Use of shock mats for mitigating degradation of railroad ballast

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Abstract
In Australia, increasing demand for High Speed Rail (HSR) and heavier freight transport is a technical and economic challenge for practicing engineers, designers and researchers. Because of this increased train speed and axle load, high undue stresses are transferred to the ballast and underlying formation. Ballast degradation is a major factor affecting track longevity and stability. Use of energy absorbing shock mats to reduce noise and vibrations is an established practice. The shock mat is sometimes called as Under Sleeper Pad (USP) and Under Ballast Mat (UBM) depending upon their placement position. However, studies to analyse their effectiveness in minimising ballast degradation are limited. A series of large-scale laboratory tests were conducted on ballast using a high-capacity drop-weight impact testing equipment to understand the performance of energy absorbing shock mats in the attenuation of impact loads and subsequent mitigation of ballast degradation. A numerical model was developed based on the modified stress-dilatancy approach to capture particle breakage during impact loading. Model predictions are compared with laboratory results. This paper presents state-of-the-art review of laboratory studies and numerical modelling illustrating benefits of USPs and UBMs in the practice.

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USE OF SHOCK MATS FOR MITIGATING DEGRADATION OF RAILROAD BALLAST

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ABSTRACT: In Australia, increasing demand for High Speed Rail (HSR) and heavier freight transport is a technical and economic challenge for practicing engineers, designers and researchers. Because of this increased train speed and axle load, high undue stresses are transferred to the ballast and underlying formation. Ballast degradation is a major factor affecting track longevity and stability. Use of energy absorbing shock mats to reduce noise and vibrations is an established practice. The shock mat is sometimes called as Under Sleeper Pad (USP) and Under Ballast Mat (UBM) depending upon their placement position. However, studies to analyse their effectiveness in minimising ballast degradation are limited. A series of large-scale laboratory tests were conducted on ballast using a high-capacity drop-weight impact testing equipment to understand the performance of energy absorbing shock mats in the attenuation of impact loads and subsequent mitigation of ballast degradation. A numerical model was developed based on the modified stress-dilatancy approach to capture particle breakage during impact loading. Model predictions are compared with laboratory results. This paper presents state-of-the-art review of laboratory studies and numerical modelling illustrating benefits of USPs and UBMs in the practice.

Keywords: Ballast, Impact load, Shock mats, Degradation, Deformation

1. INTRODUCTION

Energy absorbing mats such as Under Sleeper Pad (USP) and Under Ballast Mats (UBM) are resilient pads placed under the sleepers and under the ballast, respectively. The most significant applications of these resilient pads in railways are: 1) reduce the structure-borne vibration and noise to protect nearby structures, and 2) reduce the ballast degradation to improve stability and maintain track geometry, thereby increasing the service life of the rail track. The resilient material used as the USP and UBM to improve the overall vertical elasticity of the track substructure. In recent years, use of elastomeric soft pads underneath concrete sleepers have become increasingly popular and is the primary focus of track research (Marschling and Veit 2011). The elastic pad embedded under the sleeper avoids a hard interface with the ballast, allowing the ballast to bed into the padding material. This increases the contact surface area of the ballast with other interfaces such as, USP (increases the contact area of ballast with sleeper), and UBM (increases the contact area of ballast with sub-ballast or formation soil). Consequently, this avoids excessive contact forces between the interfaces and ballast particles, leading to increased stability, less settlement and reduced wear of the track sub-structure. In the case of USP, it extends the bending length of the rails. Therefore, the axle load from the train is distributed over a larger number of sleepers compare with sleepers without USPs. Since the compression load distributed over large area (Figure 1), it further reduces the force acting on sleeper-ballast interface and inter-ballast particle forces (Bolmsvik 2005; Plášek et al. 2007; Loy 2008; Dahlberg 2010).

The wheel and rail irregularities such as wheel flat, rail corrugation, dapped rail, defective rail weld, insulation joints and rail expansion gap causes higher impact load than the cyclic load exerted by moving wheels (Nielsen and Johansson 2000; Bruni et al. 2009; Nimbalkar et al. 2012). Change of stiffness where the track passages from ballasted track to the bridge approach, track transition locations such as road crossing and change of subgrade condition (weak subgrade to bedrock) is accelerating track degradation due to this high impact loading (Li and Davis 2005; Nimbalkar et al. 2012). Therefore the use of energy absorbing resilient pads in the track structure to attenuate the rail track degradation is becoming increasingly popular in railroad industries (Esveld 2001). This paper presents overview of various methods of analysis on use of shock mats in track structures in recent years. Few preliminary research studies on the assessment of shock mats using large scale impact testing equipment at the University of Wollongong, Australia are also presented.

Figure 1 Distribution of Axle Load.
(a) Without Shock Mats; (b) With Shock Mats
2. LITERATURE SURVEY

2.1 History: Development of shock mats

The track improvements by using shock mats have been in use since 1980s and it is increasingly in use last 10 to 20 years specially in Central Europe (Bolmsvik 2005; Schneider et al. 2011). Initially it was used to reduce vibrations transmitted from the rail track to nearby buildings, then it has been in wide use to reduce the sleeper-ballast contact stresses. Since 2005, in Austria, USPs are used as a standard component in turnouts to improve track quality and reduce rail corrugation growth on small radius curves in category A tracks (curve radius > 250m and traffic load > 30,000 tons/day) (Schneider et al. 2011). Recent studies by Loy (2008) and Marschning and Veit (2011) confirms the use of USP lessening the maintenance requirements and thereby dramatically reducing Life Cycle Cost (LCC) of track structure. Indraratna et al. (2012) found that the use of shock mats reduced up to 50% strain of the ballast layer subjected to impact forces owing to the wheel rail imperfections. Detailed overview of important studies on the use of shock mats in rail track improvement is presented in the following sections.

2.2 Shock Mats for Vibration reduction

In the beginning of 1980s, thin elastic pads as a USP material was used to cover the wooden sleeper to minimize the vibration transmitted to the houses near the rail tracks. Then in 1990s, the French railway started the testing by introducing thin layer of polyurethane as a USP material to minimize the sleeper-ballast contact stresses (Bolmsvik 2005). A study by Auersch (2006) suggest that the ballast mats (i.e. UBM) are an efficient measure to reduce the vibration near the rail tracks. In his study, numerical method of track dynamics using three dimensional and an improved simple two dimensional FEM models were used to analyse ballast track with and without ballot mat. Auersch (2006) reported that the resonance frequency depend on the stiffness of the ballast mat and the insertion of an elastic mat under the ballast layer shifts the vehicle–track resonance frequency between 20 and 50 Hz, thereby considerably improving the reduction of dynamic forces. Loy (2008) reported that the use of USP significantly improve the ballast track vibration behaviour compared with traditional track without USPs, especially the frequencies above 40 Hz. Medium frequency range of 50-150 Hz vibration tend to liquefy the ballast material and become unstable. Therefore the use of USP is a beneficial effect on the stability of ballasted track. A research study by Loy (2012) on mitigating vibration by USP suggest that appropriate USPs can reduce the vibration and also improve the track bed geometry. A sandwich type of USP consist of a soft and acoustically highly-effective elastic layer embedded to the concrete sleeper on one side and a visco-plastic material layer on the ballast side recommended to cater for above two requirements.

2.3 Reduction of Life Cycle Cost

The Austrian mainline network sections data analysed by the Technical University of Graz shows that the installation of padded sleepers significantly reduce the LCC for the track (Marschning and Veit 2011). This can be achieved by three main cost portions (1) prolonged service life by reducing depreciation, (2) higher track availability by reducing obstructions of operational cost and (3) reduced maintenance needs as shown in Figure 2. Therefore, Marschning and Veit (2011) concluded that the use of soft padded track system is a major step towards cost efficient and sustainable ballasted track.

Since the stiffness of the track is reduced by the installation of USPs on concrete sleepers which lessen the corrugation in small-radius tight curves and reduce the higher maintenance cost required at the curves. Soft padded concrete sleepers reduce the ballast wear and extend the intervals between two tamping cycle by at least 2 and thereby increase the service life of the ballast (Marschning and Veit 2011). The comfort of the rail transport also increase by the soft padded sleepers in the track structure.

2.4 Mitigating Ballast Degradation

Ballast is a major load bearing layer in the track bed which also facilitate the water draining easily from top of the track bed to the underlying formation or adjacent native ground. As the speed of the rail and the axle load increases, the ballast material used in the track bed needs considerable maintenance or a way of protect the ballast from high stresses. Limiting the generated stress on ballast is an economical option which many railway agencies and authorities are currently more interested on. This can be achieved by the use of energy absorbing shock mats such as USP and UBM. As mentioned previously, the use of USP in the concrete sleepers reduces the ballast stresses by two mechanisms: 1) Increase the contact area of the ballast to concrete sleeper interface, and 2) increase the number of load bearing sleepers per axle load (Bolmsvik 2005; Dahlberg 2010). Each of two mechanisms reduces the maximum load carried by each sleeper and thereby reduces the ballast stresses. Bolmsvik (2005) reported that USP increase the contact area of ballast to the sleeper by more than 36% for soft USP (stiffness 30 kN/mm) and by more than 18% for stiff USP (stiffness 70 kN/mm), which is otherwise far lower than 12%. As of the study by Loy (2008), the contact area between the sleeper and the ballast increases 30-35% with sleeper pads which is 5-8% without sleeper pads, at a bedded modulus C=0.2 N/mm² and reducing the pressure on the ballast by 10-25%. Dahlberg (2010) found that the higher stiffer tracks transmit the wheel–rail contact forces to the ballast through fewer number of the sleepers. Therefore, the ballast-sleeper contact stress is very high. This can be minimized by introducing USPs which distribute the stresses over more number of sleepers and thereby decrease the ballast stresses. The maximum contact force 57 kN without USP is reduced to 48 kN, 32 kN and 22 kN for stiff pad (stiffness 3000 kN/mm), medium stiff pad (stiffness 400 kN/mm) and soft pad (stiffness 50 kN/mm), respectively (Figure 3). It was concluded from the study by Dahlberg (2010) due to significant reduction of ballast stress, these USPs can be used to protect the ballast material in the track bed and the detrimental effects of hanging sleepers can also be reduced by these USPs.

2.5 Field Study on Use of Shock Mats

An extensive full-scale field test to investigate the influence of under sleeper pads (USPs) on track quality and track dynamics was conducted by Schneider et al. (2011) on the Schweizerische Bundesbahnen test site at Kiesen in Switzerland. This study concluded that the placement of USPs in a ballasted track changes the track performance. The track settlement increased with time when track was without USPs, and needed renewal of sleepers and re-tamping of ballast. The settlement restarted over again when the track was loaded. But when USPs were used, the track settlement appeared to decrease with time. The authors reported that the varying subgrade condition between padded and unpadded test track.
sides made it difficult to draw any specific recommendation. It was also mentioned in the study, the resilient layer reduced sleeper flexural strains but increased rail and sleeper accelerations and the contact forces between the USP and the ballast bed were related to the stiffness of the USPs.

3. DYNAMIC WHEEL-RAIL IMPACT FORCES

The wheel and rail undergo significant irregularities during the life time of the track structure. These irregularities are discrete in nature and usually at the surface of the rail and wheel. The higher frequency forces created by these irregularities are known as dynamic wheel-rail impact forces, which are higher in magnitude than quasi-static forces. If the wheel and rail surfaces are in good condition, then the wheel-rail contact force would be similar to the static wheel load (Steffens 2005).

3.1 Sources of impact load

The wheel-rail impact forces are caused by various sources such as wheel flat, wheel shells, worn wheel and rail, dipped rails, turnouts, crossings, insulated joints, expansion gap between two rail segments, rail joint misalignment, imperfect rail weld and rail corrugation (Indraratna et al. 2011). Figure 4 shows some of typical sources of irregularities.

![Figure 4 Wheel-Rail Irregularities causes impact forces](image)

These abnormalities on the wheel and rail can generate large impact forces between wheel and rail. The impact load caused by defects on the wheel subsequently rotates with each wheel rotation and roll over when the defects are in the rail. A large wheel impact forces generated at the turnout and crossings due to traversing of wheel over the rail discontinuity (Anastasopoulos et al. 2009). Besides, the rapid change of track stiffness at the road crossing, bridge approach and track transition such as concrete slab track merging to ballasted track or vice versa, the rise of high impact energy accelerate the track degradation and settlement (Li and Davis 2005). The magnitude of the impact forces is very high within the very short (2–10 msec) impulse duration (Lee et al. 2005). Therefore, the effects of impact forces are very significant in the design and utilization of concrete sleepers as parts of the railway track structures (Kumaran et al. 2002).

3.2 Impact Forces

Usually, the track degradation is driven by the wheel/rail impact loads, referred to as static load and peak loads. Two distinct types of peaks (1) an instantaneous sharp peak; and (2) a much longer duration gradual peak of smaller magnitude were observed during impact loading. Jenkins et al. (1974) termed these force peaks as P1 and P2, respectively. These P1 and P2 are respond to how a wheel rolling over a short-pitch irregular defects. These notations were adopted by industry and are in common use today to describe limitations on forces applied to the track structure (Indraratna et al. 2011). P1 and P2 forces observed from wheel/rail impact force time histories when the train vehicle passes a typical rail joint on Chinese mainline tracks at various train speeds are shown in Figure 5 (Zhai and Cai 1997).

The P1 force is due to the inertia of the rail and sleepers resisting the downward motion of the wheel and compression of the contact zone between the wheel and rail and the force is a very high frequency (>100 Hz) force of less than half a millisecond in length. Its effects are mostly filtered out by the rail and sleepers, therefore, its direct effect on ballast or subgrade settlement is very minimum (Frederick and Round 1985). On the other hand, the P2 force occurs at a lower frequency (30 – 90 Hz) than the P1 force, but in comparison to static forces this P2 force still classified as high frequency force. This P2 force is due to the downward movement of the vehicle unsprung mass and the rail/sleeper mass and causing compression of the ballast mass underneath the sleeper which increases the contact stresses, and the loads on sleepers and ballast. Therefore, the P2 forces are of great interest to the track designers.

![Figure 5 Wheel/Rail Impact Force](image)

Since the P2 forces are of greater importance in the assessment of track degradation, Jenkins et al. (1974) proposed a theoretical equation to calculate P2 forces at dipped joints. The P2 force in the equation shown below is dependent on the vehicle unsprung mass, track mass, track stiffness, vehicle speed and joint dip angle.

$$P_2 = P_0 + 2αV\sqrt{M_u + M_i} \left[1 - \frac{C_i\pi}{4\sqrt{k(M_u + M_i)}}\right]\sqrt{k_i}$$

where:

- $P_0$ = Vehicle static single wheel load (kN)
- $M_u$ = Vehicle unsprung mass (kg)
- $2α$ = Total joint angle (rad)
\[
V = \text{Speed of vehicle (m/s)}
\]

\[
K_s = 2K_t \beta
\]

\[
C_t = \frac{3C_s \beta}{2}
\]

\[
M_t = \frac{3M_s \beta}{2}
\]

\[
\beta = \left( \frac{K_s}{4EI} \right)^{0.25}
\]

\[
K_{td} = \text{Ballast Stiffness per metre (MN/m/m)}
\]

\[
C_{td} = \text{Ballast Damping per metre (kNs/m/m)}
\]

\[
M_{td} = \text{Rail + Sleeper mass per metre (kg/m)}
\]

Internationally similar limits are placed for the safety of the track. The British Railway Safety and Standards Board (RSSB) Railway Group Standard (GM/TT0088): Permissible Track Forces for Railway Vehicles (1993) states that when a vehicle (Class 55 Deltic locomotive) negotiates a vertical ramp discontinuity at its maximum design operating speed (160 km/h) the total \( P_2 \) force produced should not exceed 322 kN per wheel. Australian standards recommend Jenkins’s formula to calculate \( P_2 \) forces and specify the guidelines shown in Table 1 to limit \( P_2 \) forces as a function of track and vehicle characteristics (Indraratna et al. 2011).

Table 1 Limiting \( P_2 \) forces (QR 2001; RIC 2002; ARA 2003)

<table>
<thead>
<tr>
<th>Track Class</th>
<th>Maximum ( P_2 ) Force Locomotives (kN)</th>
<th>Maximum ( P_2 ) Force Other Rolling Stock (kN)</th>
<th>( K_r ) (MN/m)</th>
<th>( C_r ) (kNs/m)</th>
<th>( M_r ) (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>295</td>
<td>230</td>
<td>110</td>
<td>52.5</td>
<td>135</td>
</tr>
<tr>
<td>2</td>
<td>230</td>
<td>230</td>
<td>110</td>
<td>48</td>
<td>117</td>
</tr>
<tr>
<td>3</td>
<td>200</td>
<td>230</td>
<td>95.8</td>
<td>45.9</td>
<td>106</td>
</tr>
<tr>
<td>4</td>
<td>180</td>
<td>180</td>
<td>90.3</td>
<td>43.2</td>
<td>95</td>
</tr>
<tr>
<td>5</td>
<td>130</td>
<td>130</td>
<td>83.6</td>
<td>40</td>
<td>85</td>
</tr>
</tbody>
</table>

### 4. LABORATORY TESTING

In this study the use of energy absorbing shock mats to mitigate the ballast degradation under impact loading was assessed by a series of laboratory testing. The typical dynamic stresses in the range of 400-600 kPa caused by wheel-flat and dipped rail (Jenkins et al. 1974; Steffens and Murray 2005; Indraratna et al. 2010) was simulated by using the large-scale impact load test facility available at the University of Wollongong, Australia.

#### 4.1 Test apparatus, Impact Loading and Instrumentations

The impact loading test facility available at the University of Wollongong (Figure 6) is a high capacity drop weight impact test machine. It can be hoisted mechanically to the height which corresponds to the required impact load magnitude and drop height through guided roller on vertical column fixed to the strong concrete floor. The efficiency of the hammer velocity is 98% due to the friction of the guiding column (Kaewunruen and Remennikov 2010). Therefore, the actual hammer drop height \((h = V^2/2g)\) is calculated multiplying the theoretical drop height by a factor 1.04 (i.e., 1.098). The free fall drop hammer is a weight of 592 kg and it can be dropped from a maximum height of 6 m from the base of the concrete floor. The impact load was measured and recorded by a dynamic load cell of a capacity of 1,200 kN mounted at the bottom of the hammer and connected to a data acquisition system. Ballast deformation and transient acceleration of the impact loads were captured by a piezoelectric accelerometer of a capacity of 10,000g (g is the gravitational acceleration) connected at the top of the sample load plate shown in Figure 7.

#### 4.2 Material Specifications

The materials used in this study are the ballast, shock mats and the weak and hard base. The specifications of these materials are given in following sections.

### 4.2.1 Ballast

The railway ballast material commonly used in New South Wales (NSW), Australia is Latite basalt, a common igneous rock can be found in the south coast of NSW and closer to Wollongong City, Australia. The aggregates made from crushed volcanic basalt are dark, fine grained and very dense with sharp angular corners suitable for fresh railway ballast material. The physical and index properties of the fresh ballast were evaluated as per AS 2758.7 (1996) and discussed by Indraratna et al. (1998) in a previous study. The ballast material for this study was prepared in accordance with current practice in Australia as per AS 2758.7 (1996). The raw ballast material was thoroughly cleaned by water and dried before sieving. The particle size distribution (PSD) of the ballast material is shown in Figure 8. The basic martial parameters from the PSD are listed in Table 2.
### 4.2.2 Sand Subgrade

In order to simulate a typical weak base condition, a thin layer of sand subgrade cushion was used in the laboratory testing. The sand parameters are listed in Table 2.

#### 4.2.3 Shock Mats

There are many manufacturers of the USP and UBM around the world and some of the manufacturers listed by (Bolmsvik 2005). One of such manufacturer’s USP and UBM with its material parameters are shown in Figure 9 (a). USPs are generally stiffer than UBMs as they are placed adjacent to higher stress zones i.e. sleeper-ballast interface. The rubber shock mats used in this study was a 10 mm thick made of recycled rubber granulates of 1 to 3 mm particle size, bound by polyurethane elastomer compound. A sample of shock mat and its material parameters are shown in Figure 9 (b).

#### Table 2 Material Parameters of Ballast and Sand

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Fresh Ballast</th>
<th>Sand Subgrade</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type of Gradation</td>
<td>Uniformly graded</td>
<td>Poorly graded</td>
</tr>
<tr>
<td>Max. particle size, mm ($D_{max}$)</td>
<td>63.0</td>
<td>4.75</td>
</tr>
<tr>
<td>Min. particle size, mm ($D_{min}$)</td>
<td>19.0</td>
<td>0.075</td>
</tr>
<tr>
<td>Effective size, mm ($D_{ef}$)</td>
<td>24.0</td>
<td>0.24</td>
</tr>
<tr>
<td>Uniformity Coefficient ($C_u$)</td>
<td>1.6</td>
<td>2.3</td>
</tr>
<tr>
<td>Coefficient of Gradation ($C_c$)</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>

#### Figure 8 Particle Size Distribution (PSD) of the ballast material

#### 4.3 Laboratory Test Setup

The thickness of the ballast layer in Australian rail track is 250-300 mm (the lower thicknesses are at the bridge deck). Therefore a 300 mm thick ballast layer was selected as the specimen height in this study. 300 mm ballast thickness is found to be more realistically simulating site condition as per the previous study on ballast material conducted on large scale triaxial or cubical test apparatus by Brown et al. (2007) and Indraratna et al. (2007). The inclusion of shock mats at the top and bottom of ballast layer brings the total height of the track foundation more realistic value. In order to simulate the field density (approximately 1560 kg/m³) for heavy haul tracks, the ballast material was compacted in several layers by using a rubber padded hammer. The low lateral confining pressure for the ballast was simulated by placing a cylindrical rubber membrane around the specimen. The rubber membrane (thickness of 7 mm) was capable of prevent piercing or cutting the membrane by sharp corners of ballast particles.

The two types of base condition used were, 1) relatively weak base represented by a 100 mm thick sand layer vibro-compact to a density of 1620 kg/m³ and placed under the ballast bed, 2) hard base condition represented by a rigid steel plate of thickness 50 mm. This hard base condition is represented by the tracks running on steel bridge deck or track foundation located on hard bed rock. Three layers of shock mats (total thickness of 30 mm) were used at the top and bottom of the ballast specimen (Figure 7).

#### 4.4 Test Procedure

Each test specimens were placed on the concrete floor under the impact load hammer. The hammer was hoisted to the required drop height and released by an electronic quick release system. The ballast specimens were tested with and without shock mats placed at the top and the bottom of the ballast layer. The impact loading was repeated for 10 times for each sample. It was found that the strain due to impact loading is attenuating after certain number of blows (typical 8 or 9 blows). Automatic triggering of impact loading signal was enabled and data at sampling frequency of 50,000 Hz was collected by the data acquisition system. To remove the noise in the data, the raw impact load-time history data were digitally filtered using low-pass fourth order Butterworth filter with a cut-off frequency of 2,000 Hz. Ballast deformation and transient acceleration of the impact load data were collected by data acquisition system by the piezoelectric accelerometer connected at the top of the sample plate.

#### 4.5 Impact load-time history

The impact load was dropped on the sample and after the first impact the hammer rebounded on the sample couple of time then the impact load attenuated with time as shown in Figure 10. Two distinct types of peaks were observed during impact loading and named as P1 and P2 as per Jenkins et al. (1974). The peak P1 related to the multiple impacts including the first impact from the free fall hammer drop and the hit from rebounded hammer. The single peak P2 is related to the mechanical resistance of the ballast leading to its significant compression (Saxton et al. 1974). The P2 peak is lesser
than the instantaneous $P_1$ peaks. It is evident from Figure 10, the shock mats are attenuating the impact force (reduces the $P_2$ peak) and extending the time duration of impact load.

U.K. Railway group standards recommends considering $P_2$ force in the track design criteria as it is the direct influence on the degradation of track bed. Therefore the $P_2$ forces variation with continuous impact loading is the major concern in this study with respect to ballast degradation. The $P_2$ forces plotted with each blow is shown in Figure 11 showed a gradual increase with the increased number of blows. As the ballast particle get rearranged and become a densely packed after each blow, which offer a higher inertial resistance, leads to increased $P_2$ force values. When ballast particle rearrange and stabilise completely, the changes of $P_2$ forces become insignificant. This is apparent from Figure 11, the changes of $P_2$ forces very minor after 8th blow.

By comparing the impact forces with and without shock mats, the results shows that the shock mats attenuated the impact forces for both base conditions. It is also evident from the results shown in Figure 11 for the weak base without shock mat and hard base with shock mat, the weak base itself acted as a shock absorbing material. Therefore, the impact forces were more distinct for hard base.

\[ \varepsilon_q = \frac{2}{3}(\varepsilon_1 - \varepsilon_3) \]  
\[ \varepsilon_v = \varepsilon_1 + 2\varepsilon_3 \]

The variation of shear and volumetric strain with the number of impact blows are shown in Figure 12a and 12b, respectively. In general both the shear and volumetric strains increased in the initial impact loadings and eventually become constant at the end of impact blows 9 and 10. This is because the ballast layer displays a strong tendency to compact under repetitive loading due to rearrangement, reorientation and breakage of corners of the ballast particles (Lackenby et al. 2007; Indraratna et al. 2010) and become stable when the ballast particles are completely rearranged and densified.

4.6 Ballast deformation and strain response

Vertical and lateral deformation data were collected after each impact blow. The shear strain ($\varepsilon_q$) and volumetric strain ($\varepsilon_v$) for axisymmetric loading were calculated by using the following equation by Timoshenko and Goodier (1951).

\[ \varepsilon_q = \frac{2}{3}(\varepsilon_1 - \varepsilon_3) \]  
\[ \varepsilon_v = \varepsilon_1 + 2\varepsilon_3 \]

The inclusion of shock mats in the ballast bed reduced the shear and volumetric strain of the ballast layer. The permanent strains were more pronounced for the hard base condition. However when shock mats are placed at the top and bottom of the ballast layer the shear and the volumetric strains are reduced in the order 40% to 50%. The ballast breakage can be related to the number of blows as well as accumulated impulse (area under the transient impact loading curve). In order to abbreviate in view of scope of this paper, Figures 11 and 12 are plotted against number of blows.
4.7 Ballast Breakage under Impact Loading

Ballast particle breakage takes place under repetitive impact loading. Initially, breakage of corners of the angular ballast at the inter-particle contacts takes place, followed by complete fracture of the particles depends on the strength of the raw ballast and level of the load increase. This affects the overall deformation characteristics and ultimate strength of the ballast layer (Selig and Waters 1994; Indraratna et al. 2011). This breakage of ballast particles contributes to increased vertical and lateral deformations and differential track settlement. To quantify the particle breakage under impact loading, an evaluation of ballast breakage was performed. After 10 impact blow, the ballast from the specimen was recovered and particle size analysis was performed to compare the degraded ballast with the fresh ballast initially used in the testing. To quantify the ballast breakage, the following equation from the method proposed for Ballast Breakage Index (BBI) by Indraratna et al. (2005) was used.

\[
BBI = \frac{A}{(A + B)}
\]

The parameters defining the BBI are shown in Figure 13. The BBI for both hard and weak base condition with and without shock mats are summarized in Table 3. The values shown in parentheses in Table 3 are the percentage reduction of BBI by the use of shock mats at the top and the bottom of the ballast layer.

5. NUMERICAL MODELLING

The dynamic response of this layered system attributed to transient impact load is analyzed by a 2-dimentional (2D) axisymmetric dynamic finite element analysis by using PLAXIS (PLAXIS 2D: Ver. 8.6). The main features of this dynamic finite element analysis includes, introduction of modified stress-dilatancy relationship to capture the ballast particle degradation and incorporation of material damping for various track materials tested. The specimen of this study was modeled as an elasto-plastic model of a composite layered system including ballast, shock mat, base and steel plate. A typical axisymmetric specimen model simulated in finite element discretization using PLAXIS 2D is shown in Figure 14. All 3 layers are modeled using 15-node cubic strain elements and the interaction between granular media and the shock mats are modelled using 5-node interface elements. The 15-point cubic element provides a fourth order interpolation for displacements. The numerical integration by the Gaussian scheme involves 12 Gauss points.

The digitally filtered (by using a low-pass Butterworth filter) transient impact load-time histories obtained from the laboratory testings are used for the dynamic finite element analysis. Lateral distributed loads are applied to the right boundary to represent the confining effects of thick rubber membrane (Henkel and Gilbert 1952). The following boundary conditions are adopted for the numerical analysis. The left (axis of symmetry) and bottom boundaries are restrained in lateral and vertical directions, respectively. The top and right boundaries are free to move. The node at the left bottom corner of the mesh is restrained in both vertical and horizontal directions (pinned support - standard fixity). The right and bottom boundaries are considered absorbent boundaries. Two different soil models have been adopted are (1) classical Mohr-Coulomb elastic-perfectly-plastic model for the base material and (2) isotropic Hardening Soil model (Schanz et al. 1999) for ballast. The constitutive model parameters adopted here are based on the available laboratory test results.
5.1 Mohr-Coulomb Elasto-Plastic model

The Mohr-Coulomb (MC) model is used to represent the weak base. The following key parameters and values were used to represent a relatively weak base (i.e., poorly graded sand).

\[ E = 45 \text{ MPa}, \quad \nu = 0.33, \quad c' = 0, \quad \phi' = 24^\circ \quad \text{and} \quad \psi = 0. \]

5.2 Hardening Soil Model

The hardening soil (HS) model is used to simulate the strain-hardening behaviour of ballast under impact loading. The mobilised dilatancy angle \( \varphi_m \) is defined as follows:

\[
\sin \varphi_m = \frac{q}{q + 2\sigma'_s} \tag{5}
\]

The mobilised dilatancy angle \( \psi_m \) is given by (Nimbalkar et al. 2012):

\[
\sin \psi_m = \left( \frac{\sin \varphi'_m - \sin \varphi'_f}{\sin \varphi'_m - \varphi'_f} \right) = \frac{\kappa(dBBI)(1 - \sin \varphi'_m)}{2\sigma'_s d\varepsilon'_f (1 + \tan^2 \varphi'_f)} \tag{6}
\]

The symbols are explained in the notation section of this paper. Further details of the HS material parameters and breakage parameters are given in Table 4.

<table>
<thead>
<tr>
<th>Material Parameters</th>
<th>Hard Base</th>
<th>Weak Base</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample 1</td>
<td>Sample 2</td>
<td>Sample 3</td>
</tr>
<tr>
<td>( E'_{ISO} ) (MPa)</td>
<td>11.04</td>
<td>13.12</td>
</tr>
<tr>
<td>( E'_{cord} ) (MPa)</td>
<td>11.04</td>
<td>13.12</td>
</tr>
<tr>
<td>( E'_{cfr} ) (MPa)</td>
<td>10.20</td>
<td>12.09</td>
</tr>
<tr>
<td>( \psi' ) (degrees)</td>
<td>73.34</td>
<td>73.60</td>
</tr>
<tr>
<td>( \psi' ) (degrees)</td>
<td>21.27</td>
<td>16.15</td>
</tr>
<tr>
<td>( P'_{ref} ) (kN/m³)</td>
<td>19.70</td>
<td>12.67</td>
</tr>
<tr>
<td>( dBBI/d\varepsilon'_f )</td>
<td>0.81</td>
<td>0.68</td>
</tr>
<tr>
<td>( \kappa )</td>
<td>882.44</td>
<td>728.54</td>
</tr>
<tr>
<td>( (dE_x/\varepsilon'_f) )</td>
<td>714.78</td>
<td>495.41</td>
</tr>
</tbody>
</table>

5.3 Linear Elastic Model and Interface Elements

Steel plates located at the top and bottom of the test sample are considered as linear elastic. The shock mat is also modelled as a linear elastic material. Zero-thickness interface elements are used to model the frictional behaviour between various layers and are simulated by 5-node line elements. The following material parameters were used for Steel and shock mat.

\[ E = 210 \text{ GPa}, \quad \nu = 0.15, \quad \gamma = 77 \text{ kN/m}^3 \]

\[ Shock \ Mat: E = 6.12 \text{ MPa}, \quad \nu = 0.48, \quad \gamma = 12.04 \text{ kN/m}^3 \]

5.4 Finite Element Model Predictions

Figure 15 shows the finite element model prediction of the axial strain using the impact pulse data obtained in the laboratory impact testing. The axial strain values are compared with laboratory measured data for with and without the placement of shock mats for both hard and weak base conditions. As from Figure 15, the finite element analysis able to predict the strain hardening behaviour of ballast under repeated impact load. The FE simulation is closely captured the plastic yielding of the ballast which influenced by amount of vicious damping of the ballast material. The close comparison of FE model predicted and laboratory measured axial strain values reveal that the influence of \( P_1 \) forces on the response of the ballast is negligible, as the digitally filtered \( P_2 \) force load-time history was used as an input for the finite element analysis.

6. CONCLUSION

The performance of ballasted track with shock mats has been described through laboratory experiments and numerical models. The impact load causes accelerated ballast breakage was confirmed by experiment and numerical model data. Two base conditions tested in this study confirm that the hard base conditions such as bridge deck, rail track-road crossing and track on rock foundation cause comparatively higher ballast degradation compare to weak base condition. Initially, the impact induced strain of the ballast is very high and it eventually stabilizes and become constant after certain number of load application.

The insertion of shock mats at the top and bottom of the ballast reduces the impact induced stresses on ballast and considerably reduces the ballast degradation. As the hard base condition produces more breakage, the benefits of shock mats are greater in hard base conditions compared to weak bases. Weak base itself act as a shock absorbing layer, therefore the use of additional shock mats are not more pronounced for softer foundations. The finite element model analysis is capable of predicting strain responses measured for ballast under impact loading with and without shock mats. It is evident from this study, by placing shock mats, loads on the ballast bed can be reduced by a more homogenous mounting of the sleepers and ballast and track stability can be improved. This leads to reduced track misalignment, which in turn leads to a reduced number of maintenance operations.

Results could vary for different PSD and impact force \( P_1 \) and \( P_2 \). Also in reality, material used for USP and UBM can vary, usually stiffer mats preferred under sleeper. No study has yet been reported.

![Figure 15 Axial Strain: Measured vs FE predicted values](data sourced from Nimbalkar et al. 2012)
on quantitative or qualitative analysis of ballast degradation by placing USP and UBM under cycling loading condition. Currently an investigation is undertaken at the University of Wollongong testing facility to evaluate the effectiveness of USP and UBM in mitigating ballast degradation.

7. ACKNOWLEDGMENTS

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8. NOTATION

The symbols used in this paper are listed below:

\[ A = \text{Shift in the PSD curve after the test} \]
\[ B = \text{Potential breakage or the area between the arbitrary boundary of maximum breakage and the final PSD} \]
\[ C_v = \text{Coefficient of curvature} \]
\[ C_u = \text{Coefficient of uniformity} \]
\[ c' = \text{Cohesion (kPa)} \]
\[ D_{10} = \text{Effective particle size (mm)} \]
\[ dE_B = \text{Incremental energy consumption by particle breakage per unit volume (kN m/m}^3) \]
\[ D_{max} = \text{Maximum particle size (mm)} \]
\[ D_{min} = \text{Minimum particle size (mm)} \]
\[ E = \text{Young’s modulus (kPa)} \]
\[ E_{s0d} = \text{Stress-dependent tangent stiffness modulus for primary loading (kPa)} \]
\[ E_{s0r} = \text{Stress-dependent secant stiffness modulus for unloading and reloading (kPa)} \]
\[ E_{s0p} = \text{Stress-dependent secant stiffness modulus for primary loading (kPa)} \]
\[ k = \text{Constant of proportionality} \]
\[ N = \text{Number of blows} \]
\[ P_I = \text{High-frequency impact force (kN)} \]
\[ P_2 = \text{Low-frequency impact force (kN)} \]
\[ P_{ref} = \text{Reference pressure (kPa)} \]
\[ q = \text{Deviator stress (kPa)} \]
\[ q' = \text{Average vertical strain (major principal strain) in ballast layer} \]
\[ q'' = \text{Average lateral strain (minor principal strain) in ballast layer} \]
\[ v = \text{Poisson’s ratio} \]
\[ \sigma_1 = \text{Major principal effective stress (kPa)} \]
\[ \sigma_3 = \text{Minor principal effective stress (kPa)} \]
\[ \phi' = \text{Friction angle (degree)} \]
\[ \phi'_c = \text{Friction angle at critical state (degree)} \]
\[ \phi'' = \text{Mobilized friction angle (degree)} \]
\[ \phi''_p = \text{Peak friction angle obtained from peak stress ratio- (c'/\sigma_1)} \]
\[ \psi = \text{Dilatancy angle (degree)} \]
\[ \psi_m = \text{Mobilized dilatancy angle (degree)} \]

9. REFERENCES


RIC (2002), RSU120 General Interface Requirements, Version: 2.0, Rail Infrastructure Corporation (RailCorp).


