Regulation of additional settlements of dense urban infrastructure objects during execution of deep excavations and raft-pile foundation of high-rise buildings

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Abstract. The paper describes the calculation and design steps of deep excavations and foundations of a high-rise building on a territory with a high groundwater level and existing buildings with a physical wear of their bearing structures and foundations located in the zone of mutual influence. The aim is to find a rational solution for the diaphragm wall, pile-raft foundation of the high-rise building in order to protect adjacent objects and consider various factors that determine the nonlinear behavior of subsoil layers and the elements of geotechnical protection. As an example, a project of a high-rise building with a multi-level underground parking located in the central part of Krasnodar is shown. It is designed and calculated considering the staged construction of the pile-raft foundation, non-linear behavior of the subsoil layers, the influence of a variable groundwater level, the risk of technological settlements and liquefaction of the underlying sandy soil layers. Due to the performed researches and by following the construction methods, it became possible to keep the existing buildings and structures safe during the construction period and further operations. The observed methods of construction of the pile-raft foundations and deep excavations in difficult subsoil conditions using the example of the central part of Krasnodar make it possible to recommend the obtained results as a rational solution applicable for similar cases in conditions of a dense urban infrastructure.

1. Introduction

Today more and more deep excavations become necessary for foundations of high-rise buildings, underground structures, engineering utilities [1-4]. Technical parameters of bottoms and walls of deep excavations must provide a reliable protection from groundwater and additional technological settlements [5-11]. Due to high costs of building plots in large cities, investors prefer projects with an increasing number of underground floors. The design also considers the location of the project in relation to some existing facilities, subsoil conditions of the construction site, groundwater behavior, etc.

Under dense urban infrastructures, a choice of an excavating method for a deep excavation depends largely on parameters of adjacent buildings. The state of such buildings determines the protection from an influence of new constructions [12-15]. These methods may include sheet piling or diaphragm walls with various fixing elements (ground anchors, struts made of metal or reinforced concrete), the "top-down" technology, etc. Horizontal and vertical deformations of the terrain appearing during construction are almost inevitable, especially in conditions of weak and water-saturated soils, so they should be limited to a level that excludes a negative impact on the surrounding structures.
It is well-known that the main reasons for the additional deformations of the bottoms and walls of deep excavations, as well as adjacent objects, most often include inaccuracies in geological investigations, errors in calculating deformations at the design stage, underestimation of the influence of technological settlements in the process of diaphragm wall execution and piling, lack of reliable information about technical conditions of the surrounding buildings. Eliminating and correcting the consequences of these underestimations is much more expensive than a preliminary detailed design based on reliable initial data and multivariate analysis of possible geotechnical solutions, and, therefore, special attention should be paid to selection and justification of important options in advance.

Soil conditions in large cities are quite diverse, and they determine appropriate approaches to solving regional problems, however, the developed calculation algorithms and adopted technical solutions can be applied for the construction sites with similar urban principles [8, 9].

The paper provides an algorithm of substantiation of geotechnical solutions for the engineering protection of a deep excavation and implementation of the CPRF for the high-rise building located on a plot with highly compressible soils in the central part of Krasnodar with dense urban developments.

2. Materials and Methods

Geomorphologically, the territory of the city is located on 4 large elements: the floodplain of the Kuban River; II above floodplain terrace of the Kuban River; III above floodplain terrace of the Kuban River; floodplain of the Karasun river. The territory has a very complex hydrogeological structure. In the section of Quaternary sediments several tiered aquifers are distinguished from confined to sub-confined ones. Due to the presence of "filtration windows" of natural and technical reasons, these horizons in a part of the territory are hydraulically interconnected.

The presence of weak layers in the central part of the city makes difficulties in the design and construction of deep excavations with the bottom far below the water table. A technological removal of soil during the execution of piles is often a cause of deformations of existing buildings occurring in the process of excavation, if the project does not provide the protection of the borehole with bentonite. Or in case of using the continuous flight auger technology without considering the increasing resistance while installing the auger when passing through the layer boundary, which causes a slowdown in the vertical movement of the auger per unit time at a constant rotation speed. In this case, the rotating blades of the auger lead to a significant volumetric removal of the soil from the upper water-saturated sand layers, which is enhanced by the closure of the aquifers previously separated by the water-resistant layer and additional pressure. As a result, the volume of the soil from the borehole can be several times greater than the geometric volume of the borehole, and this inevitably leads to decompaction of bases of the adjacent buildings.

Technological settlements can appear both during the execution of a pile field or during development of deep excavations, if impermeability on contacts between vertical protection elements of the excavation (piles of a diaphragm wall) and jet-grouting layer (or waterproof layer) is not achieved.

Geotechnical codes regulate the limit values of technological settlements for existing buildings and structures, depending on the type of technology used for the newly constructed foundations or engineering protection structure [11], and also establish the minimum allowed distance without considering additional settlements. Practice shows that the arranged recommended distances do not always solve the problem of exceeded additional deformations.

Figure 1 shows the observation data of the construction process in 2019 of the pile foundations for a high-rise building located on a site with a thick stratum of water-saturated sands overlapped from above by a water-resistant clayey horizon. The values of the technological settlement of directly adjacent existing buildings were measured at different distances from the border of the pile field. Settlements were caused by piling without the evenly drilling technology of the auger to protect against the technological removal of fine water-saturated sand along the surface of the borehole during drilling. Figure 1 shows distribution of the technological settlements after the first 7-10 days of piling, which led to the necessity of construction suspension in order to find urgent technical solution.
Figure 1. Monitoring of technological settlements caused by piling without the evenly drilling technology of the auger.

After the completion of the pile field drilling, the technological settlement of the existing foundations was from 350 to 400% in comparison with the permitted technological value. This fact led the existing buildings to an emergency state even at the stage of the foundation construction and required their decommissioning.

Figure 2. The graph of increments of deformations of the nearby foundation during development of a deep excavation of a multi-storey residential building in conditions of high groundwater level.
The example on figure 2 shows the impact of the construction in 2019-2020 of a two-level excavation for a multi-storey residential building on the deformations of foundations of the adjacent buildings. A jet-grouting curtain with a thickness of 1500 mm made at a depth of 10 meters after the excavation of the waterproof clay layer located above was damaged by pushing groundwater pressure. Fine sand entering the excavation through the waterproof damages led the existing foundations to an emergency state and caused significant vertical and horizontal movements of the diaphragm wall. All the facts completely stopped the construction process and caused the revision of technical solutions. Figure 2 shows a graph of the nearby foundation deformations increment in the process of excavation.

The value of the technological settlement at the moment of construction suspension was about 200-300% from the maximum allowable value.

The above shown properties of the subsoil and hydrogeological conditions, as well as typical damages of the surrounding buildings during the drilling of a pile field and engineering protection for deep excavations safe methods of geotechnical constructions. Such methods should:

- prevent from the technological removal of the water-saturated soil by the pressurized action of groundwater during borehole drilling for the pile-raft foundations or perimetral diaphragm walls;
- ensure the safety of the waterproof jet-grouting layer during the soil excavation;
- consider the lowering of the groundwater level on the deformation of the surrounding buildings;
- consider of the influence of new diaphragm walls on additional deformations of the existing buildings;
- application of the soil models to correctly describe the stages of excavation and the application of loads.

A complex approach has been developed and implemented based on the example of the project of the high-rise buildings in the central part of Krasnodar (figure 3). The task was complicated by the proximity of historical buildings (figure 4) of the III category of the structures’ state, the presence of a high level of groundwater and the necessity to develop a deep multileveled excavation.

![Figure 3. Perspective view of the high-rise complex in Krasnodar, Krasnaya Street ("Aport" LLC, architects: Churilov, V. A., Churilov, A. V., Churilova E. Yu. [16]).](image)

The high-rise building has a height of 99.95 m and a framed structural scheme with columns and diaphragms as main vertical load-bearing elements. The foundation has CFA piles with a length of 17 m and a foundation slab with a thickness 1500 mm (figure 5).
Figure 4. Layout of foundations of the high-rise and stylobate parts combined with objects of urban infrastructure: yellow – shows the high-rise building; pink – shows the stylobate block; gray – shows the surrounding buildings.

Figure 5. Geological section 1-1 combined with the cross-section of the high-rise building and its foundation: grey – shows the high-rise building, green – shows the stylobate block.

The excavation development in conditions of a dense urban infrastructure is carried out under the protection of the diaphragm wall. Two main options were considered while choosing a final geotechnical solution: sheet piling made of Larsen 5 with a depth of five meters below the level of the excavation bottom and a 640 mm thick diaphragm wall with a depth of five meters below the level of the excavation bottom.

The excavation wall has 3 levels of struts. The depth of the excavation is 11.85 m from the ground level. Below the foundation slab the jet-grouting waterproof layer is provided. This layer with a thickness of 2000 mm has its main function of the anti-seepage barrier and additionally serves as a spacer for the diaphragm wall [12].

Figure 6. Geological cross-section of the construction site.
For the most complete consideration of all factors included in the system and also for a sufficient comparison of the obtained results, the calculations were carried out using several specialized geotechnical software (Midas GTS NX, Plaxis, Wall-3).

Analysis and generalization of the geological survey data characterizing the age, genesis, condition, deformation and strength parameters of the soil layers forming the construction site are shown on the geological cross-sections in figure 6. 15 geotechnical elements have been identified. In Table 1 main basic physical and mechanical parameters of layers are shown.

| Number of geological elements | Name of geotechnical elements | Specific weight of soil, kN/m$^3$ | Adhesion, kPa | Friction angle, degrees | Deformation modulus, MPa | Poisson's ratio |
|--------------------------------|--------------------------------|----------------------------------|---------------|------------------------|--------------------------|----------------|
| 1                              | The soil is modern, loamy, subsidence | 18.2 | 17.7 | 19.9 | | | |
| 2                              | Loess loams, hard-plastic | 18.2 | 18.0 | 18.1 | 18 | 14 | 15 | 23 | 22 | 22 | 8.9 | 0.37 |
| 3                              | Loess loams, hard-plastic | 19.3 | 19.1 | 19.2 | 16 | 11 | 13 | 24 | 22 | 23 | 16.0 | 0.37 |
| 4                              | Loess loams, hard | 19.5 | 19.4 | 19.4 | 14 | 10 | 11 | 26 | 25 | 25 | 27.0 | 0.36 |
| 5                              | Clays alluvial, hard-plastic, sandy | 18.6 | 18.4 | 18.5 | 56 | 45 | 49 | 20 | 19 | 19 | 17.0 | 0.39 |
| 7                              | Peat | 17.1 | 16.7 | 16.8 | 32 | 28 | 29 | 9 | 7 | 8 | 3.2 | 0.36 |
| 9                              | Semisolid clays, with an admixture of organic substances | 19.4 | 19.2 | 19.3 | 58 | 48 | 52 | 14 | 12 | 13 | 20.0 | 0.29 |
| 10                             | Semisolid clays, with an admixture of organic substances | 18.3 | 18.0 | 18.1 | 60 | 47 | 52 | 11 | 9 | 10 | 14.0 | 0.29 |
| 14                             | Clays alluvial, hard, sandy | 19.3 | 19.0 | 19.1 | 95 | 67 | 78 | 20 | 18 | 19 | 67.0 | 0.21 |
| 15                             | Alluvial loams, harde | 20.0 | 19.7 | 19.8 | 72 | 51 | 59 | 21 | 19 | 20 | 57.0 | 0.36 |
| 8                              | Buried peat | 12.6 | 12.1 | 12.3 | 34 | 31 | 34 | 8 | 7 | 7 | 3.1 | 0.24 |
| 6                              | Sand, variegated, medium density, saturated with water | 20.0 | 19.9 | 20.0 | 0 | 0 | 0 | 32 | 31 | 31 | 25.0 | 0.33 |
| 11                             | Fine sand, medium density, saturated with water | 20.2 | 20.1 | 20.1 | 0 | 0 | 0 | 34 | 33 | 33 | 29.0 | 0.32 |
| 12                             | Medium sand, dense, saturated with water | 20.5 | 20.4 | 20.4 | 0 | 0 | 0 | 35 | 34 | 34 | 34.0 | 0.30 |
| 13                             | The sand is gravelly, dense, saturated with water | 20.5 | 20.5 | 20.5 | 0 | 0 | 0 | 36 | 36 | 36 | 40.0 | 0.30 |

Thy following problems were solved while choosing the final geotechnical solution for the deep excavation:
- the forecast of the additional settlement of the surrounding buildings was carried out when the groundwater level was lowered to the level of the foundation slab corresponding to the stage of the postconstruction operation and drainage along the perimeter of the building. At the construction stage, the groundwater level behind the diaphragm wall was taken to be equal to the maximum predicted GWL according to geological surveys;
- the analysis of three design situations were carried out: excavation of a pit, construction of the underground part with filling of the excavation sinuses, completed construction (styrolobe + high-rise building);
- considered a design situation with an accidental exclusion of one of the most loaded spacers in three different parts of the deep excavation.
The geotechnical finite element model is shown in figure 7. A detailed modeling of the deep excavation, the defending diaphragm wall, struts and system of strapping beams, as well as objects of the surrounding developments have been carried out.

**Figure 7.** Detailed design scheme of the excavation with struts and spacers (racks are not shown): 1 – is the diaphragm wall; 2 – is the system of struts; 3 – is the foundation of the stylobate part; 4 – is the foundation of the high-rise part; 5 – is CFA piles of the high-rise building; 6 – are strapping beams; 7 – are the foundations of the surrounding building.

### 3. Results
As the main results of these calculations, stresses, forces and deformations occurred in all elements of the system at each stage of construction were analyzed. Figure 8 shows the additional settlements of the buildings of the surrounding development at the final stage of the excavation are shown. The calculated values of additional deformations of the surrounding buildings did not exceed the permitted values.

**Figure 8.** Additional settlements of the adjacent building at the 4th stage of excavation.
After geotechnical calculations and their analysis, it was found that the most effective and safe construction of the excavation fence was a 640 mm thick diaphragm wall with three tiers of metal spacers and jet-grouting waterproof layer.

However, the biggest challenge was to find a solution to prevent decompaction of sandy layers under the foundations of the existing buildings while excavating and piling. So, we developed the presented method for the construction of the deep excavation and pile-raft foundations.

The groundwater head level established during the survey was about 9 meters above the bottom of the excavation. It required an appropriate consideration to provide necessary protection of the jet-grouting waterproof layer against floating. Another difficulty was the necessity to drill out the jet-grouting layer with the designed piles. With the actual groundwater level, it could lead to an uncontrolled removal of the underlying sand by the auger blades and could cause technological settlements of the surrounding area. The necessary watertight contact between the lateral surface of the pile and jet-grouting body would not have been achieved.

The developed method (figure 9 and 10) was based on a following sequence. Immediately after the diaphragm wall was completed in the ground, piles were drilled from the mark of the excavated soil corresponding to the first tier of spacers and the established groundwater level. The reinforcement cage of the upper part of the piles had sufficient rigidity to install it into the borehole filled with concrete (CFA technology) and assumed the possibility of partial cutting at further stages of the excavation. Made from the same level, the jet-grouting layer with a thickness of 2000 mm formed a reliable contact with the lateral surface of all piles during the piling process. It ensured a sufficient anchoring of the waterproof layer and made it possible not to increase its thickness and also its depth in order to create an additional load from the overlying soil located between the foundation slab and the upper edge of the jet-grouting layer.

![Figure 9](image.png)

**Figure 9.** Construction stages of the deep excavation and pile-raft foundation: I – execution of a diaphragm wall; II – excavation to the level of piles execution, piling process using the CFA technology; III – execution of a waterproof level with the jet-grouting technology.
Figure 10. Construction stages of the deep excavation and pile-raft foundation. IV – assembling of struts and beams following the excavation stages and partial pile cutting; V – execution of a foundation slab of the high-rise building; VI – construction process of the high-rise building.

4. Conclusion

Geotechnical protection of the deep excavation and pile field of the high-rise building carried out by the developed method resulted in a significantly smaller impact on the surrounding buildings. Reduction of the jet-grouting layer height became possible due to its interaction with the piles having a temporary anchoring function.

In the process of the stage-by-stage assessment of the mutual influence of the constructed and existing buildings on each other when applying the proposed design solution, on the basis of the geotechnical calculations performed in various software complexes, it was concluded that the final additional deformations of the supporting structures and foundations of the existing buildings became acceptable.

The initially chosen construction of the excavation fence using the Larsen sheet pile assuming the dynamic installation of the vertical elements was rejected due to a risk of liquefaction of the underlying sands.

A special program of the staged geotechnical monitoring has been developed with necessary instructions in a case of emergency [6, 7] if the permitted parameters are exceeded during a construction period.

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