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BACK ANALYSIS OF SOFT CLAY BEHAVIOUR UNDER HIGHWAY EMBANKMENTS

Zs. Rémai*
Department of Geotechnics, Budapest University of Technology and Economics, Hungary

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ABSTRACT

The experiences of embankment settlement measurements and the correlation of CPT resistance and deformation properties of soft Holocene clays are discussed in this paper. Embankment settlement measurement results of nine different sites have been studied, the significant parameters have been defined, and the deformation properties have been back calculated. The correlation between the determined deformation properties (Eoed, E50) and the obtained CPT results are evaluated and an empirical coefficient α is determined for each test. The scatter of this coefficient is determined and a formula is proposed to obtain more reliable α values for the studied Holocene clays.

Keywords: Deformation properties; oedometer modulus; embankment settlement; CPTu; empirical correlation

1. INTRODUCTION

In the last decade many embankment settlement measurements have been performed at several highway projects in Hungary. These monitoring results provide useful feedback during construction and also enable back analysis of the soil behavior. However, at most sites the diverse stratification makes the back analysis extremely difficult and complex; at these sites the subsoil is composed of many different thin layers, all having different deformation characteristics. Therefore a settlement calculation requires either a vast number of input parameters or estimation of averaged deformation properties.

Nevertheless there were a few sites at the highway M43 construction where the stratification was very simple; only three or four layers were located in the significant soil zone. At these sites the back analyses of embankment settlement provide useful information about the in situ behavior of the soil, thereby the preliminary in situ and laboratory test

* E-mail address of the corresponding author: remai@mail.bme.hu (Zs. Remai)

results can be evaluated and compared to the monitoring results.

This paper summarizes the experiences gained during the back analysis of the embankment settlement; special emphasis is given to the deformation characteristics of the soft clayey soils located near surface and their correlation with CPT results. Cone penetration test, being a popular and widely used soil exploration tool [1] is frequently used for many geotechnical purposes, such as pile capacity prediction [2, 3], soil classification [4, 5] or shallow foundation design [6, 7]. One of the earliest and most popular CPT application is to obtain design parameters for soft soils [8, 9] which is again in the focus of research nowadays as more and more advanced parameters are used to describe soil behaviour [10, 11].

2. TEST SITES, SETTLEMENT MONITORING, SOIL CONDITIONS

2.1. Test sites, settlement monitoring

Nine embankment settlement measurement and the relevant laboratory and in situ test results are discussed in this paper. The test sites are located near the Hungarian-Serbian border, in the southern part of the Great Hungarian Plain, the "Nagy Alföld". The cross sections where the settlement measurements were performed and the basic geometrical data of the embankments are summarized in Table 1. The base widths of the embankments varied between 34 and 90 m, the heights between 5 and 16 m. The settlements had been monitored for nearly two years; the stopping criterion of the measurements was to have a settlement rate as low as 1.0 cm/month, but some control measurements had been performed even after reaching this rate. The monitoring was finished 9−12 months after fulfilling the requirement, at that time the settlement rates were about 0.05−0.10 cm/month. The typical results of the settlement measurements and the consolidation process are illustrated in Figure 1 and Figure 2, respectively.

Table 1: Measurement locations and embankment dimensions

Cross section	**Crown width [m]**	**Base width [m]**	**Height [m]**
3+145	30	54	9
3+212	30	54	8
6+440	26	52	5
9+059	30	78	12
9+183	30	90	16
0+268	8	50	8
0+336	8	34	8.5
52+171	16	48	9.5

52+229	16	48	10

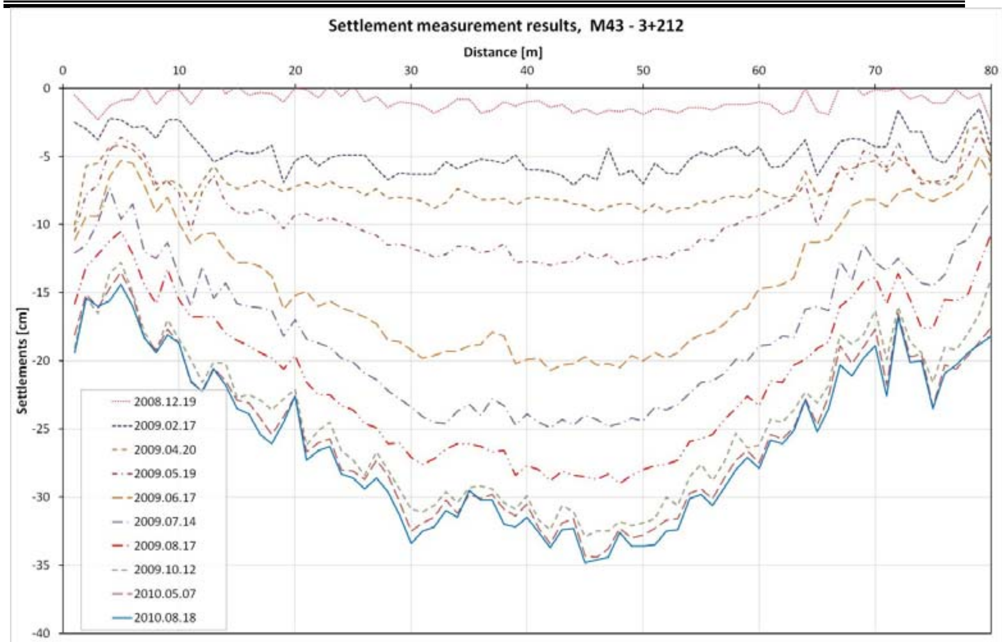

Figure 1. Settlements below the embankment

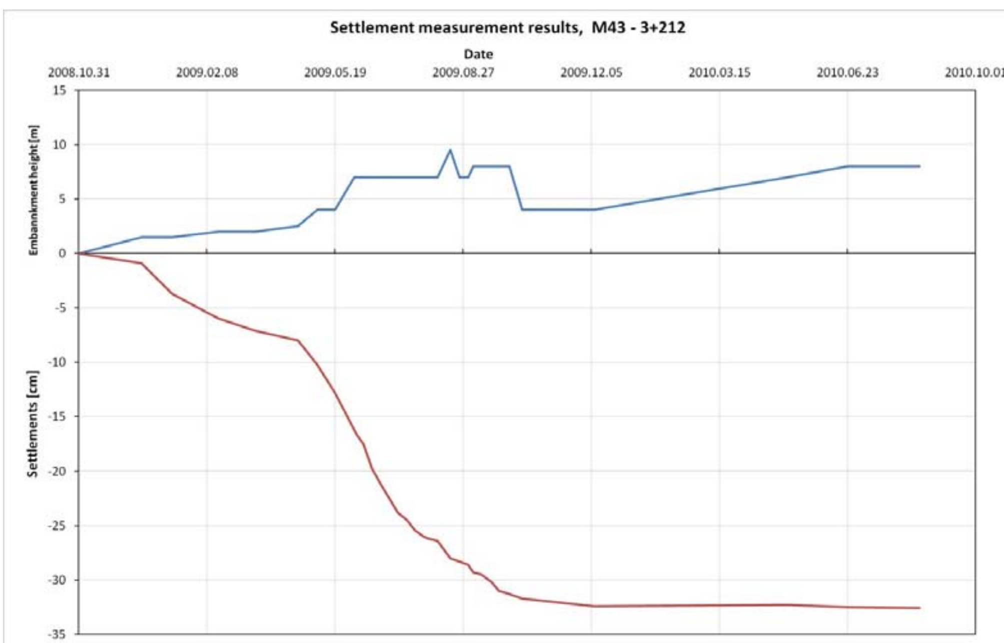

Figure 2. Progress of construction and consolidation

2.2. Soil conditions

At each site drillings and CPTu tests were performed to explore the soil conditions. The stratification was very similar at each site: a soft clay layer was located below the surface, underlain by less compressible deposits. The thickness of the compressible clay layer varied between 10.6 and 18 m. The index properties and the typical CPT resistances of the soft clay layer are summarized in Table 2.

Table 2: Properties of the soft clay

Cross section	Plasticity index, I_p [%]	CPT tip resistance, q_c [kPa]	CPT friction ratio,R_f [%]
3+145	15−28	1150	2.54
3+212	14−31	1140	2.55
6+440	16−23	1170	2.86
9+059	15−36	1490	3.15
9+183	19−30	1220	3.31
0+268	16−20	1160	3.25
0+336	15−20	1090	2.45
52+171	12−33	1210	3.12
52+229	18−36	1320	3.15

The underlying deposits consisted of medium dense granular soils (sand, silty sand) and semi solid clay layers. Because of their deeper location and moderate compressibility these layers were of less importance from the settlement point of view.

3. FINITE ELEMNT ANALYSES

The calculation of embankment settlement requires proper understanding of the soil's stress-strain characteristics and adequate modeling of its behavior. The latter includes not only the selection of a suitable soil model and its parameters that fit the experimental results, but also accounting for the thickness of compressible soil and other factors affecting the limiting depth. The soil deformation below the limiting depth is considered to be insignificant; therefore the bottom of the finite element model is generally defined at this depth. A previous study [12] has shown that determining the limiting depth in a reliable way is a complicated task, and that the "hardening soil with small strain" or shorter HSsmall [13] model enables a fairly reliable settlement prediction and the calculated results are nearly independent on the depth of the geotechnical FEM model. Thus the HSsmall soil model was used in the calculations.

3.1. Model parameters

The HSsmall soil model requires the following six deformation parameters:

- E_{oed}^{ref} is the oedometer modulus at a particular reference stress.
- m is an exponent of the power function which describes the stress-strain relationship for oedometric conditions.

- E_{50}^{ref} is the secant stiffness in standard drained triaxial test, at 50% of the final deviatoric stress.
- E_{ur}^{ref} is the secant stiffness modulus for unloading and reloading.
- G_0 is the small strain or dynamic shear modulus (for $\gamma_s < 10^{-6}$).
- $\gamma_{0.7}$ is the shear strain level at which the secant shear modulus reaches $G=0.7G_0$.

In addition to isotropic stress hardening, this soil model takes into account the much stiffer soil behavior in the small strain range. This stiffening is described by a stiffness modulus degradation curve, where the stiffness modulus is a function of the actual shear strain increment [14]. It has been found that the normalized curves for different soil types have a very similar shape, and the curve can be defined by two parameters: the small strain shear modulus, G_0, and the shear strain at which $G=0.722G_0$, i.e. $\gamma_{0.7}$. These parameters can be determined by specific laboratory or in situ tests, most typically resonant column, torsional shear, seismic CPT and downhole or crosshole tests are used. Unfortunately, at the investigated sites no such tests were performed, so the small strain parameters were estimated using empirical correlations.

The G_0 values were obtained based on the correlation proposed by Mayne and Rix [15], which estimates the small strain (or dynamic) shear modulus based on the CPT tip resistance q_t and the void ratio e:

$$G_0=49.4 \cdot q_t^{0.695} \cdot e^{-1.13} \tag{1}$$

In addition to G_0, it is the shape of the degradation curve that defines deformation characteristics in the small strain range. As mentioned above, this can be described with the parameter $\gamma_{0.7}$.

There are many factors that have an effect on the degradation curve (e.g. confining stress, loading history, void ratio, plasticity), and consequently many empirical correlation exist for estimating it. For this study the results of Wichtmann and Triantafyllidis [16] were used to estimate $\gamma_{0.7}$ of granular soils, and the results of Benz [17] were used for cohesive soils.

The common geotechnical parameters (such as oedometer or Young modulus) were obtained based on the available laboratory test results.

3.2. Back analysis

The sites adopted for the current investigation share a common feature: the majority of the soil deformations, about 70−90% of the total settlement, develop in the first, upper clay layer. Thus the deformation of the underlying layers has only slight influence on the measured settlements. Therefore the deformation properties of these layers were kept constant during the back analysis and the aforementioned correlations were used to obtain the most probable parameters. Thus the variables of the back analysis were limited to the deformation parameters of the soft clay.

The nature of the loading process (i.e. there's no unloading) implies that E_{ur}^{ref} does not affect the calculated parameters, so this value was not changed for the analyses. The small strain parameters (G_0, $\gamma_{0.7}$) neither affect the analysis results, because the typical strain range in the clay layer varies between 1% and 10%, and no significant small strain stiffening can be expected in this strain range, so these parameters were kept constant too.

Hence the number of varying parameters reduced to three: E_{oed}^{ref}, E_{50}^{ref} and m. Before the detailed back analysis, the influence of these parameters on the calculated settlements (i.e. shape of the settlement curve) has been investigated.

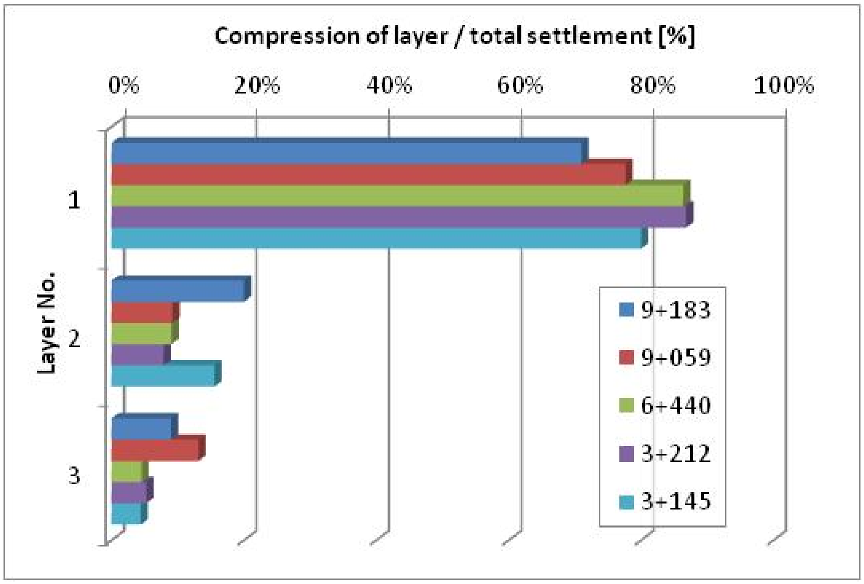

Figure 3. Compression of layers

3.2.1. Influence of the E_{50}/E_{oed} ratio on the computed settlements

The values of E_{50} and E_{oed} describe the stress-strain relationship in different stress states (triaxial and K_0 respectively). Consequently, reducing their values results in larger settlement and increasing their values results in smaller settlements. On the other hand, the ratio of the two values, which actually defines a stress dependent Poisson ratio, influences the result in a more complex way. A simplified parametric study has been performed to evaluate the effect of different E_{50}/E_{oed} ratios.

Five parameter combinations have been used to calculate the settlements. The values have been chosen in a way that the mean of E_{oed} and E_{50} was kept constant, but their ratios were set to 0.6, 0.8, 1.0, 1.2 and 1.4. The vertical displacement values have been obtained for a horizontal section at the depth where the settlement measurements were performed. The results of the calculations are shown in Figure 4 with the measured settlements also indicated. Different E_{50}/E_{oed} ratios results in different settlement values: an increasing ratio leads to larger maximum settlements, but the shapes of the curves remain still quite similar. In order to compare the shapes, normalized settlement profiles were computed by dividing the calculated settlement values by their maximum value of the same analysis. The results (Figure 5) show that the ratio has only minor effect on the shape of the settlement curve. This fact implies that any of these ratios can be used when there is no detailed information (e.g. laboratory test results) on the deformation characteristic.

In such cases the use of $E_{50}=E_{oed}$ can be the simplest choice, because the same empirical correlation can be used to estimate both parameters, furthermore the software manual also recommends to use this ratio when no other data is available. A proposal for such empirical correlation is given in the next paragraph.

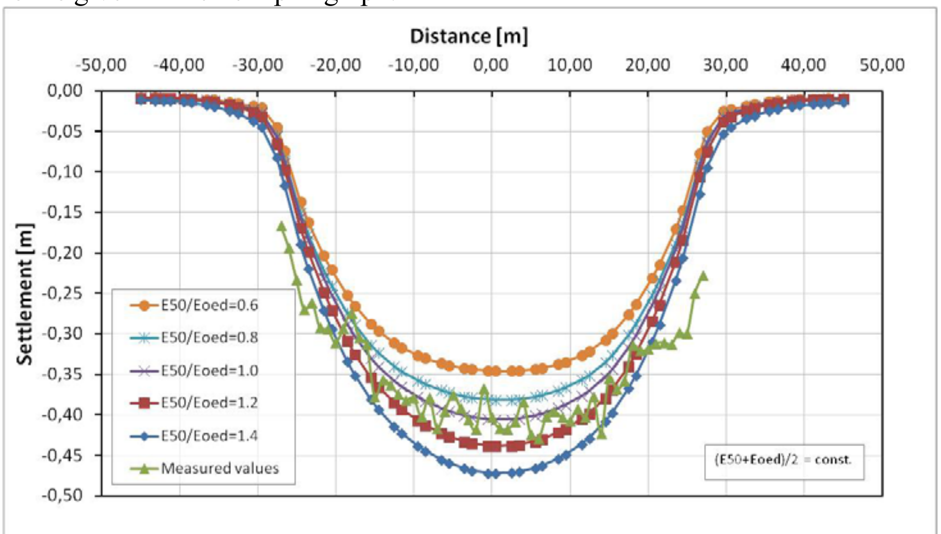

Figure 4. Effect on E_{50}/E_{oed} ratio on the calculated settlements

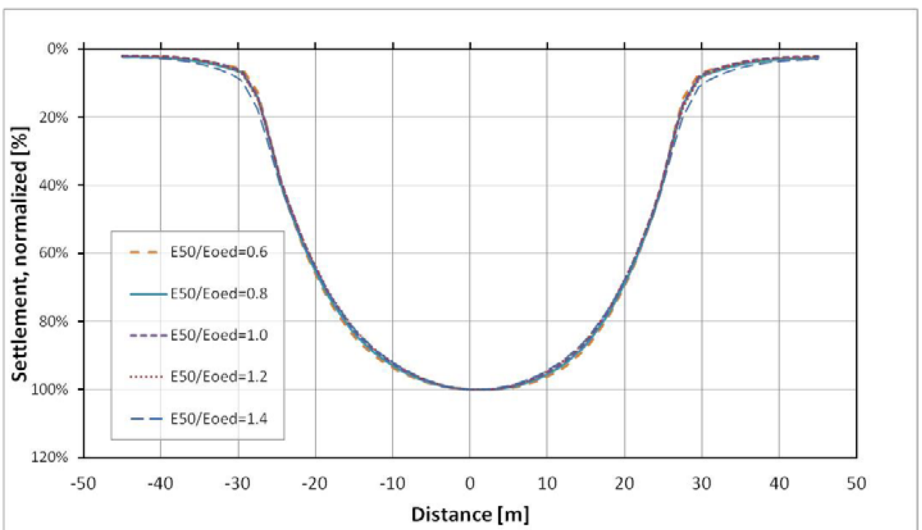

Figure 5. Effect of E_{50}/E_{oed} ratio on the shape of the normalized settlement curves

3.2.2. Influence of the exponent of power function ("m") on the computed settlements

A similar analysis was performed to evaluate the effect of the power function's exponent, and similar trends were observed (Figure 6): it does not have significant influence on the calculated settlements, neither on the maximum value nor on the shape of the settlement curve. It must be also noted that the chosen reference stress can play an important role in the analysis; if it is set improperly changing the exponent can cause significant differences. If the reference stress is chosen in accordance with the stress level of the loading process, the results are not largely influenced by the exponent. For the calculations, the exponents were obtained based on the recommendations of Viggiani and Atkinson [18] and were kept constant.

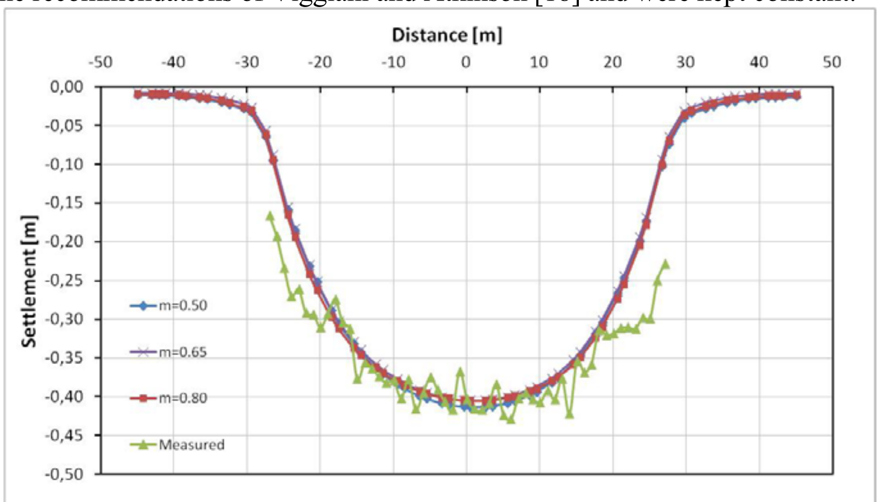

Figure 6. Effect of the power function's exponent, "m" on the shape of the normalized settlement curves

3.2.3. Varying parameters of FE analyses, evaluation criterion

As it has been shown earlier, the settlements are most importantly influenced by the oedometer modulus (E_{oed}^{ref}) and the secant modulus from the triaxial tests (E_{50}^{ref}). It has also been shown that the ratio of these values has some, but not a significant influence on the calculated results, so the same values have been used for the two parameters.

The measured settlement values show significant scatter, therefore the mean of the measured and calculated settlements were calculated for the zone within 10m distance from the embankment axis. These average values were compared, and the parameters of the back analyses were chosen in a way that the measured and calculated mean settlements are the same.

3.3. Correlation of deformation parameters and CPT resistance

The CPT results are commonly used to obtain design parameters of soft clays such as strength [19] and deformation properties. The correlation between oedometer modulus and CPT tip resistance can be expressed by the following formula proposed by Sanglerat [20]:

$$E_{oed} = \alpha \cdot q_c \qquad (2)$$

where α is an empirical coefficient depending on soil type. Sanglerat [20] recommended the use of α=2−5 for soft clays (700kPa < q_c < 2000kPa). There are many similar correlations available, as well in international [21, 22] and in Hungarian literature [23, 24]. However, these correlations either define a wide range for α values or apply to certain geological conditions which differ from the current situation (e.g. stiffer clays).

The α values have been back calculated for each site, the results are summarized in Table 3. The values varied over an acceptably narrow range: between 3.62 and 5.14. The mean of the experienced values is 4.25; these results show good agreement with earlier studies and provide a narrower range for the analyzed soft Holocene clays.

Further investigations have been carried out in order to analyze if the empirical “α“ factor could be estimated in a more reliable way, as a function of another soil property. A good correlation has been found between the α values and CPT friction ratio (R_f). In the case of soils having larger friction ratios, smaller α values have been observed, and lower values came with larger friction ratios. (Figure 7). The best fitting line can be expressed by the following formula:

$$\alpha = 8.3 - 138 \cdot R_f \tag{3}$$

Table 3: Calculated α values

Location	CPT tip resistance q_c [kPa]	Friction ratio R_f [%]	Oedometer modulus, secant stiffness E_{oed}, E_{50} [kPa]	E_{oed}/q_c ratio α [-]
3+145	1 150	2.54%	5 200	4.52
3+212	1 140	2.55%	5 500	4.82
6+440	1 170	2.86%	5 500	4.70
9+059	1 490	3.15%	5 400	3.62
9+183	1 220	3.31%	4 500	3.69
0+268	1 160	3.25%	4 700	4.05
0+336	1 090	2.45%	5 600	5.14
52+171	1 210	3.12%	5 000	4.13
52+229	1 320	3.15%	5 400	4.09

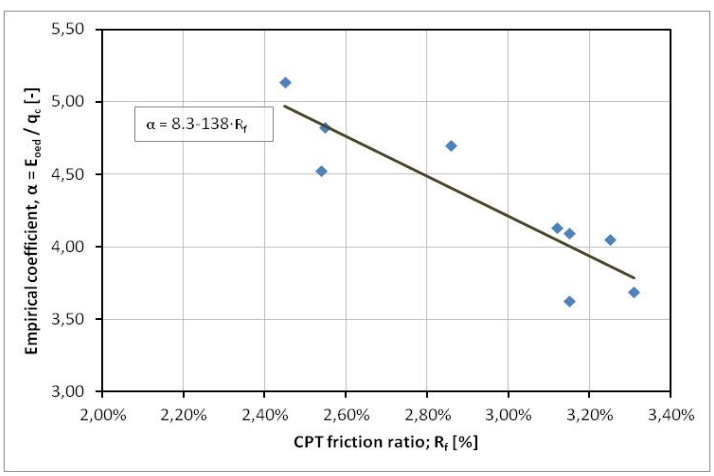

Figure 7. CPT friction ratio (R_f) vs. back calculated α coefficients

Taking into account a friction ratio dependent α value (i.e. obtaining the value by equation 2) enables a more reliable estimation of oedometer modulus, E_{oed}. The oedometer modulus values have been calculated by using the mean value of α (α=4.25) and the empirical coefficient values obtained by equation 2. The results are plotted against the back calculated E_{oed} values (Figure 8).

When using the mean α=4.25 value, the calculated oedometer moduli vary over a range of ±20%,. This range decreases to ±10% when using equation 2 to obtain a friction ratio dependent coefficient. Thus using this equation enables a more reliable estimation of the oedometer modulus.

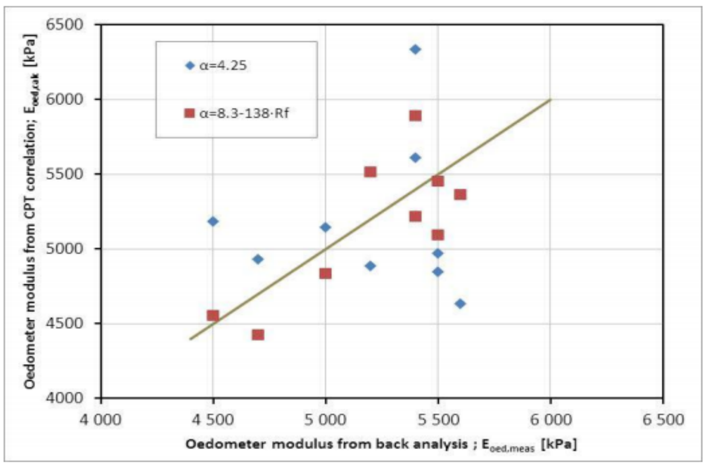

Figure 8. Measured vs. calculated oedometer modulus

4. CONCLUSION

The embankment settlement measurement results and their FEM analysis possibilities have been discussed in the paper. Special emphasis has been put on the deformation characteristics of soft Holocene clays, which constitutes the 10−18 m thick cover layer at each site, so the major portion of the deformation developed in this layer. Based on the conclusions of a previous study, the "HSsmall" soil model has been used to model the soil behavior. Regarding its parameters, the following conclusions have been drawn: (1) the exponent "m" of power function describing the stress–level dependency of stiffness does not influence significantly the shape of the settlement curve; (2) the E50/Eoed ratio slightly affects the magnitude of the settlements, but has only a minor effect on the shape of the settlement curve. These conclusion are valid only, if the reference stress is chosen properly.

Back analyses of the embankment settlement measurements have been carried out considering the ratio E50/Eoed=1.0, and the soft clays' oedemeter moduli have been determined.

An empirical correlation has been proposed between the back calculated oedometer moduli and cone penetration resistance for soft Holocene clays in the Carpathian Basin. Using the proposed value α=4.25, the oedometer modulus can be estimated with an accuracy of ±20%.

It has also been stated that the empirical coefficient, α can be expressed as a function CPT friction ratio (equation 2). Using this empirical correlation, the oedometer modulus can be estimated in a more reliable way. This empirical correlation improved the accuracy to ±10%.

Such estimation methods provide a helpful guideline when there is no other data available about the deformation characteristics of the soil, or when more information is needed to assess the uncertainty of the deformation of a certain layer. Thus it enables to have more realistic and more detailed information about the soil behavior.

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