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Abstract

Ballasted rail tracks are the most important mode of transportation in terms of traffic tonnage serving the needs of bulk freight and passenger movement, but under train loads, the particles degrade due to breakage and the progressive accumulation of external fines or mud-pumping under the subgrade, all of which reduce its shear strength and increase track instability. These actions adversely affect the safety, passenger comfort and efficiency of tracks, as well as enforcing speed restrictions and more frequent track maintenance. In spite of advances in rail track geotechnology, the optimum choice of ballast for track design is still considered critical because ballast degradation is influenced by the amplitude and number of load cycles, particle gradation, track confining pressure and the angularity and fracture strength of individual grains. One of the most effective methods of enhancing track stability and reducing the stresses transmitted to a soft subgrade layer is to increase the stiffness of the overlying granular media. This paper presents our current knowledge of rail track geomechanics, including important concepts/topics related to laboratory testing and computational modelling approaches used to study the load-deformation behaviour of ballast improved with waste tyres, synthetic geogrids and geocells.

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Current Research into Ballasted Rail Tracks: Model Tests and Their Practical Implications

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Current Research into Ballasted Rail Tracks: Model Tests and Their Practical Implications

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Abstract: Transport infrastructure must now perform over the long term because heavy haul transport networks are expected to withstand higher speeds and heavier axle loads. Ballasted rail tracks are the most important mode of transportation in terms of traffic tonnage serving the needs of bulk freight and passenger movement, but under train loads the particles degrade due to breakage and the progressive accumulation of external fines or mud-pumping under the subgrade, all of which reduces its shear strength and increases track instability. These actions adversely affect the safety, passenger comfort and efficiency of tracks, as well as enforcing speed restrictions and more frequent track maintenance. In spite of advances in rail track geotechnology, the optimum choice of ballast for track design is still considered critical because ballast degradation is influenced by the amplitude and number of load cycles, particle gradation, track confining pressure, and the angularity and fracture strength of individual grains. One of the most effective methods of enhancing track stability and reducing the stresses transmitted to a soft subgrade layer is to increase the stiffness of the overlying granular media. The Centre for Geomechanics and Railway Engineering (CGRE) has developed new design and construction concepts for track upgrading by applying theory to practice to enhance track longevity and minimise the maintenance costs. Research conducted at CGRE has shown that understanding the load transfer mechanisms and their effect on ballast breakage are important pre-requisites for decreasing track maintenance costs. This paper presents our current knowledge of rail track geomechanics, including important concepts/topics related to laboratory testing and computational modelling approaches used to study the load-deformation behaviour of ballast improved with waste tyres, synthetic geogrids, and geocells. A sustainable approach is proposed to increase the stability of the ballast layer by integrating it with a cellular rubber membrane fabricated from used tyres. The use of geogrids and energy absorbing rubber mats to improve track performance requires further study before incorporating them into existing design routines catering for future high speed trains and heavier heavy hauls. This paper explains the studies carried out at the University of Wollongong on track infrastructure by highlighting some examples where innovation from theory is put into practice.

Keywords: Ballast; Discrete Element Method; Finite Element Method; Particle breakage; Granular material

Introduction

Along with China, USA, Canada, and India, Australia is also playing a leading role in developing heavy haul railway operations with 3–4 km long trains and axle loads exceeding 35 tonnes that will optimise the efficiency of supply chains in the mining and agricultural sectors. Track structure is divided into superstructure and substructure; the superstructure consists of steel rails, fastening systems, and concrete or timber sleepers (ties) and the substructure consists of ballast, subballast and subgrade. Under moving train loading these components interact to control the efficiency and safety of ballasted railway track.

As the sharp corners of aggregates break and weaker particles are crushed under heavy cyclic loading, differential track settlement is almost inevitable (Selig and Waters 1994), so when tracks are laid on stiff foundations such as hard rock terrains or concrete bridge decks, where large dynamic (impact) loads are encountered, these problems are often exacerbated (Indraratna, Ngo, and Rujikiatkamjorn 2011; Kaewunruen and Remennikov 2009, 2011; Griffin et al. 2015). This in turn, leads to a rapid fragmentation of ballast aggregates which affects the strength and drainage of track. Moreover, the ballast layer is also fouled by an upward migration of subgrade clay fines and a downward migration of coal spilling from wagons (Indraratna et al. 2014a; Tennakoon et al. 2015), all of which seriously affects the drainage capacity of track.

Since ballasted tracks have minimum lateral support, the lateral confining pressure must be increased to control lateral stability (Indraratna, Nimbalkar, and Neville 2014, Indraratna, Nimbalkar, and Rujikiatkamjorn 2014a, 2014b; Lackenby et al. 2007). The behaviour of ballast is also affected by the overall characteristics of this granular mass such as particle size distribution, the void ratio, and its relative density (Indraratna, Sun, and Nimbalkar 2016). While the properties of individual grains of ballast such as size, shape, and angularity govern its degradation under traffic loading, deformation is also influenced by the magnitude of wheel (or axle) load, the number of load cycles, frequency (equivalent to train speed) (Selig and Waters 1994; Sun, Indraratna, and Nimbalkar 2014, 2016), and the impact loads (Indraratna et al. 2014b; Nimbalkar et al. 2012). The magnitude of the impact loads depends on the type and nature of surface imperfections on the wheels and rails, as well as on the track's dynamic response (Indraratna et al. 2014b; Le Pen et al. 2014).

In conventional track design, ballast is regarded as an elastic medium whose degradation and associated plastic deformation is ignored. This problem stems from not understanding the complexity of particle breakage and fouling mechanisms, and not having a proper constitutive model. The constitutive behaviour of granular materials has already been studied by various researchers using the discrete element modelling approach (Ahmed et al. 2015; Huang and Tutumluer 2011; Lu and McDowell 2010; Mirghasemi, Rothenburg, and Matyas 2002; Thornton 2000, Indraratna et al. 2014a); this approach to model the granular media is appealing because it can handle particles of different shapes and sizes.

The use of planar geosynthetic products such as geogrid, geotextile or geocomposite (bonding geogrid with geotextile) has drawn more attention because they are economical and relatively easy to install. In fact several studies have already shown that geogrid reinforcement reduces the settlement and degradation of ballast by providing interlocking that restricts its lateral movement (Indraratna and Salim 2003; Indraratna, Shahin, and Salim 2007; Indraratna et al. 2010). A layer of geogrid and geotextiles placed at the interface between ballast and subballast often gives

encouraging results (Giroud and Han 2004; Shin, Kim, and Das 2002), so much so that recent studies have shown that a geocell (three dimensional, polymeric cells, interconnected at the joint) can provide much better lateral track confinement than planar reinforcement (Indraratna, Biabani, and Nimbalkar 2015; Leshchinsky and Ling 2013).

As train speeds increase the track is often unable to withstand the substantially increase in vibration and the cyclic and impact loads, which is why synthetic energy absorbing mats in rail track foundations has become increasingly popular (Indraratna et al. 2014b). These mats are also called Under Sleeper Pads (USP) or Under Ballast Mats (UBM) depending on whether they are placed under the sleepers and beneath the ballast. These resilient mats are commonly used in railways: (i) to reduce structure-borne vibration and noise and thus improve the vertical elasticity of the track substructure, and (ii) to reduce ballast degradation and improve the stability of the track foundation, and thus increase the service life of rail tracks (Nimbalkar et al. 2012). Rubber tyres have a three dimensional cylindrical structure, so they can be used to stabilise foundations by increasing the bearing capacity and reducing the settlement of transport infrastructure. It is therefore expected that in railroad engineering, a capping layer reinforced with rubber tyres could help to reduce the thickness of the granular layer (i.e. ballast), improve the track bearing capacity, and reduce the frequency of maintenance. However, there is a notable gap between conceptual theories and real-life applications pertaining to the mechanisms of foundations reinforced with rubber tyres.

At present, only limited field studies which quantify the relative performance of combined geosynthetics and shock mats exist (Indraratna et al. 2010; Indraratna, Salim, and Rujikiatkamjorn 2011; Indraratna, Nimbalkar, and Rujikiatkamjorn 2013; Nimbalkar and Indraratna 2016), although large scale experiments combined with full-scale field trials often represent an appropriate strategy for assessing track degradation. In view of this, extensive field trials on sections of instrumented railway track at Bulli and Singleton, New South Wales (NSW) in Australia have been carried out. This paper describes the results of these field trials and a series of large scale laboratory tests supplemented by numerical analyses to assess the performance of ballasted tracks at increased speeds and axle loads; it will also quantify the benefits of using geoinclusions in track.

Laboratory Testing

Large-Scale Triaxial Test on Ballast under Different Frequency Loading

The load frequency of a train is expressed as $f = V/L$, where V is train speed and L is the characteristic length between axles. A typical freight wagon has multiple axles (e.g. four axles in NSW heavy haul) that impart independent load cycles. As the axle distance is much smaller than the bogie distance, the two rear axles of a leading wagon and two front axles of a trailing wagon would generate the maximum frequency (Indraratna, Nimbalkar and Rujikiatkamjorn 2014a; Sun, Indraratna and Nimbalkar 2016). Therefore, considering an axle distance of 2.02 m, frequency f is obtained as $0.138 V$. The influence of train speed on the permanent deformation and degradation of ballast during cyclic loading has been studied using the large scale cylindrical triaxial apparatus which could accommodate a 300 mm diameter by 600 mm high specimen (Figure 1). These test specimens are isotropically consolidated to confining pressures (σ_3') of 10, 30 and 60 kPa, and then frequencies (f) varying from 5 Hz to 60 Hz are selected to simulate train

speeds from about 40 to 400 km/h; maximum cyclic deviator stresses ($q_{max,cyc}$) of 230 and 370 kPa are then used to represent axle loads of 25 and 40 tonnes, respectively.

Figure 2(a&b) presents the variation of axial strain (ϵ_a) versus the number of cycles (N) for different frequencies (f) and load amplitudes ($q_{max,cyc}$) of cyclic loading. For a specific f , the ϵ_a increases with the increase of N , but for different f , ϵ_a increases as f increases at a specific N . Figure 2(a&b) shows there are four regimes of permanent deformation based on the cyclic loading applied: (i) a zone of elastic shakedown with no accumulation of plastic strain, (ii) a zone of plastic shakedown characterised by a steady state response and a small accumulation of plastic strain, (iii) a ratcheting zone with a constant accumulation of plastic strain, and (iv) a plastic collapse zone where plastic strains accumulate rapidly and failure occurs in a relatively short time (Sun, Indraratna, and Nimbalkar 2016). Moreover, three different deformation ranges appeared in response to the loading frequency, namely: Range I-Plastic shakedown at $f \leq 20$ Hz (or $V \leq 145$ km/h), Range II-Plastic shakedown and Ratcheting at $30 \text{ Hz} \leq f \leq 50 \text{ Hz}$ (or $220 \text{ km/h} \leq V \leq 360 \text{ km/h}$), and Range III-Plastic collapse at $f \geq 60 \text{ Hz}$ (or $V \geq 400 \text{ km/h}$) as shown in Figure 2(a&b). Cyclic triaxial data on ballast from Lackenby et al. (2007) indicates that similar regimes of permanent deformation response exist based on the applied stress ratio ($q_{max,cyc}/p'$).

A range of critical frequency is identified as 20-30 Hz for $\sigma_3' = 30$ kPa and 30-40 Hz for $\sigma_3' = 60$ kPa, respectively. Figure 2c shows that critical frequency decreases as particle breakage increases, and ratcheting failure (Range II) of the specimen would occur with significant particle breakage ($BBI > 0.10$) even at a relatively low value of frequency (i.e. $f = 25$ Hz). Three deformation and degradation mechanisms occurred as the increase of frequency (Sun, Indraratna and Nimbalkar 2014). For specimens tested at relatively lower frequencies (i.e. $f \leq 10$ Hz), the plastic strain was most probably influenced by particle reorientation. Some grinding or attrition of asperities occurred (Figure 2d), but as it was not expected to contribute very much to the plastic deformation; however, as f increased to 20 Hz, the axial strain rate attained a maximum value. This may be attributed to the breakage of angular corners or projections as well as particle reorientation. Following this, the permanent deformation rate declined to an insignificant value with N , while the corresponding axial strain stabilised to a relatively steady value that was defined as the plastic shakedown zone. As shown in Figure 2(a&b), when the frequency locates in Range II ($30 \text{ Hz} \leq f \leq 50 \text{ Hz}$) the triaxial specimens were initially unstable and reached plastic shakedown within in 2000 load cycles, but in this plastic shakedown zone, the specimens attained an optimum packing arrangement. Larger particles were unable to slide very much, which meant the specimen produced a resilient vibration that resulted in a stable deformation. The smaller particles located in the void between the larger ones could experience high-frequency vibration. This result could induce a high degree of attrition of asperities for the larger and smaller particles, as shown in Figure 2d. Once this kind of corner degradation has developed to a certain extent, the larger particles will begin to slide again. Moreover, after 20000-100000 load cycles, the fatigue failure of particles commences and some particle splitting takes place through planes of weakness such as microcracks and other flaws (Figure 2d). Both these mechanics could contribute to any further deformation with a constant rate that is defined as the ratcheting zone. At extremely high frequencies, each load application resulted in a progressive increment of axial strain, such that the specimen reached the displacement limit of the actuator and failure occurred within 1000 load cycles. This is defined as plastic collapse. With insufficient lateral confinement, the specimen may have had poorly established contacts and relatively small particle to particle contact areas, so at such high frequency, the coordination number is greatly reduced, which could induce a relatively large scale particle reorientation and

an unstable aggregate skeleton to occur, which elucidates the rapid development of permanent axial strain.

Large-Scale Cubical Test on Ballast Reinforced with a Cellular Rubber Membrane

One of the most effective methods of enhancing track stability and reducing the stresses transmitted to a soft subgrade layer is to increase the stiffness of the overlying granular media. A sustainable approach to increase the stability of the ballast layer is suggested by integrating it with a cellular rubber membrane fabricated from used tyres. The infilled cellular rubber membrane is evaluated as a ‘unit cell’ and tested in the Track Process Simulation Apparatus (TPSA) designed and built at the University of Wollongong to mimic realistic field conditions (Figure 3).

Figure 4a shows the lateral displacement recorded throughout the reinforced and unreinforced tests, and as expected, the lateral displacement of ballast confined by the membrane decreased significantly; in fact this reduction in lateral displacement is more pronounced for larger number of cycles ($N > 10000$ cycles) and can reach up to 20%. The resilient modulus (M_R) is an important property that is routinely adopted for track substructure design. Figure 4b shows the variation of resilient modulus (M_R) with the number of cycles (N); note that M_R generally increases with N , but this increase for reinforced ballast is much larger than for the unreinforced test.

A ballast layer that is usually composed of angular aggregates which sustain and transmit the cyclic stresses applied by moving loads to the substructure, so to examine track performance and longevity, the incidence of ballast breakage must be evaluated. Figure 5 shows the particle size gradations obtained by sieving the ballast at the end of the unreinforced and reinforced tests (i.e. $N=500000$ cycles); note the shift in the gradation of unreinforced ballast, which indicates some breakage, whereas ballast gradation remains relatively unchanged for the reinforced tests. While this observation may not appear to be intuitive, it can be explained in view of the additional confinement and energy absorption capacity the rubber can provide because when aggregates are confined, their lateral spreading is reduced and the energy that would otherwise be transferred to cause ballast breakage is now absorbed as strain energy of the rubber membrane. Visual observations while the load is applied during the tests also confirmed this result. The total extent of breakage can be computed using the procedure proposed by Indraratna, Lackenby, and Christie (2005) adopting the ballast breakage index (BBI). The BBI can be computed based on the changes in particle gradation determined before and after the tests; for the unreinforced test, the BBI is 0.1 and for the reinforced test the BBI=0.04, which is much lower.

Computational Modeling of Ballasted Tracks

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 quasi-static and dynamic conditions (McDowell and Bolton 1998; Lobo-Guerrero and Vallejo 2006; O'Sullivan, Cui, and O'Neill 2008; McDowell and Li 2016; Ngo, Indraratna, and Rujikiatkamjorn 2016, 2017a). 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:

$$\vec{F}_N = K_N U^n \quad (1)$$

$$\delta \vec{F}_T = -K_T \cdot \delta U^s \quad (2)$$

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 the two particles A and B and is determined by:

$$\vec{M}_r = \begin{cases} K_r \vec{\omega}_r & \text{if } K_r \|\vec{\omega}_r\| < \|\vec{M}_r\|_{lim} \\ \|\vec{M}_r\|_{lim} \frac{\vec{\omega}_r}{\|\vec{\omega}_r\|} & \text{if } K_r \|\vec{\omega}_r\| \geq \|\vec{M}_r\|_{lim} \end{cases} \quad (3)$$

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 e incremental rolling; here η_r is the dimensionless coefficient, and γ_r is the rolling resistance coefficient.

Modeling Irregularly-Shaped Ballast Particles

Ballast particles of varying shapes and sizes are simulated by clumping many spheres together to represent actual ballast gradation (Ngo, Indraratna, and Rujikiatkamjorn 2017b), as shown in Figure 6(a). 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 a mass $m^{[p]}$, a radius $R^{[p]}$, and centroid location $x_i^{[p]}$, the mass properties are defined by Itasca (2014) as:

$$m = \sum_{p=1}^{N_p} m^{[p]} \quad (4)$$

$$x_i^{[G]} = \frac{1}{m} \sum_{p=1}^{N_p} m^{[p]} x_i^{[p]} \quad (5)$$

$$I_{ii} = \sum_{p=1}^{N_p} \left\{ m^{[p]} \left(x_j^{[p]} - x_j^{[G]} \right) \left(x_j^{[p]} - x_j^{[G]} \right) + \frac{2}{5} m^{[p]} R^{[p]} R^{[p]} \right\} \quad (6)$$

$$I_{ij} = \sum_{p=1}^{N_p} \left\{ m^{[p]} \left(x_i^{[p]} - x_i^{[G]} \right) \left(x_j^{[p]} - x_j^{[G]} \right) \right\}; \quad (i \neq j) \quad (7)$$

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:

$$F_i = m(\ddot{x}_i - g_i) \quad (8)$$

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):

$$\{M\} - \{W\} = [I]\{\alpha\} \quad (9)$$

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

$$[W] = \begin{Bmatrix} \omega_2\omega_3(I_{33} - I_{22}) + \omega_3\omega_3I_{23} - \omega_2\omega_2I_{32} - \omega_1\omega_2I_{31} + \omega_1\omega_3I_{21} \\ \omega_3\omega_1(I_{11} - I_{33}) + \omega_1\omega_1I_{31} - \omega_3\omega_3I_{13} - \omega_2\omega_3I_{12} + \omega_2\omega_1I_{32} \\ \omega_1\omega_2(I_{22} - I_{11}) + \omega_2\omega_2I_{12} - \omega_1\omega_1I_{21} - \omega_3\omega_1I_{23} + \omega_3\omega_2I_{13} \end{Bmatrix}$$

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, respectively

Modelling Direct Shear Tests for Ballast in DEM

Figure 6 shows how DEM is used to model geogrid-reinforced ballast in a direct shear test. The model dimensions are similar to those carried out 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 (Figure 6a). This method is widely used to simulate ballast aggregates (Ngo, Indraratna, and Rujikiatkamjorn 2017b), which are then placed at random locations within the specified wall boundary and without overlapping. A biaxial geogrid with 40 mm x 40 mm geogrid is modelled by bonding a number of small spheres together, like geogrids tested in a laboratory, as shown in Figure 6a. The Micromechanical parameters used to model the ballast, geogrid, and coal fines are adopted from Indraratna et al. (2014c). To quantify levels of ballast fouling, the void contaminant index (VCI) which considers the specific gravity of the fouled material, as proposed earlier by Tennakoon *et al.* (2012), is used and given by:

$$VCI = \frac{1+e_f}{e_b} \times \frac{G_{sb}}{G_{sf}} \times \frac{M_f}{M_b} \times 100 \quad (10)$$

where e_f = 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.

DEM simulations of direct shear tests have been carried out under three normal stresses of $\sigma_n = 27\text{kPa}$, 51kPa , and 75kPa for fresh and fouled ballast ($VCI=40\%$) with and without the inclusion of geogrid (Ngo *et al.* 2014). Figure 7 shows comparisons of the shear stress-strain behaviour of geogrid reinforced ballast from the DEM analysis, and those measured experimentally. Note that the simulation results agree reasonably well with the laboratory data carried out by (Indraratna, Ngo, and Rujikiatkamjorn 2011) for a given normal stress and level of fouling. The strain softening behaviour of ballast and volumetric dilation is observed in all the simulations and show that the higher the normal stress (σ_n), the greater the shear strength and the smaller the dilation (Ngo and Indraratna 2016). The ability of geogrid reinforcement to increase the shear strength of fresh and fouled ballast can be seen by comparing it with an assembly with unreinforced ballast; this is due to the interlocking effect that occurs between the ballast grains and geogrid.

Finite Element Analysis for Geocell-Reinforced Subballast

Salim and Indraratna (2004) introduced an elasto-plastic stress-strain constitutive model which incorporated dilatancy, breakage, and the plastic flow rule to determine ballast deformation and degradation. This model applies a generalised 3D system to define the contact forces, and the stresses and strains in granular media, including the plastic potential, the hardening function, and particle breakage. This model is developed 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 presented earlier by Salim and Indraratna (2004), as follows:

$$d\epsilon_s^p = \frac{2\alpha\kappa\left(\frac{p}{p_{cs}}\right)\left(1 - \frac{p_{o(i)}}{p_{cs(i)}}\right)(9 + 3M - 2\eta^*M)(\eta - \eta_i)d\eta}{(M - \eta_i)^2(1 + e_i)\left(\frac{2p_o}{p} - 1\right)\left[9(M - \eta^*) + \frac{B}{p}\{\chi + \mu(M - \eta^*)\}\right]} \quad (11)$$

$$d\epsilon_v^p = \frac{9(M - \eta)}{9 + 3M - 2\eta^*M} d\epsilon_s^p + \left(\frac{B}{p}\right)\left[\frac{\chi + \mu(M - \eta^*)}{9 + 3M - 2\eta^*M}\right] d\epsilon_s^p \quad (12)$$

where p : effective mean stress; p_{cs} : value of p on the critical state line at the current void ratio; p_o : 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 constitutive model contains 11 parameters for monotonic loading and 4 additional parameters for cyclic loading. These parameters can be evaluated using the results of the drained triaxial test and the measurements of particle breakage. This model has been validated using large-scale triaxial tests (Figure 8).

A laboratory study where geocells are used to reinforce sub-ballast used the large-scale Track Process Simulation Apparatus (TPSA) shown in Figure 9a; the results were presented earlier by Indraratna, Biabani, and Nimbalkar (2015). Numerical studies using the Finite element method (FEM) were also carried out to investigate the reinforcement behaviour of geocells where the material properties obtained from laboratory tests and model geometry followed the TPSA

carried out in the laboratory (800 mm × 600 mm × 450 mm), as shown in Figure 9. Cyclic loads exerted beneath the ballast and then loaded directly onto the subballast surface exhibited the same the loading characteristics as those applied in the laboratory. An elasto-plastic constitutive model with non-associative behaviour was also adopted to simulate subballast in the analysis (Chen et al. 2016). Drucker-Prager yield criterion was used to capture the elasto-plastic behaviour of subballast (Biabani, Indraratna, and Ngo 2016). The model parameters (i.e., ϕ , ψ) were determined in the laboratory using triaxial equipment (i.e. friction angle $\phi=39^\circ$, angle of dilation $\psi=9^\circ$, cohesion yield stress = 2 kPa, Poisson's ratio $\nu=0.3$). The geometry of the geocell pockets were modelled as a hexagonal shape because it is similar to the actual shape of the geocell used in the laboratory. The input parameters used to model geocell are given as: density = 950 (kg/m³), secant modulus (3% strain) = 0.3-5 (GPa), Poisson's ratio = 0.3. Details of the FEM model are described in detail by Biabani *et al.* (2016a). Due to the high computation time needed to simulate a cyclic model, in this study all simulations were conducted up to 10,000 cycles, where most subballast deformation had already occurred, as observed in the laboratory (Biabani, Ngo, and Indraratna 2016). Cyclic loading and the dynamic behaviour of subballast and geocell were modelled using predetermined sinusoidal functional loading and with a dynamic amplification factor of 1.45 (Biabani, Indraratna, and Ngo 2016).

The contours where subballast has spread laterally under a confining pressure of $\sigma_3'=10$ kPa are shown in Figure 10(a) where maximum lateral deformation occurred in the subballast beneath the geocell. As discussed by Biabani, Indraratna, and Ngo (2016), the lateral deformation of subballast increases as the number of load cycles increases and then maximum lateral spreading occurs beneath the geocell-reinforced subballast. It is noted that the tensile strength of geocell is an important parameter governing the performance of geocell-reinforced subballast where it is commonly considered to remain unchanged in conventional design practices (Ngo, Rujikiatkamjorn, and Indraratna 2017; Leshchinsky and Ling 2013). However, data captured from this study shows that during cyclic loading the mobilised tensile stress of geocell varies significantly, as shown in Figure 10(b); in fact during the loading stage, maximum tensile stress is mobilised in the geocell to prevent the infill subballast from excessive lateral spreading (Ngo et al. 2016). 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 10 also shows that 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).

Finite Element Analysis of Railway Track with Rubber Tyre-Reinforced Capping Layer

Rubber tyres have a three dimensional cylindrical structure, and as such could be used to stabilise foundations by increasing the bearing capacity and reducing the settlement of transport infrastructure. A finite element 3D analysis was carried out to model the behaviour of a capping layer composed of infilled rubber tyres.

The FEM results of the load-deformation relationships for tyre confined and unconfined plate load tests agreed reasonably well with the experimental results as shown in Figure 12 (Indraratna, Sun and Grant 2017). Having calibrated the FEM model with a single tyre unit, a plane strain section of half a ballasted substructure was simulated to examine how the foundation

would react under load, with or without being confined by tyres. The application of rubber tyres in rail track has not been studied before, so currently there is no field experimental result available. The authors are on the process of conducting field trial. The field results with tyre confined capping layer application will be reported in further paper. As with a traditional railway track system, the steel rails in this study are supported on reinforced concrete sleepers spaced at 0.60 m centres; the rail head is 0.075 m wide, the web is 0.018 m wide, and the base is 0.15 m wide, and concrete sleepers are embedded into a layer of coarse granular aggregate (ballast). For a standard gauge track, a 2.50 m wide sleeper was bevelled to a maximum height of 0.20 m at the ends and 0.15 m at the centre. The ballast layer was 4 m wide at the base, 3 m wide at the crest, and 0.35 m high, with a slope of 1:1. The layer of subballast confined by rubber tyres was 6 m wide at the base and 0.25 m thick. A typical passenger car tyre was simulated as being 0.15 m wide, 0.56 m in diameter, and 0.01 m thick. The simulated geometry of the track is shown in Figure 11(a). A plane strain slice of the cross section of half a ballasted railway substructure was modelled by a finite element mesh refined to observe track settlement, lateral displacement of the ballast slope, and the subgrade stress of the foundation with or without rubber tyres reinforcing the subballast (Figure 11(b&c)). Figure 13a shows the effect that reinforcement offered by rubber tyres has on the deviator stress at the surface of the subgrade. As expected, the highest deviator stress occurs near the end of the sleeper and then decreases towards the centre of the sleeper. With tyre-reinforcement, a train running with the same axle load (i.e. 25 ton) and speed (i.e. 100 km/h), experiences a maximum deviator stress of 46.2 kPa, which is almost a 12% reduction compared to an unreinforced section. Intuitively, the confining effect causes the tyres and gravel infill composite to act as a stiffer, flexible “mattress” which allows a reduced and more uniform stress to be transmitted to the subgrade. Figure 13b shows the effect that tyre reinforcement has on the distribution of deviator stress with the depth of subgrade. The deviator stress in the subgrade decreases with depth, a result which indicates that using rubber tyres as reinforcement can reduce the vertically distributed deviator stress in the subgrade and prevent its excessive deformation and yielding. As expected, lateral deformation along the slope of the embankment also decreases considerably due to tyre reinforcement (Figure 14). The contours of lateral displacement for the unreinforced and reinforced layers of subballast are also shown in Figure 15, where the largest lateral movement of subballast developed beneath the edge of the sleeper in the reinforced subballast. These comparisons show that the maximum lateral displacement of unreinforced subballast (0.037m) was much more than reinforced subballast (0.0018m) under the same load.

Field Study on Instrumented Track at Singleton

To study the effects of different types of geosynthetics and rubber mats on overall track stability, an extensive field study was undertaken on instrumented sections of track near Singleton, NSW Australia. This track is owned and operated by the Australian Rail Track Corporation (ARTC) and is about 80 km away from the south coast (Indraratna et al. 2014d).

Track Construction and Instrumentations

Eight experimental sections were built on subgrades viz. (i) relatively soft general fill and alluvial silty clay deposits, (ii) a stiff reinforced concrete bridge deck, and (iii) intermediate siltstone. The track substructure consists of 300 mm thick ballast underlain by a 150 mm thick layer of subballast. A structural-fill layer with a minimum of 500 mm thickness was placed

above the subgrade. Three biaxial geogrids and one geocomposite (biaxial geogrid + nonwoven geotextile) were installed below the ballast layer. Three biaxial geogrids: (i) geogrid 1 (aperture size = 44×44 mm, peak tensile strength = 36 kN/m), (ii) geogrid 2 (aperture size = 65×65 mm, peak tensile strength = 30 kN/m), (iii) and geogrid 3 (aperture size = 40×40 mm, peak tensile strength = 30 kN/m). A geocomposite layer consisted of biaxial geogrid (aperture size = 31×31 mm, peak tensile strength = 40 kN/m) and nonwoven polypropylene geotextile (mass per unit area = 150 g/m^2 , thickness = 2.9 mm). A layer of shock mat was installed between the ballast and bridge deck to minimise any degradation of the ballast. More detailed information on track Sections and the properties of geosynthetics and shock mats used in this study is given elsewhere by Indraratna et al. (2014d); and Nimbalkar and Indraratna (2016). Different types of sensors were used to measure the required data during field tests. Strain gauges were used to record mobilised strains along the layers of geogrid, and traffic induced vertical stresses were monitored by pressure cells. Transient deformation of the ballast was measured by potentiometers mounted on a custom built aluminium frame, and settlement pegs were installed to measure the vertical deformation (i.e. track settlement) of ballast. Electrical analogue signals from the strain gauges, pressure cells, and potentiometers were captured using a mobile data acquisition system.

Permanent Deformations of Ballast

The vertical deformation (S_v) of ballast is plotted against the number of load cycles (N) as shown in Figure 16(a-b) for soft embankment and hard rock, respectively. These results indicated that the relationship between S_v and N was non-linear for every section of the track. The rate of increase of S_v diminished as the number of load cycles increased, as observed in the laboratory by (Indraratna and Nimbalkar 2013) where the values of S_v for the reinforced sections were 10-32% smaller than those without reinforcement. This could be attributed to the particle-grid interlock which confines and restrains particles from free displacements (Rujikiatkamjorn et al. 2012). The results also indicated that S_v becomes larger as the subgrade becomes weaker. It was apparent that geogrid reinforcement was very effectively reduced the deformation of track located on softer subgrades.

Ballast Fouling and Breakage

Ballast samples were recovered from beneath the sleeper because this location was considered to be most appropriate (Indraratna, Ngo, and Rujikiatkamjorn 2013; Indraratna et al. 2016). A visual inspection of the samples revealed fragments of crushed rock which probably resulted from particle breakage. The value of VCI was less than 4-6 %, which indicated a minimum level of fouling when these measurements were obtained. Particle breakage was quantified in terms of BBI and its values are given in Table 1. As expected, BBI was highest at the top, followed by significant reduction with depth. The largest values of BBI obtained with hard rock verified that particle breakage was influenced by the type of subgrade. The larger stresses also caused much more breakage of individual particles of ballast (Xiao et al. 2016), as was anticipated. At the alluvial deposit with hard rock, there was a larger vertical settlement which was attributed to larger particle breakage in these sections. The least amount of breakage occurred at the concrete bridge deck, thus confirming the ability of shock mats to mitigate ballast degradation.

Conclusion

This paper is a review of our current knowledge of rail track geo-technology resulting from tests and studies carried out at the University of Wollongong over the last two decades. The shear stress-strain and deformation of ballast and the use of waste tyres, geosynthetics, and geocells to enhance the performance of rail tracks has been reviewed and discussed through large scale laboratory tests, field trials, theoretical modelling, and numerical simulations. A series of tests using the large-scale triaxial equipment, large-scale direct shear box, Track Process Simulation Apparatus (TPSA) were carried out to study the role played by recycled tyres, geosynthetics and geocells in relation to the stress-strain and degradation response of ballast under monotonic and cyclic loads.

The large scale triaxial tests revealed that permanent deformation and degradation increases with the frequency and magnitude of load cycles. Three different deformation mechanisms occurred in response to the frequency of loading; in Range I: plastic shakedown at $f \leq 20$ Hz; in Range II: plastic shakedown and ratcheting at $30 \text{ Hz} \leq f \leq 50 \text{ Hz}$; and in Range III: plastic collapse at $f \geq 60 \text{ Hz}$. Based on a number of large scale process simulation tests on segments of ballasted track mimicking a typical railway substructure, it was noted that a rubber tyre membrane could enhance track performance, and there was a substantial reduction in axial deformation of ballast confined by the cellular rubber membrane, i.e. a reduction in settlement of almost 15% at $N=500,000$ cycles.

Discrete element modelling of large-scale direct shear tests for fresh and fouled ballast ($VCI=40\%$) were carried out to determine exactly how geogrids improve the performance of ballast. The shear stress-strain behaviour obtained from the DEM analysis matches reasonably well with the measured data and shows that coal fines in the ballast assembly reduces the interlock between the ballast grains and geogrids and thus decreases its shear strength. It is concluded that the interlocking of ballast aggregates with geogrid is the primary factor responsible for the enhanced performance of the geogrid-stabilised ballast assembly.

The results of a comprehensive field monitoring program undertaken at Singleton track in NSW, Australia, to study the ability of various geosynthetics and rubber mats to improve track stability have been discussed. In these studies, different types of geosynthetics (geogrid, geotextile and geocomposite) and rubber mats were installed beneath a ballast layer constructed on various subgrade conditions. The Singleton study showed that geogrids were better at curtailing deformation on soft subgrade whereas rubber mats were better when placed above a concrete bridge deck. The findings of these studies provide a better understanding of crucial aspects such as the ballast-geogrid interface mechanism, long-term deformation and degradation, as well as the benefits of using geosynthetics and rubber mats to enhance the performance of ballasted tracks.

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1. List of Tables

Table 1 Track variable values used in the FEM analysis

Track variable	Value
Rail Properties	
Density (kg/m^3)	2000
Young's modulus E (MPa)	500000
Poisson's ratio, ν	0.3
Sleeper Properties	
Density (kg/m^3)	2000
Young's modulus E (MPa)	30000
Poisson's ratio, ν	0.25
Ballast Layer	
Density (kg/m^3)	1530
Young's modulus E (MPa)	20
Poisson's ratio, ν	0.3
Internal angle of friction, ϕ (degrees)	45
Angle of dilation, ψ (degrees)	15
Cohesion, c (kPa)	1
Thickness (m)	0.35
Subballast Layer	
Density (kg/m^3)	2100
Young's modulus E (MPa)	20
Poisson's ratio, ν	0.3
Internal angle of friction, ϕ (degrees)	39
Angle of dilation, ψ (degrees)	15
Cohesion, c (kPa)	1
Subgrade Layer	
Density (kg/m^3)	1700
Young's modulus E (MPa)	20
Poisson's ratio, ν	0.35
Tyre Properties	
Density (kg/m^3)	1500
Young's modulus E (MPa)	750
Poisson's ratio, ν	0.35
Wheel Load	
Static wheel load (kN)	122.5
Train Speed (km/h)	100
Dynamic amplification factor (DAF)	1.54
Dynamic wheel load (kN)	188.2

Table 2. Assessment of ballast breakage (data sourced from Indraratna, Nimbalkar, and Neville 2014)

Section No.	subgrade	Ballast breakage index (<i>BBI</i>) for		
		Top layer	Middle layer	Bottom layer
1	alluvial silty clay (Section A)	0.17	0.08	0.06
2	concrete bridge deck (Section B)	0.06	0.03	0.02
3	Siltstone (Section C)	0.21	0.11	0.09

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Figure 1 Large-scale triaxial testing system at University of Wollongong

Figure 2 (a) Variation of axial strain ϵ_a verse number of cycles N for $q_{max,cyc}$ of 230 kPa; (b) variation of axial strain ϵ_a verse number of cycles N for $q_{max,cyc}$ of 370 kPa; (c) variation of ballast breakage index BBI with various frequencies f ; (d) examples of ballast degradation (Sun, Indraratna, and Nimbalkar 2016; with permission from ASCE)

Figure 3 (a) Tyre before experiment; (b) tyre filled with ballast; (c) testing set up; (d) schematic illustration

(a)

(b)

Figure 4 (a) Lateral displacement for unreinforced and reinforced specimen; (b) resilient modulus for unreinforced and reinforced specimen

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Figure 8: Model prediction compared with experimental data for drained triaxial shearing (Salim and Indraratna 2004)

(a)

(b)

Figure 9. (a) Track Process Simulation Apparatus (TPSA); (b) finite element modeling for geocell-reinforced subballast (modified after Biabani, Indraratna, and Ngo 2016)

(a)

(b)

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Figure 11. (a) Track geometry with rubber tyres reinforced capping layer; (b) FEM mesh of ballasted railway track; (c) FEM mesh of rubber tyres

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(a)

(b)

Figure 13 (a) Subgrade stress distribution below reinforced and unreinforced track; (b) distribution of subgrade deviator stress with depth for reinforced and unreinforced track

Figure 14 Lateral displacement of ballast layer with and without reinforcement

Maximum displacement=0.0018m

(a)

Maximum displacement=0.037m

(b)

Figure 15 Maximum lateral displacement of capping layer with and without reinforcement

Figure 16 Vertical deformation of ballast for (a) soft embankment; (b) hard rock (data sourced from Indraratna, Nimbalkar, and Neville (2014) with permission from ASCE)