# Design of Double Track Railway Bidor-Rawang on AuGeo Piling System according to BS8006 and PLAXIS numerical analysis

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**ABSTRACT**

This paper describes the design of a piled embankment constructed according to the AuGeo-method for sections of the proposed double railway track RAWANG – IPOH. The railway has to be extended to two tracks. Some sections of the second track will be located adjacent to the existing track. At other sections, both tracks of the railway will be relocated to a more suitable area. The subsoil of a number of sections requires soil improvement works to stabilise the embankment and avoid long-term post-settlements.
The basic design was made according to the British Standard BS8006. This standard describes the load systems that have to be included in the calculations. The design was verified using a numerical analysis program called PLAXIS version 7.2. Some remarkable differences were found in the results of the calculations.

## AUGEO PILING SYSTEM

AuGeo piled embankment system consists of lightweight piles with an enlarged pile cap and pile foot. The piles are founded in stable sand or gravel layer. On top of the pile caps, a geogrid mattress is placed to transfer the load of the embankment to the pile caps. In this way, the loads are directly transferred to the hard layers, and the compressible soft layers are not exposed to loads. Settlements are avoided and the construction time is limited to a minimum. The piles are installed with what is known as a drain stitcher. The stitcher is converted to install a plastic casing, instead of vertical drains, being the usual enduse. This casing consists of a corrugated HDPE pipe with an outer diametre of 174 mm and an inner diametre of 150 mm. The casing is resistant to soil pressure to a depth of 12m.
The bottom of the casing is provided with a watertight cap to prevent ingress of groundwater. The working method is as follows:

- A Ø174 mm double wall HDPE tube is cut to the required length
- A polypropylene cap with a 230*230*5 mm steel plate is attached to the tube
- The tube is inserted in a round mandrel 220*10 mm and pushed into the soil
- The mandrel stops at a certain depth, reaching a resistance of 350 kN
- The mandrel is retracted leaving behind the plastic casing in the soil
- The casing is cut-off at the required level
- The pile is provided with a steel reinforcement and filled with concrete

Important factors of this production method are:

- High installation speed, and therefore large production capacity (30 piles/hour)
- Monitoring system on installation force
- No disturbance by vibration or noise
- No handling of heavy prefab piles

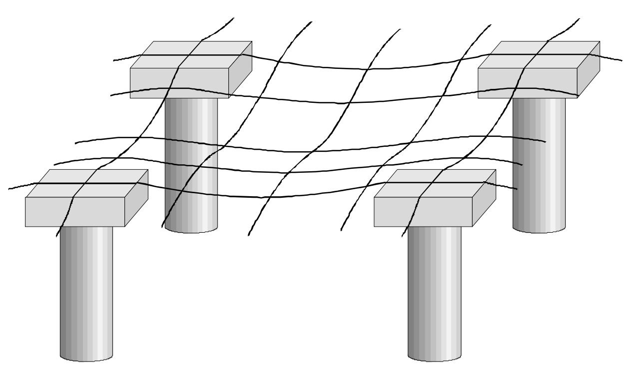

Figure 1. Schematic view of the system

The concrete has to be self-compacting and should have compression strength of 30 N/mm² after 28 days. This results into a bearing capacity of the pile Ø150 mm of 520 kN. The casing does not contribute to the bearing capacity. The reinforcement consists of 6 bars with a diametre of 6 mm, which are positioned by spacers.
During the installation of the casing, the force and depth are constantly measured and stored in

the memory of a data logger. In this way, a detailed image of the subsoil, and thus of the allowable load of the pile point are created.
Figure 2 shows the installation force as a function of the installation depth as it is registered at each pile installation. Tests have shown that theoretical bearing capacity calculations are comparable with the measurements during installation.
The available boreholes are used to determine:

- The expected pile point depth
- Thickness of the foundation layer
- Presence of hard intermediate layers
- Presence of soft layers under the foundation layer

The risk of punching depends on the thickness of the foundation layer. Because of the proportionally small area of the pile point and the relatively low load on every pile, the risk of punching is rather low. At the available boreholes there are, however, neither intermediate layers nor foundation layers with a limited thickness. Settlements in the foundation layers are not expected neither. The piles will be installed with a maximum force of 300 to 350 kN. This means that there will be a safety factor 2 as compared to the allowable bearing capacity. The safety factor on bearing capacity of the pile itself is almost a factor 3.
A resistance of 350 kN will be reached in a soil condition with a SPT value of 12 - 15 blows per 300 mm, or in soils with a CPT value of 6 MPa.

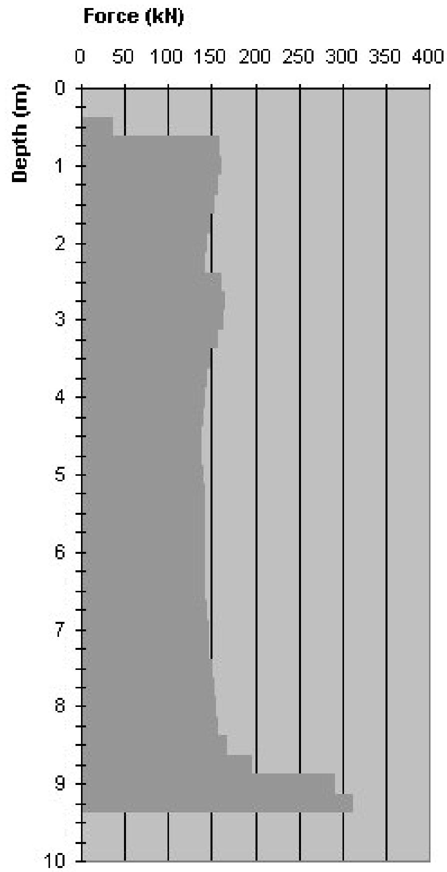

Figure 2. CPT graph from the data logger

## DESIGN CRITERIA FOR THE EMBANKMENT

For new railroads, the design criteria are described as follows:

- Settlement free conventional foundation
- Settlement free compacted fill for embankment construction
- Life time > 100 years
- 2 Tracks for electrified standard railway traffic
- Width top of embankment ± 15 m
- Slopes 1:2
- Train load according to European Standard prEN 1991-2 Eurocode 1. Section 6, Rail traffic actions and other actions specifically for railway bridges and approaches.
  Load Model 71 describes a 4 point load each 250 kN c.t.c. 1.6 m. 0.8 m from each point load a q-load of 80 kN/m per track. Effective width is 3 m on 0.7 m depth (base ballast bed). This equals a maximum traffic load of 52 kN/m². Due to spreading of the load in the embankment, this value will be reduced with 4 kN/m² over the first metre of the embankment height, and furthermore, decreases with 3 kN/m² per 0.5 m increase of the embankment height.
- Load of ballast bed = 15 kN/m² based on a width of 5 m.
- According to the BS8006, the following load factors should be applied:
  Embankment fill: $f_{fs} = 1.3$
  Dead loads: $f_f = 1.2$
  Life loads: $f_q = 1.3$
- Construction time of 10 months
- Rest settlement over 24 months < 25 mm
- Design standard BS8006 for embankment, piles and geogrid
- Pile length 3 – 9 m
- Weight of fill material 18 kN/m³
- Modulus of elasticity of fill material > 18,000 kN/m²
- Max. allowable pile load = 150 kN

## BENDING FORCES IN THE PILES

The British Standard BS8006 does not provide a code for the calculation of bending forces in the piles. Therefore, the magnitude of the horizontal forces on the pile caps is calculated with the numerical analysis and the bending moments are calculated with the program GROND.
Bending forces in the piles can occur due to:

- Horizontal loads in the embankment
- Horizontal movement of soil
- Eccentric load of the pile

In Eurocode 1 the following horizontal active train loads are given:
Acceleration: $Q_{lak}$ = 33 (kN/m) * L (m) < 1000 (kN)
Breaking Load: $Q_{lak}$ = 20 (kN/m) * L (m) < 6000 (kN)
Based on this code and on the assumption of a load spreading over the width of the track (2.6 m), the horizontal load on top of the embankment is 7 to 13 kN/m². If spreading of this load in the embankment is not taken into account, the horizontal load on the subsoil could reach a value of 10 to 15 kN/m². Calculation of the load spreading is complicated. Large scale breaking tests in Germany have shown that at a depth of 1.5 m below the top of the rail no significant horizontal loads can be determined.
The extra tensile force due to the tendency of horizontal 'sliding' of the embankment, as a result of the extra shear stresses in connection with the shoulder of the embankment, is superimposed on the stresses calculated for the axi-symmetric state for the full embankment height and loading.
The horizontal shear stresses from the embankment are transferred to the geogrid, and due to the elongation of the geogrid, part of the horizontal forces will be transferred to the pile caps which in turn will transfer this to the foundation in the soil.
Because of the stress concentration in the soil on the pile cap, the associated frictional force between the pile cap and the geogrid will be very high and, therefore, it is assumed that there will be no sliding over the pile cap. As a result, there will be a direct interaction between the horizontal behaviour of the pile head and the geogrid. This has been modelled with a horizontal spring element (anchor element) that is available in the finite element program PLAXIS. (An EA-value with an anchor length of 1.0 m gives the stiffness of the spring element.) At the pile locations of 0.0 to 9.0 m from the centre-line, a horizontally oriented spring element is introduced into the model.
The plane strain model assumes that the reaction force from the horizontal spring support is evenly distributed in the direction parallel to the track. However, since the supports are not continuous in the direction parallel to the track (the pile caps are spaced at a distance of 1.15 m), there will be some stress concentration in the geogrid in the horizontal direction at the cap locations. However, it is assumed that this effect will not substantially affect the results of the pile cap displacements.

| Pile location w.r.t. centre-line (m) | Spring constant (kN/m) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 50 | | 150 | | 300 | | 600 | |
| | No Load | Train Load | No load | Train Load | No load | Train Load | No load | Train Load |
| | Horizontal pile cap force (kN/m) | | | | | | | |
| 0 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| -1 | 0.05 | 0.44 | 0.07 | 0.40 | 0.10 | 0.22 | 0.14 | 0.11 |
| -2 | 0.14 | 1.18 | 0.23 | 1.67 | 0.32 | 1.72 | 0.42 | 1.66 |
| -3 | 0.27 | 2.10 | 0.52 | 3.56 | 0.73 | 4.55 | 0.95 | 5.59 |
| -4 | 0.37 | 2.87 | 0.71 | 5.20 | 1.01 | 6.80 | 1.34 | 8.37 |
| -5 | 0.40 | 3.34 | 0.76 | 6.05 | 1.08 | 7.55 | 1.47 | 8.98 |
| -6 | 0.41 | 3.54 | 0.77 | 6.21 | 1.10 | 7.51 | 1.51 | 8.82 |
| -7 | 0.41 | 3.55 | 0.76 | 6.17 | 1.09 | 7.41 | 1.48 | 8.57 |
| -8 | 0.40 | 3.54 | 0.74 | 6.11 | 1.04 | 7.27 | 1.40 | 8.28 |
| -9 | 0.38 | 3.50 | 0.68 | 5.94 | 0.92 | 6.91 | 1.19 | 7.60 |

***Table 1 Horizontal pile cap forces***

A cross-section with an embankment height of 2.5 m has been selected for the computations. At this location, about 5 to 6 m of soft silty clay with an SPT-value of 0 to 4 exists directly beneath the surface. For all cross-sections the pile rows are spaced 1.00 m apart, centre-to-centre, perpendicular to the track. A sand blanket with a maximum thickness of 1.0 m will be laid down directly on the soil surface, if the existing surface soil is too soft for the construction loads. At this location, about a metre of sand will be applied to enable installation of the piles and to construct the geogrid-reinforced embankment.

The bending moments in the piles have basically been determined in two steps. Firstly, for the cross-section with a height of 2.5 m, varying values of the spring constant are used in the PLAXIS calculation. The different values of the spring constant will result into a horizontal cap reaction force (and displacement) at each cap location. Secondly, the extremes in the horizontal behaviour of the pile head are assumed to be very stiff, very flexible or soft, and the associated moments are determined using a Winkler model. In the calculations, spring constants of 50, 150, 300, 600 are used to adjust the horizontal resistance of the pile cap to the stiffness of the soil. A value 50 kN/m equals very soft soil.
Program GROND calculates the forces and displacements in a horizontally loaded pile on the basis of a Winkler-type system. In this case it is assumed that the pile is horizontally loaded at the pile head by a horizontal force. The calculations are based on a circular pile with a diametre of 0.15 m and a modulus of elasticity of the pile concrete of 2.85 x $10^7$ kN/$m^2$. In order to determine the range of the bending moments that may be expected in the piles, extreme cases have been considered.
Two extreme cases have been used in the calculations. (see table 2)
A situation is considered in which the modulus of subgrade reaction and associated parametres for the clay layer are expected to relatively low, and in another situation they are expected to be relatively high. In both cases, the layer thicknesses are equal. It is assumed that the pile has penetrated 1 m into the foundation layer. No information is available concerning the phreatic water level, so that again two extremes are assumed.

***Table 2 Values for the extreme cases***

| Layer | Elevation | Arching Factor 1) | Soft Soil | | | | Stiff Soil | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | Unit weight | Angle of internal friction | Cohesion | Mod. of subgrade reaction | Unit weight | Angle of internal friction | Cohesion | Mod. of subgrade reaction |
| | [m] | | [kN/$m^3$] | [deg] | [kN/$m^2$] | [kN/$m^3$] | [kN/$m^3$] | [deg] | [kN/$m^2$] | [kN/$m^3$] |
| Sand | 38.7 – 37.7 | 2.5 | 16 | 30 | 0 | 500 | 18 | 35 | 0 | 15000 |
| clay + silt | 37.7 – 32.7 | 1.5 | 12 | 15 | 2 | 500 | 15 | 22 | 5 | 5000 |
| clayey silt | 32.7 – 31.7 | 2.5 | 19 | 30 | 0 | 10000 | 21 | 35 | 0 | 30000 |

1) ratio of the maximum horizontal soil pressure on pile and the Coulomb horizontal pressure for the plane strain case

With the use of the parametres in table 2, a force-displacement relationship can be generated for both the ‘soft’ and the ‘stiff’ case. The relationships refer to a single pile without tension cracks. The piles are spaced centre-to-centre at a distance of 1.15 m in the track direction. Table 3 shows selected values of the calculated spring constants per metre for the two extreme cases.

***Table 3 Calculated foundation spring constants (horizontal)***

| ‘Soft’ foundation soil | | ‘Stiff’ foundation soil | |
|---|---|---|---|
| Horizontal force on pile head (kN/m) | Foundation spring constant (kN/m per metre) | Horizontal force on pile head (kN/m) | Foundation spring constant (kN/m per metre) |
| 1.74 | 75 | 2.17 | 833 |
| 3.48 | 75 | 4.35 | 758 |
| 5.22 | 75 | 6.52 | 683 |
| 6.96 | 73 | 8.70 | 590 |
| 8.70 | 71 | 10.87 | 529 |

Combining the spring constants determined for the extreme cases with the relationship of the maximum pile cap force versus the value of the spring constant, the results in table 4 are obtained by manual iteration.

***Table 4 Horizontal force on head of pile***

| Foundation | *PLAXIS* calculation | | *GROND* calculation | | | |
|---|---|---|---|---|---|---|
| | Max. horizontal force on pile head | Horizontal spring constant | Horizontal force on pile head | Horizontal spring constant | Horizontal force on pile head 1) | Maximum moment in pile |
| | kN/m | kN/m per m | kN/m | kN/m per m | kN | kNm |
| Soft | 4.4 | 75 | 4.4 | 75 | 5.1 | 4.5 |
| Stiff | 8.9 | 580 | 8.9 | 580 | 10.2 | 5.4 |

1) the piles are spaced 1.15 m apart in the direction of the track

In the case of ‘soft’ foundation soil, the spring constant is more or less constant over the full range of expected forces. In the case of ‘stiff’ foundation soil, the constant varies with head force. Therefore, strictly speaking, the PLAXIS calculation should be performed with a variation in the spring constants, depending on the pile location and horizontal force. However, in most cases, the variation in horizontal cap force in the range of x= – 4 m to –9 m is not large for most cases. Therefore, it is expected that such a refined calculation would lead to little or no change in the final result.

## CALCULATIONS OF THE GEOGRID

The design is executed with an Excel file based on the analytic method according to BS8006, in which above mentioned parametres are used. According to BS8006, the following limit states have to be considered:

- Stability of the embankment fill
- Pile group capacity
- Pile group extent
- Vertical load shedding
- Lateral sliding stability
- Overall stability
- Excessive strain in the geogrids
- Settlement of the foundation
- Bonding lengths of geogrid

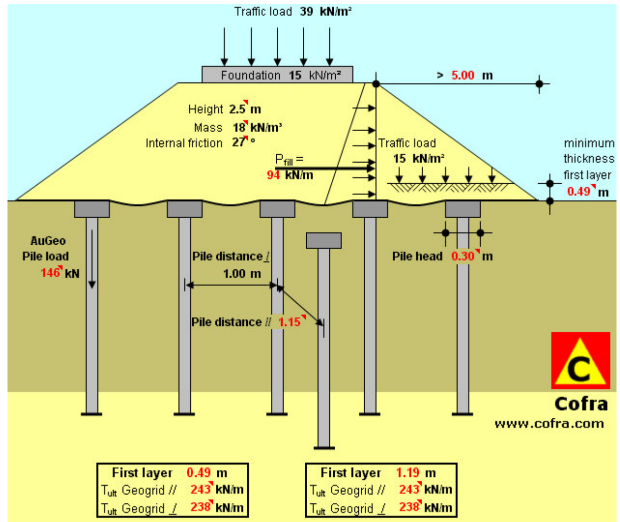


Figure 3. Input sheet BS8006 calculation program

For dimensioning of the pile configuration, the lateral sliding forces, strain in the geogrids, bonding length and pile group extent, an Excel worksheet is developed that automatically calculates all required values instantly.
A major factor in the variation of pile distance and expected loads in the geogrid is the height of the embankment. The height of the embankment is also responsible for the configuration of the cross-section due to the slope of 1:2. The height between the top of the pile cap and the base of the foundation is 2.5 m. In the perpendicular direction, pile distance configuration will be uniform per section to avoid misalignment of pile rows. If misalignment occurs, the forces in the geogrid cannot be transferred to the pile caps, according to the calculated system. This requires a rectangular layout of the pile locations. The geogrid will be assumed to be taking only unidirectional forces. Therefore, a minimum overlap in parallel direction of only 1.3 m is necessary. Overlaps in machine direction are calculated according to par 8.3.3.8. of the BS8006, resulting in overlaps varying from 1.5 to 3 m.
The calculation of the geogrid can be divided into two sections:

- Forces created by transfer of vertical loads onto the pile caps
- Lateral forces in the embankment due to shear in the embankment

If the height of the embankment changes, the pile distance also changes. This means that the piles are not in line at the transition of areas with different pile distances. The load transfer from the geogrid onto the pile cap is not according to the theoretical design. In the design a rectangular pattern is chosen, with a standard pile distance perpendicular to the track of 1 m. The forces in the geogrid are mainly dependent on the arching in the embankment. The higher the embankment, the lower the forces in the geogrid. Various calculation methods have been developed to determine the arching rate. The method used is according to BS8006. However, due to the fact that a rectangular pile system is projected, the calculation method was slightly modified.
Four different load configurations will be calculated:

- Serviceability state during construction
- Ultimate limit state during construction
- Final serviceability state
- Final ultimate limit state

For dimensioning the geogrid, the most unfavourable result is used.

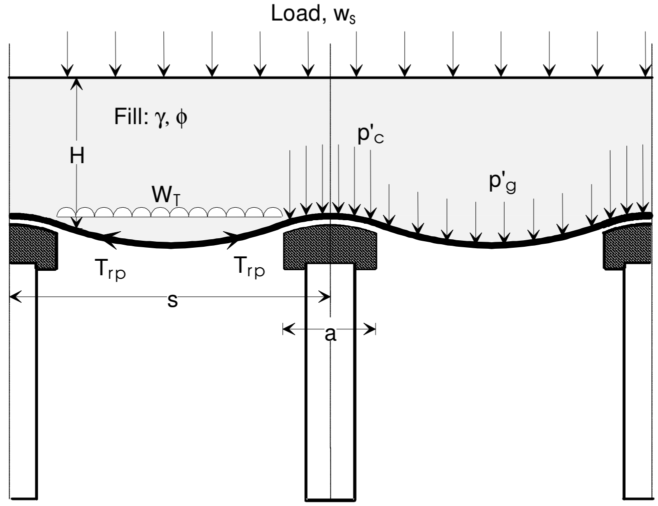


Figure 4. Load scheme

*Table 5 Calculation sheet BS8006 XLS-file*

| | | Ultimate limit state | | Serviceability limit state | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | const. | final | const. | final | | | | |
| **Vertical load shedding** | | | | | | | | | |
| $\sigma'_v = \gamma H f_{fs} + w_s f_q + q_s f_f$ | (Avrg. vertical stress in soil) | 31 | 127 | 24 | 99 | kN/m² | | | |
| $C_c = 1.5 H/a - 0.07$ | (Arching coeff. friction piles) | 2 | 12 | 2 | 12 | | | | |
| $p'_c = (C_c \cdot a/H)^2 \cdot \sigma_v$ | (Vertical stress on pile caps) | 66 | 283 | 51 | 220 | kN/m² | | | |
| **Vertical load between pile caps $\perp$ embankment** | | | | | | Thickness first layer | | | |
| | | | | | | **0.5** | **-1.0** | **m** | |
| $W_{T\perp} = [s_1(f_{fs}\gamma H+f_q w_s)/(s_1.s_2-a^2)]\cdot[s_1.s_2-a^2(p'_c/\sigma_v)]$ | $W_T =$ | 28 | | 22 | | $0.7(s_1\text{-}a)<H<1.4(s_1\text{-}a)$ | | | |
| $W_{T\perp} = [1.4 s_1 f_{fs}\gamma(s_1\text{-}a)/(s_1.s_2-a^2)]\cdot[s1.s2-a^2(p'_c/\sigma_v)]$ | $W_T =$ | 21 | 21 | 16 | 16 | kN/m | $H>1.4(s_1\text{-}a)$ | | |
| | | | | | | | **1.0** | **m** | |
| $W_T = 0$ if $s_1.s_2/a^2 < p'_c/\sigma_v$ | $s_1.s_2 / a^2 =$ | 13 | 13 | 13 | 13 | | | | |
| | $p'_c / \sigma'_v =$ | 2 | 2 | 2 | 2 | | | | |
| **Vertical load between pile caps // embankment** | | | | | | Thickness first layer | | | |
| | | | | | | **0.6** | **-1.2** | **m** | |
| $W_{T//} = [s_2(f_{fs}\gamma H+f_q w_s)/(s_1.s_2-a^2)]\cdot[s_1.s_2-a^2(p'_c/\sigma_v)]$ | $W_T =$ | 32 | | 25 | | $0.7(s_2\text{-}a)<H<1.4(s_2\text{-}a)$ | | | |
| $W_{T//} = [1.4 s_2 f_{fs}\gamma(s_2\text{-}a)/(s_1.s_2-a^2)]\cdot[s_1.s_2-a^2(p'_c/\sigma_v)]$ | $W_T =$ | 29 | 29 | 22 | 22 | kN/m | $H>1.4(s_2\text{-}a)$ | | |
| | | | | | | | **1.2** | **m** | |
| **Average tensile load in geogrid $\perp$ embankment** | | | | | | | | | |
| $T_{rp} = [W_T (s_1\text{-}a) / 2a] \cdot \sqrt{(1+1/6\varepsilon)}$ | $T_{rp} =$ | 68 | 50 | 63 | 46 | kN/m | **68** | **0.5** | **m** |
| | | 50 | | 47 | | | layer 1 | **1.0** | **m** |
| **Required breaking strength geogrid $\perp$ embankment** | | | | | | | | | |
| $T_D = T_{rp} \cdot f_n$ | $T_D =$ | 75 | 55 | 69 | 51 | kN/m | layer 1 | **0.5** | **m** |
| | | 55 | | 51 | | | layer 1 | **1.0** | **m** |
| $f_m = f_{m11} \cdot f_{m12} \cdot f_{m21} \cdot f_{m22}$ | $f_m =$ | 1.43 | 1.43 | 1.43 | 1.43 | | | | |
| $T_{ult} = T_D * f_m * f_{cr} \cdot f_{dyn}$ layer 1 **0.5 m** | $T_{CR} =$ | 171 | 144 | 144 | 122 | Max. | **171** | kN/m | |
| layer 1 **1.0 m** | $T_{CR} =$ | 126 | 144 | 107 | 122 | Max. | **144** | kN/m | |
| **Average tensile load in geogrid // embankment** | | | | | | | | | |
| $T_{rp} = [W_T (s_2\text{-}a) / 2a] \cdot \sqrt{(1+1/6\varepsilon)}$ | $T_{rp} =$ | 95 | 84 | 88 | 78 | kN/m | **95** | **0.6** | **m** |
| | | 85 | | 79 | | | layer 1 | **1.2** | **m** |
| **Required breaking strength geogrid // embankment** | | | | | | | | | |
| $T_D = T_{rp} \cdot f_n$ | $T_D =$ | 104 | 92 | 97 | 86 | kN/m | layer 1 | **0.6** | **m** |
| | | 93 | | 87 | | | layer 1 | **1.2** | **m** |
| $f_m = f_{m11} \cdot f_{m12} \cdot f_{m21} \cdot f_{m22}$ | $f_m =$ | 1.43 | 1.43 | 1.43 | 1.43 | | | | |
| $T_{ult} = T_D * f_m * f_{cr} \cdot f_{dyn}$ layer 1 **0.6 m** | $T_{CR} =$ | 237 | 243 | 200 | 205 | Max. | **243** | kN/m | |
| layer 1 **1.2 m** | $T_{CR} =$ | 213 | 243 | 180 | 205 | Max. | **243** | kN/m | |
| **Resistance against lateral sliding** | | | | | | | | | |
| $K_a = \tan^2 (45° - \phi_{cv}/2)$ | $K_a =$ | 0.4 | 0.4 | 0.4 | 0.4 | | | | |
| $T_{ds} = 0.5 K_a[f_{fs}\gamma H+2(f_q w_s+f_f q_s)]H$ | $P_{ds} =$ | 5 | 94 | 4 | 73 | Max. | **94** | kN/m | |
| **Required breaking strength geogrid $\perp$ embankment** | | | | | | | | | |
| $T_{tot} = T_{ds} + T_{ult}$ layer 1 **0.5 m** | $T_{tot} =$ | 175 | 238 | 148 | 195 | Max. | **238** | kN/m | |
| layer 1 **1.0 m** | $T_{tot} =$ | 131 | 238 | 110 | 195 | Max. | **238** | kN/m | |

From the calculation results, it is concluded that the average tensile force in the geogrid perpendicular to the track is 68 kN/m. Along the track, the average tensile force is 95 kN/m.
According to the PLAXIS calculation, the tensile distributions in the bottom geogrid and the top geotextile are summarised in table 7. The reinforcement consists of three different layers of geogrid. A layer geogrid 250/50 along the track and a second layer geogrid 250/50 perpendicular to the track are placed directly over the pile caps. A layer of 110 kN/m geogrid is placed on top of the 600 mm gravel layer perpendicular to the track

*Table 6 Results PLAXIS calculations perpendicular to embankment*

| Spring constant kN/m | Lower geogrid 250 | | geogrid 110 | Total required force perpendicular to track with train load kN/m | Required force without train load kN/m |
|---|---|---|---|---|---|
| | Lower layer kN/m | Upper Layer kN/m | Max. force kN/m | | |
| 50 | 33.5 | 1.3 | 18.1 | 52.9 | 6.9 |
| 150 | 24.3 | 1.0 | 12.2 | 37.5 | 5.2 |
| 300 | 20.1 | 0.8 | 10.1 | 31.0 | 4.2 |
| 600 | 14.7 | 0.5 | 7.3 | 22.5 | 3.2 |

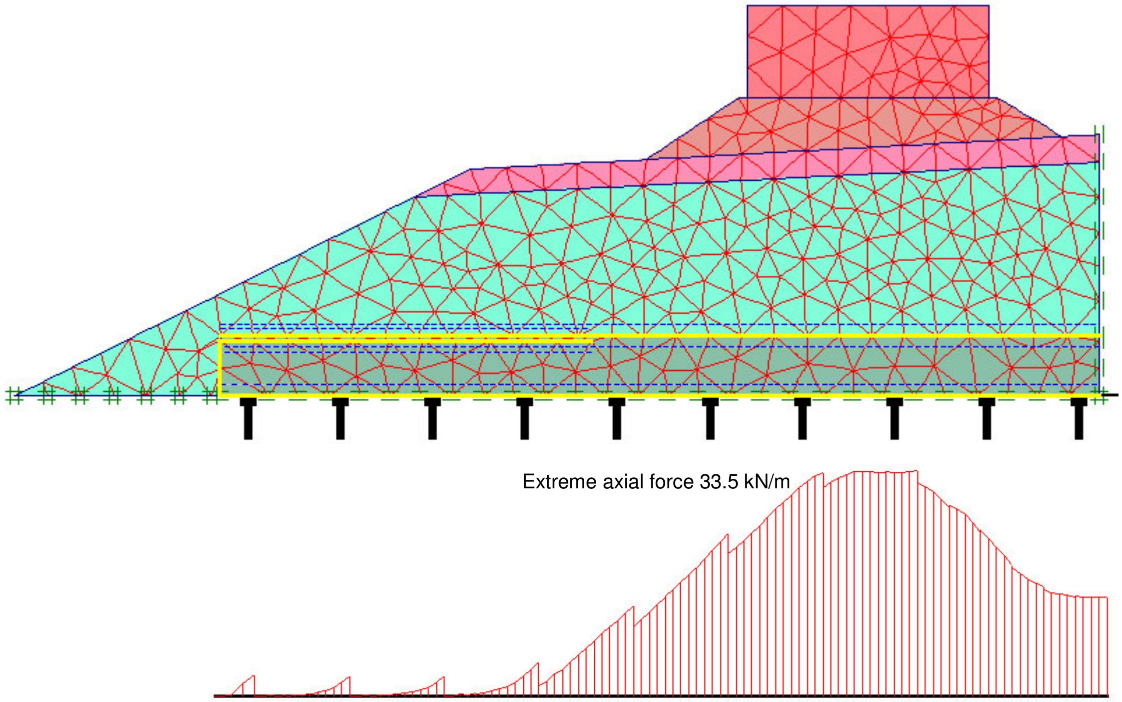


Figure 5. Force in 250 kN/m geogrid over cross-section of a half embankment using a spring constant of 50 kN/m

From the plot in figure 5 of the axial force in the main (bottom) geogrid over the cross-section of the embankment we can see the influence of resistance to sliding. The BS8006 gives a sliding force of 94 kN/m. The main reason for the difference (apart from the use of partial factors in BS8006 formula) lies in the fact that specific influences such as boundary conditions and stress-strain behaviour of the soil and the geogrid are not taken into account. Another remarkable fact was determined during calculation of several embankment heights. In the PLAXIS calculations, the force in the geogrid increases with the height of the embankment, whereas the force in the geogrid in the BS8006 stays constant for a specific pile spacing.

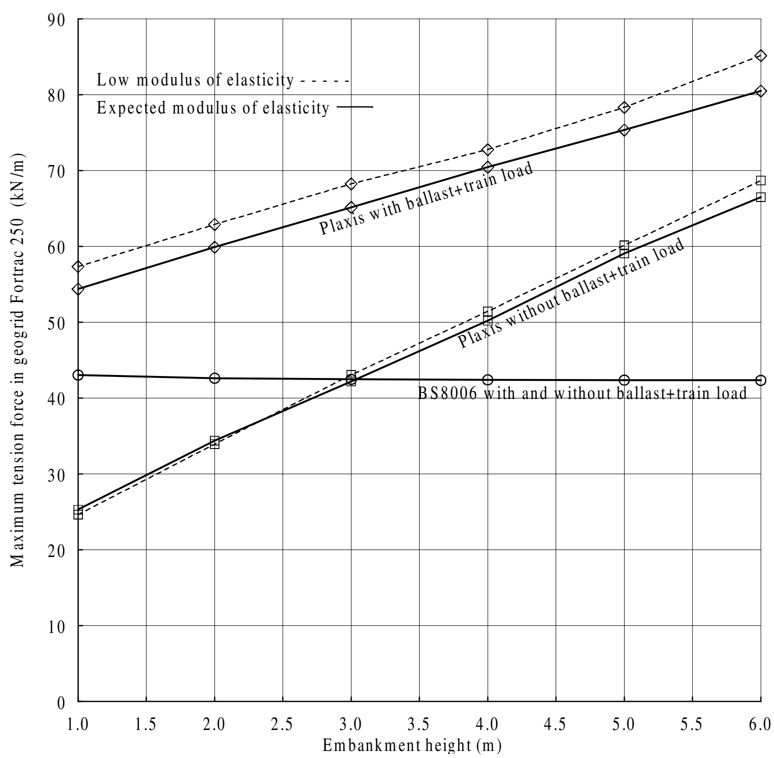


Figure 6. Geogrid tension forces versus embankment height

Figure 6 gives the relation between the tensile force in the geogrid as a function of the height of an infinite fill on a square pile grid of 1m. For both the BS8006 calculation and the PLAXIS calculation no load factors are applied. It is clear that complete arching does not exist if $H \geq 1.4\ (s - a)$ in which H = embankment height, s = pile spacing and a is the pile cap size.

Figure 7 explains why. The BS8006 assumes that if the embankment reaches a certain height, the extra load is transferred directly to the pile caps by arching. However if a stiff membrane is placed over the pile caps the area that carries the arching forces increases and part of the arching load is transferred to the geogrid adjacent to the pile cap.

The PLAXIS analysis also shows that there is a large difference in tensile force over a very short distance adjacent to the pile cap. This is caused by the assumption that the geogrid is fully bonded to the pile cap. Very high tensile forces occur at the edge of the pile caps. In situ measurements have to show if the calculations correspond to the actual forces.

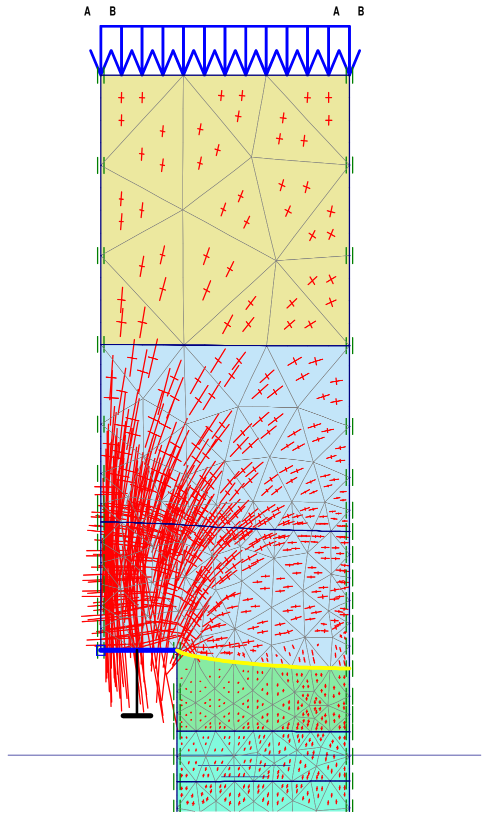

Figure 7 Arching in a numerical analysis

## CONCLUSIONS

The BS8006 has been used to determine the design of a piled embankment with basal reinforcement consisting of a geogrid. A check of the tensile forces in the geogrid using numerical analyses indicate that, depending on the embankment height, the force at the edge of the pile cap may be substantially higher than that which is determined with the use of the BS8006.
The effect of lateral sliding on the tensile forces can be determined more accurately using numerical methods which take into account the proper boundary conditions and the stress-strain behaviour of the embankment material and the geogrid. For example, an extreme load configuration may result in very high horizontal forces on the pile cap and can cause large horizontal displacements of the pile head.
The BS8006 does not directly determine the horizontal forces on the pile caps although they seem to be of much more importance than lateral sliding.

## REFERENCES

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