Geotechnical Safety and Risk V
T. Schweckendiek et al. (Eds.)

doi:10.3233/978-1-61499-580-7-628

Evaluation Settlement Models Test Embankments Bloemendalerpolder – Geo-Impuls Program

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Abstract. In the Bloemendalerpolder two test embankments were constructed, starting October 2010, to study the long term behavior of embankments on very soft soils with respect to settlement and lateral pile loading resulting from horizontal soil deformations. A detailed description of the test embankments, performed soil investigation, laboratory tests and instrumentation is given in an accompanying paper. This paper provides a brief description of both test embankments consisting of sand fill with a height of 3.0 m and a plan area 26 x 36 m^2 with slopes 1:2. At the embankment overlying 5.7 m of clay/peat, wick drains are installed at a triangular spacing of 1.0 m. The second embankment overlying 4.0 m of clay/peat is not provided with any additional drainage accelerating measures. From the laboratory tests settlement parameters were derived for three settlement models; Koppejan-Terzaghi-Buisman, NEN-Bjerrum isotache model and the a,b,c-isotache model. Additionally parameters for Terzaghi and Darcy consolidation models were derived as well: consolidation coefficient, permeability and strain dependent permeability strain factor. Based on applied field loading increments predictions were made. Test results of both embankments will be shown including predictions. Because of the limited dimensions of the test embankments a correction of the monitoring results is applied to match one-dimensional conditions. Based on the measurements a fit of input parameters for all models have been generated matching the settlement of both embankments. Based on these results recommendations are given for application of the models for the site preparation in the Bloemendalerpolder. Researchers, however, are invited to analyse the data to develop more general settlement models for such very soft soils.

Keywords. Evaluation Settlement models Test Embankments Bloemendalerpolder – GeoImpuls program

1. Introduction

Settlement of embankments for urban development or road construction on soft soils is a well-known issue. Common applied settlement models however give rise to a wide range of predicted settlements. Especially long-term behavior of very soft clays and peats is not easy to predict. A conservative approach on settlement predictions may increase direct construction costs whereas an optimistic approach may result in long term unexpected deformations. Validation of settlement models, in general, suffers from a lack of well-described field test data. There certainly is a quest to mitigate risks associated with settlement predictions.

For an urban development in the Bloemdalerpolder in the Netherlands field tests were designed to study the settlement behavior for a year. The GeoImpuls program took the opportunity to extend the field test to a five-year monitoring period as well as to include test piles for monitoring their behavior with respect to horizontal soil deformations.

A 4 to 6 m thick very soft peat layer underlies the existing grasslands at the Bloemendalerpolder with a groundwater table at 0.3 m below ground level. Urban development requires 0.5 to 1.0 m clearance of the ground level above the groundwater table. Two test embankments consisting of sand fill were designed with a height of 3.0 m and a ground area 26 x 36 m^2 with slopes 1:2. The GeoImpuls program aimed to provide a well-described field test with long time monitoring results.

2. Stratification

The soil investigation shows a constant thickness of the peat layer for each embankment but a relatively large difference between both embankments; details are given in Hoefsloot (2015a). A schematic stratification is given in Table 1 and Table 2. The groundwater table lies at approximately NAP -2.1 m.

Table 1. Soil stratification at Embankment 1.

Top of Layer m+NAP	Soil Description
-1.7	CLAY slightly organic, unsaturated
-2.0	PEAT soft
-5.7	SAND loose to medium dense
-10.5	Maximum exploration depth

Table 2. Soil stratification at Embankment 2.

Top of Layer m+NAP	Soil Description
-1.8	CLAY slightly organic, unsaturated
-2.0	PEAT soft
-7.5	SAND loose to medium dense
-11.0	Maximum exploration depth

3. Settlement parameters

In total 5 Incremental Loading (IL) Consolidation tests and 10 K_0-CRS tests have been performed. Based on these results settlement parameters have been derived for three different settlement models; Koppejan-Terzaghi-Buisman, NEN-Bjerrum isotache and a,b,c-isotache model . The selected settlement parameters are given in Table 3, 4 and 5.

Table 3. Settlement parameters Koppejan-Terzaghi-Buisman.

Soil	C_p	C'_p	C_s	C'_s	POP[1)] kPa
Clay	28	7	320	80	7
Peat	10	6	102	102	7
Sand	∞	∞	∞	∞	7

1) POP = pre-overburden pressure

Table 4. Settlement parameters NEN-Bjerrum isotache.

Soil	RR	CR	C_α	POP kPa
Clay	0.10	0.31	0.014	7
Peat	0.061	0.493	0.020	7
Sand	$1{\cdot}10^{-6}$	$2{\cdot}10^{-6}$	$1{\cdot}10^{-6}$	7

Table 5. Settlement parameters a,b,c Isotache.

Soil	a	b	c	POP kPa
Clay	0.013	0.16	0.008	7
Peat	0.04	0.327	0.014	7
Sand	$1{\cdot}10^{-6}$	$2{\cdot}10^{-6}$	$1{\cdot}10^{-6}$	7

General soil parameters are given in Table 6.

Table 6. General Soil Parameters.

Soil	γ kN/m^3	γ_{sat} kN/m^3	e_0
Clay	14.0	14.0	-
Peat	-	10.3	15.6
Sand	-	20.0	-
Sand Fill	17.0	19.0	-

4. Consolidation parameters

Dissipation of pore water pressure is described by the consolidation process. Parameters for three different consolidation models were derived:

- Terzaghi model with constant consolidation coefficient
- Darcy model with constant consolidation coefficient
- Darcy model with strain dependent permeability

For the first two models the consolidation coefficient for the peat layer has been determined for loading steps 3, 4 and 5 of the incremental loading tests, all above the pre-consolidation stress. Figure 1 presents the results for five samples. Since the load increment of the fill matches the load increment from stage 2 to stage 4 in the incremental loading test, the average consolidation coefficient at stage 3 has been selected for predictions; $c_v = 1.0 \cdot 10^{-7}$ m^2/s.

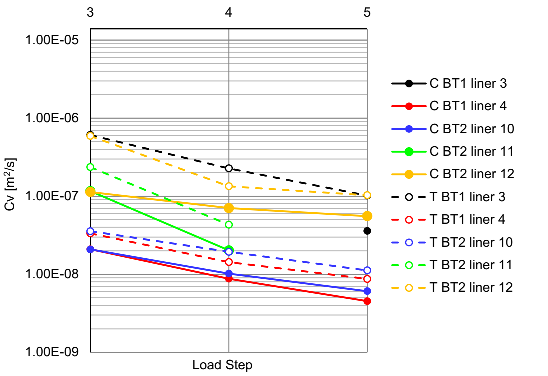

Figure 1. Consolidation coefficient determined with Casagrande (C) and Taylor (T).

The strain dependent permeability is described with equation (1) where k is the momentary permeability, k_0 is the permeability before straining and C_k is the permeability strain factor. Figure 2 and 3 show typical test results.

$$k = k_0 10^{-\frac{1+e_0}{C_k}\varepsilon} = k_0 10^{\frac{e-e_0}{C_k}} \quad (1)$$

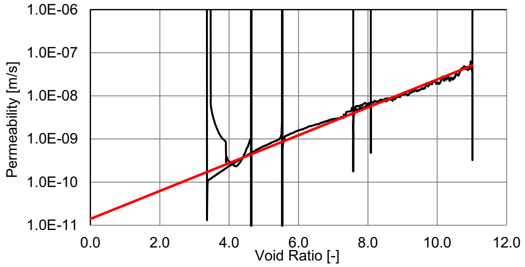

Figure 2. Permeability versus void ratio, K_0-CRS test.

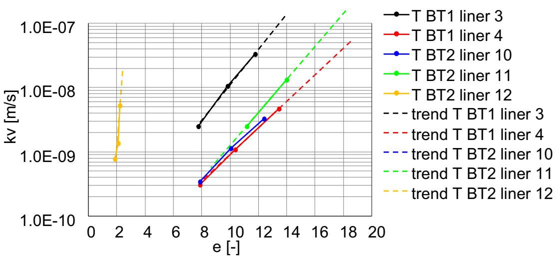

Figure 3. Permeability (Taylor) - void ratio IL tests.

A summary of interpreted test results is given in Table 7. The selected strain dependent permeability parameters for the peat layer are: $k_0 = 5.0 \cdot 10^{-8}$ m/s and $C_k/(1+e_0) = 0.25$.

Table 7. Results strain dependent permeability peat layer.

Test Type	Sample ID	e_0 -	k_0 m/s	$C_k/(1+e_0)$ -
IL	T BT1 liner 3	13.9	$1.3 \cdot 10^{-7}$	0.242
IL	T BT1 liner 4	18.5	$5.3 \cdot 10^{-8}$	0.245
IL	T BT2 liner 10	16.2	$2.0 \cdot 10^{-8}$	0.277
IL	T BT2 liner 11	18.2	$1.6 \cdot 10^{-7}$	0.203
K_0-CSR	BT1 2B	11.2	$5.1 \cdot 10^{-8}$	0.253

Embankment 2 is equipped with Wick drains at a spacing of 1.0 m in a triangular grid. For the ratio c_h/c_v a value of 2.0 has been selected. For the consolidation coefficient of the thin upper clay layer a value of $7.9 \cdot 10^{-8}$ m^2/s has been selected based on IL test results.

5. Prediction

Predictions were made with three settlement models which are often applied in The Netherlands; the traditional Dutch Koppejan-Terzaghi-Buisman model, NEN-Bjerrum isotache and a,b,c-isotache model using Deltares DSettlement software. Detailed descriptions of the settlement models are given in Deltares (2014). Within each model three consolidation models according to section 4 have been applied resulting in nine different predictions.

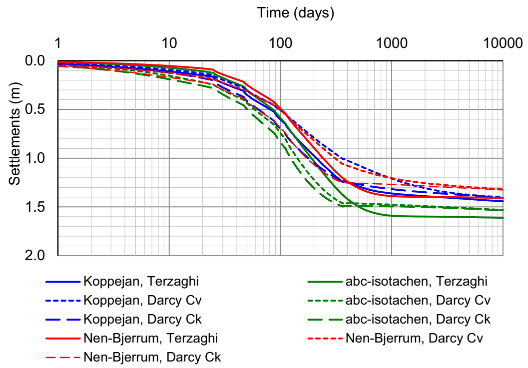

Figure 4. Settlement prediction Embankment 1 including removal preload 0.5 m after 365 days.

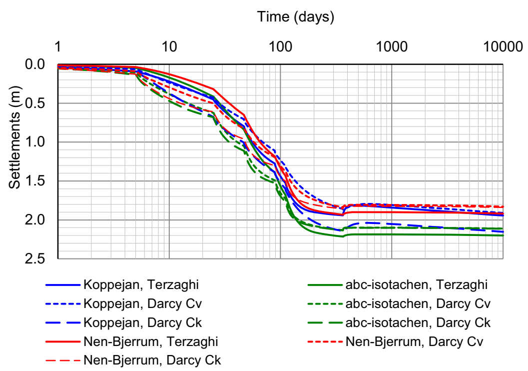

Figure 5. Settlement prediction Embankment 2 including removal preload 0.5 m after 365 days.

The settlement prediction of Embankment 2 in Figure 5 clearly shows faster consolidation than Embankment 1 in Figure 4 despite the larger thickness of the peat layer as a result of applied Wick drains.

6. Settlement monitoring Results

In Figure 6 the settlement of all gauges of both embankments is given in time as well as the fill height. The description of the construction stages is given in Hoefsloot (2015a). It is clear that the total settlement of Embankment 2 is much larger than for Embankment 1 as a result of the difference in thickness of the peat layer.

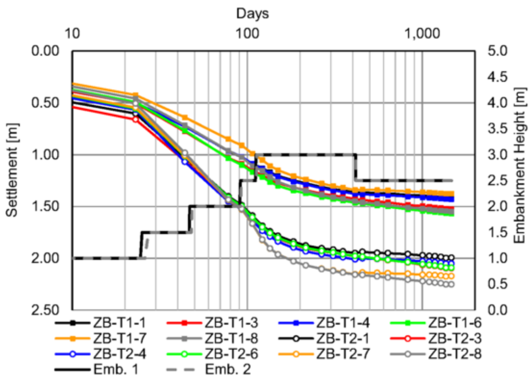

Figure 6. Settlement and fill height versus log time

In order to compare both embankments the settlement at the gauges in the center of the embankments is converted to vertical strain. The results are given in Figure 7. At day 416 a preload of 0.5 m fill, located at gauge T1-7 and T2-7, has been removed over half the top areas of both embankments.

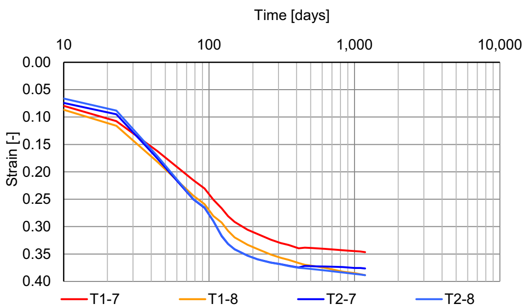

Figure 7. Strain results center gauges embankment 1 and 2

7. Selection Evaluation Data

The horizontal dimensions of both embankments are relatively small with respect to the thickness of the peat layer and therefore there might be a non one-dimensional situation at all gauges. The inclinometers and settlement tubes give rise to this hypothesis and even suggest that this is at least the case at Embankment 2 with a 5.7 m thickness of soft layers. For the evaluation of the settlement parameters the settlement data of Embankment 2 have been multiplied with a factor 0.90 to compensate for horizontal deformations. Additionally, in order to compare the results of gauge T1-7 and T1-8 of Embankment 1 with respect to removal of the preload, the settlement data of gauge T1-7 have been multiplied with a factor 1.08.

8. Postdiction

An extensive evaluation of settlement models and consolidation models haven been performed. Parameters, of the peat layer only, have been varied in order to arrive at a consistent set of parameters meeting the settlement data. Although not in all cases a perfect fit was reached, the best set of parameters is given in Tables 8, 9 and 10. The deviating figures from Tables 3, 4 and 5 are given in bold italics.

Table 8. Settlement parameters Koppejan-Terzaghi-Buisman.

Soil	C_p	C'_p	C_s	C'_s	POP kPa
Clay	28	7	320	80	7
Peat	***36***	***6***	***276***	***46***	7
Sand	∞	∞	∞	∞	7

Table 9. Settlement parameters NEN-Bjerrum isotache.

Soil	RR	CR	C_α	POP kPa
Clay	0.10	0.31	0.014	7
Peat	0.061	***0.45***	***0.042***	7
Sand	$1\cdot10^{-6}$	$2\cdot10^{-6}$	$1\cdot10^{-6}$	7

Table 10. Settlement parameters a,b,c Isotache.

Soil	a	b	c	POP kPa
Clay	0.013	0.16	0.008	7
Peat	0.04	***0.22***	***0.029***	7
Sand	$1\cdot10^{-6}$	$2\cdot10^{-6}$	$1\cdot10^{-6}$	7

With respect to the consolidation parameters the best fit is found for both the Terzaghi and Darcy model with constant consolidation coefficient c_v = 5.0 $\cdot 10^{-7}$ m^2/s. For the strain dependent permeability model the best-fit parameters are different for both embankments; without drains: $k_0 = 1.0 \cdot 10^{-6}$ m/s and $C_k/(1+e_0)$ = 0.11 and with drains: $k_0 = 1.0 \cdot 10^{-7}$ m/s and $C_k/(1+e_0)$ = 0.25. The best fit for the ratio of c_h/c_v applicable to the Wick drains is 1.0.

Postdictions have been performed with Deltares DSettlement software for all three settlement models, each with three consolidation models. Figure 8 and 9 give examples for the postdiction of both embankments with the NEN-Bjerrum isotache model with strain dependent permeability. In both figures Vert 7 and Vert 8 apply to the calculation results at settlement gauge 7 and 8 respectively.

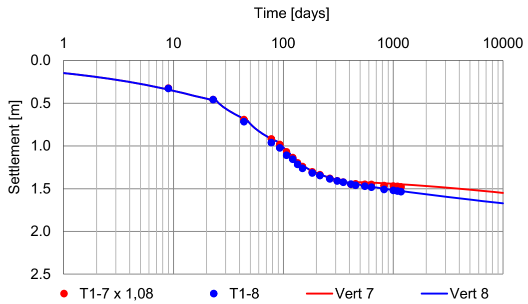

Figure 8. Postdiction NEN-Bjerrum isotache model with strain dependent permeability, Embankment 1

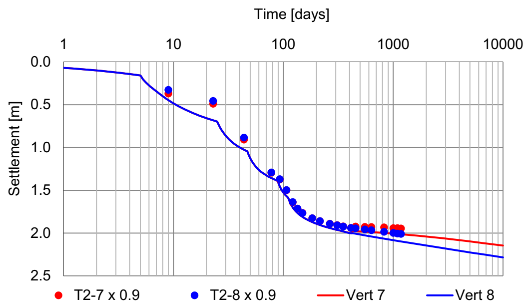

Figure 9. Postdiction NEN-Bjerrum isotache model with strain dependent permeability, Embankment 2

9. Conclusions

At Embankment 1, without Wick drains, the consolidation process is best described with the Darcy model with strain dependent permeability. The strain dependency parameter $C_k/(1+e_0)$ = 0.11 deviating from the laboratory test results (0.25). The Terzaghi and Darcy models with constant consolidation coefficient behave poor as a result of decreasing permeability with vertical strain. At Embankment 2, with Wick drains, the strain dependent model and Terzaghi constant consolidation coefficient model behave both satisfactory. The Darcy, constant consolidation coefficient model behaves poor. Application of a constant consolidation coefficient for a staged fill is allowed, though conservative, when the consolidation coefficient applies to the effective stress at the final loading stage.

The best fit for the primary compression parameter is in the NEN-Bjerrum isotache model 10% smaller and for the a,b,c isotache model 30% smaller than the laboratory test results.

In all models creep is underestimated on application of the parameters resulting from the laboratory tests. The best fit for the creep parameter results in approximately a 2.1 times larger creep contribution; the prediction parameters are given in parentheses:

- Koppejan-Terzaghi-Buisman: C'_s = 46 (102)
- NEN-Bjerrum isotache: C_α = 0.042 (0.020)
- A,b,c isotache: c = 0.029 (0.014).

Possibly the additional creep is a result of gas development in the peat layer resulting in additional time dependent settlement.

10. Recommendations

For the site preparation of the Bloemendalerpolder it is recommended to take the additional creep settlement resulting from this field test into account.

The GeoImpulse Program aimed for long term monitoring excluding detailed interpretation of results. Therefore researchers are invited to analyse the data and try to find explanations for the somewhat unexpected results and come to improved model descriptions for settlement of these very soft soils. Important questions are:

- Do two- or three-dimensional effects play an important role?
- How can the large creep with respect to laboratory test results better be described?

11. Acknowledgement

This research project has been funded by Project organization Bloemendalerpolder and the GeoImpuls Program. Geo-Impuls is a five year long, joint industry program which aims at reducing geotechnical failure substantially in 2015. The author wishes to thank contributing partners in the project Maurik Groot-Ammers as well as Deltares.

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