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Geotechnical constructions analysis based upon static load test curve

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Introduction

- In 2010 Meyer and Kowalow presented a method of full description of static load test curve, called M-K curve (Meyer, Kowalow 2010).
- M-K curve has two asymptotes and their location can be described as follows:

$$\boldsymbol{s(N) = C \cdot N_{gr} \cdot \frac{\left(1 - \frac{N}{N_{gr}}\right)^{-\kappa} - 1}{\kappa}}$$

$$\lim_{N \to 0} s(N) = C \cdot N; \quad \lim_{N \to N_{gr}} s(N) = \infty$$

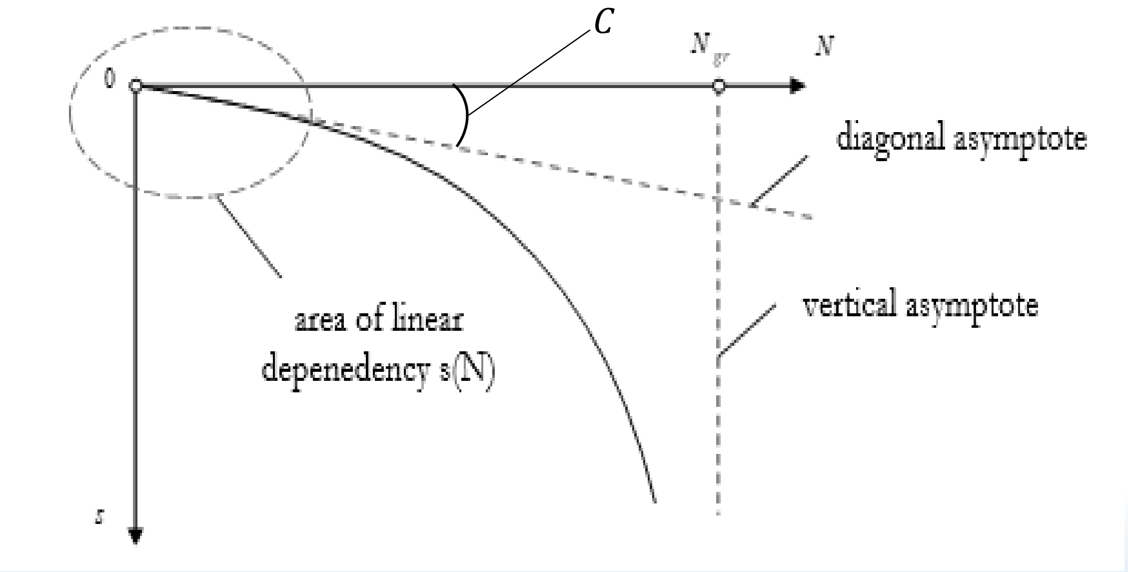


Fig. 1 Asympototes limiting curve $s(N)$ for static load test of a pile (Meyer 2017).

$\boldsymbol{C}$ – parameter that represents inversed aggregated Winkler's modulus, in Fig. 1 it is diagonal asymptote of M-K curve

$\boldsymbol{N_{gr}}$ – limit load capacity of a pile, in Fig. 1 it is vertical asymptote of M-K curve

$\boldsymbol{\kappa}$ – parameter that indciates proporotion between toe and skin resistance of a pile.

Introduction

M-K curve parameters:

$N_{gr2} = 12500\ [kN]$

$C_2 = 7{,}46 \cdot 10^{-3} \left[\frac{m}{MN}\right]$

$\kappa_2 = 0{,}8\ [-]$

N_i $[kN]$	$s_{i,mes}$ $[mm]$	$s_{i,calc}$ $[mm]$
1600	0,7	1,350
2200	1,3	1,952
3000	2,2	2,862
3600	3,1	3,640
4400	4,3	4,837
5000	5,1	5,884
5500	6,3	6,879
5800	7,9	7,540
6300	9,1	8,769
6500	10	9,312
7900	15,1	14,278
8900	20	19,897
9400	24,8	23,906
9900	30	29,279
10300	34,9	35,133

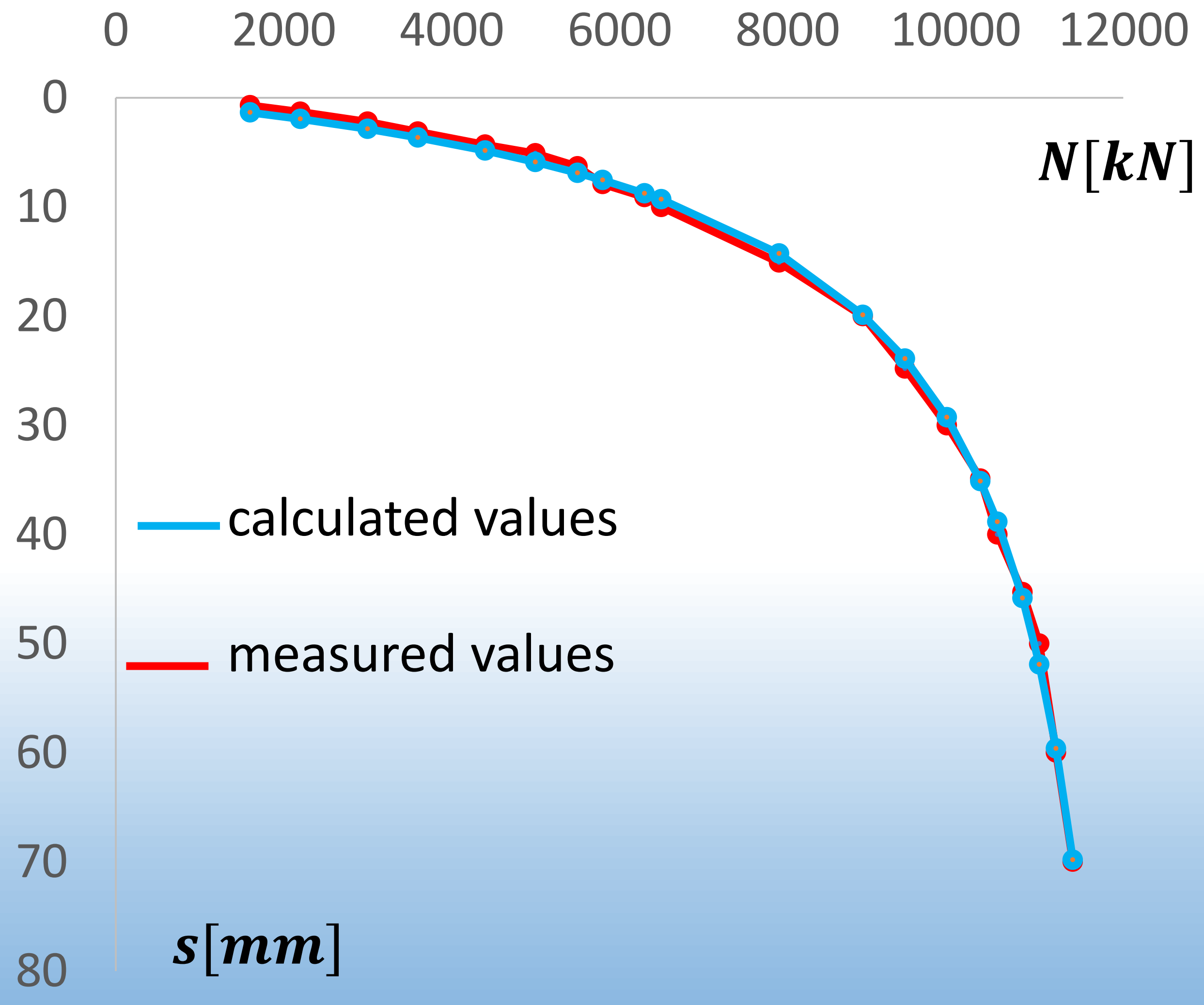


Fig.3 Comparision between settelment values obtained as a result of static load test and values caluculated using M-K curve equation.

Introduction

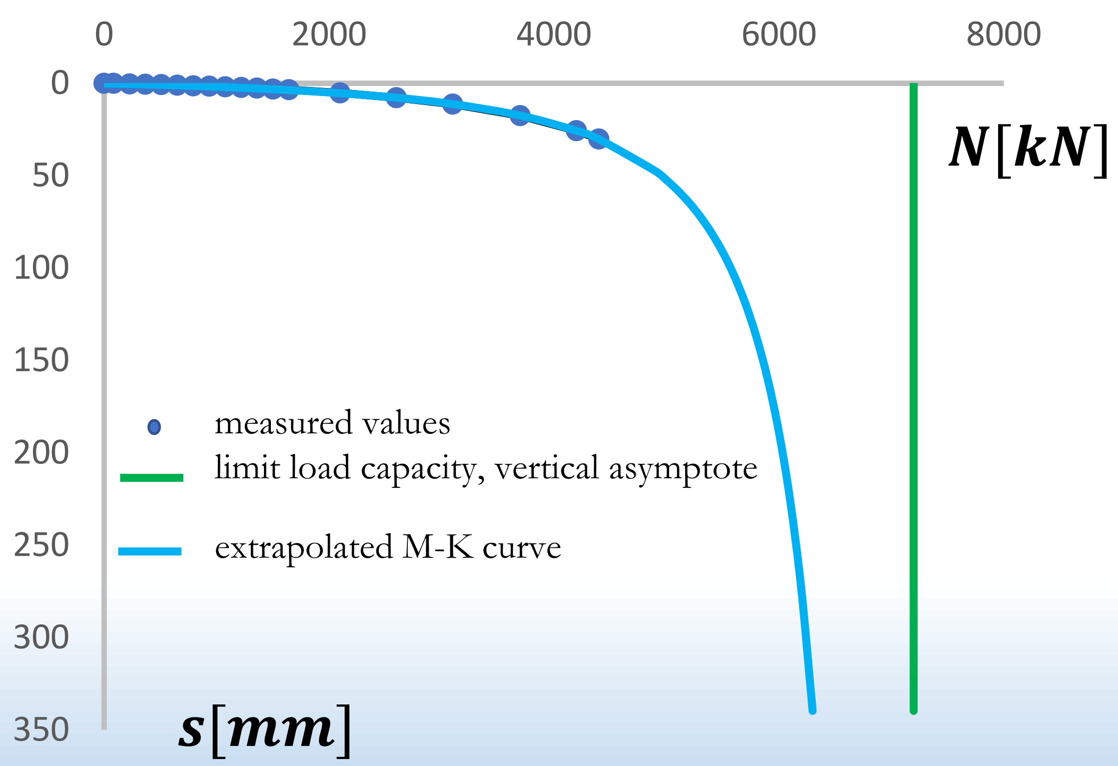


Fig. 3 Extrapolation of static load tests results using M-K curve.

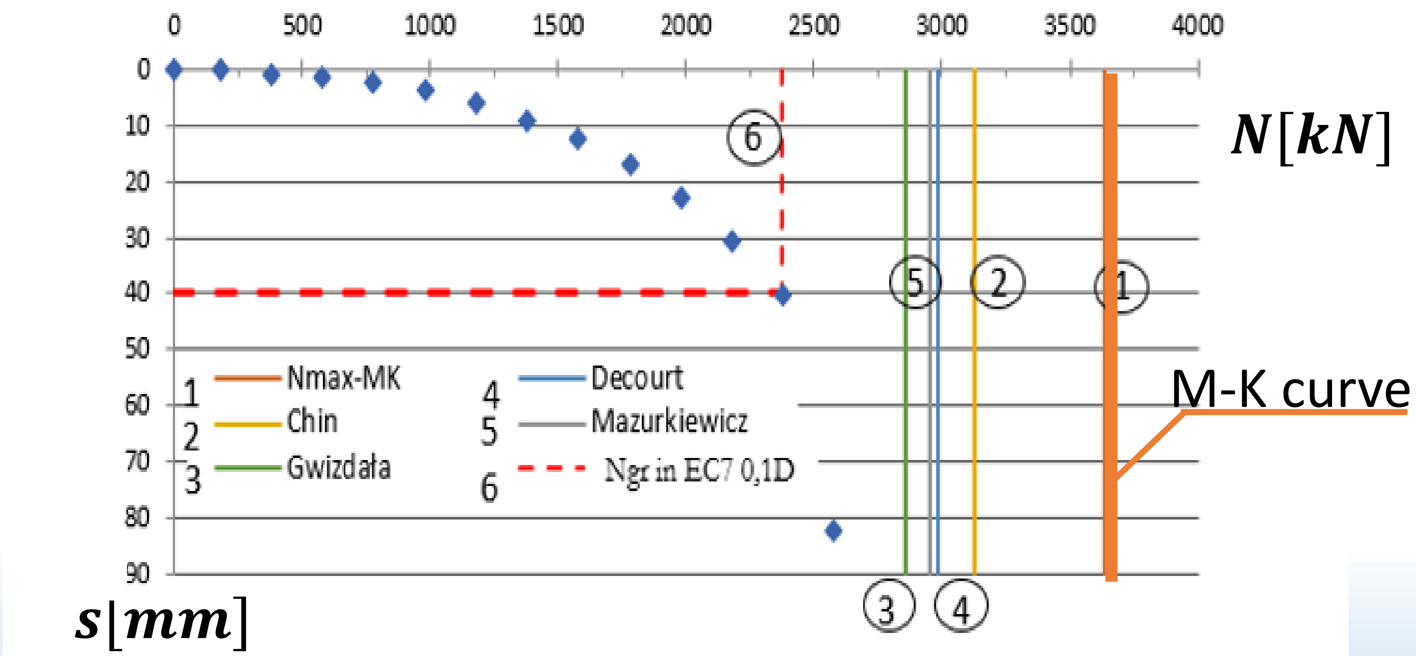


Fig.4 Comparision of limit load capcity obtained using diffrent methods (Szmechel 2015)

Szmechel has proven that using M-K curve we can obtain full description of load-settelment curve even in case when static load test performes only limited load range (Szmechel 2014).

Laboratory test

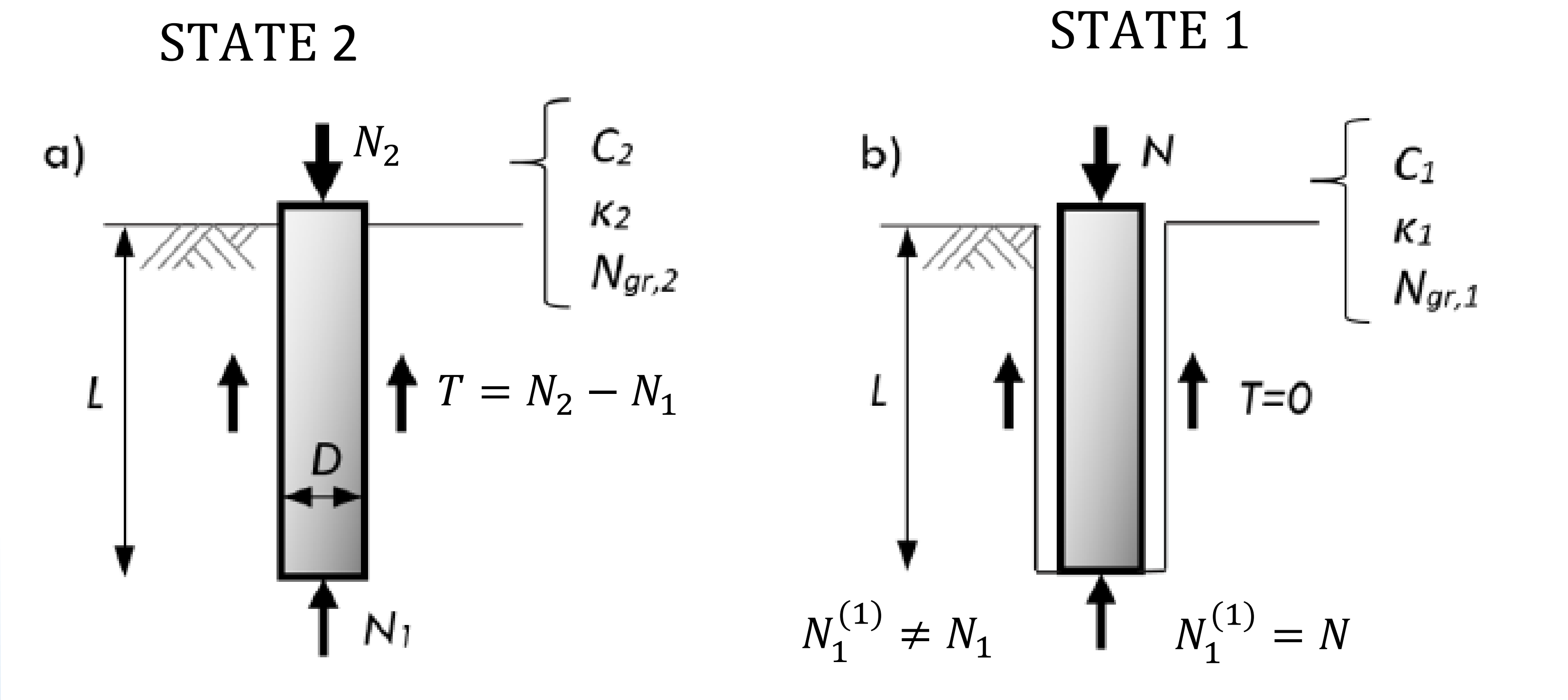


Fig.5 Model for laboratory tests performed by Żarkiewicz a) soil adheres to skin of a pile; b) soil does not have contact with a pile skin (Żarkiewicz 2017).

Laboratory test results

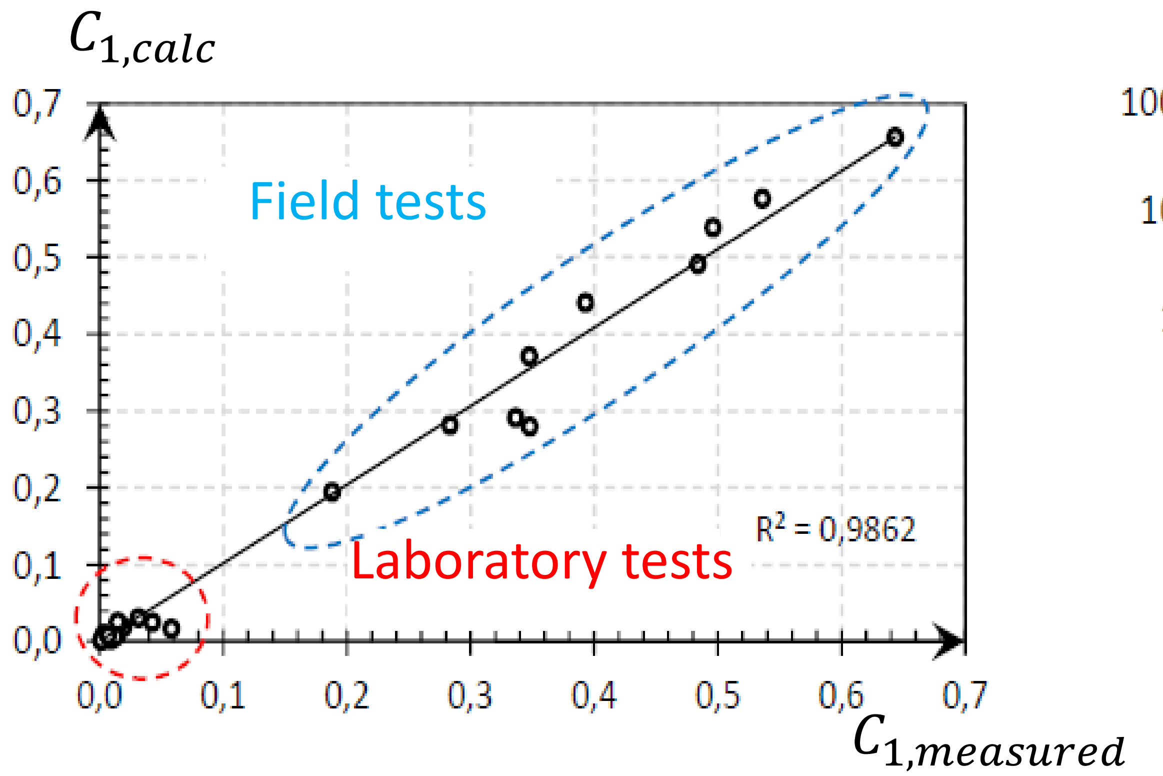


Fig.6 Relation between C_1 measured and C_1calculated (Żarkiewicz 2017)

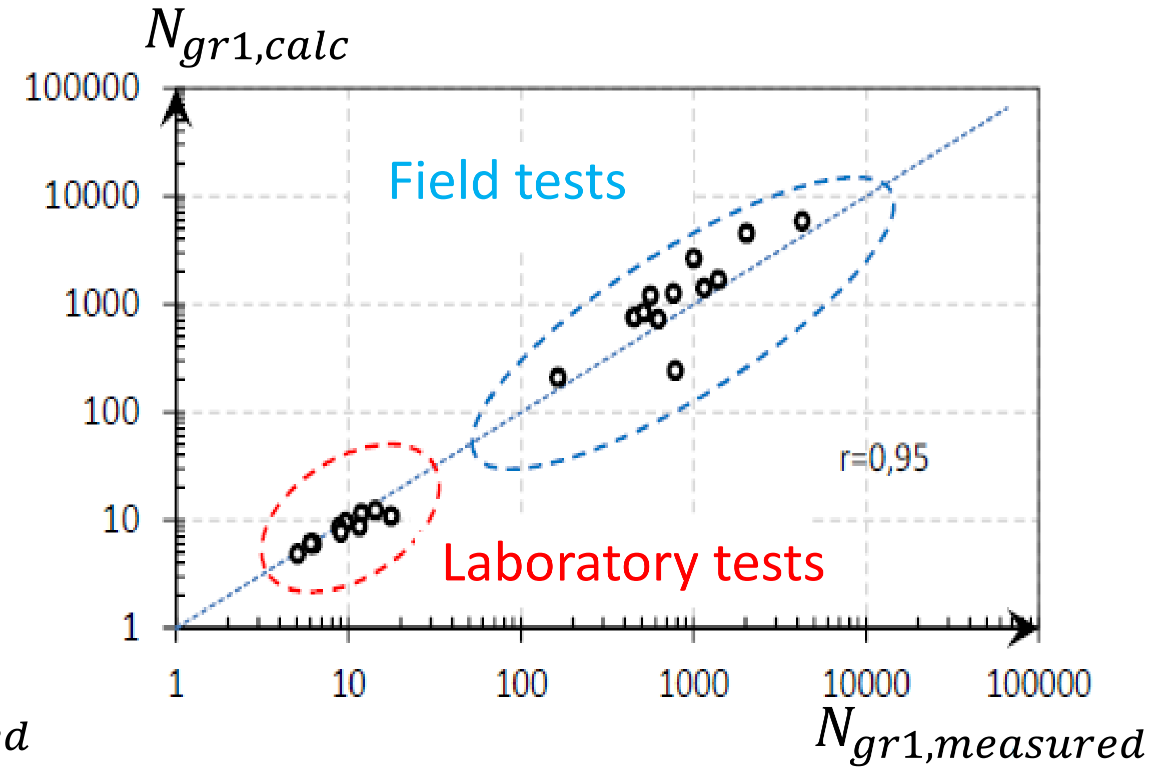


Fig.7 Relation between N_{gr1} measured and N_{gr1}calculated (Żarkiewicz 2017)

Relations between M-K curve parameters

Tests conducted by Żarkiewicz proves that there are relations between M-K curve parameters considering toe and skin resistance, which can be written as follows (Żarkiewicz 2017):

$$C_1 = C_2 \cdot (\kappa_2 + 1)^2; \qquad \kappa_1 = \ln(1 + \kappa_2)$$

$$N_{gr1} = \frac{C_1}{C_2} N_{gr2} \cdot \left[1 + \beta \cdot \left(\frac{H}{D}\right)^{1/3} \kappa_2^{n_1}\right]$$

Finally it gives:

$$N_{gr1} = \frac{N_{gr2}}{(\kappa_2 + 1)^n}, \text{where } n \cong \frac{4}{3}$$

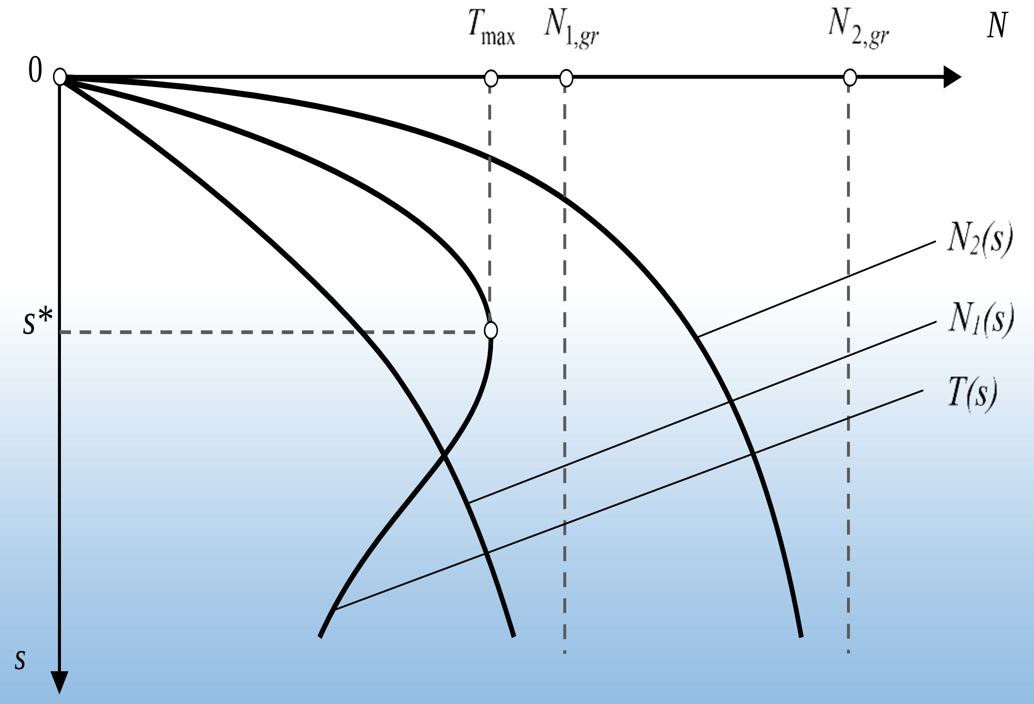


Fig. 8 M-K curves describing skin (T) and toe (N_1) resistances as well as a load-settelment dependency (N_2) (Meyer 2015)

Analysis based on soil field investigations

Meyer and Siemaszko analyze possibility of obtainig κ_2 parameter based directly on CPTU field tests depeneding on soil behavior under the base of a pile under the assumption that skin and toe resistance can be written as follows (Meyer, Siemaszko 2019):

$$\frac{T}{N_1} = \frac{4\pi\beta}{3{,}68} \cdot \left(\frac{H}{D}\right)^{0{,}785} \cdot D \cdot s \cdot \overline{q_c}$$

where:

T –skin resistance

N_1 –toe resistance

β –parameter depending on pile boring technology

$\overline{q_c}$ –average vertical CPT cone resistance

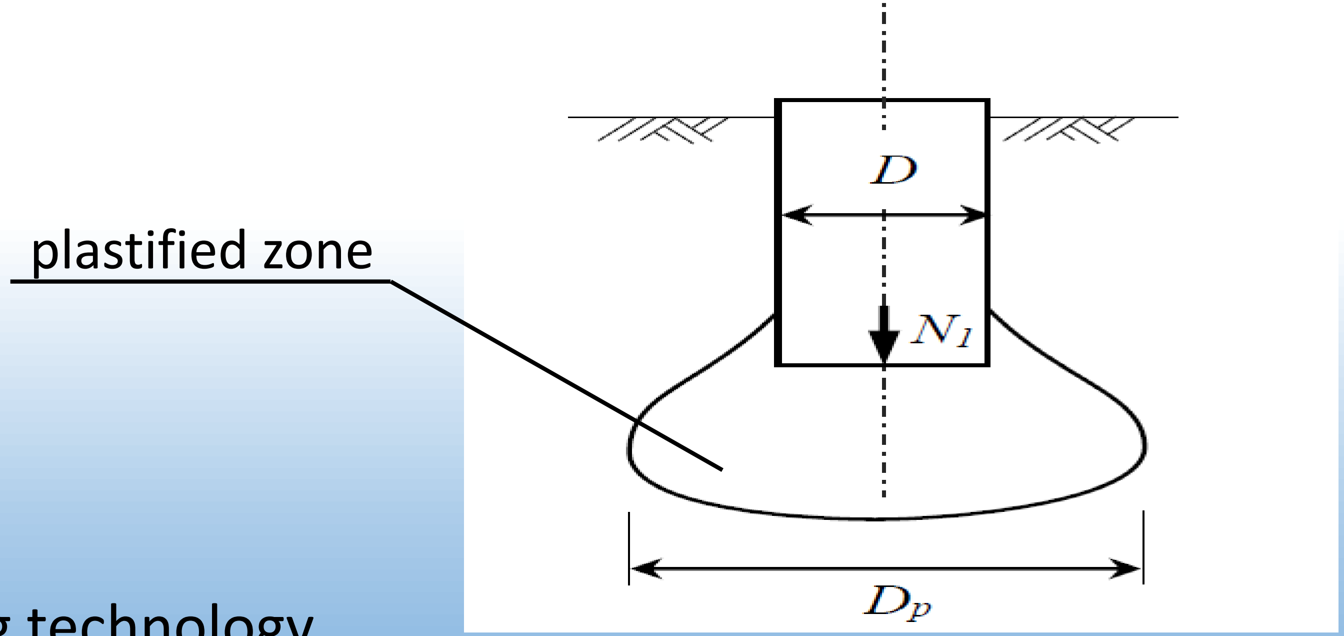


Fig.9 Example of soil beavior under the pile base (Meyer, Siemaszko 2019)

Analysis based on soil field investigations

Equations can be written as follows:

1) $$\kappa_2 = \left[\frac{4\beta}{3{,}68} \cdot \left(\frac{H}{D}\right)^{0{,}785} \cdot \frac{\overline{q_c}}{q_b} \cdot \frac{1}{1 + \frac{1}{4} q_b^{1/3}}\right]^{1/2} - 1$$

2) $$\kappa_2 = \left[\frac{4\beta}{3{,}68} \cdot \left(\frac{H}{D}\right)^{0{,}785} \cdot \frac{\overline{q_c}}{q_b} \cdot \frac{1}{\left(1 + \frac{1}{4} q_b^{1/3}\right)\left(1 + \frac{1}{4} q_b^{1/3}\right)}\right]^{1/2} - 1$$

3) $$\kappa_2 = \left[\frac{4\beta}{20{,}86} \cdot \left(\frac{H}{D}\right)^{0{,}785} \cdot \frac{\overline{q_c}}{q_b} \cdot \frac{1}{1 + \frac{1}{4} q_b^{1/3}}\right]^{3/5}$$

4) $$\kappa_2 = \left[\frac{4\beta}{3{,}68} \cdot \left(\frac{H}{D}\right)^{0{,}785} \cdot \frac{\overline{q_c}}{q_b} \cdot \frac{D}{D_1} \cdot \frac{1}{1 + \frac{1}{4} q_b^{1/3}}\right]^{1/2} - 1$$

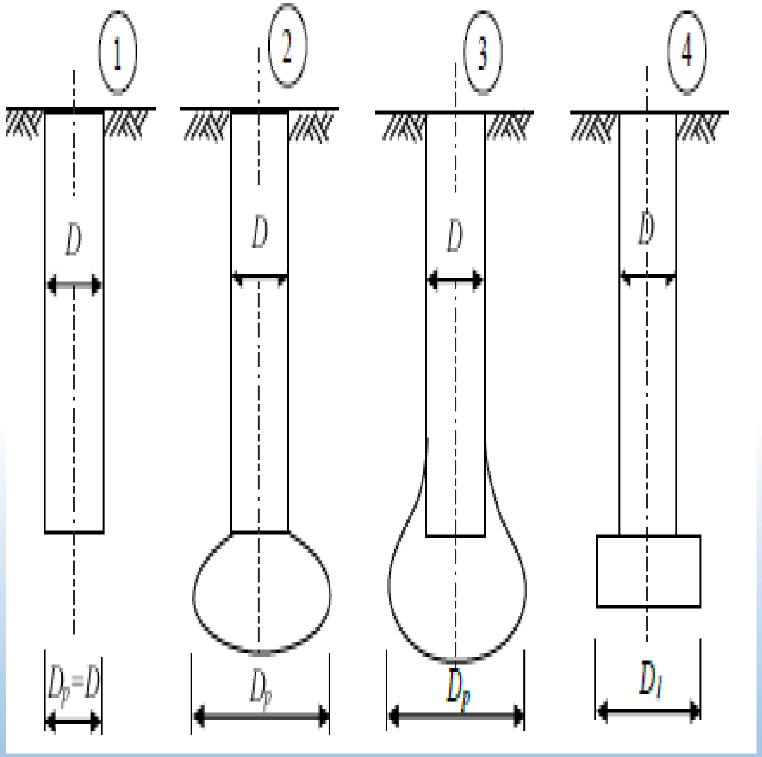


Fig.10 Various examples of soil behavior under the pile base.

Conversion of M-K curve

According to the original M-K method for small load-settelment we have:

$$N_2 = \frac{s}{C_2}; \quad N_1 = \frac{s}{C_1}; \quad T = \frac{s}{C_t}$$

it allows to obtain basic relations (Meyer 2010):

$$C_t = \frac{C_1}{\left(\frac{2}{1+\nu} \cdot \frac{E_t}{E_q} \cdot \frac{H}{D}\right)^{2/3}} \qquad C_1 = C_2 \cdot \left[1 + \left(\frac{2}{1+\nu} \cdot \frac{E_t}{E_q} \frac{H}{D}\right)^{\frac{2}{3}}\right]$$

For further calculation it is convenient to enter *U* parameter, as a material constant of soil (Meyer, Stachecki 2018):

$$U = \left(\frac{2}{1+\nu} \cdot \frac{E_t}{E_q}\right)^{2/3} = \frac{\frac{C_1}{C_2} - 1}{\left(\frac{H}{D}\right)^{\frac{2}{3}}}$$

Conversion of M-K curve

Relation that allows for conversion of the M-K curve in homgenous soil can be written as follows (Meyer, Stachecki 2018):

$$\frac{\frac{C_1^{(0)}}{C_2^{(0)}} - 1}{\left(\frac{H^{(0)}}{D^{(0)}}\right)^{\left(\frac{2}{3}\right)}} = \frac{\frac{C_1^{(1)}}{C_2^{(1)}} - 1}{\left(\frac{H^{(1)}}{D^{(1)}}\right)^{\left(\frac{2}{3}\right)}}$$

where:

H –length of a pile

D –diameter of a pile

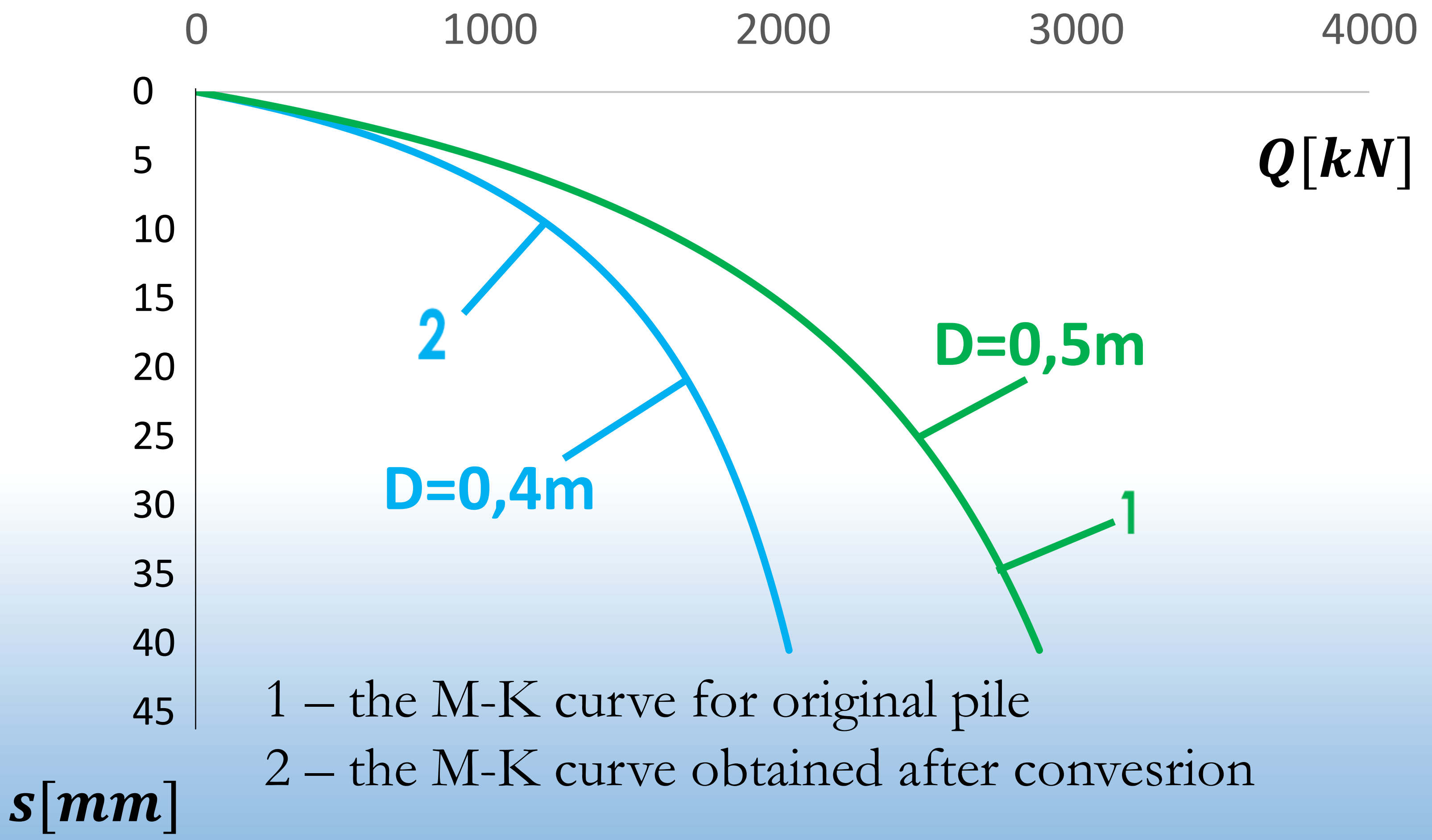
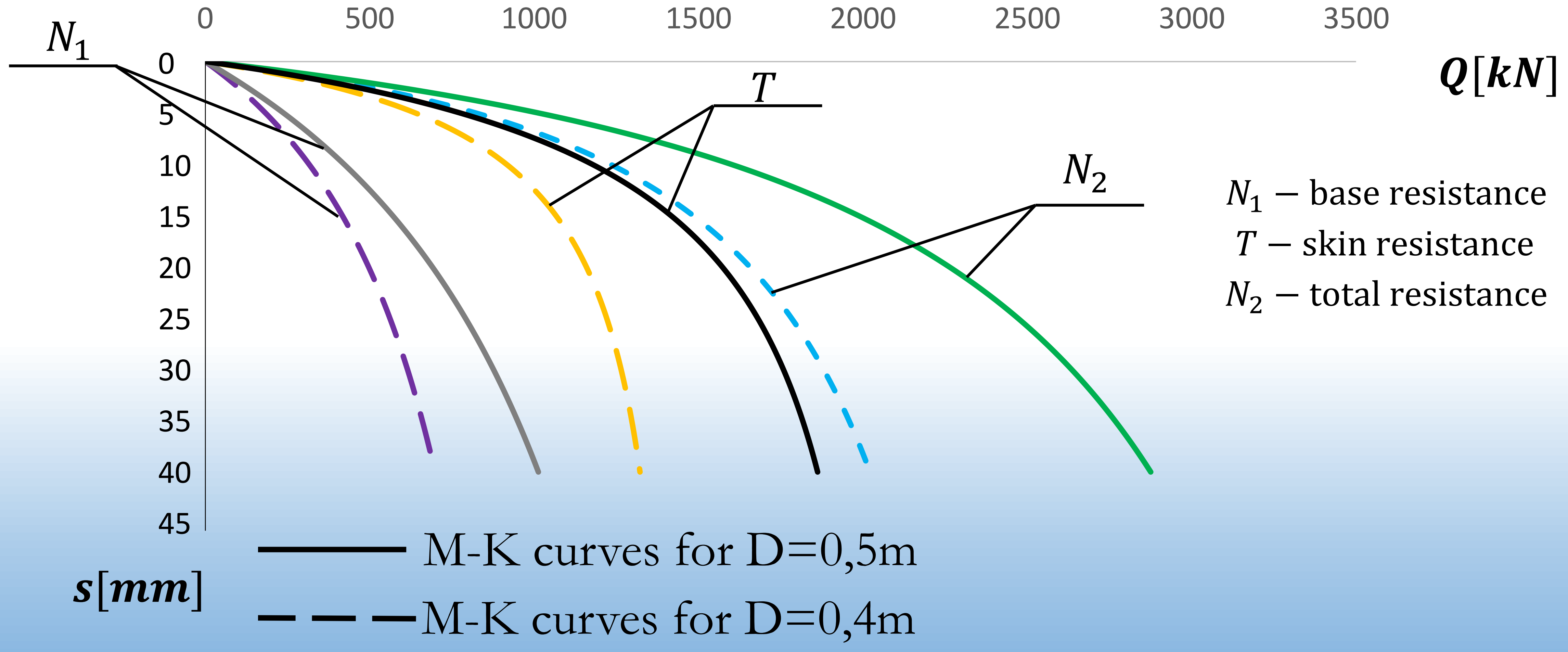


Fig. 11 Static load test curve conversion in case of diameter change

Conversion of the M-K curve parameters



N_1 – base resistance
T – skin resistance
N_2 – total resistance

Fig.12 Conversion of M-K curves in case of diameter changes

Interaction between raft and piles based upon static load test

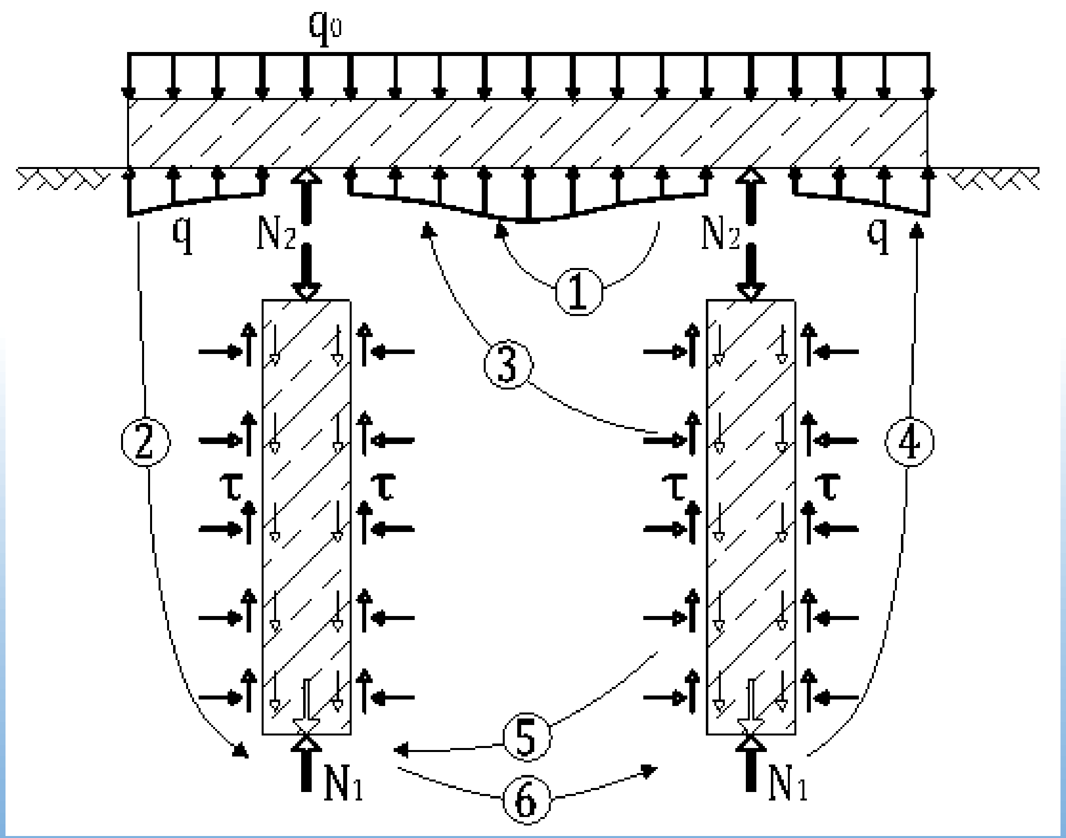


Fig.13 Scheme of slab-pile foundation (Cichocki 2018)

Interactions:

1. raft → raft
2. raft → pile
3. pile skin → raft
4. pile base → raft
5. pile skin → pile
6. pile base → pile

q_0 – load applied to the raft
q – pressure on the raft-subsoil surface
N_2 – load applied to the pile head
τ – skin resistance
N_1 – base resistance

Interaction between raft and piles based upon static load test

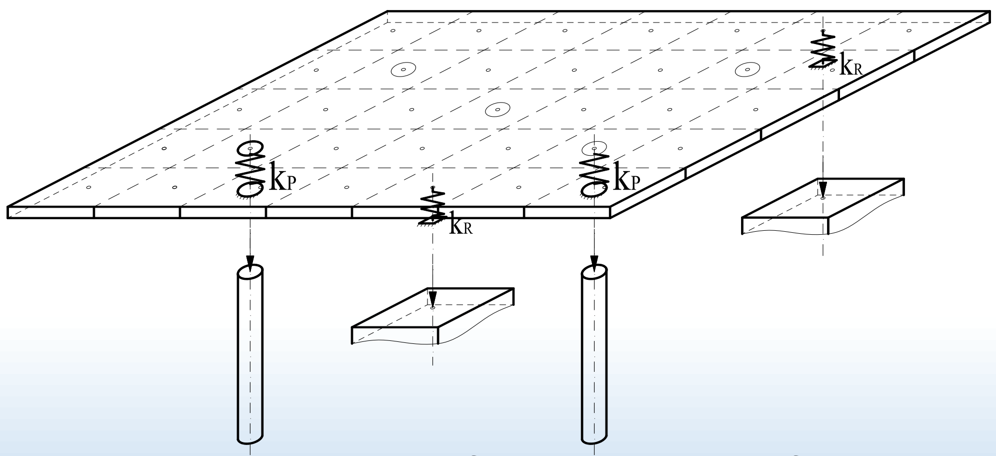


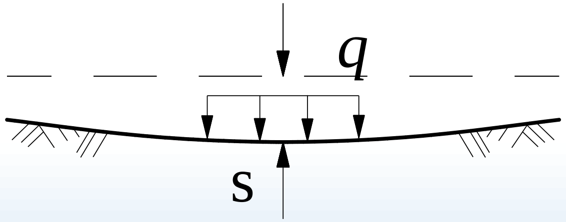
Fig.14 Assumptions for the mathematical model of cooperation of a combined pile raft foundation with subsoil (Cichocki 2018)

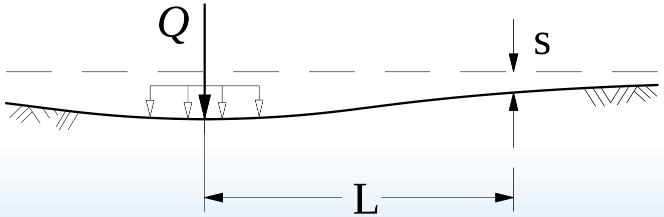
D_b – bending stiffness of the raft [kNm]
k – vertical stiffness of the elastic supports modelling subsoil or pile [kPa/m]
q_0 – load applied to the raft [kPa]
s – unknown vertical displacements of the raft [m]
x,y – axes of a rectangular coordinate system

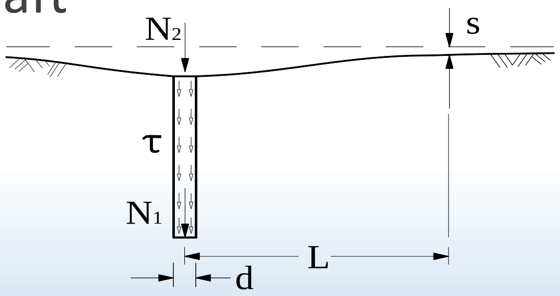
Interaction between raft and piles based upon static load test

The following impacts were taken into account in the model in the form of partial settlements:

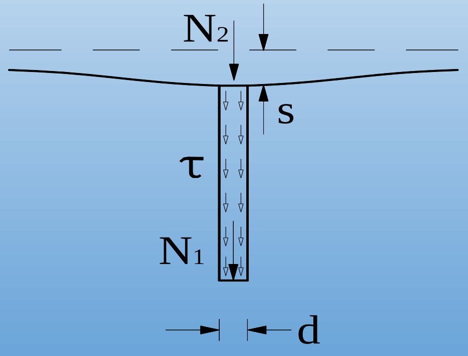
- settlement of the field of the discrete raft

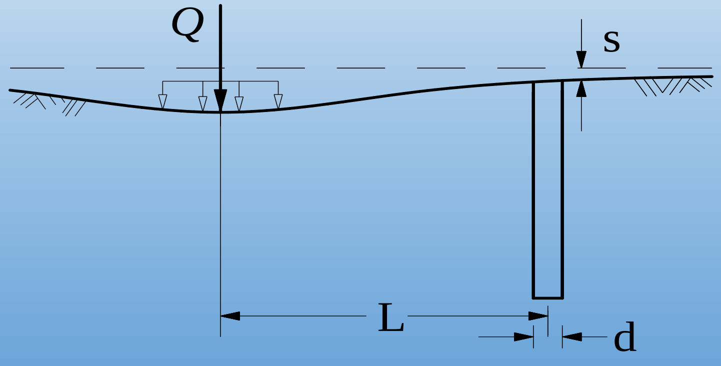


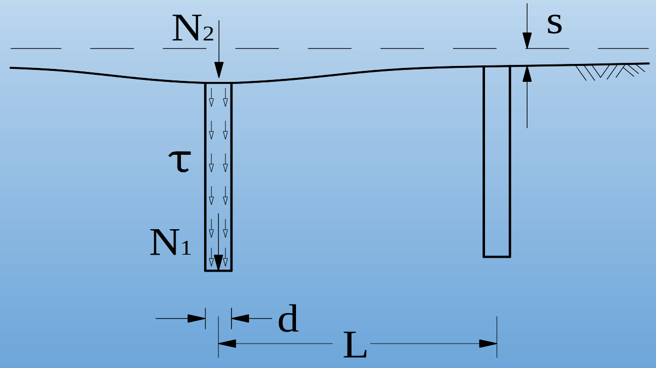




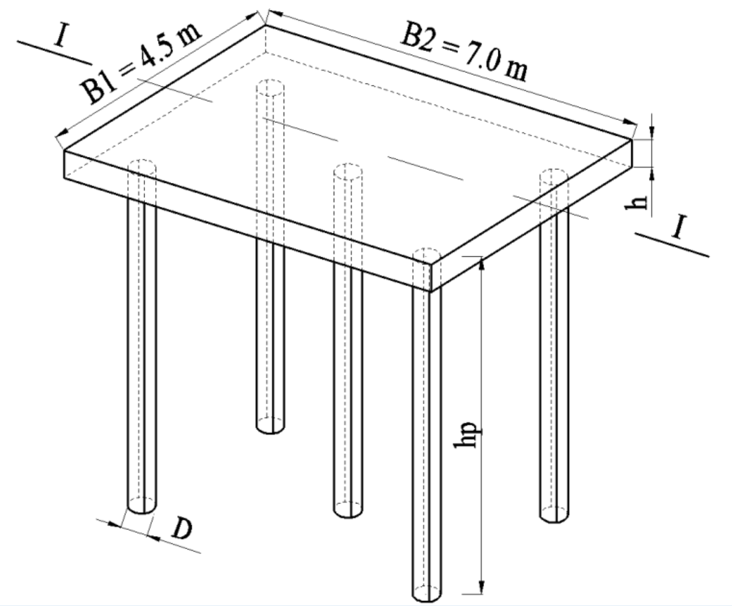
- settlement of the pile



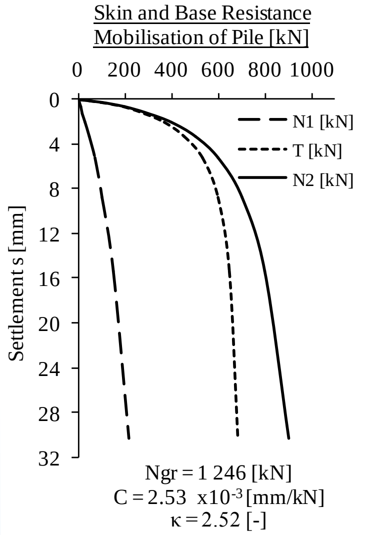




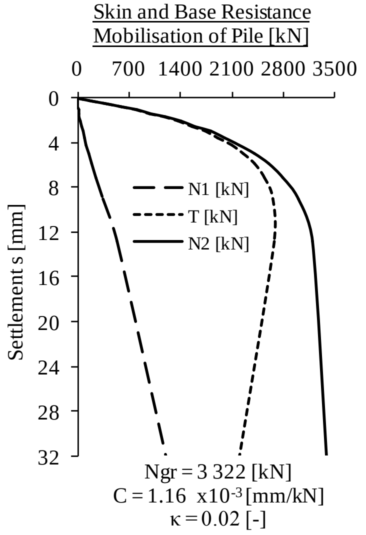
Interaction between raft and piles based upon static load test



Pile no. I
(*hp=10.0m, D=0.60m*)



Pile no. II
(*hp=13.5m, D=0.508m*)



Geotechnical parameters – pile no. I

Type of soil	Thickness of layer [m]	Vol. weight γ [kN/m³]	Angle of friction ϕ [°]	Young's modulus E_o[MPa]
silty clay	3.0	17	10	7.0
sandy siltyclay	6.5	20	21	24.6
silty clay	–	19	10	12.5

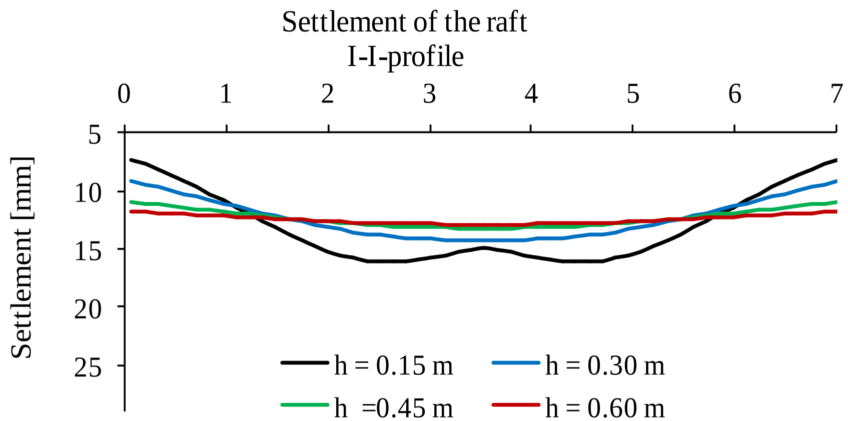
Geotechnical parameters – pile no. II

Type of soil	Thickness of layer [m]	Vol. weight γ [kN/m³]	Angle of friction ϕ [°]	Young's modulus E_o [MPa]
silty clay	5.0	17	10	7.0
fine sand	7.0	20	32	56.4
fine sand	–	20	32	60.0

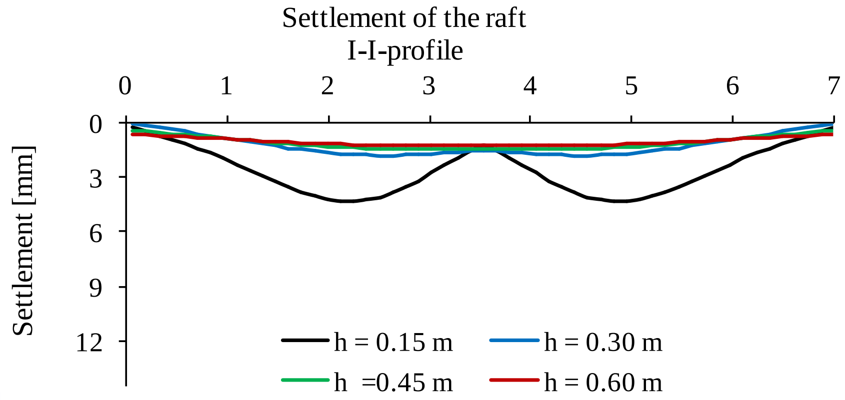
Pile thickness influence on vertical displacement

Impact of bending stiffness of the raft on its settlement and deflection qo=125kPa :

- for N_2-s curve of pile no. I

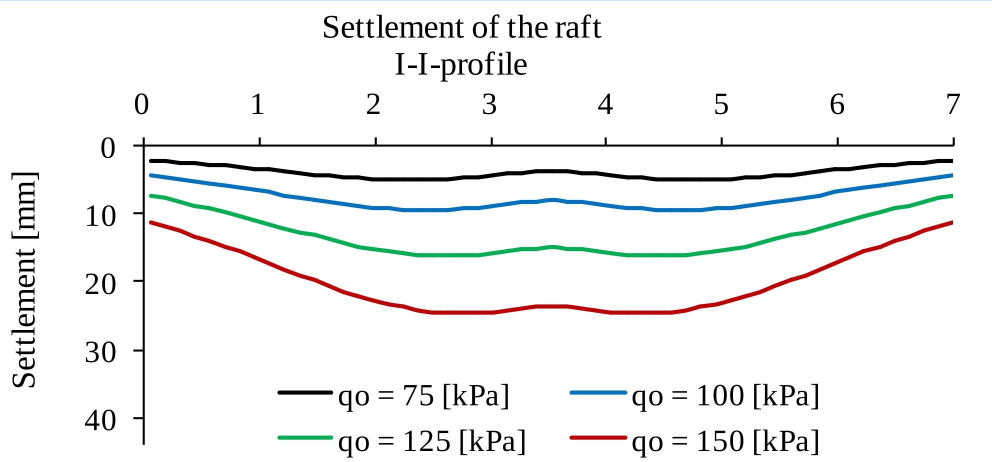


- for N_2-s curve of pile no. II

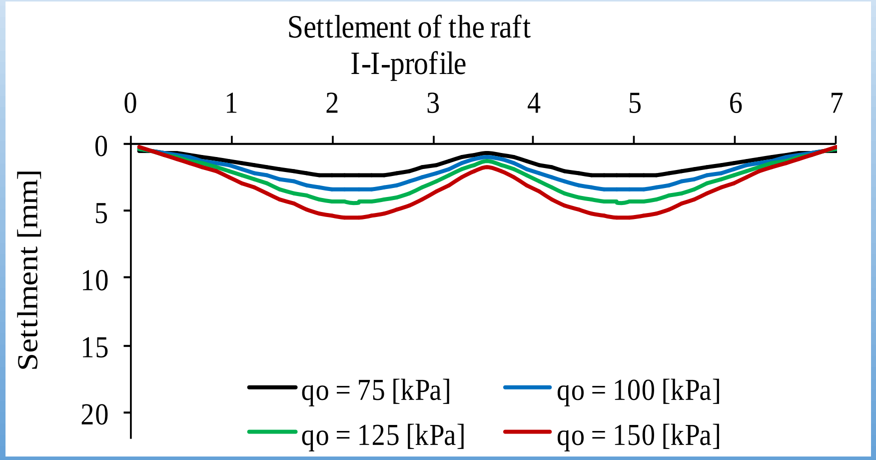


Impact of load applied to the raft on its settlement and deflection (h_{raft}=15cm)

- for N_2-s curve of pile no. I



- for N_2-s curve of pile no. II



Field measurment verification

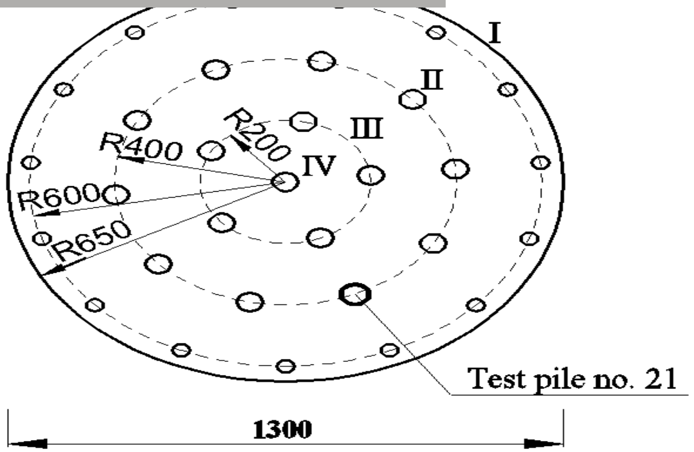


Fig. 15 Geodetic monitoring of emergency tank (Cichocki 2018).

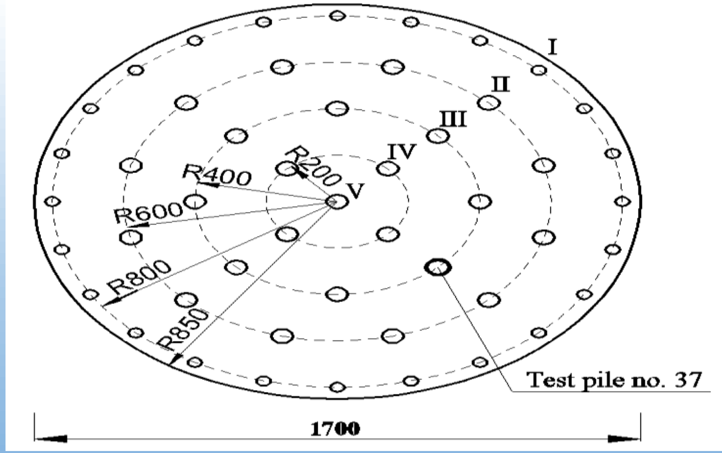


Fig. 16 Geodetic monitoring of final cleaning tank (Cichocki 2018).

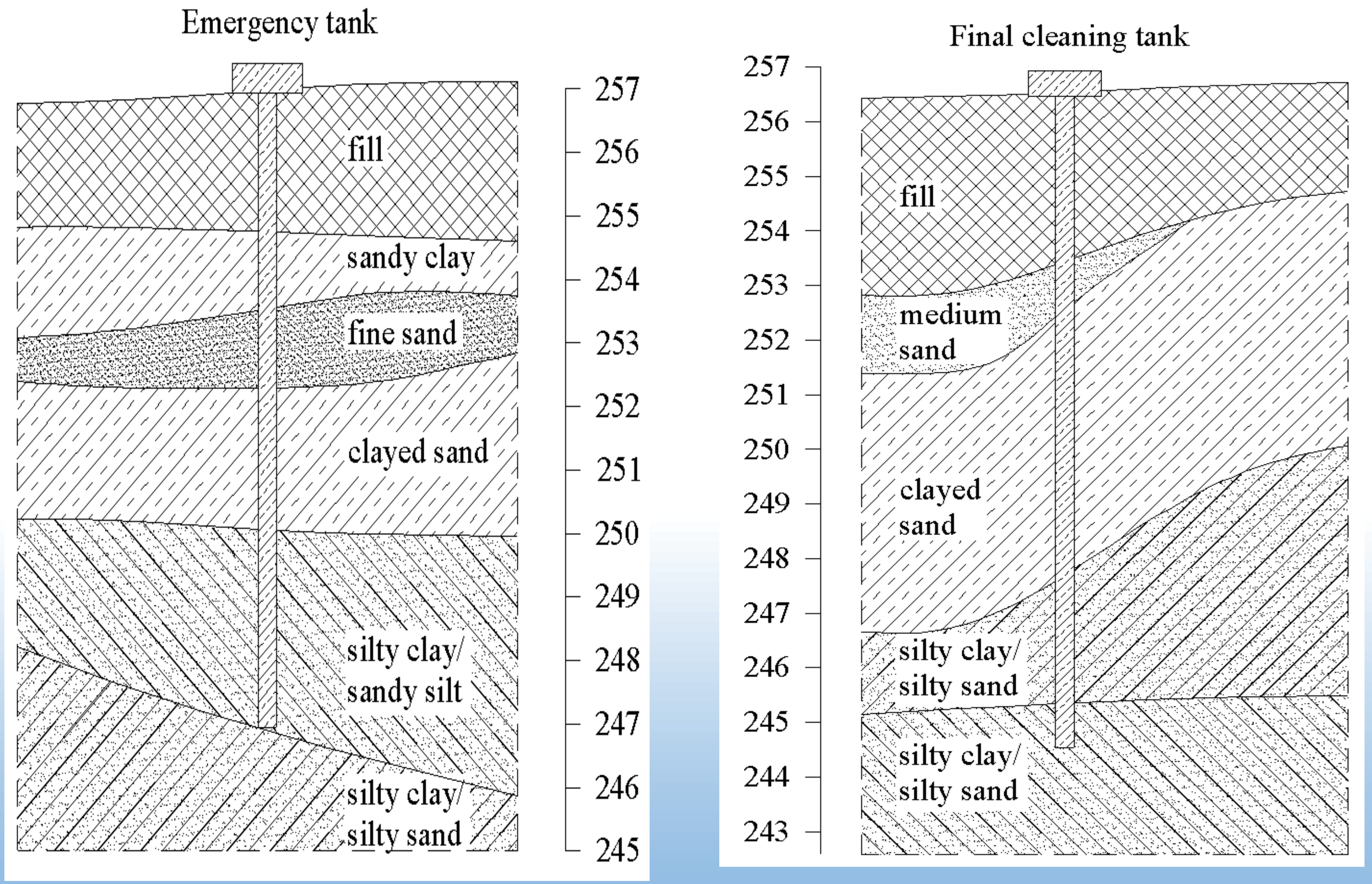


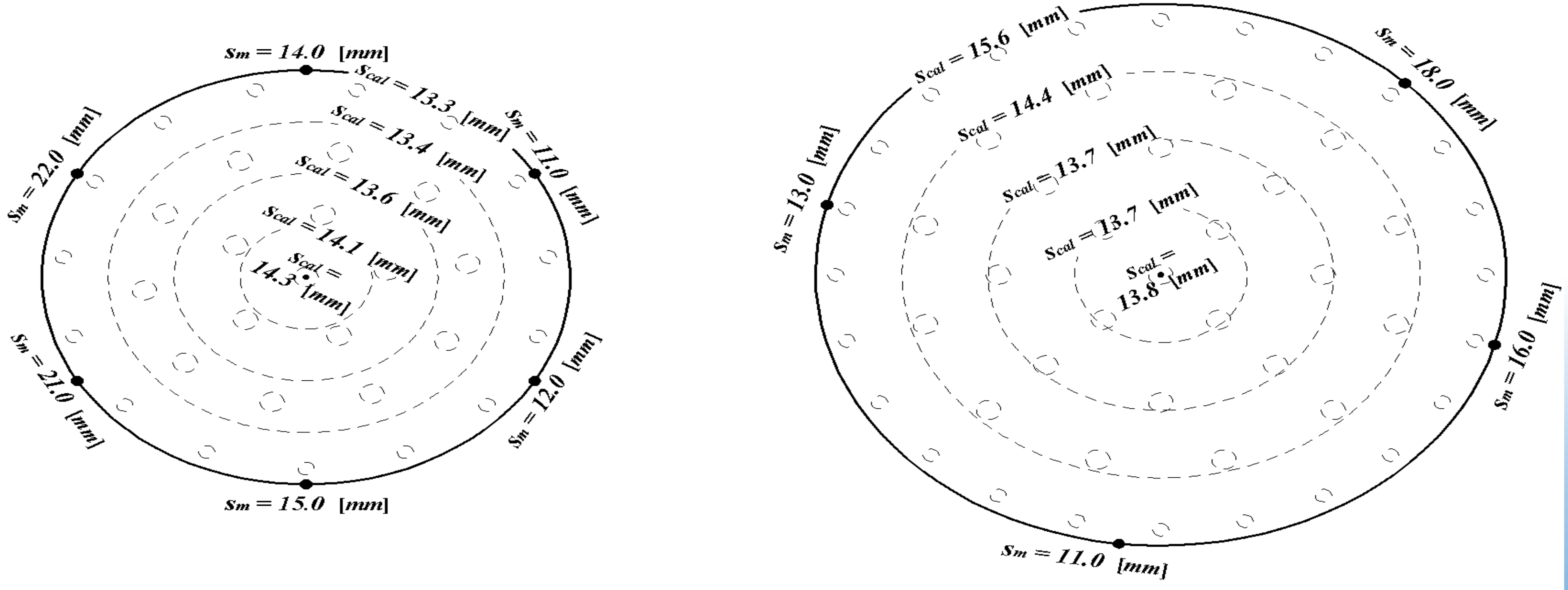
Fig. 17 Geotechnical profiles. Sewage Treatment Tanks – Tychy Brewery Plant Complex (Cichocki 2018).

Field measurment verification

Sewage Treatment Tanks – Tychy Brewery Plant Complex

Emergency tank

Final cleaning tank



s_{cal} – calculated settlement

s_m – measured settlement

Fig. 18 Calculated and measured settelment of foundation rafts (Cichocki 2018)

Geotubes

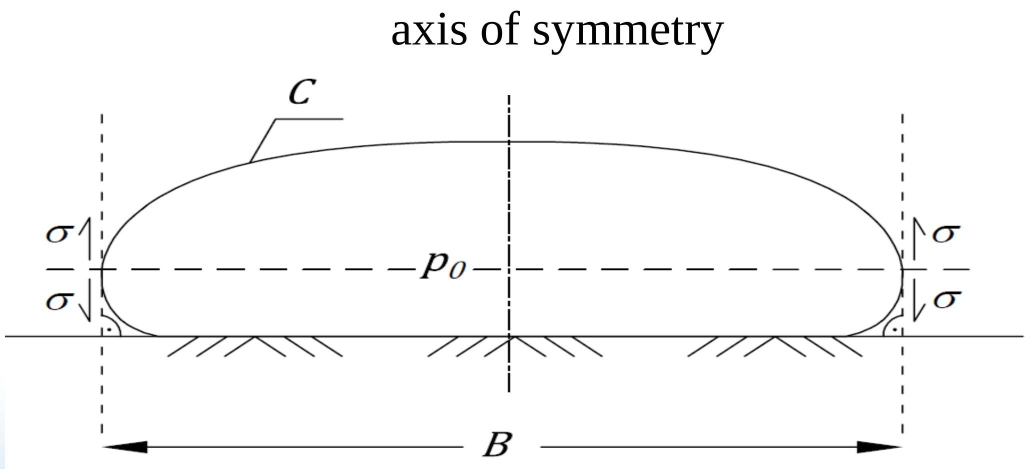


Fig. 19 The scheme according to Pilch (Pilch 2018)

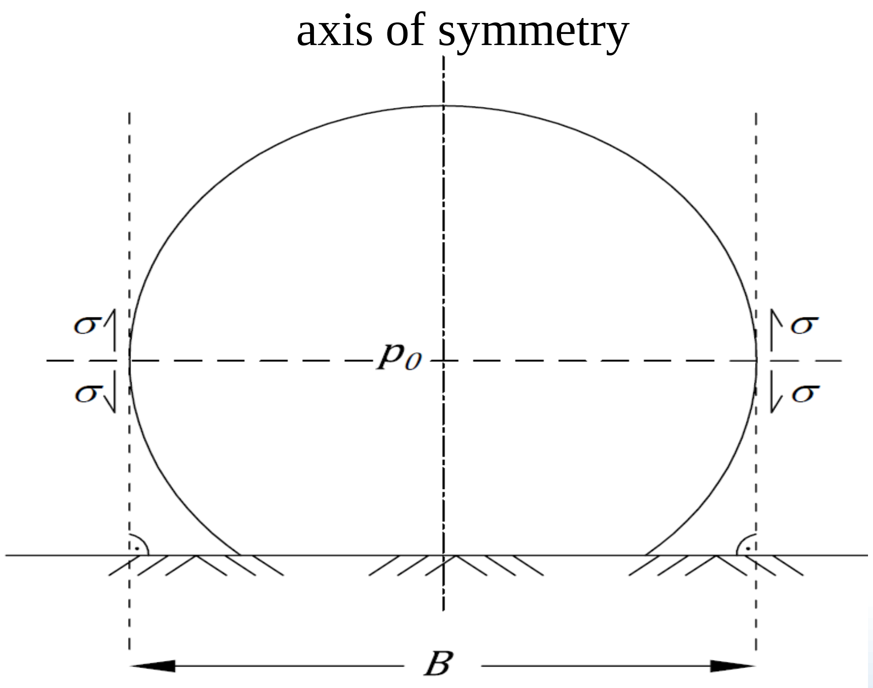


Fig. 20 The scheme according to Leshchinsky (Pilch 2018)

$$1m\, p_0\, B = 2\, \delta\, \sigma\, 1m$$

$$p_0 = \frac{2\, \delta\, \sigma\, 1m}{B\, 1m}$$

where:

p_0 – pressure at the reference level to assume that stresses σ operate vertically along tangent [kPa]

δ – casing thicknes [m]

σ – stress [kN/m^2]

B – width of geotube[m]

C – geotube circuit[m]

Geotubes

Fig. 21 Securing dunes on the beach in Rowy.
(Pilch 2018)

Geotubes

Fig. 22 The end result – recrating the natural landscape.
(Pilch 2018)

Geotubes

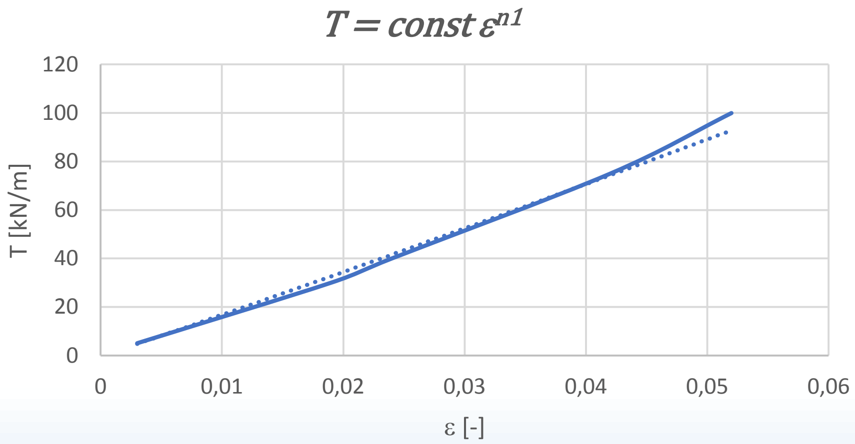
- Effective dune protection.
- The core is intact
- Temporary protection SoilTain Geotubes thanks to use of geotextile.
- Easy restoration.

Fig. 23 Section protected after Ksawery hurricane.
(Pilch 2018)

Geotubes

After performing field tests Pilch described following relations (Pilch 2018):

$$p_0 = 0,000773\ T^{1,05}$$



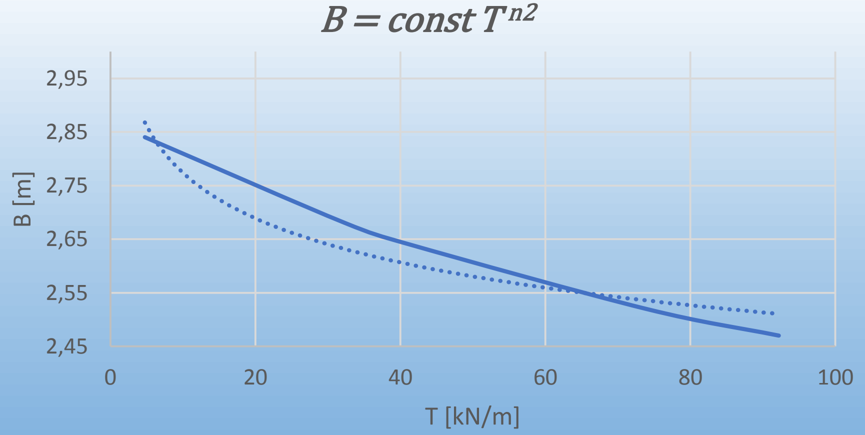
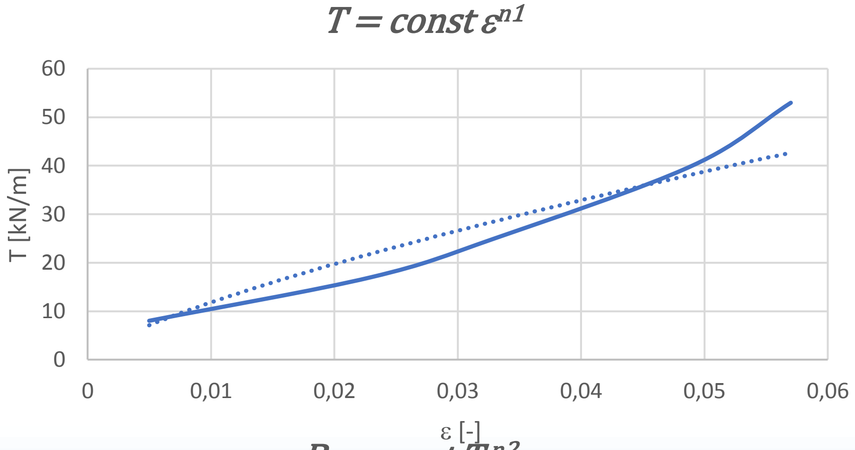


Fig. 24 Results for small geotube (Pilch 2018)

$$p_0 = \text{const}\ T^{n_3}$$

measured
calculated

$$p_0 = 0,00104\ T^{1,017}$$



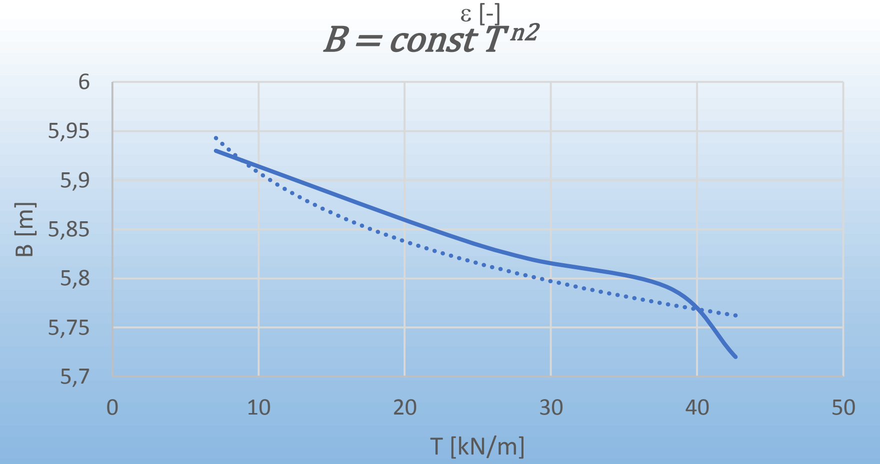


Fig. 25 Results for large geotube (Pilch 2018)

Incased sand columns

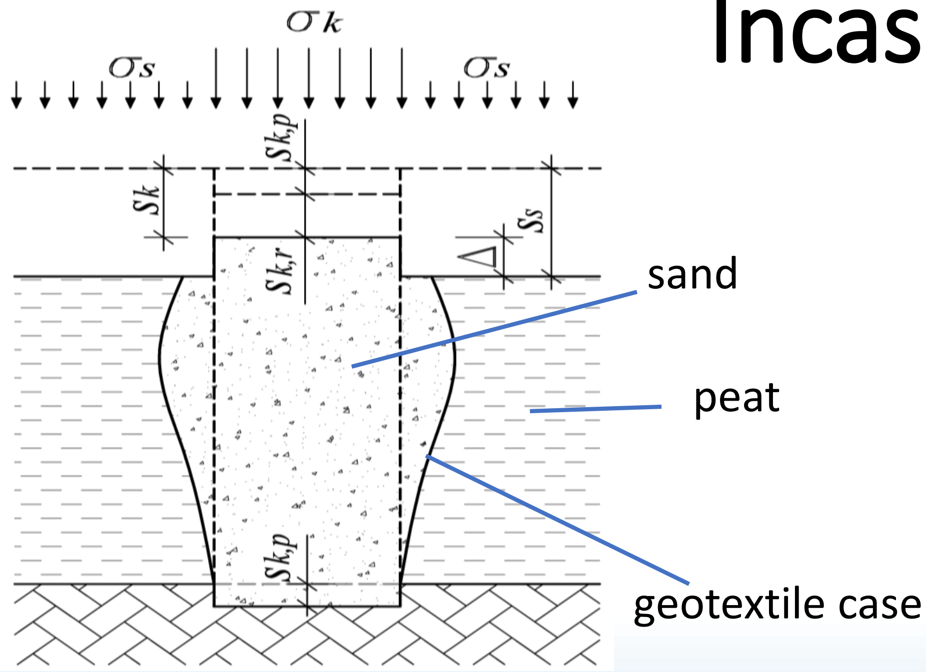


Fig. 26 The scheme of incased sand column (Łopatko 2016)

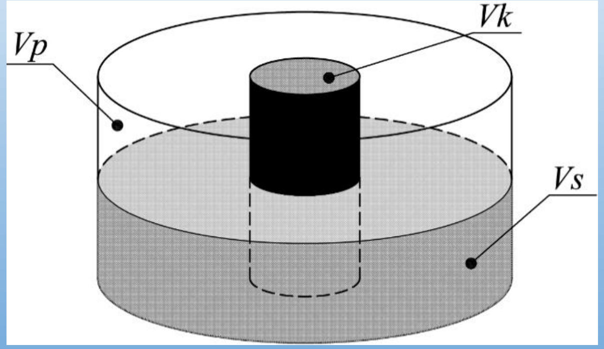


Fig. 27 The scheme of incased sand column (Łopatko 2016)

Łopatko made following assumptions (Łopatko 2016):

- inhomogeneous settlement of the terrain surface is allowed;
- the supporting layer is compressible;
- the mineral filling of the column is subjected to resting pressure;
- the weak soil is non-linearly elastic

Basic equations used in analysis:

$$M(s) = M_0\left(1 - \frac{s}{n_0 H_0}\right)^{-\kappa} \qquad \alpha = \frac{V_k}{V_p + V_s} \qquad n_0 = \frac{V_p}{V_p + V_s}$$

Incased sand columns

Fields tests were performed by Łopatko in case of ground reinforcement under the A-2 highway (Łopatko 2016):

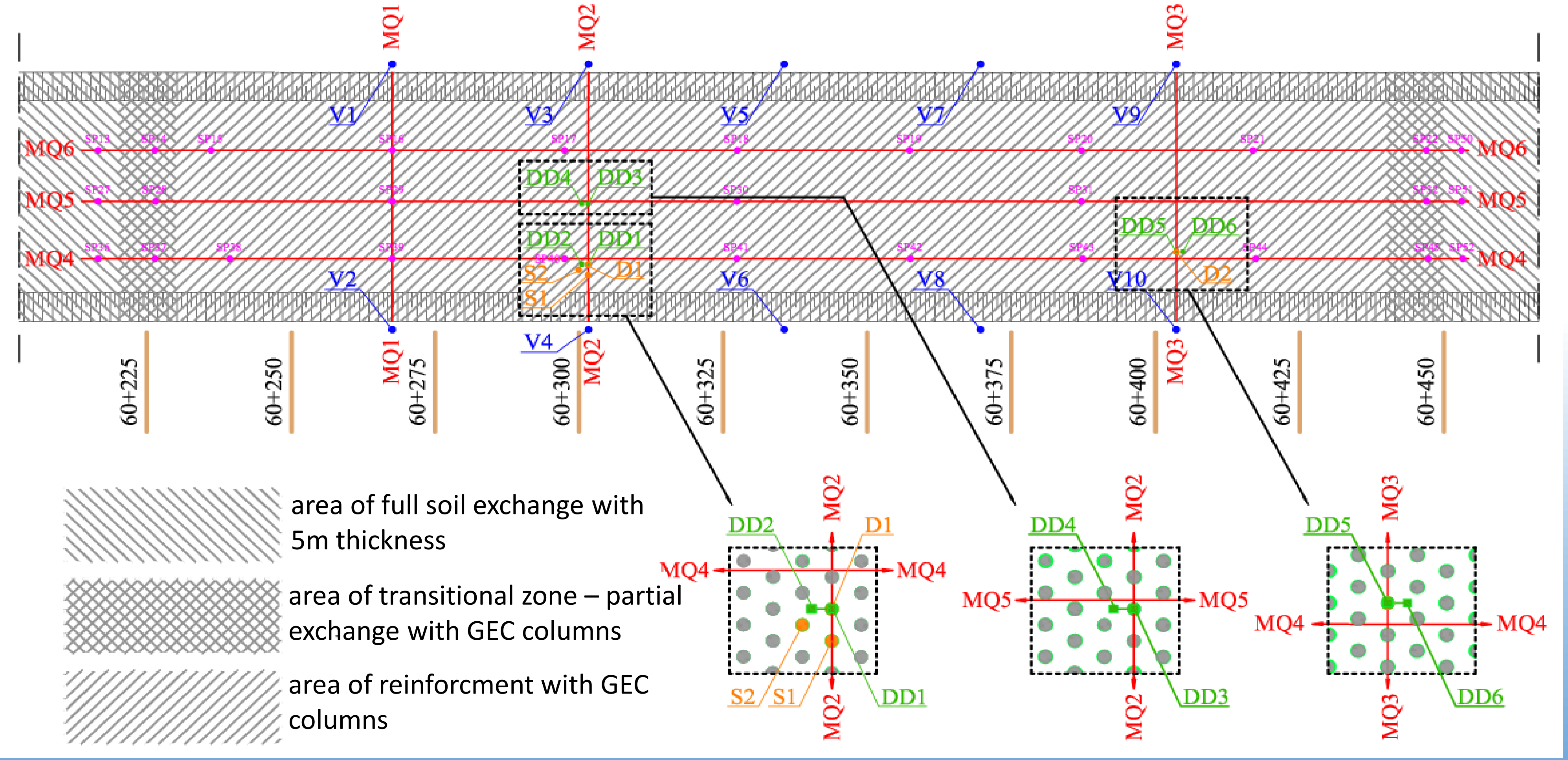


Fig. 28 The scheme of soil reinforcemnt (Łopatko 2016)

Incased sand columns

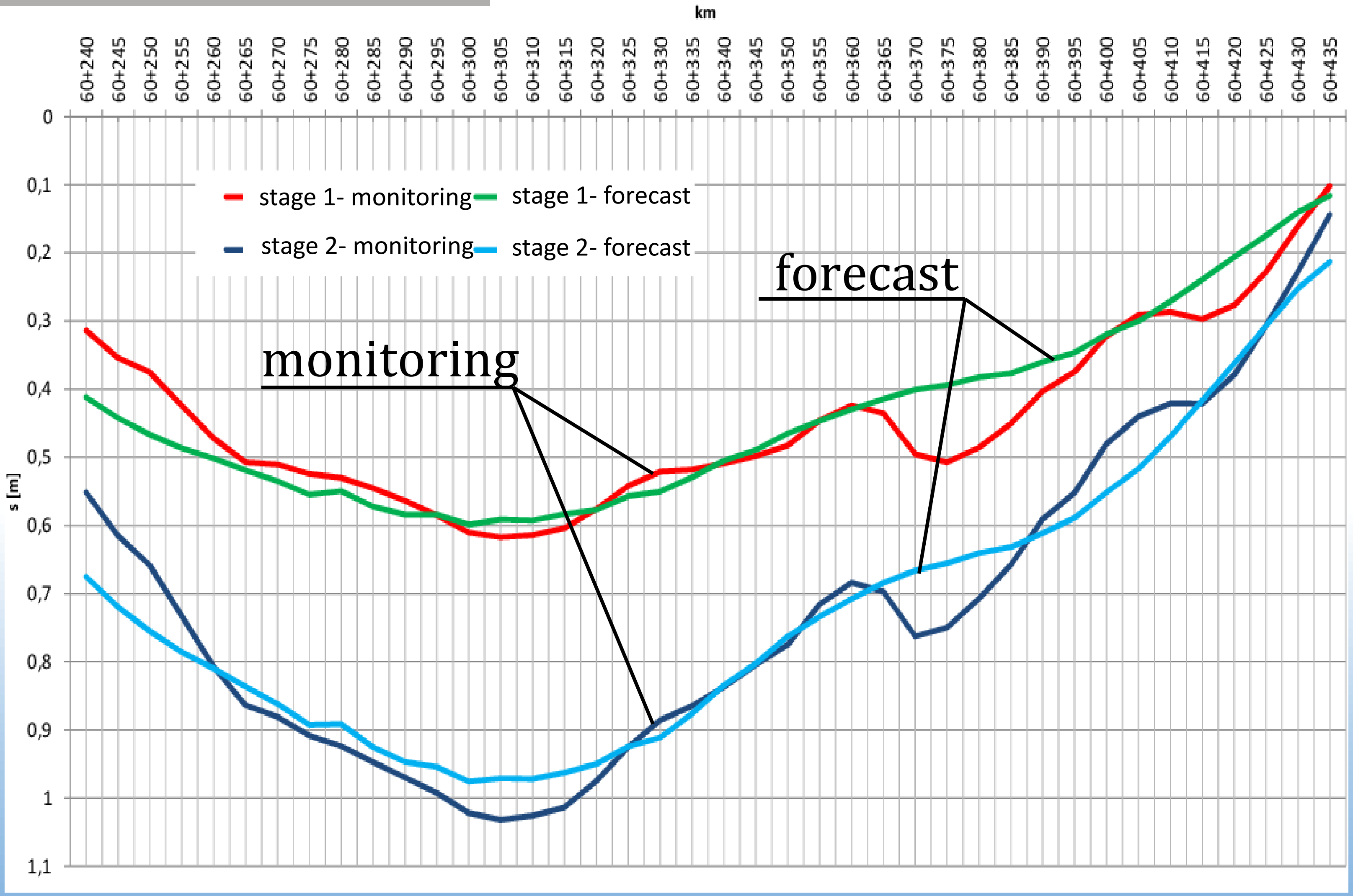


Fig. 29 Measured and predicted settlement of incased sand columns (Łopatko 2016)

soil	γ/ γ' [kN/m³]	Φ' [°]	c' kN/m²]	$M_{0,ref}$ [kN/m²]
peat	11/1	15	5	500
silty clay	14/4	20	5	750
silty sand	20/10	32,5	-	-

Fig. 28 Geotechnical profile (Łopatko 2016)

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