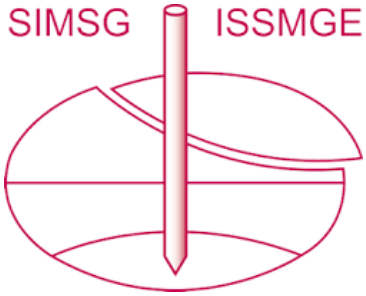
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Foundation of a new bridge over the Göta River in Gothenburg

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ABSTRACT
A new bridge, 1500 m long, is planned to replace the existing bridge, Göta älvbron, in order to secure a good connection between the downtown of Gothenburg and the large industrial Hisingen region to the North in the years to come. The new bridge, named Hisingsbron, is designed with a central section that can be lifted to provide a 28 m high free boat passage. Unlike other bridge spans, this movable part and the sections adjacent to it to the North and the South have to be constructed to high dimensional tolerances, and cannot accommodate any differential settlement. The site consists of a very deep deposit of soft clay, from 40 m to more than 100 m, overlaid by about 3 m fill on either sides of the river. This setting poses a challenge to the foundation work because of creep settlement. This article describes the relevant geotechnical conditions, the foundation for the different supports of the bridge, the calibration of the soft soil creep (SSC) models, and the utilisation of the models to numerically analyse the creep settlement with time as well as its effects on the piles (drag load and downdrag). Suggestions are made to minimize the drag forces by allowing a certain settlement of the piles in the southern and northern bridge sections, eventually in combination with other methods such as protecting the piles, in the part subjected to drag forces, with bitumen or with a sleeve. The protective sleeve will be in contact with the pile at a number of points.

Keywords: Bridge, Pile foundation, soft soil creep, drag

1 THE NEW HISINGSBRO

A new bridge, named Hisingsbron, is planned in downtown Gothenburg. By allowing an efficient connection over the Göta River between the city center and the large island of Hisingen to the North, it will contribute to the development of the city and create a strong region with conditions favorable to the shipping industry.

Figure 1 shows the four present links connecting the northern and southern shores of the Göta Älv River in central Gothenburg.



Fig 1. Present connections between the main land and Hisingen.

In 2013 The Municipality of Gothenburg, owner of the bridge, announced a competition for the design of a new bridge across the Göta River. Five teams were qualified for the final round, and the winner

was the team who presented a design named "Arpeggio" for the specific bridge section across the river. The criteria for the decision were: viability, development and functionality.

In total, the new bridge will be approximately 1500 m long, out of which the Arpeggio part will have a length of 440 m. The movable section is carried by four 50 m high pylons. It will be 12 m above mean water level, but can be lifted up to provide a 28 m high free ship passage.

The existing Göta Älvbron is in bad condition due to ageing brittle steel, which requires a high maintenance cost every year. It is, in fact, necessary to monitor it on a 24 hour basis in order to guarantee traffic safety.

The new bridge must be built and opened for traffic before the existing bridge can be demolished, since this is the only tramway connection between Hisingen and the mainland.

1.1 Layout and location of the bridge

Figure 2 shows an artist rendering of the bridge. The location of the bridge is about 60 m to the east of the aging Göta Älv Bridge that the new bridge will replace.



Figure 2 Hisingsbron looking East (courtesy Trafikkontoret, 2014).

As mentioned above, the bridge will be 12 m high. This is 7 meters lower than the existing bridge. The reason why the bridge will be built lower is that it enables traffic to be rerouted closer to the river and thus release more than 70 000 square meters for housing development. The low height also makes it easier for pedestrians and cyclists to cross the bridge.

Since the bridge in the future will be surrounded by buildings on the southern shore of the Göta River, not much of it will be visible there. However, on the northern shore the bridge will be surrounded by green areas and become a more prominent sight.

The waterway between the two pairs of pylons is about 30 m wide with a depth of about 6.5 m. The bridge central section can be lifted up to 28 m, as shown in Figure 3, to allow tall ships to pass. This operation will be necessary for cargo ships, which can clear the existing 19 meter high bridge. The bridge will not be opened on morning and afternoon at rush hours. This decision has met with opposition from ship owners, as ocean-crossing ships must have a minimum height.



Figure 3 The Bridge in uplift position 28 m above water level (courtesy Trafikkontoret, 2014).

Building of the bridge will start in 2016 and will be completed in 2020, after which the existing bridge can be broken down (an operation that takes about 6 months to complete).

1.2 Historical review

The decision to build a high level bridge was taken in 1933. The bascule bridge, named "Göta Älvbron", started operation in 1939, see Figure 4. It has almost reached its life span and has to be replaced at the latest in 2020, when the new Hisingsbron will be completed.



Figure 4 The Göta Älv Bridge from 1939 onwards (courtesy Trafikkontoret)

1.3 Foundation of the existing Göta Älv Bridge

The bridge is founded on 46 supports, ten of which are over water. Thus the river section has 9 spans, see Figure 5.

The supports are founded on jointed timber piles with a length of 36 meters. The shear strength of the clay at this depth is 4-5 times higher than in the shallow layers. Most of the piles are friction piles except for the ones at the bascule bridge support and the support north of that in the middle of the river section. Since the bedrock rises steeply in this stretch these piles are end bearing. Many of the piles are driven with an inclination of 5:1 and 6:1 (v:h), due to horizontal (mostly dynamical) loads, in addition to gravity loads.

The adjoining fenderings are 110 m and 265 meters long and they are founded on 18 m long timber piles.

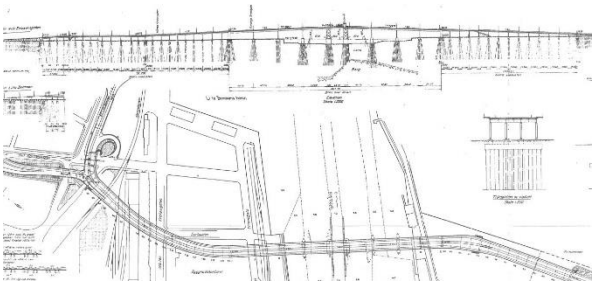


Figure 5 The Göta Älv Bridge. Assembly drawing from 1955 (courtesy Trafikkontoret).

2 THE SITE

The Göta River valley is a low-lying area with a very deep deposit of clayey sediments. The thickness of the clay layers amounts generally to 90 -100 m at the southern shore of the river. In the middle part of the river, the clay deposit is less thick, 30-40 m, because of the presence of a high local bedrock, whose surface is very steep southward. North of this bedrock, the soil depth increases steadily and attains around 70 meters near the northern shore. Further north the clay thickness diminishes again, see Figure 6.

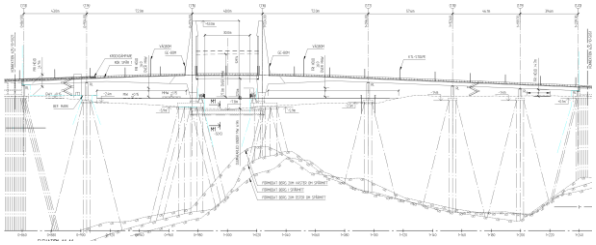


Figure 6 Estimated bedrock surface.

2.1 Geological setting and hydrology

The Southern part of future Hisingsbro will be founded in an area with a very thick clayey deposit, up to about 90 m. The clay there is overlaid by a ca 3 m thick fill and rests on a 3 m thick till layer (sand/gravel) that covers the bedrock. The groundwater level (GWL) is between 0.5 and 1.5 meters below the ground surface (GS).

The undrained shear strength, c_u, is very low to low from GS to 20-25 meter depth. It becomes medium at 40 m depth, and high at greater depths. The clay layers are normally to slightly preconsolidated. The upper part of the clay deposit is post-glacial, whereas the lower part is glacial. Fill materials have been laid out on top of the clays more recently, probably in 1860 and in 1890.

The situation is similar in the North River bank. Here the clay deposit has a thickness of about 50 m, and the fills are laid out probably in 1885 and 1945. In both the northern river and the southern banks the transition zone between glacial and post glacial clay can be found about 5- 10 m above the middle line of the deposit.

2.2 The geotechnical features

The clay at the southern part of the river bank has an undrained shear strength c_u which varies from 15 to 25 kPa at 3 m below GS. From there it increases to 60 ±5 kPa at a depth of about 35 meters and attains 100 kPa at a depth of 60 m, see Figure 7.

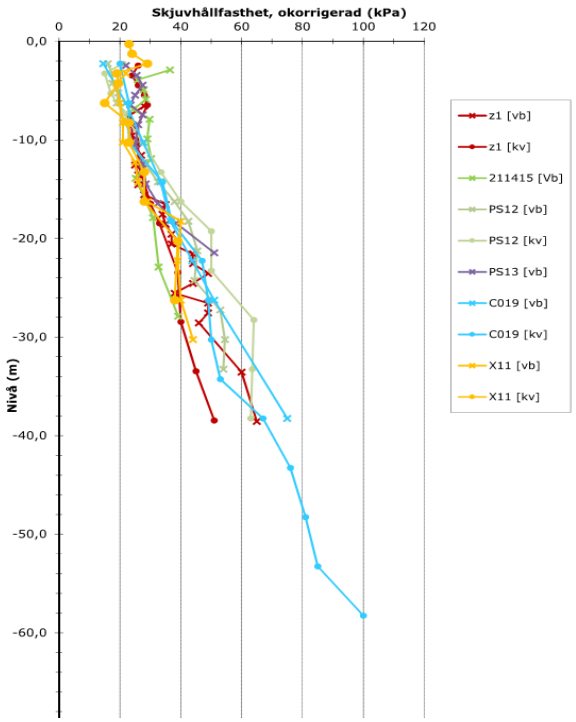


Figure 7 Undrained shear strength at the southern bank.

The bulk density of the clay varies between 1.55 and 1.7 ton/m³; the higher value can be found in the deep soil layers. The water content varies between about 80% and 55%; it decreases generally with depth.

At the Northern Bank, the undrained shear strength c_u increases with the depth, from about 17 to 20 kPa just below the fills to 45 - 55 kPa at about 25 meters below GS, see Figure 8.

The bulk density of the clay varies between 1.5 and 1.7 ton/m³; the higher value belongs to the deep layers. The water content varies between about 90% and 50%; a general decrease with depth has been observed.

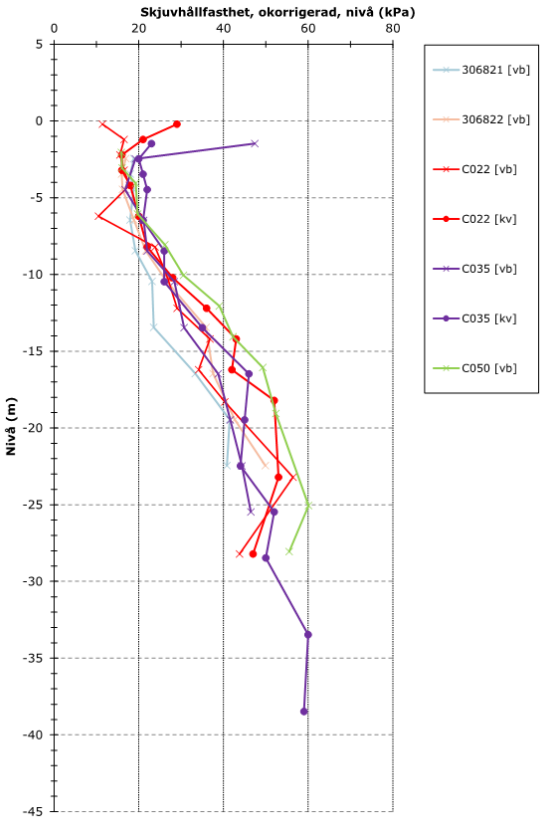


Figure 8 Undrained shear strength at the northern bank.

3 THE FOUNDATION

The pylons of the movable bridge section and the supports of the adjacent bridge spans will be founded on point bearing piles. Most probably circular bored steel piles will be utilized with their toes resting on the bedrock or socketed in the bedrock. The supports of the other spans can be founded on friction piles to allow a certain settlement which reduces the effects of negative skin friction.

3.1 Overview

Topographical survey carried out the last 2-3 decades at different places in the southern river bank reveals a GS settlement of 3-5 mm/year; the higher figure relates to places with 5 m fill. When designing a pile foundation in such a geological setting one must not ignore the drag forces on the piles and the downdrag on the foundation. The negative friction develops with time and must be carefully studied. The situation at hand differs from the usual cases where settlement

of the soil surrounding a pile foundation is essentially caused by fills which are laid out at the construction time. In such a case the settlement and the ensuing negative skin friction can be analysed in a conventional manner using directly the soil parameters at hand. In our case, the surcharge was put on the post glacial clay which rests on the glacial deposit in the 1880s. It induced a new consolidation and creep process, the effects of which are still noticeable today.

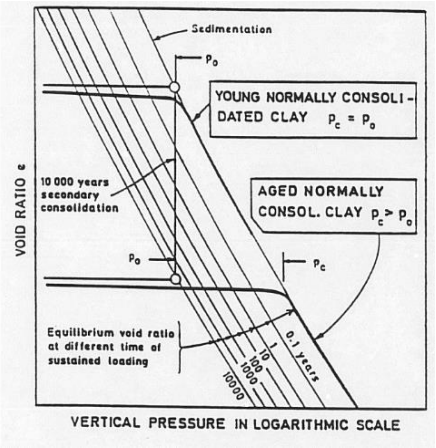


Figure 9 Geological history and compressibility (Bjerrum, 1973).

3.2 Soil models and their calibration

The issue is illustrated in Figure 9 in which Bjerrum (1973) outlined the effect of a recent loading on a clayey deposit that has consolidated and crept for thousands of years. If the loading is large enough, it brings the aged clay to the normally consolidated stage (OCR=1). The quantitative assessment of the process requires the construction of soil models which reflect the geological history and compressibility of the different parts of the construction site and adequately reproduce their behaviour during the last 2-3 decades. Such a soil model will form a basis on which prediction of creep settlement and its effects on the piles can be calculated for the life time (120 y) of the new bridge. The task was done by employing the Soft Soil Creep (SSC) constitutive law which is incorporated in the finite element (FE) program Plaxis 2D-2015. Its application requires input, for each layer, of the strength parameters c', φ', ψ (dilatancy) and the stiffness parameters λ^*, μ^*, and κ^*. The latter are called modified indexes for compressibility, creep, and swelling respectively – modified with respect to similar parameters defined in Critical Soil Mechanics. They are related to the initial void ratio e_i, and the more familiar parameters C_c, C_α, and C_s from incremental oedometer testing.

$$\lambda^* = C_c/2.3(1+e_i) \quad (1)$$

$$\mu^* = C_\alpha/2.3(1+e_i) \quad (2)$$

$$\kappa^* \approx 2C_s/2.3(1+e_i) \quad (3)$$

$$log(k/k_i) = \Delta e/c_k \quad (4)$$

Equation (4) defines the decrease of the coefficient of permeability *k* with the compression ($\Delta e < 0$). Plaxis (2015) recommends setting $c_k \approx C_c$.

In the modelling and calibration work, attention must be given to the geology of the deposits, the loadings in the 1880s, the soil properties, the GS settlement rate during the last 2-3 decades, and the recent observations of settlement pattern and pore water pressures. A major issue remains, however, with the soil properties, because they are derived from field and laboratory investigations conducted in 2014 and do not directly reveal the conditions of the clay layers prior to the loadings in the 1880s. Obviously the void ratio e and the coefficient of permeability k must be higher then. The other parameters were also different since the clays were weaker and more compressible. The work included several series of trial and error FE analyses accompanied by a number of minor hand calculations, always keeping in mind the conditions cited above. Also the empirical values or relationships concerning C_c, C_α, and C_s were utilized. These are reported by P. von Soos and J. Bohác (2002), by K. Terzaghi et al. (1996), and by Trafikverket (2014).

3.2.1 The Northern part

Figure 10 represents the soil model for the Northern part in which a vertical pile row (or

pile wall, PW) is installed 3 m above the bottom till layer. The horizontal lines in the figure delimit the different soil layers whereas the fine red curves represent the magnitude of 120-year settlement within the soil mass. All points on a curve have the same settlement (iso-curves).

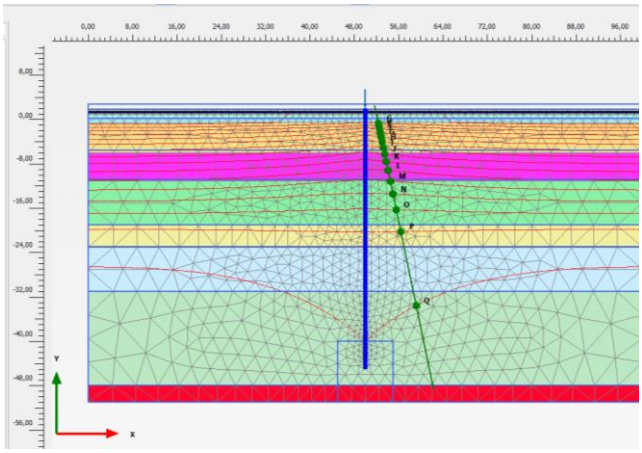


Fig 10 N-soil model for northern river bank.

The model consists of 6 clay layers denoted from top to bottom CL1 to CL6. On top of CL1 are 2 layers of fill, and below CL6 is the bottom till layer. It has been assigned to all clays $c' = 0.1\ c_u$ *(kPa)*, $\varphi' = 30^o$, and $\psi = 0^o$. The other relevant SSC parameters are compiled in Table 1.

Table 1 N-soil model. Relevant SSC parameters

Clay	c'	λ^*	$10^3\ \mu^*$	$10^3\ \kappa^*$	e_i
CL1	1.7	0.1133	0.7931	3.399	2.07
CL2	2.2	0.1226	0.8519	3.672	1.96
CL3	3.8	0.1318	0.5599	4.743	1.64
CL4	5.3	0.1142	1.939	5.255	1.78
CL5	5.9	0.1251	3.128	5.255	1.78
CL6	7.2	0.1210	3.024	6.048	1.66

When calibrating the model, the parameters C_c, C_α, and C_s were taken into consideration, and the ratio $C_s/\ C_c$ was kept within 0.02 ±0.01. As to the fills and the very stiff bottom till layer, their behaviour was simplified to a drained Mohr-Coulomb soil type.

Figure 11 shows the result of the analysis of consolidation-creep in the period from 1885 to 2134. Preceding that calculation phase was a consolidation-creep analysis of the glacial and post glacial deposits without any fill over a period of 10 000 years. It was a preparatory analysis to bring the deposits to the state they were supposed to have in 1885 when the first 1.5 m thick fill was laid out. It was assumed that the second fill (1m) was put on place in 1945.

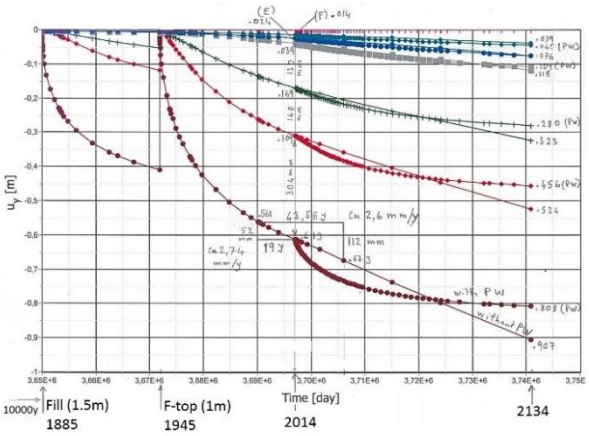


Figure 11 Settlement in period 1885- 2134.

The curves in Figure 11 represent the time settlement of the upper boundary of CL1 to CL6. The lowest curve belongs to CL1, the curve above it to CL2, etc.… The Plaxis technique of resetting displacements to zero while preserving the obtained stress state was utilized in the beginning of the calculation phase 1885-1945 and again in the beginning of the phase 1945-2014 in order to bring the ground surface to the elevation from which it settled down to the 2014 level. It can readily be seen that the GS settlement rate amounts to 2.7 - 2.8 mm/y during the last 2 decades. This result is deemed realistic since the clay deposit here is not as thick as in the south.

Another result worth mentioning is the calculated settlement until year 2014 of the upper boundary of the uppermost clay layer CL1: 613 mm. This means that the GS which was assigned the elevation of +2.7 in the very beginning of the analysis is now lowered to 2.7 – 0.613 ≃ +2.09. If one accounts for some compression of the fills, then the calculated GS elevation is very close to +2.0 which is the result of survey in 2014.

Inspection of figure 11 reveals that 94% of the settlement until 2014 is caused by the compression of the 3 upper clay layers: 304+140+130 = 574 mm. This means that most settlement occurred within the upper 20 m. Also this calculated result correlates well

with field measurements conducted the last few years.

The model was also checked with respect to pore water pressures. These were monitored in 2014 with piezometers installed at elevations -8, -18, and -28. The relative difference between calculated values and field data does not exceed 1.1 %.

Table 2 Pore water pressures (kPa) in 2014

Point (elev.)	Field Data	Calculated value	Relative difference
K (-8)	99.0	99.7	+0.7 %
L (-18)	198.0	200.2	+1.1 %
M (-28)	299.0	297.5	-0.5 %

The results presented in Table 2 are a further validation of the N- soil model.

Figure 10 shows the application of the model for analysing the effects of creep settlement on a pile row over a period of 120 years. The pile row is modelled here as a rough thin concrete pile-wall (PW) loaded on the top with a vertical line load of 600 kN/m. It can readily be seen that the upper soil layers settle down and hangs on the PW (negative skin friction) while the deep seated layers support the PW (positive skin friction). The neutral plane can be found at elevation -20. This result is consistent with the compression pattern described above. The analysis yields also the settlement of PW's toe: 56 mm and PW's head: 70 mm. PW's compression is caused by the load and the drag forces.
The results are deemed realistic. The N-soil model presented here can, therefore, be utilized for investigating the behaviour of a pile group. This task is currently being performed.

3.2.2 The Southern part

The S-soil model for the southern river bank was constructed in the same manner as the N-soil model, but it is less elaborate and has fewer clay layers although the clay deposits are thicker (90m). This simplification diminishes the calibration work load.

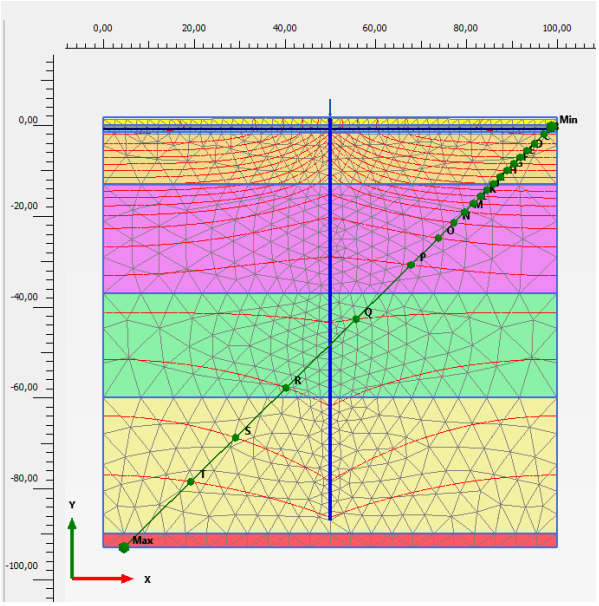


Fig 12 S-soil model for the southern river bank.

The four clay layers in the S-model are denoted from top to bottom CL1 to CL4. Below CL4 is the stiff till layer whose upper boundary is located at elevation -90. On top of CL1 are two fill layers called Fill and F-top. Fill is believed to be deposited in year 1860; it rests now on CL1 at elevation -1.6 and is about 1.6 m thick. F-top was laid upon Fill probably in 1890; it is now about 1.7 m thick. So GS is presently at elevation +1.7. The till and fills are modelled as Mohr-Coulomb materials. Attention on consolidation and creep was given only to the clayey layers; their SSC parameters are collected in Table 3.

Table 3 S-soil model. Relevant SSC parameters

Clay	c'	λ^*	$10^3\,\mu^*$	$10^3\,\kappa^*$	e_i
CL1	2.45	0.1296	4.928	5.188	2.00
CL2	4.50	0.1733	6.242	6.925	1.80
CL3	8.05	0.1854	4.634	7.415	1.58
CL4	11.7	0.1572	3.143	6.286	1.49

The analysis method is the same as for the Northern river bank reported above. It includes several steps: (i) a preparatory consolidation-creep analysis of the glacial and post glacial deposits without any fill over a period of thousands of years to bring them to a state they were supposed to have at the time when the fills were applied; (ii) the application of the first fill in year 1860 followed by a consolidation-creep analysis

until 1890; (iii) the application of the second fill in 1890 followed by a consolidation-creep analysis until 2014 then until 2134; (iv) the installation of a pile row or a pile group in 2014 for studying the effects of creep settlement on it over its life time (120 y).

The results of the steps (i) to (iii) are qualitatively similar to those for the Northern river bank:

- Settlement rate calculated at the upper boundary of the uppermost clay layer CL1: ca 3.2 mm/y during the last 2 decades
- Settlement pattern: about 87% of the settlement until 2014 and beyond 2014 is caused by the compression of the upper 2 clay layers CL1 and CL2 which are post glacial clays (down to elevation -37)
- Calculated pore water pressures: relative difference from the 2014 readings of the piezometers K and L installed at elevation -32 and -48: for K (-32): +3.7%; for L (-48): -0.8%.

The obtained results correlate with field observations. The S-soil model is deemed realistic; it can be utilized for studying the behaviour of structures to be built in the South River bank.

Figure 12 shows the results of such a study with a pile row modelled as a thin concrete pile-wall (PW). PW is loaded on its head with a vertical line load of 600 kN/m; its toe stands 3 m above the stiff till layer. As expected the two upper clay layers settle and drag down PW (negative skin friction) whereas the lower layers support PW (positive skin friction). The neutral plane can be found at about elevation -40. As to the settlement the calculated values are given in Table 4.

Table 4 S-soil model. Settlement after 120 years

PW13	V= 0 kN/m	V= 600 kN/m
Cap	74 mm	102 mm
Toe	58 mm	76 mm

In the case of a non-loaded PW, the toe settlement is less, i.e., in comparison with the loaded pile, the surrounding soil settles more relative to the pile. As a consequence the zone of negative friction becomes longer, and the neutral plane is lowered to elevation ca -45. The same is true in the case where the PW is installed 1 m into the till layer. The toe settlement will be negligible, and the neutral plane will move downwards to elevation -51. These results are logical. They show the applicability of the model in the design of pile foundations.

Figures 13 and 14 exemplify a tentative design of the pile group PG 13 for the pier No. 13 of the future bridge. The pile cap is a 1.5 m thick reinforced concrete plate with the dimension L x W =30 m x 6 m. Parallel to L are four pile rows in equal spacing, and parallel to W seven rows on either side of the middle row which has fewer piles than the other rows. The outer piles are slightly inclined (V: H=7:1), but the inner piles are vertical. The pile rows are modelled as thin vertical concrete plates. The vertical permanent load acting on PG13 amounts to 56 MN.

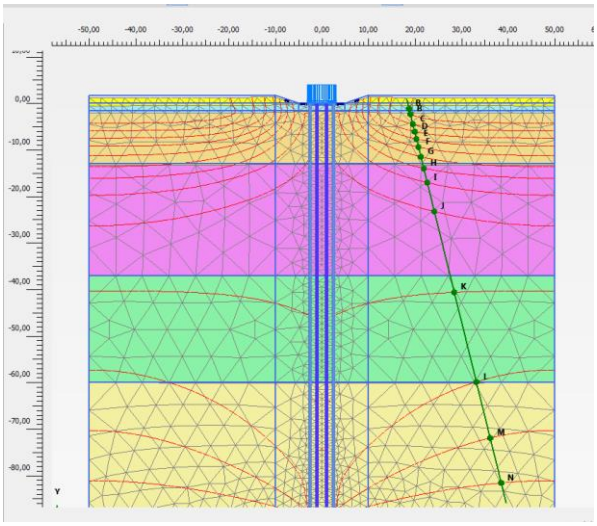


Fig 13 Pile group PG13 in S-soil model.

The pile group PG13 stands 3 m above the bottom till, at elevation -87. It undergoes, over a period of 120 year after installation, a settlement of 156 mm at the toe and 183 mm at the cap. The neutral plane appears approximately at elevation -37 in figure 13.

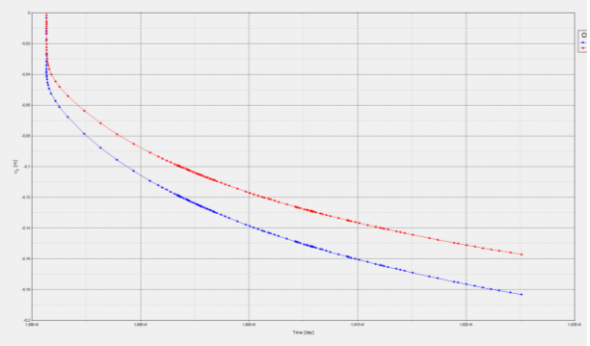


Fig 14 Settlement of PG13 in S-soil model.

It can be seen in figure14 that about 29 % settlement occurs during the first year at the cap: 53 mm. The settlement continues at the rate of about 3.1 mm/y until year 10, then 1.9 mm/y until year 20, and 1.2 mm/y until year 50. With time the settlement rate decreases to less than 1 mm/year.

The above are the results of a 2D analysis of PG13 seen perpendicular to the bridge axis (pile cap cross section 6 m x 1.5 m with 4 pile rows). The real problem is, however, 3D. Another investigation was carried out in the bridge direction with a cap cross section 30 m x 1.5 m. As expected the settlement is more important. The results for both cases are summarized in Table 5.

Table 5 S-soil model. Settlement after 120 years

PW13	**// bridge axis**	**⊥ bridge axis**
Cap	280 mm	183 mm
Toe	259 mm	156 mm
Neutral plane	Elevation -20	Elevation -37

The behaviour in the direction transverse to the bridge axis is believed to better reflect the real situation. This in analogy with the behaviour of a strip footing.

4 DESIGN RECOMMENDATIONS AND CONCLUSIONS

The soil models presented here reflect the site's current behaviour. They can cope with the phenomenon of consolidation and creep settlement as well as their effects on a pile foundation. It was time consuming to construct models, but once done, they constitute a useful tool for studying different pile group designs. Different geometries, loadings, and foundation depths can be readily investigated. The central issue is the settlement including downdrag and the magnitude of drag forces. Parameter studies are being currently conducted to assess the sensitivity of different parameters considering also their uncertainties.

The on-going study focuses on methods to reduce the drag forces by allowing, for example, a certain settlement and/or applying bitumen on the part of the piles which is subjected to negative friction. Also a protective sleeve can be considered. That alternative can, however, cause production problems. Should bitumen be utilized, its effect can be easily simulated using the above soil models by giving the interfaces of the pile portion in question a reduced shear strength.

5 ACKNOWLEDGEMENT

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