Performance of Ballasted Track under Impact Loading and Applications of Recycled Rubber Inclusion

Sanjay Nimbalkar[1], Sujit Kumar Dash[2], and Buddhima Indraratna[3]
[1]*School of Civil & Environmental Engineering, University of Technology Sydney, Sydney NSW 2007, Australia*
[2]*Department of Civil Engineering, Indian Institute of Technology Kharagpur, Kharagpur 721302, India*
[3]*Centre for Geomechanics and Railway Engineering; University of Wollongong, NSW 2522, Australia*
[1]*E-mail*: Sanjay.Nimbalkar@uts.edu.au
[2]*E-mail*: sujit@civil.iitkgp.ernet.in
[3]*E-mail*: indra@uow.edu.au

ABSTRACT: In this paper a review of the sources of impact loads and their effect on the performance of ballasted track is presented. The typical characteristics and implications of impact loading on track deterioration, particularly ballast degradation, are discussed. None of the procedures so far developed to design rail track incorporate the impact that dynamic loading has on the breakage of ballast and therefore it can be said to be incomplete. An intensive study on the impact of induced ballast breakage is needed in order to understand this phenomenon and then use the knowledge gained to further advance the design methodology. A stiff track structure can create severe dynamic loading under operating conditions which causes large scale component failure and increases maintenance requirements. Installing resilient mats such as rubber pads (ballast mat, soffit pad) in rail tracks can attenuate the dynamic force and improve overall performance. The efficacy of ballast mats to reduce structural noise and ground vibration has been studied extensively, but a few recent studies has reported how ballast mats and soffit pads reduce ballast degradation, thus obviating the necessity of a comprehensive study in this direction.

KEYWORDS: Impact loads, Recycled materials, Degradation, Rubber mats

1. INTRODUCTION

Railways form one of the major worldwide transportation networks and they continue to provide quick, affordable and safe means of transportation. The railroad industry has experienced repeated convulsions from changing economic needs, including the rise of automobile sector. With perpetually increasing traffic, rail tracks continue deteriorating fast because the induced vertical and lateral dynamic loads increase with an increase in the axle load. It was found that increasing the axle load from 18 tonnes to 25 tonnes shortened the rail life by 86.7% and increased the track maintenance workload by 79% (Zhai et al., 1993).

Ballasted tracks are the most common railroad structures, owing to the relatively low cost associated with their construction and maintenance operations (Janardhanam and Desai, 1983; Jeffs and Marich, 1987). Figure 1 shows the main components of a typical ballasted track structure which are grouped into two primary categories, superstructure and substructure. Superstructure consists of the rails, the fastening system, and the sleepers (ties). The substructure consists of the ballast, subballast (capping, structural or general fill) and the subgrade soil. Ideally, this structure should be elastic in nature because it is less prone to deterioration but compromises in design are often made due to budgetary limitations and therefore the ideal elasticity is not realised, which then often leads to a more rapid degradation of track quality and earlier renewals (Esveld, 2009). In view of this, the resilient components such as rail pad, soffit mat, and ballast mat are inserted in the track structure to mitigate vibration. Further details of these resilient components will be discussed in later sections of this paper.

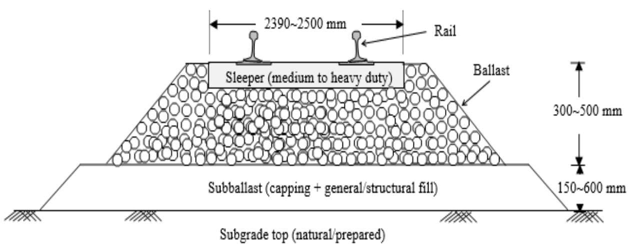

Figure 1 Typical ballasted rail-track system.

Ballast is a free draining granular material composed of coarse sized angular/rounded particles ranging in size from 10 to 60 mm (Selig and Waters, 1996). It can absorb a large proportion of the dynamic load (Chebli et al., 2008) and thereby offers a desirable resiliency to repeated wheel loads. It efficiently dissipates the energy generated from high frequency dynamic excitation (Grassie and Cox, 1985), distributes sleeper pressure over a wider area onto the subgrade (Ravitharan et al., 1998), and helps to hold the track in the desired alignment by resisting the vertical and lateral (transverse and longitudinal) forces exerted on the track from train movements. It also drains away the water falling onto the track. However ballasted layers need periodic maintenance due to deformation and degradation associated with breakage and fouling (Nimbalkar et al., 2012a). Constant vibration of the ballast layer from repeated wheel loads causes differential settlement which eventually alters the track geometry, and hence the cost of maintaining the track. According to statistics from the Chinese Railways (CR), about 75% of the daily maintenance work on track structures is due to the ballast and its deformation (Zhai et al., 2004). This problem becomes severe under impact loading because it accelerates the breakage of ballast particles. These impact loads can occur at various frequencies and at various amplitudes depending on the nature of wheel or rail irregularities, as well as on the dynamic response of the track (Jenkins et al., 1974; Remennikov et al., 2014). Therefore, understanding the complex mechanisms involved with the transfer of impact loads on the substructure and their effect on ballast breakage and degradation is essential when designing new tracks and rehabilitating existing ones.

In this paper, a state-of-art review of the above mentioned problems associated with high impact loads generated through wheel irregularities and track abnormalities is presented. The serious implications of impact loading on track deterioration in general, and ballast breakage in particular, are discussed. The use of resilient materials to reduce vibration in the track beds and the need to include impact based ballast degradation in track design methods has been brought out.

2. IMPACT LOADING ON RAIL TRACKS

2.1 Evaluation of train loads

An understanding of the sources that cause impact loads is the first essential step towards accurate estimation of them and how best they may be mitigated. In general, impact loads are generated from the interaction between a moving train and the track superstructure. The car body is connected to the bogie via the secondary suspension which in the case of modern passenger trains, usually consists of an air bag. The weight of the car body is then transferred to the wheels via a bogie frame that is connected to the wheels by the primary suspension system. The wheels in turn transfer the load to the rails. The vertical forces exerted on the track (F_V) comprises of six different components (Bahrekazemi, 2004),

$$F_V = F_{V0} + F_{Vk} + F_{Vds} + F_{Vdh} + F_{Vdb} + F_{Vj} \tag{1}$$

where, F_{V0} is the static wheel force (100%), F_{Vk} is the quasi-static contribution in curves (0-40%), F_{Vds} is the dynamic contribution due to rail irregularities (0-300%), F_{Vdh} is the dynamic contribution due to wheel flat (0-300%), F_{Vdb} is the contribution due to braking (0-20%) and F_{Vj} is the contribution due to asymmetries (0-10%).

Dynamic impact loads are caused by abnormalities in the wheels or rails, such as wheel flat, dipped rails, turnouts, crossings, insulated joints, an expansion gap between two segments of rail, imperfect welds and corrugations on the rail, etc. A diagrammatic representation of these typical sources of impact is shown in Figure 2. A wheel flat is a flat spot on the periphery of a wheel caused by its unintentional sliding on the rail, possibly due to poorly adjusted, frozen, or defective brakes, or braking forces that were too for the adhesion between the wheel and the rail (Jergeus et al., 1999). The potential causes of wheel flats and the damage they cause to the tracks are brought out by Nielsen and Johansson (2000). The clearance between the flange on the wheel at the intersecting point on the rail, called railway turnout, is another potential source of large wheel impact (Anastasopoulos et al., 2009).

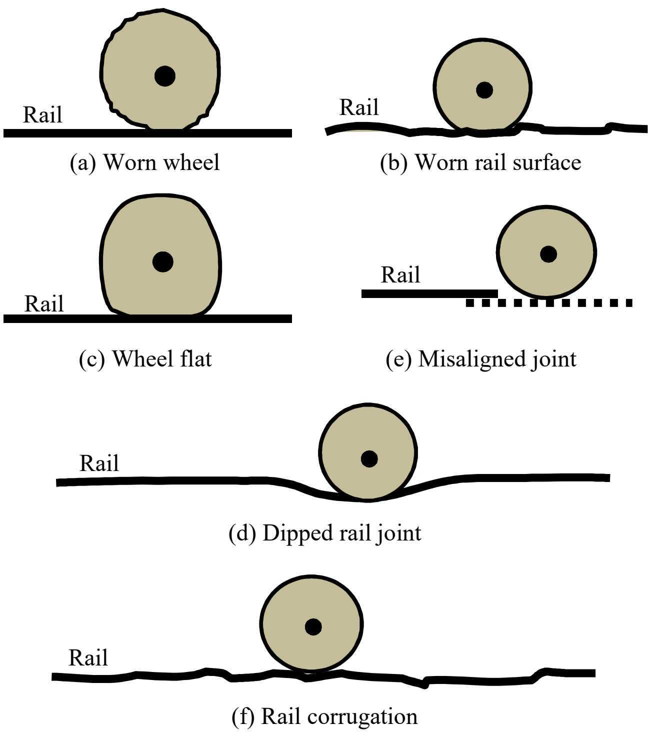

Figure 2 Various sources for impact loads in rail tracks.

The rapid change in wheel to rail contact at these turnouts, coupled with sudden variations in flexibility causes the wheels to jump up and down momentarily and impart impact loads onto the rails. These impact loads may also occur at the switch points due to the shape and flexibility of the movable blades used to control the direction of train passage (Bruni et. al., 2009). Besides, at transition points such as approaches to a bridge, road crossings, and concrete slab track to ballast track, the large change in track stiffness causes high impact forces which accelerate settlement (Hunt, 1997; Li and Davis, 2005; Mishra et al., 2014).

The impact load history is the combined effect of the response of inertial forces and bending resistance. The magnitude and frequency of these impact loads are generally much higher than the cyclic dynamic loads caused from the repeated passage of wheels. Indraratna et al. (2010) carried out in situ measurement of pressures transmitted into the ballast bed in a full scale track and found that where trains had wheel flats, pressures as high as 415kPa were transmitted to the ballast bed. The typical loading duration produced by the wheel flats can vary from 1 msec to 10 msecs, while the magnitude of the impact force could be as high as 600 kN per rail seat (Kaewunruen, 2007). Through analysis of field data collected over a year, Murray and Leong (2006) have observed that the maximum impact force is of the order of 400-500 kN. The overall wheel to rail impact data obtained, showed a logarithmically linear distribution which indicates that the occurrence of impact forces are random events.

Generally the impact response shows two distinct peak forces called P_1 and P_2, so designated by British Rail researchers (Jenkins et al., 1974; Dukkipati and Dong, 1999). Typical P_1 and P_2 forces in a simulated impact force time history caused by a dipped rail joint are shown in Figure 3. The inertia of the rail and sleeper resisting the downward motion of the wheel and compression between the wheel and the rail at the contact zone produces the first peak (P_1). This increases sharply as the striking drop mass impacts on the contact zone but it decreases quickly as the velocity of the test specimen increases due to the deformation induced by the impact. During this the wheel vibrates out of phase with the rail (Rochard and Schmid, 2004).

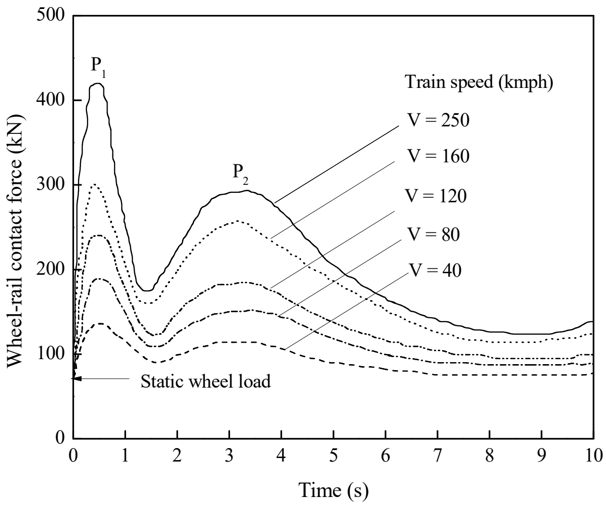

Figure 3 Simulated wheel/rail impact force history of a rail joint [data sourced from Zhai and Cai (1997)]

The P_1 force generally has a frequency greater than 100 Hz and can reach up to 200 Hz, but it only lasts for less than half a milli-second. Its effects are mostly filtered out by the rail and sleepers and therefore, do not directly affect the ballast or the subgrade (Frederick and Round, 1985), although it does have a visible influence on interaction between the wheel and the rail. The second peak (P_2) is gradual (frequency 30-90 Hz) but it lasts longer because of the mechanical resistance of the test specimen (Saxton et al., 1974). This force is due to the unsprung mass and rail-sleeper mass moving down together (vibrating in phase), severely compressing the underlying ballast (Rochard and Schmid, 2004). Furthermore, it increases the load on the sleepers, particularly at the joints, which leads to ballast breakage and track deflection, and since it remains longer, its potential to cause damage is very high. Indeed it has been observed that the second peak (P_2) has a significant influence on the formation of primary cracks in the sleeper and degradation of the underlying ballast (Bona, 2004). In view of this the P_2 forces are of greater importance in track design. The British Rail Safety and Standards Board (RSSB) Railway Group standard (GM/RC2513) suggest that for the safety of the track, the P_2 force should not exceed 322 kN. The P_2 force can be lowered by reducing the unsprung mass of the train that comprises the wheel set and other associated components that are not dynamically isolated from the track by suspension arrangements (Rochard and Schmid, 2004, 2005). Indeed Dukkipati and Dong (1999) have shown that the amplitude of forces, at resonance of rail track system reduces when the unsprung mass is reduced. In fact the P_2 force will reduce from 4 to 5 kN with every reduction of 1 kN of unsprung mass per axle, while the P_1 force will reduce about 1 kN (Steffens 2005). In order to understand the influence that different vehicle-track parameters have on the impact loads in railway tracks, several studies have recently been conducted. Based on the methodology adopted, these studies are discussed under three different sections i.e. analytical, numerical, experimental, and are presented below.

2.2 Analytical studies

Zhai and Cai (1997) simulated the dynamic interaction between train and ballasted rail track using the analytical model. The model considers the rail as an infinitely long beam discretely supported at rail and sleeper junctions by a series of springs, dampers, and masses representing the elasticity and damping effects of the rail pads, ballast, and subgrade soil, respectively (Figure 4). The shear interaction in the interlocked ballast system was simulated by connecting the individual masses of ballast to each other through shear springs and dampers. The dynamic interaction between wheels and rail was achieved through wheel-rail force compatibility using the non-linear Hertzian theory widely used in wheel-rail interaction problems, wherein the wheel rail contact force $P_{(t)}$ is,

$$P_{(t)} = \left[\frac{1}{G}\delta Z_{(t)}\right]^{\frac{3}{2}} \qquad (2)$$

where G is the Hertzian wheel-rail contact coefficient, $\delta Z_{(t)}$ is the elastic wheel deformation in compression mode along the vertical direction, and is given as

$$\delta Z_{(t)} = Z_{w(t)} - Z_{r(x,t)} - z_{0(t)} \qquad (3)$$

where $z_{0(t)}$ denotes geometric irregularities along the surface of the rail or around the circumference of the wheel, such as a rail joint, a corrugated rail, or a wheel flat, $z_{0(t)}$ can be any kind of deterministic function, either spatial or time function.

The results of a parametric study indicate that both P_1 and P_2 forces increase rapidly as the speed of the train increases (Figure 3), indicating that increased speed leads to increased impact and hence accelerated degradation of ballast and sleepers. Furthermore, while the density of the ballast has no effect on the impact force (P_2), its acceleration is substantially reduced as its density increases, indicating that in order to reduce the level of vibration in the track, the ballast should be compacted to a higher density. A reduced level of vibration reduces maintenance of the ballast and subgrade.

Suzuki et al. (2005) used their spring-dash-pot model to simulate the joint behaviour of ballasted rail tracks under vehicle movement. They found that with the vehicle running over the ascending convex part of the rail end, the peak wheel load was obtained when the wheel moved to the next rail and jumped down under gravity after it lost contact with the previous rail. The other factor contributing to impact is the resonance of the unsprung mass (M) of the vehicle and the stiffness of the track (K). Here, although the peak dynamic wheel load increased with an increase in train speed, beyond 30 km/h it tended to decrease.

However, it should be noted that the discrete spring support cannot transmit waves because it happens in a continuous layer of ballast (Esveld, 2001). Three types of waves are generated in a rail track under dynamic loadings, two body waves (a compression wave and a shear wave), and one surface wave that propagates along the surface of the track. Of these three, the surface wave generally conveys the largest part of the energy generated by the train. When the train reaches the same velocity as the surface wave in the ballast, the energy generated remains close to the train. This causes an accumulation of energy under the train over time which results in large amplifications, called resonance, in the track. The commonly used beam and spring models cannot capture these phenomena so any description of the track dynamics is inadequate. Indeed Sun et al. (2009) reported that for a dip of 1.852 mm in depth, 572 mm in length, and at a speed of 93 km/h, the peak impact acceleration value obtained through mass-spring-dashpot model is about 60 m/s^2 while the actual value measured on the rail track is about 510 m/s^2.

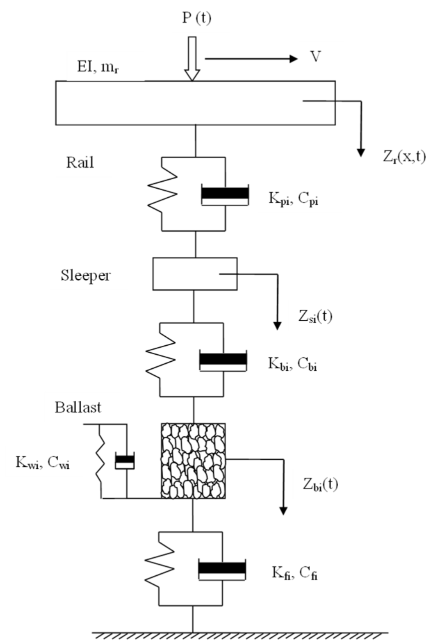

Figure 4 Vehicle and track vertical interaction system model [data sourced from Zhai and Cai (1997)]

The dynamic amplification predicted by the beam and spring model therefore, becomes significant only for velocities far beyond the range of operational train speeds. This would lead to the misconception that the dynamic track amplification caused by the train is generally negligible. Alternatively, the beam-half space model can accommodate all these phenomena more realistically because the ballast and supporting subgrade are modelled as a continuous half-space. However, this analysis is more complicated in nature and is generally solved through numerical techniques such as the finite element method (FEM) and finite difference method (FDM).

2.3 Numerical studies

Numerical methods such as FEM and FDM are powerful and versatile tools for analysing many problems in geomechanics for which mathematical solutions are not possible. One major advantage of these methods is that a physical appreciation of the behaviour of the continuum can be achieved at various stages of loading. Besides, complexities such as non-homogeneous media, non-linear material behaviour, arbitrary geometries, boundaries, discontinuities etc., are well taken care of in these methods but would be extremely difficult using conventional methods.

Through a finite element simulation of urban rail track turnout under train passage, Bruni et al. (2009) observed that at a speed of 55 km/h the vertical acceleration due to impact at the heart of the turnout is as high as 49g. A similar such observation was made in the field where peak acceleration was recorded as 44g. A parametric study indicates that an increase in the ballast or stiffness of the subsoil, and radiation damping, tend to visibly decrease acceleration and displacement at the turnout. It is of interest to note that the influence of these parameters is more pronounced in adjacent rails than at the heart (the point where rails intersect) of the turnout. This is because the heart region is directly affected by the impact and therefore the influence of ballast and foundation is not pronounced. However, for adjacent rails to be affected, the impact generated waves must pass through the sleepers to actuate the rails so the underlying ballast-soil foundation plays a major role.

Andersson and Dahlberg (1998) studied the vertical dynamics of turnout under a moving load using a linear finite element model with modal damping. The problem of interaction between the train and the track was solved in the time domain using an extended state, space vector approach and a modal superposition of the turnout. An increase in the force between the wheel and the rail was found at discontinuities such as junctions in the track. Where there was an irregular transition, the force was significant and its magnitude increased with velocity. Indeed Bruni et al. (2009) observed that the impact of the wheel at the heart of the turnout generates stress as high as 150 MPa.

Kassa et al. (2006) studied the dynamic interaction between the train and a railway turnout crossing using two multi-body system models. A detailed finite element modelling of the track was performed that included a variation in cross sections of the rail and multi-point contacts with the wheel. It was found that the variation in cross sections of the rail along the turnout play a substantial role in the dynamic force generated. Pang (2007) developed a three dimensional finite element model to investigate the impact of the wheel-rail contact at insulated joints in the rail. The explicit FE method was used in this dynamic analysis. The Lagrange Multiplier method and the Penalty method for contact constraint enforcement were adopted for the static and dynamic analyses, respectively. The findings of the model were similar to the experimental observations. The finite element model of a vehicle track system by Rao and Dong (1999) simulated the experiments performed by British Rail and the Canadian Pacific Rail system for obtaining the forces and stresses generated under the dynamic interaction between the wheel and rail components caused by wheel flats and shells. They found that the shape and size of the wheel flat or shell, the axle load, speed of the vehicle, and the stiffness of the rail pad, mainly affected the impact load. At a low velocity the track and unsprung mass acted simultaneously such that impact load peaked at 30 km/h, but at speeds higher than 60 km/h, the peak impact force initially increased with velocity to a maximum value, and then decreased when the train picked up high speed and literally flew over the flats.

The non-linear model of Wu and Thomson (2004) developed under the frame work of a finite element method shows that the impact forces rise dramatically when the pads used between the rail and sleeper were stiff. On the other hand, soft pads reduce the impact forces significantly.

2.4 Experimental studies

Kaewunruen and Remennikov (2009) performed a series of impact tests on full size concrete sleepers supported on thick rubber mat that simulated the ballast bed. The impact was simulated through a large drop-weight impact test facility (Figure 5), currently the largest in Australia. The impact of various magnitudes were applied by a mass of 5.81 kN dropped from various heights, i.e. 200 mm, 400 mm, 600 mm, and 800 mm. Two distinct peaks were observed under all these impacts (Figure 6), both P_1 and P_2 increased in magnitude as the drop height increased. It is of interest to note that at a low impact energy (200 mm drop height) the magnitude of P_1 was higher than for P2 while the reverse was the case for high impact energy (800 mm drop height). Moreover the peaks were relatively flatter at low drop heights but became sharper as the drop height increased, which indicated that the duration of impact reduces as the drop height increases. However, this variation is very small.

Ahlbeck and Hadden (1985) observed through measurements of full size tracks that the impact stemming from worn wheels and rails increases as the speed of the vehicle increases. For example the impact load increased from 100 kN to 133 kN when the speed increased from 32 km/h to 177 km/h, while the duration of the load, reduced from 40 ms to 5 ms when the speed increased from 32 km/h to 177 km/h. Frederic (1978) reported an increase of 30% in the P_1 force and an increase of 15% in P_2 when the speed of the train increased from 60 km/h to 96 km/h. Wu and Thompson (2001) reported similar findings through an analytical simulation of impact responses due to wheel flats.

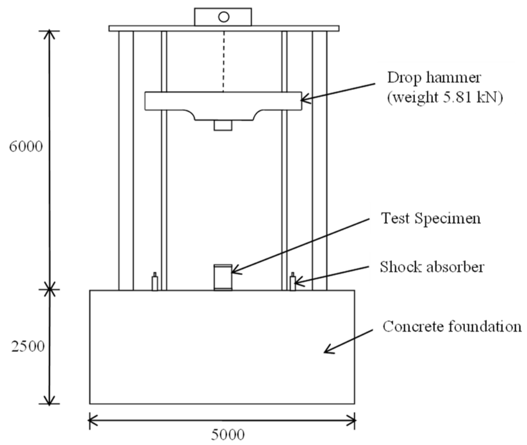

Figure 5 Drop-weight impact test setup [data sourced from Kaewunruen and Remennikov (2009)]

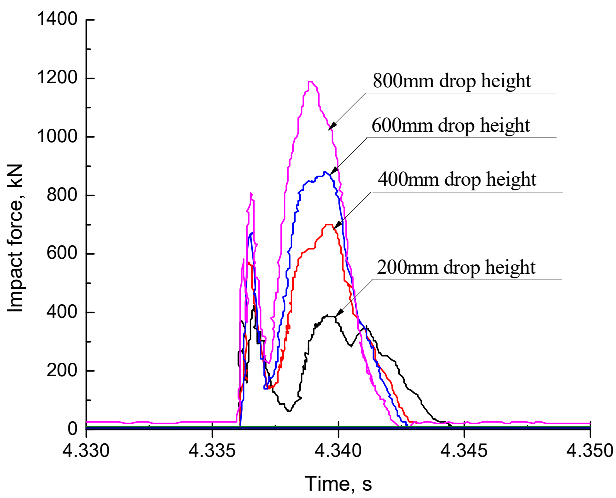

Figure 6 Typical impact forces from impact tests [data sourced from Kaewunruen and Remennikov (2009)]

At low speeds, the wheel, rail, and tie move together without separation during impact, but at speeds higher than 60 km/h, the wheels separate from the rails, which increases the force of impact as the speed increases further. At very high speed the wheel just flies over the flat which reduces the impact force on the rail (Dukkipati and Dong, 1999). However, with a light axle load, such as an empty car, the impact load increases with speed before it reaches a certain value, beyond which it remains almost constant at higher speeds (Dukkipati and Dong, 1999).

More details of different track and wheel irregularities causing impact, e.g. rail corrugation, wheel flats and shells, worn wheel and rail profiles, bad welds or joints, and track imperfections etc., are discussed in details by Remennikov and Kaewunruen (2008). The authors of this paper present a comprehensive review of the characteristics of different loading conditions for rail track structures, with a particular emphasis on impact loads generated due to wheel-rail interaction.

3. BEHAVIOUR OF BALLAST UNDER IMPACT

3.1 Evaluation of particle breakage

The primary functions of ballast are to resist the vertical, lateral, and longitudinal forces imposed on the sleeper-rail system and thereby hold the track in place. Ballast also redistributes the bearing pressure of sleepers over a wider area onto the subgrade (Selig and Waters, 1996; Gallagher et al., 1999). Its performance depends on properties such as the angularity of particles, and its shear strength, toughness, and hardness (Le Pen et al., 2013; Sun et al., 2014, 2016). Good quality ballast, apart from being strong, should have minimal cracks, rough surfaces, and contain low fouling materials. It should have at least two crushed faces, sharp angularity, and a coarse surface texture to help it interlock with the ties (Raymond, 2000). Although ballast is usually quarried from high strength unweathered rocks, large repeated wheel loads and occasional excessive impact loads often results in severe degradation. This degradation can occur in three ways (Lackenby, 2006):

(a) grinding-off of small asperities (abrasion) where the resulting fines cause fouling and reduce drainage
(b) breaking into fragments and angular projections, which influence initial settlement
(c) fracturing or splitting of individual particles.

This ballast breakage has detrimental effects on the long term stability and safety of the track. Experimental investigations (Hardin, 1985; Lade et al., 1996; Le Pen et al., 2013; Sun et al., 2014; Indraratna et al., 2016) have shown that the potential for particle breakage increases with the size of the ballast, because larger particles contain more flaws. Smaller particles are generally produced from larger particles fracturing along their defects, and smaller particles are less likely to fracture because they contain fewer defects. However studies by Redden et al. (2002) recommended the use of larger particles of ballast because they reduce vibration and less particles flying away under dynamic acceleration.

Impact produces a series of physical phenomena such as elastic shock and plastic wave propagation, fracture and fragmentation, perforation and spallation (Meyers, 1994) which degrades the ballast. When the dynamic load reaches 355.8 kN a track undergoes large scale degradation and or failure (Redden et al., 2002). Nimbalkar et al. (2012b) analysed ballast from a rail crossing at Thirroul, NSW, Australia and reported a ballast breakage index as high as 37%, which is a significant breakage near the impact zone. Degrading ballast does not give uniform support which in turn leads to over stressing in the soil subgrade below. Over stressing weakens the subgrade over a large depth, which eventually leads to track failure. The problem is more acute when the subgrade is of a lower strength (Grabe et al., 2005). In view of this, several researchers have studied the impact loading induced behaviour of ballast.

Tolppanen et al. (2002) conducted a three dimensional study of ballast degradation using the Los Angeles milling and 3-D scanning techniques. The Los Angeles Abrasion is a measure of an aggregate's resistance to fracture as it occurs under the impact of wheels (Barke and Chiu 2005). Seven different types of rock were tested. The resulting change in surface roughness for hard rock was found to be 20-35% while the change in maximum dimensions was in the order of 10%. The greatest change noted in degradation was for relatively soft limestone which rapidly became smoother in the first 500 cycles, after which degradation continued to produce shorter and rounder particles. The sharp edges of harder rocks were removed during the first 100 cycles, after which there were only limited changes in roughness and maximum dimensions. This indicates that in a newly formed track there shall be more breakage in the initial stages of traffic but this rate of breakage will subsequently reduce. The American Railway Engineering and Maintenance-off-way Association (AREMA) stipulates that for a stiff track modulus environment, ballast with a resistance to degradation of less than 20% should be selected, as determined by the Los Angeles Abrasion Test at 1000 revolutions (Redden et al., 2002). Ford (1995) used his analytical model to study the physical phenomena underlying the way the interaction between train and track contributes to the deterioration of track geometry. Indeed a relatively small change in the condition of the ballast produces major changes in the geometry of the track. Stewart and Selig (1982) reported that permanent deformation of the track results mostly from compression of the ballast alone, while other components are almost elastic in their response to wheel loading.

Li and Selig (1995) carried out experiments and analytical simulations to study the effect of repeated dynamic wheel loads on ballast degradation and plastic deformation of the track substructure. They found that the breakdown of ballast increased at an increasing rate with an increase in the magnitude of the load (P), as shown in Figure 7. Furthermore, the track modulus has a significant influence on the dynamic wheel load generated, i.e., the more resilient (softer) the track, the smaller is the impact load generated due to wheel and rail interaction. Dong (1995) reported similar observations where the peak ballast P_2 force due to impact, increases almost proportionately as the stiffness of the ballast mass increases. This indicates that in order to reduce the load on the track induced by impact, the track modulus that represents the overall stiffness of the rail foundation should not be too high. The analytical model data shows that the factor that affecting the track modulus most, is the character of the subgrade layer overlying the bedrock. Therefore, a low track modulus indicates that a softer rail foundation can be achieved by weakening the ballast bed and subgrade, but this would not help the stability of the track. Therefore, the authors suggest that rubber pads, elastic fasteners, or wooden ties should be used throughout the track superstructure to provide sufficient resiliency to absorb and reduce the dynamic wheel loads.

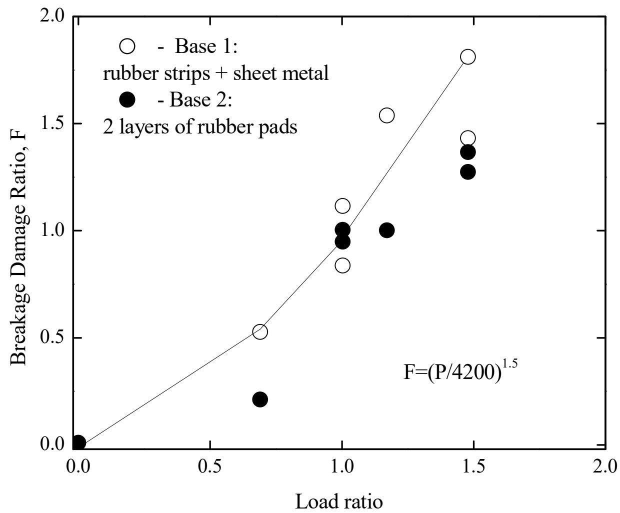

Figure 7 Effect of wheel load on ballast degradation [data sourced from Li and Selig, (1995)]

Aursudkij et al. (2009) reported that ballast breakage increases with a decreasing level of dilation, a phenomenon that takes place under low confining pressure. This lead to an interesting observation by Redden et al. (2002) regarding the spacing of ties; for a given wheel load, decreasing the spacing decreases the load on the rail seat and vertical pressure on the ballast, but it increases the track modulus by increasing the confinement. This increased track modulus increases the impact of the wheel load which in turn increases pressure on the ballast, which in turn offsets any benefits from decreasing the spacing of the ties.

Through a field study at four different concrete bridges with ballasted decks, Li and Davis (2005) saw abrupt changes in stiffness at the track to bridge transitions which accelerated the deterioration of the track geometry and large differential settlements between the bridge and the approach. Since the subgrade was well compacted and strengthened, and the subgrade had not failed, the fast deterioration in the track geometry (i.e. 100-180 mm within 6 months) was attributed to degradation of the layers of ballast and subballast.

3.2 Modelling particle breakage induced by dynamic loading

Through an extensive survey of literature related to impact Qiao et al. (2008) suggested that analytical models of impact mechanics can be classified into four different categories. (i) models based on the impulse-momentum law for rigid bodies (Goldsmith, 1960; Kolsky, 1963; Brach, 1991). In these models when a force is applied to a point in a body, every point in the body is instantaneously set in motion and the relative distances between any two material points never changes. However, this model cannot describe transient stresses, forces, or deformation, which is a serious limitation when simulating impact where the contact forces act for a very short period of time and local deformation is significant. (ii) models for propagation of stress waves in perfectly elastic materials. This model can simulate the transient stress condition that transmits strain energy away from the region of impact through stress wave propagation (Goldsmith, 1960; Zukas et. al., 1992). Coupled with Hertz theory (Johnson, 1985) it can simulate the duration of contact and maximal indentation. (iii) models for propagation of stress waves through solids which are not perfectly elastic. In this model the force-deformation equation is modified by adding a damping term to accommodate energy dissipation in the contact area from plastic deformation. This is often achieved by modelling the contact area as a spring damper system (Zhong, 1993) (iv) non-local and non-classical models (Silling, 2000; Silling et al., 2007). The prominent feature of these models is the spontaneous formation of discontinuities that can capture material behaviour at impact such as spallation and fragmentation. Among these four models, the last one would be of great use in simulating the degradation of ballast in rail tracks under the impact of wheels.

Several computational techniques incorporating the above models have been developed to simulate impact phenomenon, among which the finite element method (FEM) is the most widely used. Commercial FEM software such as ABAQUS, LS-DYNA, and PAM-Crash, have incorporated algorithms that include the modelling of contact, and are quite capable of simulating impact conditions (Hallquist et al., 1985; Hallquist, 1993). Finite difference methods (FDM) too have been used to simulate impact (Zukas 2004). However, it is extremely difficult to simulate fracture and material failure with these grid based methods (FEM and FDM), although this problem can be overcome to a large extent by using mesh-free methods (Qiao et al., 2008). The mesh-free method is a numerical one with no fixed connectivity between the discretisation nodes, which helps when simulating impact failure and fragmentation (Libersky and Petschek, 1991; Swegle et. al., 1995; Banerjee et al., 2005).

An elasto-plastic stress-strain constitutive model that takes into account the particle breakage of ballast during shearing was developed (Indraratna et al., 2014a). The effect of particle breakage and fouling on both plastic deviatoric and volumetric strain was included in the model (Indraratna et al., 2014b). This model is based on the critical state concept which considers non-associated flow, the kinematic type of yield locus, and a plastic flow rule incorporating the energy consumed from particle breakage and fouling. A constitutive model based on fractional calculus is an efficient tool for modelling long term deformation and therefore is incorporated into a constitutive model for predicting the geotechnical behaviour of ballast (Indraratna et al., 2017). Numerical studies were performed using PLAXIS, a finite element based computer code, to simulate the response of a ballast specimen under triaxial loading (Indraratna and Nimbalkar, 2013; Nimbalkar and Indraratna, 2014). An extended hardeningsoil model with hardening was used to represent the stress-strain behaviour of the granular material. The predicted stress-strain and change in volume were found to agree with the observed ones.

4. CURRENT DESIGN PRACTICES FOR BALLASTED TRACKS

Several simplified analytical and empirical methods have been proposed for designing rail tracks. The most prominent of these are from the American Railway Engineering Association (AREA, 1996), the European Union (EU) Rail (UIC, 1994), British Rail (Heath et al., 1972) and the Japanese National Railways (Okabe, 1961). However, these design methods assume the whole track bed to be a homogeneous half-space so they cannot simulate the influence of individual layers of track layers with properties that vary. This limitation is overcome in multi-layer track models such as, ILLITRACK (Robnett et al., 1975), GEOTRACK (Chang et al., 1980), KENTRACK (Huang et al. 1984) and FEARAT (Fateen, 1972; Colville, 1978) which are capable of analysing stress and deformation in all the major components of track and subgrade, i.e., rails, fasteners, sleepers (ties), ballast, subballast, and subgrade. The three dimensional multi-layer elastic GEOTRACK model proposed by Chang et al. (1980) can provide the resilient stresses and strains required to design the depth of ballast, select which type of ballast, analyse track stability and estimate any long term deformation. But these methods assume an elastic behaviour for every layer of track, including the ballast bed, which is a serious drawback.

Oscarsson and Dahlberg (1998) proposed a numerical model for simulating the vertical dynamic interaction of a railway track with a moving train. It was a linear finite element model with the rail as a Rayleigh-Timoshenko beam discretely supported by two dimensional sleepers with freedom through the rail pads. Two field experiments were performed to validate the model. The model can calculate deflection, acceleration, and forces in various components, and also enables engineers to investigate why parameters such as train speed, axle load, bogie wheelbase, rail corrugations, and wheel flats etc. affect both track and vehicle components.

In almost all track designs to date, impact is considered to be just a pseudo static component to be added to the static load, which means it is just a static design. Design engineers mostly use empirical relations to obtain the impact load from the static load. Track design in Australia design is based on the permissible stress concept where the dynamic wheel load (P_D) is obtained by multiplying the static wheel load (P_o) with a dynamic impact factor (φ), which accounts for the interaction between the vehicle and the track (Jeffs and Tew, 1991). The dynamic impact factor is always greater than one, in fact typical values are usually 1.4-1.6 times the static wheel load and in cases of high frequency this load might exceed 1.5 times the static wheel load. Therefore, being the sum of the static and dynamic wheel load, the total design wheel load is more than 2.5 times the static load.

The U.K. design standard (GM/TT0088) suggests that the P_2 component of the impact force that causes maximum damage to the track can be estimated using the formula given below, which is a relationship between the track and vehicle characteristics

$$P_2 = Q + (A_z \times V_m \times M \times C \times K) \quad (4)$$

$$M = \{M_v/(M_v + M_z)\}^{0.5} \quad (5)$$

$$C = 1 - [\pi C_z/4\{K_z(M_v + M_z)\}^{0.5}] \quad (6)$$

$$K = (K_z M_v)^{1/2} \quad (7)$$

where the maximum static wheel load (Q) in kN can be taken as 0.13 times the wheel diameter (D in mm) and shall not exceed 125 kN (i.e. half the maximum axle load generally accepted in railway lines). It has been observed (GM/RC2513, GM/TT0088) that the wheel diameter has a linear relationship with its static load. V_m is the maximum normal operating speed. M_v is the effective vertical unsprung mass per wheel. A_z is total angle of vertical ramp discontinuity. M_z is the effective vertical mass per wheel. C_z is the effective rate of rail damping per wheel. K_z is the effective vertical rail stiffness per wheel.

Dynamic effects begin when the train speed exceeds 10% that of the Rayleigh wave speed, V_c, of the subgrade (Yang et al., 2009). At a speed of $0.5V_c$ the shear stresses are underestimated by 30% in a static analysis and at speeds greater than V_c the stresses due to dynamic effects increase dramatically. A static analysis which ignores the dynamic effects could be acceptable for speeds up to 240 km/h, but for high speed trains the static analysis would incur large errors in the design (Grabe and Clayton, 2009). Besides, vertical loads exerted by a moving railway vehicle may be higher or lower than the static value, depending on whether the vehicle is momentarily accelerating downward and upward (Powrie et al., 2007).

None of the design procedures developed so far truly incorporate the influence of dynamic loading such as impact on the breakage of ballast leading to track instability, and therefore can be said to be incomplete. Hence, an intensive study on ballast breakage caused by impact is necessary in order to understand the phenomenon leading to advanced design methods.

5. HIGH ENERGY ABSORBING MATERIALS: BALLAST MATS AND SOFFIT PADS

The ballast in a typical ballasted track provides resiliency for low frequency loading (secondary suspension) but for high frequency loading (i.e. primary suspension), other resilient components such as rail pads, soffit mats, and ballast mats are necessary (Figure 8).

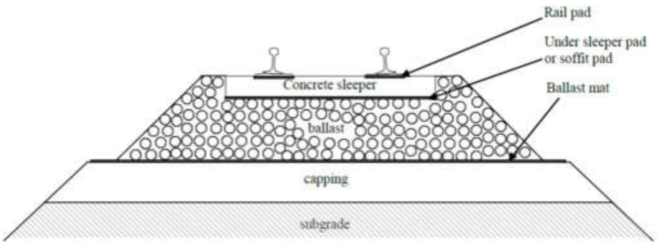

Figure 8 Overview of various elastic elements in a ballasted track [modified after Esveld (2009)]

In fact these additional resilient components actually restore the elasticity to the ballast. Ballast mats below the ballast layer are mostly fitted to help mitigate ground vibration on viaducts (bridge) and protect the concrete deck, and for mitigating structural noise. They also prevent particles of ballast from being crushed which improves the durability of the track (Esveld, 2009). In situations where it is highly imperative to use reduced thickness of the ballast such as on the deck of a bridge, ballast mats are preferred because they protect against degradation (Bachmann et al., 1997). Soffit pads are usually used below concrete sleepers so they are also called under sleeper pads (USPs). Soffit pads are quite effective at reducing the vertical transfer of dynamic stresses because they increase the contact area which subsequently reduces the contact stresses between the sleeper and particles of ballast. The use of USPs has increased in recent years, mainly in the newly built high speed railway tracks in Central Europe (Esveld, 2009).

A stiff track structure can create severe dynamic loading under operating conditions, which leads to large scale component failure and increased maintenance. Indeed it is a common observation that wood sleepers are much less stiff than concrete sleepers and produce lower ballast pressure in the dynamic load environment (Esveld, 2001; Redden et al., 2002). The analytically simulated results of Suzuki et al. (2005) showed that soft rail pads substantially reduced ballast settlement at track joints. In view of this evidence, installing resilient mats at the bottom of ballast in tunnels and on the decks of bridges is being practiced to improve the performance of tracks. Different types of mats are used, single or multiple layer rubber called profiled mats, granular mats made of old tires bound with a high grade elastomer, and foam mats consisting of a single layer or multi-layer polyurethane foam whose flexibility can be varied by altering the ratio of open cell to closed cell pores. The objective of introducing rubber mats into rail track is to create resilience and generate a mass-spring effect that will carry away the impact energy and redistribute it over a wider area, and thus reduce ballast degradation and sleeper cracking.

When ballast is placed onto a stiff subgrade, a large part of the body wave transmitted from the ballast to the subgrade is reflected back to the ballast which gives rise to dynamic amplification that accelerates ballast degradation. When this dynamic amplification from the impact load becomes superimposed with that caused by train speed, the response of the track becomes critical at much lower magnitudes of train velocity. When a ballast mat is fitted between the ballast and stiff substratum, its damping characteristics will reduce the wave reflections and reduce dynamic amplification (Esveld, 2001). Apart from improving the stiffness and damping characteristics, a rubber mat between the sleeper and the ballast (soffit pad), will reduce potential impacts between the sleeper and ballast and thereby reduce ballast degradation.

It should be noted that the layers of rubber layers change the motion of the ballast rather than absorbing substantial amounts of the impact energy. Fundamentally, the softer the mat the larger the attenuation obtained. Similar responses were observed by Esveld (2001), Dukkipati and Dong (1999), David (1988) and Grassie (1989), whereby reducing the stiffness of the rail pad (rubber pad between rail and sleeper) reduces the impact force on a concrete sleeper. However, if the rubber mats are too soft, stress in the rail will increase and the ballast will destabilise and require frequent tamping. In view of this a careful evaluation and assessment of the static and dynamic properties of the ballast/soffit mats, such as the effective mass, stiffness, damping etc. is highly desirable. There is no standard method available at present to evaluate these properties under dynamic loading conditions. Remennikov and Kaewunruen (2005), Kaewunruen and Remennikov (2008) have reported experimental techniques to evaluate such properties for rail pads and similar procedure could be adopted to evaluate the properties of the ballast/soffit mats.

Li and Davis (2005) carried out field tests to study the efficacy of rubber mats in improving track performance under impact loads in bridge approach transitions. They found that the introduction of a rubber tie mat reduced the track modulus from 61 N/mm/mm to 14 N/mm/mm, while the damping coefficient increased from 30 N/mm/s/tie/rail to 50 N/mm/s/tie/rail. This reduction in stiffness and increased damping establishes the beneficial effect of rubber mats in improving the interaction between vehicle and track.

The dynamic behaviour and transfer of force in railway tracks with and without ballast mats, has been studied by Auersch (2006) using a coupled finite element method of analysis. It was observed that a ballast mat inserted beneath the ballast lowers the resonance frequency between the vehicle and the track which in turn substantially reduces the dynamic forces transmitted to the underlying soil subgrade (Figure 9).

At higher frequencies the ballast mats result in a clear reduction of force amplitudes. The softest (i.e. low stiffness, k_M) ballast mats with the lowest resonance frequency result in the lowest amplitudes at high frequencies. But when the stiffness (i.e. shear wave velocity, V_s) of the subgrade soil of a ballasted track changes, the resonance frequency between the vehicle and the track remains almost unchanged, however, the resonance amplitude increases when the subgrade soil becomes stiffer (Figure 10). This indicates that a soft soil under the ballast mat reduces the resonance amplitude considerably. It was found that a relatively stiffer ballast mat induces higher damping in the system. However, the beneficial effect of the ballast mat, a reduction in the force transmitted into the subgrade soil at high frequencies, is almost the same for soft soil and stiff soil because the resonance frequency and high frequency damping are not influenced by the stiffness of the soil. It is of interest to note that with a standard track, the influence of the wheel set mass is very strong but with a ballast mat there is only a minor shift in the resonance frequency which indicates that it contributes quite a lot to interaction between vehicle and track.

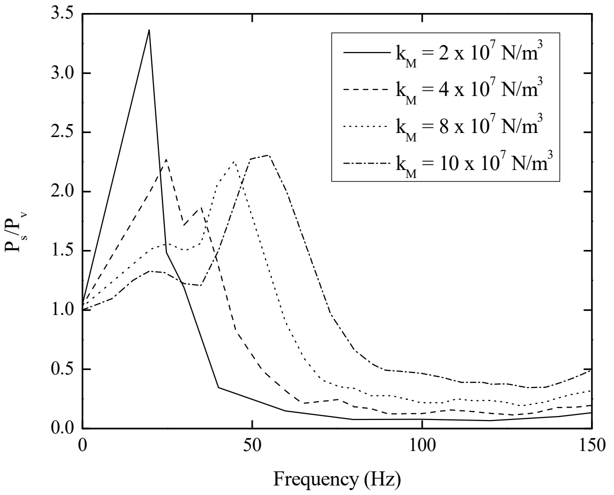

Figure 9 Force transfer function, P_s/P_v, of a wheel mass m_w = 1500 kg on tracks with ballast mats of different stiffness, k_M [data sourced from Auersch (2006)]

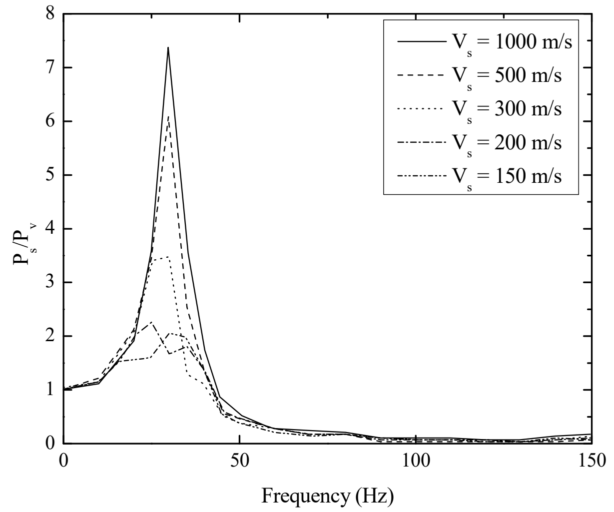

Figure 10 Force transfer function, P_s/P_v, of a wheel mass m_w = 1500 kg on a track with ballast mat $k_M = 4 \times 10^7$ N/m^3 on soils with varied shear wave velocity, V_s [data sourced from Auersch (2006)]

Anastasopoulos et al. (2009) carried out a finite element simulation to study the effectiveness of rubber mats placed at the sleeper to ballast (under sleeper mat) and ballast to subgrade soil interface (ballast mat), in rail tracks at the turnouts. The results showed that an increase in the elastic modulus of the rubber mat under the sleeper reduces the maximum vertical acceleration (a_{max}) of the rail under impact. But any increase in the stiffness of the ballast mat does not alter the acceleration on the rail head at the crossing nose, the point close to impact. However, in the region just above the ballast mat, the increase in the elastic modulus of ballast mat from 0.75 MPa to 1.5 MPa reduces the maximum vertical acceleration from 2.1g to 1.2g. This substantial effect is attributed to wave reflections. In the analysis, the modulus of the subsoil bed ($k_{subsoil}$) was varied from 500 to 3500 MN/m^3, which represents a medium dense foundation soil and a rigid base, respectively. It is observed that an increase of $k_{subsoil}$ leads to an increase of a_{max} on the rail head at crossing nose, from 3.2g to 4g. But in the region just above the ballast mat, very close to the interface with the subsoil, a_{max} is significantly affected by the $k_{subsoil}$. With the rigid subbase ($k_{subsoil}$ = 3500 MN/m^3), a_{max} tends to be zero because the impact induced waves cannot penetrate the rigid interface. With $k_{subsoil}$ = 500 MN/m^3, a_{max} is as high as 4g at the rail and 0.5g in the region just above the ballast mat. This substantial reduction in vibration from top to bottom indicates that the under sleeper mat-ballast-ballast mat system acts as a resilient system that can effectively absorb substantial amounts of impact energy. The quantification of this benefit indicates that, in the frequency range above 40Hz, a resilient track significantly reduces vibration in the soil in the range of -10dB (Figure 11). This agrees with the experimental observations recorded from field tests where a prototype of the resilient track concept was tested. However, in the low frequency range (< 40 Hz), vibration in the rail was slightly higher in the resilient track than the ballasted track without any rubber mat.

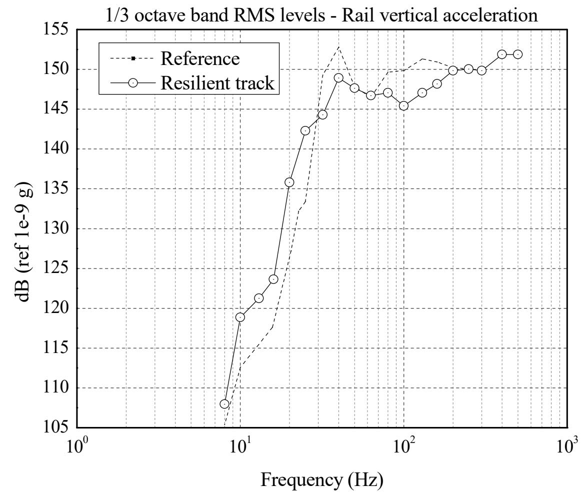

Figure 11 Resilient track concept: comparison of vibration levels [data sourced from Anastasopoulos et al. (2009)]

Indraratna et al. (2010) carried out field tests on an instrumented track at Bulli, New South Wales, Australia. The vertical and horizontal stresses developed in the track bed under repeated wheel loads were measured by placing pressure cells in the ballast. A typical plot of vertical cyclic stress transmitted to the ballast, under an axle load of about 25 tons and a train speed of about 60 km/hour, is depicted in Figure 12.

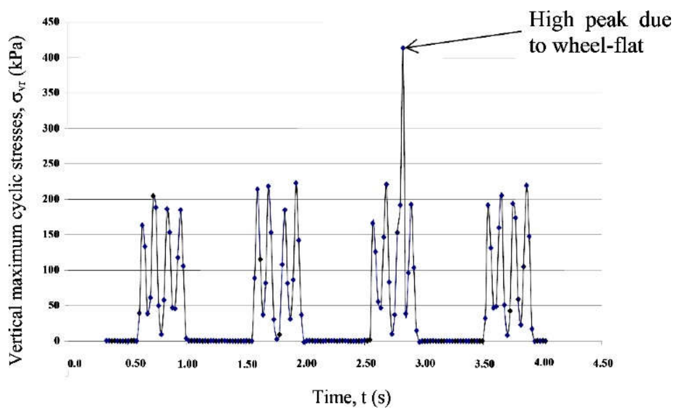

Figure 12 Typical measured vertical cyclic stresses transmitted to the ballast by coal train with wagons (100 tons) with wheel irregularity [Indraratna et al., 2010, with permission from ASCE]

It could be observed that, while most of the maximum vertical cyclic stress increased to 230 kPa, one peak reached as high as 415 kPa. This high intensity pressure was subsequently found to correspond with the arrival of a wheel flat, which indicated that high impact pressures are generated in the ballast by wheel imperfections, and should be carefully assessed and accounted for in the design and maintenance of ballasted track beds.

Nimbalkar and Indraratna (2016) investigated the 'in-situ' performance of rubber mat through a full-scale field monitoring of an instrumented track near Singleton, NSW. The details of track construction and material specifications can be found in Indraratna and Nimbalkar (2015). The ballast deformations (S_V) are plotted against the number of load cycles (N), as shown in Figure 13. These results indicated that the non-linear variation irrecoverable deformations in the ballast layer. The rate at which the permanent deformations in the ballast layer increased actually decreased as the number of load cycles increased.

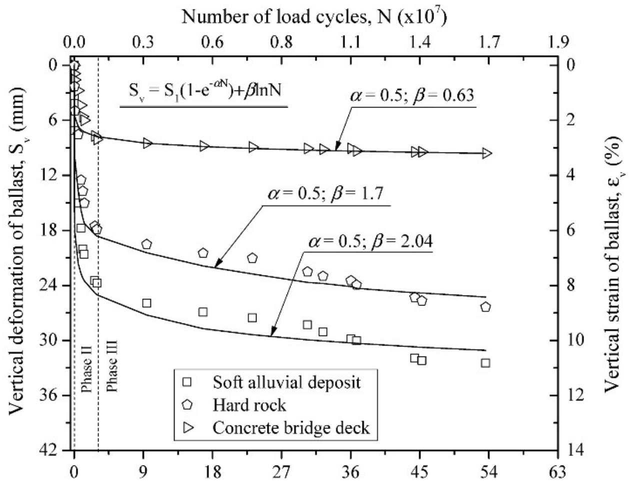

Figure 13 Vertical deformations and strains of ballast layer plotted versus number of load cycles for varying subgrade types [Nimbalkar and Indraratna 2016, with permission from ASCE].

Figure 13 also show the comparison vertical deformation in the experimental sections for different types of subgrade. The vertical deformation was smallest on the deck of the concrete bridge compared to other locations (i.e. soft alluvial clay deposit and hard rock). The relatively smaller deformation at Mudies Creek Bridge resulted, in part, from the ballast contained within the barriers of the bridge and where the shockmat was placed, which resulted in insignificant lateral spreading of ballast.

Samples were recovered to assess the particle breakage subjected to train load repetitions i.e. number of load cycles, N = 780,000 (Indraratna et al. 2014c]. Samples were recovered from three equal portions (top, middle and bottom layers). As expected, the amount of particle breakage was largest at the top and reduced as depth increased. Values of ballast breakage index (BBI) for various subgrade and shock mat combinations are listed in Table 1. The larger stresses also caused much more breakage of individual aggregates of ballast layer. At concrete bridge deck, least breakage was observed, thus confirming the ability of shock mat to mitigate ballast degradation. At other locations, a larger vertical deformation was observed thus attributing to larger particle breakage.

Nimbalkar et al. (2012a,b) through experiments and numerical simulations have investigated the performance of shock mats in the attenuation of dynamic impact loads and subsequent mitigation of ballast degradation. The investigations include different locations of shock mats considering stiff and weak subgrade conditions. With the provision of shock mats, the magnitude of impact forces decreases, and the time duration of impact gets prolonged. In the case of stiff subgrade, the efficiency of the shock mat in reducing the impact forces is greater when it is located at the bottom of ballast rather than at the top, whereas the reverse is true for weak subgrade. However, the provision of shock mats both at the top and bottom of the ballast bed, irrespective of the subgrade condition, is the best solution for minimising the impact force. The shock mats can bring down the impact-induced strains in the ballast bed by as much as 50%, apart from substantially reducing the ballast breakage.

Table 1 Measured particle breakage of ballast [data sourced from Indraratna et al. 2014c,d].

Subbase	Top layer	Middle layer	Bottom layer
Alluvial deposit	0.170	0.078	0.064
Concrete bridge deck	0.064	0.031	0.022
Hard rock	0.210	0.110	0.087

Qiao et al. (2008) reported the use of high energy absorbing materials such as lattice and truss structures, hybrid sandwich composites, metal foams, magneto rheological fluids, and porous shaped memory alloys in aerospace structures. It would be of interest to examine the utility of these materials as shock absorbers in rail tracks. The use of crumb rubber (derived from scrap tires) mixed with ballast layer could reduce ballast degradation and consumption of natural aggregates (Sol-Sánchez et al. 2015). Recently, a mixture of gravel and rubber pneumatically injected under the sleeper has found to improve longevity of ballast (Chan and Johan, 2016; Sol-Sánchez et al. 2017). Such diverse research is imperative to develop new materials with the aim of increasing the service life of the track.

6. SUMMARY

This paper presents a detailed review of studies on the influence of dynamic impact loading on ballasted railway tracks. The sources of these impact forces such as wheel flats, dipped rails, turnouts, and crossings, etc., have been brought out. Generally these impact forces show two distinct peaks called P_1 and P_2, of which P_2 has the most significant influence on the formation of crack in sleepers and degradation of the underlying ballast. Data from computational models, laboratory experiments, and instrumented field tests have shown that the maximum impact loading on the ballasted rail tracks can be as high as 500 kN, and when the dynamic loading reaches 355 kN, the track undergoes large scale degradation.

In almost all design methods, impact is only considered as a pseudo static component to be added to the static load, after which design becomes just a static design. Apart from which, impact loads are obtained through over simplified empirical relations. None of the track design methods developed so far, ever consider ballast breakage from dynamic impact loading and therefore, can be said to be incomplete. Hence, an intensive study on ballast breakage induced by impact is necessary in order to develop an understanding of the phenomenon leading to advanced design methods.

Over the past four decades many researchers have developed analytical/numerical models of the vehicle-wheel-rail-pad-sleeper-ballast-mat-subballast-subgrade system and dynamic forces, but there is still a need for a simple and reliable tool to simulate the impact induced ballast degradation and assess its implications on the function and long term serviceability of rail track. The advanced non-local and non-classical models which are able to capture the degradation of material under impact, such as spallation and fragmentation, can simulate the degradation of ballast very well. When incorporated into mesh free numerical codes, these models can be used to design ballasted tracks subjected to impact loading.

A stiff track structure 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 (ballast mat, soffit pad) in rail tracks can attenuate the dynamic forces and improved overall performance. The ability of ballast mats to reduce structural noise and vibration under the rail tracks has been studied extensively, but the benefits of ballast mat and soffit pad reducing ballast degradation has also recently been investigated as demonstrated in the paper.

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