2013

Rail track infrastructure for enhanced speed - analysis design and construction challenges

Buddhima Indraratna
University of Wollongong, indra@uow.edu.au

Sanjay Shrawan Nimbalkar
University of Wollongong, sanjayn@uow.edu.au

Jayan Sylaja J S Vinod
University of Wollongong, vinod@uow.edu.au

Publication Details
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Abstract
Rail is one of the largest transportation modes offering freight and passenger traffic in rapidly developing nations, including India. Conscious efforts to improve productivity, modernization and technology upgrading have led to an impressive growth in railways. Large-scale physical modeling, sophisticated numerical modeling and full-scale field monitoring often provide significant knowledge to better understand track performance and to extend the current state-of-the-art in design. A series of large-scale laboratory tests were conducted to establish relationships between (i) ballast breakage and train speed, (ii) ballast fouling and strength and (iii) interface strength and geogrids. Comprehensive field trials were carried out on instrumented track sections in Bulli and in Singleton, New South Wales, Australia. This keynote paper provides an insight to design and construction challenges of rail tracks capturing the effects of particle degradation, fouling, geosynthetics, and increased train speed.

Keywords
design, construction, challenges, track, enhanced, analysis, rail, speed, infrastructure

Disciplines
Engineering | Science and Technology Studies

Publication Details

This conference paper is available at Research Online: http://ro.uow.edu.au/eispapers/1858
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INTRODUCTION

A typical ballasted track is composed of graded layers of rock fragments placed below timber or concrete sleepers (ties). Compared to other substructure components, ballast is subjected to high stresses under heavy freight and high-speed passenger trains. In recent times, ballast degradation and track deformation under these high stresses have led to significantly high maintenance costs. In many countries including North America, Australia, China and India, a large proportion of the maintenance budget is usually spent on ballast replacement and related activities.

Unless the proper role of geotechnical parameters which controls stress-strain behaviour of ballast under repeated loads is carefully assessed, the cost of track maintenance will continue to escalate. The breakage of asperities and crushing of rock aggregates under high cyclic and impact loading, causes differential track settlement that adversely affects track geometry and results in more frequent maintenance [1, 2, 3, 4]. The large lateral deformation of ballast due to insufficient track confinement, fouling of ballast by coal from freight trains, soft formation soils (clay pumping) as well as ballast breakage, are the primary causes of track deterioration.

The base reinforcement applications of geosynthetics in rail track have been studied in the past through laboratory studies [5, 6, 7, 8, 9, 10, 11, 12, 13] and field studies [14, 15, 16, 17, 18, 19, 20]. The effectiveness of geogrid reinforcement is highly dependent on the stiffness and size of the apertures. A 30% increase in the stiffness of geogrid has resulted in up to 20% smaller vertical strain on the ballast [21], while the apertures providing the best frictional interlock are influenced by the median particle size ($D_{50}$) of ballast [22].

The infiltration of fouling material decreases the void ratio and restricts track drainage. Extensive field trials on sections of instrumented railway track at town of Bulli and Singleton, New South Wales (NSW), Australia have been conducted. This paper describes the results of large-scale laboratory tests, analytical-numerical methods and full-scale field trials.

DESIGN PRACTICES AND LOGISTICS FOR HIGH SPEED TRAIN

Macroeconomic statistics reveal that transport alone accounts for 3 to 8% of the GDP of countries in Asia and the Pacific [23]. Public investment in
transport typically accounts for 2.0 to 2.5% of GDP [24] and may rise as high as 4% or more in countries modernizing or building new transport infrastructure. High-speed rail corridors have proven beneficial in establishing links to major cities, airports and harbors for increased freight opportunities and regional benefits. In Australia and India, substantial upgrading in transportation infrastructure is essential for long distance routes and high speed services.

Rail tracks undergo millions of loading cycles with varying magnitudes and frequencies during their service life. The transfer of moving wheel loads at higher speeds ($V > 150$ km/h) results in a significant increase in track damage because the quasi-static response at relatively low speeds transforms to a dynamic (increases vibrations) state at elevated speeds. In trials conducted in France and Japan, high speed tracks on flexible ballast foundations have proven to be the safest and most economical, in contrast to rigid concrete pavements that may cause a sudden change of gradient between slabs, thus elevating the risk of derailment of long carriages. A field trial on the Est-Européenne line in France demonstrated that speeds exceeding 400 km/h on ballast beds would be feasible, if particle gradations could be optimised to minimise damage to the track. The construction of a proposed high speed train network (length > 1600 km, speed 350 km/h) in Australia is estimated to cost up to $120 billion. This design faces significant geotechnical challenges due to soft clay deposits in coastal regions, ballast fouling, and track drainage.

Over many years, the design of railway tracks has remained almost unchanged, even though the demand for increased train speed and heavier freight traffic has increased. Several shortcomings, such as quasi-static loading instead of cyclic, overly uniform particle sizes, while ignoring particle breakage, prevail in the current design practices. This often leads to high track maintenance and interruption of rail services. Under the influence of cyclic loading, mechanisms of particle breakage, interface behaviour, and time dependent changes of the fouling and track drainage have to be studied in detail.

EFFECTS OF BALLAST BREAKAGE AND CONFINING PRESSURES ON TRACK STABILITY

Ballast is usually composed of medium to coarse gravel sized aggregates (10 - 60 mm). Ballast should possess angular particles of high specific gravity, high shear strength, high toughness and hardness, high resistance to weathering, and subjected to a minimum of hairline cracks [25].

Ballast Breakage

The ballast degradation is a complex mechanism that usually initiates with the breakage of asperities (sharp corners/projections), followed by complete crushing of weaker particles under further loading. Indraratna et al. [2] introduced a new Ballast Breakage Index (BBI) specifically for rail ballast to quantify the extent of degradation, and it is given by:

$$BBI = \frac{A}{A+B}$$

Where, $A$ is the area due to shift in the particle size distribution curve before and after loading, and $B$ is the area between the arbitrary boundary of maximum breakage and the final particle size distribution.

Confining Pressure

Indraratna et al. [2] proposed that the degradation of ballast under cyclic loading can be categorised into three distinct zones, namely: (I) the Dilatant Unstable Degradation Zone (DUDZ: $\sigma_3' < 30$ kPa), (II) the Optimum Degradation Zone (ODZ: $\sigma_3' = 30 - 75$ kPa), and (III) the Compressive Stable Degradation Zone (CSDZ: $\sigma_3' > 75$ kPa), as shown in Fig. 1.

![Fig. 1 Effect of confining pressure on particle degradation(data sourced from Indraratna et al. [2])](image-url)
In the DUDZ zone, specimens are characterised by limited particle-to-particle contact areas and undergo considerable degradation as a result of shearing and attrition of angular projections. In the ODZ region, the particles are held together with enough lateral confinement to provide increased inter-particle contact areas, which in turn reduces the risk of breakage.

At higher $\sigma'_3$ (CSDZ region), the particles are forced against each other, which limits any sliding or rolling, and therefore breakage is significantly increased. Due to the large lateral forces being applied to the samples in this region, volumetric compression is enhanced, which is partly attributed to the increased particle breakage.

**EFFECTS OF BALLAST FOULING ON TRACK DRAINAGE AND STABILITY**

Track substructure should be designed and constructed so as to drain the water into nearby drainage ditches or pipes, but fouling of ballast due to intrusion of fine material either from surface or subgrade impairs its drainage capacity. In the case of poor drainage, problems may occur in the track such as (i) reduced ballast shear strength, stiffness, and load bearing capacity, (ii) increased track settlement, (iii) softening of subgrade, (iv) formation of slurry and clay pumping under cyclic loading, (v) ballast attrition, and (vi) sleeper degradation [6, 25]. All these problems will degrade the performance of the track and demand additional maintenance.

**Ballast Fouling Assessment**

Several fouling indices are used in practice to measure fouling. Selig and Waters [6] have defined the fouling index as a summation of percentage (by mass) passing the 4.75 mm sieve and 0.075 mm sieve. They also proposed a percentage of fouling which is the ratio of the dry mass of fouled material passing through a 9.5 mm sieve to the dry mass of the fouled ballast sample. These mass based indices represent a false measurement of fouling when the fouling material has a different specific gravity. Therefore, Feldman and Nissen [26] defined the Percentage Void Contamination (PVC) as the ratio of bulk volume of fouling material to the volume of voids of the clean ballast. However, PVC does not consider the effects of the void ratio, gradation and specific gravity of the fouling material, which is the main factor affecting ballast drainage.

Therefore, a parameter Void Contaminant Index (VCI) has been proposed to incorporate effects of void ratio, specific gravity and gradation of fouling material and ballast [27, 28]:

$$VCI = \frac{(1+e_f) \times G_{sb} \times M_f \times 100}{G_{sf} \times M_b}$$ (2)

where $e_b$ is the void ratio of clean ballast, $e_f$ is the void ratio of fouling material, $G_{sb}$ is the specific gravity of the ballast material, $G_{sf}$ is the specific gravity of the fouling material, $M_f$ is the dry mass of the fouling material, and $M_b$ is the dry mass of clean ballast. For example, a value of VCI = 100% indicates that total voids in the ballast are occupied by the fouling material. More details of VCI including the field evaluation method are available elsewhere [28].

**Effects of Fouling on the Track Drainage**

**Laboratory Testing**

A series of large scale constant head permeability tests [29] were carried out on fouled ballast. Pulverised coal, clayey sand, and kaolin were mixed with ballast in different percentages to simulate fouling levels adequate for railways. The large scale permeability test chamber could accommodate specimens 500 mm in diameter and 500 mm high (Fig. 2). The fouled specimen was saturated for at least 24 hours prior to testing. A constant head was ensured with a steady flow subjected to a 1500 mm head of water by an adjustable overhead tank.

Figure 3 shows the variations of hydraulic conductivity of coal fouled and sand fouled ballast with VCI where the fouling material was distributed non-uniformly. As expected, the overall hydraulic conductivity of fouled ballast always decreased with an increase in VCI. The current test
Effects of Fouling on the Track Stability

In order to understand the effect of clay fines on the stress-strain and degradation behaviour of ballast, for different levels of fouling (VCI = 0 to 80%), a series of large scale monotonic triaxial tests were carried out for confining pressure ($\sigma_3$) in the range of 10-60 kPa. Owing to the larger physical dimensions of ballast, a large-scale triaxial apparatus that could accommodate a specimen of 600 mm in height and 300 mm in diameter was used (Fig. 4).

![Fig. 4](image)

A 7 mm thick cylindrical rubber membrane was used to confine the specimen. The Young’s modulus of the membrane was determined to be 4300 kPa following the method described earlier [30, 31]. The initial particle size distribution (PSD), density and void ratio of ballast were kept almost identical in all specimens and were selected in accordance with the recommended Australian Standards [32] to capture realistic track conditions. As discussed in earlier studies, the sample size ratio (i.e. diameter of the test specimen to the maximum particle size) exceeds 6, the sample size...

Fig. 3 Variation of the hydraulic conductivity with VCI (data sourced from Tennakoon et al. [28])

results showed that a 5% increase in the VCI decreased the hydraulic conductivity by a factor of at least 200 and 1500 for ballast contaminated by coal and clayey sand, respectively. However, this reduction in permeability would not significantly affect the minimum drainage capacity needed for acceptable track operations. Beyond a VCI of 75%, any further reduction in hydraulic conductivity becomes insignificant, because, it approaches the hydraulic conductivity of the fouling material.

**Fig. 2** Schematic diagram of large scale permeability test apparatus

**Fig. 4** Schematic illustration of large-scale triaxial chamber
effects become increasingly insignificant [33, 34]. Two specimen preparation methods were adopted for different levels of VCI as described below:

**Method 1: Clean Ballast**
The clean ballast specimen was divided into four equal portions. Each portion was placed inside the rubber membrane and compacted using a vibrating plate to a height of 150 mm. All layers were compacted until the final height of 600 mm was attained. The specimen was then kept under saturation for less than half hour.

**Method 2: Clay-Fouled Ballast**
The amount of clay needed for predetermined VCI was calculated for each test specimen. Then a quarter of the clay was mixed with a quarter portion of ballast using the concrete mixer, and then placed inside the cylindrical membrane. A vibrating plate was used to compact the specimen following the sequential procedure for the subsequent layers.

After preparing the test specimen, the outer cell chamber was placed and connected to the axial loading actuator. For all the tests, back pressure of 80 kPa was applied to obtain sample saturation with Skempton’s B value approaching unity (B > 0.98). During testing, the required membrane correction was carried out [30]. Sieve analysis was carried out to measure BBI at the end of each test.

**Stress-Strain Behavior**
Figure 5(a) illustrates the stress-strain behaviour of clay-fouled ballast (VCI = 10 and 80%) in contrast to fresh ballast (VCI = 0%) at increasing confining pressure. As expected when VCI increases, the peak deviator stress decreases significantly. Highly fouled ballast (VCI = 80%) shows an increasingly more ductile post-peak response. Figure 5(b) shows the volumetric strain changes with the axial strain for varying levels of fouling and increasing confining pressure. In the compression zone, the increasing VCI generally shows a reduced compression of the clay-fouled ballast specimen as the clay acting as filler of voids between the ballast aggregates.

For low levels of fouling (VCI @ 10%), an exception is observed for all three specimens indicating a slightly increased compression compared to clean ballast. This may be attributed to the small amount of clay that is coating the ballast grains as a lubricant, thereby facilitating the specimens to attain a slightly higher compression.

**Fig. 5** Stress-strain behaviour of clean and fouled ballast at confining pressures (σ_c) of (a) 10 kPa, and (b) 30 kPa, respectively (data sourced from Indraratna et al. [35])
With respect to dilation, the highly fouled specimens show a decrease in the rate and magnitude of dilation at axial strains exceeding @ 20%, while the increase in $\sigma_3$ from 10 to 30 kPa significantly suppresses dilation of all specimens. The addition of kaolin in sufficient quantities appears to contribute to a ‘binding’ effect that diminishes the tendency of the aggregates to dilate. Moreover, the specimens that are highly fouled (VCI = 80%) begin to dilate swiftly at a lower axial strain after showing a reduced compression initially compared to clean ballast.

The empirical relationship that represents the normalized shear strength of clay-fouled ballast may be used in the preliminary assessment of track conditions in view of track maintenance [35]:

$$q_{peak,f} = \frac{1}{q_{peak,b} + \beta \sqrt{VCI}}$$  \hspace{1cm} (3)

where, $q_{peak,b}$ and $q_{peak,f}$ are peak deviator stresses for clean and clay-fouled ballast respectively; $\beta$ is an empirical parameter.

**Shear Strength Envelope**

The clay-fouled ballast specimens exhibit a non-linear shear strength envelope similar to other rockfills [34, 36]. A non-linear shear strength envelope for clay fouled ballast can be presented in a non-dimensional form [35]:

$$\frac{\tau}{\sigma_c} = m \left( \frac{\sigma_n}{\sigma_c} \right)^n$$  \hspace{1cm} (4)

where, $\tau$, $\sigma_n$, and $\sigma_c$ are shear stress, effective normal stress and uniaxial compressive strength of the parent rock. The empirical coefficients $m$ and $n$ in Eq. (4) vary with VCI, and they are expressed as [35]:

$$m = 0.07 \left[ 1 + \tanh \left( VCI / 21.5 \right) \right]$$  \hspace{1cm} (5)

$$n = 0.56 \left[ 1 + 0.3 \tanh \left( VCI / 21.5 \right) \right]$$  \hspace{1cm} (6)

**Allowable Train Speeds**

The dynamic load ($P_d$) can be obtained from the American Railway Engineering Association method [37] as given below:

$$P_d = \left[ 1 + \frac{0.0052 V}{D_a} \right] \times P.$$  \hspace{1cm} (7)

Using Eqs. (3) and (7), the allowable train speed is determined as shown in Figs. 6(a) and 6(b). Figure 6(a) shows that the track can sustain small amount of fouling (VCI $\leq$ 15%) at $\sigma_3$ of 10 kPa for train speed of 70 km/h. However, when $\sigma_3$ exceeds 30 kPa, the track is stable at all levels of fouling. Figure 6(b) shows that for $\sigma_3$ greater than 30 kPa, fouling cannot exceed 50% VCI for train speed of 70 km/h. Nevertheless, increasing the confining pressure beyond 60 kPa makes the track stable enough against all levels of fouling. However, achieving a confining pressure of 60 kPa in the field is often impractical as this involves the use of sheet piles on either side of track or overly close sleeper spacing [3].

![Fig. 6](image_url) Allowable train speeds for (a) 25 t and (b) 30 t axle loads at different $\sigma_3$. 
Assessment of Coal-Fouled Ballast using DEM Approach

The shear strength and apparent angle of the shearing resistance of clean ballast and coal-fouled ballast was evaluated in a past study [38]. Tests were conducted using the large scale direct shear apparatus (Fig. 7). Coal fines were used as fouling material and the degree of fouling was measured in terms of VCI defined in previous section.

A three-dimensional discrete element method (DEM) was employed in Particle Flow Code (PFC\textsuperscript{3D}) to study the shear behaviour of fresh and coal fouled ballast in direct shear testing. ‘Clump logic’ in PFC\textsuperscript{3D} was incorporated in a MATLAB code to simulate irregular shaped particles in which various groups of spherical balls were clumped together in appropriate sizes to simulate ballast particles.

A large scale shear box (300 mm long \times 300 mm wide \times 200 mm high) separated horizontally into two equal boxes was simulated in PFC\textsuperscript{3D}. A free loading plate that allowed the particles to displace vertically during shearing was placed on the top boundary. This plate was used to apply a normal load and monitor normal displacement during shearing.

![Fig. 7 Direct shear box apparatus (after Indraratna et al. [38])](image)

The DEM simulation of this direct shear box for both clean and coal-fouled ballast (VCI = 40%), is shown in Fig. 8. Fouled ballast with VCI ranging from 20% to 70% were modeled by injecting a specified number of miniature spherical particles into the voids of fresh ballast.

![Fig. 8 Initial assembly of large-scale direct shear test with irregular shaped particles, (a) fresh ballast and (b) coal-fouled ballast (VCI = 40%) (Indraratna et al. [39], with permission from ASCE)](image)

DEM simulations were conducted to model fresh ballast subjected to three different normal stresses of 27 kPa, 51 kPa, and 75 kPa. The shear stress and volumetric changes were monitored during shearing. Figure 9 shows the plots of shear stress, shear strain and volumetric strain predicted by DEM, compared to the laboratory data reported...
earlier by Indraratna et al. [38]. The predicted results agreed with the laboratory test findings. The post peak behaviour of ballast was similar to other rockfill aggregates of comparable sizes [1, 36, 40].

**Fig. 9** Comparisons between the DEM simulation and experiment, (a) shear stress versus shear strain, (b) volumetric strain versus shear strain (Indraratna et al. [39], with permission from ASCE)

The DEM analysis shows a noticeable discrepancy in stress-strain curves, i.e. markedly decreased stress and retarded dilation shown by the experimental data at a shear strain of 4-8% compared to the predicted line. This difference may be attributed to some particle degradation that could not be accurately captured in the DEM simulation. Indeed, owing to the breakage of ballast aggregates, the reduction in shear strength would also be accompanied by a decrease in dilation.

**BEHAVIOUR OF COAL-FOULED BALLAST STABILISED WITH GEOGRID UNDER CYCLIC LOAD**

In order to study the effects of coal fines on the behaviour of fresh and fouled ballast, a series of triaxial tests employing the large scale Process Simulation Apparatus (PSA) was performed [41]. PSA can accommodate a ballast specimen 800 mm in length, 600 mm in width and 600 mm in height (Fig. 10).

**Fig. 10** Process simulation testing apparatus

A cyclic load was applied by a servo-dynamic hydraulic actuator at a frequency of 15 Hz with a maximum cyclic stress of 420 kPa. The hydraulic jacks mounted on the sides of the apparatus were used to provide confining pressures in two horizontal directions (perpendicular and parallel to the sleeper). The load cells connected to hydraulic jacks were used to control the confining pressures to a prescribed range. The initial stresses were kept constant (about 7 kPa) in the transverse direction (parallel to sleeper), and about 10 kPa along the longitudinal direction (perpendicular to sleeper). The geogrid was always at the base of the ballast.
layer. This was carried out in accordance with current industry practice in Australia. All tests were performed on the fresh ballast and coal-fouled ballast at different values of VCI. Tests were continued up to 500,000 load cycles.

Influence of Fouling and Geogrids on Relative Deformation Factors

Figure 11 shows the variation of relative deformation factors at 500,000 cycles with varying VCI. The relative deformation factors are determined as [41]:

\[ R_s = \frac{S_{\text{unreinforced}} - S_{\text{reinforced}}}{S_{\text{unreinforced}}} \times 100 \] (8)

\[ R_{h2} = \frac{S_{2\text{unreinforced}} - S_{2\text{reinforced}}}{S_{2\text{unreinforced}}} \times 100 \] (9)

\[ R_{h3} = \frac{S_{3\text{unreinforced}} - S_{3\text{reinforced}}}{S_{3\text{unreinforced}}} \times 100 \] (10)

A beneficial aspect of geogrid on reducing ballast deformation is clearly revealed by the values of \( R \) presented in Fig. 11. The positive effects of the geogrid reduced with an increase of VCI and became insignificant when VCI exceeded 40%. Geogrids can decrease the deformation of fresh ballast, but their effectiveness diminishes with an increase of VCI.

At VCI = 40% and beyond, coal fines occupy most of the ballast voids and geogrid apertures. This phenomenon prevents the ballast particles from effectively interlocking with the geogrid. A threshold value of VCI = 40% is proposed as a limiting value where the effect of geogrid becomes marginal and track maintenance would become mandatory exercise.

INFLUENCE OF TRAIN SPEED

In this investigation, the influence of train speed on the deformation and degradation of ballast during cyclic loading was studied using the large scale triaxial cylindrical equipment [42]. Latite ballast was thoroughly cleaned, dried, and compacted in four layers to a density of 1500 kg/m³. These specimens were then isotropically compressed to a confining pressure of 60 kPa. Cyclic tests up to 100,000 load cycles were conducted at 10 Hz, 20 Hz, 30 Hz, and 40 Hz frequency. Numerical model was developed in PFC²D to apply a stress controlled cyclic biaxial test at the desired frequency (\( f \)) and amplitude of cyclic loading. Sub-routines were developed to represent irregular ballast particles as shown in Fig. 12.
in $\varepsilon_a$ with $f$ was observed. For a particular value of $f$, $\varepsilon_a$ rapidly increased to maximum value (e.g. 6.1% at $N = 2500$ for $f = 10$ Hz) in the initial cycles, after which $\varepsilon_a$ attained a stable value at large $N$. The sudden increase in $\varepsilon_a$ at low values of $N$ could be attributed to the particle re-arrangement and corner breakage. In addition, it was evident that with an increase in $f$, higher values of $N$ are required to stabilise $\varepsilon_a$.

The influence of frequency ($f$) and number of load cycles ($N$) on ballast breakage is presented in Fig. 14. Ballast breakage is measured using BBI for different values of $f$ and $N$. The BBI is found to increase with $f$, but at lower values of $f$ (e.g. $f \leq 30$ Hz) BBI is not influenced by $N$. This clearly highlights that most ballast breakage occurs within 1000 cycles.

A significant increase in BBI can be observed in the range $10$ Hz $\leq f \leq 20$ Hz, a marginal increase in BBI in the zone $20$ Hz $< f \leq 30$ Hz, and a rapid increase in BBI for higher range ($f > 30$ Hz).

It has been observed from the laboratory tests that in the zone $10$ Hz $\leq f \leq 30$ Hz, corner breakage is more pronounced, while considerable splitting of ballast particles can be observed at $f > 30$ Hz. In the range $20$ Hz $\leq f \leq 30$ Hz, the ballast becomes denser under cyclic loading without much additional breakage. However, more pronounced breakage occurs as $f > 30$ Hz. High permanent
deformation observed at higher frequencies can be attributed to an increase in particle degradation.

The cumulative bond breakage ($B_r$) is defined as a percentage of bonds broken compared to the total number of bonds. Figure 15 shows the variation of $B_r$ at different $f$ and $N$. It is observed that $B_r$ increases with increase in $f$ and $N$. Most of the bond breakages occurred during the initial cycles of loading, causing higher permanent $\varepsilon_a$. Once the bond breakage ceases, there is marginal increase in $\varepsilon_a$. This clearly highlights that particle breakage is one of the major sources responsible for permanent deformation.

STABILISING BALLASTED TRACK USING GEOGRIDS: DIRECT SHEAR TEST APPARATUS

In order to investigate the effect of the size of the geogrid aperture on the strength of the ballast-geogrid interface for different types of geogrids, a series of large scale direct shear tests were conducted [22]. Fresh latite basalt with recommended gradations ($D_{50} = 35$ mm) and geogrids with different aperture sizes ($A$) were used for this study. Their physical characteristics and technical specifications are given elsewhere [22]. A geogrid was placed at the interface of the upper and lower sections of the shear box assembly with the machine direction placed parallel to the direction of shearing. Tests were conducted at normal stresses of 26.3, 38.5, 52.5, and 61 kPa, using a shear rate of 2.75 mm/min. All tests were conducted to a maximum shear displacement of 36 mm.

The behaviour of the ballast-geogrid interface could be examined on the basis of the interface efficiency factor ($\alpha$) which is defined as the ratio of the shear strength of the interface to the shear strength of the soil. Figure 16 shows the variation of $\alpha$ with $A/D_{50}$ ratio. It was observed that $\alpha$ increased with $A/D_{50}$ until it attained a maximum value of 1.16 at $A/D_{50}$ of 1.21, and then decreased towards unity as $A/D_{50}$ approached 2.5. The value of $\alpha < 1$ indicated that the particles were not interlocked, whereas when $\alpha > 1$ they were, which effectively increased the shear strength. Based on this variation of $\alpha$, the ratio $A/D_{50}$ was classified into three primary zones, as explained below:

**Feeble Interlock Zone (FIZ)**

In this zone ($0.95 > A/D_{50} > 0$), the particle-grid interlock was weaker than the inter-particle interaction achieved without geogrid, because, the particle-grid interlock was only attributed to smaller particles ($< 0.95D_{50}$) compared to the particle-particle interlock with respect to all sizes. An examination after testing showed insignificant particle breakage, which suggests the interface failure originated from a loss of particle-grid interlock during shearing.

**Optimum Interlock Zone (OIZ)**

In this zone ($1.20 > A/D_{50} > 0.95$), the interlocking of relatively larger particles occurred, which contributed to values of $\alpha$ exceeding unity. The value of $\alpha$ attained a maximum of 1.16 at an optimum $A/D_{50}$ ratio of about 1.20. An examination after testing showed there were many broken particles at the interface, suggesting
that the failure was caused by the breakage of initially interlocked particles.

**Diminishing Interlock Zone (DIZ)**

In this zone \((A/D_{50} > 1.20)\), the values of \(\alpha\) were greater than unity but the degree of interlocking decreased rapidly, leading to a reduction in \(\alpha\) with an increasing \(A/D_{50}\) ratio. It was observed that \(\alpha\) decreased to almost unity when \(A/D_{50}\) exceeded 2.50. This implies that the interface responds in a similar manner as unreinforced ballast as the apertures increase in magnitude in relation to the size of the ballast particles.

The minimum and maximum size apertures of geogrid required to achieve maximum efficiency was 0.95\(D_{50}\) and 2.50\(D_{50}\), respectively. For all practical purposes, the optimum size aperture of geogrid can be considered to be 1.15-1.3\(D_{50}\).

**STABILISING BALLASTED TRACK USING GEOGRIDS: MODIFIED PROCESS SIMULATION APPARATUS**

The modified process simulation apparatus (MPSA) was used to simulate lateral deformations of load bearing ballast [13]. The modification involved the replacement of the side wall with a setup of five movable plates each of 64 mm in height (Fig. 17). Cyclic tests were carried at a frequency of 20 Hz that corresponds to a train speed of about 150 km/h, for an axle spacing of 2.02 m. The geogrids were selected based on the interface efficiency factor \((\alpha)\), obtained from direct shear tests explained in previous section. A vertical stress of 460 kPa corresponding to an axle load of 225 kN was applied by means of a dynamic actuator. A confining pressure of 10 kPa was applied to the modified side wall.

**Lateral Spread Reduction Index (LSRI)**

The improved performance of geogrid-reinforced ballast can be represented by a normalized parameter called lateral spread reduction index (LSRI), defined as [13]:

\[
LSRI = \frac{S_{(\text{unreinforced})} - S_{(\text{reinforced})}}{S_{(\text{unreinforced})}}
\]  

(11)

where, \(S_{(\text{unreinforced})}\) and \(S_{(\text{reinforced})}\) are lateral displacements of unreinforced and reinforced ballast, respectively. LSRI of zero indicates unreinforced condition whereas a value of unity represents complete arrest of particle spreading. On the other hand, a negative value of LSRI indicates an increase in lateral displacement due to the inclusion of reinforcement.

**Effects of \(A/D_{50}\) on LSRI**

Figure 18 shows the variation of average LSRI along the ballast depth with the ratio \(A/D_{50}\), for both the geogrid placement positions (i.e. at \(z = 0\) and 65 mm). The values of \(A/D_{50}\) for different geogrids have been adapted from Indraratna et al. [22], wherein the \(A/D_{50}\) of geogrid G2 was based on the diameter of the largest circle inscribed within the triangular aperture.

It was noted that for geogrid placed at subballast-ballast interface, the average LSRI increases significantly from 0.06 to 0.25 as \(A/D_{50}\) increases.
from 0.6 to 1.20. This may be attributed to the better ballast/geogrid interlock attained as the geogrid aperture size increases for a given ballast size. The geogrid G4 with $A/D_{50}$ of 1.21 gives a maximum LSRI of 0.25. However, with the further increase in $A/D_{50}$ from 1.21 to 1.85 the average LSRI decreases from 0.25 to 0.20. For the geogrid placed at 65 mm above the subballast, the average LSRI follows an almost similar trend with $A/D_{50}$ except that the geogrid G2 exhibits a negative LSRI.

![Fig. 18 Variation of average LSRI with $A/D_{50}$ (data sourced from Indraratna et al. [13]).](image)

**Effects of LSRI on Settlement and BBI**
It was observed that as the average LSRI increases both the settlement and the particle breakage reduce significantly. The settlement and BBI decrease from about 23.5 to 9.8 mm and 9.89 to 4.6%, respectively, as the average LSRI increases from zero to 0.37. A linear increase in settlement was observed with the increase in BBI as given by [13]:

$$S_{v\text{ reinforced}} = 0.0243 BBI - 1.74$$  \hspace{1cm} (12)

Equation (12) also highlights that a portion of settlement also occurs due to the particle breakage.

**FIELD STUDY: BULLI**
In order to assess the in-situ response of composite track substructure to repeated wheel loads of moving trains and the advantages of using geosynthetics in fresh and recycled ballast, a field trial was undertaken at Bulli in the State of New South Wales (NSW), Australia [16, 43].

**Track Construction**
The proposed location for track construction was selected between two turnouts at Bulli, along the south coast of NSW. The instrumented section of track was divided into four 15 m long sections. The load bearing ballast and subballast were 300 mm and 150 mm thick, respectively. A layer of geocomposite was used below the fresh and recycled ballast (Fig. 19).

![Fig. 19 Installation of geocomposite at Bulli.](image)

The grain size distributions of various materials and technical specifications of geosynthetics used during the track construction are given in Indraratna et al. [16].

**Fig. 20 Installation of settlement pegs and displacement transducers in experimental sections of track at Bulli (data sourced from Indraratna et al. [27], with permission from ASCE).**
Maximum cyclic stresses under rail, $\sigma_v, \sigma_h$ (kPa)

Depth below base of sleeper, $z$ (mm)

$N = 9.1 \times 10^5$

**Fig. 21** Vertical and horizontal maximum cyclic stresses ($\sigma_v, \sigma_h$) (data sourced from Indraratna et al. [16], with permission from ASCE).

**Track Instrumentation**

The performance of each section of track under the repeated loads was monitored using sophisticated instruments. The vertical and horizontal stresses were measured by pressure cells. Vertical deformations of the track at different sections were measured by settlement pegs and lateral deformations were measured by electronic displacement transducers connected to a computer controlled data acquisition system. The settlement pegs and displacement transducers were installed at the top and bottom of load bearing ballast layer (Fig. 20).

**Peak Stresses in Ballast**

Figure 21 shows the vertical and horizontal maximum cyclic stresses ($\sigma_v, \sigma_h$) recorded in an unreinforced Section, under the rail and the edge of the sleeper, from a passenger train travelling at 60 km/h. The large vertical stresses and relatively small lateral stresses caused large shear strains in the track.

**Average Lateral Deformations**

The average lateral deformations were determined from the mean of measurements between the sleeper and ballast, and between the ballast and sub-ballast. The average lateral deformations are plotted against the number of load cycles ($N$) in Fig. 22. The recycled ballast exhibited less vertical and lateral deformations, because of its moderately graded particle size distribution ($C_u = 1.8$) compared to very uniform fresh ballast ($C_u = 1.5$).

**Fig. 22** Average lateral deformations of the ballast (data sourced from Indraratna et al. [16], with permission from ASCE).

The apertures of the geocomposite offered a strong mechanical interlock with the ballast, forming a highly frictional interface. The apertures of the geogrid offered strong mechanical interlocking with the ballast. The capacity of the load bearing ballast to distribute loads was improved by the placement of the geocomposite layer, which substantially reduced settlement under high repeated loading.

**FIELD STUDY: SINGLETON**

To investigate the performance of different types of geosynthetics to improve overall stability under in-situ track conditions, an extensive study was undertaken on instrumented track sections near Singleton, NSW [17, 18].

**Track Construction**

Nine experimental sections were included in the Third Track while it was under construction, on three different types of sub-grades, including (i) the relatively soft general fill and alluvial silty clay deposit (Sections 1-4 and Section A), (ii) the intermediate cut siltstone (Sections 5 and C), and (iii) the stiff reinforced concrete bridge deck...
supported by a piled abutment (Section B), as shown in Table 1. A layer of geosynthetics was placed below the fresh ballast (Fig. 23). The details of track construction and material specifications can be found in Indraratna et al. [17].

**Table 1** Reinforcement at experimental sections using geogrids, geocompolistes, and shock mats.

<table>
<thead>
<tr>
<th>Section</th>
<th>Location (Chainage)</th>
<th>Type of Geosynthetics</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>234.66</td>
<td>Geogrid 1</td>
</tr>
<tr>
<td>2</td>
<td>234.40</td>
<td>Geogrid 2</td>
</tr>
<tr>
<td>3</td>
<td>234.22</td>
<td>Geogrid 3</td>
</tr>
<tr>
<td>4</td>
<td>234.12</td>
<td>Geocomposite</td>
</tr>
<tr>
<td>B</td>
<td>232.01</td>
<td>Shock mat</td>
</tr>
<tr>
<td>5</td>
<td>228.44</td>
<td>Geogrid 3</td>
</tr>
</tbody>
</table>

**Track Instrumentation**

Different types of equipment were used as listed below. Strain gauges were used to study mobilised strains along the layers of geogrid. Traffic induced vertical stresses were monitored by pressure cells. Transient deformations of the ballast were measured by five potentiometers mounted on a custom built aluminum frame. Settlement pegs were installed between the sleeper and ballast and between the ballast and sub-ballast to measure vertical deformations of the ballast. Electrical analogue signals from the strain gauges, pressure cells, and potentiometers were obtained using a mobile data acquisition system.

**Settlements and Strain of Ballast**

Settlement ($S_v$) and strains ($\varepsilon_v$) of ballast layer after $N = 230,000$ load cycles are reported in Table 2. The vertical settlements of sections with reinforcement are generally smaller than those without reinforcement. This phenomenon is mainly attributed to interlocking between the ballast particles and grids, thus creating larger track confinement. When Sections A, B, and C are compared, the results indicate that $S_v$ and $\varepsilon_v$ are larger when the subgrade stiffness becomes smaller, i.e. $S_v$ are smallest on the concrete bridge deck and largest at the alluvial deposit.

**Table 2** Settlements and strains in ballast.

<table>
<thead>
<tr>
<th>Section</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_v$ (mm)</td>
<td>16.3</td>
<td>21.2</td>
<td>20.6</td>
<td>14.8</td>
<td>16.0</td>
</tr>
<tr>
<td>$\varepsilon_v$ (%)</td>
<td>5.4</td>
<td>7.1</td>
<td>6.9</td>
<td>4.9</td>
<td>5.3</td>
</tr>
</tbody>
</table>

It is also observed that geogrid is more effective in terms of reducing track settlement for softer subgrades. Similar observation was found from full scale prototype testing conducted elsewhere [44]. Geogrid at Section 4 performed better, although the tensile strength does not differ much with other types. This is attributed to optimum aperture size (40 mm) which would enable better interlocking between the ballast particles and geogrid.

**Longitudinal and Transverse Strains Accumulated in Geogrids**

Accumulated longitudinal ($\varepsilon_l$) and transverse ($\varepsilon_t$) strains after $N = 230,000$ load cycles are given in Table 3. The transverse strains were generally larger than longitudinal strains, which is attributed to the relative ease for lateral spreading of the ballast. It was also observed that $\varepsilon_l$ and $\varepsilon_t$ are mainly influenced by the subgrade deformation.

The strains of geogrid at Section 4 were relatively large although its higher stiffness could have resulted in smaller strains. This is because the thick general fill underwent large lateral deformations shortly after the track was commissioned. Induced transient strains in both longitudinal and transverse directions due to the passage of trains (axle load of
30 t) travelling at 40 km/h were of magnitude 0.14-0.17%.

Table 3 Accumulated longitudinal and transverse strains in geogrids.

<table>
<thead>
<tr>
<th>Section</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon_l$ (%)</td>
<td>0.80</td>
<td>0.78</td>
<td>0.92</td>
<td>0.61</td>
<td>0.60</td>
</tr>
<tr>
<td>$\varepsilon_t$ (%)</td>
<td>0.85</td>
<td>1.50</td>
<td>0.85</td>
<td>0.80</td>
<td>1.80</td>
</tr>
</tbody>
</table>

Traffic Induced Vertical Stresses
The vertical stresses ($\sigma_v$) due to the passage of trains with an axle load of 30 tonnes travelling at about 40 km/h were about 280 kPa at Section B (mat-deck interface) and between 30 to 40 kPa at Sections 1, 5, A, and C (ballast-sub-ballast interface). Vertical stresses at the sleeper-ballast interface of the latter sections were between 170 to 190 kPa, which indicate that the traffic-induced stresses were considerably larger in the track with a stiffer subgrade.

The larger stresses also caused much more breakage of individual particles of ballast, as was anticipated. The ballast breakage index (BBI) after $N = 750,000$ load cycles for Sections B was 17%, while Sections A and C were 9.8% and 13.1%, respectively. This finding appears to contradict the general perception that ballast subjected to higher stresses (Section B) would undergo larger settlements and vertical strains due to a larger extent of particle breakage [3, 4, 25]. This is because the ballast at Section B was confined within the barriers of the Mudies Creek bridge, which meant that the ballast could not spread laterally. At Sections A and C however, ballast was allowed to expand more freely in a horizontal direction, and as a result, a larger vertical settlement was observed. This observation also confirms that the ability of ballast to expand horizontally also influences the magnitude of track settlement as well as the degree of ballast breakage.

CONCLUSIONS
This keynote paper presented results of large scale laboratory tests, numerical modeling as well as the findings from a full-scale instrumented track. The detrimental effects of fouling on the strength and drainage characteristics were assessed using large-scale triaxial shear tests. Based on the laboratory findings, a novel empirical relationship between the peak deviator stress and VCI was proposed with the aim of assisting the preliminary track assessment. Using this relationship, the recommended train speeds for fouled ballast was also proposed. While the geogrid reduces the deformation of ballast due to effective interlocking occurring at the ballast geogrid interface, coal fines increase the ballast deformation. Geogrid was more useful at reducing the breakage of ballast when the VCI was less than 40%, but beyond that its effect was found to be marginal.

The triaxial tests showed that permanent deformation and degradation increased with the frequency and number of cycles, but in a $20 \text{ Hz} \leq f \leq 30 \text{ Hz}$ zone, cyclic densification was observed. The DEM based micro-mechanical investigation showed that most particles break during the initial cycles, which leads to higher initial axial strains.

It was also observed that the normalised aperture ratio ($A/D_{50}$) had a profound influence on the interface efficiency factor ($\alpha$). The best size geogrid aperture to optimise the interface shear strength was $1.20D_{50}$. Lateral spread reduction index (LSRI) was proposed to assess the deformations of geogrid-reinforced ballast. It was shown that LSRI is influenced by the type of geogrid. For geogrids placed at the subballast-ballast interface, the LSRI varies from 0.06 to 0.25. However, LSRI increases significantly and attains a maximum value of 0.37 for geogrid placed at 65 mm above the subballast. Both ballast breakage and associated settlements exhibited a significant reduction with the increase in LSRI.

The results of the Bulli case study indicated that the use of geocomposite as reinforcing elements for recycled ballasted tracks proved to be a feasible and effective alternative. The test results demonstrated that geocomposite was able to reduce the lateral deformation of fresh ballast by about 50% and recycled ballast by 10%. The results of
the Singleton study showed that the effectiveness of the geosynthetic reinforcement increased as the subgrade decreased in stiffness. The strains accumulated in the geogrids were influenced by the placement density of the ballast and deformation of the subgrade. The findings of these field studies allows for a better assessment of the ability of geosynthetic reinforcement, as well as more economical and effective design and maintenance of ballasted rail tracks.

ACKNOWLEDGEMENTS

The authors are grateful to the CRC for Rail Innovation (established and supported under the Australian Government’s Cooperative Research Centres program R3.106 & R3.117) for the funding of this research. The authors express their sincere thanks to the Australian Research Council, Sydney Trains (previously RailCorp), Australian Rail Track Corporation (ARTC) and Aurizon (previously Queensland Rail National) for their continuous support. The Authors would like to thank Dr Pongpipat Anantanasakul (Lecturer, Mahidol University, Thailand) for his help during the tenure of his postdoctoral fellowship at the University of Wollongong. A number of past PhD students, namely Dr Nayoma Tennakoon, Dr Sd K Karimullah Hussaini, Dr Ngoc Trung Ngo, Dr Joanne Lackenby, Dr Wadud Salim and Dr Pramod Kumar Thakur have participated to the contents of this paper and their contributions are greatly acknowledged. The Authors would also like to thank A/Professor Cholachat Rujikiatkamjorn (University of Wollongong, Wollongong). The assistance of Mr David Christie (formerly Senior Geotechnical Consultant, RailCorp), Mr Tim Neville (ARTC), Mr Michael Martin (Aurizon), Mr Damien Foun (Aurizon), and Mr Sandy Pfeiffer (formerly Senior Geologist, RailCorp) is gratefully acknowledged. The authors would like to thank Mr Alan Grant (laboratory manager), Mr Cameron Neilson and Mr Ian Bridge (technical staff) at the University of Wollongong for their assistance throughout the period of this study. The on-site assistance provided by Mr David Williams of ARTC, Carol Bolam, Tony Miller, and Darren Mosman of Hunt8r Alliance (Newcastle) is much appreciated.

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