Design of Double Railway Track on AuGeo Piling System.

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ABSTRACT

This paper describes a preliminary design of the piled embankment according 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 at a more suitable area. The subsoil of a number of sections need soil improvement to stabilise the embankment and avoid long-term post-settlements. Following soil improvement systems are proposed;

1. Removal / Soil replacement
2. Stone Columns
3. Piled embankment with geogrid (pre-cast piles)
4. Piled embankment with geogrid (AuGeo pile system)

The choice of soil improvement method is based on local properties of the subsoil. The AuGeo system is proposed for areas with a limited thickness of the compressible layers and areas where a new alignment is foreseen.

AUGEO PILING SYSTEM

AuGeo piled embankment system exists 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. This way the loads are directly transferred to the hard layers and the compressible soft layers are not loaded. Settlements are avoided and the construction time is limited to a minimum. The piles are installed with a so-called drain stitcher. The stitcher is converted to install a plastic casing instead of vertical drains. This casing consists of a corrugated HDPE pipe with an outer diameter of 174 mm and an inner diameter of 150 mm. The casing is resistant against soil pressure up to a depth of 12m. The casing is at the bottom provided with a watertight cap to prevent the entrance of groundwater. The working method is as follows:

- An Ø174 mm double wall HDPE tube is cut on 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 in the soil
- The mandrel stops at a certain depth reaching a resistance of 350 kN.
- The mandrel is retracted leaving the plastic casing behind 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:

- Very quick installation and therefore large production capacity (30 piles/hour)
- Monitoring system on installation force
- No vibration or noise
- No handling of heavy prefab piles

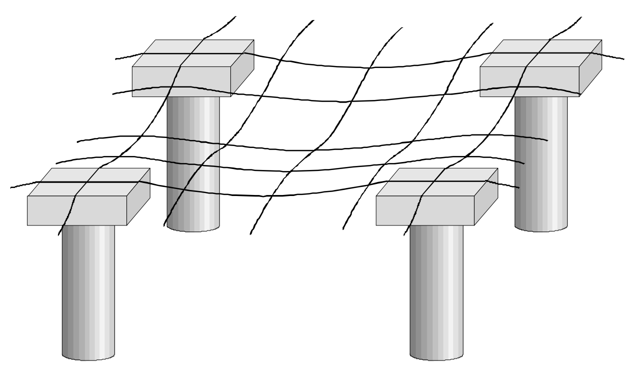

Figure 1. Schematic view of the system

The concrete has to be self-compacting and have compression strength of 25 N/mm² after 28 days. This results in a bearing capacity of the pile Ø150 mm of 440 kN. The casing does not contribute to the bearing capacity. The reinforcement exists of 4 bars with a diameter 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. This way a detailed image

of the subsoil and thus the allowable load of the pile point are created. Figure 2 the installation force is given as function from 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 thickness of the foundation layer determines if punching can occur. Because of the proportionally small area of the pile point and the relative low load on every pile the chance on punching is rather low. At the available boreholes there is however neither layer nor foundation layers intermediate with a limited thickness. Also settlements in the foundation layers are not expected. The piles will be installed with a max force of 300 to 350 kN. This means there will be a safety factor 2 compared with 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 soil with a SPT value of 12 - 15 blows per 300 mm or soils with a CPT value of 6 MPa.
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Bending forces can occur due to:

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

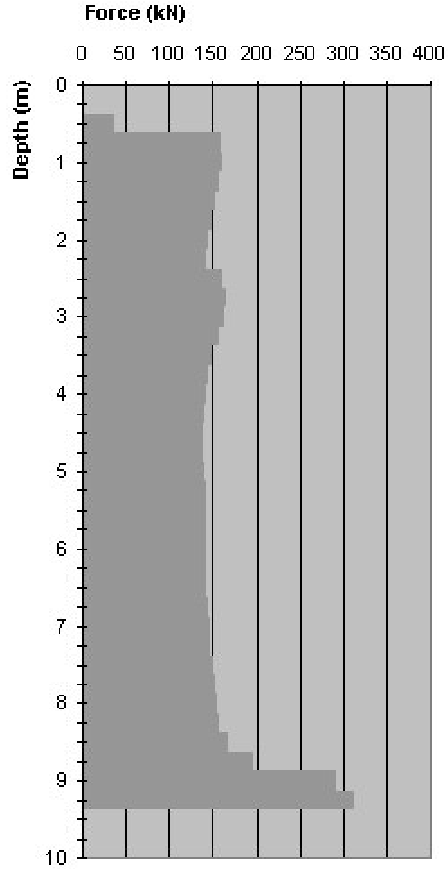


Figure 2. CPT graph from the data logger

In Eurocode 1 following 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. Breaking tests in Germany have shown that on a depth of 1.5 m below the top of the rail no significant horizontal loads can be determined. The horizontal load results in a bending moment in the pile. Maximum bending moment will occur at places here the pile distance is at its maximum.
A horizontal force of 15 kN on a pile with a length of 10 m in weak soil results in a maximum bending force of 1.1 kNm. On 3 m below the pile cap the moment is nil.
If horizontal movements in the soil can occur at certain cross-sections, the pile loads have to be calculated and the reinforcement in the piles has to be adjusted to the required moment. Eccentric loads due to bad construction should be avoided. In Figure 4 the allowable moments in the pile are given based on different sizes of reinforcement.

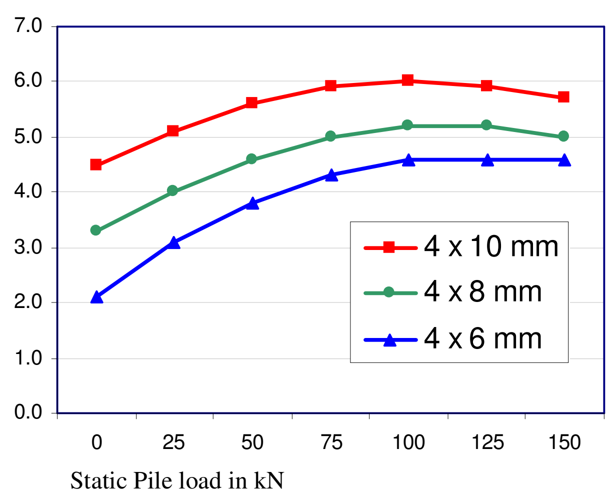


Figure 3. Allowable moment in pile

DESIGN CRITERIA

For new railroads the design criteria are described as follows:

- Settlement free conventional foundation
- Settlement free compacted fill for embankment construction
- Life time > 100 year
- 2 tracks
- Width top of embankment >16,360 mm
- Slopes 1:2
- Trainload 4 point loads 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. This equals a maximum traffic load of 62.5 kN/m² (Effective width track = 2500mm). Due to spreading of the load in the embankment this value at the base will be reduced to 30 kN/m² at 5 m embankments to 48 kN/m² at a 1 m embankment.
- Load of ballast bed = 15 kN/m² based on a C.T.C. width of 5 m.
- Safety against stability during service time > 1.4
- Construction time 10 months
- Rest settlement 24 months < 25 mm
- Design standard BS8006 for embankment, piles and geogrid
- Pile length 3 – 17 meter
- Weight of fill material 18 kN/m³
- Max. Allowable pile load = 150 kN

DESIGN BASICS

The design is executed with an excel file based on the analytic method according the BS8006 whereby above mentioned parameters are used. According the BS8006 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

Stability of the embankment is designed according standard soil mechanical principles. 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.

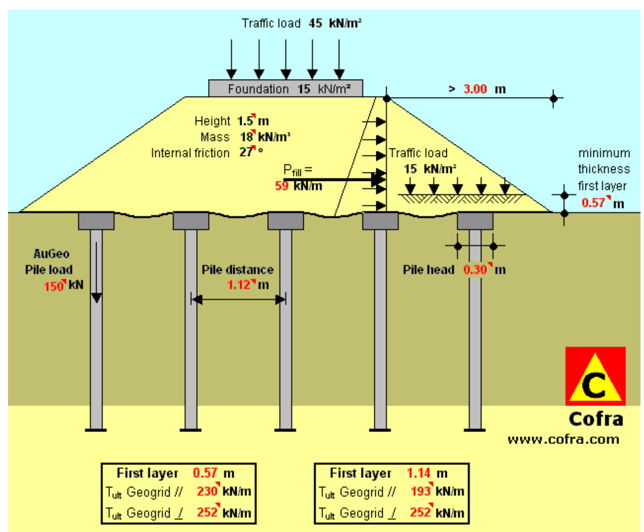


Figure 4. Imput sheet BS8006 calculation program

Mayor 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 height of the track varies from 2.5 to 7.5 m. 0.5 m can be deducted due to the distance between top of the embankment to the top of the rail. Therefore the embankment height varies from 2 to 7 m. For ten different embankment heights, in steps of 0.5 m the loads in the embankment are calculated, resulting in a range of pile distances in the track direction. 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 the calculated system. This results in a rectangled layout of the pile locations. The geogrid will be assumed to be taken only unidirectional forces. Therefore a minimum overlap in parallel direction of only 500 mm is necessary. Overlaps in machine direction are calculated according par 8.3.3.8. of the BS8006 resulting in overlaps varying from 1 to 8 m.

Required breaking load of the Geogrid
According BS 8006

		Fase		
		constr.	final	
Allowable load on piles	F_p =	150	150	kN
Size pile cap (square)	a =	0.30	0.30	m
Pile distance	s =	1.15	1.15	m
Strain angle of friction fill	ϕ_{cv} =	27	27	°
Unit weight fill	γ =	18	18	kN/m³
Height embankment	H =	0.60	1.0	m
Traffic load	w_s =	15	48	kN/m²
Weight foundation	q_s =		15	kN/m²
Reduction factor creep	f_{cr} =	1.45	1.67	

		State	
		Ultim.	Serv.
Load factor embankment fill	f_{fs} =	1.30	1.00
Load factor dead loads	f_f =	1.20	1.00
Load factor live loads	f_q =	1.50	1.00
Strain in Geogrid after 10 hr (%)	ε =	5.00	3.14
Strain in Geogrid after 1M hr (%)	ε =	6.00	3.81
Partial factor manufacturing	f_{m11} =	1.30	1.30
Partial factor extrapolation test data	f_{m12} =	1.00	1.00
Partial factor installation damage	f_{m21} =	1.10	1.10
Partial factor degradation	f_{m22} =	1.00	1.00
Partial factor safety	f_n =	1.10	1.10
Partial factor safety dynamic load	f_{dyn} =	1.10	1.00

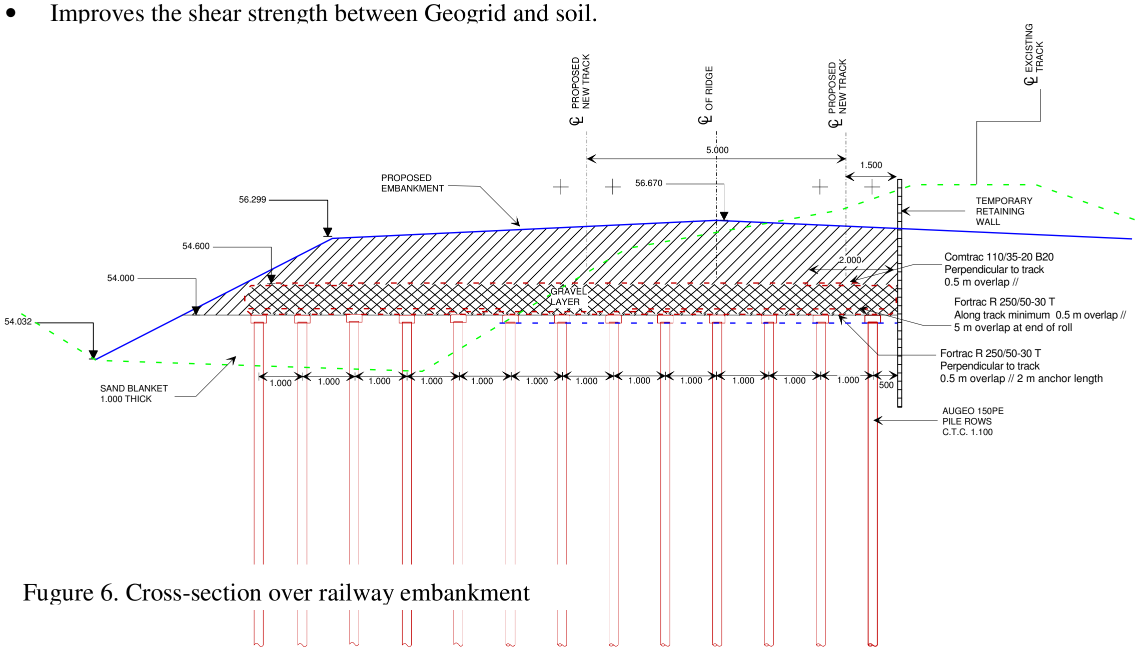
		Ultimate limit state		Serviceability limit st.				
		constr.	final	constr.	final			
Vertical load shedding								
$\sigma'_v = \gamma H f_{fs} + w_s f_q + q_s f_f$	(Average vertical stress in soil)	36.42	113.40	25.71	81.00	kN/m²		
$C_c = 1.95\ H/a - 0.18$	(Arching coeff. concrete piles)	3.69	6.32	3.69	6.32			
$p'_c = (C_c \cdot a/H)^2 \cdot \sigma_v$	(Vertical stress on pile caps)	125.91	407.65	88.88	291.18	kN/m²		
Vertical load between pile caps							Thickness first layer	
							0.6 -1.2 m	
$W_T = [s(f_{fs}\gamma H + f_q w_s)/(s^2-a^2)] \cdot [s^2-a^2(p'_c/\sigma_v)]$	W_T =	34.38		24.27			0.7(s-a)<H<1.4(s-a)	
$W_T = [1.4 s f_{fs}\gamma(s-a)/(s^2-a^2)] \cdot [s^2-a^2(p'_c/\sigma_v)]$	W_T =	26.28	25.96	20.22	19.97	kN/m	H>1.4(s-a)	
							1.2 m	
$W_T = 0$ if $s^2/a^2 < p'_c/\sigma_v$	s^2/a^2 =	14.70	14.70	14.70	14.70			
	p'_c/σ'_v =	3.46	3.59	3.46	3.59			
Average tensile load in geogrid								
$T_{rp} = [W_T\ (s-a)/2a] \cdot \sqrt{(1+1/6\varepsilon)}$	T_{rp} =	101.40	76.57	86.35	71.07	kN/m	layer 1	0.6 m
		77.52		71.95			layer 1	1.2 m
Required breaking strength geogrid // embankment								
$T_D = T_{rp} \cdot f_n$	T_D =	111.54	84.23	94.99	78.17	kN/m	layer 1	0.6 m
		85.28		79.14			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} =	254.40	221.27	196.96	186.68	Max.	254	kN/m
layer 1 1.2 m	T_{CR} =	194.50	221.27	164.11	186.68	Max.	221	kN/m
Resistance against lateral sliding								
$K_a = \tan^2(45° - \phi_{cv}/2)$	K_a =	0.38	0.38	0.38	0.38			
$P_{ds} = 0.5\ K_a[f_{fs}\gamma H + 2(f_q w_s + f_f q_s)]H$	P_{ds} =	6.70	38.84	4.63	27.50	Max.	39	kN/m
Required breaking strength geogrid / embankment								
layer 1 0.6 m	T_{tot} =	261.1	260.1	201.6	214.2	Max.	261	kN/m
layer 1 1.2 m	T_{tot} =	201.2	260.1	168.7	214.2	Max.	260	kN/m

Figure 5. Calculation sheet of Excel BS8006 Workbook.

DESIGN OF EMBANKMENT

The load spreading layer consist of a 600 mm thick gravel layer with a geogrids/geotextile composite on top and a two layer geogrids at the bottom. The function of the mattress is threefold.

- Create a mattress that can transfer loads from the embankment to the piles.
- Take care of the anchor loads at the border of the embankment
- Improves the shear strength between Geogrid and soil.



Fugure 6. Cross-section over railway embankment

The lower geogrid layers run in two directions to provide adequate vertical load shedding to the pile caps. The upper geogrid layer is a composite of a geogrid and a geotextile type Comtrac. The textile prevents migration of fines in the gravel bed. The design of the pile pattern and the Fortrac® geogrids are based on following assumptions.

- The pile caps are in line
- The required force is the Fortrac® geogrids will not fluctuate to much over the width of the embankment

For the centre of the embankment the spreading of the traffic load increases at higher embankments. This increase in load is given in following table:

Beside the traffic load following parameters are taken into account:

- Max. Allowable pile load 150 kN
- Mass of the fill material 18 kN/m³
- Internal friction fill material 27°
- Traffic load construction layer 15 kN/m²
- Weight ballast bed 15 kN/m²
- Pile cap size 300 mm

Embankment height (m)	Traffic load distribution (kN/m²)	Pile distance in square pattern based on load schedule (m)
1.0	48	1.15
1.5	45	1.12
2.0	42	1.08
2.5	39	1.05
3.0	36	1.03
3.5	33	1.00
4.0	30	0.98

Based on these assumptions the tensile force of the geogid was calculated for the centre of embankments with heights varying from 1 to 4 m in steps of 0.5m. Figure 7 gives the results of these calculations.

Since the required pile distance under the slopes is larger than under the centre of the embankment also the forces in the geogrid will be larger. For the centre of the embankment a geogrid with a tensile strength of 240 to 280 kN/m is required in perpendicular direction. If the pile distance is maximised and based on expected loads, a Geogrid® with strengths up to 800 kN would be necessary. To avoid high strength Geogrid® and determine the most economic construction calculations have been made on maximum pile distance based on pile cap size and Geogrid® strength.

Height fill (m)	Pile spacing: Square pattern (m)	Rectangled pattern: along track (m)	Rectangled pattern: perpendicular track centre (m)	Rectangled pattern: perpendicular track slopes (m)
6.0	0.90	0.80	1.00	1.25
5.5	0.92	0.84	1.00	1.25
5.0	0.94	0.88	1.00	1.25
4.5	0.96	0.92	1.00	1.25
4.0	0.98	0.96	1.00	1.25
3.5	1.00	1.00	1.00	1.25
3.0	1.03	1.05	1.00	1.25
2.5	1.05	1.11	1.00	1.25
2.0	1.08	1.17	1.00	1.25
1.5	1.12	1.24	1.00	1.25
1.0	1.15	1.32	1.00	1.25

	Required Geogrid (kN/m) Perpendicular track: Geogrid	Perpendicular track: Comtrac	Along track: Geogrid
6.0	400	110	100
5.5	350	110	100
5.0	300	110	100
4.5	250	110	150
4.0	250	110	150
3.5	250	110	200
3.0	250	110	200
2.5	250	110	250
2.0	250	110	300
1.5	250	110	350
1.0	250	110	400

Based on these results a 250 kN geogrid is chosen combined with 300 mm pile caps under the slopes as well as the centre embankment. To keep the pile caps in line in both directions under embankments with a variable height there is chosen for a 1 m pile distance under the centre of the embankment. The pile rows in the direction of the embankment vary depending on the embankment height. Based on the mentioned assumptions the in following table given pile distances are going to be used.

Based on this pile configuration, the required Geogrid loads can be determined. In following table these values are summarised

Under the gravel two layers of geogrid will be used. The required strength is given in the table above. Perpendicular to the track the required force will increase with embankment height due to higher lateral forces in the embankment. Along the track the required force will decrease with embankment height due to smaller pile distance. On top of the gravel layer Comtrac® 110/35-20 B20 will be used to avoid migration of the fines of the fill material in the gravel layer. In annex 1 the concept drawing of the cross-section over the embankment show the pile spacing and the configuration of the geogrid layers.

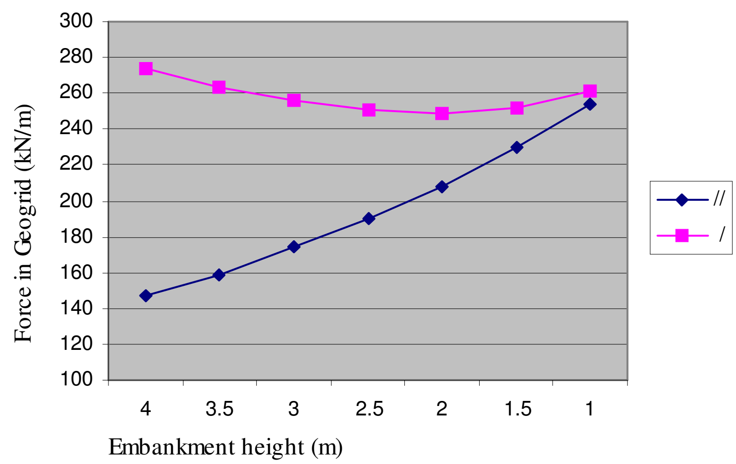


Figure 7. Relation between embankment height and required Geogrid forces

The reinforcement should achieve an adequate bond with the adjacent soil at the extremities of the piled area. This is to ensure that the maximum limit state tensile loads can be generated (across the width and along the length of the embankment) between the outer two rows of piles. The anchor length can be calculated with following equation:

$$L_b = \frac{f_n f_p (T_{rp} + T_{ds})}{(f_{fs}\gamma.h + f_q w_s)\left(\frac{\alpha'_1 \tan \varphi'_{cv1}}{f_{ms}} + \frac{\alpha'_2 \tan \varphi'_{cv2}}{f_{ms}}\right)}$$

Where

T_{rp} = load in geogrid due to vertical load shedding
T_{ds} = load in geogrid due to lateral sliding (only in cross direction)
L_b = bond length beyond outer pile row
f_n = partial load factor economic ramification

f_p = partial load factor pull-out resistance
h = average height of fill over the bond length
γ = unit weight of the embankment
α'_1 = interaction coefficient fill
α'_2 = interaction coefficient gravel
f_{ms} = partial material factor applied to tan φ_{cv}
φ_{cv} = internal friction of fill
w_s = uniformly distributed surcharge
f_q = partial load factor external load
f_{fs} = partial load factor soil

Based on this equation the following bond lengths are calculated:

Load w_s	Pile distance //	Pile distance ⊥	Height h	// T_{rp}	// $\boldsymbol{L_b}$	⊥ $T_{rp}+T_{ds}$	⊥ $\boldsymbol{L_b}$
(kN/m²)	(m)		(m)	(kN)	**(m)**	(kN)	**(m)**
63.0	1.30	1.00	1.00	139	**5.51**	102	**4.05**
61.5	1.25	1.00	1.25	131	**5.04**	110	**4.23**
60.0	1.25	1.00	1.50	131	**4.90**	122	**4.56**
58.5	1.20	1.00	1.75	102	**3.71**	133	**4.83**
57.0	1.15	1.00	2.00	102	**3.61**	142	**5.02**
55.5	1.15	1.00	2.25	102	**3.51**	154	**5.30**
54.0	1.10	1.00	2.50	87	**2.92**	164	**5.50**
52.5	1.10	1.00	2.75	87	**2.84**	175	**5.72**
51.0	1.05	1.00	3.00	76	**2.42**	186	**5.93**
49.5	1.00	1.00	3.25	63	**1.96**	196	**6.10**
48.0	1.00	1.00	3.50	63	**1.91**	208	**6.32**
46.5	1.00	1.00	3.75	63	**1.87**	220	**6.53**

Pile group extent:
The piled area should extent to a distance beyond the edge of the shoulder of the embankment to ensure that differential settlement or instability outside the piled area will not affect the embankment crest. The edge limit of the outer pile cap is given by the following equation:

$L_p = H(n - \tan\theta_p)$ and $\theta_p = 45° - \varphi_{cv}/2$

Based on these equations the minimum distance between the edge of the shoulder and the outer limit of the pile cap is given in following table:

Embankment height at outer edge (m)	Distance L_p (m)
1.50	2.08
1.75	2.43
2.00	2.77
2.25	3.12
2.50	3.47
2.75	3.81
3.00	4.16
3.25	4.51
3.50	4.86
3.75	5.20
4.00	5.55

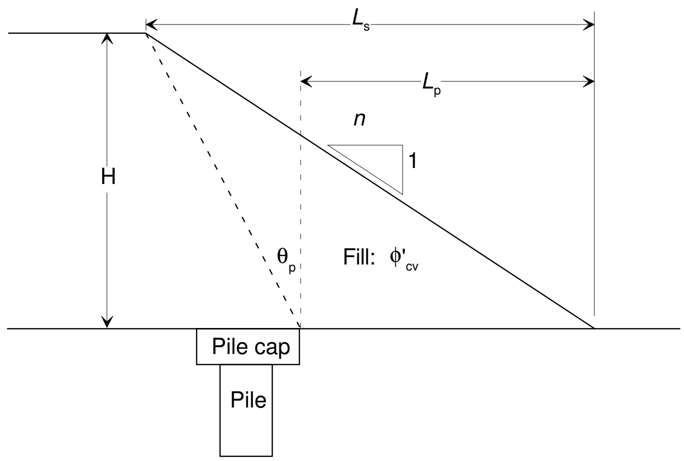


Figure 8. Outer limit of pile caps

CONCLUSIONS

The BS8006 is very suitable tool to determine the forces in geogrids at piled embankments. However to calculate the loads on the pile caps a finite element calculation program has to be used to determine the load spreading in the embankment. Then it will be possible to create variation in the pile distance in the cross direction as well. Now the pile distance and geogrids force are dictated by the maximum allowable pile load.

REFERENCES

British Standard 8006 Section 8. “Design of embankments with reinforced soil foundations on poor ground”