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Improved performance of geosynthetics enhanced ballast: laboratory and numerical studies

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Abstract

Ballasted rail tracks form one of the most important worldwide transportation modes in terms of traffic tonnage, serving the needs of bulk freight and passenger movement. High impact and cyclic loads can cause a significant deformation leading to poor track geometry. In order to mitigate these problems, the concept of the inclusion of geosynthetics in rail tracks is introduced. This paper presents the current state-of-the-art knowledge of rail track geomechanics, including results obtained from laboratory testing, field investigations and numerical modelling to study the load-deformation behaviour of ballast improved by geosynthetics. The shear stress-strain and deformation behaviour of geosynthetic-reinforced ballast are investigated in the laboratory using a large-scale direct shear test device, a track process simulation apparatus and a drop-weight impact testing equipment. Computational modelling using the discrete-element method is employed to simulate geosynthetic-reinforced ballasted tracks, capturing the discrete nature of ballast aggregates when subjected to various types of loading and boundary conditions. Discreteelement modelling is also used to conduct micromechanical analysis at the interface between ballast and geogrid, providing further insight into the behaviour of ballast subjected to cyclic loadings. These results provide promising approaches to incorporate into existing track design routines catering for future high-speed trains and heavier heavy hauls.

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Improved performance of geosynthetics enhanced ballast: laboratory and numerical studies

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Ballasted rail tracks form one of the most important worldwide transportation modes in terms of traffic tonnage, serving the needs of bulk freight and passenger movement. High impact and cyclic loads can cause a significant deformation leading to poor track geometry. In order to mitigate these problems, the concept of the inclusion of geosynthetics in rail tracks is introduced. This paper presents the current state-of-the-art knowledge of rail track geomechanics, including results obtained from laboratory testing, field investigations and numerical modelling to study the load–deformation behaviour of ballast improved by geosynthetics. The shear stress–strain and deformation behaviour of geosynthetic-reinforced ballast are investigated in the laboratory using a large-scale direct shear test device, a track process simulation apparatus and a drop-weight impact testing equipment. Computational modelling using the discrete-element method is employed to simulate geosynthetic-reinforced ballasted tracks, capturing the discrete nature of ballast aggregates when subjected to various types of loading and boundary conditions. Discrete-element modelling is also used to conduct micromechanical analysis at the interface between ballast and geogrid, providing further insight into the behaviour of ballast subjected to cyclic loadings. These results provide promising approaches to incorporate into existing track design routines catering for future high-speed trains and heavier heavy hauls.

Notation

a	coefficient of contact anisotropy
a_n	coefficient of normal force anisotropy
a_s	coefficient of shear force anisotropy
C_n	coordination number
$d_{\max}$	maximum particle size
$E(\theta)$	density distribution function of fabric tensor
$\mathbf{F}_{ij}$	fabric tensor
f	frequency
f_n^k, f_s^k	contact normal force and shear force
$\tilde{f}_n(\theta)$	density distribution function of contact normal force tensor
$\tilde{f}_s(\theta)$	density distribution function of contact shear force tensor
$\tilde{f}_0$	average contact normal force

k_n	contact normal stiffness
$k_{n\text{-wall}}$	contact normal stiffness of wall–particle
k_s	contact shear stiffness
$k_{s\text{-wall}}$	contact shear stiffness of wall–particle
N	number of load cycles
N_c	total number of contacts
$\mathbf{N}_{ij}$	contact normal force tensor
$\mathbf{n}$	unit normal vector
$\mathbf{S}_{ij}$	contact shear force tensor
$\mathbf{t}$	vector perpendicular to $\mathbf{n}$
Δh	shear displacement
ε_s	shear strain
θ_a	major principal directions of contact anisotropy
θ_n	major principal directions of contact normal force anisotropy

θ_s	major principal directions of contact shear force anisotropies
μ	inter-particle friction coefficient
σ_{cyc}	maximum cyclic stress
σ_n	normal stress
σ'_3	lateral confining stresses

1. Introduction

The lack of capacity of current tracks in many parts of Australia to support increasingly heavier and faster trains is a concern in national railway practices. As a result of excessive track degradation, the Australian rail industry invests a very large budget in frequent track repair and maintenance operations, apart from significant ground-improvement efforts where soft and saturated subgrade poses challenges (Indraratna *et al.*, 2011a). Ballast is a free-draining granular medium designed as a load-bearing layer in rail tracks; its main functions are: (a) transmitting induced train loads to the underlying layers at a reduced and acceptable level of stress, (b) providing lateral resistance and (c) facilitating drainage for tracks (Indraratna *et al.*, 2011b; Selig and Waters, 1994; Tutumluer *et al.*, 2012). Since ballasted tracks have minimum lateral support, the lateral confining pressure must be increased to control lateral stability (Indraratna *et al.*, 2016; Ngo *et al.*, 2017a; Remennikov and Kaewunruen, 2008).

In addition to the repetitive wheel loads, track structures are often subjected to impact forces induced by wheel and rail irregularities (e.g. wheel-flats, out-of-round wheels, rail corrugation, dipped rails, defective rail welds, insulation joints and expansion gap between two rail segments), whose magnitude depends on the type/nature of the wheel or rail imperfections and on the dynamic track response (Correia, 2001; Le Pen *et al.*, 2014; Mishra *et al.*, 2014). Impact forces can also occur at stiffness transition zones, such as bridge approaches, tunnels and road crossings, where ballasted tracks merge into concrete slabs or vice versa, resulting in exacerbated track deterioration and more frequent maintenance operations (Indraratna, 2016; Li and Davis, 2005).

Geosynthetics have successfully been used for a variety of geotechnical engineering purposes including the reinforcement of retaining walls, embankments and slopes, leachate drainage, stiffener for sandy subgrade to increase vehicle mobility and to assist in the construction of roads (Arulrajah *et al.*, 2015; Brown *et al.*, 2007; Ferreira *et al.*, 2015, 2016a; Halvordson *et al.*, 2010; Hosseinpour *et al.*, 2016; McDowell *et al.*, 2006; Ngo *et al.*, 2017b; Sitharam and Sireesh, 2004; Voottipruex *et al.*, 2010; Warren *et al.*, 2010, among others). Geosynthetics have been applied in new rail tracks and track rehabilitation for approximately three decades, but they have not been widely accepted as a common component to be consistently added throughout the railway structure (Arulrajah *et al.*, 2009; Indraratna *et al.*, 2013; Ngo *et al.*, 2016a). This is partly due

to the interference caused by geosynthetics during maintenance operations. The installation of synthetic inclusions (i.e. geogrids, geocomposites, geocells, rubber mats) at the interface between the ballast and subballast layers has been proved as an effective approach to mitigate ballast degradation and improve track longevity (Amsler, 1986; Raymond, 2002). To date, very limited fundamental research has been carried out to explain the effect of geosynthetic inclusions on the breakage of ballast under cyclic and dynamic impact loads (Indraratna *et al.*, 2011b, 2017; Kaewunruen and Remennikov, 2011). Recently, evaluation of the track performance was carried out at the University of Wollongong (UOW) using extensive instrumentation for identifying ballast degradation and deformation, and further extended by comprehensive field measurements at the Singleton track (Indraratna *et al.*, 2014a).

The discrete-element method (DEM) introduced by Cundall and Strack (1979) has been increasingly used to study micro-mechanical characteristics of granular materials (Cui and O'Sullivan, 2006; Huang *et al.*, 2009; Indraratna *et al.*, 2005; McDowell and Li, 2016; Ngo *et al.*, 2016b, among others). Chen *et al.* (2015) applied the DEM to simulate a box test of geogrid-reinforced ballast under confined and unconfined conditions. They reported that the geogrid reinforcement effectively prevents ballast displacements. Han *et al.* (2011) used the DEM to model geogrid-reinforced embankments and reported that the geogrid-reinforced pile-supported embankment experienced up to 50% less total settlements than those for unreinforced cases. These simulations did not properly capture the irregularly shaped aggregates where either only two-ball clumps or circular balls were simply used to model particles. It is noted that most of the previous studies have been conducted to assess the behaviour of geosynthetic-reinforced soils and there have been only limited attempts to study geogrid-reinforced ballast either in the laboratory or through numerical modelling (Biabani *et al.*, 2016a; Ferreira *et al.*, 2016b; Jeon, 2010; Lopes *et al.*, 2014; Sitharam and Sireesh, 2004).

Tutumluer *et al.* (2012) adopted an aggregate image aided DEM modelling approach to study the interaction and inter-lock mechanisms between geogrids and aggregates. Results of this study showed that the use of geogrids increased the shear strength of the aggregate assembly by constraining the movement of aggregates in the shear zone. Huang *et al.* (2009) used the DEM to study the effect of coal dust fouled railway ballast and concluded that when ballast samples were fouled, the shear strength significantly decreased. Goodhue *et al.* (2001) investigated the role of geosynthetics in increasing the strength of foundry sands using a series of direct shear and pullout tests and found that typical interface friction angles ranged from 25° to 35°, with efficiencies ranging between 0.5 and 0.9, while the interaction coefficients derived from the pullout tests ranged from 0.2 to 1.7. Tatlisoz *et al.* (1998) studied the interaction between reinforcing geosynthetics and soil–tyre chip

mixtures and the results indicated that embankments can potentially be constructed with steeper slopes and a smaller volume of material when soil–tyre chip fill is used, while providing greater resistance against lateral sliding and foundation settlement. McDowell *et al.* (2006) applied the DEM to study the interaction between geogrids and ballast aggregates and reported that the ratio of the geogrid aperture size to the particle diameter of around 1.4 yields optimum results in terms of peak resistance mobilised at the smallest displacements, as a result of good ballast–geogrid interlock.

This paper presents a study to examine the use of geogrids as an effective measure to control track degradation and enhance track performance under harsh rail environments. A series of large-scale laboratory tests were carried out on geogrid-reinforced ballast under monotonic, cyclic and impact loading conditions. An extensive field-monitoring programme was also undertaken on fully instrumented track sections constructed in Singleton, New South Wales, Australia. Four different geosynthetic materials were installed at the ballast–capping interface in track sections involving different types of subgrade to characterise the effect of geosynthetics in actual railway tracks. Discrete-element modelling was carried out to simulate large-scale direct shear tests on geogrid-reinforced ballast to study the interaction mechanism and interface behaviour between the geogrid and ballast from a micro-mechanical perspective. The DEM model was calibrated and validated by the laboratory data obtained at UOW.

The shearing process imparted using either the direct shear box or a process simulation triaxial apparatus leads to dilation and/or compression of the granular mass. Along the corresponding straining history, particle-to-particle attrition and stress concentration at sharp edges and corners during the deformation process can initiate sudden or gradual degradation. During the real-life passage of trains, in addition to shearing-based degradation and deformation, inevitable particle breakage occurs due to impact forces generated by wheel-flats, rail corrugations and worn rail pads, among others. These effects can be simulated in the laboratory using an appropriate impact testing equipment.

To assess the complete degradation over a given period of time (cycles), the cumulative effects of all types of tests must be essentially analysed and quantified, and this can be conducted by a mathematical process adopting for instance a fractional calculus approach (Sun *et al.*, 2017), which is not discussed within the scope of this paper. The use of geogrids that reduce particle movement invariably decreases the extent of attrition-based degradation, while the particle–geogrid interlock (additional friction) translates to an equivalent internal confining pressure that also assists in reducing grain damage by restricting dilation. In this manner, the extensive review presented in this paper can be considered as an attempt to demonstrate to the readership the comparative and contrasting types

of geotechnical testing that hones our insight regarding ballast deformation, degradation and stabilisation mechanisms, corroborating the specific geomechanical processes occurring in harsh track environments.

2. Laboratory study

2.1 Large-scale direct shear tests of geogrid-reinforced ballast

Experiments were carried out using a large-scale direct shear box which consisted of two square units (300×300 mm). The upper box (100 mm in height) is fixed, while the lower box (100 mm in height) is free to move to apply shear loads, as illustrated in Figure 1. The recommended ballast gradation proposed by Indraratna *et al.* (2011a) with a mean particle size $d_{50} = 35$ mm was used in the study. This ballast grading is consistent with the gradation limits specified in Australian Standard AS 2758.7 (SA, 1996). Ballast aggregates were cleaned, sieved, weighed and then thoroughly mixed following the recommended particle-size distribution, as shown in Figures 2(a) and 2(b). Table 1 presents the grain-size characteristics of the ballast samples used in the laboratory, where the maximum particle size of ballast was selected as $d_{\max} = 40$ mm to minimise the influence of boundary conditions. This satisfies the requirement that the ratio of the size of the shear box to the maximum size of ballast particles is not less than six, so that the sample size effects become negligible (Indraratna *et al.*, 1998; Marsal, 1967). An electric motor with a set of gears was used to control the displacement of the lower box. A predetermined amount of ballast was placed inside the lower shear box and compacted in several layers to achieve the desired density that represented field conditions ($\rho = 15.5$ kN/m³). Two types of geogrids with different aperture sizes (e.g. triaxial-TG1 and biaxial-BG2) were used in the current study (Figures 2(c) and 2(d)), and their physical characteristics and technical specifications are listed in Table 2. Geogrids were placed at the interface between the upper and lower boxes along the shearing direction (Figure 1). Several clamping blocks were used to fix each end of the geogrids to the lower shear box, and then the upper box was filled with ballast. In the field, only a small confining pressure (hence effective normal stress) is exerted by the ballast shoulder (i.e. $\sigma'_3 \leq 40$ kPa), as reported by Indraratna *et al.* (2011a). The experiments were conducted under relatively low effective normal stresses ($25 \leq \sigma_n \leq 75$ kPa) to simulate a realistic track environment (i.e. under low confining pressure), and at a constant shearing rate of 2.75 mm/min (i.e. selected based on technical specifications of the shear box apparatus). Under actual Australian track conditions, the lateral confinement provided by the shoulder ballast is relatively low, approximately 15–50 kPa, as measured by Indraratna *et al.* (2014a, 2014b) at Singleton and Bulli tracks. Therefore, the confining pressure (25–75 kPa) used in this study is considered to be appropriate, representing typical Australian track conditions. The direct shear apparatus, data-acquisition system and measuring

Figure 1. Large-scale direct shear apparatus used in the laboratory

devices (e.g. load cells, dial gauges) were calibrated prior to each test. All tests were conducted up to a shear displacement of $\Delta h = 37$ mm, which corresponds to a shear strain of $\varepsilon_s = 12.3\%$.

Figure 3 presents the stress–strain and compression–dilation curves of ballast with and without geogrid under three normal stresses of $\sigma_n = 27, 51$ and 75 kPa. As expected, the peak shear stresses increase with the applied normal stress. The shear strength drops after the maximum shear stress value is reached, showing the post-peak strain-softening behaviour of dense granular materials. This finding is in agreement with the typical stress–strain behaviour of rockfill materials. In general, the peak shear stresses (i.e. shear strength) of the geogrid-reinforced ballast are greater than those of the unreinforced specimens, which is a result of the interlocking effect occurring at the ballast–geogrid interface, leading to increased mobilised shear strength.

Generally, geogrid-reinforced ballast specimens exhibit less dilation than the unreinforced ones. Geogrids decrease dilation simply because they prevent ballast particles from moving in the horizontal direction by mechanical interlocking; ballast particles at the interface are not pushing others up and they are trapped in the geogrid apertures, creating a

non-displacement boundary condition at the ballast–geogrid interface. This means that the geogrid interlocks well with ballast particles to result in decreased dilation. It is seen that the triaxial geogrid did not perform well compared to the biaxial geogrid; and this is believed to be due to the opening areas of the triangular apertures that did not facilitate an effective interlock with ballast grains. Indeed, if the geogrid apertures are small, they could actually create a slipping plane at the interface and decrease the shear strength of the geogrid–ballast assembly, as discussed earlier by Indraratna *et al.* (2012). It is known that the inclusion of geogrids having appropriate aperture sizes decreases the dilation of geogrid–ballast assemblies (Indraratna *et al.*, 2012; McDowell *et al.*, 2006), among others, which is due to the interlocking between the ballast and geogrid that decreases the freedom of particles to displace, leading to reduced dilation. The influence of the relationship between geogrid aperture and ballast on its dilative behaviour was not examined in this study. However, in a previous study conducted by Indraratna *et al.* (2012), the authors proved that the best geogrid aperture size to optimise the interface shear strength is approximately $1.20 \times d_{50}$.

2.2 Cyclic tests of geogrid-reinforced ballast

A series of cyclic tests was also carried out on ballast with and without the inclusion of a biaxial geogrid

(a)

(b)

(c)

(d)

(e)

Figure 2. Particle-size distributions of ballast and materials used in the study: (a) particle-size distributions of ballast tested in the laboratory; (b) sieving and drying ballast; (c) triaxial geogrid, TG1; (d) biaxial geogrid, BG2; (e) biaxial geogrid, BG3

(BG2: 40 mm × 40 mm) using a novel track process simulation apparatus (TPSA) (800 mm long, 600 mm wide and 600 mm high), as shown in Figure 4. The equipment is large enough to accommodate a unit cell that represents the length of a typical sleeper ($L = 800$ mm) and the distance between two adjacent sleepers ($B = 600$ mm). A 150 mm thick layer of dry gravel and sand was used as subballast, which was compacted to a representative field unit weight of about 19.5 kN/m³. A geogrid sample was placed onto the subballast layer, which was then covered by ballast compacted to a field unit weight of 15.5 kN/m³. Ballast aggregates were placed in multiple layers to a total thickness of 300 mm, each of which was compacted with a hand-held vibratory hammer to achieve the desired unit weight.

To mimic the lateral confinement in the field, which in a real track is generated by the weight of crib and shoulder ballast, a minor principal stress ($\sigma_3 = 15$ kPa) was applied to the vertical walls of the TPSA by way of horizontal jacks. To simulate a realistic plane strain condition along the long straight section of track, any lateral movement of the vertical walls in the direction of the intermediate principle stress (σ_2) was prevented by locking the castors (i.e. $\epsilon_2 = 0$). To carry out the test, initially, a monotonic displacement-controlled load was applied at a rate of 1 mm/min until the mean level of cyclic deviator stress of $\sigma_1 = 210$ kPa was achieved. Then, a cyclic load was applied by a servo-dynamic hydraulic actuator at a frequency of $f = 15$ Hz with a maximum cyclic stress of $\sigma_{cyc} = 420$ kPa to simulate an Australian train travelling at

Table 1. Grain-size characteristics of ballast

Test type	Particle shape	$d_{\max}$: mm	d_{10} : mm	d_{30} : mm	d_{50} : mm	d_{60} : mm	C_u	C_c	Size ratio
Ballast gradation	Highly angular	40	16	28	35	39	2.4	1.3	5.7

Table 2. Physical and mechanical properties of geogrids

Properties	Data	Data	Data
Structure	Triaxial geogrid, TG1	Biaxial geogrid, BG2	Biaxial geogrid, BG3
Unit weight	562 g/m ²	420 g/m ²	250 g/m ²
Aperture shape	Triangle	Square	Square
Open area	65.7%	78.9%	62.5%
Geogrid mesh size	37 mm	40 × 40 mm	39 × 39 mm
Rib thickness (MD/CMD)	2 ^a /2 ^a mm	2.2 ^a /1.3 ^a mm	0.9 ^a /0.9 ^a mm
Tensile strength at 2% strain	7.5 kN/m	10.5 kN/m	16 kN/m
Tensile strength at 5% strain	16.3 kN/m	20.4 kN/m	32 kN/m
Peak tensile strength	24 ^b kN/m	30 ^b kN/m	40 kN/m
Strain at peak	11%	13%	8%

^aASTM D4885-01 (ASTM, 2011a)^bASTM D6637-11 (ASTM, 2011b)

approximately 80–90 km/h. All tests were carried out up to $N = 500\,000$ cycles, which is enough to capture the long-term performance of ballasted tracks (Biabani *et al.*, 2016b; Indraratna *et al.*, 2013).

The vertical settlements of ballast were measured by settlement pegs, excluding any deformation of the subballast layer. Figure 5(a) shows the variation in the vertical settlement (S) of ballast with and without the inclusion of geogrid against the number of load cycles. Generally, at a given number of cycles, the geogrid-reinforced ballast experiences less settlement than the unreinforced assembly, which can be justified by the mechanical interlock at the ballast–geogrid interface that minimises the deformation. It is seen that ballast settles rapidly as soon as the loading cycles start, but the rate of settlement decreases once it reaches a relatively ‘stable’ zone after a certain number of cycles (i.e. 300 000 cycles). This indicates that ballast aggregates undergo considerable rearrangement and densification during the initial load cycles (Indraratna *et al.*, 2011b; Rujikiatkamjorn *et al.*, 2012), but after achieving a threshold compression, any further loading would cause subsequent settlement.

The lateral deformations of the ballast assembly with and without the inclusion of the geogrid were measured by 16 electronic potentiometers (Figure 4) and the recorded data are presented in Figure 5(b). It can be observed that the inclusion of the geogrid significantly decreases the lateral displacements of ballast during cyclic loading. Indeed, when ballast particles are compacted over the geogrid, they penetrate and project through the geogrid apertures, creating a strong mechanical interlock (Ngo *et al.*, 2015; Qian *et al.*, 2013). This interlock

enables the geogrid to work as a presumable non-horizontal displacement boundary that prevents the ballast from lateral displacement.

The effect of the geogrid in reducing the vertical stresses transmitted to the underneath subballast layer is presented in Figure 6, where the stresses at the ballast–subballast interface with and without the geogrid inclusion are compared. As expected, the inclusion of the geogrid results in lower stress values when compared to those for the unreinforced ballast assembly. This can be justified by the fact that, through a better locking at the ballast–geogrid interface, the geogrid decreases the maximum pressure at the ballast layer resulting in reduced ballast breakage. In fact, by comparing the amount of ballast breakage after cyclic tests for the unreinforced and geogrid-reinforced ballast assemblies (i.e. using the ballast breakage index (BBI) introduced earlier by Indraratna *et al.* (2005)), the effect of the geogrid can also be proved. By sieving the ballast before and after every test, the amount of ballast breakage can be quantified based on the differences in the particle-size distribution curves. The values of BBI for unreinforced and geogrid-reinforced ballast samples were determined as 0.125 and 0.103, respectively. It is noted that the primary effect of the geogrid is to interlock with ballast particles creating a stiffened zone and preventing ballast from moving horizontally, while its effect in reducing particle degradation could be marginal.

2.3 Impact tests of geogrid-reinforced ballast

The effectiveness of a biaxial geogrid, BG3 (Figure 2(e)) in attenuating the ballast deformation and degradation under impact loading was evaluated using a high-capacity

Figure 3. Measured stress–strain behaviour of ballast with and without geogrid inclusion: (a) shear stress against shear strain; (b) normal strain against shear strain

drop-weight impact testing apparatus developed at UOW (Kaewunruen and Remennikov, 2010). The equipment consists of a 5.81 kN weight free-fall hammer that can be released from a maximum height of 6 m, with an equivalent maximum drop velocity of 10 m/s (Figure 7(a)). The hammer is attached to rollers and is guided through low-friction runners on vertical steel columns fixed to a reinforced concrete floor. The transient impact forces and accelerations are recorded by a dynamic load cell and a piezoelectric accelerometer, respectively, which are mounted on the free-fall hammer and connected to an automatic data-acquisition system.

Figure 7(b) presents the schematic illustration of a typical test sample, where a geogrid was installed at the interface between the subballast and ballast layers. To simulate a low lateral confining pressure in the field, test samples were confined in a cylindrical rubber membrane which was thick enough ($t=7$ mm) to prevent piercing by sharp ballast aggregates

under high-impact loads. A 150 mm thick layer of dry gravel and sand (subballast layer) was compacted to an initial unit weight of 18.8 kN/m^3 . The ballast was then compacted over the subballast to a representative field unit weight (15.3 kN/m^3) in three layers of 100 mm thickness. The different ballast layers were distinguished through colour coding to allow the assessment of ballast degradation with depth (Figure 7(c)). An electric vibratory hammer was used to compact the subballast and ballast materials and a rubber pad was installed underneath the vibratory hammer to minimise the particle breakage during compaction. A steel top plate was placed over the ballast particles to receive the load from the free-fall hammer, as shown in Figures 7(b) and 7(d). While the use of the steel plate may reasonably reproduce the effect of the steel rails in the field, the effect of the sleeper has not been considered in the current study. Tests were conducted with and without geogrid reinforcement to evaluate the role of the geogrid in the attenuation of impact-induced damage. The placement position of the geogrid was altered within the test sample (i.e. either at the ballast–subballast interface or at 100 mm height from the base of the ballast layer) to assess its possible effect on the ballast behaviour.

The drop hammer was raised mechanically to a drop height of 150 mm (Figure 7(d)) and then released by an electronic quick release system. The drop height was selected to produce dynamic stresses representative of typical wheel-flats in the field. As reported by Indraratna *et al.* (2010) based on the results of a field study on an instrumented track at Bulli (New South Wales, Australia), large dynamic impact stresses can be transmitted to the ballast by wheel or rail imperfections, with a peak stress of 415 kPa being detected as a result of a wheel-flat of a 25 t axle load coal train. The impact stresses simulated in the current study, with magnitudes of about 650 kPa in the first impacts, aimed to account for higher-impact loads that can be generated by increasingly heavier and faster trains having wheel irregularities. For data-recording purposes, an automatic triggering was enabled using the impact loading signal obtained during the hammer release and the sampling frequency was set to 50 000 Hz. After each blow, the vertical and lateral strains of the test samples were manually measured at strategic locations. The tests were discontinued after 12 blows due to the attenuation of ballast strains. To characterise the extent of particle degradation (breakage) upon impact loading (Figure 7(e)), the three ballast layers were individually sieved and the change in gradation was obtained. The particle breakage was then quantified using the parameter BBI introduced originally by Indraratna *et al.* (2005).

Figure 8(a) shows the typical impact force–time histories recorded in the first and last impact blows of one representative test. Distinct types of force peaks can be clearly identified, that is multiple instantaneous sharp peaks followed by a gradual peak of lower magnitude and longer duration.

Figure 4. Details of TPSA used for cyclic tests for geogrid-reinforced ballast: (a) TPSA; (b) data-acquisition logger; (c) installation of geogrid, ballast and instrumentations; (d) geogrid-reinforced ballast before testing

These peak forces are usually designated as P_1 and P_2 , respectively (Jenkins *et al.*, 1974). P_1 peaks represent a quasi-instantaneous reaction of the sample to the impact load. These forces are attributed to the inertia of the top plate resisting the downward movement of the free-fall hammer and the compression of the contact zone between the drop hammer and the sample top plate. The multiple P_1 peaks occur when the drop hammer is vertically unrestrained so that after the first blow it rebounds and hits the sample again. The effect of these forces is generally filtered out by the load assembly and, hence, they are not considered to have a significant influence on the ballast degradation (Frederick and Round, 1985). On the other hand, the force P_2 prevails over a longer duration and its occurrence is associated with the mechanical resistance of ballast against impact loading, resulting in its significant compression. Thus, P_2 forces are of greater relevance in the assessment of track deterioration (e.g. Rochard and Schmid, 2004). According to the British Rail Safety and Standards Board (BRSSB, 1995), P_2 forces should not exceed 322 kN to ensure track safety.

The evolution of P_2 along the different tests is plotted in Figure 8(b), which indicates that these forces increase

progressively throughout the repeated impact blows. Indeed, with increasing number of blows, the ballast develops a denser assembly due to the rearrangement/reorientation and breakage of aggregates, which offers higher inertial resistance leading to higher P_2 values. Figure 8(b) also shows that the peak forces P_2 do not vary significantly in the tests conducted with and without the geogrid inclusion.

Figure 9 compares the permanent strain response of ballast in the tests conducted on unreinforced and geogrid-reinforced samples. As expected, both the axial (Figure 9(a)) and radial (Figure 9(b)) strains increase with the successive impacts. Higher increments of deformation are obtained at the initial stage of impact loading, due to the rearrangement and corner breakage of aggregates. It is evident from Figure 9 that, although the provision of a geogrid at the ballast–subballast interface can considerably reduce the ballast permanent strains, in comparison with those for the unreinforced case, higher efficiency can be achieved by installing the geogrid at a height of 100 mm from the base of the ballast layer. As previously mentioned, the main principle of the geogrid reinforcement is to provide better interlocking that attenuates the lateral spreading of ballast, thereby significantly reducing

Figure 5. Measured deformation of ballast with and without geogrid inclusion: (a) vertical settlement; (b) lateral displacement

its vertical settlement. The reduction in ballast strains as the location of the geogrid is changed from the ballast–subballast interface to 100 mm above the subballast may be justified by the enhanced ballast–geogrid interaction, as the particles above and below the geogrid can penetrate its apertures in contrast to when the reinforcement is installed directly above a dense subballast layer.

The values of the BBI measured after the tests for each of the three individual layers (100 mm high) are summarised in Table 3. The results confirm that particle degradation is more pronounced in the top ballast layer, where higher impact-induced stresses are generated. Ballast reinforced with geogrid experiences considerably less breakage in comparison with the unreinforced sample, thus confirming the beneficial effect of the geogrid in reducing impact-induced degradation. Similar to what was observed in terms of permanent deformations, the

provision of the geogrid within the ballast layer (i.e. 100 mm above the subballast) resulted in the highest ballast performance (i.e. the lowest average BBI value) due to better interlocking and increased ballast confinement. The type of test simulating impact-induced degradation carried out in the laboratory may not perfectly model the rail conditions, similarly to the cylindrical triaxial test that suffers from the same limitation because of equal lateral stresses that are hardly representative of field conditions. Notwithstanding, the stress–strain behaviour derived from this test has been successfully employed over recent decades. The current test belongs to a similar category but has a low confinement provided by a relatively thick rubber membrane which supports the ballast and subballast materials. The main purpose of the testing was to demonstrate how the geogrids can effectively be used to mitigate ballast deformation and degradation under successive impact loads.

Figure 6. Measured stress distribution underneath ballast layer with and without geogrid inclusion

3. Use of geosynthetics in Singleton track

3.1 Track instrumentation and monitoring

An extensive field trial was carried out on fully instrumented track sections constructed near the city of Singleton, New South Wales (Australia), to investigate the potential benefits of artificial inclusions, such as geosynthetics and shock mats on the performance of ballasted rail tracks (Indraratna *et al.*, 2014a). These experimental track sections were built on different types of subgrade soils and various types of geosynthetics were installed at the ballast–capping interface, including: (a) geogrid 1 (aperture size = 44×44 mm, peak tensile strength = 36 kN/m), (b) geogrid 2 (aperture size = 65×65 mm, peak tensile strength = 30 kN/m), (c) geogrid 3 (aperture size = 40×40 mm, peak tensile strength = 30 kN/m) and (d) geocomposite reinforcement formed by a biaxial geogrid (aperture size = 31×31 mm, peak tensile strength = 40 kN/m) and a non-woven polypropylene geotextile (mass per unit area = 150 g/m^2 , thickness = 2.9 mm). A layer of shock mat was placed underneath ballast on a concrete bridge deck to minimise the degradation of ballast. Three field-experimental sites, each 500 m long, were instrumented along the proposed Singleton track constructed by the Australian Rail Track Corporation (ARTC), where the different geosynthetics were installed according to the recommended UOW layouts as examined and accepted by ARTC (Indraratna *et al.*, 2014a). This proposed Singleton track up to 8 km in length forms one of the world's longest trial tracks of an operating heavy haul railway for evaluating performance on the basis of track degradation and deformation (Figure 10(a)). The track monitoring instruments including electronic pressure cells, settlement pegs and strain gauges were linked to a data-logging system, as presented in Figures 10(b)–10(f). The subsurface investigation record was obtained prior to the commencement of the work. Details of the track configurations, types of subgrades, field measurements, data records and testing

programme were presented earlier by Indraratna *et al.* (2014a, 2014b).

3.2 Vertical settlement of ballast

The vertical settlement (S_v) of ballast along approximately 300 000 load cycles is presented in Figure 11. When the results for sections on similar subgrades are compared, vertical deformations of the reinforced sections are 15–30% lower than those without reinforcement. This pattern is similar to that observed in the laboratory (Brown *et al.*, 2007; Indraratna *et al.*, 2013; Shin *et al.*, 2002), and this is mainly attributed to the interlocking effect between ballast particles and geogrids, as discussed earlier. These results also indicate that the relationship between the settlement of ballast and the number of load cycles (N) is non-linear, regardless of how the track was reinforced. When the results for sections with similar geogrids are compared, it is apparent that the ability of the geogrid reinforcement to reduce ballast deformation is generally higher for softer subgrades (i.e. low track substructure stiffness). Such an observation is in agreement with the results of the full-scale laboratory tests presented by Ashmawy and Bourdeau (1995). Moreover, of the four types of geosynthetics used, the geogrid 3 (at Section 4) performed most effectively. Although the stiffness of this geogrid was equal to or lower than that of the others, its aperture size ($40 \text{ mm} \times 40 \text{ mm}$) enabled better interlocking between the ballast particles and the grid. This finding is consistent with the criteria for optimum geogrid aperture size proposed by Indraratna *et al.* (2012).

The accumulated longitudinal (ϵ_l) and transverse (ϵ_t) strains after 100 000 and 300 000 load cycles, as measured by the strain gauges attached to the geogrids and located below the edges of the sleepers, are given in Table 4 (Indraratna *et al.*, 2014a). Here, most of the permanent strains in the geogrids in

Figure 7. Details of the drop-weight impact testing apparatus and test samples: (a) drop-weight impact testing apparatus (after Kaewunruen and Remennikov, 2010); (b) schematic diagram of a test sample; (c) preparation of a test sample; (d) test sample before testing; (e) particle degradation upon impact testing. GGR, geogrid reinforcement

both directions developed when the track was being constructed, particularly when the ballast was being placed. In general, the strains did not significantly change with the number of load cycles. As shown in Table 4, the transverse strains were generally larger than the longitudinal strains, probably due to the higher level of longitudinal restraint relative to the transverse direction. The values of ε_l and ε_t also appeared to be mainly influenced by the deformation of the subgrade.

4. Discrete-element modelling of geogrid-reinforced ballast

The DEM developed by Cundall and Strack (1979) has been widely used to examine the shear behaviour of granular materials (Bhandari *et al.*, 2015; Han *et al.*, 2011; Lobo-Guerrero and Vallejo, 2005; McDowell *et al.*, 2006;

Ngo and Indraratna, 2016; Ngo *et al.*, 2015; O'Sullivan *et al.*, 2008; Tutumluer *et al.*, 2012, among others). Pioneering works on the use of DEM to model ballasted tracks were initiated by McDowell and Bolton (1998) at Cambridge University, and then by Lu and McDowell (2006) at Nottingham University, Huang and Tutumluer (2011) at University of Illinois and both Indraratna *et al.* (2014b) and Ngo *et al.* (2017c) at the University of Wollongong, among others. In DEM, the force–displacement law derives from the contact forces which apply on particles that are in contact with the relative deformation between them. Irregularly shaped grains are modelled by connecting a number of spherical balls of different sizes and positions together (Itasca, 2014; Ngo *et al.*, 2016a, 2016b). In these studies, monotonic and cyclic tests of railway ballast under varying confining pressures were simulated. In the cyclic

Figure 8. Impact force response: (a) typical impact force variation with time; (b) variation of impact force P_2 with the number of blows for unreinforced and geogrid-reinforced ballast samples

Figure 9. Permanent strain behaviour of ballast with and without geogrid inclusion: (a) axial strains; (b) radial strains

triaxial test simulations, a servo-control was applied to the top and bottom walls to maintain the required loads (Lu and McDowell, 2010). Following Lackenby *et al.* (2007), ballast assemblies under confining pressures ranging from 10 to 240 kPa were modelled and sinusoidal load pulses were applied to the samples with a minimum deviatoric stress of 45 kPa for each test. The results of load–deformation responses of ballast obtained from the DEM simulations were comparable to those measured experimentally. Details of the modelling procedure and results can be found in Lu and McDowell (2010).

The recent DEM analysis of ballast reinforced by geogrids (i.e. triaxial or biaxial geogrids) conducted at the University of Wollongong is shown in Figure 12. A library of different ballast particles was generated, as illustrated in Figure 12(a). A triaxial geogrid (TG1) having an aperture size of 37 mm and a biaxial geogrid with 40 mm $\times$ 40 mm apertures, similar to those used in the laboratory, was modelled in DEM by bonding small spherical balls together (Figure 12(b)). These balls were connected by contact and parallel bonds that represent the tensile strength of the actual geogrids. A large-scale

direct shear box (300 mm long $\times$ 300 mm wide $\times$ 200 mm high) was simulated in DEM with rigid walls and then used to model fresh ballast with the geogrid reinforcement, as shown in Figure 13.

The micromechanical parameters of ballast were selected by comparing the shear stress–strain responses obtained from DEM simulations with those measured by direct shear tests in the laboratory. The contact stiffness (k_n , k_s) and inter-particle coefficients of friction (μ) used to simulate the ballast were varied interactively, until the predicted shear stress against shear–strain curves agreed reasonably well with the laboratory data. A similar approach was used for determining the input parameters for the geogrids, based on the results of a series of tensile tests (Ngo *et al.*, 2014). The set of micromechanical parameters selected to model the ballast and the geogrids in the current analysis is shown in Table 5.

4.1 Shear stress–strain analysis

The simulations of direct shear tests were carried out on reinforced and unreinforced ballast assemblies subjected

Table 3. Assessment of BBI after impact tests

Test	BBI – bottom	BBI – middle	BBI – top	BBI – average
Without geogrid	0.155	0.109	0.187	0.150
BG3 at the interface	0.111	0.122	0.190	0.141
BG3 at 100 mm height	0.091	0.143	0.155	0.130

Figure 10. Use of geogrids to reinforce ballast at the Singleton track (Indraratna *et al.*, 2014a): (a) location of the Singleton track; (b) placing geogrids and installation of strain gauges; (c) placing ballast on top of geogrid; (d) installation of a pressure cell underneath sleepers; (e) displacement monitoring frame mounted on base; (f) data logger module connected to mobile personal computer

to a normal stress of $\sigma_n = 51$ kPa, similar to one of the values applied in the laboratory. During the shearing process, the lower part of the shear box was moved at a velocity of 8.35×10^{-5} mm/time step to a horizontal displacement of $\Delta h = 37$ mm, while the upper part of the shear box was

fixed, and the shear forces and corresponding strains were recorded.

Figure 14 shows comparisons of the shear stress–strain curves of unreinforced and reinforced ballast obtained from DEM

Figure 11. Vertical deformation of ballast for: (a) soft embankment; (b) hard rock (data sourced from Indraratna *et al.* (2014a))

Table 4. Typical values of accumulated longitudinal and transverse strains in geogrids (data source from Indraratna *et al.* (2014a))

At 100 000 load cycles				
Section	1	2	3	4
ϵ_l : %	0.8	0.7	0.9	0.6
ϵ_t : %	0.9	1.6	0.8	0.8
At 300 000 load cycles				
Section	1	2	3	4
ϵ_l : %	0.8	0.7	1.0	0.7
ϵ_t : %	1.0	1.6	1.0	0.9

simulations and laboratory measurements. It is seen that the predicted stress–strain responses match reasonably well with the laboratory data, thus indicating that the proposed DEM model can capture the shear stress–strain behaviour of ballast with and without the geogrid inclusion. The strain-softening behaviour of ballast was also observed in all simulations. The role of the geogrids (both for triaxial and biaxial grids) in increasing the shear strength of ballast can be proved by comparing the results for reinforced and unreinforced ballast assemblies. The simulated data also show a sudden reduction in the shear stress at a shear strain of around $\epsilon_s = 3\text{--}7\%$ before picking up the load again, which may be related to the initiation of particle breakage at this level of shear strain.

The strains developed in the geogrids could not be measured in the laboratory due to the complexity of the installation of strain gauges on geogrids and difficulty in protecting them from the damage caused by sharp edges of ballast particles during compaction. In the current DEM analysis, the strains

Figure 12. Modelling ballast aggregates and geogrids in DEM: (a) template ballast particles simulated in DEM; (b) triaxial and biaxial geogrids simulated in DEM

Figure 13. DEM simulations for direct shear tests of geogrid-reinforced ballast: (a) placement of the biaxial geogrid in shear box; (b) DEM model of biaxial geogrid-reinforced ballast in shear box; (c) placement of the triaxial geogrid in shear box; (d) DEM model of triaxial geogrid-reinforced ballast in shear box

in the horizontal shearing direction developed in the geogrids (triaxial and biaxial grids) at a shear strain of $\epsilon_s = 7\%$ are captured and presented in Figure 15. It is observed that the strains developed non-uniformly across the geogrids and the magnitude of strain is dependent on the interlock occurring between the geogrids and ballast aggregates.

4.2 Micromechanical analysis

The load transfer in a ballast assembly depends on the contact orientations where the applied load is transmitted by an interconnected network of force chains at contact points (Oda and Iwashita, 1999). Under shear loads, the evolution of contact forces in ballast assemblies takes place and consequently, changes are induced in the coordination numbers (i.e. load-carrying contacts) and orientation of contacts. The second-order density distribution tensor introduced by Rothenburg and Bathurst (1989) was used to examine the anisotropy of

Figure 14. Comparisons of shear stress against strain between DEM simulation and laboratory test data for unreinforced and reinforced ballast

contact forces for unreinforced and reinforced ballast assemblies. These tensors were incorporated into the DEM models and are given as follows

$$1. \quad \mathbf{F}_{ij} = \int_0^{2\pi} E(\theta) \mathbf{n}_i \mathbf{n}_j d\theta = \frac{1}{N_c} \sum_{k=1}^{N_c} \mathbf{n}_i^k \mathbf{n}_j^k$$

$$2. \quad \mathbf{N}_{ij} = \frac{1}{2\pi} \int_0^{2\pi} \frac{\bar{f}_n(\theta)}{\bar{f}_0(\theta)} \mathbf{n}_i \mathbf{n}_j d\theta = \frac{1}{N_c} \sum_{k=1}^{N_c} \frac{f_n^k}{\bar{f}_0} \mathbf{n}_i^k \mathbf{n}_j^k$$

$$3. \quad \mathbf{S}_{ij} = \frac{1}{2\pi} \int_0^{2\pi} \frac{\bar{f}_s(\theta)}{\bar{f}_0(\theta)} \mathbf{t}_i \mathbf{n}_j d\theta = \frac{1}{N_c} \sum_{k=1}^{N_c} \frac{f_s^k}{\bar{f}_0} \mathbf{t}_i^k \mathbf{n}_j^k$$

where $\mathbf{F}_{ij}$, $\mathbf{N}_{ij}$ and $\mathbf{S}_{ij}$ 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

Table 5. Micromechanics parameters of ballast and geogrid adopted for DEM simulations

Parameter	Ballast	Geogrids
Particle density: kg/m ³	2700	850
Coefficient of friction	0.85	0.55
Contact normal stiffness, k_n : N/m	0.82×10^8	2.64×10^7
Contact shear stiffness, k_s : N/m	0.41×10^8	1.32×10^7
Contact normal stiffness of wall-particle, k_{n-wall} : N/m	2.5×10^8	2.5×10^8
Shear stiffness of wall-particle, k_{s-wall} : N/m	1.25×10^8	1.25×10^8
Parallel bond radius multiplier, r_p	—	0.5
Parallel bond normal stiffness, k_{np} : kPa/m	—	5.68×10^8
Parallel bond shear stiffness, k_{sp} : kPa/m	—	5.68×10^8
Parallel bond normal strength, σ_{np} : MPa	—	358
Parallel bond shear strength, σ_{sp} : MPa	—	358

Figure 15. Contour strains developed across the geogrid in horizontal shearing direction at the shear strain of $\varepsilon_s = 7\%$, $\sigma_n = 51$ kPa: (a) biaxial geogrid; (b) triaxial geogrid. A full-colour version of this figure can be found on the ICE Virtual Library (www.icevirtuallibrary.com)

functions; f_n^k and f_s^k are the contact normal force and shear force, respectively; $\mathbf{n} = (\cos\theta, \sin\theta)$ is the unit normal vector, $\mathbf{t} = (-\sin\theta, \cos\theta)$ 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

$$4. \quad \bar{f}_0 = \frac{1}{2\pi} \int_0^{2\pi} \bar{f}_n(\theta) d\theta = \frac{1}{N_c} \sum_{k=1}^{N_c} f_n^k$$

The force fabric is characterised by the distribution of inter-particle contact orientations, which can be described by the

following Fourier series approximations proposed by Rothenburg and Bathurst (1989), as given below

$$5. \quad E(\theta) = \frac{1}{2\pi} [1 + a \cos 2(\theta - \theta_r)]$$

$$6. \quad \bar{f}_n(\theta) = \bar{f}_0 [1 + a_n \cos 2(\theta - \theta_n)]$$

$$7. \quad \bar{f}_s(\theta) = \bar{f}_0 [-a_s \cos 2(\theta - \theta_s)]$$

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

4.3 Variations of coordination number, C_n and contact anisotropy

The micromechanical analysis presented herein focuses on the evolution of coordination numbers, C_n and contact orientations of ballast particles with and without geogrid inclusion. Figure 16 shows the evolution of the coordination number, which is the average number of contacts per particle of ballast, during the shearing of unreinforced and geogrid-reinforced ballast. It is noted that the initial C_n values for all ballast assemblies are nearly the same (i.e. about $C_n = 4.5$) since the initial void ratios of reinforced and unreinforced ballast assemblies are similar. Once the shearing begins, the value of C_n for the unreinforced ballast keeps almost unchanged, whereas the C_n values for the geogrid-reinforced ballast assemblies increase considerably up to about $C_n = 7$, followed by a reduction to the initial value of $C_n = 5$ toward the end of shearing. The increase in the coordination numbers due to the inclusion of the

Figure 16. Variations of coordination numbers, C_n of unreinforced/reinforced ballast during the shearing progress

geogrid may be attributed to the interlocking between the geogrid and surrounding ballast grains that results in an increased number of contact forces.

Besides the coordination numbers, the orientation of contact normal and shear forces (i.e. fabric anisotropy) is also

an important aspect affecting the shear strength of granular materials (Fu and Dafalias, 2011; Ngo *et al.*, 2017c). Figure 17 illustrates the polar histograms of contact orientation, normal and shear force orientations for unreinforced and geogrid-reinforced ballast captured in the DEM simulation at a given shear strain of $\varepsilon_s = 7\%$. The polar histograms of the

Figure 17. Polar histograms of contact and force orientations in the ballast assembly with and without geogrid reinforcement: (a) unreinforced ballast; (b) with triaxial geogrid; (c) with biaxial geogrid

contact forces are determined by counting contact force information at the predefined bin angle, $\Delta\theta = 10^\circ$. It is seen that the contact orientation in the unreinforced ballast assembly is coaxial with the vertical axis, having a principal direction of about $\theta_r = 6^\circ$, while the triaxial and biaxial geogrid-reinforced ballast assemblies show contact orientations of $\theta_r = 12^\circ$ and $\theta_r = 14^\circ$, respectively. The inclusion of geogrids results in an increased number of contacts in both the vertical and horizontal directions and they distribute almost uniformly in all orientations. The normal force orientations of both unreinforced and reinforced ballast exhibit a predominant distribution in the vertical direction, having principal directions of $\theta_n = 16^\circ$, 22° and 24° , respectively. When the ballast is subjected to shearing, contact force chains develop to resist shear and disperse the loads from the surface into the ballast grains. Anisotropies of average contact normal and shear forces grow and rotate considerably during the shearing progress and reach their values of $\theta_s = 18^\circ$, 20° and 23° for unreinforced, triaxial and biaxial-reinforced ballast assemblies, respectively (at the corresponding shear strain of $\varepsilon_s = 7\%$). It is believed that the inherent anisotropy affects the overall shear strength of ballast assemblies, while stresses within the ballast medium are composed of multiple stress chains, which can lead to localised high stress concentrations (Chang and Yin, 2010; Pan and Dong, 1999). Such high stress concentrations may induce crushing of single particles even under relatively low applied stresses. Therefore, understanding the contact force distribution in ballast and its evolution is most beneficial and provides more insight into the orientation of contacts transmitted in ballast assemblies.

5. Conclusion

This paper shows the results of a series of large-scale laboratory tests, field investigation and numerical modelling using the DEM, carried out to evaluate the improved performance of railway ballast reinforced by geogrids. Direct shear test results clearly indicated that the inclusion of geogrids increases the interface shear strength and decreases the deformation of ballast. This was attributed to the interlock occurring between the geogrids and surrounding ballast aggregates at their interfaces. Cyclic tests were conducted using a novel TPSA to study the load–deformation response of biaxial geogrid-reinforced ballast. The measured data indicated that the settlement and lateral displacement of ballast reinforced by the geogrid decreased significantly. The ballast samples experienced a considerable amount of deformation within 100 000 cycles, followed by a gradual increase in settlement up to 300 000 cycles, and then remained relatively stable. Additionally, the inclusion of the geogrid decreased the maximum pressure underneath the ballast layer, resulting in reduced ballast breakage. The role of a biaxial geogrid in improving the deformation and degradation behaviour of ballast under impact loading was also investigated using a high-capacity drop-weight impact testing apparatus. The results showed that the impact forces increase progressively over the repeated blows, which is associated with the densification of the ballast assembly. Higher increments in

the permanent axial and radial strains of ballast were observed during the initial impacts due to the rearrangement and corner breakage of aggregates. Although the installation of a geogrid at the ballast–subballast interface may considerably attenuate the ballast deformation and particle breakage, the highest efficiency was achieved when the geogrid was placed within the ballast layer, at 100 mm height from its base. This occurrence was attributed to the enhanced ballast–geogrid interaction obtained when the particles on both sides of the geogrid could penetrate its apertures, in contrast to when the geogrid was placed directly above a dense/compacted subballast mass.

The results of a comprehensive field-monitoring programme undertaken in Singleton (New South Wales, Australia) to examine the ability of various geosynthetics in improving track stability were discussed. Different types of geosynthetics (i.e. three geogrids and a geocomposite) and shock mats were installed beneath the ballast layer constructed on varying subgrade conditions. The measured data at the Singleton track indicated that the geogrids were more effective in curtailing ballast deformations on the soft subgrade, while the rubber mats were effective when placed above the concrete bridge deck. When the results for sections involving similar subgrades were compared, the vertical deformations of the reinforced sections were found to be 15–30% smaller than those without reinforcement.

DEM simulations were implemented to study the shear stress–strain behaviour of ballast subjected to direct shear loads. Irregularly shaped ballast particles were simulated by connecting many spherical balls together. Similarly, triaxial and biaxial geogrids with different geometries were modelled by bonding numerous small balls together. Appropriate sets of micromechanical parameters to simulate the ballast and geogrids were determined by calibration with the data measured in the laboratory. The micromechanical parameters were then used to simulate the unreinforced and geogrid-reinforced ballast in the direct shear test. For a given normal stress, the DEM simulation captured the shear stress–strain behaviour of ballast and, as expected, the geogrid-reinforced ballast exhibited higher shear strength than the unreinforced ballast. It was observed that the strains developed non-uniformly across the geogrids during the shearing process, and the magnitude of the strain depends on the interlock between the geogrid and ballast particles. The coordination number, C_n , and the contact force orientations were also examined, which provided more insight into the orientation of contacts transmitted in a granular assembly.

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