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B Indraratna

University of Wollongong, indra@uow.edu.au

S Nimbalkar

University of Wollongong, sanjayn@uow.edu.au

C Rujikiatkamjorn

University of Wollongong, cholacha@uow.edu.au

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Abstract

An efficient transportation infrastructure has become utmost priority for global economic reforms. Railways are designed to provide high speed passenger and heavy haul freight transportation. Ballast is one of important constituents of the rail track however, it experiences excessive deformation and degradation from trains operating at high speeds. In addition, tracks built along coastal areas often undergo large settlements due to soft compressible clay deposits. This leads to progressive track deterioration and necessitates frequent and costly track maintenance. The use of artificial inclusions such as geogrids, geocomposites, shock mats and prefabricated vertical drains (PVDs) is often an attractive design alternative for track practitioners. However performances of these inclusions are predominantly governed by their technical specifications in addition to geotechnical characterization of the track substructure including ballast and subgrade. Therefore, full scale field trials were conducted on instrumented track sections built along the south-east coast of Australia (e.g. Bulli and Singleton). The performance of geogrids and geocomposite was demonstrated in terms of specific key parameters such as stiffness and aperture size of geogrids, placement location of geogrids, as well as subgrade types. Placement of shock mats (rubber pads) in rail tracks has also lead to the mitigation of particle breakage. Empirical approaches to relate ballast strains with the number of load cycles are presented. Bearing capacity analyses of track substructure is discussed. The use of PVDs to dissipate the excess pore pressure for increased stability of the soft clay subgrade is presented.

Keywords

speeds, higher, freight, tracks, modernisation, rail, greater

Disciplines

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Modernisation of Rail Tracks for Higher Speeds and Greater Freight

B. Indraratna¹, S. Nimbalkar² and C. Rujikiatkamjorn³

¹Professor of Civil Engineering and Research director, Centre for Geomechanics and Railway Engineering; Program Leader, ARC Centre of Excellence for Geotechnical Science and Engineering; University of Wollongong, Wollongong, Australia

²Research Fellow, Centre for Geomechanics and Railway Engineering; University of Wollongong, Wollongong, Australia

³Associate Professor and Research Fellow, School of Civil, Mining and Environmental Engineering, University of Wollongong, Wollongong, Australia

Abstract

An efficient transportation infrastructure has become utmost priority for global economic reforms. Railways are designed to provide high speed passenger and heavy haul freight transportation. Ballast is one of important constituents of the rail track however, it experiences excessive deformation and degradation from trains operating at high speeds. In addition, tracks built along coastal areas often undergo large settlements due to soft compressible clay deposits. This leads to progressive track deterioration and necessitates frequent and costly track maintenance. The use of artificial inclusions such as geogrids, geocomposites, shock mats and prefabricated vertical drains (PVDs) is often an attractive design alternative for track practitioners. However performances of these inclusions are predominantly governed by their technical specifications in addition to geotechnical characterization of the track substructure including ballast and subgrade. Therefore, full scale field trials were conducted on instrumented track sections built along the south-east coast of Australia (e.g. Bulli and Singleton). The performance of geogrids and geocomposite was demonstrated in terms of specific key parameters such as stiffness and aperture size of geogrids, placement location of geogrids, as well as subgrade types. Placement of shock mats (rubber pads) in rail tracks has also lead to the mitigation of particle breakage. Empirical approaches to relate ballast strains with the number of load cycles are presented. Bearing capacity analyses of track substructure is discussed. The use of PVDs to dissipate the excess pore pressure for increased stability of the soft clay subgrade is presented.

Keywords: ballast, geosynthetics, deformation, drainage, bearing capacity.

1 Introduction

Frequent traffic congestion and the demand for quicker and safer transport have made the railways the most demanded means of public transportation. The ballast layer should provide the optimum resiliency, thereby transmitting the imposed wheel loading to an acceptable depth of the subgrade soils while preventing excessive settlement and lateral spread [1]. However, the progressive deterioration and breakdown of ballast due to increased train speeds and heavier axle loads is a key factor to cause track geometry change and increased track maintenance costs [2]. In addition, soft compressible clays found along coastal regions of Australasia often exhibit extremely low bearing capacity. In order to improve track conditions and optimise the life-cycle cost of track, the use of geosynthetics (geogrids, geocomposites), resilient mats (under ballast mats), and prefabricated vertical drains is desirable.

The use of geosynthetics (geogrids, geocomposites) for track confinement, and as separation layer between the ballast and subballast, is much preferred. Geogrids can reduce the lateral spreading and fouling of ballast, as well as its degradation [3, 4, 5]. A layer of geocomposite stabilised recycled ballast much better than standard geogrids, and also prevented the ballast from being fouled due to fines migrating from layers of subballast and subgrade [6, 7, 8, 9]. Comprehensive field studies on instrumented tracks at Bulli (near Wollongong) and Singleton (near Newcastle) supported by RailCorp and ARTC, were carried out to measure the in-situ stresses and deformation of ballast embankments and to study the effectiveness of different types of geosynthetics installed at the ballast-capping interface [10, 11, 12].

Two distinct types of peak forces were seen during impact loading, an instantaneous sharp peak with very high frequency, and a gradual peak of smaller magnitude with relatively lesser frequency. Jenkins et al. [13] termed these peak forces as P_1 and P_2 respectively. P_1 is a high-frequency dynamic load that occurs when a vibration mode between the wheel and rail is excited, while P_2 is a low-frequency dynamic load that occurs when the coupled wheel-rail vibrates in phase on the ballast [14]. U.K. Railway group standards [15] recommends consideration of the P_2 force in the track design criteria. Installing shock mats in rail tracks can mitigate the breakage of ballast substantially [7, 16].

The radial drainage can be much effective in dissipating the excess pore pressure [17, 18]. It was found that the radial drainage decelerated the rate of excess pore pressure build up to its critical value. The test results also suggested that for newly constructed railway lines, a train with a lower speed is preferred initially, until the track becomes stable for the next loading stage. The prefabricated vertical drains assist the dissipation of the excess pore pressures both during and after cyclic loading [19, 20]. A reduced void ratio due to the drainage of the pore water can prevent the generation of excess pore pressure in the following cyclic loading. It shows that drainage during cyclic loading provides dissipation of excess pore

pressure and increases shear strength and therefore normally consolidated clays are more resistant to the following cyclic shear stress. This paper presents the results of laboratory testing, full-scale field monitoring, and theoretical modelling, which demonstrated the beneficial use of geosynthetic grids, shock mats and drains for rail infrastructure.

2 Bearing Capacity of Track Substructure

In this section the limit equilibrium approach for determining the bearing capacity of rail track is presented.

2.1 Friction Angle of Ballast

Rowe [21] studied the effect of dilatancy on the friction angle of granular aggregates and concluded that the interparticle friction angle ϕ_μ , should be replaced by ϕ_f , which is the friction angle of aggregates after correction for dilatancy. The friction angle ϕ_f , varies from ϕ_μ at very dense state to ϕ_{cv} at very loose condition, where deformation takes place at a constant volume. The energy spent on the rearrangement of particles during shearing has been attributed to the difference between ϕ_f and ϕ_μ [22]. The dense assemblies of cohesionless particles deform in such a way that the minimum rate of internal energy (work) is absorbed in frictional heat [21, 22]. The following relationship was proposed to evaluate the basic friction angle (ϕ_f) of the ballast [23, 24]:

$$\frac{q}{p'} = \frac{\left(1 - \frac{d\varepsilon_v}{d\varepsilon_1}\right) \tan^2\left(45^\circ + \frac{\phi_f}{2}\right) - 1}{\left[\frac{2}{3} + \frac{1}{3}\left(1 - \frac{d\varepsilon_v}{d\varepsilon_1}\right) \tan^2\left(45^\circ + \frac{\phi_f}{2}\right)\right]} + \frac{dE_B (1 + \sin\phi_f)}{p' d\varepsilon_1 \left[\frac{2}{3} + \frac{1}{3}\left(1 - \frac{d\varepsilon_v}{d\varepsilon_1}\right) \tan^2\left(45^\circ + \frac{\phi_f}{2}\right)\right]} \quad (1)$$

where q/p' is the stress ratio, $(1-d\varepsilon_v/d\varepsilon_1)$ is the dilatancy, and ϕ_f is the basic friction angle, which excludes the effects of both dilatancy and particle breakage.

In order to evaluate the basic friction angle (ϕ_f) for the ballast, the last term of Equation (1) containing the energy consumption due to particle breakage is set to zero. The resulting apparent (equivalent) friction angle is denoted by ϕ_b , which naturally includes the contribution of particle breakage but excludes the effect of dilation. The value of ϕ_f of the latite aggregates based on the triaxial testing is found to be approximately 44° [23]. By using the Mohr-Coulomb failure criterion and considering the peak principal stress ratio $(\sigma_1/\sigma_3)_p$, the peak friction angle (ϕ_p) could be conveniently calculated. The peak friction angle ϕ_p can also be considered as the summation of the basic friction angle ϕ_f , and the effects of dilatancy and particle breakage.

2.2 Bearing Capacity of Ballast

The maximum bearing capacity of the ballast q_{max} , is obtained as [25]:

$$q_{max} = 0.5\gamma BN_{\gamma}S_{\gamma} \quad (2)$$

$$N_{\gamma} = (N_q - 1)\tan(1.4\phi) \quad (3)$$

$$N_q = K_p e^{\pi \tan \phi} = \left(\frac{1 + \sin \phi}{1 - \sin \phi} \right) e^{\pi \tan \phi} \quad (4)$$

$$S_{\gamma} = 1 + 0.1K_p (B/L) \quad (5)$$

where γ is the bulk unit weight of ballast, B is the length of the sleeper, L is the width of the sleeper, ϕ is the angle of effective shearing resistance of the ballast, and N_q , N_{γ} and S_{γ} are the bearing capacity factors.

By using the various friction angles (ϕ_{fb} , ϕ_f , ϕ_p), together with $B = 2.5$ m, $L = 0.285$ m, $\gamma = 16$ kN/m³, the maximum bearing capacity q_{max} , can be calculated as shown in Figure 1. As the confining pressure increases the bearing capacity of ballast show the same trends as the friction angles do.

Figure 1: Effect of confining pressure on the maximum bearing capacity.

2.3 Bearing Capacity of Subgrade

The ultimate bearing capacity for shallow foundations with an applied vertical load is calculated using the following approach [26]:

$$q_u = cN_cS_c d_c + \gamma_1 H N_q S_q d_q + 0.5 \gamma_2 B N_\gamma S_\gamma d_\gamma \quad (6)$$

where q_u is the ultimate load per unit area of the foundation (kPa), γ_1 is the effective unit weight of the granular layer (kN/m³), γ_2 is the effective unit weight of the subgrade layer (kN/m³), H is the thickness of the granular layer under the sleeper (m), B is the width of foundation (m), c is the cohesion of soil (kPa), $N_{c,q,\gamma}$ are bearing capacity factors relative to the soil friction angle (ϕ), $S_{c,q,\gamma}$ are shape factors and $d_{c,q,\gamma}$ are depth factors. This approach is used for different types of failure mechanisms viz. (1) Failure for the complete sleeper-track system ($B = 2.5$ m), (2) Failure beneath the highest stressed location (the outside third of the sleeper) ($B = 0.83$ m, $L = 2.02$ m), (3) Failure below a freight wagons arrangement ($B = 2.02$ m, $L = 2.5$ m).

Figure 2 shows effects of subgrade cohesion and friction angle on the bearing capacity of subgrade for various failure mechanisms. Figure 2 reveals that increased subgrade friction angle caused a larger bearing capacity. Similarly, increased cohesion of subgrade also increased the bearing capacity. Thus cohesion and friction angle of the subgrade are the most critical parameters in calculating the bearing capacity. This Figure also shows that the failure mechanism resulting into the minimum bearing capacity is mostly Failure Mechanism 1.

Figure 2: Bearing capacity of subgrade.

3 Use of prefabricated vertical drains as subsurface drainage

To cater for rapid development, railways will inevitably be built on soft subgrade such as soft clay subgrade. It is imperative to understand the behaviour of soft clay subgrade subjected to cyclic loads. Upon cyclic loading, excess pore pressures and axial strains can develop with the increasing number of cycles, resulting in a decreased bearing capacity and excessive settlement. Prefabricated vertical drains can be used to dissipate excess pore pressures by radial consolidation before they can develop to critical levels. These PVDs continue to dissipate excess pore water pressures even after the cyclic load ceases [17].

3.1 Use of Short PVDs under Railway Track

To stabilise the subgrade, short prefabricated vertical band drains can be employed where a short radial drainage path is reduced to dissipate the excess pore pressure so that the soft clay subgrade becomes more stable subjected to train loads [18, 20].

3.2 Cyclic Triaxial Test Procedure

Large scale cyclic triaxial tests were carried out on specimens of reconstituted Kaolinite using the cylindrical dynamic triaxial equipment (accommodating 300 mm diameter and 600 mm height samples). While the clay was being placed into the membrane, four miniature pore pressure transducers *T1* to *T4* were inserted from the base plate through cable adapters and fixed at predetermined radial (20, 60, and 130 mm from the drain) and vertical distances (150 and 450 mm from the bottom) (see Figure 3). A single PVD was installed in the centre of the soil cylinder to allow for radial drainage during and after the cyclic tests.

Three types of partially drained cyclic triaxial tests were conducted on specimens of soft kaolin (see Figure 4): (a) cyclic loading without a rest period, (b) cyclic loading with a rest period, and (c) cyclic loading with a changing loading frequency. The usefulness of PVDs in dissipating the excess pore pressure during and after cyclic loading was investigated. The cyclic stress ratio ranged from 0.4 to 0.8 and the loading frequency ranged from 0.1 to 5 Hz (Table 1). These tests were stopped when either failure occurred or it reached 15,000 cycles. Failure is considered to occur when the excess pore pressure increases to a critical value.

Figure 3: The location of the excess pore pressure transducers (all units are in mm)
(data sourced from Ni [19])

Figure 4: Large scale cyclic triaxial tests: (a) Cyclic loading without a rest period,
and (b) Cyclic loading with a rest period.

Sr. No.	σ_{1c} (kPa)	σ_{3c} (kPa)	f (Hz)	CSR	Load cycles (N)	Failed ?
D_{01}	40	24	1	0.4	15000	No
D_{02}	40	24	2	0.6	15000	No
D_{03}	40	24	5	0.6	15000	No
D_{04}	40	24	0.1	0.6	3024	Yes
D_{05}	40	24	1	0.6	15000 + Rest period + 15000 + Rest period + 15000	No

Table 1: Test conditions for partially drained cyclic loading.

3.3 Test results

The effectiveness of radial drainage in dissipating the excess pore pressure was examined. It was found that for a high cyclic stress ratio, the radial drainage decelerated the rate of excess pore pressure build up to its critical value, so the soil could undergo more loading cycles prior to failure. With a low cyclic stress ratio, radial drainage could prevent the excess pore pressure from accumulating to its critical value, so the soil would not fail. The test results also suggested that for newly constructed railway lines, a train with a lower speed is preferred initially, until the track becomes stable for the next loading stage.

Detailed test results for partially drained cyclic loading without a rest period are given in Figure 5. For each loading condition the development of excess pore pressures against the number of cycles obtained from the four miniature pore pressure transducers is provided, along with the corresponding undrained curves. As expected, the values of all the four pore pressure transducers were lower than the undrained value obtained from the undrained cyclic triaxial test. $T1$ has the lowest value as it has the shortest drainage path, followed by $T2$, $T3$, and $T4$. However, the sample failed after 3,024 cycles when a critical level of excess pore pressure of 0.68 was detected at $T4$. This indicates that more loading cycles can run before failure occurs at a high cyclic stress ratio with a centrally installed PVD.

The generation and dissipation of excess pore pressures under partially drained cyclic loading with rest periods are given in Figure 6. After the first 15,000 cycles of cyclic loading, the excess pore pressure ratio increased to 0.4 for $T4$ which is furthest from the drain, followed by 0.35 for $T3$, 0.3 for $T3$, and 0.2 for $T1$. The first rest period of 2 days was allowed for the excess pore pressure to dissipate. After the pore water flowed out of the specimen the excess pore pressure ratios decreased to 0.3, 0.25, 0.1, and 0.05 for $T4$, $T3$, $T2$, and $T1$ respectively.

Upon the application of the second 15,000 cycles of cyclic loading, the incremental excess pore pressure ratio due to the second set of cyclic shear decreased compared to the first set due to a decreased void ratio caused by the dissipation of excess pore pressure during and after the first set of cyclic loading.

Figure 5: Generation of excess pore pressures at different locations.

The increments in the excess pore pressure ratio were 0.25, 0.18, 0.15, and 0.08 for $T4$, $T3$, $T2$, and $T1$ respectively. However, with the residual excess pore pressure, the accumulated excess pore pressure ratios after the second set of cyclic loading for $T4$ and $T3$ were higher than the values obtained after the first set, while the accumulated values for $T2$ and $T1$ were smaller compared to the first set. Then another rest period of 2 days was allowed for drainage, during which the void ratio decreased further due to the drainage of pore water. After the second rest period of two days, the last 15,000 cycles of cyclic loading was imposed on the specimen. The incremental excess pore pressure ratios were even smaller and the accumulated excess pore pressure ratios were smaller than those after the second application of cyclic loading.

To summarise, the prefabricated vertical drains assist the dissipation of the excess pore pressures both during and after cyclic loading. A reduced void ratio due to the drainage of the pore water can prevent the generation of excess pore pressure in the following cyclic loading. After three sets of cyclic loading, the accumulated excess pore pressure began to reduce. This suggests that no substantial excess pore pressure will be observed if more sets of cyclic loading are applied. This study shows that drainage during cyclic loading provides dissipation of excess pore pressure and increases shear strength and therefore normally consolidated clays are more resistant to the following cyclic shear stress.

Figure 6: Generation and dissipation of excess pore pressures under partially drained cyclic loading with a rest period.

4 From Theory to Practice: Field Trial at Bulli

In order to assess the benefits of using geosynthetics in fresh and recycled ballast, a field trial was undertaken on a section of instrumented track at Bulli, NSW [10].

4.1 Track construction and material specifications

The field trial was carried out on a section of instrumented track located between two turnouts at Bulli, part of RailCorp's South Coast Track. The total length of the instrumented track section was 60 m, which was divided into four equal sections. The particle gradation of fresh ballast ($d_{50} = 35$ m, $C_u = 1.5$) and recycled ballast ($d_{50} = 38$ m, $C_u = 1.8$) were in accordance with the Industrial Standard [27]. The technical specifications of the geosynthetic are shown in Table 2. Technical specifications of various instruments used at the site can be found in Indraratna et al. [10].

Type	Geocomposite			
	Biaxial Geogrid		Nonwoven Geotextile	
Direction	MD	TD	MD	TD
Tensile strength (kN/m)	30	30	-	-
Strain at break (%)	11	10	-	-
Aperture size (mm)	40	27	-	-

Table 2: Mechanical properties of geocomposite used during the field trial.

4.2 Vertical strains in ballast

Vertical deformations were measured in the field, against time. A relationship between the annual rail traffic in million gross tons (*MGT*) and axle load (A_t) was used to determine the number of load cycles [1]:

$$C_m = \frac{10^6}{(A_t \times N_a)} \quad (7)$$

where C_m = number of load cycles per *MGT*, A_t = axle load in tons, and N_a = number of axles per load cycle. Considering an annual tonnage of 60 *MGT* of traffic, and four axles per load cycle, an axle load of 25 tons gives 600,000 load cycles for 60 *MGT* per annum.

Under repeated loading, the ballast layer undergoes compression in the vertical direction and expands in the two lateral directions. The vertical deformations (S_v) of ballast were determined by subtracting displacements of the ballast-capping interface from those at the sleeper-ballast interface. The mean vertical strain (ε_v) is defined as the ratio of S_v to the initial ballast thickness (i.e. 0.3 m). The mean vertical strains (ε_v) are plotted against the number of load cycles (N) in Figure 7. The

vertical strain is highly non-linear under cyclic loading and its non-linear variation against the number of load applications is best described by a semi-logarithmic relationship [2, 12, 28] such as:

$$\varepsilon_v = a + b(\ln N) \quad (8)$$

where, a and b are two empirical constants, depending on the type of ballast, type of geosynthetics used, and the initial placement density. The non-linear variation of ε_v with increasing load cycles becomes linear in the semi-logarithmic plot (Figure 7). When the results obtained from Sections 1 and 2 are compared, the vertical strain of ballast with geosynthetics is found to be about 40% smaller than that without reinforcement. The similar trend is observed also in the laboratory [8, 29, 30, 31], and is mainly attributed to the interlocking between ballast particles and the grid apertures, thus creating an enhanced track confinement.

The recycled ballast showed less vertical strains, because of its moderately graded particle size distribution compared to the highly uniform fresh ballast. However, the fresh ballast exhibited lesser vertical strains when reinforced with geogrid. This is attributed to a better interlock of fresh ballast with the geogrid (aperture size of 40×27 mm). Thus, the results of the field trial demonstrated the potential benefits of using a geocomposite at the base of the ballast layer in track and the use of moderately graded recycled ballast.

Figure 7: Vertical strains of the ballast layer plotted in semi-logarithmic scale (data sourced from Indraratna et al., [10]).

5 From Theory to Practice: Field Trial at Singleton

The sections of experimental track in this recent study were part of the track that extends from Bedford to Singleton, New South Wales. An extensive program of sub-surface exploration, consisting of 33 bore holes and 107 test pits, indicated that the Third Track was located on a massive sedimentary outcrop of rock, between 224.2 to 229.0 km, and later on the flood plain of the nearby Hunter River [32]. The rock outcrop was part of the Branxton Formation and mainly composed of medium to high strength siltstone. The flood plain consisted of a layer of an alluvial deposit of silty clay 7-10 m thick, underlain by heterogeneous layers of medium dense sand and silty clay with a total thickness of 7-9 m. Medium strength siltstone, similar to the first part of track, was found beneath the layer of sand and silty clay.

5.1 Track construction and material specifications

The experimental sections were constructed on subgrades viz. (i) the relatively soft general fill and alluvial silty clay deposit (Sections A and 1-4), (ii) the stiff reinforced concrete bridge deck (Section B), and (iii) the intermediate siltstone (Sections C and 5). The track substructure consisted of a 300 mm thick ballast ($d_{50} = 36$ mm) underlain by a 150 mm thick layer of sub-ballast ($d_{50} = 4$ mm). A structural layer of fill with a minimum of 500 mm thickness ($d_{50} = 3$ mm) was placed below the sub-ballast.

Three commercially available geogrids and one geocomposite were installed in a single layer at the ballast-sub-ballast interface. A layer of shock mat was installed between the ballast and bridge deck to minimise any degradation of the ballast. The properties of the geosynthetics and shock mats are listed in Table 3. For comparison purposes, no geosynthetic was installed at Sections A and C. A layer of shock mat was installed between the ballast and bridge deck at Section B to minimise any degradation of the ballast. Technical specifications of various instruments and shock mat used at the site can be found in Indraratna et al. [11, 12].

Type	Geogrid 1		Geogrid 2		Geogrid 3		Geocomposite			
							Geogrid		Geotextile	
Direction	MD	TD	MD	TD	MD	TD	MD	TD	MD	TD
Tensile strength (kN/m)	36	36	30	30	30	30	40	40	6	10
Strain at break (%)	15	15	15	15	15	15	15	15	60	40
Aperture size (mm)	44	44	65	65	40	40	31	31	-	-

Table 3: Mechanical properties of geogrids and geocomposite.

5.2 Vertical strains in ballast

The ballast deformation and strains in geogrids were measured against time. Delany (2011) reported a total traffic tonnage of 64 million gross tonnes (*MGT*) on the third track section during the period of measurement. The majority of traffic was imparted from coal trains having four axles, and axle loads between 25 and 30 tonnes. This results in 3.3×10^5 load cycles using Equation (7).

The deformation of ballast was determined by subtracting the vertical displacement of the ballast-capping interface from that at the sleeper-ballast interface. The mean vertical strain is defined as the ratio of ballast deformation to the initial ballast thickness. Vertical strains (ε_v) of the ballast layer are plotted against the number of load cycles (N) as shown in Figure 8 (a and b). The vertical deformation of the ballast is highly nonlinear under cyclic loading as also shown in previous studies [8, 10, 29, 33, 34]. The non-linear variation of vertical strains with increasing load cycles becomes linear in the semi-logarithmic plot. The values of empirical constants are obtained by performing a linear regression analysis. It is observed that Equation (8) fits the vertical strains of ballast reasonably well for a wide range of numbers of load cycles.

Figure 8(a): Vertical strains of ballast layer plotted versus number of load cycles in semi-logarithmic scale for soft embankment (data sourced from Indraratna et al., [12]).

Figure 8(b): Vertical strains of ballast layer plotted versus number of load cycles in semi-logarithmic scale for hard rock (data sourced from Indraratna et al., [12]).

When the results for sections on similar subgrades are compared, the vertical deformation of ballast with geosynthetics is 10-32% smaller than that without reinforcement. This is mainly attributed to the interlocking between ballast particles and the grid apertures, thus creating an enhanced track confinement. When the results for sections with similar geogrids are compared, it is observed that the effectiveness of a geogrid to reduce track settlement becomes higher for softer subgrades. This observation is in agreement with previous study by Ashmawy and Bourdeau [35].

5.3 Vertical stresses induced by trains

The maximum vertical cyclic stresses (σ'_v) recorded in Section C (i.e. fresh ballast) are shown in Table 4. These stresses were measured during the passage of coal freight trains (axle load of 25 and 30 tons) travelling at 40 km/h and 60 km/h. As expected, the greater axle load induced a higher σ'_v . It was also found that higher train speeds increased the stresses at the sleeper-ballast and ballast-capping interfaces. The effect of increased train speed was more pronounced at the sleeper-ballast interface. This study clearly highlights the implications of the increased train speeds on the ballast contact stresses.

Axle load (tons)	$A_t = 25$		$A_t = 30$	
Speed, V (km/h)	40	60	40	60
Sleeper-ballast	290	301	315	338
Ballast-capping	85	89	94	102

Table 4: Vertical cyclic stresses measured under the rail (σ'_v).

5.4 Ballast breakage

Samples were recovered from load bearing ballast beneath the rail seat as vertical stresses are usually the largest beneath the rail seat [8, 10]. A sampling pit with plan area of about $1.8 \text{ m} \times 1.3 \text{ m}$ was formed by excavating the ballast from the crib, shoulder and load bearing segments of the rail track. Samples were recovered from three equal portions of load bearing ballast in order to assess the variation of ballast breakage with depth. A proper care was taken to collect fine particles trapped inside voids of ballast. The ballast profile was then reinstated using clean ballast and tamped using a tamping head on the excavator.

Visual inspection of the samples suggested that fouling of the ballast layer due to spillage of coal from passing trains and 'slurry pumping' of the fines from the underlying subgrade had not taken place at this relatively new track. The breakage is quantified using the parameter, Ballast Breakage Index (BBI), proposed by Indraratna *et al.*, [33].

Sr. No.	Section	subgrade	Ballast breakage index (BBI)		
			Top	Central	Bottom
1	A	alluvial silty clay	0.17	0.08	0.06
2	B	concrete bridge deck	0.06	0.03	0.02
3	C	siltstone	0.21	0.11	0.09

Table 5: Assessment of ballast breakage for $N = 7.8 \times 10^5$ load cycles.

The value of ballast breakage index (BBI) for Sections A, B and C are shown in Table 5. As expected, the ballast breakage was highest at the top and decreased with depth. The variations in the BBI with depth were found quite similar to those observed in stresses and displacements of load bearing ballast layer. Largest values of BBI at Section C revealed that particle breakage was influenced by the type of subgrade. The particle degradation phenomenon is more pronounced for stiff subgrade than that for the relatively soft or weak subgrade [11, 12, 16]. This observation was also in agreement with previously published results on track transition zone [36]. In practice, the sleeper on the approach between soft and stiff subgrade has largest settlement, which imply most particle breakage. Although the

track at Section B was much stiffer than that at Section A, larger lateral confinement from the barriers of Mudies Creek bridge most likely resulted in a significantly smaller value of BBI. These results may also suggest the effectiveness of UBM in reducing ballast degradation when placed above the concrete deck. However, sufficient data from a similar bridge without any UBM is necessary for more convincing validation.

6 Conclusions

Benefits of geosynthetics (geogrid, geocomposite), shock mats (under ballast mats) and geodrains (prefabricated vertical drains) for improved track performance through laboratory studies and field trials are demonstrated in this Special Lecture. The use of large scale triaxial apparatus and precise instrumentation schemes adopted at long instrumented sections of rail track at Bulli and Singleton has advanced the state of the art knowledge in Transport geotechnics.

The confining pressure, shear strength and bearing capacity of track substructure and use of resilient track elements (grids and mats) have a significant influence on the engineering behaviour of ballasted rail track. Bearing capacity analyses of track substructure including ballast and subgrade are presented. The prefabricated vertical drains can decrease the excess pore water pressure and continue to dissipate excess pore water pressure during the rest period. This dissipation of pore water pressure during the rest period made the track more stable for the next train passage (loading stage).

The findings of the Bulli Study demonstrated the potential benefits of using a geocomposite to minimise the deformation and degradation of rail tracks. The recycled ballast performed satisfactorily compared to the fresh ballast due to its broader gradation. The results of the Singleton Study showed that geogrids with an optimum aperture size can significantly reduce deformations of ballast layer by providing improved interlock with the particles. The effectiveness of geosynthetics appeared to increase, as the stiffness of the subgrade decreased. Empirical approaches to relate ballast vertical strains with the number of load cycles are useful for practicing engineers. Results of large scale laboratory tests and field trials demonstrated benefits of using geosynthetics, shock mats and vertical drains for more resilient and durable track substructure.

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