Memo

To
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Subject
Back analyses of dikes that withstand a high water level

Introduction

In the present note the slope stability of four dikes that withstand a loading of a high water level is analysed. These analyses are part of the research towards a new method for slope stability analysis for the safety assessment of primary dikes in The Netherlands. This new method includes the characterisation of the behaviour of clay and peat with undrained shear strength parameters.

The slope stability analyses described in this note are a continuation of a validation study of the new slope stability method (Van Duinen, 2010). The previous validation study was based on the recalculation of seven failed slopes of dikes. In the discussion of the results of that validation study one of the points of discussion was the fact that in the validation study only failed slopes were analyzed. This was chosen because a failed slope by definition must have had a slope stability factor less then 1,0, which gives a sound reference value for the results of the back analyses. In the present study the slope stability of four dikes that did withstand a certain high water level is analyzed. These additional analyses give a more complete picture of the different calculation methods and how the results relate to observations and engineering judgment.

Context

This research is a part of the SBW-project Macrostabiliteit, which is a part of the programme `Sterkte en Belastingen Waterkeringen´ (SBW) of Rijkwaterstaat. This research programme covers the development of the knowledge which is needed to improve the assessment of the safety level of the primary dikes in the Netherlands. Deltares is carrying out this research programme by order of Rijkswaterstaat.

In the context of improving the safety assessment of primary dikes in the SBW programme Deltares is developing a new method for slope stability analysis. The main specific elements of this method are:

- Undrained behaviour of clay and peat is taken into account, which gives cause for excess pore water pressures be generated. This is an important phenomenon in the behaviour of clay and peat, but this effect is not in the prevailing method.
- The strength at failure of the soil is taken into account whereas the prevailing method (with effective stress strength parameters) uses a strength at 2 to 5% vertical strain of the laboratory test results, at which the strength of the soil is not yet fully mobilised.
- Is in line with the international state of the art on characterising soft soil behaviour.

The reasons why the development of this new method for slope stability analysis is necessary are:

- The unclear situation about the strength of a soil layer because of the differences between the strength from (multi stage) triaxial tests and from cell tests and the discussions about this subject due to the termination of cell tests.
- The opinion that the current methods do not suits well for describing the behaviour of soft organic clay and peat.
- The opinion that the strength at failure of the soil has to be taken into account for the safety assessment of a dike, including the undrained behaviour.
- To prevent for unjust approval or disapproval of a dike in a safety assessment.

Validation study on failed slopes

A validation study in 2010 (Van Duinen, 2010) has shown that the current methods with use of strength parameters from the cell test and the (multi stage) triaxial test can be unsafe. In this validation study seven locations of slopes where slope instability had occurred were investigated extensively with field and lab tests. The results of the validation study are showed in Figure 1.

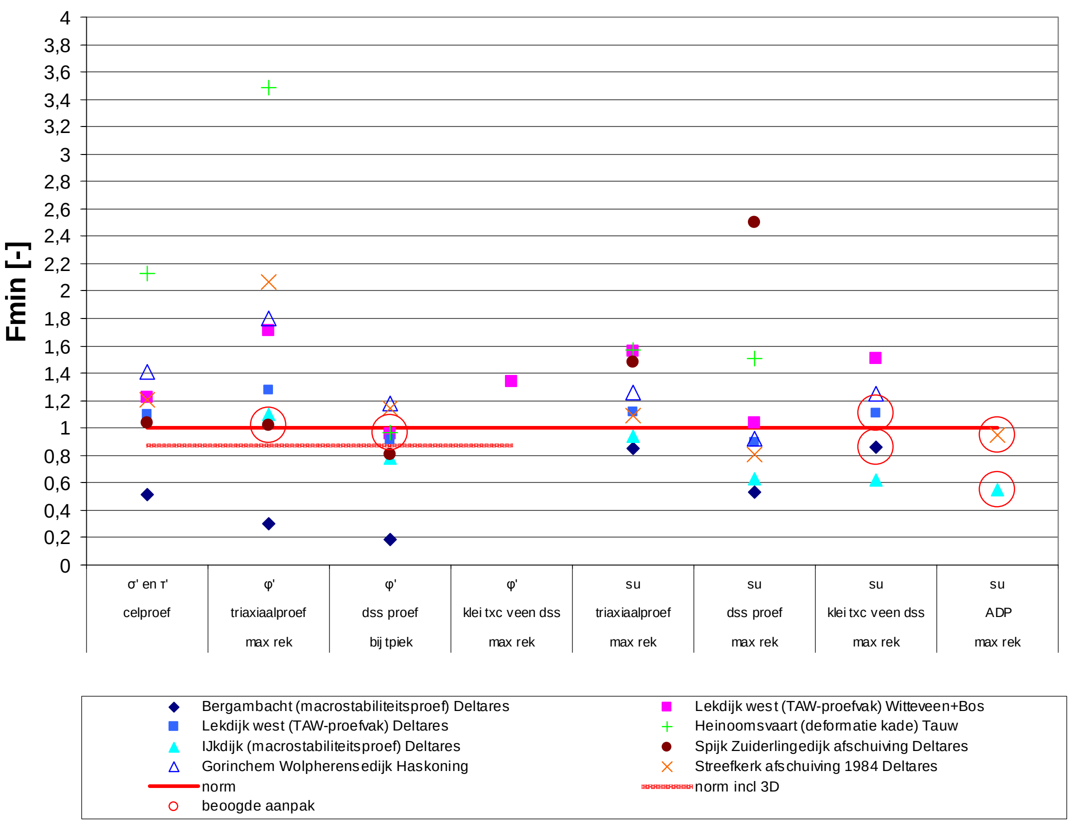

Figure 1 Results of the validation study based on seven locations of slopes where slope instability had occurred. On the y-axis is F_{min} the stability factor (Van Duinen, 2010).

From the results of the slope stability analyses it was concluded that the current methods (left side in Figure 1) often result in a safety factors by which the slope could not have failed. But they did fail, so the strength of the soil layers derived from the lab tests which are carried out by the standard procedures is on average too high, which is unsafe. The new method for undrained slope stability analyses on the other hand show good results for the seven cases. Two of the seven cases are supposed to behave drained during failure. These are the cases Heinoomsvaart, where the slope deformed very slow during many years, and Spijk Zuiderlingedijk, where a leakage of a water pipe caused the slope failure. For these two cases an effective stress slope stability analysis with a friction angle (critical state with cohesion is zero) resulted in a good calculated safety factor.

Analyses on withstand high water levels
The selected locations for the slope stability analyses for the situations with the high water level which is withstand by the dikes are presented in Table 1.

Location	Specifications high water level
Markermeerdijk Dp. 63 dijk 23	November 26, 1928 Highest water level NAP +2,10 m Duration 48 hours (source Infram, 2000)
Lekdijk Nieuw-Lekkerland Dp. 182+040m	February 1, 1953 Highest water level NAP +3,67 m Duration 34 hours (estimated for Krimpen aan de Lek by Dekker, 2003)
Lekdijk Bergambacht Dp. 84 - Dp. 87	January – February 1995 Highest water level NAP +2,95 m Duration 21 days (source Waterdata.nl)
Wolpherensedijk Gorinchem Dp. 391	January – February 1995 Highest water level NAP +4,0 m (estimated from NAP +4,81 m at Vuren and NAP +3,13 m at Werkendam) Duration 21 days (source Waterdata.nl)

Table 1 Selected locations and specifications of high water level that was withstand.

These locations are selected because a lot of information about soil layers, soil parameters and geohydrological parameters of these locations is available (See Appendix 1 and Van Duinen, 2008 and Van Duinen, 2010).

The geometry of the dikes in the slope stability calculations is a reconstruction of the geometry of the dikes at the moment of the high water level which is back analysed. The effect of land subsidence and water level decrease in the polder is not included in the calculations. This is expected to be a conservative assumption.

The shear strength parameters adopted in the slope stability analyses are:

- Sigma-tau stress tables bases on cell test results which are available in D-Geo Stability (represents the common engineering practise);
- Normal consolidated undrained shear strength ratio and pre-overburden pressure derived from the laboratory tests of previous studies (Van Duinen, 2008 and Van Duinen, 2010).

No cohesion and friction angle parameters from the common multi-stage triaxial tests are available for the selected cases.

Calculations are performed with mean values (best estimates) and design values of the shear strength. The mean values of the shear strength are based on laboratory tests of previous studies. The design values of the shear strength account for the requested safety level. The design values are calculated by determining the characteristic values of the shear strength from the mean values and standard deviations and dividing these characteristic values by a material factor (partial safety factors). The design values are therefore lower than the mean values of the shear strength. To analyse an event in the field, such as a slope failure or a dike that withstood a high water level, the mean values (best estimates) of the shear strength are the preferred parameter values.

The material factors for the shear strength parameters from the cell test results are adopted from the Technical Report `Waterkerende Grondconstructies´ (TAW, 2001). The material factors for the undrained shear strength parameters are temporary set to 1,15 for the undrained shear strength ratio and 1,15 for the pre overburden pressure (Rohe et al, 2012). The Technical Reports do not provide partial safety factors for undrained shear strength parameters yet.

The slope stability analyses are carried out with the LiftVan slip plane model in the D-Geo Stability slope stability program. In the calculations, multiple tangent lines are used, so all combinations of both deep and shallow as well as both long and small slip surfaces can be calculated.

Other assumptions are described in Appendix 1.

The results of the slope stability calculations are presented in Table 2 and Figure 2.

Location	Cell test (stress tables)		Undrained shear strength	
	Mean values (best estimates)	Design values	Mean values (best estimates)	Design values
Markermeerdijk	1,22 (1,17)	0,90 (0,86)	1,01 (0,96)	0,50 (0,48)
Lekdijk Nieuw-Lekkerland	1,44 (1,37)	1,10 (1,05)	1,06 (1,01)	0,66 (0,62)
Lekdijk Bergambacht	1,54 (1,46)	1,14 (1,08)	1,13 (1,08)	0,75 (0,71)
Wolpherensedijk Gorinchem	1,64 (1,56)	1,26 (1,20)	1,17 (1,11)	0,78 (0,75)

Table 2 Stability factors F_{min} calculated with LiftVan slip plane model. The values between brackets are the values wherein the factor 1,05 for three-dimensional effects is included.

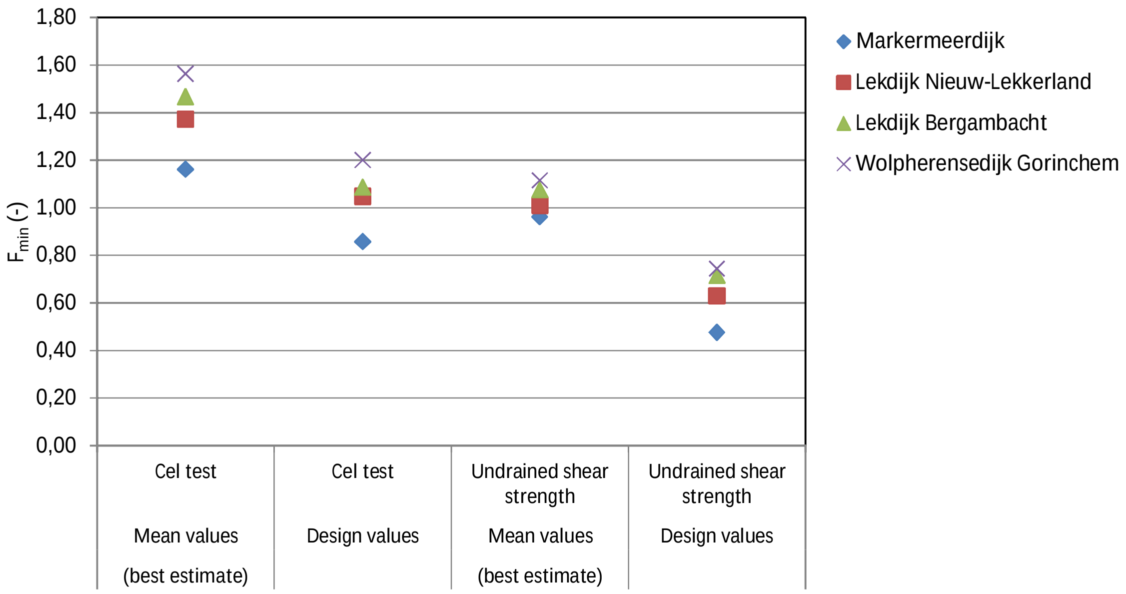

Figure 2 Stability factors F_{min} calculated with LiftVan slip plane model. The presented stability factors are the values wherein the factor 1,05 for three-dimensional effects is included.

The calculated slip planes are presented in Appendix 2.

Discussion
Some observations regarding the calculated stability factors presented in Table 2 and Figure 2 are:

- The stability factors F_{min} calculated with mean values of the shear strength from the cell tests are much higher then the stability factors calculated with the mean values of the undrained shear strength (factor 1,21 – 1,40).
- The stability factors F_{min} calculated with design values of the shear strength from the cell tests are also much higher then the stability factors calculated with the design values of the undrained shear strength (factor 1,52 – 1,80).
- The ratio of the stability factors F_{min} calculated with mean values of the shear strength from the cell tests divided by the stability factors calculated with the design values of the shear strength from cell test is 1,30 – 1,36, while the ratio of the stability factors calculated with mean values of the undrained shear strength divided by the stability factors calculated with the design values of the undrained shear strength is 1,50 – 2,02. This wider range of the ratio´s for the undrained strength calculations is due to the wider variation in test results and the less test results available of undrained shear strength.
- The stability factors of the calculations of all locations with mean values of the cell test results are largely above F_{min} = 1,0, so based on these results it is evident that the high water levels could be withstand.
- The stability factors of the calculations of the locations Nieuw-Lekkerland, Bergambacht and Gorinchem with design values of the cell test results are above F_{min} = 1,0, so based on these results it is explainable that the high water levels could be withstand.

- The stability factors of the calculations of the locations Nieuw-Lekkerland, Bergambacht and Gorinchem with mean values (best estimates) of the undrained shear strength are above F_{min} = 1,0, so based on these results it is understandable that the high water levels could be withstand.
- The stability factors of the calculations of all four locations with design values of the undrained shear strength are beneath F_{min} = 1,0. Since the stability factors of the calculations of the locations Nieuw-Lekkerland, Bergambacht and Gorinchem with best estimate values of the undrained shear strength are above F_{min} = 1,0 the stability factors beneath F_{min} = 1,0 with design values of the undrained shear strength do not mean that these dikes could have failed, but that these dikes do not have the requested safety level.
- The stability factor of the calculation of the Markermeerdijk location with mean values of the undrained shear strength is just above F_{min} = 1,0 when the factor for three-dimensional effects is assumed to be 1,0. When the factor for three-dimensional effects is assumed to be 1,05 the calculated stability factor is less then F_{min} = 1,0 (0,96). Since for this location a circular critical slip plane is calculated a factor of 1,0 for three-dimensional effects can be proposed. When adopting this value the calculation result of the Markermeerdijk with mean values of the undrained shear strength also fits with the observation that the high water level could be withstood. But also this dike does not have the requested safety level.
- At the Nieuw-Lekkerland case the calculated critical slip planes with the strength parameters from the cell tests are deep failure mechanisms but the calculated critical slip planes with the undrained shear strength are shallow mechanisms. In the slope stability calculations with undrained shear strength at this case a deep failure mechanism have much higher stability factors then a shallow mechanism.
- The calculated stability factors with mean values of the undrained shear strength are in a narrow band just above F_{min} = 1,0 (the Markermeerdijk case excepted as commented earlier). This may be explained by the fact that the dikes in the past were improved by experience. So this was also the situation at the moment at which the high water levels occurred which are analysed in the present study.

Combining the results of the validation study based on stability analyses of failed dike slopes (Figure 1 and Van Duinen, 2010) and the results of the analyses of the present study give the following observations:

- The mean values of the shear strength from cell test results leads to stability factors above F_{min} = 1,0 both for the failed slopes and the slopes that withstood a high water level. The only exception is the Bergambacht case for the slope failure at the slope stability test with F_{min} = 0,51.
- The mean values of the undrained shear strength leads to stability factors close to F_{min} = 1,0 and less then F_{min} = 1,0 for the failed slopes and above F_{min} = 1,0 for the slopes that withstood a high water level. The Markermeerdijk case has also a stability factor above F_{min} = 1,0 (F_{min} = 1,01) when the factor for three-dimensional effects is set to 1,0, which can be proposed because a circular critical slip plane is calculated.
- Two cases in the validation study are supposed to behave drained during failure (Heinoomsvaart and Spijk Zuiderlingedijk) . For these two cases an effective stress slope stability analysis with mean values of the friction angle (critical state with cohesion is zero) gave stability factors close to 1,0.
- Due to the conservative assumptions about the land subsidence and the water level decrease in the polder the calculated stability factors could be slightly higher.

It is important here to note that the current slope stability analyses with the effective stress shear strength parameters cohesion and friction angle derived from (multi-stage) triaxial tests often (but not always) give higher stability factors then stability analyses with shear strength parameters from cell tests (Rohe et al, 2012).

Conclusions

From the results of the previous validation study based on the back analyses of the failed slopes and the present study on the back analyses of slope stability at high water levels which are withstood by the studied dikes it can be concluded, that:

- The common method of slope stability analyses with shear strength parameters derived from cell tests is not a safe method. These stability analysis leads to relatively high stability factors, which can not be justified by the results of the back analyses of the failed slopes in the validation study.
- The proposed slope stability method with undrained shear strength gives the most reliable results. With this method and mean values (best estimate) of the undrained shear strength, the back analyzed stability factors F_{min} are less then 1,0 or around 1,0 in cases where a slope failure had occurred and the back analyzed stability factor is (just) above 1,0 for cases where a high water level is withstood (also mean values of the undrained shear strength).

Note that the current method of slope stability analyses with the effective stress shear strength parameters cohesion and friction angle derived from (multi-stage) triaxial tests is also not a safe method. This method often (but not always) gives higher stability factors then stability factor of analyses with shear strength parameters from cell tests (Rohe et al, 2012). These current stability analysis leads also to high stability factors, which can not be justified by the results of the back analyses of the failed slopes in the validation study.

References

Dekker, J. 2003. De Stormvloedramp van 1 februari 1953. Crempene. Historische Vereniging van Krimpen aan de Lek. 2003 – Uitgave 2.
Van Duinen, T.A. 2008. Grensverleggend onderzoek macrostabiliteit bij opdrijven - Fase 2.C. Deltares report 419230-0040. Version 2. September 2008.
Van Duinen, T.A. 2010. SBW Werkelijke sterkte van dijken - validatie WS15. Synthese resultaten stabiliteitsanalyses onderzoekslokaties. Deltares report 1202121-003-GEO-0022, Version 1, 7 september 2010.
Infram. 2000. Markermeer II, Bewezen sterkte met betrekking tot geotechnische stabiliteit. Infram report i 221. December 2000.
Rohe, A. and T.A. van Duinen. 2010. Consequentieanalyse Macrostabiliteit. Vergelijking oude en nieuwe toetsmethode. Deltares report 1204203-005-GEO-0002, Version 3, 27 april 2012, definitief.
TAW. 2001. Technisch Rapport Waterkerende Grondconstructies, Technische Adviescommissie voor de Waterkeringen. Juni 2001.

Appendix 1
Assumptions

General assumptions
Information about soil layers, soil parameters and geohydrological parameters for the back analyses of the four locations is available in Van Duinen (2008) and Van Duinen (2010).

The geometry of the dikes in the slope stability calculations is a reconstruction of the geometry of the dikes at the moment of the high water level which is back analysed. The effect of land subsidence and water level decrease in the polder is not in the calculations. This is expected to be a conservative assumption.

The shear strength parameters adopted in the slope stability analyses are:

- Sigma-tau stress tables bases on cel test results which are available in D-Geo Stability (represent the common engineering practise);
- Normal consolidated undrained shear strength ratio and pre overburden pressure derived from the laboratory tests of earlier studies (Van Duinen, 2008 and Van Duinen, 2010).

Calculations are performed with mean values and design values of the shear strength. The material factors (partial safety factors) for the shear strength parameters from the cell test results are derived from the Technical Report `Waterkerende Grondconstructies´ (TAW, 2001). The material factors for the undrained shear strength parameters are temporary set to 1,15 for the undrained shear strength ratio and 1,15 for the pre overburden pressure (Rohe et al, 2012).

No traffic load is taken into account.

The slope stability analyses are carried out with the LiftVan slip plane model in the D-Geo Stability slope stability program. In the calculations multiple tangent lines are used, so all combinations of both deep and shallow as well as both long and small slip surfaces can be calculated.

Markermeerdijk Dp. 63 dijk 23
High water level NAP +2,10 m at November 26, 1928 (duration 48 hours) (Infram, 2000).
Geometry, soil layers and pore water pressures according to Van Duinen (2008).
Soil parameters from cell tests from parameter set Noord-Holland in D-Geo Stability library.
Soil parameters undrained shear strength as in Table 1.1 (modified from Van Duinen, 2008).

Soil layer	Mean values		Design values	
	su-ratio (-)	POP (kN/m^2)	su-ratio (-)	POP (kN/m^2)
Dijksmateriaal	0,26	24	0,17	14
Hollandveen	0,40	14	0,22	0
Calais clay	0,27	14	0,21	0
Calais sandy clay	0,25	14	0,19	0

Table 1.1 Soil parameters undrained shear strength case Markermeerdijk.

Lekdijk Nieuw-Lekkerland Dp. 182+040m
High water level NAP +3,67 m at February 1, 1953 (duration 34 hours) (Dekker, XXXX)
Geometry of present dike modified to the situation in 1953; i.e.: new dike from 80´s removed.
Soil layers according to Van Duinen (2008).
Phreatic level in the dike: NAP +2,50 m (1 m below high water).
Stationary hydraulic head in aquifer NAP -0,20 m (Van Duinen, 2008).
Hydraulic head in aquifer at high water level as in Table 1.2 (not corrected for uplift) (calculated with Watex based on geohydological parameters in Van Duinen, 2008).
Consolidation length at the bottom of aquitard: about 1 m because of short duration of high water.
Soil parameters from cell tests from parameter set Krimpenerwaard+Alblasserwaard in D-Geo Stability library.
Soil parameters undrained shear strength as in Table 1.3 (modified from Van Duinen, 2008 and Van Duinen, 2010).

Distance to dike axis (m)	Hydraulic head PL 3 to NAP (m)
-25	2,21
0	1,95
25	1,66
50	1,41
75	1,20
100	1,03

Table 1.2 Hydraulic head in aquifer at high water level at case Lekdijk Nieuw-Lekkerland (not corrected for uplift).

Soil layer	γ_{wet} (kN/m^3)	Mean values		Design values	
		su-ratio (-)	POP (kN/m^2)	su-ratio (-)	POP (kN/m^2)
Dijksmateriaal	16 – 20	0,25	9	0,19	0
Tiel klei	14 – 18	0,25	9	0,19	0
Hollandveen	10 – 11	0,36	18	0,27	9
Gorkum klei venig	11 – 12	0,33	17	0,17	7
Gorkum klei organisch	12 – 14	0,24	17	0,17	7
Gorkum klei zwaar	14 – 18	0,25	15	0,19	5
Basisveen	10 – 12	0,36	15	0,27	3
Kreftenheye	16 – 18	0,25	15	0,19	3

Table 1.3 Soil parameters undrained shear strength case Lekdijk Nieuw-Lekkerland.

Lekdijk Bergambacht Dp. 84 – Dp. 87
High water level NAP +2,95 m at January – February 1995 (duration 21 days) (source Waterdata.nl)
Geometry of present dike modified to the situation in 1995.
Soil layers according to Van Duinen (2010).
Phreatic level in the dike: NAP +1,95 m (1 m below high water).
Stationary hydraulic head in aquifer NAP +0,0 m (Van Duinen, 2008).
Hydraulic head in aquifer at high water level as in Table 1.4 (not corrected for uplift) (calculated with Watex based on geohydological parameters in Van Duinen, 2008).

Consolidation length at the bottom of aquitard: about 2 m because of long duration of high water.
Soil parameters from cell tests from parameter set Krimpenerwaard+Alblasserwaard in D-Geo Stability library.
Soil parameters undrained shear strength as in Table 1.5 (modified from Van Duinen, 2010).

Distance to dike axis (m)	Hydraulic head PL 3 to NAP (m)
-25	0,99
0	0,94
25	0,89
50	0,85
75	0,81
100	0,76

Table 1.4 Hydraulic head in aquifer at high water level case Lekdijk Bergambacht (not corrected for uplift).

Soil layer	γ_{wet} (kN/m^3)	Mean values		Design values	
		su-ratio (-)	POP (kN/m^2)	su-ratio (-)	POP (kN/m^2)
Dijksmateriaal	16 – 20	0,25	26	0,19	17
Tiel klei	14 – 18	0,25	26	0,19	17
Hollandveen	10 – 11	0,36	19	0,27	13
Gorkum klei venig	11 – 12	0,33	14	0,17	8
Gorkum klei organisch	12 – 14	0,24	18	0,17	13
Gorkum klei zwaar	14 – 18	0,25	27	0,19	14
Basisveen	10 – 12	0,36	27	0,27	10
Kreftenheye	16 – 18	0,25	14	0,19	6

Table 1.5 Soil parameters undrained shear strength case Lekdijk Bergambacht.

Wolpherensedijk Gorinchem Dp. 391
High water level NAP +4,0 m at January – February 1995 (estimated from NAP +4,81 m at Vuren and NAP +3,13 m at Werkendam) (duration 21 days) (source Waterdata.nl)
Geometry of present dike modified to the situation in 1995; i.e.: new dike from 90´s removed.
Soil layers according to Van Duinen (2010).
Phreatic level in the dike: NAP +3,50 m (Van Duinen, 2010).
Water level in the channel behind the dike: NAP +0,50 m (Van Duinen, 2010).
Stationary hydraulic head in aquifer NAP +0,50 m.
Hydraulic head in aquifer at high water level as in Table 1.6 (not corrected for uplift) (calculated with Watex based on geohydological parameters in Van Duinen, 2008).
Consolidation length at the bottom of aquitard: about 2 m because of long duration of high water.
Soil parameters from cell tests from parameter set Krimpenerwaard+Alblasserwaard in D-Geo Stability library.
Soil parameters undrained shear strength as in Table 1.7 (modified from Van Duinen, 2010).

Distance to dike axis (m)	Hydraulic head PL 3 to NAP (m)
-25	2,00
0	1,93
25	1,86
50	1,80
75	1,74
100	1,68

Table 1.6 Hydraulic head in aquifer at high water level at case Wolpherensedijk (not corrected for uplift).

Soil layer	γ_{wet} (kN/m^3)	Mean values		Design values	
		su-ratio (-)	POP (kN/m^2)	su-ratio (-)	POP (kN/m^2)
Dijksmateriaal	16 – 20	0,25	47	0,19	19
Tiel klei	14 – 18	0,25	16	0,19	5
Hollandveen	10 – 11	0,36	10	0,27	7
Gorkum klei venig	11 – 12	0,33	39	0,17	18
Gorkum klei organisch	12 – 14	0,24	45	0,17	12
Gorkum klei zwaar	14 – 18	0,25	40	0,19	16
Basisveen	10 – 12	0,36	40	0,27	16
Kreftenheye	16 – 18	0,25	40	0,19	16
Cunet (kanaalbodem)	20	--	--	--	--

Table 1.7 Soil parameters undrained shear strength case Wolpherensedijk.

Appendix 2
Calculation results

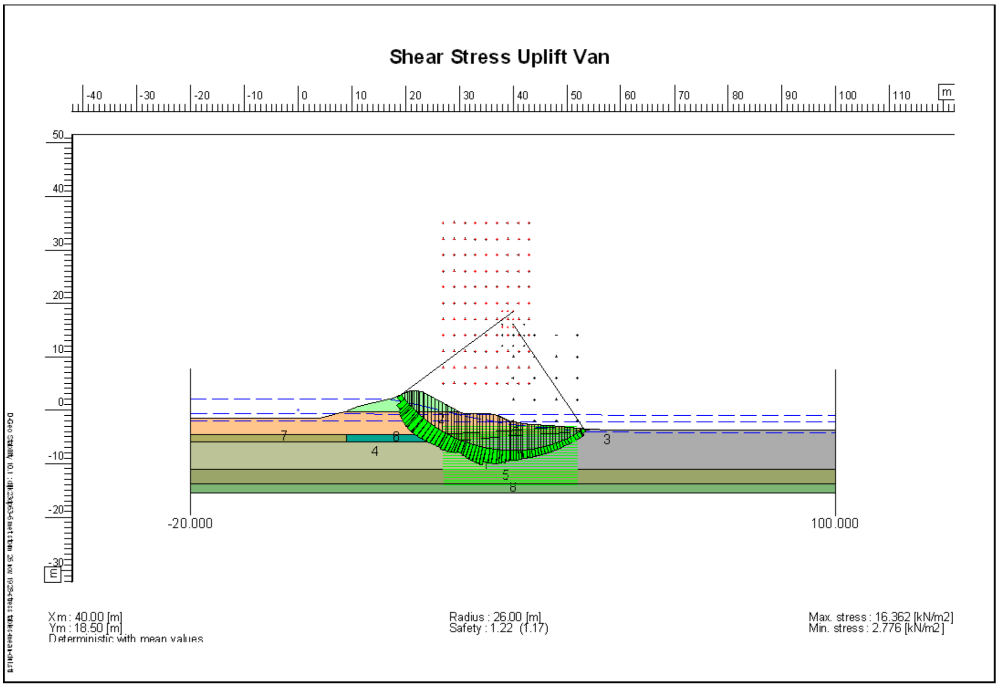

Figure 2.1 Critical slip plane at case Markermeerdijk with mean values of shear strength parameters from cell tests.

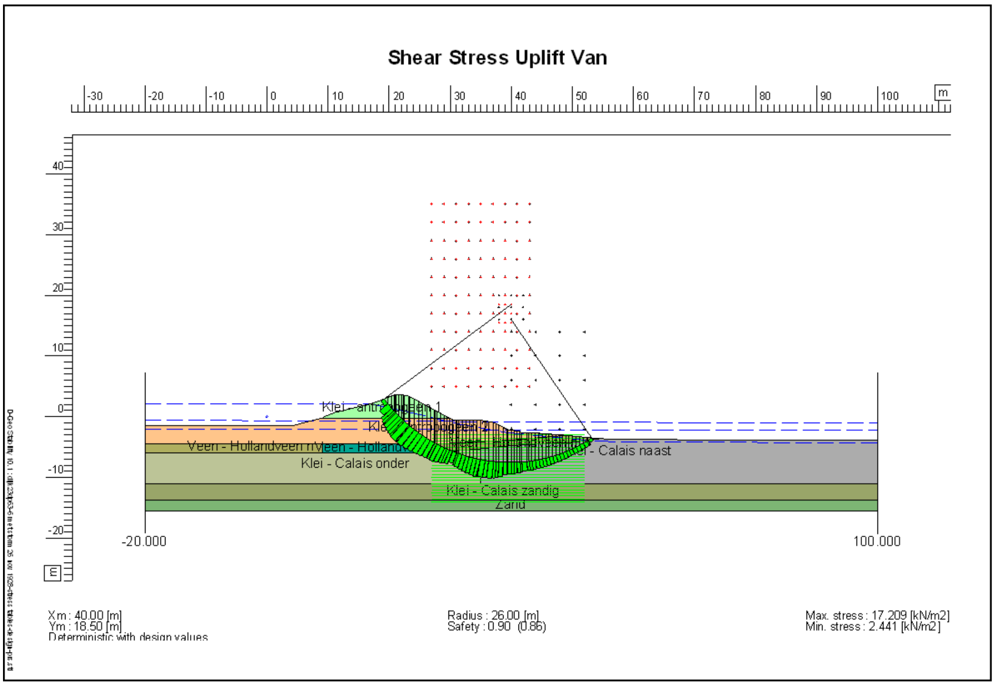

Figure 2.2 Critical slip plane at case Markermeerdijk with design values of shear strength parameters from cell tests.

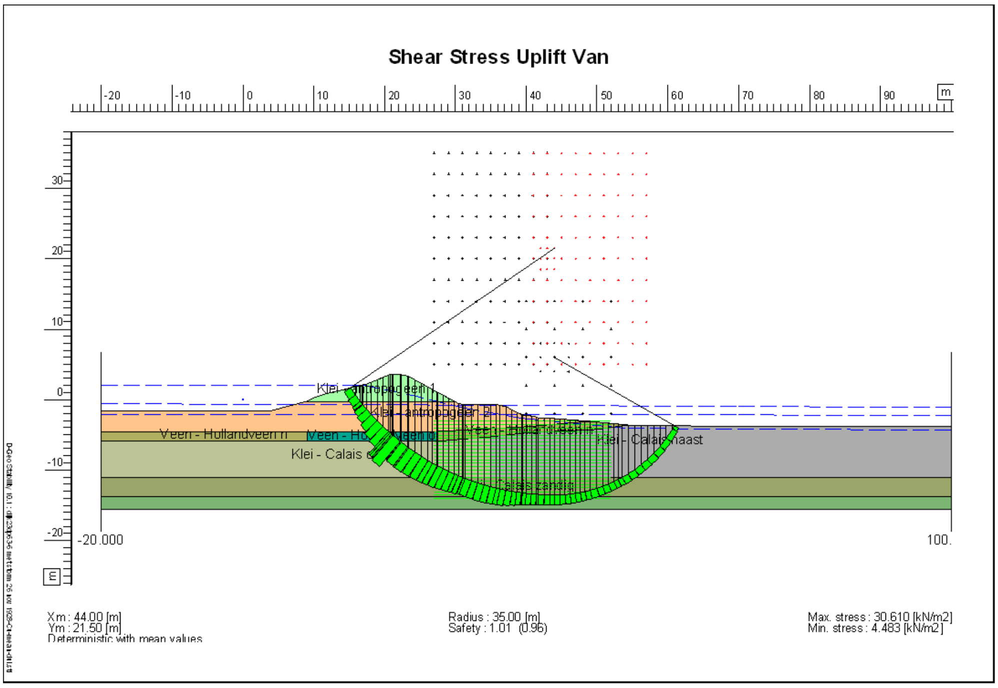

Figure 2.3 Critical slip plane at case Markermeerdijk with mean values of undrained shear strength.

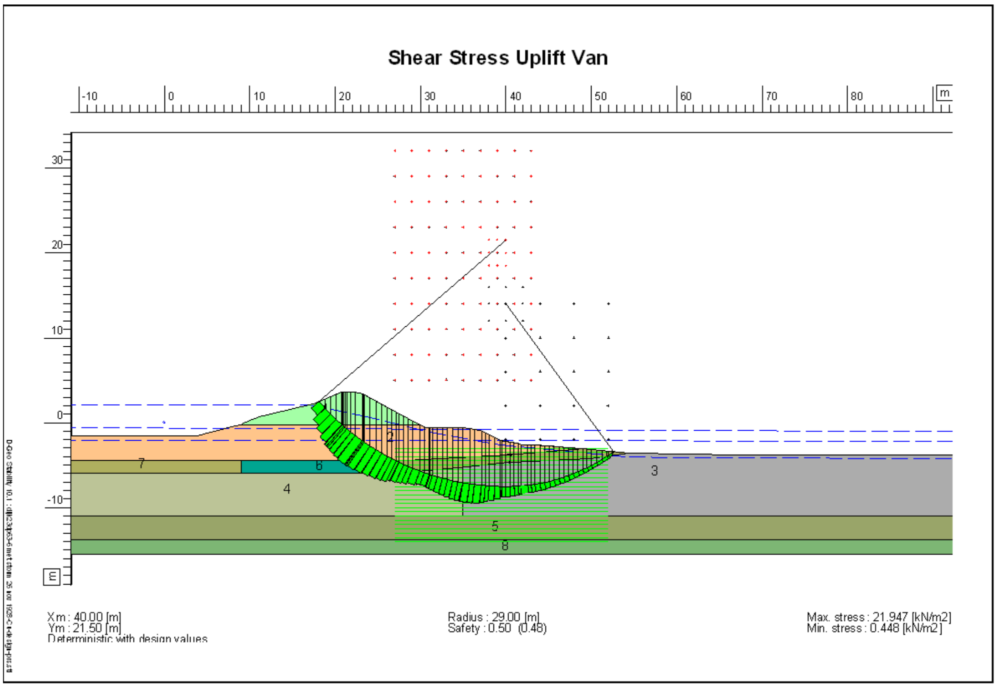

Figure 2.4 Critical slip plane at case Markermeerdijk with design values of undrained shear strength.

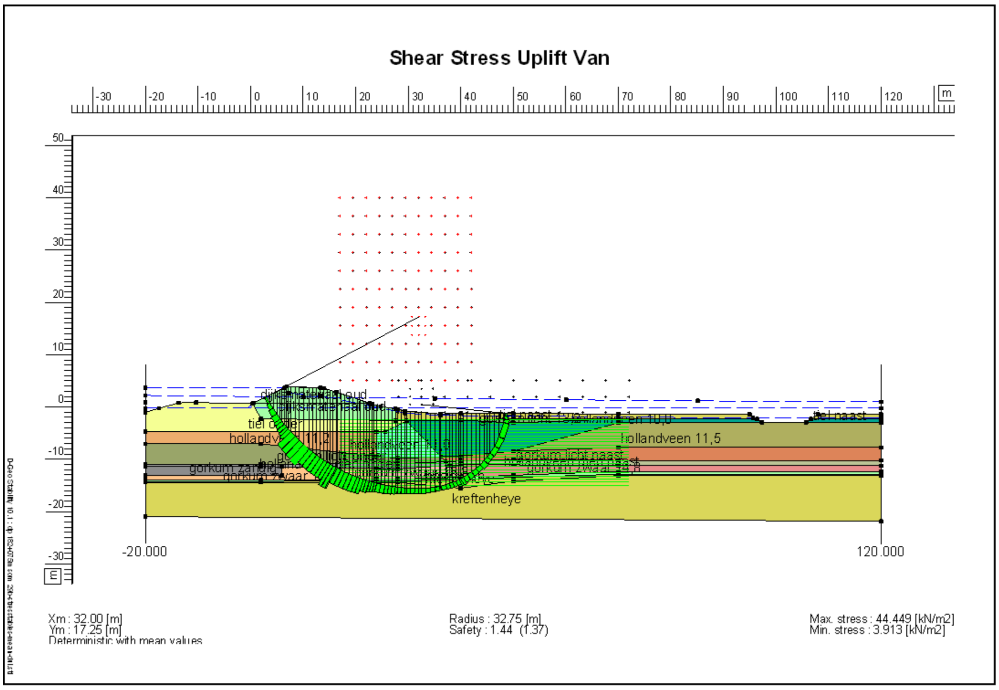

Figure 2.5 Critical slip plane at case Lekdijk Nieuw-Lekkerland with mean values of shear strength parameters from cell tests.

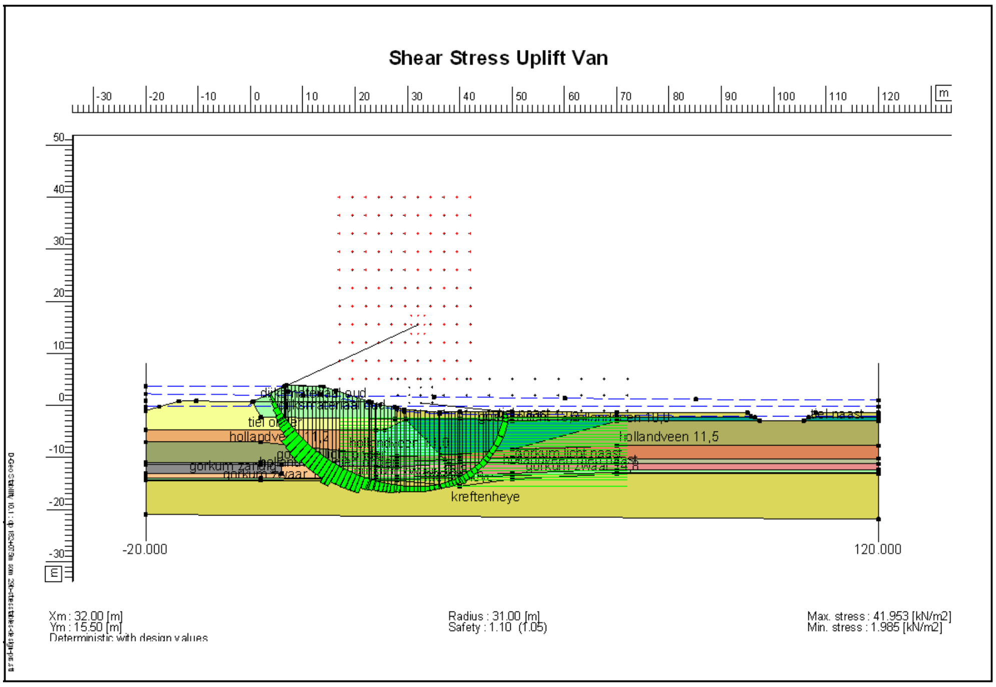

Figure 2.6 Critical slip plane at case Lekdijk Nieuw-Lekkerland with design values of shear strength parameters from cell tests.

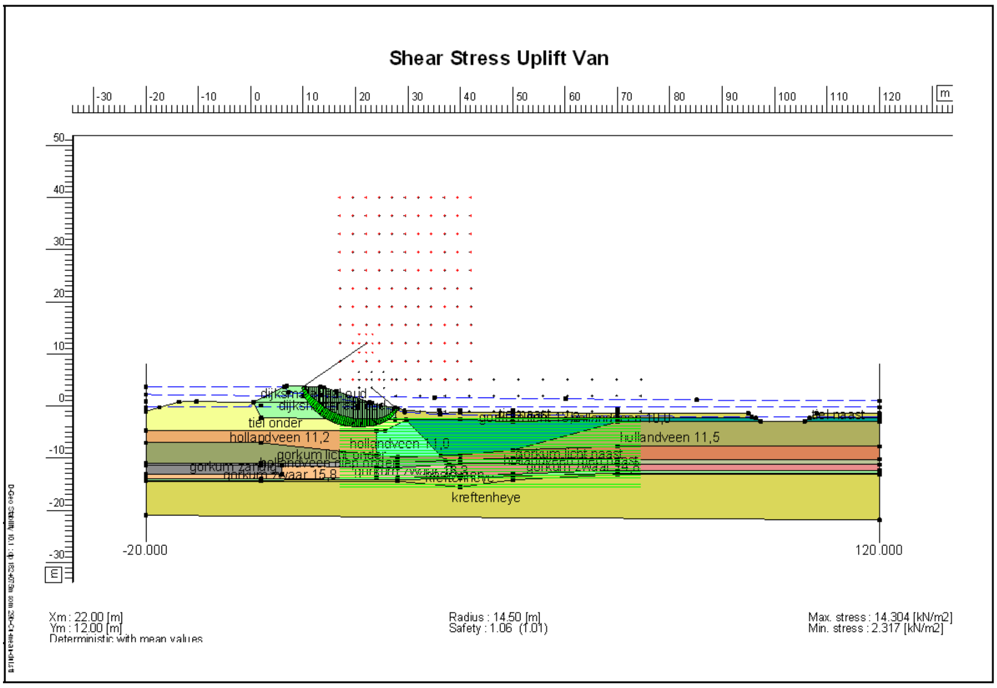

Figure 2.7 Critical slip plane at case Lekdijk Nieuw-Lekkerland with mean values of undrained shear strength.

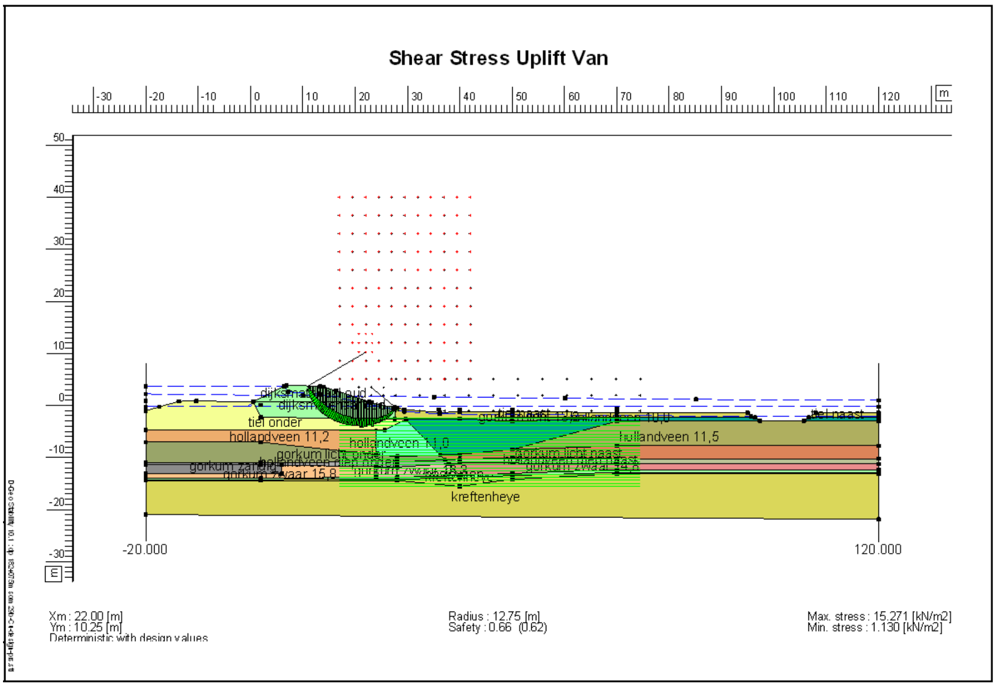

Figure 2.8 Critical slip plane at case Lekdijk Nieuw-Lekkerland with design values of undrained shear strength.

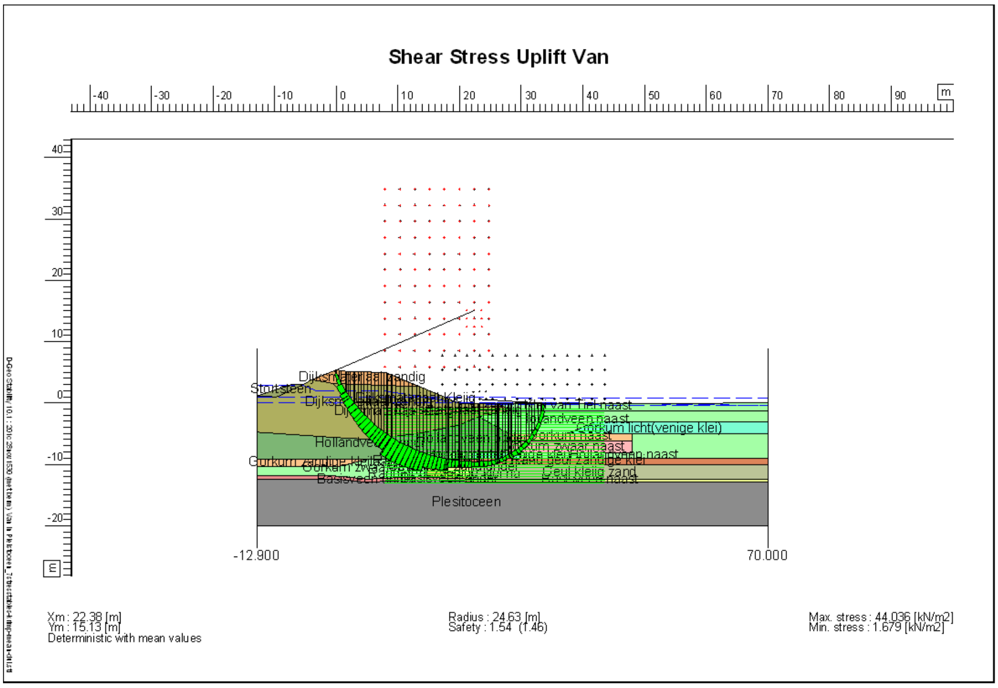

Figure 2.9 Critical slip plane at case Lekdijk Bergambacht with mean values of shear strength parameters from cell tests.

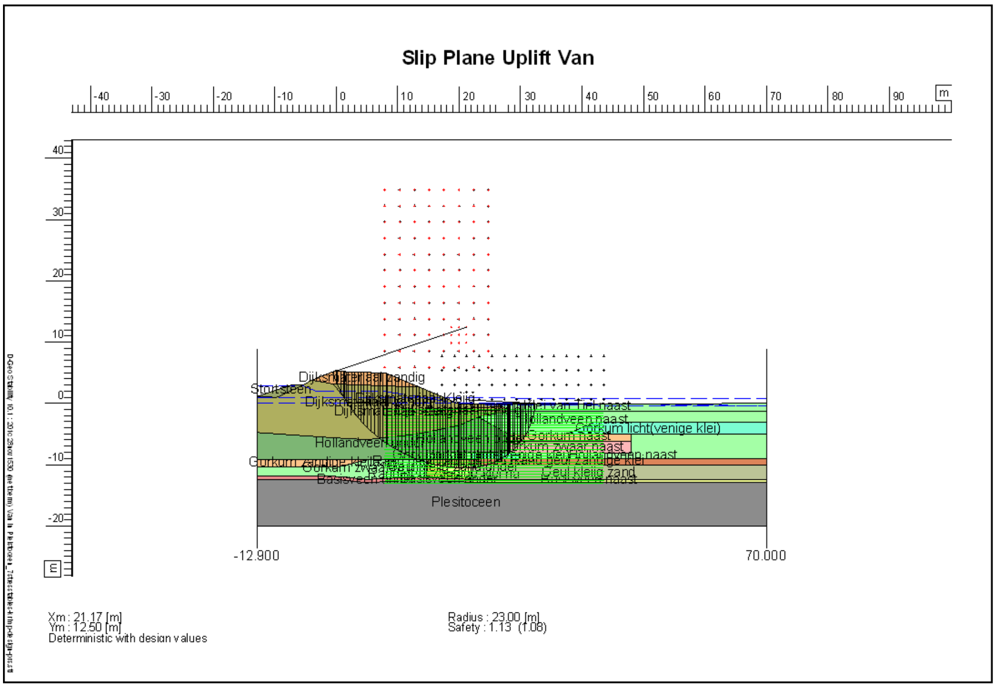

Figure 2.10 Critical slip plane at case Lekdijk Bergambacht with design values of shear strength parameters from cell tests.

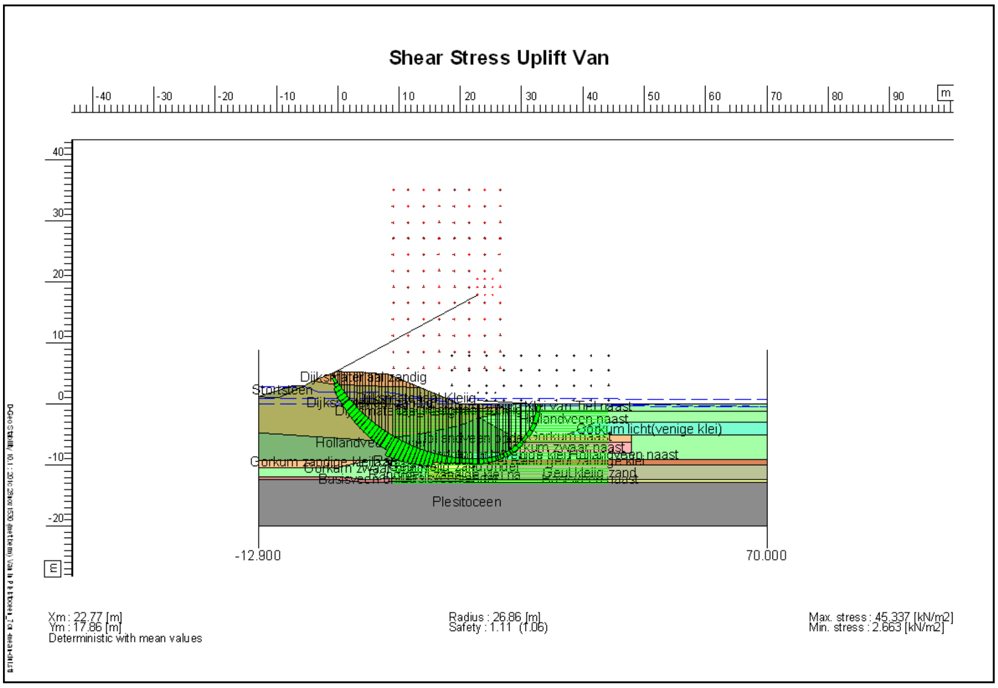

Figure 2.11 Critical slip plane at case Lekdijk Bergambacht with mean values of undrained shear strength.

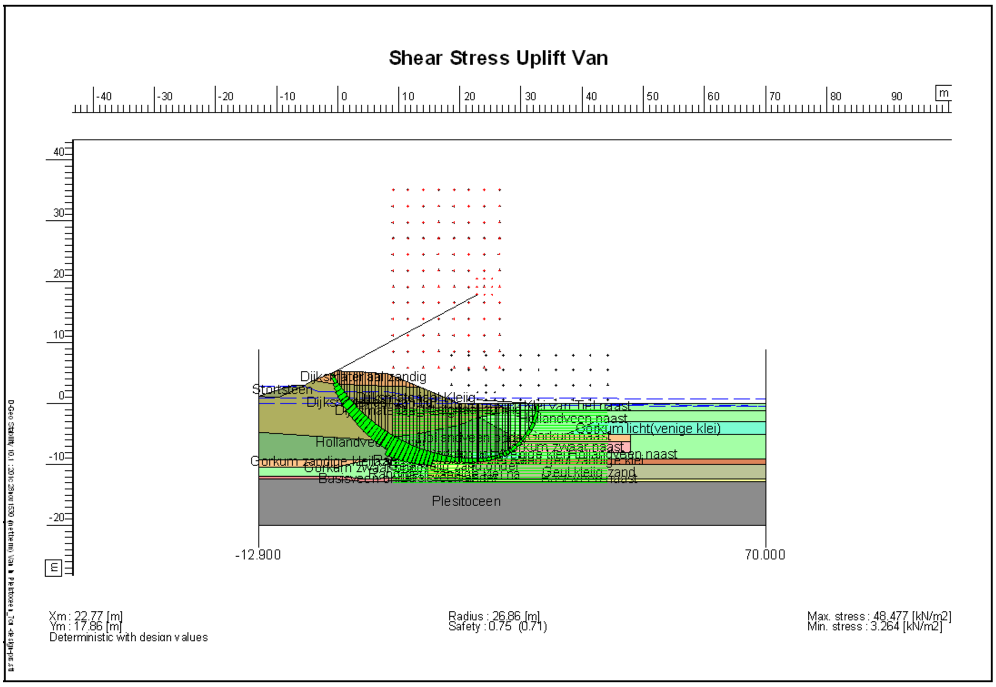

Figure 2.12 Critical slip plane at case Lekdijk Bergambacht with design values of undrained shear strength.

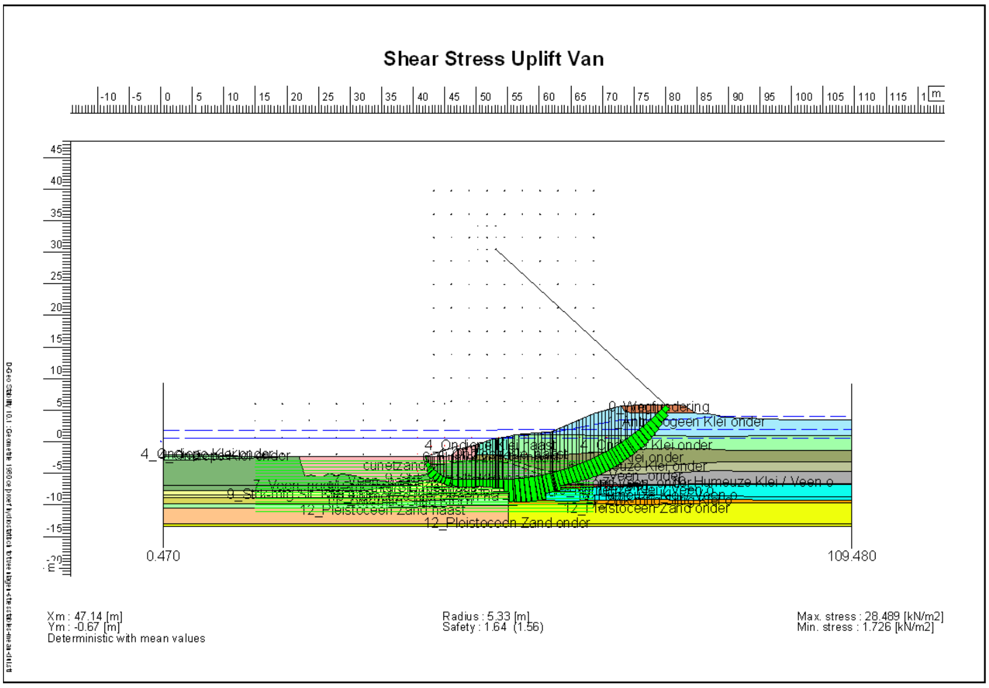

Figure 2.13 Critical slip plane at case Wolpherensedijk with mean values of shear strength parameters from cell tests.

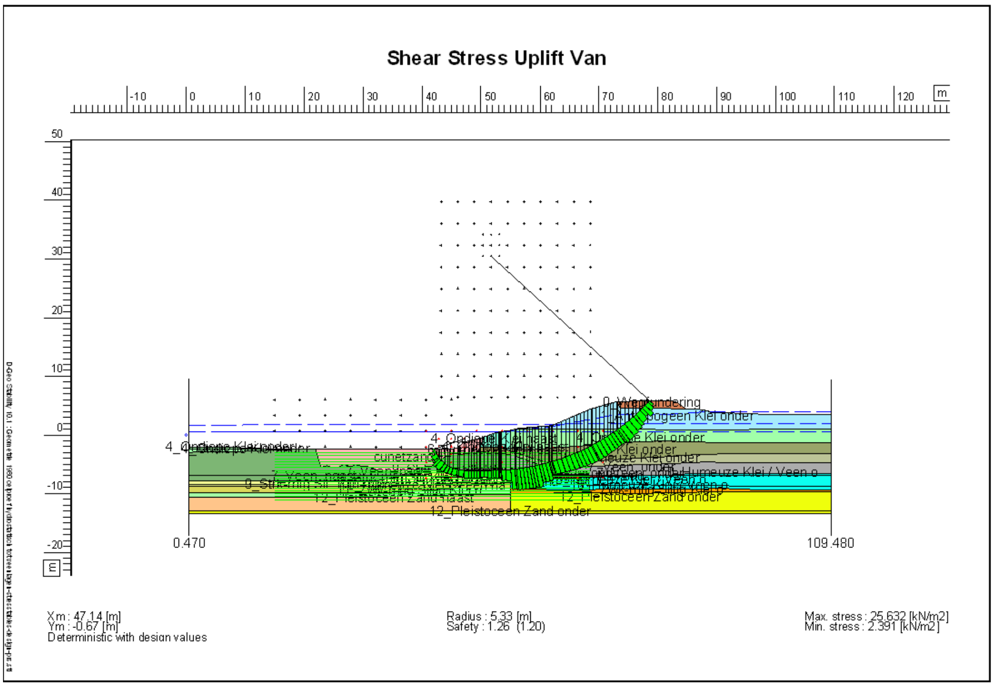

Figure 2.14 Critical slip plane at case Wolpherensedijk with design values of shear strength parameters from cell tests.

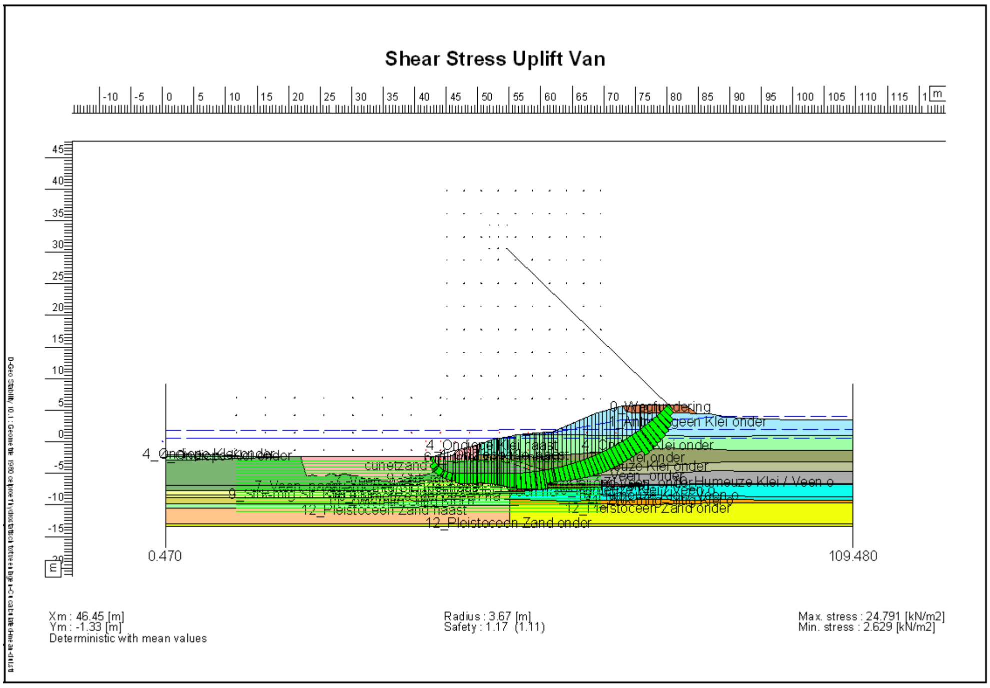

Figure 2.15 Critical slip plane at case Wolpherensedijk with mean values of undrained shear strength.

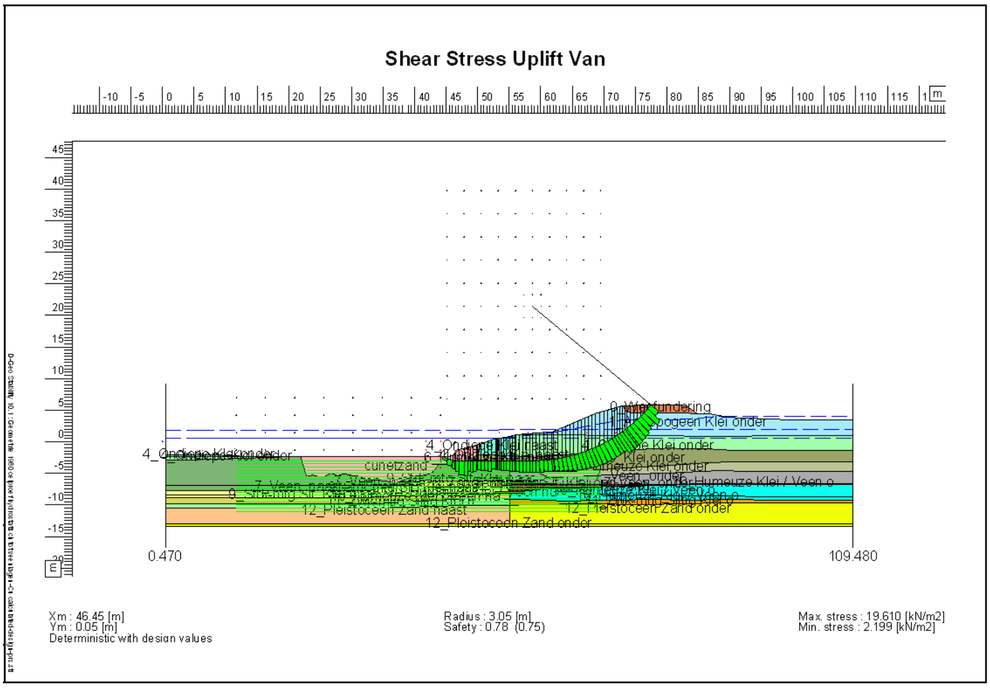

Figure 2.16 Critical slip plane at case Wolpherensedijk with design values of undrained shear strength.