Geotechnical Aspects of Underground Construction in Soft Ground, Kastner, Emeriault, Dias, Guilloux (eds)
© 2002 Spécifique, Lyon. ISBN 2-9510416-3-2

Assessment of underground construction and deep excavation influence on deformation of structures constructed on unstable soils

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ABSTRACT: The article features the analysis of underground construction and deep excavation influence on changes in stressed-strained conditions of soft clay in subsoils of the existing buildings. Using the example of a new construction in St. Petersburg the article covers the results of studies in the influence of various construction works, e.g. sheet piling, bored piling, bulk excavation, on deformations of the existing buildings.

1 INTRODUCTION

The number of underground construction and reconstruction projects carried out in historical centres of European cities every year has been recently growing. Such projects usually comprise works that have to be conducted in congested conditions adjacent to buildings of historical value. According to European grading (Eurocode 7 Geotechnics) such works classify under Category III of the highest geotechnical complexity demanding geotechnical supervision at all construction stages, especially during earthworks.

The largest recent construction site in the historical city centre of St. Petersburg has been that of Transportation and Commercial Centre of High Speed Railways on Ligovsky Prospect. The site is surrounded by Moscow Railway Station, a famous tenement building (Pertsov House), Ligovsky Prospect roadway and railway platforms.

Within the project scope it was envisaged to construct 11 buildings from 17 to 37 m high with underground structures positioned at max 6 m below ground level. It was supposed that the new complex would incorporate the fragments of partially demolished Buildings 26 and 30 on Ligovsky Prospect. The construction comprised 3 stages (Figure 1). The foundation was designed with 620 mm bored piles.

The geological stratification of the site is typical of the central city areas: 1 – 3 m made-up ground with the absolute upper level at + 8.5 m, postglacial strata comprising 0.5 – 2 m sands of various grade (normally being subsoils of the existing buildings and structures) with inclusions of soft and floating clays; lacustrine-glacial layered clay sands of floating to plastic composition down to – 6 – 8.5 m with underlying clay sands and loams of *Luzhskaya* moraine. The highest unbound ground water level is expected at appr. 2.5 m from the ground level.

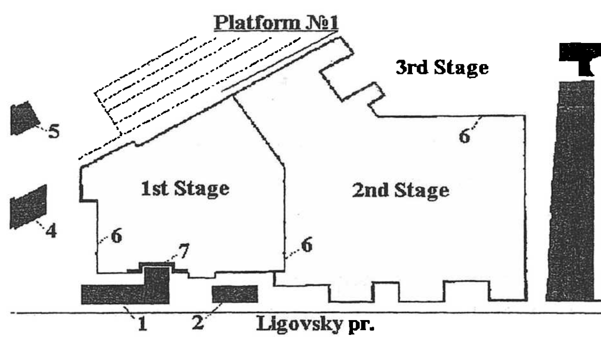

Figure 1. Transportation and Commercial Centre site.

1, 2, 3 –buildings 26, 30 and 44 (Pertsov House) on Ligovsky Prospect respectively; 4 – Administration Building; 5 – Moscow Railway Station Building; 6 – sheet piling cofferdam; 7 – secant wall of CFA piles.

Geotechnical monitoring was introduced to ensure safety of the adjacent buildings and structures. The scope therefore comprised the adjacent Administration Building of October Railway, Platform One of Moscow Railway Station, buildings 26, 30 and 44 (Pertsov House) on Ligovsky Prospect. The scope of works included settlement monitoring on buildings and structures, visual survey, vibration parameters control on buildings under preservation restrictions and ground water level monitoring around the site.

The study of the drawings allowed definition of risk factors for the surrounding buildings and structures: A – remoulding of ground at dynamic impact occasioned by bored piling or sheet piling in adjacency to the existing buildings; B – instability of the sheet piling cofferdam and the anchorage during bulk excavation; C – increase of critical loads in the ground and development of suffusion (wash-out of fines) in the event of permeability of the cofferdam during dewatering; D – remoulding of soft clay and taking out the excess amount during bored piling.

2 RISK FACTOR A – DYNAMIC IMPACT

Earthworks had begun with reduced level excavation down to 2 m from which level the sheet piles were driven down to 12 – 15 m throughout the entire perimeter of each construction stage. The sheet piles were constructed at 9-15 m from Pertsov House, 9 m from Building 30 and approximately 6 m from Building 26 on Ligovsky Prospect.

We shall now look into the works impact on the adjacent area. Figure 2 presents the settlement data measured on the surrounding structures during the works and following the suspension thereof.

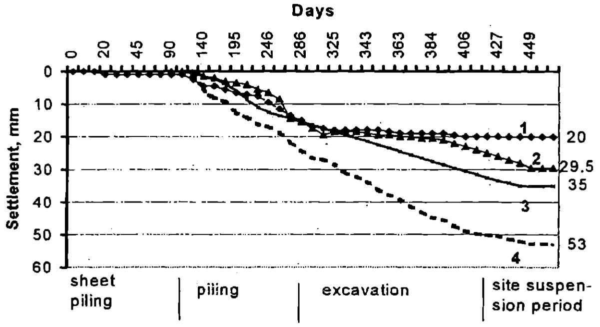

Figure 2. Settlement development in time curves.

1- Station Administration Building; 2 – Ligovsky Prospect roadway; 3 – Railway platform; 4 – Subway Tunnels.

By means of operative adjustment of sheet pile driving regimes and distribution of hammers continuous monitoring of sheet piling made sure that no excess vibration was occasioned on structures of the surrounding buildings. An interesting phenomenon was identified during monitoring. The ground vibration from a hammer was transmitted with negligible attenuation over the considerable distance of more than 45 m onto the foundation of Moscow Station Building. The medium for waves propagation was the subway pedestrian tunnel leading to the Station from the Platform alongside which the sheet piles were being driven. Resulting from oscillation superposition from two hammers located at a considerable distance from each other vibration acceleration reached 0.15 m/s^2. In some station premises the former cracks concealed under the superficial repairs reopened. Health and safety vibration norms were also violated. To rule out any damage to superficial layers and provide comfortable working medium for the station personnel it was suggested to limit the vibration level to 0.05 - 0.07 m/s^2 which was possible to implement with only one hammer engaged in work. This recommendation was rather effective as the settlement of the Station Building never exceeded the accuracy range of instrumentation over the entire sheet piling period. Sheet piling in the vicinity of Pertsov House and Building 30 was likewise successful with settlement not exceeding 1.5 mm and 3.5 mm respectively.

In the immediate adjacency to Building 26 the wall of secant 620 mm bored piles was envisaged by the design. The piles were designed as 'soft' (uncaged) 10.5 m piles placed at 900 mm between centres and 'hard' (caged) 18 m piles. The auger with 250 mm rod with three flights per meter length was used to construct the secant pile wall.

The standard procedure consists of screwing the auger and the tremie pipe into the ground for the entire pile length following which the auger is extracted by 10-15 cm, the valve in the tremie is opened and the concrete mix is pumped into the bore whereat the auger is extracted without rotation. The reinforcement cage is subsequently oscillated into the concreted bore.

This method internationally known as CFA (Continuous Flight Auger) had been successfully applied in many countries, however no previous experience of its application existed in the ground conditions of St. Petersburg. That is why the local geotechnical experts tried to advocate research targeted at adjustment of CFA method to the ground conditions of the site. It would have been pertinent to establish the most reasonable auger geometry (the angle and pace of flight), the speed of rotation and immersion, as well as the mix pumping pressure. Quite deplorably, no such research had been conducted and the application of CFA commenced in the most sensitive location as the construction of the secant pile wall adjacent to Building 26. The effect was not in the least delayed in its manifestation as the settlement soared by 50 mm in the fourth day of piling works.

Settlement stabilisation in remoulded weak clays is a process of considerable duration. In order not to suspend construction by an indefinite period we suggested a sheet piling option *"in situ"* of the secant pile wall. We also put forward an appeal for either strengthening or demolition with subsequent replication to be carried out on Buildings 26 and 30 and proposed a 20 m restriction area for CFA piling until either strengthening or demolition has been completed.

Within the next two months the settlement development on Building 26 continued as a flattening out curve peaked at 100 mm with no corresponding areas for sheet piling impact. But the attempt to construct several CFA piles along the border of the restriction area resulted in rapid settlement surge on the unstrengthened Buildings 26 and 30 in the order of 70 – 90 mm. Such increment practically ruled out the possibility of preserving the buildings.

The reason whereby the critical situation was brought about was the CFA piles. As witnessed on site the amount of soil augered out of the bore three times exceeded the auger volume. This effect was ushered into regrettable reality by the complexity of establishing the optimal flight pace which would have allowed passing through various strata (sand, silty sand clay and stiff moraine) in the elementary 'screw' mode when each turn by 360° would cause the auger to immerse by one flight. When drilling through moraine the speed of auger immersion was considerably lower and the drilling regime approached that of rigid rotation. As a result an excess amount of soft silty clay was taken up by the auger. This created strain in the ground which caused local remoulding in soft strata around the bore. The following concrete pumping at maximum and average pressure levels of 0.9 MPa and 0.4 – 0.5 MPa respectively caused compression almost two times as strong as the natural rigidity of soft clay and the corresponding progressive remoulding. That is how the pressurised concrete pumping not only failed to restore the ground properties present prior to augering, but actually made things worse.

Concrete pumping actually precluded relaxation of strain in the ground and the following fact is demonstrative thereof. The construction of each subsequent pile in a pile cap took more time to pass through the moraine strata whereat the concrete volume increased in proportion to the time used to drill through the moraine. At the theoretical bore volume of approximately 8 m^3 the first pile in a pilecap took 10-12 m^3 of concrete, whereas the last one took 20-25 m^3 (in some cases as much as exceeding 40 m^3 of concrete).

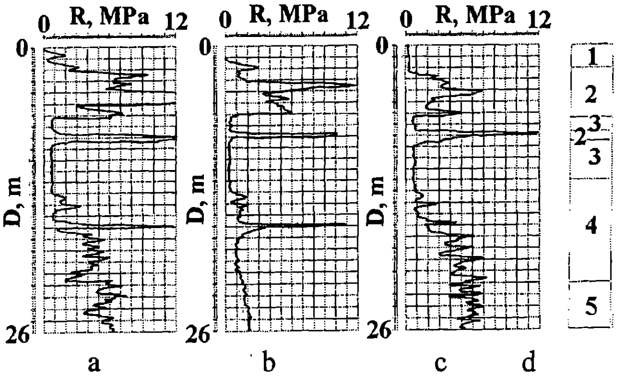

Figure 3. Results of cone penetration test (CPT):

a – prior to works commencement (1997); b – during CFA piling (August 1998); c – following eight months after CFA piles construction (May 1999); d) – core strata: 1 – made-up ground; 2 – medium grain sand; 3 – floating soft clay sand; 4 – floating soft layered clay loam; 5 – moraine loam.

Changes in the ground properties during CFA piling were identified during CPT tests the series thereof being conducted during initial site investigation prior to piling (1997), following construction of each pile in a pilecap in August 1998 and eight months after piling in May 1999. Cone resistance was twice as low in soft strata and diminished by one third in the moraine, the peak values having reduced also in surface sands (Figure 3). The registered situation repeated the results of CFA piling research conducted by prof. Van Impe in 1980s. In the following six months the mechanical properties of clays was practically completely restored. The cone resistance curve practically repeated the initial.

The conducted research made it possible to establish that in order to make CFA piling a more sparing method in the ground conditions of the central part of St. Petersburg the following changes must be introduced to the standard technology:

- rotation and immersion of the auger to be coordinated so that each turn by 360° would cause the auger to immerse by one flight (to fulfil this requirement it is necessary to calculate the flight pacing and angle as well as the speed of rotation and immersion;
- additional vertical load on the auger to be ensured to drill through stiff moraine strata;
- pumping of concrete to be carried out with minimum pressure necessary for satisfactory mix delivery through tremie (0.1 – 0.2 MPa);
- the possibility of replacing the wet slump mix with a more rigid mix to be considered and appropriate changes in mix delivery through tremie to be introduced.

Restriction in using CFA technology was effective in providing safety of Building 44. However, the scale of influence of CFA technology on soil defeated all expectations. Piles constructed under CFA technology at the distance of 30 m from the building caused its settlements despite *Larsen IV 12 m* sheet piles driven between the building and the piling area.

To certain extent the sheet pile wall acted as a retaining wall for the foundation of the residential building to protect it from CFA piling, however it failed to protect it from settlement as the length of the sheet piles was insufficient to have their toes embedded in the roof of stiff strata. As a result of remoulding the shear resistance of soil was reduced, which was attested by CPT, and caused respective reduction of soil bearing on the surface of the sheet piles. Despite small depth of the reduced level excavation the sheet pile cofferdam developed lateral movement by 40-50 mm and even up to as much as 140 mm in nearest proximity to CFA piles (Figure 4). In the conditions of soil bearing reduction the sheet piles failed to counteract the active charge effected by the ground bulk loaded with the weight of a seven storey building.

Unfortunately, these proposals could not be realised without a major change in standard equipment. The only practicable method of controlling CFA piling in order to ensure safety of the surrounding buildings was abandoning CFA method in immediate adjacency to existing structures. CFA piles were substituted for casing protected bored piles in 30 m restriction area adjacent to Pertsov House.

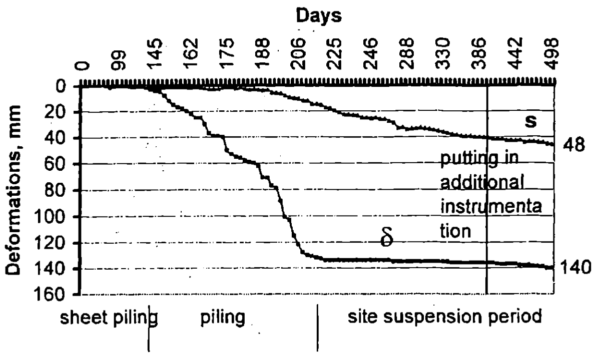

Figure 4. Plots of settlement of the residential building and displacement δ of the sheet piles against time.

The above graph shows that the process of the settlement development started three weeks after beginning of sheet piles deflection. Settlement of the adjacent buildings and structures started to flatten out 6 months after the site suspension. This was related to deformation stabilisation in soft clay due to time-dependent thixotropic hardening of soft clay.

3 RISK FACTOR B – SHEET PILING

Another risk factor on the given site was instability of the sheet piling cofferdam. It was constructed of 12-15 m *Larsen IV* and *V* piles anchored in one or two levels with 100 mm diameter bored anchors of 8-9 m length and 15° angle in horizontal plane.

According to in situ measurements, dewatering within the cofferdam had not resulted in any considerable ground water level reduction in the observed wells which was demonstrative of reliable integrity and water tightness of the cofferdam.

During bulk excavation near the eastern border of the site some particular tension cracks appeared alongside the cofferdam in the former Platform One with opening up to 3 cm and on the nearest railway line with opening up to 2 cm. The overall crack opening value corresponded with deflection of sheet pile tops into the cofferdam, this being 3-4 cm according to routine geodetic readings. The most probable cause for deformation development was the movement of the ground bulk together with the sheet piling and the anchors into the excavation.

Geometry and structure of the cofferdam and the anchorage were taken by the designers based on widely attested calculation methods. The engineering methods are normally used to calculate the loads in the elements of the cofferdam and the anchorage, which are subsequently used in selection of required parameters fro these temporary structures. The employed methods however do not allow any assessment of the stress-strain state of the surrounding soil. We carried out precise finite-element solution of the modelled situation. According to calculations the maximum deflection of the sheet piles at bulk excavation would total 4.8 cm which would stand in good correlation with the actually measured values.

The conducted analysis confirmed the reliability of the actual cofferdam solution as far as the bearing was concerned. However, the other extreme parameter – the deformation – had escaped the designers' attention. This parameter was especially important for evaluation of safety of the surrounding buildings, particularly of Pertsov House, and the ensuring its stability with no corresponding stability of the cofferdam proved a difficult task to resolve.

The conducted numerical investigation showed that the design solution provided for stability of the batter but failed to rule out the building deflections. The maximum calculated displacement of unanchored sheet piles at complete bulk excavation was established at 10.1 cm, whereas with the designed anchorage this would have reached 5.2 cm. Thereat in the second case the land slide behind the sheet piles was ruled out but the effect of engaging into action the soft clay underlying the sand strata was manifested. The additional foundation settlement was 4.0-4.3 cm with lateral displacement of 1.9-2.3 cm which exceeded the permissible values for buildings of such category.

The building deflection at bulk excavation can be explained in the following way. The sheet pile toes below the level of the excavation bottom were embedded completely in the soft soil. These strata have low deformation modulus and are characterized by a higher instability factor. The sheet piles got deflected into the excavation without much ground resistance. During sheet pile deflection particularly below the excavation bottom the deformation process engaged the ground mass at distances corresponding to the length of sheet piles.

As shown by calculations the sheet piles have to be perfectly stable to rule out any building deflection. The most well pronounced effect would have been reached with rigid connection at the absolute level +4.8 m BS. Resulting from application of numerical experiments practical recommendations as to the works implementation were prepared.

4 CONCLUSIONS

1. The actual construction carried out in St. Petersburg showed the importance of the issue of ensuring minimum impact of construction on the surrounding area. To ensure minimum impact it is necessary to conduct geotechnical monitoring as well as experimental and analytical investigations.

2. Such construction must be accompanied by the complete geotechnical supervision comprising the following activities:

- geotechnical justification and back up of the design solution taken for implementation, which will make it possible to estimate the risk factors for surrounding buildings;
- complex instrumentation monitoring at all work stages below ground level ensuring the possibility of emergency adjustments of the works schedule;
- adjustments of complex earthworks to local ground conditions using trial sites perfectly fitted with all types of appropriate instrumentation.