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The role of geosynthetics in improving the behaviour of ballasted rail tracks

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Abstract
This chapter contains a laboratory assessment of the role of geosynthetics in the performance of railway ballast. The main objectives of this testing were to examine the actual potential of selected geosynthetic products for ballast stabilisation. A series of cyclic drained tests were carried out using a large scale prismoidal triaxial apparatus that was designed and built at the University of Wollongong, and is the most innovative cyclic process simulation testing equipment available in the world today. The effects of different types of geosynthetics, including bi-axial geogrid, non-woven geotextile and geocomposite (a combination of bi-axial geogrid and non-woven geotextile) have been evaluated as single layer and dual layer arrangements. The test findings revealed that bi-axial geogrid with aperture size of $40 \times 40$ mm and with minimum bi-directional strength that complies with the University of Wollongong recommendations, would be a suitable grid reinforcement to be placed under the ballast layer for track stabilisation. This particular non-woven geotextile offered an optimum separation function between the ballast and capping layers and maintained a higher resiliency during cyclic loading. Among the three types of geosynthetics tested, the use of geocomposite resulted in the least strain and particle breakage. The dual layer configuration reduced deformation and degradation better than the single layer configuration.

Keywords
rail, tracks, ballasted, behaviour, role, improving, geosynthetics

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The Role of Geosynthetics in Improving the Behaviour of Ballasted Rail Tracks

Buddhima Indraratna and Sanjay Nimbalkar

SUMMARY
This chapter contains a laboratory assessment of the role of geosynthetics in the performance of railway ballast. The main objectives of this testing were to examine the actual potential of selected geosynthetic products for ballast stabilisation. A series of cyclic drained tests were carried out using a large scale prismoidal triaxial apparatus that was designed and built at the University of Wollongong, and is the most innovative cyclic process simulation testing equipment available in the world today. The effects of different types of geosynthetics, including bi-axial geogrid, non-woven geotextile and geocomposite (a combination of bi-axial geogrid and non-woven geotextile) have been evaluated as single layer and dual layer arrangements. The test findings revealed that bi-axial geogrid with aperture size of $40 \times 40$ mm and with minimum bi-directional strength that complies with the University of Wollongong recommendations, would be a suitable grid reinforcement to be placed under the ballast layer for track stabilisation. This particular non-woven geotextile offered an optimum separation function between the ballast and capping layers and maintained a higher resiliency during cyclic loading. Among the three types of geosynthetics tested, the use of geocomposite resulted in the least strain and particle breakage. The dual layer configuration reduced deformation and degradation better than the single layer configuration.
1. INTRODUCTION

Ballasted rail track is designed to provide an economical and safe transportation system for passenger and freight traffic. The track normally consists of superstructure (rails, railpads, fastenings, sleepers (ties), and substructure (ballast, sub-ballast (capping and structural-fill), and subgrade). Ballast is a granular material with a high bearing capacity that is placed above the sub-ballast or subgrade to act as a platform to support the track superstructure. During the passage of a train, large stresses induced from the superstructure are transmitted to the ballast. Consequently the ballast layer must be thick enough to hold the track in position and to provide protection to subgrade soils, while aggregates must be tough enough to resist abrasion and degradation, but high traffic induced stresses always result in large plastic deformation and degradation of the ballast. The recent introduction of faster and heavier trains in countries like Australia and India has also resulted in track deterioration, lateral instability, and increased maintenance costs. This problem becomes more severe under conditions of ballast fouling (Selig and Waters 1994, Indraratna et al. 2011).

During track operations, fine particles can accumulate within the ballast voids (ballast fouling) due to: (a) breakage of sharp angular corners (edges), (b) infiltration of fines from the surface (e.g. coal spillage from wagons), and (c) pumping of soft saturated subgrade soils under excessive cyclic loads. Fouling of ballast makes the granular mass effectively less angular, decreases its shear strength, and impairs track drainage. In the worst case scenarios, fouled ballast must be cleaned or replaced with fresh ballast in order to keep the track at its desired level and alignment. In order to compete with other modes of transportation, rail industries face challenges to minimise maintenance costs and find alternative materials to improve track performance. The application of geosynthetics in granular materials is well known (Koerner 1990). Various types of geosynthetic reinforcements placed in fresh ballast have usually improved the performance of rail transportation systems (Shin et al. 2002,
Indraratna and Salim 2003, Brown et al. 2007, Indraratna et al. 2007, 2012). The purpose of adding a geosynthetic layer in the fresh ballast is to compensate for the loss of bearing capacity, shear strength, and dynamic resiliency that occurred during previous cycles of degradation and fouling.

This chapter presents the results of cyclic tests conducted on fresh ballast stabilised with three types of geosynthetics. The deformation and degradation of fresh ballast with and without geosynthetics was assessed using a large scale prismatic triaxial chamber that was designed and built at University of Wollongong, under one distinct cyclic loading history. The results of these tests alone are not sufficient to generalise the behaviour of these geosynthetics under alternative cyclic loading conditions or ballast properties. In the field, the lateral displacement of ballast, particularly parallel to the sleeper, is not restricted, and hence the prismatic triaxial chamber with unrestrained sides provides an ideal facility for physical modelling of the deformation of ballast under cyclic loading. The vertical and lateral deformations and the degradation aspects of fresh ballast stabilised with different types of geosynthetics were compared with those of unreinforced ballast. The relative benefits of dual layer reinforcement were also assessed.

2. MATERIALS AND SPECIFICATION

2.1 Ballast, Capping and Subgrade characteristics

The fresh ballast used in the present investigation is Latite Basalt, a common ballast aggregate obtained from a designated quarry in Bombo (near Wollongong city), Australia. The particles represent sharp angular coarse aggregates of crushed volcanic basalt (latite), and their physical properties were evaluated using the standard test procedures as per AS2758.7 (1996) [Indraratna et al. 1998]. This basalt is a fine-grained, dense-looking black aggregate, with the essential minerals being plagioclase (feldspar) and augite (pyroxenes).
The durability, shape, and strength of the fresh ballast used in this laboratory study are summarised in Table 1.

Table 1. Characteristics of fresh ballast (data sourced from Indraratna et al. 1998)

<table>
<thead>
<tr>
<th>Test Parameters</th>
<th>Values</th>
<th>Recommendations by Australian Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Durability</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Aggregate crushing value</td>
<td>12%</td>
<td>&lt; 25%</td>
</tr>
<tr>
<td>• Los Angeles Abrasion</td>
<td>15%</td>
<td>&lt; 25%</td>
</tr>
<tr>
<td>• Wet attrition value</td>
<td>8%</td>
<td>&lt; 6%</td>
</tr>
<tr>
<td>Strength</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Point load index</td>
<td>5.39 MPa</td>
<td>-</td>
</tr>
<tr>
<td>Shape</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• Flakiness</td>
<td>25%</td>
<td>&lt; 30%</td>
</tr>
<tr>
<td>• Misshapen particles</td>
<td>20%</td>
<td>&lt; 30%</td>
</tr>
</tbody>
</table>

The particle size distribution of fresh ballast, capping, and subgrade materials is shown in Figure 1. The selected particle size distribution used in the laboratory testing is typical of ballast gradations used in Australia [AS 2758.7 (1996), TS 3402 (2001)].

Figure 1. Particle size distribution of fresh ballast, capping and subgrade materials
The particle size distribution for capping was selected in accordance with industry specification [TS3422 (2001)]. A thin layer of compacted clayey sand was used in the laboratory model to simulate the subgrade of a real track. A capping layer comprising sand-gravel mixture was used between the ballast and the subgrade layers. Table 2 shows the particle size characteristics of fresh ballast, capping and the subgrade materials used in the cyclic triaxial tests.

Table 2. Grain size characteristics of ballast, capping and subgrade materials

<table>
<thead>
<tr>
<th>Material</th>
<th>Particle shape</th>
<th>(d_{\text{max}}) (mm)</th>
<th>(d_{\text{min}}) (mm)</th>
<th>(d_{10}) (mm)</th>
<th>(d_{30}) (mm)</th>
<th>(d_{50}) (mm)</th>
<th>(d_{60}) (mm)</th>
<th>(C_u)</th>
<th>(C_c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fresh Ballast</td>
<td>Highly angular</td>
<td>63.0</td>
<td>19.0</td>
<td>24.0</td>
<td>30.0</td>
<td>35.0</td>
<td>38.0</td>
<td>1.6</td>
<td>1.0</td>
</tr>
<tr>
<td>Capping (Sand-gravel)</td>
<td>Angular to rounded</td>
<td>19.0</td>
<td>0.075</td>
<td>0.8</td>
<td>2.2</td>
<td>3.7</td>
<td>6.2</td>
<td>7.8</td>
<td>1.0</td>
</tr>
<tr>
<td>Subgrade (clayey sand)</td>
<td>-</td>
<td>4.75</td>
<td>-</td>
<td>0.06</td>
<td>0.15</td>
<td>0.18</td>
<td>0.22</td>
<td>3.67</td>
<td>1.7</td>
</tr>
</tbody>
</table>

### 2.2 Geosynthetic characteristics

Three types of geosynthetics were used to stabilise the fresh ballast in the laboratory test apparatus. These included: (a) polyester bi-axial geogrid, (b) polypropylene staple fibre non-woven geotextile, and (c) geocomposite, which is a combination of bi-axial geogrid and non-woven geotextile. The physical and mechanical characteristics of these geosynthetics are described below.

#### 2.2.1 Bi-axial geogrid

A knitted polyester (PET) bi-axial geogrid was selected for the current study (Figure 2). It was manufactured from select grades of high tenacity, high molecular weight, and low carboxyl end group polyester yarn. The yarns are formed into a grid structure with uniform apertures and are then coated with a specially formulated PVC plastisol to enhance dimensional stability, resistance to mechanical damage, and durability. These geogrids have high tensile strength, low creep and excellent durability, and are generally suitable for the
reinforcement of soils and other granular materials when strength in both directions is important, including: the reinforcement of unbound aggregate courses of paved and unpaved roads, area stabilisation/reinforcement and track stabilisation. Having relatively large apertures (40 mm), geogrids provide a strong mechanical interlock with coarse ballast grains ($d_{50} = 35$ mm). The properties of the tested geogrid are given in Table 3.

![Figure 2. Typical knitted and PVC coated bi-axial polyester geogrid](image)

Table 3. Properties of bi-axial geogrid [data sourced from Techfab (India) Industries Ltd.]

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Test Method</th>
<th>Machine Direction</th>
<th>Cross machine Direction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mechanical</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ultimate tensile strength</td>
<td>kN/m</td>
<td>ASTM D 6637</td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>Tensile strength at 2% strain</td>
<td>kN/m</td>
<td>ASTM D 6637</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>Tensile strength at 5% strain</td>
<td>kN/m</td>
<td>ASTM D 6637</td>
<td>18</td>
<td>18</td>
</tr>
<tr>
<td>Elongation at break</td>
<td>%</td>
<td></td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>Physical</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Aperture size</td>
<td>mm</td>
<td></td>
<td>40</td>
<td>40</td>
</tr>
<tr>
<td>Mass per unit area</td>
<td>g/m²</td>
<td>ASTM D-5261</td>
<td>390</td>
<td></td>
</tr>
</tbody>
</table>

Notes: $^1$Minimum average roll value; $^2$Typical value
2.2.2 Non-woven geotextile

A typical polypropylene non-woven geotextile was also used, as shown in Figure 3. It was manufactured from high quality polypropylene staple fibres that were mechanically bonded through needle-punching to form a strong, flexible, and dimensionally stable fabric structure, with optimum pore sizes and high permeability. The geotextile is resistant to chemicals and biological organisms normally found in soils and has been stabilised against degradation via a short term exposure to ultraviolet radiation. The properties of non-woven geotextile are summarised in Table 4.

![Figure 3. Typical polypropylene non-woven geotextile](image)

2.2.3 Geocomposite (bi-axial geogrid + non-woven geotextile)

A bi-axial geogrid was placed over a non-woven polypropylene staple fibre geotextile, and this combination of materials was installed at the ballast-capping interface to serve as a geocomposite layer. Earlier studies indicated that a geocomposite layer (geogrid bonded to woven geotextile) stabilised recycled ballast much better than standard geogrids (Indraratna and Salim 2003, Indraratna et al. 2011). In a geocomposite, the geogrid provides a strong
mechanical interlock with the angular ballast particles and produces reinforcement, whereas the geotextile provides both filtration and separation functions, and allows partial in-plane drainage. The geotextile also prevents the fines moving up from the capping and subgrade layers, thus, keeping the ballast layer relatively clean.

Table 4. Properties of non-woven geotextile [data sourced from Techfab (India) Industries Ltd.]

<table>
<thead>
<tr>
<th>Property</th>
<th>Unit</th>
<th>Test Method</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mechanical Grab Tensile strength</td>
<td>N</td>
<td>ASTM D-4632</td>
<td>1570</td>
</tr>
<tr>
<td>Mechanical Elongation at break</td>
<td>%</td>
<td>ASTM D-4632</td>
<td>60</td>
</tr>
<tr>
<td>Mechanical Trapezoidal tear</td>
<td>N</td>
<td>ASTM D-4533</td>
<td>600</td>
</tr>
<tr>
<td>Mechanical Puncture strength</td>
<td>N</td>
<td>ASTM D-4833</td>
<td>910</td>
</tr>
<tr>
<td>Hydraulic Mullen Burst</td>
<td>kPa</td>
<td>ASTM D-3786</td>
<td>4700</td>
</tr>
<tr>
<td>Hydraulic Permeability / Flow rate</td>
<td>litres/m²/sec</td>
<td>ASTM D-4491</td>
<td>35</td>
</tr>
<tr>
<td>Hydraulic Apparent Opening Size (AOS)</td>
<td>μm</td>
<td>ASTM D-4751</td>
<td>90</td>
</tr>
<tr>
<td>Physical Mass per unit area</td>
<td>g/m²</td>
<td>ASTM D-5261</td>
<td>500</td>
</tr>
<tr>
<td>Physical Thickness</td>
<td>mm</td>
<td>ASTM D-5199</td>
<td>2.9</td>
</tr>
<tr>
<td>Physical Endurance</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Physical UV Resistance</td>
<td>% @ 500 hrs</td>
<td>ASTM D-4355</td>
<td>70</td>
</tr>
</tbody>
</table>

Note: Above values are average values with a -10% tolerances

3. PREPARATION OF TEST SPECIMENS

The large scale prismoidal triaxial chamber used in this study can accommodate specimens 800 mm long, 600 mm wide, and 600 mm high (Figure 4). In order to model real railway track, the prismoidal test chamber was filled in four layers, as shown in Figure 5. This is a true triaxial apparatus where three independent principal stresses can be applied in three mutually orthogonal directions. Since each wall of the test chamber can move independently in the lateral directions, the ballast specimen is free to deform laterally under cyclic vertical stress and relatively smaller lateral stresses. The lateral confinement offered by the shoulder and crib ballast in an actual track is not sufficient to restrain any lateral movement of the
ballast, and hence this prismoidal test chamber with unrestrained sides is an ideal facility for physical modelling of ballast under cyclic loading. Although the actual stress states may not be simulated exactly, especially in the regions of lateral boundaries, this particular design of the chamber reasonably simulates realistic track boundary conditions, and is the most innovative cyclic process simulation testing equipment available in the world today. It was designed and built at the University of Wollongong (Indraratna and Salim 2003).

Figure 4. The large scale triaxial chamber at the University of Wollongong

Figure 5. Schematic illustration of geosynthetics layout: (i) single layer, and (ii) dual layer
The bottom layer consisted of compacted clayey sand of 50 mm in thickness to simulate the layer of subgrade soil in the track (Figure 6). A 100 mm thick layer of compacted gravel and sand was used to represent the capping (sub-ballast) layer (Figure 7). The upper two layers viz. load bearing ballast (300 mm thick) and crib ballast (150 mm thick) consisted of fresh ballast. A timber sleeper and rail segment was placed above the compacted load bearing ballast layer. The space between the sleeper and walls was filled with crib ballast (Figure 8). The ballast layer was compacted in 75 mm thick layers and the capping layer was compacted in 50 mm thick layers to represent the field densities. Compaction was carried out using vibratory hammer. A rubber pad was placed underneath the vibratory hammer in order to prevent particle breakage during placement. The bulk unit weights ($\gamma_{\text{bulk}}$) of the compacted ballast layer and capping layer were 15.3 kN/m$^3$ and 23.8 kN/m$^3$ respectively. The initial void ratio ($e_0$) of the ballast and capping layer were 0.74 and 0.52, respectively.

Figure 6. Compacted bottom subgrade layer (clayey sand)
Figure 7. Compacted capping layer (mixture of gravel and sand) on the top of subgrade

Figure 8. Crib ballast, rail and sleeper assembly
4. LAYOUT OF GEOSYNTHETICS

4.1 Single layer arrangement

The relative benefits of different geosynthetics, i.e. bi-axial geogrid, non-woven geotextile, and geocomposite, when placed as a single layer at the ballast-capping interface, were evaluated.

4.2 Dual layer arrangement

Apart from the bi-axial geogrid and the non-woven geotextile placed at the ballast-capping interface, a layer of geocomposite was placed at the capping-subgrade interface to evaluate the relative advantages of a dual layer arrangement.

5. INSTRUMENTATION

To accurately measure transient stresses and strains induced in the model track layers along the vertical and lateral directions, high precision equipment were used during testing. In order to make sure that these instruments were in full contact with the surrounding layer of ballast and capping, a vibrating plate was used for compaction. Details of these equipment are given below.

5.1 Pressure cells

The vertical stresses induced in the ballast layer were measured by pressure cells. The pressure cells were rapid-response hydraulic earth pressure cells with grooved, thick, active faces. Several factors, such as the aspect ratio and size of the cell, placement effects, corrosion, and temperature can influence the accuracy of the measurements (Weiler and Kulhawy 1982, Dunnicliff 1988 among others). Therefore, relatively thin but robust pressure cells made from steel (230 mm diameter by 12 mm thick) were adopted. The pressure cells were placed at the sleeper-ballast and ballast-capping interfaces with due care taken to avoid any damage during placement and subsequent ballast compaction.
5.2 Settlement pegs

Vertical deformations of the track were measured by settlement pegs. Track deformation is considered to be a primary indicator for predicting the stability and longevity of track. Excessive deformations cause accelerated movements and breakage of ballast particles. The settlement pegs consisted of 100 mm × 100 mm × 6 mm steel base plates attached to 10 mm diameter steel rods. A typical arrangement of the settlement pegs is shown in Figure 9.

![Settlement pegs on top of geogrid layer](image)

Figure 9. Placement of settlement pegs on top of geogrid layer

5.3 Displacement transducers

Lateral deformations were measured by the linear variable differential transducers (LVDT) (also called differential transformer). A typical arrangement of LVDTs is shown in Figure 10. About 4 LVDTs were connected near corners of each movable vertical wall to measure the lateral deformations. This arrangement also enabled any tilting in the wall resulting from differential movements of ballast particles to be measured. Lateral deformation was
determined from the mean of the measurements thus obtained. Data loggers were connected to the LVDTs to obtain a continuous record of permanent lateral deformations.

![Image](image1.jpg)

**Figure 10.** Typical arrangement of displacement transducers to measure lateral deformations

6. **TEST PROCEDURE**

The cyclic vertical stress (σ′ \(_{cyc}\)) was provided by a servo-hydraulic dynamic actuator and transmitted to the ballast through a 100 mm diameter steel ram and a rail-sleeper assembly. In rail track environments, low confining pressure is of major concern (Lackenby et al. 2007). Under normal rail track environments, there is significant lateral movement in the ballast layer due to reduced lateral restraint at the edge of the sleeper (Indraratna et al. 2010). Small lateral pressures (intermediate principal stress, \(σ′_{h2} = 10\) kPa and minor principal stress \(σ′_{h3} = 7\) kPa) were applied to the triaxial prismoidal specimens through hydraulic jacks to simulate field confinement. Confinement in a real track is generally developed by the weight of the crib and shoulder ballast, along with frictional interlock between angular ballast particles. Full scale field trials on instrumented track sections near Bulli in New South Wales (NSW,
Australia) indicated that the lateral confining pressure rarely exceeds 60 kPa (Indraratna et al. 2010, 2012). A confining pressure range of about 10-70 kPa was found most appropriate for European rail tracks (Suiker et al., 2005). The minimum cyclic stress ($\sigma'_{\text{min,cyc}}$) was kept at 45 kPa which represents the unloaded state of the track but it includes the weight of the sleepers and rails (Lackenby et al. 2007). An initial static load was applied at a rate of 1 mm/s to a stress equal to the average of the minimum and maximum cyclic deviator stress was reached. Afterwards, a stress-controlled test with a harmonic sinusoidal cyclic stress amplitude of ($\Delta\sigma'_\text{cyc} = \sigma'_{\text{max,cyc}} - \sigma'_{\text{min,cyc}}$) was carried out. A reduced frequency conditioning phase (5 Hz) was employed at the commencement of cyclic loading (during rapid vertical deformation) to prevent impact loading and loss of actuator contact with the top surface of the rail-sleeper assembly. After this stage the initial readings of the load cells, pressure cells, LVDTs, and settlement pegs were taken. A cyclic load corresponding to a 25 tonne axle load calculated in accordance with AREA method was applied to produce the same average contact stress at the sleeper-ballast interface in real tracks. The tests were conducted at a frequency of 15 Hz, simulating 109 km/hour with a wheel diameter of 0.97 m and assumed distance between wheels of common rolling stock bogies as 2.02 m. The maximum cyclic stress ($\sigma'_{\text{max,cyc}}$) at the sleeper-ballast interface obtained by the AREA method (Jeffs and Tew 1991) was 447 kPa compared to 335 kPa using the European method (Esveld 2001). The total number of load cycles applied in each test was $2 \times 10^5$. The cyclic loading was halted at a selected number of load cycles, and the readings of settlement, lateral movement of walls, loading magnitudes and stresses were recorded. 6 tests were conducted to investigate the response of cyclic loading on ballast with and without geosynthetics. Initially, the test on fresh ballast without geosynthetics was carried out. A single reinforcement configuration was adopted in 3 tests, while the effect of a double reinforced model track was studied in the remaining 2 tests.
In order to ensure repeatability, the same experimental procedure was maintained for all tests and the same amount of ballast was also used.

7. TEST RESULTS

7.1 Deformation characteristics

The deformations of fresh ballast with and without geosynthetics are presented in Figure 11. The vertical deformations (and strains) of ballast were computed by excluding the deformation of the capping and subgrade layers. In this respect the limited thickness of subgrade layer was expected to have an insignificant influence on the test results, especially when the response of different ballast specimens with and without geosynthetics was compared. As expected, there was a rapid deformation of ballast at the onset of the loading cycles. The rate of ballast deformation diminished to a controlled steady state, after a certain number of load cycles defined as the ‘stable zone’ (Figure 11). The granular materials displayed a strong tendency to compact under cyclic loading which is in agreement with the findings from previous studies (Lackenby et al. 2007, Indraratna et al. 2010). Compared with the unreinforced ballast, the reinforced ballast exhibited a lower vertical deformation. The knitted polyester bi-axial geogrid appeared to be more effective than the polypropylene staple fibre non-woven geotextile. This may be attributed to the fact that highly frictional, angular particles of fresh ballast develop strong mechanical interlock with the geogrid layer, whereas the performance of geotextile depended largely on the tension membrane effect. As expected, the fresh ballast stabilised with the geocomposite exhibited the least vertical deformation. This is because, a non-woven geotextile offers an optimum separation function between the ballast and capping layers maintaining a higher resiliency, whereas a bi-axial geogrid provides a strong interlock (Indraratna et al. 2010). Dual layer reinforcement, i.e. geogrid at the ballast-capping interface and geocomposite at the capping-subgrade interface, reduced
deformation better than the single layer reinforcement. Rail track deformation is related to the number of load cycles by a semi-log relationship (Raymond et al. 1976, Jeff and Marich 1987, Indraratna et al. 2011). Figure 12 shows the deformation of fresh ballast with and without geosynthetics, plotted in a semi-logarithmic scale. Ballast deformation under cyclic loading is represented by a semi-logarithmic relationship (Indraratna et al. 2011):

\[ S_v = a + b \ln N \]  

where \( S_v \) is the vertical deformation of ballast, \( N \) is the number of load cycles, and \( a \) and \( b \) are two empirical constants. As evident from Figure 12, the vertical deformation of ballast is characterised by three phases. The first phase is immediate deformation under the first loading cycle. The second phase is an unstable zone where rapid deformation occurs, and the reorientation and rearrangement of ballast aggregates along with significant breakage results in a denser (compressive) packing assembly. In the third phase the rate at which deformation increases is marginal, with an almost linear relationship between deformation and the number of load cycles. This third phase is often characterised as ‘stable shakedown’. Thus, the ballast deformation \( (S_v) \) can be modelled in terms of the number of load cycles \( (N) \) (Indraratna and Nimbalkar 2013) as:

\[ S_v = S_{v1} \left( 1 + c \ln N + 0.5d \ln N^2 \right) \]  

where \( S_{v1} \) is the vertical deformation of ballast after the first load cycle, and \( c \) and \( d \) are two empirical constants. The first term of Equation (2) refers to deformation due to the first cycle \( (N = 1 \text{ cycle}) \), the second to an unstable zone \( (N < 10^4 \text{ cycles}) \), and the third term to a stable shakedown zone \( (N > 10^4 \text{ cycles}) \). Figures 13 and 14 show the lateral deformations of ballast measured using LVDTs. The negative sign indicates that ballast always deformed outwards. The lateral deformations of ballast parallel to the sleeper were significantly higher than ballast parallel to the rail. This was due to reduced lateral restraint \( (\sigma_{h1} < \sigma_{h2}) \).
Figure 11. Vertical deformations of ballast with and without geosynthetics

Figure 12. Vertical deformations of ballast with and without geosynthetics plotted in semi-logarithmic scale
Figure 13. Lateral deformations (parallel to rail) of ballast with and without geosynthetics plotted in semi-logarithmic scale

Figure 14. Lateral deformations (parallel to sleeper) of ballast with and without geosynthetics plotted in semi-logarithmic scale
7.2 Strain characteristics

With the application of cyclic loading, ballast undergoes compression in a vertical direction (major principal strain, $\varepsilon_v$), and expands in the two lateral directions (intermediate principal strain, $\varepsilon_{h2}$, and minor principal strain, $\varepsilon_{h3}$). Figure 15 shows the variation in the major principal strain ($\varepsilon_v$) with an increasing number of load cycles for fresh ballast with and without geosynthetics. All specimens showed almost similar trends in variation of $\varepsilon_v$. The geocomposite appeared to be the most effective, for the same reasons explained earlier. The polyester bi-axial geogrid alone decreased the vertical strain of fresh ballast moderately and the polypropylene staple fibre non-woven geotextile stabilised the fresh ballast to a lesser extent.

The vertical strain of ballast increased linearly with the logarithm of load cycles, and may be expressed by a function similar to Equation 1 (Indraratna et al. 2011):

$$\varepsilon_v = e + f (\ln N)$$  \hspace{1cm} (3)

where $e$ and $f$ are two empirical constants. The vertical strain of ballast can also be expressed by a function similar to Equation 2:

$$\varepsilon_v = \varepsilon_{v1} \left(1 + g \ln N + 0.5h \ln N^2 \right)$$  \hspace{1cm} (4)

where $\varepsilon_{v1}$ is the major (vertical) principal strain after the first load cycle, and $g$ and $h$ are two empirical constants. The lateral strains of ballast (intermediate principal strain $\varepsilon_{h2}$, and minor principal strain $\varepsilon_{h3}$) were obtained from the measurements of lateral deformation of the vertical walls and the initial lateral dimensions of the test specimens.
The lateral strain perpendicular to the sleeper (i.e. parallel to the rails) is the intermediate principal strain ($\varepsilon_{h2}$), which corresponds to the intermediate principal stress ($\sigma'_{h2}$). The strain parallel to the sleeper is the minor principal strain ($\varepsilon_{h3}$) and it corresponds to the minor principal stress ($\sigma'_{h3}$). The variations of lateral strains ($\varepsilon_{h2}$, $\varepsilon_{h3}$) of fresh ballast with and without geosynthetics are shown in Figures 16 and 17. It is important to note that these lateral strains are based on the rigid body movements of the walls of the prismoidal triaxial chamber, and therefore they only represent the average strains across the depth of the sample and not at the interface (in particular, the ballast-capping interface in a single layer arrangement and the capping-subgrade interface in a dual layer arrangement where the geosynthetics are placed), where the lateral strains were expected to be least, or even negligible.
Figure 16. Intermediate (lateral) principal strain of fresh ballast layer with and without geosynthetics

Figure 17. Minor (lateral) principal strain of fresh ballast layer with and without geosynthetics
The knitted polyester biaxial geogrid decreased the lateral strains of ballast by an appreciable amount, thus proving to be more effective than the polypropylene staple fibre non-woven geotextile. Geogrid helps to confine (lateral stability) the ballast layer, thus improving its vertical stress distribution characteristics. Confinement is achieved as the geogrid restrains the lateral strains when placed near the ballast. Part of the ballast layer located in the immediate vicinity of the geogrid is locked into the apertures of the geogrid during placement and compaction of the particles. The reinforcement action (strength) of the geogrid is generated by the application of vertical stress and is responsible for reducing lateral strains of the ballast.

7.3 Shear strain and volumetric strain

The shear strain ($\varepsilon_s$) and volumetric strain ($\varepsilon_{vol}$) of the ballast is determined by (Timoshenko and Goodier 1970):

$$
\varepsilon_s = \frac{\sqrt{2}}{3} \left[ \sqrt{(\varepsilon_v - \varepsilon_{h2})^2 + (\varepsilon_{h2} - \varepsilon_{h3})^2 + (\varepsilon_{h3} - \varepsilon_v)^2} \right]
$$

(5)

$$
\varepsilon_{vol} = \varepsilon_v + \varepsilon_{h2} + \varepsilon_{h3}
$$

(6)

Figures 18 and 19 show the variations of $\varepsilon_s$ and $\varepsilon_{vol}$ against the number of load cycles (N), respectively. These results show that ballast always undergoes compression under cyclic loading (represented by a positive $\varepsilon_{vol}$). Under the application of monotonic loading, the initial compression is usually followed by dilation (i.e. $\varepsilon_{vol}$ becomes negative) at increasing shear strains (Indraratna et al. 1998, Indraratna et al. 2013). In general, both the shear strain and volumetric strain accumulated steadily with an increasing number of cycles, but their rates of increase were reduced with progressive accumulations of strain. Less permanent strains ($\varepsilon_s$, $\varepsilon_{vol}$) were induced in the ballast layer reinforced with single and double layers of geosynthetics.
Figure 18. Shear strains of fresh ballast layer with and without geosynthetics

Figure 19. Volumetric strains of fresh ballast layer with and without geosynthetics
7.4 Ballast breakage

The breakage of ballast particles due to repeated (cyclic) wheel loading can occur due to: (a) particle splitting, (b) breakage of angular projections, and (c) grinding of small-scale asperities (Raymond and Diyaljee 1979). In Australia, most breakage of latite ballast is primarily attributed to the quarried aggregates having highly angular corners (Lackenby et al. 2007, Nimbalkar et al. 2012). This breakage contributes to differential track settlement and increases the vertical and lateral deformation. In order to analyse the degradation of fresh ballast under cyclic loading, an assessment of ballast breakage was performed. After each test was completed the crib ballast and load bearing ballast aggregates were removed from the triaxial chamber separately and then sieved to determine the changes in particle gradation. This breakage was quantified using the Ballast Breakage Index (BBI) parameter proposed by Indraratna et al. (2005). By utilising a linear hypothetical size axis as reference, the BBI was calculated using Equation (7). The BBI values obtained from all the tests are presented in Table 5.

\[
BBI = \frac{A}{A + B}
\]  

(7)

Table 5. Assessment of ballast breakage during cyclic loading

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Material Type</th>
<th>Ballast breakage index (BBI)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Fresh ballast (FB)</td>
<td>0.163</td>
</tr>
<tr>
<td>2</td>
<td>Ballast + bi-axial geogrid (FB + GG)</td>
<td>0.109</td>
</tr>
<tr>
<td>3</td>
<td>Ballast + non-woven geotextile (FB + GT)</td>
<td>0.126</td>
</tr>
<tr>
<td>4</td>
<td>Ballast + geocomposite (FB + GC)</td>
<td>0.083</td>
</tr>
<tr>
<td>5</td>
<td>Ballast + geogrid + geocomposite (FB + GG + GC)</td>
<td>0.071</td>
</tr>
<tr>
<td>6</td>
<td>Ballast + geotextile + geocomposite (FB + GT + GC)</td>
<td>0.094</td>
</tr>
</tbody>
</table>

The fresh ballast stabilised with the polypropylene staple fibre non-woven geotextile reinforcement exhibited marginally more degradation in this range of particle sizes compared
to ballast stabilised with the knitted polyester biaxial geogrid. This is in agreement with the less displacement observed for the geogrid. In a single layer configuration the use of geocomposite (combination of geogrid and geotextile) resulted in the least ballast breakage. As expected, the dual layer reinforcements were most effective at reducing particle degradation than single layer reinforcement.

8. CONCLUSIONS AND RECOMMENDATIONS
A series of large-scale triaxial tests were conducted on fresh ballast with and without geosynthetics to assess its deformation and degradation under cyclic loading. The single layer and dual layer configuration was selected in order to study their relative benefits. Polypropylene nonwoven geotextile, polyester bi-axial geogrid, and geocomposites were placed at the ballast-capping interface while in dual layer configuration an additional layer of geocomposite was placed at the capping-subgrade interface. The geogrid and non-woven geotextile demonstrated sufficient capacity to reduce deformations of the ballast (vertical and lateral) under applied cyclic loads, and also reduced any grain breakage. The geogrid was more effective than the non-woven geotextile due to sound mechanical interlock with ballast particles. It is concluded that in a single layer configuration the biaxial geogrid would be a suitable grid reinforcement to be placed below the ballast layer for track stabilisation. The nonwoven geotextile offers an optimum separation function between the ballast and capping layers and maintains a higher resiliency during cyclic loading.

A very large aperture geogrid may not be effective as a separator when used above the capping layer, unless placed in conjunction with a bonded geotextile. The non-woven geotextile is an excellent material that provides separation of different gradations as well preventing soft subgrade material from being pumped into the ballast layer. The tested geocomposite (Polyester geogrid + polypropylene needle-punched nonwoven geotextile)
appeared to offer a number of favourable qualities that enhance the performance of railway ballast. On the basis of this study, geocomposite was shown to be very effective at controlling both strain and particle breakage. It was also demonstrated that the dual layer reinforcements, i.e. geogrid at the ballast-capping interface, and geocomposite at the capping-subgrade interface, are better at reducing vertical and lateral deformations as well as particle degradation, than single layer reinforcement.

9. LIST OF NOTATIONS

- **a, b, c, d**: Empirical constants relating $S_v$ and the logarithm of $N$
- **BBI**: Ballast breakage index
- **$C_c$**: Coefficient of curvature
- **CMD**: Cross machine direction (across the width of the roll)
- **$C_u$**: Coefficient of uniformity
- **$d_{10}, d_{30}, d_{50}, d_{60}$**: Particle sizes at percent finer of 10, 30, 50 and 60% respectively (mm)
- **$d_{\text{max}}, d_{\text{min}}$**: Maximum and minimum particle size respectively (mm)
- **$e, f, g, h$**: Empirical constants relating $e_v$ and the logarithm of $N$
- **$e_0$**: Initial void ratio
- **FB**: Fresh ballast
- **GC**: Geocomposite (bi-axial geogrid + nonwoven geotextile)
- **GG**: Knitted polyester bi-axial geogrid
- **GT**: Polypropylene nonwoven geotextile
- **MD**: Machine direction (longitudinal to the roll)
- **$N$**: Number of load cycles
- **PSD**: Particle size distribution
- **$S_h2$**: Lateral displacement of ballast parallel to rail (mm)
- **$S_h3$**: Lateral displacement of ballast parallel to sleeper (mm)
- **$S_v$**: Vertical deformation of ballast (mm)
- **$\sigma'_{\text{vcyc}}$**: Cyclic vertical (principal) stress (kPa)
\( \sigma_{h2} \quad \text{Intermediate principal stress (kPa)} \)

\( \sigma_{h3} \quad \text{Minor principal stress (kPa)} \)

\( \gamma_{\text{bulk}} \quad \text{Bulk unit weight (kN/m}^3) \)

\( \varepsilon_{h2} \quad \text{Intermediate (lateral) principal strain acting parallel to rail (\%)} \)

\( \varepsilon_{h3} \quad \text{Minor (lateral) principal strain acting parallel to sleeper (\%)} \)

\( \varepsilon_s \quad \text{Shear strain of ballast (\%)} \)

\( \varepsilon_v \quad \text{Major (vertical) principal strain of ballast (\%)} \)

\( \varepsilon_{\text{vol}} \quad \text{Volumetric strain of ballast (\%)} \)

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11. REFERENCES


