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Performance assessment of reinforced ballasted rail track

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Performance assessment of reinforced ballasted rail track

Abstract

In coastal Australia, high population density and increased traffic volumes have promoted rapid expansion of urban transportation infrastructure including railways. Coastal soft clays pose significant construction challenges. Therefore, the search for innovative ground improvement techniques imperative for more resilient and sustainable transport infrastructure has become an obvious priority in research and development. Use of artificial inclusions such as polymeric geosynthetics and energy-absorbing shock mats is described in this paper as a suitable alternative for reducing unacceptable track degradation and for ensuring sustainable track alignment. An extensive monitoring programme was undertaken on fully instrumented track sections constructed near Singleton, New South Wales, Australia. Four types of geosynthetics were installed at the ballast-capping interface of track sections located on different types of subgrades. It was found that geogrids could decrease the vertical settlement of the ballast layer with the obvious benefits of improved track stability and decreased cost of maintenance. It was also found that the effectiveness of reinforcing geogrids is greater when the subgrade is soft.

Keywords

Field testing & monitoring, geotextiles, membranes & geogrids, railway systems

Disciplines

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Performance assessment of reinforced ballasted rail track

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In coastal Australia, high population density and increased traffic volumes have promoted rapid expansion of urban transportation infrastructure including railways. Coastal soft clays pose significant construction challenges. Therefore, the search for innovative ground improvement techniques imperative for more resilient and sustainable transport infrastructure has become an obvious priority in research and development. Use of artificial inclusions such as polymeric geosynthetics and energy-absorbing shock mats is described in this paper as a suitable alternative for reducing unacceptable track degradation and for ensuring sustainable track alignment. An extensive monitoring programme was undertaken on fully instrumented track sections constructed near Singleton, New South Wales, Australia. Four types of geosynthetics were installed at the ballast–capping interface of track sections located on different types of subgrades. It was found that geogrids could decrease the vertical settlement of the ballast layer with the obvious benefits of improved track stability and decreased cost of maintenance. It was also found that the effectiveness of reinforcing geogrids is greater when the subgrade is soft.

Notation

A	shift in particle size distribution (PSD) curve
A_t	axle load (tonne)
a	aperture size of geogrids (mm)
a', b'	empirical constants relating deformation and the logarithm of load cycles
B	potential breakage or area between arbitrary boundary of maximum breakage and final PSD
c', d'	empirical constants relating vertical strain and the logarithm of load cycles
$d_{\max}$	maximum particle size (mm)
$d_{\min}$	minimum particle size (mm)
d_{50}	particle size at percent finer of 50% (mm)
d_{95i}	particle size at percent finer of 90% (mm) of $d_{\max}$
N	number of load cycles
N_a	number of axles per load cycle
S_v	vertical deformation of ballast layer (mm)
S_{tv}	transient vertical deformation of ballast layer (mm)
S_{th}	transient horizontal deformation of ballast layer (mm)
T	total traffic tonnage (tonne)
ε_{gt}	cumulative transverse strain in geogrids or geocomposites (%)
ε_{gl}	cumulative longitudinal strain in geogrids or geocomposites (%)
ε_{th}	transient average horizontal strain of ballast layer (%)
ε_{tv}	transient vertical strain of ballast layer (%)
ε_{tt}	transient transverse strain in geogrids or geocomposites (%)

ε_{tl}	transient longitudinal strain in geogrids or geocomposites (%)
ε_v	vertical strain of ballast layer (%)
σ'	traffic-induced vertical stress (kPa)
σ'_v	vertical stresses

1. Introduction

Ballasted rail tracks serve as one of the major infrastructures for freight and passenger transport in Australia. In recent years, an increasing demand for such transport has led to the use of considerably heavier and faster trains. Large cyclic stresses from train traffic can induce rapid deformations and degradation of the ballast layer (Indraratna *et al.*, 2011b; Lackenby *et al.*, 2007; McDowell and Harireche, 2002). This, in turn, adversely affects track stability and increases the frequency of track maintenance.

It has been observed from several laboratory studies (Chen *et al.*, 2012; Indraratna and Nimbalkar, 2013; Indraratna *et al.*, 2007; McDowell *et al.*, 2006; Rowe and Jones, 2000; Shin *et al.*, 2002) that layers of geosynthetics when placed in the track substructure can increase the stability and therefore the longevity of ballasted rail tracks. Nevertheless, only a few studies have investigated the benefits of geosynthetic reinforcement under ‘field’ conditions. Among these studies, Indraratna *et al.* (2010) performed a field trial on a track, in which layers of geocomposite (combination of biaxial geogrids and non-woven polypropylene geotextiles) were installed at the ballast–capping interface. They reported a reduction of up to 40% in vertical and horizontal strains of the ballast layer for the reinforced track section.

The aperture size of geogrids in relation to the mean particle size of the ballast mainly governs the effectiveness of the reinforcement mechanism. McDowell *et al.* (2006) recommended the ratio between grid aperture size and nominal size of the aggregate to be 1.4, whereas Brown *et al.* (2007) recommended this ratio to be 1.2–1.6 for minimising the settlement of ballast employed in the UK (Railtrack, 2000). In view of rail industry practices in Australia (Standards Australia, 1996), this ratio was proposed to be 1.15–1.3 for optimising the shear strength (Indraratna *et al.*, 2011a). However, there is a lack of sufficient knowledge of how geogrids interact with ballast particles when subjected to the stress and strain environment resulting from moving wheel loads.

A stiff track structure (e.g. concrete bridges and tunnels) can create severe dynamic loading under operating conditions, leading to large-scale failure of components and a subsequent increase in maintenance. Installing resilient mats, such as rubber pads (shock mats) in rail tracks can attenuate the dynamic forces and improve overall performance. A shock mat is called an ‘under ballast mat’ (UBM) when provided below the ballast layer, and an ‘under sleeper pad’ (USP) when provided below the sleeper. Recently, the use of USPs has increased as they reduce dynamic stresses and vibrations in high-speed railway tracks (Esveld, 2009; Ferreira and López-Pita, 2013). The ability of UBMs to reduce structural noise, vibration (Anastasopoulos *et al.*, 2009; Auersch, 2006) and ballast breakage (Indraratna *et al.*, 2012; Nimbalkar *et al.*, 2012) has been studied extensively. Minimising ballast degradation is imperative to sustain its primary functions and the overall working of the substructure.

The field performance of different geosynthetics used as reinforcing elements for ballasted tracks, and the use of UBMs for mitigating particle degradation has not been investigated in a systematic manner. The present study was undertaken to fill this gap. Fully instrumented track sections were constructed near the city of Singleton, New South Wales, Australia. Permanent and transient strains of ballast, breakage of particles and variation of vertical stresses in the track substructure were routinely monitored. The details of an experimental programme as well as field monitoring results of this study are presented herein

2. Experimental track sections

2.1 Site layout and subsurface exploration

The experimental track sections were part of the Minimbah Bank Stage 1 Line that extended from Bedford (chainage 224.2 km) to Singleton (235.06 km). The location of the instrumented test track was about 200 km from Sydney, New South Wales, Australia. A subsurface exploration programme indicated that the rail track was located on an extensive medium- to high-strength siltstone outcrop between 224.2 and 229.0 km and on the flood plain of the nearby Hunter River thereafter (RCA, 2008). The flood plain consisted of a layer of alluvial silty clay deposit 7–10 m thick underlain by heterogeneous layers of medium dense

sand and silty clay with a total thickness of 7–9 m. Medium-strength siltstone was found beneath the sand-silty clay layer.

2.2 Track construction

The track substructure consisted of a ballast layer which extended about 300 mm below sleepers. It was underlain by a 150–300 mm thick capping layer. A structural fill layer of 400–600 mm thickness was placed below the capping layer. The thickness of the structural fill was varied depending on the type of subgrade, and the thickness of the capping layer was commensurate with that of the structural fill in order to satisfy the minimum track formation depths as specified by the adopted construction provisions (ARTC, 2007b). Eight experimental sections were included in the track at the time of construction. Figure 1 shows the locations of experimental sections on different parts of the rail track. These experimental sections were located on subgrades, namely (a) the relatively soft general fill and alluvial silty clay deposit (sections 1–4 and A), (b) the intermediate siltstone (sections 5 and C) and (c) the stiff reinforced concrete bridge deck (section B). This was carried out to study the effects of varying the subgrade type on the performance of geosynthetic reinforcement. Figure 2 shows the details of the geosynthetics and UBM placement locations at the trial sections. Single layers of three types of geogrids were installed in sections 1–3 and 5, and a layer of geocomposite was installed at section 4 to study their

Figure 1. Locations of instrumented track sections at Singleton (data sourced from RCA, 2008)

Figure 2. Reinforcement of track substructure with different geosynthetics at (a) sections A to 4, (b) section B, and (c) sections 5 and C

potential benefits for improving the overall track stability. These geosynthetic reinforcing materials were installed at the ballast–capping interface, as in actual practice the depth of this placement (about 300 mm from the base of the sleeper) is adequate to prevent any damage from the tamper tines. For comparison purposes, no geosynthetic was installed at sections A and C. A layer of UBM was installed at the ballast–deck interface at section B (Figure 2(b)) to minimise particle degradation.

2.3 Material specifications

2.3.1 Ballast and capping characteristics

The particle gradations of the ballast and capping were in accordance with the construction provisions (ARTC, 2006a, 2006b, 2007a) and are shown in Figure 3. The particle gradation and USCS classification of these materials (at the time of track commission) are reported in Table 1. The ballast consisted of compacted highly angular latite basalt particles. The underlying capping layer was compacted sandy gravel. The percentage of silt and clay-sized particles (by weight) of the capping was 10%. The ballast and capping were obtained from the same quarry (Allandale, 20 km north-west of Singleton) and comprised similar mineral components.

Figure 3. Particle gradations of fresh ballast and capping used in construction of track

2.3.2 Geosynthetics and UBM characteristics

Four types of geosynthetics, namely three biaxial geogrids and one geocomposite (biaxial geogrid + non-woven geotextile), were used to stabilise the railway ballast. The biaxial geogrids were

Material	Description	$d_{\max}$: mm	$d_{\min}$: mm	d_{50} : mm	USCS classification	California bearing ratio (CBR): %
Fresh ballast	Compacted angular latite basalt	63	19	36	GP	—
Capping	Compacted sandy gravel	26.5	0.008	4	GP-GM	50

Table 1. Grain size characteristics and USCS classification of ballast and capping materials

made of polypropylene, and were generally suitable for soil stabilisation and reinforcing applications because of their high tensile strength and high tensile stiffness. Adding a non-woven geotextile to the biaxial geogrid enabled the geocomposite to provide filtration, separation and partial in-plane drainage (Indraratna *et al.*, 2007; Indraratna and Nimbalkar, 2013). The UBMs (elastic pads or rubber mats) had high impact strength and are typically installed as a protective layer over several bridges in Australia. The UBMs are composed of bonded rubber granulates with typical particle sizes between 1 and 3 mm with polyurethane elastomer binding. Table 2 lists the physical, strength and dimensional characteristics of these materials as employed in this study.

3. Track instrumentation

3.1 Pressure cells

Traffic-induced vertical stresses in the track were monitored by pressure cells that were suitable to measure compressive stresses up to 600 kPa. Two pressure cells were installed at sections 1, 5, A and C (Figure 4(a)). One pressure cell was installed at the sleeper–ballast and another at the ballast–capping interface. At section B, however, three pressure cells were installed at the UBM–deck interface. Two cells were located below the up rail, whereas the other was below the down rail.

3.2 Strain gauges

Strain gauges were used to study longitudinal and transverse strains accumulated in the geogrid layers. The strain gauges were of post-yield type and suitable to measure strains in the range of 0.1 to 15%. They were installed in a group, on the top and bottom sides of grids in both longitudinal and transverse directions (Figure 4(b)).

3.3 Settlement pegs

Settlement pegs were installed to monitor vertical deformation of the ballast layer. The settlement pegs were installed at the sleeper–ballast and ballast–capping interfaces (Figure 4(c)). A simple survey technique was used to track the movements of pegs and deformations of the ballast layer were later determined.

3.4 Displacement monitoring frame

Transient deformations of the ballast layer were measured by electronic potentiometers (POTs) mounted on a custom-made

aluminium frame as shown in Figure 4(d). Two POTs – one monitoring the vertical movement of sleepers another for that of the ballast-capping pegs – determined the transient vertical displacements of the ballast layer. The other three POTs mounted in an inclined fashion monitored both vertical and horizontal deformations of the ballast shoulder at different locations. The deformation frame was used at all experimental sections, except at section B. The frame was held in place by support bases installed in the capping and structural-fill layers (i.e. more than 150 mm depth).

3.5 Data acquisition

Data from the pressure cells, strain gauges and POTs as well as levels of the settlement pegs were obtained immediately after the instruments were installed. Subsequently, the data were obtained daily for three days, weekly for three weeks, monthly for three months and then quarterly for three quarters.

4. Field monitoring results

The ballast deformation and strains in geogrids were measured against time. In order to establish a suitable correlation with other research methodologies, an appropriate scale of ‘number of load cycles’ was selected in lieu of the ‘time’ scale (Indraratna *et al.*, 2010). The number of load cycles (N) is determined from the total traffic tonnage (T), axle load (A_t) and number of axles per load cycle (N_a) (Selig and Waters, 1994) as given below

$$1. \quad N = \frac{T}{A_t \times N_a}$$

Delaney (2011) reported a total traffic tonnage of 64 million gross tonnes (MGT) on the third track section during the period of measurement. The majority of traffic was imparted from coal trains having four axles, and axle loads between 25 and 30 t. This resulted in 3.3×10^5 load cycles. The results were plotted against the number of load cycles as discussed below.

4.1 Vertical deformations of ballast

The deformation of ballast was determined by subtracting the vertical displacement of the ballast–capping interface from that at the sleeper–ballast interface. The mean vertical strain is defined as the ratio of ballast deformation to the initial ballast thickness. Vertical deformation (S_v) and strains (ϵ_v) of the ballast

(a) Geogrids and geocomposite

	Geogrid 1	Geogrid 2	Geogrid 3	Geocomposite	
Physical characteristics					
Material	Polypropylene			Polypropylene	
Type	Biaxial	Biaxial	Biaxial	(Grid) biaxial	(Fabric) non-woven
Technical characteristics					
Tensile stiffness ^a : MN/m	1.8/1.8	1.5/1.5	1.5/1.5	2.0/2.0	0.3/0.5
Tensile strength ^a : kN/m	36/36	30/30	30/30	40/40	6/10
Strain at break ^a : %	15/15	15/15	15/15	15/15	60/40
Dimensional characteristics					
Aperture size ^a : mm	44/44	65/65	40/40	31/31	—
Thickness: mm	3	3	4	3	2.9
Specific mass: g/m ²	—	—	—	—	150

(b) Shock mat (UBM)

Physical characteristics

Material	Polyurethane elastomer
Type	Bonded rubber granulates
Particle size: mm	1–3
Technical characteristics	
Tensile strength: kN/m ²	600
Strain at break: %	80
Dimensional characteristics	
Thickness: mm	10

^a Values are given as MD/CMD, where MD is the machine direction (longitudinal to the roll) and CMD is the cross-machine direction (across roll width).

Table 2. Properties of (a) geogrids and geocomposite and (b) shock mat (UBM)

layer were plotted against the number of load cycles (N) as shown in Figure 5(a) and (b). The vertical deformation of the ballast was highly non-linear under cyclic loading, as also shown in previous studies (Indraratna *et al.*, 2007, 2010; Indraratna and Nimbalkar, 2013; Jeffs and Marich, 1987; Lackenby *et al.*, 2007). A rapid increase in vertical deformation was observed during the first 200 000 load cycles, beyond which the deformation showed only a marginal increase. The zone during which the rate of deformation diminished to a controlled steady state after a certain level of load repetitions had passed, is defined as the ‘stable’ zone (Figure 5(a) and (b)). When the results for sections on similar subgrades were compared, the vertical deformation of ballast with geosynthetics was 10–32% smaller than that without reinforcement

(Figure 5(a)). This trend was observed also in the laboratory (Brown *et al.*, 2007), and is mainly attributed to the interlocking between ballast particles and the grid apertures, thus creating an enhanced track confinement.

When the results for sections with similar geogrids (i.e. section 3 and section 5) were compared, it was observed that the effectiveness of a geogrid to reduce track settlement became higher for softer subgrades (Figure 5(a) and (b)). This observation is in agreement with the results of full-scale tests presented by Ashmawy and Bourdeau (1995). It was also observed that among the four types of geosynthetic used, the geogrid at section 3 performed most effectively. Although the tensile strength of this

Figure 4. Details of track instrumentation using (a) pressure cells, (b) strain gauges, (c) settlement pegs and (d) potentiometers (POTs) mounted on deformation frame

Figure 5. Vertical deformations and strains of ballast layer plotted against number of load cycles for (a) soft embankment and (b) hard rock

geogrid was not more than that of the others, its aperture size (a) of 40 mm would enable better interlocking between the ballast and grid. This finding also agrees well with the criteria for optimum aperture sizes for reinforcing geogrids proposed in earlier studies (Brown *et al.*, 2007; Indraratna *et al.*, 2011a).

Usually, the non-linear variation of ballast deformation against the number of load applications is best described by a semi-logarithmic relationship (Indraratna and Salim, 2003) such as

$$2. \quad S_v = a' + b'(\ln N)$$

where a' and b' are two empirical constants, depending on the type of ballast, type of geosynthetics used and the initial placement density. Similarly, the relationship between the vertical strain of ballast and the logarithm of load cycles can be expressed by Indraratna *et al.*, 2011b

$$3. \quad \epsilon_v = c' + d'(\ln N)$$

where c' and d' are two empirical constants. The non-linear variation of ballast deformation and strains with increasing load cycles becomes linear in the semi-logarithmic plot (Figure 6(a) and (b)). The values of empirical constants were obtained by performing a linear regression analysis as shown in Figure 6(a) and (b). It is observed that Equations 2 and 3 fit the vertical deformations and strains of ballast reasonably well for a wide range of numbers of load cycles.

4.2 Transient deformation of ballast

Transient deformation of the ballast layer was measured by the custom-made deformation frame. It was observed that the passage of trains travelling at 40 km/h resulted in vertical deformations (S_{tv}) of the ballast layer in the range of 1.5 to 3.0 mm resulting in average vertical strains (ϵ_{tv}) between 0.5 and 1.0%. The transient horizontal deformation (S_{th}) of ballast was always expansive and in the range -0.5 to -0.3 mm. This resulted in average horizontal strains (ϵ_{th}) of -0.05 to -0.02% . The horizontal

Figure 6. Vertical deformations and strains of ballast layer plotted in semi-logarithmic scale for (a) soft embankment and (b) hard rock

strains were larger near the crest and smaller near the toe of the ballast layer. The transient strains (ϵ_{tv} , ϵ_{th}) of track sections with reinforcement were about 15% smaller than those without reinforcement regardless of the type of geosynthetics employed.

4.3 Strains mobilised in geogrids

Accumulated transverse (ϵ_{gt}) and longitudinal (ϵ_{gl}) strains measured from the strain gauges below the edges of sleepers are plotted in Figure 7(a) and (b). The results indicate that both transverse (ϵ_{gt}) and longitudinal (ϵ_{gl}) strains generally increased with the number of load cycles, suggesting the accumulation of permanent deformation in the geogrids. It should be highlighted that the majority of strains in both directions probably took place during the construction of the experimental sections, particularly when the ballast was placed. The transverse strains were generally larger than the longitudinal strains. This is attributed to the relative ease for lateral spreading of the track substructure caused by smaller track restraints in the transverse direction. As shown in Figure 7(a), the strains of geocomposite (section 4) were relatively large. This is because the thick general fill at this

location underwent large lateral deformations shortly after track commissioning, resulting in excessive transverse strains in the geocomposite.

Induced transient strains in synthetic grids in both longitudinal (ϵ_{tl}) and transverse (ϵ_{tt}) directions due to the passage of trains at a speed of 40 km/h were of the order of 0.14–0.17%. Unlike the case of accumulated strains, smaller values of ϵ_{tl} and ϵ_{tt} were observed in grids with higher values of stiffness.

4.4 Vertical stresses

The vertical stresses (σ_v) resulting due to the passage of trains travelling at a speed of 40 km/h for sections 1 and A were in the range of 170 to 180 kPa and 30 to 35 kPa at the sleeper–ballast and ballast–capping interfaces, respectively. At section B, the vertical stress at the mat–deck interface was about 150 kPa. The vertical stresses for sections 5 and C were in the range of 215 to 230 kPa and 90 to 110 kPa at the sleeper–ballast and ballast–capping interfaces, respectively. As anticipated, the stresses measured at the sleeper–ballast interface were larger than those at the ballast–capping interface. These results indicate that the induced stresses were considerably larger in a track with stiffer subgrade; that is, the stresses were highest at section B (concrete deck) and lowest at sections A and 1 (silty clay deposit).

4.5 Ballast breakage

Digital photographs of the track formation condition prior to excavation were taken together with a photograph of the sampling pit (Figure 8(a)). Samples were recovered from beneath the sleeper and the rail seat. This sampling location was considered to be most appropriate, as vertical stresses are usually the largest beneath the rail seat (Indraratna *et al.*, 2010; Indraratna and Nimbalkar, 2013). Samples were collected from three equal portions of load-bearing ballast in order to assess the variation of ballast breakage with depth as shown in Figure 8(a). The ballast profile was then reinstated using clean ballast and tamped using a tamping head on the excavator. Visual inspection of the samples showed no apparent foreign materials in the ballast. Only rock fragments, which most likely resulted from particle breakage, were found in these ballast samples. This suggested that fouling of the ballast layer due to spillage of coal from passing trains and ‘slurry pumping’ of the fines from the underlying subgrade had not taken place on this section of the relatively new track.

The breakage is quantified using the parameter, ballast breakage index (BBI), proposed by Indraratna *et al.* (2005). By utilising a linear hypothetical size axis as reference, BBI is calculated using Equation 4

$$4. \quad BBI = \frac{A}{A + B}$$

where A is shift in the particle size distribution (PSD) curve due to breakage; and B is the potential breakage or the area between

Figure 7. Strains accumulated in geogrids and geocomposite in (a) transverse and (b) longitudinal directions plotted against number of load cycles

(a)

Figure 8. (a) Photograph showing sampling pit for ballast breakage assessment: A, sleeper; B, ballast; B1, top zone; B2, central zone; B3, bottom zone; C, capping; D, structural-fill. (b) Determination of ballast breakage index (data sourced from Indraratna *et al.* (2005))

the arbitrary boundary of maximum breakage and the final PSD. The parameters A and B are obtained as defined in Figure 8(b). The values of BBI after 7.8×10^5 load cycles are shown in Table 3. As expected, the ballast breakage was highest at the top and decreased with the increase in depth. Although the typical train speeds of sections A and B were practically similar, the smaller

shear or deviator stresses imparted on the ballast due to larger lateral confinement from the barriers of Mudies Creek bridge most likely resulted in a significantly smaller value of BBI at section B. These results may also suggest the effectiveness of UBMs in reducing ballast degradation when placed above the concrete deck. However, sufficient data from a similar bridge without any UBM are necessary for more convincing validation.

5. Conclusions

A comprehensive field monitoring programme was undertaken on instrumented track sections to study the effectiveness of various geosynthetics at improving the overall stability of ballasted rail tracks. Four types of geosynthetics with different geometrical and mechanical properties were installed at the ballast–capping interface. The experimental track sections were located on three types of subgrades. It was found that geogrids and geocomposite could decrease vertical deformations of the ballast by as much as 35%, with obvious benefits of improved track stability and decreased maintenance cost. For the biaxial geogrids and geocomposite employed, the aperture sizes of the geogrids which resulted in least ballast deformations (both long-term and transient) were in the range of $1.1d_{50}$.

The effectiveness of reinforcement increased in the case of softer subgrade. Transient strains of the ballast layer also decreased when geosynthetics were used. The traffic-induced vertical stresses at the ballast–capping interface were larger on stiffer subgrades. The reduced ballast breakage at the concrete bridge was found to be associated with the increased lateral track confinement. The use of shock mat (UBM) also reduces ballast breakage. However, this needs further validation through collection of more field data from a similar bridge without any UBM. The findings of this field study allow for better assessment of the performance of geosynthetic reinforcement and energy-absorbing UBMs to mitigate track degradation caused by cyclic and impact wheel loads. Better understanding of such performance would allow for safer and more effective design and analysis of ballasted rail tracks with geosynthetic reinforcement.

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Serial no. Section details and subgrade conditions

Ballast breakage index (BBI)

Serial no.	Section details and subgrade conditions	Ballast breakage index (BBI)		
		Top zone 100 mm thick	Central zone 100 mm thick	Bottom zone 100 mm thick
1	Section A (relatively soft general fill and alluvial silty clay deposit)	0.17	0.078	0.064
2	Section B (stiff reinforced concrete bridge deck)	0.064	0.031	0.022

Table 3. Assessment of ballast breakage resulted from train traffic during the period of measurements ($N = 7.8 \times 10^5$ load cycles)

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