City planning of Hermanninranta – A combination of geotechnics, contamination, and climate change

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Abstract. The Hermanninranta area presents one of the most geotechnically challenging urban planning areas in the City of Helsinki. The area located on the shore of Vanhankaupunginlahdi (Old City Bay) is approx. 50 ha in size. Most of the area is reclaimed from the sea and only Kyläsaari island is a natural formation. Land reclamation in the area was performed in various stages between the 1840s to the 1980s. Natural very soft clay layers encountered in the area extend to a depth of over 40 m below ground surface. Today, the proximity to the city centre makes Hermanninranta very attractive for residential and commercial construction. Thick fill containing contaminated soil, slag and ash on top of soft deep clay, are a challenging combination. Variable fills have caused the settlement of soft clays and the displacement of the embankment towards the bay. Changing current land use for residential purposes will require changing of current ground surface elevation to approximately 2 to 2.5 m upward. This will be an issue impacting geotechnical engineering. Rising sea level caused by global warming shall also be considered. Settlement calculations show that within the next hundred years almost half of the area will be under water in case of no pile slab or soil improvement. The draft city plan has been updated based on these geotechnical, environmental and climate change studies.

1. Introduction

The Hermanninranta district is located in the Kalasatama redevelopment area in Helsinki. The area is an example of modern land-use redevelopment of an area originally reclaimed from the sea for use in industrial applications. The former industrial area will be transformed into a new, livable urban environment. The proximity of Hermanninranta to the city center (figure 1) makes it very attractive for residential and commercial construction. In this paper, land-use considerations for redevelopment of the Hermanninranta shore area in Helsinki are discussed. In a study performed separately, potential configuration and location future city blocks was also analyzed. Land-use planning with reasonable pre-construction and foundation costs relative to property value per floor square meter has proven to be a very demanding and time-intensive process due to challenging geotechnical conditions in the area. Development challenges include natural clay layers covered with various fill materials such as un-engineered soil, ash and slag. This article describes the final stage of the process during which the shore area solution was established, and locations of residential and park areas were specified further.

Except for the natural Kyläsaari island, most of the study area was reclaimed from the sea. Typical soil conditions in the area are characterised by fill units of varying thickness mantling natural clay formations. In the 1980s, a georeinforced, floating, double embankment was built on the soft clay on the eastern shore of the area.
Slag is identified as a special fill material used in Hermanninranta between the 1960s and 1980s, during which time a waste incineration plant operated in the area. Fill in most of the area is contaminated, mostly commonly with hazardous substances including copper, lead and zinc. In several locations, contaminant concentrations exceed the limit values set for hazardous waste. Additionally, PAH compounds and dioxins apparently related to the slag have been detected in the fill. In general, contamination levels are highest in deeper fill layers; topsoil in the area is generally only slightly contaminated.

This study contained a number of activities and engineering stages. The first phase of the study included collecting initial data and historic land use records and planning and specification of geotechnical investigations. New monitoring equipment was installed, and measurements collected according to a programme established during planning. Soil investigation, in-situ measurement, and laboratory data was reported (chapter 2), and these data were used to evaluate stability under drained and undrained conditions of the park area (chapter 3). Further evaluation of settlement and displacement calculations were also performed (chapter 4). The results of the exploration and analysis phase were then used to prepare forecasts of shoreline positions over the next 50 to 100 years, based on the observed conditions and on various predicted scenarios of sea level rise caused by anthropogenic climate change (chapter 5). These data and forecast scenarios were then integrated into a comprehensive whole, allowing development of a shoreline concept situation, estimation of future behavior of the shoreline area and its position, and ultimately a proposal for the recommended, optimum location of the future residential area.

2. Soil conditions and fill layers

2.1. Historical background of fill placement

Changes in positions of the shoreline in Arabianranta, Hermanninranta (formerly Kyläsaari area) and Verkkosaaari areas between the 1840s and the present are shown in figure 1. Most land reclamation activities in the study area were performed in the 1970s and 1980s. Land reclamation was completed using fill composed of a heterogeneous mass of coarsegrained soil with local inclusions of construction case, fly ash, bottom ash, and slag generated by nearby waste incineration activities. Aerial photos taken in 1932 and 1987 are shown in figure 2. Comparative analysis of these images indicates that in general shoreline position has not changed significantly since 1987. Areas of fill known to contain ash and slag are shown in red lines in figure 2; these fills are understood to date to various phases of filling between the 1940’s and the 1980’s [1; 2].

Thickness of fill ranges from 2-3 meters to more than 10 meters. In general, the thickest fill layers are located in the northern portion of the area, beneath and immediately west of the double embankment (figure 2). Fill material and the underlying soft clay have become mixed and, as a result, no clear transition between these materials is observable in many locations. However, at the southern end of the embankment area, complete mass replacement of the clay layer was partly realized [2].

A representative cross-section of the double embankment is provided in figures 3 and 5. The double embankment was built in the Hermanninranta area between 1986-1987, and in Arabianranta area (north of Hermanninranta) in 1983-1985. In the design of the embankment, the shore area was reserved for the park. Construction of Arabianranta to new residential area was realized 2000-2010; in this area, the clay was also subject to settlement and displacement due to fill weight. During development of Arabianranta, the fill and clay layer was stabilized using in situ deep mixing (column stabilization), and all structures were founded on end bearing piles supporting mat foundations [1].

Approximately 300 000 m$^3$ of slag was placed in approximately 10 ha. of the shore area that had been previously filled with surplus soil and construction waste. In the 1980s, the area failed due to soft underlying clay and uncontrolled fill placement and slid into the bay. The location of the failure is designated with a yellow arrow in figure 2. Approximately 130 000 m$^3$ of remaining slag was relocated to the western side of the double embankment and reinforced with georreinforcement and graded to prevent further failures. After placement of the slab, all fill areas, totaling 20 ha., were covered with surplus soil. [2]
Figure 1. Location of Hermanninranta in Helsinki (marked with red ellipse), and historical development of the shoreline in Arabianranta and Hermanninranta areas between 1840 and 2002 [2]. Study area considered in this report is outlined with violet dashed line. The former Kyläsaari island area is circled in a green dashed line (see also aerial photo in figure 2). The study area is approximately 50 ha in size and the area of the island is approx. 2 ha.

Figure 2. Hermanninranta area in aerial photos from 1932 and 1987 [3]. Areas were filled with ash during the 1940s and 1950s, and slag fill areas were completed from the 1960’s through 1980’s; these are marked with red lines. From 1960-1980, the waste incineration plant was located on the island shown on aerial photo of 1932. The double embankment is marked with blue lines (gross section of double embankment is presented in figure 3). The location of the failure in the 1980s is indicated with a yellow arrow.
2.2. Site geology

Generally, the bottom level of the clay layer in Hermanninranta is at approximate depth of 20-40 m (base of the deepest clay units is situated at -40 m absolute elevation). The thickest clay layers are encountered in the south-eastern part of the study area, in the location of the double embankment or to the east of it. Above the level of -15 to -18 m absolute elevation, soil composition varies from mud to clay. The clay transitions to clayey silt below this depth. Water content in the upper most clay layers varies from 80 % to 130 %, whereas in the lower most layer it varies from 60 % to 100 %. A sand layer of variable thickness (2-10 m) underlies the clay unit, which in turn is underlain by moraine and bedrock. Rock surface in the location of the former Kyläsaari island varies from +0 m to +4 m absolute elevation. [7] An example of sounding and laboratory test results representative of the study area is presented in figure 4.

The ground surface in the study area is characterized by low topographic relief, sloping gently towards the sea; ground surface elevation on the eastern side of the double embankment ranges from +1 to +1.5 m, and in the location of the double embankment the elevation of ground surface is approximately +0.5 m. Groundwater table is measured between ±0 and +1.2 m; and level of perched water corresponds to fluctuations in sea level. In the vicinity of Hermanninranta area, the level of sea bottom varies from approximately -0.9 to -1.7 m absolute elevation, increasing toward the south. [7]

2.3. Observed settlements and displacements

Settlement plates have been utilized to monitor settlement in the area since the 1980’s. In total, measurement data are available from 54 settlement plates. The following general observations are relevant to the documented settlement behavior:

- Short-term settlements (i.e. those measured during the construction of the double embankment) are approximately 1 m. Subgrade shear strain is believed to be a contributing factor to the observed settlement
- In the 1990s, the average settlement rate of the double embankment was approximately 160 mm/year. By the 2010s, the average settlement rate had slowed to approximately to 30 mm/year
- On the western side of the double embankment further from the shore, the average settlement rate is approximately 30 mm/year
- Significant variation in the settlement rate is observed over the study area; settlement rate is clearly lower in the north-western and western edges and is almost twice as high in the area between Kyläsaari island and the double embankment. Variation in the settlement rates is attributed to presence of variable thickness of fill layers and the underlying clay layers.

The area has been also monitored since 1986 using 46 inclinometers installed at various locations. The following general observations are relevant to the inclinometer data, estimates of error sources, and analyses performed using the data:

![Figure 3. Design cross-section of the double embankment. The seaside berm on the right (east) is submerged [4].](image-url)
• Between 1980 and 2000, the double embankment moved toward the sea (east). After this time, rate of movement decreased, and during the 2010’s, no clear directional trend is evident along with decreased absolute displacement, indicating that the movement of the double embankment was slowing.
• Movement in the western side of the double embankment was primarily local; displacements appear to have occurred only in random directions and over minor distances.

Figure 4. Sounding and laboratory test profiles from cross-section 1-1. From the left: fineness ratio, water content, combined static-dynamic penetration test, soil types, vane test and cone test. [7]

3. Areal stability analyses

3.1. Required safety factor
Numerous shore areas in Helsinki used for residential and other purposes have been reclaimed from the sea using fill placed over a thick clay layer in either a controlled or uncontrolled manner. The minimum acceptable factor of safety against internal failure that must be achieved during detailed design is 2.5 for building areas, and 1.8 for park areas and open spaces. These requirements are set forth, for example, in the calculation report by Länsivaara, Hartikainen & Janbu [5]. The Hermanninranta shore area was originally planned as parkland in the 1980’s, thus design safety factors are generally lower than those required for building sites.
3.2. Cross-sections, case calculations and parameters
Stability of the development area was assessed for five representative cross-sections oriented approximately west-to-east. Location of the analysis cross-sections is presented in 5a. Stability calculations were performed using the commercially available software GeoCalc using both undrained and effective strength parameters. The undrained parameters were determined using results of in situ vane shear, CPTU and fall cone tests performed in connection with this study. Summary of measurements performed and selected undrained parameters derived from the measurements is presented in 5b. Effective strength parameters were defined for clay layers in accordance with the results of triaxial compression tests performed during the study. Porewater pressure applied in the effective stress calculations was determined from values measured by piezometers.

The calculations were made for three loading conditions: 1) existing situation without external loads, 2) with 20 kPa additional load, and 3) with 40 kPa additional load. The loads chosen are selected to represent additional fill thickness of 1m and 2m for the 20kPa and 40kPa load conditions respectively. In the calculations, sliding surfaces terminate offshore, in the sea. In all calculations, sliding surfaces can develop randomly.

![Cross-sections diagram](image)

**Figure 5.** a) Location of five stability analysis cross-sections is shown by thick violet lines. b) Vane test results and undrained shear strength determined for stability calculations. Above the level of -15 to -18 m, soil varies from mud to clay. Residential area (blocks in the figure) will we constructed to the west from blue lines. [7]

3.3. Calculation results, shore area
Calculations are performed with and without inclusion of the georeinforcements in the double embankment.

For calculation in which georeinforcement is not included: in this scenario, only the onshore double embankment is considered in the first calculation model; the georeinforcement installed in 1987, and the underwater berm constructed simultaneously with the double embankment, are not included. Due to absence of these elements, the calculation outcome is considered conservative, producing calculated safety factor against internal stability failure that is lower than in reality. In all cross-sections of the calculation for the existing situation, safety against internal stability failure is at least 1.8. In the event that additional loads of 20 or 40 kPa are applied corresponding with possible filling plans, the calculated safety factor is less than 1.8, in most cases.
For calculation in which georeinforcement is included: in this scenario the georeinforcement is added based on data from construction. The design tensile strength of georeinforcement required for the construction time in 1986 was 600 kN/m, and long-term strength requirement for 50-100 years was 200 kN/m. Based on stability calculations, factor of safety against internal stability failure in the existing situation exceeds 1.8. However, if additional loads are applied to the double embankment, the required designed strength of the georeinforcement must be higher than the existing strength capacity of the installed georeinforcement currently in place. The additional load of 20 kPa applied to the double embankment was analyzed by stability calculations for the three following scenarios: 1) no georeinforcement, 2) georeinforcement with designed strength of 100 kN/m, and 3) georeinforcement with designed strength of 380 kN/m. In case 1, safety factor in undrained and drained conditions varied from 1.39 to 1.8. In case 2, safety factor in undrained conditions is 1.43 (in figure 6 below, safety without georeinforcement equals 1.39). In case 3, the safety factor in undrained conditions was calculated to be 1.8.

According to settlement monitoring and forecasts, settlements in the area will continue for an extended period of time. Piezometer data indicates that pore water pressure will remain elevated far into the future, and potential new fill layers will further increase the pore pressure level. Impacted by settlements, some fill will settle below the groundwater and seawater level, and consequently loads generated by the existing fill will gradually decrease. At the same time, the shore area will flood more often. Therefore, considering stability and settlement issues, it is not recommended to place additional fill in the location of the double embankment. Elsewhere in the shore area, placement of additional fill in a controlled manner is possible, but related settlements and lateral displacements must be studied on a case by case basis.

![Figure 6](image)

**Figure 6.** Result of stability calculations for undrained conditions. Cross-section 4-4 in figure 5. Applied surface load is 20 kPa, designed strength of georeinforcement is 100 kN/m. Spacing between horizontal lines is 5 m. [7]

### 3.4. Stability calculations of residential area

In figure 5, the residential area with new buildings founded on piles or pile supported slabs (in soft soil conditions), is shown with a blue line. Top level of the pile supported slab is estimated to be approximately 2-2.5 m below future fill grading level. In the residential area, elevations of future ground surface will presumably vary from +3 to +4.5 m; as such, the bottom level of the pile-supported slab will likely vary from +1 to +1.5 m. Elevations of existing ground surface in the area vary from +1 to +2 m, meaning that top surface of fill below the pile supported slab will be on the existing ground surface level or slightly lower. In the previous study [3], lightweight fill structures shown in figure 7 were proposed for the eastern part of the residential area. If lightweight structures are utilized, additional loads are not expected outside the residential blocks.

Construction of the residential blocks will not change the load condition from the fill and may in some cases reduce it. Piling must be designed and performed in a way that produces no net increase of pore water pressure. In this situation for drained conditions with 0 kPa additional loads to ground surface the safety factor against internal stability failure is ≥2.5 (required for residential blocks). Stability calculations performed within the current study [7] are consistent with calculations reported previously [3].
Figure 7. Pile supported slab and lightweight fill in eastern part of residential area (not to scale, distance between houses and sea is several hundred meters). New houses and streets in the western side of the Hermanninranta area shall be founded on piles and pile supported slabs. [3]

4. Settlement calculations and forecasts

4.1. Calculation cases
Settlement calculations have considered five representative points. In connection with the study, samples from the points subjected to oedometer testing to assess consolidation behavior. Calculations were performed for the existing ground level and for the ground level raised by approximately 1 m (additional load of 20 kPa). Stability calculations were performed using the GeoCalc program. Historical background of the points was assessed using maps, aerial photos, cross sections developed in historical and current soil investigations, historic reports and documents, etc. [7]

Calculation results were compared with settlements measured in the adjacent monitoring point. Considering lack of historical data on one hand and accuracy of calculation parameters on other hand (good quality samples and laboratory tests), the calculated and measured settlement values have been confirmed as quite similar, and calculated consolidation settlements have been considered as reliable. [7]

4.2. Estimated settlements in 50, 100 and 500 years
Consolidation settlements predicted over the next 50 years (through 2070) and 100 years (through 2120) are estimated as follows. Providing additional load of 0-20 kPa, settlements in the northern part of the island (north-east and east) will be between 0.5-0.7 m and 0.9-1.1 m. In the southern part (double embankment west) of the latest reclaimed areas, settlements will vary from 0.8-1.0 m to 1.3-1.6 m. In the location of the double embankment, settlements will be in the range of 0.5-0.6 m and 0.8-1.0 m.

Due to presence of a substantially thick clay layer with poor water permeability, secondary settlements are not considered. Porewater measurements indicate that the thick clay layer present on site remains in the early stages of primary consolidation after 50 years loading. Based on calculations and measurement data, primary consolidation settlements produced by current loads will continue for hundreds of years into the future. Primary and secondary settlements will occur simultaneously due to the low permeability of the clay unit. When secondary consolidation begins to develop in the top and bottom levels of the clay layer adjacent to coarse, water-bearing soil, most of the clay layer will remain in the primary consolidation state. If loads are increased by lifting the ground level corresponding to rate of settlement, pore water pressure will rise, and pore water will discharge slowly; in this case the clay in
the middle of the unit will never reach the secondary consolidation stage because primary consolidation continues indefinitely. In reality, secondary consolidation is probably developing currently in plastic/liquid-plastic surface layers of clay; thus, actual settlements may be larger than calculated consolidation settlements presented above.

A sample from one point was used to estimate settlements over a period of 500 years. It is assumed in calculations that load generated by additional fill is 20 kPa and fill is placed gradually to balance with buoyancy that caused fill material sinking beneath the water. Between 2020 and 2470, calculated settlement is approximately 2.5 m and continues beyond 2470.

4.3. Forecasted shoreline position in next 50 and 100 years

Settlement calculations and measurements were used to forecast shoreline position over the next 50 and 100 years (2017 and 2120) for two scenarios of sea level (figure 8). In scenario 1 it is assumed that the sea level does not change relative to the current elevation, due to ongoing isostatic rebound of ground surface level in Helsinki. In scenario 2, the sea level increases significantly, outpacing isostatic rebound. Scenario 2 proposes that by 2070, the sea level rises approximately 0.5 m, and in 2120 the increase is 1.0 m. Estimations of sea water mean level are based on the sea level forecasts made by the Finnish Meteorological Institute; projected isostatic rise of the ground surface level in Helsinki area has been also notified by these forecasts. [6]

![Figure 8](image)

**Figure 8** Forecasted shoreline position in next 50 years (2070) and 100 years (2120) based on scenarios 1 and 2 [6]. In scenario 1 sea level does not rise, and in scenario 2 sea level rises appr. 1 m in 100 years (considering rising of ground surface level) to appr. level +1.1 m (N2000). Settlements are assumed to occur in accordance with settlement calculations and no new fills are considered. [7]

The relationship between ground surface level in the settlement calculation and the project change in sea level was analyzed, using both existing and estimated ground surface levels and sea levels in 2070 and 2120 based on the two scenarios. The data was used to determine the ground surface level in the absence of new fill in the analyzed points in comparison with the sea level in 2070 and 2120. Depending
on the point, time, and sea level scenario, ground surface may be either above or below the water. Calculation outcomes are summarized in a map (in figure 8) showing the shoreline position in 2070 and 2120 estimated by different scenarios. No additional fills are considered in the eastern part of the residential area after 2019 in preparation of the map.

5. Summary and conclusions
Based on the calculations, in the current situation the safety factor of the entire shore area exceeds 1.8, and therefore the shore area can be used for the park construction. The double embankment structure will, however, be under water in next 50-100 years due to a combination of settlement and projected sea level rise.

Due to the substantially thick clay layer encountered in the study area, application of ground reinforcement methods in the shore area in order to improve areal stability and to reduce settlements is challenging and expensive. Therefore, in the park area, ground improvement is not recommended and, in fact, not needed.

Based on the performed study, it is possible to guide the land use planning of Hermanninranta area in an optimal cost-effective direction. A distinct and memorable urban area can be created if an extensive part of the shore area is allocated for the park, and slow transformation of the area into a flooding zone lined with reeds is accepted by city planners. By allocating future residential areas further from the shore, excessive foundation costs will be avoided, and housing needs will be secured. Additionally, treatment requirements of contaminated soil for the park area are reduced in comparison to demands required for a residential area, and thus in the park case, related treatment costs are substantially lower.

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Acknowledgements
Authors would like to acknowledge the City of Helsinki.