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Long term environmental effect of tunnel construction in weak soils

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ABSTRACT: In the present paper the environmental effects of the construction of Tallinn's Merimetsa—Mehhaanika—Tihase sewage tunnel are analyzed, examining in closer detail the changes that occurred place during the construction works in 1987–1992, but also the later effects that only subsided during the last decade. Since three different technologies were used, acknowledging these effects might facilitate future endevours in similar circumstances.

1 INTRODUCTION

The tunnel is 2,2 meters in diameter is and it is supported with reinforced concrete elements; the length of the tunnel is 3 kilometres.

The geotechnical conditions on the line of the sewage tunnel are very complicated. It is situated in a buried primordial valley between Männiku and Pelguranna. According to data received from the Geological Survey of the Estonian SSR, the depth of the valley is about 100 meters. In the base of the valley, Lower Cambrian and Vendian sandstones are exposed. The valley is filled with fluvio-and limnoglacial deposits of different ice ages. The various glacial, fluvioglacial and layers are covered by Holocene deposits, which consist of marine sediments. A thin layer of fill and topsoil cover the marine sediments.

A typical geological section of the area is given in Figure 1.

2 GEOLOGICAL SECTION

2.1 *Soil layers*

Typical geological section of the area would be as follows (figure 1).

1. a thin layer of fill and topsoil (max 1,5 m); often the fill is missing;
2. a complex of marine sand that comprises mostly silty and fine sand interchanging with layers of medium and coarse sand;
3. a complex of weak clay layers; judging by the geotechnical properties, the most prevailing layer there should be limno-glacial deposits, which vary largely in their grading and thickness; the layer mostly consists of clay and silt; in general, clay content in clay complexes tends to diminish with depth and clay is usually located under marine sands; under the clay lies clayey silt;
4. fluvio-glacial sands with a variety of grading. In general, the grain size grows with depth: under the weak clayey silt lies silty- and fine sand, in deeper parts medium sand under which there can be found silty sand and clayey silt layers. Taking into account the grading curve and water content of latter, it can be asserted that the layer is mostly made up of redeposited till from the early formational stages of a glacial river;
5. Till—located 25 to 30 m from the ground level—is a dense, coarse grained sandy silt till; that lies in the depth 40 meters. The soil layers under the till were not geotechnically investigated.

2.2 *Ground water conditions*

On the investigated tunnel line there are two groundwater horizons above the till layer. The upper water horizon above the clay complex lies in marine sands and the fill deposit and gets its recharge from precipitation. The water table lies 0,3…1,0 meters beneath the ground level and the water there is fairly polluted, containing nitrates and phosphates. Under the clay complex there lies a layer of artesian groundwater that is connected with the upper unconfined groundwater horizon via the occasional "openings" in weak clay. From the onset of the excavation works, the pressure level

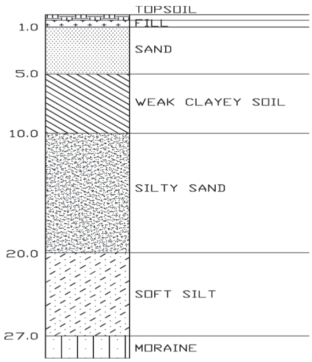

Figure 1. Geological section of the area.

of the artesian water horizon practically coincided with that of the upper groundwater horizon. The recharge area of the artesian groundwater layer extends relatively far—into the Männiku fluvioglacial delta—and has minimally polluted water.

3 GEOTECHNICAL PROPERTIES OF SOILS

The geotechnical properties of soils were examined with field and laboratory tests.

Meremeeste complex. The complex consists of silty-, fine- and medium sands. The upper part of the 0,5–1,2 m consists of silty sand (q_c = 1,2 MPa and $E_{0-0,8}$ = 4 MPa). Beneath the loose sand there lie dense sands with fine geotechnical properties – q_c = 4,0 to 15 MPa, E_{0-3} = 15 to 50 MPa, φ = 30 to 35° and **c** = 10–30 kPa.

Weak clay soils, silty clays and clayey silts and their geotechnical properties must be examined separately.

The natural water content of silty clay (with clay interlayers) is 45 to 60% (mean W_n = 57%). These soils are slightly overconsolidated, with the pressure (p_c) reaching up to 30 to 40 kPa. The yield limit during the plate load test was q_y = 70 kPa and the ultimate load q_t = 100 KPa. The modules of deformation reached 0,3 to 0,4 MPa. The shear strength parameters during the unconsolidated drained test were φ = 9 to 12° and c = 5 kPa. The undrained unconsolidated parameters based on field tests were the following: yield point C_{uy} = 14 kPa and maximum shear strength C_{uf} = 35 kPa.

The geotechnical properties of clayey silt improve with depth. In the upper layers, W_n = 27 to 28% and q_c = 2 to 5 MPa. Downwards, the water content becomes W_n = 22 to 25% and q_c = 12 to 18 MPa. In plate load tests, q_y = 120 kPa, q_f = 500 kPa and $E_{0-1,2}$ = 2,0 to 2,5 MPa. The unconsolidated drained shear test gave the following result φ = 30 and c = 0. The undrained shear strength parameters were C_{uy} = 20 kPa and C_{uf} = 50 kPa.

The soils that lie under the weak clayey layers were investigated only using cone penetration tests. Based on a field test, q_c = 9 to 23 MPa.

The coefficient of permeability in the upper ground water layer in silty sands was 3 m/d and 7 m/d in fine sand which corresponds with experiences from similar tunneling works. The water conductivity parameters in the artesian water layer depend on the grain-size distribution of the soil. The coefficient of permeability in silty sands was 2 to 3 m/d (lab tests and also field tests), in fine sands 5 to 10 m/d and in medium sands 10 to 30 m/d. The given data characterizes the industrial water lowering on the line of the sewage tunnel quite precisely.

4 GROUND WATER

During the first part of the excavation works the complex method of lowering the water table and caisson was used for penetrating the sewage tunnel. An extra pressure of 0,5 at was established in front of the tunnel and continuous water table lowering was implemented, lowering the pressure in such a way that the groundwater level was fixed to 1 to 2 m below the tunnel base. The complex method enabled the execution of normal and accident free excavation works and also the satisfactory advancement speed of 3,5 m/d. In the constructing conditions discussed above, this complex method can be considered the most optimal and best possible option.

The water table lowering along the sewage line was planned with 20 m deep boreholes (Ø = 500 mm) that did not penetrate the artesian water layer. The distance between the boreholes was 25 to 30 m. The boreholes in the artesian layer were equipped with ceramic filters, which worked quite well under the given geological conditions. The coefficient of permeability in medium sands was 20 to 30 m/d. Using this module in calculations coincided with the actual field works.

In September 1987 a groundwater monitoring grid was constructed in order to observe the settling of ground surface due to lowering groundwater levels, discussed further below. This borehole grid encompasses both upper and lower artesian water horizons. At the beginning, measurements were taken daily; later, the interval was 2 to 3 days. The grid was constructed in a beam-like pattern: two boreholes were situated perpendicularly across the construction line and one further downwards on the construction line. The length of the "beam" was 0,5 km and it gave good insight into the spreading of the drawdown and the change of pressure in the water horizons underneath the weak clay layers.

The boreholes describing the upper water horizon showed that the two water horizons were connected. The boreholes near the tunnel line reacted to changes in the artesian water horizon with a delay of one day. A borehole which was located 60 m from the tunnel line reacted to the pressure drop of 1,5 m with a water level drop of 10 cm. No further reaction in the upper water horizon was detected. The reaction of the upper water horizon to precipitation was highly distinctive. A rainfall in September 1987 lifted the upper water table by 0,5 m in two days and this also resulted in an equivalent rise in the water table of the lower horizon (Figure 2). The drawdown of the upper horizon runs in an almost straight line. The pressure drop is 1 m per the 500 m long line, which results in a drawdown incline of 1:500.

The conditions were more complicated in the artesian horizon. Nearly all the boreholes on the "beam" reacted almost immediately to any changes in water pressure. In the reaction times between the boreholes nearest to the line and their neighboring boreholes, a few delays were registered. The formed depression curve can be tentatively divided into two. The first curve describes the distance of 0…130 m; at the minimal level the pressure drop was 2 m and 4 m during the maximum level. For the following 370 m, the corresponding figures were 0,8 and 2 m.

The extension of the depression curve indicates to the fact that the regular calculation schemes are not applicable in the cases of extensive and long-term lowering of water tables. The methods that give good results in calculating the criteria a single borehole tend to give a much lower figure than the actual result when used in cases of extensive water lowering works—hence the need for a separate hydrodynamic analyses of the obtained figures in order to be able to work out more realistic calculation schemes for the future.

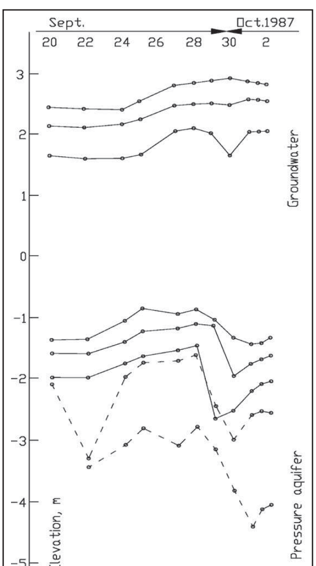

Figure 2. Changes in water table and pressure.

5 SETTLEMENTS

5.1 *Settlements caused by excavation*

Intensive water table lowering and excavation works resulted in settlements in the ground surface. These facts can be linked with two different processes. The first one of these is connected with the excavation works that caused the settlements directly above the tunnel line and the adjoining areas. The settlements were caused by an additional compaction of weak clay layers brought on by caisson and its pressure, the displacement of weak layers due to pressure and the soil outflow to the front of the tunnel. Settlements were smaller in sandy layers due to the greater excavation depth and the dilatation effect, but more significant in the clayey layer due to additional compaction and the soil outflow.

The deformations caused by excavation works were examined in 5 different profiles; the effect of excavations was also analyzed perpendicularly along the tunnel line. The selected profiles were situated in the area of the park, under the street and under the railroad.

The observations made along the line of the tunnel (Figure 3) revealed that the settling of the ground surface started to occur 15 to 20 m from the tunnel line. The main settlements occurred before the face (8 to 18 cm) and after and may continue for some years. The difference in benchmark behaviour cannot be explained with the geological situation, because geological conditions have not changed between the benchmarkers. Extensive ground settlements can be attributed to with the onset of excavation works.

Three of the four cross-sections behaved similarly (figure 3). The settlements on the axis of those three cross-sections were 8 to 14 cm and the extent of the settlement funnel was 7 m from the axis of the tunnel. The benchmarkers situated further on practically did not settle. The fourth cross-section that was located under the railroad settled up to 27 cm and the extension settlement funnel was 7,5 m. To minimize the effect of the tunnel works on the nearby building, a sheet-pile wall was constructed. Its effect was sufficient to prevent any further settlement on the building. An abandoned laboratory building got further settlements of 2 to 8 cm (less than on tunnel axis 12 cm) and developed cracks.

5.2 *Additional causes of settlements*

5.2.1 *Artesian water horizon*

Another cause for the settlements to take place was the pressure drop in the underlying artesian water horizon. It results in a rise of effective stress in weak

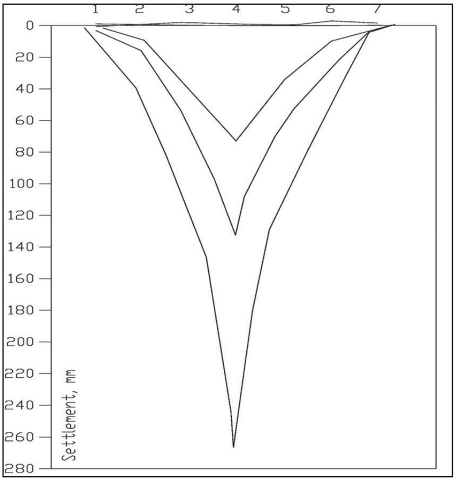

Figure 3. Settlements on cross-section a.

soils and in turn, consolidation takes place. In such cases, it is very important if the rise of effective stress exceeds p_c or not. Preliminary calculations estimated the settlement to be 4…10 cm and the lateral extension to be 0,5 to 1,0 km from the tunnels axis.

The development of actual settlements is a little different and smaller than in theoretical examples. Weak clay layers under pressure (less than p_c) are considerably less compressible than laboratory tests show (because of structural damage that gets done when obtaining probes and the preparation of probes in the laboratory). Analyzing settlements was difficult due to the fact that the geotechnical survey of the tunnel line started later than the actual excavation works, but was eventually somewhat facilitated by the relevant data that could be acquired from the one-time geodetical surveys that had been carried out there.

The settlements on the tunnel line lasted for 6 to 10 years. In area of the park, the settlements lasted for 3 years and in addition to the initial 18 cm settlements there were additional settlements of 5 cm during that time span. The street section with heavy traffic suffered an additional 10 cm settlements in the course of 10 years (initial 18 cm and additional 10 cm). The road surface needed additional maintenance every 3 to 4 years. Under the railroad, immediate settlements were 27 cm; during the next 8 years, the area settled another 6 cm.

5.2.2 *Construction and traffic*

Considering the common occurrence of weak clay layers and intensive construction activities and heavy traffic load, there has always been some settlement of buildings and ground surface. The intensity of the settlements has been quite big. For example bolts driven into asphalt constructed on fill settled for 3,5 years (1984 to 1987). These settlements of 20 to 90 mm depended on the geological section, the thickness of fill and the intensity of traffic: the settlements were 20 mm in places were the fill was absent, 60 mm in places where there was 1,5 m of fill with a low traffic load and 30 mm with the same fill thickness but a heavy traffic load.

Ground surface settlement took place before the lowering of water tables. In locations of thicker fill there may have been cases where the combination of hydrodynamic pressure drops and the loads inflicted by the fill caused the rise of effective stress that exceeded p_c values. However, considering the fact that weak soil layers have been compacted by the load of the fill material and its p_c has risen, it is unlikely that these spots could be very wide spread.

5.3 *Case examples of settlements*

The intensity of the settlement processes can be best described with the example of a school that was constructed on shallow foundations, which consist of differently loaded blocks. In the end of 1970s the maximum settlement for the school was 2,5 cm and minimal 1,5 cm, factoring in also the period during construction works and the time after completing the construction. Due to the uneven load scheme designed for the school, crack appeared in this particular building. The benchmark was levelled again in 1980 and 1986 and the settlement remained 4 mm. After the drop in groundwater levels, the settlements intensified and in the beginning of 1988 it had reached 39 mm. During the following three months, the settlement process slowed down.

The settlements in the Navy Hospital remained 7 mm during a 10 year period. During the first phase of water lowering works, the settlements were 25 mm; later, the settlements continued at the pace of 1 to 2 mm per month, until 1995.

An analysis of the two of afore-mentioned benchmarkers allows us to draw the conclusion that the settlement of 20 to 40 mm was caused by lowering the water table and the consolidation that had been induced by a pressure drop. Currently the consolidation caused by filtration has subsided and settlements occur due to creep (secondary consolidation). The settlements for the past half year were 3 to 15 mm (figure 8). Taking into account other similar surveys in the city district of Lilleküla, these settlements should subside in steady conditions in 2 to 3 years of time. Hence, the predicted settlements were 20 to 30% bigger than the maximum settlements in the area and 50 to 70% bigger than average settlements.

However, some of the benchmarkers behaved differently. These are the benchmarkers in the catering block of the Navy Hospital and in the boiler-house of the Children's Hospital.

The floors in the catering block suffered the settlement of 11 mm, which can be explained by the fact that the backfill material under the floor had never been compacted evenly and thus it compacted unevenly after the completion of floors. The fill material also put a load on the weak clay layers, these compacted more and hence supplemented to the existing settlements. The maximum settlement under the floor happened in the middle zone of the floors, while the minimum settlement occurred near the grillage due to friction between the piles and the soil. During the lowering of the water table, the floors cracked off the grillage and there occurred additional settlements.

6 BUILDINGS ON PILE FOUNDATION

Two types of behaviour can be observed in buildings founded on pile foundations, depending on the soil layer the pile tips are inserted into. on to which. The first group consists of piles rammed into the fluvioglacial sands through weak clay. In accordance with the initial project, these piles were loaded with 30 to 40 tons (considerably less than prop limit). The buildings on these piles got settlements of 1 to 3 cm during the excavation works. The settlements faded in 1 to 2 years after the excavation works. The pressure drop in the artesian water horizon did not cause any additional settlements.

Different conditions appeared in Children's Hospital. The piles were driven into silty sand. The middle row of the piles was loaded over the yield limit. Due to this fact a large web of cracks developed in this building during its construction. After the completion of construction works these cracks were covered and forgotten. However, after the water table lowering these cracks reappeared and caused panic among the staff of the hospital, the children of the hospital were immediately evacuated. The immediately started geotechnical survey established that there were very little additional settlements (during the survey these never exceeded 2 to 6 mm).

The onset of cracks due to a drop in pressure has proved that those initial cracks must have been already there beforehand. At first, the cracks appear quite intensively, however, these are generally not dangerous and do not interfere with the normal exploitation of the building, except in the cases when the buildings have had dangerous cracks before the water lowering process. If this is the case, the building needs additional reinforcements, because the drop in pressure can cause the structure to collapse.

Additional cracks can also appear between two building blocks that are loaded differently, especially if the heavier blocks are loaded above the p_c level and the intensity of settlements rises abruptly. Fortunately these circumstances never appeared during this project. However, in the future these facts must be kept in mind.

Considering all the aspects discussed above, one can say that the technology chosen was the correct one. The problem with using this kind of technology is associated with the need for reinforcements in existing buildings and some renovation works; the expenses were small. The other option would have been to freeze the soil. It would have resulted in a somewhat lesser settlement caused by excavations and a reduction in the water pressure. These effects however would not be too significant.

7 BUILDINGS ON SHALLOW FOUNDATIONS

In the line of the excavations in question the buildings founded on shallow foundations received the settlement of 10 to 25 mm (depending on their weight). It is distinctive for this particular area that the settlements caused by consolidation cease within 2 to 3 years and the creep continues. Before the water lowering began, most of the older buildings had had settlements of 0,5 to 0,8 mm. Buildings on pile foundations had settlements of 2 to 4 cm (piles in sand layer) and 4 to 6 cm (piles in silty sand layer). The settlements had ceased before the water lowering process began.

The further behaviour of buildings was determined by their loading history.

A 3 to 5 storey building on shallow foundations had the additional settlements of 2 to 3 cm. The settlements in question were equable and no new cracks appeared, only some old ones reopened.

One-storey buildings developed further settlements of 3 to 4 cm, however no new cracks appeared either. The ground surface settlements were in the same magnitude. The sewage wells of the systems developed somewhat greater settlements of 4 to 6 cm.

The buildings that were founded on piles inserted into sand layers did not settle. However, the buildings with pile foundations which were inserted into silt settled 1 to 2 cm. The cellar floors behaved in a similar manner for both pile foundations, which received settlements of 4 to 6 cm. On the whole, the technology used induced settlements, but these were not dangerous and only reopened over existing cracks.

The geotechnical investigations conducted 25 years later revealed that the lowering of the water table had not permanently affected the properties of clay layers. However, a very substantial effect was discovered in silty layers. The values of q_c had increased from 1 to 2 MPa up to 6 to 8 MPa. The water content had lowered by 3 to 6% and the bearing capacity of the pile tip had grown 3 times.

8 CONCLUSIONS ON DIFFERENT SEGMENTS OF THE TUNNEL LINE

8.1 *First segment*

The line segment ran along the streets where the private residences were dominant. The ditches to drain rain water had been dug to both sides of the street, between the streets and the residences. Since the thickness of the fluvioglacial sand layer was only 0,1 to 0,6 m, it was decided that no water lowering was to be used. Thus, the excavation works were carried out by using caisson technology. It proved to be successful, because no new cracks opened on private residences that were located in a 12 m radius from the tunnel axis. An exception was a terrace house situated 10 m from the tunnel axis that settled 3 to 10 cm and also developed some cracks. No other damage was registered on this line segment.

Considerably greater settlements occurred directly on the tunnel line. During the excavation works the area between the two drainage ditches settled 15 to 20 cm. After the completion of the excavation works, it settled another 10 cm in the course of 6 years.

On this segment a test polygon was established. Its aim was to register stresses accompanied with the excavation of the tunnel. The measuring station was situated 130 m from the tunnel front and ended at the excavation front. Pore water pressure and dead weight pressure were examined in the given depths. The pressure cells were installed into 4,3 m depth in the clay layer which lay above the tunnel ceiling. The next cell was installed in the depth of 6,0 m into silty layers that lay little above the tunnel base. The last cell was installed in the depth of 9,0 m, just under the base of the tunnel.

The pore pressure above the tunnel did not show any change during the excavation. In the depth of 6,0 m, a drop in pore pressure was registered. Probably the tunnel started to work as a drain. On the other hand, a slight rise in pore pressure was registered in a cell located 9,0 m deep.

8.2 *Second segment*

A situation concerning total stress is more complex. After the installation the cells showed that stabilisation had taken place. This was the case until the excavation works started. The beginning of excavation works is described in cells situated in the depth of 4,3 and 9,0 m with a sudden rise in pore pressure. The cell that was located 6.0 m deep did not react to the onset of the excavation works. This rise continued until the excavation shield was located 80 m from the cells. From this point forward, a slow decline in the total stress was registered. After the shield passed the monitoring station, a slow rise in total stress was observed and it probably continued until it reached the initial value.

The behaviour of the pore water pressure is understandable because all the changes appeared when the excavation shield passed the monitoring station. It is difficult to understand, however, why total stress grew in the course of the penetration of the excavation shield into the soil and then decrease until the shield reached the monitoring station. These measurements also do not explain the settlements under the streets.

8.3 *Third segment*

In the third section the thickness of fluvio-glacial sands and water pressure increased. Since the water conductivity of fluvio-glacial sand in this region is quite big, the cryogenic method was tested. Boreholes with the cap of 0,8 m were established and they reached 4 m into the fluvio-glacial sands. During the excavations, ground settlements were rare but after the melting, the ground settling continued and reached 35 cm in 12 years. The cryogenic method was also preferred because the constructed tunnel had to be connected with other collectors in the city.

One existing water pipe broke due to freezing and caused an outwash of soil and a settlement of 20 cm. Another problematic factor was the loss of strength in clay and silty sand layers after the melting. The decrease was 30 to 45%. To date, it is 85% from the initial strength. Due to loss in strength on one gathering shaft, a breakdown occurred during the construction works.

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