THE USE OF GEOYNTHETICS TO IMPROVE SOFT SUBGRADE RAILWAY FOUNDATIONS

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SUMMARY

This paper describes the methodology followed to evaluate the ability of generic geosynthetic products to improve the bearing capacity and resistance to differential settlement of track formations on poor subgrades. Extensive laboratory testing, finite element analyses, test site construction and monitoring formed part of this research and are described in the paper. Measurements over a 34 month period, through dry and wet seasons, showed how the different products as well as a special dumprock design, possess significant superior strength over the conventional design with no geosynthetic reinforcement. The function of a separation geosynthetic to reduce natural moisture content changes within the formation also played a vital role in the performance of the various alternatives. The site was evaluated using electronic survey, subgrade moisture content measurements, track geometry measurements by the EMV80 track geometry car over a period of time, dynamic stiffness modulus evaluations with the LWD (Light Weight Drop Apparatus), formation bearing strength measurements, predictions of heave, geological evaluation and visual observations. Lastly, maintenance history data also assisted in drawing conclusions. From the gathered measurements, meaningful scientific results were obtained to support the use of geosynthetics to reduce track geometry deterioration on poor subgrades. As a result, superior formation designs have been adopted where problem soils associated with heaving, compressible and expansive clays were encountered. This approach has been used successfully in other geographical locations on Transnet Freight Rail (TFR) lines in Southern Africa where the specific formation designs have doubled the duration of the tamping maintenance cycles. Recommendations for various formation design scenarios and options for new and existing track formations subjected to similar geological, geotechnical and climatic conditions, have been developed based on the findings from this research.

1 INTRODUCTION

1.1 Background

A Geotechnical investigation was carried out in 1998 by Spoornet Technology Management (Track Technology) [1] on the Pendoring to Thabazimbi Railway Line from km 57.21 to km 248.78. Formation problems and the possibility of increased axle loading and annual tonnage necessitated this investigation. The report identified the following four causes for the formation problems on the line:

- Non-conformance with the S410 Specification for Railway Earthworks
- Swelling and shrinking of the subgrade material (especially the so-called black turf)
- Lack of shear resistance in the subbase material
- Insufficient bearing capacity of the subgrade

This paper addresses the improvement of the formation conditions on the Pendoring to Thabazimbe line by making use of geosynthetic products and selective soil reconstruction. A cost-effective solution is sought that will address the formation problems as discussed above in the context of future upgrading of the line.

1.2 Literature

The Literature contains numerous references to the successful use of geogrids, geocells and geotextiles as reinforcement over soft subgrades. In summary, they are used in the following applications:

- Geogrids and geocells are used to provide tensile reinforcement and shear resistance to increase the effective bearing capacity of the subgrade.
- Geogrids are used to interlock with and confine the ballast, increasing its resistance to both vertical and lateral movement.
- Geotextiles are used for separation and filtration to prevent contamination of the ballast and provide quick relief of pore water pressures. In addition, they can also minimize changes in subgrade field moisture content.

1.3 Research Methodology

The methodology that was followed to investigate the possible use of geosynthetic products and selective soil reconstruction to improve formation conditions consisted of three different phases:

1. Laboratory testing to evaluate a range of geosynthetic products under controlled conditions and to single out good performers in the three categories of geotextiles, geocells and geogrids.
2. Finite element analyses to study the expected effectiveness of different geosynthetics and combinations of those.
3. The construction of selected test sections and the monitoring of those at a suitable site on the Pendoring to Thabazimbe railway line.

Each of these three phases and the research findings from each [2, 3, 4], will be described in detail in the following paragraphs.

2 GEOSYNTHETICS PRODUCTS

This section of the paper gives an overview of the geosynthetic product groups and particular products that have used to increase the track stability and bearing capacity over soft soils. For background purposes a brief from various related publications is outlined in the paragraphs below:

Geocells – Rajagopal et al. [5] showed that a granular soil develops significant apparent cohesive strength due to the confinement by a geocell. Their tests included triaxial compression tests on different samples of granular soil encased in woven and nonwoven geotextiles and soft mesh. It was also shown that the magnitude of apparent cohesive strength was dependent on the strength characteristics of the geocell material.

Reasearch by the Association of American Railway Technology Department [6] showed that a geocell product can be used to significantly reduce traffic-induced stresses in the track subgrade. A test section with soft, deformable clay subgrade under heavy axle loading of 36-ton was used to demonstrate how the use of geocells increased the average tamping cycle of about 15 MGT (million gross tons) to 85 MGT with only minimal loss of track geometry. In these tests, a 400mm subballast layer reinforced with geocells replaced the clayey layer.

Luo et al. [7] carried out full-scale dynamic tests to evaluate the effects of geocells in stabilization of railway subgrades. With geocells embedded in the upper layer of the subgrade, maximum static and dynamic stresses in the subgrade were reduced. Similarly, vertical deformation and permanent strain accumulation were greatly reduced.

British Rail [8] chose a cellular confinement system (i.e. a geocell) to upgrade the track subballast on their Edinburgh to London Intercity line in Northcumberland. Two layers of geocells were used to stiffen the track subballast over soft subgrade consisting of peat up to a depth of 5m. The geocells were placed on top of a geocomposite drainage layer and fast construction resulted in the lifting of the speed restriction on this 200 km/h passenger route.

At different locations in Denmark, plate load tests were carried out to determine the effect of reinforcement on top of granular soils due to the choice of geogrids or geotextiles. Vanggaard [9] showed that nonwoven geogrids placed in the substructure significantly increased the stiffness of the granular soil in comparison to woven geogrids, geotextiles and unreinforced granular soil.

In a paper describing the laboratory tests on a large-scale model of a single tie-ballast system over artificial subgrades of variable compressibility, Buthurst et al. [10] concluded that the rate of permanent deformation accumulation was substantially lower for reinforced sections over compressible subgrades. An open-grid polymer based geogrid was seen to decrease the permanent deformation accumulation rate by up to 50% when compared to unreinforced test sections.

Chang et al. [11] carried out a laboratory study with a dynamic test system on geogrid reinforced subgrade soil. Repeated load test results showed that geogrid reinforcement is highly effective as reflected in factors related to foundation stiffness and the amount of deformation associated with repeated addition of heavy loads.

In a study of bearing capacity of a reinforced sand layer overlaying soft clay subgrade, Kenny [12] suggested that in order to function effectively as a reinforcement layer over very soft clays, the geosynthetic should possess a significant component of in-plane bending stiffness.

British Rail Research trials carried out on the rolling road rig at its laboratory in Derby demonstrated that inserting geogrids in the ballast can help extend maintenance intervals by minimising settlement where tracks lie over soft ground. The use of a geogrid with high profile ribs which provide a good interlock with the ballast can limit creep of the ballast particles and hence reduce the settlement and rate of deterioration of the vertical track geometry. Field tests on the East Coast Main line showed the effectiveness of the geogrids for ballast reinforcement [13].

Successful subgrade reinforcement with the use of geogrids has also been reported by Jain and Azeem [14] and Tanabashi et al. [15].

Forsman et al. [16] used a combination of nonwoven geotextiles, geocells and geogrids to increase low bearing capacity and large soil settlement in the construction of secondary road over deep peat deposits.

A combination of geotextiles and geogrids were used for track rehabilitation in Alabama, USA. Geogrids was used to provide tensile reinforcement and shear resistance to increase the effective bearing capacity of the subgrade. Another function of the geogrid was to interlock with and confine ballast, increasing its resistance to both vertical and lateral movement. A geotextile was used for separation and filtration to prevent contamination of the ballast and provide quick relief of pore water pressures (see Walls and Nerby (1987)) [17].

3 LABORATORY TESTING

In this investigation, geosynthetic products were inserted into the formation at a depth of 200 mm below formation level. The strengthening of the layers due to the reinforcement was investigated by observing the pressures transferred to the layers below the reinforced layer.

The placement of a geosynthetic product with a granular fill may be a solution to inadequate bearing capacity by strengthening the top subballast layer and the entire formation profile. In the case of good quality material, such an application may result into a reduction in the number of structural layers or a reduction of the formation layer depth. Comparisons were made between the stress distribution within a good track formation structure without any reinforcement and the stress distribution within an identical formation with the addition of a particular geosynthetic product.

A detailed account of the laboratory testing is given in a technical report entitled "The use of Geosynthetics as Reinforcement in Formation layers" [2].

3.1 Laboratory test set-up

The laboratory tests were carried out in a purpose-built, 2.4 m by 2.4 m steel box with adequate reinforcement on the sides of the box to limit deformation of the box during loading. The earthworks layers in the box were selected and built according to the S410 Railway Earthworks specification [18]. Figure 1 shows a soil box test set-up used in the investigation.

Figure 1: Soil test box

Figure 2 shows a diagrammatic representation of the laboratory set-up used in the investigation.

The top 200 mm layer (SSB-layer), was placed as shown in Figure 2. This box configuration (with no geosynthetic present in the upper SSB-layer) was used as the "control" test with which all the subsequent tests (with various geosynthetic configurations) were compared. The SSB-layer was changed to achieve the various product configurations at the centre of the box. Eight different product configurations were constructed.

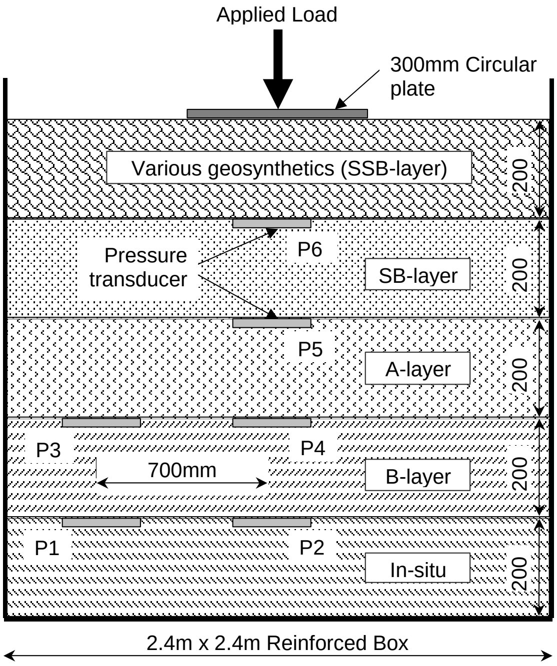

Figure 2: Laboratory test set-ups

3.2 Load application and measurements

An actuator with a 300 mm diameter plate, applied the load. The load was applied at a constant rate from 0 kN up to 14 kN (i.e. a pressure of 198kPa). The pressures at the different interfaces and the total deflection of the structural layers were recorded as the load increased. The measured and applied pressures were recorded and plotted for the various depths.

3.3 Material properties

A summary of material properties used in the Box-test is shown in Table 1.

Table 1: Actual material properties of laboratory layerworks

Layer	Grading Modulus	Plasticity Index	% < 0.075 mm	CBR (%)
SSB	2.18	SP	14	98
SB	2.12	10	18	31
A	2.12	10	18	31
B	1.73	11	16	–
In-situ	1.73	11	16	–

3.4 Results

A summary of the laboratory results is shown in Table 2 below. The table gives the percentage vertical formation pressure reduction for the different geosynthetic products compared to the "control" set-up which did not have any geosynthetic reinforcement. The actual geosynthetic product names have been replaced with numerals for reasons of impartiality and objectivity.

Table 2: Percentage pressure reduction at various depths for selected geosynthetics products

Products	Depth (mm)		
	200	400	600
Geotextiles			
GT1	51%	61%	87%
GT2	22%	52%	83%
Geogrids			
GG1	54%	34%	66%
Geocells			
GC1	24%	5%	24%
GC2	25%	5%	85%
GC3	1%	16%	96%
GC4	34%	53%	93%

The results in Table 2 show that a substantial improvement in load bearing capacity can be obtained by reinforcing the formation with various geosynthetic products. Insertion of a geotextile in the formation layer showed a pressure reduction of 85% on average at 600 mm depth. The geogrid reduced the pressure by 66% at 600 mm depth. The geocells reduced the pressure by between 24% and 93% at 600m depth. To evaluate different combinations and configurations of geosynthetics, the Finite element method was chosen.

4 NUMERICAL ANALYSIS

4.1 General

Finite element analyses (FEAs) were carried out to evaluate the effectiveness of a number of rehabilitation design alternatives. These designs incorporated the use of three different generic types of geosynthetic products, namely a geogrid, a geocell and a geotextile. The replacement of selected layers of subgrade material with quality granular material was also incorporated in the designs.

The report "Thabazimbi Formation Rehabilitation Test Sections"[3] contains full details of these analyses.

4.2 Models for numerical analysis

Figure 3 shows the basic finite element model that was used to create the different design alternatives. The basic model was modified to include the various geosynthetics and new formation layers as per design alternative.

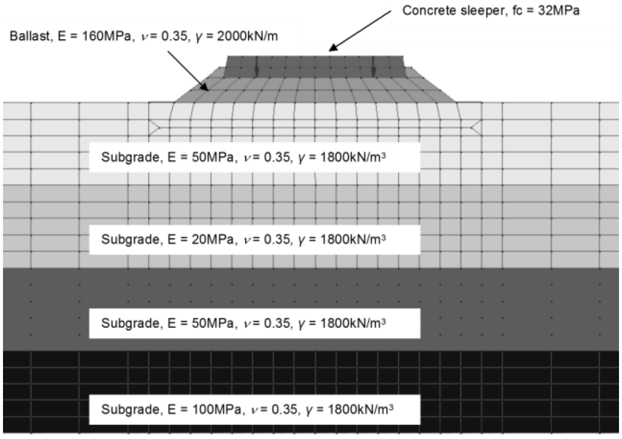

Figure 3: Basic finite element model of track structure and substructure

A brief description of the 10 different FEA models follows:

Table 3: Summary of different FEA models

Model	Description
1	No geosynthetic reinforcement
2	Geogrid directly below the ballast
3	Geogrid at 150 mm and 150 mm thick granular layer
4	300 mm thick granular layer between two layers of geogrid directly below the ballast
5	150 mm geocell at 300 mm below the ballast filled with granular material up to formation level
6	Woven geotextile at 300 mm with granular material up to formation level
7	150 mm geocell at formation level filled with granular material and a layer of geogrid at 300 mm
8	Geogrid at 300 mm and 300 mm thick granular layer
9	400 mm thick granular layer between two layers of geogrid directly below the ballast
10	400 mm thick granular layer between two layers of geogrid at 400 mm with granular material up to formation level

4.3 Loading

Two types of loading were investigated in these analyses, namely (1) a static vertical load of 100 kN per rail, representing 20 ton/axle loading and (2) swelling of the subgrade material resulting in heave.

4.4 Results

Finite element runs were carried out to model 2 different scenarios, namely (1) bearing capacity improvement and (2) clay heaving improvement. The results were then averaged to express each design's effectiveness as a percentage and finally it was combined with a unit product cost (2003 figures) to arrive at a cost-effectiveness factor. The higher this factor, the higher the cost-effectiveness of the specific solution. Table 4 summarizes these results.

The results in Table 4 were then used as a basis for the design of a limited number of test sections to be constructed on the Pendoring to Thabazimbi railway line.

Table 4: Cost effectiveness comparison

Model	Bearing capacity improvement	Heaving improvement	Effectiveness	Product cost/km (xR1000)	Cost-effectiveness factor
1	-	-	-	-	-
2	6%	0%	3%	120	0.3
3	29%	17%	23%	220	1.0
4	57%	34%	46%	440	1.0
5	51%	34%	43%	280	1.5
6	51%	34%	43%	250	1.7
7	57%	34%	46%	400	1.2
8	51%	34%	43%	320	1.3
9	63%	40%	52%	490	1.1
10	80%	69%	75%	740	1.0

5 TEST SECTION AND MONITORING

5.1 Geology of the area

The geology of the area comprises of the Transvaal system and the Bushveld complex. Basic crystalline rocks in this wet region weather into diorite, gabbro, norite, diabase, dolerite and basalt. The end product of these rocks is known to be high in montmorillonite clay, which is high in water adsorption and considered as highly expansive clay exhibiting considerable volume change as a result of moisture content change. The activity of the clay is normally greater than 1.25. Heave predictions of 78mm were calculated from the Van der Merwe method [19].

5.2 General description of the test section

The description of the test sections and monitoring thereof is described in the report "Geotechnical and field Measurements at Amandelbult Test Section"[4].

Amandelbult test section is situated on the Northam - Thabazimbi railway line. During the monitoring period the track section was carrying 2.43 million axles and 9.93 million gross tonnes (MGT) per annum. The test and control sections are between km 212.100 to 213.700 and are each approximately 100 m in length. The track section under consideration consists of 57kg rails on P2 sleepers at a sleeper spacing of 650 mm – 700 mm.

Table 5 gives a summary of the 5 constructed test sections as well as their control sections.

Table 5: Summary of Amandelbult test sections

Section No.	Design	Km Start	Km End
Test 1	Geocell *(GC4)*	212.150	212.250
Control 1 and 2	–	212.250	212.350
Test 2	Conventional	212.350	212.450
Test 3	Geogrid *(GG1)*	212.500	212.600
Control 3 and 4	–	212.600	212.700
Test 4	Geotextile *(GT1)*	212.700	212.800
Test 5	Dumprock	213.200	213.300
Control 5	–	213.300	213.400

In addition to the three geosynthetic test sections, a conventional section was also constructed with no geosynthetic product in the design as well as a fifth test section where dump rock was used. All test sections, except Test 5, was based on a 400 mm structural depth as well as a 200 mm backfill for undercutting. Test 5 consisted of a 1000 mm rock fill below the ballast. In all cases, a separation geosynthetic was placed between the structural layers and the subgrade material (i.e. the material to backfill the undercuts as well as the remainder of the clay layer). The subgrade in all the test sections had CBR of between 1.8% and 4% and average depth of between 990mm and 1570mm. The plasticity index of the subgrade in the test sections ranged from 36% to 68%.

5.3 Monitoring

The periods from track renewal as a result of tamping or reconstruction to the track deterioration will be regarded as a “Cycle”. The entire test site was not tamped for 10, 11 and 13 months in Cycles 1, 2 and 3 respectively and the results presented in this paper covers these periods.

The following tests were carried out at the various test sections and their individual control sections since their construction in July 2004, until June 2007 (end of monitoring), representing three cycles totalling 34 months of monitoring:

- DCP testing
- Natural moisture content (NMC): The subgrade moisture content was measured on a quarterly basis at the bottom of the formation at all test and control sections.
- Light Drop Weight Tester
- Survey levels: Formation levels of both the test and control sections were taken on a monthly basis.
- EMV80 track geometry measuring vehicle results: The geometry of the track on the test and control sections was initially measured on a monthly basis and was later extended to a two monthly cycle.
- Ballast Evaluation: Representative samples of ballast were taken at every test and control section. Details of the results are not discussed in this report.

5.4 Results

5.4.1 Natural moisture content of clay subgrade

Table 6 gives a summary of the average subgrade moisture content and formation gradient of the test and control sections. Subgrade moisture contents were measured on a two monthly basis and the minimum and maximum values are presented in Table 6 below.

Table 6: Summary of average subgrade moisture content

No.	Min. Moisture Content (%)	Max. Moisture Content (%)	Thickness of Clay subgrade (mm)	M.C. change (%)
T1	33.0	45.4	1410	12.4
C1	25.0	43.1	1570	18.1
T2	25.3	41.0	990	15.7
C 2	25.0	43.1	1570	18.1
T3	29.0	41.5	1090	12.5
C3	29.0	44.0	1320	15.0
T4	33.1	45.9	1180	12.8
C4	29.0	44.0	1320	15.0
T5	31.9	44.3	1290	12.4
C5	28.4	42.5	1230	14.1

Overall the changes in moisture content of the test sections are slightly lower when compared to the control sections. This observation can be ascribed to the functioning of a separation geotextile that was placed below the structural layers to prevent water from entering the clay subgrade and at the same time also restricting drying out of subgrade.

5.4.2 Dynamic deflection modulus (E_{vd})

Table 7 gives a summary of the formation deflection modulus and mean formation settlement values per test section during Light Drop Weight testing. The test section shows smaller deflection values as compared to their control sections. The higher stiffness values of between 44 MPa and 61 MPa were obtained at the test sections whereas their control sections yielded stiffness values of between 8 MPa and 17 MPa. This clearly indicates that larger permanent deformations and instability of the track in terms of the track geometry are to be expected at the control sections.

Table 7: Formation dynamic deflection modulus test results

Test/Control	Mean Deflection (mm)	E_{vd} (MPa)
Test 1	0.430	52.5
Control 1 and 2	2.696	8.4
Test 2	0.461	49.0
Test 3	0.394	57.3
Control 3 and 4	1.334	16.9
Test 4	0.465	49.0
Test 5	0.518	43.5
Control 5	1.607	14.0

The deflections obtained during Falling Weight testing were reduced to dynamic deflection modulus (E_{vd}) and are plotted graphically in Figure 4 for comparitive analysis of the formation modulus of the test and control sections.

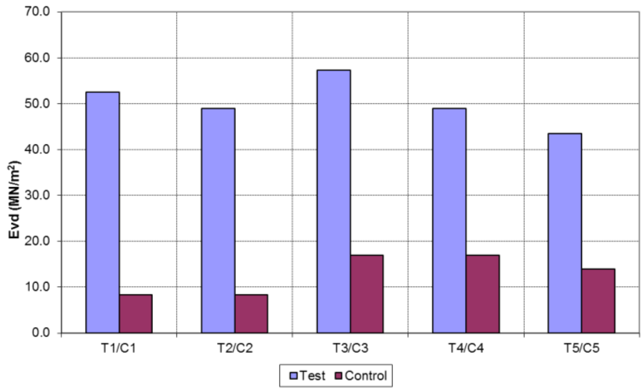

Figure 4: Dynamic Deflection Modulus

5.4.3 Formation and rail levels

Figure 5 shows the standard deviation of the formation settlement taken at the end of Cycles 1, 2 and 3. The standard deviation of the formation settlement ranged from 1.6 to 7.1, 1.1 to 8.3 and 3.3 to 14.9 during Cycles 1, 2 and 3 respectively. The standard deviation of the control section formation settlement ranged from 5.1 to 14.5, 2.7 to 12.0 and 14.8 to 20.4 during Cycles 1, 2 and 3 respectively. High standard deviation values were observed in Cycle 3, however the test sections still performed much better as compared to the control sections.

At the end of Cycle 3 all test section showed good resistance to heaving and shrinkage in absolute terms when compared to their control sections. In terms of the formation settlement the best reinforcement in decreasing order were T1 (Geocell), T5 (Dumprock), T4 (Geotextile) and T2 (Conventional).

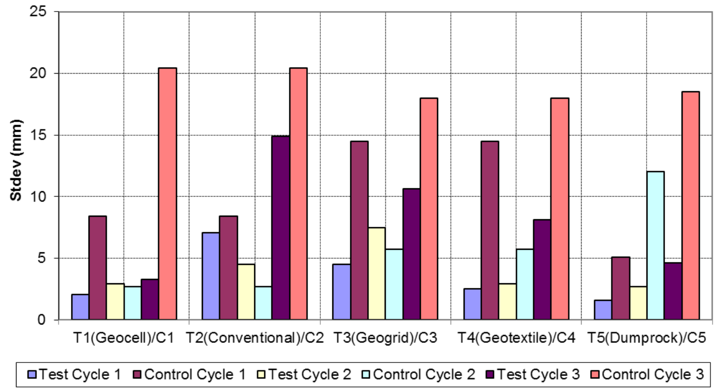

Figure 5: Standard deviation of settlement measurements per test section

The results show that the variance in settlement, or differential settlement for all test sections was less than that of their individual control sections. Lower standard deviations of settlement indicate a more stable formation and should result in a track of higher geometric standard.

Figure 6 shows the average rail settlement (permanent) of the test and control sections for the three cycles (as a result of tamping). The large rail settlements of the test sections during Cycles 1 and 2 are most probably due to the unconsolidated ballast beds of the new sections. In Cycle 3 large rail settlements of the control sections were observed. These can be ascribed to the (unstable) flowing of ballast during train loading as a result of low formation deflection stiffness and poor ballast quality of the control sections.

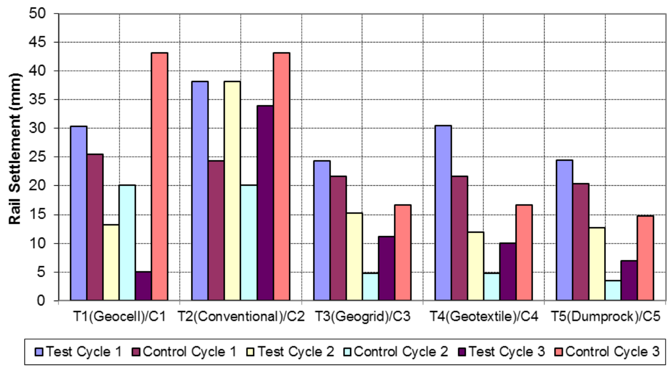

Figure 6: Average rail settlement measurements per test section

The high magnitude of the rail settlements are due to normal ballast settlement or punching of ballast into the subgrade material and should not be ascribed to the settlement of the clay subgrade layer alone.

5.4.4 EMV80 track geometry measurements

Table 8 gives a summary of the tamp TQI (Track Quality Index) values at the end of each cycle. The tamp TQI represents the summation of standard deviation taken over a track distance of 100 m and includes the following parameters: twist, super elevation, vertical profile and vertical alignment. This value is therefore an indication of the geometric condition of the track and the higher the value (normally greater than 9) the greater the need for ballast tamping (correction of the track geometry).

Table 8: A summary of tamp TQI at the end of each cycle

Test/Control	07/04/05 End of Cycle1	24/03/06 End of Cycle 2	17/04/07 End of Cycle 3
T1(Geocell)	8.0	8.0	7.1
Control 1	8.6	8.5	4.8
T2 (Conv.)	13.9	8.3	6.2
Control 2	8.6	8.5	4.8
T3 (Geogrid)	9.3	7.3	5.8
Control 3	10.4	9.0	9.3
T4(Geotextile)	11.9	11.7	9.9
Control 4	10.4	9.0	9.3
T5 (Dumprock)	9.8	8.0	7.0
Control 5	13.5	9.6	15.9

Tables 9 gives a summary of the profile geometry deterioration of the different test sections compared to their individual control sections during Cycles 1, 2 and 3.

Table 9: Track geometry deterioration (Cycles 1, 2 and 3)

Test	Cycle	Profile Deterioration (Std.Dev)(mm)		
		Test	Control	Ratio
Geocell	1	0.62	1.02	0.61
	2	0.93	1.47	0.64
	3	0.32	0.64	0.50
Conventional	1	2.50	1.02	2.45
	2	0.61	1.47	0.42
	3	0.48	0.64	0.76
Geogrid	1	1.09	1.31	0.83
	2	0.92	1.32	0.70
	3	–	0.87	–
Geotextile	1	0.96	1.31	0.74
	2	1.18	1.32	0.90
	3	0.40	0.87	0.46
Dumprock	1	0.84	2.73	0.31
	2	0.23	1.12	0.20
	3	0.32	0.64	0.49

Figure 7 graphically presents the profile deterioration as a ratio of the control section for the three cycles.

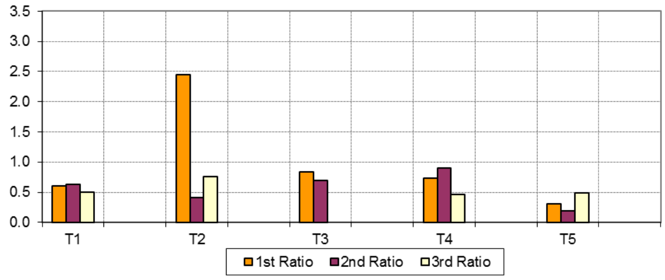

Figure 7: Geometric deterioration as a ratio of control section

The profile results in Cycle 3 show that all test sections had relatively low geometry deterioration ratios and T3 (Geogrid) performed exceptionally well with no signs of deterioration.

Tables 10 gives a summary of the twist geometry deterioration of the different test sections compared to their individual control sections during Cycles 1, 2 and 3.

Table 10: Track geometry deterioration (Cycles 1, 2 and 3)

Test	Cycle	Twist Deterioration (Std.Dev)(mm)		
		Test	Control	Ratio
Geocell	1	0.30	0.42	0.71
	2	0.47	0.31	1.54
	3	0.50	0.31	4.00
Conventional	1	1.27	0.42	3.10
	2	0.47	0.31	1.54
	3	0.31	0.31	2.50
Geogrid	1	0.61	0.64	0.95
	2	0.69	0.87	0.79
	3	0.67	0.52	0.13
Geotextile	1	0.50	0.64	0.78
	2	0.70	0.87	0.80
	3	0.56	0.52	1.08
Dumprock	1	0.92	0.71	1.30
	2	0.20	0.39	0.51
	3	0.78	1.90	0.41

Figure 8 shows a graphical representation of the twist deterioration as a ratio of the control section.

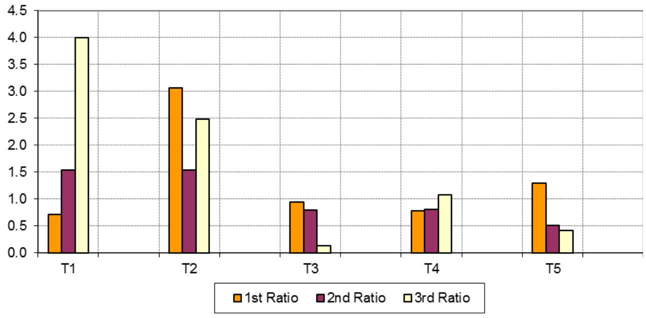

Figure 8: Geometrical deterioration as a ratio of the control section

The twist results show that T3 (Geogrid), T5 (Dumprock) and T4 (Geotextile) had relatively low deterioration ratios compared to T1 (Geocell) and T2 (Conventional).

Figure 8 indicates that the conventional section performed worse than all the other test sections in Cycle 3 in terms of the differential formation settlement and that the geocell, dumprock, geotextile and geogrid sections proved to possess specific benefits in terms of differential settlement and geometry. The standard deviation of the formation settlements from the last measurement indicates improved resistance to geometrical deterioration at the test sections compared to the control sections. The order of deterioration in terms of performance of the sections was not consistent in the last two cycles (see Figures 7 and 8), however the test sections have shown low geometry deterioration ratios. When deterioration is expressed as a ratio of the control section in terms of the profile and the twist, all test sections performed exceptionally well.

6 CONCLUSIONS

This paper described the laboratory tests, finite element analyses and test sections that were constructed and monitored to evaluate the ability of specific geosynthetic products to improve the bearing capacity, differential settlement and geometry of poor track formations over a period of 34 months.

The following conclusions can be drawn from the results described in this paper:

a) The results showed that the generic geocell, geogrid and geotextile products as well as the dumprock design possess superior strength over the conventional design with no geosynthetic reinforcement. Geosynthetics improved the bearing capacity (dynamic stiffness modulus) and resistance to differential settlement.
b) The function of a separation geosynthetic to reduce NMC changes within the formation also played a vital role in the performance of the various alternatives.
c) The introduction of the geosynthetic products has the ability to nearly double the ballast tamping cycle. Tamping cycles were eventually increased by a factor of 2, from 6 months to 12 months since the introduction of the test sections.
d) Cost effectiveness factors of 1.3, 1.5 and 1.7 were obtained for the geogrid, geocell and geotextile products respectively from FE analyses. The FEA performance and laboratory analysis results correlate well with the performance (over 34 months) of the geosynthetic products in the field. Overall the best three performing test sections over three cycles (34 months of monitoring period) were T1 (Geocell), T5 (Dumprock) and T3 (Geogrid). The geotextile product was not far from the rest of the best performing products.
e) The change in formation deflection modulus between the test and control sections may also have an influence on overall behaviour of the entire track structure. Replacement of the entire formation with any of the three best performing products should be adopted.
f) Formation design scenarios and options for newly built and formation rehabilitation of existing track formations, situated in similar geological, geotechnical and climatic conditions have been developed based on the findings from this research.

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