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Track Stabilisation with Geosynthetics and Geodrains, and Performance Verification through Field Monitoring and Numerical Modelling

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Abstract

All over the world, ballasted railway tracks form one of the major transportation networks designed to provide heavy haul freight and passenger traffic. However, large cyclic loading from heavy axle trains operating at high speeds often causes excessive deformation and degradation of ballast, as well as unacceptable differential settlement of compressible foundation and, or pumping of the soft subgrade soils. The problem becomes more severe under high impact loads due to rail or wheel imperfections, causing accelerated ballast breakage. A proper understanding of load transfer mechanisms and their effects on track deformations are essential prerequisites for minimising maintenance costs. The field trial at Bulli demonstrated that for trains with wheel flats, extremely high stresses were transmitted to the ballast layer. Installing resilient mats such as rubber pads (shock mats) in rail tracks can attenuate impact forces and consequently mitigate particle degradation. In view of this, a series of laboratory tests were carried out using a unique large-scale drop-weight (impact) rig to evaluate the role of shock mats. The field trial also showed that the moderately-graded recycled ballast, when used with a geocomposite layer, could perform well in comparison with traditionally uniform fresh ballast.

Both Class A predictions and field measurements at Sandgate proved that relatively short vertical drains would be sufficient to dissipate cyclically induced pore pressures, curtail the lateral movements, and increase the shear strength and bearing capacity of the subgrade. In summary, this invited Special Paper describes in detail the large-scale laboratory tests imperative for material characterisation, full-scale instrumented field trials for performance verification, elasto-plastic finite element analyses for predicting the behaviour of tracks stabilised using shock mats, and geosynthetic products including grids and prefabricated drains.

Keywords: ballast, geosynthetics, impact forces, rail track, deformations, subsurface drainage.

1 Introduction

Railway systems continues to be an important mode of transportation for both passengers and cargo. Ballast is the largest component of a rail track by weight and volume. It plays an important role in supporting heavy traffic loading, preventing track deformation, and providing adequate drainage of water in the track structure. However, the progressive deterioration and breakdown of ballast due to increased train speeds and heavier axle loads is a key factor in track geometry change and increased track maintenance costs. Discontinuity of the rail or wheel irregularities: such as flat wheels, dipped rails, *etc.*, cause large dynamic impact loads which are detrimental to the track structure. These impact loads are of very high magnitude and short duration, depending on the nature of wheel or rail irregularities, and on the dynamic response of the track [1, 2]. Many coastal regions of Australasia contain very soft clays which possess undesirable geotechnical properties: such as high compressibility, low permeability and low bearing capacity. The railway embankments built on such soft soil deposits are severely affected by the excessive differential settlements, lateral displacements. In order to improve track conditions and optimise the life-cycle cost of track, the use of geosynthetic grids, resilient mats, and prefabricated vertical drains, in the form of the following methods, can be considered:

- (a) inserting geosynthetics at the ballast-subballast interface or subballast-subgrade interface to improve the track stability and to provide optimum filtration, adequate separation, and better drainage for the ballast layer simultaneously.
- (b) placing shock mats at the top and bottom of the ballast layer to attenuate extremely high transient impact loads and mitigate associated ballast degradation.
- (c) using prefabricated vertical drains (PVDs) for rapid excess pore pressure dissipation in order to increase shear strength of the soft formation soils.

The application of geosynthetics beneath the ballast layer for drainage and internal track confinement, and as separation layer between the ballast and subballast, are highly desirable [3, 4]. A layer of geocomposite (biaxial geogrid bonded with non-woven geotextiles) stabilised recycled ballast much better than standard geogrids, and also prevented the ballast from being fouled due to fines migrating from layers of subballast and subgrade [5, 6, 7, 8]. 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 nonwoven polypropylene geotextile) installed at the ballast-subballast interface. The relative performance of moderately graded recycled ballast compared to the very uniform fresh ballast, was also evaluated.

Installing resilient mats, such as rubber pads (shock mats), in rail tracks can attenuate the dynamic impact force substantially. The effectiveness of shock mats in reducing noise along stiff tracks (*e.g.* concrete bridges and tunnels), and controlling vibration along open tracks, has been studied previously [9, 10].

The combination of preloading with prefabricated vertical drains (PVDs) is widely considered to be a low cost solution to improving the performance of thick deposits of soft clays [11, 12]. When PVDs are installed on the ground, the drainage path length (radial flow) is shortened, thereby reducing the consolidation time [13, 14, 15]. Another field trial at Sandgate, NSW Australia, revealed that relatively short prefabricated vertical drains (PVDs), between 6m and 8m long, can dissipate pore pressures induced by trains, limit lateral movements, and increase the shear strength and bearing capacity of soft formation soil. This paper presents the results of laboratory testing, full-scale field monitoring, theoretical modelling, and finite element analyses, demonstrating the beneficial use of geosynthetic grids, shock mats and drains for rail infrastructure.

2 Use of shock mats for mitigating ballast breakage

In this study a series of laboratory tests were carried out to evaluate the effectiveness of shock mats in the attenuation of high frequency impact loads and subsequent mitigation of ballast deformations and degradation.

2.1 Test apparatus and instrumentation

The large scale drop-weight impact testing equipment consists of a free-fall hammer of 5.81kN weight that can be dropped from a maximum height of 6m with an equivalent maximum drop velocity of 10m/s. The hammer was hoisted mechanically to the required drop height and released by an electronic quick release system (Figure 1a). To eliminate surrounding noise and ground motion, an isolated concrete foundation (5.0m × 3.0m × 2.5m) was designed to have a significantly higher

Figure 1: (a) Drop weight impact testing equipment; (b) Instrumentation details (load-cell and accelerometer)

fundamental frequency than the test apparatus. The transient impact forces were recorded by a dynamic load cell (with a capacity of 1200kN) mounted on the drop-weight hammer. A piezoelectric accelerometer (capacity of 10,000g, where g is the gravitational acceleration) (Figure 1b) was used to capture records of transient accelerations, and sample deformations were obtained after each blow. The load cell and accelerometers were connected to a computer controlled data acquisition system. The details of material specifications and test procedures are given elsewhere [16].

2.2 Impact loading

The impact load-time history under a single impact load (1st drop of the free-fall hammer) is shown in Figure 2a. Two distinct types of peak forces were seen during impact loading: an instantaneous sharp peak with very high frequency, and a gradual peak of smaller magnitude with relatively lesser frequency. Jenkins *et al.* [1] termed these peak forces as P_1 and P_2 respectively, universal terminology that is now used widely by track engineers. P_1 is a high-frequency dynamic load that occurs when a vibration mode between the wheel and rail (coupled by the very stiff contact zone) is excited, while P_2 is a low-frequency dynamic load that occurs when the coupled wheel-rail vibrates in phase on the ballast [17, 18]. In the laboratory model, the impactor-shaft simulates the wheel-rail contact zone. Since P_2 is the force that has direct influence on the degradation of track bed, U.K. Railway group standards [19, 20] recommends consideration of the P_2 force in the track design criteria.

It was also evident that multiple P_1 type peaks followed by the distinct P_2 type peak often occurred. The multiple P_1 peaks occurred when the drop hammer was not restrained vertically, so that it rebounded after the first impact and hit the specimen again. The observed benefits of a shock mat are therefore twofold: (a) it attenuates the impact force and (b) it reduces the impulse frequencies, thereby extending the time duration of impact. Figure 2b shows the variation of P_2 with repeated hammer blows (N) for both stiff and weak subgrades.

The P_2 force showed a gradual increase with the increased number of blows. This was because the ballast develops a denser (compressed) packing assembly arising from reorientation and rearrangement of ballast aggregates. A dense aggregate matrix offers a higher inertial resistance, which leads to an increased value of P_2 . A rapid increase of P_2 occurred at the initial stages of impact loading, but became almost insignificant after the eighth blow. This indicates that the ballast mass stabilises after a certain number of impacts to produce an almost constant P_2 . Even without a shock mat, a ballast bed on a weak subgrade leads to a decreased magnitude of impact force compared to a stiffer subgrade.

(a)

(b)

Figure 2: (a) Typical impact force responses for stiff subgrade; (b) Variation of impact force with number of blows (data sourced from Nimbalkar *et al.*, [16])

2.3 Shear strain and volumetric strain response

Vertical and lateral deformations of the ballast sample were recorded after each blow. The shear strain (ε_s) and volumetric strain (ε_v) for axisymmetric loading condition are presented in Figures 3a and 3b respectively. In general, both the shear strain and volumetric strains increase with successive impacts. This is because the ballast layer displays a strong tendency to compact under repetitive loading [21, 22], and a similar behaviour is observed in the current study.

Figure 3: Permanent strain response of ballast with and without shock mat: (a) shear strain; (b) volumetric strain (data sourced from Nimbalkar *et al.*, [16])

Large permanent strains (shear and volumetric strains) are induced in the ballast bed for stiff subgrade. However, with shock mats placed at the top and bottom of the ballast, the shear and volumetric strains are reduced significantly (*i.e.* in the order of 40% to 50%). For weak subgrade conditions, this improvement is relatively less marked. Placement of shock mats at the top and bottom of the ballast mass provides the significant reduction of the impact induced strains.

2.4 Ballast breakage

Breakage of corners of sharp angular ballast particles and attrition of asperities occur progressively under heavy impact loading. Particle breakage severely affects the strength and deformation of ballast [23, 24, 25]. After each test the ballast sample was sieved and a change in gradation was obtained. Breakage was quantified using the ballast breakage index (*BBI*), proposed by Indraratna *et al.* [25]. The *BBI* values are presented in Table 1.

Test No.	Base type	Shock Mat Details	<i>BBI</i>
1	Stiff	Without shock mat	0.170
2	Stiff	Shock mat at top and bottom of ballast	0.091
3	Weak	Without shock mat	0.080
4	Weak	Shock mat at top and bottom of ballast	0.028

Table 1: Ballast breakage under impact loading (data sourced from Nimbalkar *et al.*, [16])

The higher breakage of ballast particles can be attributed to the considerable non-uniform stress concentrations occurring at the corners of the sharp angular particles of fresh ballast under high impact induced contact stresses. The application of just 10 impact blows caused considerable ballast breakage (*i.e.* *BBI* = 17%) when a stiff subgrade was used, but when a shock mat was placed above and below the ballast bed, particle breakage was reduced by approximately 47% for a stiff subgrade, and approximately 65% for a weak subgrade.

3 Stabilising a recycled ballasted track using geosynthetics

Geosynthetics have been widely and successfully used in road, rail, port and airport transport infrastructure. When appropriately designed and installed, geosynthetics are a cost effective alternative in construction of new tracks and in track rehabilitation schemes. The applications of geosynthetics within railway construction can be subdivided into (1) separation, (2) reinforcement, (3) filtration, (4) drainage, (5) moisture barrier/waterproofing, and (6) protection. The geocomposites can provide reinforcement to the ballast layer, as well as

simultaneous filtration and separation functions [27]. The combination of reinforcement by the geogrid and the filtration and separation functions provided by the bonded non-woven geotextile, reduce the lateral spreading and fouling of ballast, as well as ballast degradation, especially in wet conditions. The non-woven geotextile also prevents the fines moving up from the subballast and subgrade layers (subgrade pumping), thereby keeping the recycled ballast relatively clean.

In order to investigate train traffic induced stresses and deformations inside multi-layer rail track systems, and the 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 [22, 28]. The University of Wollongong provided technical specifications for the design, while RailCorp (Sydney) 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 south coast of NSW. The length of the instrumented section of track was 60m, which was divided into four 15m long sections. The layers of load bearing ballast and subballast were 300mm and 150mm, respectively. The particle size, gradation, and other index properties of fresh ballast used at the Bulli site were in accordance with the Technical Specification [29], which represents sharp angular coarse aggregates of crushed volcanic basalt (latite). Recycled ballast was collected from spoil stockpiles of a recycled plant commissioned by RailCorp at their Chullora yard near Sydney. Concrete sleepers were used. Electrical Friction Cone Penetrometer (EFCP) tests reported that the subgrade soil is a stiff, over-consolidated, silty clay, and had more than sufficient strength to support the train loads [30]. The bedrock is a highly weathered sandstone having weak to medium strength [31]. 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. Technical specifications of various test materials and instruments used at the site can be found in Indraratna *et al.* [22].

3.2 Track deformations

In order to investigate the overall performance of the ballast layer, the average vertical and lateral deformations were determined from the mean of measurements at sleeper-ballast and ballast-capping interfaces. The values of average ballast deformation are plotted against the number of load cycles (N) in Figure 4. This particular recycled ballast performed very well *i.e.* showed less vertical and lateral deformations because of its moderately-graded particle size distribution, compared to the very uniform fresh ballast. Recycled ballast often has less breakage because they are less angular, which prevents corner breakage resulting from high contact stresses.

Figure 4: Average deformations of the ballast layer: (a) vertical; (b) lateral (data sourced from Indraratna *et al.*, [22])

It was evident that geocomposite reduced the vertical deformation of fresh ballast by 33% and that of recycled ballast by 9%. It also reduced lateral deformation of fresh ballast by about 49% and that of recycled ballast by 11%. The apertures of the geogrid offered strong, mechanical interlock with the ballast, forming a highly frictional interface. The capacity of the ballast layer to distribute load was improved by the placement of a flexible and resilient geocomposite layer, which also substantially reduced settlement under high cyclic loading.

3.3 Traffic induced stresses in ballast

Figure 5(a) shows the maximum cyclic stresses (σ_v , σ_h) recorded in Section 1 arising from the passage of a coal train with 100T wagons (25 tons axle load), where the stresses were measured under the rail and at the edge of the sleeper. The large vertical stress and relatively small lateral (confining) stress caused large shear strains in the track. The corresponding ease of lateral spread arising from the absence of sufficient confinement, increased the vertical compression of the ballast layer. Also, σ_v and σ_h increased with an increase in the number of load cycles, which further degraded the track bed.

Figure 5: Cyclic stresses induced by coal train with wagons (100 tons) (data sourced from Nimbalkar *et al.*, [2])

A typical plot of vertical cyclic stress transmitted to the ballast under an axle load of about 25 tons and a train speed of about 60km/h, is shown in Figure 5(b). While most of the maximum vertical cyclic stresses were up to 230kPa, one peak reached 415kPa. This high magnitude of stress was associated with a wheel flat, which proved that large dynamic impact stresses can be generated in the ballast by wheel imperfections: a fact that should be carefully assessed and accounted for in the design and maintenance of ballasted track beds.

4 Numerical modelling using PLAXIS

4.1 Model formulation

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 [32], to predict track behaviour with and without geosynthetics. PLAXIS has already demonstrated its success with limit analysis of geotechnical problems. A typical plain strain track model was numerically simulated by finite element (FE) discretisation, as shown in Figure 6.

Figure 6: (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 track layers were modelled using 15-node linear strain quadrilateral elements (LSQ). 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 3m high and 6m

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.* they were restrained in both vertical and horizontal directions (standard fixities). The left and right boundaries were restrained in the horizontal direction to represent smooth vertical contacts. 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 was 1.68m. The shoulder width of ballast was 0.35m, and the side slope of the rail track embankment was 1:2.

4.2 Modified flow rule incorporating particle breakage

The flow rule adopted in Hardening Soil (HS) model is characterised by a classical linear relation:

$$d\varepsilon_v^p = \sin \psi_m d\varepsilon_s^p \quad (1)$$

where, $d\varepsilon_v^p$ and $d\varepsilon_s^p$ are the plastic components of volumetric and shear strain increments respectively. The mobilised dilatancy angle ψ_m is given by [33]:

$$\sin \psi_m = (\sin \phi_m - \sin \phi_{cv}) / (1 - \sin \phi_m \sin \phi_{cv}) \quad (2)$$

where ϕ_{cv} is a material constant (the friction angle at critical state) and the mobilised friction angle ϕ_m is given by :

$$\sin \phi_m = (\sigma'_1 - \sigma'_3) / (\sigma'_1 + \sigma'_3) \quad (3)$$

Indraratna and Salim [34] described the dependence of particle breakage and dilatancy on the friction angle of ballast. They developed a continuum-mechanics-based constitutive model incorporating dilatancy and plastic flow rule to predict particle breakage. Salim and Indraratna [35] used a generalised three-dimensional system to define contact forces, stresses and strains in granular media, including the plastic potential, hardening function and particle breakage. The model is based on the critical state concept, and the theory of plasticity with a kinematic-type yield locus (constant stress ratio). The increments of plastic distortional strain $d\varepsilon_{ps}$ and volumetric strain $d\varepsilon_{pv}$ were given as follows [35]:

$$d\varepsilon_s^p = \frac{2\alpha\kappa \left(\frac{p}{p_{cs}} \right) \left(1 - \frac{p_{o(i)}}{p_{cs(i)}} \right) (9 + 3M - 2\eta^* M) \eta d\eta}{M^2 (1 + e_i) \left(\frac{2p_o}{p} - 1 \right) \left[9(M - \eta^*) + \frac{B}{p} \{ \chi + \mu(M - \eta^*) \} \right]} \quad (4)$$

$$d\varepsilon_v^p = \frac{9(M - \eta)}{9 + 3M - 2\eta^* M} d\varepsilon_s^p + \left(\frac{B}{p}\right) \left[\frac{\chi + \mu(M - \eta^*)}{9 + 3M - 2\eta^* M} \right] d\varepsilon_s^p \quad (5)$$

where p is the effective mean stress, p_{cs} is the value of p on the critical state line at the current void ratio, p_o is the value of p at the intersection of the undrained stress path and the initial stress line, and subscript i indicates the initial value at the start of shearing: η is the stress ratio ($\eta = q/p$); q is the deviator stress, and $\eta^* = \eta(p/p_{cs})$; M is the critical state stress ratio; e_i is the initial void ratio; κ is the negative slope of the compression curve ($e - \ln p$); α and B are two dimensionless constants; and χ and μ are the material constants defining the rate of particle breakage.

A modified flow rule that includes the energy consumption due to particle breakage during shear is given by [35]:

$$\frac{d\varepsilon_v^p}{d\varepsilon_s^p} = \frac{9(M - \eta)}{9 + 3M - 2\eta M} + \frac{dE_B}{pd\varepsilon_s^p} \left(\frac{9 - 3M}{9 + 3M - 2\eta M} \right) \left(\frac{6 + 4M}{6 + M} \right) \quad (6)$$

Assuming that the incremental energy consumption due to particle breakage per unit volume is proportional to the corresponding increment of breakage index (*i.e.*, $dE_B = \beta dB_g$, where β is a constant of proportionality, and B_g is Marshal's breakage index), Equation 6 becomes:

$$\frac{d\varepsilon_v^p}{d\varepsilon_s^p} = \frac{9(M - \eta)}{9 + 3M - 2\eta M} + \frac{\beta dB_g}{pd\varepsilon_s^p} \left(\frac{9 - 3M}{9 + 3M - 2\eta M} \right) \left(\frac{6 + 4M}{6 + M} \right) \quad (7)$$

Replacing B_g with a more accurate ballast breakage index (*BBI*) defined by Indraratna *et al.* [25] gives:

$$\frac{d\varepsilon_v^p}{d\varepsilon_s^p} = \frac{9(M - \eta)}{9 + 3M - 2\eta M} + \frac{\beta d\text{BBI}}{pd\varepsilon_s^p} \left(\frac{9 - 3M}{9 + 3M - 2\eta M} \right) \left(\frac{6 + 4M}{6 + M} \right) \quad (8)$$

The material parameters and constitutive models used for each component of the track section are given in Table 2.

The experimental values of η , p , M and the computed values of $dE_B/d\varepsilon_s^p$, which are linearly related to the rate of particle breakage $d\text{BBI}/d\varepsilon_1$, can be readily used to quantify the flow rule. The values of stress-dependent stiffness moduli were obtained from previously published results of large scale drained triaxial compression tests under monotonic loading conditions [2]. The hardening soil model showed better agreement with the strain-hardening behaviour of ballast observed in large scale triaxial tests which indicated ballast breakage [26, 36, 37]. The current formulation of finite element is incapable of conducting post-peak analysis into the strain-softening region, however, in reality, such large strains, or large deformations, are not permitted.

Material Parameter	Rail Track component				
	Rail	Concrete Sleeper	Ballast	Sub-ballast	Subgrade
Model	LE	LE	HS	MC	MC
Material Type	Non-porous	Non-porous	Drained	Drained	Drained
E (MPa)	210,000	10,000	-	80	34.2
E_{50}^{ref} (MPa)	-	-	21.34	-	-
E_{oed}^{ref} (MPa)	-	-	21.34	-	-
E_{ur}^{ref} (MPa)	-	-	64.02	-	-
γ (kN/m ³)	78	24	15.6	16.67	18.15
ν	0.15	0.15	-	0.35	0.33
ν_{ur}	-	-	0.2	-	-
c (kN/m ²)	-	-	0	0	5.5
ϕ (degrees)	-	-	58.47	35	24
ψ (degrees)	-	-	12.95	0	0
P_{ref} (kN/m ²)	-	-	50	-	-
m	-	-	0.5	-	-
k_0	-	-	0.3	-	-
R_f	-	-	0.9	-	-

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

Note: *HS* = Hardening-Soil model, *MC* = Mohr-Coulomb model, *LE* = Linear Elastic model, γ = unit weight, E_{50}^{ref} = secant stiffness at 50% strength for loading conditions, E_{ur}^{ref} = triaxial unloading/reloading stiffness, E_{oed}^{ref} = tangent stiffness for primary oedometer loading, EA = elastic normal (axial) stiffness, ν = Poisson's ratio for loading conditions, ν_{ur} = Poisson's ratio for unloading/reloading conditions, c = effective cohesion, ϕ = effective friction angle, ψ = dilatancy angle, P_{ref} = reference confining pressure, m = stress dependent stiffness factor, k_0^{nc} = coefficient of earth pressure at rest for normal consolidation, R_f = failure ratio.

4.3 Comparison of field measurements with the finite element predictions

In order to validate the findings of the elasto-plastic finite element (FE) analysis, a comparison was made between the FE analysis and the field data. Figures 7 and 8 show variations in the vertical stress and vertical ballast deformations predicted by the 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-subballast interfaces using settlement pegs. The values predicted

by elasto-plastic analysis showed a 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 approximately represented by equivalent dynamic plain strain analysis in FE studies.

Figure 7: Variation of vertical stress of ballast with the depth

Figure 8: Variation of vertical deformation of ballast with the depth

5 Use of geosynthetic vertical drains as subsurface drainage

Low lying areas with large volumes of plastic clays can sustain high excess pore water pressures during static and repeated loading. In low permeability soils, the increase in excess pore pressures has an adverse effect on the effective load bearing capacity of the soil formation. Under certain circumstances, clay pumping beneath rail tracks may cause pumping of the soil upwards, thereby fouling the ballast bed and promoting undrained shear failure [38, 39]. Geosynthetic prefabricated vertical drains (PVDs) can be used to dissipate excess pore pressures by radial consolidation before they can develop to critical levels. These PVDs continue to dissipate excess pore water pressures even after the cyclic load ceases [40].

5.1 Use of short PVDs under railway tracks

The Sandgate Rail Grade Separation Project is located in the town of Sandgate, in the Lower Hunter Valley of NSW (Figure 9). New railway tracks were required to ease the traffic in the Hunter Valley Coal network. In this section the use of short PVDs in the soft subgrade for track stabilisation is presented together with the background of the project including: the soil improvement details, the design methodology, and the FE analysis. The effectiveness of PVDs in improving soil conditions has been demonstrated by Indraratna *et al.* [41]. Preliminary site investigations were carried out for the soil profile along the track. In-situ and laboratory testing programs were conducted to provide relevant soil parameters. The site investigation included 6 boreholes, 14 piezocones (CPTU) tests, 2 in-situ vane shear tests, and 2 test pits: including laboratory testing such as soil index property testing, standard oedometer testing, and vane shear testing.

Figure 9: Site location [41] (reproduced with permission from ASCE)

A typical soil profile showed that the thickness of the soft compressible soil varied from 4m to 30m (Figure 10). Soft residual clay lies beneath a layer of soft soil, followed by shale bedrock. The groundwater level is located at the ground surface. The moisture contents of the soil layers were almost the same as their liquid limits. The soil unit weight varied from 14kN/m^3 to 16kN/m^3 . The undrained shear strength increased from about 10kPa to 40kPa with depth. The clay deposit at this site can be considered as lightly overconsolidated ($OCR \approx 1-1.2$). The horizontal coefficient of consolidation (c_h) was about 2-10 times the vertical coefficient of consolidation (c_v).

Figure 10: Soil profiles at Sandgate Rail Grade Separation Project [41] (reproduced with permission from ASCE)

Based on a preliminary numerical analysis conducted by Indraratna *et al.* [42], PVDs 8m long were suggested and installed 2m apart in a triangular pattern. An extensive field instrumentation scheme that included settlement plates, inclinometers, and vibrating wire piezometers, was installed to monitor the track. The settlement plates were installed above the layer of subgrade to directly measure the vertical settlement of the subgrade. The main aims of the field monitoring were to: (a) ensure the stability of the track; (b) validate the design of 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).

5.2 Preliminary design

Due to the time constraints in the contractual agreements, the rail track was constructed immediately after PVDs were installed. A train load moving at very low speed was allowed as an external surcharge. The equivalent dynamic loading with an impact load factor was utilised to predict the behaviour of the track. In this analysis

a static pressure of 104kPa, with an impact factor of 1.3, was applied according to the low train speed (60km/h) for axle loads up to 25 tons, based on Australian Standards [43]. The Soft Soil model and Mohr-Coulomb model were both used in the FE code, PLAXIS [44]. The over-compacted surface crust and fill layer were simulated by the Mohr-Coulomb theory, whereas the soft clay deposit was modelled using the Soft Soil model. The formation was separated into 3 distinct layers, ballast and fill, Soft soil-1 (depth 1.0m – 10.0m), and Soft soil-2 (depth 10.0m – 20.0m). The relevant soil parameters are given in Table 3.

Soil layer	c (kPa)	ϕ	$\lambda/(1+\varepsilon_0)$	$\kappa/(1+\varepsilon_0)$	k_h ($\times 10^{-4}$ m/day)
Soft soil-1	10	25	0.131	0.020	1.4
Soft soil-2	15	20	0.141	0.017	1.5

Note: ϕ is back-calculated from the Cam-clay M value

Table 3: Selected parameters for soft soil layer used in the FEM [41]
(reproduced with permission from ASCE)

A cross section of the FE mesh discretization of the formation beneath the track is shown in Figure 11. A two-dimensional plane strain FE analysis was used with linear strain triangular elements with 6 displacement nodes and 3 pore pressure nodes. A total of 4 drain walls were used in the analysis. An equivalent plane strain analysis, with appropriate conversion from axisymmetric to two-dimensions, was used to analyse the multi-drain analysis [37]. In this method, the corresponding ratio of the smear zone permeability to the undisturbed zone permeability is given by:

$$\frac{k_{s,ps}}{k_{h,ps}} = \frac{\beta}{k_{h,ps}/k_{h,ax} [\ln(n/s) + k_{h,ax}/k_{s,ax} \ln(s) - 0.75] - \alpha} \quad (9)$$

$$\alpha = 0.67(n-s)^3 / n^2(n-1) \quad (9a)$$

$$\beta = 2(s-1)[n(n-s-1) + 0.33(s^2 + s + 1)] / n^2(n-1) \quad (9b)$$

$$n = d_e / d_w \quad (9c)$$

$$s = d_s / d_w \quad (9d)$$

In the above expressions d_e = the diameter of unit cell soil cylinder, d_s = the diameter of the smear zone, d_w = the equivalent diameter of the drain, k_s = 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.

Figure 11: Vertical cross section of rail track and foundation [41]
(reproduced with permission from ASCE)

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

$$k_{h,ps}/k_{h,ax} = 0.67(n-1)^2 / [n^2 [\ln(n) - 0.75]] \quad (10)$$

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

5.3 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. The 8m long vertical band drains were spaced 2m apart. The field data and numerical predictions were compared and discussed. The calculated and observed vertical consolidation settlements at the centre line are presented in Figure 12.

The predicted settlement agrees with the field data. The in-situ lateral displacement at the toe of the rail embankment after 6 months is illustrated in Figure 13. As expected, the maximum lateral displacements occurred within the top layer of clay, *i.e.* the softest soil below the 1m crust. Lateral displacement was curtailed to the topmost compacted fill (0-1m deep). The Class A prediction of lateral displacements also agreed with the field observation. The effectiveness of PVDs in reducing the effects of undrained cyclic loading through the reduction in lateral movement, is undeniably evident.

Figure 12: Predicted and measured at the centre line of rail tracks [41] (reproduced with permission from ASCE)

Figure 13: Measured and predicted lateral displacement at the embankment toe at 180 days [41] (reproduced with permission from ASCE)

6 Conclusions

The observations recorded in the instrumented section of track at Bulli and Sandgate validate the analytical and numerical investigations carried out at the University of Wollongong, and highlight the successful application of geosynthetics and geodrains for improved track performance. The results highlight that particle breakage, confining pressure, soft formation in addition to train loading patterns (cyclic and impact) have a significant influence on the engineering behaviour of ballasted rail track.

The field tests carried out on the instrumented track at Bulli demonstrate the potential benefits of using a geocomposite (combination of biaxial geogrid and nonwoven polypropylene geotextile) to minimise the deformation and degradation of rail tracks. The sophisticated instrumentation scheme used during field monitoring led to a significant understanding of the mechanisms of stress-transfer and accumulated lateral deformations that may affect track stability arising from the loss of lateral confinement. The field results highlighted the fact that recycled ballast performs satisfactorily under repeated train loads compared to the fresh ballast as a consequence of its broader gradation. The test results reported that geocomposite reduced the vertical deformation of fresh ballast by 33% and recycled ballast by 9%. It also reduced the lateral deformation of fresh ballast by approximately 49% and recycled ballast by 11%. The complex deformation and degradation mechanisms were modelled by elasto-plastic constitutive relationships that incorporated dilatancy and particle breakage. It was shown that the predictions of elasto-plastic finite element analysis were in good agreement with the field data.

It was also shown that the benefits of shock mats were better for stiff subgrades than relatively soft foundations for attenuating the impact forces. The laboratory test results revealed that the shock mats could decrease impact induced strains in ballast by as much as 50% when placed both at the top and bottom of the ballast layer. Impact caused the most significant damage to ballast, especially under high repetitive loads: just 10 impact blows caused considerable ballast breakage (i.e. BBI = 17%) when a stiff subgrade was used. However, when a shock mat was placed at the top and bottom of the ballast bed, particle breakage was reduced by approximately 47% using a stiff subgrade, and by approximately 65% for a weak subgrade.

The field trial at Sandgate showed that geosynthetic prefabricated vertical drains can decrease the build-up of excess pore water pressure during cyclic loading, and continue to dissipate excess pore water pressure during the rest period. This dissipation of pore water pressure during the rest period made the track more stable for the next train passage (loading stage). Even with relatively short drains, both the predictions and field data proved that lateral displacement can be curtailed. The equivalent plane strain FE analysis was adequate enough to predict the behaviour of track improved by short PVDs, as long as the soil parameters obtained from laboratory and field testing are accurate. A longer maintenance cycle is possible using geosynthetics and geodrains in rail tracks, which in turn helps defray the high costs associated with maintaining ballasted tracks.

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