Soil Data Inflation in Analysis of Settlements and Tilts of Structures

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ABSTRACT: During geotechnical investigations (GI) geologists test only "infinitesimal" volumes of soil. GI operations i.e., soil sampling, in situ and lab tests, data interpretation, etc. distort the data, and then the geologists subjectively "inflate" this fuzzy data further on over the whole subsoil volume. The geologists are paid only ≈0.05-0.1% of the total capital construction cost for their efforts. Luckily, most structures are highly insensitive to these uncertainties with the exception of soil local disruptions (cracks, shears), which are usually neglected. The paper describes a simple computerized method to evaluate these uncertainties by determining the representative number of boreholes and the respective values of settlements, tilts and their scatter.

1 INTRODUCTION

During geotechnical investigations (GI) geologists test “infinitesimal” volumes of subgrade ($\approx 10^{-6}$ fractions of the total subgrade volume), GI cost only 0.05-0.1% of the total capital construction cost (Barvashov, 2007). All tests and operations in situ or in laboratory: drilling holes, CPT, soil sampling, transportation, preparation of samples for laboratory tests, the tests per se, etc. degrade soil properties. The geologists creatively “inflate” this fuzzy GI soil data into continuous stratification on several cross sections. Then the structural engineers (designers) “inflate” the GI data between cross sections. They often reduce the values of soil parameters to be “on the safe side”. However, this is often unsafe. But GI data is so fuzzy that it can differ much even between very closely located boreholes, as well (Figure 1).

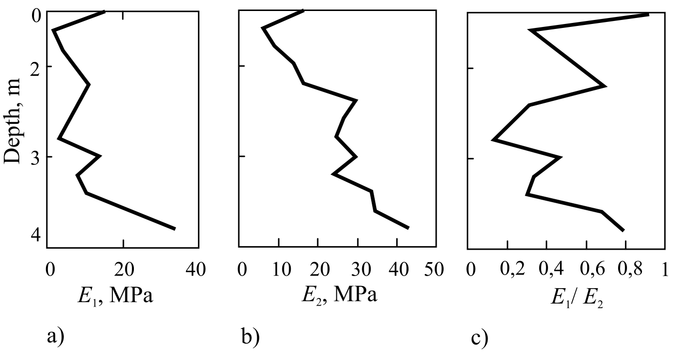

Figure 1. Real dilatometer moduli profiles in test holes, spaced 1.5 m from each other, and their ratio: a - deformation modulus E_1 profile versus depth in the 1st test hole; b – E_2 profile in the nearby 2nd test hole; c – E_1/E_2 ratio profile

Figure 1 shows that the values of deformation moduli E_1, E_2, E_1/E_2, measured in closely spaced test holes (at 1.5 m), differ very much. It is so for c and φ either.

Most structures are “robust” and “insensitive” i.e., do not ”feel“ soil data scatter, and even simplified subsoil models yield acceptable results. “Don’t save on footings” wisely advised Peter I (Russia’s Emperor, XVIII century). However, failures of structures are very rare and are mostly due to scarce and erroneous GI data, human error and neglect of formation of soil distruction (“plastic”) zones under footing edges. The latter ones were firstly visualized by Mikheev et al. who pushed a 10 cm wide test plate into transparent paraffin (Figure 2). 50 years later Photo Imaging Velocimetry (PIV) technique made it possible to obtain much better images (Figure 3).

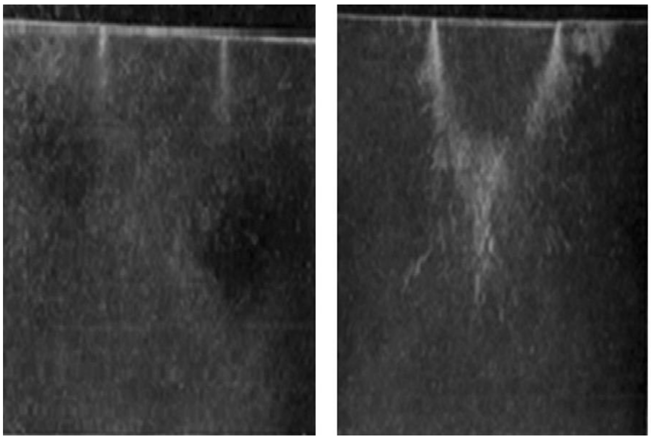

Figure 2. Plastic zones under the edges of 10 cm plate, pushed into highly transparent paraffin

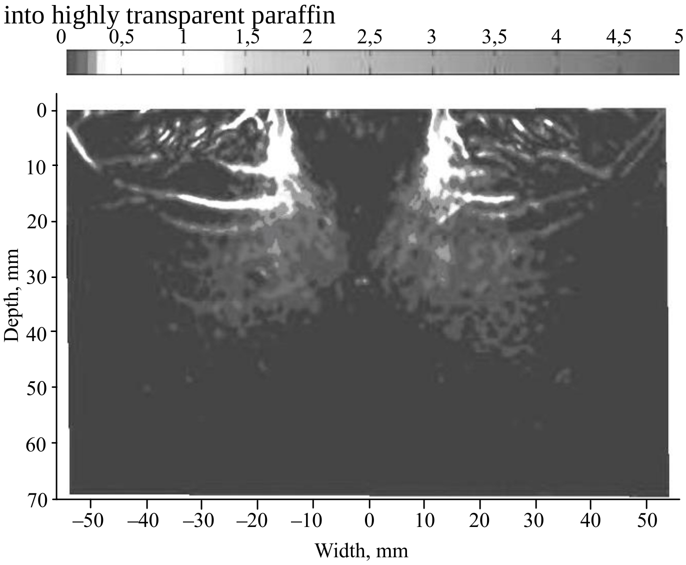

Figure 3. Shear cracks under a test plate in clay

FEM software (PLAXIS) can simulate elastoplastic soil behavior in shear only for very fine FEM grids (on the right). It does not “see” such shear cracks if FEM grid is not fine enough. Figure 4 shows a deep “crack” under the right side of a footing (very fine FEM grid) and almost no shear on the right (rough FEM grid). A simple method (see below) to assess the depth of such “crack” is described below.

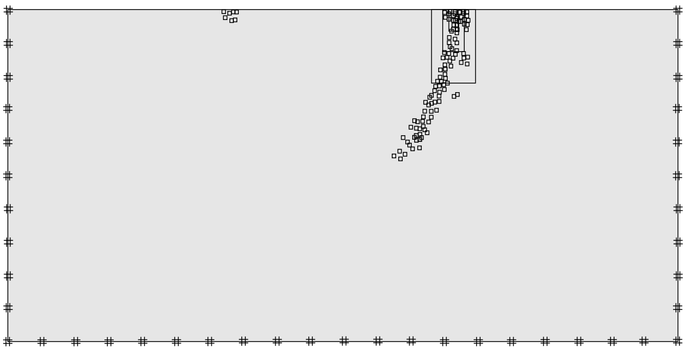

Figure 4. “Cuts” in sand

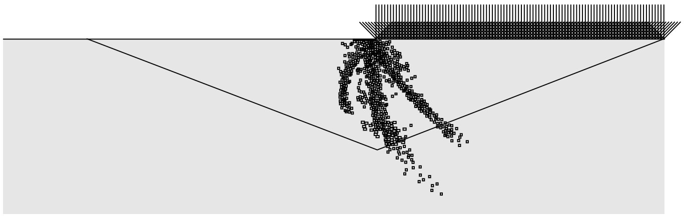

Figure 5. “Cuts” in clay

Fig. 5 shows Plaxis analysis output with similar shear cracks in clay soil. They look even more intricate than the images above on Figure 3. The “cuts” block distribution capacity of the layer below the footing i.e., this layer behaves in the same way as a Winkler layer. Therefore, if any subgrade elastic model such as elastic space is covered with a Winker springs layer then it would simplify and improve analysis of subsoil-structure interaction. A Winkler model with laterally variable subgrade ratio is might be also valid.

1 THEORY

The paper describes a new analytical method for evaluating virtual settlements and tilts of a designed structure, depending on the number of drilled GI boreholes and the soil data volume. The method applies the simplest Winkler model with non-uniformly distributed subgrade modulus $K=K(x,y)$, calculated by extrapolating the scarce data from N boreholes. Firstly, $K_1, K_2, \ldots K_N$ values are calculated on the tops of N boreholes. Then K_i values are extrapolated over the whole of the subgrade surface with the help of Shepard continuous approximation function $K(x,y)$ (Shepard, 1968), having a free parameter of shape n. Variation of n yields different distributions $K=K(x,y)$ that simulate the fuzziness of the approximation between the test points. The scatter of settlements and tilts is determined in order to decide if the current number of boreholes is representative. The algorithm was programed in MathCad, and two numerical cases for E, c and φ distributions in 5 and 9 boreholes were analyzed. The analysis showed that the values of mean settlements have low sensitivity to the differences of boreholes number while the tilts are rather sensitive.

2 ANALYSIS OF STRUCTURE SETTLEMENTS, ACCOUNTING FOR “DATA INFLATION” AND “CUT”

The proposed method is illustrated by the following case of a stiff rectangular 40x20 m structure, loaded uniformly with q=300 kPa to be built on highly heterogeneous subsoil, both horizontally and vertically. E, c, φ vertical profiles are given in 9 boreholes, uniformly spaced 20x10 m over the subsoil surface with one borehole in the center and other ones – at the corners and at the sides midpoints. The vertical profiles of E, c and φ values in boreholes were computed by a random numbers generator and shown on Figure 6 below. (We could not get “real” data from geologists, only “inflated” data was available).

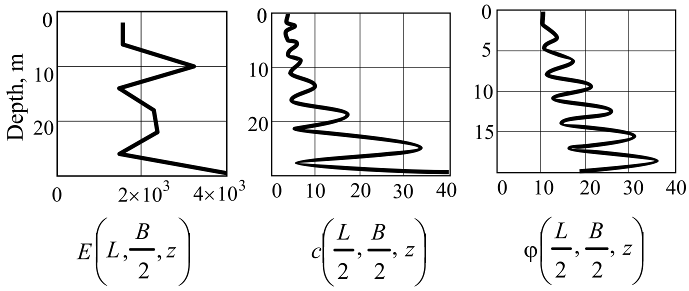

Figure 6. An example of E, c φ vertical profiles in the center of the L = 40 and B = 20 m stiff rectangular structure

Subsoil settlements (S) at points under uniformly distributed load q below the structure, are calculated with the help of the equation for determination the depth of the cut (plastic zone under the edge of the foundation):

$$Z_{\max} = \frac{q-\gamma h}{\pi\gamma}\left(\cot\varphi + \varphi - \frac{\pi}{2}\right) - \frac{c\cdot\cot\varphi}{\gamma} - h \quad . \qquad (1)$$

where q = pressure under the footing edge; h = the footing depth; c, φ, γ = soil parameters.

As per equation (1) the Mohr-Coulomb plasticity criterion is fulfilled just along the boundary of the egg-shaped zone while it is exceeded inside this zone that is impossible. This equation give realistic solution only in the case if the plastic criterion is fulfilled just in one “plastic” point, above which a fracture appears as it happens in the experiment and in situ (Figure 3). Then it is possible to calculate the respective value of the critical load $p_{кр}$ that produces one plastic point.

In order to do it consider a heavy soil layer “cut” to the depth H that behaves both as a Winkler layer as well as an additional soil weight load that sums up with the applied evenly distributed load γH in addition to the surcharge γh. Under the loaded interval it is the evenly distributed load γH that add up to the applied load, resulting in $\gamma\cdot(h+H)$. If $p+\gamma H = p_{кр}$ then a point-like plastic point is generated at depth H under the load edge, whose depth from the surface is $Z_{\max}=0$. Then it follows from equation (1) that

$$0 = \frac{p+\gamma H - \gamma(h+H)}{\pi\gamma}\left(\cot\varphi + \varphi - \frac{\pi}{2}\right) - \frac{c\cdot\cot\varphi}{\gamma} - (h+H). \qquad (2)$$

Hence the cut depth is

$$H_p = \frac{p-\gamma h}{\pi\gamma}\left(\cot\varphi + \varphi - \frac{\pi}{2}\right) - \frac{c\cdot\cot\varphi}{\gamma} - h \quad . \qquad (3)$$

Hence, we have obtained the equation for the cut H_p depth that coincides with $Z_{\max}$ if $K_0 = 1$. However, the meaning of the equation is different: H_p is the depth of the shear cut ("cut") with a plastic point at its bottom end rather than "pseudoplastic" N.P.Puzirevsky zone. Application of the contact model with the cut reduces the footing-structure interaction problem to Fredholm integral equation of the first type that ensures convergence of Schwarz iteration process (Barvashov, 2005).

Then the settlement is computed from equation below:

$$S(x,y,L,B,q) := \left\| \begin{array}{l} H_c \leftarrow H_p Z(q,L,B) \\ S \leftarrow \left(\int_{Z_{\max}}^{H} \frac{q\sigma_z(x,y,z,B)}{E(x,y,z)} dz \right) \\ S \leftarrow S + \left(\int_0^{Z_{\max}} \frac{q}{E(x,y,z)} dz \right) \text{ if } \; xy = 0 \vee xy = LB \end{array} \right. \tag{4}$$

where: σ_z = vertical compressive stresses in subsoil at point (x,y,z) under uniform load q; $E(x,y,z)$ = deformation modulus at point (x,y,z); $\sigma_z(x,y,z)$ = vertical normal stress at point (x,y,z); H_c = compressible layer thickness; $Z_{\max}$ = depth of the cut under points of the footing contour, see eq. (1). As is seen on Figure 7 equation (1) gives two Z values, among which $Z_{\max} = H_p = 4$ m is chosen, as a more realistic one to avoid iterations as c and φ values are highly variable versus depth. Otherwise, concentration of stresses would be too high i.e., enough to punch through to a greater 4 m depth that results in disappearance of the concentration of stresses.

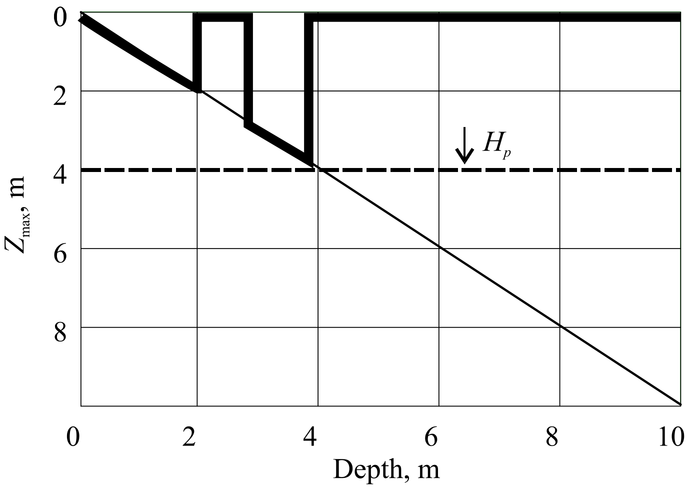

Figure 7. Cut depth H_p=4 m

The settlements $S_i=S(x_i,y_i)$ i=1,2, ... N under uniformly distributed load q=300 kPa at N=9 borehole points are determined by integrating vertical deformations (4).

Then subgrade modulus values at discrete points of the test boreholes:

$$K_i = K(x_i,y_i) = q/S(x_i,y_i) \quad i=1,2,...9 \tag{5}$$

Then Shepard 2D approximation is applied:

$$K(x,y,n,K) := \frac{\sum_{i=0}^{N} \frac{K_i}{\left[\left(x - XY_{i,1}\right)^2 + \left(y - XY_{i,2}\right)^2\right]^n + 0.001}}{\sum_{i=0}^{N} \frac{1}{\left[\left(x - XY_{i,1}\right)^2 + \left(y - XY_{i,2}\right)^2\right]^n + 0.001}} \tag{6}$$

where $XY_{i,1}$ and $XY_{i,2}$ = i^{th} borehole coordinates; x, y = approximation points between boreholes; n = shape parameter of interpolation; N>1 = number of boreholes; K_i = subgrade ratio value at the i^{th} borehole location.

Settlements and tilts of the stiff structure are then determined from its 3 static equilibrium equations (one force, and two moments), as follows:

$$Ga = F, \tag{7}$$

with G = matrix of the system of equilibrium; a = column of unknown values of the tilts and the settlement under the structure center; F = the column of resultant moments and load from the structure:

$$G = \begin{vmatrix} I(2,0) & I(1,1) & I(1,0) \\ I(1,1) & I(0,2) & I(0,1) \\ I(1,0) & I(0,1) & I(0,0) \end{vmatrix}; \quad F = \begin{pmatrix} Q\frac{L}{2} \\ Q\frac{B}{2} \\ Q \end{pmatrix} \tag{8}$$

the values of matrix G elements are calculated by integration:

$$I(i,j) = \int_0^L \int_0^B K(x,y,n,N) x^i y^j \, dy dx \tag{9}$$

with $Q=qLB$. (10)

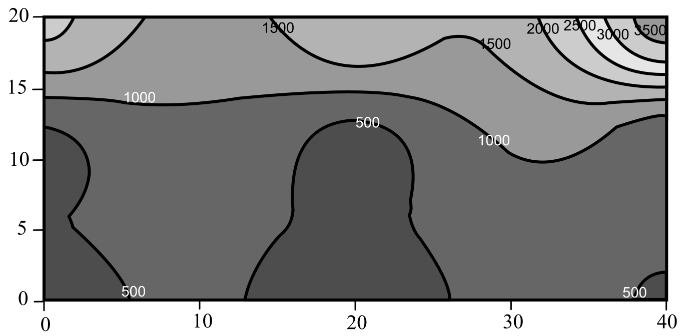

Figure 8. Isolines of the subgrade modulus $K=K(x,y,2,9)$ (overview)

Here the considerable tilt is due to random choice of soil parameters profiles versus depth distribution, imitating those from CPT data. We had to simulate these profiles randomly, because the "raw" data is not available in GI reports.

Analytical settlements and tilts were compared for different values of shape parameter n=1, 2, 3, 4. It was so done to simulate the fact that the values of subgrade modulus *K* were only computed at the locations of the test boreholes but they are unknown in-between boreholes. The analytical results are given in Table 1. Positive tilt directions coincide with positive directions of coordinate axes (*x*,*y*).

Table 1 shows that the values of lateral tilts largely depend on the value of the shape parameter *n* and show that the values of mean settlements closely coincide with each other irrespective of the number of boreholes (5 or 9) and parameter *n* values. The only exception is *n*=1 for the case of 9 boreholes. The tilts, both longitudinal and lateral, differ tangibly between the cases of 5 and 9 boreholes, with the case of *n*=1, dropping out again of other more “typical” values. It means that voluntary unambiguous “inflation” of soil data over the whole volume of subgrade might be not representative.

Table 1. Settlements and tilts of the structure for two test boreholes numbers

Subgrade distribution shape parameter *n*	Longitudinal tilt (along axis *X*)	Lateral tilt (along axis *Y*)	Settlement of the center (cm)	Number of test boreholes
1	0.00191	0.00365	18.8	9
2	0.00244	0.00552	22.9	
3	0.00255	0.00604	23.9	
4	0.00258	0.00620	24.3	
1	0.0322	0.00079	22.4	5
2	0.00375	0.00113	23.8	
3	0.00384	0.00122	23.5	
4	0.00388	0.00124	23.5	

CONCLUSIONS

1. Uncertainties of GI data are an inevitable and essential factor that can result in major errors of subsoil-structure analysis.
2. The relative volume and cost of GI are “infinitesimal” as compared to the capital cost of the project so it is profitable to increase the number of boreholes.
3. GI data in boreholes are extrapolated (“inflated”) into a stratification on a few cross sections. Then engineers subjectively “inflate” this data further on – between cross sections.
4. A new concept is proposed: how to apply “raw” data from boreholes directly (no “inflation” is required) to quantify subgrade-structure interaction.
5. An analytical method is described to calculate stiff structure settlements and tilts, accounting for scatter of input GI soil data from boreholes.

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