USE OF GEOFABRIC IN BALLINA BYPASS CONSTRUCTION

Thevaragavan Muttuvel, Richard Kelly and Kim Chan
Coffey Geotechnics Pty. Ltd, Lane Cove West, NSW 2066

ABSTRACT

This paper presents the use of geofabric in the construction of a 14 m high embankment on soft ground at Cumbalum Flood Plain Bridge (CFPB) for the Ballina Bypass Alliance (BBA) project. The geofabric was used to improve the embankment stabilisation. In this paper the performance of the geofabric has been assessed using limit equilibrium and finite element methods. Limit equilibrium methods are found to provide similar factors of safety to finite element methods at the ultimate stability state. Finite element methods show that large embankment deformations are required to mobilise the ultimate capacity of the geofabric.

1 INTRODUCTION

Geofabric has been widely used for several civil engineering applications including construction of embankment and construction of temporary working platform on weak soils. The selection of the appropriate geofabric for an application depends on several factors such as strength and stiffness, soil-geofabric interaction and creep characteristics. During the design phase, all potential failure mechanisms such as deep seated failure surface and lateral sliding on top of geofabric need to be considered to assess the geofabric requirements.

This paper presents a case study for the use of geofabric to improve the stability of a 14 m high embankment constructed on soft soils improved by dynamic replacement ground treatment. The embankment forms the northern abutment of the Cumbalum Flood Plain Bridge, currently being constructed for the Ballina Bypass Alliance project. Despite the presence of the dynamic replacement ground treatment, structural geofabric is required to achieve the design factors of safety for embankment stability. Failure modes of the embankment at the ultimate state have been assessed using SLOPE/W (Version 7.10) and BS8006 (1995) limit equilibrium methods as well as the PLAXIS 2D (Version 8.6) finite element package. The performance of the structural geofabric during serviceability is assessed using PLAXIS 2D.

2 PROJECT DESCRIPTION

As a part of the Roads and Traffic Authority's (RTA) ongoing program of works to upgrade the Pacific Highway, the Ballina Bypass Alliance (BBA) was established to undertake the upgrading works for the Ballina Bypass. Ballina town is situated 740 km north of Sydney. The southern end of the Ballina Bypass section of the Pacific Highway (the Upgrade) joins the existing Pacific Highway about 6 km west of Ballina, just east of the junction of the existing Pacific Highway with the Bruxner Highway. The northern end of the Bypass joins the existing Pacific Highway at Ross Lane, approximately 9.4 km north of Ballina. The bypass involves the construction of a four lane dual carriageway from Bruxner Highway to Ross Lane.

A 14 m high embankment needs to be constructed on soft ground at the northern approach of Cumbalum Floodplain Bridge (CFPB) along northbound and southbound carriageways. Dynamic Replacement (DR) with Surcharge and Wick Drains (SWD) has been adopted as ground treatment for the northern approach to achieve the proposed settlement criteria. 2.5 m diameter DR columns have been installed at 5 m equi-triangular spacing and wick drains have been installed at 1.25 m equi-triangular spacing up to 1 m below the bottom of soft clay.

3 FAILURE MODES CONSIDERED

An embankment 14 m high is to be built on top of an approximately 4.5 m thick soft clay treated with Dynamic Replacement (DR) and Surcharge with Wick Drains (SWD) at northern abutment of Cumbalum Flood Plain Bridge (CFPB) as shown in Figure 1.

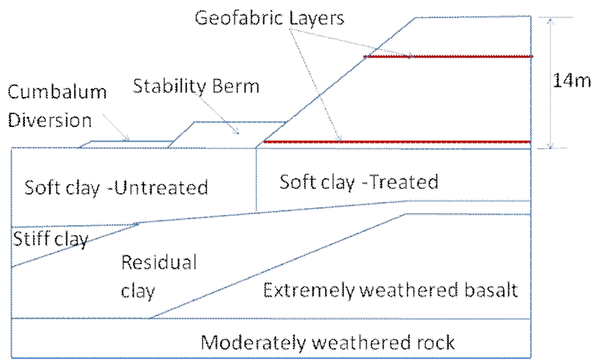

Figure 1: Schematic long section along MC10 main alignment at CFPB northern abutment.

Thorough analysis using SLOPE/W has been carried out to assess the geofabric requirements considering non-circular failure mode of the embankment. In addition, the use of geofabric can lead to lateral sliding failure of the embankment on top of the geofabric. BS8006 approach, SLOPE/W and PLAXIS 2D analyses have been carried out to examine the potential lateral sliding of the embankment.

4 DESIGN PARAMETERS FOR GEOFABRIC

Geofabric with a short term ultimate tensile strength of 600 kN/m at 10% strain has been adopted to provide additional resistance against embankment instability. The short term ultimate tensile capacity of the geofabric has been assessed based on the division of this short term strength by two partial factors including (i) construction damage factor and (ii) an environmental factor. The resulting short term capacity has been conservatively assessed (using a material strength factor of 1.7) to be about 350 kN/m, which corresponds to about 6% strain for polyester type geofabrics. The geofabric properties as listed below have been used to calculate the bond resistance:

- Interface factor (f_i) = 2
- Bond safety factor (f_b) = 2 and
- Fabric safety factor (f_f) = 1.7.

Bond resistance per meter, F_b (kN/m/m) can be calculated using Equation 1.

$$F_b = \left(f_i \sigma'_v \tan \phi'_b \right) \big/ f_b f_f \quad \textbf{(1)}$$

where, σ'_v (kPa) is the overburden pressure from soil above the geofabric; ϕ'_b (deg) is the friction angle of the material in contact with geofabric.

Once the required fabric force is estimated to achieve a target Factor of Safety (FoS) using SLOPE/W software, the required length of fabric has been assessed using the bond resistance according to Equation 1.

5 LIMIT EQUILIBRIUM METHODS

5.1 SLOPE STABILITY ASSESSMENT

Stability assessment has been carried out using the computer program SLOPE/W for the embankment geometry shown in Figure 1 to achieve required FoS in the short term. The DR treated soils are considered in the stability assessment as a block of equivalent strength material calculated based on an approach introduced by Priebe (1995). Taking into account stress concentration on stone columns, Priebe (1995) expressed the short term equivalent strength of the treated ground (without yielding of DR columns) as provided in Equations 2 and 3.

$$\phi_{eq} = \tan^{-1}[(1-\mu_c)\tan\phi_{dr}] \quad \textbf{(2)}$$

$$c_{eq} = \mu_c s_u \quad \textbf{(3)}$$

where, ϕ_{eq} (deg) and c_{eq} (kPa) are equivalent friction angle and equivalent cohesion of the DR treated ground, respectively; ϕ_{dr} (deg) is the friction angle of DR column; s_u (kPa) is the undrained shear strength of soft clay in between DR columns and μ_c is a reduction factor calculated based on the stress concentration and area replacement ratio of DR treated soil. This reduction factor for DR columns will increase under higher embankment load (due to yielding of DR columns), leading to a reduction in the equivalent friction angle and an increase in the equivalent cohesion. The boundary for possible yielding under embankment load has been marked in Figure 2. A summary of geotechnical model parameters adopted for the assessment are presented in Table 1.

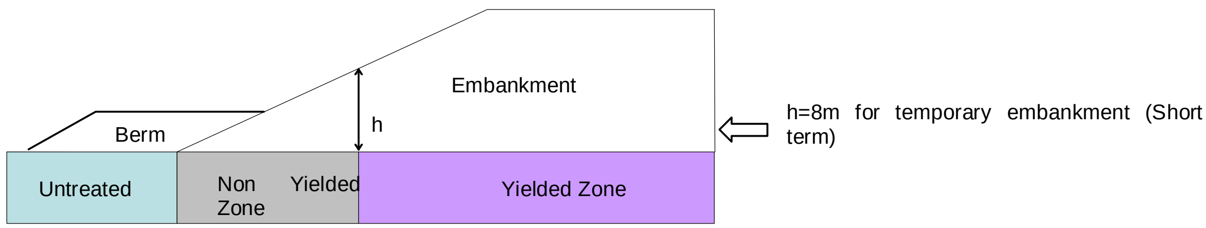

Figure 2: Yielded and Non Yielded zones considered for SLOPE/W.

Table 1: Geotechnical Model Parameters adopted for SLOPE/W assessment.

Soil Type/ Unit	Sub-layer Thickness (m)	Untreated ground		DR treated ground			
				Non Yielded		Yielded	
		C (kPa)	ϕ (deg.)	C (kPa)	ϕ (deg.)	C (kPa)	ϕ (deg.)
Upper crust – Unit 2b	1.0	30	0	16.2	17.8	16.2	17.8
	0.5	20	0	10.8	17.8	10.8	17.8
Soft clay – Unit 2a	1.5	11	0	6.0	17.8	8.5	9.0
	1.5	12	0	6.5	17.8	8.5	9.0
Residual Soil - Unit 6	Varies from 1.5	100	0	-	-	-	-

The critical non-circular failure surface through the boundary of residual clay and soft clay at the full embankment height shown in Figure 3a has been identified and additional stability requirements including stability berm and geofabric to achieve the target FoS have been assessed. The assessment showed that two layers of geofabric with short term ultimate strength of 600 kN/m and a stability berm of 3 m height with 10 m width as shown in Figure 3a are required. One of the layers of structural fabric provides stability during the initial stages of construction and the second layer provides stability for the full embankment height. In addition, potential failure surface on top of geofabric (Figure 3b) has been explored to compare the FoS with that observed from non-circular failure of embankment on top of residual clay. A thin soil layer with friction angle of 20 degrees has been introduced to simulate the interface between the embankment fill and the geofabric, and to assess the FoS of lateral sliding on top of geofabric. The FoS obtained from lateral sliding assessment is 1.9, while the FoS obtained from the deep seated failure of embankment on top of residual clay is 1.5.

Based on these analyses, it can be concluded that the conventional slope stability analysis does not indicate the lateral sliding as the critical failure mode. However, considering the 14 m high embankment, an additional assessment using BS8006 (Code of Practice for Strengthened/Reinforced Soils and Other Fills) has been carried out to assess the possibility of lateral sliding.

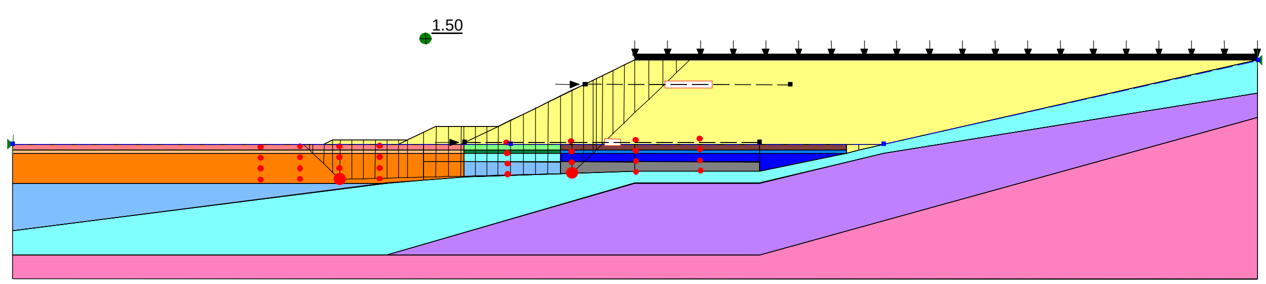

Figure 3a: Failure surface observed from SLOPE/W - Sliding on top of residual clay.

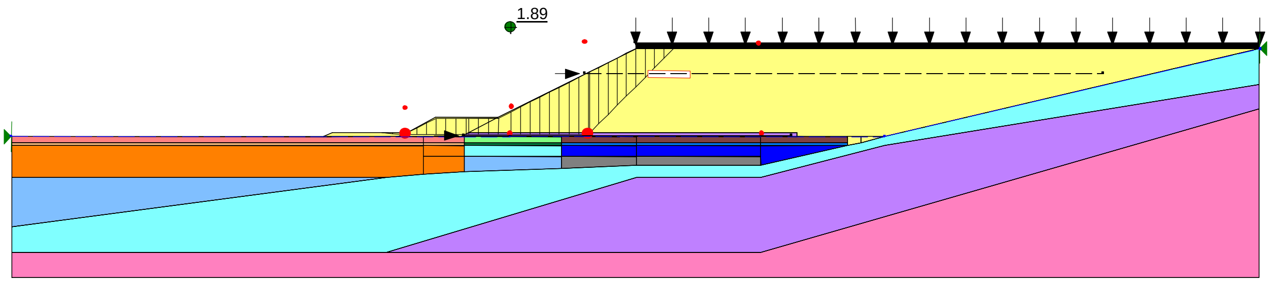

Figure 3b: Failure surface observed from SLOPE/W - Sliding on top of lower geofabric.

5.2 LATERAL SLIDING REQUIREMENTS (BS8006)

BS8006 recommends checking possible lateral sliding over structural geofabric as shown in Figure 4 and examining the minimum reinforcement length and reinforcement load.

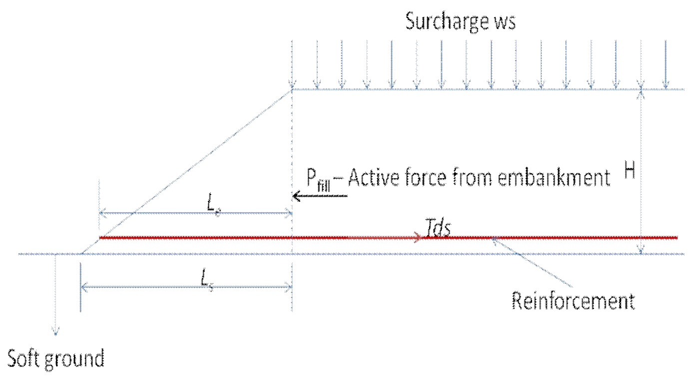

Figure 4: Forces considered for lateral sliding assessment.

The reinforcement load (T_{ds}) acting on the structural geofabric under lateral sliding failure is given by Equation 4.

$$T_{ds} = 0.5 K_a H (f_{fs} \gamma H + 2 f_q w_s) \tag{4}$$

Where K_a is the active earth pressure coefficient; H (m) is the embankment height; f_{fs} is the partial factor for soil unit weight; γ (kN/m^3) is the unit weight of embankment fill; f_q is the partial load factor for external applied loads and w_s (kPa) is the surcharge intensity on top of the embankment.

The minimum reinforcement bond length (L_e) required to resist T_{ds} is given by Equation 5.

$$L_e = \frac{0.5 K_a H (f_{fs} \gamma H + 2 f_q w_s) f_s f_n}{\gamma h \left[\alpha' \tan\phi'_{cv} / f_{ms} \right]} \tag{5}$$

Where f_s is the partial factor for reinforcement sliding resistance; f_n is the partial factor governing the economic ramifications of failure; h (m) is the average height of embankment fill above the reinforcement length L_e; α' is the interaction coefficient relating the soil/reinforcement bond angle to $\tan\phi'_{cv}$; ϕ'_{cv} is the large strain angle of friction of the embankment fill under effective stress conditions and f_{ms} is the partial material factor applied to $\tan\phi'_{cv}$. The values of the lateral sliding parameters adopted in this analysis are presented in Table 2.

Table 2: Parameters adopted for lateral sliding assessment (BS8006).

Parameters	f_s	f_n	α'	f_{ms}	f_{fs}	f_q	w_s **(kPa)**
Value	1.3	1.0	0.67	1.0	1.3	1.3	10

The required tensile load to avoid lateral sliding as per BS8006 is approximately 950 kN/m and it is significantly higher than that calculated by SLOPE/W and used in the design (design strength is 350 kN/m). As shown in Figures 3a and 3b, SLOPE/W results demonstrate that rotational instability leading to non-circular type slip failure instead of lateral sliding failure on geofabric is the critical mode. Since the required reinforcement load predicted from SLOPE/W and BS8006 differ significantly, rigorous finite element analyses using PLAXIS 2D have been carried out to assess the potential failure mechanism and to confirm the design.

6 FINITE ELEMENT METHODS

6.1 PLAXIS MODELLING AT ULTIMATE STATE

6.1.1 Sliding of embankment fill over lower geofabric layer

The main objectives of the PLAXIS 2D modelling are:

- to assess the likely modes of failure using Phi/c reduction approach and compare them with results of SLOPE/W and BS8006. As part of the assessment, analyses have been carried out to assess any possible sliding of fill at the interface of geofabric layer. Lateral sliding is only possible at the interface of embankment fill and geofabric as a result of reduced friction between fill and the geofabric. Therefore, this interface has been modelled in PLAXIS 2D as a thin embankment fill layer with varying friction angles ranging from 30° to 7.5°. Geofabric is placed in the middle of this layer; and
- to predict the mobilised tensile load in the geofabric at ultimate state and compare with values determined from BS8006 and SLOPE/W.

6.1.2 Numerical Modelling and Model Parameters

A two-dimensional plane-strain model was adopted incorporating 15 nodes triangular elements. The geotechnical units were modelled as linearly elastic-perfectly plastic materials with a Mohr-Coulomb (M-C) failure criterion and plastic analysis has been carried out. In the M-C model, the modulus (E_u) of untreated soil has been calculated using Equation 6:

$$E_u = 150 s_u \tag{6}$$

where s_u is the undrained shear strength of untreated soil.
Equivalent modulus of DR treated ground has been calculated using Equation 7.

$$E_{eq} = E_1 a_r^2 + E_2 (1 - a_r^2) \tag{7}$$

where E_1 is the modulus of DR columns; E_2 is the modulus of untreated soil and a_r is the area replacement ratio, which is 0.23 for the adopted configuration. Modulus of DR columns was assumed to be 30 MPa. A summary of geotechnical parameters adopted for the analysis is provided in Table 3. As PLAXIS 2D assessment considers yielding of DR treated ground during the finite element calculation, only non yielded properties of the DR treated ground have been input.

Table 3: Geotechnical parameters adopted for PLAXIS 2D modelling.

Soil Type/ Unit	Sub-layer Thickness (m)	DR treated ground				Untreated ground			
		c (kPa)	ϕ (deg.)	E (MPa)	γ (kN/m³)	c (kPa)	ϕ (deg.)	E (MPa)	γ (kN/m³)
Upper crust – Unit 2b	1.0	16.2	17.8	5.8	16.1	30	0	4.5	15.0
	0.5	10.8	17.8	4.3	15.8	20	0	3.0	14.5
Soft clay – Unit 2a	1.5	6.0	17.8	3.1	15.8	11	0	1.65	14.5
	1.5	6.5	17.8	3.2	15.8	12	0	1.8	14.5
Residual Soil - Unit 6	1.5	-	-	-	-	100	0	15	19

The geofabric was modelled as elasto-plastic material with design strength of 350 kN/m after applying a material strength factor of 1.7 for the short term analysis. Stiffness of the fabric was assumed to be 6000 kN/m.

6.1.3 Comparison of PLAXIS and SLOPE/W results

Results of stability analysis using Phi/c reduction approach and SLOPE/W are summarised in Table 4. Possible failure surfaces for varying interface friction angles from 30 degrees to 7.5 degrees are given in Figures 5a to 5d, which show the observed total displacement after the Phi/c reduction.

Table 4. Analysis Results from PLAXIS 2D and SLOPE/W

Interface Friction Angle (Deg)	Failure Mode from PLAXIS	Factor of Safety		Reinforcement load (kN/m) from		
				PLAXIS		
		PLAXIS	SLOPE/W	Working Load[1]	After Phi/c reduction	SLOPE/W
30	Through the bottom of treated ground (Refer Figure 5a)	1.58	1.50	32	350[2]	350[2]
15	Through the bottom of treated ground (Refer Figure 5b)	1.54	N/A	36	350[2]	N/A
10	Over the fabric interface - Possible sliding (Refer Figure 5c)	1.37	N/A	38	≈350	N/A
7.5	Over the fabric interface - Possible sliding (Refer Figure 5d)	1.26	N/A	39	260	N/A

[1] The working load is at the end of embankment construction with 10 kPa traffic load without Phi/c reduction.

[2] Part of the geofabric reached its design limit of 350 kN/m.

Figures 5a to 5d show the potential modes of embankment instability for varying interface friction angles. According to the results provided in Table 4 and the failure surfaces observed from PLAXIS 2D analyses given in Figures 5a to 5d, sliding failure at the interface of geofabric and embankment fill is only critical when the interface friction angle is less than 15 degrees. However it is considered that the typical interface friction angle is around 2/3 of embankment fill friction angle, i.e. 20 degrees and therefore sliding failure along the embankment fill and geofabric interface is unlikely to happen. The likely critical failure mode is therefore a deep seated rotational failure mode with a non-circular slip surface which is similar to the non-circular SLOPE/W results.

As provided in Table 4, the mobilised working load on reinforcement based on PLAXIS 2D is in the range of 32- 39 kN/m under the load of a 14 m high embankment and 10 kPa traffic load. The mobilised reinforcement load at failure for possible lateral sliding is approximately 350 kN/m. However BS8006 predicts the required tensile capacity of the geofabric as 950 kN/m. Minimum bond length (L_e) required to avoid lateral sliding based on BS8006 is approximately 22 m, which is less than the available reinforcement length of 28 m in batter area.

Based on these analyses and results, it can be concluded that BS8006 is a more conservative analysis than the slope stability approach and the rigorous Finite Element analysis. It overestimates the required reinforcement capacity compared to conventional limit equilibrium analysis (e.g. Slope/W) or the rigorous Finite Element analysis (e.g. PLAXIS 2D). In addition, sliding failure mode is unlikely to be the failure mechanism as described in BS8006, and rotational instability with non-circular failure surface is the critical case according to results of SLOPE/W and PLAXIS 2D analyses.

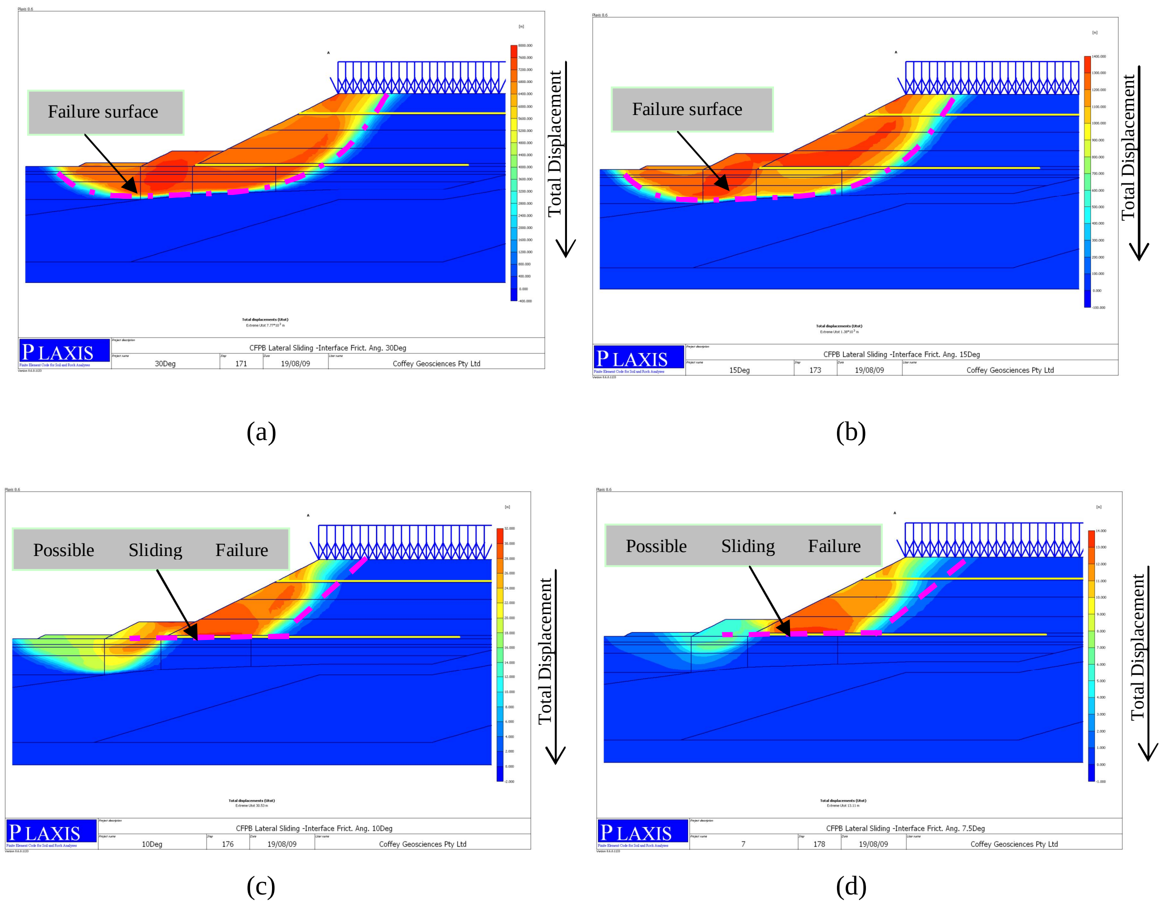

Figure 5: Possible failure surface for interface friction angle of (a) 30 degrees, (b) 15 degrees, (c) 10 degrees and (d) 7.5 degrees

6.2 PLAXIS MODELLING AT SERVICEABILITY STATE

It is clear from the results presented in Table 4 that the geofabric load under a 14 m high embankment and traffic load of 10 kPa (under working load) is less than 40 kN which is induced by approximately 0.7% of strain in the soil. This strain compatibility between soil and geofabric limits the use of ultimate geofabric strength of 350 kN/m which corresponds to 6% of strain in geofabric in design. However, the 350 kN geofabric load, which is induced by approximately 6% of strain in the geofabric, has been utilised at the ultimate stage of failure under the Phi/c reduction approach. This can be achieved under large deformation of the embankment at the ultimate stage of failure.

7 CONCLUSIONS

The following conclusions are drawn from the results of the limit equilibrium and finite element studies:

1. The BS8006 method for assessment of lateral sliding of fill on structural geofabric is conservative.
2. The limit equilibrium and the finite element methods can give similar factors of safety for embankment stability at the ultimate state. The limit equilibrium methods are able to be utilised more rapidly than finite element methods, i.e. saving time and money, and are therefore more suitable for stability design.
3. Large embankment deformations are required to mobilise the ultimate capacity of the structural geofabric prior to the ultimate state. Strain compatibility should be taken into account during design to assess the mobilised geofabric force under working load if any assessment is required to be carried out to match field performance of the embankment during construction.

8 ACKNOWLEDGEMENTS

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9 REFERENCES

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