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Settlements of spread footing foundations on quick clay stabilized with lime and cement

E S Juvik¹, B K F Bache² and S S Berre³

¹ Norwegian Public Roads Administration, Prinsensgate 1, 7013 Trondheim, Norway

² Norwegian Geotechnical Institute, Høgskoleringen 9, 7034 Trondheim, Norway

³ Multiconsult, Sluppenveien 15, 7037 Trondheim, Norway

E-mail: eivind.juvik@vegvesen.no, bjorn.kristian.fiskvik.bache@ngi.no, stian.berre@multiconsult.no

Abstract. Since the early 70's ground improvement by use of stabilizing agents such as lime and cement has been widely used for solving geotechnical problems; either as a way to improve slope stability, reduce settlements or vibrations, or to reinforce the inside of braced excavations. On a recently constructed motorway project in south of Trondheim, two bridges were founded with spread footings on a medium stiff, quick Norwegian clay stabilized with lime and cement. The post-tensioned concrete bridges are 78 m and 110 m long, with respectively 4 and 5 spans. In this article, evaluation of various foundation solutions is performed based on cost estimates. This showed that the cost of founding the bridges on stabilized ground to be the most viable alternative. Further, the article discusses the calculation methods used to evaluate settlement properties of the foundations. Settlement measurement is performed for about two years after completion of the structures. A comparison of the calculated deformations and measured settlements is conducted in this work. Settlements are measured to be 3-10 mm after one and a half year, except for one abutment that measured 21 mm of vertical displacements. Then the deformations nearly stop for many of the foundations. The measurements show a higher deformation rate in the first six months after casting of the bridge slab, than expected from the calculations. After some time, the measurements coincide well within the upper and lower bounds in the calculations done in the geotechnical design. Possible reasons for these observations are discussed.

1. Background

1.1. About the project

About 10 km south of Trondheim city centre, a new four-lane motorway has recently been constructed. The city of Trondheim is situated in central Norway. The 8.1 km motorway runs from Tiller in Trondheim municipality to Jaktøya in Melhus municipality. In addition, the project consists of 4.6 km of local roads, 6.2 km of pedestrian and cycle paths and 7.2 km of noise reduction mounds and walls. Apart from the roads and constructions, the project includes river diversion and erosion control of the streams Søra and Klasbekken.

The soil conditions along the road is dominated of marine deposits with a broad variety of properties. In the northern part, there is also a glaciofluvial deposit following the road alignment and deep deposits of peat.



About one fourth of the road runs through areas with quick and sensitive clay. This necessitated large efforts to reinforce the ground. In two sections the road cuts down into quick clay, where the stability of the cuttings was secured with single and double panels of lime and cement columns. Two bridges, which are subject to this paper, are founded directly on quick clay stabilized with lime and cement. Soil reinforcement was also used to improve the stability of the roadbed and to secure the water supply and sewage ditches during excavation.

1.2. Ground improvement with lime and cement

Ground improvement of inorganic clays with lime has been used in the Nordic countries since the 1970s. From mid 1980s it became more and more common to use lime and cement as stabilizing agents. [1]

The lime and cement columns are mixed with a whisk. First the whisk is drilled down into the ground to a desired depth. Then the whisk is pulled back up, still rotating, while the binder is pumped into the stirred soil. The binder and the soil react, and during the curing period the strength increases with time.

The method can be used to fulfil many purposes. It can be used as a mean to reduce settlements of embankments, buildings and roadbeds. It can be used to reduce vibrations from traffic. It can be used to reinforce the inside of braced excavations or to ensure the stability of ditches during excavation. Furthermore, it is a suitable method to improve the stability of embankments, cuttings and sometimes natural slopes.

Ground improvement by this method is not only conducted with lime and cement as stabilizing agents. Other types of binders are also in use, e.g. residual products from industry such as fly ash and cement kiln dust.

2. Introduction

In the project E6 Trondheim-Melhus, two overpass bridges are founded directly on reinforced quick and sensitive clay. Klettrø bridge is 78 m long and Leinstrand bridge is 110 m long, with respectively 4 and 5 spans. Both constructions are post-tensioned concrete bridges. All the pillars and abutments rests on shallow spread footing foundations.

The northern abutment (foundation 1 in figure 1) and the two adjacent pillars (foundation 2 and 3) for the Klettrø bridge are located in the area of an old ravine which has been filled with organic and inorganic waste several years ago. An embankment was constructed to elevate the terrain below the northern abutment and the local road. The height is 8 m and it is wide in extension. Calculations indicates that the additional loads from the fill will induce settlements more than 50 m down in the soil layers [2]. The fill has been preloaded with a surcharge of 4 m to reduce the settlements to a minimum after construction. The other two foundations are located close to the surface in original, undisturbed deposits.

The Leinstrand bridge are founded in original, undisturbed deposits, 7-13 m below the original surface.

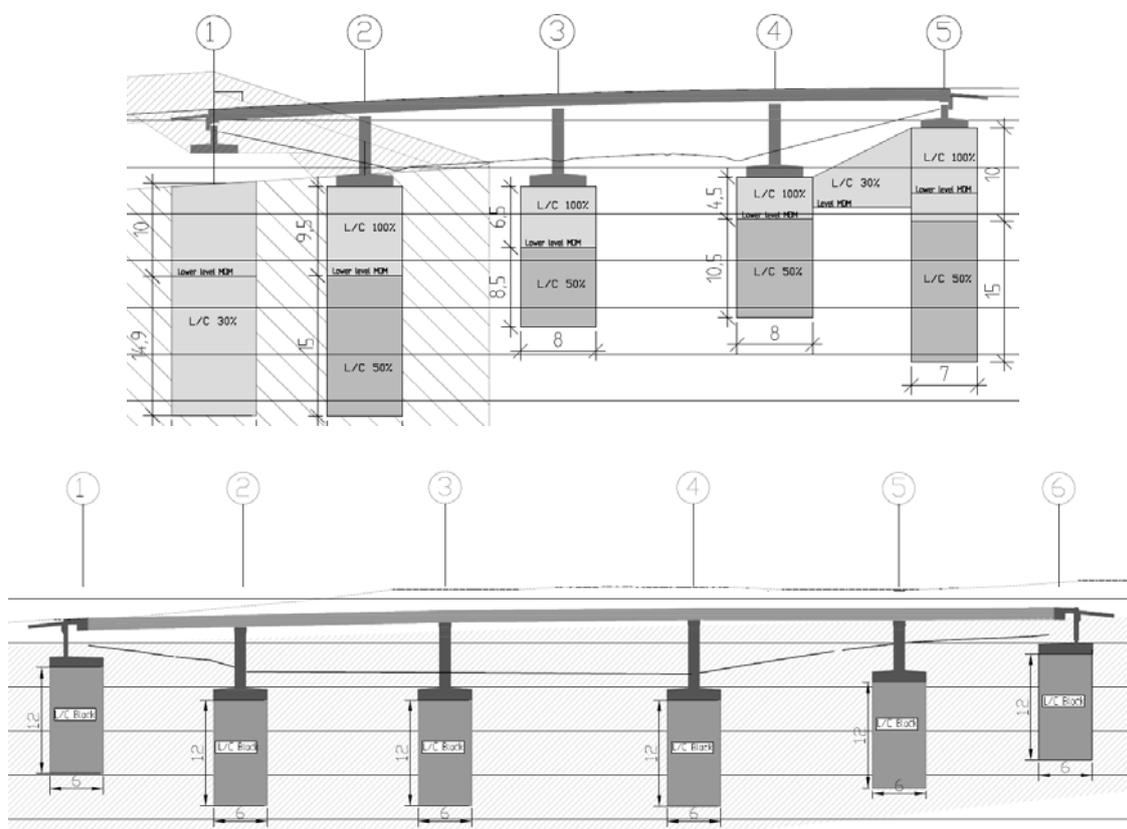


Figure 1. Construction plans of Klettør bridge (above) and Leinstrand bridge (below).

3. Soil properties

The distance between the two bridges is only 250 m. They are founded in the same marine deposits. However, there have been discussions whether the sediments are natural deposits, or if they are avalanche debris from large scale quick clay slides thousands of years ago [3]. A description of the soil conditions is given in this section.

3.1. Klettør bridge

As mentioned in the introduction, Klettør bridge is located in an old ravine which has been filled. The thickness of waste deposits below the foundations varies between 0 and 9 m. Below there is approximately 25 m of quick silty clay. Further down there is non-sensitive silty clay to unknown depth.

The ground water table is assumed to be 2 m below soil surface before construction. After construction the ground water table adjusts to the lower edge of the foundations. The pore pressure increases with 9.2 kPa/m down to 50 m depth. From 50 m it is assumed an increase of 10.0 kPa/m.

3.2. Leinstrand bridge

At Leinstrand bridge the thickness of the dry crust is approximately 2-4 m. Below there is a 20-25 m thick layer of sensitive and quick silty clay. Further down there is non-sensitive silty clay to unknown depth.

The assumptions for ground water table and pore pressure distribution are the same as for Klettør bridge, with the exception that the pore pressure increase is 9.0 kPa/m down to 50 m depth.

3.3. Summary

Table 1 summarizes the material properties of the original marine deposits. A thorough description of the soil properties, including the dry crust, fill material and stabilized clay, are given in [2] and [4].

Table 1. Soil properties of sensitive and quick silty clays. Additional properties and properties of dry crust, fill material and stabilized clay are given in [2] and [4].

	Klettrø bridge	Leinstrand bridge
w (%)	32.0	32.0
PI (%) ⁴	7.0	7.0
γ (kN/m ³)	19.5	19.5
OCR (-)	7.0-1.3	8.0-1.3
$c_{u,a}$ (kPa)	$45 + 0,70 \times OCR^{0.27} \times \sigma'_{v0}$	$40 + 0,70 \times OCR^{0.27} \times \sigma'_{v0}$
c_{uD}/c_{uA} (-)	0.63	0.63
c_{uD}/c_{uA} (-)	0.35	0.35
a (kPa)	2.0	2.0
φ (°)	31.0	31.0
k_0 (m/year)	0,063	0,032
M_0 (MPa)	$3,3 \times 13 \times p'_c$	$3,3 \times 13 \times p'_c$
m	23	23
r_s (-)	510	-
Clay content (%)	~30	30-35

⁴Plasticity index = 14.0 in non-sensitive silty clay

Where w (%) is water content, PI (%) is plasticity index, γ (kN/m³) is density of soil, OCR (-) is overconsolidation ratio, $c_{u,a}$ (kPa) is characteristic active undrained shear strength, c_{uD}/c_{uA} (-) is the ratio between direct/passive and active shear strength, a (kPa) is attraction, φ (°) is friction angle, k_0 (m/year) is permeability, M_0 (MPa) is oedometer modulus in the overconsolidated area, m (-) is modul number and r_s (-) is time resistance number.

4. Evaluation of foundations alternatives

Based on nearby reflection seismology in the area, the depth to bedrock is estimated to be several hundred meters. Therefore, a foundation with piles to bedrock is not an option. Two solutions were considered in the design phase; 1) foundations with open steel pipes and 2) foundations on reinforced quick clay.

A cost analysis was made between the two alternatives for Klettrø bridge. This showed that a foundation with steel pipes was almost 3 times more expensive than a foundation on reinforced soil. [7]

The final design of the reinforced blocks was a bit more extensive than assumed in the cost estimate. Non the less, the method with foundations on reinforced soil showed significantly cheaper than steel pipes.

Ground improvement was used in a large extent in this project, and the machinery was already needed on the construction site. The reliability of the solution was also considered to be high. Therefore, ground improvement was selected as a viable solution and details are given in the next section.

5. Ground improvement

This section summarizes the ground improvement methods that was used for the foundations of the bridges.

5.1. Klettrø bridge

The ground improvement under the foundations of the Klettrø bridge was carried out with two different patterns and methods. The upper 5-10 m of the blocks is located partially in the old landfill which has relatively low water content of approximately 20 %. This was stabilized with the Modified Dry Mixing (MDM) method [5]. Water is injected into the soil together with the binder, which is 100 % cement in this case. The coverage is 100 % for the upper part, hence the entire soil volume is stabilized. 100 kg/m³ cement was used.

Regular dry mixing was conducted in the lower part of the blocks. The binder consisted of 50 % lime and 50 % cement. The amount of binder was 90 kg/m³. The coverage is 30 % in foundation 1 and 50 % in the rest. Total length of the reinforced blocks is 25 m in foundation 1, 2 and 5, and 15 m in foundation 3 and 4.

5.2. Leinstrand bridge

The reinforced blocks below the foundations of Leinstrand bridge was stabilized with 50 % lime and 50 % cement. The amount of binder for all foundations was 90 kg/m³, and regular dry mixing was used. The coverage is 75 %, and the length of the blocks are 11,5 m.

6. Calculation methods

There are some uncertainties related to how the settlements will develop with time. As a basis for settlement predictions, two different approaches were used for each bridge, giving an upper and lower bound for the estimated settlements.

All settlement calculations were cut off at a depth of 50 meters. At this level the additional stress from the foundations will be small and be spread over a large area. Strains at this level will not result in differential settlements on the bridge, which was the design criteria for the bridges.

6.1. Klettrø bridge

For Klettrø bridge, it was assumed that the construction of the embankment, including the preload, would take 4 months. Then, the consolidation phase was assumed to last for 12 months, followed by 4 months of unloading and 4 months of construction time for the bridge. [2] It is important to note that there are vertical drains under the embankment. In reality, the construction time for the embankment was 7 months, and the consolidation phase was reduced to 8 months. Unloading was done within a month, and construction time for the bridge was about 4 months. This gives a total construction time of 20 months, whereas 24 months was assumed in design. This will influence how the settlements develop with time after construction.

6.1.1. Model 1. In this model, primary settlements are calculated with Janbus soil model in the program GeoSuite Settlement. Creep deformations are manually calculated with Janbus concept of time resistance numbers. [6]

The concept underestimates deformations during the consolidation phase, as it does not account for creep. This means that deformations during construction can be larger than calculated. Model 1 are considered to be an upper limit for expected deformations in the long term. The assumptions regarding vertical drains and consolidation are conservative, which means that the calculated consolidation time is a high estimate. [6]

6.1.2. Model 2. In this model, settlements in foundation 1 and 2 are calculated with the Krykon soil model in the program GeoSuite Settlement. The Krykon soil model represents a more realistic development of settlements with time, as it calculates creep deformations during the consolidation phase. However, the calculations are sensitive to the input parameters. Small changes in a single parameter can produce large variations in calculated creep deformations. The model is also sensitive to model depth and the preconsolidation stresses. [6]

Foundation 3, 4 and 5 are calculated with Janbus soil model for primary settlements in the program GeoSuite Settlement. [6] Creep deformations are not included for these foundations, as this assumption gives the largest differential settlements between foundations.

This combination of Krykon model in foundation 1 and 2, and Janbus model in foundation 3-5 represents an unfavourable situation for the bridge. [6] After consolidation there will be no more deformations in foundation 3-5, while foundation 1 and 2 will continue to settle due to creep deformations. This creates increased differential settlements between the foundations, which will induce increased constraining forces in the construction.

6.2. *Leinstrand bridge*

The foundations for Leinstrand bridge are located 7-13 m below the original surface. It was assumed that the excavation phase lasts for 3 months, and then another 3 months of construction time for the bridge. Settlements are calculated with Janbus soil model in the program GeoSuite Settlement. Creep deformations are not calculated, as the loads from the bridge are smaller than the unloading caused by excavation. [4]

6.2.1. Model 3. The excavation will lead to swelling in the underlaying clay. This will again induce a negative pore pressure, or suction in the clay. It will take time before the negative pore pressure dissipates and the swelling stops, most likely more than 3 months.

Model 3 takes this effect into account. The negative pore pressure is introduced in the calculations. When the load from the construction is applied, it will result in excess pore pressure at the top of the model and suction at the bottom. Both will dissipate over time until the pore pressure has stabilized. In this period, the upper part of the model will compress, and the lower part will continue to heave. [4]

There are uncertainties regarding the depth where the excess pore pressure is zero. The transition depth will vary over time and will affect the calculated deformations. The deformation parameters for swelling and compression are also uncertain. Especially the swelling parameter could be too low in the calculations. If the parameter is in reality higher, the actual deformations will be larger than calculated in model 3. [4]

6.2.2. Model 4. In this model it is assumed that the swelling is finished, and the negative pore pressure has been neutralized before the loads from the construction is applied. [4] This implies a compression in the entire soil volume during loading.

[4] indicates that it will take more than 2 years before the negative pore pressure has dissipated. This means that model 4 represents a conservative assumption of the settlements, and the real settlements should not exceed these values.

There will most likely still be negative pore pressures in the ground when the construction is finished. Total settlements after construction and the deformation rate will depend on the magnitude of this.

7. Settlement measurements

Vertical deformations have been measured on all foundations on both bridges since before casting of the deck. The results are presented in this section.

7.1. *Bolts and levels*

Bolts were drilled into the abutments and pillars during the construction period. They were measured with digital levels. The starting point of the levelling was an old, solid house close to the construction site. Ideally, one should have used a reference mark on bedrock as starting point, but it is too far to the nearest bedrock in this area. The house is therefore assumed to be the best reference, although it is most likely subject to settlements due to creep. The accuracy of the measurements with levels are 2-4 mm.

7.2. Prisms and total station

The bolts were replaced with prisms after the opening of the motorway, because the traffic made it impossible to use levels. The prisms are measured with a total station. The total station uses known reference points nearby to calibrate the position and height. Then it measures the absolute height on the prisms. The accuracy of the measurements with total station is 2-4 mm.

7.3. Results

Results from the measurements are plotted in the following graphs, together with calculated settlements discussed in the previous section.

Measurements from Klettør bridge is plotted in figure 2 to 6. Measurements from Leinstrand bridge is plotted in figure 7 to 10. The zero point at the timeline is the day the deck formwork was removed, and the loads were fully transferred to the foundations.

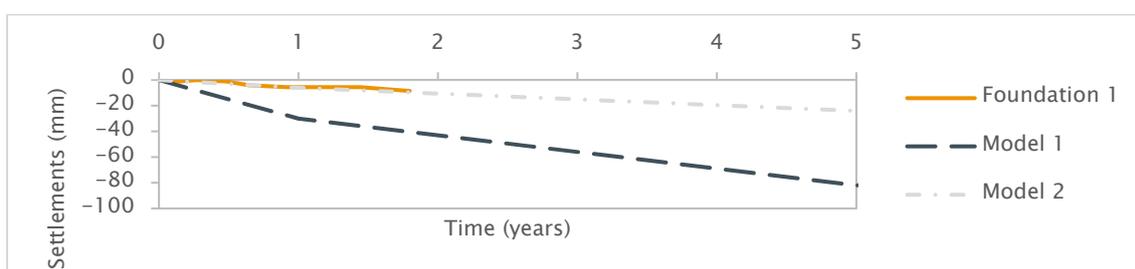


Figure 2. Klettør bridge. Measured and calculated settlements for foundation 1.

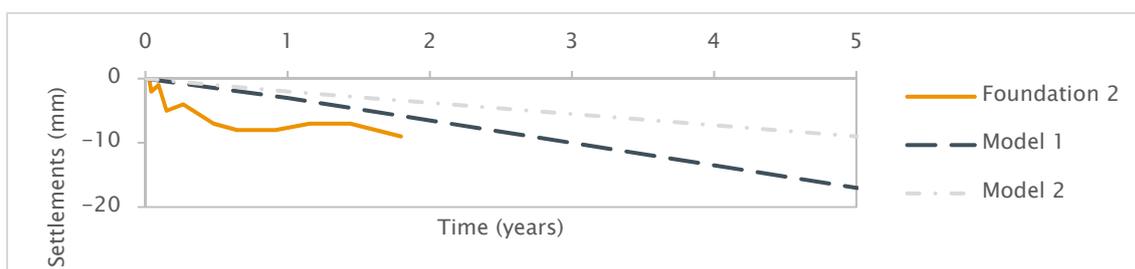


Figure 3. Klettør bridge. Measured and calculated settlements for foundation 2.

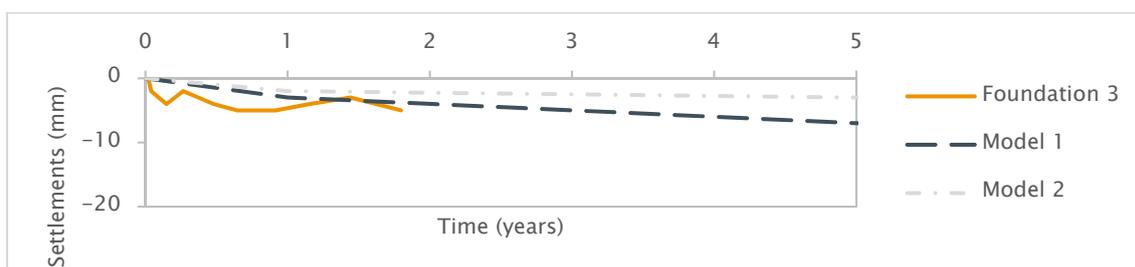


Figure 4. Klettør bridge. Measured and calculated settlements for foundation 3.

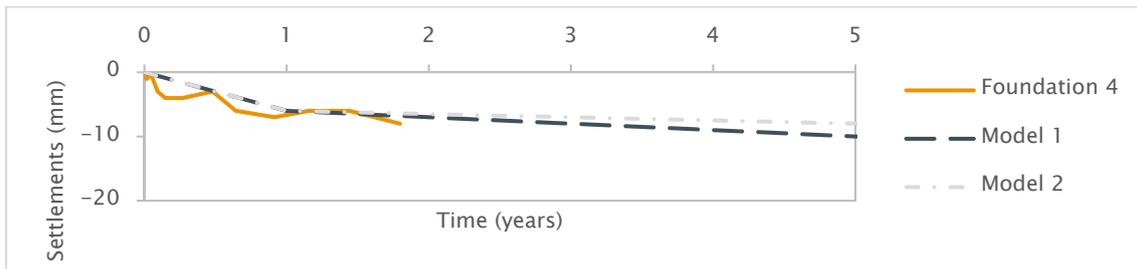


Figure 5. Klettør bridge. Measured and calculated settlements for foundation 4.

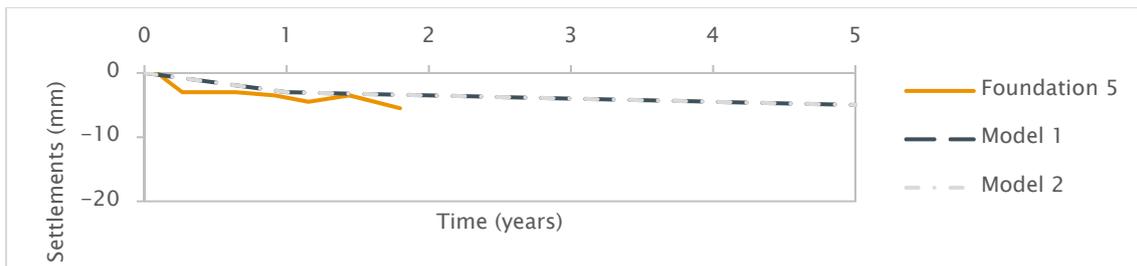


Figure 6. Klettør bridge. Measured and calculated settlements for foundation 5.

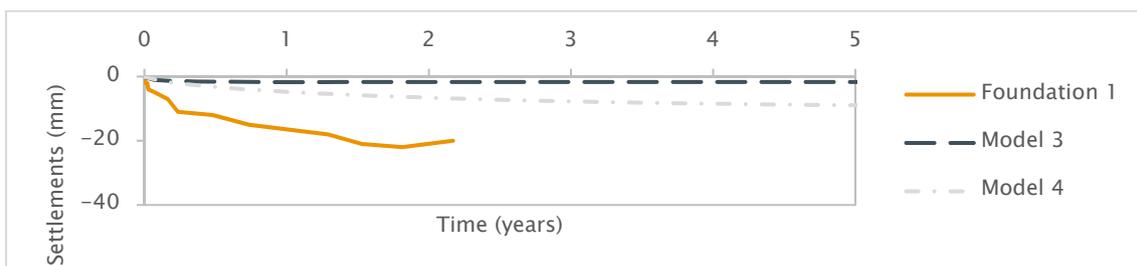


Figure 7. Leinstrand bridge. Measured and calculated settlements for foundation 1.

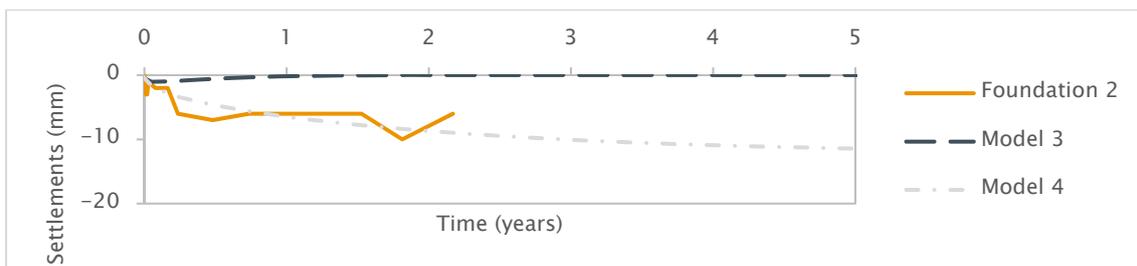


Figure 8. Leinstrand bridge. Measured and calculated settlements for foundation 2.

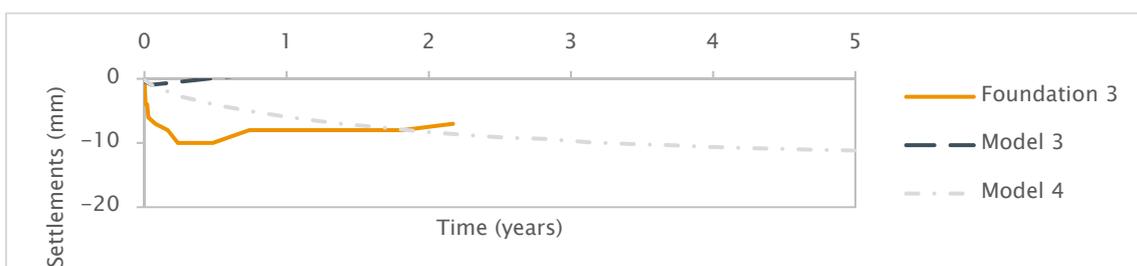


Figure 9. Leinstrand bridge. Measured and calculated settlements for foundation 3.

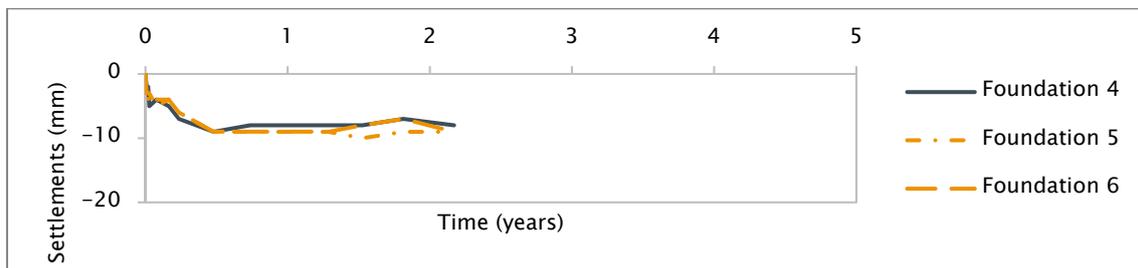


Figure 10. Leinstrand bridge. Measured settlements for foundation 4, 5 and 6.

8. Comparison of calculated and measured deformations

A comparison between the calculated and measured deformations is made in this section.

8.1. Klettro bridge

The loading and deformation situation in and around foundation 1, and partially 2, is complicated. Vertical drains under the embankment and soil reinforcement below the foundations affects the way the soil reacts to loading. The duration of the construction of the embankment, consolidation phase and unloading phase affects how the settlements develop with time. Furthermore, the duration of the construction of the bridge also affect the development.

Figure 2 indicates that *foundation 1* behaves similar to the assumptions in model 2. This calculation was done with the Krykon model, that includes creep deformations during the consolidation phase. Both the total amount of settlements after construction and the rate of creep deformations are similar. This confirms the assumption that the manually calculated creep deformations in model 1 overestimates the settlements.

The deformations in *foundation 2* are larger than both models to this point (figure 3). The deformation rate the first year has been faster than calculated, but it looks like the rate has slowed down significantly. Within 1-2 years it might be possible to see if also foundation 2 approaches the behaviour of model 2. As mentioned in section 6.1, the real construction time was 4 months shorter than assumed in the calculations. This could be a reason why the deformation rate is faster the first year. Foundation 1 does not show this behaviour as one should expect, as both of them were subject to preloading. Then again, the preloading in foundation 1 was greater than in foundation 2, and that could explain the difference.

The behaviour of *foundation 3* (figure 4) is quite similar to foundation 2. The deformations are touching the line of model 1 and might draw near model 2. In this foundation, model 2 only calculates primary settlements, and it can be expected that the long-term deformations will exceed the calculated values.

The measurements in *foundation 4 and 5* coincide with both model 1 and 2 (figure 5 and 6). Both show the same deformation rate in the first years after construction.

8.2. Leinstrand bridge

To make a precise prediction on the vertical deformations for Leinstrand bridge, it is essential to reproduce the pore pressure distribution in the calculations. After construction of the bridge, there will be excess pore pressure in the upper layers and suction in the lower layers because of the excavation. Calculated settlements will depend on how much the negative pore pressure has dissipated.

The assumption for model 4 was that, before applying the load from the bridge, the negative pore pressure had been neutralized and the swelling had finished. Calculations indicates that it would take several years until this happens. However, the measurements in *foundation 2 and 3* (figure 8 and 9) is close to the behaviour of model 4, even though it only took about 7 months from unloading until the construction was finished.

Leinstrand bridge is situated in a cutting in quick clay. The whole area around the bridge is stabilized with lime and cement columns. There are panels in the cuttings and grids in the roadbed. There is a drainage effect alongside the columns since they have higher permeability than the surrounding soil. This could be a reason why the pore pressure distribution has normalized in such a short time.

The settlements in *foundation 1* is now about 3 times larger than what was expected as maximum in the geotechnical design (figure 7). One reason for this could be the terrain adjustment around the abutment, where the extra load from the fill is causing the additional settlements. Hence, the extra load might not have been considered in the calculations. Another reason could be the fact that foundation 1 is located at the edge of the cutting, which could cause difficulties reproducing the pore pressure distribution or the in-situ stress distribution in the calculations.

9. Conclusions

The construction of bridges in sensitive and quick clay can be challenging, especially when it comes to fulfilling requirements on vertical deformations. One common way to solve this is a foundation with piles into the bedrock. In the project E6 Trondheim-Melhus, the marine sediments are probably more than hundred meters thick and piling to bedrock is not an option. Founding the bridge on open end steel piles was considered. Based on a cost estimate the Kletttrø bridge and Leinstrand bridge was founded on blocks of reinforced quick clay.

Settlement measurements on the foundations after completion of the bridge, shows that the method is reliable and viable. In general, the measurements coincide well with the upper and lower bounds in the calculations done in the geotechnical design.

Cost estimates has shown that the method is significantly cheaper than foundations on open steel pipes, especially when ground improvement is needed to fulfil other purposes on the construction site.

There are many uncertainties, both in material properties and the execution of the construction work, and it can be difficult to predict which models that best represent the reality. However, the measurements on the two bridges will continue for years. They will give more insight on how the deformations and creep settlements will develop with time.

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