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# Influence of the initial soil stress state on the cone factor

Reducing the uncertainties in the design of dikes on peat and clay





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## Reducing the uncertainties in the design of dikes on peat and clay

By

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## Preface

*“He is like a man who, in building his house, dug deep and laid the foundation on rock. The river flooded over and hit that house but could not shake it, because it was well built.”*

Luke 6:48 (GNB, 1992)

Last year I worked on the concluding part of my study at the TU Delft. The result is a master thesis report that shows the technical process and results of my graduation research. It does not show the personal growth and difficult times during this thesis, of which I learnt maybe even more than the technical issues. I believe that looking at everything I learnt and experienced during my study time, at the TU, at my student society (VGSD) and in my personal life, I can look back with satisfaction and gratitude. After roughly seven and a half year at the TU Delft I am ready to finish this chapter of my life and start with the next one.

I would like to thank several people for their support during my thesis. Erik, thank you for being available every week for all my questions and problems, technical, organisational and personal. Cor, many thanks for all the times you were able to meet me and provide knowledge and data from practice. Cristina, Bert and Arny, thank you for your constructive criticism and helping me to keep the right focus during my thesis.

Further I would like to thank my parents for supporting me during my thesis, but also during my whole study. Whenever I was in doubt about important choices I always had (and have) a place to go to. Thanks to my friends who were there for me, also during harder times, and who were always in for a beer to cheer me up or have a good talk.

Finally I want to thank God for everything he gives. For my family and friends, for the opportunity to receive education, for the amazing world around us and the possibility to investigate and discover how it works.

*S.H. Alkema  
Delft, 3 April 2018*

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## Abstract

The uncertainties in undrained macro-stability analysis of dikes lead to overdimensioned and inefficient designs. The undrained shear strength ( $s_u$ ), which describes the strength of clay and peat layers, is important information for the macro-stability analysis of a dike. The cone factor ( $N_c$ ) gives a linear correlation between undrained shear strength ( $s_u$ ) of the soil and the cone resistance ( $q_c$ ) from a cone penetration test (CPT). In practice the uncertainty on this correlation, given by the variation coefficient, is high, especially it overestimates the  $s_u$  for high values of  $q_c$ . As a result, a large reduction factor (RF) is taken into account between the expected value of  $s_u$  and the characteristic value of  $s_u$  used in calculations.

In literature the cone factor ( $N_c$ ) is assumed to depend on the soil stiffness, the cone roughness and the initial soil stress state ( $\Delta$ ). Including the initial soil stress state, or more specifically the horizontal soil stresses ( $\sigma'_{h0}$ ), in the cone factor determination can lead to a reduction of the variation coefficient. Based on literature and a numerical analysis it is found that including the initial soil stress state in the determination of  $N_c$  can lead to a difference of 10% in cone factor. However, the difference is less than 10% for most measures due to the relatively similar stress conditions over a large part of the cross section of a dike. In addition, a theoretical application of the initial soil stress state ( $\Delta$ ) shows that the overestimation of  $s_u$  for high values of  $q_c$  cannot be solved by including it as found in literature.

A case study on the Hollandse IJsseldijk project has been conducted to back up the theoretical approach with experimental measurements. As a conclusion it is found that the variation coefficient is reduced slightly taking into account the initial horizontal soil stresses, resulting in a decrease of the reduction factor (RF) of 2-3%. Taking this small improvement and remaining unknowns in the calculation into account, it is concluded that including a measure of the in-situ soil stress state ( $\Delta$ ) in the determination of the cone factor ( $N_c$ ) does not result in a significant reduction of the present uncertainties in dike assessment with existing approaches.

## Samenvatting

De onzekerheden in ongedraineerde analyses met betrekking tot macro-stabiliteit leiden tot overgedimensioneerde en inefficiënte ontwerpen. De ongedraineerde schuifsterkte ( $s_u$ ) is belangrijke informatie voor de macro-stabiliteit van een dijk. De conusfactor ( $N_c$ ) geeft een lineaire correlatie tussen de ongedraineerde schuifsterkte ( $s_u$ ) van de grond en de conusweerstand ( $q_c$ ) uit een sondering (CPT). In de praktijk is de onzekerheid in deze correlatie, gegeven door de variatiecoëfficiënt, hoog. In het bijzonder doordat de correlatie de waarde van  $s_u$  overschat voor hoge conusweerstand. Dit resulteert in een hoge reductiefactor (RF) tussen de verwachte en karakteristieke waarde van  $s_u$  die gebruikt wordt in de berekeningen.

In de literatuur wordt de conusfactor ( $N_c$ ) beschreven afhankelijk te zijn van de grondstijfheid, de ruwheid van de conus en de initiële spanningstoestand van de grond ( $\Delta$ ). Het meenemen van de spanningstoestand, of meer specifiek de horizontale grondspanningen ( $\sigma'_{h0}$ ), in de bepaling van de conusfactor, leidt mogelijk tot een vermindering van de variatiecoëfficiënt.

Op basis van de literatuur en een numerieke analyse is bepaald dat het meenemen van  $\Delta$  in de bepaling van  $N_c$  kan leiden tot een verschil van 10% in de conusfactor. Het effectieve verschil is echter minder dan 10% doordat dezelfde spanningstoestand geldt voor een groot deel van de dwarsdoorsnede van een dijk. Verder laat een theoretische beredenering zien dat de overschatting van de ongedraineerde schuifsterkte ( $s_u$ ) voor hoge waarden van de conusweerstand ( $q_c$ ) niet opgelost kan worden door de initiële spanningstoestand van de grond mee te nemen zoals in de literatuur wordt voorgesteld.

Gegevens van het project Hollandse IJsseldijk zijn gebruikt om de theoretische analyse te ondersteunen met experimentele waarnemingen. Op basis hiervan is geconcludeerd dat de variatiecoëfficiënt enigszins verkleind wordt door de initiële horizontale grondspanningen mee te nemen. Dit resulteert in een verlaging van de reductiefactor (RF) van 2-3%. Rekening houdend met deze slechts kleine verbetering en de overblijvende onzekerheden kan worden geconcludeerd dat het meenemen van de initiële spanningstoestand van de grond ( $\Delta$ ) in de bepaling van de conusfactor ( $N_c$ ) niet resulteert in een significante verkleining van de onzekerheden in huidige macro-stabiliteit berekeningen.



# 1 Introduction

The Netherlands is a country that is well known for its fight against water. In the past and the present, dikes protect a large part of the population against flooding caused by high water levels at sea, in rivers or in delta's. While the sea level rises and weather conditions are getting more extreme, the art of designing dikes is relevant as ever.

Of the many dikes in The Netherlands, a large number is built in areas where there are soft peat- or organic clay layers present in the subsoil. In these situations, macro stability is a normative failure mechanism for the dike (Figure 1.1). The failure mechanism is largely determined by the shear resistance of the soft layers in the soil beneath and next to the dike. Therefore, the shear resistance is an important parameter in dike design.

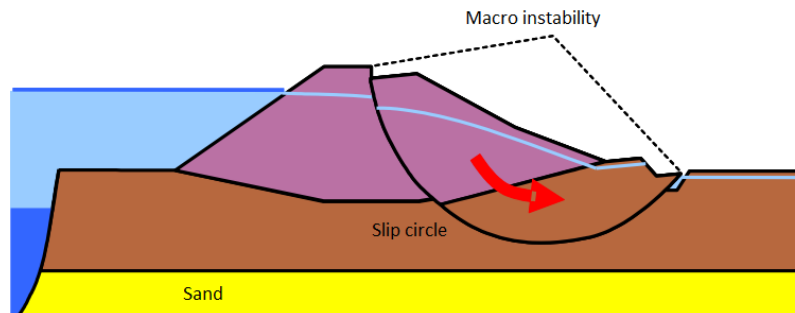


Figure 1.1 - Macro instability of a dike (van Duinen, 2014)

There are numerous ways to determine the shear strength of a peat- or organic clay layer. In the laboratory, the shear strength can be found by different testing methods, and multiple relationships have been proposed to correlate the shear strength of the soil to other soil properties (Ceccato, 2015). In practice, the shear strength can be derived directly from field data, by correlating it to the cone resistance of a Cone Penetration Test (CPT). An important advantage of the latter method is the continuous, undisturbed image of the subsoil the CPT gives at a relatively low cost. In this way, it is possible to obtain a dataset which gives an integral view on the shear strengths beneath the dike. The largest uncertainty in this method resides in the correlation between the shear strength of the soil ( $s_u$ ) and the cone resistance of a CPT ( $q_c$ ) (Rijkswaterstaat, 2016, p. 159).

## 1.1 Current practice in design of dikes on peat and organic clays

In the Netherlands the current practice of designing and assessing dikes is described in the ‘Wettelijk Beoordelingsinstrumentarium’ or WBI 2017 (De Waal, 2017) and the report ‘Dijken op Veen II’ or DoV II (Zwanenburg, 2014). The WBI 2017 describes the complete assessment procedure of primary dikes, while the DoV II report is focussed on the macro stability of the dikes. Although there are some minor differences between the method of the WBI and that of DoV, these are not relevant in this research. Therefore the method described in DoV II will be taken as leading in the description of the current practice.

The following steps are taken in the DoV II method:

1. Choice of the profiles where the macro stability has to be calculated.
2. Measurements in-situ, containing:
  - a. Geological description of the peat and organic clay layers;
  - b. CPT’s, BPT’s (Ball Penetration Tests), and borings at specific test fields for the purpose of composing the correlation;
  - c. Borings for the purpose of laboratory tests;
  - d. CPT’s and BPT’s at all the chosen profiles for parameter determination.
3. Laboratory tests.
4. Determination of the cone factor(s).
5. Determination of the local parameters.
6. Determination of the remaining assumptions for the stability analysis, such as extreme water level.
7. Design of the ‘zero-variant’, with which the current safety of the dike is calculated.
8. Design of the dike improvement if the current safety is not sufficient.

In this process the steps 2-5 provide a local profile of the undrained shear strength ( $s_u$ ) as shown in Figure 1.2. First CPT, BPT and laboratory tests from specific test fields are used to determine the cone factors ( $N_{kt}$  or  $N_b$ ). With this factor the correlation between the cone resistance ( $q_c$ ) and shear strength ( $s_u$ ) is described by the following formulas:

$$s_u = \frac{q_{net}}{N_{kt}} \quad (1.1)$$

$$q_{net} = (q_c + u_2(1 - a)) - \sigma'_{v0} \quad (1.2)$$

where  $q_{net}$  is the corrected cone resistance,  $q_c$  is the measured cone resistance,  $u_2$  is the measured water pressure,  $a$  is the cone factor for the water pressure filter and  $\sigma'_{v0}$  is the initial vertical stress. For a Ball Penetration Test (BPT) the correlation factor  $N_b$  is found taking the  $q_{net}$  the same as the measured resistance  $q_{ball}$ .

In order to find the undrained shear strength at other locations where no borings are conducted, local CPT’s are used as shown at the right side of Figure 1.2. The local CPT is linearized and with the found N-factor, the expected value of  $s_u$  for a specific location can be calculated using the same formula (1.1).

In the described process there is a number of uncertainties which have to be dealt with as mentioned in Figure 1.2. Therefore the standard deviation of the shear strength at the test fields is calculated. This is done by the spatial and the transformational variability. The spatial variability deals with the uncertainties in the linearization of the CPT and the differences in space

due to heterogeneity, while the transformational variability looks at the deviation for the correlation. A more detailed description of this calculation can be found in ‘Eindrapport heterogeniteit’ (De Bruijn, Visschedijk, & Van der Ham, 2014). Both variabilities together form the following standard deviation:

$$VC_{average} = \sqrt{VC_{average.spatial}^2 + VC_{average.transformation}^2} \quad (1.3)$$

$$SD_{\ln(s_u)} = \sqrt{\ln(1 + (VC_{average})^2)} \quad (1.4)$$

With the standard deviation the initial characteristic value for  $s_u$  is found. Secondly, the ultimate value is calculated using the over consolidation ratio (OCR) and strength increase exponent (m) from the laboratory tests. This step takes into account the changed loading conditions of the soil during the ultimate design situation with respect to the daily measurement situation. To obtain the design value of  $s_u$  the material factor can be applied before the calculation (Zwanenburg, 2014), or a safety factor is taken into account after the calculation (Rijkswaterstaat, 2016).

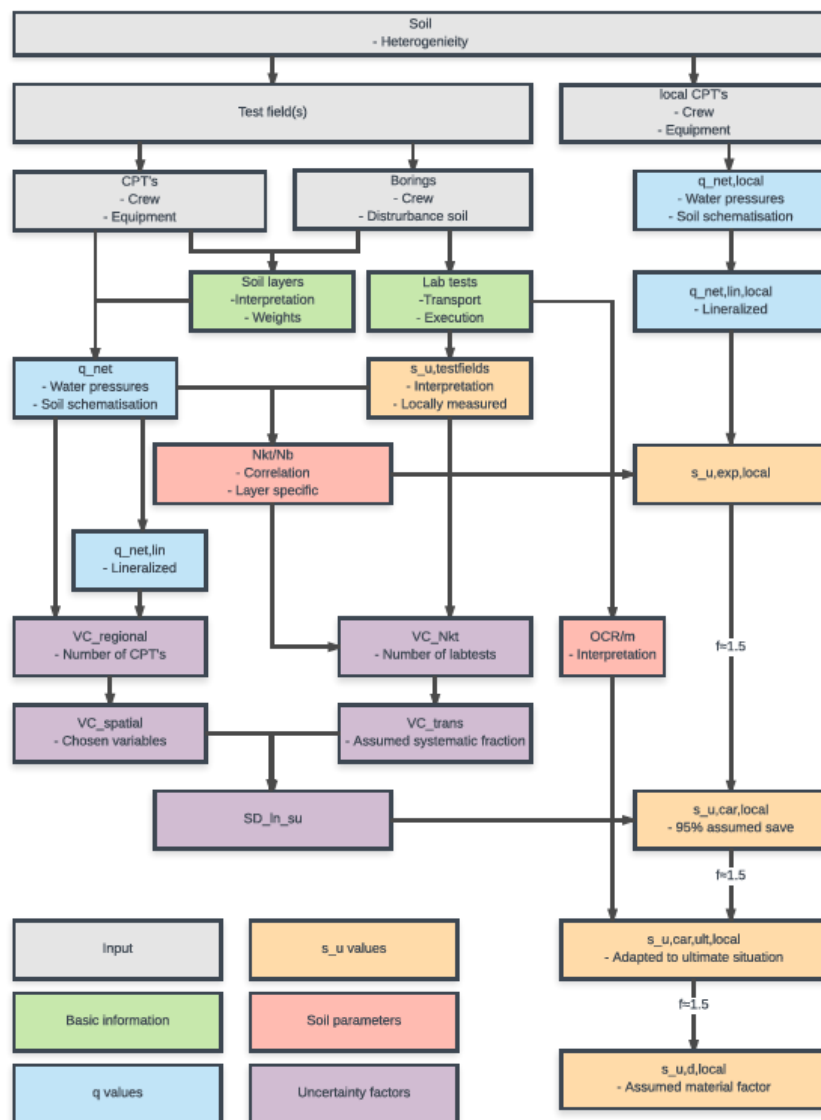


Figure 1.2 - Process scheme determination local  $s_u$  based on DoV II

## 1.2 Problem statement

Looking at the whole process from measurements to calculation results there are three reduction steps in which the initial  $s_u$  value is corrected (Figure 1.2).

1. The first step contains the standard deviation regarding the spatial variability and transformational variability.
2. The second step takes into account the differences in loading conditions between the daily measuring situation and the ultimate design situation.
3. Thirdly a safety factor is applied before or after the calculation.

The need for the first and last reduction lays in the safety philosophy. In order to mitigate the risks a safe value is chosen for the parameters used in the calculation and a safety factor is taken into account during the calculation. When the uncertainties become higher, the safe value for a strength parameter like  $s_u$  must be lowered in order to guarantee that the probability of macro instability will be below the acceptable one. As a result, each uncertainty in the process of determining  $s_u$  leads to the choice of a value that is lower than a given percentage of all measurements (often 95 or 99%) and therefore lower than the expected value. In practice these reduction steps result in a total reduction of the shear strength by a factor between 2 and 4. Some of the reduction may be technically correct due to the changed loading condition or heterogeneity, but most of it is due to the uncertainties in the calculation process. As a result, the design value of  $s_u$  used is much lower than the value it probably has in reality. While this is reasonable due to uncertainties, this causes a lot of problems in practice or at least inefficient designs.

One of the uncertainties lies in the relation between the net cone resistance  $q_{net}$  and the undrained shear strength  $s_u$ . Which is included in the first step mentioned above. As stated by Rijkswaterstaat (2016, p. 159), 90% of the standard deviation ( $SD_{In(s_u)}$ ) is due to the transformational variability of this correlation. The large influence of the transformational variability is also shown by looking at the spread of the found values for  $SD_{In(s_u)}$  in practice (den Arend, 2017), (Zwanenburg, 2014). Following the WBI, the reduction in the first step can be calculated based on the standard deviation. This reduction is further referred to as the Reduction Factor (RF). As shown in Figure 1.3 following equation 1.3 and 1.4, a difference in  $SD_{In(s_u)}$  between 0,15 and 0,35 is found and resulting a difference between 1.2 and 1.7 for the RF.

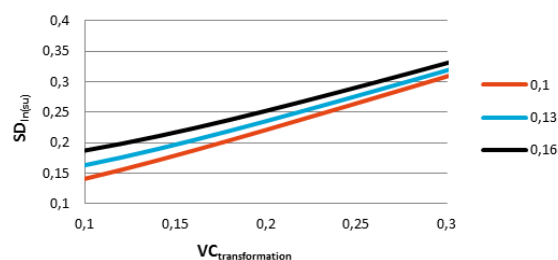


Figure 1.3 –  $SD_{In(s_u)}$  as a function of  $VC_{transformation}$  for three values of  $VC_{spatial}$

A significant part of the problem described therefore lies in the correlation between  $q_{net}$  and  $N_c$ . In the current practice this correlation is assumed linear and dependent on the initial vertical stresses. While literature describes an influence of initial horizontal stresses on the cone factor, this is not taken into account in the current practice in the Netherlands. It is unknown whether a more specific inclusion of the stress state of the soil will lead to a smaller variation coefficient and therefore a smaller reduction factor for the undrained shear strength.

### 1.3 Research goal

When the uncertainties in the calculation are reduced, the standard deviations become smaller and as a result the safe value of  $s_u$  can be taken higher. In order to limit the extension of this research, it is chosen to focus on the influence of the initial soil stresses on the cone factor. In literature (Teh & Houlsby, 1991), (Lu, Randolph, Hu, & Bugarski, 2004) the cone factor is described including the initial effective soil stresses. Further in Figure 1.4 CPT data from the Markermeerdijken project is plotted against the shear strength. Part of the scatter may be due to measurement errors in-situ and in the lab, but what can be seen is the relative closeness of location specific clusters, which suggests an influence of location specific parameters. Therefore the correlation could be improved by including the local initial stresses. It is also observed that the  $s_u$  corresponding with a higher  $q_{net}$  shows more test results below the expectation line than above it. If including the initial soil stresses gives an explanation for this observation, this improves the correlation.

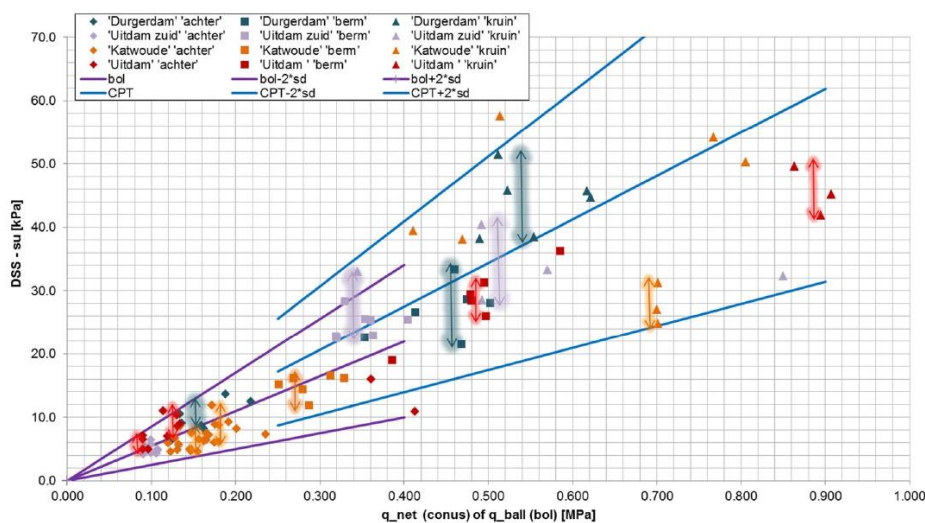


Figure 1.4 -  $q_{net/ball}$  plotted versus  $s_u$  with the expectation value and the 95% interval (De Buijn, Visschedijk, & Van der Ham, 2014)

The relation between  $s_u$  and  $q_c$  is now described by the factor  $N_c$ , in practice taken as a constant. By improving the knowledge about the influence of the initial stress state on the  $N_c$  value, a better correlation can be proposed. Therefore the research goal of this thesis is stated as follows:

*Investigate if the uncertainties in macro stability analysis for dikes on peat can be reduced by including the initial stress state in the correlation between shear strength and cone resistance.*

In order to reach this goal, the following research questions are listed:

- What is known about the cone factor?
- What is known about the initial horizontal soil stresses?
- What is the theoretical influence of the initial soil stresses on the cone factor?
- Which stress states can be found under or next to a dike?
- Which local characteristics influence the stress state of the soil?
- What is the influence of the initial soil stresses expected in practice?
- How much can the correlation be improved by including the initial horizontal stress?
- To what extent the horizontal stresses should be included in practice?

## 1.4 Layout of the thesis

### 1.4.1 Method

#### Literature study

To pursue the stated research goal the first step is a thorough understanding of the subject and knowledge of the most up to date research on the topic. With this knowledge a probable outcome of the research goal is stated. This phase will focus on understanding the geotechnical behaviour of peat and clay during a CPT, the possible factors that influence the cone factor  $N_c$  and the characteristics of the factors of influence. Further the horizontal stresses are subjected to a literature study.

#### Preliminary numerical analysis

In order to investigate the influence of a non-horizontal surface on the horizontal stresses a basic PLAXIS calculation will be executed. In order to do this first the best suitable soil model is selected. Following a preliminary calculation is conducted. In this way the horizontal stresses over a dike profile can be estimated. Following typical maximum and minimum values of the horizontal stress and the initial soil stress ratio ( $\Delta$ ) are presented. In this way a first substantiated estimation of the total influence of the horizontal stress on the cone factor in practice can be given. In addition the progression of the cone factor over a basic dike profile and a homogeneous soil profile can be shown.

#### Case study

In order to test if taking the horizontal stresses into account following earlier results would decrease the variation of the  $N_c$  correlation, the method is applied on a case study, which will be the Hollandse IJsseldijk. The known profiles of this case will be analysed with PLAXIS and the horizontal stresses on the spots of CPT's and borings will be obtained. With the results, the initial deviation of the results can be compared to the results based on the calculation including horizontal stresses. As a result the improvement of an extended calculation can be discussed.

#### Implementation possibilities

The last step in the thesis is looking at the implementation of the found results. An analysis of the found results is given to place them in the larger context of the WBI. Also the benefits and problems of implementing the improved correlation in practice are discussed.

### 1.4.2 Reading guide

Chapter 1 introduces the subject, the problem, research goal and how it is handled with in this thesis. The following chapter 2 states a summary of the literature study about the cone factor and horizontal stresses. Chapter 3 describes the numerical PLAXIS analysis of the horizontal stresses in a dike. The fourth chapter states the results of the case study of the Hollandse IJsseldijk. Chapter 5 discusses the implementation possibilities and the last and sixth chapter summarises the conclusions from the present study and recommendations.

## 2 The cone factor and initial stresses

### 2.1 Literature study on the cone factor

#### 2.1.1 Definition

In the literature study (Appendix I) the main focus was on the correlation between  $q_c$  and  $s_u$  and its values or formulas presented earlier. The correlation between the net cone resistance ( $q_c - \sigma_0$ ) and the undrained shear strength ( $s_u$ ) is given by the cone factor  $N_c$ . Which is defined as follows (Lunne, Robertson, & Powell, 1997, p. 63):

$$N_c = \frac{q_c - \sigma'_0}{s_u} \quad (2.1)$$

This cone factor  $N_c$  can be a value based on empirical data, as shown in current practice. It can also be a theoretical formula, taking into account several soil parameters and therefore differentiating specific situations. As shown by Lunne et al. (1997) in most literature the  $\sigma_0$  is taken as  $\sigma'_{v0}$  but in some cases  $\sigma_{mean}$  or  $\sigma'_{h0}$  is used. In Dutch practice  $N_{kt}$  is based on the initial vertical stress and a corrected pore pressure value ( $q_t$ ) is used.

$$q_t = q_c + (1 - a)u_2 \quad (2.2)$$

$$N_{kt} = \frac{q_t - \sigma'_{v0}}{s_u} = \frac{q_{net,v}}{s_u} \quad (2.3)$$

#### 2.1.2 Proposed formulas

In the past, a lot of different methods to estimate or describe  $N_c$  are used such as

- **Classical bearing capacity theory** by Prandtl (1920), Terzaghi (1943) and Skempton (1951).
- **Cavity expansion theory** (Vesic, 1975)
- **Strain path theory** (Baligh M. M., 1985)

Teh and Houlsby (1991) improved the last method by incorporating its merits with the finite element method in order to diminish the error in the stress equilibrium. As a result the cone factor is described in two ways based on the rigidity index ( $I_r$ ), the cone and shaft roughness ( $\alpha_c$  or  $\alpha_f$  and  $\alpha_s$ ) and the dimensionless initial horizontal stress factor  $\Delta$ . At first Teh and Houlsby described the correlation based on strain path theory, using a quasi-analytical approach to add the cone tip roughness, resulting in the following equations:

$$N_c = 1,25 + 1,84 \ln(I_r) + 2\alpha_c - 2\Delta \quad (2.4)$$

$$I_r = \frac{G}{s_u} \quad (2.5)$$

$$\Delta = \frac{\sigma'_{v0} - \sigma'_{h0}}{2s_u} \quad (2.6)$$

$$\alpha_c = \frac{a}{s_u} \quad (2.7)$$

Combining the strain path theory with finite element analysis, the resulting expression for the cone factor becomes:

$$N_c = N_s \left( 1.25 + \left( \frac{I_r}{2000} \right) \right) + 2,4\alpha_f - 0,2\alpha_s - 1,8\Delta \quad (2.8)$$

$$N_s = \frac{4}{3}(1 + \ln(I_r)) \quad (2.9)$$

Following the assumed values in the report of Teh and Houlsby,  $N_{kt}$  typically range from 9 to 17. More recently, **numerical approaches** are used to correlate  $q_c$  to  $s_u$  instead of the analytical approaches described above. In these cases numerical approaches are needed which can handle large deformations. Ceccato (2015) lists the Arbitrary Lagrangean Eulerean method or ALE (van den Berg, 1994) (Lu, Randolph, Hu, & Bugarski, 2004), the Coupled Eulerian-Langrangian method or CEL (Qui, 2014) and the Material Point Method or MPM (Beuth, 2012). Based on the Tresca soil model, these approaches led to the general equation (2.10). However the gradient factors in the equation differ for each model and report. The general formula stated by Lu et al. is as follows:

$$N_c = 3,4 + 1,6 \ln(I_r) + 1,3\alpha_c - 1,9\Delta \quad (2.10)$$

In which the rigidity index ( $I_r$ ), the dimensionless factor  $\Delta$  and the cone factor ( $\alpha_c$ ) are defined by (2.5), (2.6) and (2.7).

### 2.1.3 Factors of influence

In the most recent approaches there are three main influences on the value of the cone factor: first the cone roughness, second the soil rigidity and third the initial state of the soil. The **cone roughness** ( $\alpha_c$ ), as defined by (2.7) influences the direction of the stresses at the cone face. As shown in Figure 2.1 (Ceccato, 2015) the principal stress direction for a rough cone ( $\alpha_c = 1$ ) is faced more downwards, and therefore creating a higher cone resistance with respect to the smooth cone. This effect is also shown in the general formula for  $N_c$  (2.10). The factor with which the cone roughness is multiplied, ranges from 1,3 to 2,4. If the same cone is used in practice the effect of the cone roughness on the cone factor will be constant.

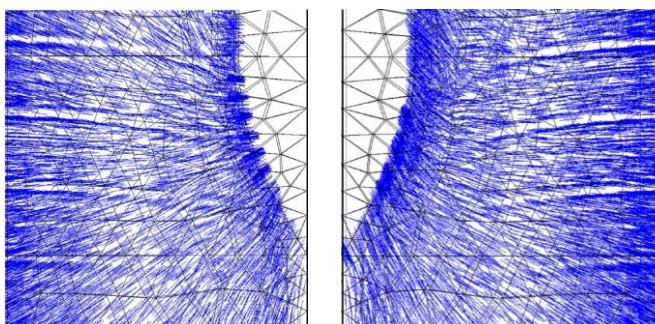


Figure 2.1- Principal stress direction around the cone in case of  $\alpha_c = 0$  (left) and  $\alpha_c = 1$  (right). (Ceccato, 2015)

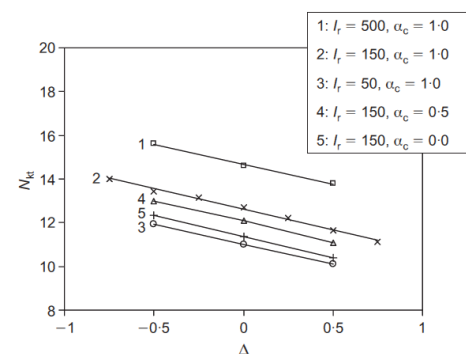


Figure 2.2 - Variation of  $N_c$  with stress anisotropy (Lu, Randolph, Hu, & Bugarski, 2004)



The **soil rigidity** ( $I_r$ ) as defined by (2.5) is the primary soil parameter in most theories presented in the previous paragraph. The correlation between the soil rigidity and the cone factor is presented as logarithmic in (2.10) and other formulas. The multiplication factor is found 1,84 (Teh & Houlsby, 1991) or 1,6 (Lu, Randolph, Hu, & Bugarski, 2004). Although research shows that this factor can differ based on the other influence factors, Lu et al. indicates that the variation is less than 5%, recommending to use the gradient 1,6 for simplicity. In practice it can be stated that the soil rigidity is constant for a soil layer.

The influence of the **initial soil stresses** on the cone factor is first investigated by Teh and Houlsby (1991) by varying the in-situ horizontal and vertical stresses. In order to present their results the dimensionless factor  $\Delta$  is introduced, which is defined in formula (2.6). The possible values of  $\Delta$  are in the range -1 to 1, because the stresses in the initial soil state are limited by the shear strength of the soil. As a result it is stated that it would be more consistent to define the cone factor in terms of horizontal stresses instead of the vertical stresses. Lu et al. also investigated the influence of  $\Delta$ , concluding with a gradient of -1,9 as supported by the gradient found in Figure 2.2. In later research the in-situ soil stresses are often kept constant at  $\Delta = 0$ , as shown by Ceccato (2015, p. 126). Reason for this could be the fact that in practice  $\sigma'_{ho}$  is not known accurately.

#### *2.1.4 In-situ soil properties*

In order to improve the estimation of the cone factor for specific locations, the in-situ soil properties should be included in the correlation. The cone roughness and soil rigidity are considered constant over a soil layer. Therefore the influence of local soil properties as heterogeneity, anisotropy, vertical and horizontal stresses on the cone factor are discussed. The focus of this discussion is the possibility to estimate the cone factor based on the specific in-situ soil properties.

##### Heterogeneity

The heterogeneity of the soil, and therefore of the soil rigidity and indirect the undrained shear strength value, can have a big impact on the safety risks of a slope failure (Griffiths & Fenton, 2000). In most cases, it is suggested that the vertical scale of fluctuation is small relative to the horizontal scale of fluctuation and the height of the slope (Hicks & Samy, 2002). Therefore, the vertical variability is most relevant for the stability of clay and peat dikes. When the soil fails around a CPT, the zone in which the failure takes place extends over 6-8 times the cone radius (Teh & Houlsby, 1991), and therefore in a way averages the shear strength values over that zone. In current practice this effect is neglected even more by linearizing the cone resistance over a layer, deleting the first and last few values which are affected by soil layers above or below (Zwanenburg, 2014). This linearization seems to have no effect on the reliability of the correlation between the cone resistance and the undrained shear strength (De Bruijn, Visschedijk, & Van der Ham, 2014, p. 59). In order to diminish the risks due to heterogeneity, a material factor is applied in practice, although it is argued that in some cases this is not enough (Hicks, Nuttall, & Chen, 2014). However, further research about the effect of heterogeneity on the cone factor and slope failure lies out of the scope of this thesis because they are assumed to be the same throughout the whole soil layer.

##### Strength anisotropy

Due to the effect of deposition, natural clays are usually anisotropic. When soil elements are subjected to principal stress rotation by a cone, the anisotropy of the clay can have an effect

on the cone resistance (Baligh M. M., 1984). Su & Liao (2002) concluded that taking the influence of anisotropy into account gives a difference of maximum 20% for the cone factor. Although this is a difference that should be taken into account in order to find a more reliable cone factor, it is assumed that this factor will not differ significant within a stated soil layer. The strength anisotropy is considered the same for each location as long as the same soil layer is considered. Therefore it is taken into account implicitly when using empirical results and does not influence the local differences in the cone factor.

### Vertical stresses

From an early stage the vertical stresses are taken into account in the formula (2.1) of the cone factor (Vesic, 1975). In this way, the cone factor is already correlated with the cone depth and the corresponding stress. Further, the vertical stress is taken into account as a part of the dimensionless factor  $\Delta$  (2.7), which function is to describe the stress anisotropy in the soil rather than the specific vertical stresses. In none of the researches mentioned before another correlation of the cone factor with depth is observed. Therefore it is assumed that the theoretical approach which subtracts the vertical stress from the measured cone resistance is in line with the empirical results.

### Horizontal stresses

While the vertical stress is easy to determine from the soil weight above, the horizontal stresses are not as easy to find (Verruijt, 2007). Therefore most of the stated values and formulas for the cone factor use  $\sigma'_{v0}$  as variable (Lunne, Robertson, & Powell, 1997), while it is stated by Teh & Houlsby (1991) that using the horizontal initial stresses would be more consistent. Taking into account the dimensionless factor  $\Delta$  with a gradient of 2 gives the same result, replacing  $\sigma'_{v0}$  with  $\sigma'_{h0}$ . In general it can be stated that the stresses in the soil have influence on the cone factor in two ways, firstly direct by subtracting it from the cone resistance and secondly due to the influence of the initial stress anisotropy in the soil. When the gradient of  $\Delta$  is determined, both ways are taken into account. The factor  $\Delta$  can differ for the same soil layer because it is effected by local non-horizontal surfaces. Therefore in order to find a more location specific value of  $N_c$ , the horizontal stresses need to be taken into account.

In order to find the horizontal stress for a soil at rest, the factor  $K_0$  is used. Formula (2.7) can be described in terms of  $K_0$  as follows:

$$\Delta = \frac{\sigma'_{v0} - \sigma'_{h0}}{2s_u} = \frac{(1 - K_0)\sigma'_{v0}}{2s_u} \quad (2.11)$$

While this may be correct for a horizontal soil body at rest, the horizontal stresses in reality may differ, specifically for situations with a non-horizontal surface, which is the case for dikes. Therefore the next section will focus on the value of  $\sigma'_{h0}$  and  $\Delta$  with presence of non-horizontal surfaces.

## 2.2 Literature study on horizontal stresses

Following the findings in the previous chapter about the cone factor, the horizontal (or lateral) stresses are subjected to a literature study (Appendix I). In this chapter, the horizontal stresses in general are discussed, followed by specific situations where the horizontal stresses are influenced. While the general description concerns a 1-dimensional situation, the influence of historical and spatial stresses should be taken into account. The history of the soil is described by the OCR in the second section and the influence of a non-horizontal surface in the third section. Concluding a first estimation is made concerning the influence of the horizontal stresses on the cone factor.

### 2.2.1 Horizontal stresses in general

The calculation of stresses in the soil is one of the main problems in soil mechanics (Verruijt, 2007). Even in the easiest situation of a soil mass without load, the stresses are not unequivocal. In general, a good assumption is that the vertical soil stresses increase linearly with the soil weight. For this research, it is also assumed that the horizontal stresses are equal in all horizontal directions. In general the horizontal stresses are related to the vertical stresses using the coefficient of lateral earth pressure (K). Therefore the horizontal stresses can be described as:

$$\sigma_{xx} = \sigma_{yy} = K\sigma_{zz} = K\gamma z \quad (2.12)$$

Where  $\sigma_{xx}$  and  $\sigma_{yy}$  are the lateral stresses,  $\sigma_{zz}$  the vertical soil stress, K the coefficient of lateral earth pressure,  $\gamma$  the soil weight and z the depth. Although this seems to simplify the problem, K can still be a random function. Following Verruijt, a number of factors influence the value of K, such as material properties, geological history and current conditions. If a soil is modelled elastic following Hooke's law, the value of K can be found based on the Poisson's ratio ( $\nu$ ):

$$K = \frac{\nu}{1-\nu} \quad (2.13)$$

In wet conditions the water pressure is considered hydrostatic and the vertical and horizontal water pressures are equal, therefore in this situation formula (2.12) becomes (2.14) so only the effective stresses are multiplied by factor K.

$$\sigma'_{xx} = \sigma'_{yy} = K\sigma'_{zz} = K(\gamma_s - \gamma_w)z \quad (2.14)$$

In order to find the boundaries for the value of K, Rankine (1857) used the Mohr-Coulomb failure criterion and defined  $K_a$  and  $K_p$  as minimum and maximum values for K based on the angle of internal friction and cohesion of a material:

$$K_a = \frac{1-\sin\phi'}{1+\sin\phi'} = \frac{1}{K_p} \quad (2.15)$$

Because  $K_a < K < K_p$  as stated by Verruijt, the ultimate boundaries for K are roughly 0,2 and 5 and depending on  $\phi'$ . Because of the big ratio between these factors the horizontal stresses are still mostly undefined. Looking at a soil body at rest, in the situation of a CPT, the K value is defined as  $K_0$ . Jaky (1944) suggested to estimate the value for  $K_0$  with formula (2.16) and this is used often in practice, but there is no theoretical or empirical base for this formula

(Verruijt, 2007) (Michalowski R. L., 2005). Zhao et al. (2010) discusses formula (2.17) as a way to deal with the differentiation of  $K_0$  that is observed when increasing the axial pressure in a triaxial test.

$$K_0 = 1 - \sin\phi' \quad (2.16)$$

$$K_0 = K'_0 + k \cdot \ln \frac{\sigma_1}{10P_a} \quad (2.17)$$

In which  $K'_0$  is the initial coefficient of lateral earth pressure,  $k$  a material factor,  $\sigma_1$  the axial pressure and  $P_a$  the atmospheric pressure. This formula would suggest very low values for  $K_0$  at low axial pressures, which is also visible in the measurements of Zhao, as values below 0.3 are observed. However Zhao also mentions that the formula of Jaky for the initial value of  $K_0$  is only suitable for certain soils. Therefore in order to find the value for  $K_0$  lab tests are performed in practice. Because local conditions as stress history and non-horizontal surfaces are not included in the results of this test, the following  $K$  value is defined as  $K_0^{nc}$ . Hayashi et al. (2012) stated a formula based on the ignition loss of the soil which describes the normally consolidated coefficient of lateral earth pressure for peat and organic clays:

$$K_0^{nc} = 0,47 - 0,0025L_i \quad (2.18)$$

In which  $L_i$  is the ignition loss in [%]. Hayashi also mentions the found results for peat and (organic) clays match the stated formula of Jaky. Therefore the combination of (2.16) and (2.18) could give a reasonable estimation of the  $K_0^{nc}$  value for peats and organic clays.

### 2.2.2 Over consolidation and horizontal stresses

In order to find the horizontal stress in-situ, the consolidation of the soil must be taken into account. Due to the history of the soil it may be over consolidated, this can be due to loads in the past or seasonal change in water level (Hayashi, Yamazoe, Mitachi, Tanaka, & Nishimoto, 2012). This soil state is mostly described with the Over Consolidation Ratio (OCR) or the Pre Overburden Pressure (POP). While in an over consolidated soil the stiffness parameters change, a change is also observed in the  $K_0$  factor of the soil. Brinkgreve (2017) describes the process as shown in Figure 2.3. While a plastic process took place during loading, in the unloading phase an elastic process is observed.

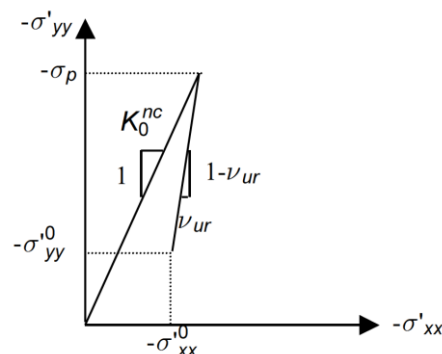


Figure 2.3 -  $K_0$  for an over consolidated soil (Brinkgreve, Kumarswamy, & Swolfs, 2017)

While in Figure 2.3 the process of unloading and reloading is described elastic, other researchers suggest a logarithmic relation between the  $K_0^{nc}$ ,  $K_0^{oc}$  and the OCR. Schmidt (1966) presented formula (2.19) and Mayne and Kulhawy (1982) found a value for the OCR gradient  $\alpha$  (2.20).

$$K_0^{oc} = K_0^{nc} \cdot OCR^\alpha \quad (2.19)$$

$$\alpha = \sin\phi' \quad (2.20)$$

While the first method is theoretically motivated, the logarithmic relation is empirically found. Because the empirical results are probably better in line with the real behaviour of the soil, the logarithmic solution is preferred for this research, in Appendix I the small difference between both solutions is discussed. Hayashi et al. found that for organic soils such as peat and organic clay, the value of  $\alpha$  can also be described based on the ignition loss. Combined with the normally consolidated coefficient ( $K_0^{nc}$ ) based on the ignition loss, the  $K_0^{oc}$  can be described with equation (2.21).

$$K_0^{oc} = K_0^{nc} \cdot OCR^\alpha = (0,47 - 0,0025L_i) \cdot OCR^{0,005L_i+0,45} \quad (2.21)$$

Although this last formula is not generally used, it may give a relative easy way to describe the  $K_0^{oc}$  coefficient. Using previous formulas the factor  $OCR^\alpha$  determines the influence of the over consolidation on the horizontal stresses. While OCR values can range from 1-5 for soils in practice, and following (2.20) the value for  $\alpha$  will be between 0,3 and 0,6, the influence of over consolidation on the horizontal stress potentially is a factor 2,5. Although formula (2.19) and (2.20) are widely accepted, Jefferies et al. (1987) stated that the  $K_0^{oc}$  in some cases seems to be uncorrelated with the OCR, and the elastic properties could be influenced by geologic-time processes, so the  $K_0^{oc}$  depends on the history and not just the OCR (Michalowski R. L., 2005). In order to deal with this criticism, besides the mathematical possibility of finding the over consolidated coefficient, it can also be found in the lab. In that case a sample can be tested with the constant rate of strain (CRS)  $K_0$  oedometer test (den Haan & Kamao, 2003).

### 2.2.3 Horizontal stresses under a non-horizontal surface

While in previous chapters the soil was considered a one-dimensional column, assuming that the surface was horizontal, in the case of dikes the height profile of the dike has an influence on the stress distribution in the soil (Michalowski R. L., 2005). Following Verruijt (2007) and as shown in section 2.2.1 the K-value presents the ratio between the vertical and horizontal stress. Based on the Mohr-Coulomb model the K-value changes due to deformations in the soil (Figure 2.4), with as extreme values the  $K_a$  and  $K_p$ . It is logical to assume that a man-built non-horizontal surface leads to deformations in the soil, and therefore to a changed stress state of the soil.

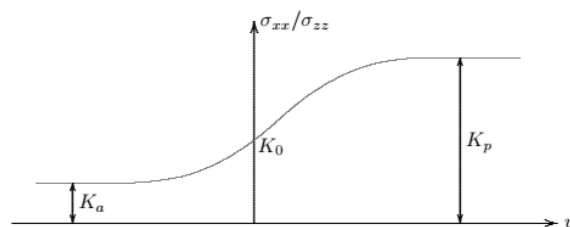


Figure 2.4 - K-value based on the horizontal deformation of the soil. (Verruijt, 2007)

In early experiments it was already found that underneath a pile of sand the distribution of base stress has a depression at the centre (Abramov, Kruzhanovskii, & Petrova, 1968). When the non-horizontal surface is modelled as a load, the stresses following will spread out into the soil. Although in the literature study some theoretical methods are discussed, it is concluded that a Finite Element Method (FEM) will give an integral solution of the stress state of the soil. Most of these models use Hooke's law to model the isotropic linear elastic behaviour of the soil stresses and are extended with deformation models such as the Hardening Soil and Soft Soil model (Brinkgreve, Kumarswamy, & Swolfs, 2017). Den Haan and Feddema (2009) modelled the deformations and stresses based on a viscous version of the Cam-clay model. They showed the good correlation between the model and the CRS  $K_0$  oedometer test for Slidrecht peats and added two case studies for which the correlation was found to be good.

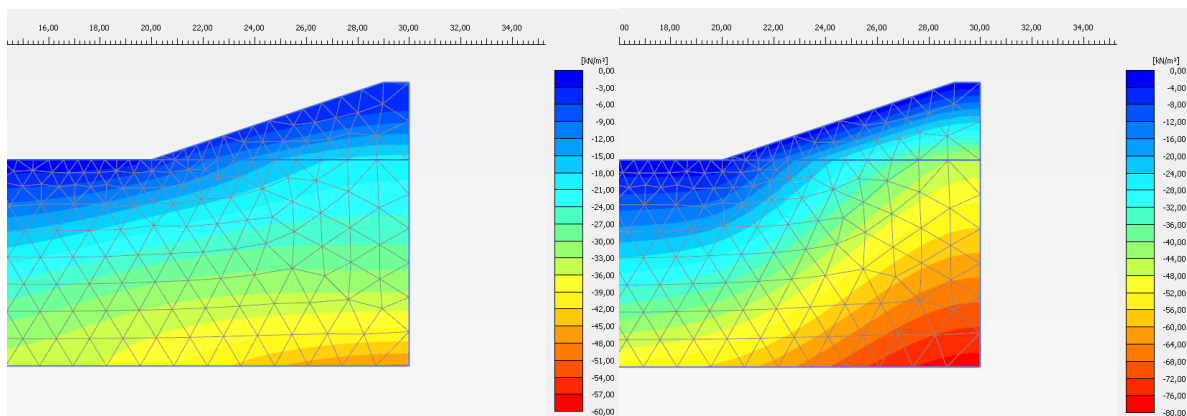


Figure 2.5 - Example of the vertical (left) and horizontal (right) stresses in a basic dike, based on SSC model FEM calculations in PLAXIS

Concluding, it can be said that the horizontal stresses under a non-horizontal surface are governed by the deformations following the load history. While some general values for  $K$  can be found, for a specific case FEM models can give a good approximation of the vertical and horizontal stresses in the soil. However the right model and correct modelling are important.

#### 2.2.4 Influence of horizontal stresses on the cone factor in dikes

Based on the results of the findings in the earlier sections of this chapter, an estimation can be made of the extreme values of  $\Delta$  and  $N_c$ . In order to do this, first the formula for  $N_c$  is adapted so that the resulting parameters are considered constant for a soil layer. Doing so with formula (2.10) gives the following equations:

$$N_c = N_{soil} - 1,9\Delta \quad (2.22)$$

$$N_{soil} = 3,4 + 1,6 \ln(I_r) + 1,3\alpha_c \quad (2.23)$$

Due to the fact that  $\Delta$  has a range from -1 to 1, this gives a theoretical boundary of the maximum influence of  $\Delta$  on the cone factor of 3,8 [-]. In the following paragraph this range is limited further by finding a range of  $\Delta$  based on values that theoretically can be expected.

Expected maximum and minimum values of  $\Delta$ 

In order to find the extreme expected values around a dike, the differences between under and next to a dike are mentioned. First, the OCR can differ due to the load history. If the soil has a certain OCR before the dike is built, the OCR remains similar next to the dike, while the OCR decreases under the dike. If a maximum OCR of 5 is assumed next to the dike and no horizontal deformations are taken into account, the  $K_0^{oc}$  of a soil with a  $\phi'$  of  $25^\circ$  such as clay can be estimated based on (2.19) and (2.20). Following (2.11) a  $\Delta$  can be calculated based on a undrained shear strength and vertical stress, which results in (2.24). If horizontal stresses due to the non-horizontal surface are taken into account, a K-value more in the direction of the passive state can be taken into account. At the toe of the dike, this may lead to values of K above 2. Therefore, locally the  $\Delta$  can become more negative than shown in (2.24). This effect will be investigated during the PLAXIS analysis.

$$\Delta_{min} = \frac{(1-1.14)\sigma'_{v0}}{2s_u} = -0.14 \frac{\sigma'_{v0}}{2s_u} \quad (2.24)$$

For the other extreme, the value under a dike is taken into account. The OCR at this location is assumed 1 there, while the horizontal stresses are even lower due to the effects of the non-horizontal surface. In that case, the K-value is assumed almost in the active state, so a value following (2.15) is taken as minimum. Following from (2.11) the maximum value becomes:

$$\Delta_{max} = \frac{(1-0.41)\sigma'_{v0}}{2s_u} = 0.59 \frac{\sigma'_{v0}}{2s_u} \quad (2.25)$$

Because  $s_u$  and  $\sigma'_{v0}$  are unknown, it is not directly clear which  $\Delta$  values follow from the found maximum and minimum. However, if  $s_u$  is defined by  $\sigma'_{v0} \cdot \tan(\phi')$ , neglecting cohesion, the value of  $s_u/\sigma'_{v0}$  becomes  $1/2\tan(\phi')$ . For a soil with a  $\phi'$  of  $25^\circ$ , this leads to a value of about 1,1. Therefore the range of -1 to 1 stated at the start of the chapter, may be reduced to roughly estimated -0.2 next to the dike to 0.7 under the dike in practice, although at local peaks of the horizontal stress, the  $\Delta$  may become lower than -0.2. It should be noted that with this definition of  $s_u$ , the values of  $I_r$  and  $\alpha_c$  are also influenced. Because the  $\Delta$  is subtracted in the equation, this leads to a higher value of  $N_c$  next to the dike and a lower  $N_c$  value under the dike. It must be noted that this does not explain the relatively lower  $s_u$  values for high  $q_{net}$  measurements observed in section 1.3. Based on this observation the  $N_c$  under the dike is expected higher. The values result in a difference of around 1.8 in  $N_c$  factor in practice excluding the other parameters. This is around 10% of the total value. However, it must be noted that this is a rough estimation. Additional research using FEM-analysis described in the following chapters provides better insight on the real influence of OCR and horizontal stresses on the cone factor.

## 2.3 Conclusion and hypothesis

The goal of this literature report is to give an overview of the current knowledge about the cone factor. In the first section, the cone factor is discussed and formula (2.25) is given as physical description. In this factor, a number of influences is taken into account such as cone roughness, soil properties and the  $\Delta$ -factor. Where the cone roughness ( $\alpha_c$ ) and rigidity index ( $I_r$ ) are assumed constant in the same soil layer, the  $\Delta$  may differ specially, because it is based on ratio between horizontal and vertical stress (2.26).

$$N_c = 3.4 + 1.6 \ln(I_r) + 1.3\alpha_c - 1.9\Delta \quad (2.25)$$

$$\Delta = \frac{\sigma'_{v0} - \sigma'_{h0}}{2s_u} \quad (2.26)$$

In order to get a better understanding about the horizontal stresses, the second paragraph discussed the coefficient of horizontal stress and the influence of the OCR and non-horizontal surfaces on the lateral stresses. This resulted in an estimation of the maximum and minimum values of  $\Delta$  that can be found around Dutch dikes. It is stated that the  $N_c$  factor can differ up unto 10% because of the horizontal stresses when the other factors are excluded. This does not give an explanation for the relatively lower  $s_u$  values for higher  $q_{net}$ . The hypothesis is stated as follows:

*The horizontal stresses as a result of the OCR and the non-horizontal surface of a dike influence the cone factor, resulting in a higher cone factor next to the dike, and a lower factor in the dike.*

In order to test this hypothesis, a FEM investigation is executed in order to gain a better understanding about the stress state of a dike. This is firstly done by a preliminary analysis for a basic dike. In addition a case study is used to compare the results with data from measurements. This may lead to a more accurate prediction of the cone factor, and therefore a lower uncertainty in macro stability calculations.



### 3 Numerical analysis

Based on the findings in the literature study, a numerical analysis using PLAXIS is executed. The goal of this analysis is to find some general estimations about the horizontal stresses under a non-horizontal surface. With these estimations, a general conclusion can be drawn about the possible effect of taking the horizontal stresses into account for the estimation of the shear resistance of the soil using the cone factor. In this chapter, first the relevant available models in PLAXIS are discussed. Secondly, some basic parameters in the PLAXIS model are discussed. Following, the results of the PLAXIS analysis are presented, concluding with a discussion about the estimated influence of the lateral stresses.

#### 3.1 Models

The choice of a model is ideally backed up with test results from the field or lab. In this research, due to limited time and capacity, it was not possible to set up an own experiment which measures the horizontal stresses in a dike body, so they are unknown as shown in Figure 3.1. Therefore, the choice of the model used is only backed up afterwards with the results in the next phase, which connects the found horizontal stresses with the value of the cone factor and therefore with the cone resistance measured in the field. Because the preliminary analysis is carried out without the implementation of these test results, it is most important to argue strongly on a theoretical basis that the model is suitable for the determination of horizontal stresses in a dike body.

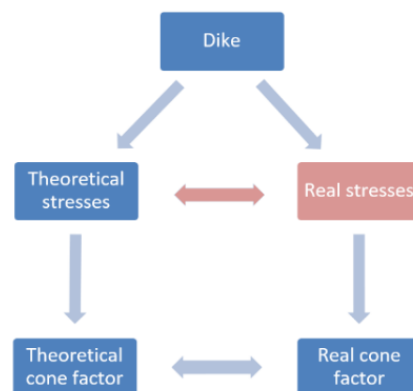


Figure 3.1 - Theoretical and real properties of a dike

For this analysis, it is chosen to use the 2D version of PLAXIS. It is assumed that the problem can be modelled two dimensional, because the length of a dike body far surpasses its width in general. Therefore, the soil properties, strains and stresses will be constant over the length of the embankment. Although the stresses are assumed constant, the horizontal stress in the direction perpendicular on the dike may be different from the stress parallel to the dike. This may result in a lower or higher average lateral stress than found in a 2D analysis. However, the lateral stress parallel to the dike will always be in between  $\sigma'_1$  and  $\sigma'_3$ , and mostly similar to  $\sigma'_3$  in cases where the stress state is governed by  $K_0^{nc}$ . At the toe of the dike, where horizontal stresses could be higher than vertical stresses this is uncertain. Further, it is assumed that the horizontal stress is the same in all directions, but additional research is recommended to evaluate this assumption.

In PLAXIS-2D, numerous models are present which are stated in the PLAXIS manual (Brinkgreve, Kumarswamy, & Swolfs, 2017). Some of the models such as the Linear Elastic, Jointed Rock and Hoek-Brown are clearly unsuitable for this research, so further discussion on these models is discarded.

The models taken into account as for further discussion are the following:

1. Mohr-Coulomb model (MC)
2. Hardening Soil model (HS)
3. Hardening Soil model with small-strain stiffness (HSsmall)
4. Soft Soil model (SS)
5. Soft Soil Creep model (SSC)
6. Modified Cam-Clay model (MCC)
7. NGI-ADP model (NGI-ADP)
8. Sekiguchi-Ohta model (Sekiguchi-Ohta)

The level of detail in the description and discussion of each model is depending on the usability of the models for this research.

The features the model ideally incorporates are as follows:

- describes the behaviour of soft soils as (organic) clays and peats well;
- includes the (stress) history of the soil;
- gives a good description of the stress state of the soil, distinguishing the vertical and lateral stresses;
- has a short calculation time.

The eight models mentioned above are presented and discussed in Appendix II. As a conclusion, only four models are taken into account for further research. The HS and HSsmall model are probably comparable for this application, so both are compared with a slight preference for the HSsmall model. Further, the SSC model and the Sekiguchi-Ohta model are used, they respectively incorporate the (creep) compression and the anisotropy of the soil. Because there are no test results on which the choice for one of the models can be made, all are taken into account in the preliminary analysis. Based on the results, the differences are discussed and one model is chosen to continue with the research.

## 3.2 Model and parameter investigation

For the preliminary analysis of a general case, some basic geometries with corresponding parameters and soil layering are set up in PLAXIS. The goal of this analysis is to find differences between the chosen models, investigate the influence of different parameter configurations and give a general description of the stress state at different positions under a dike body. The calculations conducted are described in Appendix IV.

The results of the preliminary analysis are presented in this section. Before looking at the needed boundaries of the model, a number of models are compared in order to see the differences influencing the outcome of this research. After that the resulting model is investigated, the boundaries are discussed and concluding a estimation of the horizontal and vertical stresses and the  $\Delta$  value is made.

### 3.2.1 Comparison various models

The models are compared looking at the effective horizontal and vertical stresses next to and under the dike. In Appendix IV, the geometry and model parameters are stated. The model is assumed drained, because the dike is at its place long enough for the excess pore pressures to distribute. The models are tested with and without a present OCR. As a conclusion in Appendix IV, it is found that the models show the same soil behaviour in general. The Sekiguchi-Ohta model is discarded, because there are too many unknowns of which the values cannot be found in a reliable way. The HS and HSsmall model show practically the same results. The differences between the HS(small) model and the SSC model are visible (see Figure 3.2), but due to the lack of experimental data, it is not possible to choose one of them. Because of the relative small differences and the lack of data to test the models, one model is chosen in order to reduce the work load. There are no clear distinctions found in the analysis on which the choice can be made. It is known that the SSC model is created for soft soils which are mostly present under dikes in practice. Further the CUR 228 (2010) suggest to use SSC for horizontal deformations and stresses. Therefore, it is chosen to continue with the Soft Soil Creep model as basic model for the following analysis. The 'Updated Mesh' function is used because large deformations are present and consolidation calculations are conducted.

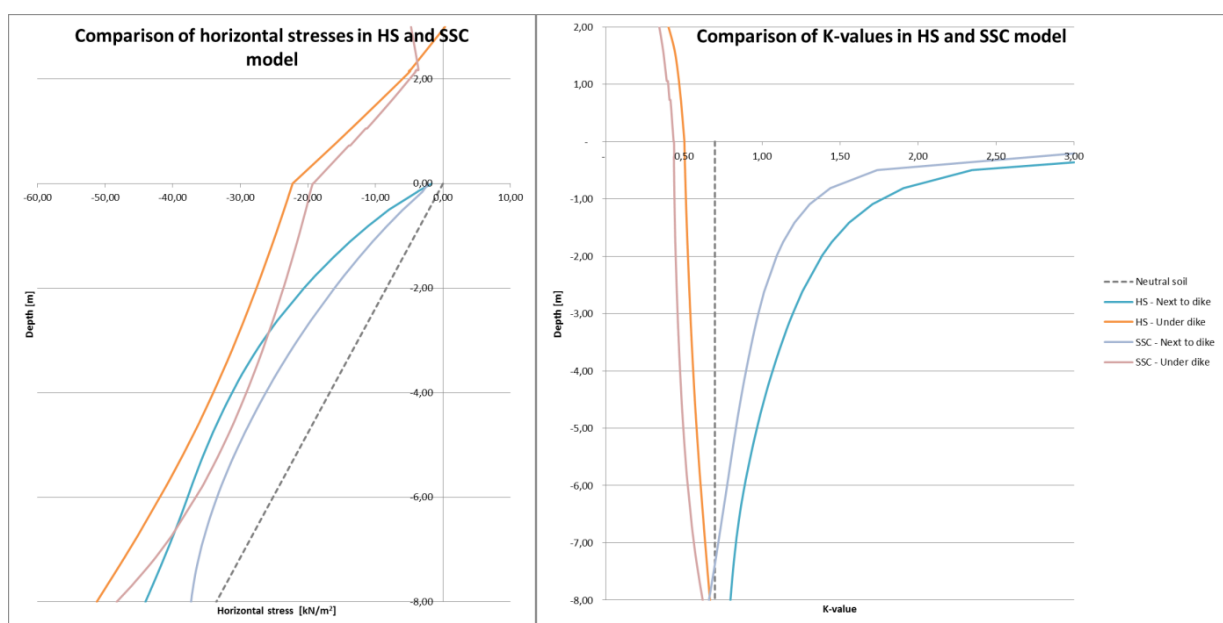


Figure 3.2 - Horizontal stresses and K-values for the HS and SSC model (normally consolidated)

### 3.2.2 Soil behaviour in the SSC model

Before continuing to the model boundaries and profile calculations, with the test module in PLAXIS the soil behaviour of the SSC model is studied, which is elaborated in Appendix III. In this case the focus was on the horizontal and vertical stresses. The influence of the friction angle mainly appears as a consequence of its influence on  $K_0$  following Jaky (1944). However it also governs when the behaviour becomes plastic. For a material with cohesion the effect in case of small stresses is found to be high. Due to the cohesion the material follows a stress path that is parallel to the  $K_0$  line, but with a higher vertical stress. Especially at the start of loading this may lead to vertical stresses in the model while the horizontal stresses are still practically zero. This is not in line with reality and therefore results in situations with low stresses with respect to the cohesion should be viewed critically.

It was found that the ratio between horizontal and vertical stress is largely influenced by the initial stress conditions because the slope of  $\Delta^*s_u (= (\sigma'_{yy} - \sigma'_{xx})/2)$  plotted against increasing pressure is different for loading and reloading. This is logical because the stress path in a normally consolidated situation follows the  $K_0$  line, while the path for unloading/reloading is governed by  $v'_{ur}$  following formula (3.1).

$$\frac{v'_{ur}}{1-v'_{ur}} = \frac{\Delta\sigma'_{xx}}{\Delta\sigma'_{yy}} \quad (3.1)$$

This can be seen clearly if a oedometer test is simulated with a unloading/reloading phase as shown in Figure 3.3. If a preconsolidation pressure is applied or a soil is unloaded until zero-stress condition the behaviour of the soil becomes unrealistic. Appendix III shows a comparison between the stress paths in the SSC model and in reality. It is concluded that for a preconsolidated soil that is unloaded to an extend that the lower failure boundary is reached and the horizontal stresses decrease faster, the resulting stress path becomes unreliable. This adds to the earlier mentioned unrealistic influence of cohesion for low stresses. Therefore in this research, no reliable statements can be made about zones with low stresses.

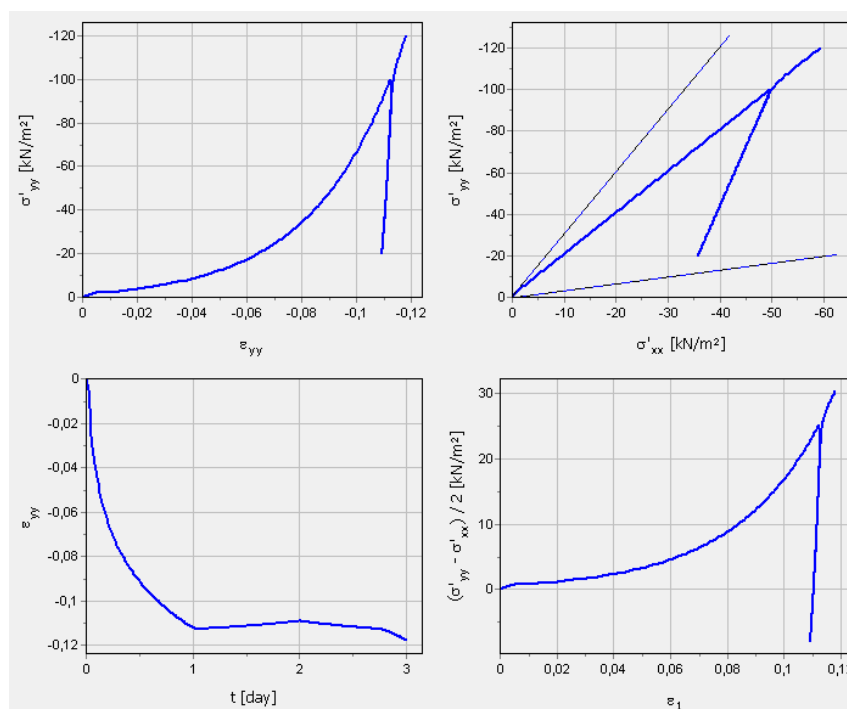


Figure 3.3 – Oedometer test with a unloading/reloading phase

Further in Appendix III it is stated that the stiffness parameters  $\lambda^*$  and  $\kappa^*$  do not influence the stress paths, while it must be noted that in a 2D mesh with various material the stiffness may influence the stress distributions.

Following the influence of creep is investigated as shown in Appendix III. It is concluded that creep does not influence the stress state in the normally consolidated situation. From literature, presented in Appendix I, it is known that creep is a similar process as the relaxation of the soil. In the test module of PLAXIS it is found that the creep parameter also governs the relaxation of the soil. As shown in Figure 3.4 the horizontal stresses decrease faster during unloading and horizontal stresses dissipate if a waiting time is added before reloading, this is in line with theories found in literature. However it also raises questions about the stress state in over consolidated soils. While on one hand the OCR can be seen increasing over time, on the other hand the horizontal stresses seem to dissipate. This effect is also visible in the test module if a waiting time is implemented before loading. In this case the stresses remain zero, but the soil behaves over consolidated from the beginning of the loading. The effect this may have on over consolidated soils is kept in mind. Further elaboration on this effect lies out of the scope of this thesis.

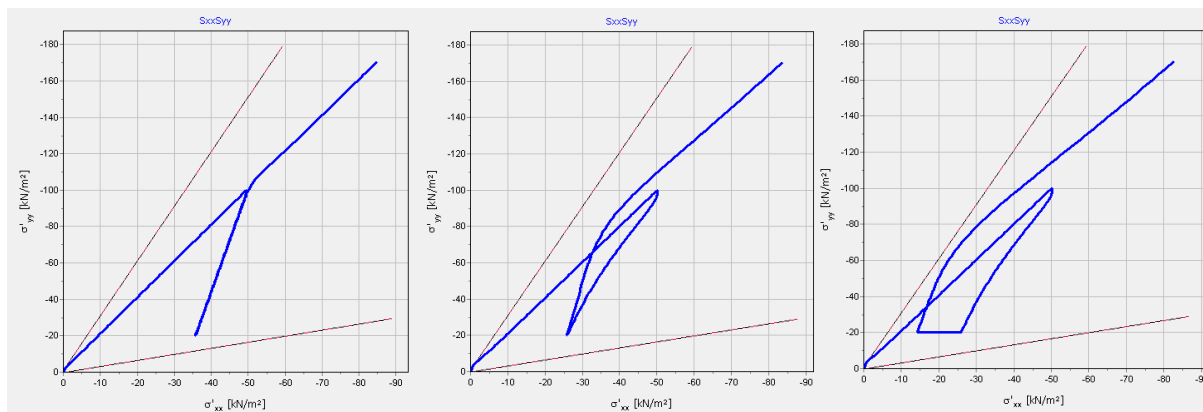


Figure 3.4 - Stress state in the oedometer test for a soil with low (left) and high (middle, right) creep index and waiting time before reloading (right)

It is concluded that the main influences on the stress distributions in a one dimensional test are the  $K_0$  in case of normally consolidated soil and  $v'_{ur}$  in case of unloading and reloading. While  $K_0^{nc}$  can be defined following the friction angle, it should be taken into consideration to manually import the  $K_0^{nc}$  value. Further it is found that the model becomes unrealistic regarding the effects of cohesion and preconsolidation in case of low stresses. Therefore in areas where the stresses are low the results of SSC-FEM calculations should be considered unreliable. Further research lies out of the scope of this thesis, but additional research could compare the results of the test module concerning horizontal and vertical stresses with laboratory tests.

### 3.2.3 Model boundaries

When using the Soft Soil Creep model it is important to define the boundaries of the model needed to prevent them from influencing the results. For the comparison analysis a model with a depth of 8 meter is used, but as can be seen in some primary calculations concerning the model choice, the influence of the dike exceeds that depth. Therefore symmetry, width, depth and element mesh are discussed and constraints are given.

#### Symmetry

It is assumed that a symmetric dike can be modelled as done in the previous paragraphs. In order to substantiate this assumption the analysis is conducted for a simple geometry. Figure 3.5 shows the assumption is correct, other images provided in Appendix IV neither show any differences which could lead to doubt the assumption.

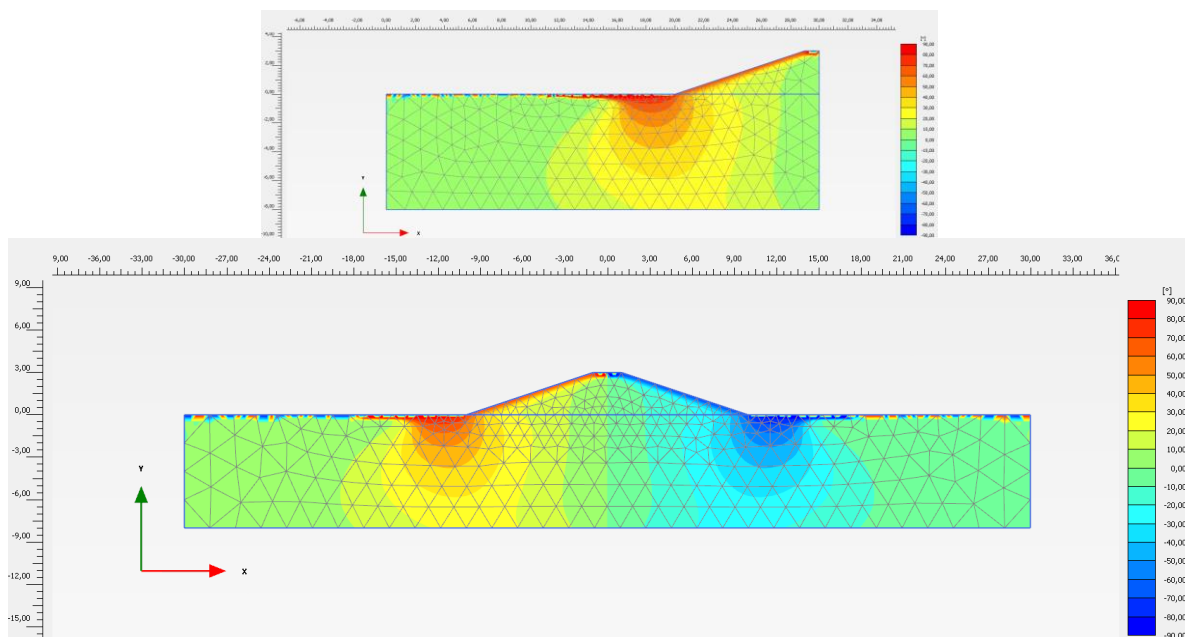


Figure 3.5 - Symmetry of the principle stress directions in the SSC model for DG2

#### Model depth and length

It is assumed that the strength of the Pleistocene layer will surpass the strength of the parent layers. Therefore the lower boundary of the model can be placed the start of the Pleistocene layer in practice. In Appendix IV is it shown that this assumption is correct. In order to look at the influence of the model length two dike geometries are used to see how far the influence reaches. Firstly the influence of the depth of the model is investigated. It is concluded that the depth has an effect, but for the model boundaries it does not play an important role. Following the width of the model is varied and resulting it is found that a model length of 10 times the dike height or 2,5 times the half-dike width plus the dike width itself is a good outer range for the boundary. Because of the relative short calculation times a more exact determination of the right boundary limits is unnecessary.

#### Element Mesh

In earlier calculations the time needed to perform a calculation was very limited. Therefore the 'fine mesh' option was chosen in PLAXIS continuously. Following there were no results which made a finer mesh necessary. Therefore in the resulting analysis the 'fine mesh' function in PLAXIS is used to generate meshes.

### 3.2.4 Influence of model parameters

Following the model boundaries the influence of the various model parameters is investigated. In this way insight is given in the sensitivity of the water level, input coefficient of lateral stress ( $K_0^{nc}$ ) and variation of OCR. The influence of a variation in these parameters may lead to more or less attention on them during further calculations. The influence of other model parameters such as soil stiffness and strength are discussed earlier and it is assumed they have a relative small effect. In this report a small summary on the findings is given, a more broad description can be found in chapter 3 of Appendix IV.

#### Consolidation and creep time

It is found that the consolidation and creep have a large influence on the stress distribution within the soil. In Figure 3.6 the situation after plastic loading, during consolidation and after a long period of creep are shown. It is concluded that for practical applications the situation after creep relaxation is representative. With a low permeability and low creep index the process still is mostly finished within 25 years, which is the case for dikes in practice. Therefore in the following calculations the situation after 50 years is taken normative. However, with young dikes this effect should be taken into account.

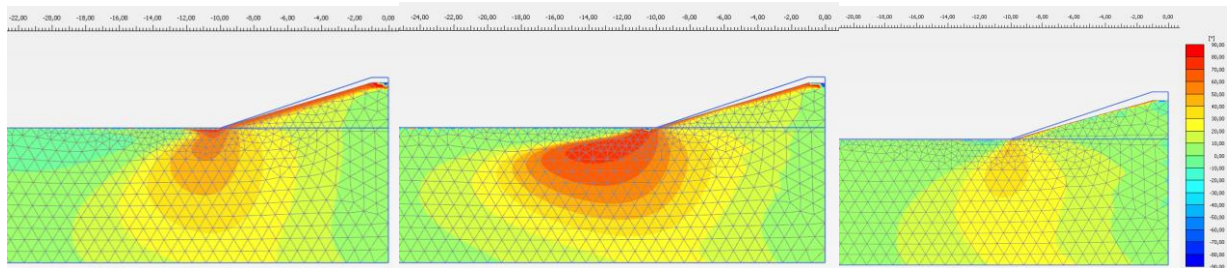


Figure 3.6 – Stress direction in the soil after plastic loading, during consolidation and after relaxation.

Additional it is found that for a unloaded soil, after 50 years the time has an effect on the initial stress ratio ( $K_i$ ) of the soil. In this situation the soil will undergo relaxation after which the horizontal stresses will dissipate. Figure 3.7 shows the K-values over depth for a unloaded soil with a OCR of 1,1. It can be seen that the K-value after creep becomes very low in the top layers where relaxation is most present. Therefore the unloaded 'initial' situation (U) as a comparison measure to the situation with dike load, is taken as the situation where consolidation and creep have occurred. These values are presented in grey in Figure 3.8, 3.9 and 3.10. Further, if present, the cohesion will have a unrealistic influence in the upper layer of the soil, leading to an effect over the total depth, therefore in the numerical analysis a low  $c'_{ref}$  value of 0,2 [kN/m<sup>2</sup>] is used.

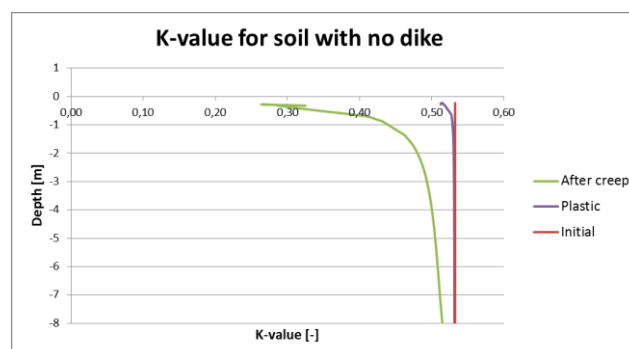


Figure 3.7 – K-value for a unloaded soil, with OCR 1,1,  $K_0^{nc}$  of 0,5 and water level of 0,0 [m] after different calculation phases

### Influence of water level

The water level influences the way the load is distributed over the soil. Therefore for different water levels the K-value changes. As shown in Appendix IV it is found that even in a unloaded horizontal situation (U) the stress ratio changes slightly if the water level is changed. This is due to the degree of relaxation. In general, for a unloaded soil, it can be said that the lower the water level, the bigger the deviation from the initial state. After a dike loading is applied as shown in Figure 3.8, it is found that the biggest effect is found for a submerged soil. This can be explained by the fact that for a submerged soil the effective stresses are lower and therefore the ratio is influenced more by higher horizontal stresses.

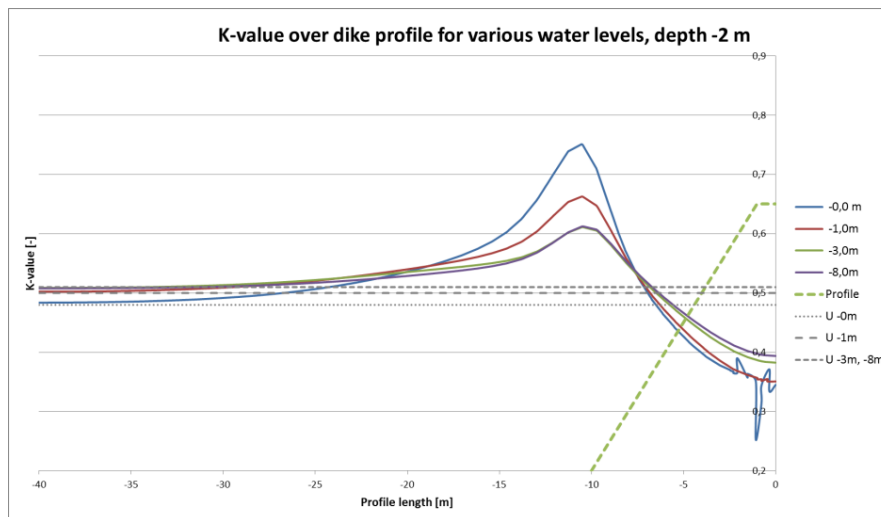


Figure 3.8 – K-value at depth -2 [m] over dike profile for various water levels with

If the water level changes during the consolidation and creep phase it could have an effect on the stress state. Investigation of this process however is discarded due to limited time.

### Influence of $K_0^{nc}$

As for the water level, the influence of a different stress ratio ( $K_0^{nc}$ ) is investigated. In the unloaded horizontal situation it was found that the influence of creep is similar for each  $K_0^{nc}$ . Following the effect of the dike loading is shown in Figure 3.9, which also shows the K-value at -2 m for a unloaded soil after 50 years. It can be seen that the behaviour is mostly the same for each  $K_0^{nc}$ , but the differences decrease under the dike, because plastic behaviour takes place there. Concluding it can be said that the initial normally consolidated stress ratio has a large influence on the K-value itself, but the variation does not have a very big impact on how the soil behaves under loading.

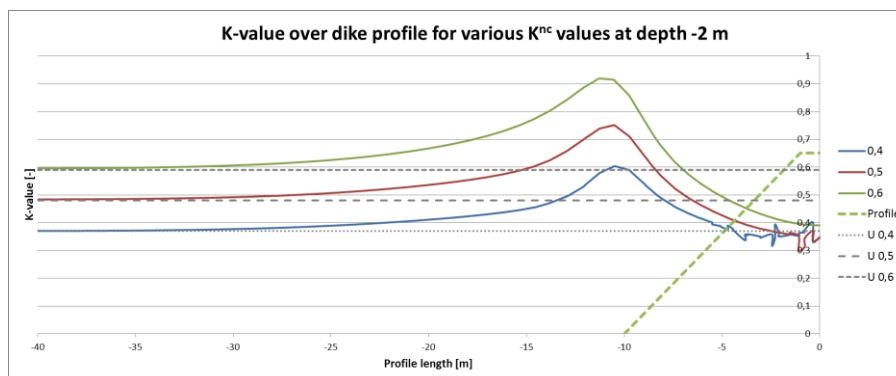


Figure 3.9 – K-value at depth -3 [m] over dike profile for various  $K_0^{nc}$  values



### Influence of over consolidation

The last model parameter discussed in this paragraph is the Over Consolidation Ratio (OCR) or Pre Overburden Pressure (POP). Like the water level and stress ratio firstly the unloaded situation after 50 years was evaluated. As shown in Appendix IV it is found that in case of a slightly over consolidated soil due to the relaxation in the SSC model the horizontal stresses dissipate over time, so the resulting K-value is lower than in the initial situation. However as the soil is highly over consolidated (OCR of 2,0 [-] or 3,0 [-] or POP of 30 [kN/m<sup>2</sup>]) the higher stiffness prevents the relaxation of the soil. Additional PLAXIS soil tests show the same behaviour in oedometer tests. Therefore in these situations the creep index and the time over which it is calculated influence the K-values. For the case study it is important to take this into account.

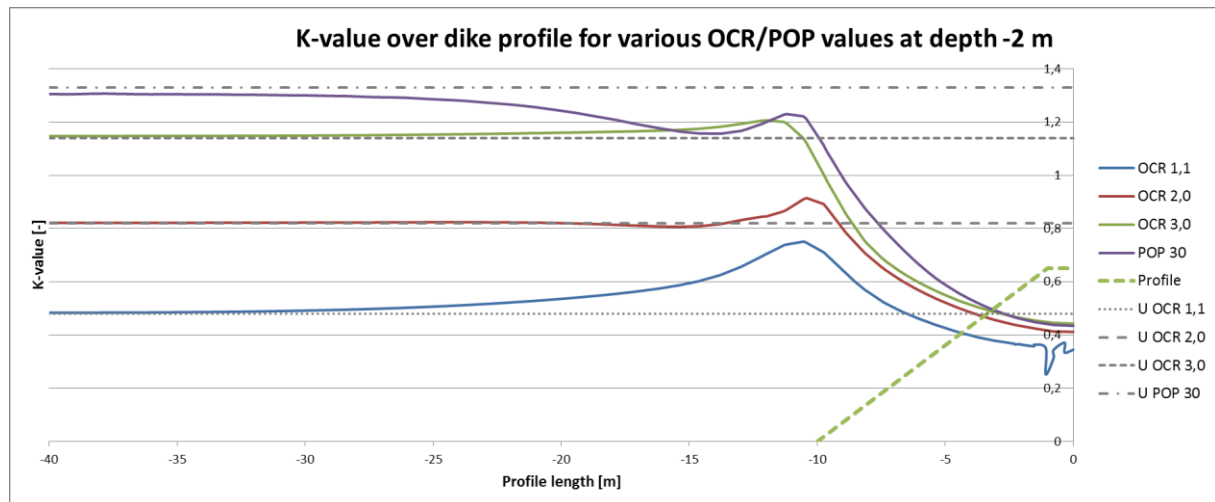


Figure 3.10 - K-value at -2 [m] depth for various OCR/POP values

Figure 3.10 shows the stress state with a dike load. It can be seen that the K-value does not increase next to the dike in case of largely over consolidated soil. This is a logical response to the already high horizontal stresses next to the dike. Under the dike however it is found that the difference between the various OCR's becomes very small. Like observed for the varied  $K_0^{nc}$  values this is due to the large deformations and therefore the plastic behaviour of the soil.

### Conclusion

The model parameters discussed can have a large effect on K in most of the situations. The calculations in Appendix IV give insight in the way their influence is present. In general it can be said that the influence next to the dike is large for the discussed parameters, while under the dike the K-value is mostly depending on the basic soil parameters because under the load it behaves plastic. During the case study it is important to watch the influence of the following:

- The stress state in soils with a low permeability and small creep index may still be changing after 50 years, this is also the case for highly over consolidated soils;
- Apart from the dike itself, the unloaded soil state is influenced by the discussed parameters;
- Combinations of model parameters that are not discussed in this paragraph, for example low water level with high OCR.

### 3.3 Theoretical effect of OCR and stress state

Following the first calculations, a more precise estimation of the influence of the initial stresses can be made. Also some definitions are needed in order to keep a clear sight on the topic. Therefore in the current paragraph a theoretical approach to  $N_c$  is stated which uses the knowledge from literature and previous calculations in order to show the effect of the initial stresses.

#### 3.3.1 Approach and definitions

In order to see the relative influence of the horizontal stresses on the cone factor in literature the  $\Delta$ -factor is used. If formula's (2.3), (2.22) and (2.26) are combined the following is found.

$$\frac{q_t - \sigma'_{v0}}{s_u} = N_{soil} - \frac{1,9}{2} \frac{\sigma'_{v0} - \sigma'_{h0}}{s_u} \quad (3.2)$$

For the preliminary calculations this formula is simplified by  $1,9/2 \approx 1$ , which rewritten results in (3.3). Which adds to the conclusion of Teh and Houlsby (1991) that the description of the cone factor would be better using the horizontal stress, in their approach  $N_{soil}$  is mentioned as  $N_h$ .

$$N_{soil} * s_u = q_t - \sigma'_{h0} = q_{net,h} \quad (3.3)$$

The value for  $q_{net}$  in formula (3.3) is corrected for the horizontal stresses and not for the vertical stresses. Therefore a different definition is used for the net cone resistance corrected for the vertical initial stresses ( $q_{net,v}$ ) and the net cone resistance corrected for the horizontal initial stresses ( $q_{net,h}$ ). Because the horizontal stress is found to be lower than the vertical stress in almost all cases, some general statements can be made about the effect of this approach. Especially when it is included that horizontal stresses are relatively high next to the dike and low under the dike, while under the dike the shear strength and cone resistance are mostly found higher. Because  $\sigma'_{h0}$  is expected lower than  $\sigma'_{v0}$ , the value of  $q_{net,h}$  will be between the  $q_t$  and  $q_{net,v}$  value. In this case it is expected that for high values, as found under the dike, the  $q_{net,h}$  will be closer to the  $q_t$  value due to relatively low horizontal stresses, while next to the dike it is closer to  $q_{net,v}$  as shown in Figure 3.11. In the toe of the dike,  $q_{net,h}$  may even become lower than  $q_{net,v}$  because the horizontal stresses can be higher than the vertical stresses. This results in the conclusion that the relatively lower values for  $s_u$  for high cone resistance as mentioned in section 1.3 cannot be explained by including the stress state, in contrary, the values would be expected relatively high under the dike due to low horizontal stresses.

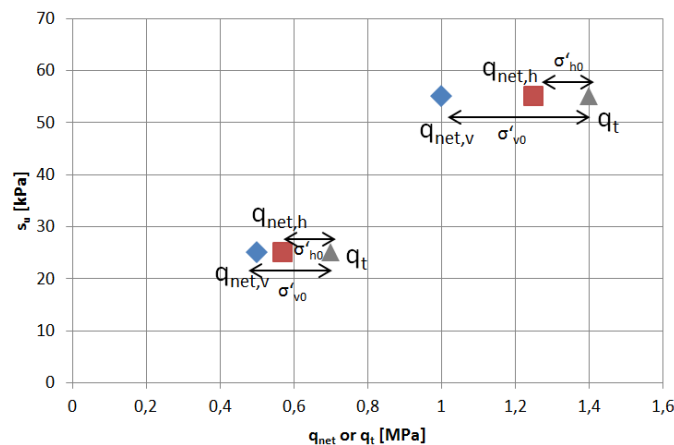


Figure 3.11 – Theoretical effect including horizontal stresses instead of vertical stresses

Through (2.22) and (3.2) the difference between the cone factor in practice ( $N_c$ ) and the cone factor independent from local stresses ( $N_{soil}$ ) can be described as follows.

$$-2\Delta = \frac{\sigma'_{v0} - \sigma'_{h0}}{s_u} = N_c - N_{soil} \quad (3.4)$$

The difference between current practice and using the horizontal stress can therefore be given by the effective stress difference divided by the undrained shear strength. However  $s_u$  is unknown in this case, and therefore it is not possible to give exact values. In order to give a reasonable estimation, the  $s_u$  is assumed to be approximated by (3.5) for the first approach (De Bruijn, Visschedijk, & Van der Ham, 2014). Subsequently the difference will be defined as  $N_\Delta$  following (3.6). In this case  $\beta$  is introduced as a constant factor which varies between 0.5 for peats and 1 for clays.

$$s_u \approx \beta \tan(\varphi') \sigma'_{v0} * OCR^m \quad (3.5)$$

$$N_c - N_{soil} = N_\Delta \approx -\frac{\sigma'_{v0} - \sigma'_{h0}}{\beta \sigma'_{v0} \tan \varphi' * OCR^m} = -\frac{(1-K_i)}{\beta \tan \varphi' * OCR^m} \quad (3.6)$$

In this formula the  $N_c$  is defined as the cone factor,  $N_\Delta$  as the stress dependent part and  $N_{soil}$  as the soil specific value part.  $K_i$  is defined as the coefficient of initial soil stress, including stresses due to the dike load. The value of  $m$  is defined as the strength increase exponent (Rijkswaterstaat, 2016). The previous parameter gives a measure for the influence of the horizontal stresses, however this is only based on an estimation of  $s_u$  and therefore only gives a estimation of  $N_\Delta$ .

Under a horizontal soil surface without load the  $N_\Delta$  can be calculated easily using equation (3.7), which leads to (3.8) and suggest that  $N_\Delta$  is independent of depth. In reality OCR is varying with depth and  $s_u$  is not directly linear to depth, so this is only a general approximation. It must be noted that in some literature  $\alpha$  is also written as  $m$ , in this case  $\alpha$  is used to prevent confusion with the other  $m$  parameter.

$$K_0^{oc} = K_0^{nc} * OCR^\alpha = (1 - \sin(\varphi')) * OCR^\alpha \quad (3.7)$$

$$N_\Delta \approx -\frac{(1-K_i)}{\beta \tan \varphi' * OCR^m} \approx -\frac{(1-(1-\sin(\varphi')) * OCR^\alpha)}{\beta \tan \varphi' * OCR^m} \quad (3.8)$$

As stated in the literature study, the effect of changing  $I_r$  and  $\alpha_c$  following a change in  $s_u$  are not taken into account in this way because these are assumed constant for the same soil layer. If however this is done the same way as for  $N_\Delta$ , the complete formula for  $N_c$  in this case becomes (3.9).

$$N_c \approx 3,4 + 1,6 \ln \frac{G}{\beta \sigma'_{v0} \tan \varphi' * OCR^m} + 1,3 \left( \frac{a}{\beta \sigma'_{v0} \tan \varphi'} \right) - \frac{(1-K_i)}{\beta \tan \varphi' * OCR^m} \quad (3.9)$$

In this way the expected N-value can be calculated under a horizontal soil surface without load. As mentioned in the literature study, the rigidity index ( $I_r = G/s_u$ ) is expected constant, in this way it has no effect on the variability of  $N_c$  and therefore a general value of 100 [-] can be chosen for this analysis (Ceccato, 2015). The value of 'a' governs the effect of the cone roughness and is also depending on the vertical stress. Therefore  $\alpha_c$  is not varied and taken 0,5 [-] as an average value (Ceccato, 2015). Further m is defined by the compression and swelling index, which for a clay or peat is around 0,8 [-] (De Bruijn, Visschedijk, & Van der Ham, 2014). The variable values become  $\phi'$  and OCR, where  $K_0$  can be added as an independent variable for non-horizontal surfaces or calculated following for a horizontal surface (3.7). Therefore a preliminary test value of  $N_{c,p}$  can be defined by (3.10). Which suggests a value of 11,4 [-] for  $N_{soil}$ . In Dutch practice a higher value of  $N_{soil}$  would be expected, and a therefore higher values for  $I_r$  and/or  $\alpha_c$ , but as this part is assumed constant and has no further influence, a value of 11,4 [-] is used in this theoretical approach.

$$N_{c,p} = 3,4 + 1,6 \ln(100) + 1,3(0,5) - \frac{(1-K_i)}{\beta \tan \varphi' * OCR^m} = 11,4 + N_\Delta \quad (3.10)$$

### 3.3.2 Theoretical effect

If the  $N_\Delta$  part of equation (3.10) is independent of depth, because OCR is kept constant with depth, the whole  $N_c$  value should be constant with depth. When the OCR is calculated based on a POP the  $N_\Delta$  varies with depth, this still has a relevant effect at a depth of around 5 meters. Further, if the  $\beta$  is taken 0,5 the  $s_u$  value becomes half as big and the effect of  $N_\Delta$  on  $N_c$  is doubled.

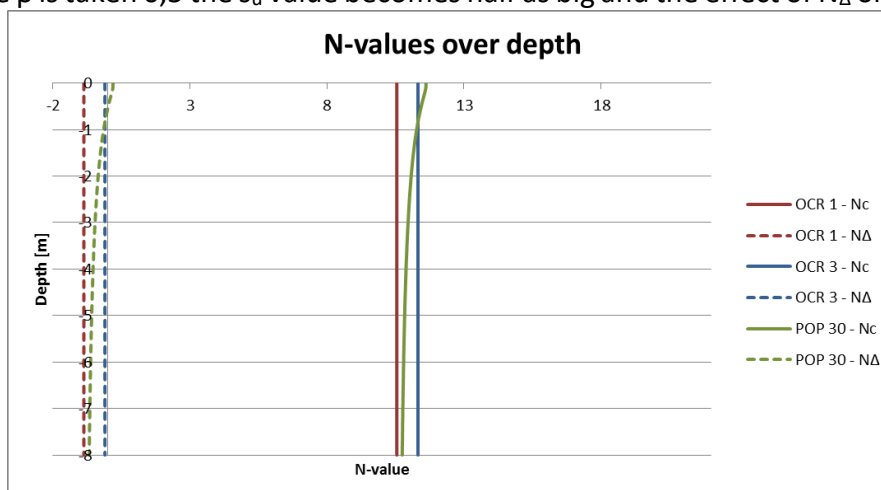


Figure 3.12 – Theoretical N-values over depth for different OCR and POP values

In case of a non-horizontal surface the K-values under and next to the dike are influenced as discussed earlier. If the  $K_i$  found in paragraph 3.2.4 are used as input in equation (3.10) the expected  $N_c$  at a depth of 2 meter can be shown. However under the dike the over consolidation is not applicable anymore because a load is applied. Therefore the OCR is calculated based on the current stresses, which leads to an OCR of 1,0 under the dike, because the dike load surpasses the over consolidation. In Figure 3.13 the results are shown for a basic dike with OCR 1,1 and the same geometry with an OCR of 3,0. A result using  $\beta = 0,5$  is added to show the effect of this parameter in the theoretical approach.

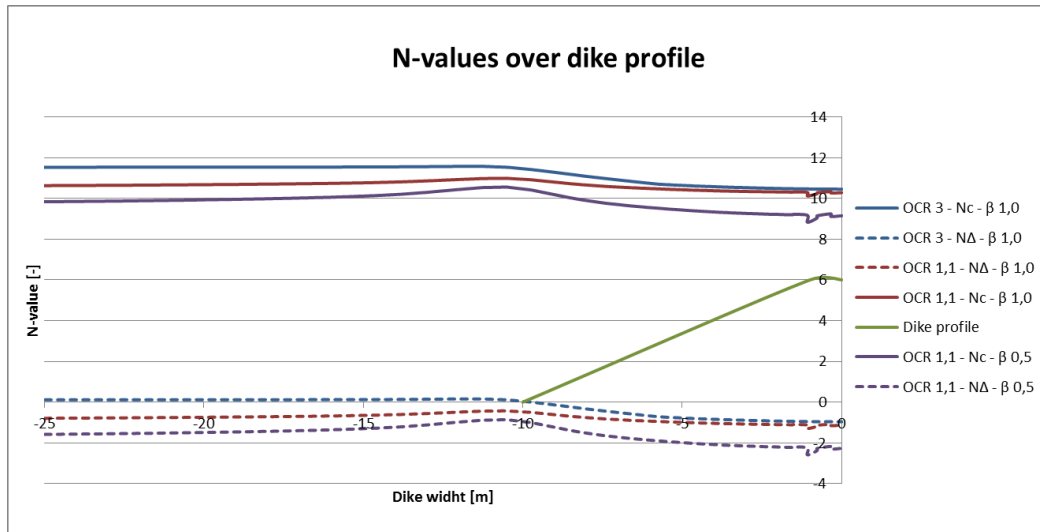


Figure 3.13 – Theoretical  $N$ -values at depth -2 m over a dike profile with and without OCR based on FEM

It can be seen that the theoretical approach leads to a  $N_c$  value that is 0,5 [-] higher next to the dike without over consolidation up to 1,0 [-] in case of a lower  $\beta$  and 1,5 for a higher OCR, however the calculated  $K_i$  value is for a clay soil and therefore the  $\beta$  should be typically 1,0 [-]. This is in line with the stated hypothesis from literature. Looking at the  $N_\Delta$  it can be seen that the lowest values occur just next to the dike, in line with the highest found  $K$ -values. Therefore the largest differences are expected between under the dike and in the toe of the dike.

### 3.4 Stress distributions under a dike

Now that most model parameters are discussed qualitatively, and a theoretical approach with some basic values is given, calculations are conducted in order to find the expected  $N_{\Delta}$  values under different dikes. In the calculations the dike profile is varied in slope and height. Very soft and strong soil is added in order to see the effect on the stress distribution. Appendix IV shows the results of the calculations more elaborated, in this chapter these results are summarised. In order to find  $N_{\Delta}$  in these cases, the  $s_u$  value is estimated by the shear strength ( $\tau_{max}$ ) provided by PLAXIS, which includes the stress history and current stress condition.

#### 3.4.1 Basic dike

Firstly for a basic dike the OCR and water level are varied. The range of values found for  $N_{\Delta}$  in normally consolidated situations are -2,0 to 0,0 [-] as can be seen in Figure 3.14. Without water level the values for  $N_{\Delta}$  become less extreme due to the fact that the absence of water results in higher effective stresses and therefore a lower relative difference between horizontal and vertical stresses. This effect is shown in Figure 3.15. For over consolidated soils this can become -2,0 to 1,2 as shown in Figure 3.16 and Figure 3.17. However the highest values for  $N_{\Delta}$  are found in the upper layer which are assumed unreliable in the calculations. The largest differences that can be found that are relevant are between under and next to the dike (2,0 difference in  $N_{\Delta}$ ) and a difference in water level (0,5 difference in  $N_{\Delta}$ ). The differences between the most extreme situations can give a difference in  $N_{\Delta}$  of almost 3,2, but this situation is unlikely to happen in practice. The zone that is influenced by the stress distributions at the toe of the dike is found to be around 10 meters from the toe for the basic dike of 3 meters high.

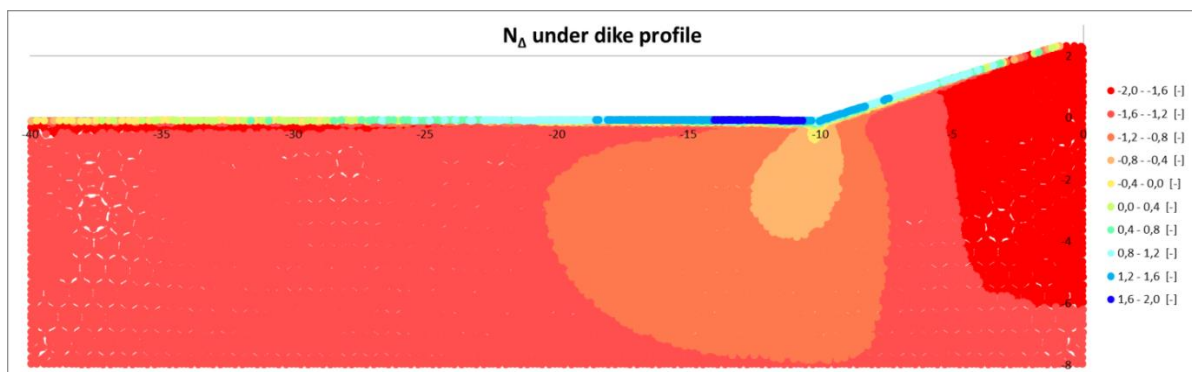


Figure 3.14 –  $N_{\Delta}$  under and next to a basic dike with OCR 1,1 and water level 0,0

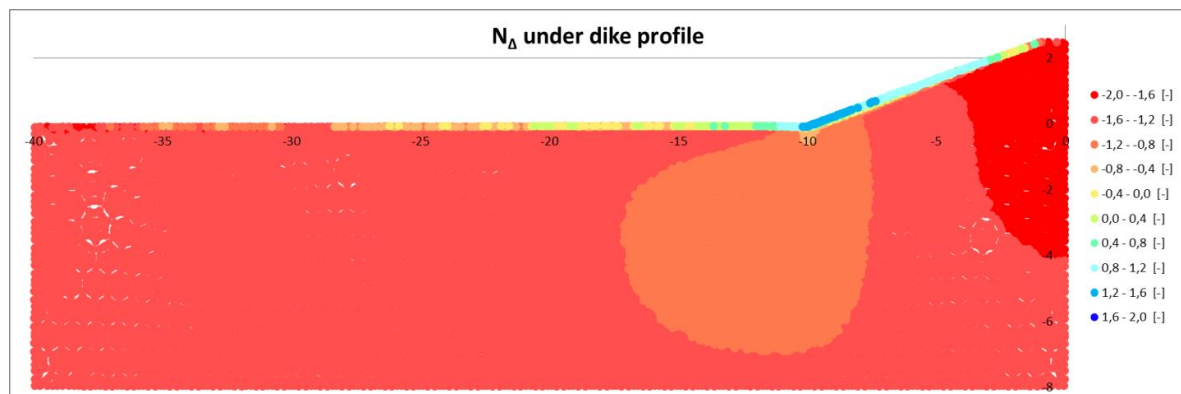


Figure 3.15 -  $N_{\Delta}$  under and next to a basic dike with OCR 1,1 and water level -8,0

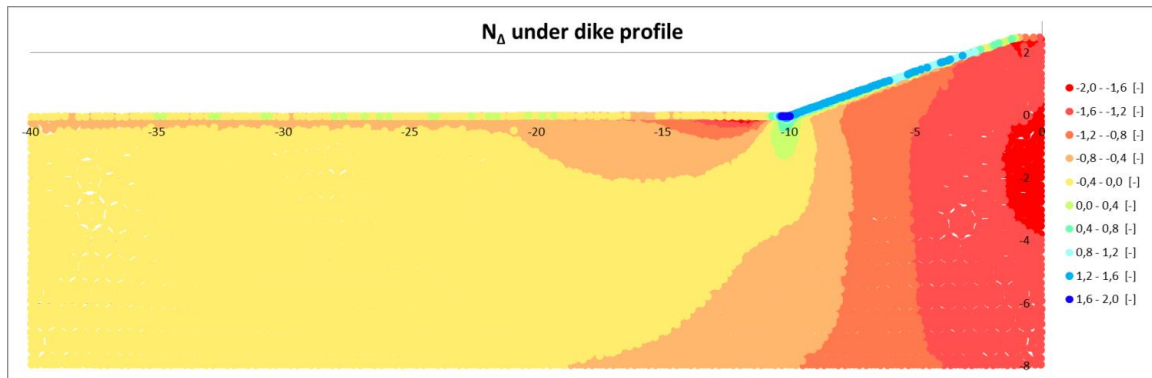


Figure 3.16 -  $N_{\Delta}$  under and next to a basic dike with OCR 2,0 and water level 0,0

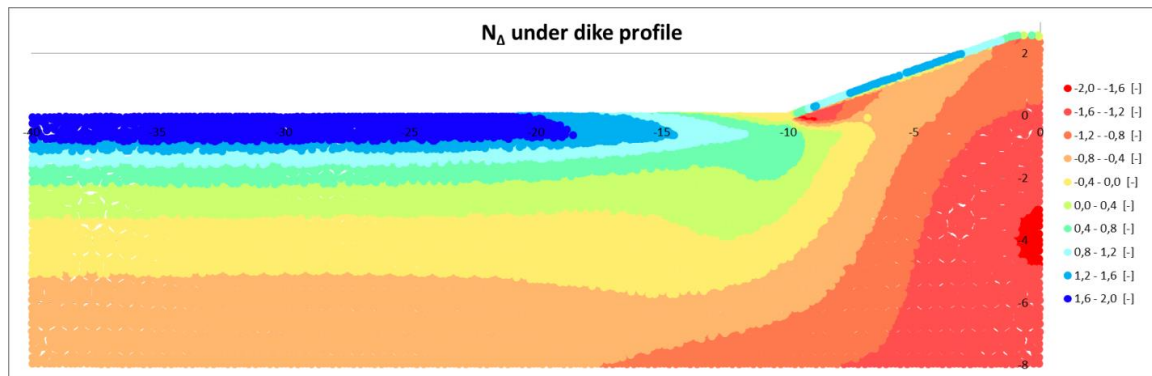


Figure 3.17 -  $N_{\Delta}$  under and next to a basic dike with POP 30 and water level 0,0

### 3.4.2 Influence dike slope, width and height

Following the influence of the slope, width and height of a dike are investigated. It is found that the slope does not have a big influence on the stress distributions. Figure 3.18 shows the  $N_{\Delta}$  values under a higher dike, showing slightly higher values at the toe. Further the zone of influence for a larger dike naturally is also larger. The extreme values however seem to be comparable with a smaller dike. This is logical due to the already maximum value under the dike.

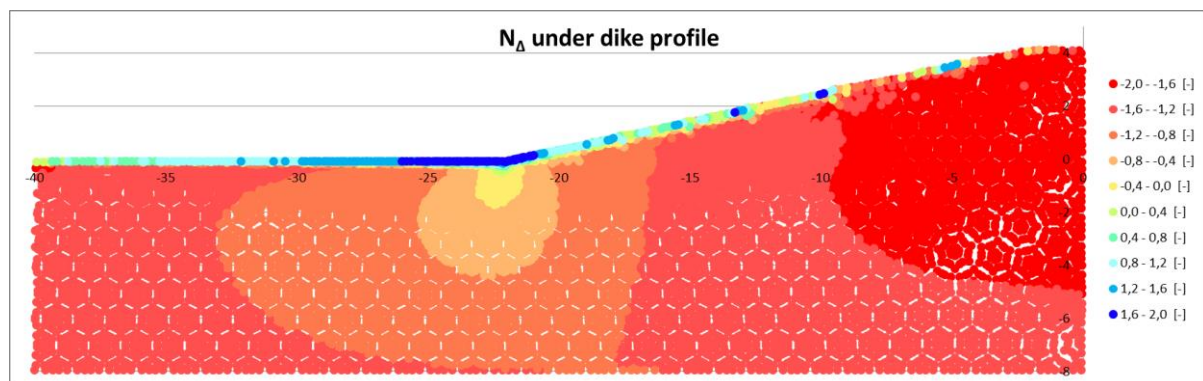


Figure 3.18 -  $N_{\Delta}$  under and next to a dike of 5 meters high with OCR 1,1 and water level 0,0

As shown in Figure 3.19 the effect of the width of the dike is mainly found under the dike itself, although the zone of influence becomes slightly bigger for a wider dike. Because the lack of horizontal support the  $N_{\Delta}$  becomes really low near to the slope, while further under the dike the  $N_{\Delta}$  becomes less extreme, in the end resulting in a neutral situation if the dike is wide enough. In an over consolidated situation this is also the case, which would result in a situation where the  $N_{\Delta}$  becomes 'neutral' again, leading to a slight increase in  $N_c$  in the middle of a wide dike in comparison with the side of the dike. A more elaborated description is given in Appendix IV.

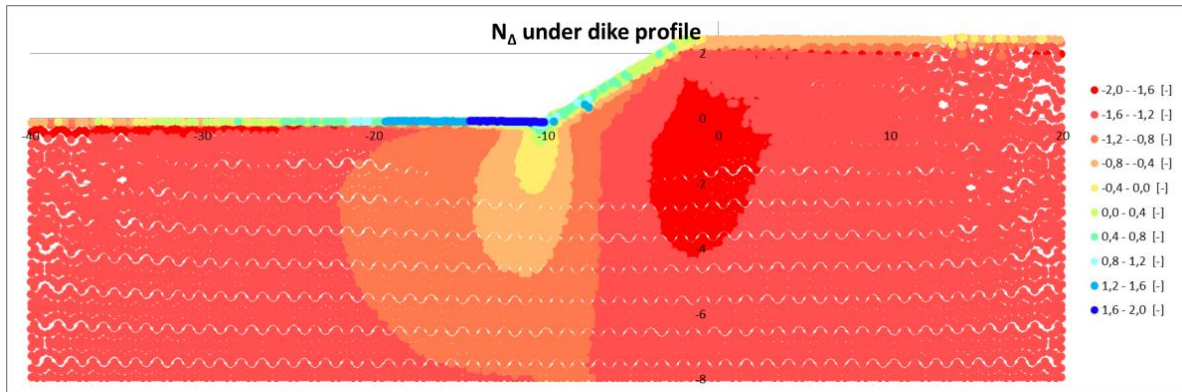


Figure 3.19 -  $N_{\Delta}$  under and next to a basic dike with large width, with OCR 1,1 and water level 0,0

### 3.4.3 Influence various layers

Finally the influence of a soft or stiff layer in the soil is investigated. The results show an expected response where the stresses are distributed in the stiffer layers. In case of a peat layer in the soil this leads to a higher  $N_{\Delta}$  in the clay just above and below the peat layer as shown in Figure 3.20. The values above the peat layer should be discarded because they are assumed unreliable. In case of a stiff sand layer the horizontal stresses become higher in the sand layer, leading to a higher  $N_{\Delta}$  in that layer as can be seen in Figure 3.21. The effect in this case becomes relatively high for the sand layer itself because of the influence reaching until 40 meters from the dike. However in practice the  $N_c$  value of a sand layer is irrelevant. The further effect of a stiff layer seems to be small, leaving the values for  $N_{\Delta}$  the same in the clay below the sand.

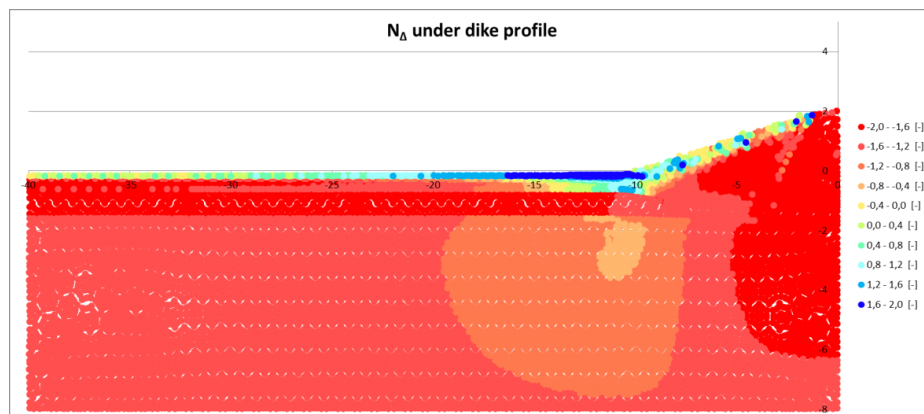


Figure 3.20 -  $N_{\Delta}$  under and next to a basic dike with a peat layer from -0,5 to -2,0

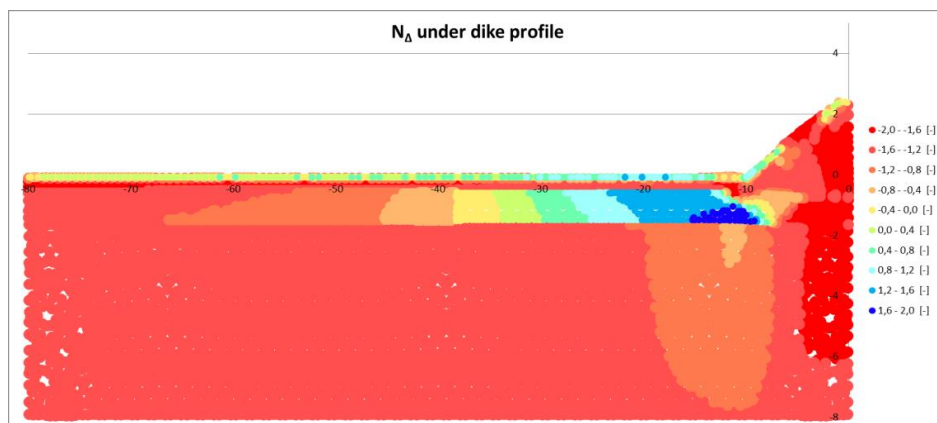


Figure 3.21 -  $N_{\Delta}$  under and next to a basic dike with a sand layer from -0,5 to -2,0



### 3.5 Conclusions numerical analysis

The preliminary calculations are conducted in order to give insight on three subjects. Firstly the model with which the calculations should be executed, secondly the workings of the model and which model parameters are important and thirdly the expected influence of horizontal stresses on the cone factor. The Soft Soil Creep model was chosen from a selection of available PLAXIS models. It is noted that due to lack of time it was not possible to conduct laboratory experiments. Therefore the choice of the model cannot be substantiated empirically. Differences between the SSC model and other considered models were present but all showed the same general soil behaviour. The effect of consolidation and creep is found to be significant and can only be taken into account using SSC. Based on literature the SSC model was preferred above other models. However it is noted that empirical data from the case study has to show a clear effect to verify this choice.

Numerous soil parameters were varied in the PLAXIS soil test module as well as 2D calculations. In this way a good understanding of the SSC model is obtained and the following statements about the soil behaviour and resulting horizontal stresses can be made.

- The rate in which the stress ratio changes differs for loading, reloading and failing of the soil, therefore the preconsolidation has a large influence on the ratio;
- The influence of stiffness parameters is relatively small;
- Due to creep relaxation in the soil, the horizontal stresses decrease over time;
- The  $K_0^{nc}$ , OCR and water level have a big influence on the stress ratio next to the dike, but a small influence under it due to the loading;
- Concerning geometry, especially the width of the dike results in a other situation under the dike;
- In case of various layers, the horizontal forces are distributed to the stiffer layers, resulting in lower horizontal stresses in soft layers.

In order to give a measure for the effect of horizontal stresses on the cone factor,  $N_\Delta$  is defined by (3.11). It gives the value of the part of the cone factor which is influenced by the stress ratio. Because a cone factor of 15-25 is common in practice, the  $N_\Delta$  can give insight on the relative influence of the horizontal stresses. A theoretical approach shows a decrease in  $N_c$  value under the dike. Which is in contrast with the relatively low  $s_u$  values observed in section 1.3. Therefore including the initial soil stresses cannot explain this observed behaviour.

$$N_\Delta = - \frac{\sigma'_{v0} - \sigma'_{h0}}{s_u} = N_c - N_{soil} \quad (3.11)$$

As a result of the preliminary calculations it is found that  $N_\Delta$  can vary between both extremes - 2,0 to 2,0 [-] in different cases. However because local parameters as OCR and water level will be comparable in practice, and the value in a sand layer is not relevant, the difference in cone factor is assumed maximum 2,0 [-] for next to and under a dike. Further differences between  $N_\Delta$  values on different locations can differ up to 0,5 [-] due to dike height, width and water level. This difference is too small to explain the total deviation of all the results presented in Figure 1.4. Especially because the differences between similar measurements cannot be explained by the small deviations in  $N_\Delta$ . As a conclusion of this chapter it is stated that the effect of horizontal stresses on the cone factor is expected to be 0-10%, which is in line with the hypothesis stated at the end of the literature study. But the effect on the variation coefficient is expected much lower due to limited differences between measurement spots.

## 4 Case study Hollandse IJsseldijk

The literature study and preliminary calculations show an influence of the horizontal stresses on the cone factor of up to 10%. However, this is not substantiated with empirical results. Therefore a case study is conducted on the Hollandse IJsseldijk. In this case Deltares used a constant cone factor to calculate the local  $s_u$  values from CPT data. The goal of the case study is to investigate if the cone factor corrected for the horizontal stresses gives a better correlation in the found data. In this chapter, firstly the approach is stated, following the situation and input parameters are described. After that, the results are presented and a conclusion is formed. The calculation and results are more widely described in Appendix V.

### 4.1 Approach

#### Gathering and interpreting data

Firstly the data available from the Deltares project is gathered. For the purpose of this research lab tests, geometries and history of the dike and available CPT's and BPT's are needed. In order to build the model in PLAXIS the  $\phi'$ ,  $c'_{ref}$ ,  $\lambda^*$ ,  $\kappa^*$ ,  $\mu^*$ ,  $K_0^{nc}$ , OCR, permeability and soil weights are needed. Further, the dike geometry is needed to accurately model the current situation. For the resulting analysis, the  $s_u$  and CPT's on calibration fields need to be known. Based on the previous analysis, it can be estimated how important the parameters are and unknowns should be filled with reasonable guesses. As a result of this phase for each calibration field the parameters, geometry, history and available penetration test are stated and ordered.

#### Modelling cross sections

With the data gathered, the cross sections can be built in PLAXIS. Each cross section should be modelled in a way the result is in line with the found geometry. Subsequently, the stress conditions in the dikes are found in the calculation results. As a result, the stress conditions at each spot where a  $s_u$  lab test is conducted are known.

#### Calculation following WBI

Firstly in order to compare the differences between the different methods, the  $N_{kt}$  is calculated based on the current WBI practice. The results of this step can be compared with the project results of Deltares. In this step the lab results are connected to the corresponding averaged  $q_{net}$  value and the least squares method is used to find  $N_c$  and its coefficient of variation.

#### Calculation including local stress situation

The horizontal and vertical stresses found in the PLAXIS calculation are added. Subsequently, all parameters in equation (3.2) are known except  $N_{soil}$ . Again, the least squares method is used in order to find one  $N_{soil}$  for all locations.

#### Comparing both methods

With  $N_c$  and  $N_{soil}$  known, both methods can be compared. If the hypothesis is correct, the standard deviation following from the calculation including horizontal stresses will be lower.

## 4.2 Input data and parameters

The data provided by Deltares contained four cross sections of the Hollandse IJsseldijk, referred to as raai 1,2,4 and 5. For each cross section, numerous CPT's are available, as shown for raai 1 in Figure 4.1. At each of the cross sections, a boring is conducted in the top of the dike and in the toe. A total of 36 triaxial tests and 16 DSS tests on peat are done. The results of these laboratory tests are provided by Deltares. Although some questions were raised, and discussed in Appendix V concerning the test results it is chosen to copy the results from Deltares rather than interpreting the laboratory results again in order to reduce the needed time. The used parameters are listed completely in Appendix V. Based on the laboratory tests Deltares provided the calculation parameters for the SSC model, such as  $\phi'$ ,  $c'_{ref}$ ,  $\lambda^*$ ,  $\kappa^*$ ,  $\mu^*$  and  $K_0^{nc}$ . In comparison with the numerical analysis, it was found that the  $c'_{ref}$  found was much higher than the 0,2 [kN/m<sup>2</sup>] used in the numerical calculations. Based on the results found in the preliminary calculations this means that the results in the upper layers should be handled with care. Further, the OCR was not defined for the cross sections. However, in the calculations it was found that the settlements were in line with a low OCR that did not exceed 1,2 [-]. Further the determination of  $N_c$  of Deltares is discussed in Appendix V. The results of this calculation are compared to the results of the own WBI calculation and the calculation including  $\sigma'_{h0}$ .

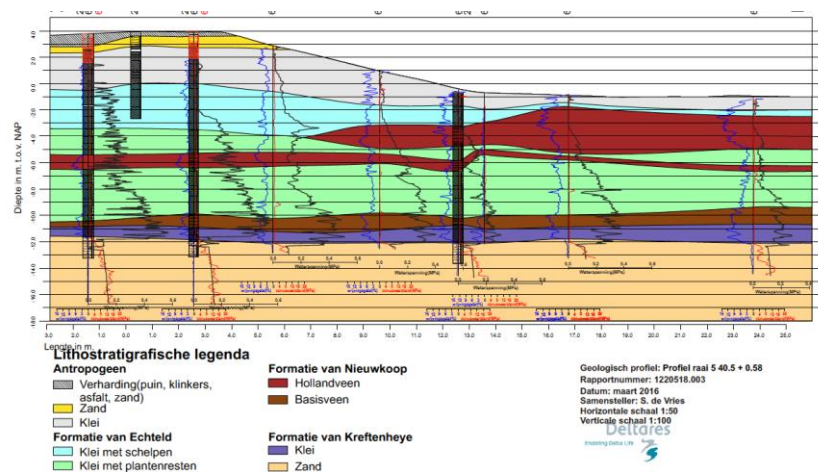


Figure 4.1 – Deltares model with CPT's and borings – raai 1

Based on the data from Deltares, PLAXIS models were made which included the available parameters and geometries. In order to find a result corresponding to the current situation, the model was iteratively changed in height and additionally in OCR. Some layer geometries, for example raai 1, are interpreted differently than Deltares. Figure 4.2 shows an input and result geometry from the PLAXIS calculation. All geometries can be found in Appendix V.

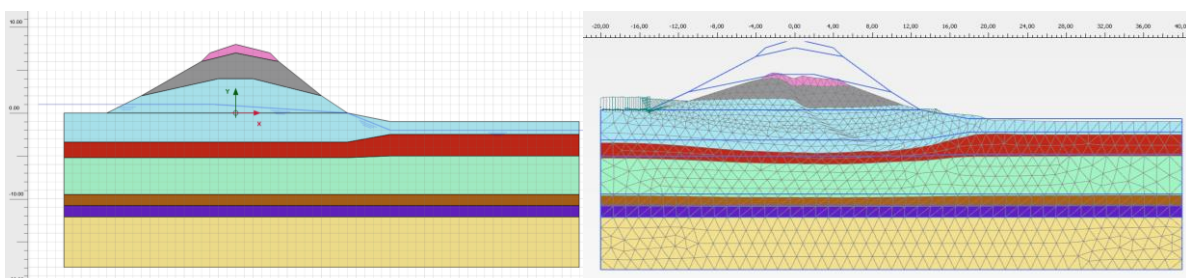


Figure 4.2 - Plaxis input (left) and result (right) corresponding with available data

### 4.3 Calculation

As mentioned in the approach firstly the calculation is executed following the WBI. In comparison with the Deltares calculation two changes are present:

- The linearization of Deltares dealt with the *absolute* variation following the ‘trend line’ function in Excel. In order to find the lowest variation coefficient the *relative* variation should be taken into account. Therefore an additional iterative Excel file is made which finds the best fit based on the relative difference following (4.1) instead of the absolute difference.
- In the Deltares calculation one CPT measure was used for each laboratory test, while an average over the sample should give a more stable result. Therefore the average of three values, corresponding to a height of 6 centimetres, is used in the WBI calculation.

The results of the WBI calculation are a more relevant comparison for the stress-state included calculation.

$$VC_{Nc} = \sqrt{\frac{\sum_{j=1}^n \left( \frac{s_{u,i} - \frac{q_{net,i}}{N_c}}{\frac{q_{net,i}}{N_c}} \right)^2}{n-1}} \quad (4.1)$$

In the new method, which is referred to as the ‘ $\sigma'_{h0}$  method’ there is a number of ways to find the  $q_{net}$ . For the WBI method,  $q_{net}$  is defined by (4.2). As shown in (3.3) the vertical effective stress can be replaced by the horizontal effective stress in order to find the new  $q_{net,h}$  value. In order to remove the simplification  $1,9/2 = 1$ , the factor  $\delta$  is introduced. Following the literature study in Appendix I, the value for  $\delta$  should always be between 0,9 and 1. For this purpose it is kept 1 [-]. Resulting, the cone resistance corrected for horizontal stresses instead of vertical stresses can be found following (4.3).

$$q_{net,v} = (q_c + u_2(1 - a)) - \sigma'_{v0} \quad (4.2)$$

$$q_{net,h} = q_{net,v} + \delta * \Delta\sigma'_{v0-h0} \quad (4.3)$$

Where  $q_{net,v}$  is defined as the  $q_{net}$  value used in the WBI, which is corrected for the vertical stresses.  $\Delta\sigma'_{v0-h0}$  is the difference between vertical and horizontal effective stress before the start of cone penetration, which is provided by the PLAXIS calculations.

In order to give insight in the resulting influence of changing the variation coefficient the reduction factor is calculated. Following the WBI the  $s_u$  should be divided by this reduction factor in order to find a safe characteristic value for  $s_u$ , and therefore it give as measure of the improvement that is reached. The equations for this calculation given in the WBI can be found in Appendix V.

## 4.4 Results

The results presented in Appendix V can be summarised by Figure 4.3 and Figure 4.4. In these figures the laboratory results are plotted against the  $q_{net}$ . In case of the WBI calculation (red), this is done with respect to  $q_{net,v}$ , while for the  $\sigma'_{ho}$  method  $q_{net,h}$  is taken. Further the linearized correlation based on  $N_c$  or  $N_{soil}$  is shown as well as the 5% SD interval.

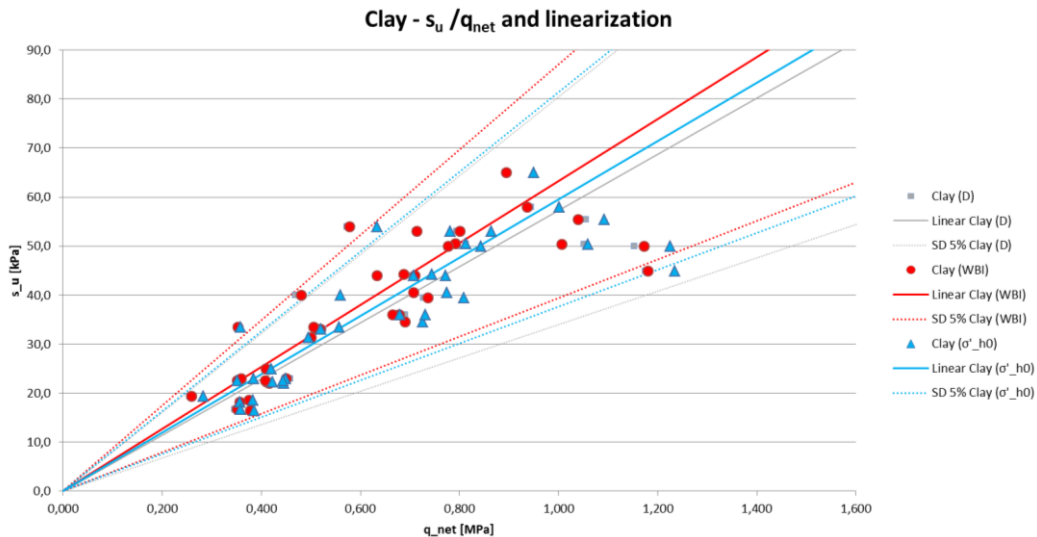


Figure 4.3 – Laboratory results  $s_u$  and  $q_{net,v}/q_{net,h}$  with linearization and 5% standard deviation

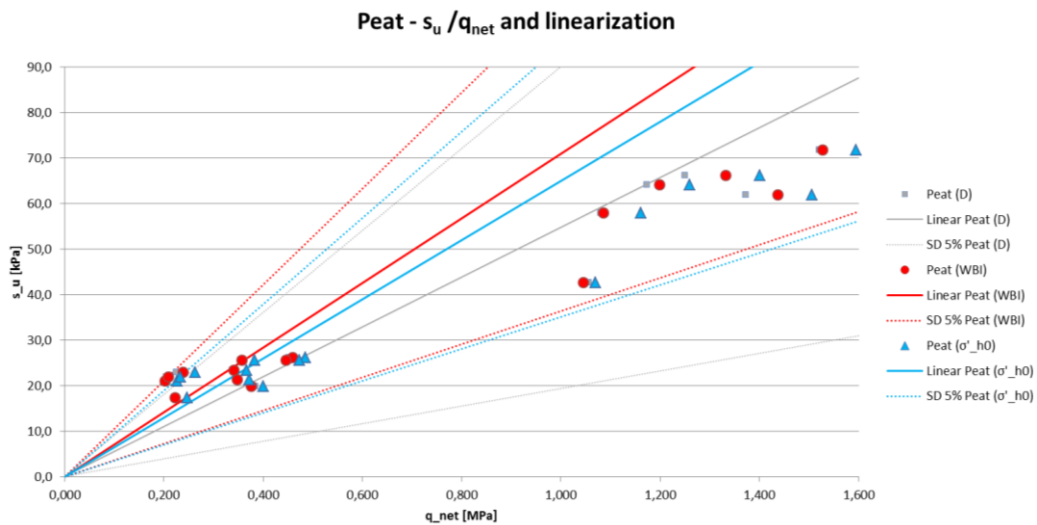


Figure 4.4 - Laboratory results  $s_u$  and  $q_{net,v}/q_{net,h}$  with linearization and 5% standard deviation

Following the linearization the  $N_c$  and  $N_{soil}$ , different VC's and difference in RF are shown in Table 4.1. The reduction factor (RF) is defined as the factor between expected and characteristic value, and is further explained in Appendix V. In this appendix an example is given using some basic values in order to show the calculation process.

	Clay			Peat		
	$N_c$ or $N_{soil}$	VC $N_c$ or $N_{soil}$	RF	$N_c$ or $N_{soil}$	VC $N_c$ or $N_{soil}$	RF
<b>Deltares (<math>N_c</math>)</b>	17,4	0,23	1,50	18,3	0,44	1,91
<b>WBI method (<math>N_c</math>)</b>	15,8	0,21	1,46	14,1	0,30	1,63
<b><math>\sigma'_{ho}</math> method (<math>N_{soil}</math>)</b>	16,8	0,20	1,44	15,4	0,28	1,58

Table 4.1 - Resulting values for  $N_c/N_{soil}$ , VC and reduction factor

## 4.5 Discussion and conclusion

Based on the found results from the case study two separate comparisons can be made. Firstly, the comparison between the Deltares calculation and the WBI calculation, with the biggest difference found in the way of finding the smallest deviation. Although this difference lies not within the scope of this research, it is important to mention the large difference in reduction factor, from 1,91 [-] to 1,63 [-] for peat and 1,50 [-] to 1,46 [-] for clay. However, especially for peat, the linearization seems to overestimate the shear strength for higher  $q_{net}$  values. This is also observed in the Markermeerdijken data as discussed in section 1.3. As mentioned earlier, including the initial stress state does not explain this observation. A method which uses two values for the cone factor, one for under and one for next to the dike, may give a more correct result. However, no theoretical base for this method is found in this research.

Secondly, the comparison between the WBI method and  $\sigma'_{h0}$  method is more relevant for this research. In line with earlier results, the  $N_{soil}$  differs maximally 2,0 [-] from the  $N_c$  value, and is 1,0 [-] higher on average. A slight improvement of variation coefficient is observed and therefore a smaller reduction factor follows from the calculation. However, the improvement is rather small in comparison with the difference found between  $q_{net,v}$  and  $q_{net,h}$ . In line with literature and preliminary calculations, the found  $N_{\Delta}$  is between 0,2 [-] and -2,0 [-]. The average value of  $N_{\Delta}$  is -0,9 [-] for clay and -1,1 [-] for peat. As a result most values shift to the right, resulting in a higher value for  $N_{soil}$  than  $N_c$ . As also shown by the example in Appendix V. If the effect of  $N_{\Delta}$  would have been maximal, the values under the averaged line should have a low negative or positive  $N_{\Delta}$  value, while the values above the averaged line should have a high negative value. This would result in a clear shift in direction of the averaged line. As can be seen in Figure 4.3 and Figure 4.4, this is not the case. In most situations, the high  $q_{net}$  values are mostly associated with a relative high  $N_{\Delta}$ , where for low values, the  $q_{net,v}$  and  $q_{net,h}$  are almost the same. In these 'groups', the deviation present is probably a result of measurement uncertainties.

The decrease in variation coefficient is so small that with this number of measurements, it is not directly possible to conclude that the effect is a result of adding the horizontal stresses. At first sight the improvement could also be the effect of just randomly changing the  $q_{net}$  values slightly. However, looking at the relative variation of the single data points as provided in Appendix V, there seems to be a clear decrease in deviation for most of the points. Therefore, it is assumed that the improvement is an effect of including the horizontal stresses. Additional research should back up this claim.

Based on the case study, the following conclusions can be drawn:

- A constant value for the cone factor overestimates the undrained soil strength for higher  $q_{net}$  values, especially in peat, this cannot be explained by including the horizontal stresses;
- The improvement of the correlation due to including the horizontal stresses is not maximal due to correlation between  $q_{net}$  value and  $N_{\Delta}$  value;
- The slight decrease in variation coefficient is assumed to be the result of taking the horizontal stresses into account;
- In practice, this small improvement leads to a 2-3% higher characteristic value of  $s_u$  as shown in Appendix V by the case study and an example calculation;
- The biggest part of the variation cannot be explained by the horizontal stresses.

## 5 Implementation possibilities

### 5.1 Calculating horizontal stresses

Following the previous results, there are numerous possibilities to implement the horizontal stress in  $N_c$  determination. Two methods are distinguished in this case:

- The ‘first guess’ WBI method, taking a predetermined value for  $N_c$  and the variation coefficient (VC). In this case a  $N_c$  value of 20 [-] and VC of 0,25 [-] are recommended by Rijkswaterstaat (2016);
- the extended WBI method, where the  $N_c$  and VC are found based on test fields and laboratory results. This method is also described by Rijkswaterstaat (2016).

In both situations, the horizontal stresses need to be known, which is not a standard defined parameter. Therefore additional measurements or calculations are needed in order to implement the  $\sigma'_{h0}$  method. Four options are taken into account.

1. Measure the horizontal stresses in the field;
2. Estimate the horizontal stresses based on a FEM or other calculation;
3. Estimate the difference between horizontal and vertical stress with a calculation;
4. Estimate the difference based on a ‘rule of thumb’.

In order to measure the horizontal stresses in the field, for example the Flat Dilatometer Test (DMT) can be used (Marchetti, Monaco, Totani, & Calabrese, 2001). Although these tests give direct empirical data, some uncertainties will arise using them. Lutenegger (1990) found very different  $K_0$  values for different test methods and states that DMT overestimates the in-situ stress. The horizontal stresses based on a FEM calculation can give an estimate of the stresses, as well as the difference between horizontal and vertical stress, as used in the case study. While these calculations can give a more general insight on the effect, they are costly and time-consuming, especially considering the relatively small effect found in the case study. Lastly an estimation can be made based on a ‘rule of thumb’ following the numerical analysis. In this case a estimation of the initial soil stress ratio ( $K_i$ ) can be made based on earlier PLAXIS calculations as shown in Figure 5.1. In this case the effect may not be maximal due to a non-specific approach, but it may be cost effective and lead to a small improvement of the results.

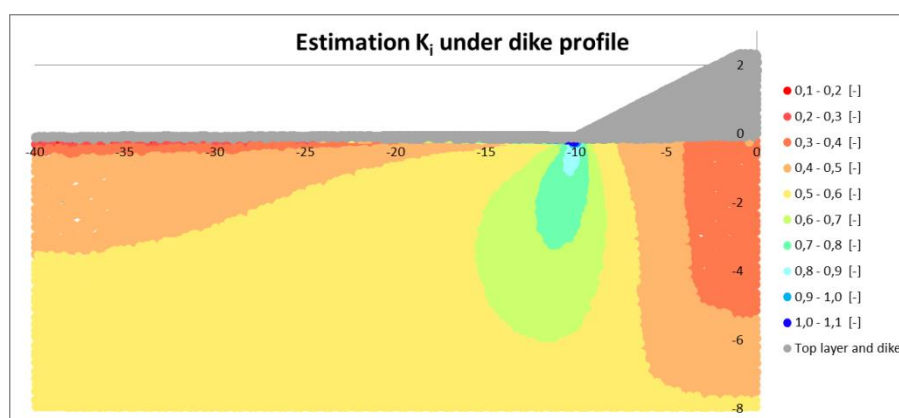


Figure 5.1 – Estimation  $K_i$  under a basic dike profile based on PLAXIS calculations

## 5.2 Best practice

A best practice is suggested based on ‘rule of thumb’ values for initial soil stress ratio ( $K_i$ ). While the exact values possibly can be improved, a first starter is given by defining the rules-of-thumb for the initial soil stress ratio ( $K_i$ ) around a dike. Which could be implemented using equation (5.1) instead of (1.2).

$$q_{net,h} = (q_c + u_2(1 - a)) - K_i * \sigma'_{v0} \quad (5.1)$$

In which  $q_{net,h}$  is the cone resistance corrected for horizontal stresses,  $q_c$  the measured cone resistance,  $u_2$  the water pressure, ‘ $a$ ’ a cone factor and  $\sigma'_{v0}$  the initial effective vertical stress.

Based on the numerical analysis and case study, the following rules-of-thumb are made:

- For all places further than 3 times dike height from the toe  $K_i = K_0^{oc}$ , the soil stresses are not significantly influenced by the load of the dike;
- For under a dike  $K_i \approx 0,25$ , the soil is deformed by the dike load and horizontal stresses are relatively low. This is at least until a depth of -6, but depends weight of the dike, deeper in the soil the stress state returns to  $K_0^{oc}$ ;
- At the toe of the dike:
  - o For relatively stiff layers (clay)  $K_i \approx 1,2 - (0,1 * \text{depth})$  until  $K_i = K_0^{oc}$ , the dike body creates high horizontal stresses at the toe, decreasing over depth;
  - o For relatively soft layers (peat)  $K_i \approx K_0^{oc}$ , while the stiffer layers prevent most of the horizontal deformations, the horizontal stress in softer layers remains almost  $K_0^{oc}$ .

Given estimations for  $K_i$  may be improved based on additional research and can also be estimated using Figure 5.1.  $K_0^{oc}$  can be found using  $K_0^{nc}$  and the OCR based on equation (2.19).

The implementation of this ‘rule of thumb’ differs for the ‘first guess’ and extended WBI method. The implementation in the extended WBI method would be similar to the case study. In that case, instead of using the found values from the PLAXIS calculation,  $K_i$  values based on the ‘rule of thumb’ are used as input. Following a value for  $N_{soil}$  and the variation coefficient are found. If these ‘rule of thumb’ values are used in the case study data, the improvement is reduced with a third, but still a small improvement of 1-2% is present. Therefore with minimal work load, some improvement is reached.

For the ‘first guess’ WBI calculation, a new predetermined  $N_c$  (in this case defined as  $N_{soil}$ ) and VC value must be found in order to implement equation (5.1). It is not possible to give a reasonable and reliable estimation of the VC and  $N_{soil}$  based on one case study. Additional projects using the extended method could give a more substantiated value for  $N_{soil}$  and the variation coefficient, but due to the small effect in practice, this is not expected to be profitable to do.



## 6 Conclusion and recommendations

### 6.1 Conclusion

The undrained shear strength ( $s_u$ ) is important information for the macro-stability analysis of a dike. The cone factor ( $N_c$ ) gives a correlation between undrained shear strength ( $s_u$ ) of the soil and the cone resistance ( $q_c$ ). In practice, the deviation of the results is high and therefore, a large reduction factor is taken into account from expected to characteristic value of  $s_u$ . Further is observed that for higher values of the cone resistance, the  $s_u$  is mostly overestimated by a linear correlation.

In literature, the cone factor is described in terms of soil stiffness, the cone roughness and the initial soil stress state ( $\Delta$ ). While soil stiffness and cone roughness can be assumed constant over a layer in a project area, the stress state can differ for each measurement of  $s_u$ . Including the initial stress state ( $\Delta$ ), governed by the horizontal and vertical effective stress, in the correlation could reduce the deviations of the results.

While the vertical effective stress is calculated relatively easily, the horizontal stress is harder to determine. Literature describes the ratio between horizontal and vertical stress with the coefficient of lateral earth pressure ( $K$ ), which in an one-dimensional soil can be measured as  $K_0$ . However, under a two- or three-dimensional loading, such as a dike, the initial coefficient of lateral earth pressure ( $K_i$ ) can vary between active pressure ( $K_a$ ) and passive pressure ( $K_p$ ) and is not easy to estimate analytically. Based on literature it is found that the maximal and minimal values for  $K_i$  could lead to a difference in  $N_c$  of 10%.

With FEM calculations using the Soft Soil-creep Model (SSC), the stress state under a dike is described. It is found that in the toe of the dike  $K_i$  shows a peak up to a  $K_i$  value of 1,5 [-] followed by a minimum  $K_i$  value under the dike of 0,25 [-]. This results in an estimated difference in  $N_c$  value of 2,0 [-] in practice. Geometry,  $K_0^{nc}$  and OCR are found to have a clear effect on the horizontal stresses. In line with the literature study, the change in cone factor ( $N_c$ ) is therefore estimated to be up to 10%. However, based on the numerical analysis, it is assumed that the final effect is limited due to relatively small differences between measurement locations. Further, a theoretical application shows that the including the initial stresses cannot explain the relatively low  $s_u$  values for a high cone resistance.

A case study is conducted to include experimental data in the process. The results of the case analysed are in line with previously obtained results. The difference between the cone factor ( $N_c$ ) and the cone factor corrected for horizontal stresses ( $N_{soil}$ ) is found to be 6-9%. A small improvement concerning variation coefficient is observed, resulting in a 2-3% lower reduction factor between expected and characteristic value for  $s_u$ .

As it is expected that the time-consuming calculations in comparison with the small improvement lead to a unprofitable implementation in practice, the horizontal stresses can be included using a 'rule of thumb' values. For the case study, this 'rule of thumb' would lead to a small improvement of 1-2%.

In general, it can be said that taking into account the horizontal stresses slightly improves the correlation between cone resistance and undrained shear strength ( $s_u$ ). However the total effect is small and numerous uncertainties remain in the calculations. In addition, the largest deviation observed, showing relatively low  $s_u$  values under the dike, is not explained by including the stress state. Therefore, it is concluded that including a measure of the in-situ soil stress state ( $\Delta$ ) in the determination of the cone factor ( $N_c$ ) does not result in a significant reduction of the present uncertainties in dike assessment with existing approaches.

## 6.2 Recommendations

The following general recommendations are stated:

- In the case study data, it was found that the linear correlation between  $s_u$  and  $q_{net}$  seems to overestimate the undrained shear strength for high values of  $q_{net}$ . This is also the case for the Markermeerdijken data discussed in section 1.3.
- If both the influence of the initial soil stress state on the cone factor ( $N_c$ ) as described in literature is correct as well as the findings that the horizontal stresses are relatively low under the dike and high next to the dike, it can be stated that the relatively low undrained shear strength ( $s_u$ ) for high cone resistance cannot be explained by variations in the initial soil stress state as discussed in section 3.3.1.
- A fit of the results using two different cone factors for under and next to the dike, or a non-linear correlation, could decrease the variation coefficient found. However no theoretical base for this approach was investigated.
- During the thesis, no clear research is found that verifies the horizontal stresses found from FEM calculations. Further research is recommended, verifying the horizontal stress output of a FEM calculation with experimental data.

Recommendations regarding the implementation of horizontal stresses in the cone factor:

- The effect of including the horizontal stress is found to be very small, in most situations a complete calculation will cost a lot of time and effort and will be out of proportion with the gained improvement.
- If the horizontal stresses are taken into account during macro-stability analysis following the WBI, it is recommended to do it based on a general rule as presented in chapter 5. However, this decreases the already small effect, and therefore only gives a slight increase (1-2%) in characteristic value.
- Numerous soil parameters were considered to remain constant for different geometries. Using local parameters for  $K_0^{nc}$  and OCR, and thereby improving the estimation of  $K_i$ , may increase the effect of including the horizontal stresses;
- Water levels in the numerical analysis and case study were modelled as being constant. Varying the water level over time may have an influence on the  $K_i$  value under and next to the dike.

Recommendations regarding the use of the Soft Soil Creep (SSC) model in PLAXIS:

- In the PLAXIS soil test module, it was found that the SSC model does not model cohesion realistically at low stresses. The vertical stress increases while the horizontal stress remains zero at the start of loading in a oedometer test. In order to find reliable results for soils subjected to low stresses, improvement of the model is advised;
- The soil test module in PLAXIS shows decreasing horizontal stresses after unloading over time due to relaxation, coinciding with increasing OCR due to creep over time for the SSC model. The combination results in a soil that behaves pre-consolidated, but without the relatively high horizontal stresses normally present in this case. During loading this leads to quick failure of the soil. This effect is also found in a 2-D model, where over time the additional horizontal stresses due to OCR decreases due to creep. This effect seems to be incorrect and improvement is recommended.

## Abbreviations

BPT	Ball Penetration Test
CPT	Cone Penetration Test
CRS	Constant Rate of Strain
DMT	Flat Dilatometer Test
DoV II	Dijken op Veen II
FEM	Finite Element Method
HS	Hardening Soil model
HSsmall	Hardening Soil model with small-strain stiffness
MCC	Modified Cam-Clay model
OCR	Over Consolidation Ratio
POP	Pre Overburden Pressure
RF	Reduction Factor
SD	Standard Deviation
SSC	Soft Soil Creep model
WBI	Wettelijk Beoordelingsinstrumentarium

## List of symbols

$a$	Water pressure filter factor
$c'_{ref}$	Cohesion
$I_r$	Rigidity Index
$K$	Coefficient of lateral earth pressure
$K_a$	Coefficient of active lateral earth pressure
$K_p$	Coefficient of passive lateral earth pressure
$K_i$	Coefficient of initial lateral earth pressure $\sigma'_{h0}/\sigma'_{v0}$
$K_0$	Coefficient of lateral earth pressure at rest
$K_0^{nc}$	Normally consolidated coefficient of lateral earth pressure at rest
$K_0^{oc}$	Over consolidated coefficient of lateral earth pressure at rest
$L_i$	Ignition loss
$m$	Strength increase exponent
$N_c$	Cone factor
$N_{kt}$	Cone factor for CPT
$N_b$	Cone factor for BPT
$N_{soil}$	Cone factor without local stress state
$N_{\Delta}$	Part of cone factor based on initial soil stress state
$q_c$	Cone resistance CPT
$q_t$	Pore pressure corrected cone resistance CPT
$q_{net}$	Net cone resistance
$q_{net,v}$	Net cone resistance, corrected for vertical effective stress
$q_{net,h}$	Net cone resistance, corrected for horizontal effective stress
$q_{ball}$	Cone resistance BPT
$s_u$	Shear strength
$u_2$	Water pressure
$VC_{average}$	Average variability of the cone factor
$VC_{average,spatial}$	Spatial variability of the cone factor
$VC_{average,transformation}$	Transformational variability of the cone factor
$z$	Soil depth
$\alpha$	OCR gradient
$\alpha_{c/s/f}$	Cone and shaft roughness factor
$\beta$	Constant for correlation $s_u$ and $\phi'$
$\gamma_{unsat}$	Unsaturated soil weight
$\gamma_{sat}$	Saturated soil weight
$\Delta$	Dimensionless initial soil stress ratio
$\delta$	Constant for $\Delta$ -factor in cone factor correlation
$\phi'$	Friction angle
$\kappa^*$	Modified swelling index
$\lambda^*$	Modified compression index
$\mu^*$	Modified creep index
$\nu$	Poisson's ratio
$\sigma_0$	Initial soil stress
$\sigma_{mean}$	Mean soil stress
$\sigma'_{v0}$	Initial vertical effective soil stress
$\sigma'_{h0}$	Initial horizontal effective soil stress

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*“Arguing with an engineer is like wrestling with a pig in the mud.  
After a few minutes, you realize the pig likes it”*