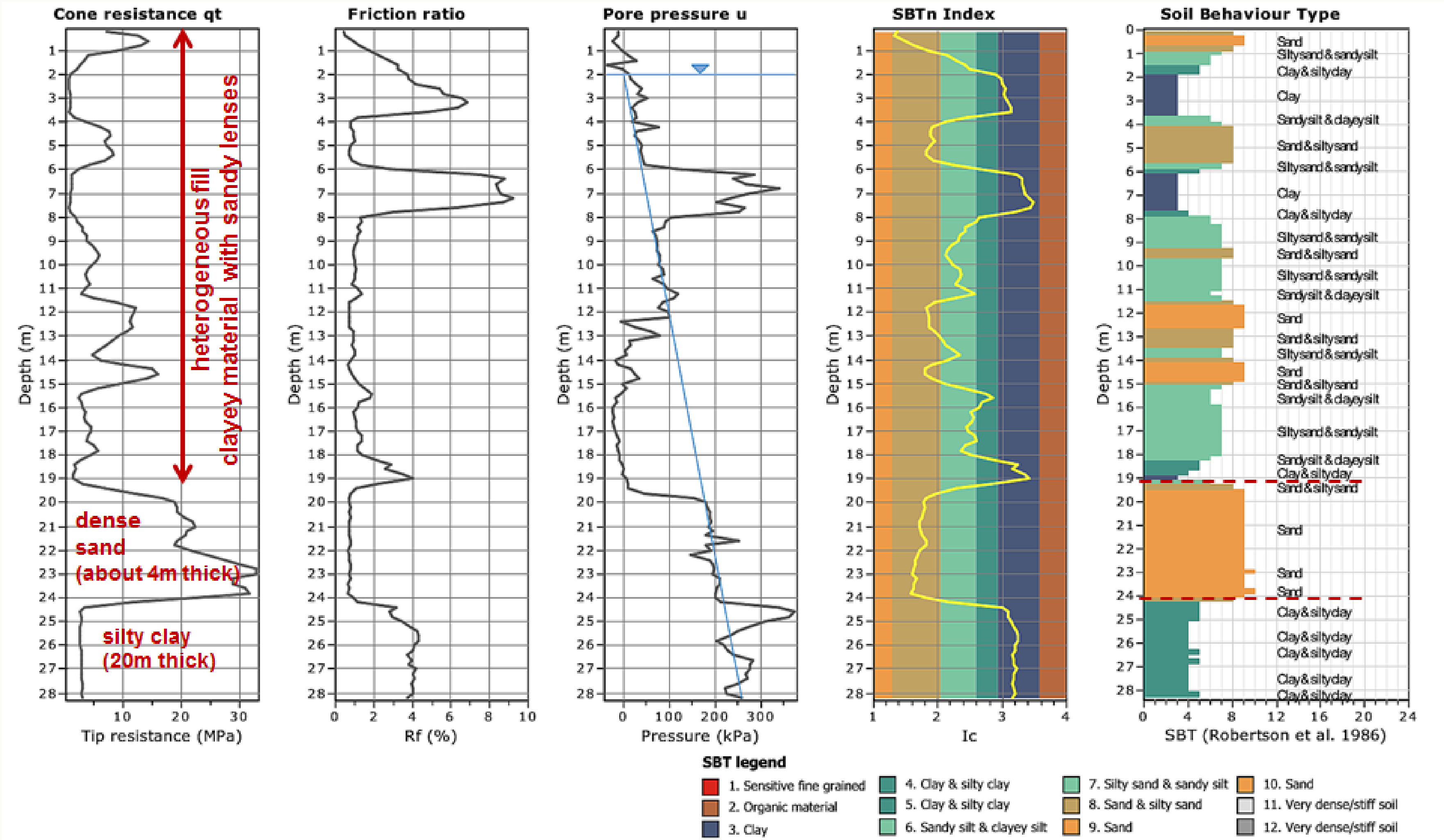
Soil conditions at the site

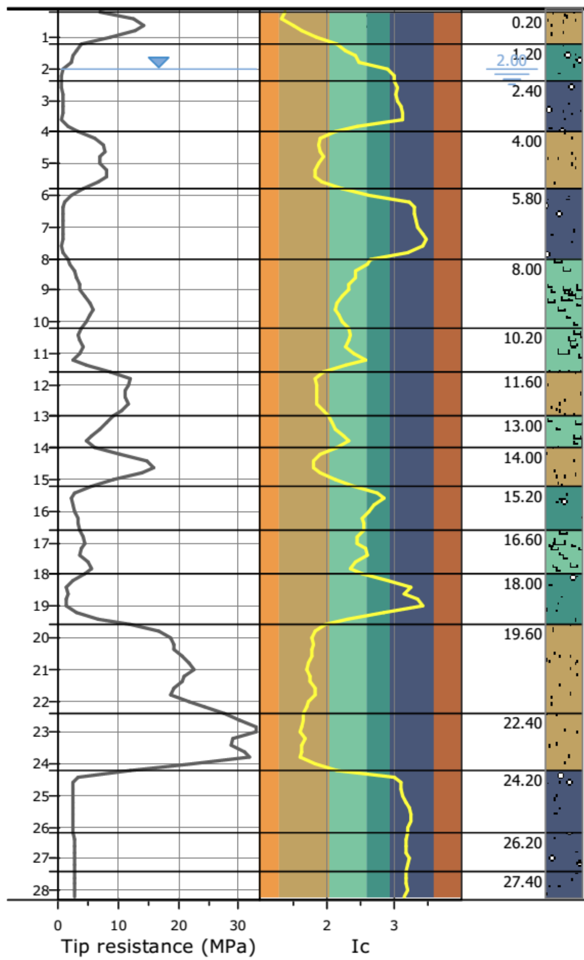


Soil conditions at the site

Example of soil data estimation from CPT interpretation (Robertson 2010)

Project: **CPT: CPT-04 - Total depth: 28.20 (m)**

Depth (m) Elevation: 0.00 (m)



Depth (m)	Description	qt (MPa)	Ksbt (m/s)	N60	Es (MPa)	Dr	Phi (°)	M (MPa)	Go (MPa)	Su (kPa)	Su ratio	OCR	Gamma (kN/m³)
0.20	Sand & silty sand	9.0	9.38E-5	16.2	51.4	83.0	43.8	64.4	64.4	-	-	-	19.0
1.20	Clay & silty clay	1.4	1.09E-7	3.9	35.3	-	-	19.0	34.0	69.6	1.7	17.7	19.0
2.40	Clay	0.9	1.42E-8	3.0	-	-	-	11.5	30.4	47.0	1.0	8.9	19.0
4.00	Sand & silty sand	6.4	1.10E-5	12.8	52.7	48.5	38.6	65.6	66.1	-	-	-	19.0
5.80	Clay	1.1	3.44E-9	4.2	-	-	-	9.2	45.5	60.3	0.7	5.9	19.0
8.00	Silty sand & sandy silt	3.6	7.40E-7	8.7	50.7	33.1	35.0	48.0	59.3	-	-	-	19.0
10.20	Silty sand & sandy silt	4.1	8.40E-7	9.5	52.9	32.1	34.7	52.3	65.0	-	-	-	19.0
11.60	Sand & silty sand	10.7	1.56E-5	21.1	83.1	50.1	39.0	104.2	104.2	-	-	-	19.0
13.00	Silty sand & sandy silt	6.6	2.50E-6	14.4	69.5	37.1	36.1	82.7	87.1	-	-	-	19.0
14.00	Sand & silty sand	10.1	6.86E-6	20.7	89.6	45.5	38.0	107.8	112.3	-	-	-	19.0
15.20	Clay & silty clay	3.2	1.01E-7	8.3	-	-	-	39.0	70.1	166.1	1.0	9.3	19.0
16.60	Silty sand & sandy silt	4.3	2.41E-7	10.8	66.7	26.7	32.9	55.0	82.8	-	-	-	19.0
18.00	Clay & silty clay	2.8	2.07E-8	9.0	-	-	-	20.8	78.7	99.8	0.5	4.0	19.0
19.60	Sand & silty sand	20.0	3.77E-5	37.6	131.9	59.0	40.5	165.3	165.3	-	-	-	19.0
22.40	Sand & silty sand	26.9	6.11E-5	49.6	163.0	66.9	41.7	204.3	204.3	-	-	-	19.0
24.20	Clay	3.0	4.18E-9	10.8	-	-	-	21.9	108.8	151.9	0.6	4.7	19.0
26.20	Clay	2.7	1.83E-9	10.1	-	-	-	18.2	112.2	157.3	0.6	4.6	19.0
27.40	Clay	2.9	1.95E-9	11.0	-	-	-	19.8	117.5	166.6	0.6	4.8	19.0

Soil conditions at the site

All raw CPT and CPTU data are available numerically to the TC207, for own interpretation of the soil data; additional soil testing in the lab; as well as additional CPTU with dissipation curves, as well as SCPT might be considered at the location.

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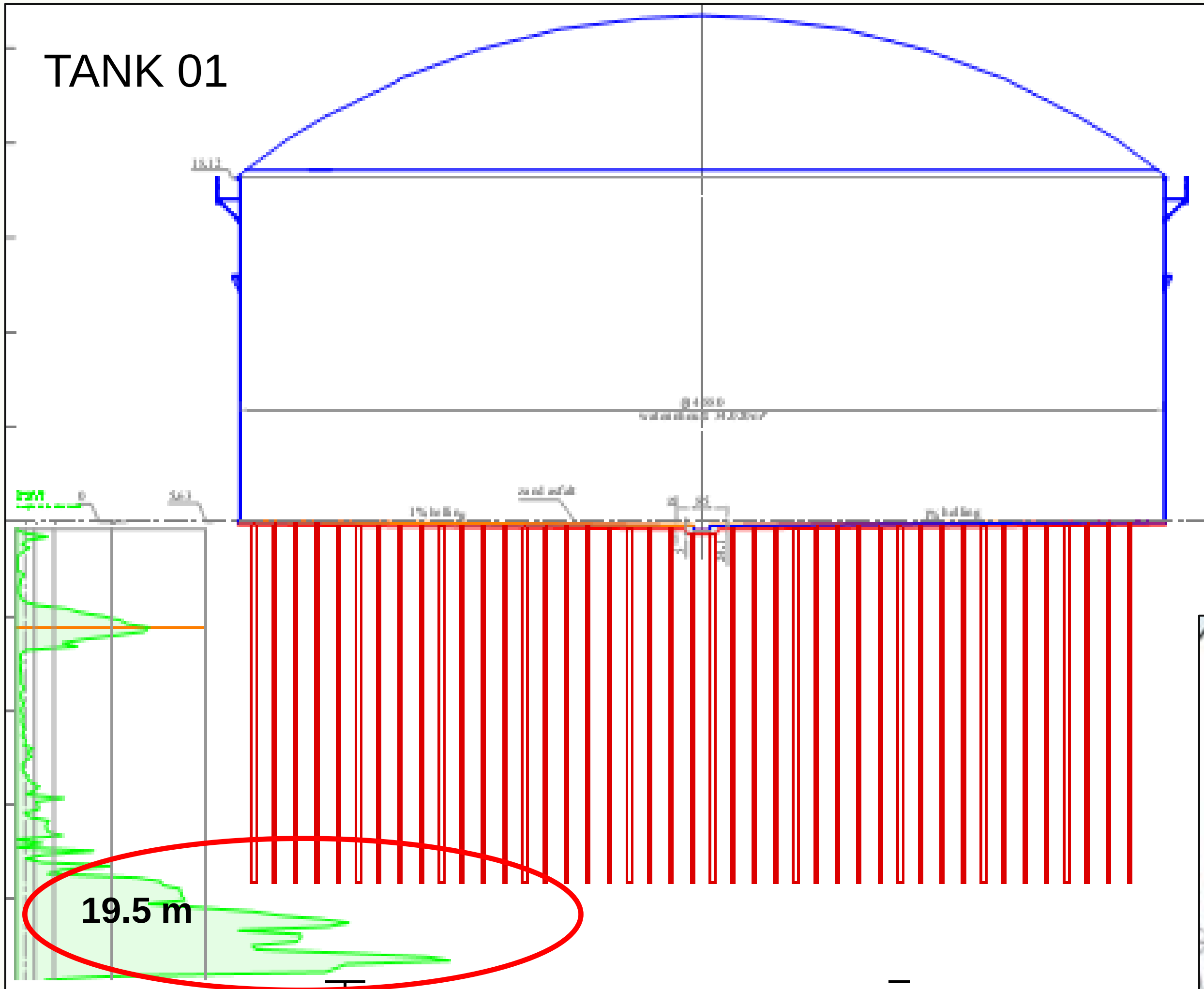
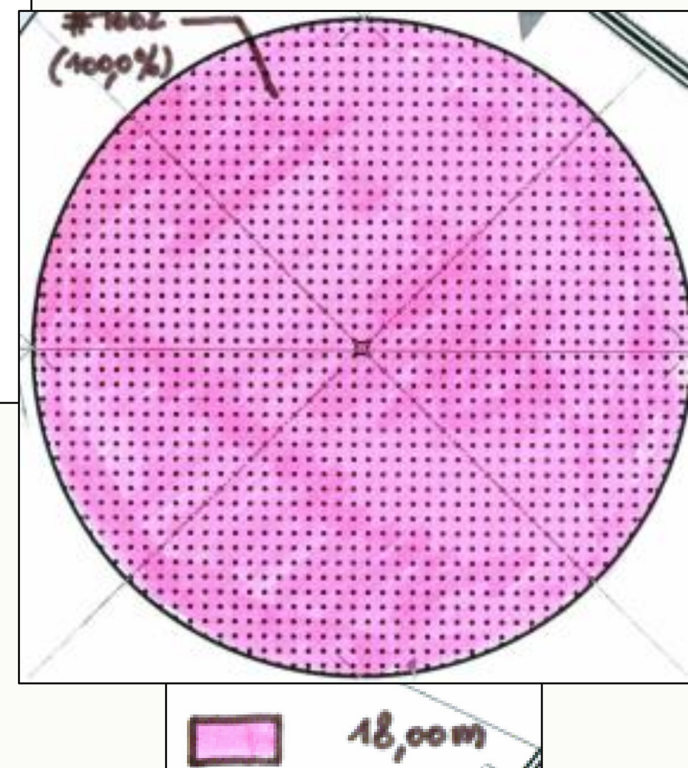
THE ORIGINAL FOUNDATION DESIGN

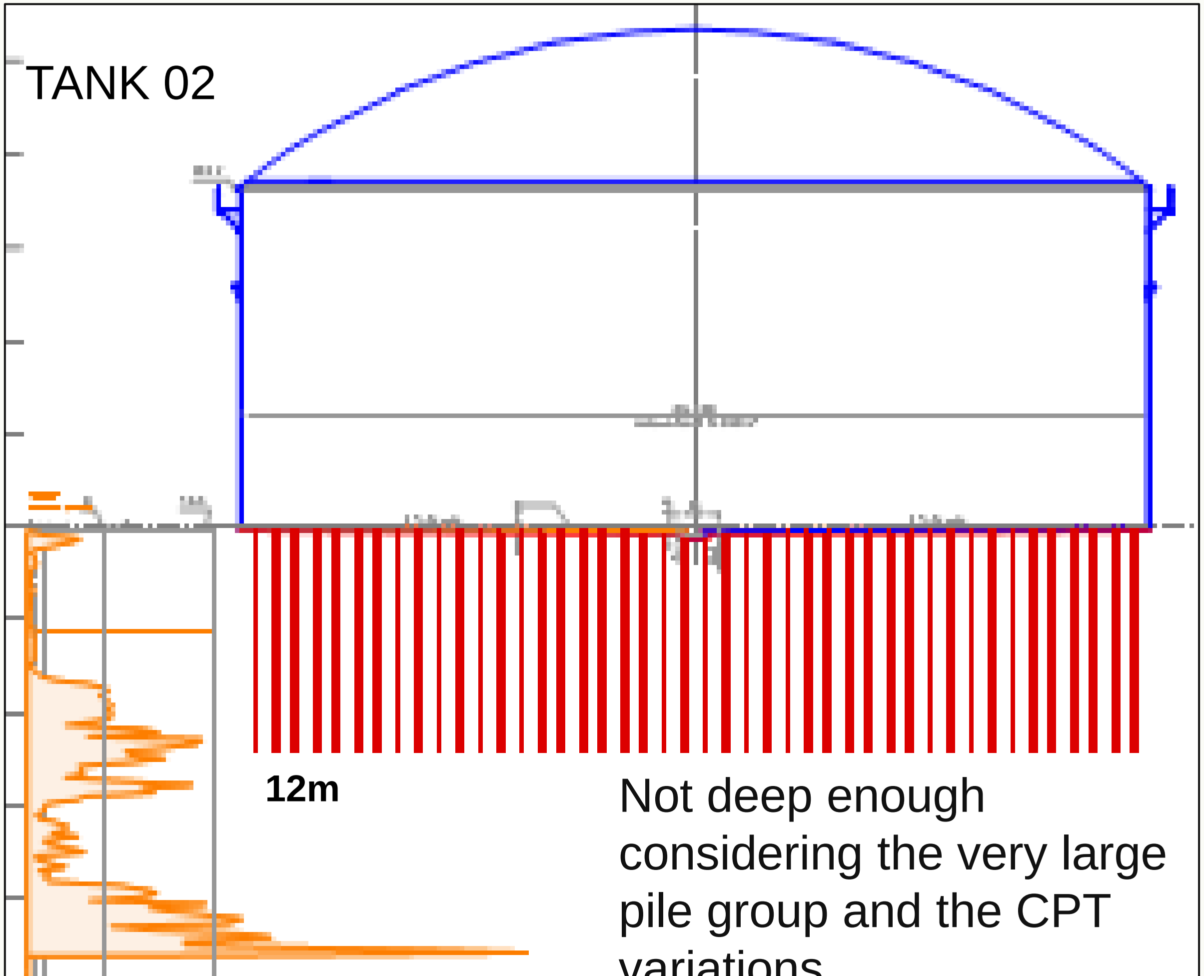
- 1800 small diameter mortar piles (vibratory installation) were supposed to handle the load bearing and deformation capacity of each of the tanks.
- The small diameter HSP piles (diam 180mm), were positioned in a grid of 1.1 x 1.1 m²; covered by a thin concrete sheet – 180 mm – as foundation slab.
- The small slender HSP piles were basically supposed in the design to work mainly as real deep foundation – tip load carrying piles



In total : ~1800 small diameter (180 mm) piles, each with ~230 kN of load

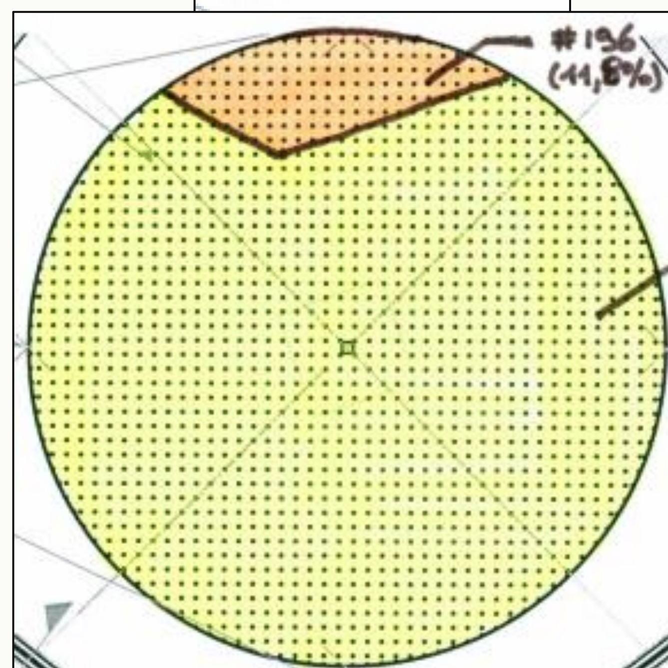


This suggests a foundation concept of the tank **on end bearing (root)piles;** **however with a very thin slab on top;** **???group effect???**



Not deep enough considering the very large pile group and the CPT variations

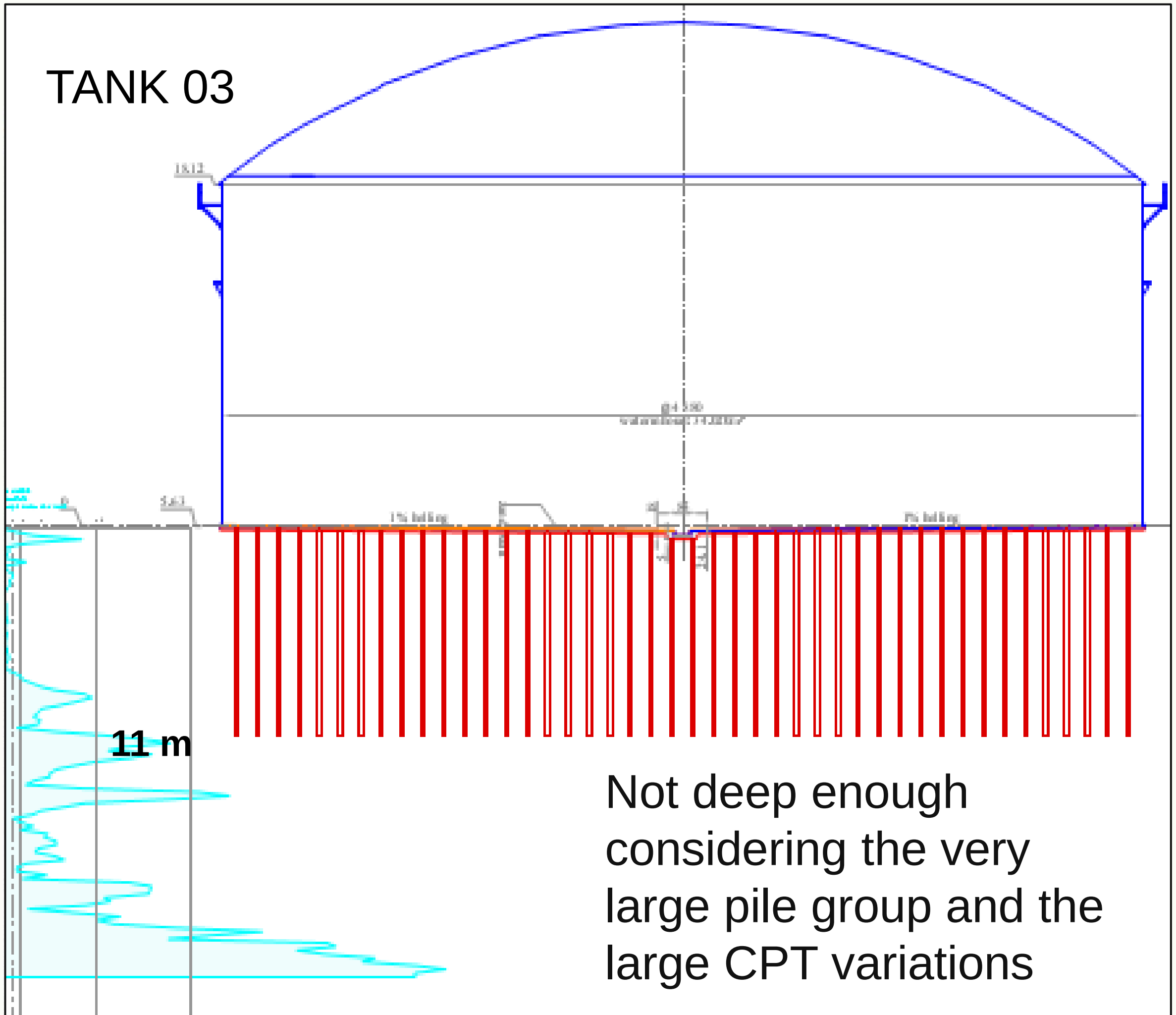
In total : ~1800 small diameter (180 mm) piles, each with ~230 kN of load

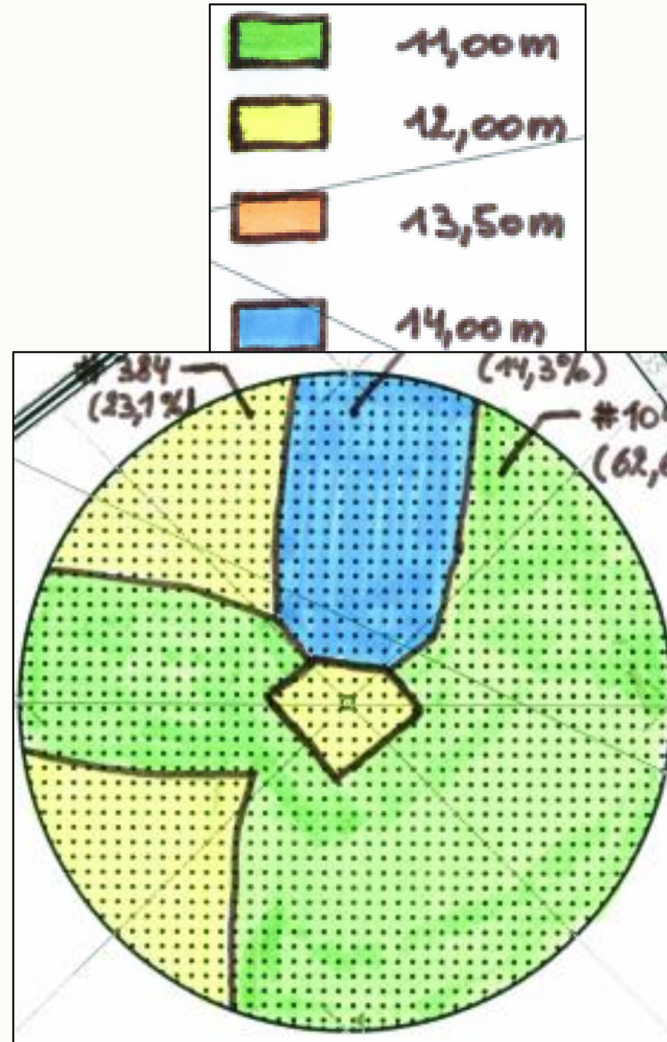


This suggests an equivalent raft foundation of the tank **on (for almost the total pile amount) short (root) piles ???acting as settlement reducers???**;

incompatible with the **design of end bearing piles** with a very thin slab on top

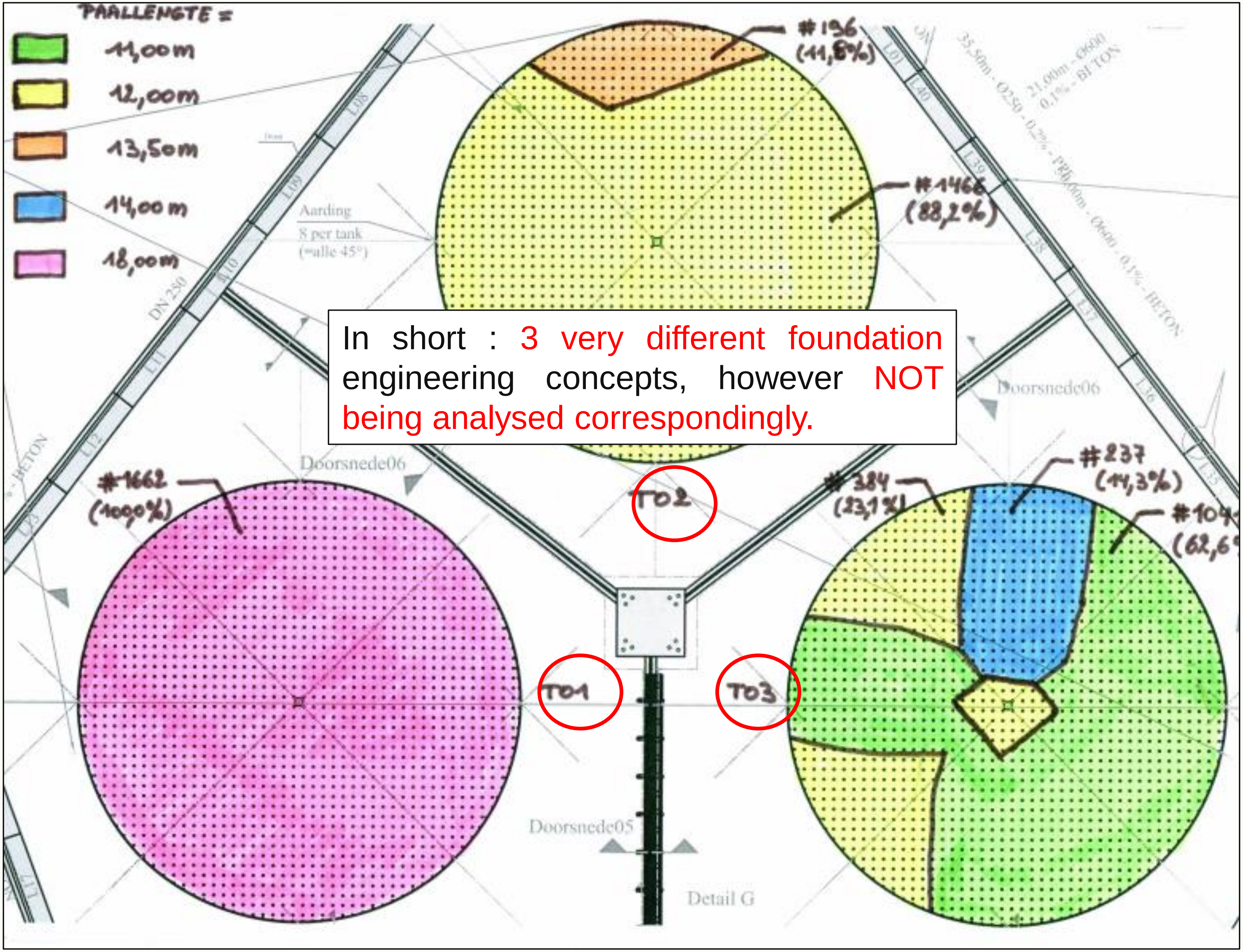
In total : ~1800 small diameter (180 mm) piles, each with ~230 kN of load





This suggests an equivalent raft foundation of the tank on **very short(~root) piles ?short end bearing with sub-groups of piles at various depths?**

<u>Incompatible with the design: an end bearing pile group concept</u>


PAALLENGTE =
11,00m
12,00m
13,50m
14,00 m
18,00m
#196
(11,8%)
#1466
(88,2%)
Aarding
8 per tank
(=alle 45°)
Doorsnede06
#1662
(100,0%)
#237
(14,3%)
#384
(23,1%)
(62,6
TO2
TO1
TO3
Doorsnede05
Detail G
In short : 3 very different foundation engineering concepts, however NOT being analysed correspondingly.

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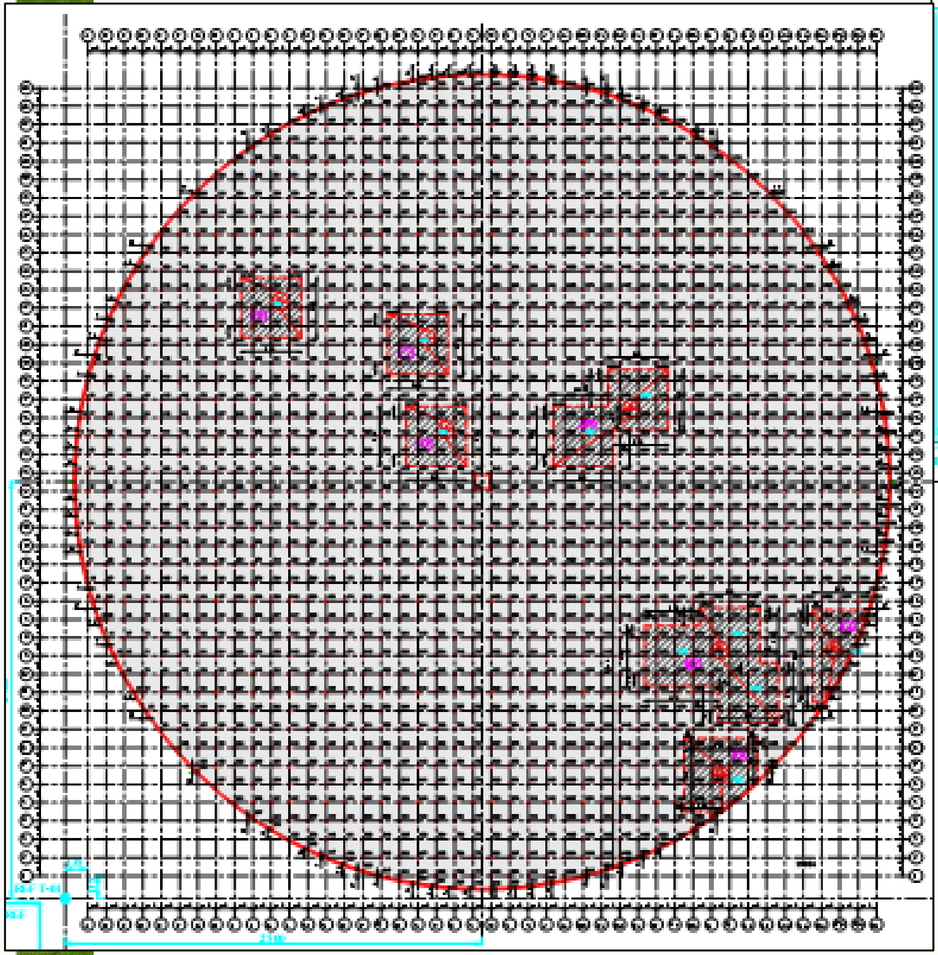
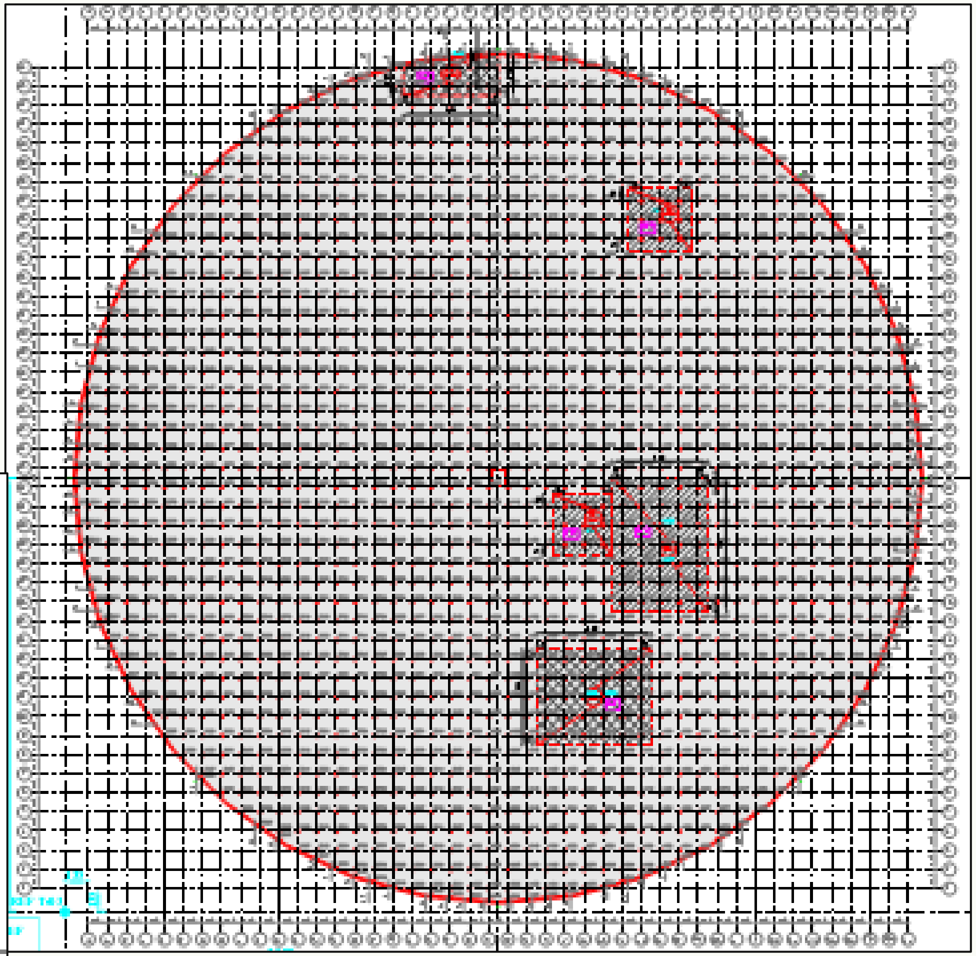
Notice of defects in HSP piles

It concerns mainly the necking/interruption of the pile shaft (ideally here only 180mm), as a result of the often high speed pile casing withdrawal and casting combined with the relatively high W/C ratio of the mortar.

Deficiencies were observed near the pile top but one could expect that lower located deficiencies would appear and cannot be detected nor repaired.

Tank 01 : small-diam pile deficiencies (upper ~2m)





Tank 03 : small-diam pile deficiencies (upper ~2m)

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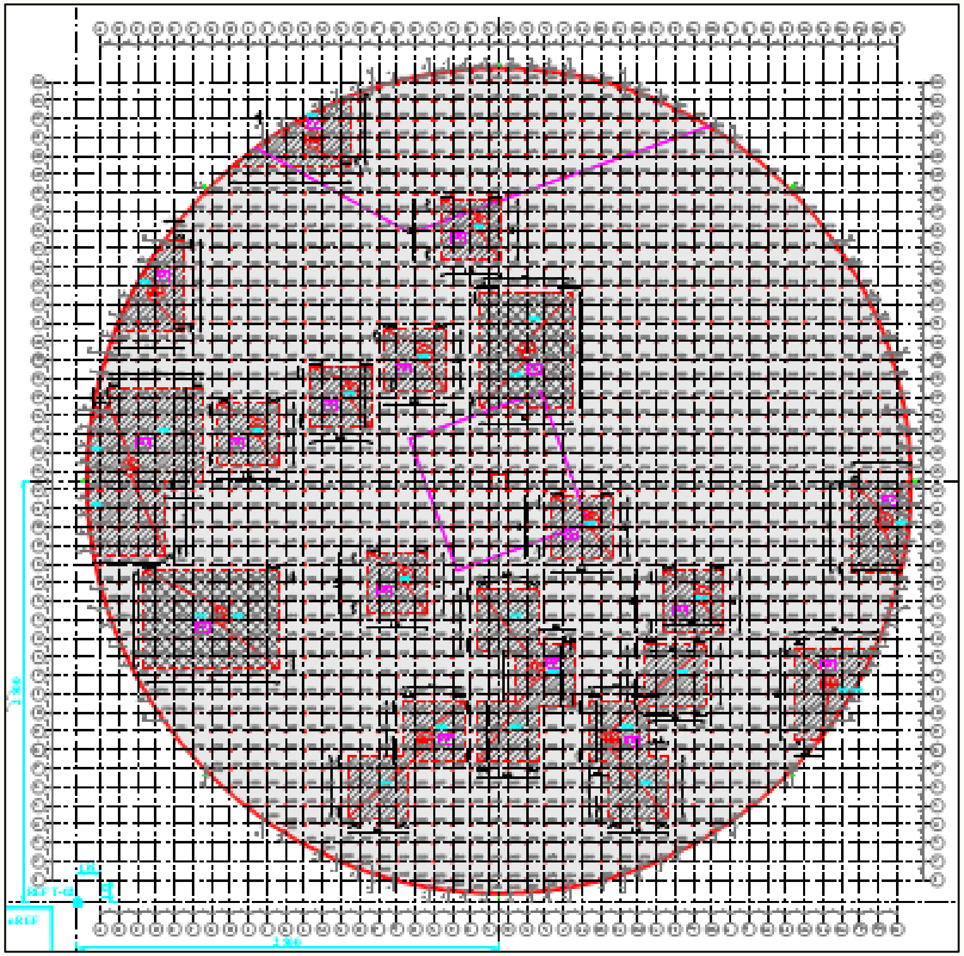
Tank 02 :Pile deficiencies in the top area (upper ~ 2 m)



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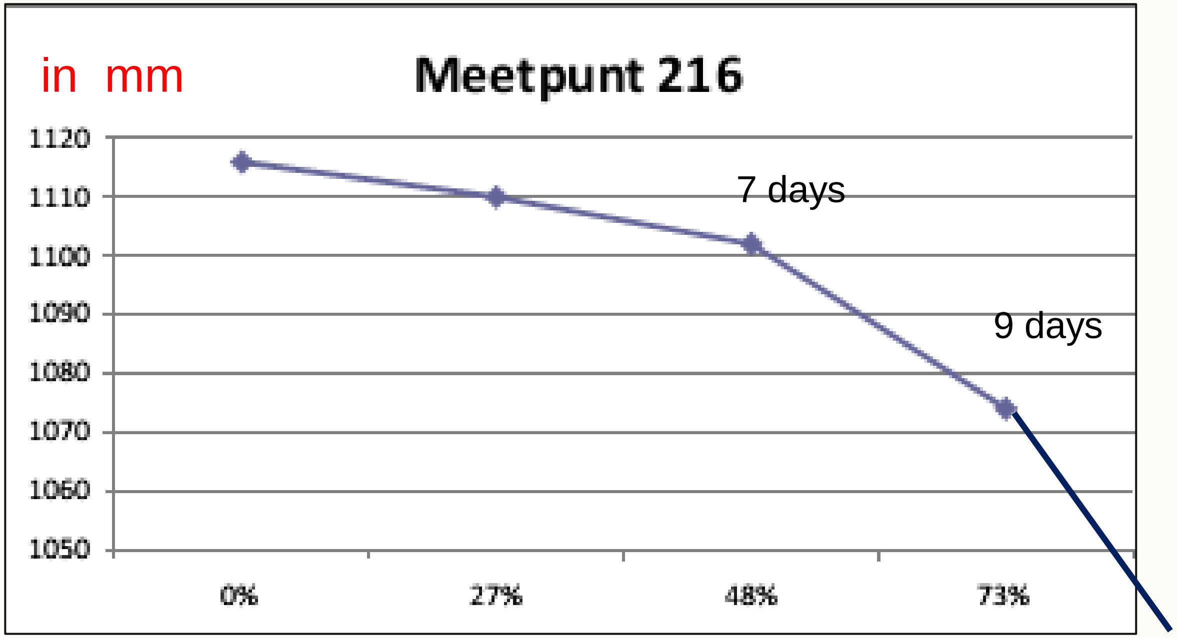
HYDRO-TESTS AND PROBLEMS OBSERVED

Movements during the hydrotest

Differential settlements are measured at several points along the perimeter of tank 2 during the water test



Increasing deformation gradient starting at ~50% loading,

but fully uncontrolled increase of the settlements – initiating 2 to 12 days after reaching the 92% level of loading ;

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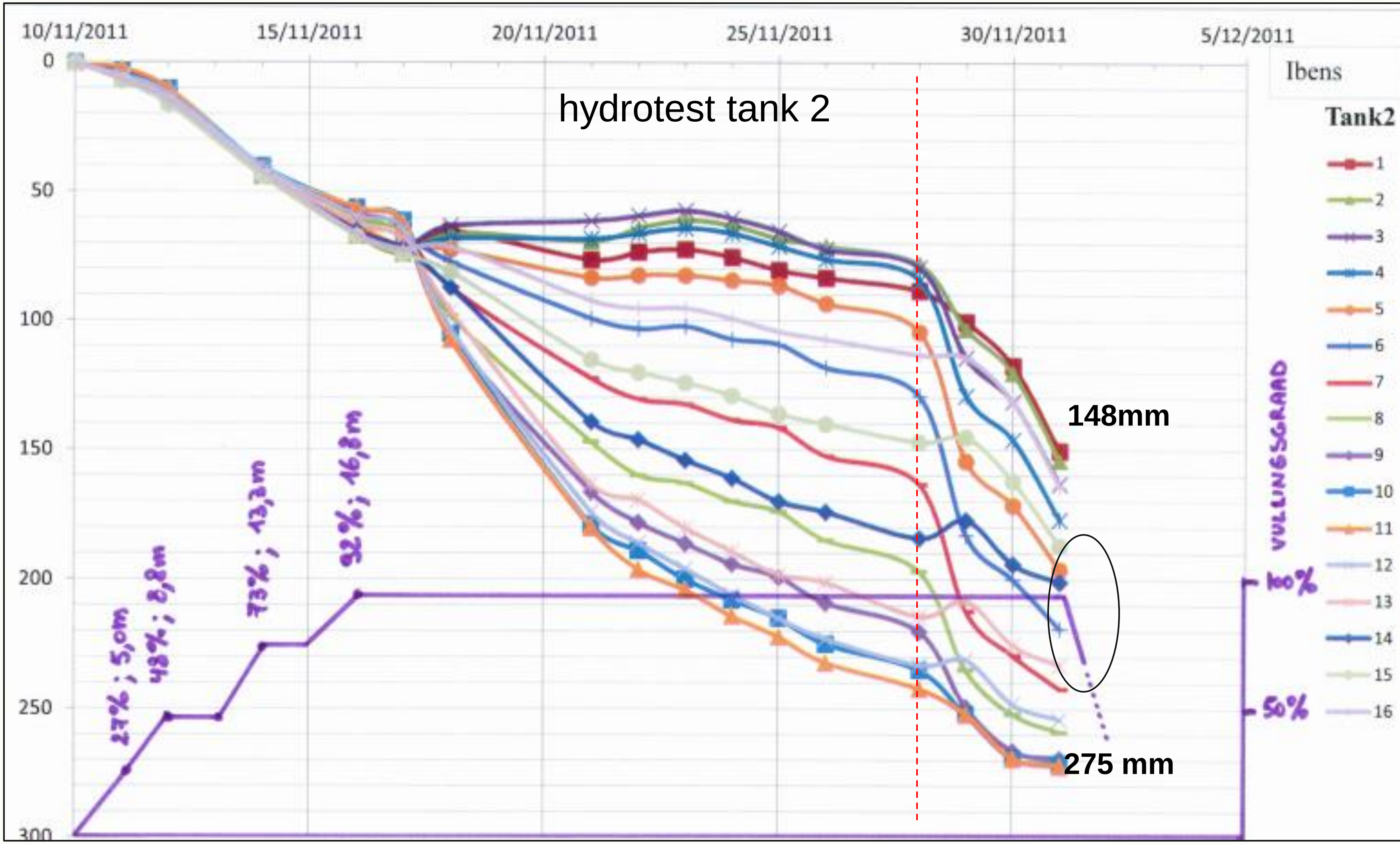
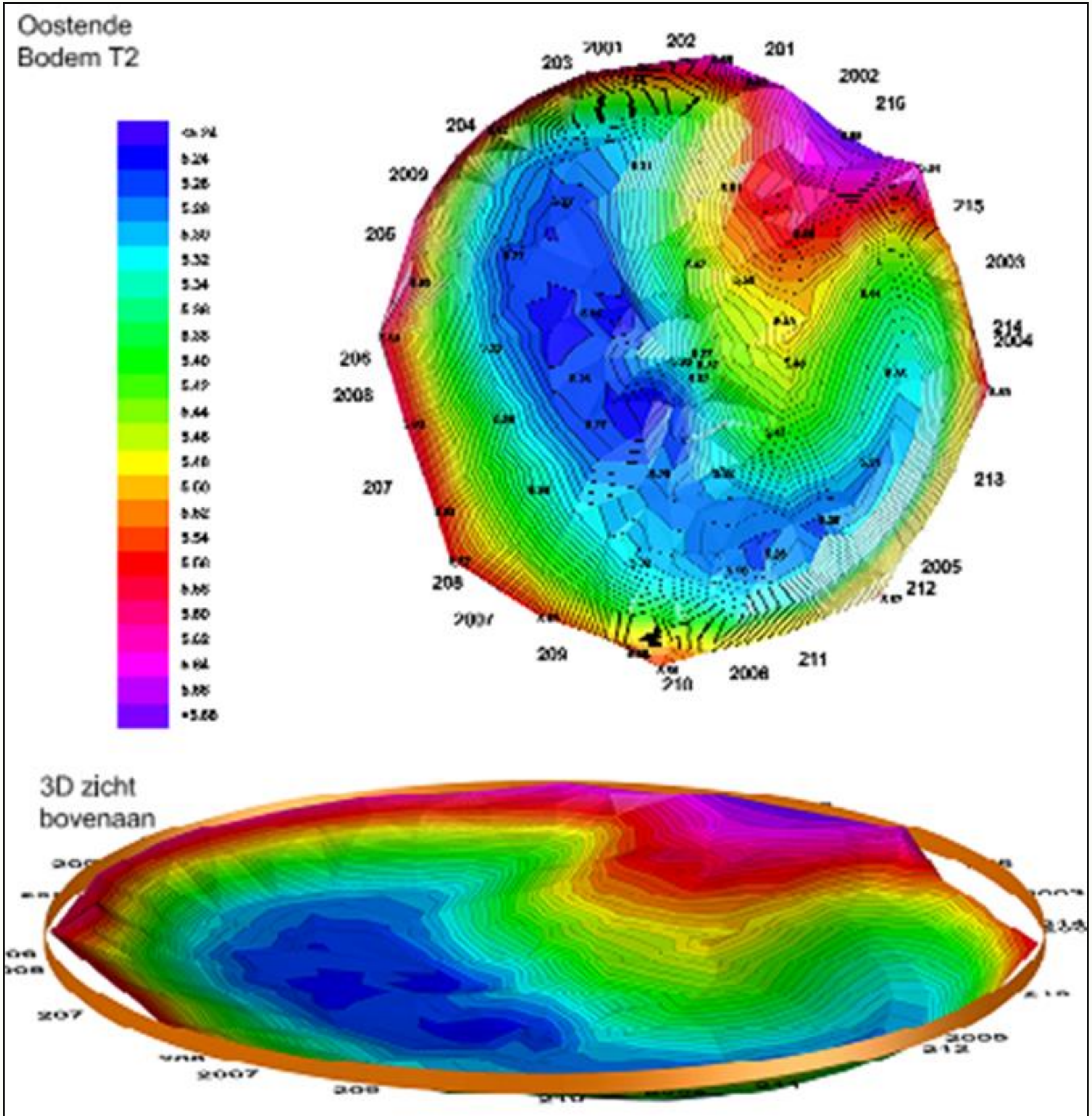


Increase in time of the settlements along the perimeter of tank 02, finally reaching the 92% loading level.

Rapid unloading required because of uncontrollable differential settlements at various perimeter points initiating 2 to 12 days after the 92% loading level.



Shape of the deformed bottom plate of tank 02 after loading test

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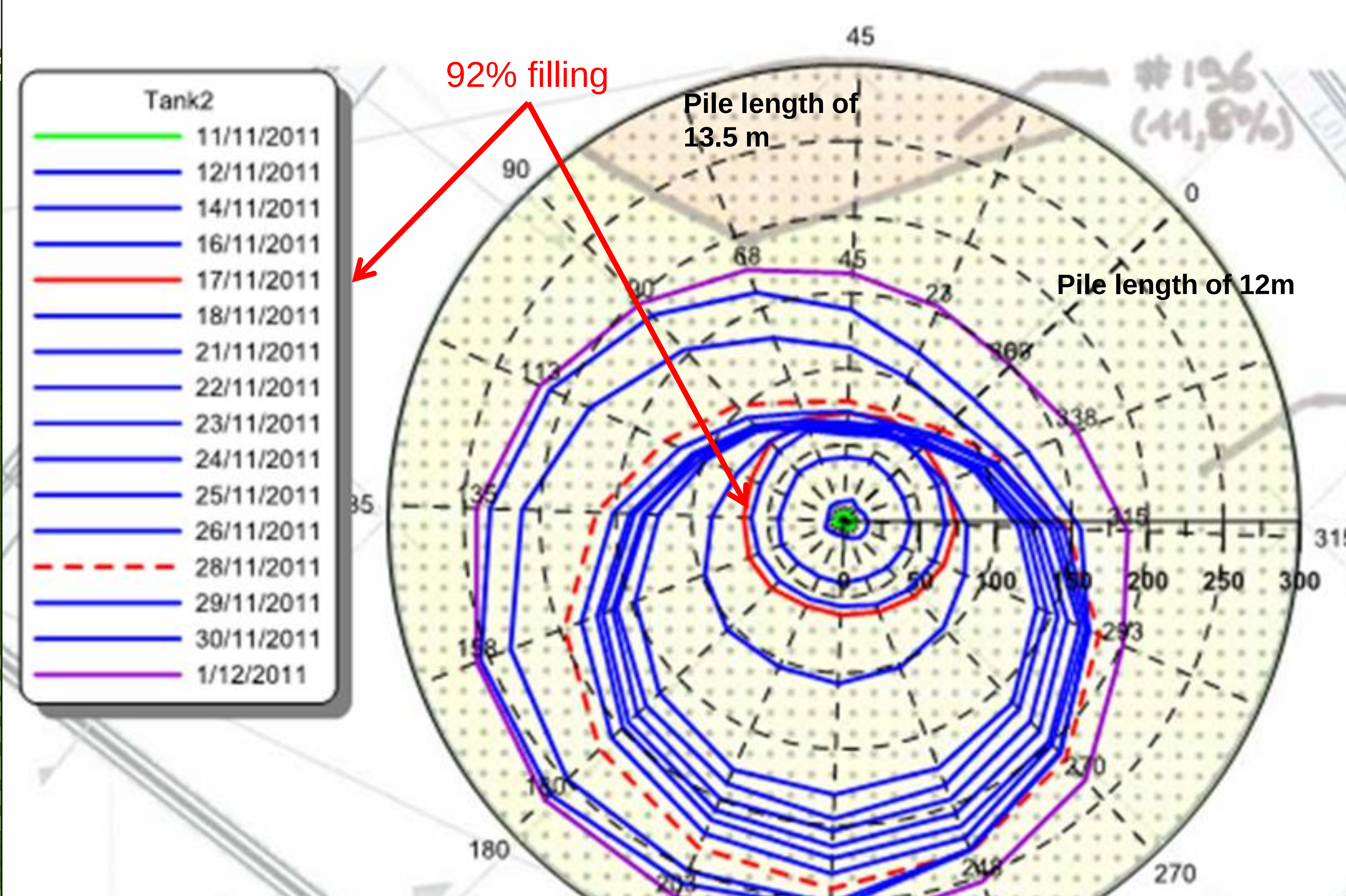
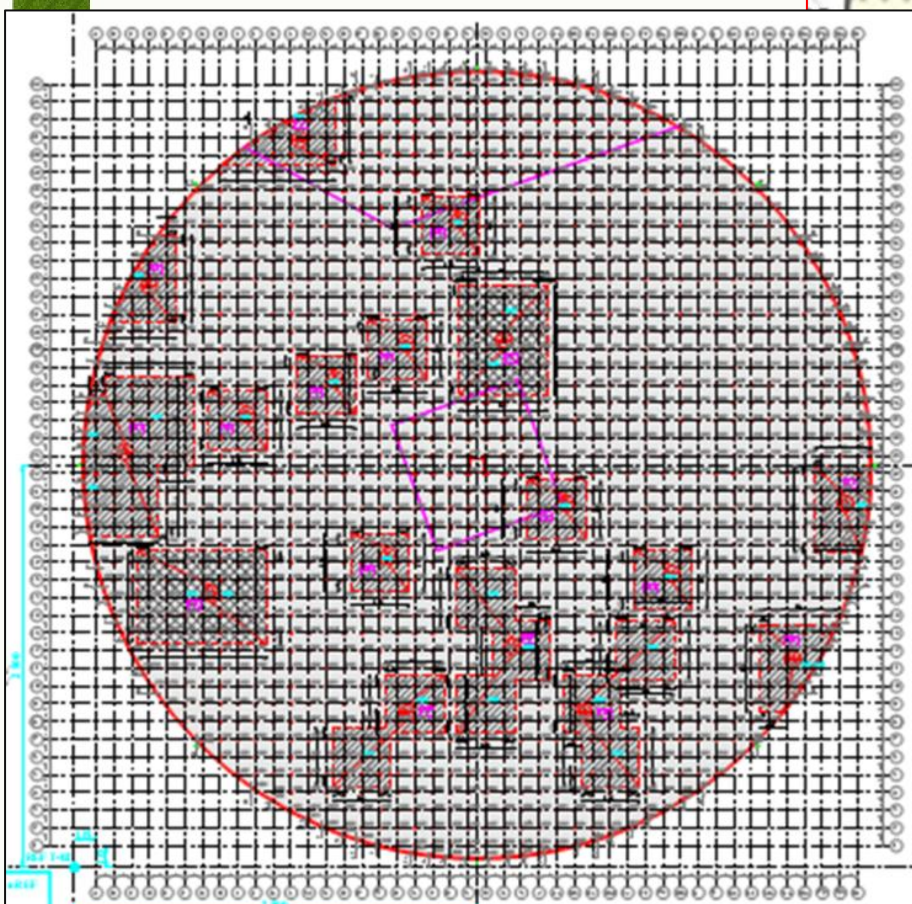
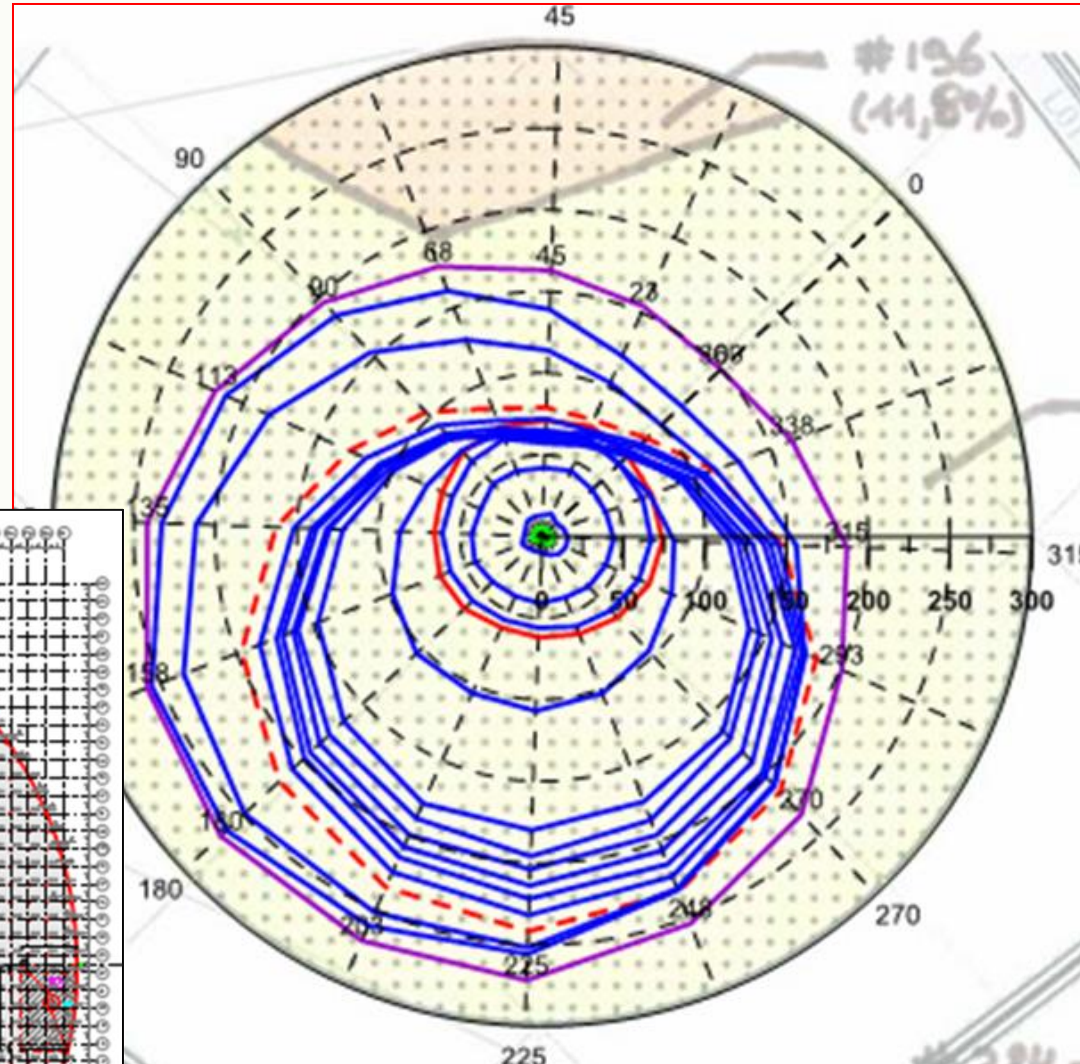
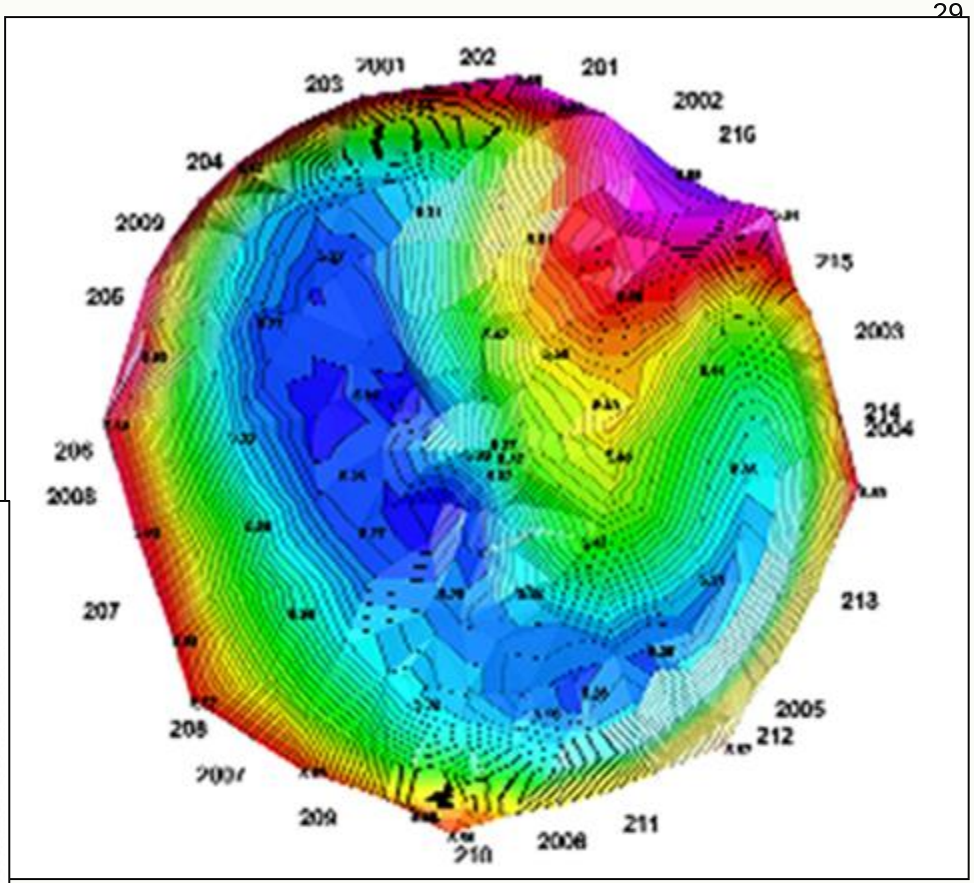


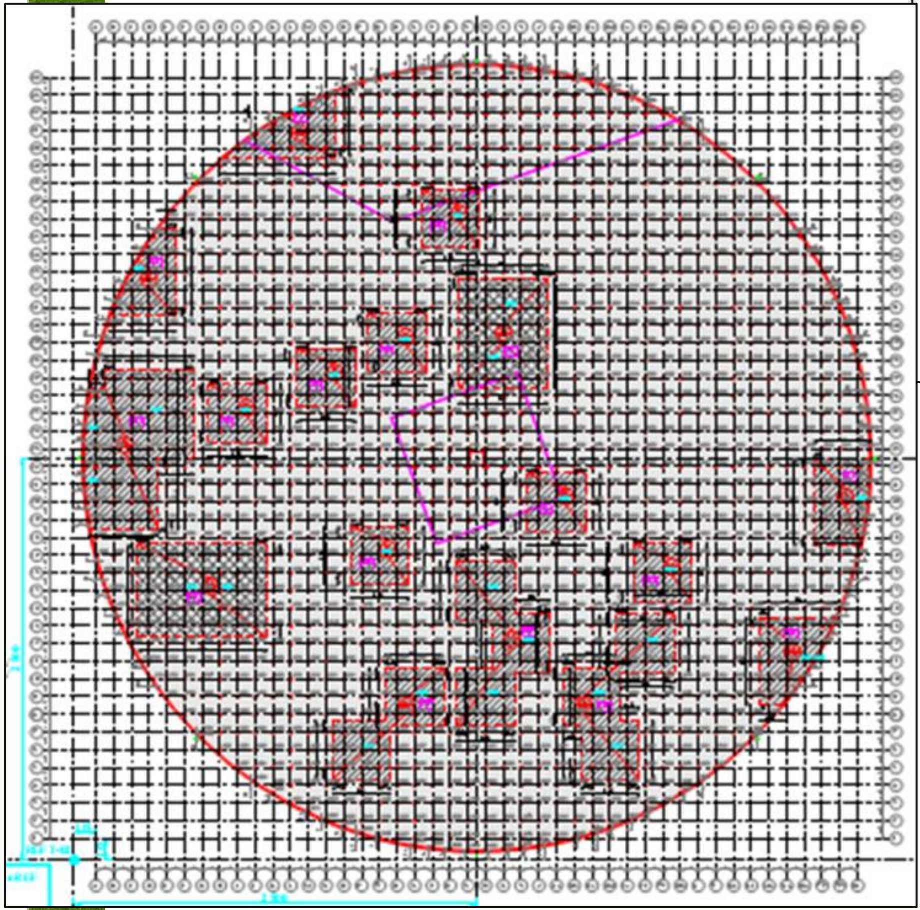
Measured increase of the differential settlements along the perimeter of the tank 02 during the hydro test and its evolution until 01/12/2011





Differential settlements due to local « failure conditions » is obviously asymmetrically distributed, however not following fully the top deficiencies (also deeper defic. are influencing)





The supporting small diameter pilles, being of questionable quality in many zones, « collapsed » one after another, from 17/11/2011 on, creating a domino effect on the overall capacity failure – with some squeezing failure of the soft soil under the tank bottom (cfr the upheave in some zones around).


Connections largely out of line in
x –y –z direction

Starting fissuring failure of tank 01
during the water test filling

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Failure of tank 02 during the water test filling

Failure of tank 02 during the water test filling

First conclusions of the forensic geotechnical engineering work in this case

1. Incorrect pile length mostly under tanks 2 and 3, even with best possible pile type
2. No group effects possible (pile diameter too small, inter-distance too large, pile too slender, pile raft too thin)
3. Different pile lengths under one and the same tank
4. End bearing pile-concept of foundation engineering actually impossible with this pile type; such piles can only serve as ground reinforcement in another design type
5. Unadapted foundation concept

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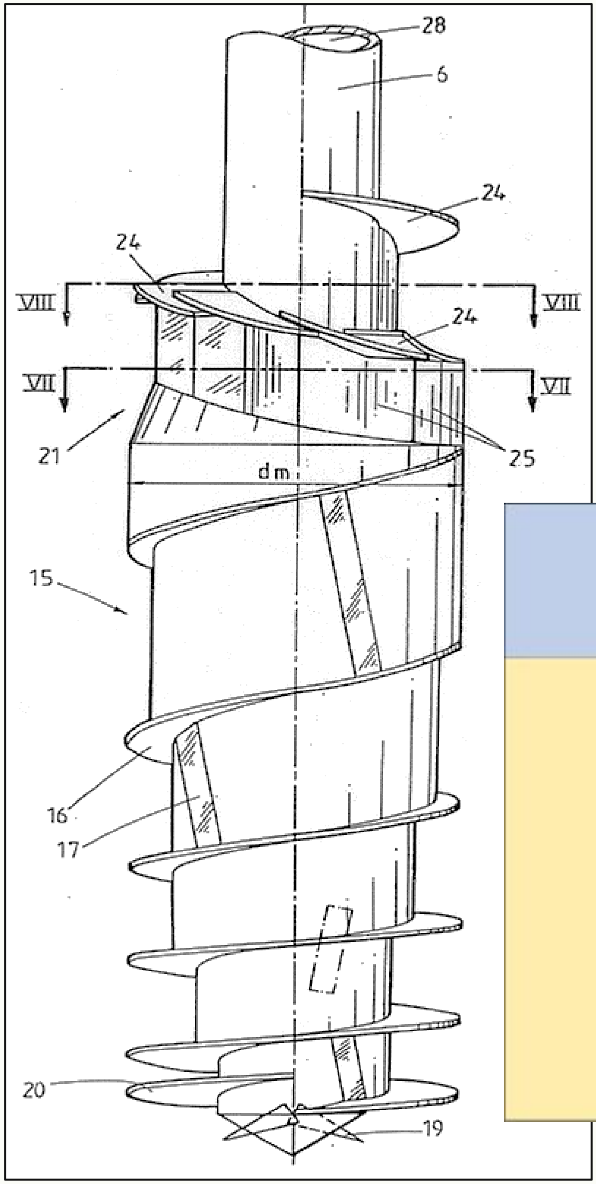
THE NEW FOUNDATION DESIGN

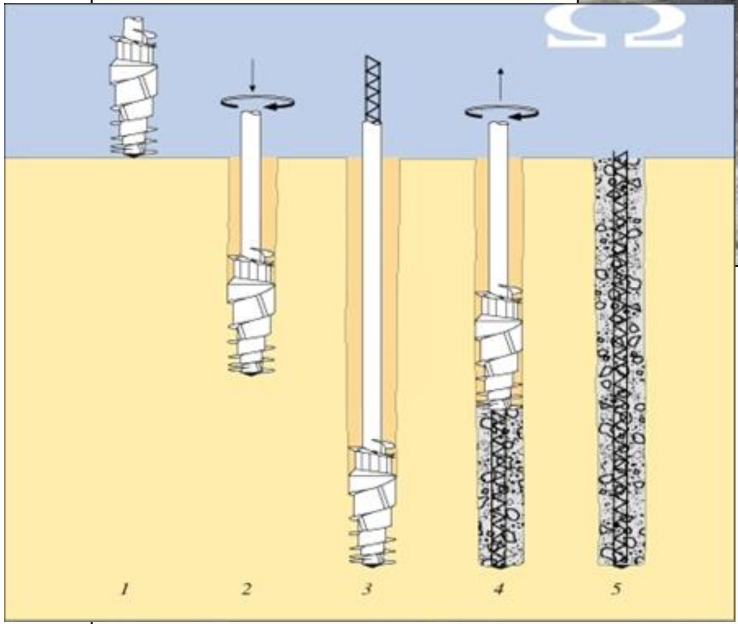
The foundation design

- foundation on a large group of ~ 420 end bearing displacement screw piles per tank, based in the dense tertiary dense sand at ~ 22m of depth
- 460mm diameter Franki Omega pile type designed (at an overall safety factor of >2) to allowable load of 1000 kN/pile (negative skin friction included); pile installation sequence from outer to inner rings

Omega screw pile installation

Displacement Franki Omega- screw pile type





Franki piling

The foundation design

A 600mm thick reinforced concrete raft on top; connecting all displacement reinforced Omega screw piles of 460mm diameter
+ an asphalt layer cover

Pile bearing capacity – Belgian practice as "translated" in the Eurocode National application document

Ultimate base resistance

$$R_{bu} = \beta \cdot \alpha_b \cdot \varepsilon_b \cdot q_{bu}^{(m)} \cdot A_b = \beta \cdot \alpha_b \cdot \varepsilon_b \cdot Q_{bu}$$

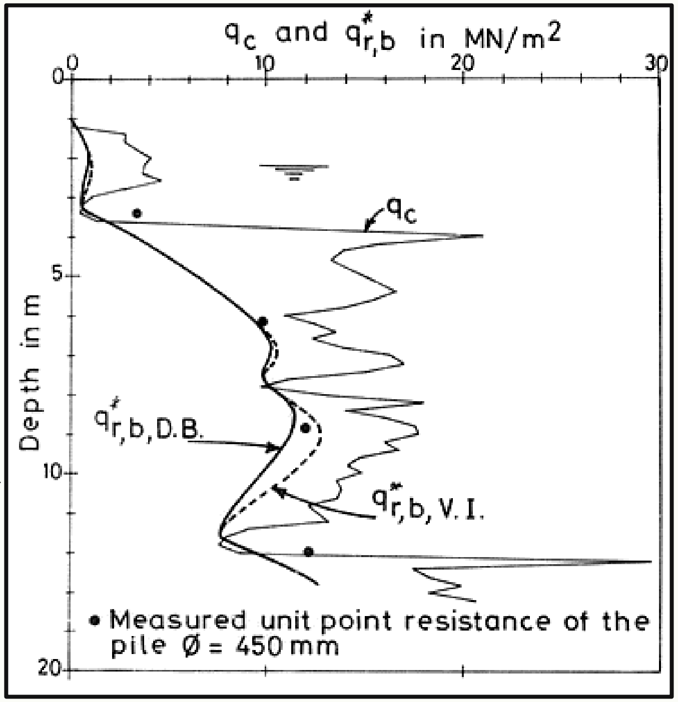
β = 1,0

α_b= installation factor

ε_b = parameter for stiff clays

$q_{bu}^{(m)}$or $q_{r,b}$= ultimate unit pile base resistance
(De Beer method)

A_b= nominal pile base cross section area



Ultimate shaft resistance

$$R_{su} = \xi_f \cdot X_s \cdot \sum H_i \cdot q_{su,i} = \xi_f \cdot X_s \cdot \sum H_i \cdot \eta_p^* \cdot q_{c,i}$$

ξ_f = pile installation factor =1.0

η_p^* = soil parameter, here in the tertiary dense sand: 1/150

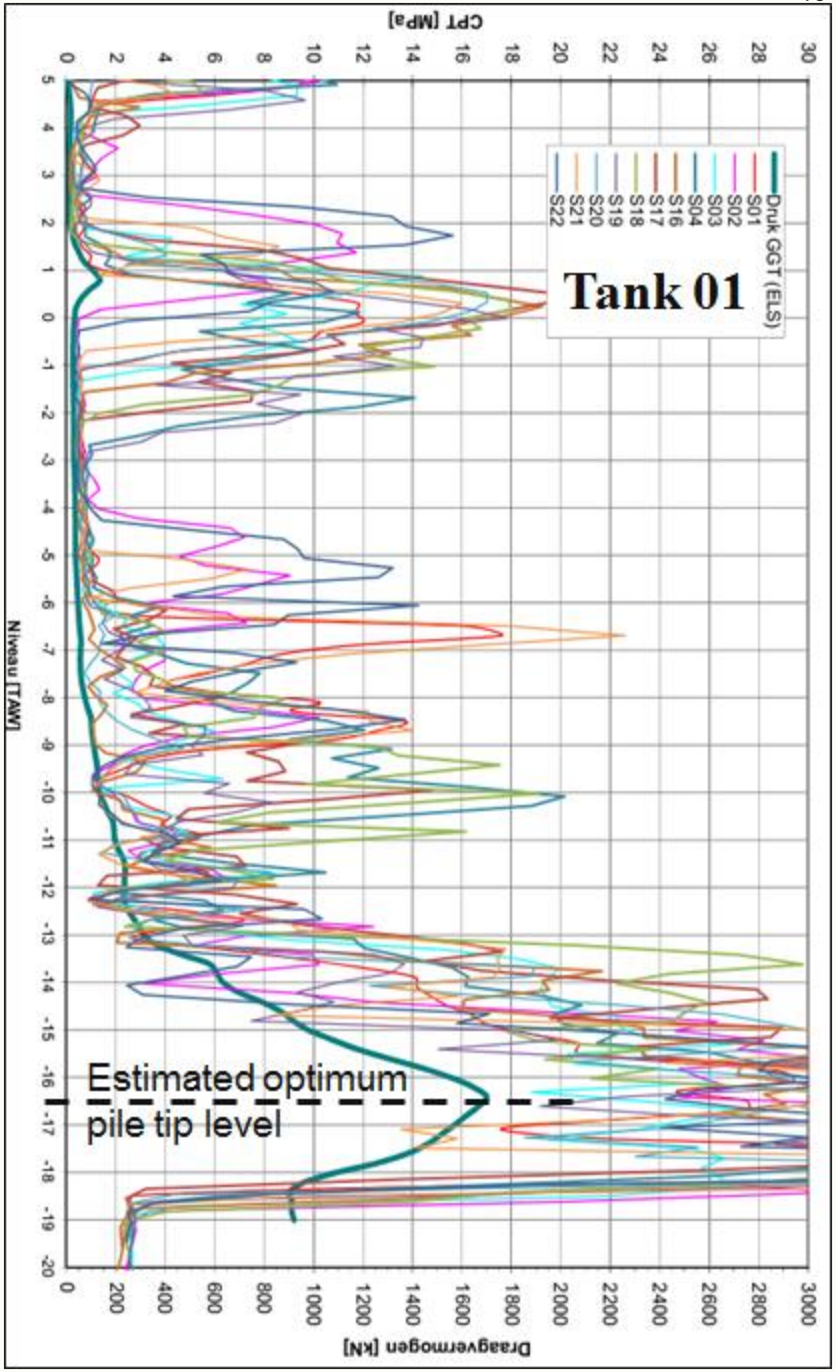
$q_{c,i}$ = cone resistance at the considered depth i

H_i = pile shaft height corresponding to the considered layer i

X_s = pile shaft perimeter

The foundation design

Ultimate unit pile tip resistance of a displacement pile of 460mm diameter, as a function of depth, using all CPT results relevant to tank 01 using the adapted Van Impe - De Beer method (ISC Orlando-1986)



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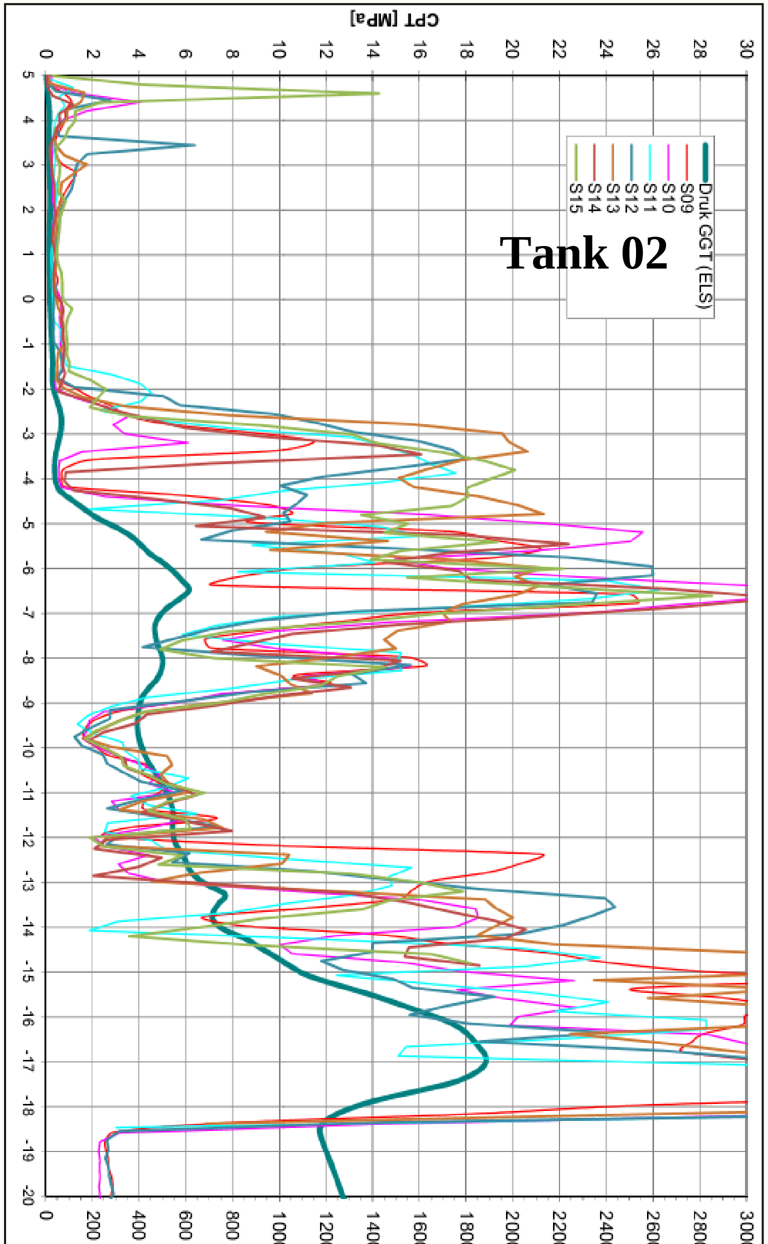
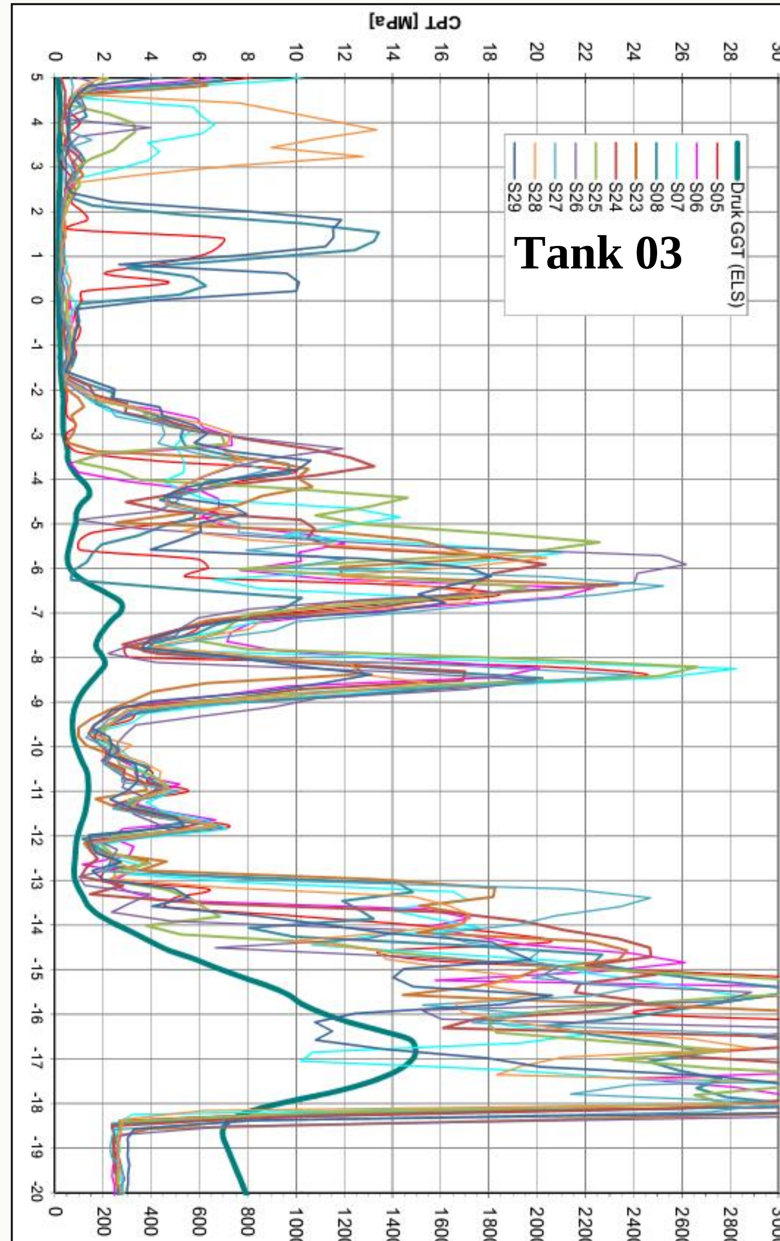
The foundation design





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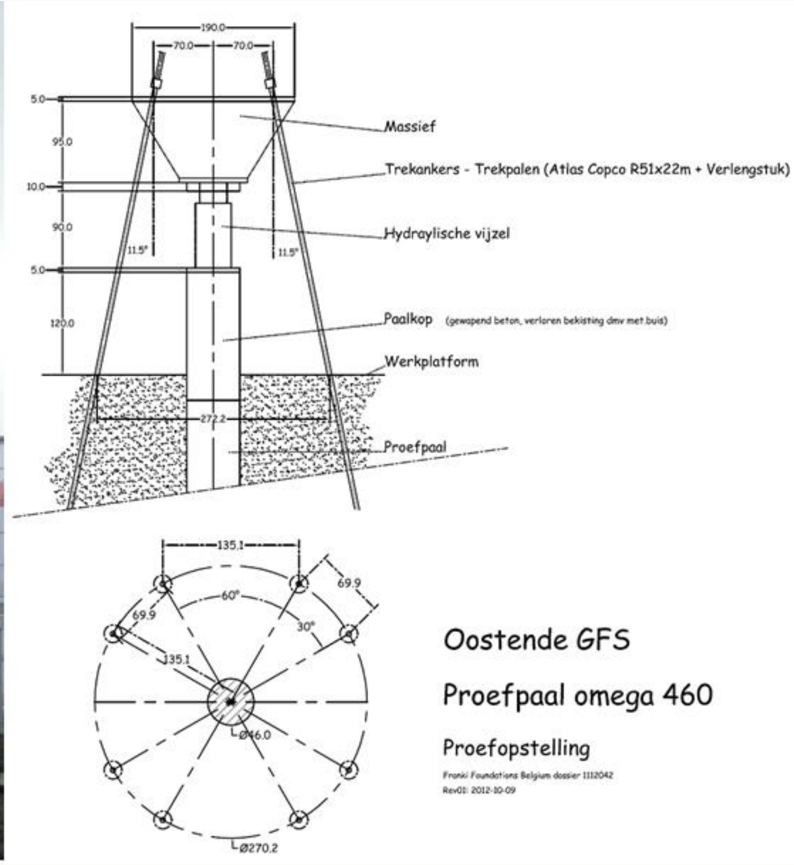
AVAILABLE INSTRUMENTED SINGLE PILE TEST LOAD

Instrumented single pile test load

Test pile for fully (extensometer) instrumented pile test until complete ‘failure’ was being set-up in the neighbourhood of the tanks 2 and 3

Instrumented single pile test load

Fully instrumented test pile (to be loaded up to a pile base settlement of 10% of pile diameter)



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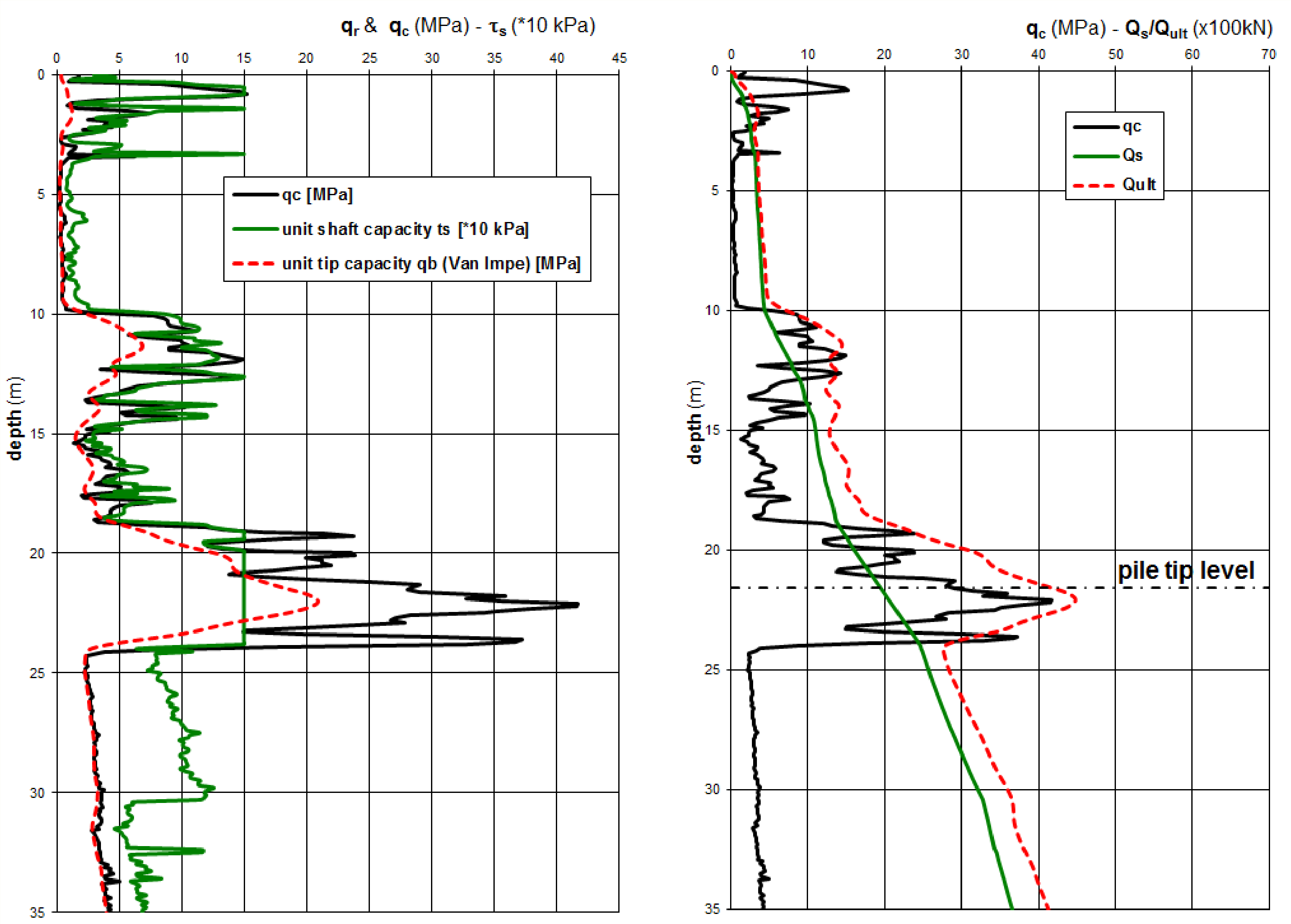
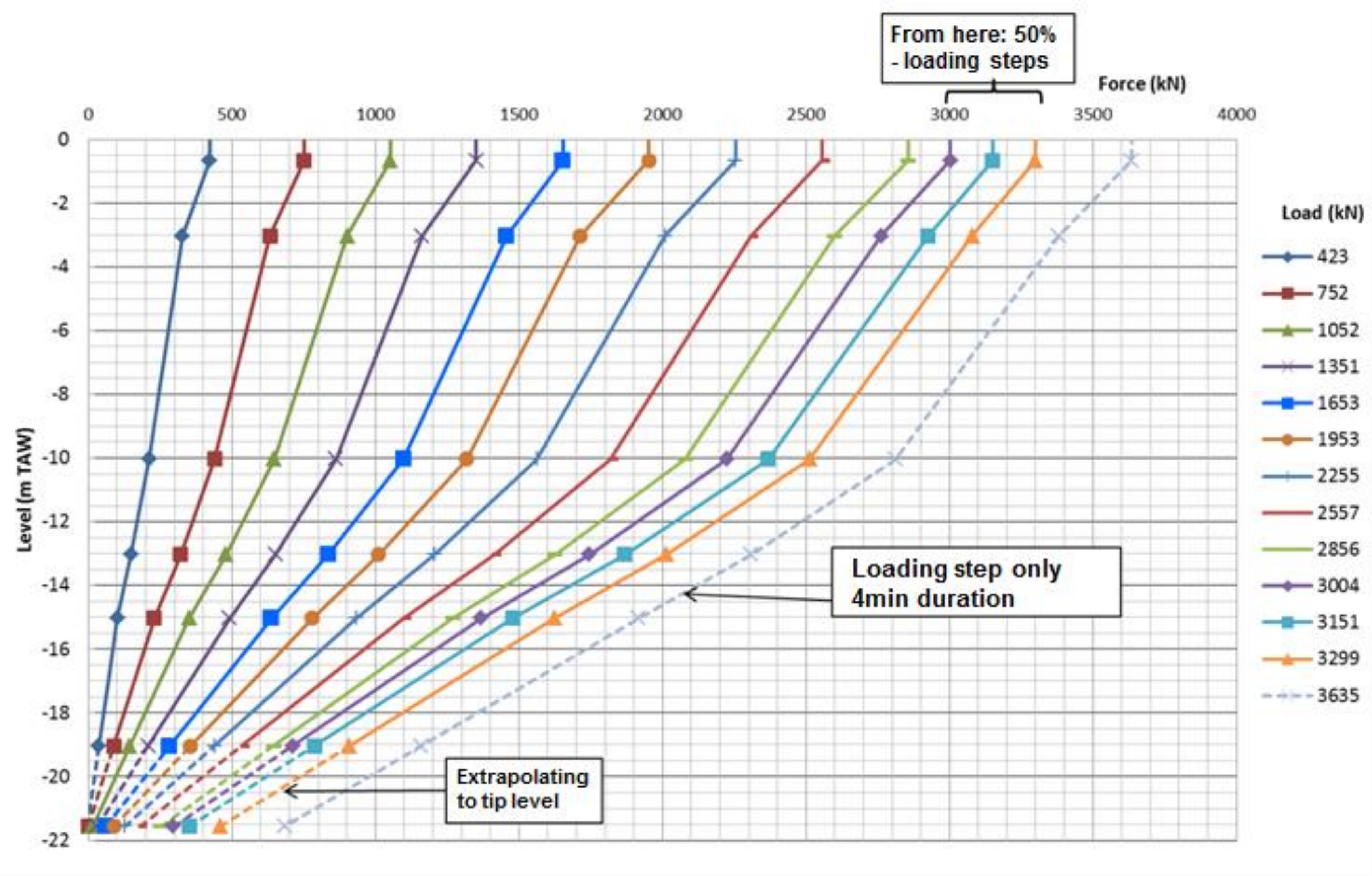
Instrumented single pile test load



On the left: the unit pile tip and unit pile shaft values; on the right: the total ultimate pile capacity Q_{ult} vs depth at the location of the test pile axis

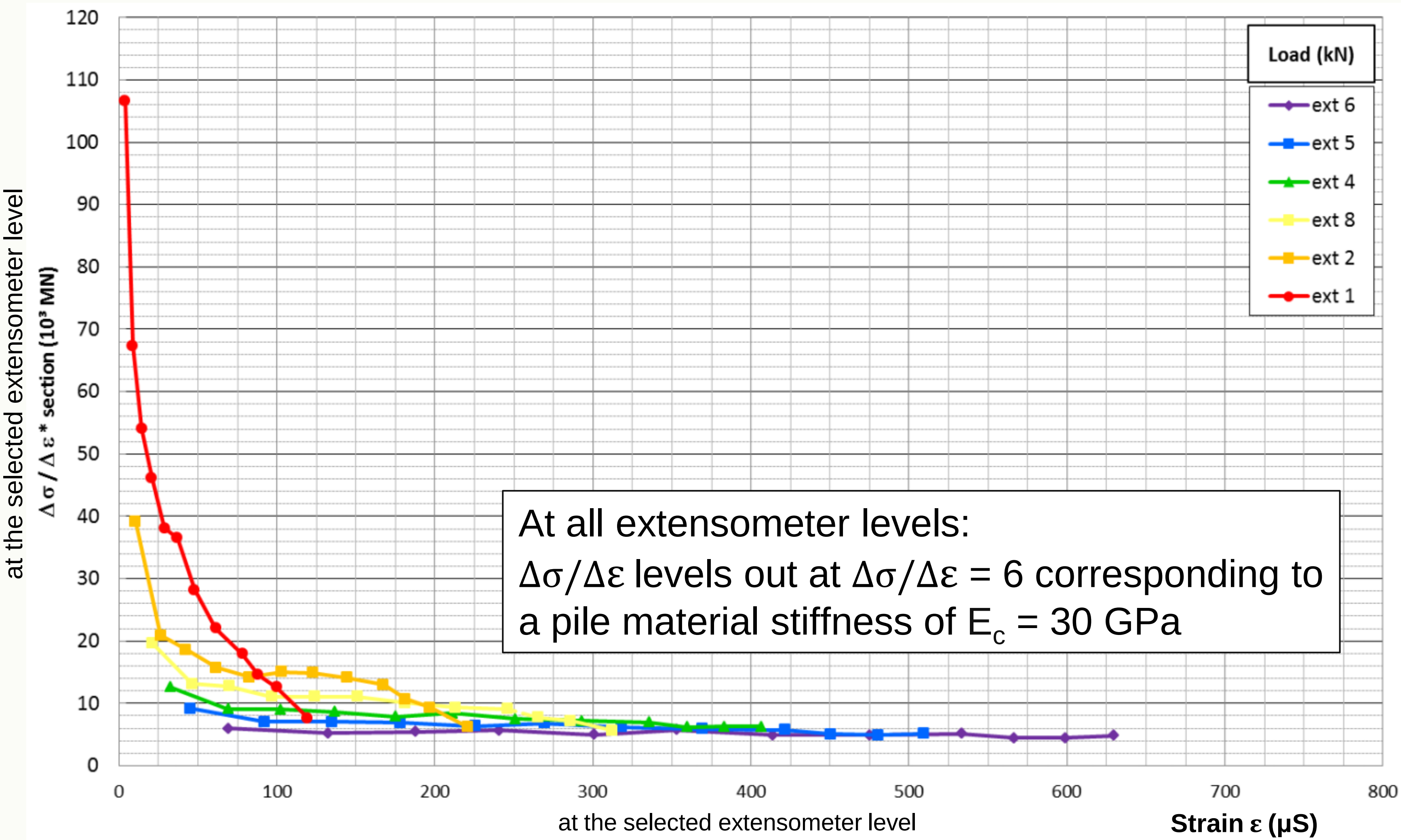
Instrumented single pile test load

Pile head loads, as transferred to the various pile section levels (extensometer levels)



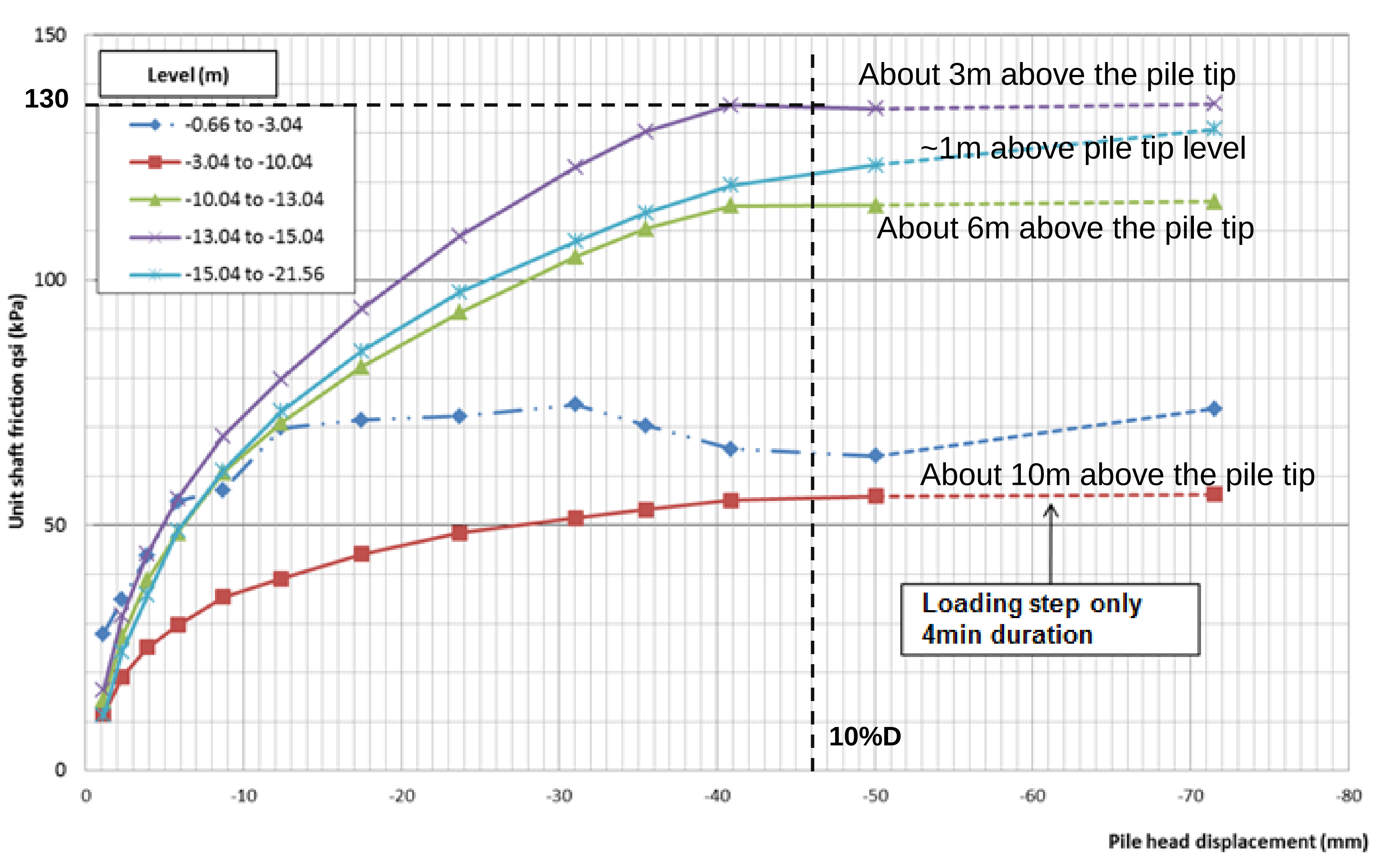
Instrumented single pile test load

Stiffness evaluation of the test pile material during loading – at the various levels of the extensometers (adapted Fellenius 2001 method)



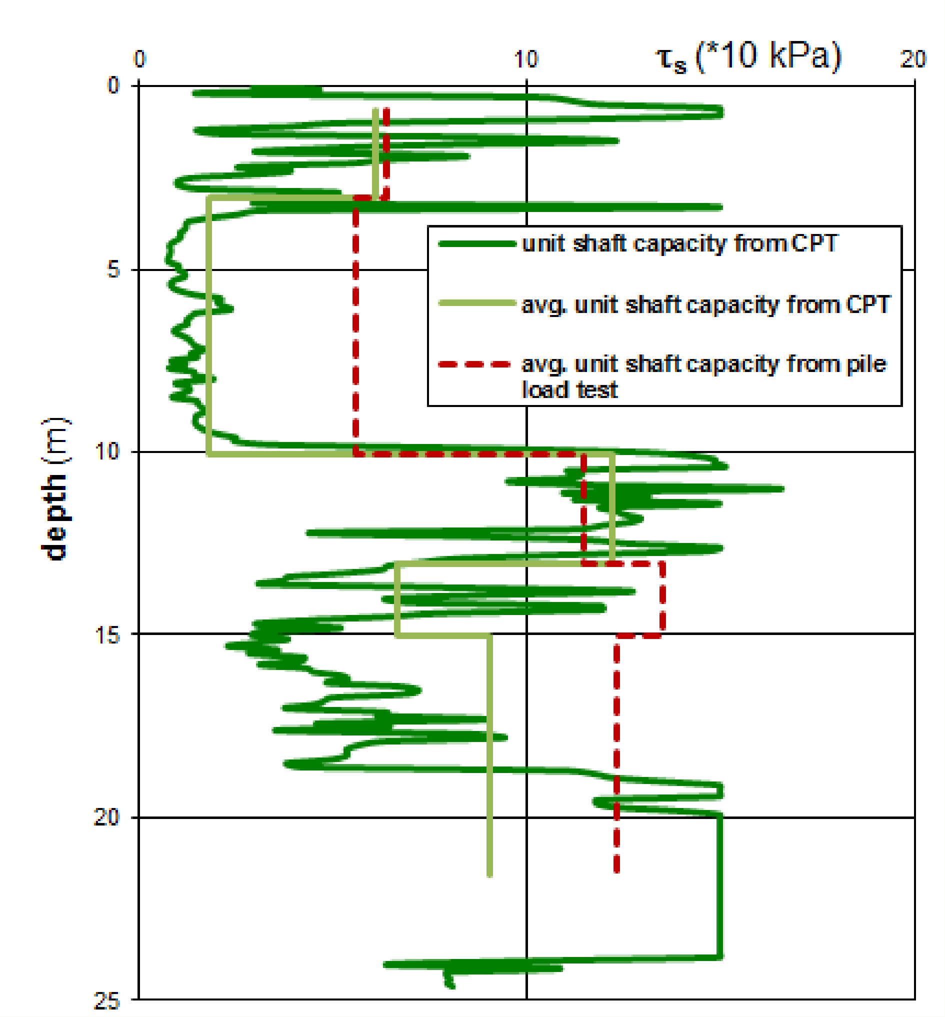
Instrumented single pile test load

Mobilizing the pile shaft capacity at increasing pile deformations, in the sections in between the extensometer levels.

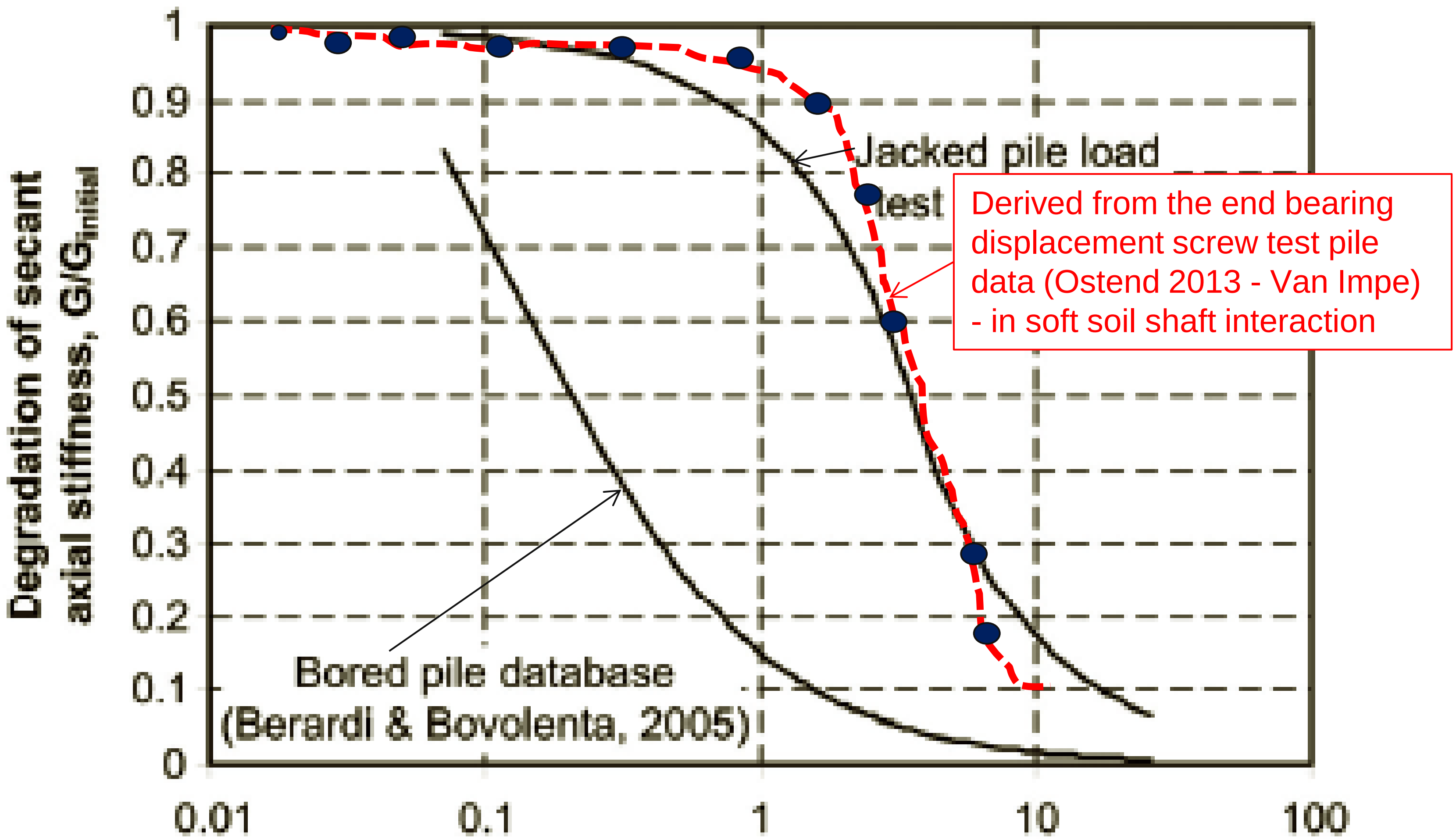


Instrumented single pile test load

Comparison of the mobilized unit shaft friction along the test pile shaft: CPT based predicted mobilized friction versus the measured values during the load test



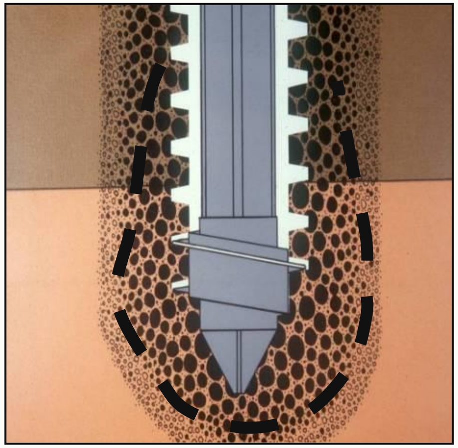
Strain level dependency of the interaction shear stiffness



$$G = G_0 \cdot \left[1 - 0,95 \cdot \left(\frac{\tau_s}{\tau_{s,max}}\right)^{0,3}\right]$$

The strain level dependency varies with installation method because of the very different nature of failure pattern

Impact of shear surface curvature

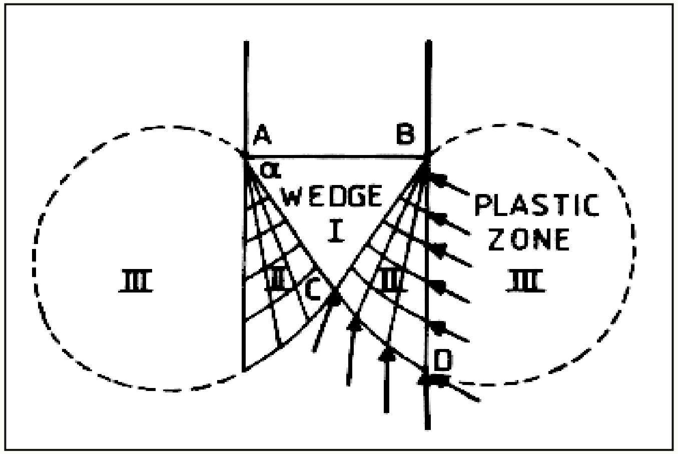


Displacement screw piles

- less curvature of failure surface
- so, less progressive failure

Driven piles

- very high level of curvature
- very progressive failure
- larger interaction displacement allowable up to failure



Impact of shear surface curvature

Displacement screw piles do show a by far less curved potential shear surface in the same type of soil, due to a less pronounced tip-soil interaction + a greater (installation governed) disturbed “soil paste zone” along the shaft (and so an easier and more pronounced upward soil movement along the tip/shaft)

Impact of shear surface curvature

This means that, as compared to driven piles the progressive nature of the “failure” is much less outspoken for screw piles.

Another “failure criterion” is imposing itself , since for screw piles (outspokenly in case of a non-displacement type), the “failure” is of a more “brittle” nature.

- for example : s_r occurs at even 2.5%diam instead of the 10%diam

TANK SETTLEMENT BEHAVIOUR

Initial settlement estimation

Initial settlement estimation for a single loaded tank using method of the **equivalent raft** and soil parameters out of CPT

Depth (m)	q_c (MPa)	E_s (**) (MPa)	M (*) (MPa)
0.66	4.60	34	40
3	0.66	4	5
10	10.07	77	97
14.5	5.28	63	66
18	15.80	118	143
21.56	35.68	225	282
22.48	26.18	170	209
24	3.3	27	27
35	5.23	51	51

…NO data available below 35m depth !!!

(*) $$M = \alpha_M \cdot (q_t - \sigma_{v0})$$

where α_M varies with Q_t as suggested by Robertson (2009):

when $I_c > 2.2$ (fine-grained soils)use:
$\alpha_M = Q_t$ when $Q_t < 14$ and $\alpha_M = 14$ when $Q_t > 14$;

when $I_c < 2.2$ (coarse-grained soils):

$$\alpha_M = 0.0188 \cdot \left[10^{0.55 \cdot I_c + 1.68}\right]$$

(**) $$E_s = \alpha_E \cdot (q_t - \sigma_{v0})$$

where

$$\alpha_E = 0.015 \cdot \left[10^{0.55 \cdot I_c + 1.68}\right]$$

Initial settlement estimation

The method predicted an upper level of the overall elastic oil tank deformation of 27 mm, to be increased by an upper level of the soil plastic deformations of about 110 mm, due to the consolidation effects of the relevant interfering layers into the foundation engineering problem.

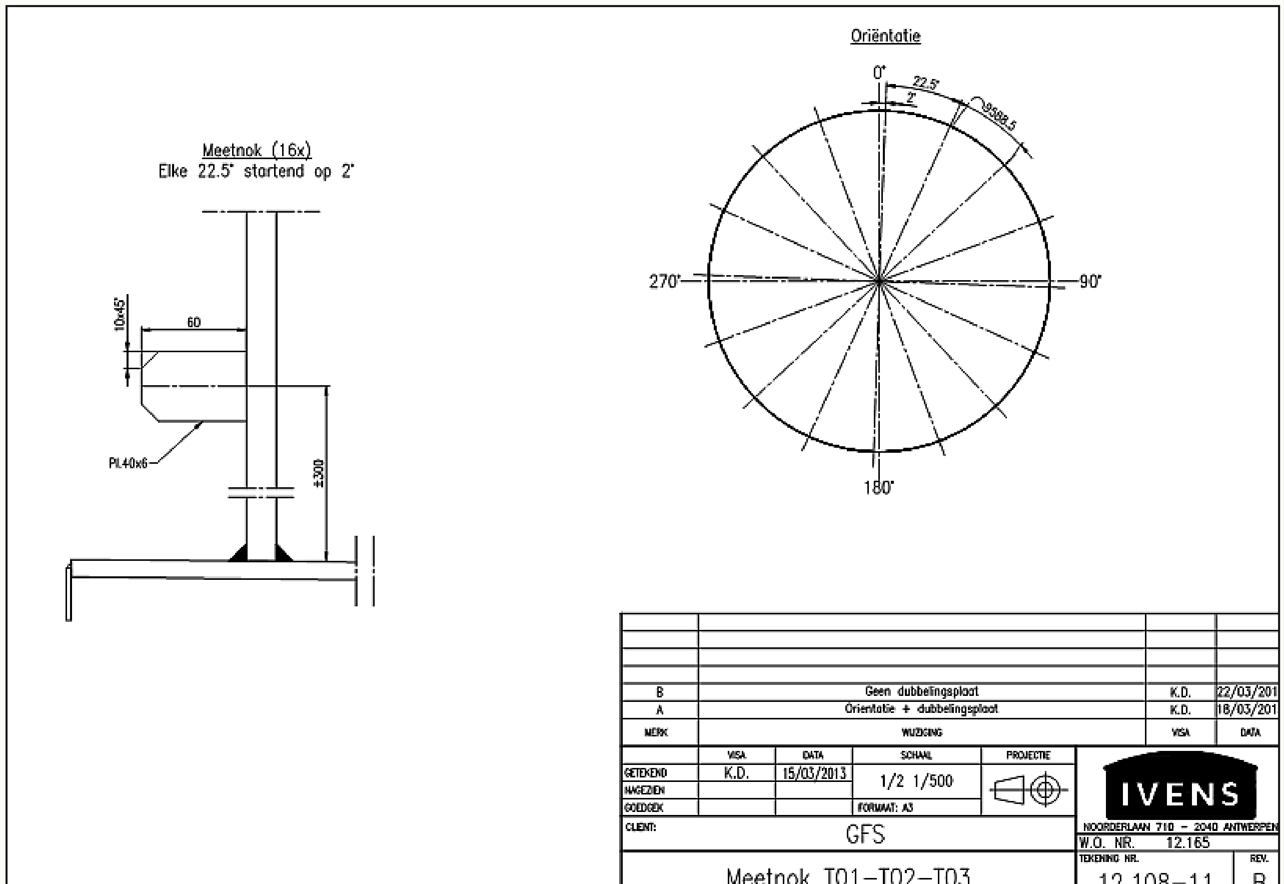
HOWEVER,

- such initial prediction is very much depending on the actual compressibility of the unknown clayey layers (below 35 m depth)
- interaction of the 3 tanks' loading will lead unavoidably also to settlement trough

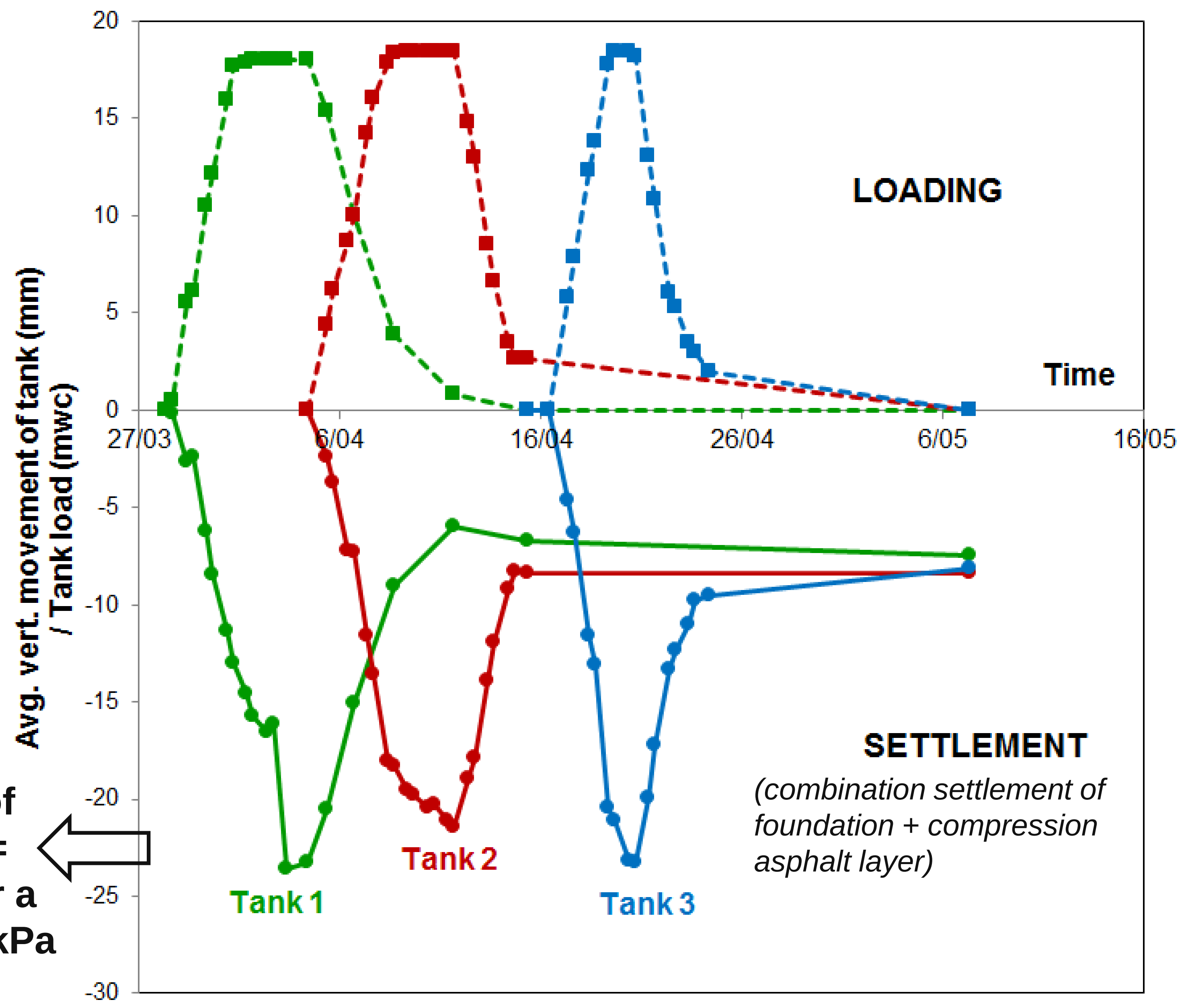
⇨ settlement **monitoring** of tanks was deemed to be essential

Monitoring tank settlement

16 points along tank’s perimeter, equally divided at a center angle of 22.5° from each other

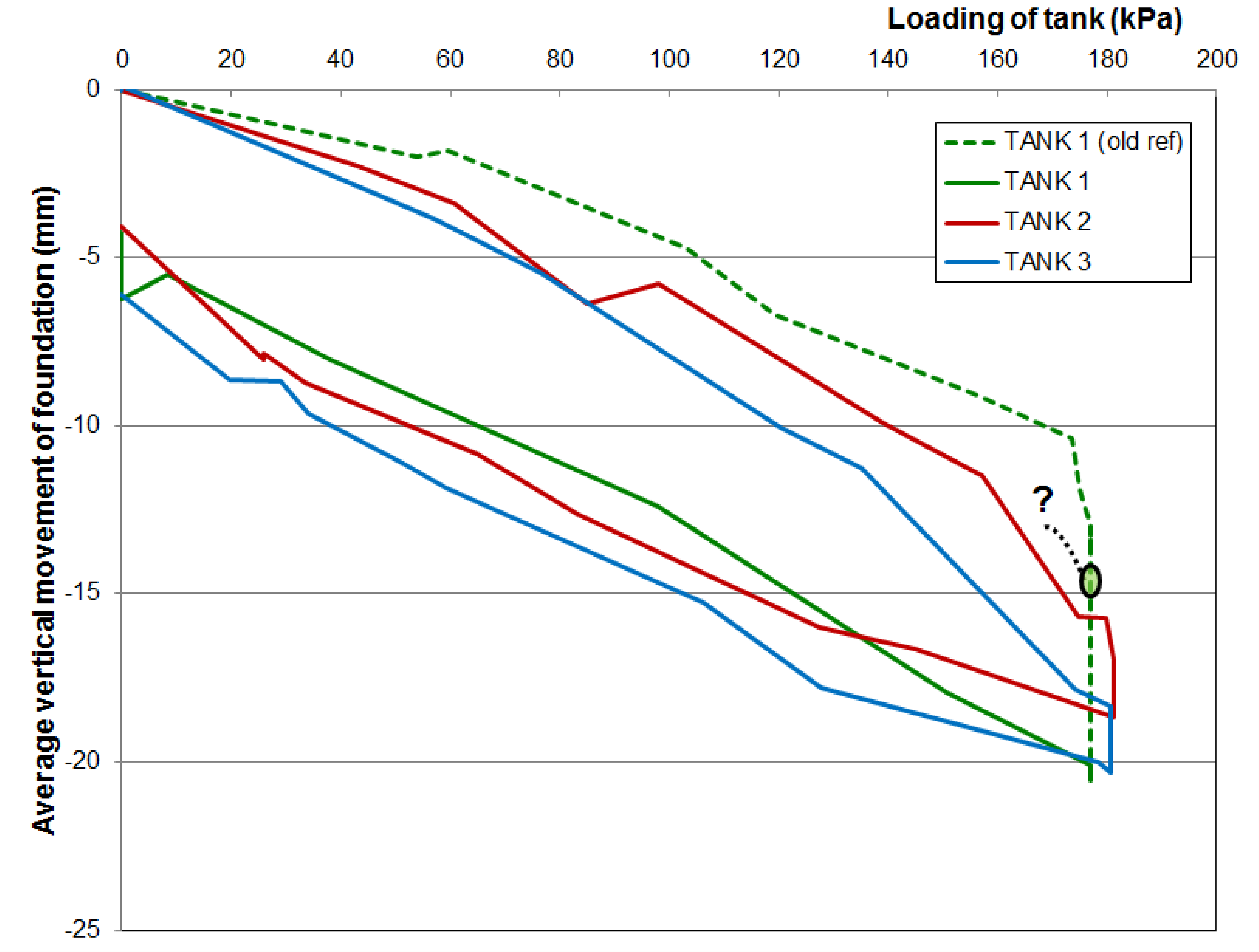


Tank settlement during hydro-test



settlement of foundation = 21mm under a load of 180 kPa

Full load-settlement curves at hydro test with unloading



Tank settlement during hydro-test

- As each tank was tested separately and for a very short period, the impact of the load is presumably limited to the immediate response of the stiff sand layer and the upper part of the underlying silty clay layer

- no real interaction between tanks

  = limited tilt of the tanks (2-3mm)

- residual average deformation at the end of hydro-test

  ~ 8mm for all tanks

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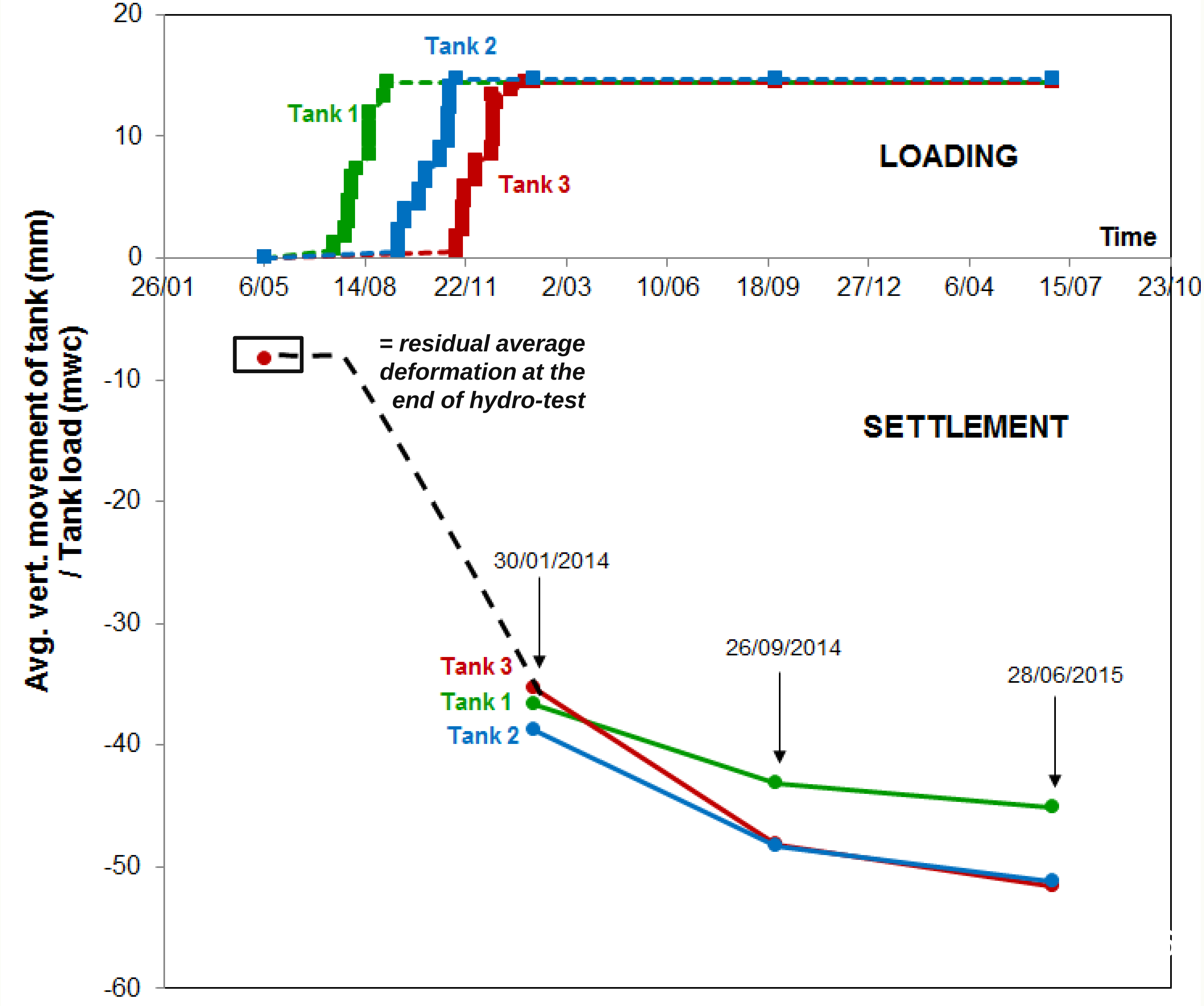
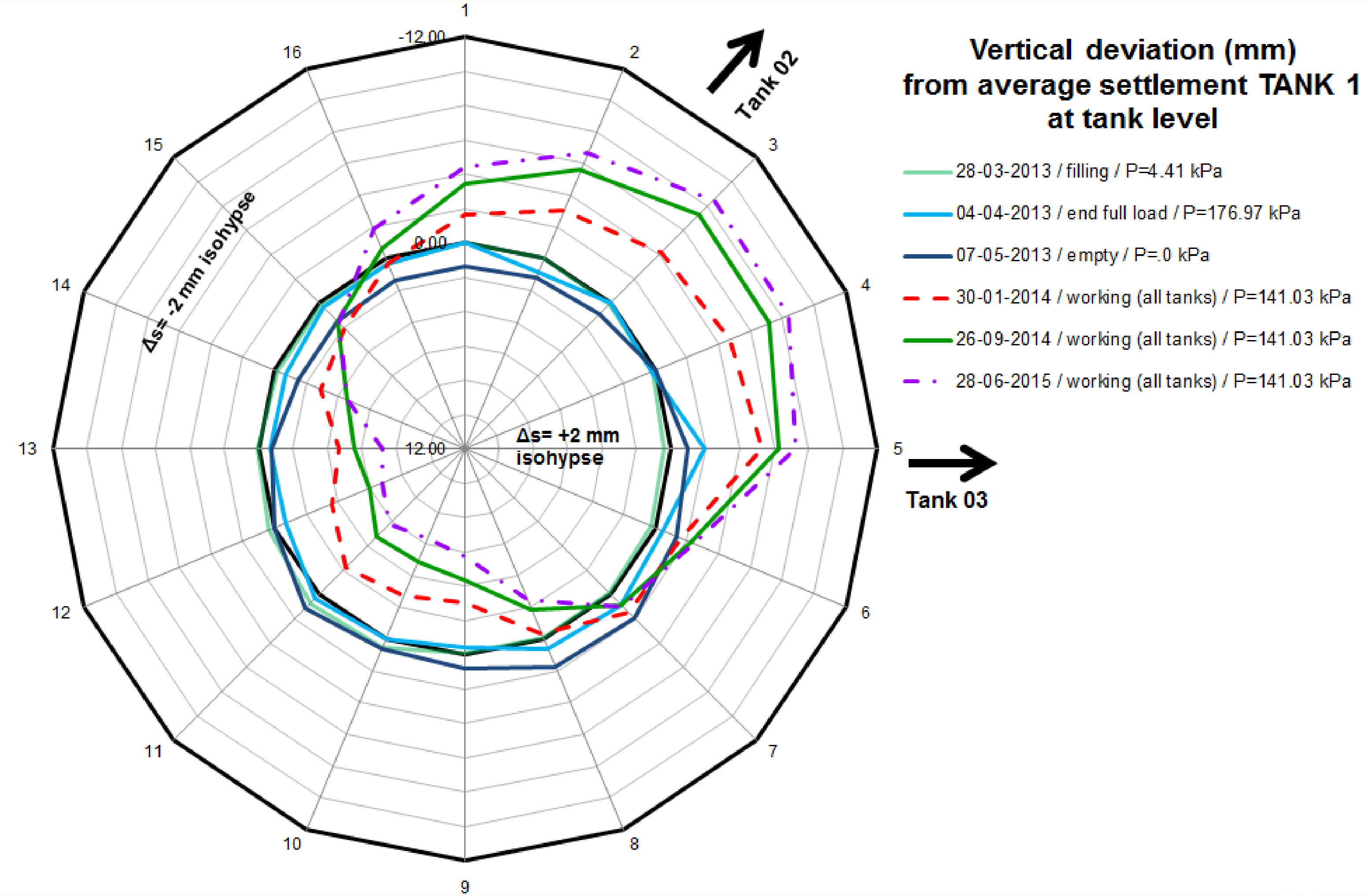
Tank settlement during operation



Vertical deviation from average settlement (mm) of Tank 1 - during hydro test and during operation



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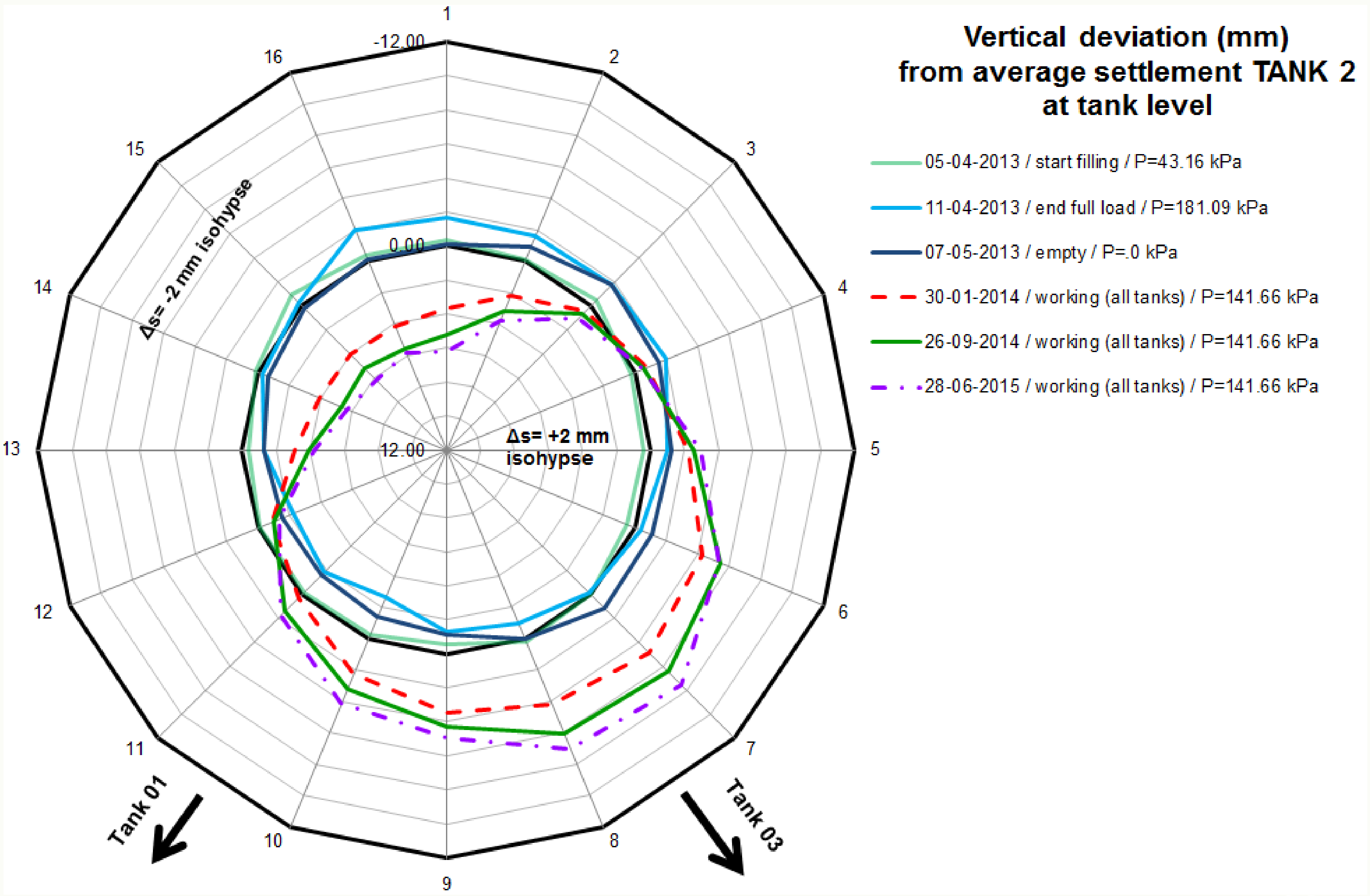
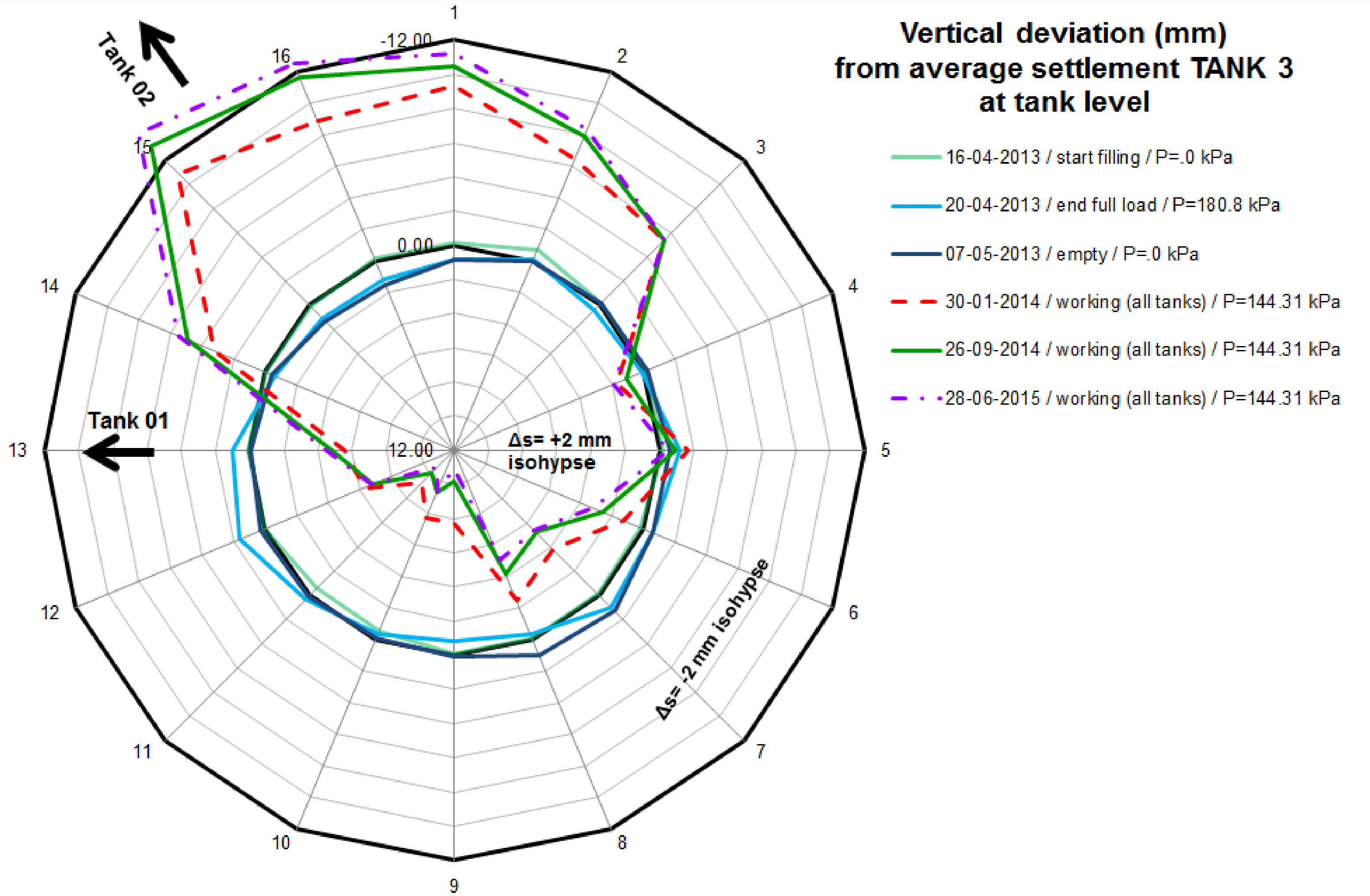
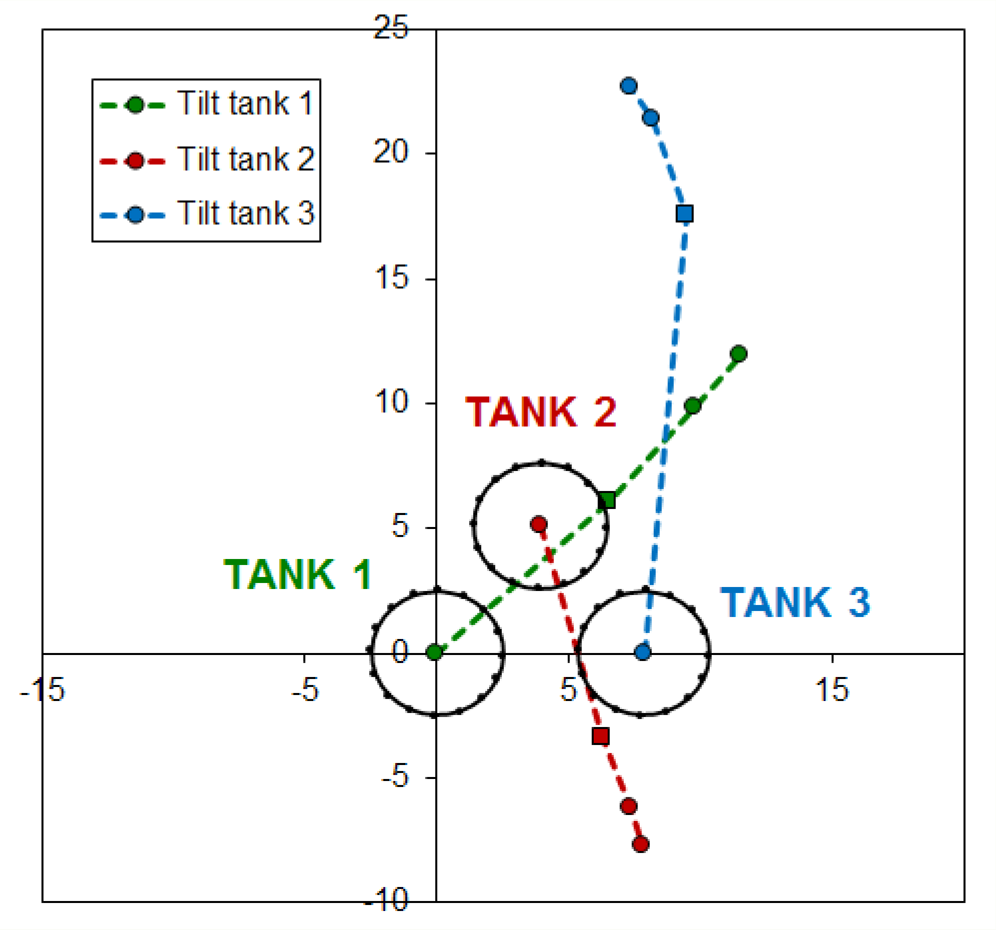
Vertical deviation from average settlement (mm) of Tank 2 - during hydro test and during operation



Vertical deviation from average settlement (mm) of Tank 3 - during hydro test and during operation



Direction and size (mm) of tilt of the tanks during operation



Both **tank 1** and **2** exhibit about 15 mm of nearly perfectly planar tilt (0.00031 m/m) towards the central area in-between the tanks, while **tank 3** tilts almost directly north for about 22 mm (0.00046 m/m)

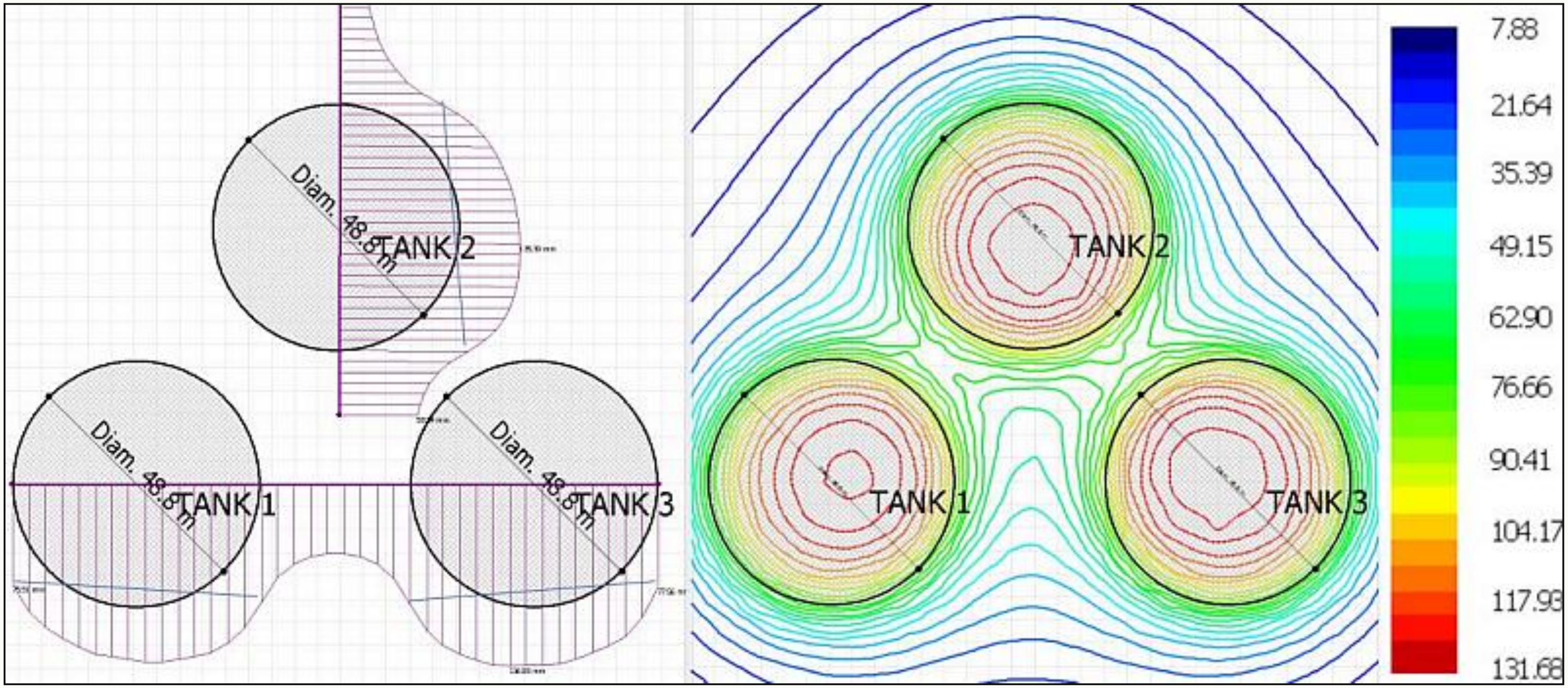
!!!! local subsoil heterogeneities below tank 3

Tank settlement during operation

- additional average settlement at this point has reached values of 34 to 40mm
- higher value of the tilt (compared to hydro-test) as expected due to interaction of different loads
- values of average settlement, tilt and distortion still far below critical values

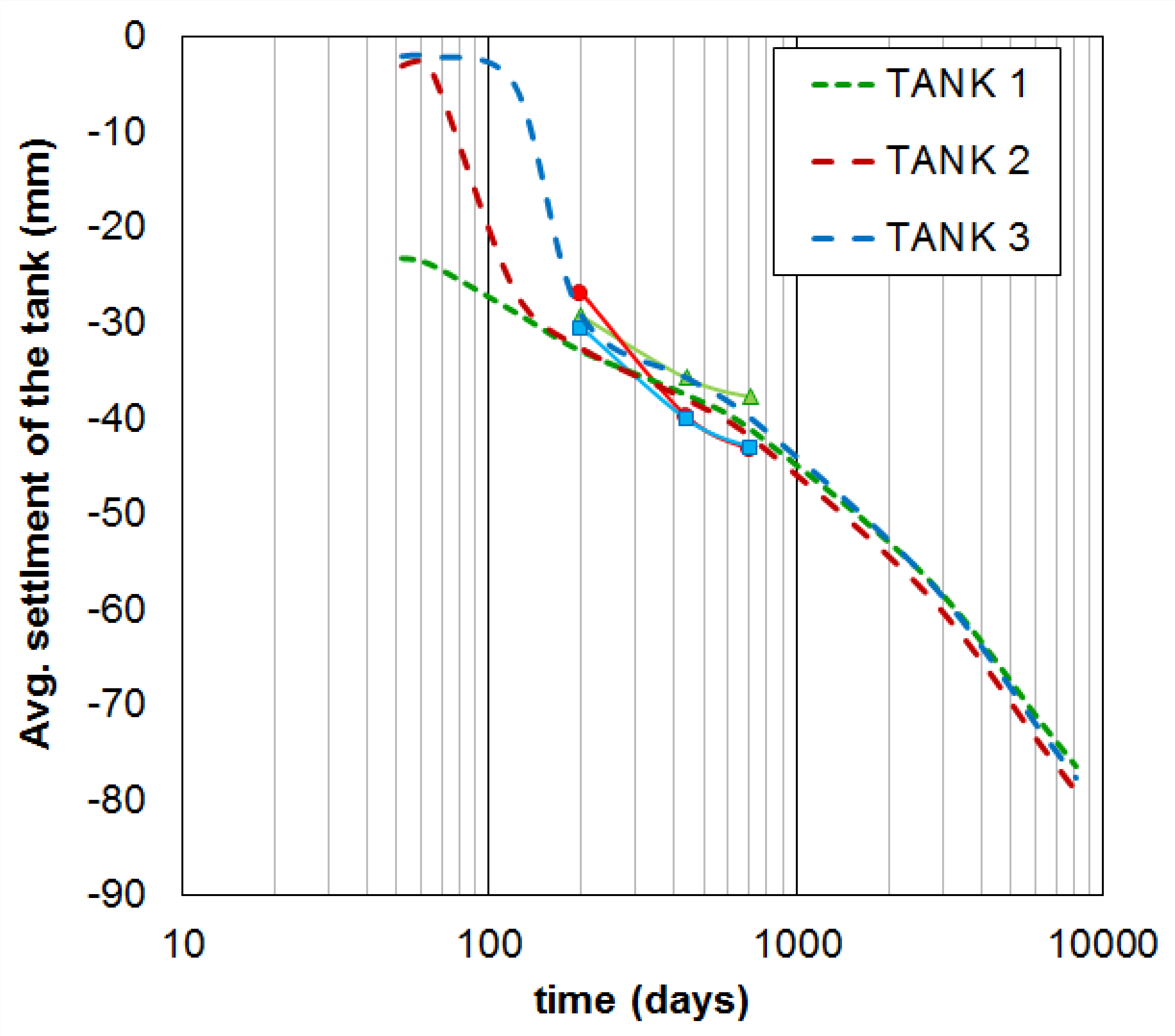
Analysis of the time settlement behavior

Based on the compressibility parameters from CPT + hydro-testing, a single value of c_v for the silty clay was found to get the best fit between predicted and measured average settlements



(SteinP 3DT program)

Analysis of the time settlement behavior –
Predicted (lines) vs measured (dots) average settlement



Fitting done on the basis of c_v/d^2 (also the drainage path length is indeed unknown!!!)

This leads so far to a value of the time factor $c_v/d^2 = 0.0023\ \text{month}^{-1}$

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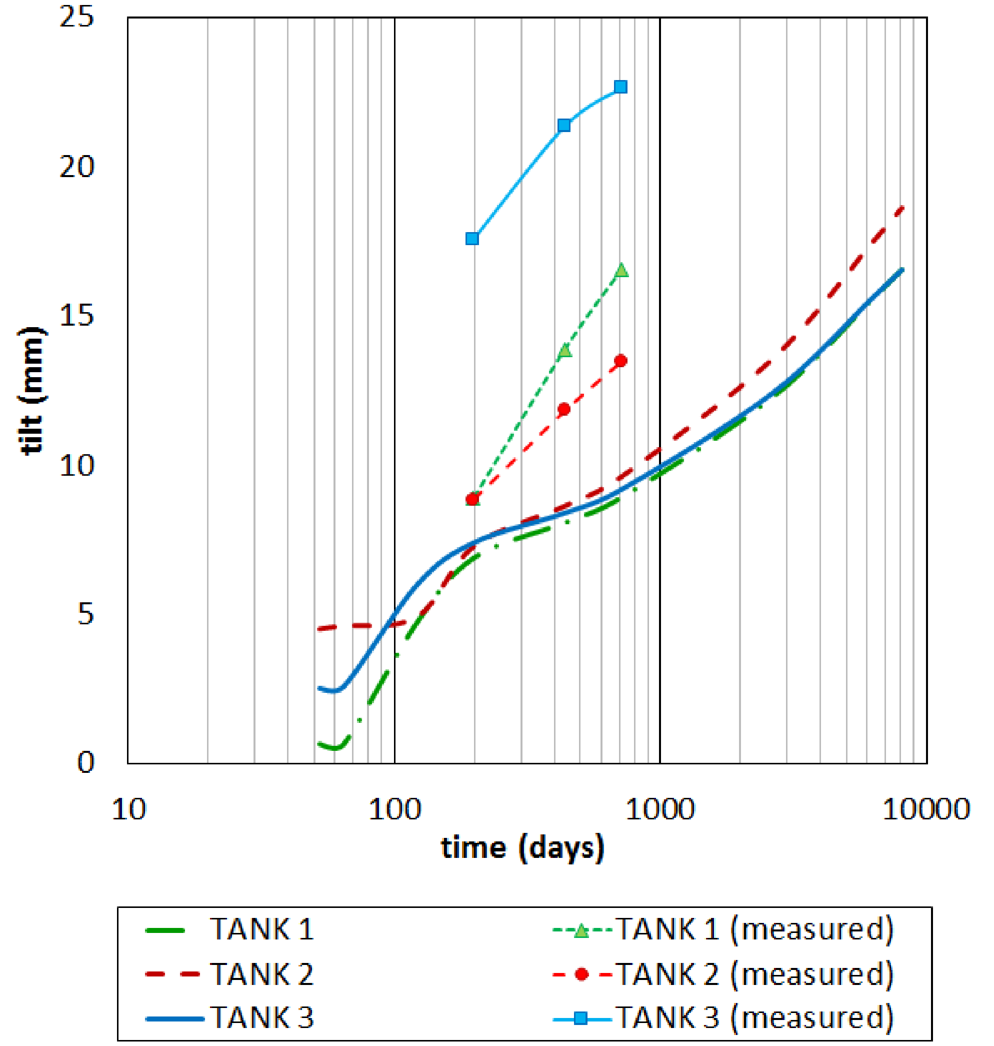
Analysis of the time settlement behavior –

Predicted (lines) vs measured (dots) tilt of the tanks under operational load



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CONCLUSIONS

Conclusions (1)

- Deformation behavior is a governing factor for foundation design.
- Selection of foundation concept and subsequent design should be made accordingly.
- In this case a stiff raft and displacement screw piles were an adequate solution

Conclusions (2)

- The pile load test has confirmed the load-capacity of the Omega displacement single pile.
- Prediction of the elastic settlement of the pile group gives ~ 27 mm, which is acceptable and confirmed by the settlement measured. Moreover, the stiff raft will force the interaction between the individual piles, leading to minimal differential settlements.
- Consolidation settlements are estimated at 110 mm. + the creep (secondary cons. settl.)

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Conclusions (3)

- Hydro test data illustrate that the end bearing displacement screw pile group underneath each 3 of the tanks can guarantee quite uniform settlement of each tank to a very similar level
- Due to the large scale of the construction, the influence depth is considerably larger then the extent of the soil investigation and monitoring was deemed essential
- Additional measurements will allow for further optimization of the model to better extrapolate the long term behavior

Thank you
for your attention