Procedia Engineering

Volume 143, 2016, Pages 1007–1015

Advances in Transportation Geotechnics 3 . The 3rd International Conference on Transportation Geotechnics (ICTG 2016)

Assessment of interface shear behaviour of sub-ballast with geosynthetics by large-scale direct shear test

M. Mahdi Biabani[1*], Buddhima Indraratna[2] and Sanjay Nimbalkar[3†]

[1]*Geotehnical Engineer, Coffey Geotechnics; Formerly PhD candidate, Centre for Geomechanics and Railway Engineering, Univ. of Wollongong, Wollongong City, NSW 2522, Australia.*

[2]*Professor of Civil Engineering and Research Director, Centre for Geomechanics and Railway Engineering, Faculty of Engineering, Univ. of Wollongong, Wollongong City, NSW 2522, Australia.*

[3]*Research Fellow, Centre for Geomechanics and Railway Engineering, Faculty of Engineering, Univ. of Wollongong, Wollongong City, NSW 2522, Australia.*

mmb958@uowmail.edu.au, indra@uow.edu.au, sanjayn@uow.edu.au

Abstract

A series of large-scale direct shear test were conducted to study the interface shear strength of subballast reinforced with different types of geomembranes and geogrids. The impact of normal stress (σ_n), shearing rate (S_R), relative density (D_R) and open area (OA%) on the behaviour of granular material was investigated in unreinforced and reinforced condition. The results revealed that the performance of material was markedly influenced by σ_n and OA. The results also showed that geogrids provided a greater value of passive resistance owing to have transverse ribs, but the mobilised passive resistance became smaller with increase in OA. The triaxial grids offered more passive resistance than biaxial geogrid.

Keywords: interface shear strength; subballast; geogrid; passive resistance; direct shear tests

1 Introduction

The railroad industry in Australia is currently undergoing transformation in order to create a competitive edge through imaginative ideas, innovative research leadership and cutting-edge technology. The rail authorities spend hundreds of millions dollars annually to maintain existing tracks. The use of frontier ground improvement technologies (i.e. use of artificial inclusions) is among key priorities for the railways operating in the coastal areas of Australia. The use of artificial inclusions in the form of planar geosynthetic reinforcement is a commonly established practice (Indraratna et al., 2015; 2010). Recent studies have shown that geocell can provide much better lateral

* Created the draft of this document

† Provided comments and feedback

Selection and peer-review under responsibility of the Scientific Programme Committee of ICTG 2016

doi:10.1016/j.proeng.2016.06.094

confinement compared to planar reinforcement. The interface shear strength between the granular material and the geocell strip is one of the most design important factors that need careful attention. Several key parameters, such as low normal stress (σ_n), shearing rate (S_R), relative density (D_R) and percentage of opening (OA%) can affect the interface shear strength. The present study is aimed to fulfil this gap by analysing effects of these important factors on the performance of sub-ballast stabilized with different types of geosynthetics. A series of monotonic drained tests were conducted using large-scale direct shear box apparatus design and built at the University of Wollongong. Both unreinforced and reinforced sub-ballast material were tested at different relative densities and shearing rates and low normal stresses indicative of the in-situ track conditions.

2 Laboratory procedure

Granular materail used in this study was crushed basalt. The particle size distribution adopted for the subballast was within the rail industry specified range (D_{50}=3.3 mm, D_{max}=19 mm, D_{min}=0.075 mm, C_u=16.3, C_c=1.3, γ_d=19 kN/m^3). A predetermined amount of the granular material was laced inside the shear box (300×300 mm) and compacted in several layers to achieve a relative density of about ρ =2100 kg/m^3. Two types of geomembrane and four types of geogrid were selected to investigate the influence of normal stress (σ_n), shearing rate (S_R), relative density (D_R) and percentage of opening (OA%) on the unreinforced and reinforced subballast (Biabani & Indraratna, 2015). Physical and mechanical properties of different types of geosynthetics are provided in **Table 1**.

Table 1: Mechanical and physical characteristics of geosynthetics used for the study.

Geosynthetic type	Geomembrane		Geogrid			
	GC1	GC2	GG1	GG2	GG3	GG4
Material	PE	PE	PP	PP	PP	PP
Structure	Perforated, textured strip	Perforated, textured strip	Triaxial	Biaxial	Biaxial	Biaxial
Mechanical Characteristics						
Tensile strength at 5% strain (kN/m)	7.5	5	11	16.5	17.5	15.5
Ultimate strength (kN/m)(MD/CMD)	9.5[a]/-	6.5[a]/-	19[b]/19[b]	30[b]/30[b]	30[b]/30[b]	30[b]/30[b]
Physical Characteristics						
Open Area (%)	19.19	29.65	65.74	78.9	84.01	81.03
A/D_{50}	3.03	3.03	10.90	11.21	19.54	13.33
Aperture shape	circle	circle	Triangle	Square	Rectangle	Square
Aperture size (mm)	10	10	37	37	63.5×64.5	44
Cell depth (mm)	150	150	—	—	—	—
Thickness (mm)	1.5[c]	1.5[c]	—	—	—	—
Rib thickness (mm) (MD/CMD)	-/-	-/-	2[c]/2[c]	2.2[c]/1.3[c]	2.3[c]/1.3[c]	1.0[c]/1.0[c]

Note: PP: polypropylene, PE: Polyethylene, MD: Machine Direction, CMD: Cross Machine Direction Note: [a](ASTM D4885); [b](ASTM D6637); [c](ASTM D5321).

For the reinforced subballast, two layers of geomembrane having the dimensions of 150×300 mm or one layer of geogrid (300×300 mm) were placed at the interface of upper and lower boxes, along the shearing direction. Two ends of the geosynthetics were clamped at the front edge of the lower shear box using several clamping blocks, and the top half of the shear box was then filled with subballast. All laboratory experiments were conducted in dry condition. Considering railway track environment, only a small confining pressure (hence normal stress) exerted to the ballast shoulder and sleeper ($\sigma_3' \leq 30$ kPa) (Indraratna et al. 2015). Accordingly, a small degree of normal stress applied to the specimen during the testing ($1 \leq \sigma_n \leq 45$ kPa). Different shearing rates ($1 \leq S_R \leq 12$ mm/min) were applied to the specimens to simulate different cyclic stress levels upon train speeds. To obtain optimum relative density of granular material, experiments were carried out at different relative densities ($40\% \leq D_R \leq 85\%$). All specimens were subjected to a maximum horizontal strain (ε_h) of 10%. Shear force, vertical and horizontal displacements were recorded by three mechanical gauges (Biabani and Indraratna 2015).

3 Results and Discussions

3.1. Stress ratio

The laboratory results showed that normal stress had a significant impact on the subballast performance. The magnitude of stress ratio (τ/σ_n) and normal strain (ε_n) are plotted at different horizontal displacements (ΔH) for GG1 shown in **Figure 1** (a & b). Based on the results, higher stress ratios (τ/σ_n) were happened at relatively lower normal stress, which is due higher ratio of apparent friction angle in granular media. The magnitude of τ/σ_n decreased as σ_n increased, which can be justified due to diminishing of dilation. **Figure 1** (c & d) presents the corresponding stress ratio (τ/σ_n) of the unreinforced sample and subballast reinforced with different types of geosynthetics at a normal stress of σ_n=11.50 kPa. The results showed that using the geosynthetics led to improving subballast performance at different magnitudes. Based on the results, geogrid GG1 had the highest impact on the subballast performance in terms of improving its behaviour. This highlights the effectiveness of aperture shape (triaxial ribs) and aperture size with respect to gradation of granular material. Nevertheless, geogrid GG4, did not provide a notable increase in the value of τ/σ_n. Also by utilizing geosynthetics, the magnitude of dilation was decreased, compared to unreinforced specimen. The values of bounding coefficient for different types of geosynthetics at different normal stress are presented in Table 2.

Table 2: Bounding coefficient for different types of geosynthetics (S_R = 1 mm/min and D_R = 77 %)

	Geosynthetic type					
Normal stress	GC1	GC2	GG1	GG2	GG3	GG4
6.7 kPa	1.06	1.12	1.22	1.2	1.03	1.04
11.5 kPa	1.04	1.05	1.22	1.19	1.02	1.03
20.5 kPa	1.08	1.11	1.29	1.21	1.06	1.08
29.5 kPa	1.03	1.09	1.25	1.20	1.02	1.05
45 kPa	1.04	1.09	1.22	1.16	1.04	1.10

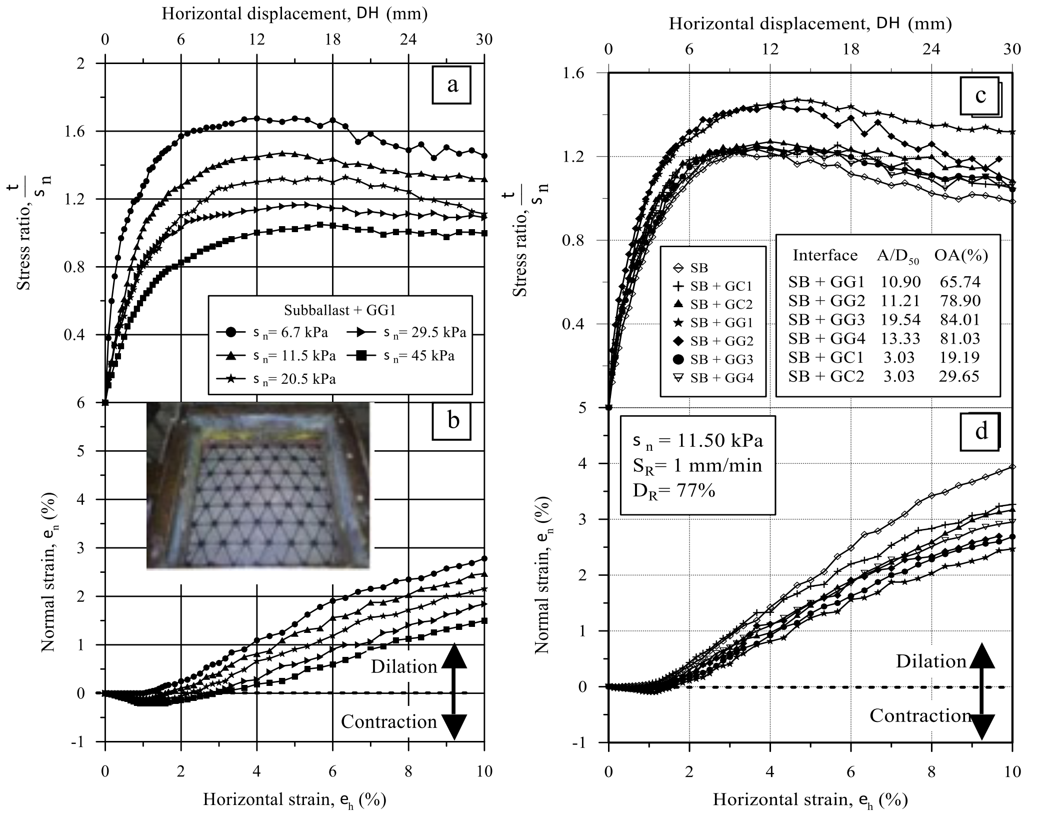

Figure 1: Plots of stress ratios (τ/σ_n) and normal strain (ε_n) of (a&b) GG1 and (c&d) different types of geosynthetic conducted in large-scale direct shear box.

3.2. Plastic work

To highlight the influence of geosynthetic reinforcement on the plastic work and dilation, W_p is plotted against the dilatancy factor, which is defined as $D_p=1-(\delta y / \delta x)_p$ (Rowe, 1962; Indraratna et al., 1998), as shown in Figure 2. By increasing the normal stress, the ratio of dilation was decreased. However, in reinforced subballast, the dilation factor is larger than for unreinforced subballast. This is due to better interlocking induced by the geosynthetic reinforcement. The relationship between plastic work (W_p) and the dilation factor (D_p) for unreinforced and reinforced subballast is nonlinear. Using the hyperbolic fit, the following equation can be derived to measure the dilatancy factor for reinforced subballast with respect to the dissipation of plastic work in large-scale direct shear as (Indraratna et al., 1998):

$$D_p = \frac{1}{c + d/W_p} \tag{1}$$

where D_p is the dilatancy factor, W_p is the plastic work, and c and d are experimental parameters (c=0.2 and d=0.83). It can be seen that the nonlinear curve of the plastic work and dilation factor tended to become asymptotic at about D_p=0.92.

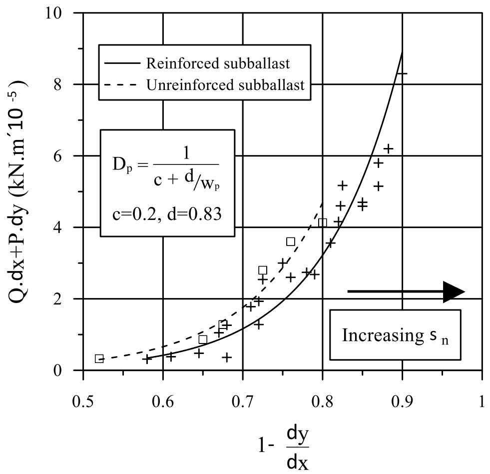

Figure 2: Dissipation of plastic work in large-scale direct shear

Considering the cohessionless of granular material, the normalised shear strength of rockfills can be expressed by (Indraratna et al. 1998):

$$\left(\frac{\tau}{\sigma_c}\right) = \alpha\left(\frac{\sigma_n}{\sigma_c}\right)^{\beta} \tag{2}$$

where, τ/σ_c is the normalised shear strength ratio, σ_n/σ_c is the normalised stress, α and β are empirical parameters, and σ_c is the uniaxial compressive strength of the parent rock. The merit of Eq. (2) is that the shear strength of the subballast can be estimated based on the recommended values of α and β, just by knowing the value of σ_c. The values of α and β are provided in Figure 3. It is evident that β controls the non-linearity or curvature of the envelopes. The maximum (initial) curvature of the shear envelopes is attributed to the dilation behaviour of subballast at very low normal stress. Accordingly, β approaches unity and α approaches the tangent of the interface peak friction angle. Figure 3 shows that all the experimental results of the subballast were within the same range of other rockfill and ballast. This was because the subballast material was sourced from similar parent rock (i.e. basalt).

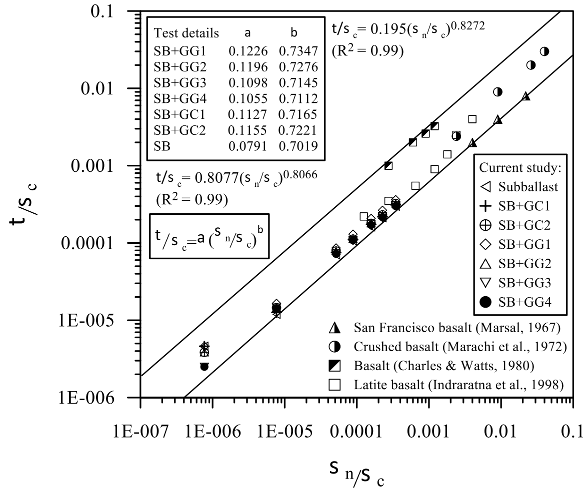

Figure 3: Variation of normalized shear strength vs. normal stress relation (data sourced from Biabani & Indraratna, 2015).

3.3 Friction (ϕ) and Dilatancy angle (ψ)

The laboratory results showed that inclusion of geosynthetics had a remarkable impact on the friction angle and dilatancy angle of subballast. Figure 4 shows variation of peak friction and peak dilatancy angle at different normal stresses. The results confirmed that both dilatancy (ψ) and friction angle (ϕ) were decreased as normal stress (σ_n) was increased. Also Figure 4 shows the rate of reduction in friction angle was lower in reinforced subballast, compared to unreinforced specimen.

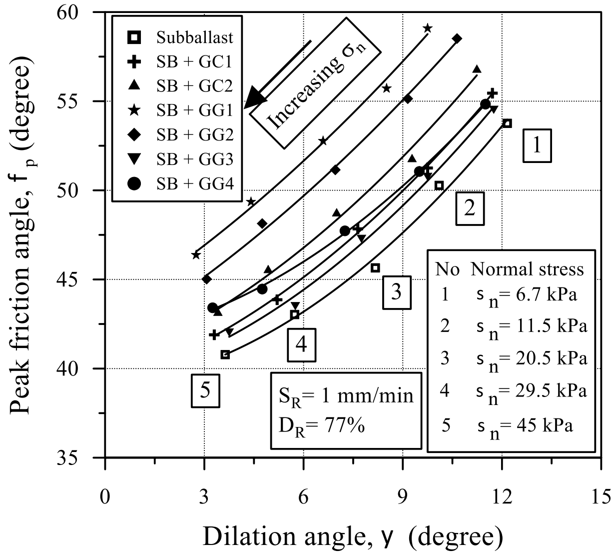

Figure 4: Variation of dilation angle (ψ) against peak friction angle (ϕ_p) (data sourced from Biabani & Indraratna, 2015).

3.4. Shearing rate and relative density

The results revealed that the performance of sub-ballast specimen markedly influenced by relative density (D_R) and shearing rate (S_R). Figure 5 shows variation of interface coefficient in subballast reinforced with GC1 at different shearing rates and relative density at σ_n=20.5 kPa. Based on the laboratory results, the magnitude of interface coefficient was decreased by increasing shearing rates. This is because at higher shearing rate, there will be higher and faster particle rearrangement and densification. Also Figure 5 shows that the diminishing rate of interface coefficient was reduced at higher S_R. Also the laboratory results confirmed that by increasing relative density, reinforced specimens exhibited an improvement in their performance. As Figure 5 shows, marginal improvement was observed at lower density (D_R=40-50%). However, the performance of specimen was substantially improved as D_R was increased. Marginal improvement was observed by increasing D_R from 75% to 85%.

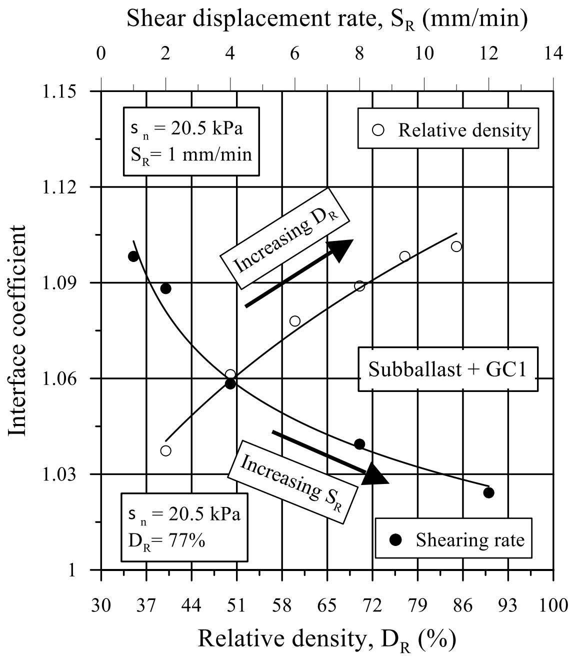

Figure 5: Variation of bounding coefficient at different relative densities and shearing rates (data sourced from Biabani & Indraratna, 2015).

3.5. Frictional and passive resistance

It is well known that shear resistance developed between particles is markedly influenced by (i) frictional resistance between the soil and reinforcement, (τ_{fri}), (ii) passive resistance due to transverse ribs (τ_{pas}) and (iii) internal resistance between the soil particles (τ_{int}) (Bergado et al. 1993; Liu et al. 2009). The outcomes of this study are significant in the view of a safe and economical design of sub-ballast reinforced with different types of geosynthetics. In order to compare the impact of passive and interface resistance in different reinforcement, τ_{fri} can be determined as Bergado et al. (1993) and Liu et al. (2009):

$$\tau_{frictionl} = \sigma_n \times \lfloor (1-OA)\tan\delta + OA \times \tan\phi_{p(u-sb)} \rfloor \tag{3}$$

where δ=interface friction angle of subballast-geosynthetic (degree), σ_n = normal stress (kPa), OA (%) is the open area of the geosynthetic and $\phi_{p(u-sb)}$=peak friction angle of unreinforced subballast obtained from direct shear test (degree). Passive resistance can be obtained by subtracting the frictional resistance (τ_{fri}) and subballast internal resistance (τ_{int}) from the total shear strength (τ_{sb-r}) of reinforced subballast [$\tau_p= \tau_{sb-r} - (\tau_{fri}+\tau_{int})$]. Laboratory tests were performed using procedures given elsewhere (Bergado et al. 1993; Liu et al. 2009). Figure 6 (a & b) shows the variation of frictional and passive resistance for different types of reinforcement at different σ_n. The magnitude of τ_{fri} was increased as normal stress was increased, shown in Figure 6(a). As expected, the value of frictional resistance was markedly decreased as open area (OA%) of reinforcement was increased. Nevertheless, the results showed that the magnitude of τ_p was varied at different types of geosynthetics. As shown in Figure 6(b), GG1followed by GG2 provided the maximum value of τ_p. This can be explained due to effectiveness of transverse ribs in these reinforcements. Also Figure 6(b) shows that GG3 and GC1 provided the minimum value of $\tau_{p.}$ Based on these results, it can be concluded that the optimum OA for the subballast can be at the range of 60-80%. From these results, it can be concluded that maximum interface shear resistance developed in geomembrane GC1, can be effectively, when GC1 utilized in a vertical direction in geocell mattress.

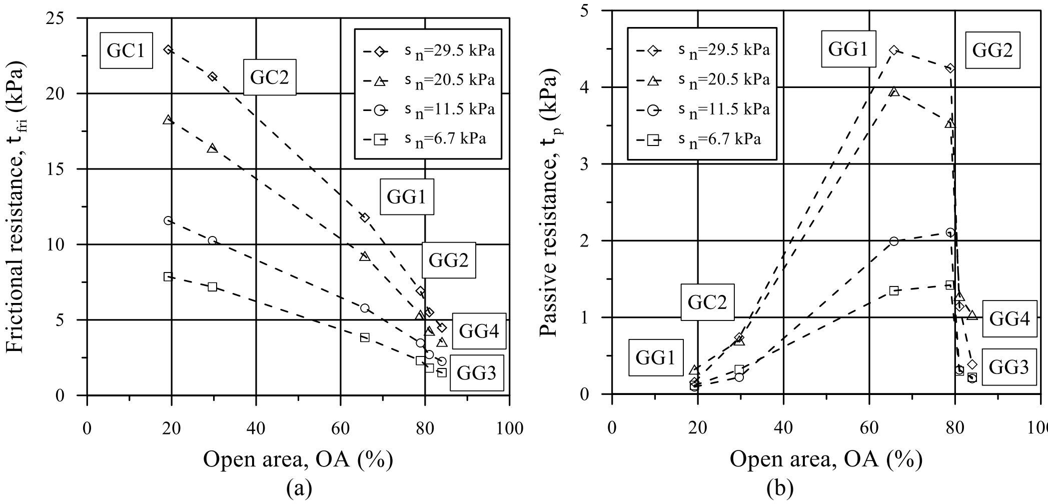

Figure 6: Computation of (a) frictional resistance (τ_{fri}) and (b) passive resistance (τ_p) at different opening area (OA) of different geosynthetics (data sourced from Biabani & Indraratna, 2015).

4. Conclusion

The performance of subballast in unreinforced and reinforced condition was studied using large-scale direct shear testing. The laboratory results confirmed that the behaviour of specimen was significantly influenced by normal stress (σ_n), relative density (D_R), type of geosynthetic and shearing displacement rate (S_R). The results confirmed that specimen behaviour was improved by increasing σ_n. The maximum performance was at the subballast reinforced with GG1, owing to more favourable size of apertures maximising the particle interlock. Also the result showed that GG3 provided minimum improvement. Also the results showed that the specimen performance was improved remarkable as relative density of specimen was increased from 40% to 77%. However, at D_R > 77%, the influence of relative density diminished for both unreinforced and reinforced subballast. On the other hand, interface coefficient was decreased notably as shearing displacement rate was increased from 2 to 12 mm/min. Geogrids provided a greater value of passive resistance compared to geomembrane reinforcement owing to have transverse ribs, but the mobilised passive resistance became smaller with increase in OA%. Considering the opening area, the frictional resistance mobilised against a vertical wall in a geocell mattress made of geomembrane (GC1) is significantly greater than a geocell made by geosynthetics with larger aperture size. Also the result revealed that triaxial grids offered more passive resistance than biaxial geogrid.

References

ASTM Standard D4885. (2011). *Standard Test Method for Determining Performance Strength of Geomembranes by the Wide Strip Tensile Method.* West Conshohocken, PA: ASTM International.

ASTM Standard D5321. (2012). *Standard Test Method for Determining the Shear Strength of Soil-Geosynthetic and Geosynthetic-Geosynthetic Interface by Direct Shear.* West Conshohocken, PA: ASTM International.

ASTM Standard D6637. (2011). *Standard Test Method for Determining Tensile Properties of Geogrids by the Single or Multi-Rib Tensile Method.* West Conshohocken, PA: ASTM International.

Bergado, D. T., Chai, J. C., Abiera, H. O., Alfaro, M. C., & Balasubramaniam, A. S. (1993). Interaction between cohesive-frictional soil and various grid reinforcements. *Geotextiles and Geomembranes , 12* (4), 327-349.

Biabani, M. M., & Indraratna, B. (2015). An evaluation of the interface behavior of rail subballast stabilized with geogrids and geomembranes. *Journal of Geotextiles and Geomembranes , 43* (3), 240-249.

Indraratna, B., Biabani, M. M., & Nimbalkar, S. (2015). Behavior of geocell-reinforced subballast subjected to cyclic loading in plane strain condition. *Journal of Geotechnical and Geoenvironmental Engineering ASCE , 141* (1), 04014081-16.

Indraratna, B., Ionescu, D., & Christie, H. (1998). Shear Behavior of Railway Ballast Based on Large-Scale Triaxial Tests. *Journal of Geotechnical and Geoenvironmental Engineering , 124* (5), 439-449.

Indraratna, B., Nimbalkar, S., Christie, D., Rujikiatkamjorn, C., & Vinod, J. S. (2010). Field assessment of the performance of a ballasted rail track with and without geosynthetics. *Journal of Geotechnical & Geoenvironmental Engineering, ASCE , 136* (7), 907-917.

Liu, C., Zornberg, J., Chen, T., Ho, Y., & Lin, B. (2009). Behavior of Geogrid-Sand Interface in Direct Shear Mode. Journal of Geotechnical and Geoenvironmental Engineering. *135* (12), 1863-1871.

Rowe, P. W. (1962). The Stress-Dilatancy Relation for Static Equilibrium of an Assembly of Particles in Contact. *Proceedings of the Royal Society of London. Series A. Mathematical and Physical Sciences, 269* (1339), 500-527.