project	**Geluidswal Hooggelegen**
betreft	notitie beantwoording opmerkingen RWS op rapportage
door	
datum	13 mei 2022

1 Inleiding

Gemeente Utrecht is voornemens het terrein tussen de lokale verbindingsweg Stadsbaan Leidsche Rijn en de geluidswal langs de A2 verder uit te vlakken tot bovenkant geluidswal en de kavel vervolgens uit te geven zodat er vervolgens bebouwing op geplaatst kan worden.

Door ABT zijn een 2-tal notities opgesteld ten aanzien van de stabiliteit van de gewapende grondconstructie, ten gevolge van de ophogingen van het tussenterrein en het aanbrengen van funderingspalen voor de toekomstige bebouwing.

Deze twee notities zijn in oktober 2021 door Arcadis, namens RWS, geotechnisch getoetst. Op 29 november 2021 is er een overleg geweest tussen Gemeente Utrecht, Rijkswaterstaat als eigenaar van de geluidswal, Arcadis als toetser namens RWS en ABT.
Naar aanleiding van dit overleg zijn de 2 notities samengevoegd tot één rapport d.d. 17 december 2021 en is één en ander nader toegelicht.

Op het rapport heeft een eerste hertoetsing plaatsgevonden door RWS-GPO. In voorliggende memo wordt puntsgewijs een reactie gegeven op de punten die zijn open blijven staan of zijn toegevoegd naar aanleiding van deze eerste hertoetsing.

In de tussentijd heeft de Duitse leverancier van de geogrids, waarmee de gewapende grondconstructie is opgebouwd, een briefrapport opgesteld. Dit briefrapport d.d. 22 maart 2022 beantwoordt enkele van de vragen en is als bijlage bij de memo toegevoegd.

2 Beantwoording van de vragen

Hieronder wordt per onderwerp kort gereageerd op de opmerkingen van RWS. Deze zijn samengevat en onderstreept opgenomen.

2.1 Maatvoering geluidswal

<u>Er wordt opgemerkt door RWS dat de schetsen in de rapportage van ABT niet overeenkomen met de werkelijke situatie.</u>

Ter illustratie van de mogelijke plaatsing van funderingspalen t.b.v. de nieuwbouw ten opzichte van de geluidswal, is door de constructeur van de toekomstige nieuwbouw een schets gemaakt. Deze schets is toegevoegd ter illustratie aan de rapportage van ABT. In deze schets is inderdaad een onjuiste maatvoering van de geluidswal aangegeven.

Voor de geotechnische berekeningen, welke door ABT en door BBG zijn uitgevoerd, is altijd uitgegaan van de oorspronkelijke tekening D-IV-01-rev03 d.d. 02-12-2009. De constructeur van de nieuwbouw is er inmiddels op gewezen, voor een correcte maatvoering ook deze tekening aan te houden.
Vanuit de gemaakte geotechnische berekeningen is de randvoorwaarde opgelegd dat funderingspalen ten minste 6,5 m uit de kruin van de geluidswal geplaatst dienen te worden.

<u>Ter plaatse van de portaalconstructies hebben de geogrids een langere lengte in de grondwal. Hier dient rekening mee gehouden te worden.</u>
De nieuwbouw is achter een portaalconstructie (VDC) met onderliggend wegkantstation (WKS) geprojecteerd. Ter plaatse van de WKS is een andere layout van de geogrids toegepast, waardoor de onderste grids 2 m langer doorlopen in de geluidswal. Het palenplan van de nieuwbouw is hierop aangepast door ter hoogte van de WKS de beoogde funderingspaal eveneens 2 m verder naar achter te positioneren. Op tekening in de bijlage is dit weergegeven.

2.2 *Palen aanbrengen*

Het advies van NAUE is nog niet opgenomen in de notitie.
De berekening welke door het ingenieursbureau van NAUE is gemaakt is toegevoegd in de bijlage.

Boren door de geogrids op 7,5 meter diepte geeft onzekerheid wat er met de grids gebeurt op deze diepte.
Door BBG, het ingenieursbureau van Naue, is een uitgebreide analyse gemaakt van de stabiliteit van de bestaande, met Secugrid gewapende, grondconstructie tijdens en na het aanbrengen van funderingspalen voor de nieuwbouw.

Op basis van berekeningen en ervaringen met vergelijkbare projecten concluderen zij dat voor het project A2 de randvoorwaarden niet kritiek zijn voor het doorboren van de lagen geogrids.
Nadere gevoeligheidsanalyses worden daarom niet nodig geacht.

2.3 *Bovenbelasting*

Hijswerkzaamheden tussen het nieuw te realiseren gebouw en de geluidswal zijn niet toegestaan.
In de analyses van BBG is nadrukkelijk de bovenbelasting vanuit een boorstelling voor de realisatie van de paalfundering in beschouwing genomen. Hierbij is uitgegaan van het toepassen van dragline-schotten voor het spreiden over een groter oppervlak.

Gemeente Utrecht zal een overdrachtsdocument van de grond opstellen. Hierbij wordt opgenomen dat tussen het gebouw en de rand van de geluidswal maximaal 10 kN/m2 aan bovenbelasting toelaatbaar is. Dit betekent dat hier geen zwaar verkeer of grote stempelkrachten kunnen worden toegelaten.

Als randvoorwaarde dient te worden aangehouden dat op de 5-6 meter bende onderhoudsstrook vanaf de kruin geen bouwactiviteiten en geen bovenbelastingen worden geplaatst.
Met betrekking tot de onderhoudsstrook zijn nadere afspraken gemaakt met Rijkswaterstaat en deze zijn vastgelegd in het matenplan.
Voor de bouwperiode zal in het overdrachtsdocument worden opgenomen dat op deze onderhoudsstrook geen bouwactiviteiten mogen plaatsvinden of bovenbelasting mag worden aangebracht.

2.4 *Monitoring*

Afgesproken is dat voor, tijdens en na de werkzaamheden monitoring, visuele inspecties van de gewapende grondwal opgenomen worden. Deze metingen en foto’s dienen in een beknopt verslag te worden vastgelegd.
Voor, tijdens en na de bouwerkzaamheden dient de gewapende grondconstructie gemonitord te worden door de realiserende partij van de nieuwbouw. Ook dit zal als randvoorwaarde in het overdrachtsdocument opgenomen worden.

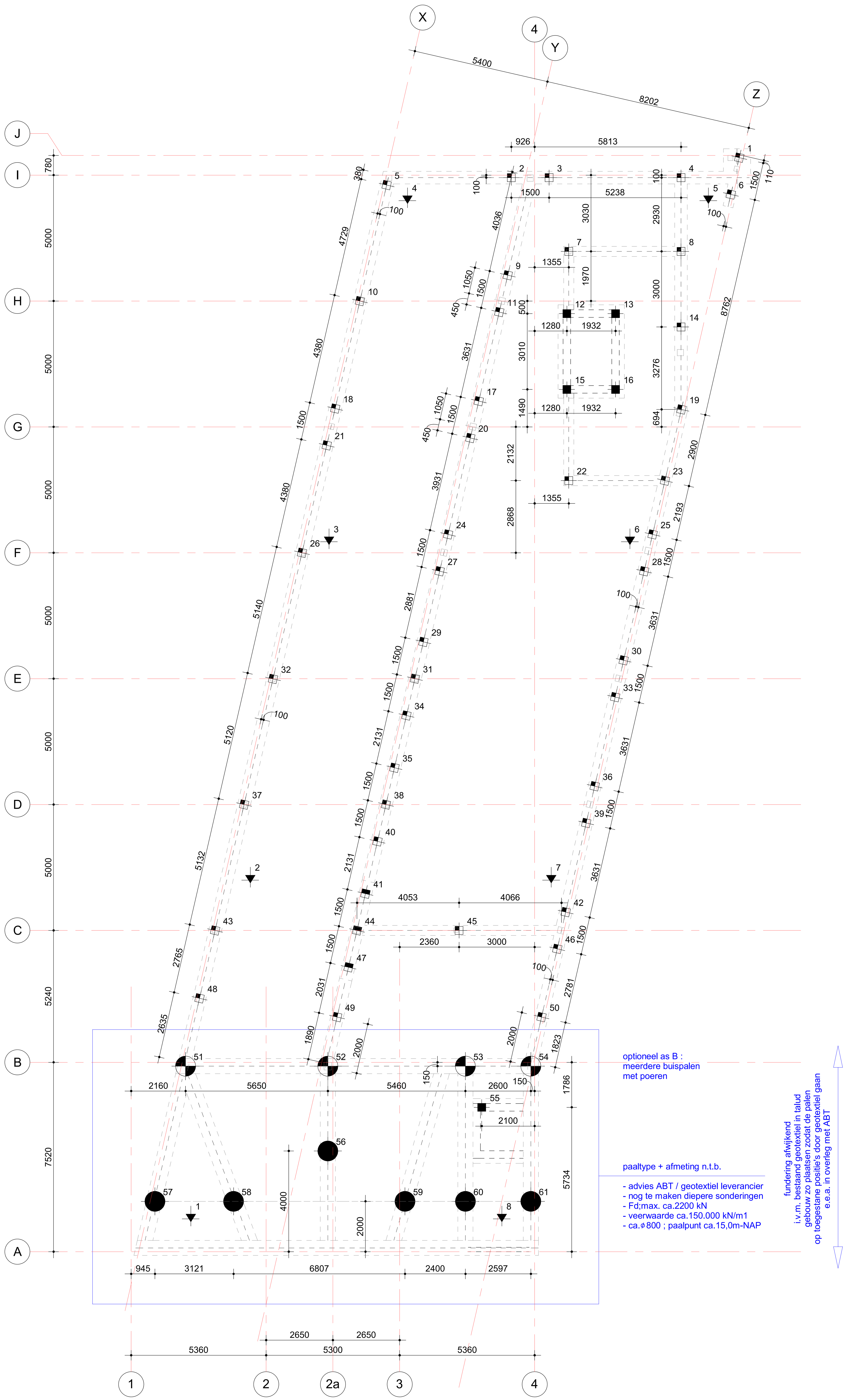
De monitoring dient te bestaan uit visuele inspecties, foto’s en deformatiemetingen aan de geluidswal. Deze metingen zijn minimaal metingen van de kruin en de teen van de geluidswal en de rand van het asfalt van de parallelbaan. Van deze monitoring/inspecties dienen verslagen te worden gemaakt.

De precisie invullen van de monitoring zal in afstemming met RWS gaan plaatsvinden.

3 Conclusie

De verschillende vragen en opmerkingen welke bij de beoordeling van de rapportage betreffende de stabiliteit van de grondwal langs de A2 bij werkzaamheden op het naastgelegen gebied Hooggelegen zijn in voorliggende notitie allen beantwoord en/of toegelicht.
Aandachtspunten welke betrekking hebben op de toekomstige werkzaamheden zullen door middel van een overdrachtsdocument worden gecommuniceerd met de realiserende partijen op het uit te geven kavel.

Bijlage 1 **Palenplan nieuwbouw**



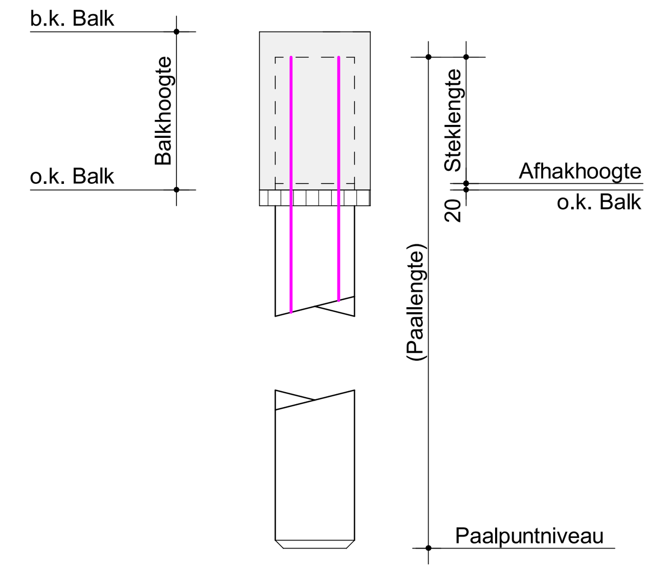
Peil = +6,70 m t.o.v. NAP

Palenstaat prefab betonpalen

type paal	aantal	P.P.N. tov NAP	b.k. paal t.o.v. NAP	b.k. paal t.o.v. peil	bestellengte	steklengte	afhakniveau tov Peil	Fd; druk
prefab betonpaal ø320mm								
	5	-7,000 m	5,750 m	-0,950 m	12,750 m	0,400 m	-1,480 m	660 kN
	3	-7,000 m	6,000 m	-0,700 m	13,000 m	0,400 m	-1,340 m	660 kN
	43	-7,000 m	6,250 m	-0,450 m	13,250 m	0,400 m	-1,040 m	660 kN
Totaal	51	niet dieper !						

Palenstaat boorpalen

type paal	aantal	P.P.N. tov NAP	afstortniveau tov Peil	steklengte	paallengte	Fd; druk
boorpaal ø800mm						
	6	-15,000 m	-2,240 m	400 mm	19,460 m	2200 kN
	4	-15,000 m	-1,640 m	400 mm	20,060 m	2200 kN
Totaal	10					



prefab betonpaal

sonderingen:
zie sondeerrapport — Hoogveld Geo
projectnummer — HA-17758 d.d. 21-04-2020
n.b. positie sonderingen bij benadering!

tekening status: voorlopig

Van Roekel Van Roekel
INGENIEURS IN CONSTRUCTIES

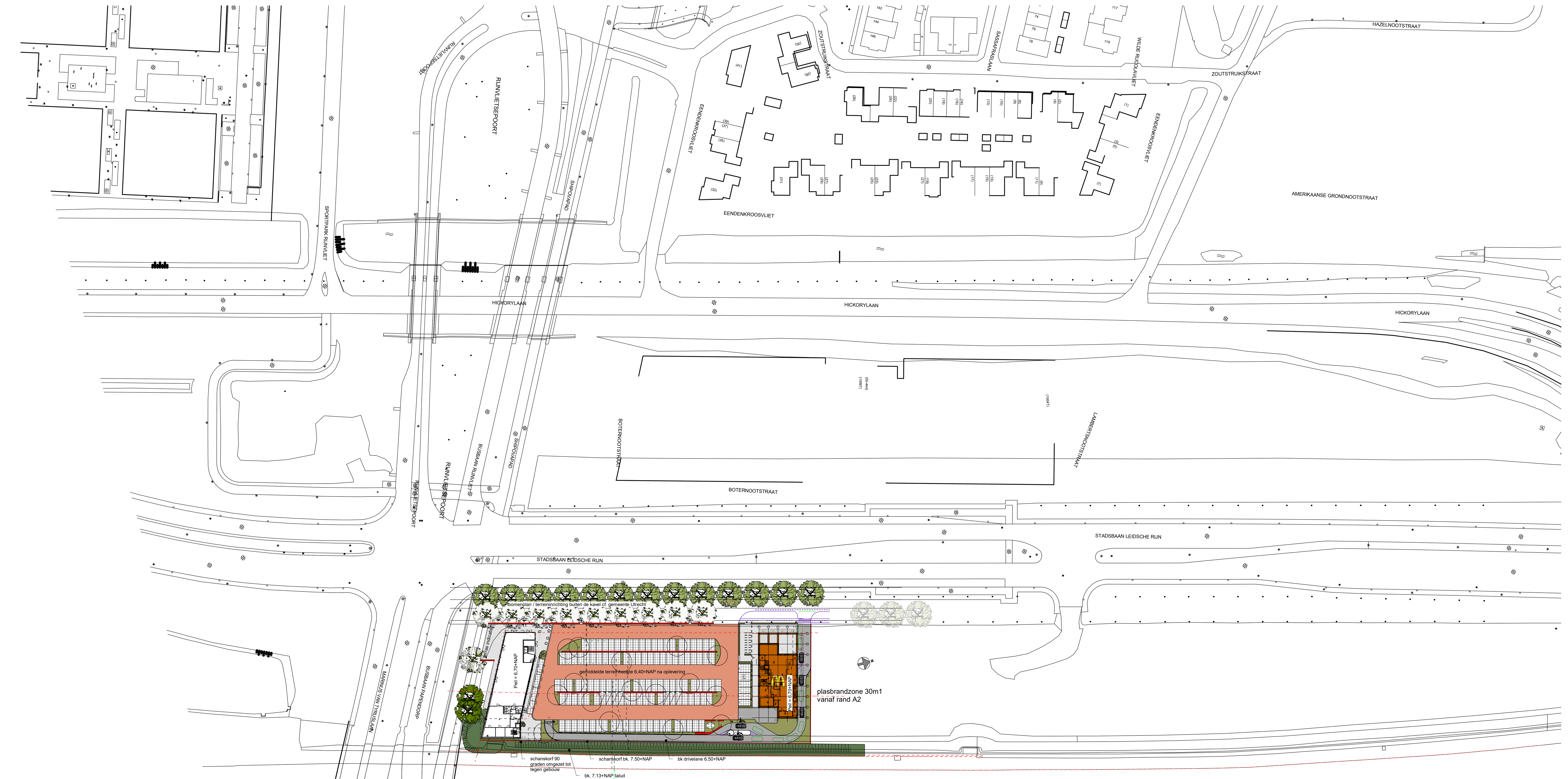
project
Restaurant Hooggelegen te Utrecht

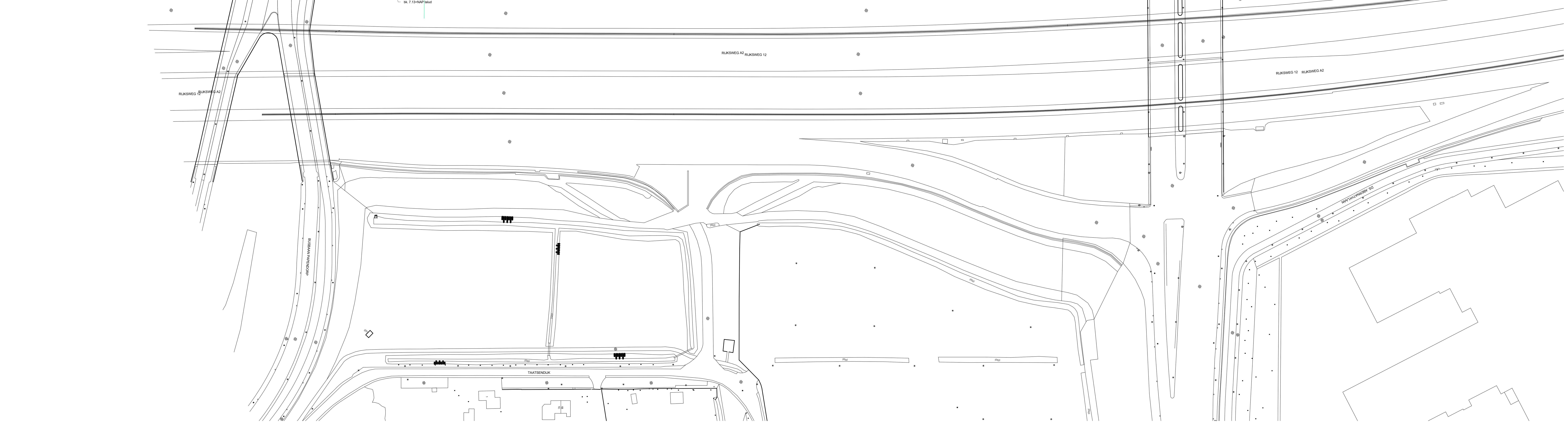
onderwerp
Palenplan

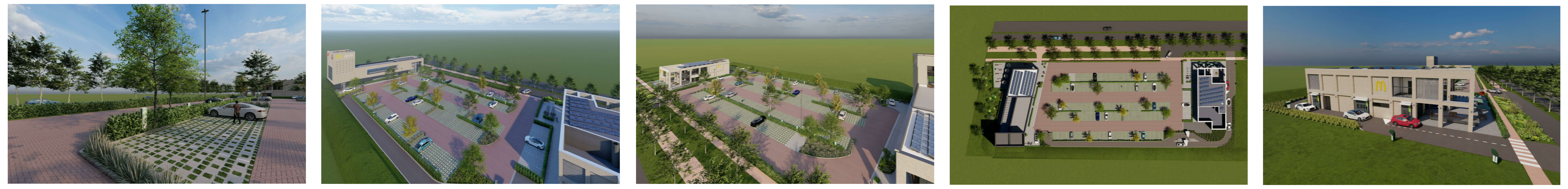
opdrachtgever: bouwbedrijf De Vree en Sliepen te Tiel

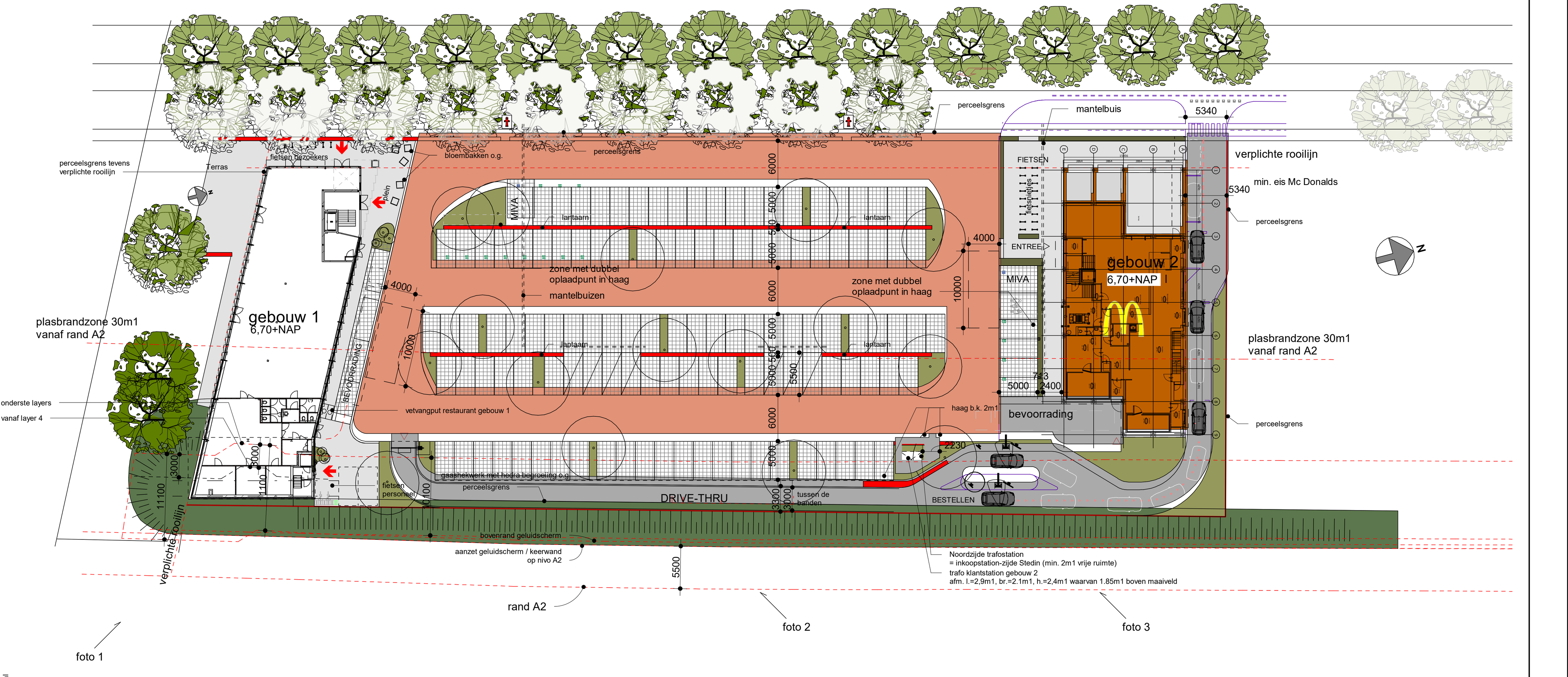
wijz.	:
tek.	:js 10-02-2022
constr.	:
schaal	:1:100
projectnr.	:37548
tekeningnr:	B-01
formaat	: A1

Van Roekel & Van Roekel
Wilhelminastraat 27; 3911MB Rhenen
telefoon (0317) 681100; e-mail: info@roekel.nl







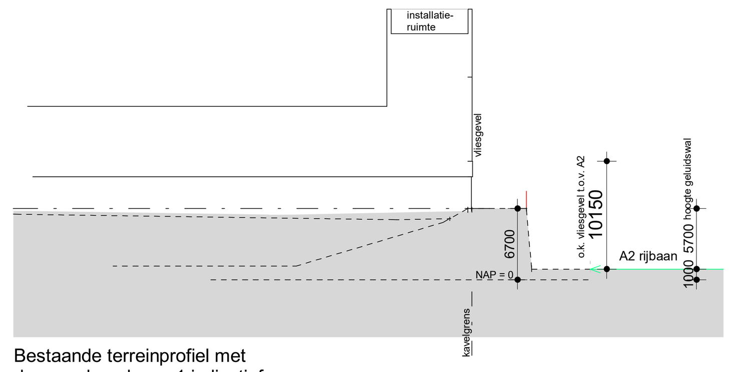


Renvooi brand gebouw 1

	brandweeringang
	voorstel locatie bluswatervoorziening - in overleg met bevoegde gezag en waterleidingbedrijf locatie nader vaststellen. - doorgang door haag voorzien.
	opstelplek brandweer voertuig

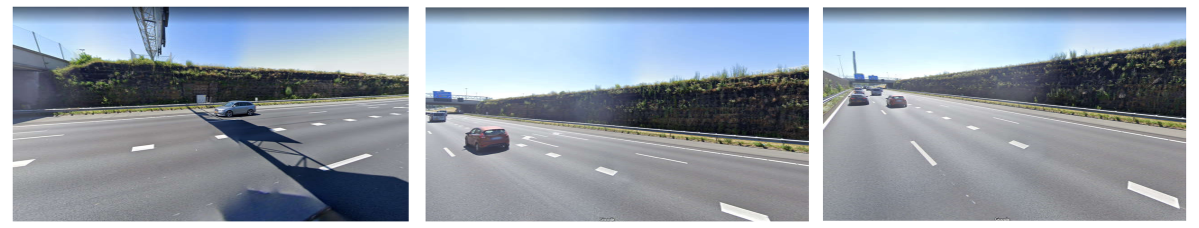
Renvooi terrein

	hagen cf. plan Donker Design
	laag groen cf. plan Donker Design
	bomen cf. plan Donker Design
	parkeerplek 2500x5000mm voorzien van grasbetonstenen Struyk Verwo model Square
	parkeerplek 2500x5000mm voorzien van grasbetonstenen Struyk Verwo model Square
	Struyk Verwo Infra standaard betonstenen kleur Standaard Rood
	Tegels Belpasso Premio firma Metten antracietgrijs
	Struyk Verwo Infra standaard betonstenen kleur Standaard Zwart
	principe verplichte mantelbuizen t.b.v. optioneel plaatsen oplaadpunten naar 1 op de 5 parkeerplaatsen (eis minimaal 1 oplaadpunt, aanwezig minimaal 3)
	zone's oplaadpunten voorzien van betontegels o.g. met symbool,
	minder valide (MIVA) parkeerplekken voorzien van betontegels o.g. met symbool t.p.v. MIVA en gezinsplekken voorzien van klinkervulling ipv gras ivm gebruik



Bestaande terreinprofiel met
doorsnede gebouw 1 indicatief

foto 1 foto 2 foto 3



PEIL gebouw 1 en 2 = 6.70 m1 + NAP

restaurant Hooggelegen **Utr003**
Definitief Ontwerpfase **BV000**

tekening	Situatie
datum	11-08-2021
versie	5: 14-02-2022
status	Definitief
schaal	1:500 / 1000
formaat	A2
getekend	BS

architectenbureau
visser en bouwman bv
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form : A1(1189x594mm)

Bijlage 2 **Rapportage NAUE**

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10 pages + 4 enclosures

March 22th 2022

Project: Utrecht Hooggelegen A2 geogrid - pile foundation advise
Here: Verification of the stability for the existing Secugrid® geogrid reinforced wall during and after completion of the construction of the pile foundation for the planned new building

Dear

BBG has been asked by you to give advises regarding the planned pile foundation which will affect the bottom geogrid layers of the existing geogrid reinforced wall structure and carry out a verification of the stability for the existing geogrid reinforced wall during and after completion of the pile foundation works for the above-mentioned project in Utrecht, Netherlands. The design works are to be coordinated and checked by the client (resp. third party organised by the client). It is the scope of the client (resp. third party organised by the client) in the design phase as well as on site to verify the completeness and harmonisation of all design works and health and safety requirements according to national regulations and state of the art, where this design is a limited part of.

1. Introduction

In 2009/2010 the A2 Hooggelegen project in Utrecht has been executed with reinforced soil using Secugrid® geogrids. The function of the geogrid reinforced structure was noise protection and it was built with an inclination of 70° on the motorway side and with a natural slope inclination on the side facing away from the road.

For the geogrid reinforced noise protection embankment constructed in 2009/2010 the facing system Naue Wrap was carried out. This facing system is considering a wrap-around of the geogrid layers on the facing side of the structure.

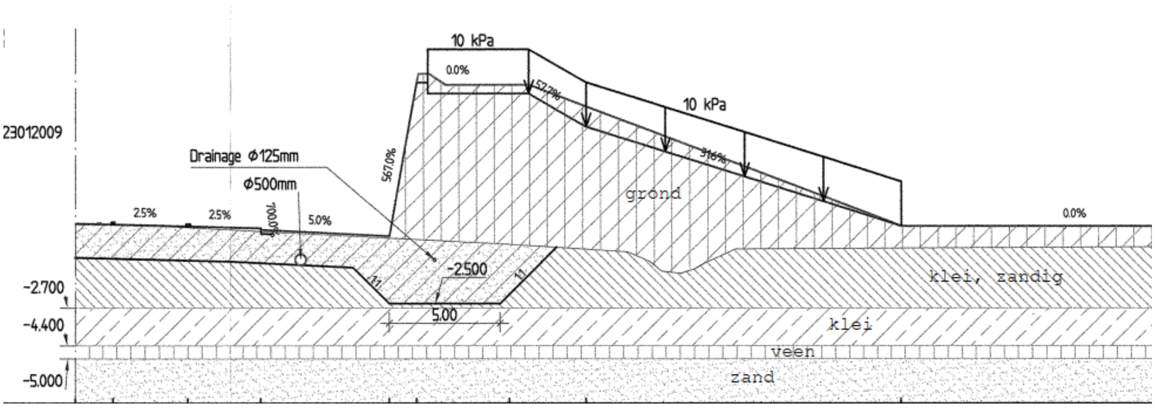

Figure 1: Cross-Section of the noise protection embankment as constructed in 2009/2010

In 2018, the area behind the noise protection embankment was filled up to the upper edge of the noise protection embankment. There are currently plans to construct a new building in the area of the fill from 2018. The building will be founded on piles.

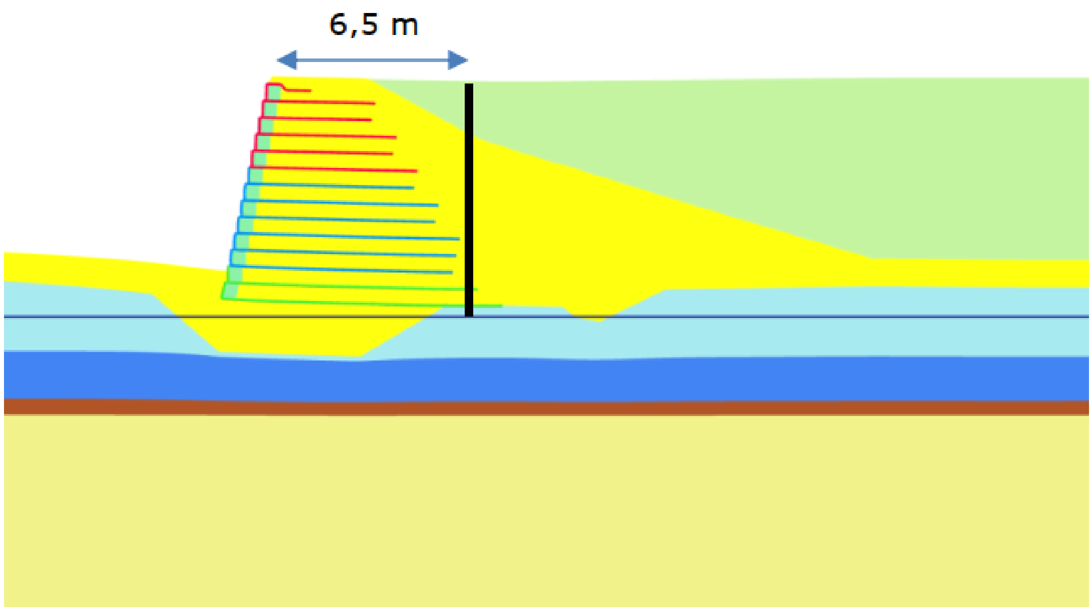

Figure 2: Cross-Section of the noise protection embankment with back fill from 2018

The first row of foundation piles next to the highway is projected at the noise protection embankment in such a way that in the first case the piles are 6.5 m away from the embankment edge and have to be drilled through the bottom five geogrid layers and in the second case the bottom three layers will be penetrated.

The second case is the area of the third pile from left hand (see Figure 3), which is placed back 2 meters extra due to the location of the indenting for the WKS. The indenting for the WKS is positioned on the right side of the gantry (road sign structure) (see Figure 4).

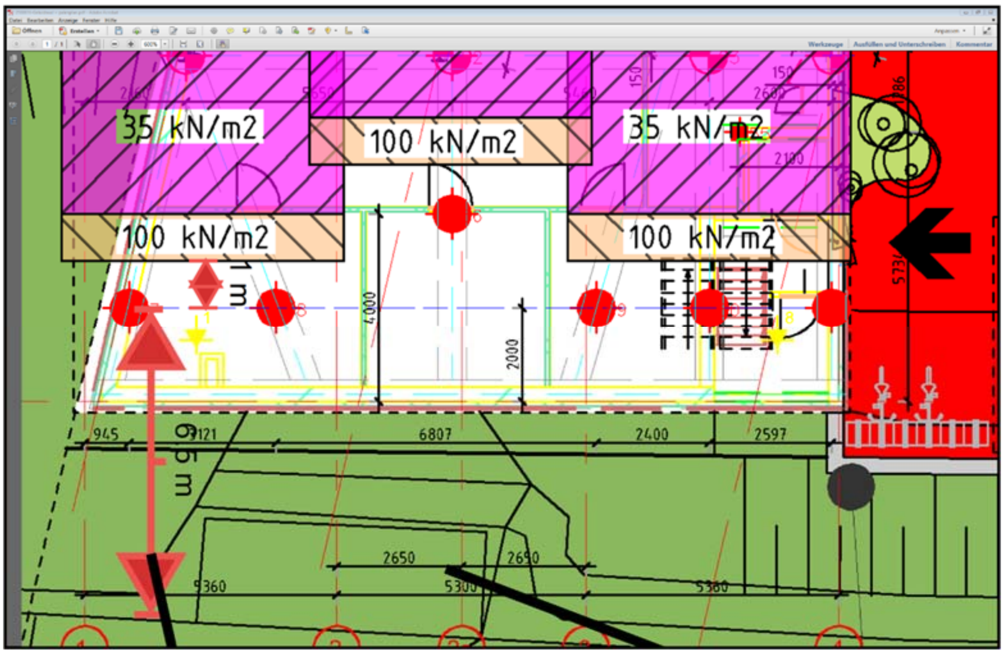

Figure 3: Layout plan for the first row of foundation piles

Figure 4: Position of the indenting for the WKS

The Secugrid® 120/40 R6 geogrid layers will be locally penetrated by the planned pile foundation. Therefore, a separate reduction factor (A_2') must be taken into account for these locations in order to account for the local weakening.

The reduction factor A_2' results from the ratio of the building width and the total sum of the pile diameters. The reduction factor results from the percentage of the pile diameters to the total width of the considered area. The determination of the Secugrid® design strength for the geogrid layers under consideration of the extra partial reduction factor A_2' follows the design principles of EBGEO (2010) [2] is given in Enclosure [E3].

Within the scope of this design, the structure is analysed for its internal, global and external stability, taking into account the additional loads and the weakening of the geogrids caused by the pile foundation in the temporary and final state.

For this purpose, the structure is designed for permanent condition (DS-P design) and the loads caused by the construction of the pile foundation (DS-T design). The results for ultimate limit state (ULS) analysis are provided and discussed in this statement.

2. Soil Parameters & Loading Conditions

According to the project information in [1], the design is carried out based upon the minimum shear parameters of the subgrade and fill soil. The soil characteristics including the shear strengths and unit weights of the underground soil present on site, fill soil and backfill soil are summarised in table 2 below.

Table 2: Subsoil characteristics considered in this design according to [1]

Soil	$\varphi_{,k}$ [°]	$c_{,k}$ [kN/m²]	$\gamma_{,k}$ [kN/m³]	max ψ_A [°]	Designation
	32.50	1.00	18.00	75.00	Reinforced Fill
	22.50	2.00	17.00	75.00	Clay, sandy
	17.50	2.00	15.00	75.00	Clay
	15.00	2.00	13.00	75.00	Peat
	30.00	0.00	20.00	75.00	Sand
	27.00	0.00	17.00	75.00	Backfill

It has to be assured that the given information regarding the shear parameters and unit weights, as used in this design, are representative for the soil types have been used and on site. In case that lower shear parameters are examined, a new design will have to be carried out.

On top of the reinforced earth structure a uniform live load with magnitude of 10 kPa has been considered for the final state. For the transient design situation uniform live loads of 35 kN/m² and 100 kN/m² were considered according to [1] as shown in Figure 3.

Seismic actions have not been considered in this design.

3. Ultimate Limit State (ULS) Design

The following partial factors have been used for the Ultimate Limit State design according to DIN 1054:2021 [3]:

Permanent condition (DS-P):

Reinforcement Material Factors:		= see Enclosure [E3]
Soil Strength:	Applied to tan ϕ	= 1.25
	Applied to c_u	= 1.25
	Applied to c’	= 1.25
Soil Weight:		= 1.00
Pull Out Resistance:		= 1.40
Direct Sliding:		= 1.10
Surcharge Loads (GEO-3):	Dead Load	= 1.00
	Live Load	= 1.30
Surcharge Loads (GEO-2):	Dead Load	= 1.35
	Live Load	= 1.50

Transient design (DS-T):

Reinforcement Material Factors:		= see Enclosure [E3]
Soil Strength:	Applied to tan ϕ	= 1.15
	Applied to c_u	= 1.15
	Applied to c’	= 1.15
Soil Weight:		= 1.00
Pull Out Resistance:		= 1.30
Direct Sliding:		= 1.30
Surcharge Loads (GEO-3):	Dead Load	= 1.00
	Live Load	= 1.20
Surcharge Loads (GEO-2):	Dead Load	= 1.20
	Live Load	= 1.30

The design is carried out using GGU STABILITY design software for the Ultimate Limit State design to determine the global safety factor of the reinforced soil structure. The results for both analyses for “Internal and Global Stability” (Bishop-Slip-Circle analyses) and “Internal and External Stability” (Two-Part-Wedge-Method) are performed and shown in Enclosures [E1] and [E2] respectively for cross sections 1 to 2.

According to [3] & [4] circular and planar (Slices Method according to Bishop and Two-Part Wedge Method) failure mechanisms have been examined as well as the proof of sliding, overturning, bearing failure and eccentricity. The following design is based on limit state principles for the Ultimate Limit State (ULS) according to [2]. The Ultimate Limit State is associated with collapse or

other similar forms of structural failure (STR + GEO). This state is attained, for a specific mode of failure, when disturbing forces equal or exceed restoring forces. Margins of safety, against attaining the Limit State of collapse are provided by the use of partial material resistance factors and partial load factors (as documented in [E1], [E2] and [E3]).
The stability of the Secugrid® reinforced embankment has been investigated regarding the permanent load case DS-P and the transient load case DS-T.
Disturbing forces are increased by multiplying with prescribed load factors to produce design loads. Restoring forces are decreased by dividing with prescribed material factors to produce design strengths. If the design strength equals or exceeds the design load then there is deemed to be an adequate margin of safety against attaining the Ultimate Limit State of collapse.

The design is carried out using STABILITY design software which is distributed by the company GGU/Germany (Version 13.25). The most critical failure plains which have been achieved during the examination of the Internal, External & Global Stability Analyses are shown in Enclosures [E1] and [E2].
The examination of all failure mechanisms leads to satisfactory results of $\mu_{max.} \leq 1.0$ in the Ultimate Limit State.

4. Penetration of Secugrid layers with pile foundation

According to the information in [1], the planned pile foundations will be constructed in a vibration-free soil replacement procedure using an auger. The diameter of the piles is 800mm.
Experience has shown that with a sufficiently contact pressure (soil ballast >1.0m) on the Secugrid® geogrid layers, there is no danger of the geogrid bars winding up on the auger and being pulled out of the surrounding soil. Due to the sufficiently high ground load, the resulting retention force in the ground is sufficiently high, which leads to the geogrid bars being cut within or at the edge of the borehole.
A risk of damage or unacceptable deformation of the facing of the geogrid reinforced structure is considered to be very low for the following reasons:

- The pile foundation method is vibration-free and is carried out using the soil replacement method rather than the soil displacement method.
- The distance between the slope edge and the pile axis is at least 6.50m.
- The soil ballast on the affected geogrid layers is significantly more than 1.0m. The minimum thickness of the soil ballast is 6.0m.

Two project references are presented in the following chapter.

5. Project references

a. Werdau bypass

In 2008, the Werdau bypass was built for the S239 road. In the framework of this project, a steep embankment was built as a geogrid reinforced earth structure. For the foundation of the noise protection wall planned on top of the geogrid reinforced earth structure, steel casing was installed using the Cased Continuous Flight Auger drilling method. The steel beams of the noise protection wall were later concreted into the steel casing.

Photo 1: Steel casing installed into the geogrid reinforced earth structure

Photo 2: Reinforced slope with noise protection wall on top, Google Earth screen shot

b. Baden-Baden

The second reference project is a geogrid reinforced slope project in Baden-Baden, constructed in 2004.

For a planned structure on top of the geogrid reinforced slope a concrete pile foundation was required. The pile foundation was constructed using the Continuous Flight Auger drilling method.

Photo 3-5: Geogrid reinforced slope, drilling machine and reinforcement for the concrete column

The Photo 5 is showing very well that the geogrid bars are ruptured within or at the edge of the borehole. The geogrid structure within the soil remains undamaged.

6. Final Comments

In this design, the stability of the existing geogrid reinforced earth structure under the influence of the planned pile foundation was verified in the transient load situation and the permanent load situation. It could be shown that despite the weakening of the lower geogrid layers, the required stability for the construction is given.

Based on the experience gained in other projects, we can conclude that the boundary conditions in this project are not critical for the penetration of the geogrid layers. The soil ballast on top of the geogrid layers is significantly higher than required and the distance to the embankment shoulder is relatively large with at least 6.50m.

Taking into account the current boundary conditions, there is in our view nothing to object to the application of the planned pile foundation method.

Should there be any changes to the boundary conditions considered here, then the feasibility assessment would have to be carried out again.

If you have questions concerning this design, please do not hesitate to contact us.

With best regards,

BBG - Bauberatung Geokunststoffe
GmbH & Co. KG

Project related information and standards

[1] Email from Mr. Marco Kimenai (ABT B.V.) with project related information, from the 23rd of Dezember 2021 and 10th of March 2022

[2] EBGEO, 2010: Recommendations for the Design of soil structures using geosynthetic reinforcement: Working Group 5.2 of the German Society for Geotechnics (DGGT)

[3] DIN 1054:2021-04. Verification of the safety of earthworks and foundations – Supplementary rules to DIN EN 1997-1.

[4] DIN 4084:2021-11: Sliding and slope failure calculations

Enclosures

[E1] Proof of stability – Reinforced soil structure

[E2] Proof of stability – Reinforced soil structure – Area of indenting for WKS

[E3] Long-term design strength for Secugrid® R6

[E4] Data sheet: Secugrid® R6 Product Range (NAUE GmbH & Co. KG)

Calculation: Internal & Global Stability - Bishop Slip Circle Analysis - DS-P

Section/Detail: Grondwal - Final situation

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Project: Utrecht Hooggelegen A2 geogrid pile foundation advise

Designer:

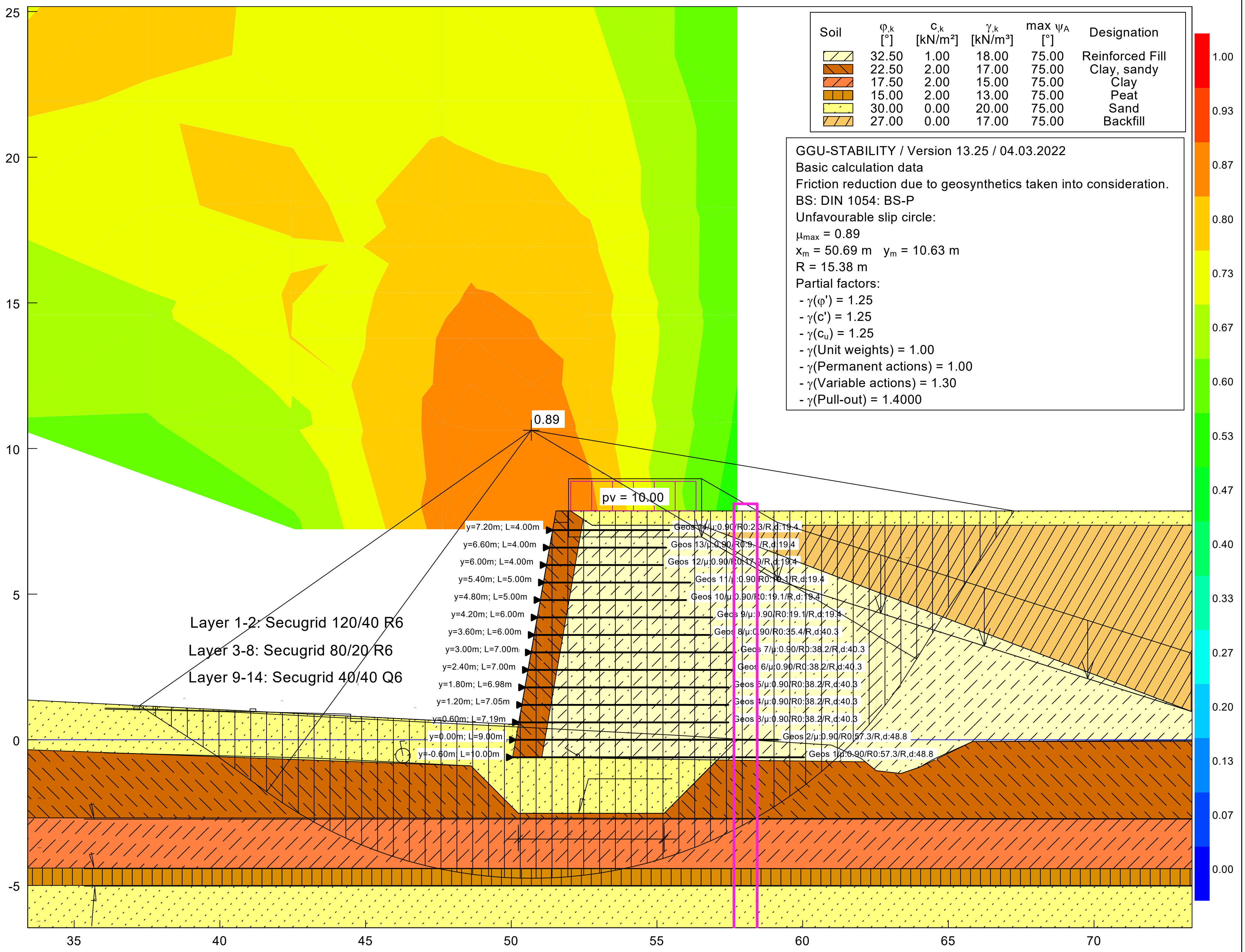
Reviewer:

Rev-No: --

Scale:

Date: 22.03.2022

Enclosure: 1.1



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Calculation: Internal & External Stability - Geeral Wedge Analysis - DS-P

Section/Detail: Grondwal - Final situation

Project: Utrecht Hooggelegen A2 geogrid pile foundation advise

Designer:

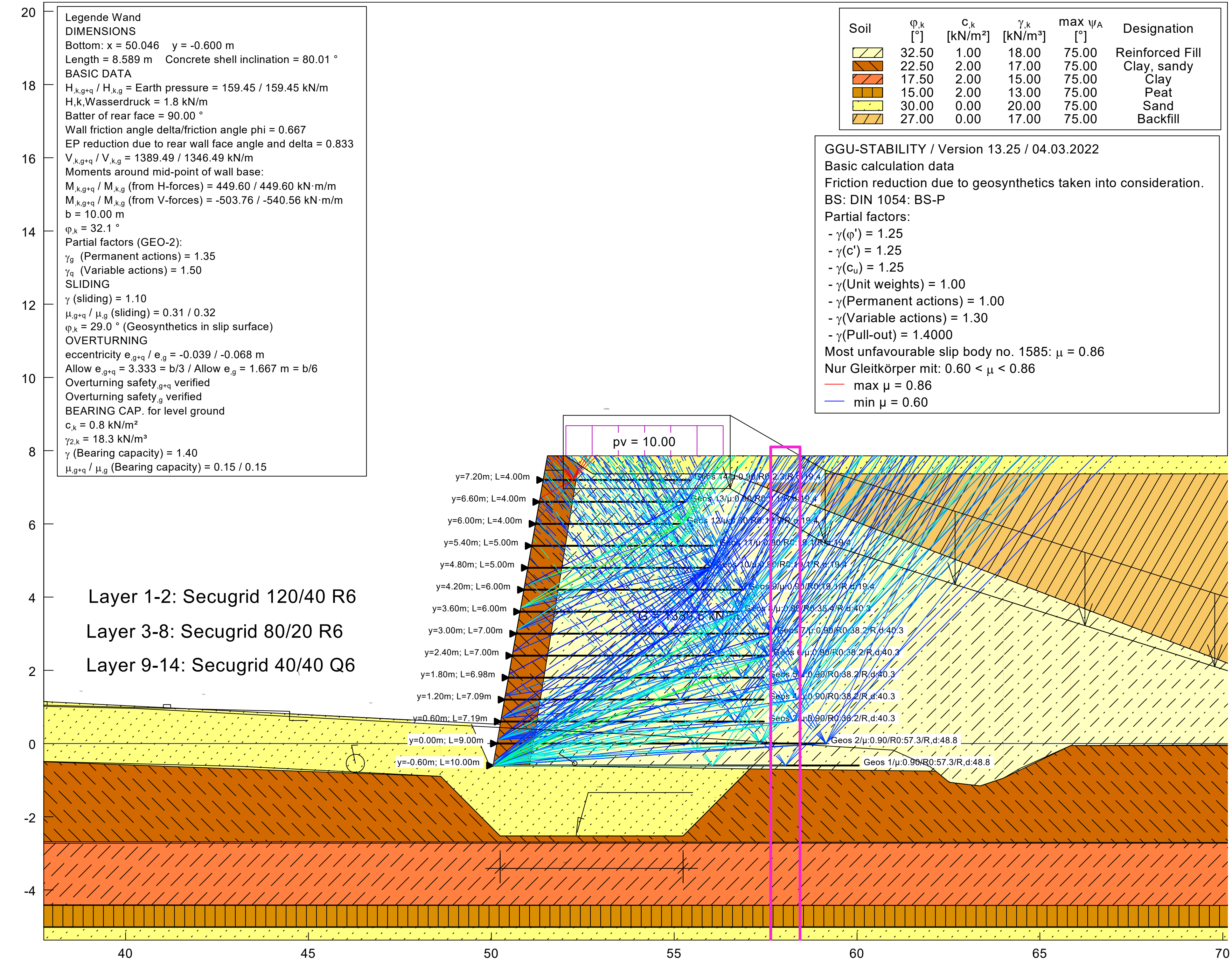
Reviewer:

Rev-No: --

Date: 22.03.2022

Scale:

Enclosure: 1.2



Legende Wand
DIMENSIONS
Bottom: x = 50.046 y = -0.600 m
Length = 8.589 m Concrete shell inclination = 80.01 °
BASIC DATA
$H_{,k,g+q}$ / $H_{,k,g}$ = Earth pressure = 159.45 / 159.45 kN/m
H,k,Wasserdruck = 1.8 kN/m
Batter of rear face = 90.00 °
Wall friction angle delta/friction angle phi = 0.667
EP reduction due to rear wall face angle and delta = 0.833
$V_{,k,g+q}$ / $V_{,k,g}$ = 1389.49 / 1346.49 kN/m
Moments around mid-point of wall base:
$M_{,k,g+q}$ / $M_{,k,g}$ (from H-forces) = 449.60 / 449.60 kN·m/m
$M_{,k,g+q}$ / $M_{,k,g}$ (from V-forces) = -503.76 / -540.56 kN·m/m
b = 10.00 m
$\varphi_{,k}$ = 32.1 °
Partial factors (GEO-2):
γ_g (Permanent actions) = 1.35
γ_q (Variable actions) = 1.50
SLIDING
γ (sliding) = 1.10
$\mu_{,g+q}$ / $\mu_{,g}$ (sliding) = 0.31 / 0.32
$\varphi_{,k}$ = 29.0 ° (Geosynthetics in slip surface)
OVERTURNING
eccentricity $e_{,g+q}$ / $e_{,g}$ = -0.039 / -0.068 m
Allow $e_{,g+q}$ = 3.333 = b/3 / Allow $e_{,g}$ = 1.667 m = b/6
Overturning safety$_{,g+q}$ verified
Overturning safety$_{,g}$ verified
BEARING CAP. for level ground
$c_{,k}$ = 0.8 kN/m²
$\gamma_{2,k}$ = 18.3 kN/m³
γ (Bearing capacity) = 1.40
$\mu_{,g+q}$ / $\mu_{,g}$ (Bearing capacity) = 0.15 / 0.15

Soil	$\varphi_{,k}$ [°]	$c_{,k}$ [kN/m²]	$\gamma_{,k}$ [kN/m³]	max ψ_A [°]	Designation
	32.50	1.00	18.00	75.00	Reinforced Fill
	22.50	2.00	17.00	75.00	Clay, sandy
	17.50	2.00	15.00	75.00	Clay
	15.00	2.00	13.00	75.00	Peat
	30.00	0.00	20.00	75.00	Sand
	27.00	0.00	17.00	75.00	Backfill

GGU-STABILITY / Version 13.25 / 04.03.2022
Basic calculation data
Friction reduction due to geosynthetics taken into consideration.
BS: DIN 1054: BS-P
Partial factors:
- $\gamma(\varphi')$ = 1.25
- $\gamma(c')$ = 1.25
- $\gamma(c_u)$ = 1.25
- γ(Unit weights) = 1.00
- γ(Permanent actions) = 1.00
- γ(Variable actions) = 1.30
- γ(Pull-out) = 1.4000
Most unfavourable slip body no. 1585: μ = 0.86
Nur Gleitkörper mit: 0.60 < μ < 0.86
max μ = 0.86
min μ = 0.60

Calculation: Internal & Global Stability - Bishop Slip Circle Analysis - DS-T

Section/Detail: Grondwal - Driving & Drilling

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Project: Utrecht Hooggelegen A2 geogrid pile foundation advise

Designer:

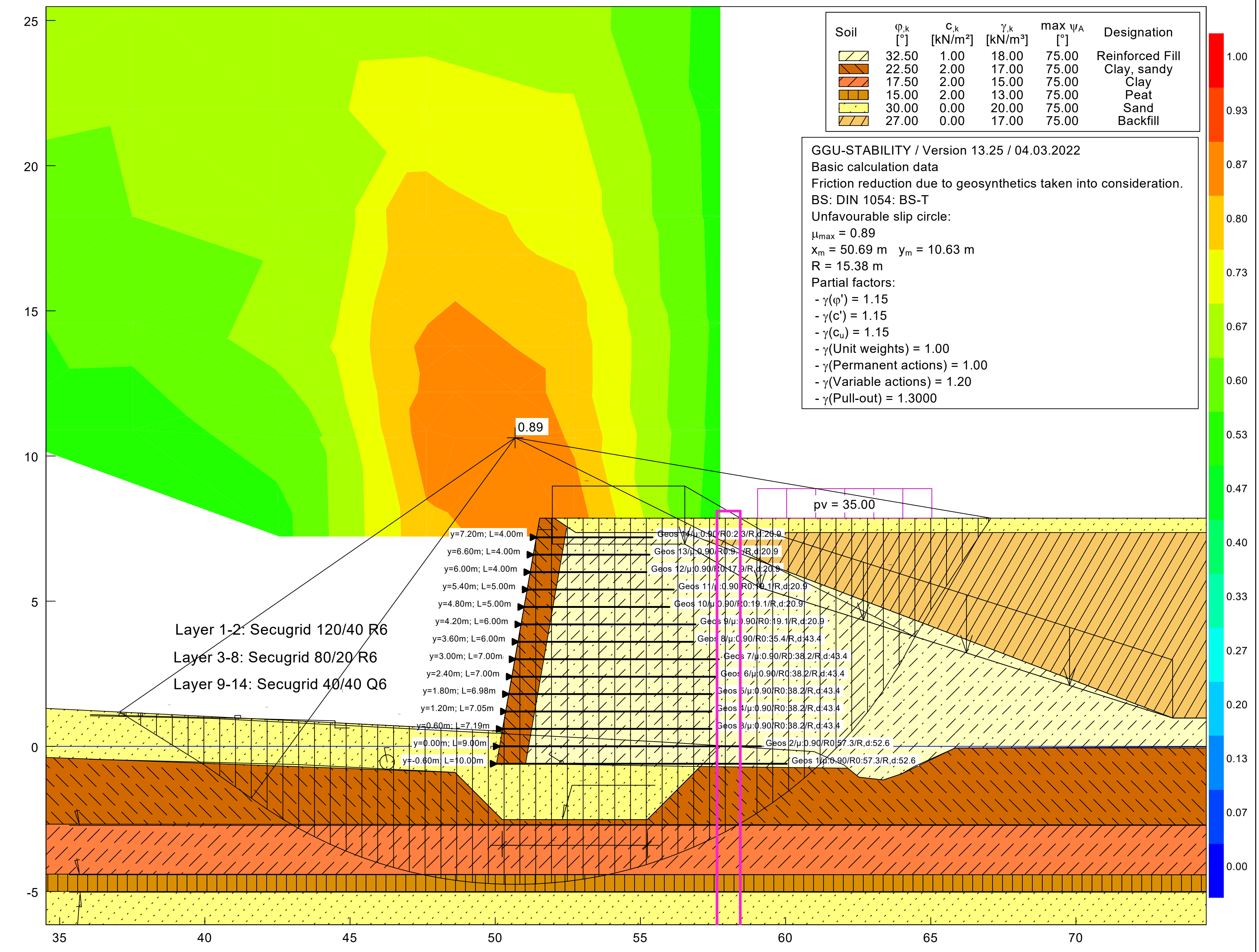
Reviewer:

Rev-No: --

Scale:

Date: 22.03.2022

Enclosure: 1.3



Calculation: Internal & External Stability - Geeral Wedge Analysis - DS-T

Section/Detail: Grondwal - Driving &Drilling

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Project: Utrecht Hooggelegen A2 geogrid pile foundation advise

Designer:

Rev-No: --

Reviewe

Scale:

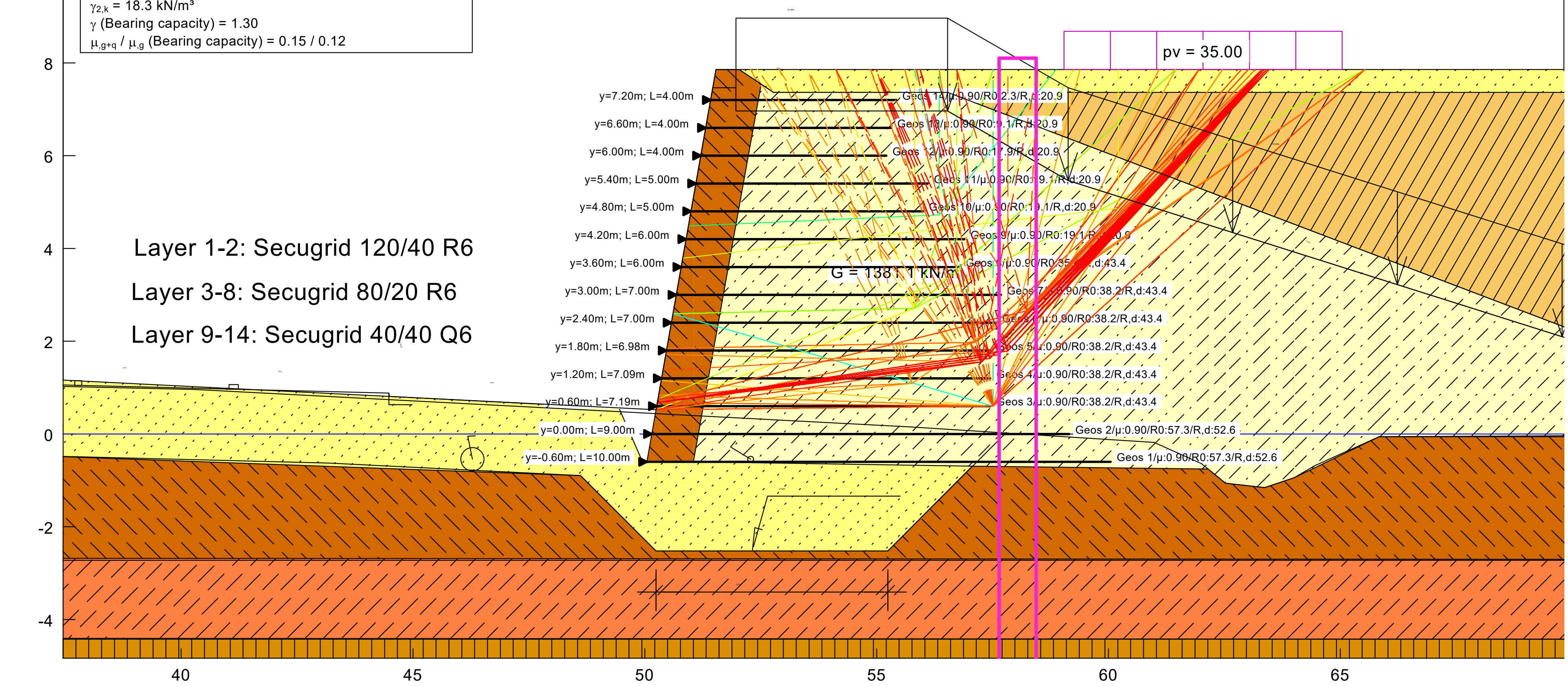
Enclosure: 1.4

Date: 22.03.2022

Legende Wand
DIMENSIONS
Bottom: x = 50.046 y = -0.600 m
Length = 8.589 m Concrete shell inclination = 80.01 °
BASIC DATA
$H_{,k,g+q}$ / $H_{,k,g}$ = Earth pressure = 240.98 / 160.95 kN/m
Batter of rear face = 90.00 °
Wall friction angle delta/friction angle phi = 0.667
EP reduction due to rear wall face angle and delta = 0.833
$V_{,k,g+q}$ / $V_{,k,g}$ = 1381.14 / 1345.93 kN/m
Moments around mid-point of wall base:
$M_{,k,g+q}$ / $M_{,k,g}$ (from H-forces) = 679.48 / 453.82 kN·m/m
$M_{,k,g+q}$ / $M_{,k,g}$ (from V-forces) = -700.63 / -542.28 kN·m/m
b = 10.00 m
$\varphi_{,k}$ = 32.1 °
Partial factors (GEO-2):
γ_g (Permanent actions) = 1.20
γ_q (Variable actions) = 1.30
SLIDING
γ (sliding) = 1.10
$\mu_{,g+q}$ / $\mu_{,g}$ (sliding) = 0.43 / 0.29
$\varphi_{,k}$ = 29.0 ° (Geosynthetics in slip surface)
OVERTURNING
eccentricity $e_{,g+q}$ / $e_{,g}$ = -0.015 / -0.066 m
Allow $e_{,g+q}$ = 3.333 = b/3 / Allow $e_{,g}$ = 1.667 m = b/6
Overturning safety$_{,g+q}$ verified
Overturning safety$_{,g}$ verified
BEARING CAP. for level ground
$c_{,k}$ = 0.8 kN/m²
$\gamma_{2,k}$ = 18.3 kN/m³
γ (Bearing capacity) = 1.30
$\mu_{,g+q}$ / $\mu_{,g}$ (Bearing capacity) = 0.15 / 0.12

Soil	$\varphi_{,k}$ [°]	$c_{,k}$ [kN/m²]	$\gamma_{,k}$ [kN/m³]	max ψ_A [°]	Designation
	32.50	1.00	18.00	75.00	Reinforced Fill
	22.50	2.00	17.00	75.00	Clay, sandy
	17.50	2.00	15.00	75.00	Clay
	15.00	2.00	13.00	75.00	Peat
	30.00	0.00	20.00	75.00	Sand
	27.00	0.00	17.00	75.00	Backfill

GGU-STABILITY / Version 13.25 / 04.03.2022
Basic calculation data
Friction reduction due to geosynthetics taken into consideration.
BS: DIN 1054: BS-T
Partial factors:
- $\gamma(\varphi')$ = 1.15
- $\gamma(c')$ = 1.15
- $\gamma(c_u)$ = 1.15
- γ(Unit weights) = 1.00
- γ(Permanent actions) = 1.00
- γ(Variable actions) = 1.20
- γ(Pull-out) = 1.3000
Most unfavourable slip body no. 85: μ = 0.66
Nur Gleitkörper mit: 0.30 < μ < 0.66
max μ = 0.66
min μ = 0.30



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Project: Utrecht Hooggelegen A2 geogrid pile foundation advise

Section/Detail: Grondwal - Pull back of the steel tube

Calculation: Internal & Global Stability - Bishop Slip Circle Analysis - DS-T

Designer:

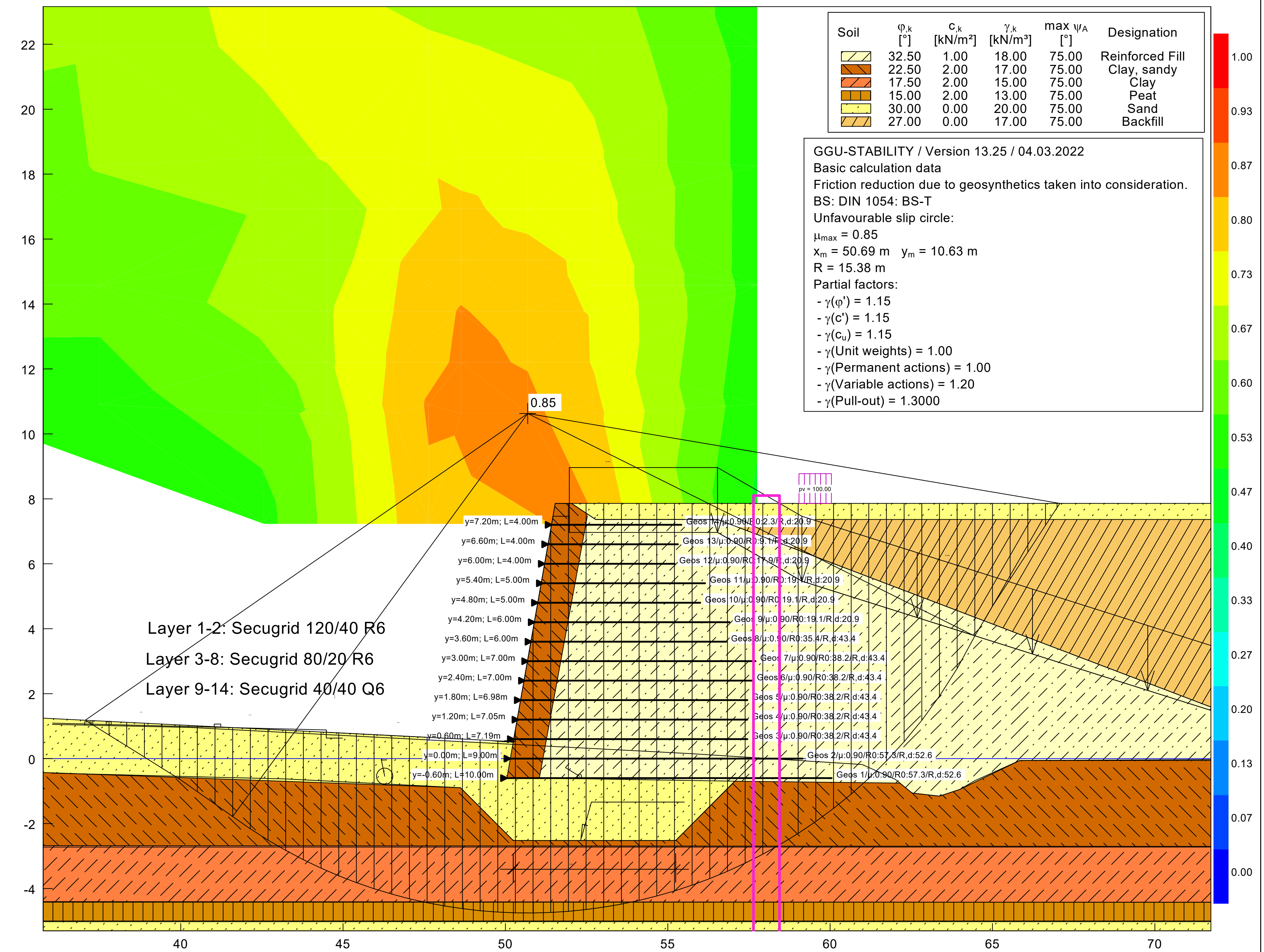
Reviewer:

Rev-No: --

Date: 22.03.2022

Scale:

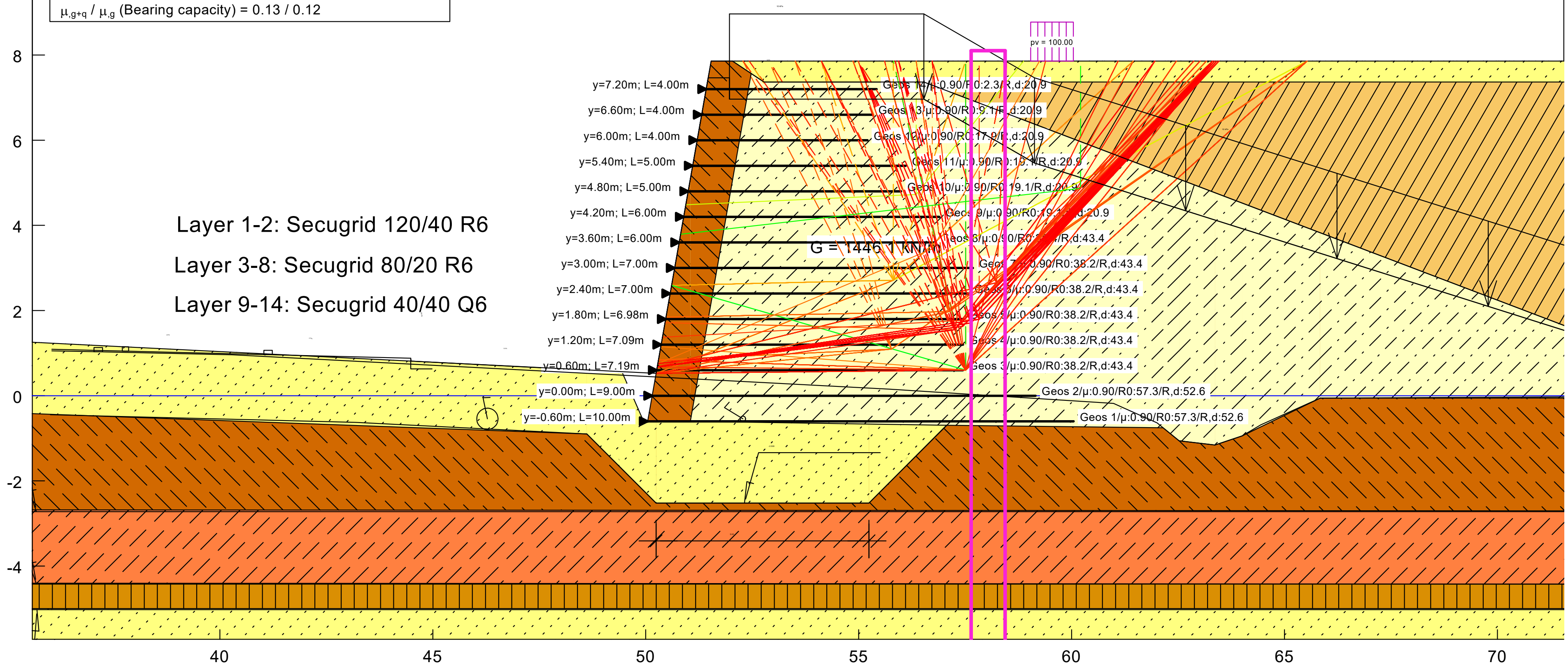
Enclosure: 1.5



Legende Wand
DIMENSIONS
Bottom: x = 50.046   y = -0.600 m
Length = 8.589 m   Concrete shell inclination = 80.01 °
BASIC DATA
$H_{,k,g+q}$ / $H_{,k,g}$ = Earth pressure = 160.95 / 160.95 kN/m
Batter of rear face = 90.00 °
Wall friction angle delta/friction angle phi = 0.667
EP reduction due to rear wall face angle and delta = 0.833
$V_{,k,g+q}$ / $V_{,k,g}$ = 1446.08 / 1346.08 kN/m
Moments around mid-point of wall base:
$M_{,k,g+q}$ / $M_{,k,g}$ (from H-forces) = 453.82 / 453.82 kN·m/m
$M_{,k,g+q}$ / $M_{,k,g}$ (from V-forces) = -991.19 / -541.81 kN·m/m
b = 10.00 m
$\varphi_{,k}$ = 32.1 °
Partial factors (GEO-2):
γ_g (Permanent actions) = 1.20
γ_q (Variable actions) = 1.30
SLIDING
γ (sliding) = 1.10
$\mu_{,g+q}$ / $\mu_{,g}$ (sliding) = 0.27 / 0.29
$\varphi_{,k}$ = 29.0 ° (Geosynthetics in slip surface)
OVERTURNING
eccentricity $e_{,g+q}$ / $e_{,g}$ = -0.372 / -0.065 m
Allow $e_{,g+q}$ = 3.333 = b/3 / Allow $e_{,g}$ = 1.667 m = b/6
Overturning safety$_{,g+q}$ verified
Overturning safety$_{,g}$ verified
BEARING CAP. for level ground
$c_{,k}$ = 0.8 kN/m²
$\gamma_{2,k}$ = 18.3 kN/m³
γ (Bearing capacity) = 1.30
$\mu_{,g+q}$ / $\mu_{,g}$ (Bearing capacity) = 0.13 / 0.12

Soil	$\varphi_{,k}$ [°]	$c_{,k}$ [kN/m²]	$\gamma_{,k}$ [kN/m³]	max ψ_A [°]	Designation
	32.50	1.00	18.00	75.00	Reinforced Fill
	22.50	2.00	17.00	75.00	Clay, sandy
	17.50	2.00	15.00	75.00	Clay
	15.00	2.00	13.00	75.00	Peat
	30.00	0.00	20.00	75.00	Sand
	27.00	0.00	17.00	75.00	Backfill

GGU-STABILITY / Version 13.25 / 04.03.2022
Basic calculation data
Friction reduction due to geosynthetics taken into consideration.
BS: DIN 1054: BS-T
Partial factors:
- $\gamma(\varphi')$ = 1.15
- $\gamma(c')$ = 1.15
- $\gamma(c_u)$ = 1.15
- γ(Unit weights) = 1.00
- γ(Permanent actions) = 1.00
- γ(Variable actions) = 1.20
- γ(Pull-out) = 1.3000
Most unfavourable slip body no. 89: μ = 0.64
Nur Gleitkörper mit: 0.20 < μ < 0.64
max μ = 0.64
min μ = 0.20



Calculation: Internal & Global Stability - Bishop Slip Circle Analysis - DS-P

Section/Detail: Grondwal - indenting for WKS - Final situation

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Project: Utrecht Hooggelegen A2 geogrid pile foundation advise

Designer:

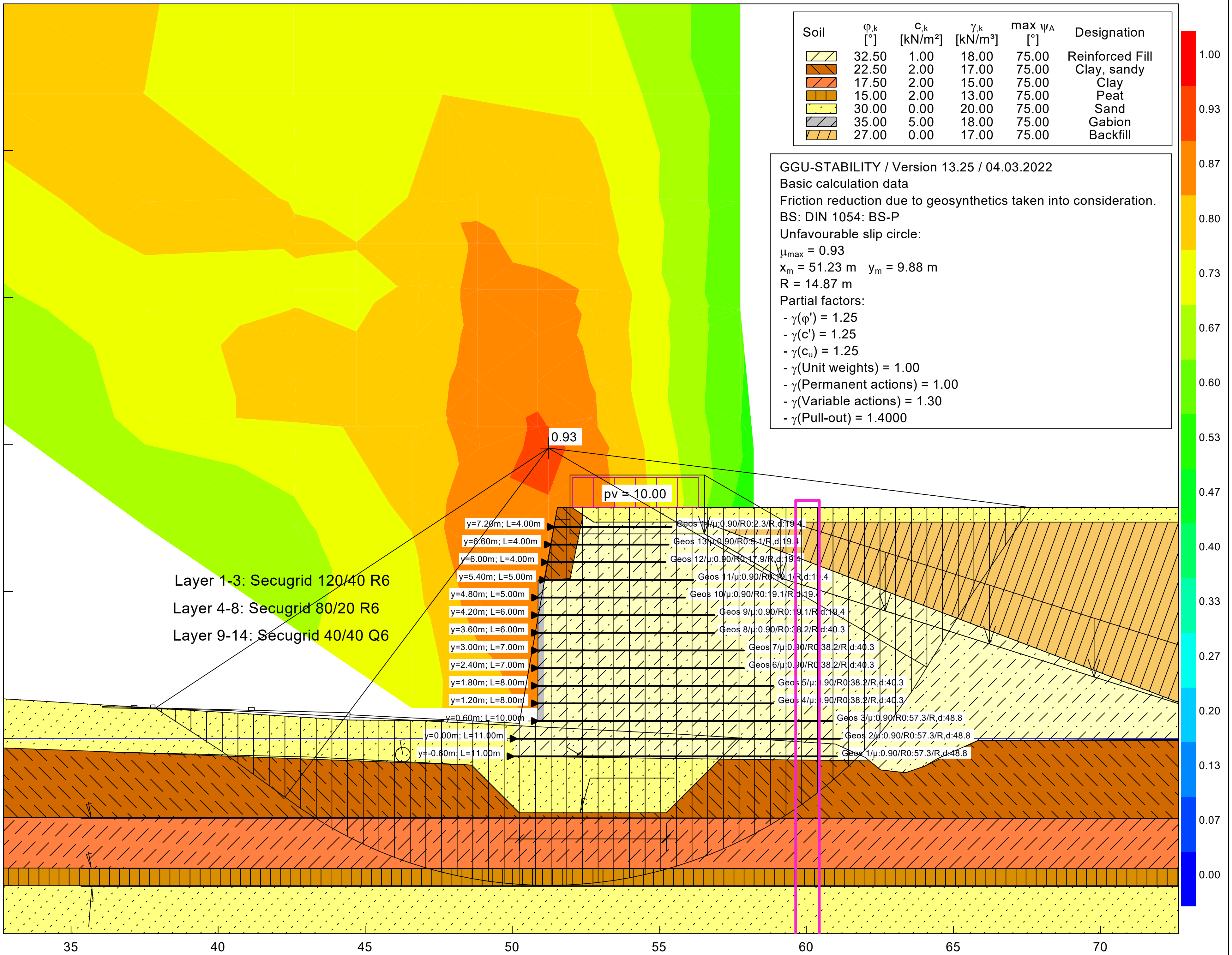
Reviewer:

Rev-No: --

Date: 22.03.2022

Scale:

Enclosure: 2.1



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Calculation: Internal & External Stability - General Wedge Analysis - DS-P

Section/Detail: Grondwal - indenting for WKS - Final situation

Project: Utrecht Hooggelegen A2 geogrid pile foundation advise

Designer:

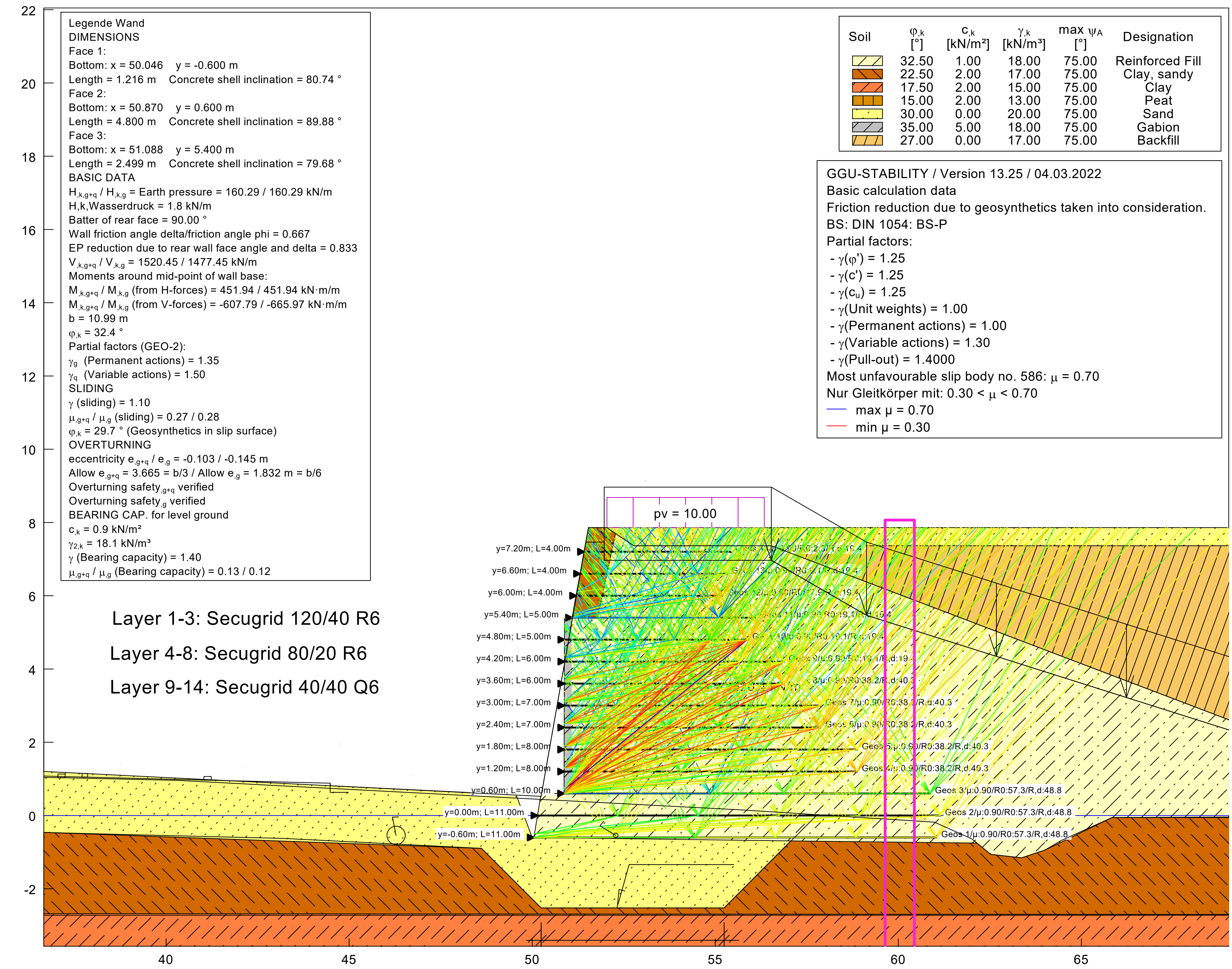
Reviewer:

Rev-No: --

Scale:

Date: 22.03.2022

Enclosure: 2.2



Legende Wand
DIMENSIONS
Face 1:
Bottom: x = 50.046 y = -0.600 m
Length = 1.216 m Concrete shell inclination = 80.74 °
Face 2:
Bottom: x = 50.870 y = 0.600 m
Length = 4.800 m Concrete shell inclination = 89.88 °
Face 3:
Bottom: x = 51.088 y = 5.400 m
Length = 2.499 m Concrete shell inclination = 79.68 °
BASIC DATA
$H_{,k,g+q}$ / $H_{,k,g}$ = Earth pressure = 160.29 / 160.29 kN/m
H,k,Wasserdruck = 1.8 kN/m
Batter of rear face = 90.00 °
Wall friction angle delta/friction angle phi = 0.667
EP reduction due to rear wall face angle and delta = 0.833
$V_{,k,g+q}$ / $V_{,k,g}$ = 1520.45 / 1477.45 kN/m
Moments around mid-point of wall base:
$M_{,k,g+q}$ / $M_{,k,g}$ (from H-forces) = 451.94 / 451.94 kN·m/m
$M_{,k,g+q}$ / $M_{,k,g}$ (from V-forces) = -607.79 / -665.97 kN·m/m
b = 10.99 m
$\varphi_{,k}$ = 32.4 °
Partial factors (GEO-2):
γ_g (Permanent actions) = 1.35
γ_q (Variable actions) = 1.50
SLIDING
γ (sliding) = 1.10
$\mu_{,g+q}$ / $\mu_{,g}$ (sliding) = 0.27 / 0.28
$\varphi_{,k}$ = 29.7 ° (Geosynthetics in slip surface)
OVERTURNING
eccentricity $e_{,g+q}$ / $e_{,g}$ = -0.103 / -0.145 m
Allow $e_{,g+q}$ = 3.665 = b/3 / Allow $e_{,g}$ = 1.832 m = b/6
Overturning safety$_{,g+q}$ verified
Overturning safety$_{,g}$ verified
BEARING CAP. for level ground
$c_{,k}$ = 0.9 kN/m²
$\gamma_{2,k}$ = 18.1 kN/m³
γ (Bearing capacity) = 1.40
$\mu_{,g+q}$ / $\mu_{,g}$ (Bearing capacity) = 0.13 / 0.12

Soil	$\varphi_{,k}$ [°]	$c_{,k}$ [kN/m²]	$\gamma_{,k}$ [kN/m³]	max ψ_A [°]	Designation
	32.50	1.00	18.00	75.00	Reinforced Fill
	22.50	2.00	17.00	75.00	Clay, sandy
	17.50	2.00	15.00	75.00	Clay
	15.00	2.00	13.00	75.00	Peat
	30.00	0.00	20.00	75.00	Sand
	35.00	5.00	18.00	75.00	Gabion
	27.00	0.00	17.00	75.00	Backfill

GGU-STABILITY / Version 13.25 / 04.03.2022
Basic calculation data
Friction reduction due to geosynthetics taken into consideration.
BS: DIN 1054: BS-P
Partial factors:
- $\gamma(\varphi')$ = 1.25
- $\gamma(c')$ = 1.25
- $\gamma(c_u)$ = 1.25
- γ(Unit weights) = 1.00
- γ(Permanent actions) = 1.00
- γ(Variable actions) = 1.30
- γ(Pull-out) = 1.4000
Most unfavourable slip body no. 586: μ = 0.70
Nur Gleitkörper mit: 0.30 < μ < 0.70
max μ = 0.70
min μ = 0.30

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Project: Utrecht Hooggelegen A2 geogrid pile foundation advise

Section/Detail: Grondwal - indenting for WKS - Driving & Drilling

Calculation: Internal & Global Stability - Bishop Slip Circle Analysis - DS-T

Designer:

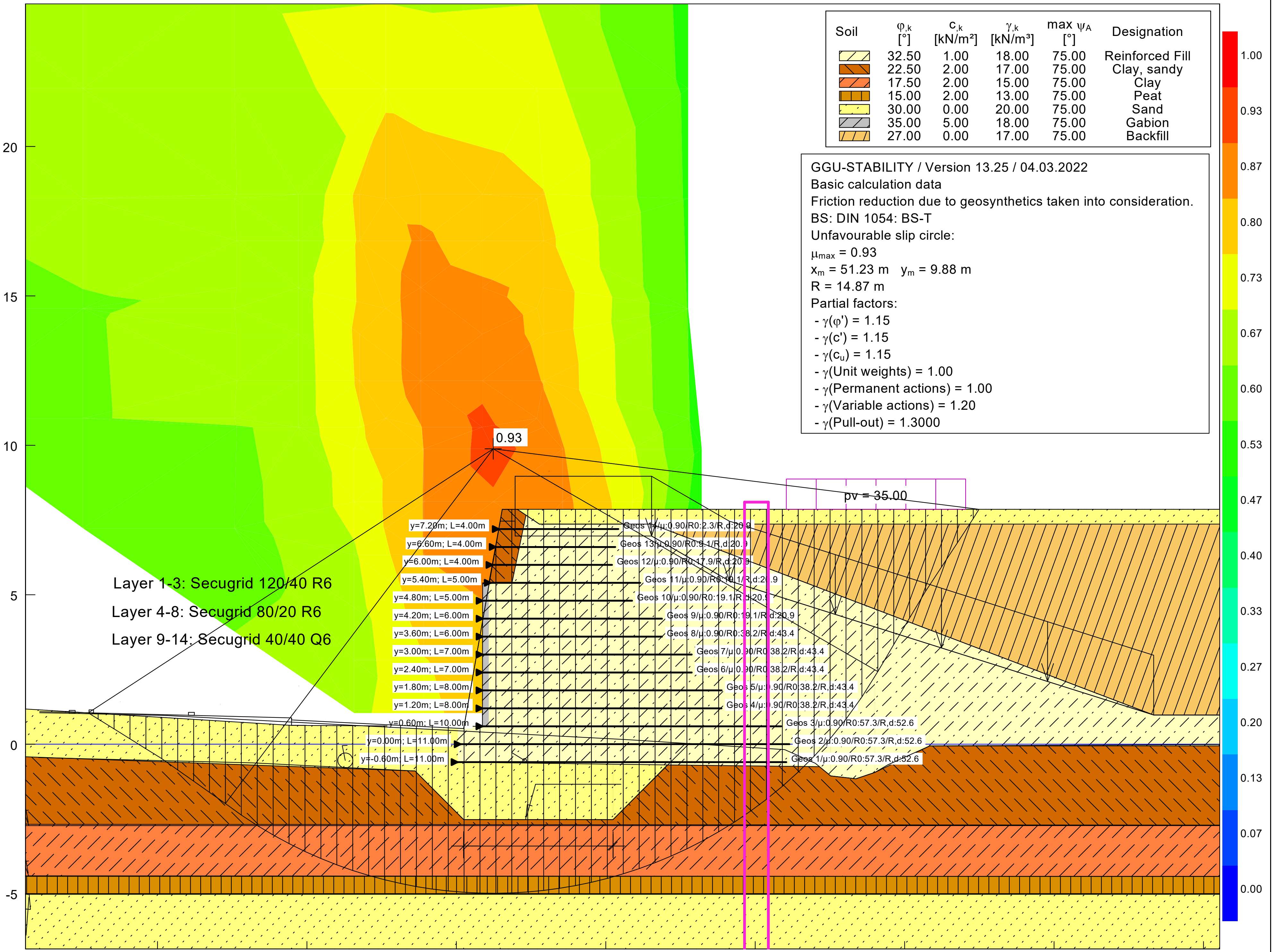
Reviewer:

Rev-No: --

Date: 22.03.2022

Scale:

Enclosure: 2.3



Calculation: Internal & External Stability - Genaral Wedge Analysis - DS-T

Section/Detail: Grondwal - indenting for WKS - Driving & Drilling

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Project: Utrecht Hooggelegen A2 geogrid pile foundation advise

Designer:

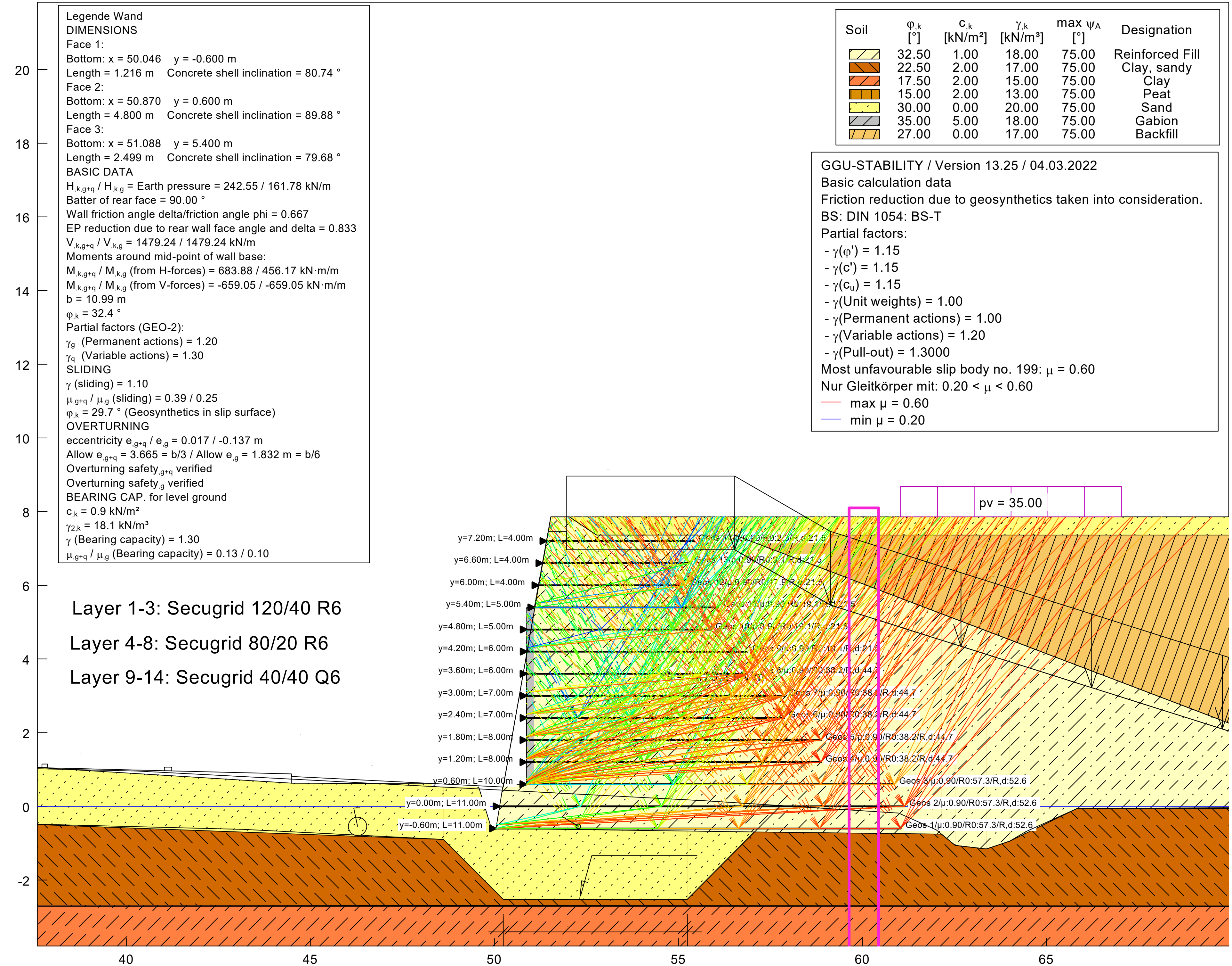
Reviewe

Rev-No: --

Scale:

Date: 22.03.2022

Enclosure: 2.4



Soil	$\varphi_{,k}$ [°]	$c_{,k}$ [kN/m²]	$\gamma_{,k}$ [kN/m³]	max ψ_A [°]	Designation
	32.50	1.00	18.00	75.00	Reinforced Fill
	22.50	2.00	17.00	75.00	Clay, sandy
	17.50	2.00	15.00	75.00	Clay
	15.00	2.00	13.00	75.00	Peat
	30.00	0.00	20.00	75.00	Sand
	35.00	5.00	18.00	75.00	Gabion
	27.00	0.00	17.00	75.00	Backfill

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Calculation: Internal & Global Stability - Bishop Slip Circle Analysis - DS-T

Section/Detail: Grondwal - indenting for WKS - Pull back of the steel tube

Project: Utrecht Hooggelegen A2 geogrid pile foundation advise

Designer:

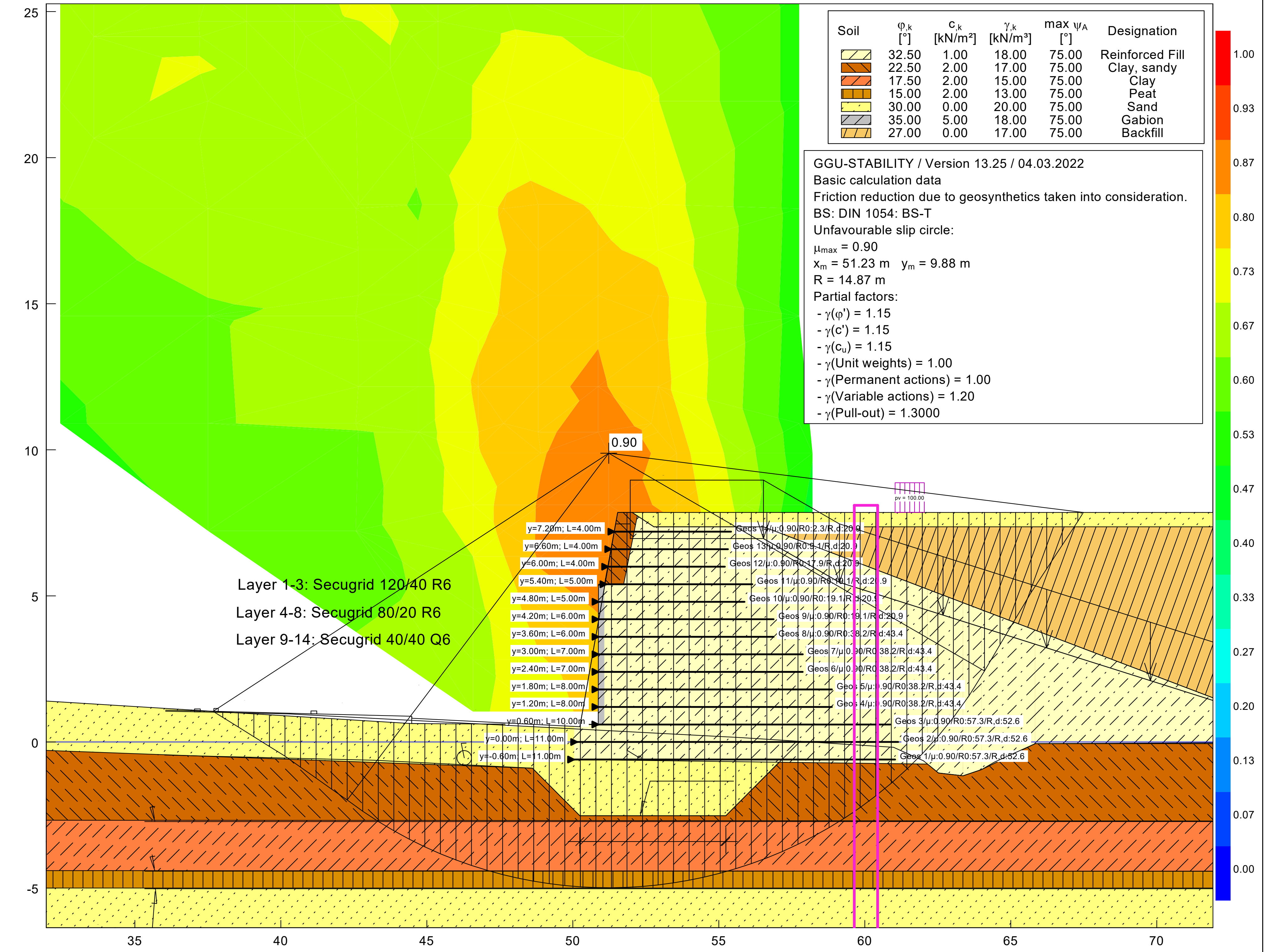
Reviewer:

Rev-No: --

Scale:

Date: 22.03.2022

Enclosure: 2.5



Soil	φ,k [°]	c,k [kN/m²]	γ,k [kN/m³]	max ψA [°]	Designation
	32.50	1.00	18.00	75.00	Reinforced Fill
	22.50	2.00	17.00	75.00	Clay, sandy
	17.50	2.00	15.00	75.00	Clay
	15.00	2.00	13.00	75.00	Peat
	30.00	0.00	20.00	75.00	Sand
	35.00	5.00	18.00	75.00	Gabion
	27.00	0.00	17.00	75.00	Backfill

GGU-STABILITY / Version 13.25 / 04.03.2022
Basic calculation data
Friction reduction due to geosynthetics taken into consideration.
BS: DIN 1054: BS-T
Unfavourable slip circle:
μ_{max} = 0.90
x_m = 51.23 m y_m = 9.88 m
R = 14.87 m
Partial factors:
- γ(φ') = 1.15
- γ(c') = 1.15
- γ(c_u) = 1.15
- γ(Unit weights) = 1.00
- γ(Permanent actions) = 1.00
- γ(Variable actions) = 1.20
- γ(Pull-out) = 1.3000

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Calculation: Internal & External Stability - Genaral Wedge Analysis - DS-T

Section/Detail: Grondwal - indenting for WKS - Pull back of the steel tube

Project: Utrecht Hooggelegen A2 geogrid pile foundation advise

Designer:

Reviewer:

Rev-No: ---

Date: 22.03.2022

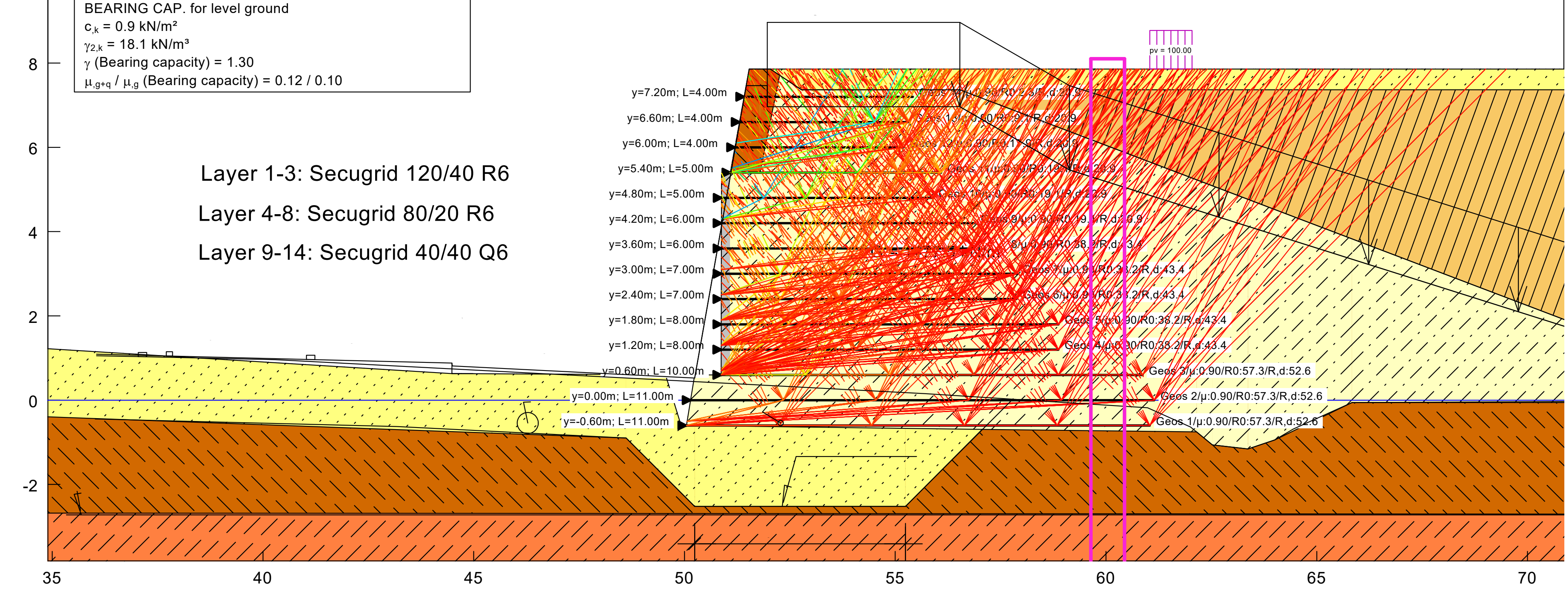
Scale:

Enclosure: 2.6

Legende Wand
DIMENSIONS
Face 1:
Bottom: x = 50.046 y = -0.600 m
Length = 1.216 m Concrete shell inclination = 80.74 °
Face 2:
Bottom: x = 50.870 y = 0.600 m
Length = 4.800 m Concrete shell inclination = 89.88 °
Face 3:
Bottom: x = 51.088 y = 5.400 m
Length = 2.499 m Concrete shell inclination = 79.68 °
BASIC DATA
$H_{,k,g+q}$ / $H_{,k,g}$ = Earth pressure = 211.23 / 160.29 kN/m
H,k,Wasserdruck = 1.8 kN/m
Batter of rear face = 90.00 °
Wall friction angle delta/friction angle phi = 0.667
EP reduction due to rear wall face angle and delta = 0.833
$V_{,k,g+q}$ / $V_{,k,g}$ = 1477.46 / 1477.46 kN/m
Moments around mid-point of wall base:
$M_{,k,g+q}$ / $M_{,k,g}$ (from H-forces) = 595.59 / 451.94 kN·m/m
$M_{,k,g+q}$ / $M_{,k,g}$ (from V-forces) = -665.92 / -665.92 kN·m/m
b = 10.99 m
$\varphi_{,k}$ = 32.4 °
Partial factors (GEO-2):
γ_g (Permanent actions) = 1.20
γ_q (Variable actions) = 1.30
SLIDING
γ (sliding) = 1.10
$\mu_{,g+q}$ / $\mu_{,g}$ (sliding) = 0.34 / 0.25
$\varphi_{,k}$ = 29.7 ° (Geosynthetics in slip surface)
OVERTURNING
eccentricity $e_{,g+q}$ / $e_{,g}$ = -0.048 / -0.145 m
Allow $e_{,g+q}$ = 3.665 = b/3 / Allow $e_{,g}$ = 1.832 m = b/6
Overturning safety$_{,g+q}$ verified
Overturning safety$_{,g}$ verified
BEARING CAP. for level ground
$c_{,k}$ = 0.9 kN/m²
$\gamma_{2,k}$ = 18.1 kN/m³
γ (Bearing capacity) = 1.30
$\mu_{,g+q}$ / $\mu_{,g}$ (Bearing capacity) = 0.12 / 0.10

Soil	$\varphi_{,k}$ [°]	$c_{,k}$ [kN/m²]	$\gamma_{,k}$ [kN/m³]	max ψ_A [°]	Designation
	32.50	1.00	18.00	75.00	Reinforced Fill
	22.50	2.00	17.00	75.00	Clay, sandy
	17.50	2.00	15.00	75.00	Clay
	15.00	2.00	13.00	75.00	Peat
	30.00	0.00	20.00	75.00	Sand
	35.00	5.00	18.00	75.00	Gabion
	27.00	0.00	17.00	75.00	Backfill

GGU-STABILITY / Version 13.25 / 04.03.2022
Basic calculation data
Friction reduction due to geosynthetics taken into consideration.
BS: DIN 1054: BS-T
Partial factors:
- $\gamma(\varphi')$ = 1.15
- $\gamma(c')$ = 1.15
- $\gamma(c_u)$ = 1.15
- γ(Unit weights) = 1.00
- γ(Permanent actions) = 1.00
- γ(Variable actions) = 1.20
- γ(Pull-out) = 1.3000
Most unfavourable slip body no. 200: μ = 0.63
Nur Gleitkörper mit: 0.05 < μ < 0.63
max μ = 0.63
min μ = 0.05



1. Long-Term Design Strength

The Long-term design strength $R_{B,d}$ of a geosynthetic reinforcing element are calculated according to TL Geotex E-StB 05 – Technical conditions of delivery for geosynthetics in earthwork of the road construction (FGSV, 2005) respectively according to EBGEO [4]:

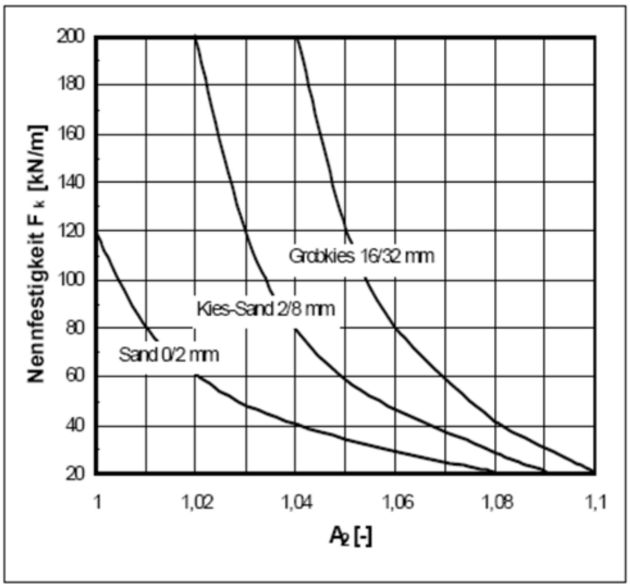
$$\text{Long-term design strength } R_{B,d} = R_{B,k0} / (A_1 \cdot A_2 \cdot A_3 \cdot A_4 \cdot \gamma)$$

$R_{B,k0}$ = Characteristic strength of the geogrid [kN/m]
A_1 = Partial factor for creep
A_2 = Partial factor of installation damage
A_3 = Partial factor for connections
A_4 = Partial factor for environmental effects
γ = Additional factor of safety according DIN 1054:2005-01 / EC 7
γ = 1.4 (DS-P), **γ = 1.3 (DS-T)**, γ = 1.2 (DS-A), γ = 1.0 (DS-E)

Secugrid Q6 and R6							
Raw Material		A_1 1,2 [-]	A_2 1,3 [-]	A_3 1,4 [-]	A_4 1,5,6 [-]		
	identifier		d_{max} < 32 mm		pH 2,0 - pH 4,0	pH 4,1 - pH 8,9	pH 9,0 - pH 12,5
Polyester	6	**1.35**	**1.05-1.09**	**1.0**	1.1	**1.0**	1.18

Tab 1. Recommended reduction factors for Secugrid® - products

1 calculate product-specific considering project boundary conditions
2 according the TL Geotex E-StB 05 from creep rupture test and SIM (linear regression), given for 114 years
3 For soils with d_{max} > 63 mm it is basically recommended to calculate project-specific values; the mentioned values can be used for preliminary designs
4 no connection or overlapping in main stress direction
5 In cases of high concentrations of chemical compounds at the ground the constancy is to be proven in individual cases
6 In case of a period of free weathering > 0,5 years have to be considering another reduction by UV radiation where required



The long-term design live is calculated for a permanent load over a period of 120 years, requiring proper protection of the geogrids against UV-influences.

The Secugrid 120/40 R6 geogrid layers will be locally penetrated by the planned pile foundation. Therefore, a separate reduction factor (A_2') must be taken into account for these locations in order to account for the local weakening.
The reduction factor A_2' results from the ratio of the building width and the total sum of the pile diameters. The reduction factor results from the percentage of the pile diameters to the total width of the considered area.

With:

- Width of considered area = 16.67 m
- Sum of pile diameter = 4.0 m
- Percentage of penetration = 24.6 %
- Partial reduction factor **A_2'** **= 1.25**

The resulting design strength for the chosen products is as follows:

Static Condition (DS-P):

Secugrid® 40/20 R6: **$R_{B,d}$ = 40 / (1.35 · 1.09 · 1.0 · 1.0 · 1.4)= 19.42 kN/m**
Secugrid® 80/20 R6: **$R_{B,d}$ = 80 / (1.35 · 1.05 · 1.0 · 1.0 · 1.4)= 40.31 kN/m**
Secugrid® 120/40 R6: **$R_{B,d}$ = 120 / (1.35 · 1.05 · 1.0 · 1.0 · 1.25 · 1.4)= 48.37 kN/m**

Seismic Condition (DS-T):

Secugrid® 40/20 R6: **$R_{B,d}$ = 40 / (1.35 · 1.09 · 1.0 · 1.0 · 1.0 · 1.3)= 20.91 kN/m**
Secugrid® 80/20 R6: **$R_{B,d}$ = 80 / (1.35 · 1.05 · 1.0 · 1.0 · 1.0 · 1.3)= 43.41 kN/m**
Secugrid® 120/40 R6: **$R_{B,d}$ = 120 / (1.35 · 1.05 · 1.0 · 1.0 · 1.25 · 1.3)= 52.10 kN/m**

The characteristics of the geogrids beside the design strength as stiffness, nod-strength, structure and stress-strain-characteristics are implemented factors for the design of this projects and cannot be changed without permission of the designer.

2. Coefficients of Interaction

The friction coefficient λ was chosen for the case pullout of the reinforcement for Secugrid® geogrids according to table 2 with 0.9. Thus, all useable soils are covered.

Soil Groups GE, GW, GI, SE, SW, SI, GU, GT, SU, ST	λ = 0.9 ... 1.0
Soil Groups GU*, GT*, SU*, ST* and fine-grained soils with d ≤ 0.06 mm > 40 %	λ = 0.8 ... 0.9

Tab. 2 Friction coefficient λ for Secugrid® depending on the soil type; Secugrid®-handbook

Geogrid

Secugrid® R (PES/PET)

Product description:
Laid geogrid made of stretched, monolithic polyester (PET) flat or profile bars with welded junctions used for the reinforcement in many fields of civil engineering including landfill engineering, road construction and hydraulic engineering

Property	Test method*	Unit	40/20 R6	60/20 R6	80/20 R6	120/40 R6	150/40 R6	200/40 R6	400/40 R6
Raw material	-	-	polyester/PET, transparent						
Mass per unit area	EN ISO 9864	g/m²	285	420	380	580	680	810	1.420
Tensile strength, md / cmd**	EN ISO 10319	kN/m	≥ 40 / ≥ 20	≥ 60 / ≥ 20	≥ 80 / ≥ 20	≥ 120 / ≥ 40	≥ 150 / ≥ 40	≥ 200 / ≥ 40	≥ 400 / ≥ 40
Elongation at nominal strength, md / cmd**	EN ISO 10319	%	≤ 7 / ≤ 7						
Tensile strength at 1% elongation, md**	EN ISO 10319	kN/m	8	12	16	24	30	40	80
Tensile strength at 2% elongation, md**	EN ISO 10319	kN/m	14	21	28	42	53	70	140
Tensile strength at 5% elongation, md**	EN ISO 10319	kN/m	28	42	56	84	110	140	280
Aperture size (approx.), md x cmd**	-	mm x mm	73 x 33	73 x 31	73 x 30	71 x 28	71 x 27	71 x 25	70 x 14
UV-resistance (remaining tensile strength)	EN 12224	%	96.3						
Weather resistance	FGSV	class	high						
Production specific elongation	-	%	0						
Roll dimensions, width x length	-	m x m	4.75 x 100						4.75 x 50

*based on, **md = machine direction, cmd = cross machine direction*

The listed technical values are guiding values, achieved in our laboratories and/or independent testing institutes. Our products are subject to changes without prior notice.