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Geotextile encased columns – verification of the analytical design method

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ABSTRACT

There exist numerous methods for the design of geotextile encased columns and for the prediction of settlements of improved soil. Numerical ones may be more accurate, but at the same time far less convenient for use. Thus, a derivation of an analytical method, which would be in good agreement with the numerical solution, would be more appropriate. In this paper the analytical method for calculating settlement reduction and stresses in soil and column for one soft soil layer proposed by Pulko et al. (2011) was modified to calculate settlements of multiple soft soil layers and respective stresses in soil and column. The method is based on the unit cell assumption and considers column as an elasto-plastic material, while soil and geotextile are treated as elastic materials. Documented case studies from literature were investigated and used for the validation of the analytical method. Measured settlements for each case were compared to settlements calculated using the analytical method. The method verification presented in the paper is helpful in predicting the right values for the critical design parameters that are difficult to measure in situ or in the laboratory.

Keywords: GEC, geotextile encased columns, stone columns, soil improvement, settlement reduction, analytical solution, elasto-plastic model

1. INTRODUCTION

In the past 20 years ground improvement technique of installing geotextile encased stone columns to reduce settlements has become an established practice in a variety of projects all over the world. Experiences prove them to be very useful, even when post construction settlements pose considerably stringent limitations.

This paper focuses on an analytical method for calculating settlements of multilayered ground improved with geotextile encased stone columns (GECs). The method is derived from the analytical method proposed by Pulko et al. in 2011, which was in its final form developed for a single layer of improved soil. A detailed description of the proposed method's principal together with the most important equations is presented.

Next, the paper focuses on documented case history projects presented by Alexiew & Raithel (2015). Four projects are described and soil and column parameters, which were included

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in the calculations, are presented. All cases are analyzed according to the newly proposed method. Results and conclusions are presented individually, compared to each other and discussed.

2. ANALYTICAL METHOD FOR CALCULATING SETTLEMENTS OF GROUND WITH GEC

2.1. General description

The analytical method for calculating settlements of multilayered ground improved by GEC is based on a commonly known "Unit cell" concept (Figure 1) introduced by Priebe (1976). Soil is considered as an elastic material and column as an elasto-plastic material (Balaam and Booker, 1985). The method assumes drained condition, since the columns assure rather quick consolidation of the surrounding soil.

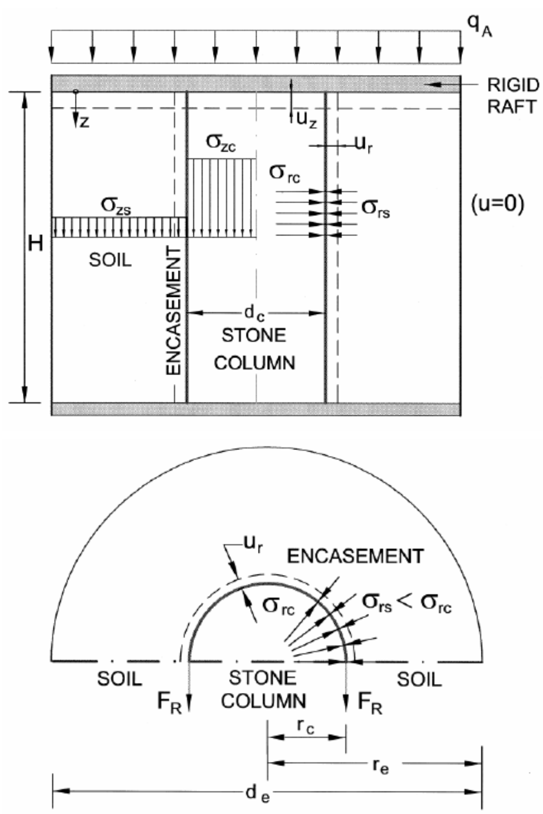

Figure 1: Unit Cell (Pulko et al., 2011)

For the purposes of this analysis the original method proposed by Pulko et al. (2011) was extended to take into account multilayered ground. The proposed method is user friendly, since new layers of soil can be easily added and all the respective parameters can be taken into account.

Beside previously mentioned assumptions, the method also takes into account:

- The load is assumed rigid, therefore vertical settlements of soil and column are supposed to be equal.
- Settlements of the bearing ground are neglected.
- The soil remains in elastic state.
- Stone column is modeled as an elastoplastic material, satisfying Mohr-Coulomb's yield criterion with constant dilation angle ψ.
- Geotextile encasement is modeled as an elastic material with constant deformation modulus (Pulko et al., 2011)

2.2. Basic equations for one layer ground

In order to describe the response of stone column and the surrounding soil under the applied load q_A the basic principles are presented below. The method proposed by Pulko et al. (2011) is based on equations for stress and strain increments for the elastic and elasto-plastic response of the slice of "Unit cell", as given in Table 1.

2.3. Extension of the method for multilayered ground

Extension of the method enables us to calculate total settlements of improved multilayered ground. For this purpose ground is divided in layers of different types of soil. each having its individual soil properties. Each soil layer can further be divided into sublayers, if necessary. For each sublayer stresses and strains are calculated by using equations from Chapter 2.2 along with equations presented below.

The stone column yield criteria is given by:

$$K_{pc} = \frac{1+\sin\varphi_c}{1-\sin\varphi_c} = \frac{\sigma_{zc,ini} + \Delta\sigma_{zc}^{e}}{\sigma_{rc,ini} + \Delta\sigma_{rc}^{e}} \quad (1)$$

$\sigma_{zc,ini}$ and $\sigma_{rc,ini}$ are initial vertical and radial stresses in the stone column after installation at a selected depth.

Table 1: Basic equations for calculating stresses and strains (Pulko et al., 2011)

	elastic response	elasto-plastic response
$\begin{Bmatrix}\Delta\sigma_{zc}\\ \Delta\sigma_{rc}\end{Bmatrix}$	$\begin{bmatrix}\lambda_c+2G_c-2\lambda_c F\\ \lambda_c-2\left(\lambda_c+G_c\right)F\end{bmatrix}\left\{\varepsilon_z^e\right\}$	$\dfrac{D}{C_5}\begin{bmatrix}K_{pc}\left[2k_0+K_\psi\left(C_2+T\right)\right]\\ 2k_0+K_\psi\left(C_2+T\right)\end{bmatrix}\left\{q_A\right\}$
$\begin{Bmatrix}\Delta\sigma_{zs}\\ \Delta\sigma_{rs}\end{Bmatrix}$	$\begin{bmatrix}\lambda_s+2G_s-2\lambda_s\dfrac{FA_r}{(1-A_r)}\\ \lambda_s+\dfrac{2A_rF}{(1-A_r)}\left(\lambda_s+G_s+\dfrac{G_s}{A_r}\right)\end{bmatrix}\left\{\varepsilon_z^e\right\}$	$\dfrac{1}{C_5}\begin{bmatrix}D\left(C_1K_\psi+2\right)+E_{oed}\left(C_3+T\right)\\ DC_2K_\psi+2Dk_0+E_{oed}Tk_0\end{bmatrix}\left\{q_A\right\}$
ε_z	$\dfrac{q_A}{\left(\lambda_c+2G_c\right)A_r+\left(\lambda_s+2G_s\right)\left(1-A_r\right)-2A_r\left(\lambda_c-\lambda_s\right)F}$	$\dfrac{q_A\left[2D+E_{oed}\left(C_2+T\right)\right]}{C_5E_{oed}}$
ε_r	$F\varepsilon_z^e$	$\dfrac{q_A\left(DK_\psi-k_0E_{oed}\right)}{C_5E_{oed}}$
F_R	$J\varepsilon_r^e$	$J\varepsilon_r^p$

Table 2: Equations for abbreviations used in Table 1

F	$\dfrac{\left(\lambda_c-\lambda_s\right)\left(1-A_r\right)}{2\left[A_r\left(\lambda_s+G_s-\lambda_c-G_c\right)+\lambda_c+G_c+G_s\right]+\left(1-A_r\right)\left(2G_s+\lambda_s\right)T}$
T	$\dfrac{J}{E_{oed}r_c}$
D	$\dfrac{E_c}{2+K_\psi K_{pc}-2\upsilon_c\left(1+K_{pc}+K_\psi\right)}$
C_2	$\dfrac{1-2\upsilon_s+A_r}{\left(1-A_r\right)\left(1-\upsilon_s\right)}$
C_5	$E_{oed}\left(1-A_r\right)\left(C_3+T\right)+D\left[\left(1-A_r\right)\left(C_1K_\psi+2\right)+A_rK_{pc}\left(K_\psi\left(C_2+T\right)+2k_0\right)\right]$
C_1	$\dfrac{2k_0A_r}{1-A_r}$
C_3	$C_2-k_0C_1$

When the applied load q_A exceeds the yield load, Eq. (1) becomes:

$$K_{pc}=\frac{\sigma_{zc,ini}+\Delta\sigma_{zc}}{\sigma_{rc,ini}+\Delta\sigma_{rc}}=\frac{\gamma_c z+\Delta\sigma_{zc}}{K_{ini}\gamma_s' z+\Delta\sigma_{rc}} \qquad (2)$$

where stress increments represent the sum of elastic and plastic response at the depth of interest. Until the criteria (Eq. (1)) is met, the stress state of the column will remain in the elastic state. When the criteria (Eq. (1)) is violated, the stress and strain state can be determined as a linear combination of elastic and elasto-plastic solutions (Table 1) with regard to the yield criteria given by Eq. (2). In the latter case the applied load can be presented as a linear combination of elastic *(e)* and plastic *(p)* load (Eq. (3)), where the plastic part of the load induces plastic strains inside the stone column.

$$q_A = q^e + q^p = \left(1-\delta\right) q_A + \delta q_A \tag{3}$$

$0 < \delta < 1$ represents the proportion of the plastic load. Following this principle the following relations for stresses can be obtained:

$$\begin{aligned} \Delta\sigma_{ij} &= \Delta\sigma_{ij}^e\left(q^e\right) + \Delta\sigma_{ij}^p\left(q^p\right) = \\ &= \left(1-\delta\right)\Delta\sigma_{ij}^e\left(q_A\right) + \delta\Delta\sigma_{ij}^p\left(q_A\right) \end{aligned} \tag{4}$$

where *i* = {z, r} and *j* = {c, s}.

By using equations for elastic and elasto-plastic stress increments from Table 1, Eq. (1) and Eq. (2) can be rewritten:

$$\begin{aligned} &\left(1-\delta\right)\Delta\sigma_{zc}^e + \delta\Delta\sigma_{zc}^p + \sigma_{zc,ini} - \\ &-K_{pc}\left[\left(1-\delta\right)\Delta\sigma_{rc}^e + \delta\Delta\sigma_{rc}^p + \sigma_{rc,ini}\right] = 0 \end{aligned} \tag{5}$$

and solved for *δ:*

$$\delta = \left\{ \begin{matrix} 0 \\ \dfrac{\sigma_{rc,ini}K_{pc} + \Delta\sigma_{rc}^e K_{pc} - \Delta\sigma_{zc}^e - \sigma_{zc,ini}}{\Delta\sigma_{rc}^e K_{pc} - \Delta\sigma_{rc}^p K_{pc} - \Delta\sigma_{zc}^e + \Delta\sigma_{zc}^p} \end{matrix} \right\} \tag{6}$$

When $\delta = 0$ for $\dfrac{\Delta\sigma_{zc}^e + \sigma_{zc,ini}}{\Delta\sigma_{rc}^e + \sigma_{rc,ini}} \le K_{pc}$ and else when $\dfrac{\Delta\sigma_{zc}^e + \sigma_{zc,ini}}{\Delta\sigma_{rc}^e + \sigma_{rc,ini}} > K_{pc}$.

Once *δ* is determined, stresses $\Delta\sigma_{ij}$ in the soil/column can be calculated as a linear combination of solutions, presented in Table 1 by using Eq. (4). Total stresses are calculated as:

$$\sigma_{ij} = \sigma_{ij,ini} + \Delta\sigma_{ij} \tag{7}$$

Vertical strain ε_z can be obtained similarly for any chosen depth:

$$\varepsilon_i = \left(1-\delta\right)\varepsilon_i^e\left(q_A\right) + \delta\varepsilon_i^p\left(q_A\right) \tag{8}$$

Finally, the total settlement of improved multilayered ground, which can in general be expressed as

$$u_z = \int_0^H \varepsilon_z dz \tag{9}$$

can be obtained numerically by summing up the strain contributions at various preselected depths (usually at 0.5 or 1 m).

3. MODEL VERIFICATION BASED ON CASE HISTORY DESCRIPTION

Designing GEC is a great challenge due to many material properties of the soil and column which can be difficult to determine. Back calculation of well documented case histories is useful and sometimes a necessity in the prediction of reinforced ground behaviour.

In this chapter measured settlements of four cases described by Alexiew & Raithel (2015) were back calculated with the proposed analytical method. Parameters for soil were mainly given in the original paper, while stone column parameters E_c and φ_c' were assumed and varied to approach the settlements measured in the field.

The stone column data (E_c and *φ'*) found in the literature are often back calculated based on the past case histories. Estimations for stone column modulus E_c range from 7 up to 58 MPa, with stone shear angle φ_c' ranging between 30° for sand columns and up to 50° for stone columns (Barksdale & Bachus, 1983). Another restriction concerning GEC elastic modulus is mentioned in literature as a ratio E_c/E_s. Authors normally use a value between 10 and 40. (Pulko et al., 2011), (Sexton et al., 2014).

3.1. Bastions West, Netherlands

3.1.1. General description

The landscape embankment was built on very soft soil in a new residential area in Houten-Zuid. The predicted settlements of non-treated ground were between 1.6 and 1.9 meters, which was unacceptable due to lack of time for such an extended consolidation and also because the adjacent building's foundation would be endangered (Alexiew & Raithel, 2015).

Figure 2. Embankment at Bastions West one year after construction (Huesker, 2013)

3.1.2. Soil and column properties

The soft ground on which the embankment was built consists of 7.5 meters of organic clay and peat. The parameters given by Alexiew & Raithel (2015) are listed in Table 3.

The load of 93.5 kPa was assigned from the embankment height of 5.5 meters and was assumed to be infinitely wide. Then E_{oed} was calculated by the following equation (Hardening Soil Model):

$$E_{oed} = E_{oed,ref}\left(\frac{c' ctg\varphi_s' + \sigma'}{c' ctg\varphi_s' + p_{ref}}\right)^m \tag{10}$$

$E_{oed,ref}$ represents a reference value of oedometer modulus of soil at a reference value of stress p_{ref} = 100 kPa (Brinkgreve R.B.J. et al, 2011).

Properties of columns are shown in Table 4. Stone columns at Bastions West were installed using displacement technique and filled with sand, which led to significantly lower values of the friction angle in comparison to the ones filled with gravel. Alexiew & Raithel (2015) proposed a value of φ' = 32.5°.

Two values were given for tensile stiffness of geotextile. J_d represents a value for long term stiffness and J_k a value for short term stiffness.

Table 3: Soil properties at Bastions West project

Soil layer	Depth [m]	γ_s' [kN/m^3]	$E_{oed,ref}$ [kPa]	p_{ref} [kPa]	c' [kN/m^2]	φ_s' [°]	m [-]	v [-]
Organic clay&peat	7.5	4	2000	100	20	2	1	0.4

Table 4: Properties of geotextile encased stone columns at Bastions West project

	Depth [m]	d [m]	γ_c' [kN/m^3]	v [-]	A_r [%]	J_d / J_k [kN/m]
GEC	7.5	0.8	9	0.3	15	2100/3500

3.1.3. Results and discussion

Settlements of improved ground were calculated according to the analytical method described in Chapter 2 and are presented in Table 5. Values outside the brackets were calculated for different combinations of stone column stiffness (E_c) and shear strength (φ_c') and various geotextile stiffness J_d and J_k (values in brackets).

Table 5: Settlements of treated ground [cm] at Bastions West calculated with the proposed analytical method

φ_c' [°] \ E_c [MPa]	8	10	12	15	20
32.5	**31 (29)**	**30** (27)	**29** (26)	28 (25)	27 (23)
35	30 (28)	28 (26)	27 (25)	26 (23)	25 (21)
40	28 (27)	26 (24)	24 (22)	23 (21)	21 (19)

Measured settlements at Bastions West were 30 cm on top of GEC and 32 cm in between GEC. It can be seen that by using modest values of E_c and φ_c' good agreement between calculated and measured settlements can be achieved. The modest values of E_c and φ_c' are also in good agreement with the values proposed by Alexiew & Raithel (2015), hence with values expected to mobilize inside a sand column.

3.2. Railroad embankment Bothnia line, Sweden

3.2.1. General description

The next case study is from Sweden, where a 190 km long high-speed railway line runs along Bothnia Bay. The route was opened in 2010 and it allows trains to travel as fast as 250km/h.

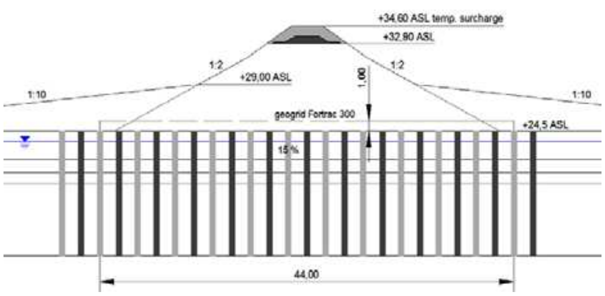

Figure 3: A typical cross section at Bothnia line GEC project (Alexiew & Raithel, 2015)

On a part of the route where the railway line crosses a valley of very soft soils GEC foundation was used to reduce great settlements. A typical cross section of the embankment is shown in Figure 3 (Alexiew & Raithel, 2015).

3.2.2. Soil and column properties

Soil properties used in the calculations are given in Table 6. In the case of Bothnia line, soil's E_{oed} of each layer was calculated using Hardening Soil Model following Eq. (10) in Chapter 3.1.3.

The load was assumed to be infinitely wide and was calculated from the embankment height. The value of 230 kPa was used for the analysis of settlements.

GECs were installed using the displacement method and filled with crushed rock basalt, very common for the area. Thus, a high internal friction angle of the column material was to be expected.

3.2.3. Results and discussion

Expected settlements of treated ground at Bothnia project were a combination of actual measurements and a prognosis based on them which resulted in approximately 44 cm. Based on the given number and the type of rock that was installed in stone columns Alexiew & Raithel (2015) proposed an internal friction coefficient of GEC $\varphi_c' = 45°$.

Table 6: Soil properties at Bothnia line project

Soil layer	Depth [m]	γ_s' [kN/m^3]	$E_{oed,ref}$ [kPa]	p_{ref} [kPa]	c' [kN/m^2]	φ_s' [°]	m [-]	v [-]
Clay, silty	2.75	6	1300	100	3.5	30	0.5	0.4
Silt, clayey	1.25	6.5	1100	100	2.5	30	1	0.4
Clay, silty	1.00	5	700	100	2	30	1	0.4
Clay, silty	0.85	7	1200	100	1.75	30	0.9	0.4
Clay, silty	1.65	7	800	100	1.75	30	1	0.4

Table 7: Properties of geotextile encased stone columns at Bothnia Line project

	Depth [m]	d [m]	γ_c' [kN/m^3]	v [-]	A_r [%]	J_d / J_k [kN/m]
GEC	7.5	0.8	9	0.3	15	2100/3500

Table 8: Settlements of treated ground [cm] at Bothnia line calculated with the proposed analytical method

φ_c' [°] \ E_c [MPa]	10	15	20	25	30
35	62	54	50	47	45
40	57	48	43	40	38
45	56	44	38	35	32

In the calculations shear angle φ_c' and column modulus E_c were varied to see which combination gives calculated settlements similar to those measured on the site. The results are gathered in Table 8.

By looking at the results from a GEC designer's perspective, the best choice for E_c according to the proposed value of $\varphi_c' = 45°$ would be 15 MPa, which gives the ratio of E_c/E_s between 35 and 21 for the stiffest and the softest soil layer, respectively. These ratios are in agreement with the common values presented in the beginning of Chapter 3, hence confirming our choice.

When designing GECs a bit more conservatively, a combinations of 40° – 42.5° for φ_c' and 15 – 20 MPa for E_c would also predict settlements in agreement with the measured ones.

3.3. Hamburg, Germany

3.3.1. General description

The extension of Airbus site at Mühlenberger Loch in Hamburg was the biggest GEC project ever accomplished in Germany until the end of the year 2015. By enclosing an area of extremely soft soils and building a 2.4 km long dyke to protect the site form tide, around 60,000 GECs were installed in a total length of about 650 km. (Alexiew & Raithel, 2015)

Figure 4: Airbus site at Mühlenberger Loch in Hamburg (Fit Fuer Innovation)

3.3.2. Soil and column properties

Properties for soil and GEC used in the calculations are presented in Table 9 and Table 10. In a typical cross section layers of sludge, clay and peat exchange depth-wise.

3.3.3. Results and discussion

Measured settlements at the end of primary consolidation were approximately 105cm. Calculated settlements are shown in Table 11. They are calculated for different combinations of stone column friction angle φ_c' and elastic modulus E_c. Results close to the measured ones are colored blue.

Table 9: Soil properties at Hamburg Airbus site

Soil layer	Depth [m]	γ_s' [kN/m^3]	$E_{oed,s}$ [kPa]	v [-]
Sludge	2.1	4	450	0.4
Clay	1.3	6	600	0.4
Peat	2.8	1	550	0.4
Clay	1.8	6	600	0.4

Table 10: Column properties at Hamburg Airbus site

	Depth [m]	d [m]	γ_c' [kN/m^3]	v [-]	A_r [%]	J [kN/m]
GEC	8	0.8	10	0.3	15	2800

Table 11: Settlements [cm] of treated ground at Hamburg Airbus site calculated with the proposed analytical method

φ_c' [°] \ E_c [MPa]	6	8	10	12	14
35	128	113	**104**	97	92
40	120	**103**	92	85	79
45	120	99	85	77	71

The results direct us into choosing lower values of E_c (8 – 10 MPa) and combining them to a friction angle between 35° and 40°. Since the soil's E_s is very low, ranging from 200 – 300 kPa, lower values of E_c inside a stone column are more realistically expected. In order to effectively asses the material parameters which ought to be chosen during project planning, the creep should also be closely investigated. Measured settlements at Hamburg airbus site progressed with time for further 35 cm due to creep behavior of these extremely soft soils.

3.4. Jordanovo, Poland

3.4.1. General description

During construction works at a section of a highway in Poland the first "State of the art" project of GEC installation was successfully executed. Maximum depths of soft soil encountered were up to 28 m. The upper 5 meters of the soil consisted of peat and bellow it a layer of sensitive soil named gyttja was found (Alexiew & Raithel, 2015).

Coupling a quite flat final embankment geometry with the proposed highway speed limit of 130 km, a serviceability limit state was very strict and thus maximum allowed post-construction settlements were extremely low. Due to the mentioned reasons a temporary preload was applied to accelerate the consolidation process (Alexiew & Raithel, 2015).

3.4.2. Soil and column properties

The parameters for soil and for columns used in the model are gathered in the tables below. The total load of 114 kPa was calculated from the embankment height. Since the value of the geotextile modulus was not given in the original source, two different values were adopted for the analysis, i.e. 2500 kN/m and 3000 kN/m.

Table 12: Soil properties at Jordanovo site

Soil layer	Depth [m]	γ_s' [kN/m^3]	$E_{oed,s}$ [kPa]	v [-]
Peat	5	1	500	0.3
Gyttja	23	4	750	0.3

Table 13: Properties of geotextile encased stone columns at Jordanovo site

	Depth [m]	d [m]	γ_c' [kN/m^3]	v [-]	A_r [%]	J_d / J_k [kN/m]
GEC	28	0.8	10	0.3	15	2100/3500

3.4.3. Results and discussion

In comparison to expected settlements of 230 cm, the maximal settlements measured at Jordanovo site were only 105 cm. Besides a probable compression modulus underestimation it was assumed that the difference also occurred because the settlements caused during the GEC installation were not considered. These settlements are worth mentioning, since due to the length of columns which led to a time consuming installation process, heavy equipment was stationed in one position for a considerable amount of time (Alexiew & Raithel, 2015). For this reason the calculated highlighted settlements in Table 14 include values that deviate up to 25% from those measured on site.

Table 14: Settlements of treated ground [cm] at Jordanovo site calculated with the proposed analytical method

φ_c' [°] \ E_c [MPa]	10	15	20	25	30
35	172 (164)	151 (143)	140 (131)	134 **(124)**	**129 (119)**
40	151 (146)	**128 (122)**	**116 (109)**	**108** (100)	102 (95)
45	141 (139)	**112 (108)**	98 (93)	89 (84)	8 (77)

Due to many uncertainties in the description of this project, it is difficult to give a trustworthy conclusion. By assessing calculated settlements and simultaneously keeping in mind the suggested values for E_c and φ_c' from Chapter 3, only rough assumptions can be drawn as in keeping the value of E_c under 15MPa and a value of φ_c' around 40°.

4. CONCLUSIONS

Based on analyzed case history projects it can be concluded that the method for ground improved with GEC is capable of yielding good settlement predictions. Like in all similar geotechnical situations it is clear that the selection of the input parameters is of key importance for the credibility of results. Therefore, their validation in the laboratory and on the field is crucial for effective and safe design of GEC.

From the analyzed cases we can conclude that in extremely soft soils (like in the Hamburg case) their very low stiffness modulus E_s leads to relatively low stone stiffness E_c (between 6 and 8 MPa). When the elastic modulus of soil increases, so does the one in GEC, like in the case of Bothnia line (E_c = 15 MPa).

When analyzing stiffness modulus ratio E_c/E_s, all results stay inside boundaries found in literature (E_c/E_s = 10 – 40) which indicates our choices of E_c are valid. The only exception is ratio values in the Jordanovo case. This could be explained with already mentioned and very probable underestimation of the soils' modulus.

The expected shear strength of the column is much lower for sands (32°, i.e. Bastions West) than for stone (gravel) material columns (40° – 45°, i.e. Bothnia Line), as suggested by Alexiew & Raithel (2015) and confirmed with our back analysis.

For the future development of GEC it would be essential to monitor new GEC projects carefully and to share the collected data with the scientific community, since case history project analysis offers an excellent opportunity to verify analytical methods and input parameters.

NOTATION

The following symbols are used in this paper:

Subscripts / superscripts

c, s	column, soil
r, z	radial, vertical coordinate
e, p	elastic, plastic
ini	initial value

Symbols

A_r	replacement ratio
c	cohesion
C_1, C_2, C_3, C_5	material/geometrical constants
D	material constant
E_{oed}	eodometer modulus of soil
E	elastic modulus of soil / column
F	material/geometrical constant
F_R	encasement hoop force
G	shear modulus of soil / column
J	geosynthetic encasement stiffness
k_0	coefficient of earth pressure at rest
K_{ini}	initial lateral pressure coefficient after columns installation
K_{pc}	passive earth pressure coefficient

K_{ψ} dilation constant

T dimensionless encasement stiffness

q_A applied load

u_z total settlement of improved ground

δ proportion of the plastic load

γ unit weight

λ Lame's parameter

ε strain

σ stress

$\Delta\sigma$ stress increment

υ Poisson's ratio

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