STATE-OF-THE-ART DESIGN ASPECTS OF BALLASTED RAIL TRACKS INCORPORATING PARTICLE BREAKAGE, ROLE OF CONFINING PRESSURE AND GEOSYNTHETIC REINFORCEMENT

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ABSTRACT

Railways are expected to play a very important role in future transport in Australia, and its large network should capture the essential needs for quick and safe, passenger and freight mobility. In recent years, the increased demand of heavier and faster trains has posed greater challenges to the railway industry to improve efficiency and stability of track while reducing the track maintenance costs. Centre for Geomechanics and Railway Engineering (GRE) has been the primary Research and Development unit in the Australasia for developing and implementing new design and construction concepts for modern track upgrading with clear emphasis on applying theory to practice, with the key objectives of ensuring enhanced track longevity and minimising track maintenance costs.

In spite of recent advances in rail track geotechnology, the optimum choice of ballast for track design is still considered critical. The major reason is that, ballast aggregates progressively degrade under heavy cyclic loading. Research at GRE has shown that a proper understanding of load transfer mechanisms and their effect on ballast breakage are important pre-requisites for minimising track maintenance costs. Ballast degradation is influenced by various factors including the amplitude and number of load cycles, particle gradation, track confining pressure, and the angularity and fracture strength of individual grains.

Recent research projects at GRE and field trials in Bulli (near Wollongong) demonstrated that the discarded aggregates from ballast tips could be effectively reused in track construction, if regraded and reinforced with geogrids to rejuvenate their internal friction and load carrying capacity. This recycling practice would directly decrease the accumulation of discarded ballast, minimise the cost of track maintenance and reduce environmental degradation (i.e. less quarrying). Moreover, the use of effective sub-surface drainage via geosynthetic drains has been very effective in rapidly dissipating cyclic-induced pore water pressures in the soft subgrade (e.g. clay and silts) during the passage of trains, and these drains have effectively prevented soil liquefaction (mud pumping). The corresponding track behaviour models have been also developed through large-scale laboratory simulations and computer-based numerical modelling.

This state-of-the-art paper describes field trials and prototype laboratory studies carried out to quantify the geotechnical behaviour of ballast, including shear strength, particle breakage, effects of increased confining pressure, supplemented with predictive and design models for practitioners adopting user-friendly analytical and numerical approaches. The paper also highlights the proposed changes to current standards of track design and how these new concepts have been implemented through actual field trials that demonstrated better performance, in terms of reducing settlement and improving drainage. Two case studies are elaborated including the Bulli and Sandgate sites enhanced by synthetic grids and geosynthetic drains.
1. INTRODUCTION

Railway track formations generally consist of a layer of ballast intended to provide a free draining load bearing base which is stable enough to maintain the track alignment with minimum maintenance. Consequently the ballast layer must be thick enough to hold the track in position and to provide protection to subgrade soils, while aggregates must be tough enough to resist the abrasion and degradation. However 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 degradation of ballast is influenced by many factors, including the amplitude, frequency and number of load cycles, gradation of aggregates, track confining pressure, and the angularity and fracture strength of individual grains. The cost of track maintenance can be significantly reduced if the geotechnical behaviour of the ballast layer is properly assessed. Geosynthetic materials play an important role in improving the efficiency and performance of ballasted rail track. The track reinforcement using geogrids provide confinement to the ballast layer, therefore leads to significant reduction in the downward propagation of stresses vertical and lateral plastic deformations, and assures more resilient long-term performance of the ballast layer (Raymond 2002, Indraratna et al. 2007). The geocomposite provides reinforcement to the ballast layer, as well as filtration and separation functions simultaneously (Indraratna and Salim, 2005; Indraratna et al., 2010a). Moreover, the use of effective sub-surface drainage via geosynthetic drains has been very effective in rapidly dissipating cyclic-induced pore water pressures in the soft subgrade (e.g. clay and silts) during the passage of trains, and these drains have effectively prevented soil liquefaction (mud pumping) (Indraratna et al., 2010c).

A field trial was conducted on a section of instrumented railway track along the New South Coast in the town of Bulli, NSW Australia, with the specific aims of studying the effectiveness of a geocomposite (combination of biaxial geogrid and non-woven polypropylene geotextile) installed at the ballast-capping interface, and to evaluate the performance of moderately graded recycled ballast in comparison with traditionally used very uniform fresh ballast. 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. Another field trial at Sandgate site has revealed that relatively short prefabricated vertical drains (PVDs) between 6 to 8 m length can still 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 satisfied in terms of maintenance practices (e.g. in new railway tracks where continuous ballast packing may be required), the rate of settlement can still 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 to adopt user-friendly analytical and numerical approaches simulating the field track behavior are presented.

2. GEOTECHNICAL BEHAVIOUR OF BALLAST

Ballast is a free-draining granular material used as a load-bearing material, which forms the largest component of a rail track by weight and volume. Ballast materials usually include dolomite, rheolite, gneiss, basalt, granite and quartzite (Raymond, 1979). It is composed of medium to coarse gravel sized aggregates (10 - 60 mm) with a smaller percentage of cobble-sized particles. North American Railway systems use typical ballast sizes ranging from 4.76 mm to 51 mm. Australian Railways (AS 2758.7, 1996; TS 3402; 2001) uses ballast sizes varying from 13.2 mm to 63 mm. 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 (Indraratna et al., 1998). The main functions of ballast include distributing and damping the loads received from sleepers, producing lateral resistance, and providing rapid drainage (Selig & Waters, 1994). 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 design depth of the ballast layer needs to be determined maintaining a balance between the bearing capacity and drainage. The major geotechnical properties of ballast are briefly discussed in this section.

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. Indraratna et al. (1997) and Ramamurthy (2001) have shown that the shear strength is a function of confining pressure, and is non-linear at high stresses. Indraratna et al. (1993) 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:

\[
\tau = \tau_0 + \left(\frac{\sigma'_n}{\sigma_0}\right)^n
\]

(1)
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, $n$ varies from 0.65 to 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. (1998) to verify the non-linearity of shear stress. The results of his study on latite basalt are plotted in Figure 2 in a normalised form, with data obtained by other researchers (Marsal, 1973; Marachi et al., 1972; Charles & Watts, 1980).

2.2 Ballast Degradation and Breakage Quantification

Railway tracks are deformed by the degradation of ballast particles (Selig & Waters, 1994; Indraratna et al. 1998). The breakage of sharp angular ballast particles under wheel loads is a complex mechanism that usually starts at the inter-particle contacts (i.e. breakage of small-scale asperities), then breaking of angular projections 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. Chrismer & Read (1994) concluded that the degradation of aggregate is the primary cause of contamination, and accounts for up to 40% of the fouled material. 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).

In order to quantify ballast breakage, Indraratna et al. (2005a) introduced 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 equation $BBI = (A+B)/A$ 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 Constitutive Modelling of Ballast

Salim & Indraratna (2004) developed an elasto-plastic stress-strain constitutive model incorporating dilatancy, breakage and the plastic flow rule to predict ballast deformation and degradation. The model uses a generalised 3D system to define the contact forces, stresses and strains in granular media, including the plastic potential, the hardening function, and particle breakage. This model is based on the critical state concept and theory of plasticity with a kinematic-type yield locus (constant stress ratio). The increments of plastic distortional strain $\varepsilon_{PS}$, and volumetric strain $\varepsilon_{V}^{p}$, were given by Salim & Indraratna (2004), as follows:
Challenge C: Increasing Freight capacity and services

\[
dc_s^e = \frac{2\alpha \left( \frac{p - p_{\alpha}}{p_{\alpha}} \right) \left( 1 - \frac{p_{\alpha}}{p_{\alpha}^i} \right) (9 + 3M - 2\eta^* M)(\eta - \eta) \delta}{(M - \eta)^2 + (1 + \epsilon_i) \left( \frac{2p - 1}{p} \right) \left( \frac{9(M - \eta^*) + \frac{B}{p} [\chi + \mu (M - \eta^*)]}{9 + 3M - 2\eta^* M} \right) dc_s^p}
\]

(2)

\[
dc_y^p = \frac{9(M - \eta)}{9 + 3M - 2\eta^* M} dc_y^p + \left( \frac{B}{p} \right) \left( \frac{\chi + \mu (M - \eta^*)}{9 + 3M - 2\eta^* M} \right) dc_s^p
\]

(3)

Figure 3: Ballast breakage index (BBI) calculation method (Indraratna et al., 2005a)

The parameter \( p \) is the effective mean stress and \( p_{\alpha} \) is the value of \( p \) on the critical state line at the current void ratio, \( p_{\alpha} \) is the value of \( p \) at the intersection of the undrained stress path and the initial stress ratio line. The sub-script \( i \) indicates the initial value at the start of shearing. The parameter \( \eta \) is the stress ratio \( (\eta = q/p) \), \( q \) is the deviator stress, \( \eta^* = \eta (p/p_{\alpha}) \), \( M \) is the critical state stress ratio, \( \epsilon_i \) is the initial void ratio, \( \chi \) is the negative slope of compression curve \((e-I_{cp})\) and \( \alpha, B, \chi \) and \( \mu \) are dimensionless constants. The evolutionary techniques of these constants are explained in detail elsewhere (Indraratna & Salim, 2002, Salim & Indraratna, 2004). This model was verified using large-scale triaxial tests (Figure 4). The above constitutive model contains 11 parameters for monotonic loading and additional 4 parameters for cyclic loading. These parameters can be evaluated using the results of the drained triaxial test and the measurements of particle breakage.

![Figure 4: Model prediction compared with experimental data for drained triaxial shearing (Salim & Indraratna, 2004)](image)

2.4 Effect of Confining Pressure

Although the effect of confining pressure on various geotechnical structures is significant and is considered to be key design criteria, 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 ballast to investigate the effect of confining pressure. 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 5),...
and using intermittent lateral restraints at various parts of the track (Figure 6). The effect of confining pressure to reduce the amount of ballast breakage has been studied by Indraratna et al. (2005a) and Lackenby et al. (2007), in order to determine the optimum confining pressure based on cyclic loading and track conditions.

Figure 5: Sleepers with enlarged ends to increase the confining pressure

Figure 6: Increasing confining pressure using intermittent lateral restraints

Specimens were prepared to the recommended gradation (AS 2758.7, 1996) and initial porosity (i.e. $d_{50} = 38.5$ mm, $C_u = 1.54$. $e_0 = 0.76$ where $d_{50}$ is the particle size corresponding to 50% finer in the particle size distribution curve and $C_u$ is the coefficient of uniformity). Effective confining pressures ($\sigma'_c$) ranging from 1 to 240 kPa with $q_{\text{max}} = 500$ kPa were applied. Figure 7 shows the results of confining pressure ($\sigma'_c$) on the axial and volumetric strains of ballast achieved at the end of 500,000 cycles. As expected, the axial strains decreased with an increasing confining pressure and the specimens dilated at a low confining pressure ($\sigma'_c < 30$), but became progressively more compressive as the confining pressure increased from 30 to 240 kPa. The effect of confining pressure ($\sigma'_c$) on particle degradation is shown in Figure 8, where breakage was divided into three regions: (I) a dilatant degradation zone (DUDZ); (II) an optimum degradation zone (ODZ); and (III) a compressive stable degradation zone (CSDZ). Lackenby et al. (2007) has shown that the specimens were subjected to rapid and considerable axial and expansive radial strains that resulted in an overall volumetric increase or dilation at low confining pressure of DUDZ region ($\sigma'_c < 30$ kPa). Particles in this region were given insufficient time to rearrange and due to the excessive axial and radial strains, a considerable degradation occurred as a result of shearing and attrition of angular projections. Due to the low confining pressures applied in this region, specimens in this degradation zone are characterised by limited particle-to-particle areas of contact.

As the confining pressure increased to the ODZ region ($\sigma'_c = 30$ - 75 kPa), the rate of axial strain was greatly reduced due to an apparent increase in stiffness, and the overall volumetric behaviour was slightly compressive. Particles in this region were held together in an optimum array with sufficient lateral confinement so that to provide an optimum distribution of contact stress and increased areas of inter-particle contact which reduced the risk of breakage associated with concentrations of stress. As $\sigma'_c$ increased further in the CSDZ region ($\sigma'_c > 75$ kPa), the particles were forced against each other which limited sliding and rolling but increased their breakage considerably. Particles in this region failed not only at the beginning of loading when axial strain rates are greatest; but also by fatigue as the number of cycles increase. Due to the large lateral forces applied onto the samples in this region, volumetric compression was enhanced, which was partly due to an increase in particle breakage.

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3. USE OF GEOCOMPOSITE UNDER RAILWAY TRACK

3.1 Track measurements

The comprehensive knowledge of the complex mechanisms associated with track deterioration is essential in the accurate prediction of a typical rail track maintenance cycle. Many simplified analytical and empirical design methods have been used to estimate settlements and stress-transfer between the track layers. However, these design methods are based on the linear elasticity approach, and thus often give crude estimates. Given the complexities of the behavior of the composite track system consisting of rail, sleeper, ballast, sub-ballast and subgrade under repeated traffic loads, the current track design techniques are overly simplified. The potential use of geosynthetics in the improvement of track stability and reducing the maintenance cost is well established (Selig and Waters 1994, Indraratna and Salim 2005).

In order to investigate train traffic induced stresses, vertical and lateral track deformations and advantages of using geosynthetics below fresh and recycled ballast in rail track, a field trial was carried out on a real instrumented track (Indraratna et al. 2010a). The instrumented track was constructed between two turnouts at Bulli along the coast of New South Wales, north of Wollongong city. The total length of the instrumented track section was 60 m, and it was divided into four sections of 15 m length each. Two sections were built without geocomposite layer, while remaining two sections were built by placing a geocomposite layer at the ballast-capping interface (Figure 9). To measure the vertical and horizontal deformations of ballast, settlement pegs and displacement transducers were installed at the sleeper-ballast and ballast-capping interfaces in different track sections. The vertical and horizontal stresses developed in the track bed under repeated wheel loads were measured by pressure cells (230 mm diameter) installed at different locations in section of fresh ballast.

The overall thickness of granular layer was kept as 450 mm including a ballast layer of 300 mm and a capping layer of 150 mm in thickness. 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 (RailCorp, Sydney), which represents sharp highly angular coarse aggregates of crushed volcanic latite basalt. Recycled ballast was collected from a recycled plant at Chullora yard near Sydney. Table 1 shows the grain size characteristics of fresh ballast, recycled ballast and the sub-ballast materials used in the instrumented track at Bulli. Concrete sleepers were used in the test track. Electrical Friction Cone Penetrometer (EFCP) tests reported that the subgrade soil is a stiff overconsolidated silty clay and had more than sufficient strength to support the train loads. The bedrock is a highly weathered sandstone having weak to medium strength.

![Figure 9: Section of ballasted track bed with geocomposite layer](image1)

![Figure 10: Installation of geocomposite at the ballast-capping interface](image2)

Table 1. Grain size characteristics of fresh ballast, recycled ballast and capping materials (Indraratna et al., 2010a)

<table>
<thead>
<tr>
<th>Material type</th>
<th>Particle shape</th>
<th>$d_{50}$(mm)</th>
<th>$d_{10}$(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>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>
</tr>
<tr>
<td>Sub-ballast</td>
<td>Semi-rounded</td>
<td>19.0</td>
<td>0.05</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 (Figure 10). The technical specifications of geosynthetic material used at the site can be found elsewhere (Indraratna and Salim, 2005). In order to investigate the overall performance of the ballast layer, the average vertical deformation was considered by deducting the vertical displacements of sleeper-ballast and ballast-capping interfaces. The vertical displacement at each interface was obtained by taking the mean of measurements taken under the rail and the edge of sleeper. The values of average vertical and lateral deformations of the ballast layer were plotted against the number of load cycles (N) as shown in...
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Figures 11 and 12 respectively. In the recycled ballast, they were smaller compared to the case of fresh ballast. The better performance of selected moderately-graded recycled ballast can benefit from less breakage as they are often less angular, thereby preventing corner breakage due to high contact stresses. The geocomposite inclusion induced decrease in average vertical deformations of recycled ballast at a large number of cycles. The load distribution capacity of ballast layer was improved by the placement of a flexible and resilient geocomposite layer which results in a substantial reduction of settlement under high cyclic loading.

![Graphs](image)

**Figure 11:** Vertical deformation of ballast (modified after Indraratna et al., 2010a)

**Figure 12:** Lateral deformation of ballast (modified after Indraratna et al., 2010a).

3.2 Finite element analysis

An elasto-plastic constitutive model of a composite multi-layer track system including rail, sleeper, ballast, sub-ballast and subgrade is proposed. Numerical simulations are performed using a two-dimensional plane-strain finite element analysis PLAXIS (2007) to predict the track behaviour with and without geosynthetics. PLAXIS has demonstrated its success in the limit analysis of geotechnical problems. A typical plain strain track model is numerically simulated in a Finite Element discretisation as shown in Figure 13a.

![Finite element mesh discretisation](image)

**Figure 13:** (a) Finite element mesh discretisation of a rail track and (b) 15-node continuum soil, 10-node Interface and 5-node line element

The subgrade soil and the track layers were modeled using 15-node linear strain quadrilateral elements ‘LSQ’. Figure 13b shows details of these elements used in finite element simulations. The 15-node isoparametric element provides a fourth order interpolation for displacements and the numerical integration by Gaussian integration scheme involves twelve Gauss points (stress points). A 3 m high and 6 m wide finite element model is discretised to 1464 fifteen-node elements, 37 five-node line elements and 74 five-node elements at the interface. The mesh generation of PLAXIS version 8.6
used here follows a robust triangulation procedure to form ‘unstructured meshes’, which are considered to be numerically efficient when compared to regular ‘structured meshes’.

The nodes along the bottom boundary of the section were considered as pinned supports, i.e., were restrained in both vertical and horizontal directions (i.e. standard fixities). The left and right boundaries are restrained in the horizontal directions, representing smooth contact vertically. The vertical dynamic wheel load is simulated as a line load representing an axle train load of 25 tons 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 constitutive models and material parameters are given in Table 2. The flow rule adopted in HSM is characterised by a classical linear relationship, with the mobilised dilatancy angle given by (Schanz et al., 1999):

$$\sin \psi_m = \left( \frac{\sin \phi_m - \sin \phi_c}{1 - \sin \phi_m \sin \phi_c} \right)$$

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

$$\sin \phi_c = \left( \frac{\sigma_s - \sigma_i}{\sigma_s + \sigma_i - 2c \cot \phi} \right)$$

According to equation (4), $\psi_m$ depends on the values of friction, $\phi$ and dilatancy angles at failure, $\psi$ which control the quantity $\phi_c$. Indraratna and Salim (2002) 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 shearing deformations is given by (Salim and Indraratna, 2004):

$$\frac{dE_p}{d\varepsilon} = \frac{9(9 - \eta)}{9 + 3M - 2\eta M} + \frac{dE_s}{d\varepsilon} \left( \frac{9 - 3M}{6 + 4M} \right)$$

Table 2. Constitutive model and material parameters adapted in FEM analysis

<table>
<thead>
<tr>
<th>Material Parameters</th>
<th>Rail Track component</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Rail</td>
</tr>
<tr>
<td>Material Type</td>
<td>Non-porous</td>
</tr>
<tr>
<td>$E$ (MPa)</td>
<td>210,000</td>
</tr>
<tr>
<td>$E_{su}$ (MPa)</td>
<td>-</td>
</tr>
<tr>
<td>$E_{ref}$ (MPa)</td>
<td>-</td>
</tr>
<tr>
<td>$E_{ur}$ (MPa)</td>
<td>-</td>
</tr>
<tr>
<td>$\gamma$ (kN/m²)</td>
<td>78</td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.15</td>
</tr>
<tr>
<td>$v_{ur}$</td>
<td>-</td>
</tr>
<tr>
<td>$c$ (kN/m²)</td>
<td>-</td>
</tr>
<tr>
<td>$\phi$ (degrees)</td>
<td>-</td>
</tr>
<tr>
<td>$\psi$ (degrees)</td>
<td>-</td>
</tr>
<tr>
<td>$P_{ref}$ (kN/m²)</td>
<td>-</td>
</tr>
<tr>
<td>$m$</td>
<td>-</td>
</tr>
<tr>
<td>$k_u$</td>
<td>-</td>
</tr>
<tr>
<td>$\kappa_f$</td>
<td>-</td>
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</tbody>
</table>

The experimental values of $\eta$, $p$, $M$ and the computed values of $dE_p/d\varepsilon$ which are linearly related to the rate of particle breakage $dB_p/d\varepsilon$ can be readily used to predict 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 (Indraratna and Salim, 2005). The hardening soil model showed better agreement with the strain-hardening behaviour of ballast observed in large scale triaxial tests representing considerable ballast breakage (Shahin and Indraratna, 2006). The current formulation of finite element is incapable of conducting postpeak analysis into the strain-softening region however such large strains or large deformations are not permitted in reality, hence the study is focused on the peak strength.

3.3 Comparison of field measurements with FE predictions

In order to validate findings of the finite element analysis, a comparison is made between the elasto-plastic analysis and the field data. Figures 14 and 15 show the variation of vertical and lateral ballast deformations predicted by FE simulations and measured values underneath the rail seat at unreinforced section of the instrumented track. The vertical deformations were monitored at the sleeper-ballast and ballast-capping interfaces using settlement pegs as mentioned earlier. The values
predicted by elasto-plastic analysis showed slight deviation in comparison with the measured values of real track behavior. This is due to the fact that the real cyclic nature of wheel loading was not considered and was approximately represented by equivalent dynamic plain strain analysis in FE studies. Nevertheless, considering the limitations of elasticity based approaches, this prediction is acceptable for the design practice.

![Figure 14: Variation of vertical stress of ballast with the depth](image1)

![Figure 15: Variation of vertical deformation of ballast with the depth](image2)

**4. DESIGN PROCESS FOR SHORT PVDS UNDER RAILWAY TRACK**

**4.1 Site location and subsurface profile**

The Sandgate Rail Grade Separation Project is located at Sandgate between Maitland and Newcastle, in the Lower Hunter Valley of New South Wales (Figure 16). 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 finite element analysis. The effectiveness of PVDS in improving soil condition was demonstrated by Indraratna et al. (2010c). Preliminary site investigations were conducted for mapping the soil condition profile at the track. Both in-situ and laboratory testing were carried out to provide relevant soil parameters. Site investigation included 6 boreholes, 14 piezocone (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 thickness of existing soft compressible soil varies from 4m to 30m. The soft residual clay is beneath the soft soil layer and followed by shale bedrock. The soil properties are shown in Figure 17. 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 varied from 14 to 16 kN/m³. The undrained shear strength increased from 10 to 40 kPa. The clay deposit at this site can be considered as lightly overconsolidated (OCR = 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. (2010c), PVDS with 8m length were suggested and installed at 2m spacing in a triangular pattern. The field instrumentation included settlement plates, inclinometers, and vibrating wire piezometers was employed to monitor the rail track responses. The settlement plate was installed on the top surface of the subgrade layer to directly provide a measurement of the vertical subgrade settlement. The aims of the field monitoring were to:

(a) ensure the stability of the tracks;
(b) validate the design of the new railway stabilised by PVDS;
(c) examine the accuracy of the numerical analysis through Class A predictions, where the field measurements were unavailable at the time of finite element modeling.

**4.2 Preliminary design**

Due to the time constraint, rail tracks were built immediately after the installation of PVDS. The train load at very low speed was used as the only external surcharge. The equivalent dynamic loading with an impact load factor was used to predict the settlements and associated excess pore pressures. In this analysis, a static pressure of 104kPa with an impact factor of 1.3 was applied according to the low train speed for axle loads up to 25 tonnes, based on the Australian Standards AS 1085.14-1997. The Soft Soil model and Mohr-Coulomb model were employed in the finite element code, PLAXIS (Brinkgreve 2002). The overconsolidated crust and fill layer was simulated by the Mohr-Coulomb theory, whereas the soft clays were conveniently modelled by the Soft Soil model. Soil formation was divided into 3 layers, namely, ballast and fill, Soft soil-1 and Soft soil-2. The soil parameters were given in Table 3.

A vertical cross-section of mesh discretisation of the formation beneath the rail track is shown in Figure 18. A plane strain finite element analysis employed triangular elements with 6 displacement nodes and 3 pore pressure nodes. Total of 4 PVDS were used in the analysis. An equivalent plane
strain analysis with conversion from axisymmetric to 2-D was adopted to analyse the multi-drain analysis (Indraratna et al., 2005b). In this method, the corresponding ratio of the smear zone permeability to the undisturbed zone permeability is obtained by:

\[
\frac{k_{h,ps}}{k_{h,ax}} = k_{h,ps}/k_{h,ax} \frac{\ln(n/s) + k_{h,ax}/k_{s,ax} \ln(s) - 0.75}{\alpha} \tag{7}
\]

\[
\alpha = 0.67(n-s)^{1/2}n^2(n-1),
\tag{7a}
\]

\[
\beta = 2(s-1)[n(n-s-1) + 0.33(s^2 + s + 1)]/n^2(n-1)
\tag{7b}
\]

\[
n = d_s/d_w
\tag{7c}
\]

\[
s = d_s/d_w
\tag{7d}
\]

In the above expressions, \(d_s\) = the diameter of unit cell soil cylinder, \(d_w\) = the diameter of the smear zone, \(d_w\) = the equivalent diameter of the drain, \(k_c\) = horizontal soil permeability in the smear zone, \(k_{h}\) = 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.

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Challenge C: Increasing Freight capacity and services

**Figure 16:** Site location (adopted from Hicks, 2005)

**Figure 17:** Soil properties at Sandgate Project (Indraratna et al. 2010c)

**Table 3.** Selected parameters for soft soil layer used in the FEM (Indraratna et al. 2010c)

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>Depth of layer (m)</th>
<th>Model</th>
<th>(c) (kPa)</th>
<th>(\phi)</th>
<th>(\epsilon_0)</th>
<th>(\lambda) / ((I+\epsilon_0))</th>
<th>(\kappa) / ((I+\epsilon_0))</th>
<th>(k_h) ((\times 10^{-4}) m/day)</th>
<th>(k_v) ((\times 10^{-4}) m/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft soil-1</td>
<td>1.0-10.0</td>
<td>Soft Soil</td>
<td>10</td>
<td>25</td>
<td>2.26</td>
<td>0.131</td>
<td>0.020</td>
<td>0.70</td>
<td>1.4</td>
</tr>
<tr>
<td>Soft soil-2</td>
<td>10.0-20.0</td>
<td>Soft Soil</td>
<td>15</td>
<td>20</td>
<td>2.04</td>
<td>0.141</td>
<td>0.017</td>
<td>0.75</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Note: \(\phi\) Back-calculated from Cam-clay M value

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

\[
k_{h,ps}/k_{h,ax} = 0.67(n-1)^{1/2}n^2[\ln(n) - 0.75]
\tag{8}
\]

In the above equation, the equivalent permeability in the smear and undisturbed zone vary with the drain spacing, \(n\).
4.3 Comparison of field results with CLASS A FE predictions

The field results were released to the Authors by the track owner (Australian Rail Track Corporation) after a year subsequent to the analysis. Therefore all prediction can be categorized as Class A (Lambe, 1973). A spacing of 2 m was adopted for Mebra (MD88) vertical drains of 8 m in length, based on the Authors’ analysis. 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 19. 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 20. As expected, the maximum displacements were measured within the top clay layer, i.e., the softest soil below the 1 m crust. It is also observed that the lateral displacement was restricted to the topmost compacted fill (0–1 m deep). The Class A predictions of lateral displacements were also in 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 evident.

![Figure 19: Predicted and measured at the centre line of rail tracks (after Indraratna et al. 2010c)](image1)

![Figure 20: Measured and predicted lateral displacement at the embankment toe at 180 days (after Indraratna et al. 2010c)](image2)

CONCLUSIONS

The performance of ballasted rail tracks with geosynthetic reinforcement and PVDs has been discussed through large-scale laboratory tests, field trials, theoretical modelling, and numerical simulations. The complex deformation and degradation mechanisms have been modelled by elasto-plastic constitutive relations incorporating dilatancy and particle breakage. The results highlight that particle breakage and confining pressure have a significant influence on the permanent deformation of ballast. During cyclic loading, breakage was most significant at low and high values of confining pressure, with minimal breakage at some intermediate value. Increasing confining pressure to an optimal range of 30–75 kPa results in minimum breakage, which improves the track performance, and reduces the maintenance costs. Among several measures used to increase the confining pressure, reinforcing geosynthetics is considered to be more practical and economic. The implications of ballast breakage on the friction angle and dilatancy of ballast layer are discussed in the context of finite element simulations. It is shown that predictions of elasto-plastic finite element analysis are in good agreement with the field data.

The finite element analysis of the track behaviour conducted prior to track construction was considered as a Class A prediction, with field monitoring since the time of construction and even as of today. It is shown that PVDs can decrease the buildup of excess pore water pressure during cyclic loading, and also continue to dissipate excess pore water pressure during the rest period. The dissipation of the pore water pressure during the rest period made the track more stable for the next loading stage. Even with the relative short PVDs, both the predictions and field data proved that the lateral displacement can be curtailed. The equivalent plane strain finite element analysis is adequate to predict the behaviour of track improved by short PVDs, as long as the soil parameters are known to a good accuracy from laboratory and field testing. The field tests carried out at Bulli and Sandgate validated the findings of the numerical studies and also proved the effectiveness of using geosynthetics and PVDs in rail track to minimise deformation and degradation. Use of synthetic grids and geosynthetic drains in ballasted track stabilisation also proved to be a feasible and effective alternative.

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REFERENCES

Challenge C: Increasing Freight capacity and services