Stabilisation of rail tracks and underlying soft soil formations

B. Indraratna* M. A. Shahin†

C. Rujikiatkamjorn‡

*University of Wollongong, indra@uow.edu.au

†University of Wollongong

‡University of Wollongong, cholacha@uow.edu.au

This paper was originally published as: Indraratna, B, Shahin, M & Rujikiatkamjorn, C, Soft soil stabilisation under railway embankments, Indian Geotechnical Conference, India, 2006, 41-50.

STABILISATION OF RAIL TRACKS AND UNDERLYING SOFT SOIL FORMATIONS

Buddhima Indraratna
Professor of Civil Engineering, Faculty of Engineering, University of Wollongong, NSW 2522, Australia.
indra@uow.edu.au

Mohamed A. Shahin
Research Fellow, School of Civil, Mining & Environmental Engineering, University of Wollongong, NSW 2522, Australia.
shahin@uow.edu.au

Cholachat Rujikiatkamjorn
Research Associate, School of Civil, Mining & Environmental Engineering, University of Wollongong, NSW 2522, Australia.
cholacha@uow.edu.au

ABSTRACT: Construction of rail tracks requires appropriate improvement techniques for ballast and underlying soft formation soil, both of which deform and degrade progressively under heavy train loads. This paper aims to demonstrate and discuss some major aspects in relation to stabilisation of ballasted rail tracks overlaying soft formation soils. The use of geocomposites (i.e. bonded geogrid-geotextile layers) for enhancing the performance of rail tracks is described, with the aim of achieving reduced track settlement, increased resilient modulus and decreased ballast degradation. The effects of increasing the confining pressure on rail tracks in regard to particle breakage are studied using laboratory tests. The utilization of prefabricated vertical drains (PVDs) to improve soft subgrade and provide better drainage system is introduced. Finite element analysis using an equivalent plane strain conversion is employed to predict settlements, excess pore pressures and lateral displacements under rail tracks. Finally, the use of native vegetation to stabilise soft formations of rail tracks is discussed, with the aid of preliminary suction models based on evapotranspiration mechanics applied to tree roots.

1 INTRODUCTION

In Australia, the railway system plays a significant role in hauling bulk commodities to ports, conveying passengers and transporting freight along major corridors. Rail tracks are conventionally founded on compacted ballast platforms, which are laid on natural subsoil. The rail track substructure can be divided into 2 major parts, namely, ballast and subgrade. Ballast is the largest component of the track by weight and volume. Ballast materials usually include dolomite, rheolite, gneiss, basalt, granite and quartzite (Raymond, 1979). Railway ballast deforms and degrades progressively under heavy train (cyclic) loading, especially on soft formation. Ballast degradation and deformation are influenced by several factors including the amplitude/number of load cycles, density of aggregates, track confining pressure, angularity, and most importantly, the fracture strength of individual grains. It is essential to have an appropriate track design and firm foundation to prevent the deviation of rail tracks alignment.

The subgrade is composed of naturally deposited soil, fill material or a combination of both. Its primary function is to provide a stable foundation to the track. In soft formation areas, high volumes of plastic clays can sustain the excess pore water pressures during both static and cyclic (repeated) loading. The excessive pore water pressures drastically decreases the bearing capacity of the undrained formation, leading to overall track failure.

In this paper, the deformation and degradation behaviour of ballast under static and dynamic loading conditions are studied. The prospective use of different types of geosynthetics to improve the performance of fresh and recycled ballast is investigated. The benefits of increasing the confining pressure on rail tracks are highlighted in relation to particle breakage. The role of short vertical drains for improving soft subgrade formation drainage conditions is investigated, and the role of vertical drains to improve the soft subgrade formation stability is demonstrated. The use of native vegetation to stabilize track soft formations is described and discussed.

2 BEHAVIOR OF BALLAST

The key functions of ballast are: distributing load from sleepers, damping of dynamic loads, increasing lateral resistance and providing free draining conditions. To explain this point, the main geotechnical properties of ballast are discussed below.

2.1 Shear Strength

The shear strength of granular materials is usually considered to vary linearly with the applied stress range based on the Mohr-Coulomb theory. Indraratna et al. (2000) and Ramamurthy (2001) have shown that a non-linear shear strength response is observed, when soils at high stress levels and rocks at low normal stresses are tested. Therefore, the constant cohesion intercept, *c*, and angle of shearing resistance, ϕ, cannot be used to accurately represent the failure envelopes corresponding to the entire stress range. Indraratna et al. (1993) introduced a non-linear strength envelope obtained during the testing of granular media at low normal stress. This non-linear shear strength envelope can be expressed by the following equation:

$$\tau_f / \sigma_c = m(\sigma'_n / \sigma_c)^n \tag{1}$$

where, $\sigma'n$ is the effective normal stress, m and n are dimensionless constants, τ_f is the shear strength at failure and σ_c is the uniaxial compressive strength of parent rock determined from the point load test. The non-linearity of the strength envelope is controlled by the coefficient *n*. For small confining pressures (< 200 kPa) representative of rail tracks, *n* is in the order of 0.65 – 0.75. Indraratna et al. (1998) used a large-scale cylindrical triaxial apparatus, accommodating specimens of 300 mm diameter and 600 mm high, to verify Equation (1) for latite basalt. The results in a normalized form are plotted in Figure 1, with other results obtained for various sources of basalt (Marachi et al., 1972; Marsal, 1973; Charles and Watts, 1980).

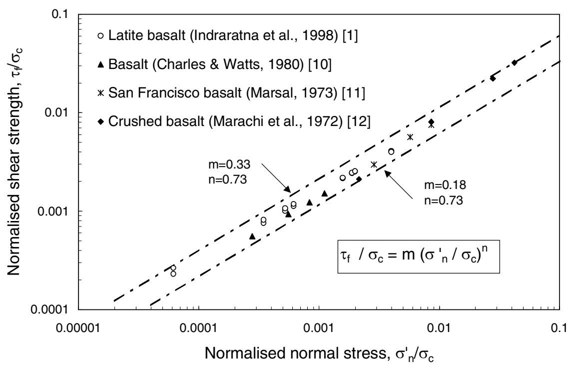

Fig. 1 Normalized shear strength for latite basalt aggregates (Indraratna et al., 1998)

2.2 Settlement

The settlement of ballast can be divided into two categories including elastic (such as the initial settlement due to the compaction of ballast) and plastic (due to breakage of ballast particles). Settlement of ballast may not be a problem if it is occurring uniformly along the length of the track (Selig and Waters, 1994). In fact, differential track settlement is more important than the total track settlement. Over 50% of the total deformation of railway lines (both differential and uniform) originates from the ballast layer (Suiker, 1997; Ionescu et al. 1998). Based on laboratory studies, Jeffs and Marich (1987) showed that the relationship between the number of load cycles and settlement of ballast is nonlinear. The rail track settlement can be determined by:

$$S_N = a(1 + k \log N) \tag{2}$$

where, S_N is the settlement of ballast at *N* load cycles, *a* is the settlement at first cycle and *k* is an empirical constant depending on initial compaction, type of ballast, type of reinforcement and degree of saturation. Indraratna et al. (2000) proposed an alternative relationship for settlement under cyclic load represented by a power-function of the number of load cycles as follows:

$$S_N = aN^b \tag{3}$$

where, *a* and *b* are empirical coefficients determined from non-linear regression analysis. Equation (3) can be used to model the ballast settlement as shown in Figure 2. It is noted that the coefficient *a* depends only on the variation of the applied load, whereas the coefficient *b* remains constant (Figure 2).

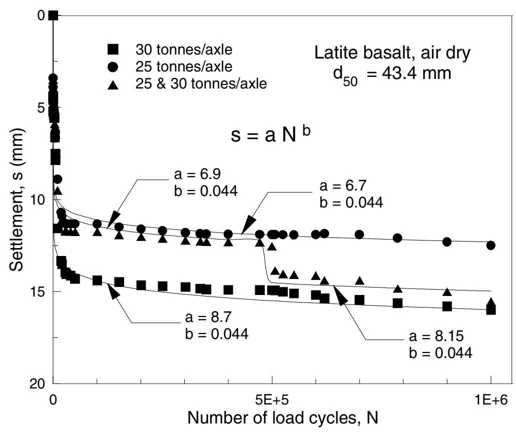

Fig. 2 Effect of load cycles and axle loads on settlement (Indraratna et al., 2000)

2.3 Ballast Degradation

Excessive cyclic loading and vibration, temperature and moisture fluctuation, as well as impact load on ballast may cause ballast degradation. The particle degradation can occur in three ways (Raymond and Diyaljee, 1979):

(a) The angular projections breakage, which influences the initial settlement;
(b) The breakage of particles into equal parts, which influences the long-term stability and safety of rail tracks; and
(c) The grinding-off of small-scale asperities, where the presence of fines can adversely affect the drainage conditions.

Since ballast particles are primarily angular, most breakage is derived from the corner degradation and attrition, although splitting is also observed. The factors governing particle degradation is described in the following section.

3 FACTORS AFFECTING BALLAST DEGRADATION

3.1 Particle Size Distribution

The gradation of ballast significantly controls ballasted track performance, thus, should provide adequate shear strength and the necessary porosity to allow proper run-off groundwater. To assess the effects of particle size distribution on deformation and degradation behaviour of ballast, large-scale cyclic triaxial tests were conducted on four different distributions of basalt (Indraratna et al. 2003a). The gradation and void ratio characteristics of the test specimens are shown in Figure 3. An effective confining pressure of approximately 45 kPa was used. To simulate the train axle loads running at relatively high speed, cyclic loading with a maximum deviator stress of 300 kPa was applied on the ballast specimens at a frequency of 20 Hz.

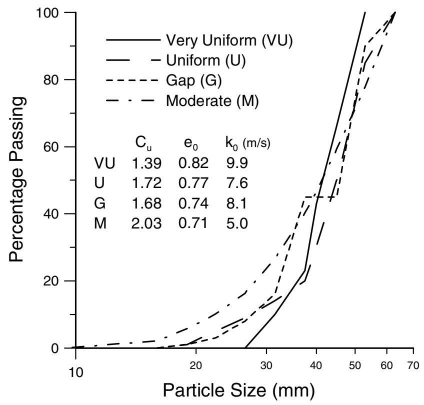

Fig. 3 Particle size distributions used in the triaxial tests, along with their uniformity coefficients, C_u, initial void ratios, e_0, and permeability coefficients, k_0 (Indraratna et al., 2004)

Figure 4 illustrates the relationship between the uniformity coefficient, C_u, and particle breakage. Ballast breakage decreases when the value of C_u increases, with the exception of the gap-graded specimen. The gap-graded ballast excluded particle sizes, which were found to be highly vulnerable to breakage. Therefore, the gap-graded specimen shows a smaller amount of breakage than the uniform and very uniform gradations.

The cyclic test results of ballast varying the gradation indicate that even a modest change in C_u significantly affects the deformation and breakage behaviour of ballast. The test results suggest that a distribution similar to the moderate grading would give improved track performance. Indraratna et al. (2004) recommended that a ballast gradation with a uniformity coefficient exceeding 2.2, but not more than 2.6, is most appropriate. This recommended gradation, which is relatively more well-graded than the current Australian Standard (AS 2785.7, 1996), is presented in Figure 5.

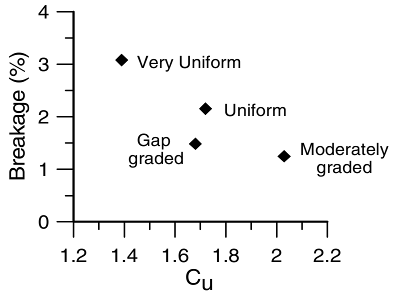

Fig. 4 Effect of grading on particle breakage (Indraratna et al., 2004)

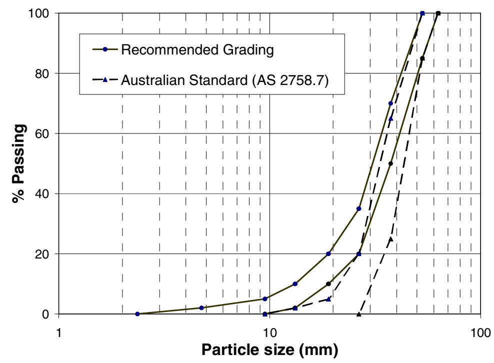

Fig. 5 Recommended railway ballast grading in comparison with the current Australian Standard grading requirements (Indraratna et al., 2003b)

3.2 Effect of Confining Pressure

The confining pressure acting on ballast layer has not often been considered as a significant factor in conventional rail track design. This is because the confining pressure applied on tracks by the shoulder ballast and sleepers is small in comparison with the relatively high vertical stress. The role of confining pressure on ballast performance under cyclic loading has been investigated by Indraratna et al. (2004; 2005a) to evaluate whether there is an optimum confining pressure in the track to reduce the amount of ballast breakage.

Triaxial testing on ballast has shown that the initial particle size distribution shifts towards smaller particle sizes, with the maximum size unaffected before and after loading (Figure 6). An arbitrary boundary of maximum breakage is governed by the availability of the smallest sieve size (2.36 mm in this case) and signifies a practical upper limit for ballast breakage, extending from d_{95} of the maximum sieve aperture d_{max} to the smallest sieve aperture d_{min}. The Ballast Breakage Index (BBI) is proposed as follows (Indraratna et al., 2005a):

$$BBI=A/(A+B) \tag{4}$$

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

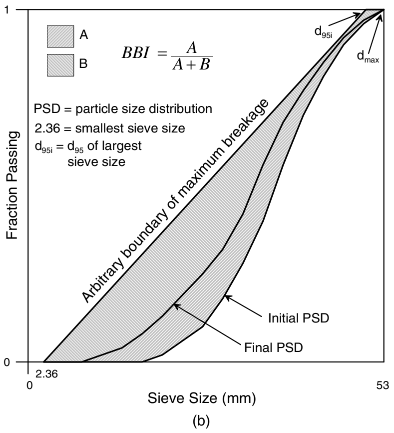

Fig. 6 Evaluation of the Ballast Breakage Index *BBI* (Indraratna et al., 2005a)

The effect of confining pressure on particle degradation using the *BBI* is shown in Figure 7. The sample breakage has been divided into three regions namely: (I) dilatant unstable, (II) optimum and (III) compressive stable degradation zones (Indraratna et al., 2005a). At low confining pressure of region (I) where $\sigma_3' < 30$ kPa, ballast specimens are subjected to rapid and considerable axial and expansive radial strains. This leads to an overall volumetric increase or dilation. In this region, particles do not have sufficient time to rearrange, and due to the excessive axial and radial strains, considerable degradation occurs via shearing and attrition of angular projections. Because of the small confining pressures applied here, specimens in this degradation zone are characterized by a limited co-ordination number as well as relatively small particle-to-particle contact areas.

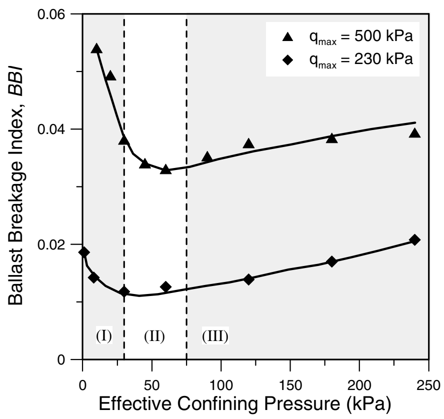

Fig. 7 Effect of confining pressure on particle degradation (Indraratna et al., 2005a)

As the confining pressure is increased to the middle (optimum) region (σ_3' = 30-75 kPa), axial strain rate is greatly reduced due to increased apparent stiffness. It is noted that the overall volumetric behaviour is slightly compressive. In this region, particles are held together in an optimum array with sufficient lateral confinement so as to provide an optimum contact stress distribution and increased inter-particle contact areas. This leads to the reduction of the risk of breakage associated with stress concentrations. As σ_3' is increased further to the compressive stable region ($\sigma_3' > 75$ kPa), particles are forced to move against each other within a limited space for sliding and rolling. Therefore, breakage is significantly increased. In this region, particles fail not only at the beginning of loading when the axial strain rates are the greatest, but also by the process of fatigue as the number of cycles increases.

4 CONSTITUTIVE MODELLING OF BALLAST BREAKAGE

In an attempt to correlate the degree of particle breakage with the strength parameters, Indraratna et al. (1998) formulated some empirical relationships between the peak principal stress ratio, particle breakage and peak friction angle. These empirical relationships have limited applications only at the failure states of axisymmetric specimens. Indraratna and Salim (2002) developed an analytical model to include the relationship of the deviator stress ratio (q/p'), rate of dilation ($d\varepsilon_v/d\varepsilon_1$), corrected friction angle to include dilatancy (ϕ_f), and the rate of energy consumption due to particle breakage. Based on the proposed model, the deviator stress ratio becomes:

$$\frac{q}{p'} = \left[\frac{3\left(1 - d\varepsilon_v/d\varepsilon_1\right)\tan^2\left(45^\circ + \phi_f/2\right) - 3}{\left[2 + \left(1 - d\varepsilon_v/d\varepsilon_1\right)\tan^2\left(45^\circ + \phi_f/2\right)\right]} + \frac{3 f\left(dB_g / d\varepsilon_1\right)\left(1 + \sin\phi_f\right)}{p\left[2 + \left(1 - d\varepsilon_v/d\varepsilon_1\right)\tan^2\left(45^\circ + \phi_f/2\right)\right]} \right] \tag{5}$$

where, $d\varepsilon_1$ and $d\varepsilon_v$ are the increments of major principal strain and volumetric strain, respectively. The parameter p' is the mean effective stress, q is the deviator stress and dB_g is the increment of breakage index associated with $d\varepsilon_1$. The function $f(dB_g/d\varepsilon_1)$ in Equation (5) remains to be determined based on laboratory triaxial testing. Equation (5) was employed along with drained triaxial test results of fresh ballast. Since this formulation employs the stress invariants, which include all normal and shear stresses in an element, it can be applied to a conventional triaxial ($\sigma_2 = \sigma_3$) and to a true triaxial state. A constitutive model that can predict particle breakage under cyclic loads in an assembly of irregular shapes including the effects of anisotropy is currently being developed at University of Wollongong.

Figure 8 shows the stress-strain and volume change predictions of latite basalt employing the developed constitutive model during drained shearing compared with the experimental data. The model predictions without any particle breakage are also shown for comparison. Excellent agreement is found between the test data and model predictions. Figure 8 demonstrates that the model predictions with particle breakage are closer to the experimental data. Since the laboratory confining pressures (50-300 kPa) were small compared to the crushing strength of aggregates (130 MPa, Indraratna et al. 1998), a small fraction of the imparted energy was consumed in particle breakage. Therefore, only a small difference is evident between the model predictions with and without particle breakage. As seen in Figure 8, the gap between the predicted curves with and without breakage increases under greater confining pressure (e.g. σ_3 = 300 kPa) when particle breakage becomes increasingly more significant.

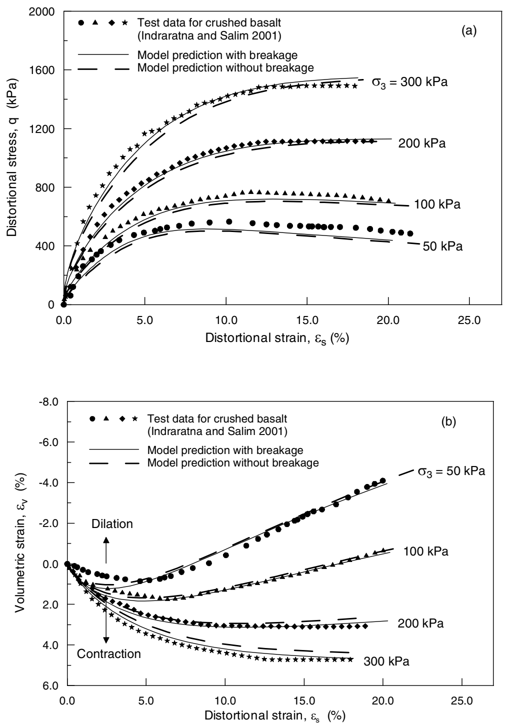

Fig. 8 Predictions of developed constitutive model compared with experimental results of drained triaxial shearing (Salim and Indraratna, 2004)

5 BALLASTED RAIL TRACK IMPROVEMENT: FROM THEORY TO PRACTICE

This section elucidates the need for improving rail tracks that are more resistant to future fast trains. The use of geogrids to stabilize the ballast bed, and the increased confining pressure on tracks to minimize lateral strains are some aspects that are imperative to consider through sound research evidence.

5.1 Improvement of Recycled Ballast Using Geosynthetics

The deformation and degradation behaviour of fresh and recycled ballast was investigated in a large-scale cubical triaxial chamber (Figure 9). The stabilization aspects of recycled ballast using various types of geosynthetics were also studied in the model tests (Indraratna et al. 2003a). Three types of geosynthetics were used including woven geotextiles, geogrids and geocomposites (bonded geogrids and non-woven geotextiles). The testing procedures together with complete findings and discussions have been reported by Indraratna et al. (2004).

Fig. 9 Cubical triaxial apparatus with dynamic actuator designed at University of Wollongong

Figure 10 represents an example of the test findings which shows the settlement of fresh ballast without any reinforcement, and recycled ballast with and without the inclusion of geosynthetics. The test results reveal that wet recycled ballast (without any geosynthetic inclusion) generates significant settlement, because, water acts as a lubricant thereby reducing the frictional resistance and promoting particle slippage. The combination of reinforcement by the geogrid and the filtration and separation functions provided by the non-woven geotextile component (of the geocomposite) reduces the lateral spreading and fouling of ballast.

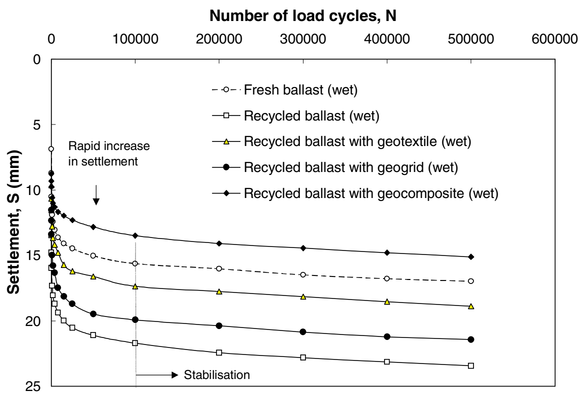

Fig. 10 Settlement of wet fresh and recycled ballast with and without geosynthetics (Indraratna et al., 2004)

In order to obtain the optimum location of geosynthetics for improving the deformation characteristics of recycled ballast, a finite element analysis (PLAXIS) was used (Indraratna et al., 2005c). The large-scale cubical triaxial apparatus shown in Figure 9 was numerically discretised using the mesh depicted in Figure 11. Due to symmetry, only one half of rig was considered in the numerical model. Full details of the finite element analysis conducted can be found in Indraratna et al. (2005c). The placement of geosynthetics beneath the sleeper was initially made at 300 mm depth (i.e. at the ballast capping interface) and then decreased at intervals of 50 mm so that the placement of geosynthetics could be examined at 250, 200, 150 and 100 mm, respectively. The results are plotted in Figure 12, which demonstrate that there is a threshold depth (between 150 to 200 mm) below which the geosynthetics do not contribute any further, but in fact, provides less assistance to settlement reduction. According to Figure 12, the optimum location of geosynthetics for improving the deformation characteristics of recycled ballast may be taken as 200 mm. Nevertheless, for a conventional ballast thickness of 300 mm, placement of geosynthetics at the optimum location (i.e. at 200 mm) may not be feasible for maintenance reasons as mentioned earlier. Consequently, in such cases, the layer of geosynthetics may still be located conveniently at the bottom of the ballast bed (i.e. ballast/capping interface).

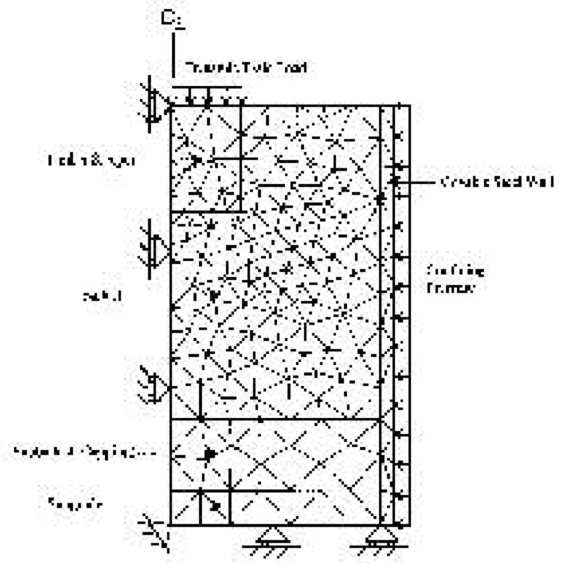

Fig. 11 Finite element mesh used in PLAXIS for the cubical triaxial apparatus (Indraratna et al., 2005c)

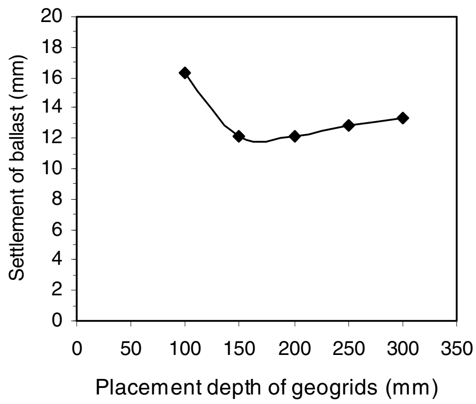

Fig. 12 Optimum location of geosynthetics by the finite elements (Indraratna et al., 2005c)

5.2 Design of Sleeper Shape

As discussed earlier, the confining pressure plays a very important role in controlling rail track deformation in both the vertical and lateral directions (Indraratna et al., 2005a). The lateral force produced by continuously welded rail on a curved track depends on the train speed, axle loads, temperature changes, curvature of the track (degree of the curve) and slope of the track. The buckling of the track is usually due to the build up of stress in the welded rail as a result of high temperature change and insufficient lateral stability (confinement) to support the track.

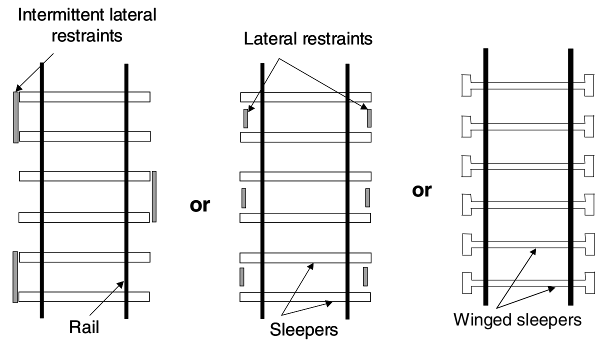

Fig. 13 Increasing confining pressure of tracks using intermittent lateral restraints (Indraratna et al., 2005b)

Some measures for increasing track confinement include (Indraratna et al., 2005b):

- Reducing sleeper spacing;
- Increasing height of shoulder ballast;
- Inclusion of a geosynthetic layer at the ballast-subballast layer interface; and
- Altering the shape of sleepers at both ends or using intermittent lateral restraints at various parts of the track (Figure 13).

6 STABILISATION OF SOFT SUBGRADE FORMATION UNDERLYING RAIL TRACKS

In the coastal areas, the rail tracks are constructed on embankments overlying soft and compressible formation soils. The passage of heavy haul trains with considerable imposed train loads over these deposits causes excessive track settlement and significant reduction in the load bearing capacity of the track. This requires an investigation into the effects of various track parameters including track substructure thickness and stiffness, shoulder width and track modulus on the overall track performance. Such an investigation is very useful for railway geotechnical engineers to arrive at optimum track design and maintenance. Shahin and Indraratna (2006) used a plane strain finite element model (PLAXIS) of railway track section (Figure 14) for this purpose, and concluded that subgrade stiffness has the most significant impact on overall track response. It was shown that when subgrade stiffness decreases, the sleeper deflection increases dramatically, which indicates that maintenance would be a crucial issue for tracks on soft formations. These results point out the importance of investigating methods for increasing stiffness of soft subgrade using different stabilization techniques, which will be described and discussed below.

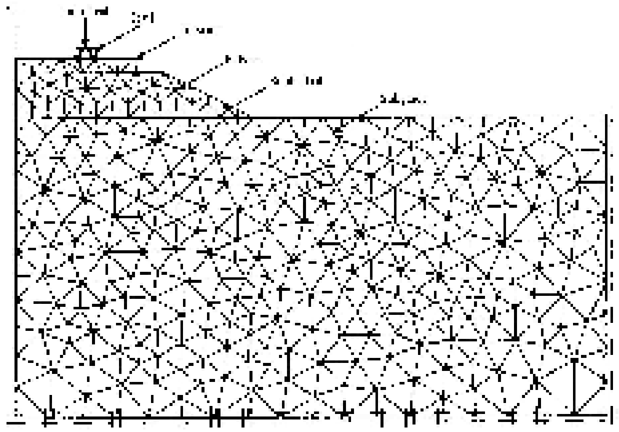

Fig 14 Finite element configurations used in PLAXIS for rail track section (Shahin and Indraratna, 2006)

6.1 Prefabricated Vertical Drains

The effectiveness of prefabricated vertical drains (PVDs) for dissipating cyclic pore water pressures is discussed here. A large-scale triaxial test was used to examine the effect of cyclic loads on the radial drainage and consolidation by PVDs (Indraratna et al., 2006a), as shown in Figure 15. The excess pore water pressure was measured via saturated miniature pore pressure transducers, fitted through the base of the cell to the desired sample locations. Reconstituted estuarine clay was lightly compacted to a unit weight of about 17 to 17.5 kN/m^3 under k_o conditions (0.6-0.7). Undrained shear strengths of softest estuarine deposits is less than 8 kPa. The tests were conducted at frequencies of 5-10 Hz. Figure 15 shows that the maximum excess pore water pressure beside the PVD during the application of cyclic load (T4) is significantly less compared to that near the cell boundary (T3). Also, the excess pore pressures close to the outer cell boundary (e.g. T1 and T3) dissipated at a slower rate than T4 and T2 closer to the PVD. The test results confirm that PVDs decrease the maximum excess pore pressure effectively under cyclic loading.

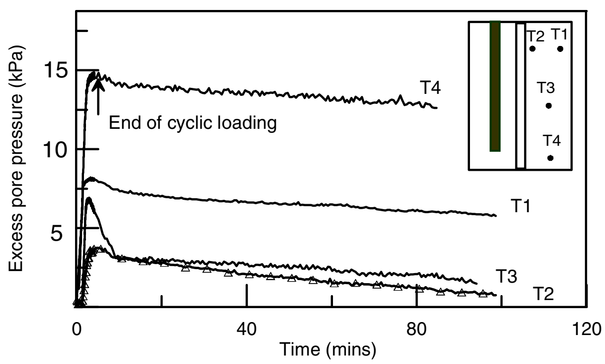

Fig. 15 Dissipation of excess pore pressure at various locations from the PVD (Indraratna et al., 2006a)

Consolidation time may be too long and often uneconomical, especially to consolidate deep estuarine soft clays (up to 30-40m). Under railway tracks, where the significant proportion of the applied load is usually propagated within the first several meters of the formation, if sufficient ballast and subballast depths are provided. In this regard, it is unnecessary for improving the entire layer of soft clay deposit, therefore, relatively short PVDs may still be suitable in design. Short PVDs (5-8m) may still dissipate the cyclic pore pressures, curtail the lateral movements and increase the shear strength and bearing capacity of the soft formation. This will also provide "stiffened" foundation up to several meters in depth, supporting the rail track within the predominant influence zone of vertical stress distribution. In order to investigate the effectiveness of short PVDs in stabilizing track soft formations, Indraratna et al. (2006a) carried out a finite element analysis based on 2D plain strain model proposed by Indraratna and Redana (2000).

In the analysis, repeated train loading is assumed as a static load corrected by an impact load factor. The value of impact load factor depends on the field conditions simulated on track (Esveld, 2001). In the following example, a static load of 70kPa with an impact factor of 1.2 is applied. A typical cross-section of the formation beneath the rail track is shown in Figure 16, where a relatively shallow very soft clay deposit is underlained by a deeper soft soil layer of slightly higher stiffness. PVDs are only used to stabilize the shallow soil layer immediately beneath the track. Table 1 shows the soil properties used in the analysis. The top compacted soil crust (1m in thickness) and the ballast layer (500mm thick) are modelled by Mohr-Coulomb theory. The two layers of soft normally consolidated clays are modelled using the modified Cam-clay theory (Roscoe and Burland, 1968). Total unit weight of artificially compacted granular fills is assumed to be around 16.5-17.0 kN/m^3 with a deformation modulus not more than 200 MPa. The saturated unit weight of the soft clay layers is assumed to be 15.5-16.0 kN/m^3.

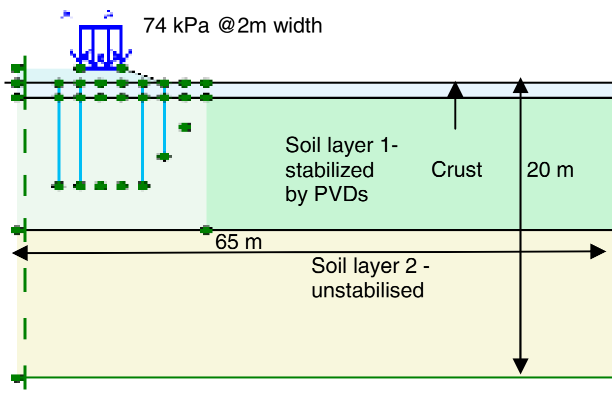

Fig. 16 Vertical cross section of track and formation (Indraratna et al., 2006a)

It is clearly shown that PVDs speed up the excess pore pressure dissipation. More than 65% excess pore pressure dissipates within first 4-5 months (Figure 17). More significant is the considerable reduction of lateral displacement in the PVD stabilised soil underlying the compacted ballast (Figure 18). While long-term lateral displacements at shallow depths (@ 1m) could be as large as 100-150mm, the PVDs are shown to decrease the lateral displacement by about 25%. This numerical example demonstrates the function of short PVDs installed beneath rail tracks.

Table 1. Soil properties used in the FEM

Depth of layer (m)	Model	c (kPa)	ϕ	$\lambda/(1+e_0)$	$\kappa/(1+e_0)$
+0.5	M-C	5	45	–	–
0-1	M-C	29	29	–	–
1-10	S-S	10	25	0.30	0.03
10-30	S-S	15	20	0.12	0.02

Note: M-C= Mohr-Coulomb, S-S= Soft Soil

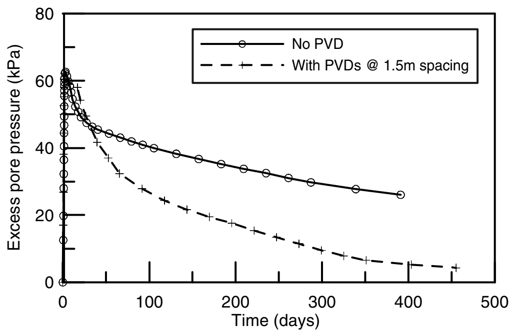

Fig. 17 Excess pore pressure dissipation at 2 m depth at centre line of loading strip

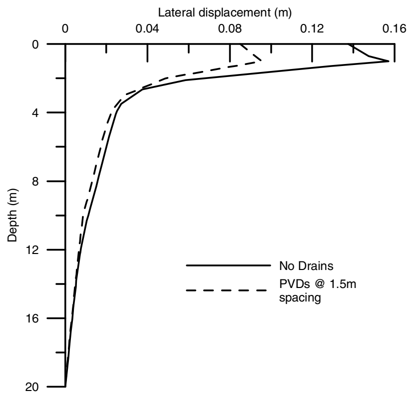

Fig. 18 Lateral displacement profiles near the embankment toe

6.2 Native Vegetation

The tree roots provide an effective form of natural soil reinforcement apart from dissipating the excess pore water pressure, and generate sufficient matric suction to increase the shear strength of the surrounding soil. In Australia, various forms of native vegetation grow along rail corridors. In order to quantify pore pressure dissipation and induced matric suction generated by transpiration, Indraratna et al. (2006b) carried out a finite element analysis using ABAQUS. A two-dimensional plane strain mesh employing 4-node bilinear displacement and pore pressure elements (CPE4P) was considered. The mesh and element geometry, and boundary conditions are shown in Figure 19. More details about the model can be found in Indraratna et al. (2006b).

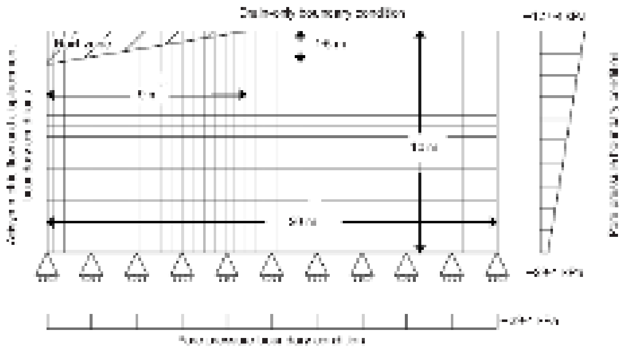

Fig 19 FE configurations used in ABAQUS for soil stabilization by native vegetation (Indraratna et al., 2006b)

Figure 20 shows the ground suction related settlement obtained from the finite element analysis at various depths, and Figure 21 depicts the maximum change in the soil matric suction. It can be seen from Figure 20 that on the surface (z = 0 m), the predicted settlement beside the tree trunk decreases by almost 4 times, at a distance 10 m away from the trunk. The location of the maximum settlement is closer to the trunk at shallower depths, which tends to coincide with the points of maximum change in suction obtained from the analysis (Figure 21). This indicates that native biostabilisation improves the shear strength of soil by increasing the matric suction, thus, decreasing the soil movements. In this regard, native vegetation generating soil suction is comparable to the role of PVDs with vacuum pressure, in terms of improved drainage (pore water pressure dissipation) and associated increase in shear strength.

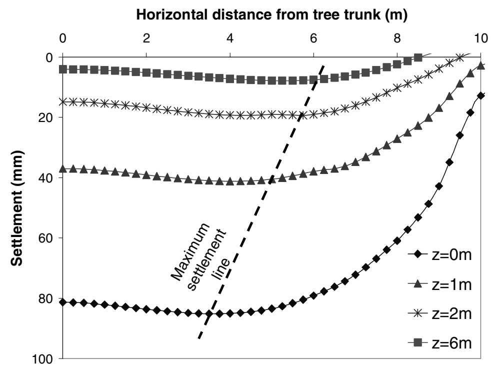

Fig 20 Ground suction related settlement at various depths (Indraratna et al., 2006b)

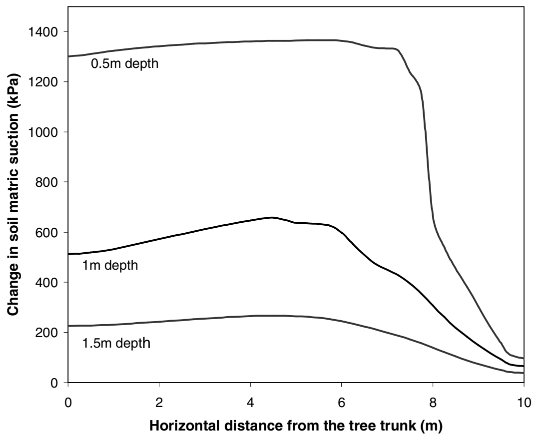

Fig 21 Predicted soil matric suction at various depths (Indraratna et al., 2006b)

7 CONCLUSIONS

The engineering mechanics of ballast and the stabilization of ballasted tracks with geosynthetics, and the consolidation-based improvement of soft formation clays with prefabricated vertical drains and native vegetation have been described through laboratory and numerical simulations. The results show that ballast particle size distribution and confining pressure have a significant influence on ballast degradation. The uniformly graded distribution of ballast is the most prone to breakage and ballast breakage is minimal at confining pressures ranges from 30-75 kPa. The deformations of fresh and recycled ballast vary with the number of load cycles and the extent of degradation and settlement in fresh and recycled ballast reduces with the insertion of geosynthetics. The use of bonded geosynthetics also prevents clay pumping into ballast voids under cyclic loads, providing an additional drainage facility. This indicates the potential use of geosynthetics for enhancing ballast characteristics.

The results also show that short prefabricated vertical drains (PVDs) can be used under rail tracks to dissipate cyclic excess pore pressure and curtail lateral displacements to improve stability, and native vegetation can also be used close to the rail track to reduce settlement and lateral movements.

ACKNOWLEDGEMENTS

The authors wish to thank the CRC for Railway Engineering and Technologies (Australia) for its continuous support. A number of current and past doctoral students, namely, Dr. D. Ionescu, Dr. J. Lackenby, Dr. W. Salim, Mr Anass Attya and Mr Behzad Fatahi have participated to the contents of this keynote paper and their contribution is greatly acknowledged.

REFERENCES

Australian Standards 2758.7 (1996). Aggregates and rock for engineering purposes. *Part 7: Railway ballast. Standards Australia*, NSW, Australia.

Charles J.A. and Watts K.S. (1980) The influence of confining pressure on the shear strength of compacted rockfill. *Geotechnique*, London, U.K., Vol. 30, N0. 4, pp. 353-367.

Esveld, C. (2001) *Modern railway track. MRT-Productions*, Zaltbommel, 653p.

Indraratna, B. and Redana, I. W. (2000). Numerical modelling of vertical drains with smear and well resistance installed in soft clay. *Canadian Geotechnical Journal*, Vol. 37, pp. 132-145.

Indraratna, B. and Salim, W. (2002). Modelling of particle breakage of coarse aggregates incorporating strength and dilatancy. *Geotechnical Engineering*, Proceedings of the Institution of Civil Engineers, London, Vol. 155, No. 4, pp. 243-252.

Indraratna, B., Fatahi, B. and Khabbaz, H. (2006b). Numerical analysis of matric suction effects of tress roots, *Geotechnical Engineering*, Proceedings of the Institution of Civil Engineers, Vol. 159, No. GE2, pp. 77-90.

Indraratna B., Ionescu D. and Christie, D., (1998) Shear behaviour of railway ballast based on large-scale triaxial tests. *Geotechnical and Geoenvironmental Engineering*, ASCE, Vol. 124, No. 5, pp. 439-439.

Indraratna B., Ionescu D. and Christie D. (2000) State-of-the-art large scale testing of ballast. *Proc. CORE2000, Railway Technology for the 21st Century, Conf. on Railway Engineering, 21-23 May, Adelaide, Australia*, pp. 24.1-24.13.

Indraratna B., Khabbaz, H. and Lackenby, J. (2003a) Behaviour of railway ballast under dynamic loads based on large-scale triaxial testing. *Proceedings of AusRAIL Plus 2003, Sydney,* 8p.

Indraratna B., Khabbaz H., Salim W. and Christie D., (2003b) Geotechnical characteristics of railway ballast, and the role of geosynthetics in minimising ballast degradation and track deformation, *Special invited lecture in: RAILTECH 2003, Railway Technology in the New Millennium*, 27-29 May 2003, Kuala Lumpur, Malaysia, pp. 3.1-3.22.

Indraratna, B., Lackenby, J. and Christie, D. (2005a). Effect of confining pressure on the degradation of ballast under cyclic loading. *Geotechnique*, Institution of Civil Engineers, UK, Vol. 55, No. 4, pp. 325-328.

Indraratna, B., Rujikiatkamjiorn, C., Wijeyakulasuriya, V., Shahin, M. A. and Christie, D. (2006a). Soft soil stabilization with special reference to railway embankments. *Keynote Paper: 4th International Conference of Soft Soil Engineering*, Vancouver, Canada, to be held in October.

Indraratna, B., Salim. W. and Khabbaz, H. (2004). Stabilisation of rail tracks employing geosynthetics with recycled ballast. 5 *International Conf. on Ground Improvement*, Kuala Lumpur, Malaysia, pp. 27-40.

Indraratna, B., Shahin, M. A. and Salim, M. W. (2005c). Use of geosynthetics for stabilizing recycled ballast in railway track substructures. *North American Geosynthetics Society (NAGS)– Geosynthetics Institute (GSI) Conference*, Las Vegas, Nevada, Proceedings on CR-ROM.

Indraratna, B., Shahin, M. A., Salim, W. and Christie, D. (2005b). Improvement of granular media and formation soil using geosynthetics with special reference to rail track environment. *Proc. 6th Int. Conf. Ground Improvement Techniques, Coimbra, Portugal,* pp. 37-57.

Indraratna B., Wijewardena L.S.S. and Balasubramaniam A.S. (1993) Large-scale testing of greywacke rockfill, *Geotechnique*, Vol. 43, No. 1, pp. 37-51.

Ionescu, D., Indraratna, B. and Christie, H.D., (1998). Behaviour of railway ballast under dynamic loads. *Thirteenth Southeast Asian Geotechnical Conference*, Taiwan, pp. 69-74.

Jeffs T. and Marich, S. (1987) Ballast characteristics in the laboratory. *Conf. on Railway Engineering*, Perth, Australia, pp. 141-147.

Marachi N.D., Chan C.K. and Seed H.B., (1972). Evaluation of properties of rockfill materials. *Journal of Soil Mech. and Found. Division*. ASCE, Vol. 96, No. 6, pp. 95-114.

Marsal R.J., (1973). Embankment dam engineering - mechanical properties of rockfill, Wiley Publication, New York, pp. 109-200.

Ramamurthy T. (2001). Shear strength response of some geological materials in triaxial compression. *International Journal of Rock Mechanics and Mining Science*s, Vol. 38, pp. 683-697.

Raymond, G.P., (1979). Railroad ballast prescription: state-of-the-art. *Journal of the Geotechnical Engineering Division*, ASCE, Vol: 105, No. GT2, pp. 305-322.

Raymond G.P. and Diyaljee V.A. (1979). Railroad ballast load ranking classification, *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 105, No.10 pp. 1133-1153.

Roscoe, K.H., and Burland, J.B. (1968). On the generalized stress strain behaviour of wet clay. *Engineering plasticity*, Cambridge Univ. Press; Cambridge, U.K., pp. 535-609.

Salim, W. and Indraratna, B. (2004). A New elasto-plastic constitutive model for granular aggregates incorporating particle breakage. *Canadian Geotechnical Journal*, Vol. 41, pp. 657-671.

Selig, E.T. and Waters, J.M., (1994). *Track geotechnology and substructure management*. Thomas Telford, London.

Shahin, M. A. and Indraratna, B. (2006). Parametric study on the resilient response of ballasted railway track substructure using numerical modelling. *ASCE Geocongress 2006*, Atlanta, Proceedings on CR-ROM.

Suiker, A. (1997). Fatigue behaviour of granular materials, Part 1-constitutive modelling and description of experiments. *Tu-Delft Report No. 7-97-119-2*. Delft University of Technology, Netherlands.