Amsterdam A2/A9 project; embankments using reinforced soft soils on very soft sub-soils

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ABSTRACT: The Amsterdam A2/A9 project is the extension, with a new connection, of an intersection between two major highways in the Netherlands. As a part of the project, a landscape art project has been designed. This landscape project is constructed over very soft soils and uses soft soils that came free during the construction of the intersection, thus preventing costly removal and deposition. In the landscape project three major embankments up to 12 m high have been constructed. These mounds of soft soil are intersected by 4 clefts. These clefts, which total 8 steep slopes up to 65 degrees, are designed incorporating a geosynthetic reinforcing system. The special design for the complex reinforcement layout, and global stability analysis using base reinforcement is described. In addition, predictions of deformation and settlements of the soft soil embankment and sub-soil are given and compared with actual deformation on the site. The design and construction of this landscape art project has resulted in valuable experience in using reinforcement both with and on soft soils.

1 INTRODUCTION

As part of the intersection between two main highways in the Netherlands with a new connection to Amsterdam South-East office park, a landscape project has been realised. At various vacant locations in this intersection three embankments have been constructed using soils arising from the project. These three embankments have the shape of, and are named after, an apple, and a banana at the Westside of the A2 highway and a pear at the Eastside of this highway (see fig. 1). The embankments have clefts in them with steep slopes. The apple has two clefts and the others one cleft.

The clefts have steep slopes of 60o on both sides. At the bottom of the clefts there is a 1 m wide footpath. Several methods exist to safeguard the stability of the clefts. Ingenieursbureau Amsterdam has compared these methods and selected the geogrid reinforced soils solution as the most economical. The fill material of reused soft soils is reinforced with layers of geogrids using the wraparound method with the geogrids anchored into the embankment. The advantage of this flexible construction is that settlements in the subsoil and the consolidation of the embankment itself have no negative influence on the construction.

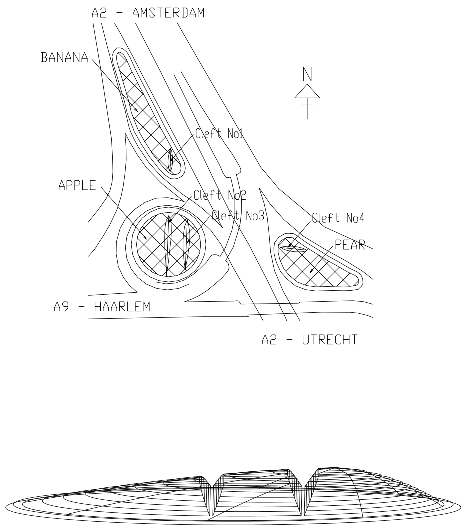


Figure 1: Overview and view mound apple

2 SUB SOIL

The subsoil underneath the three embankments varies considerably in the different layers and properties. The subsoil consists of 4-5 m of peat and clay and the fill material for the embankment comes from the same area and came free during construction of the intersection. The mechanical properties of the various layers in the subsoil are also affected by previous soil improvement techniques.

3 DESIGN

3.1 *General*

The stability analysis was considered in two phases. The first phase was taken to be the time during and shortly after the construction of the embankment, where the water pressures in the subsoil are high and the undrained situation is effective. The second phase was the situation long term after the water pressures had decreased to a normal value.

In the first phase the undrained shear strength (ƒund) was used as the main soil parameter in the $\varphi' = 0$ analysis. In the second phase was the $\varphi' – c'$ analysis used.

All height references are based on the final situation, when all settlements and consolidation has occurred. The height of the apple embankment directly after construction was higher based on the estimated settlements and consolidation (estimated 10%) of the embankment. For the other two embankments, the settlements are expected to be negligible due to earlier soil improvements and the presence of a sand layer.

3.2 *Partial factors*

The partial factors are based on the Dutch guideline CUR162: "Constructing with soils". Three types of partial factors are used:

- Load factors: the estimated loads are multiplied with these factors
- Material factors: particular material properties are divided by these factors
- Method factors: which represent the uncertainties in the design method.

3.2.1 *Load factors*

In the final design a temporary surcharge of 10 kN/m2 was imposed on the embankments in both phases. A load factor of $\gamma f;q = 1,2$ is required for this.

3.2.2 *Material factors*

This type of construction is part of the Dutch standard NEN6740 and is classified as type 1A. The material factors are determined using the CUR162 guideline. The construction is located in safety class I (low to no economical ramifications) for both phases. The following partial factors are required:

			Partial material factor γm
Soil mass	$\gamma m;g$	=	1,0
Tangent of angle of internal friction	$\gamma m;\varphi$	=	1,1
Cohesion	$\gamma m;c2$	=	1,1
undrained shear strength	$\gamma m;f und$	=	1,2

3.2.3 *Method factors*

When using the "Bishop"method a partial method factor and a length factor are required.

3.2.3.1 Method factors

For the method factor is used 1.0 for this method.

3.2.3.2 Length factor

The length factor applies for construction with a significant length which is measured parallel to the construction. In general this length has no effect on the safety of the construction; failure is mostly due to very local effects. However, over a certain length the overall stability can be affected, and in this case the partial length factor has to be applied. The length factor for a 135 m long embankment is $\gamma L = 1.04$, based on de CUR162 guideline.

4 GEOGRID MATERIAL PROPERTIES

The length and types of geogrids for this design are determined using the computer program "Winslope". The stability in 2-dimensions is checked with this program, which also allows geogrids to be entered. The design strength of each type of geogrid is determined by including partial factors for creep, chemical, biological ageing, installation damage etc. This design strength for each type of geogrid is defined as the MILTS (minimum installed long-term strength).

$$\text{MILTS} = \frac{P_c}{f_m * f_e * f_d * f_j} \qquad (1)$$

In which:

- P_c = creep limited strength of geogrid based on certain average soil temperature and design life
- f_m = partial factor for material properties (extrapolations of test results; production tolerances)
- f_e = partial factor for surrounding influences (chemical, biological, UV)
- f_d = partial factor for installation damage
- f_j = partial factor for connections

The Tensar geogrids used in this design have the advantage of a very high degree of orientation of the polymer molecules of the geogrids combined with a unique high quality production process. This results in favourable partial material factors for these types of geogrids. These partial material factors have been certified in certain countries of which the following two are the most relevant:

- the certificate z20.1-102 of the Instute für Bautechnik in Berlin, Germany
- the BBA (British Board of Agrément) certificates 99/R108, 99/109 and 99/R113 for construction both above and below a 70^o slopes.

The effect of the reinforcement is that it introduces tensile forces in the Bishops analysis. The reinforcement is laid in horizontal layers. The "Winslope" program makes a 2-dimensional analysis in which strength per linear meter embankment is determined. The geogrid strength use in the analysis is the lower of either the design strength of that particular geogrid or the pullout strength in the embankment beyond the potential failure surface.
The interaction with the surrounding fill material modifying the cohesion and the shear resistance determines this pullout strength. The forces work on both sides of the geogrid, which giving the following formula for the pullout force P:

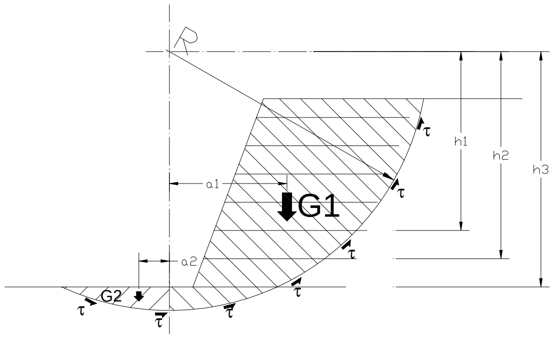
$$P = 2\, l\, \alpha p\, (c' + \sigma v' \tan \varphi') \qquad (2)$$

In which:

- l = length of the geogrid behind potential failure circle
- αp = coefficient of interaction between soil and grid (pull-out factor)
- $\sigma v'$ = vertical soil pressures on the geogrid

A safety factor of 2 has to be applied to this pullout force, resulting in the following formula:

$P = l\ \alpha p\ (c' + \sigma v' \tan \varphi')$ (3)



$$SF = \frac{G2 * a2 + \tau * R + \sum T_i * h_i}{G1 * a1} > \gamma_L = 1{,}04$$

Figure 2: Bishop method with geogrid reinforcement

5 RESULTS OF CALCULATION

The stability calculations for both phases (short-term, undrained and long-term, drained) are done using Bishop's-method. This calculation is based on a 2 dimensional cross-section of the embankment. A SF factor is used for the analysis of the stability, which gives the measure of stability of the construction. This SF factor is the result of the resisting moment divided by the driving moment and should be higher than the length factor of the construction:

$$SF = \frac{M_{resisting,soil} + M_{resisting,geogrids}}{M_{driving,soil}} \geq \text{length factor } \gamma L \quad (4)$$

In all calculations, the undrained situation is critical. This is related to the following points:

- lowest mechanical properties
- highest embankment (before settlement and consolidation)
- slope is steeper (slope reduces with settlement and consolidation)
- distances between geogrids is largest (before consolidation of the embankment fill)

The determination of the type of geogrid and the length of the geogrids are also based on practical aspects of the construction of the embankment. The length is kept equal for each type of geogrid used in order to make it very practical during construction and to avoid mistakes during the construction works.

The following stability factors were achieved for the selected geogrid layout in the undrained situation:

embank-ment	cleft	cross-section A	cross-section B
banana	1	SF(min) = 1,19	SF(min) = 1,07
apple	2	SF(min) = 1,14	SF(min) = 1,13
apple	3	SF(min) = 1,16	SF(min) = 1,20
pear	4	SF(min) = 1,06	SF(min) = 1,31

During the stability analysis it was noted that locations near the sides of the embankment and at the highest point the stability factor was too low. The slope is here nearly 2:3 and in order to achieve sufficient stability it would have been necessary to include many more layers of geogrid.
A different approach was used to solve this problem. A high strength basal reinforcement was selected to ensure the stability of the embankment. This reinforcement was applied directly on top of the sub-soil.

Photo 1: Placing high strength basal reinforcement

6 CONSTRUCTION

The slopes in the clefts are stabilised using horizontal layers of geogrid with a wraparound face detail to stabilize the front of the slopes. This results in an integral system, which is flexible enough to follow the expected settlements and consolidation. This type of construction is normally able to follow differential settlements up to 2.5%.
For the wraparound method the previous layer of geogrid is wrapped around the front and connected to the next layer of geogrid using a bodkin connection.

Figure 3: Connection between two geogrids using a bodkin connection

A geotextile was used on the inside of the wraparound to prevent erosion of the fill material. The wraparound construction requires a light temporary scaffolding in order to be able to compact the fill material.

Photo 2: Scaffolding during construction

Photo 3: Cleft after finishing construction

The distance between the layers of geogrids will be reduced due to the consolidation of the embankment. A maximum expected consolidation of 10% gives a distance of 0.54 m.