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Application of High Strength Woven Geotextile in Embankment Stability Control on Soft Soils

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ABSTRACT
In a fast paced construction world, the application of high strength woven geotextile as tensioned membranes to restrict the vertical displacement of ground underneath and subsequently increase the stability of road embankments is becoming more popular. Due to the relatively low permeability of soft soils, particularly clayey materials, the drainage and consolidation process is significantly slow. The road embankments need reinforcement to provide additional lateral resistance to the soft soil before they are able to support their own weight. The main objective of this study is to evaluate the constitutive model of finite element analysis compared to field measured results in the Brisbane River deltaic soft soils nearby Northern Bifurcation of Gateway Motorway. This area consists mainly of Holocene Clays up to 26m depth. More than 1,000,000m^2 of high strength woven geotextile has been used as reinforcement to control stability in heavily instrumented road embankments located in the Airport Interchange area of Brisbane, Australia. Lateral ground movement observed due to field horizontal displacement recorded by inclinometers installed along the embankments compared to the consolidation process by fill activity were computed using 2D finite element analysis was quantified as horizontal displacement ratio. The horizontal displacement ratio observed was ranged from 0.2 to 0.4 in reinforced embankments and 0.1 to 1.7 in unreinforced embankments. The study acknowledged that the ideal horizontal displacement ratio based on standard constitutive finite element model shall equal to one. However, the case study of reinforced embankments with high strength woven geotextile demonstrated horizontal displacement ratio is less than one.

1. INTRODUCTION

In 1986, Queensland Motorways delivered the historic Gateway Bridge which was built at a cost of $140 million and has been heralded a great engineering triumph. The Gateway Bridge and sections of the motorway are either at, or fast approaching capacity, which necessitates a need for a new Gateway Motorway deviation and airport interchange. The $1.88 billion Gateway Upgrade Project is the largest bridge and road project in Queensland's history. It is a State Government initiative being delivered by Queensland Motorways, with design, construction and maintenance by the Leighton - Abigroup Joint Venture.

The project involves the construction of a second Gateway Bridge, the refurbishment of the existing Gateway Bridge, a 12km upgrade to the Gateway Motorway and 7km's of new motorway. Construction works commenced early 2007 with the project scheduled for completion in early 2011.

A total of sixteen bridge structures will be built for the deviation however for the purpose of this paper we will focus on the Embankments for Bridges 19B and Bridges 25A & B which incorporated the use of high strength woven geotextile.

2. GEOLOGICAL PROFILE DESCRIPTION

The geological sequencing of the Northern Kedron Brook and Airport Drive areas consists of the upper and lower Holocene deposits underlain by the relict Pleistocene alluvia, residual soils and rock.

The Upper Holocene alluvia were laid down during the most recent rise in sea level, in shallow fluctuating water bodies, and are comprised of interlayered clays, silts, and sands, sometimes with peaty inclusions. They are present from the ground surface (or from the base of any site fill) and are usually between 6m and 12m thick. These alluvia are highly compressible (apart from a shallow crust) but usually consolidate relatively rapidly.

The lower Holocene alluvia were laid down in deeper water, either off-shore or in deeper stream channels. They tend to be silty clays underlain by sandy layers and extend to significant depths; in excess of 30 m in some places. They are highly compressible, and because they lack persistent layers of sand, they consolidate relatively slowly taking years or even decades to complete primary consolidation depending on their thickness.

The underlying Pleistocene deposits generally comprise stiff to hard clayey and medium dense to very dense sandy gravely materials. Their upper profile was a former land surface, shaped by erosion and stream cutting during lower sea levels. Rock, present beneath the alluvia, consists of the Tertiary-age Petrie Formation which comprises mudstone, shale, sandstone, oil shale and pebble and cobble conglomerate.

The geological profiles of the various sections of the road embankment and the airport interchange area are shown in Figures 1 and 2. General soil properties from laboratory tests at the Airport Interchange area are provided in Figure 3.

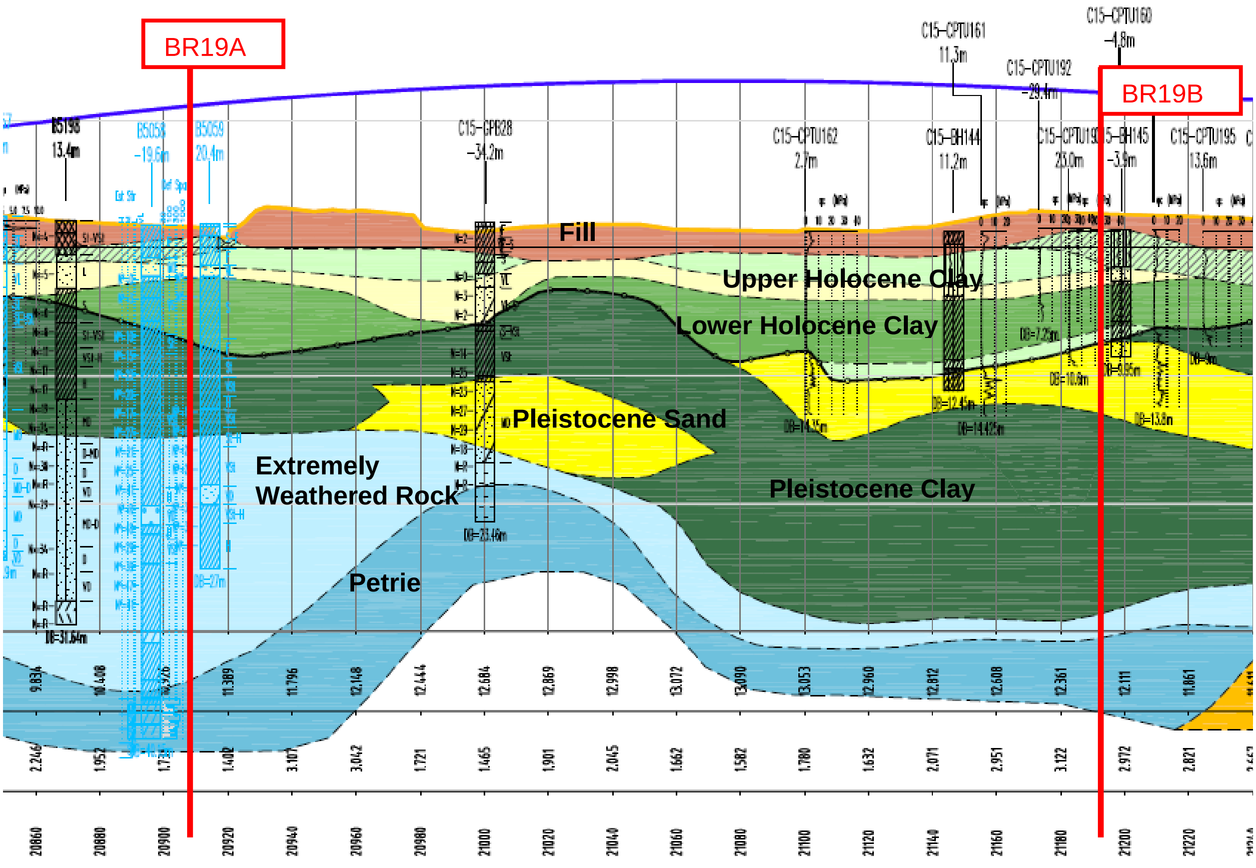


Figure 1. Geological long section profile – BR 19A & B (Coffey Report, 2007)

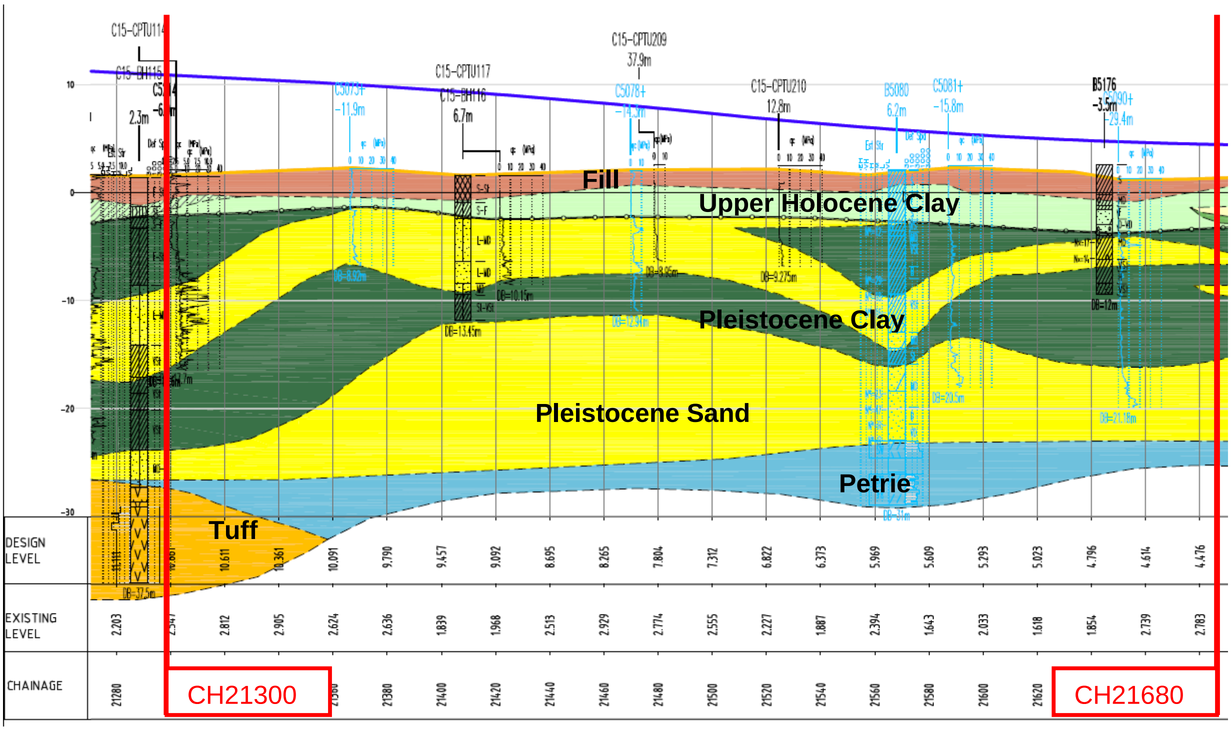


Figure 2. Geological long section profile – CH21300 & CH21680 (Coffey Report, 2007)

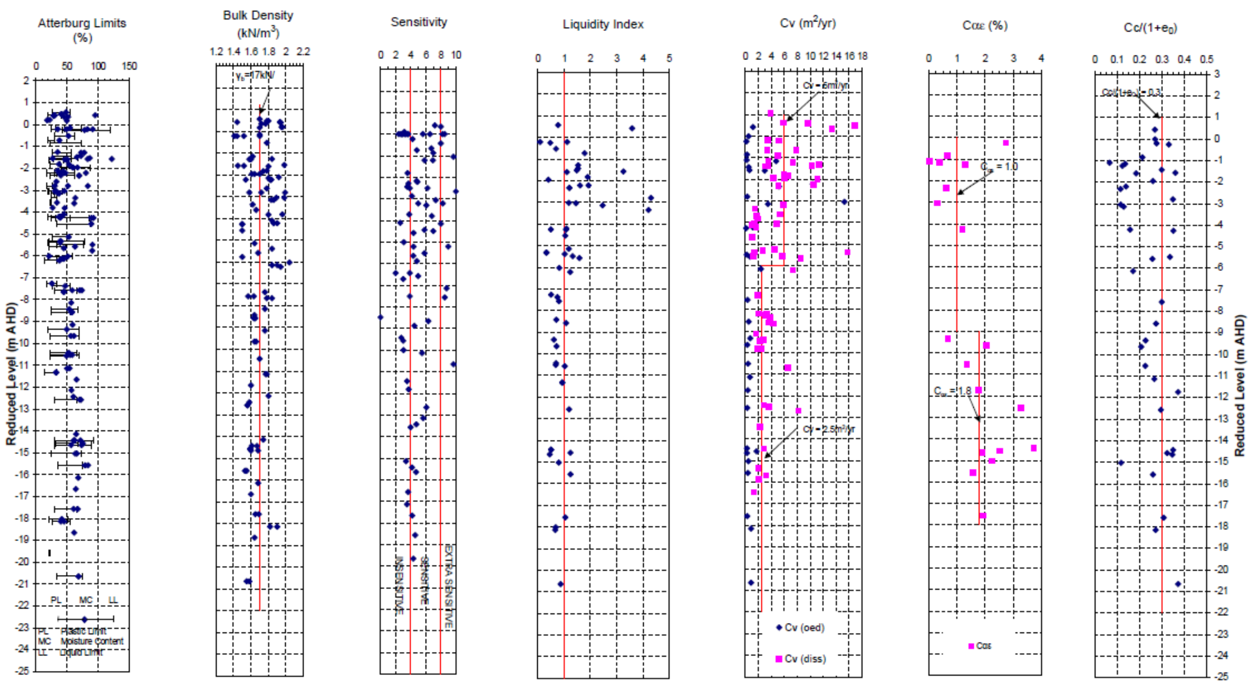


Figure 3. General soil properties from laboratory and field dissipation tests at Airport Interchange area (Coffey Report, 2007)

3. CONSTRUCTION METHODOLOGY

The embankments were constructed using a 300mm layer of fine silty sand placed on top of the existing alluvial crust which was then lightly compacted. Two layers of Polyfelt WX high strength woven geotextile were then placed on top of the fine silty sand layer and each layer of geotextile separated by another 300mm blanket of fine silty sand. The sand blanket maintains friction between the reinforced soil layers and minimizes construction damage. Embankment BR19B had an additional layer of high strength woven placed 6m above the toe of the embankment. To achieve the embankment design height within construction program, subsequent layers of engineered fill 1.5m deep were placed every 10 working days.

The cross sections of both geotextile reinforced and unreinforced embankments are shown in Figures 4 and 5.

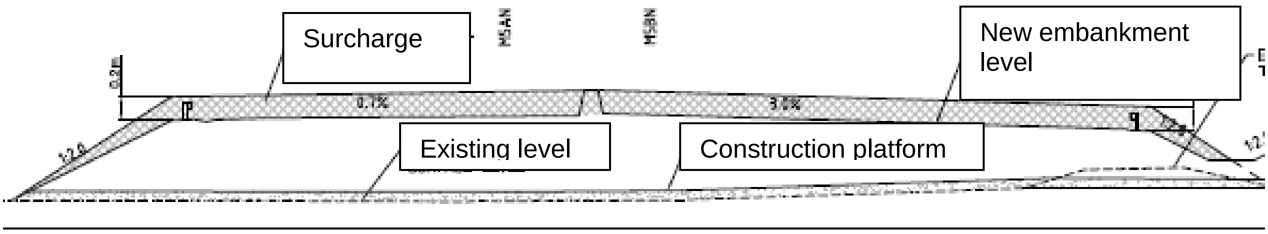


Figure 4. Typical geometry of unreinforced road embankment

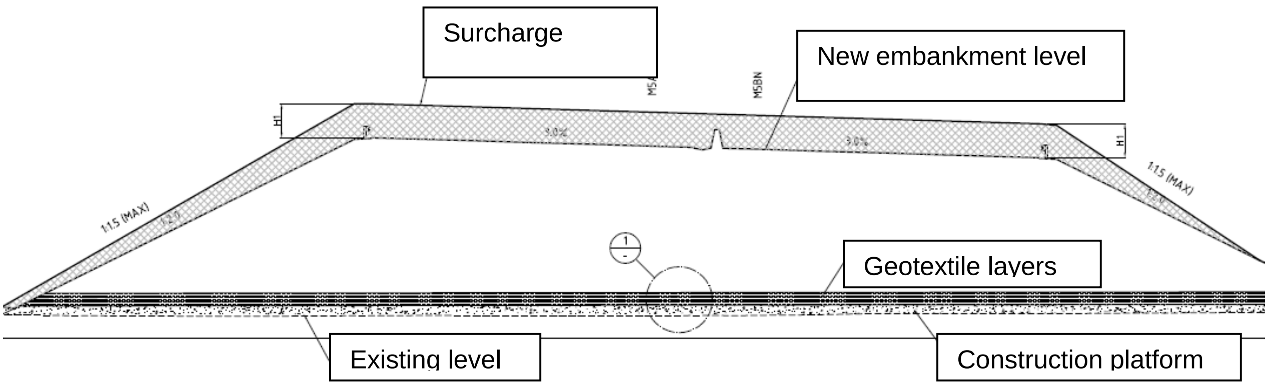


Figure 5. Typical geometry of geotextile reinforced road embankment

Table 1 below indicates the geometry of the embankments as well as preload conditions, depths of Holocene clays and the number of high strength woven geotextile layers used at selected section of the embankments.

Table 1. Embankment profiles and number of high strength geotextile layers

Area	Height of embankment (m)	Surcharge height (m)	Total height (m)	Preload period (months)	Soft clay thickness (m)	Layers of high strength woven geotextile
21+300	9.8	0.5	10.3	3.0	1.0	None
21+680	2.9	2.1	5.0	6.0	6.5	None
BR19B	11.0	3.0	14.0	3.0	5.0	3 layers of WX 800
BR25A	4.3	4.3	8.6	6.0	20.0	2 layers of WX 600
BR25B	4.3	4.3	8.6	6.0	20.0	3 layers of WX 600

Inclinometers were placed at the toe of all the reinforced and unreinforced embankments in order to measure lateral movements generated by applied overburden pressure (Figure 6). Field measured values were recorded and are shown in Table 4.

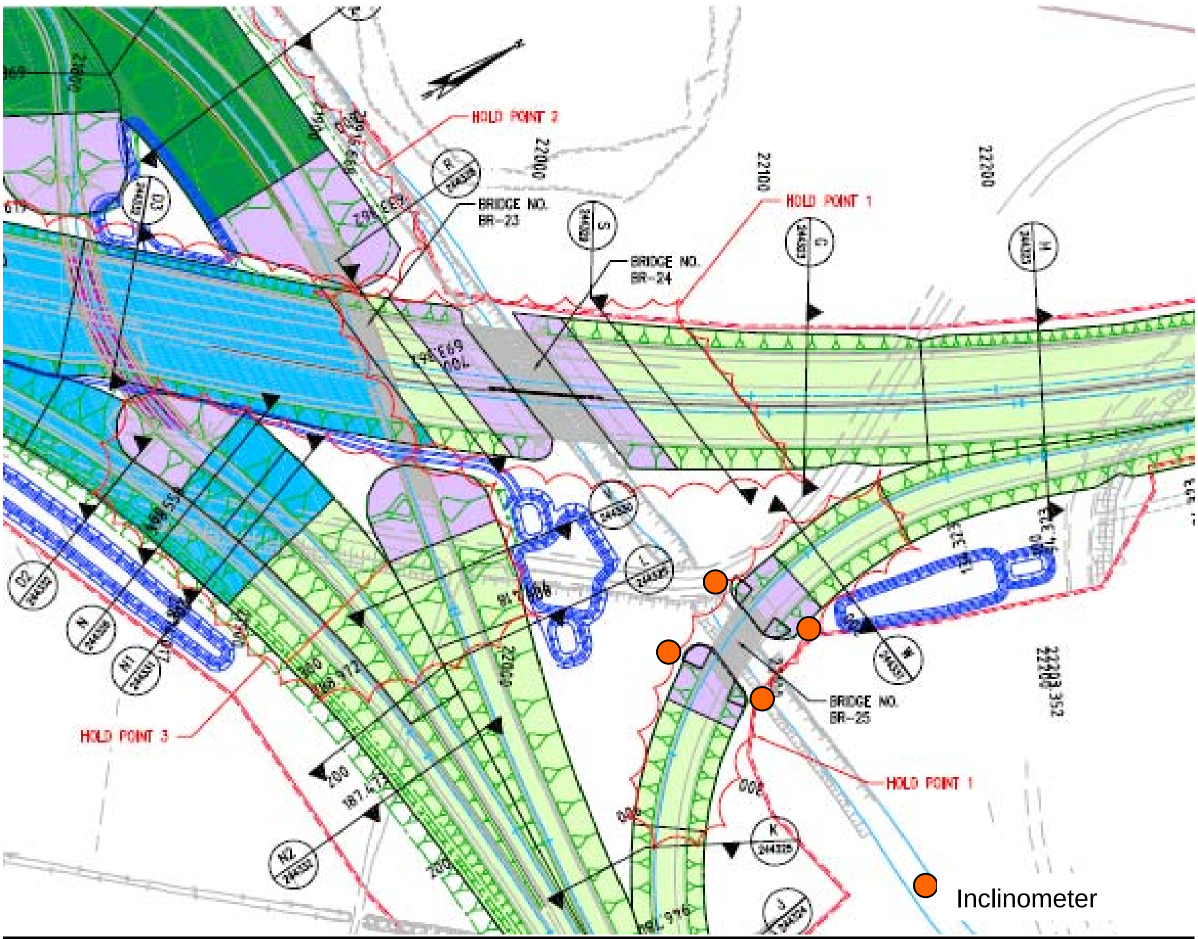


Figure 6. Plan view showing locations of inclinometers for BR25A & B

4. GEOSYTHETIC MATERIAL PROPERTIES

Polyfelt WX was selected as the most appropriate geosynthetic reinforcement to provide short and long term stability of the embankments in this project. Polyfelt WX is manufactured using high modulus polyester fibers to exhibit very low creep strains at high tensile load levels. The polyester fibers are assembled to form a directionally structured and stable geotextile that enables maximum load carrying capacity and efficiency. Partial material factors were adopted to determine the long term strength characteristics under specific load and environmental regimes. These factors are shown in Table 2. Figure 7 shows the placement of the high strength geotextile at the base of the embankment.

Table 2. Mechanical properties of Polyfelt WX high strength woven geotextiles

Area	Product	Short term tensile strength (kN/m)	Partial factor creep (F_c)	Partial factor construction damage (F_d)	Partial factor environmental Effects (F_e)	Long term tensile strength (kN/m)
BR 19B	WX 800	800	1.55	1.00	1.1	469.2
BR 25A	WX 600	600	1.55	1.00	1.1	351.9
BR 25B	WX 600	600	1.55	1.00	1.1	351.9

Figure 7. Polyfelt WX installed at BR25A and BR25B

5. 2D FINITE ELEMENT MODELING

The 2D finite element modeling approach adopts the Mohr-Coulomb drained soil model in order to calculate the maximum horizontal displacement at the toe of the embankments. Geotextile was modelled as reinforcement in the embankment with the input lateral resistance as design lateral resistance (Td) multiplied by partial factors, in accordance with BS8006:1995. A horizontal deflection plot from the analysis output was generated for each section of modelled embankments, as maximum horizontal displacement, compared to the field measurement to generate horizontal displacement ratio. Drained shear strength (Cu) was adopted as Cref in the analysis, considered the drained approach is more appropriate in predicting the maximum horizontal displacement during construction.

It is acknowledged that the Mohr-Coulomb approach has its limitations in material model to achieve the objective of this study. However, this is the industry conventional approach without involving the sophisticated modelling process. The following parameters adopted to model the 2D finite element analysis are as shown in Table 3. The results of the finite element analyses are shown in Figures 8 to 11.

Table 3: Soil Parameters Used For 2D Finite Element Models

Soil Type	γ_{sat} (kN/m³)	E_{ref} (kN/m²)	C_{ref} (kN/m²)	θ (Degrees)	υ (Poisson's Ratio)
Engineered Fill	20	20 000	5	30	0.30
In Situ Fill	16.5	15 000	1	30	0.30
Alluvial Crust	18	12 000	5	26	0.33
Soft Clay	16	3 750	0	20	0.33
Stiff Clay	17	25 000	2	24	0.33
Very Stiff Clay	18	30 000	5	28	0.33
Very Loose Sand	17	8 000	0	28	0.30
Loose Sand	17	10 000	0	28	0.30
Medium Dense Sand	19	20 000	1	30	0.30
Dense Sand	19	40 000	1	32	0.30
Very Dense Sand	19	60 000	2	34	0.30
Gravel	21	75 000	2	34	0.30

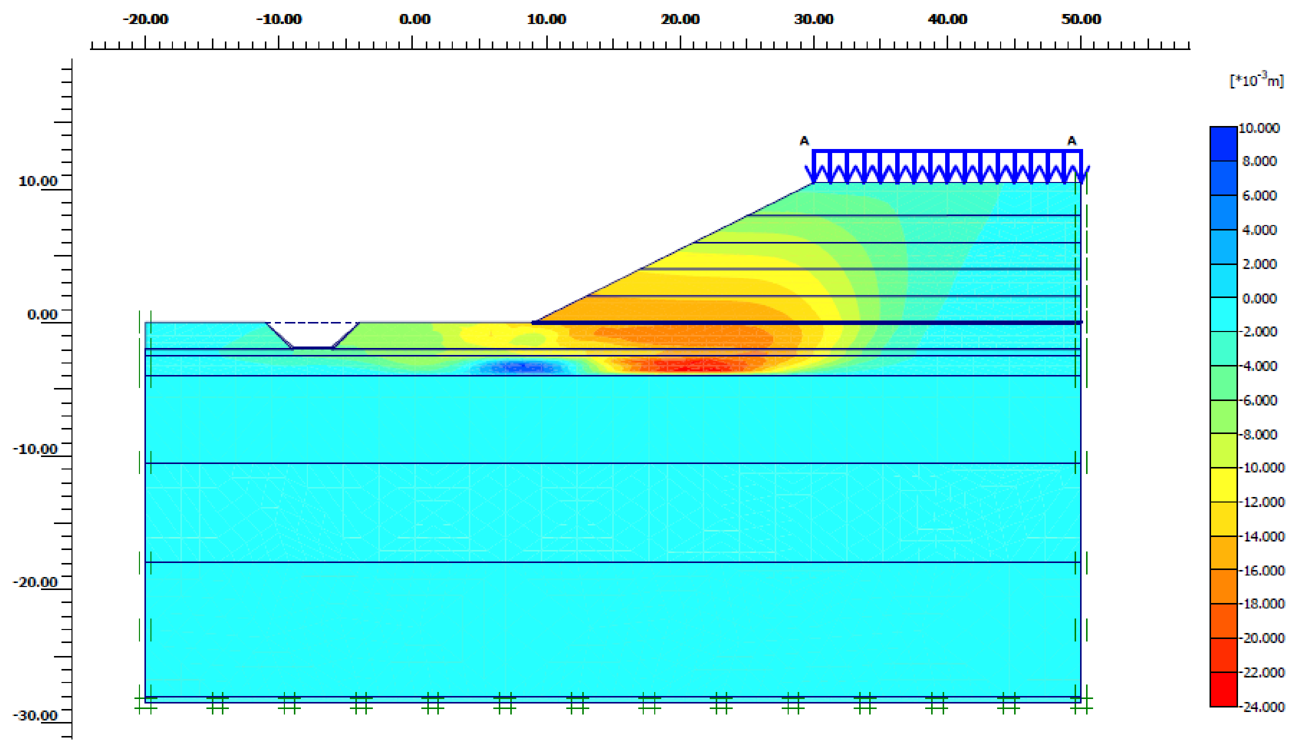


Figure 8. Results of 2D Finite Element analysis on 21+300 (unreinforced sections)

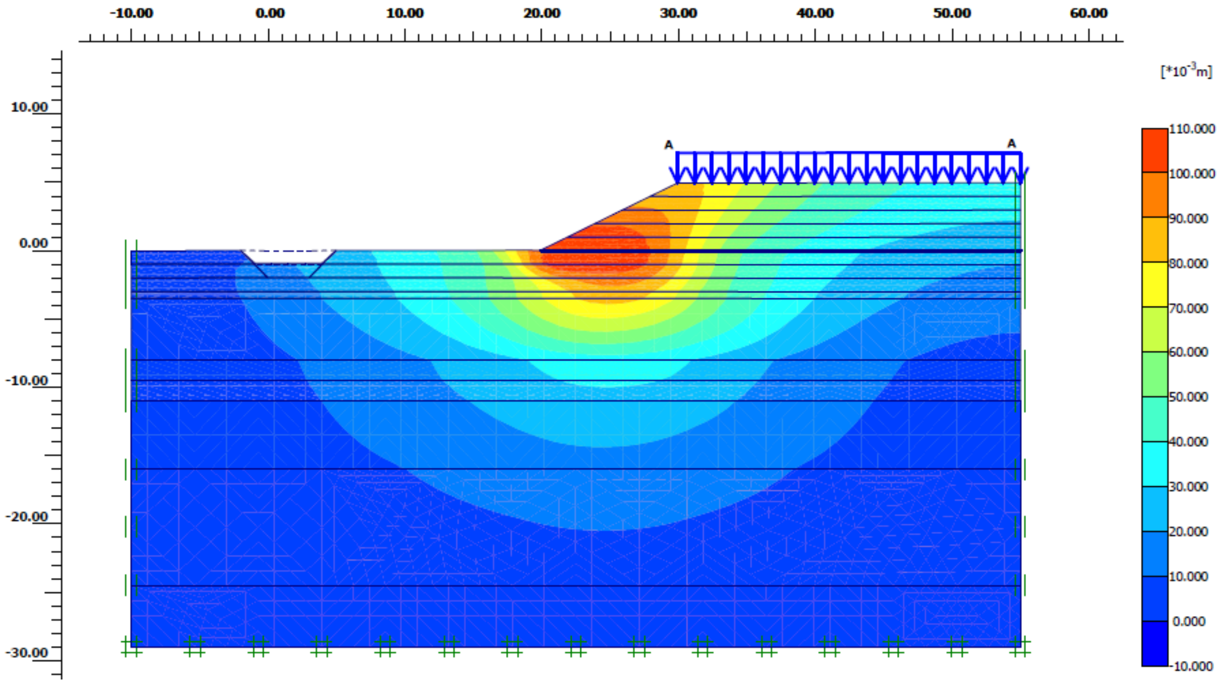


Figure 9. Results of 2D Finite Element analysis on 21+680 (unreinforced sections)

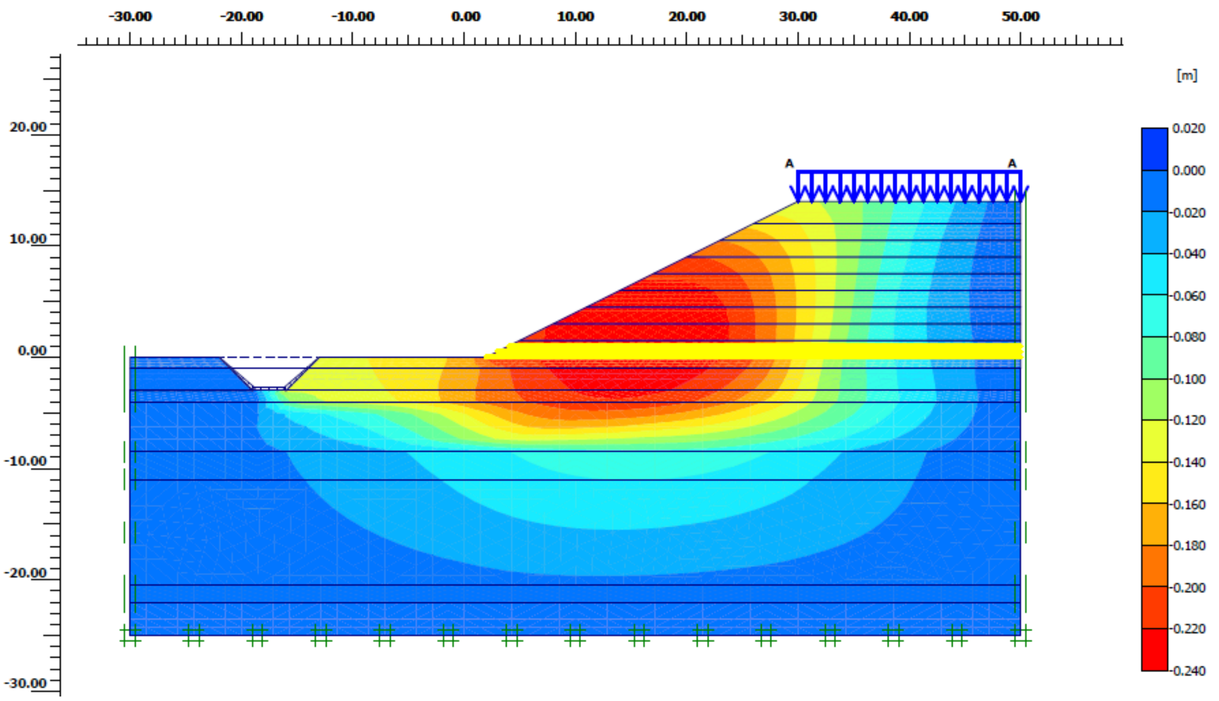


Figure 10. Results of 2D Finite Element analysis on BR19B (reinforced sections)

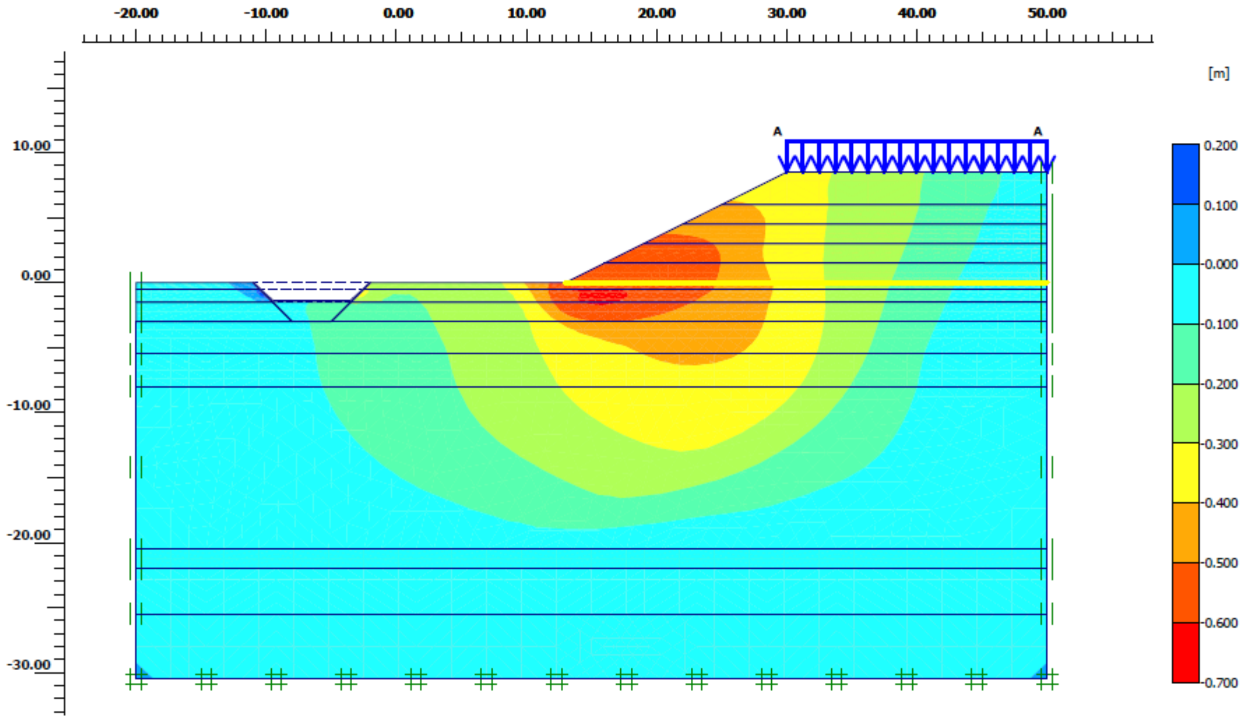
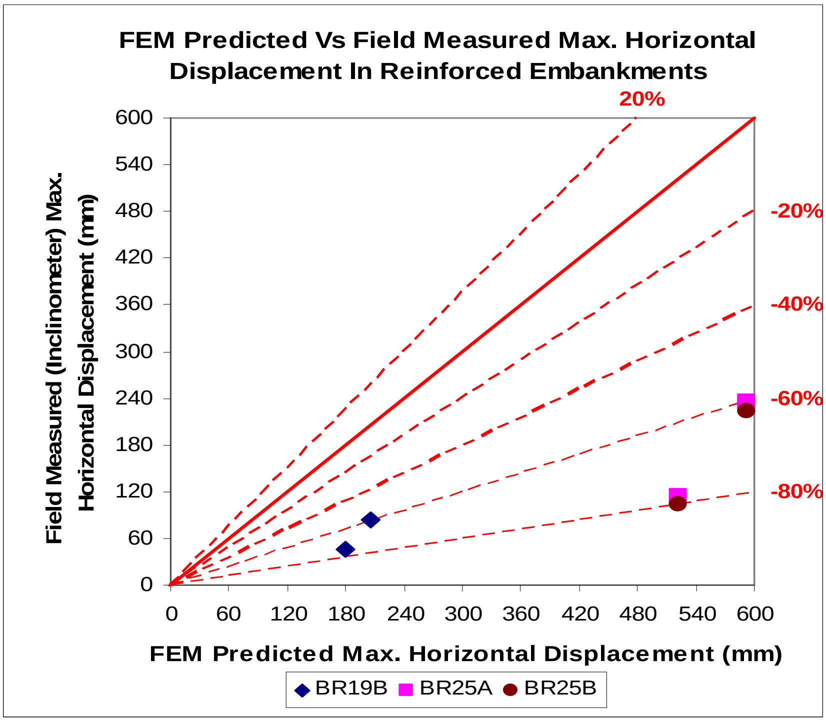


Figure 11. Results of 2D Finite Element analysis on BR25A & B (reinforced sections)

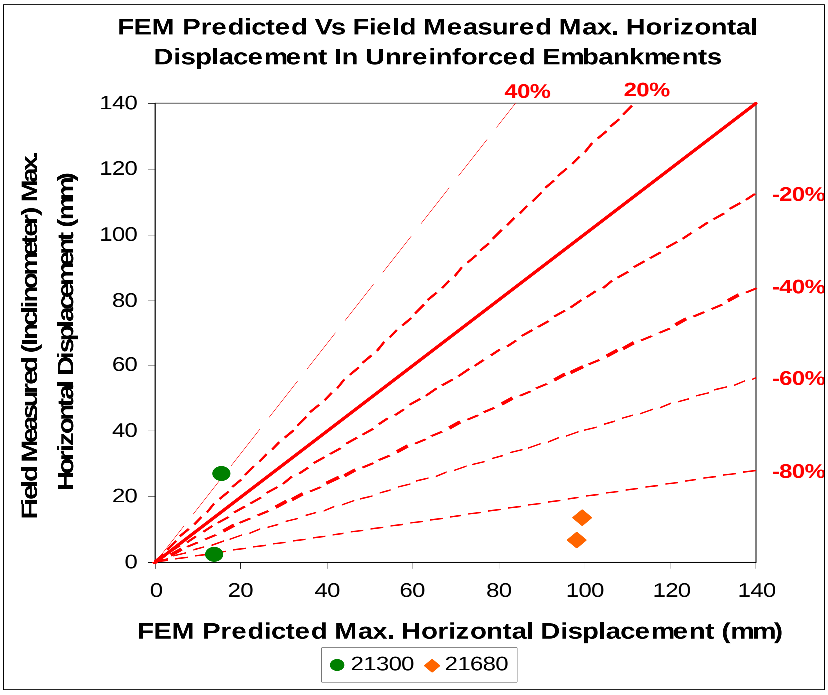
6. RESULTS

Table 4: 2D Finite Element Analysis Results Vs Field Measured Results

Area	(A) 2D FEM Predicted horizontal displacement (mm)	(B) Field measured horizontal displacement (mm)	Inclinometer ID	Horizontal Displacement Ratio (B / A)
BR19B	205.6	83.6	I44-NB	0.4
(Reinforced)	178.9	44.9	I45-SB	0.3
BR25A	523.1	113.6	I10-NB	0.2
(Reinforced)	592.1	235.5	I11-SB	0.4
BR25B	523.1	103.6	I12-NB	0.2
(Reinforced)	592.1	222.7	I13-SB	0.4
21300	15.5	26.9	I3-NB	1.7
(Unreinforced)	14.1	2.1	I4-SB	0.1
21680	98.1	7.0	I13-SB	0.1
(Unreinforced)	99.7	13.7	I14-NB	0.1



Figures 12: FEM vs Field measured horizontal displacements for reinforced embankments

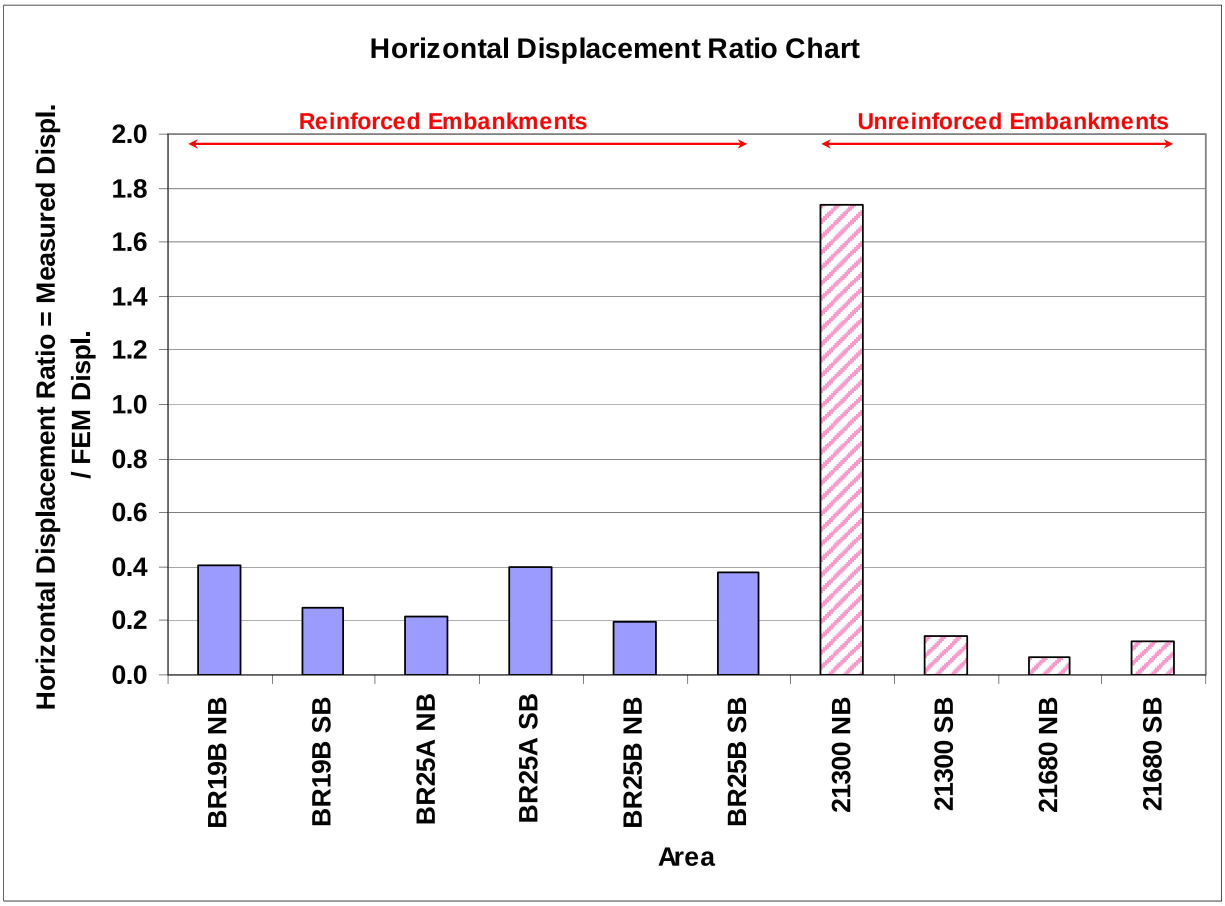


Figures 13: FEM vs Field measured horizontal displacements for unreinforced embankments

The Figure 12 above demonstrate that embankments BR19B and BR25A&B, which were reinforced with high strength woven geotextile, recorded smaller maximum lateral displacement measured by the field inclinometers compared to the lateral displacement computed by the finite element analysis. In the analysis, the models were built on the construction program in order to simulate the actual field conditions. The 2D Finite Element analysis over-estimated the maximum lateral displacement by 60% to 80%.

In the case of the embankments that were not reinforced with high strength woven geotextile (21+300 and 21+680), the results show that the maximum lateral displacement recorded in the field by inclinometers compared to computed by the finite element analysis varies significantly..

It is noticeable from Table 4 that there is a significant difference in lateral displacement measured by the inclinometers at either end of the embankment toes. This can be explained in the case of embankments BR19B and 21+300. A water pond was built along the north bound corridor to collect rainwater for construction use. The water pond may well account for the difference in the north and south bound displacement with higher lateral displacement recorded at the north bound toes of the embankment. In addition, embankments BR25 A&B were constructed over an old channel which has been relocated. Again, this may account for the difference in north and south bound displacement, although the water pond and old channel were modelled in 2D finite element analysis.



Figures 14: FEM Horizontal Displacements Vs Field Measured Horizontal Displacement

In order to examine the over-estimation of maximum lateral displacement recorded by the 2D finite element analysis compared to field recorded inclinometer readings, a horizontal displacement ratio is established and shown in Figure 14. Horizontal displacement ratio is quantified as maximum lateral displacement measured by the field inclinometers divided by 2D finite element predicted horizontal displacement.

From Figure 14, the horizontal displacement ratio ranging from 0.2 to 0.4 in reinforced embankments and 0.1 to 1.7 in unreinforced embankments.

7. CONCLUSIONS AND RECOMMENDATIONS

The reinforced embankments performed well with the assistance of additional tensile resistance provided by the high strength geotextile reinforcement as demonstrated from the observed horizontal displacement ratio ranging from 0.2 to 0.4, below the ideal value of 1. For the unreinforced embankments, an inconsistence displacement ratio was observed ranging from 0.1 to 1.7. The application of high strength geotextile provides higher level of confidence in construction as demonstrated from an average lower and consistence horizontal displacement observed in the case study.

The study acknowledged the application of Mohr-Coulomb approach in 2D finite element analysis has its limitations in material model to achieve the objective of this study. The adopted methodology is to demonstrate the conventional or standard industry approach in obtaining the modelled maximum lateral displacement without involving the sophisticated process, for further comparison with field measured values.

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