Numerical Analysis of Settlement of a Structure Situated on a Heterogeneous Loess Subsoil

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Abstract. The paper presents FEM numerical analyses of interaction of structures situated on loess subsoil. Static CPT tests on loess from Lublin, Poland, was used in the process of defining subsoil variability. The calculations assumed a variant of weak soil under individual foundation footing. It was shown that variable stiffness of loess subsoil causes heterogeneous settlements of the building, which leads to deformations and affects internal forces within the structure.

1. Introduction

Lublin is located within the Lublin Upland, which is largely composed of Pleistocene loess coverings of a considerable thickness. The thickness of the loess covering ranges from several to over thirty meters [1], [2]. Therefore, a significant number of buildings is situated on loess subsoil, which has a major impact on deformations and stress distribution in the structural elements of these buildings.

Figure 1. Geotechnical cross-sections documenting actual soil conditions in Lublin, in the area of the following streets: a) Gęśia, b) Raclawickie, c) Poligonowa.

Aeolian facies loess soils in the Lublin area in their natural state are usually in solid consistency. In solid state loesses ($I_C>1.0$), cone resistances $q_c$ vary between 4÷12 MPa, and the mean value of cone resistance is 6.5 MPa [3]. Division into geotechnical layers only on the basis of the consistency ($I_C>1.0$)
results in the adoption of one compressibility modulus for the entire loess cover. In geotechnics, in-situ tests play a very important role [4], [5]. If we work on geotechnical layers based on static CPT sounding, several zones with different compressibility moduli are distinguished. This can be very important when estimating the settlement of a building. Figure 1 shows examples of geotechnical cross-sections of loess subsoil in Lublin. Typical aeolic loesses in solid consistency are contained in layer II. Due to the large range of cone resistance values $q_c$, additional separations of these layers were introduced (dashed line).

2. Numerical model
A series of numerical analyses was carried out in order to illustrate the differences that appear in the absence of differentiation of layers in terms of compressibility. A schematic model of a reinforced concrete slab and column system based on foundation footing was created. The axial spacing between the columns was 6.0 m, the assumed floor thickness was 0.2 m, and columns were 3.0 m high with 0.4 x 0.4 m cross section. The layout diagram and basic dimensions are shown in Figure 2. Symmetry of the system was used to optimize the calculations and its half was modelled, assuming appropriate supports at the intersection. The structural elements are defined with an elastic model, assuming the following values: $E=30$ MPa, $v=0.2$, $\gamma=25$ kN/m$^3$.

![Scheme and basic dimensions of the model.](image)

Figure 2. Scheme and basic dimensions of the model.

![Numerical model of the structure with the subsoil.](image)

Figure 3. Numerical model of the structure with the subsoil.

The loess subsoil was modelled with a Modified Cam Clay model. All parameters of the model were assumed to be constant, except for the slope of the virgin consolidation line $\lambda$ corresponding to the primary constrained modulus $M$. The constrained modulus was calculated on the basis of cone resistance $q_c$ values recorded in CPT static sounding in the Lublin region and collected in the works ([3], [6]) from the formula
and the factor of $a_{m}=6$.

Due to the normal consolidated nature of loess soils, pre-consolidation stresses were assumed to be equal to the geostatic stresses at the depth of 4 m, i.e. 72 kPa, at which buildings are usually founded. The subsoil was divided into 5 layers (Figure 3), and then calculations were made by differentiating the primary constrained modulus $M$ in these zones, and thus the slope of the virgin consolidation line $\lambda$. Constant parameters of the Cam Clay model $M=1.495$, $\kappa=0.0015$, $\gamma=18$ kN/m$^3$, $a_{m}=34.5$ were assumed as in the paper [6]. The analysis was carried out using ABAQUS software, according to the guidelines from [7]–[9].

3. Numerical analysis
The average resistance cone value of the loess in Lublin is about 6.5 MPa. After taking into account the statistical probing resistance $q_c$ and an analysis of typical geotechnical cross-sections of loesses, two main geotechnical layers: “H” (hard soil) were assumed, with mean cone resistance $q_c=5.0$ MPa, and “S” (soft soil), with mean cone resistance $q_c=8.0$ MPa. With the assumed coefficient of $a_{m}=6$, the constrained modulus was, respectively, $M=30$ MPa for the “S” layer and $M=48$ MPa for the harder “H” layer. Additionally, to illustrate how settlement changes with changing constrained modulus, calculations were made for a homogeneous subsoil with 5 variants of the modulus value. For the variability $q_c$ in the range of 4÷12 MPa and the assumed coefficient $a_{m}=6$, the constrained modulus in the separated variants were assumed as follows: $M=24$ MPa (C1); $M=30$ MPa (C2); $M=39$ MPa (C3); $M=48$ MPa (C4); $M=60$ MPa (C5). For such layers, the parameter $\lambda$ was determined in accordance with the procedure described in [6]. The compressibility parameters of the all of presented in the paper cases compiled in Table 1.

| No. | Case                        | Constrained modulus $M$ [MPa] | Slope of Vigin Consolidation Line [-] |
|-----|-----------------------------|------------------------------|--------------------------------------|
|     | Subsoil layer               | L1  | L2  | L3  | L4  | L5  | $\lambda$ |
| C1  | Homogeneous $M=24$ MPa ($q_c=4.0$ MPa) | 24  | 24  | 24  | 24  | 24  | 0.011 |
| C2  | Homogeneous $M=30$ MPa ($q_c=5.0$ MPa) | 30  | 30  | 30  | 30  | 30  | 0.009 |
| C3  | Homogeneous $M=39$ MPa ($q_c=6.5$ MPa) | 39  | 39  | 39  | 39  | 39  | 0.007 |
| C4  | Homogeneous $M=48$ MPa ($q_c=6.0$ MPa) | 48  | 48  | 48  | 48  | 48  | 0.005 |
| C5  | Homogeneous $M=60$ MPa ($q_c=10.0$ MPa) | 60  | 60  | 60  | 60  | 60  | 0.004 |
| C6  | Weak layer under foot F1    | 30  | 48  | 48  | 48  | 48  | 0.009 (S) 0.005 (H) |
| C7  | Weak layer under foot F1, F2| 30  | 30  | 48  | 48  | 48  | 0.009 (S) 0.005 (H) |
| C8  | Weak layer under foot F1, F2, F3 | 30  | 30  | 30  | 48  | 48  | 0.009 (S) 0.005 (H) |
| C9  | Weak layer under foot F2    | 40  | 30  | 48  | 48  | 48  | 0.009 (S) 0.005 (H) |
| C10 | Extremely weak layer under foot F1, F2 (footing load F2 overload) | 24  | 24  | 48  | 48  | 48  | 0.011 (S) 0.005 (H) |
| C11 | Extremely weak layer under foot F1, F4 (without overload) | 24  | 48  | 48  | 24  | 48  | 0.011 (S) 0.005 (H) |
| C12 | Extremely weak layer under foot F1, F4 (footing load F1 overload) | 24  | 48  | 48  | 24  | 48  | 0.011 (S) 0.005 (H) |
| C13 | Weak layer under foot F1, F2 (load about 300 kPa) | 30  | 30  | 48  | 48  | 48  | 0.009 (S) 0.005 (H) |
| C14 | Weak layer under foot F1, F2 (load about 400 kPa) | 30  | 30  | 48  | 48  | 48  | 0.009 (S) 0.005 (H) |
| C15 | Weak layer under foot F1, F2 (load about 500 kPa) | 30  | 30  | 48  | 48  | 48  | 0.009 (S) 0.005 (H) |
The analysis was divided into the following calculation steps: “Geostatic“ – introduction of the initial conditions, i.e. the geostatic stresses, “Construction“ – introduction of the structure into the model (footing, columns, and ceiling), and 4 “Load“ steps, in which load was gradually applied to the columns simulating the load from the higher floors. The load was applied to the column surface (0.4 x 0.4 m), increasing it with each subsequent step by 1250 kPa, which corresponds to a force of 200 kN per column, i.e. an increase of stress under the footing of 2.0 x 2.0 m by approx. 50 kPa.

4. Results and discussions

Calculations were carried out for each of the assumed variants. Table 2 summarizes the settlement values under each footing and the settlement differences in relation to the F4 footing. Figure 4 shows maps of vertical displacements for a homogeneous subsoil with extreme moduli $M=24$ MPa and $M=60$ MPa. For a homogeneous subsoil and other modulus values, the displacement image did not change, only the values of settlement changed. The differences in settlement between the footings were small and decreased as the constrained modulus increased, which is an obvious and expected result. More important and valuable are the results obtained from the introduction of weakened subsoil under part of the footing (C6÷C9).

Table 2. Footings settlement values in the main variants.

| No. | Case                  | Footing settlement [mm] | Settlement differences [mm] |
|-----|-----------------------|-------------------------|-----------------------------|
|     |                       | F1  | F2  | F3  | F4  | F1  | F2  | F3  | F4  |
| C1  | Homogeneous $M=24$ MPa $(q_c=4.0$ MPa) | 18.5 | 20.3 | 20.3 | 18.5 | 0.0 | 1.8 | 1.8 | 0.0 |
| C2  | Homogeneous $M=30$ MPa $(q_c=5.0$ MPa) | 14.8 | 16.2 | 16.2 | 14.8 | 0.0 | 1.4 | 1.4 | 0.0 |
| C3  | Homogeneous $M=39$ MPa $(q_c=6.5$ MPa) | 11.9 | 12.8 | 12.8 | 11.9 | 0.0 | 0.9 | 0.9 | 0.0 |
| C4  | Homogeneous $M=48$ MPa $(q_c=8.0$ MPa) | 10.9 | 11.8 | 11.8 | 10.9 | 0.0 | 0.9 | 0.9 | 0.0 |
| C5  | Homogeneous $M=60$ MPa $(q_c=10.0$ MPa) | 9.9  | 10.6 | 10.6 | 9.9  | 0.0 | 0.7 | 0.7 | 0.0 |
| C6  | Weak layer under foot F1 | 15.7 | 13.0 | 12.9 | 11.9 | 3.8 | 1.1 | 1.0 | 3.9 |
| C7  | Weak layer under foot F1, F2 | 15.8 | 17.0 | 13.0 | 11.9 | 3.9 | 5.1 | 1.1 | 3.8 |
| C8  | Weak layer under foot F1, F2, F3 | 15.8 | 17.2 | 17.1 | 12.0 | 3.8 | 5.2 | 5.1 | 3.8 |
| C9  | Weak layer under foot F2 | 12.0 | 16.9 | 13.1 | 11.9 | 0.1 | 5.0 | 1.2 | 0.1 |

Figure 4. Map of vertical displacements [m] for a homogeneous subsoil for the constrained modulus: a) $M=24$ MPa (C1), b) $M=60$ MPa (C5).
Figure 5 shows maps of vertical displacements for computational cases (C6-C9) with heterogeneous subsoil. Weakening of subsoil parameters under part of the foundations increases their settlement and increases the vertical displacements in structural elements. In these variants, differences in settlement reach over 5 mm. When the displacement maps are analysed, there is a noticeable increase in displacement on the floor (blue) in the area of weaker layers. The given values refer to settlement at the top level of the foundation. Vertical displacements are greater in the horizontal plane of the floor because of the compressibility of the reinforced concrete column and its deflection in the middle of the floor.

![Displacement Maps](image)

Figure 5. Map of vertical displacements [m] for heterogeneous subsoil, with weakening under footing: a) F1 (C6), b) F1 and F2 (C7), c) F1, F2 and F3 (C8), d) F2 (C9).

At this stage, calculations were carried out assuming that the loads were uniformly transmitted to each footing and the symmetrical system, and the stresses did not exceed 200 kPa. This is an idea variant and corresponds to the average case. In fact, loads may transfer unevenly to the footings, and structural systems may be more complex, resulting in a different distribution of stresses on individual structural elements. Also, adopting soil layers with a cone resistance difference of $q_c=5\ldots8$ MPa is the most common occurrence, but not an extreme case. Therefore, an extremely unfavourable situation was considered for further analysis. The variant with weakening under footings F1 and F2, i.e. half of the foundations, was selected and the boundary conditions were modified. The possibility of an uneven distribution of loads on columns was assumed, and the differences in cone resistance for the assumed layers were increased. An 20% increase of the F2 footing load was assumed. For the weaker layer, cone resistances were reduced to $q_c=4$ MPa, i.e. the compressibility modulus of 24 MPa, and therefore the Cam Clay model parameter of $\lambda=0.011$ (C10). A value of 4 MPa was assumed, because according to the research described in [[10]], this corresponds to the transition limit of loess silts from solid to plastic consistency. New calculations were made for the new assumptions. The results for the extremely unfavourable variant indicate the largest settlement in the F2 footing. In this case, it reached a value of 25 mm. The displacement map shown in Figure 6 illustrates the form of vertical deformations of the structure and subsoil. Uneven settlement affects the stress distribution within the structure. Figure 7 shows stress maps in reinforced concrete elements. As can be seen, the uneven settlement of supports resulted in a change in the distribution of stresses in the floor and an increase of the maximum values. It is not a question of whether the load-bearing capacity of the structure has been exceeded, but rather of drawing attention to the impact of ground movement on the structure.
The structure deformation forms shown on the maps indicate a concave bend, i.e. a case where the inner footings subside to a higher degree than the outermost ones, which is less dangerous for the structure. However, in the case of convex bending, i.e. when the outermost feet subside to a higher degree than the inner ones, the effects of settlement recorded on buildings are more pronounced. Therefore, another analysis was carried out, assuming weakened subsoil under extreme footings F1 and F4 in variants with uniform load (C11) and overload of footing F1 (C12). As a result of the calculations, displacement maps were obtained as shown in Figure 8. The results obtained (Table 3) indicate that the extreme footings settlement approx. 5 mm more than the central ones, and in the case in which one footing is overloaded, the difference in settlement between two adjacent footings is almost 11 mm.

Figure 6. Map of vertical displacements [m] for heterogeneous subsoil in an extremely unfavourable case (C10).

Figure 7. Stress map [kPa]:
- a) minimum stresses – homogeneous subsoil C1,
- b) minimum stresses – heterogeneous subsoil C10,
- c) maximum stresses – homogeneous subsoil C1,
- d) maximum stresses - heterogeneous subsoil C10.
Table 3. Footings settlement values in the additional variants.

| No.  | Case                                      | Footing settlement [mm] | Settlement differences [mm] | Settlement differences [mm] |
|------|-------------------------------------------|-------------------------|-----------------------------|------------------------------|
|      |                                           |                         | Case                        | 1   | 2   | 3   | 1   | 2   | 3   |
|      |                                           |                         |                             |     |     |     |     |     |     |
| C10  | Weak layer under foot F1, F2 (footing F2 overload) | 18.9 25.0 13.3 11.8    | 7.1 13.2 1.5                |     |     |     |     |     |     |
| C11  | Weak layer under foot F1, F4 (without overload) | 18.3 13.0 13.0 18.3    | 0.0 -5.3 -5.3               |     |     |     |     |     |     |
| C12  | Weak layer under foot F1, F4 (footing F1 overload) | 23.8 13.1 13.0 18.2    | 5.6 -5.1 -5.2               |     |     |     |     |     |     |

Figure 8. Map of vertical displacements [m] for subsoil weakened under outermost footings F1 and F4: a) uniform load (C11); b) footing F1 overload (C12).

In the presented calculation variants, pressure under the foundations did not exceed or slightly exceeded 200 kPa, which is consistent with the conditions in most of the existing buildings. Of course, this applies to values that are characteristic of actual loads, and not design values that assume the most unfavourable situation and take into account partial coefficients. The next stage of the analysis was therefore to determine critical loads causing significant deformations of the structure situated on loess soils of “average variability”. For this purpose, the scheme with weakening of the subsoil under footings F1 and F2 was used again. Layers with an average value of $q_c=5$ MPa for the “S” soil and $q_c=8$ MPa for the “H” soil were assumed as “average variability”, which, respectively, corresponds to moduli of $M=30$ MPa and $M=48$ MPa. The load on the columns in subsequent “Load” steps was applied in such a way that the average stress under the footing (above the self-weight) increased to 200 kPa (C7), 300 kPa (C13), 400 kPa (C14), 500 kPa (C15), respectively. These are above-average values for the structure, but they may occur in real life. The results are presented in Table 4.
Table 4. Footings settlement values in the overload variants.

| No. | Case                          | Footing settlement [mm]       | Settlement differences [mm] (in relation to the F4 footing) |
|-----|-------------------------------|------------------------------|------------------------------------------------------------|
|     |                               | 1   | 2   | 3   | 4   | 1   | 2   | 3   |
| C7  | Weak layer under foot F1, F2 (load about 200 kPa) | 15.8 | 17.0 | 13.0 | 11.9 | 3.9 | 5.1 | 1.1 |
| C13 | Weak layer under foot F1, F2 (load about 300 kPa) | 27.1 | 28.1 | 20.5 | 19.2 | 7.9 | 8.9 | 1.3 |
| C14 | Weak layer under foot F1, F2 (load about 400 kPa) | 38.6 | 39.3 | 27.9 | 26.5 | 12.1 | 12.8 | 1.4 |
| C15 | Weak layer under foot F1, F2 (load about 500 kPa) | 50.3 | 50.5 | 35.3 | 33.9 | 16.4 | 16.6 | 1.4 |

The natural consequence of the increased load is the propagation of settlement. However, it appears that, under the conditions described, the settlement of individual footings exceeds 50 mm only at a load of approx. 500 kPa. This indicates that an increase in above-average loads does not cause significant deformations in the structure, unless it is associated with significant heterogeneity of the subsoil or stresses under the foundations.

5. Conclusions

In summary of the described analyses, it is stated that the heterogeneity of loess subsoil, identified in this case by CPT tests, has an impact on the settlement of the object and the distribution of forces in the structure. Zones of weakened subsoil increase the settlement of foundations placed on them, therefore it should be taken into account during the design process. Several types of variability were assumed in the analyses: subsoil, load, and load unevenness. It appears that a single occurrence of an average variation should not cause the serviceability limit states to be exceeded. However, with two types of variability at the same time, there is a high risk that the deformations or displacements of the structure will be exceeded.

The calculations show that for a thickness of weaker zones up to 4 m and stresses under foundations under 200 kPa on loess soils of "average variability" (differentiation of average $q_c$ for the layer in the range of 5÷8 MPa), there are deformations that may affect the movement of the structure. For these conditions, the designer should determine whether these deformations will be significant for the movement of the structure.

In the case of stresses exceeding 200 kPa, above-average variation of loesses and uneven distribution of stresses under foundations, the possibility of exceeding normal displacements or deformations of the structure should be taken into account. It should be noted that in order to identify the variability of loess subsoil stiffness, borehole testing is not sufficient, even with advanced laboratory testing. Identification of stiffness should be carried out by means of in-situ tests.

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