Ground Improvement for Transport Infrastructure: Geosynthetic Inclusions for Enhanced Ballasted Rail Tracks

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Abstract

Given the increasing demand for trains to carry heavier loads, current Australian ballasted rail networks require a significant amount of upgrading. Railroad ballast is an unbounded granular material that displaces laterally when subjected to repeated train loading. During track operations, ballast deteriorates due to progressive breakage and the infiltration of fine particles or mud-pumping from the underneath layers (e.g., capping, subgrade), which decreases the shear strength, impedes track drainage and increases the deformation of ballasted tracks. Rail track substructures can be reinforced by geosynthetics to reduce lateral displacements and optimise overall track performance. This paper presents the current state-of-the-art knowledge of rail track geomechanics based on research conducted at the University of Wollongong, including essential topics related to laboratory tests, computational modelling and field investigations undertaken to examine the improved performance of ballast by the use of geosynthetics. Full-scale monitoring of instrumented tracks supported by RailCorp and Australian Rail Track Corporation (ARTC) has been carried out to obtain data (i.e. measure the in-situ stresses and deformation of ballast embankments) that will reliably verify track performance as well as calibrate and validate introduced numerical simulations. This paper focuses on primary research and development of new design and construction concepts to enhance track performance using geosynthetics, whilst highlighting examples of innovations from theory to practice. These results provide promising approaches that can be incorporated into existing track design routines to cater for future high speed trains and heavier hauls.

Keywords: Ballast, Geogrid, Rail Track Infrastructure, Discrete Element Modelling.

1. INTRODUCTION

Australia relies heavily on rail to transport bulk freight and passengers; hence, the investments made in rail transport infrastructure, particularly high speed rail are essential. However, high speed rail demands safe and economic track design to withstand the large cyclic and impact loadings, whilst also protecting the subgrade soils from progressive shear failure and excessive plastic deformation (Selig and Waters 1994, Indraratna *et al.* 2013, Lim *et al.* 2005, Powrie *et al.* 2007). Conventional design methods are commonly based on the assumption of a homogeneous half-space for all the layers of track and do not

consider that individual layers have different properties (e.g., Indraratna *et al.* 2011a, Suiker and Borst 2003, Tutumluer *et al.* 2008, among others). Upon repeated train loads, ballast aggregates become deteriorated due to the breakage of angular corners and sharp edges and pumping of clayey subgrade, all of which foul the ballast, cause it to become less angular, apart from reducing its shear strength (Selig and Waters 1994, LePen 2008, Indraratna *et al.* 2011b). In addition, impact forces induced by wheel and rail irregularities or imperfections (e.g., wheel flats, dipped rails, rail corrugation, defective rail welds, insulation joints and expansion gaps between rail segments) or at stiffness transitions zones (e.g., bridge approaches, tunnels and road crossings) may lead to exacerbated degradation of the track elements and more frequent maintenance operations (Indraratna *et al.* 2011a; Nimbalkar *et al.* 2012; Ferreira and Indraratna, 2017).

Fouling materials have often been considered as unfavorable to track substructure. Selig and Waters (1994) stated that ballast breakdown, on average, accounts for up to 76% of fouling, followed by 13% of infiltration from subballast, 7% of infiltration from surface ballast, 3% from subgrade intrusion, and 1% from sleeper wear. Feddman and Nissen (2002) reported that for tracks in Australia used predominantly for coal transport, coal dust accounts for 70% - 95% of contaminants and ballast breakdown contributes from 5% - 30%. The adverse effects of fouling on the shear behaviour of ballast have been the subject of a number of studies (e.g. Budiono *et al.* 2004, Huang *et al.* 2009, Rujikiatkamjorn *et al.* 2012, among others). In fact, when the amount of contaminants is excessive, fine particles may dominate the ballast behaviour and eventually make the track unstable (Dombrow *et al.* 2009).

Past research has attempted to use cellular reinforcement (i.e. geocells) to provide lateral confinement to infill granular aggregates (Biabani *et al.* 2016a). Under induced loads, this additional confinement by the geocell helps to prevent infilled granular aggregates from spreading laterally, and by increasing infill rigidity, geocells also improve the load-carrying capacity of track embankments, which in turn enhances track performance (Ngo *et al.* 2016b, Fernandes *et al.* 2008). Planar geosynthetics (e.g., geogrids and geotextiles) have also been widely used to reinforce ballasted tracks (Figure 1) and increase the duration of track serviceability (e.g. Raymond 2002, McDowell *et al.* 2006, Brown *et al.* 2007, Ngo and Indraratna 2016). It has been reported that the mechanical interlock by geosynthetics with ballast particles can decrease the lateral displacement and degradation of ballast (Bathurst and Raymond 1987, McDowell *et al.* 2006). Current literature on the geogrid-ballast interface behaviour is still limited, both in experimental/field studies and numerical simulations, particularly when ballast becomes fouled. In this paper, the role of different geosynthetics in stabilising fresh and coal-fouled ballast is described based on the results of a series of large-scale laboratory direct shear tests, impact tests and discrete element modelling; data obtained from field trials conducted on an instrumented track at Bulli, NSW Australia are also presented and discussed.



Figure 1. Schematic of main components of track structures

2. EXPERIMENTAL STUDY OF GEOSYNTHETIC-REINFORCED BALLAST

2.1. Large-scale direct shear tests

The large-scale direct shear test apparatus used in this study consists of a 300 mm \times 300 mm steel box, 200 mm high (Figure 2). Ballast selected from Bombo quarry, New South Wales, Australia was cleaned and sieved according to Australian Standards (AS 2758.7, 1996). Coal fines were used as fouling material and the Void Contamination Index (*VCI*) introduced earlier by Tennakoon *et al.* (2012) was applied to measure the degree of fouling, as given below:

$$VCI = \frac{1+e_f}{e_b} \times \frac{G_{sb}}{G_{sf}} \times \frac{M_f}{M_b} \times 100 \tag{1}$$

where, e_f : the void ratio of fouling material; e_b : the void ratio of fresh ballast; G_{sb} : the specific gravity of ballast; G_{sf} : the specific gravity of fouling material; M_f : the dry mass of fouling material; M_b : the dry mass of fresh ballast. This method allows an accurate assessment of the degree of fouling because it incorporates the effects of void ratios, specific gravities and gradations of both fouling material and ballast.

Large-scale direct shear tests for fresh and coal-fouled ballast reinforced by a 40 mm × 40 mm geogrid were carried out to a maximum horizontal displacement of $\Delta h=37$ mm, under different normal stresses of $\sigma_n = 15, 27, 51$ and 75kPa. During the shearing process, the shearing forces and vertical displacements of the top plate were recorded at every 1mm of horizontal displacement. The shear stresses and vertical strains were then computed and plotted against the horizontal shear strain. Laboratory test results indicate that the peak shear stress of ballast increases with the normal stress and decreases with an increasing level of fouling. Strain softening and dilation have also been observed in all the tests, where a higher normal stress σ_n resulted in a greater shear strength and in smaller dilations. The coal fines reduced the peak shear stresses of the reinforced and unreinforced ballast assemblies because they coated the surfaces of ballast grains, thus inhibiting inter-particle friction and reducing the shearing resistance at the geogrid-ballast interface. Tutumluer et al. (2006) observed that the railway ballast they tested in the laboratory exhibited similar shear stress-strain responses. The variations of the normalised peak shear stress (τ_p/σ_n) and the apparent angle of shearing resistance (ϕ) with VCI for fouled ballast assemblies with and without geogrid reinforcement are shown in Figure 3. It can be observed that coal fines steadily reduce the peak shear stress of a ballast assembly, which then diminishes the apparent angle of shearing resistance. This reduction of (τ_p/σ_n) due to the presence of coal fines is significant when the VCI is less than 70%, but it becomes marginal when the VCI is higher.

The effect of fouling materials on the shear strength reduction is illustrated in Figure 4. The normalised shear strength reduction is expressed as the ratio of the decrease in peak shear stress ($\Delta \tau_p$) to normal stress (σ_n). Figure 4 shows that the decrease in shear strength is more significant for unreinforced ballast than for ballast stabilised by geogrid. This is due to the interlocking effect created at the ballast-geogrid interface (Raymond 2002, Qian *et al.* 2010). The variations of the decrease in normalized peak shear stress for ballast with and without geogrid, with respect to changes in the *VCI*, could be described by the following hyperbolic equation:

$$\frac{\Delta \tau_p}{\sigma_n} = \frac{VCI/100}{a \times VCI/100 + b}$$
(2)

where, $\Delta \tau_p$: shear strength reduction of ballast due to the presence of fines, σ_n : normal stress, *VCI*: void contamination index, *a* and *b*: hyperbolic constants.

The results obtained from direct shear tests on ballast with and without geogrid reinforcement are plotted in transformed axes to determine the hyperbolic constants (a, b), by rearranging Equation 2, as follows:

$$\frac{VCI}{100} \times \frac{\sigma_n}{\Delta \tau_p} = a \times \frac{VCI}{100} + b \tag{3}$$

The linear regression curves presented in Figure 5 prove that the decrease in normalized peak shear stress could be accurately estimated based on a hyperbolic relationship (coefficient of regression, $R^2 > 0.95$). The hyperbolic constants, *a* and *b*, for both cases are presented in tabular forms in Figure 5. It is observed that *a* and *b* are independent of the *VCI* ratio (fines content) and vary with applied normal stresses.



Figure 2. Large-scale direct shear apparatus used in the laboratory



Figure 3. Effect of VCI on the normalised peak shear strength and apparent angle of shearing resistance of ballast: (a) without geogrid; (b) with geogrid (modified after Indraratna et al. 2011b)



Figure 4. Variation of normalised peak shear stress drop for unreinforced and biaxial reinforcedballast with VCI (data source from Indraratna et al. 2011b)



Figure 5. Determination of hyperbolic constants *a* and *b* for ballast with and without geogrid reinforcement (data source from Indraratna et al. 2011b)

2.2. Geosynthetic-ballast interface behaviour

The influence of the geometry and aperture size of geogrids and confining pressure on the interface behaviour of a geogrid-reinforced ballast assembly was also evaluated by Indraratna *et al.* (2012). In their study, seven types of geogrids, namely *G1* to *G7* (Table 1) with square, rectangular, and triangular geometry and different aperture sizes (i.e. 36 mm to 70 mm) were tested by large-scale direct shear tests under varying normal stresses from 26 to 61 kPa. All the tests were conducted up to a shear displacement of 36 mm, which corresponded to a horizontal strain of 12%. The effect of applied normal stress on the friction angle of the ballast and the different ballast-geogrid interfaces is shown in Figure 6a, which indicates that the internal friction angle of ballast decreases from 64⁰ to 59⁰ when the normal stress increases from 26 to 61 kPa. It is well known that the friction angle of granular materials decreases as the confining pressure increases (Marsal 1967, Indraratna *et al.* 1998, Ngo *et al.* 2017a) and similarly, the friction angle of the ballast-geogrid interfaces also decreases with the normal stresses increase. The improvement in the behaviour of ballast-geogrid interfaces can be expressed in terms of the interface efficiency factor, which is defined as the ratio of the interface shear strength to the internal shear strength of ballast:

$$\alpha = \frac{\tan \delta}{\tan \varphi} \tag{4}$$

where, δ is the apparent friction angle of the interface and φ is the friction angle of the ballast. It should be noted that for ballast materials the cohesion intercept is omitted. The influence of the geogrid aperture size (*A*) on the shear strength of ballast-geogrid interfaces is shown in Figure 6b. Here, the values of α are plotted as a function of the *A/D50* ratio, where α increases with *A/D₅₀* until it attains a maximum value of 1.16 at *A/D50* of 1.21, and then it decreases towards unity as *A/D₅₀* approaches 2.5. The value of $\alpha < 1$ indicates an ineffective interlocking of particles, whereas $\alpha > 1$ indicates acceptable interlocking which contributes to increased shear strength. In other words, the *A/D₅₀* value at which $\alpha = 1$ represents the minimum condition required to generate the beneficial effects of geogrid reinforcement. Based on the variation of α , an optimum interlock zone is defined where the interface efficiency factor ranges from 0.95 to 1.20. The value of α attains a maximum of 1.16 at an optimum *A/D₅₀* ratio of about 1.20. According to this study, the minimum and maximum aperture sizes required to achieve the benefits of the geogrid inclusion are established as $0.95D_{50}$ and $2.50D_{50}$, respectively. Moreover, the optimum aperture size of geogrid can be considered as approximately 1.2-1.3 D_{50} .

Geogrid type	Aperture shape	Aperture size (mm)	Tensile strength (kN/m)
G1	Square	38×38	30
G2	Triangle	36	19
G3	Square	65×65	30
G4	Rectangle	44×42	30
G5	Rectangle	36×24	30
G6	Square	33 × 33	40
G7	Rectangle	70×110	20

Table 1. Physical characteristics of the geogrids used in this study



Figure 6. (a) Variation of friction angle of ballast-geosynthetic interfaces with normal stress; (b) interface efficiency factor (α) versus A/D₅₀ (*data source from Indraratna et al. 2012*)

2.3. Impact tests on geosynthetic-reinforced ballast samples

The role of a polypropylene biaxial geogrid in reducing the deformation and degradation of ballast under impact loading conditions was assessed using a high-capacity Drop-weight Impact Testing equipment designed and built at UOW (Kaewunruen and Remennikov 2010). The test rig (Figure 7) is composed of a 5.81 kN weight free-fall hammer that can be dropped from a maximum height of 6 m, which allows to simulate repeated impact loading resembling actual track conditions. The drop hammer is connected to rollers and guided through low-friction runners on vertical steel columns fixed to an isolated high-strength reinforced concrete floor. The apparatus can accommodate test samples within a working area of

 1800×1500 mm. The impact load-time histories are recorded by a dynamic load cell (capacity of 1200 kN) mounted on the drop hammer and connected to an automatic data acquisition system.

Figures 8a and 8b present the photographic and schematic illustrations of a typical test sample, where a geogrid specimen was installed at the subballast-ballast interface. To mimic a relatively low lateral confining pressure in the field, the granular materials were confined in a cylindrical rubber membrane thick enough to avoid piercing by sharp ballast particles under severe impact loads. First, a 150 mm thick subballast layer consisting of a mixture of gravel and sand was levelled and compacted in dry conditions to an initial unit weight of 18.8 kN/m³, over which the geogrid sample was positioned (Figure 8c). The ballast aggregates were then compacted on the top of the subballast mass to a representative field unit weight of 15.3 kN/m³, using a rubber-padded electric vibratory hammer. To better assess the ballast degradation (i.e., breakage) with depth, the ballast specimens were divided into three equal layers (100 mm height) through distinct colour coding (Figure 8d).

The geogrid used is composed of flat polypropylene bars with welded junctions and 31 mm square apertures, with a peak tensile strength of 40 kN/m and corresponding elongation of 8%. The impact tests were conducted with and without geogrid reinforcement to evaluate the effectiveness of the geogrid in the attenuation of impact-induced damage. The geogrid placement position within the test sample was varied (i.e., either at the base of the ballast layer or at 100 mm height) to analyse its possible influence on the ballast response. To investigate the combined use of different synthetic inclusions (i.e., geogrid and rubber mats), an additional test was conducted in which three layers of rubber mat (shock mat) accounting to a total thickness of 30 mm were provided underneath the ballast layer and a geogrid sample was placed at 100 mm height from its base.

The free-fall hammer was raised mechanically to the required drop height and released by an electronic quick release system. The drop height (150 mm) was selected to produce dynamic stresses simulating typical wheel-flats and dipped rail joints in the field (Indraratna *et al.* 2010, Jenkins *et al.* 1974). For data recording purposes, an automatic triggering was enabled using the signal obtained during the hammer free-fall and the sampling frequency rate was set to 50,000 Hz. The permanent vertical and lateral deformations of the test samples after each blow were estimated by manual measurements at strategic locations. The tests were discontinued after twelve impact blows due to the attenuation of ballast strains. To evaluate the extent of particle degradation after the tests, the three ballast layers were individually sieved and the shift in gradation was determined. The particle breakage was then quantified using the Ballast Breakage Index (*BBI*) proposed earlier by Indraratna *et al.* (2005), specifically for railway ballast.



Figure 7. Drop-weight impact test rig (designed by Kaewunruen and Remennikov 2010)



Figure 8. Impact test sample: (a) photographic view; (b) schematic illustration; (c) geogrid installed over the subballast layer; (d) ballast placed over the geogrid

Figures 9a and 9b illustrate the impact force-time histories recorded in the first and last impact blows of one representative test, respectively. Two distinct types of force peaks can be observed, i.e., multiple sharp peaks followed by a gradual peak of lower magnitude and longer duration. These peak forces are generally termed as P_1 and P_2 , respectively (Jenkins *et al.* 1974). P_1 forces represent a quasi-instantaneous reaction of the test sample to the impact load and the multiple P_1 peaks occur due to the drop hammer rebound. These forces are caused by the inertia of the top plate resisting the downward motion of the drop hammer and the compression of the contact zone between the free-fall hammer and the sample top plate. The effects of P_1 type forces are generally filtered out by the load assembly, and thus they would not directly affect ballast degradation (Frederick and Round 1985). On the other hand, the force P_2 is associated with the mechanical resistance of ballast against impact loading, leading to its significant compression. Therefore, P_2 forces are of greater importance in the analysis of track deterioration (e.g., Rochard and Schmid 2004). The specifications of the British Rail Safety and Standards Board (1995) suggest that, for the safety of the track, P_2 forces should not exceed 322 kN.



Figure 9. Typical impact force response: (a) 1st blow; (b) 12th blow

The variation of the force P_2 along the number of blows in the different tests is plotted in Figure 10. It can be observed that the magnitude of these forces increases progressively throughout the repeated impacts. In fact, with increasing number of blows, the ballast develops a denser assembly due to the rearrangement and reorientation of aggregates and particle breakage, which offers higher inertial resistance causing higher P_2 values. This finding suggests that the impact forces induced in a newly laid track will be lower than those in a heavily used track where the ballast is in a denser state. Figure 10 also shows that the values of P_2 are not significantly influenced by the inclusion of the geogrid reinforcement. However, a considerable reduction of the impact forces is achieved by installing a rubber mat below the ballast layer, which is associated with the energy-absorbing capacity or damping characteristics of this material.

The permanent axial and radial strains of ballast along the tests conducted on unreinforced and geogridreinforced samples are presented in Figures 11a and 11b, respectively. As expected, ballast deformations increase with the successive blows. A relatively rapid strain increment rate is observed during the initial impacts due to the reorientation and corner breakage of aggregates, which gradually reduces after a certain stage. As shown in Figure 11, the provision of the geogrid mitigates the ballast strains, in comparison with the unreinforced sample, and higher efficiency is achieved when the reinforcement is installed at 100 mm height from the subballast-ballast interface. This can be attributed to a better interlocking with the ballast particles, as the particles above and below the geogrid can penetrate its apertures, in comparison to when the geogrid is placed directly above a dense subballast layer. Moreover, installing a rubber mat below the ballast mass and a geogrid at 100 mm height further enhances the ballast deformation behaviour, which is related to the attenuation of the impact forces P2.



Figure 10. Evolution of P2 force with the number of blows for different test conditions



Figure 11. Permanent deformation behaviour of ballast with and without geogrid inclusion: (a) axial strain; (b) radial strain

As previously mentioned, the impact-induced degradation of the ballast particles was quantified using the parameter Ballast Breakage Index – *BBI* (Indraratna *et al.* 2005), which is estimated on the basis of the change in the particle size distribution (PSD) before and after the test, as illustrated in Figure 12. The increase in the degree of breakage causes the PSD curve to shift towards the smaller particle size region in a conventional PSD plot. An increase in the area A between the initial and final PSD curves leads to higher values of *BBI*. By referring to a linear particle size axis, *BBI* can be computed as follows:

$$BBI = \frac{A}{A+B}$$
(5)

where, A is the area described above and B is the potential breakage or area between the arbitrary boundary of maximum breakage and the final PSD curve.



Figure 12. Assessment of ballast breakage using the parameter BBI (Indraratna et al. 2005)

The values of *BBI* obtained after the tests for each of the three individual layers and for the whole ballast samples (*BBI*-average) are listed in Table 2. The particle degradation is more pronounced in the top ballast layer, where higher impact-induced stresses are generated, and generally decreases with increasing distance to the top plate. On average, geogrid-reinforced ballast experiences less degradation in comparison with the unreinforced case. Similar to what was observed in terms of ballast strains, the inclusion of the geogrid within the ballast layer (i.e., 100 mm above the subballast-ballast interface) resulted in the highest ballast performance (i.e., the lowest average *BBI* value), which is associated with the better interlocking and increased lateral confinement, as elaborated above.

		I	8	
Tast	BBI	BBI	BBI	BBI
Test	(bottom)	(middle)	(top)	(average)
Without inclusions	0.155	0.109	0.187	0.150
Geogrid (base)	0.111	0.122	0.190	0.141
Geogrid (100 mm height)	0.091	0.143	0.155	0.130
Shock mat (base) + geogrid (100 mm height)	0.089	0.130	0.171	0.129

Table 2. BBI values obtained after impact testing

3. FIELD INVESTIGATION AT BULLI TRACK

3.1. Track construction

Geosynthetics have been widely and successfully used in new rail tracks and in track rehabilitation schemes for almost three decades, and when appropriately designed and installed, they are a cost effective alternative to more traditional techniques (Bathurst *et al.* 2009, Kwon and Penman 2009, Indraratna *et al.* 2016). To investigate the stress and deformation imparted to track by train traffic, as well as the benefits of using geosynthetics in fresh and recycled ballast, a field trial has been carried out in a section of South Coast Track owned and operated by Sydney Trains (formerly RailCorp). During this period, the train-induced stresses and the vertical and lateral deformations of the track were monitored by the Centre for Geomechanics and Railway Engineering (Indraratna *et al.* 2010).

The construction and instrumentation of this track segment is shown in Figure 13; the subgrade consists of stiff, over-consolidated silty clay with shale cobbles and gravels, over bedrock of highly weathered sandstone. The instrumented track is divided into four, 15 m long sections (Figure 13a), and the layers of ballast and subballast are 300 mm and 150 mm thick, respectively. Fresh and recycled ballast without inclusion of a geocomposite are used at Sections 1 and 4, respectively, whereas the other two sections are reinforced by a layer of geocomposite at the ballast-subballast interface (Figure 13b). The geocomposite is composed of a biaxial geogrid (aperture size = $40 \text{ mm} \times 27 \text{ mm}$, peak tensile strength = 30 kN/m) placed over a layer of nonwoven polypropylene geotextile (mass per unit area = 140 g/m^2 , thickness = 2 mm), as shown in Figure 7c. Further technical specifications of the materials used during construction are reported elsewhere (Indraratna *et al.* 2010, 2016).

The vertical and horizontal stresses are measured by rapid response hydraulic earth pressure cells with thick, grooved active faces based on semi-conductor type transducers. Settlement pegs are installed between the sleeper and ballast, and between the ballast and subballast to measure the vertical deformation of the ballast layer. The settlement pegs consist of 100 mm \times 100 mm \times 6 mm stainless steel base plates attached to 10 mm diameter steel rods. Lateral deformation is recorded by potentiometric displacement transducers placed inside 2.5 m long stainless steel tubes that can slide over each other, with 100 mm \times 100 mm end caps as anchors. The pressure cells and lateral displacement transducers are connected to a computer controlled data acquisition system which can operate at a maximum frequency of 40 Hz. The positions of the settlement pegs and displacement transducers are shown in Figure 13b and the placement of geosynthetics in the track is illustrated in Figure 13c.

3.2. Measured ballast deformation

In the field, vertical and horizontal deformation is measured against time, which means that a relationship between the annual rail traffic in million gross tons (MGT) and axle load (A_t) is needed to determine the number of load cycles N, as proposed by Selig and Waters (1994). This relationship is expressed as: $N_t = 106/(A_t \times N_c)$, where N_t , A_t and N_c are the numbers of load cycles per MGT, the axle load in tonnes, and the number of axles per load cycle. When this relationship is used for a traffic tonnage of 60 MGT per year and four axles per load cycle, an axle load of 25 tonnes gives 600,000 load cycles per MGT. A simple survey technique is then used to record changes in the reduced level of tip of the settlement peg (Indraratna *et al.* 2010). Figure 14 shows the variation of average deformation of ballast against the number of load cycles (N). Unlike fresh ballast, recycled ballast exhibits less vertical and lateral deformation, possibly due to its moderately graded particle size distribution - PSD ($C_u = 1.8$) compared to the very uniform PSD ($C_u = 1.5$) of fresh ballast. These results also indicate that the geocomposite reinforcement reduced the vertical (S_v) and lateral (S_l) deformation of fresh ballast by about 33% and 49%, respectively, while decreasing the vertical and lateral deformation of recycled ballast by about 9% and 11%, respectively. Lateral deformation is one of the most important indices affecting track stability, and the use of a geocomposite layer can be an effective way of curtailing it significantly, with obvious implications for improved track performance and reduced maintenance costs.



Figure 13. (a) Construction of track sections; (b) installation of vertical settlement pegs and displacement transducers; (c) installation of geosynthetics (modified after Indraratna et al. 2010)



Figure 14. Average deformation of the ballast layer: (a) vertical settlement (S_v) ; (b) lateral displacement (S_l) (*data sourced from Indraratna et al. 2010- with permission from ASCE*)

3.3. Traffic induced stresses

Figure 15a shows the peak cyclic vertical (σ_v) and lateral (σ_i) stresses recorded at Section 1 (i.e. fresh ballast without geocomposite) after the passage of a coal train with an axle load of 25 tonnes. Here, the peak cyclic vertical stress decreased by 73% and 82% at depths of 300 mm and 450 mm, respectively. Moreover, σ_i decreased only marginally with depth, which implies that artificial inclusions are needed for additional restraints (Nimbalkar *et al.* 2012). While most of the peak cyclic vertical stresses were below 230 kPa, one value of σ_v reached 415 kPa, as shown in Figure 15b; this was later found to be associated with a wheel flat, thus proving that much larger stresses are induced by wheel imperfections (Kaewunruen and Remennikov 2010, Zhai *et al.* 2004, Ferreira and Indraratna, 2017). The resulting particle breakage could be mitigated by the use of a shock mat, as reported by Indraratna *et al.* (2014a) in the Singleton study.



Figure 15. Cyclic stresses induced by coal train with wagons (100 tonnes): (a) variation of stresses with depth; (b) additional stress due to wheel flat (data sourced from Indraratna et al. 2010)

4. DISCRETE ELEMENT MODELLING

4.1. Discrete Element Method (DEM)

The discrete element method (DEM) introduced by Cundall and Strack (1979) is widely used to study the behaviour of granular materials. DEM is often used to model ballast because it captures the discrete nature of a granular assembly which consists of a collection of arbitrarily shaped discrete particles under quasistatic and dynamic conditions (McDowell and Bolton 1998, Lobo-Guerrero and Vallejo 2006, Vallejo *et al.* 2006, O'Sullivan *et al.* 2008, O'Sullivan and Cui 2009, Bhandari and Han 2010, Han *et al.* 2011, Huang and Tutumluer 2011, Tutumluer *et al.* 2012, Indraratna *et al.* 2014b, McDowell and Li 2016, Ngo *et al.* 2017b, among others). Particle motion is determined using Newton's second law and the interaction between particles is determined using Newton's second law contact laws. At a given time, the force vector \vec{F} that represents the interaction between the two particles is resolved into normal (\vec{F}_N) and shear component (\vec{F}_T) with respect to the contact plane:

$$\vec{F}_N = K_N U^n$$

(6)

$$\delta \vec{F}_T = -K_T \cdot \delta U^s \tag{7}$$

where, K_N and K_T are the normal and tangential stiffnesses at the contact; U^n is the normal penetration between two particles; δU^s is the incremental tangential displacement; and $\delta \vec{F}_T$ is the incremental tangential force. The resistance moment \vec{M}_r is introduced to represent the restraint (i.e. interlocking) between two particles A and B and is determined by:

$$\vec{M}_{r} = \begin{cases} K_{r}\vec{\omega}_{r} & \text{if } K_{r} \|\vec{\omega}_{r} \| < \|\vec{M}_{r}\|_{lim} \\ \|\vec{M}_{r}\|_{lim} \frac{\vec{\omega}_{r}}{\|\vec{\omega}_{r}\|} & \text{if } K_{r} \|\vec{\omega}_{r}\| \ge \|\vec{M}_{r}\|_{lim} \end{cases}$$
(8)

where $\|\vec{M}_r\|_{lim} = \eta_r \|\vec{F}_r\| \frac{R_A + R_B}{2}$; $K_r = \gamma_r \left(\frac{R_A + R_B}{2}\right)^2$; $\vec{\omega}_r$ is a rolling angular vector representing the relative changes in orientation between two particles and is computed by adding the angular vectors of the incremental rolling; here η_r is the dimensionless coefficient, and γ_r is the rolling resistance coefficient.

4.2. Modeling irregularly-shaped ballast particles

Ballast particles of varying shapes and sizes are simulated by clumping many spheres together to represent actual ballast gradation (McDowell et al. 2006, Ngo et al. 2016c, Tutumluer et al. 2006, Aursudkij et al. 2009), as shown in Figure 16a. The clump approach is used to generate groups of slaved particles to model arbitrary particle shapes. Particles within a clump may overlap to any extent, but there are no contact forces between them, therefore a clump acts like a rigid body (with deformable boundary) that will not break apart, regardless of the forces acting upon it (Itasca 2014). The basic properties of a clump are its total mass m; the location of the centre of clump mass, $x_i^{[G]}$; and the moments and products of inertia I_{ii} and I_{ij} . For a clump consisting of N_p particles, each of which has mass $m^{[p]}$, radius $R^{[p]}$, and centroid location $x_i^{[p]}$, the mass properties are defined by Itasca (2014) as:

$$\boldsymbol{m} = \sum_{p=1}^{N_p} \boldsymbol{m}^{[p]} \tag{9}$$

$$x_i^{[G]} = \frac{1}{m} \sum_{p=1}^{N_p} m^{[p]} x_i^{[p]}$$
(10)

$$I_{ii} = \sum_{p=1}^{N_p} \left\{ m^{[p]} \left(x_j^{[p]} - x_j^{[G]} \right) \left(x_j^{[p]} - x_j^{[G]} \right) + \frac{2}{5} m^{[p]} R^{[p]} R^{[p]} \right\}$$
(11)

$$I_{ij} = \sum_{p=1}^{N_p} \left\{ m^{[p]} \left(x_i^{[p]} - x_i^{[G]} \right) \left(x_j^{[p]} - x_j^{[G]} \right) \right\}; \qquad (i \neq j)$$
(12)

The motion of a clump is determined by the resultant force and moment vectors acting upon it, but owing to its rigid body its motion can be described in terms of the translational motion of a point in the clump and the rotational motion of the entire clump. The equation for translational motion can be expressed in the vector form:

$$F_i = m(\ddot{x}_i - g_i) \tag{13}$$

where F_i is the resultant force, the sum of all externally applied forces acting on the clump and g_i is the body force acceleration vector arising from gravity loading. The equation for rotational motion can be written in the matrix form (Itasca 2014):

$$\{M\} - \{W\} = [I]\{\alpha\}$$
(14)
where, $[M] = \begin{cases} M_1 \\ M_2 \\ M_3 \end{cases};$ $[I] = \begin{bmatrix} I_{11} & -I_{12} & -I_{13} \\ -I_{21} & I_{22} & -I_{23} \\ -I_{31} & -I_{32} & I_{33} \end{bmatrix};$ $[\alpha] = \begin{cases} \alpha_1 \\ \alpha_2 \\ \alpha_3 \end{cases} = \begin{cases} \dot{\omega}_1 \\ \dot{\omega}_2 \\ \dot{\omega}_3 \end{cases};$ and

where,

$$\begin{bmatrix} W \end{bmatrix} = \\ \begin{pmatrix} \omega_2 \omega_3 (I_{33} - I_{22}) + \omega_3 \omega_3 I_{23} - \omega_2 \omega_2 I_{32} - \omega_1 \omega_2 I_{31} + \omega_1 \omega_3 I_{21} \\ \omega_3 \omega_1 (I_{11} - I_{33}) + \omega_1 \omega_1 I_{31} - \omega_3 \omega_3 I_{13} - \omega_2 \omega_3 I_{12} + \omega_2 \omega_1 I_{32} \\ \omega_1 \omega_2 (I_{22} - I_{11}) + \omega_2 \omega_2 I_{12} - \omega_1 \omega_1 I_{21} - \omega_3 \omega_1 I_{23} + \omega_3 \omega_2 I_{13} \end{pmatrix}$$

$$\begin{bmatrix} W \end{bmatrix} =$$

$$\begin{bmatrix} (15) \\ W \end{bmatrix} =$$

in which, [M] is the resultant moment about the centre of mass, and ω_i and $\dot{\omega}_i$ are the angular velocity and angular acceleration about the principal axes, respectively.

4.3. Modelling of geogrid-reinforced ballast

The discrete element method (DEM) developed by Cundall and Strack (1979) has been used extensively to study the behaviour of granular materials. The DEM approach is used in this study to simulate the large-scale direct shear tests of ballast reinforced by geogrids. Figure 16 shows how DEM is used to model geogrid-reinforced ballast in a direct shear test. The model dimensions are similar to those existing in the laboratory (300 mm long x 300 mm wide x 200 mm high). Ballast particles of varying shapes and sizes are simulated by clumping many spheres together to represent actual ballast gradation, which is then placed at random locations within the specified wall boundary without overlapping. The micromechanical parameters used to model ballast, geogrid, and coal fines are presented in Ngo *et al.* (2014).

DEM simulations of direct shear tests are conducted at three normal stresses of $\sigma_n = 27$ kPa, 51kPa, and 75kPa for fresh and coal-fouled ballast (*VCI*=40%), with and without the inclusion of geogrid. Fouling is modelled by injecting a predetermined number of 1.5 mm spheres (145,665 spheres for *VCI*=40%) into the voids of fresh ballast. Figure 17 shows comparisons of the shear stress-strain and vertical displacement responses of geogrid-reinforced ballast from the DEM analysis and those measured experimentally. It can be seen that the simulation results agree reasonably well with the laboratory data at any particular normal stress. The ability of the geogrid reinforcement to increase the shear strength of both fresh and fouled ballast was observed by comparing the results for the geogrid-stabilised ballast assemblies with those for the unreinforced ballast. This is believed to be due to the interlocking effect that occurs between the ballast grains and the geogrid (Ngo *et al.* 2017c).

4.4. Contact force distribution and contours of strain developed in the geogrids

Figure 18 presents the contact force distributions of fresh and fouled ballast (VCI=40%) with and without geogrid reinforcement at a shear strain of 6% and under the normal stress of 51 kPa. Contact forces between particles were plotted as lines whose thickness is proportional to the magnitude of the forces. For the purpose of clarification, only contact forces with magnitude higher than the average value of contact forces in the assembly were plotted. It is seen that the 40% VCI fouled ballast assemblies (Figures 18b and 18d) exhibit denser contact chains and reduced maximum contact forces, compared with those for the fresh ballast (Figures 18a and 18c). This is related to the presence of coal fines in the voids among large particles that partially carry and transmit contact forces across the assembly (Bolton et al. 2008, Thornton and Zhang 2010). It is also observed that, at the shearing plane, contact forces developed between the geogrid and surrounding ballast grains, which is attributed to the interlocking effect between them. Compared to the unreinforced ballast, the geogrid-reinforced ballast exhibited a significant increase both in the number and magnitude of contact forces at the geogrid-ballast interface. The mobilisation of large contact forces within the geogrid-reinforced ballast assembly comes from the interlock between the ballast and geogrid. For fouled ballast, the mobilised contact forces were lower than those for fresh ballast due to the reduced effectiveness of the geogrid apertures. In fact, the effectiveness of the geogrid-reinforcement decreases with an increase in VCI for a given normal stress, as observed in the laboratory.

The strains developed in the geogrids could not be measured during the experiments due to the complexity of the installation of strain gauges on geogrids and difficulty in preventing the damage caused by sharp edges of ballast aggregates. However, they could be captured in the numerical simulation and are presented herein for the completeness. Figures 19a-b show the horizontal contours of strain developed across the geogrid at the end of the direct shear test (shear strain of 13%) for fresh ballast and 40%*VCI* fouled ballast, respectively. The simulated and actual deformed shape of the geogrid at the end of the test is also shown in Figures 19c-d, respectively. It is clearly seen that the strains developed non-uniformly across the geogrid and the magnitude of strain depends on the degree of interlock between the geogrid and ballast particles. The geogrid in the fouled ballast assembly experienced a slightly lower maximum strain than that in the fresh ballast. This would be attributed to the reduced interlocking effect between the geogrid and ballast aggregates due to the presence of coal fines clogging the geogrid-ballast interface. Hence, for the sound design of rail tracks, it is imperative to understand the underlying mechanisms of geogrid-ballast interaction under various fouling conditions and determine a threshold value of fouling for track maintenance purposes.



Figure 16. DEM model of geogrid-reinforced ballast: (a) simulated grains; (b) geogrid; (c) fresh ballast; (d) fouled ballast (modified after Ngo et al. 2014)



Figure 17. Effect of VCI on the shear stress and vertical displacement versus shear displacement for geogrid-reinforced ballast: (a) fresh ballast; (b) fouled ballast (after Ngo et al. 2014)

4.5. Micromechanical analysis

Load transfer in a granular assembly depends on the orientation of contacts where the applied load is transmitted through an interconnected network of force chains at contact points (Oda and Iwashita 1999). When subjected to shearing, the contact forces of ballast assemblies evolve so that the number of load-carrying contacts and their orientations inevitably change. In this study, the second-order density distribution tensor introduced by Rothenburg (1980) was used to examine the anisotropy of contact forces of the ballast assembly at different settlements. These tensors were incorporated into the DEM models and are given as follows:

$$F_{ij} = \int_0^{2\pi} E(\theta) n_i n_j d\theta = \frac{1}{N_c} \sum_{k=1}^{N_c} n_i^k n_j^k$$
(16)

$$N_{ij} = \frac{1}{2\pi} \int_0^{2\pi} \frac{\bar{f}_n(\theta)}{\bar{f}_0(\theta)} n_i n_j d\theta = \frac{1}{N_c} \sum_{k=1}^{N_c} \frac{f_n^k}{\bar{f}_0} n_i^k n_j^k$$
(17)

$$S_{ij} = \frac{1}{2\pi} \int_0^{2\pi} \frac{\bar{f}_{\bar{s}}(\theta)}{\bar{f}_0(\theta)} t_i n_j d\theta = \frac{1}{N_c} \sum_{k=1}^{N_c} \frac{f_{\bar{s}}^k}{\bar{f}_0} t_i^k n_j^k$$
(18)

where, F_{ij} , N_{ij} , and S_{ij} are fabric, average contact normal force and average contact shear force tensors, respectively; $E(\theta)$, $\bar{f}_n(\theta)$, and $\bar{f}_s(\theta)$ are the corresponding density distribution functions; f_n^k and f_s^k are contact normal force and shear force, respectively; $\mathbf{n} = (\cos\theta, \sin\theta)$ is unit normal vector, and $\mathbf{t} = (-\sin\theta, \cos\theta)$ is the vector perpendicular to \mathbf{n} ; and N_c is the total number of contacts in the assembly. \bar{f}_0 is the average contact normal force determined by:

$$\bar{f}_0 = \frac{1}{2\pi} \int_0^{2\pi} \bar{f}_n(\theta) d\theta = \frac{1}{N_c} \sum_{k=1}^{N_c} f_n^k$$
(19)

The force-fabric is characterised by the distribution of inter-particle contact orientations that can be described by the following Fourier series approximations proposed by Rothenburg and Bathurst (1989):

$$E(\theta) = \frac{1}{2\pi} [1 + a\cos^2(\theta - \theta_a)]$$
⁽²⁰⁾

$$\bar{f}_n(\theta) = \bar{f}_0[1 + a_n \cos 2(\theta - \theta_n)] \tag{21}$$

$$\bar{f}_s(\theta) = \bar{f}_0[-a_s \cos 2(\theta - \theta_s)] \tag{22}$$

where, a, a_n , and a_s are the coefficients of contact normal, contact normal force and contact shear force anisotropies, respectively; θ_a , θ_n , and θ_s are the corresponding major principal directions of anisotropies, respectively.



Figure 18. Distribution of contact forces of fresh and 40% VCI fouled ballast with and without geogrid for a normal stress of 51kPa at a shear strain $\varepsilon_s = 6\%$: (a) unreinforced fresh ballast; (b) 40% VCI unreinforced ballast; (c) geogrid-reinforced fresh ballast; (d) 40% VCI geogrid-reinforced ballast



Figure 19. Strains developed across the geogrid: (a) contour strain for fresh ballast; (b) contour strain for 40% VCI fouled ballast; (c) simulated deformed geogrid; (d) photograph of deformed grid after the test

4.6. Polar histogram of contact forces

The micromechanical analysis presented herein focusses on the evolution of contact force distributions of particles in the shear box at varying shear displacements. Eqs. (16)-(18) were used to capture the contact information of every particle in the DEM model while Eqs. (20)-(22) were used for the Fourier series approximation. Figure 20 shows the polar histograms of inter-particle contact force distributions for the *VCI* fouled ballast (*VCI*=40%) at different shear displacements, Δh , captured from the DEM simulation and those obtained from the Fourier approximation. Polar histograms of the contact forces were obtained by collecting the contact force information at the predefined bin angle $\Delta\theta$ =10°. At the beginning of the shearing process the inter-particle forces were almost uniformly distributed in all orientations (i.e., isotropic), as shown in Figure 20a. The normal contact force anisotropy was coaxial with the vertical axis, having a principal direction of almost $\theta_n = 4^0$, which is the major principal stress in the assembly. At this stage the contact shear force anisotropy was very small and its direction with the vertical axis was almost zero due to very low induced shear stress. With an increase in the applied shear load the contact force chains develop to resist shear and disperse the loads from the surface into the ballast. Anisotropies of average contact normal force and shear force grow and rotate vigorously as shearing progresses, and reach the values of $\theta_n = 33^0$ and 51^0 at corresponding shear displacements of $\Delta h = 9$ mm and 18 mm,

respectively. As the shear displacement increases (Figures 20b and 20c), the contact force anisotropies tend to align towards the horizontal axis as the number of contacts in the horizontal shearing direction increases. This analysis provided more insight into the orientation of contacts where the applied load was transmitted to a granular assembly through an interconnected network of forces that are difficult to measure in the laboratory.



Figure 20. Polar histograms of contact and force orientations in the fresh ballast assembly at varying shear displacements Δh : (a) $\Delta h = 0$ mm; (b) $\Delta h = 9$ mm; (c) $\Delta h = 18$ mm

4.7. Numerical modelling and analysis for geocell-reinforced subballast

Salim and Indraratna (2004) proposed an elasto-plastic stress-strain constitutive model which incorporates dilatancy, breakage, and the plastic flow rule to determine ballast deformation and degradation. The authors used a generalised 3D system to define contact forces, stresses and strains in granular media, including the plastic potential, hardening function and the particle breakage. The model was developed based on the concept of critical state and the theory of plasticity with a kinematic-type yield locus (constant stress ratio). The increments of plastic distortional strain, $d\epsilon_s^p$ and volumetric strain, $d\epsilon_v^p$ are determined as:

$$d\varepsilon_{s}^{p} = \frac{2\alpha\kappa \left(\frac{p}{p_{cs}}\right) \left(1 - \frac{p_{o(i)}}{p_{cs(i)}}\right) (9 + 3M - 2\eta * M) (\eta - \eta_{i}) d\eta}{(M - \eta_{i})^{2} (1 + e_{i}) \left(\frac{2p_{o}}{p} - 1\right) \left[9(M - \eta *) + \frac{B}{p} \{\chi + \mu(M - \eta *)\}\right]}$$
(23)

$$d\varepsilon_{\nu}^{p} = \frac{9(M-\eta)}{9+3M-2\eta^{*}M}d\varepsilon_{s}^{p} + \left(\frac{B}{p}\right)\left[\frac{\chi+\mu(M-\eta^{*})}{9+3M-2\eta^{*}M}\right]d\varepsilon_{s}^{p}$$
(24)

where, p: effective mean stress; p_{cs} : value of p on the critical state line at the current void ratio; p_0 : value of p at the intersection of the undrained stress path and the initial stress ratio line. The subscript *i* indicates the initial value at the start of shearing. The parameter η is the stress ratio ($\eta = q/p$), q is the deviator stress, $\eta^* = \eta$ (p/p_{cs}), M: critical state stress ratio, e_i : initial void ratio, κ is the negative slope of the compression curve (e-*ln*p), and α , B, χ and μ are dimensionless constants. This model consists of 11 parameters for monotonic loading and 4 additional parameters for cyclic loading, which can be determined using the results of large-scale triaxial tests and the measured particle breakage. The model has been validated using large-scale triaxial tests, as shown in Figure 21.



Figure 21. Model prediction compared with experimental data for drained triaxial shearing (data from Salim and Indraratna 2004)

A laboratory study on the use of geocells to reinforce subballast using a large-scale Track Process Simulation Apparatus (TPSA) is illustrated in Figure 22a. The experimental results were presented earlier by Indraratna *et al.* (2015). Numerical studies using the Finite Element Method (FEM) were also carried

out to investigate the reinforcement effect of geocells where the material properties were obtained from laboratory tests and the model geometry was consistent with the TPSA used in the laboratory (800 mm \times $600 \text{ mm} \times 450 \text{ mm}$). Cyclic loads acting beneath the ballast and then loaded directly onto the subballast surface exhibited the same characteristics as those applied in the laboratory. An elasto-plastic constitutive model with non-associative behaviour was also adopted to simulate the subballast in the analysis. Drucker-Prager yield criterion was used to capture the elasto-plastic behaviour of subballast (Biabani et al. 2016a). The model parameters were determined in the laboratory using triaxial equipment (i.e. friction angle $\phi=39^{\circ}$, angle of dilation $\psi=9^{\circ}$, cohesion yied stress = 2 kPa, Poisson's ratio v = 0.3). A hexagonal shape was used to model the geometry of the geocell pockets, similar to the actual shape of the geocell tested in the laboratory. The input parameters used to model the geocell are as follows: density = 950 (kg/m^3) , secant modulus (3% strain) = 0.3-5 (GPa) and Poisson's ratio = 0.3. Additional details of the FEM model can be found in Biabani et al. (2016a). Due to the high computation time required to simulate a cyclic model, all the analyses were conducted up to 10,000 cycles, after which most of the subballast deformation had already occurred, as observed in the laboratory (Biabani et al. 2016b). The cyclic loading and the dynamic behaviour of the subballast and geocell were modelled using a predetermined sinusoidal functional loading and a dynamic amplification factor of 1.45.





Figure 22. (a) Track Process Simulation Apparatus (TPSA); (b) finite element modelling for geocell-reinforced subballast (modified after Biabani et al. 2016a)

The contours of lateral displacement of geocell-reinforced subballast under a confining pressure of $\sigma_3'=10$ kPa are shown in Figure 23(a). As discussed by Biabani *et al.* (2016b), the lateral deformation of the subballast increases with the number of load cycles and the maximum lateral spreading occurs beneath the geocell-reinforced subballast. It is noted that the tensile strength of the geocell is an important parameter governing the performance of geocell-reinforced subballast, where it is commonly considered to be constant in conventional design practices (Ngo *et al.* 2017b; Leshchinsky and Ling 2013). However, data measured in this study show that during cyclic loading the mobilised tensile stress in the geocell varies significantly, as shown in Figure 23b; in fact, during the loading stage, the maximum tensile stress is mobilised in the geocell to prevent the subballast infill from excessive lateral spreading (Ngo *et al.* 2016b). Tensile stress develops non-uniformly across the geocell, where the middle of the geocell strip (e.g. point A) exhibits the highest degree of mobilised tensile stress. Figure 23 also shows that the minimum tensile stress occurs parallel to the intermediate principal stress (e.g. point C), where the geocell mattress is prevented from moving in this direction (i.e. plane strain condition).



Figure 23. (a) Typical lateral deformation contour of geocell-reinforced subballast; (b) tensile stress mobilised in geocell mattress subjected to cyclic loading (modified after Biabani et al. 2016a)

5. CONCLUSIONS

This paper briefly reviews the current extent of knowledge on how ballasted rail tracks perform with geosynthetic reinforcement based on laboratory tests, field trials, and numerical simulations. The results of the direct shear tests indicated that geogrids increase the shear strength and apparent angle of shearing resistance, while only slightly decreasing the vertical displacement of composite geogrid-ballast assemblies. However, when ballast is fouled by coal fines, the benefits of the geogrid reinforcement decrease in proportion to the increased degree of fouling. This is believed to be due to the fact that coal fines cover the surface of ballast aggregates, acting as a lubricant, which induces the particles to slide and roll over each other, thus increasing the dilation. These coal fines infiltrate between the ballast and geogrid and become trapped between the geogrid apertures. Hence, fewer particles can interlock through the geogrid apertures which in turn leads to reduced interface shearing resistance. It was also noted that the normalised aperture ratio, (A/D_{50}) has a profound influence on the interface efficiency factor (α), where the optimum aperture size of geogrids to maximise the interface shear strength is around $1.2D_{50}$. The minimum and maximum aperture sizes required to attain the beneficial effects of geogrids are $0.95D_{50}$ and $2.5D_{50}$, respectively. Under impact loading conditions, the use of a biaxial geogrid mitigates the lateral and vertical deformation of ballast and the particle breakage. Higher efficiency is achieved when the geogrid is placed within the ballast layer, at 100 mm height from its base, in comparison to when it is installed at the subballast-ballast interface. This is associated with an enhanced ballast-geogrid interaction obtained when the particles on both sides of the geogrid can penetrate its apertures, in contrast to when the reinforcement is placed directly over a dense subballast mass. Moreover, installing a rubber mat underneath the ballast layer and a geogrid at 100 mm height may considerably attenuate the impactinduced stresses and further reduce the lateral spreading and vertical settlement of ballast, thus contributing to improved track longevity.

The results of a comprehensive field monitoring program carried out at Bulli track in NSW, Australia, to assess the ability of geosynthetics to improve track stability have been discussed. In this study, both fresh and recycled ballast were used and a geocomposite reinforcement consisting of a biaxial geogrid placed over a nonwoven geotextile was installed beneath the ballast layer. The measured data showed that the use of discarded (recycled) ballast is an attractive option. The use of the geocomposite contributed to decreased track settlement and lateral spreading, with obvious implications for improved track stability and reduced maintenance costs.

A series of DEM simulations of large-scale direct shear tests was carried out for fresh and coal-fouled ballast (*VCI*=40%) with and without the inclusion of geogrids. Irregularly-shaped ballast grains were simulated in DEM by clumping many balls together in approximate sizes and positions. The geogrids were modelled using bonded spherical particles of 2.00 mm diameter at the ribs and 4.00 mm diameter at the junctions. The coal fines were modelled by introducing a pre-determined number of miniature balls into the ballast voids. The results obtained from the DEM model for fresh and fouled ballast were in good agreement with the measured data, showing that the proposed model is able to capture the stress-strain behaviour of railway ballast. The presence of coal fines in the ballast assembly facilitated the reduced interlock between the ballast grains and geogrids which resulted in lower shear strength. The findings provide a better understanding of the ballast-geogrid interaction mechanisms, long-term deformation and degradation of ballast, as well as the benefits of using geosynthetics to enhance the overall performance of ballasted tracks.

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