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Use of High Strength Geogrid Reinforcement for Embankments on Soft Soil in Western Australia

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ABSTRACT

Design and construction of embankments over very soft to soft soil has always been a challenge to geotechnical engineers. This paper describes the use of very high strength geogrid reinforcement for the construction of a perimeter road embankment for a multiuser stockyard facility in Port Hedland, Western Australia. The existing subsurface comprised of a tidal Mangrove mud layer, up to 2m thickness, with a nominal undrained shear strength of about 10kPa or less, overlying calcarenite. The embankment was up to 7m in height and comprised of dredge fill materials. Limit equilibrium analysis was undertaken using SLOPEW computer program and load deformation analysis was undertaken using PLAXIS 2D Finite Element program. Results of the analyses indicated the requirement for multiple layers of high strength geogrid (i.e. Secugrid 120/40 and 400/40) reinforcement in the embankment for slope stability and bearing capacity purposes. PLAXIS was used for assessment of tensile force mobilised in the geogrid reinforcement and calculation of the embankment settlement with time. The method of design and construction, properties of the geo-materials and post construction performance are described in this paper.

Keywords: soft soil, geogrid, reinforcement, limit equilibrium, load deformation

1 INTRODUCTION

Australia is the world's largest iron ore exporter and as a producer, ranks third after China and Brazil (DoIR 2003). The ores from major mines in Western Australia's Hamersley Province of Pilbara region are hauled from working faces to the port sites. Port Hedland is one of the largest ore export facilities in Australia. The Facilities have been under expansion based on the growing global demand of ore materials from Australia. In 2008, Port Hedland Port Authority commenced construction of a multiuser bulk iron, manganese and chromite ore export facility comprising road, stockyard (comprising of thirteen stockpiles), conveyors and a wharf. An aerial photograph of the site and drawing showing the arrangement of the stockyard and the perimeter road are shown in Figure 1. The method of geotechnical analysis and the earthwork construction process are described in the following sections.

(a) (b)

Figure 1 (a) Aerial Photograph of Site (b) Stockpile and Perimeter Road Arrangements

2 GEOLOGICAL AND GEOTECHNICAL INFORMATION

2.1 Subsurface condition

The geotechnical investigation comprised of the excavation of test pits, hand augered boreholes, hand held dynamic cone penetrometer (DCP) tests and shear vane tests. The laboratory investigation comprised of classification tests and consolidation tests on the Mangrove mud.

The generalised subsurface profile for the stockyard and associated facilities site are presented in Table 1.

Table 1: Generalised Subsurface Profile – Stockyard and Associated Facilities Site

Layer/Unit	Typical Depth to Top of Layer (m)	Typical Layer Thickness (m)	Description/Remarks
Mangrove Mud	Surface	<0.5m to greater than 1.8m	Sandy Clay/Clayey Sand, medium plasticity, brown becoming grey, trace fine to medium grained sand content, very soft to soft
Calcarenite	<0.5m to greater than 1.8m	Not penetrated	Calcarenite, well-cemented coarse grained, white.

2.2 Geotechnical Parameters

Geotechnical parameters adopted for the analysis are presented in Tables 2 and 3. The parameters for the mangrove mud layer were derived from the results of field vane shear tests and laboratory consolidation tests. An undrained shear strength (S_u) of 10kPa was adopted for the mangrove mud as indicated in Table 2.

Table 2: Soil Unit Weight, Permeability and Strength

Material	Unit Weight (kN/m^3)		Permeability (m/day)	Strength	
	Dry	Saturated		Cohesion or undrained shear strength (kPa)	Friction Angle (°)
Mangrove Mud	17	17.5	0.000215	10	-
Calcarenite / Red Beds	18	18.5	1	75	30
Dredged Materials	18	18.5	1	-	38

Table 3: Soil Stiffness

Material	Young's Modulus (MPa)	Poisson Ratio	Consolidation		
			Compression Index, C_c	Swelling Index, C_s	Creep Index, C_α
Mangrove Mud	-	-	0.149	0.01	0.004
Calcarenite / Red Beds	100	0.3	-	-	-
Dredged Materials	40	0.3	-	-	-

Note: Initial void ratio (e_0) of the Mangrove mud is 0.8

2.3 Geogrid Reinforcement

The weak subsurface condition and loading from embankment and stockpiles warranted use of high strength geogrid reinforcements for stope stability and bearing capacity purposes. Geogrid reinforcement used for the project and their properties are presented in Table 4.

Table 4: Properties of Geogrid Reinforcements

Description	Use	Tensile Strength in Machine Direction			Raw Material	Weather Resistance
		2% Strain	5% Strain	8% strain		
Secugrid 400/40 R6	Basal Reinforcement	140 kN/m	240 kN/m	400 kN/m	Polyester	High
Secugrid 120/40 R6	Embankment Reinforcement	42 kN/m	72 kN/m	580 kN/m		

2.4 Design Loads Adopted

The following design loads were adopted in the stability and deformation analysis.

- Average vehicle traffic loads on the perimeter embankment road of 20kPa; and
- Iron ore stockpiles up to 16.5m in height with an assumed iron ore stockpile dry unit weight of 22kN/m^3. The stockyard floor will be subject to a stockpile load of up to about 300kPa.

3 ANALYSIS AND RESULTS

3.1 Methods of Analysis

Limit equilibrium method of stability analysis using computer program SLOPEW Version 7.12 and deformation and safety factor analysis using PLAXIS Version 8 finite element technique was undertaken with the following conditions and assumptions:

- Use of soil reinforcement to achieve end of construction and long term global stability;
- Embankment global slope stability factor of safety (FOS) ≥ 1.5 for static loading, FOS ≥ 1.1 for dynamic (seismic) loading and FOS ≥ 1.4 for a flood event.
- The horizontal acceleration coefficient or hazard factor $a_{(z)max}$ of 0.12g (1 in 500 years annual probability of exceedance) and $a_{(z)max}$ of 0.09g (1 in 250 years annual probability of exceedance) were considered for quasi-static earthquake analysis based on AS1170.4-2007.
- The analysis was based on a design life of 20 years.

3.2 Strength of Mangrove Mud Layer

The end of construction stability of the perimeter road embankment and stockyard floor were analysed using SLOPE/W program. The gain in strength of the mangrove mud layer due to consolidation during the construction of the embankment fill was estimated and taken into account in the analysis, as follows:

- The undrained shear strength (S_u) of the mangrove mud layer was assumed to be approximated by the following Equation (Ladd, 1986).

$$\frac{S_u}{\sigma'_v} = 0.2 \times (OCR)^{0.8}$$

  Where: σ'_V = Effective vertical stress and OCR = Overconsolidation Ratio
- An S_u of 10kPa was adopted for the mangrove mud as indicated in Table 2. The stress history of the mud was assessed to vary from an OCR of 3.6 at shallow depth, decreasing to 2.6 at about 1.8m depth.
- With the full embankment height, the resultant OCR of the mangrove mud layer was estimated. Then based on the resultant OCR and using the above equation the undrained shear strength of the mangrove mud layer underneath the embankment was calculated.

3.3 Results of Analysis

The results of global slope stability analysis at the end of construction for the road embankment cross section and stockyard are summarised and presented in Tables 5 and 7. The number, type, length and location of geogrid layers considered in the analysis are summarised in Table 6. Typical SLOPEW output is shown in Figure 2. Tensile forces mobilised in the geogrid reinforcement was verified using PLAXIS analysis.

Table 5: End of Construction Stability (SLOPE/W Analysis Results)

Fill Materials	Slope Analysed	Water Table	Geogrid	Minimum FOS for Global Stability	
				Static	Dynamic (1 in 250yrs/ 1 in 500yrs)
Dredge	Outer	0mCD	Yes	1.58	1.27 / 1.20
	Outer	7.5mCD	Yes	1.58	1.24 / 1.15
	Outer	Flood	Yes	1.50	-

Table 6: Geogrid Reinforcement Requirements

Embankment Material	Geogrid Requirement			Comment
	RL mCD	Minimum Strength at 2% strain (kN)	Length (m)	
Dredged	10.75	42	11m	Securid 120/40 or equivalent
	9.4	42	12m	
	8.1	42	13m	
	6.8	42	14m	
	5.5	42	14m	
	Top of Mud	140	Cover mud area	Securid 400/40 or equivalent

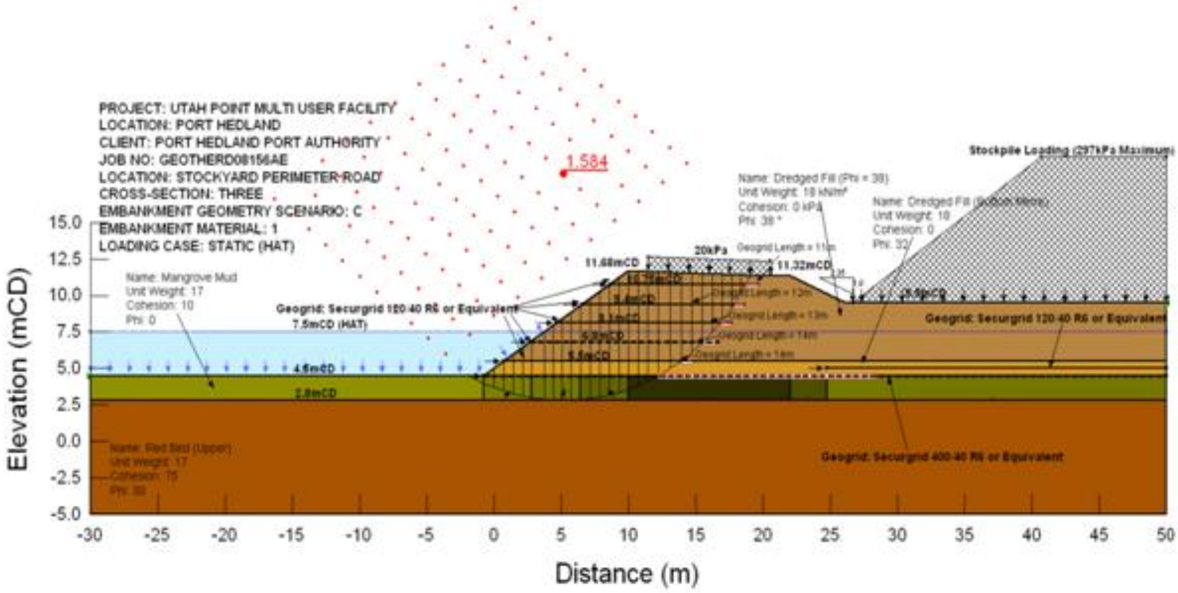

Figure 2 Typical SLOPEW Output – Road Embankment, Highest Astronomical Tide Condition

Table 7: Stockyard Floor Stability Due to Stockpile Loading

Location	Material	Water Table	Factor of Safety
Mud thickness >1.5m	Dredged	0mCD	1.74
		7.5mCD	1.54

3.4 Finite Element Modelling

Analysis of deformation and stability during construction, and deformation at the end of construction of the stockyard facility, based on an assumed 20 years design life, was undertaken using PLAXIS finite element modelling program (PLAXIS). Three cross-sections were selected for analysis based on different mud thickness (0.5m, 1m and >1.5m respectively).

3.4.1 Stress Initialisation

Gravity loading: This event modelled the deposition of the sediments over geological time and calculated the initial ground stresses. Approximately 1000 years of consolidation to model the initial deposition and creep of the Mangrove mud layer.

3.4.2 Stockyard Embankment Construction (Water level at 0mCD)

Water level at lowest astronomical tide (0mCD) was considered more critical to stability than highest astronomical tide (7.5mCD).
Step 1: Placement of basal geogrid, 1st 1m of fill materials, and secondary geogrid underneath the stockyard floor. Assumed duration of 7 days to allow for consolidation of the mangrove mud layer.

Step 2 to completion of embankment construction at Step 11: Placement of 0.5m of fill materials in each step together with placement of the geogrid reinforcement where required. Assumed duration of 7 days for each step to allow for consolidation of the mangrove mud layer. Vehicle load of 20kPa was applied at Step 11.
Step 12: Placement of iron ore stockpile to full height of 16.5m. Assumed duration of 7 days. Step 1 to Step 12 is equivalent to a construction time of about 3 months.
Step 13: Consolidation and creep during the assumed 20 years design life of the facility.

3.4.3 Results of Analysis

The PLAXIS modelling indicated the following results for three sections analysed.

- The calculated safety factors during construction (from Step 1 to Step 12) varied from 1.4 to 1.6, which are larger than the minimum required construction stability of 1.3.
- Embankment movements due to the self weight of the embankment of perimeter road, stockyard floor at conveyor location, and stockpile location for the end of construction and 20 years design life assumed for the facility are presented in Table 8.

A typical PLAXIS output is shown in Figure 3.

Table 8: Predicted Embankment Movements (PLAXIS)

Cross Section	Stage	Crest (South) (mm)		Crest (North) (mm)		Stockyard Floor, Conveyor (mm)		Stockyard Floor, Stockpile Loading (mm)	
		Horizontal	Vertical	Horizontal	Vertical	Horizontal	Vertical	Horizontal	Vertical
Section 1	EoC[1]	40	185	5	115	10	100	-	-
	Design Life	35	30	5	10	5	10	35	200
Section 2	EoC[1]	15	115	20	145	5	65	-	-
	Design Life	10	5	10	5	5	5	45	95
Section 3	EoC[1]	40	195	5	160	10	85	-	-
	Design Life	15	30	5	15	5	15	45	165

Note: EoC – End of Construction

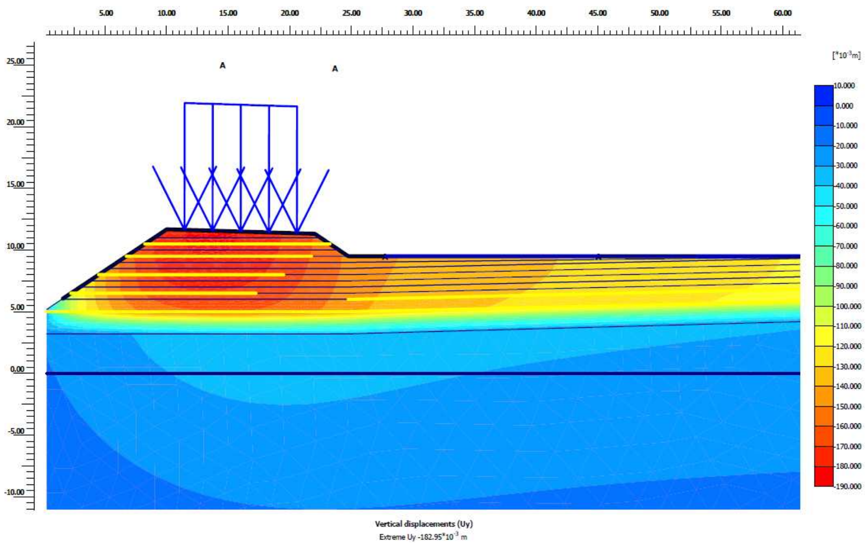

Figure 3: Typical PLAXIS Output – Vertical Displacements of Perimeter Road

3.5 Construction Procedure

The following construction sequence was adopted.

(i) Chain saw the mangrove trees to ground level without disturbing the root zone below the ground surface thus retaining some of the root system as a reinforcement zone.

(ii) Lay out the geotextile drainage separator layer on the mud with a minimum overlap of 1.0m. The geotextile shall be non-woven needle-punched 100% polyester filament fabric, such as Bidim A29, or an approved equivalent.

(iii) Lay out the basal geogrid reinforcement layer (where required) and hand tension to remove any folding or slack in the geogrid. Overlap of rolls to be 600mm with cable ties at 300mm intervals in both directions to maintain tension over joints. The geotextile reinforcement layer was Secugrid 400/40 as shown on Figure 3.

(iv) Spread the first layer of fill material over the geotextile layers in an initial 1.0m thick bridging layer using light tracked construction equipment that has low ground pressures, ensuring that all equipment only operates over at least 1.0m of fill. The bridging layer is anticipated to be compacted by loading from the construction equipment. The fill in this initial layer is to be free draining sandy or sandy gravelly material, which shall have fines content (grain sizes finer than 75μm) of not more than 8% by weight.

(v) Install geotechnical instrumentation (settlement cells, inclinometers, piezometers) and associated survey datums at selected locations

(vi) In some areas of the stockyard floor (where required), lay out the geogrid reinforcement layer Secugrid 120/40 or an approved equivalent and hand tension to remove any folding or slack in the geogrid. Overlap of rolls to be 600mm with cable ties at 300mm intervals in both directions to maintain tension over joints.

(vii) For succeeding fill layers to achieve the required elevation, dredged materials, with a soaked CBR of not less than 12%, shall be used and compacted in a loose layer lift not exceeding 250mm. These layers were compacted to achieve a dry density ratio of not less than 95% of Modified Maximum Dry Density.

Figure 3 Photograph of Basal Geogrid Reinforcement

4 PERFORMANCE

The stockyard facilities were commissioned in 2010. Instrumentation monitoring results indicated settlements to be within the expected range. In 2011 the Utah Point Multi User Bulk Export Facility project won the Australian National Engineering Excellence Award for implementing a range of innovative and new technologies.

5 ACKNOWLEDGEMENTS

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