Research Article

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A FEM analysis of the settlement of a tall building situated on loess subsoil

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Abstract: In order to correctly model the behaviour of a building under load, it is necessary to take into account the displacement of the subsoil under the foundations. The subsoil is a material with typically non-linear behaviour. This paper presents an example of the modelling of a tall, 14-storey, building located in Lublin. The building was constructed on loess subsoil, with the use of a base slab. The subsoil lying directly beneath the foundations was described using the Modified Cam-Clay model, while the linear elastic perfectly plastic model with the Coulomb-Mohr failure criterion was used for the deeper subsoil. The parameters of the subsoil model were derived on the basis of the results of CPT soundings and laboratory oedometer tests. In numerical FEM analyses, the floors of the building were added in subsequent calculation steps, simulating the actual process of building construction. The results of the calculations involved the displacements taken in the subsequent calculation steps, which were compared with the displacements of 14 geodetic benchmarks placed in the slab.

Keywords: Cam Clay model, Loess subsoil, CPT test, 3D FEM analysis, settlement, geodetic measurements

1 Introduction

The interaction between building structures and subsoil is usually analysed in two dimensions. Calculations are usually done on a characteristic cross-section of the structure in a single plane [1–3]. In the case of axially symmetrical tasks, a section of the structure can be used [4]. Analyses of three-dimensional tasks encompassing buildings with the subsoil body are rarely performed, due to their high complexity. The creation of three-dimensional building models often ends with the foundations, with the as-

sumption of a rigid foundation underneath, described by a single-parameter susceptibility [5–7], or an analysis of the subsoil body with a small fragment of the structure [8]. Generally, calculations require combining elements described by completely different constitutive laws [9–11]. The proper verification of numerical analyses is possible on the basis of geodetic observations of the movements of the building. This paper presents an example of a numerical analysis of a tall building, founded on a base slab on loess soil. The building is located on Kraśnicka street in Lublin.

2 Calculation assumptions

The analysed building is a tall office building with 14 above-ground storeys and 1 underground storey. The dimensions within the plan are approximately 70×45 m. The load-bearing structure is a reinforced-concrete framework consisting of 0.28 m thick monolithic ceilings supported on columns arranged on a fairly regular grid. The entire structure is founded on a base slab with a thickness of 2.2 m in the central part, and decreasing in steps from 1.3 m to 0.9 m towards the edge, and with a thickness of 0.5 m locally.

The calculations were performed with the ABAQUS software [12], using the Finite Element Method (FEM). The numerical model of the structural part of the building was defined as a plate with beam elements. The walls, ceilings, and binding joists, were modelled as S4R four-node slab elements. The thickness of particular structural elements was assumed according to the actual geometry with thickening in the area of the ceiling caps. The columns were modelled as beam elements. The elements were modelled in their axes. The base slab was built of C3D8R solid elements. To all the structural elements were assigned ideally elastic properties, according to the characteristics of reinforced concrete. The numerical model of the subsoil was created as a solid model with C3D20R twenty-node elements. The behaviour of the soil layer directly beneath the foundation was described using the elastico-plastic Cam Clay model. The linear elastic perfectly plastic model with

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Figure 1: The analysed building: a) a perspective of the view structure, b) a 3D numerical model

the Coulomb-Mohr failure criterion was adopted for the deeper subsoil.

The analysis was divided into the following calculation steps.

- GEOSTATIC – the introduction of geostatic stresses into the subsoil
- EXCAVATION – the removal of the soil in the location of the excavation for the base slab
- PLATE – the introduction of the slab geometry in the location of the excavation
- KP – the introduction of the underground storey to the model
- K1÷K1314 – the introduction of successive storeys into the model (10 steps)
- FINISHING – the introduction of operational loads and substitute weights from the non-structural elements.

Figure 1 presents a view of the actual structure and a 3D numerical model of the building with the body of the subsoil. The division of the building into different colours reflects the structural elements appearing in the successive calculation steps, from “PLATE” to “K14.” Two above-ground storeys were introduced in each calculation step. The first model to be created was the building model, and static calculations were made with the assumption of rigid supports; subsequently the subsoil was modelled and the two models were combined. The representative geotechnical cross-section is shown in Figure 2, and the complete

“building-subsoil” model is shown in Figure 3. Three basic layers of the subsoil were distinguished: the first a loess layer, the second a weathered-limestone layer, and the third a rocky subsoil consisting of low-strength cracked limestone.

Two Cam-Clay model parameter variants were assumed for the loess soil layer. The first variant, provisionally referred to as “EDO”, was based on the results of oedometer tests performed on samples with an undisturbed soil structure, taken from boreholes during construction. On the dependency charts $e-\ln(\sigma)$ (Figure 4) the lines of primary and secondary loading and the formulae describing the slope of these straight lines are presented. In the chart e is a void ratio, and σ is normal pressure. These charts were the basis for determining the initial parameters of the Cam-Clay model – λ and κ , where λ is the slope of the normal (virgin) consolidation line, while κ is the slope of unloading-reloading line.

In the calculations it was assumed that the soil was normally consolidated, and the preconsolidation stresses p_0 corresponded to geostatic stresses. The slope of critical state line in $p-q$ space called as M parameter of the Cam-Clay model was calculated with the assumption of the internal friction angle $\varphi = 35^\circ$, based on an analysis of the tests of Lublin loess soils [13]. The void ratios e_0 and e_1 were determined from basic laboratory tests and consolidation charts.

In the second parameter variant, referred to as “CPT,” the parameters were estimated by combining the results

Table 1: The parameters of loess layer for Cam-Clay model

Variant	λ	κ	M	a_0	p_0	e_1	e_0
EDO	0.0300	0.0042	1.495	28.1	108	0.561	0.421
CPT	0.0130	0.0042	1.495	32.6	108	0.584	0.523

Figure 2: The representative geotechnical cross-section

of laboratory tests and CPT soundings, which were transformed into Cam Clay model parameters using the procedure described in [14]. Both the lithology and heterogeneity of the subsoil, identified by CPT soundings, were taken into account in the division into geotechnical layers. Afterwards, the mean cone resistance was calculated in respect of loesses as a weighted mean, where the thickness of the separated layer was taken as the weight. The mean value of cone resistance was $q_c = 3.27$ MPa. On this basis and with the assumed coefficient $\alpha_m = 6$ the mean constrained modulus for the layer was determined as $M_{CPT} = 19.6$ MPa [14]. This modulus was the basis for determining the slope of the primary consolidation line λ . The parameters of the

Figure 3: The view of the complete FEM "building-ground" model **Figure 4:** Oedometer-sample compression curves

Cam Clay model used for calculations for both these variants are listed in Table 1.

Rocky subsoil is located in the deeper parts and its movements are limited to a very small range of deformations. For these layers the simplified model of Coulomb-Mohr was adopted and the parameters were derived from among other things, the following works [1, 9, 10, 15]. For the soil layers elastic properties were assigned with values similar to the initial deformation modulus. After the analysis of the literature data [16, 17] the angle of internal friction was assumed as $\varphi = 40 \div 45^\circ$, with a cohesion of $c = 50 \div 2000$ kPa. The value of the constrained modulus for rock layers was assumed as $E = 2000$ MPa for the lower zone of cracked carbonate rock (marl, opoka, gaize). For the upper zone – partially weathered – the modulus was estimated as $E = 300$ MPa with the use of literature data [10, 16] and on the basis of own research on the shear wave

Figure 5: Vertical displacements [m] in selected successive phases of construction

velocity V_s measured for the weathered layers in SDMT tests in Lublin.

3 Numerical analyses and results

In each of the calculation variants, successive floors were added in the calculation steps. In order to illustrate the calculation process Figure 5 presents the vertical displacements in selected calculation steps.

Figure 6 shows the comparison of vertical displacements for two soil-parameter variants – EDO and CPT. The model with EDO parameters shows greater settlement than the CPT model. The maximum vertical displacement was 89.7 mm for the EDO model, and 51.7 mm for the CPT model. These are total values, from the start of the construction works, and should not be compared directly with the measured values, which are presented in relation to the reference measurement taken after constructing part of the structure. The displacement values in relation to the first geodetic measurement were 77.4 mm for the EDO parameters and 38.5 mm for the CPT parameters. The settlement determined with the EDO parameters significantly

exceeded the actual settlement (22.0 mm), and the model with the CPT parameters was adopted for further analysis.

The numerical calculations were compared to the actual settlement values. In subsequent calculation steps, the values of vertical displacements at the nodes corresponding to the location of benchmarks in the base slab were taken. The data and comparison to the measured values are listed in Table 2. The measured and calculated settlement values were compared directly and relative to the permissible settlement, which was assumed at 50 mm. The final comparison was made on averaged-out values. Some of the benchmarks were destroyed during the construction works.

The chart of the displacements of the benchmarks prepared on the basis of the actual measurements indicates that the building settlement had not yet stabilised so, after the analysis of the literature data [18, 19] the final settlement was estimated by extrapolation of the results, which is shown in Figure 7. The chart shows the displacements of the benchmarks which remained until the last measurement, and the mean value of these measurements. The average final settlement was found to be approximately 25 mm with a maximum of 33 mm.

Figure 6: Vertical displacements [m] in the building model with the body of the subsoil: a) EDO variant model, b) CPT variant model

Figure 7: Forecast final settlement of the building

In the analysis of the displacement results presented in Table 2 it can be seen that the measured displacement values are higher than the calculated ones in the initial settlement phase alone. After crossing the calculation step KP, the calculated values exceeded the measured values. According to the author, it is caused primarily by spreading the actual settlement over time, which was not taken into account in the calculations. At this stage, the relative calculation error in relation to the value obtained in the last settlement measurement was less than 20%, which is not a very-significant error in the context of geotechnical calculations; moreover, the result of the calculations is on the safe side for the building. However, due to the fact that the actual settlement of the building is not final this error will decrease. If the author confirms the estimated final mean

settlement value of 25 mm, the error will be less than 5%, which should be considered a very-good result.

In Figure 8 the displacement map of the base slab obtained from the calculations and that created on the basis of displacement measurements are compared. The values of displacements do not overlap, because the maps for calculations are given as final and the geodetic ones as the last measurement; however one should pay attention to the shape and arrangement of isolines, which substantially overlap with each other on both maps, which confirms the correctness of the calculations.

To assess the impact of the subsoil work on structural elements, additional calculations were made with rigid supports under the base slab instead of the subsoil. Stresses in structural elements were compared for both calculations. In Figure 9 the minimum principal stresses are shown, while in Figure 10 the maximum ones are shown. The maps show increased stress zones. Significant differences can be seen primarily on the walls of underground floors. For rigid supports, these walls work as compressed. For a subsoil model defined with elasto-plastic behaviour, the bent base slab makes the walls work a bit as deep beam and the change in stress distribution is significant.

Table 2: A summary of vertical displacements in the successive calculation steps

Benchmark	Settlement in calculation step [mm]												Finishing
	Plate	KP	K1	K2	K3	K4	K5	K6	K78	K910	K1112	K1314	
6	0.0	0.4	1.3	2.2	3.0	3.9	4.8	5.9	8.8	12.2	15.6	19.2	29.1
7	0.0	1.5	2.3	3.7	5.0	6.4	7.8	9.5	12.9	16.5	20.0	23.8	34.3
8	0.0	2.5	3.9	5.2	6.3	7.4	8.4	9.6	12.8	16.6	20.7	24.8	36.0
9	0.0	2.5	3.8	5.3	6.6	8.0	9.4	11.2	15.6	20.2	24.8	29.7	40.9
11	0.0	0.5	1.6	2.5	3.3	4.0	4.7	5.5	7.2	9.7	12.5	15.3	24.8
12	0.0	0.9	1.7	2.3	3.0	3.6	4.2	4.8	6.4	8.5	10.7	13.1	20.0
13	0.0	1.9	2.7	4.0	5.0	5.9	6.9	7.9	10.0	12.4	14.9	17.6	26.2
14	0.0	2.2	3.2	4.3	5.0	5.6	6.2	6.9	8.6	10.6	12.8	15.1	22.6
15	0.0	-1.4	-1.3	-1.2	-1.1	-1.0	-1.0	-0.9	-0.7	-0.4	0.0	0.4	2.1
16	0.0	0.7	1.6	3.3	4.8	6.3	7.9	9.7	10.1	10.5	10.8	11.3	15.8
17	0.0	0.0	0.2	1.6	3.0	4.4	5.8	7.2	7.1	6.9	6.8	6.6	11.1
18	0.0	2.8	4.1	5.5	6.6	7.7	8.7	10.1	13.7	17.7	21.7	26.0	36.4
19	0.0	2.6	4.0	5.4	6.8	8.2	9.6	11.5	16.3	21.4	26.4	31.8	43.7
20	0.0	2.3	3.6	4.9	6.2	7.4	8.6	10.3	14.5	19.3	24.0	28.9	40.8
Average	0.0	1.4	2.3	3.5	4.5	5.6	6.6	7.8	10.2	13.0	15.8	18.8	27.4
Measured (average)	0.0	2.2	2.2	2.3	3.3	3.3	3.3	3.5	5.5	6.2	6.7	7.5	17.2 (25.0)*
Error [mm]	0.0	0.8	-0.2	-1.2	-1.2	-2.2	-3.2	-4.3	-4.7	-6.8	-9.2	-11.3	9.8 (2.4)*
Relative error [%]	0.0	1.6	-0.4	-2.3	-2.4	-4.4	-6.5	-8.6	-9.5	-13.7	-18.3	-22.7	19.6 (4.8)*

* - forecasted value

Figure 8: Map of the base-slab settlement on the basis of: a) measurements of the real actual settlement [mm], b) calculations in ABAQUS [m]

Figure 9: Minimum principal stress [kPa]: a) in model with rigid support, b) in model with subsoil defined by elasto-plastic Cam Clay model

Figure 10: Maximum principal stress [kPa]: a) in model with rigid support, b) in model with subsoil defined by elasto-plastic Cam Clay model

4 Conclusions

As a result of the numerical analyses the forecast settlement of the building was obtained, which was then compared with the geodetic measurements. At the time of the last geodetic measurement the settlement had not yet stabilised but the partial results indicate a high convergence of the calculation results with the actual behaviour of the building. In the case of loesses the Cam Clay model can be used to simulate the movement of the subsoil, whose parameters the author proposes to determine with tests performed using CPT soundings. A more-detailed analysis of the presented results can be found in [14].

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