Stabilisation of ballast and subgrade with geosynthetic grids and drains for rail infrastructure

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Stabilisation of Ballast and Subgrade with Geosynthetic Grids and Drains for Rail Infrastructure

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Synopsis: Rail tracks are often placed on ballast that is intended to provide a free draining and sound load bearing platform to ensure track stability and maintain proper alignment. In recent years, the increased demand of heavier and faster trains has posed greater challenges to the railway industry for improving efficiency and track stability while reducing maintenance costs. High traffic induced stresses always result in large plastic deformations and degradation of ballast, which in turn, leads to significant loss of track stability. The deformation and degradation of ballast are influenced by factors such as the amplitude and number of load cycles, gradation of aggregates, track confining pressure and the angularity and fracture strength of individual particles. Geosynthetic materials can also play an important role in improving the efficiency and performance of a ballasted rail track. This paper highlights laboratory testing and field trials carried out to demonstrate the beneficial use of geosynthetic grids and drains for rail infrastructure. The use of numerical models for analysing and predicting track behaviour is also elucidated.

Keywords: ballast, geosynthetics, rail track, deformations, fouling.

1. Introduction

Rail tracks form the largest worldwide network catering for quick and safe, public and freight transportation. In spite of recent advances in rail geotechnics, the correct choice of ballast for rail track foundation is still considered critical because aggregates progressively deteriorate under heavy cyclic loading. The excessive deformations and degradations of the ballast, unacceptable differential settlement of track and pumping (liquefaction) of underlying soft subgrade soils necessitate frequent and costly track maintenance works. Hundreds of millions of dollars are spent each year for the construction and maintenance of rail tracks in large countries like the USA, Canada and Australia. A proper understanding of load transfer mechanisms and their effects on track deformations are essential prerequisites for designing new tracks and for rehabilitating existing ones. The cost of track maintenance can be significantly reduced if the geotechnical behaviour of rail substructure, in particular the ballast layer, is better understood. The varied uses of geosynthetics beneath the ballast layer as drainage, providing internal confinement and to separate ballast from subballast, are highly desirable.

A field trial was conducted on a section of instrumented railway track in the town of Bulli, New South Wales (NSW), Australia to study the effectiveness of a geocomposite (combination of biaxial geogrid and non-woven polypropylene geotextile) installed at the ballast-capping interface. The relative performance of moderately graded recycled ballast in comparison with traditionally used very uniform fresh ballast was also evaluated. A large-scale direct shear test was conducted to study the performance of geogrid reinforced ballast under various fouling conditions. Another field trial at Sandgate, NSW, Australia has revealed that relatively short prefabricated vertical drains (PVDs) between 6 to 8 m in length can be adequate to dissipate train induced pore pressures, limit the lateral movements and increase the shear strength and bearing capacity of the soft formation. If significant initial settlement of estuarine deposits cannot be curtailed in terms of routine maintenance practices (e.g. in new railway tracks, continuous ballast packing may be required), the rate of settlement can be controlled by optimising the drain spacing and the drain installation pattern. In this way, while the settlements are controlled to an acceptable level, the reduction in lateral strains and gain in shear strength of the soil beneath the track improve its stability significantly. In this paper, predictive and design models for practitioners in view of adopting user-friendly analytical and numerical approaches simulating field track behaviour are presented.
2. Geotechnical Behaviour of Ballast

Ballast forms the largest component of a rail track by weight and volume. Ballast materials usually include dolomite, rheolite, gneiss, basalt, granite and quartzite [1]. It is composed of medium to coarse gravel sized aggregates (10 - 60 mm) with a smaller percentage of cobble-sized particles. Good quality ballast should possess angular particles, have a high specific gravity, a high shear strength, a high toughness and hardness, a high resistance to weathering, a rough surface, and a minimum of hairline cracks [2, 3].

The main functions of ballast are: distributing and damping the loads received from sleepers, producing lateral resistance, and providing rapid drainage [4]. It could be argued that for high load bearing characteristics and maximum track stability, ballast needs to be angular, well graded, and compact, which in turn reduces drainage. Therefore the use of geosynthetics as a suitable alternative for drainage and separation of ballast and subballast needs to be investigated. In this paper the geotechnical properties of ballast are discussed. The effect of geosynthetics on rail track performance for both fresh and recycled ballast is discussed. It is shown that using geosynthetics with special characteristics in the track bed improves its performance significantly.

2.1 Shear Strength

The shear strength of granular materials is generally assumed to vary linearly with the applied stress and the Mohr-Coulomb theory is usually used to describe conventional shear behaviour. The shear strength of ballast is a function of confining pressure, and is highly non-linear at high stresses [5, 6]. Indraratna et al. [7] proposed a non-linear strength envelope obtained during the testing of granular media at low normal stresses. This non-linear shear stress envelope is represented by the following equation:

\[ \frac{\tau}{\sigma_c} = m \left( \frac{\sigma'_n}{\sigma_c} \right)^n \]

where \( \tau \) is the shear stress at failure, \( \sigma_c \) is the uniaxial compressive stress of the parent rock determined from the point load test, \( m \) and \( n \) are dimensionless constants, and \( \sigma'_n \) is the effective normal stress. The non-linearity of the stress envelope is governed by the coefficient \( n \). For the usual range of confining pressures (below 200 kPa) for rail tracks, \( n \) takes values in the order of 0.65 - 0.75. A large-scale cylindrical triaxial apparatus, which could accommodate specimens of 300 mm diameter and 600 mm high (Figure 1), was used by Indraratna et al. [2] to verify the non-linearity of shear stress. The results of his study associated with latite basalt in a normalised form are plotted in Figure 2, with results obtained from other researchers [8, 9, 10].

![Figure 1. Cylindrical triaxial apparatus with dynamic actuator design at UoW](image1)

![Figure 2. Normalised shear strength for latite basalt aggregates [2]](image2)

2.2 Ballast Breakage

Railway tracks are severely affected by the degradation of ballast particles [3]. Their breakage under load is a complex mechanism that usually starts at the inter-particle contacts (i.e. breakage of asperities), followed by a complete crushing of weaker particles under further loading. A rapid fragmentation of
particles and subsequent clogging of voids with fines is commonly observed in overstressed railway foundations. The degradation of aggregate is the primary cause of contamination, and accounts for up to 40% of the fouled material [11]. Generally, the main factors that affect breakage can be divided into three categories: (a) properties related to the characteristics of the parent rock (e.g. hardness, specific gravity, toughness, weathering, mineralogical composition, internal bonding and grain texture); (b) physical properties associated with individual particles (e.g. soundness, durability, particle shape, size, angularity and surface smoothness); and (c) factors related to the assembly of particles and loading conditions (e.g. confining pressure, initial density or porosity, thickness of ballast layer, ballast gradation, presence of water or ballast moisture content, cyclic loading pattern including load amplitude and frequency). The effects of some of these factors are demonstrated in this paper.

Marsal’s method [12] was used to investigate the breakage of rock-fill materials in which the volume of particle breakage while loading a specimen is defined by the changes in particle size distribution curves measured before and after loading. Marsal [12] introduced a breakage index $B_g$, which is the sum of the difference in percentage retained on sieves, having the same sign.

Indraratna et al. [13] introduced an alternative ballast breakage index (BBI) based on particle size distribution (PSD) curves. The ballast breakage index (BBI) is calculated on the basis of change in the fraction passing a range of sieves, as shown in Figure 3. The increase in degree of breakage causes the PSD curve to shift further towards the smaller particles size region on a conventional PSD plot. The area $A$ between the initial and final PSD increases results in a greater BBI value. BBI has a lower limit of 0 (no breakage) and an upper limit of 1. By referring to the linear particle size axis, BBI can be calculated by using following equation.

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

where, $A$ is the area as defined previously, and $B$ is the potential breakage or area between the arbitrary boundary of maximum breakage and the final particle size distribution.

2.3 **Effect of Confining Pressure**

Although the effect of confining pressure on various geotechnical structures is significant and is considered to be key criteria in the design of these structures, it is usually neglected in conventional rail track design. Track substructure is essentially self-supporting with minimal lateral constraints. During a train passage, ballast and capping (subballast) materials are free to spread laterally, which increases track settlement and decreases its shear strength. At the University of Wollongong, cyclic triaxial tests have been conducted on samples of ballast to investigate the effect of confining pressure. The effect of confining pressure on ballast under cyclic loading to reduce the volume of breakage has been studied to find out the optimum confining pressure based on loading and track conditions [13,14]. Track confinement can be increased by reducing the spacing of sleepers, increasing the height of shoulder ballast, including a geosynthetic layer at the ballast-subballast interface, widening the sleepers at both ends (Figure 4), and using intermittent lateral restraints at various parts of the track (Figure 5) [15].
3. Use of Geosynthetics for Stabilising a Recycled Ballasted Track

Geosynthetics have been widely and successfully used in new rail tracks and in track rehabilitation schemes for almost three decades. When appropriately designed and installed, geosynthetics are a cost effective alternative to more traditional techniques. The application of geosynthetics within railway construction can be subdivided into (1) separation, (2) reinforcement, (3) filtration, (4) drainage, (5) moisture barrier/waterproofing and (6) protection. In order to investigate train traffic induced stresses and track deformations and benefits of using geosynthetics in fresh and recycled ballast, a field trial was carried out on a fully instrumented track in the town of Bulli (Indraratna et al. [16]). The University of Wollongong provided technical specifications for the design, while RailCorp, Australia provided funding to build a section of highly sophisticated instrumented track.

3.1 Track Construction

The proposed site for track construction was located between two turnouts at Bulli along the NSW coast. The total length of the instrumented track section was 60 m and was divided into four sections, each of 15 m length. The depths of the load bearing ballast and capping layer were 300 mm and 150 mm, respectively. Concrete sleepers were used. The detailed layout of these four sections is shown in Figure 6. Fresh and recycled ballast without inclusion of a geocomposite layer were used at two sections, while the other two sections were built by placing a geocomposite layer at the base of the fresh and recycled ballast, respectively (Figure 7).

3.2 Material Specifications

The particle size, gradation, and other index properties of fresh ballast used at the Bulli site were in accordance with the Technical Specification TS 3402 [17], which represents sharp angular coarse aggregates of crushed latite basalt. Recycled ballast was collected from spoil stockpiles of a recycled plant commissioned by RailCorp at their Chullora yard near Sydney. The capping material was comprised of a sand-gravel mixture. The particle size distribution of fresh ballast, recycled ballast and the capping
(sub-ballast) materials are given in Figure 8. Table 1 shows the grain size characteristics of fresh ballast, recycled ballast and the capping materials used in the Bulli instrumented track \[16\].

<table>
<thead>
<tr>
<th>Material</th>
<th>Particle shape</th>
<th>$d_{\text{max}}$ (mm)</th>
<th>$d_{\text{min}}$ (mm)</th>
<th>$d_{50}$ (mm)</th>
<th>$C_u$</th>
<th>$C_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fresh Ballast</td>
<td>Highly angular</td>
<td>75.0</td>
<td>19.0</td>
<td>35.0</td>
<td>1.5</td>
<td>1.0</td>
</tr>
<tr>
<td>Recycled Ballast</td>
<td>Semi-angular</td>
<td>75.0</td>
<td>9.5</td>
<td>38.0</td>
<td>1.8</td>
<td>1.0</td>
</tr>
<tr>
<td>Capping</td>
<td>Angular to rounded</td>
<td>19.0</td>
<td>0.05</td>
<td>0.26</td>
<td>5.0</td>
<td>1.2</td>
</tr>
</tbody>
</table>

A bi-axial geogrid was placed over the non-woven polypropylene geotextile to serve as the geocomposite layer, which was installed at the ballast-capping interface. The technical specifications of geosynthetic material used at the site have been discussed elsewhere by Indraratna et al. \[3\].

### 3.3 Track Instrumentation

The performance of each section under the cyclic load of moving trains was observed using sophisticated instrumentation. The vertical and horizontal stresses developed in the track bed under repeated wheel loads were measured by pressure cells. Track vertical deformations at different sections were measured by pegs and the 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 sleeper-ballast and ballast-capping interfaces, respectively, as shown in Figure 9.

### 3.4 Track Measurements

#### 3.4.1 Vertical Deformations

In order to investigate the overall performance of the ballast layer, the average vertical deformation was considered by deducting the vertical displacements of the sleeper-ballast and ballast-capping interfaces. The vertical displacement at each interface was obtained by taking the mean of measurements recorded beneath the rail and the edge of the sleeper. The values of average vertical ballast deformation are plotted against the number of load cycles (N) in Figure 10. In the recycled ballast, they are smaller compared to the case of fresh ballast, because of its moderately-graded particle size distribution compared to the very uniform fresh ballast. Recycled ballast is often subjected to less breakage as they are less angular, thereby preventing corner breakage due to high contact stresses. The inclusion of a geocomposite decreases the average vertical deformation of recycled ballast over a large number of load cycles. The load distribution capacity of the ballast layer is improved by the placement of a flexible and resilient geocomposite layer, which results in a substantial reduction of settlement under high cyclic loading.

#### 3.4.2 Lateral Deformations

Data from the displacement transducers was recorded by a data logger at regular time intervals. Figure 11 shows the average lateral deformation of ballast plotted against the number of load cycles (N). The average deformations were determined from the mean of measurements at the sleeper-ballast and ballast-capping interfaces. The ballast layer exhibits an increase in the average lateral deformation (i.e. lateral spread, represented by negative sign) in all sections. This particular recycled ballast performed well
showing less vertical deformation. This can be attributed to its more well-graded particle size distribution, compared to uniform fresh ballast. If placed as a well-graded mix, the corners may not break as much because of improved grain interlock, hence less risk of stress concentrations. The layer of geogrid also reduced the lateral deformation of both fresh and recycled ballast. The track deterioration is a result of accumulated plastic settlements in the track layers, which has serious consequences on passenger comfort, safety and efficiency (speed restriction) during train operation. The influence of geosynthetics in preventing track deterioration is appealing to the railway industry, not only due to the low cost of geosynthetics, but also because of the substantial savings generated by the extended track life-cycle and enhanced resilient behaviour of the track.

Figure 10. Average vertical deformation of the ballast layer [16]
Figure 11. Average lateral deformation of the ballast layer [16]

3.5 Numerical Modelling using PLAXIS

An elasto-plastic constitutive model of a composite multi-layer track system including rail, sleeper, ballast, sub-ballast and subgrade is proposed. Numerical simulations were performed using a two-dimensional plane-strain finite element analysis i.e. PLAXIS [18] to predict the track behaviour with and without geosynthetics. PLAXIS code has demonstrated its success in the limit analysis of geotechnical problems. A typical plain strain track model was numerically simulated as shown by the Finite Element (FE) discretisation as shown in Figure 12a.

![Figure 12. (a) Finite element mesh discretisation of a rail track and (b) 15-node continuum soil, 10-node Interface and 5-node line element](image)

The subgrade soil and track layers were modelled using 15-node linear strain quadrilateral elements ‘LSQ’. Figure 12b shows the details of these elements used in the FE simulations. The 15-node isoparametric element provides a fourth order interpolation for displacements. The numerical integration by a Gaussian integration scheme involves twelve Gauss points (stress points). A 3 m high and 6 m wide
finite element model was discretised by 1464 fifteen-node elements, 37 five-node line elements and 74 five-node elements at the interface.

The nodes along the bottom boundary of the section were considered as pinned supports i.e. were restrained in both vertical and horizontal directions (standard fixities). The left and right boundaries were restrained in the horizontal directions, representing smooth contacts vertically. The axial wheel load was simulated as a line load representing an axle train load of 25 tonnes with a dynamic impact factor of 1.4. The gauge length of the track is 1.68 m. The shoulder width of ballast is 0.35 m and the side slope of the rail track embankment is 1:2. The material parameters and constitutive models used for each component of the track section are given in Table 2. The flow rule adopted in the hardening soil model is characterised by a classical linear relation, with the mobilised dilatancy angle given by Schanz et al. [19]:

$$\sin \psi_m = (\sin \phi_m - \sin \phi_v) \sin \phi_m \sin \phi_v$$  \hspace{1cm} (3)

where $\phi_v$ is a material constant (the friction angle at critical state) and:

$$\sin \phi_v = (1 - \sigma_3) (1 + \sigma_3 - 2c \cot \phi_v)$$  \hspace{1cm} (4)

Indraratna and Salim [20] described the dependence of particle breakage and dilatancy on the friction angle of ballast. A modified flow rule considering the energy consumption due to particle breakage during shear is given by Salim and Indraratna [21]:

$$\frac{dc^p}{d\gamma} = \frac{9M - 9M}{9 + 3M - 2\eta M} + \frac{dE_p}{pd\gamma} \frac{9 - 3M}{9 + 3M - 2\eta M} \frac{6 + 4M}{6 + M}$$  \hspace{1cm} (5)

### Table 2. Parameters of the rail track materials used in the finite element analysis

<table>
<thead>
<tr>
<th>Material Parameters</th>
<th>Rail</th>
<th>Concrete Sleeper</th>
<th>Ballast</th>
<th>Sub-ballast</th>
<th>Subgrade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material Model</td>
<td>Linear Elastic</td>
<td>Linear Elastic</td>
<td>Hardening Soil</td>
<td>Mohr-Coulomb</td>
<td>Mohr-Coulomb</td>
</tr>
<tr>
<td>Material Type</td>
<td>Non-porous</td>
<td>Non-porous</td>
<td>Drained</td>
<td>Drained</td>
<td>Drained</td>
</tr>
<tr>
<td>$E$ (MPa)</td>
<td>210,000</td>
<td>10,000</td>
<td>-</td>
<td>80</td>
<td>34.2</td>
</tr>
<tr>
<td>$E_{soil}$ (MPa)</td>
<td>-</td>
<td>-</td>
<td>21.34</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$E_{con}$ (MPa)</td>
<td>-</td>
<td>-</td>
<td>21.34</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$E_{con}$ (MPa)</td>
<td>-</td>
<td>-</td>
<td>64.02</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$\gamma$ (kN/m$^3$)</td>
<td>78</td>
<td>24</td>
<td>15.6</td>
<td>16.67</td>
<td>18.15</td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.15</td>
<td>0.15</td>
<td>-</td>
<td>0.35</td>
<td>0.33</td>
</tr>
<tr>
<td>$\nu_{ur}$</td>
<td>-</td>
<td>-</td>
<td>0.2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$c$ (kN/m$^3$)</td>
<td>-</td>
<td>-</td>
<td>0</td>
<td>0</td>
<td>5.5</td>
</tr>
<tr>
<td>$\phi$ (degrees)</td>
<td>-</td>
<td>-</td>
<td>58.47</td>
<td>35</td>
<td>24</td>
</tr>
<tr>
<td>$\psi$ (degrees)</td>
<td>-</td>
<td>-</td>
<td>12.95</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>$P_{ref}$ (kN/m$^3$)</td>
<td>-</td>
<td>-</td>
<td>50</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$m$</td>
<td>-</td>
<td>-</td>
<td>0.5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$k$</td>
<td>-</td>
<td>-</td>
<td>0.3</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>$R_f$</td>
<td>-</td>
<td>-</td>
<td>0.9</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

$HS = $ Hardening-Soil model, $MC = $ Mohr-Coulomb model, $\gamma = $ unit weight, $E_{soil}^{ref} = $ secant stiffness at 50% strength for loading conditions, $E_{con}^{ref} = $ triaxial unloading/reloading stiffness, $E_{con}^{ref} = $ tangent stiffness for primary oedometer loading, $EA = $ elastic normal (axial) stiffness, $\nu = $ Poisson’s ratio for loading conditions, $\nu_{ur} = $ Poisson’s ratio for unloading/reloading conditions, $c = $ effective cohesion, $\phi = $ effective friction angle, $\psi = $ dilatancy angle, $P_{ref} = $ reference confining pressure, $m = $ stress dependent stiffness factor, $k = $ coefficient of earth pressure at rest for normal consolidation, $R_f = $ failure ratio.

Assuming that the incremental energy consumption due to particle breakage per unit volume is proportional to the corresponding increment of breakage index (i.e., $dE_p = \beta dB_p$, where $\beta$ is a constant of proportionality and $B_p$ is Marshall’s breakage index), Equation 5 becomes:
Replacing $B_g$ with a more accurate ballast breakage index (BBI) defined by Indraratna et al. [13] gives:

$$\frac{dE_g}{d\varepsilon^p} = \frac{9M - \eta}{9 + 3M - 2\eta M} + \frac{\beta dBBI}{p\varepsilon^p} \left( \frac{9M - 3M}{9 + 3M - 2\eta M} \right) \left( \frac{6 + 4M}{6 + M} \right)$$

The experimental values of $\eta$, $p$, $M$ and the computed values of $dE_g/d\varepsilon^p$, which are linearly related to the rate of particle breakage $dBBI/d\varepsilon^p$, can be readily used to quantify the flow rule. The values of stress-dependent stiffness moduli are obtained from previously published results of large scale drained triaxial compression tests under monotonic loading conditions [3]. The hardening soil model showed better agreement with the strain-hardening behaviour of ballast observed in large scale triaxial tests indicating ballast breakage [22]. The current formulation of finite element is incapable of conducting post-peak analysis into the strain-softening region; however such large strains or large deformations are not permitted in reality.

### 3.6 Comparison of Field Measurements with FE Predictions

In order to validate the findings of the FE analysis, a comparison was made between the elasto-plastic FE analysis and the field data. Figure 13 and 14 shows the variation of vertical stress and vertical ballast deformations predicted by FE simulations as well as the measured values underneath the rail seat at the unreinforced section of instrumented track. As mentioned earlier, the vertical deformations were monitored at the sleeper-ballast and ballast-capping interfaces using settlement pegs. The values predicted by elasto-plastic analysis showed slight deviation compared with the measured field values. This discrepancy may be attributed to the fact that the real cyclic nature of wheel loading was not considered but was approximately represented by equivalent dynamic plain strain analysis in FE studies.

**Figure 13. Variation of vertical stress of ballast with the depth**

**Figure 14. Variation of vertical deformation of ballast with the depth**

### 4. Use of Geosynthetic Vertical Drains as a Subsurface Drainage

Low-lying areas having large volumes of plastic clays can sustain high excess pore water pressures during both static and cyclic (repeated) loading. In poorly drained situations, the increase in excess pore pressures will decrease the effective load bearing capacity of the formation soil. Under certain circumstances, slurring of clay beneath rail tracks may initiate pumping of the soil upwards, thereby clogging the ballast bed and promoting undrained shear failure [Indraratna et al. (7)]. Geosynthetic PVDs can be installed to dissipate excess pore pressures by radial consolidation before they can build up to critical levels. These PVDs continue to dissipate excess pore water pressures even after the cyclic load stops.

#### 4.1 Design Process for Short PVDs under Railway Track

The Sandgate Rail Grade Separation Project is located at the town of Sandgate between Maitland and Newcastle, in the Lower Hunter Valley of NSW (Figure 15). The new railway tracks were required to
reduce the traffic in the Hunter Valley Coal network. In this section, the rail track stabilised using short PVDs in the soft subgrade soil is presented together with the background of the project, the soil improvement details, design methodology and FE analysis. The effectiveness of PVDs in improving soil conditions has been demonstrated by Indraratna et al. [23]. Preliminary site investigations were conducted for mapping the soil profile along the track. In-situ and laboratory testing programs were carried out to provide relevant soil parameters. Site investigation included 6 boreholes, 14 piezocene (CPTU) tests, 2 in-situ vane shear tests and 2 test pits. Laboratory testing such as soil index property testing, standard oedometer testing and vane shear testing were also conducted.

A typical soil profile showed that the existing soft compressible soil thickness varies from 4 to 30 m. The soft residual clay lies beneath the soft soil layer followed by shale bedrock. The soil properties are shown in Figure 16. The groundwater level is at the ground surface. The moisture contents of the soil layers are the same as their liquid limits. The soil unit weight varies from 14 to 16 kN/m$^3$. The undrained shear strength increases from about 10 to 40 kPa. The clay deposit at this site can be considered as lightly overconsolidated ($\text{OCR} \approx 1-1.2$). The horizontal coefficient of consolidation ($c_h$) is approximately 2-10 times the vertical coefficient of consolidation ($c_v$). Based on preliminary numerical analysis conducted by Indraratna et al. [24], PVDs 8 m in length were suggested and installed at 2 m spacing in a triangular pattern. An extensive field instrumentation scheme including settlement plates, inclinometers and vibrating wire piezometers was employed to monitor the track response. The settlement plates were installed above the surface of the subgrade layer to directly measure the vertical subgrade settlement. The main aims of the field monitoring were to:

(a) ensure the stability of track;
(b) validate the design of the track stabilised by PVDs; and
(c) examine the accuracy and reliability of the numerical model through Class A predictions (the field measurements were unavailable at the time of FE modelling).

![Figure 15. Site location (adopted from Indraratna et al. [23])](image)

![Figure 16. Soil properties at Sandgate Rail Grade Separation Project [23]](image)

### 4.2 Preliminary Design

Due to the time constraints in contractual agreements, the rail track was built immediately after installing the PVDs. The train load moving at very low speed was used as the only external surcharge. The equivalent dynamic loading using an impact load factor was used to predict the track behaviour. In this analysis, a static pressure of 104 kPa with an impact factor of 1.3 was applied according to the low train speed (60 km/h) for axle loads up to 25 tonnes, based on Australian Standards AS 1085.14-1997 [13]. The Soft Soil model and Mohr-Coulomb model were both employed in the FE code, PLAXIS [26]. The overcompacted surface crust and fill layer were simulated by the Mohr-Coulomb theory, whereas the soft clay deposit was conveniently modelled using the Soft Soil model. The formation was separated into 3 distinct layers, namely ballast and fill, Soft soil-1 and Soft soil-2. The relevant soil parameters are given in Table 3.
A cross-section of the FE mesh discritization of the formation beneath the track is shown in Figure 17. A plane strain FE analysis was employed using linear strain triangular elements with 6 displacement nodes and 3 pore pressure nodes. A total of 4 drain rows were used in the analysis. An equivalent plane strain analysis with appropriate conversion from axisymmetric to 2-D was adopted to analyse the multi-drain analysis [23]. In this method, the corresponding ratio of the smear zone permeability to the undisturbed zone permeability is given by:

$$\frac{k_{s,ps}}{k_{h,ps}} = \frac{k_{s,ps}}{k_{h,ax}} \left[ \frac{\beta}{\alpha} + \frac{k_{s,ax}}{k_{s,ax}} \ln \left( \frac{2}{0.75} \right) \right]$$

$$\alpha = 0.67 \left( \frac{a}{s} \right)^{2/n}$$

$$n = \frac{d_e}{d_w}$$

$$s = \frac{d_e}{d_w}$$

In the above expressions, $d_e$ is the diameter of unit cell soil cylinder, $d_s$ is the diameter of the smear zone, $d_w$ is the equivalent diameter of the drain, $k_s$ is horizontal soil permeability in the smear zone, $k_h$ is horizontal soil permeability in the undisturbed zone and the top of the drain and subscripts ‘ax’ and ‘ps’ denote the axisymmetric and plane strain condition, respectively.

The ratio of equivalent plane strain to axisymmetric permeability in the undisturbed zone can be attained as,

$$\frac{k_{h,ps}}{k_{h,ps}} = 0.67 \left( \frac{a}{s} \right)^{2/n}$$

In the above equation, the equivalent permeability in the smear and undisturbed zone vary with drain spacing.

![Figure 17. Vertical cross section of rail track and foundation [23]](image)

Table 3. Selected parameters for soft soil layer used in the FEM [23]

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>Depth of layer (m)</th>
<th>c (kPa)</th>
<th>$\phi$</th>
<th>$\varepsilon_p$</th>
<th>$\lambda/(1+\varepsilon_p)$</th>
<th>$\kappa/(1+\varepsilon_p)$</th>
<th>$k_h \times 10^{-6}$ (m/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft soil-1</td>
<td>1.0-10.0</td>
<td>10</td>
<td>25</td>
<td>2.26</td>
<td>0.131</td>
<td>0.020</td>
<td>1.4</td>
</tr>
<tr>
<td>Soft soil-2</td>
<td>10.0-20.0</td>
<td>15</td>
<td>20</td>
<td>2.04</td>
<td>0.141</td>
<td>0.017</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Note: $\phi$ Back-calculated from the Cam-clay M value.

4.3 Comparison of Field Behaviour with Numerical Predictions

The field results were released by the track owner (Australian Rail Track Corporation) one year after the FE predictions. Therefore, all predictions can be categorized as Class A. A spacing of 2 m was adopted for the vertical band drains 8 m in length. The field data together with the numerical predictions are
compared and discussed. The calculated and observed consolidation settlements at the centre line are presented in Figure 18. The predicted settlement matches very well with the field data. The in-situ lateral displacement at 180 days at the rail embankment toe is illustrated in Figure 19. As expected, the maximum displacements are measured within the top clay layer i.e. the softest soil below the 1 m crust. The lateral displacement is restricted to the topmost compacted fill (0–1 m deep). The Class A prediction of lateral displacements is also in very good agreement with the field behaviour. The effectiveness of wick drains in reducing the effects of undrained cyclic loading through the reduction in lateral movement is undeniably evident.

![Figure 18. Predicted and measured at the centre line of rail tracks [23]](image1.png)

![Figure 19. Measured and predicted lateral displacement at the embankment toe at 180 days [23]](image2.png)

5. Use of Geogrid for Stabilising Fouled Ballasted Track

When ballast is fouled by breakage or infiltration of fine particles, the particle interaction may change considerably as fine particles clog the voids (of the geogrid) reducing the interlocking and frictional resistance between the geogrid and ballast. Budiono et al. [27] and Indraratna et al. [28] reported that fine particles adversely affect the strength and stiffness of track structures because as fouling increases, the stiffness of the ballast is significantly reduced. When the amount of fouling materials is excessive, fine particles can dominate the ballast behaviour and ultimately make the track unstable. Dombrow et al. [29] carried out a series of direct shear tests with fresh ballast and ballast fouled by coal, and showed that as the percentage of fouling increases, the shear strength decreases. Indraratna et al. [28] provided valuable information of shear strength and apparent angle of shearing resistance of this composite system under various degrees of fouling using a large scale direct shear apparatus.

5.1 Experimental Set up and Procedure of the Large Scale Direct Shear Test

The recommended particle size distribution of ballast with a mean particle size of $d_{50} = 35$ mm was adopted. Using a parallel gradation, the maximum size of the adopted laboratory tested ballast was less than 40 mm and this size was small enough to avoid boundary effects. Coal fines were used as fouling material for the Void Contaminant Index (VCI) that was varied as 20%, 40%, 70% and 95%. The geogrid used was manufactured from polypropylene and had 40 x 40 mm$^2$ apertures. The apparatus for the direct shear test consisted of a 300 x 300 mm$^2$ square steel box 195 mm in height, divided horizontally into two equal halves. The schematic diagram of this test set up is shown in Figure 20.

The ballast was compacted in the bottom half of the shear box to a dry density of 15 kN/m$^3$. After the first layer was compacted, a sheet of geogrid was placed on top. Coal fines were spread over each layer according to the desired VCI. The remaining ballast was then compacted to the upper half of the shear box. In this study, a vertical load was then applied via a rigid and free top plate at the top of the shear box using a dead weight system attached to a lever arm. The tests were conducted at four normal stresses of 15, 27, 51 and 75 kPa. The lower section of the shear box was moved at 2.5 mm/min, while the upper section of the box remained fixed. Each specimen was subjected to 37 mm of maximum horizontal displacement. During this shearing process, the shearing forces and vertical displacement of the top plate were recorded at every 1 mm displacement.
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