Improved Performance of Ballasted Tracks at Transition Zones: A Review of Experimental and Modelling Approaches

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Abstract
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Keywords
ballasted, tracks, transition, review, experimental, zones, modelling, approaches, improved, performance

Disciplines
Engineering | Science and Technology Studies

Publication Details

This journal article is available at Research Online: https://ro.uow.edu.au/eispapers1/3052
Improved Performance of Ballasted Tracks at Transition Zones: A Review of Experimental and Modelling Approaches

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Abstract: Track transitions such as bridge approaches, road crossings and shifts from slab track to ballasted track are common locations where track degradation accelerates due to dynamic and high impact forces; as a consequence there is higher differential settlement. These types of discontinuities cause an abrupt change in the structural response of the track due mainly to variations in stiffness and track damping. Track transition zones are prone to an accelerated deterioration of track material and geometry that leads to increased maintenance costs. Track deterioration also leads to vehicle degradation due to enhanced acceleration, low frequency oscillation, and high frequency vibrations. While ballast deterioration is a major factor affecting the stability and longevity of rail tracks, the cost of tackling transition related problems that detract from passenger comfort is also high. A good transition zone lessens the impact of dynamic load of moving trains by minimising the abrupt variations in track stiffness and ensuring a smooth and gradual change from a less stiff (ballasted track) to a stiff (slab track) structure. This paper presents a critical review of various problems associated with transition zones and the measures adopted to mitigate them; it also includes critical review of research work carried out using large-scale laboratory testing, mathematical and computational modelling and field measurements on track transition zones.

Keywords: Transition zone, Ballasted track, Bridge deck, Transportation geotechnics, Modelling approach, Differential settlement, Stiffness variation, Impact load, Field measurement
1. Introduction

Track or railway transitions are locations along the track characterized by the presence of an abrupt variation of their stiffness, such as rail tracks change from a stiff structure (slab track) to soft structure (ballasted track) or vice versa. They occur when a conventional track changes to slab track to cross a roadway, a waterway (canal, river, etc.), or valleys through bridges, culverts or level crossings. Such transitions can be due to a sudden change in track substructural components (as in the case of slab track to ballast track transitions, bridge approaches, etc.), track superstructural components (as at special trackwork, level crossings, tie types, etc.) or both [96]. Figure 1 provides some examples of track transitions as a result of sudden change in substructural components. Figure 1a shows two rail track transitions indicating a ballast track to slab track and a slab track to ballast track on both sides of Berry Bridge crossing Tannery road in NSW, Australia. In contrast, Figure 1b illustrates a ballast track to slab track transition with an obvious alignment error that is often a problem associated with such transitions. Figure 2 provides some examples of track transitions as a result of sudden change in the superstructural components. Figure 2a shows a single level crossing on a conventional track at Unanderra, NSW, Australia, whereas, Figure 2b illustrates several other types of such transitions that can generate extreme dynamic loadings attributed to associated gaps and discontinuities causing variations on the rail running surface [96].

Transitions create a sudden change in the structural properties of tracks due to variations in track stiffness, track damping and subgrade reactions. This abrupt change leads to differential settlements and increased dynamic loading that accelerates track degradation through the successive deterioration of track geometry and materials [9, 10, 24, 31, 60, 95, 128]. The sudden change in structural properties at track transition can have an adverse effect on the rail deflections, dynamic loads and track acceleration due to the moving wheel loads. This effect can be seen in Figure 3 that has been reproduced from the modelling data given in references [34] and [186]. Figure 3 illustrates how these values vary suddenly in a short length at the junction point of a ballasted and slab track, while loads move from the ballast track to the slab track. It has been suggested that such abrupt variation in track acceleration causes oscillations or vibrations that further cause destructive effects [186]. If there is no proper intervention the wear and tear of track and vehicle components will increase as ride quality for all types of rail traffic will decrease, and this includes accelerated ballast degradation (breakage). The consequences will affect railway operations through restrictions in train speed, delays in train schedules, further passenger discomfort and higher maintenance costs [10, 31, 56, 95, 97, 127, 137, 159, 188, 194].

The cost of tackling problems associated with track transitions to maintain the smooth operation of
railways is often very high [64, 137, 139], for instance the annual maintenance costs for track transitions is approximately 200 million dollars in the USA and 110 million dollars in Europe [64, 139, 162]. According to Nicks [120], US$26 million per year is spent just repairing bridge related transitions, while in Spain; a major portion of their overall investment goes to track maintenance and infrastructure materials [137]. Previous studies also show that the cost of maintaining track transitions (at track discontinuities) is much higher (up to eight times) than normal conventional track [58, 79, 84, 95, 133, 137, 166, 167].

Transition zones are provided at track junctions to alleviate the problems associated with structural discontinuities [56, 79, 133, 137, 194] and to mitigate the dynamic effect of moving loads through smooth and gradual stiffness transitions [4]. With the increasing demand for long and heavy haul trains to travel at fast speeds, crossing bridge decks, concrete culverts or tunnels with stiff foundations towards softer soils or very soft estuarine plains, the precise and economic design of transition zones is a challenge for designers and practising engineers.

This article reviews various aspects of railway transition zones by first defining a track transition and its importance with respect to the structural integrity of track catering for passenger and heavy haul trains (in introduction). Second, reviewing the various problems associated with track transition to identify their causes and consequences on railway operations, and third, investigating the multiple measures taken to minimise and mitigate these problems with reference to their limitations and effectiveness. The design and modelling of tracks at transition zones, including large-scale laboratory testing and prototype experiments, mathematical and computational modelling and field measurements is also discussed, and detailed comparisons of computational modelling and field measurements are also provided in tabular forms. The paper concludes with future research recommendations for improved track design.

2. Problems of track transitions

Major problems associated with railway transitions include (i) differential settlement, (ii) enhanced dynamic load, and (iii) accelerated track deterioration; these problems are summarised as follows.

Differential settlement (also referred to as geometric irregularity) is the result of uneven deformation on both sides of track transitions where sections of ballasted tracks undergo more settlement than the stiff side, such as a slab track on a bridge, which is normally designed for minimal settlement [48, 79, 137]. Field investigations mostly show the maximum deformations at any specific point under repeated train loading. This maximum deformation includes the elastic component (i.e. fully recoverable upon unloading)) as well as plastic deformations which remain irrecoverable and continue to accumulate over successive load applications. Usually this occurs within a very short time (e.g. in 120ms), where the deformation attains its maximum value and returns to the residual
value [119]. Furthermore, the smaller values of differential settlement suggest the structural behaviour to be elastic, whereas the large differential settlements indicate plastic track response especially that of the substructural components [137]. It is also noted that some research based on computational modelling addresses the occurrence of transient deformations where the materials considered in the analysis are assumed fully elastic (i.e. small strain behaviour).

A detailed comparison of the differential settlement of various rail transitions is given in Figure 4. Figure 4a shows a sudden increase in vertical displacement [56, 57, 138, 189] when a slab track changes to a ballast track, giving rise to differential settlement at this location. Figure 4b compares the field measurements of two studies [43, 97, 174] for vertical displacements on each side of a bridge where the sudden variation in values is obvious. Figure 4c shows the increasing trend of rail displacement along the approach zones towards bridges at three different sites that could be due to hanging sleepers [172]. Figure 4d compares the vertical displacement at concrete culverts [25, 133] where approach slabs have been provided on each side. In this specific Figure 4, the part (a) indicates the elastic settlements that have been obtained by load application for a shorter duration, whereas (b), (c) and (d) include the plastic deformations as well.

Differential settlement also leads to the development of dips, bumps and undulations near the junction of tracks at transition zones, which is another source of passenger discomfort and increasing maintenance costs [43, 45, 62, 84, 120]. Fara [43] calls the development of dips and bumps, “Jump and Bump” and reports they can occur at both sides of a bridge at track transitions, as shown in Figure 5. While dips and bumps in railway and highway bridge approaches have been seen by various researchers [16, 100, 120, 187], in the USA, more than 50% of all bridge transitions face dip/bump problems, with average bumps being 33 mm high and 5.2m long [120].

The amplification of dynamic loads at track transitions due to abrupt changes in the structural properties of tracks is another major problem associated with rail transitions. Sudden variations in stiffness and differential movement at track junctions often increase the dynamic force at track transition under vehicle loading [26, 50, 106, 123, 194]. Mishra et al. [106] measured wheel loads using strain gauges at two bridge approaches and found an increase of up to 100% or more in the dynamic force on top of sleepers at bridge approaches compared to ballasted tracks. They recorded both the plastic and elastic deformations of every individual substructural layer through multi-depth deflectometer systems, but they considered only the effect of elastic (transient) response for obtaining the wheel load. This could be due to poor sleeper support at the bridge approach section, which affects the dynamic response of train suspension [106]. Lei and Mao [91] showed that differential settlement at track transitions leads to higher dynamic forces at the wheel-rail interface than sudden changes in track stiffness; the results found in [8, 50, 102] are similar. A detailed comparison of modelling result of enhanced dynamic loads in terms of wheel-rail interaction forces for various track transitions [92,
is given in Figure 6; here the very high contact forces at the wheel rail interfaces within the transition zone area are a clear indication of enhanced load impact at the transition.

The differential settlement and dynamic load at a transition zone are directly connected, and any increase in dynamic load would result in a corresponding increase in differential settlement, a process that seems to be exacerbated when moving trains with increased axle loads and faster speeds are involved [8, 45, 46, 50, 62, 64, 92, 103, 139, 194]. The cycle of track transition problems and its main components is drawn in Figure 7 (inspired by Paixão [122]). This figure illustrates the interdependency of enhanced dynamic load and differential settlement and their relationship to track degradation. If these problems are not addressed properly, they can lead to enhanced track deterioration and increased maintenance costs [95, 107, 137]. Figure 7 also shows the various causes of these problems and the probable consequences of not intervening properly; further details of these causes and consequences are discussed in subsequent sections.

It is known that rail tracks deteriorate faster at transition zones than normal ballasted tracks [30, 95], and this deterioration is triggered by the uneven settlement at rail transition zones which also increases the track degradation process. Track degradation includes rail corrugation and wear, track level irregularities, cracking sleepers, loosening ballast and rail fastenings, and hanging sleepers [8, 9, 30, 89, 91, 110, 128, 173, 189]. Moreover, the ballast breakage and particle migration adjacent to the sleeper may also lead to hanging or swinging sleepers due to increasing dynamic loads and differential settlement at transitions [5, 25, 31, 48, 58, 64, 92, 95, 106, 120, 123, 133, 154, 162, 194]. According to Pita et al. [127], track deterioration is mainly influenced by the performance of its components at track transitions subjected to higher dynamic loads and the frequent movement of high-speed trains. Track degradation also leads to vehicle degradation due to enhanced acceleration, low frequency oscillation, and high frequency vibrations [9, 30, 41, 93, 102, 190].

3. Major causes of track transition problems

Uneven stiffness and damping between two different subgrade materials, the variation of moisture and geotechnical causes are the primary sources of track degradation at any transition zone [79, 84, 95, 120]. Gallage et al. [48] divided the causes of transition related problems into two categories: (i) primary causes such as variations in stiffness and damping, geotechnical issues, subgrade failure, excessive plastic deformation, progressive shear failure, soil water response, and wetting and shrinking cycles, and (ii) secondary causes such as train loads and speed, traffic conditions, embankment heights and types of bridge abutments.

The abrupt variations in stiffness at track transition are the major reasons for track problems [22, 23, 45, 62, 79, 84, 92, 98, 103, 107, 113]. Figure 8 shows a typical example of variations in track stiffness
where the total track stiffness $k_o$ (ballast track) suddenly changes to $k_n$ (slab track on a bridge deck); these sudden variations cause differential settlement and expedite track degradation [10, 31]. High values of track stiffness can also cause hanging sleepers as sleeper-ballast contact decreases and the gap between ballast and sleepers increases [25]. A detailed comparison of variations in track stiffness/modulus at various sites is given in Figure 9. Figure 9a shows the sudden variations in track modulus on both sides of the bridge [133], whereas Figure 9b shows how the track modulus/stiffness increases when a track changes from being less stiff to stiffer [54, 120, 166]. Note that stiff tracks such as bridges have higher modulus values than tracks that are not as stiff.

Stiffer tracks can reduce track settlement and increase longevity but they are vulnerable to track deterioration because the stiffness increases the contact forces between wheel and rail and at sleeper-ballast interfaces that could increase dynamic pressures acting on track substructure [10, 24, 98]. Track stiffness is defined as the load needed to produce a unit deflection in track and is denoted by $k$, (kN/mm); it can be static (remains constant) or dynamic (depending on the load and excitation frequency) [131, 137]. Track modulus is sometimes used instead of track stiffness because it can be defined as the load to produce a unit deflection per unit length of rail [133]. The value of track stiffness depends on the type of material and height of track embankments [51]. Figure 10 shows the track stiffness at various locations on a west coast line in Sweden; note that the track on a pile-deck bridge is almost twice as stiff as the normal track. The influence that sub-ground (formation soils) has on track stiffness is also evident, hence the rapid change in stiffness for various types of track [30].

Ballast degradation at track transitions is one of the main causes of progressive track deterioration [72, 79, 95]. This degradation occurs as ballast deforms due to volumetric compaction (ballast compaction and particle breakage) and frictional sliding (lateral movement of ballast particles under sleepers) mechanisms [25, 29, 70, 78, 140, 144, 158, 164, 174]. There is always more ballast degradation on the ballasted track side due to ballast fouling (contamination by fines), plastic deformation, chemical actions, and variations in moisture and temperature [22, 66, 154, 175, 194]. However, no such degradation occurs on a slab track [4]. Because of this, the subsequent differential settlement does increase the dynamic loads and ballast stresses at transition zones [107, 137]. These sudden variations of induced stresses on ballast aggregates continually increases the rate of ballast degradation. A comparison of the various ballast pressure/stresses for different track transitions [120, 171, 174] is given in Figure 11. This figure shows that the amplitude of ballast stresses suddenly changes at the bridge abutments on both sides, thus indicating the abrupt variation in measured stresses at these locations where the stiffness is greater.

Since the settlement of the capping and subgrade layers is mostly permanent (i.e. plastic deformation) it does contribute to problems such as hanging sleepers at transition zones because the vertical movement of ballast particles that leads to differential settlement and enhanced dynamic loads [85,
Impeded track drainage and poor compaction of low quality backfill materials, as well as limited accessibility beside these structures, can also accelerate subgrade settlement [95, 120, 130, 137]. A detailed study on the settlement of subgrade under heavy axle loads can be found in [94].

Sudden variations in track damping characteristics plays an important role in the development of differential settlements at track transitions [107] and also defines dynamic interaction at the wheel-track interface [24, 40], which is why damping of track components at transition zones influences the vehicle-track-subgrade dynamic response and helps to reduce track vibrations [90, 149]. Track damping helps to dissipate the energy produced by large dynamic loads from fast moving vehicles, whereas a sudden change from a highly damped track (track on an embankment) to a low damped track (slab track on a bridge deck) can cause damage from wheel impact (due to surface deterioration) or wheel bounce (due to variations in stiffness) phenomena. The energy imparted onto embankment tracks can be dissipated through its structural components and the subgrade and surrounding ground, and while the ballast layer in a ballasted-deck bridge track will dissipate some of the energy and most of it will still reach the bridge structures [139].

4. Mitigation measures to track transition problems

A good transition zone must be able to minimise the impact of dynamic loads applied by moving trains. Different approaches for providing a smooth and gradual transition have been proposed and implemented through laboratory experiments, model testing, field investigations and mathematical and numerical modelling; they are reviewed and discussed in the following sections.

Transition wedges are widely used to smooth the tracks at transition areas; these wedge shaped backfills are combinations of cement bond granular materials (CBM), unbound granular material (UGM), graded gravels with some percentage of cement, simple graded gravels, and well graded coarse grained soils [25, 44, 59, 79, 95, 125, 126]. When this system is used at bridge approaches, there is an immediate improvement in the dynamic response of the overall track system under moving train loads [147]. This technique focuses mainly on selecting materials with a variety of characteristics (type, modulus, stiffness, cementation, among others) for the transition wedge and its geometry (thickness, slope, layer distribution, among others) so there is a gradual transition from soft to stiff material in the transition zones. Recommendations for such selections based on variations from soft to stiff, and even from sleeper to sleeper, can be found in [50]. At present there is no universal standard for the design of a transition wedge, so different countries choose their own set of parameters. A comparison of various transition wedges, including their material configurations and geometric shapes that are used in different countries, is described in [44].

Varying the size and spacing of sleepers is another common approach for reducing abrupt change in
track stiffness at transition zones; in this approach the length, width and height of sleepers gradually increase, while the spacing between them gradually decreases when track structures proceed from being less stiff to more stiff [112, 113, 133]. Larger sleepers have proven to be good at mitigating ballast settlement and contact pressure between ballast and sleepers [113, 147], but not as good at reducing the dynamic load factor [120]. Unlike maintaining the uniformity and compaction of ballast, this approach to mitigation does not help to increase track stiffness, but it may reduce vertical displacement and induced stresses by distributing train loads over wider areas [137].

Sleepers made from composite, plastic, or rubber materials can be used at the transition zone [113, 133, 139]. Rubber sleepers that can adjust the sleeper/ballast stiffness are better at reducing the ballast vibration of high speed railway lines [113]. Frame sleepers, where every two sleepers are connected by additional supports to distribute the load over a wider area can also be used in the transition zone [3]. Nicks [120] studied the effect that three different sleepers (wooden, concrete and plastic) had on the dynamic response of bridge approaches in dip and bump cases and found that wooden sleepers will help to mitigate the bumps and dips better than the other materials.

Rail pads have recently been used to reduce the vibration and noise from impact loads under train movements and improve the damping properties of track substructure [30, 133, 152]. According to Namura and Suzuki [113], installing softer rail pads on the stiffer side of track transitions makes the rider smoother; studies into the use of soft pads on the stiff side of transitions can be found in [56, 166] and studies on the use of rubber pads that are as stiff as the bridge approaches is given in [83]. Note that thermoplastic elastomer rail pads/seat plates that are used in railway maintenance works are temperature dependent, so this property must be considered in design practice [20]. The temperature-dependency of static stiffness of various types of rail pads can be found in [176], where a nonlinear variation of static stiffness of rail pads was observed with temperature ranging from -40°C to 70°C.

Under sleeper pads (USPs) are increasingly being used to mitigate the problems associated with transition zones because they actually reduce ballast degradation, enabling the stiffness on the stiffer side (concrete deck) to match the softer (ballasted track) side, and minimise the dynamic load impact [2, 30, 76, 112, 115, 120, 152, 166]. However, placing USPs on ballasted track at bridge approaches may not be that effective in reducing the stiffness variation, as it makes the softer side of the bridge transition even softer [120]; however, it can reduce the ballast stresses significantly due to the increase in sleeper-ballast contacts leading to reduced ballast degradation [115]. The use of resilient material mats (rubber mats) under slab track has proven to reduce track vibration [52]. A summary of the various effects of under sleeper pads can be found in [102, 178], and a detailed investigation for their effectiveness through laboratory experiments and numerical simulations can be found in [80, 115].

Adding an extra rail (auxiliary rail) in between or on the sides of the main rails at the ballasted track
section of the transition zone will help to distribute the dynamic loads evenly [34, 56, 84, 113, 133];
one example of this is where guard rails are extended from the bridge abutments to the bridge
approaches [133]. This technique helps to improve the bending stiffness of the track and reduce ballast
stresses by distributing the load to the sleepers [147]. According to Shahraki et al. [147], auxiliary
rails improve the dynamic response of track by providing a smooth transition over the sudden changes
in stiffness. In some cases, auxiliary rails may not be as good at reducing the dynamic response
compared to some other mitigation approaches, so proper consideration should be given to its benefit-
cost ratio before making a final selection [133, 148]. However, two extra rails along the transition
zone are the optimum number of rails needed to decrease rail deflections [56].

In some cases, concrete confinement walls (wing walls) are installed along the approaches to reduce
ballast loosening [133, 177]; these walls increase the lateral confinement on ballast and thus reduce
the problems associated with track deformation; these walls also confine the subgrade layers and
further decrease track settlement [177]. However, there may be a large increase in track modulus due
to increased confinement and associated ballast breakage and this must be considered during design
[137]. Apart from wing walls, Nicks [120] installed steel bars of varying lengths between sleepers
into the subgrade to increase the confinement and strength of ballast; this approach is much better at
reducing subgrade stress and the track deflection, and ultimately mitigating the development of dips.

While increasing the thickness of ballast and sub-ballast (capping) at transition zones will enhance
track performance, it might also cause excessive track settlement [90, 94, 95]. However, if there are
bumps in the track, increasing the thickness of ballast at the bridge approaches is the best approach
because the extra depth helps to attenuate stress and reduce the deviatoric stresses applied on the
substructural layers [120, 141]. Moreover, increasing the ballast thickness also increases the track
modulus; Selig and Li [143] report an increase in the track modulus from 24 MPa to 34 MPa after
increasing the ballast from 0.3 m to 1.07 m thick.

The use of resin and polyurethane compound to glue ballast aggregates to reduce track settlement
[55, 82, 113, 180-183] has been tested. In fact according to Kennedy et al. [82], an almost 99%
reduction in permanent settlement can be achieved with polymer treated tracks because a ballast track
performs almost the same as a slab track. Similarly, reinforcing ballast with 3D polyurethane and 3D
polymer not only improves the efficiency and safety of a railway track, it also helps to reduce the cost
of track maintenance [180, 181]. However, Stanislav et al. [153] while investigating the effectiveness
of expanding polyurethane resin at bridge transition zone, found no improvement in track dynamic
performance. While there is no convincing evidence to indicate the life-span of this polymeric
material and its resistance to harsh track environments including UV damage, this method also raises
the question about the benefits of highly angular ballast particles (i.e. intrinsic friction in the
microscale) that may be subdued by bonding of particles.
Approach slabs (submerged approach structure) are often used on both sides of buried structures such as viaducts, culverts or bridges to reduce the high impact loads associated with sudden changes in track stiffness [19, 26, 114, 139, 165, 167]. It is a common practice in European railways to have concrete slab transitions (both horizontal and inclined) between a ballasted track and a concrete culvert to provide a smooth transition [25]. However, the recent research by Coelho et al. [26] shows that track on an approach slab has about four and eight times higher vertical displacements than on an embankment or culvert, respectively. It is reported that these higher displacements could be due to hanging sleepers on the approach slab and the tracks rocking under train movement due to a pivoting action around the edges of the stiff culverts, all of which leads to high impact loading.

Placing a layer of hot mix asphalt (HMA) under ballast to improve its performance is another proven mitigation technique for transition zones [86, 95, 133, 155, 177]. When an HMA layer is placed under ballast and protected from the effects of climate it can increase the life of the track substructure and enhance track performance by reducing stress at the ballast/capping interface and reduce the maintenance cycles [135]. Moreover, since HMA is impervious, it prevents water from seeping into the underlying subgrade layer [95], so the drainage capacity of tracks increases. An HMA layer may also help to strengthen the substructure layer by improving its load bearing capacity and further reducing the stresses acting on the subgrade [142].

Improving the load bearing capacity of track embankments with soil treatment such as grouting, dynamic compaction, soil cement, geosynthetics, geocells, cement gravels, etc., has been widely adopted [95, 130, 137, 153]. Using geocells (honeycomb structure) within the sub-ballast (capping) layer will help to improve track performance by increasing the stiffness of infilled aggregates [94]. A variety of mitigation techniques commonly used at transition zones, especially bridge approaches, to improve the embankment soil can be found in [79, 129, 130, 153]; they can be divided into three categories: (i) Mechanical (excavation and replacement, preloading and surcharge, dynamic compaction), (ii) Hydraulic (sand drains, prefabricated drains, surcharge loading), and (iii) Reinforcement (columns, stone and lime columns, geo-piers, concrete injected columns, deep soil mixing columns, deep foundations, in-situ: compacted piles, continuous flight auger cast piles, driven piles: timber and concrete piles, geosynthetics, geotextiles / geogrids, geocells).

Improving the foundation of track embankments using piles made from reinforced concrete, steel, gravel, timber, sand column, and stone column, etc., can be very helpful in mitigating transition zone problems by increasing track stiffness and reducing settlement on the softer side of the transition [88, 137, 146, 168, 179]. However, this solution may not be cost effective because it depends mainly on the length of the piles and the material used [133]. The effectiveness of piles at a transition zone can be enhanced by arranging them in a proper pattern, and by varying their lengths depending on whether the structures are soft or stiff, as shown in [86]; the length of any transition zone can be optimised
by these arrangements.

Other mitigation measures may include lightweight fills [101, 130, 146], precast prestressed crossings (PPC) [113], increasing the length of the stiffness transition zone [137] and improving the treatment of subbase materials [139]. The use of lightweight fills (expanded polystyrene, geofoam lightweight concrete or aggregate, among others) at transition zones (i.e. bridge approaches) reduces the dead weight (self-weight load) of embankments, which further increases their stability and reduces track settlement [101, 130, 146]. Although this approach has been widely used for the approaches to highway bridges, it can also be used for railways provided that the selected material is suitable (i.e. high stiffness, strength, compressibility, etc.) [130]. Similarly, precast prestressed crossings (PPC), which are approximately 1m long concrete blocks with larger sleepers or rubber sleepers at each ends towards the ballast track, have also been used for level crossings [113].

For a smooth and gradual transition, more than one mitigation approach can be used to improve track performance [137]. For example, the cost of maintenance has been reduced considerably using sleepers of varying lengths, and transition slabs [133]. A combination of auxiliary rail, pads with varying stiffness, and geo-grids have been used by Kang et al. [81], and no abnormal response was observed. Similarly, longer rubber sleepers result in a larger base plane which, through the fastening system, also helps to avoid loose sleepers; this has proven to be the best countermeasure against differential settlement used by Namura and Suzuki [113].

Some countermeasures are better at fulfilling the desired function while others are either partly effective or completely unsuccessful; for example, according to Read and Li [133], pads under rails and slab is the best way to reduce structural stiffness, whereas Seara and Correia [142] and Read and Li [133] indicate that longer sleepers with a reduced spacing in the transition zones do not increase structural stiffness. However, a Hot Mix Asphalt (HMA) layer definitely improves the load-bearing capacity and reduces the stress in subgrade [142], but it does not improve the behaviour of ballast on rigid pavements [83, 133].

5. Research into track transition zones

Along with the ever-increasing demand for high-speed passenger and heavy haul freight trains goes the increasing need to design transition zones that deliver smooth and gradual changes of track stiffness at track junctions. In this regard, rail tracks have undergone dynamic analysis to better understand the response of track at transition zones under moving loads, as well as the associated track problems and possible countermeasures. This dynamic analysis of railway track transition zones sets out to understand how traffic loads affect track components in terms of stresses, strains and deformation using established theories on the interaction between vehicle components and the track. These models are powerful enough to predict the performance of track structure as well as making
According to Esveld [39], dynamic analysis is the interaction between an applied load and the structure where the structural components react according to their inherent frequencies (governed by mass elastic properties) to the applied load and large amplifications occur when the frequencies of these structural components become equal to their natural frequencies. With transition zones, the properties of components such as track damping and stiffness, rail modulus and inertia, train loads, etc., will vary in time and space, whereas the effect of train load on track components depends on the type (static, dynamic, cyclic, etc.), and velocity in which it is being applied. Other factors that must also be considered in track dynamic analysis are the mass (providing resistance to geometric changes under applied loading), inertia (proving resistance against velocity), damping (energy absorption) characteristics, stiffness (providing resistance to deflection), and the mechanical and geometric properties of track components [156].

A concise review of ongoing research into the dynamic analysis of tracks at transition zones via laboratory experiments, mathematical and computational modelling, and field investigations is presented and discussed in the following sections.

5.1 Laboratory testing and prototype modelling

A number of laboratory experiments on rail track and track components to investigate the properties and performance of different materials/components under various situations have been carried out worldwide, and an extensive number of outcomes in the field of railway engineering have been published by various researchers [35-37, 63, 69, 73, 75, 80, 99, 116, 118].

However, very little work has been carried out in laboratories to model transition zones due to limitations of size and composition. Momoya et al. [111] performed some laboratory experiments on railway track transitions between ballasted embankments and the concrete box culverts. This model was a 1/5\textsuperscript{th} scale model of a transition onto which a moving load was applied onto rail sleepers by electric-hydraulic actuators to simulate an actual load from a 10-car train with four axles each. The four models tested were (a) without any buffering, (b) with an approach block, (c) with an approach slab, and (d) with a resilient mat. Results were based on track settlement, the hanging sleeper phenomenon, deformation of the ballast layer, and the relationship between the mobilised friction angle and ballast settlement. The conclusion of this extensive laboratory study was that the countermeasures are expected to reduce any large local settlement and an approach block will reduce settlement by almost one half.

Likewise, Namura and Suzuki [113] performed the cyclic loading tests on a 1/5\textsuperscript{th} scale model to evaluate the effectiveness of precast prestressed crossing (PPC). The model represents the transition
between ballast track (consisting of sleeper, ballast and subgrade) and the slab track (consisting of concrete block and subgrade). Train loading was simulated by movement loading device with 15 actuators (nine on ballasted track and six on PPC) considering the loading pattern of single wheel load running cyclically at a speed of 1.2 m/s. Track dynamic analysis was carried out to investigate the effect of rail fastening system on the reaction forces and vertical displacements. With no rail fastening system, loose sleepers were observed on the ballasted track side soon after track maintenance by tamping (i.e. after 1000 passes of movement loading), however, no such loose sleepers were occurred even after 20000 passes of movement loading in case of rail fastening provision. It was concluded that the rail fastening system provides a better alignment to longitudinal irregularities, which minimises the disturbance of ballast components caused by tamping work.

5.2 Mathematical modelling approach

There have been a number of researchers used the theory of beams on elastic foundations (BOEF) to model railway tracks and transitions [28, 33, 47, 81, 145, 185]. This theory is based mainly on Euler-Bernoulli beam (rail of infinite length) or Timoshenko beam resting on a Winkler foundation. The mathematical framework for the motion of a track built on a viscoelastic foundation using this theory can be found in [28]. Previous studies [34, 56, 95, 107, 126, 165] used the Euler-Bernoulli to model a transition zone, while some researchers used a Timoshenko beam to consider transverse shear deformation and beam vibration theory [4, 59, 113]. However, after comparing these two conventional approaches for various cases, Czyczula et al. [28] concluded that if either monotonic or moving loads are considered, the results through the Timoshenko beam are almost the same as the Euler- Bernoulli beam. A detailed comparison of deflection of rail beams predicted by different theories subjected to varying train speeds can be found in [28].

The use of BOEF theory to analyses the dynamic response of railway substructure has several limitations. First, a foundation with distributed Winkler springs for soil reactions only gives approximate results if the speed of a moving load (train speed) is less than the critical velocity [166]; second, a Winkler springs foundation assumes there is no deformation of the adjacent soil elements, which does not always represent an actual rail track embankment [169]; third, granular materials (ballast, sub-ballast) under track substructure do not exhibit tension, whereas springs have some tension [169]; fourth, this approach does not consider the interaction between train and track while representing the train loading by a constant moving load [166, 169]. Moreover, the load-deformation response of track is frequently been assumed to be linear [120], whereas a highly non-linear response of ballasted tracks under dynamic loading often occurs, especially in stage-1 (rapid) settlement [29, 65, 66, 70, 140].

In spite of lacking of a comprehensive model to predict the actual response of rail track while
considering the complex nature of track substructure [156], the BOEF model has been used extensively in practice, albeit using an analytical approach to solve the dynamic response of tracks at transition zones is limited because the problem of sudden changes in track stiffness is complex. Walker and Indraratna [169] recently used a semi-analytical approach to solve the moving loads at transition zones; this model considers a Euler-Bernoulli beam (pinned) of finite length on viscoelastic foundations and the approach considers the spatial variation of rail characteristics (i.e. damping, mass, bending stiffness and cross-sectional area) as well as track stiffness and damping. The governing equation for a moving load used in this study is given as:

$$\frac{EI\partial^4w}{\partial x^4} + \frac{\rho A\partial^2w}{\partial t^2} + c \frac{\partial w}{\partial t} + kw = -F\delta(x - vt)$$  \hspace{1cm} (1)

where, $E =$ modulus of elasticity (N/m$^2$), $I =$ second moment of area (m$^4$), $\rho =$ rail density (kg/m$^3$), $A =$ cross sectional area (m$^2$), $k =$ track stiffness (kN/m), $c =$ damping (Ns/m$^2$), $w =$ track deflection (mm), $F =$ dynamic load (kN), $v =$ train speed (m/s) and $\delta =$ Dirac-delta function

Equation 1 is solved for normalised track displacement ($\bar{w}$), using the semi-analytical spectral Galerkin method that assumes ‘$n$’ terms truncated series. A general transition from soft (low stiffness value, $k_1$) to stiff track (higher stiffness value, $k_2$) over a given transition length ($L_t$) is investigated under single and multiple moving loads. The differential settlements are simulated by comparing the deflections on each side of the transition. The deflection amplification factor (DAF$_{wrt1}$), is calculated using Eq.2, which considers various speed ratios ($\alpha = v/v_{cr}$), damping ratios ($\beta = c/c_{cr}$), and stiffness ratios ($k_1/k_2$); these ratios can be determined as:

$$DAF_{wrt1} = \left(\frac{k_1}{k_2}\right)^{3/4} \times DAF_2$$ \hspace{1cm} (2)

$$\frac{\alpha_2}{\alpha_1} = \left(\frac{k_1}{k_2}\right)^{1/4}$$ \hspace{1cm} (3)

$$\frac{\beta_2}{\beta_1} = \left(\frac{k_1}{k_2}\right)^{1/2}$$ \hspace{1cm} (4)

To find an optimum length for a transition zone, Walker and Indraratna [169] examined the beam deflection of various transition length ratios with a characteristic length ($L_c$), as described in Eq.5; they concluded that the minimum transition length should be 8-10 times of system’s characteristic length ($L_c$) to avoid stiffness transition deflection spikes. One of the main outcomes of this study is how valid the conventional theory of BOEF is for long transitions with gradual changes in stiffness; the conclusion is that the dynamic response of transition zones can be described adequately with this theory. Furthermore, the presented model is validated by comparing the results of maximum
displacement at the transition zone with field data, as shown in Figure 12; this took place using a case study by simulating the actual variations, further details can be found in [169]. This figure shows that as the distance from the abutment increases, track stiffness decreases and peak displacement increases. This figure also indicates the abrupt variation in track stiffness and displacement at the junction between the bridge approach and the abutment.

\[ L_c = \sqrt{\frac{4EI}{k}} \]  (5)

Mass spring-dashpot models have been used in previous studies to model a multilayer track system [11, 32, 56, 149]. Sometimes these models are simplified by using over-all track stiffness and damping values by combining the values of all structural components and layers, as suggested in [10]. To understand the nature of the transition between a ballast track and a slab track, a simplified mass spring-dashpot model can be developed, as shown in Figure 13. In this model, the total stiffness of the track is represented by the “spring” with spring constants \( k_b \) and \( k_s \) for ballast track and slab track respectively, while damping of the track structure is represented as dashpots. However, to study the effect of every individual track supporting layer, full layered models can be used because they simulate all the supporting layers and also incorporate the additional elements for USPs, elastic and soft pads, geogrid, geotextile, and polystyrene, among others [39, 42, 90, 126, 139].

Varandas [166] presented a linear mathematical model for the response analysis of inhomogeneous foundation using the two-layer mass spring-dashpot system shown in Figure 14. In this model two Euler-Bernoulli beams, one for the rail and another for the concrete slab are linked together by visco-elastic elements to represent rail pads of fill material. The whole system is supported by soil represented by a visco-elastic foundation. The stiffness of the upper and lower visco-elastic elements is assumed to change abruptly at the \( x=0 \) section from \( K_{11} \) and \( K_{21} \) to \( K_{12} \) and \( K_{22} \) respectively. The vertical displacements are defined as \( U_0(x,t) \), as mentioned in Figure 14, and are calculated using the dynamic equilibrium equations for forced vibration of beams by considering the load is acting on the left side. The governing equations for the model are given in Appendix A. This mathematical model considers an inhomogeneous foundation so it can be applied to rail transitions for a dynamic response analyse under train moving loads; it can also be utilised for the design of transition zones but it would require extensive calculations that may not be solved analytically.

5.3 Computational modelling approach

The numerical modelling approach is increasingly being used to simulate rail using fully calibrated numerical models of track transitions under various loading and boundary conditions [12, 56, 184]. Various countermeasures have been modelled and analysed using FEM (finite element method) or
DEM (discrete element method). In addition to the extensive use of FEM in rail track modelling (Table 1), the DEM has also been increasingly used to study the micromechanical behaviour of railway ballast because it can capture the discrete nature of particulate materials [14, 132, 161]. Furthermore, it is capable of examining the mechanical behaviour of granular assembly of arbitrarily shaped discrete particles under quasi-static and dynamic conditions [61, 105, 117, 192]. A comparison of several numerical and analytical models used to evaluate the dynamic response of railway tracks under moving train loads can be seen in [64, 156]. Numerical modelling through proper calibration and field validation is an appropriate tool to predict the dynamic response of any transition zone with various design options, remedial measures, train speeds and loads. A detailed comparison of several computational models of transition zones is given in Table 1.

One of the benefits of computational modelling is that a single model can be utilised to work out multiple design options for a specific transition. For example, Sañudo et al. [136] placed sleepers at six different locations using 2D FEM modelling and investigated the dynamic response of track in each case to optimise the overall design. Likewise, in [174], a 3D FE model is used to analyse the dynamic response of track transitions by considering the differential settlement, stiffness variation, vehicle dynamics and hanging sleepers. Similarly using a 3D FE model, various subgrade fillings have been investigated to explore the economic filling materials for a high-speed railway transition zone [59].

Another use of numerical modelling is to investigate the effect that complex site situations can have on the dynamic response of track. These situations may include large-scale excavation close to a track transition, variations in the moisture of track substructure, and ballast fouling, among others [116, 150, 175]. Likewise, numerical modelling can be used to investigate the dynamic response of track at various levels and locations of track components at any time. Mishra et al. [107] observed deformation at various levels using a 3D FE model to fully calibrate it with field values measured with multi-depth deflectometers.

The type of model and the modelling software/program influences how reliable and accurate is the dynamic response analysis of transition zones. Various selection parameters include, (i) the type of analysis required (static or dynamic), (ii) the inclusion of non-linearity and plasticity of material, (iii) the calculation time, and (iv) the expected outcomes. Previous studies show the use of two types of finite element programs: (i) vehicle modelling packages, and (ii) track modelling packages. The vehicle modelling software packages concentrate more on vehicle dynamics while over simplifying the modelling of ballast and subgrade materials, whereas track modelling software packages mostly deal with a substructure model that over simplifies the vehicle model [120]. At transition zones, even though the main variation is in the structural properties of track, utilising the model while considering the vehicle and track responses would enable a better understanding of the dynamic response of track.
Likewise, selecting a vehicle model which considers various suspended, semi-suspended, and non-suspended loads can also help to obtain a true dynamic response of track structure. Hunt and Winkler [62] used four vehicle models with, (i) axle load only, (ii) axle and bogie, (iii) axle, bogie, and vehicle body, and (iv) two axles and bogies with the same static axle loads and found similar settlement results from every model; they then conclude there is no effect on the settlement growth rate, even for closely spaced axles. Paixão et al. [125] found a similar track response for a train with different cars in terms of the wheel/rail interaction and vertical displacements; they conclude that a 2-car model can be just as practical as a full train model, and therefore very useful at reducing the calculation time. However, a simplified (one-bogie) vehicle model is not always appropriate for considering responses such as the pitching motion of a vehicle [4].

The choice of models depends on the complexity of the analysis and the required precision. 2D models are incapable of modelling the train load distribution in a longitudinal direction, so 2D plane strain model with continuous support has been considered for a transversal track profile instead of real field conditions with the discrete support of rails by sleepers. However, a 3D model can overcome these limitations [147], which is why Galvín et al. [53] suggested using 3D models that include track non-linearity to obtain an accurate response of track transitions under moving loads.

Paixão et al. [125] used a 2D numerical model to examine how backfill settlement affects train and track interaction by measuring the wheel-rail contact force in the transition zone. They used four scenarios of maximum backfill settlement, 1mm, 5mm, 10mm and 15mm, and found large interacting forces in each case due to the negativity of existing settlement (hanging sleeper phenomenon). Similar results can also be found in various other studies [102, 191, 193].

Numerical modelling can be utilised to investigate the response of track due to various train speeds and loads. Coelho et al. [25] concluded that train speeds up to a certain limit (160 km/h for that specific case) had limited impact on the track but as the speed became critical (180km/h) the response of track became higher due to resonance. Likewise, in more recent research, Labrado Palomo et al. [86] investigated the effect of train speeds on four different kinds of approaches at embankment-bridge transition using a 3D finite element model. The characteristic parameters of ballast, sub-ballast and soil were optimised through model calibration with field results, and then the model was successfully validated. It is found that the peak and average particle velocities for vehicle speeds of 100 km/h are higher than at 160 km/h and 220 km/h, possibly due to a match between train speed and the critical speed of the entire system. However, Heydari-Noghabi et al. [56] found an increasing trend of track displacements for various sections of track as the train speed and loads increase, as shown in Figure 15. Note here that as the train speed (Figure 15a) and loads (Figure 15b) increase,
track displacement also increases. Moreover, ballasted track has a higher displacement than a slab track. Figure 15 also shows that the auxiliary rails help to smooth the differential settlement at the transition zone.

Numerical modelling can also analyse how the direction of train movement will affect the dynamic response of track in terms of enhanced train-track interactions at transition zones. Many authors believe that trains passing through a transition zone from soft to stiff medium such as embankment to bridge are the worst-case scenarios [113, 125, 174]. This could be due to trains moving from a deformable structure to a non-deformable structure (i.e. concrete bridge) which enhances the impact load. However, Chen [22] found more settlement when moving from a stiff to soft transition zone because the boundary conditions for his model could be case specific. Despite this, some authors suggest that the effect of train direction on the dynamic behaviour of track is minor, which is the case when the quality of tracks is high and there are no sharp variations in the track support conditions [4, 5, 124]

A 3D finite element model has been used by Hu et al. [59] to evaluate how effective different filling materials are for a wedge-shaped backfill at a tunnel-culvert transition zone. Three different materials are used; (i) graded gravel with 5% cement, (ii) graded gravel with no cement (c=160kPa, $\phi=39.5$), and (iii) well graded coarse grained soil with less than 30% of fine grained soil (c =200kPa, $\phi=41.8$); their properties were calculated through laboratory (for new materials) and field testings (for in-situ material). In every case, maximum deflection occurs under maximum allowed values, however lower wheel loads are used in this study.

In [7-9] the train and track interaction has been investigated by applying a 3D finite element model to the track transition mechanism. This model incorporates variations in stiffness and considers the non-linear behaviour of ballast and subgrade. The conclusion is that simple variations in stiffness at track transitions is not the primary cause of transition problems, it is the soft subgrade, voids, and other faults at transition zones that increase the interaction forces as train speeds increase that cause passenger discomfort. It is therefore suggested that difference in deflection at the junction of two different tracks over a 4-10 m long transition will lead to a smooth transition.

To study the dynamic response of bump at bridge approaches, a detailed investigation using a 3D finite element model that incorporates train and track structure/substructure is given in [120]. The response of this track is observed by varying the sizes of the bumps and dips, the thickness of the ballast, the sleeper material, train speed direction, and the type of abutment and length of the sleepers. It is found that the enhanced load impact and ballast/subgrade pressure due to variations in the track modulus cause dips and bumps to develop; this further increases the dynamic response of track at bridge transitions.
5.4 Field measurements

A number of field investigations have been carried out to evaluate the effectiveness of the approaches used to control differential settlement at the transition zones; these studies took place mainly in the USA, Europe, Japan, and China. Various instruments are used to measure/investigate the response of rail track at transition zones in real time scenarios, they include multi-depth deflectometers (MDD), uniaxial and triaxial accelerometers, strain gauges, pressure cells, settlement pegs, video gauge systems, position sensitive devices, geophones, inclinometers, linear variable displacement transducers (LVDTs), among others [15, 26, 71, 126, 153, 155, 174]. Furthermore, the structural health monitoring of rail tracks and transition zones is being carried out with the help of conventional data measuring coaches and advanced techniques including digital image correlation (DIC) device and satellite synthetic aperture radar (InSAR) system which is developed by Wang et al. [170].

Stone blowing is a process of adjusting the track geometry by adding the crushed rock to ballast surface under the lifted sleeper. It is a relatively new method involving less damage to sleepers as compared to the tamping process where adjustment is achieved by ballast rearrangement to fill the voids under the lifted sleeper [153]. The effectiveness of stone blowing instead of tamping was investigated by Boler et al. [15] where comparison was made by analysing the performance of track before and after stone blowing; the data shows that stone blowing led to an almost 60% reduction in transient peak displacement, and moreover, the vertical acceleration and gaps at the sleeper-ballast interface also decreased due to stone blowing. The results obtained by vertical space curves through track geometry car measurements also indicate the increased effectiveness and longevity of this remedial measure.

Paixão et al. [126] concluded that Under Sleeper Pads, (USPs) at the transition zones are effective based on the passage of 40 different types of trains passing the fully instrumented zone. It is noted that USPs will reduce the track stiffness values by a considerable amount. Fortunato et al. [44] used a wedge shaped approach at the transition zone and concluded that a gradual transition of vertical stiffness can be achieved with this approach, however, passenger trains at higher speeds cause more acceleration at sleepers than heavy freight at lower speeds. They also presented the various characteristics of wedge shaped countermeasures that are commonly used at transition zones in several countries around the world.

Stark and Wynn [155] presented a report on the use of geosynthetic reinforcement systems in the railway transition zones to mitigate the differential settlement at these locations. They concluded from this ongoing research that ballast reinforcement in transition approaches with a geoweb underlay helps to mitigate transition problems by providing enhanced ballast confinement and improved load distribution. They also showed there is a large reduction in cost and installation time when geoweb
underlay is used; in fact this research shows that geosynthetic reinforcement will help to reduce
differential settlement because it can increase (when used at approaches) and decrease (when used
under the bridge abutment) the stiffness values.

Coelho et al. [26] presented the results of field measurements for a track crossing with a concrete
culvert by showing that the design for the transition was not optimal. The box culvert is almost 1.5 m
deep from the track and is made from a sand embankment over soft soils. There are 4m long by
300mm thick approach slabs on both sides of the culvert above which sand is placed up to the ballast
under the actual track. The vertical displacement, axle load, and average track stiffness are measured
by geophones (mounted on top of wooden sleepers), uniaxial accelerometers (within ballast), triaxial
accelerometers (within soil below track), strain gauges and a high-speed camera. The hanging
sleepers in the transition zone that are the result of long term track differential settlements are the
main sources of the track displacement that caused increased impact loading and accelerated track
degradation.

A summary of the most recent field investigations of the transition zones in terms of the project
description, transition types, countermeasure used and the outcome of the overall research, is given
in Table 2.

6. Recommendations for improved track design

Despite the effort that has gone into studying the performance of tracks at transition zones using
advanced modelling techniques, an optimal solution to transition related problems is still not fully
understood [79, 137], hence the need to find an effective and low cost solution (to eradicate/minimise
the problems), with minimum disruption to traffic and a longer life [64]. The main aim of designing
these transition zones is to maintain track quality while reducing the maintenance cycles and costs
[137]. In order to use computational models properly for predicting the true dynamic track
performance, the model parameters require realistic calibration either using large-scale laboratory
simulations or instrumented field trials.

Vertical track stiffness at transition zones is mainly influenced by the type of materials used in the
embankment, and its slope and height in the transition zone. Natural ground beneath an embankment
also affects the stiffness of track depending on the material used in the embankment, so it should be
replaced if it is highly compressible [49-51]. Therefore, to design a transition zone efficiently, a
proper selection of materials along with the shape and height of the embankment should be
considered, as should a proper consideration of natural ground characteristics.

While the inconsistent dynamic response of ballast particles with respect to the point of load
application indicates its particulate nature [107], and while finite element modelling (FEM) cannot
model discrete particles due to continuum based solutions, discrete element modelling (DEM) can
model irregular-shaped particles, the angularity of granular material, and particle breakage; therefore
particle to particle contact for wheel load interactions can be considered properly in dynamic track
modelling using DEM. In particular, the DEM application may be most appropriate where particle
degradation is exacerbated due to impact, and while these track sections can be modelled separately
using DEM, most part of the track can generally be modelled as a continuum using FEM. A FEM-
DEM coupled model could be the best approach to investigate the ballasted rail track dynamic as
suggested by Nishiura et al. [121] who developed a sleeper model by DEM for viscoelastic multi-
body dynamics and coupled it with rail model (FEM) providing greater insight into the dynamic
response of ballasted railway tracks under impact loading.

Ideally, ballast assemblies should be tested in a prototype scale to determine how enhanced dynamic
loadings at transition zones will affect the deformation and degradation of ballast; this is because
testing smaller particles with small equipment will affect the internal angle of friction (shearing
resistance) of the granular assembly, and hence the rate of volumetric strain during the shearing
process. A large-scale triaxial testing rig (300 mm diameter by 600 mm high) has been designed and
built at the University of Wollongong (Figure 16a); it is custom made to minimise boundary effects,
so it has been widely used to evaluate the deformation and degradation of ballast with reference to
the origin, size, and shape of aggregates used in Australian tracks. Key factors affecting ballast
degradation were found to be as loading characteristics (i.e. monotonic, cyclic), frequency and
confining pressure. Details of the components of this apparatus and its measuring techniques can be
found elsewhere [67, 74, 77, 87, 151]. A similar apparatus can also be found in [27] that can be used
to investigate the resilient behavior of track ballast with particle size up to 63mm.

A large-scale process simulation testing apparatus (PSTA) has been used to study the response of
ballast track components under realistic cyclic loading (Figure 16b). This PSTA can accommodate
specimens 800 mm long by 600 mm wide by 600 mm high, these dimensions will mimic a typical
unit cell section of Australian standard gauge tracks [13, 68, 80, 115]. The PSTA can also apply a
dynamic load up to 100 kN with frequency up to 40 Hz, simulating typical Australian passenger and
heavy haul freight trains traveling up to 200 km/h. Large-scale constant normal stiffness (CNS) direct
shear tests (Figure 16c) have been designed to study the interface between ballast-rubber
mats/geosynthetics and to determine the internal friction angle of ballast. The recently funded
National Facility for Cyclic Testing High-Speed Rail (FCTHSR; Figure 16d) is now being
commissioned with double axle loading to mimic heavy haul operations. With axle loads up to 40-
tonnes and a frequency range from 5-40 Hz (i.e. speeds of 55-220 km/h), the FCTHSR will
accommodate a range of cyclic loading patterns to evaluate the actual performance of ballast under
different structural and geotechnical conditions especially for transition zones.
A large-scale permeameter has been designed to measure the hydraulic conductivity of ballast contaminated with fouling materials such as coal and subgrade mud [69, 160]. This chamber will accommodate ballast specimens of 500mm in diameter by 300-500mm high (Figure 16e). A full-scale model track (dimensions: 4.76 m by 3.48 m by 0.79 m) has been built at UOW to study the behaviour of fouled ballast (Figure 16f). This model track is used to determine how ballast fouling can influence ground penetrating radar (GPR) data, while capturing the moisture content and loading frequency [157]. These unique and novel testing devices help us to examine and quantify the influence of induced train loading characteristics on the strength, deformation, and degradation of ballast, hence could be utilised for design optimization of transition zones.

Likewise, the CEDEX track box; a full-scale (1:1) testing facility for railway tracks in Spain [104], can be utilized to optimize various maintenance works at transition zones especially for slab track to ballast track transitions. This track box is 21m long, 4m deep and 5m wide and has a capacity to model the full rail track section for various train loads and speeds up to 450km/h considering static and dynamic loading conditions. This facility has been used in the past to study various aspects of rail track performance such as track lateral stability, vertical stiffness, short and long-term settlements, and track dynamic response under high-speeds trains among others, the details of which can be found in [36-38].

The full-scale laboratory testing facility developed by the University of Nottingham [17] for railway track can also be utilized to investigate the performance of railway ballast under impact loading caused by differential settlement at transition zones. This facility involves three sleepers and the cyclic load of 94kN can be applied directly to the sleepers through hydraulic actuators. The permanent settlement as well as transient deflection can be measured in addition to the transient stresses and degree of particle degradation, the results of various tests performed on this apparatus can be found elsewhere [6, 18, 21]. Similarly, the Southampton railway testing facility [2] can also be utilized to study the track dynamic response especially the permanent (plastic) settlement of ballast particles that can cause the differential settlement as suggested by Abadi et al. [1].

To produce a decent design, the difference between the response of track before and after a transition zone is set as low as possible. Various factors found in the literature that affect the structural response of track transitions are summarised in Figure 17; these factors should receive enough attention to achieve the appropriate design of transition zones in terms of selecting different design techniques and approaches. To make a precise model, every individual component must be modelled separately by considering characteristics such as elastic/plastic, linear/non-linear, continuum/particulate (discrete), and their interaction with neighbouring components. These factors can be addressed selecting proper modelling techniques such as 1D, 2D or 3D and commercially available advanced modelling software packages. Note that each model and modelling software has some limitations that
should be considered before designing track transitions.

7. Conclusions

This paper presents a detailed review of rail track transitions, various associated issues and their solutions. After defining the importance of rail transitions, their related problems are discussed, including their causes and effects on railway operations. Numerous mitigations measures to improve the performance of ballasted tracks at transition zones are also discussed, with special reference to their effectiveness. The ongoing research into designing transition zones to minimise the effect of abrupt changes in the structural properties of track is critically reviewed by considering large-scale laboratory testing, mathematical and computational modelling, and field investigations. Recommendations for the performance of the ballasted track at transitions are also presented after reviewing the various design approaches. Following are the conclusions that can be drawn from this review of track transition zones:

- Differential settlement and enhanced dynamic loads are the main problems associated with track transition and are thus responsible for track degradation. The major cause of these problems is the abrupt change in stiffness, which can only be controlled by designing smooth and gradual transition zones.

- Most current design practices are based mainly on empiricism established through trial and error. These methods can be unreliable if the empirical parameters are calibrated only to local subgrade and ballast properties, and should not be extended to any track section without exercising caution for different soil characteristics and dynamic loading conditions. Transition zones should be designed to cope with the required variations in stiffness and possible initial settlement, which may vary depending on the case. If the variations in stiffness are known, the length of the transition zone and the type of materials can be established to provide a smooth and gradual variation in track stiffness at the junction.

- One of many reasons for not having a precise and economical design of transition zones is because the problem is complex due to the interaction of several structures and structural components. Since conventional rail track structure consists of various structural components, it is a composite structure, but in transition zones, this complexity is enhanced due to the sudden variations in the structural properties of track. This makes prediction of the dynamic performance of the overall structure a challenging task because every component behaves differently under various loading conditions. Furthermore, the interaction between these components (vehicle-track-structure) makes the model more complex.

- There is a lack of effective computational model to study the dynamic response of transition zones due to the complexity of the problem. Similarly, implementing semi-analytical
approaches based on the transformation of time and frequency is difficult owing to the sub-
structural inhomogeneity of track. Various mitigation measures have been utilised but without
any theoretical reasoning, and therefore they are not overcoming the need for frequent
maintenance.

- It is observed that deformation in most models is very small and is in the elastic range, whereas
  in actual field investigations, high deformation occurs more frequently with the passage of
time. The main reason for these discrepancies could be the assumed linear and elastic nature
of the structural components, but the ballast, sub-ballast and subgrade layers do not behave
elastically or in a linear fashion under repeated train loads; this leads to plastic deformation
and permanent settlement i.e. bumps/dips. This plastic deformation has been overlooked in
most transition models, which is another source of error for predicting the dynamic response
of track.
8. Appendix A

This appendix provides the governing equations used by [166] to solve the linear mathematical model for the response analysis of inhomogeneous foundation using two-layer mass spring-dashpot model.

The governing equation for \( x < 0 \) is as:

\[
\frac{E_1 I_1 \partial^4 u_{11}}{\partial x^4} + m_1 \frac{\partial^2 u_{11}}{\partial t^2} + c_1 \left( \frac{\partial u_{11}}{\partial t} - \frac{\partial u_{21}}{\partial t} \right) + k_{11}(u_{11} - u_{21}) = -F \delta(x - vt)
\]

and for \( x > 0 \) is as:

\[
\frac{E_2 I_2 \partial^4 u_{21}}{\partial x^4} + m_2 \frac{\partial^2 u_{21}}{\partial t^2} + c_1 \left( \frac{\partial u_{21}}{\partial t} - \frac{\partial u_{11}}{\partial t} \right) + k_{11}(u_{21} - u_{11}) + k_{21}u_{21} + c_2 \frac{\partial u_{21}}{\partial t} = 0
\]

These equations for vertical displacement in each section are solved by considering the interface conditions at \( x=0 \). The solution for these equations of differential settlements is a complex process that involves many assumptions and the substitution of many variables, it can be found in [163, 166].

9. Acknowledgements

This research was carried out by the Australian Research Council Industrial Transformation Training Centre for Advanced Technologies in Rail Track Infrastructure (IC170100006) and funded by the Australian Government. The authors thank the Australian Rail Track Corporation (ARTC) for their continuous support and cooperation. The authors also appreciate the insightful collaboration and assistance of Australasian Centre for Rail Innovation (ACRI) and Snowy Mountains Engineering Corporation (SMEC), in particular the comments and thoughtful advice provided for the current study.
### Table 1: Comparison of computational modelling approaches and summarised key research findings on track transition zones

<table>
<thead>
<tr>
<th>Reference</th>
<th>Model type</th>
<th>Numerical method</th>
<th>Foundation type</th>
<th>Model length (m)</th>
<th>Transition length (m)</th>
<th>Model validation</th>
<th>Model calibration</th>
<th>Transit speed (km/h)</th>
<th>Axle load (kN)</th>
<th>Parameters studied</th>
<th>Train direction</th>
<th>Mitigation measures</th>
<th>Innovations / Findings</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggestam and Nielsen, 2019 (Sweden) [6]</td>
<td>2D</td>
<td>FEM</td>
<td>-</td>
<td>Dyn.</td>
<td>TS</td>
<td>LE</td>
<td>60</td>
<td>23.4</td>
<td>-</td>
<td>Yes</td>
<td>BTS</td>
<td>250 - 350</td>
<td>170</td>
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<tr>
<td>Hu et al., 2019 (China) [59]</td>
<td>3D</td>
<td>FEM</td>
<td>ANSYS</td>
<td>Dyn.</td>
<td>TS</td>
<td>EP</td>
<td>52.2</td>
<td>46.2</td>
<td>No</td>
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<td>BTS</td>
<td>350</td>
<td>140</td>
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<tr>
<td>Esmaeili et al., 2018 (Iran) [34]</td>
<td>2D</td>
<td>FEM</td>
<td>ANSYS</td>
<td>Dyn.</td>
<td>EB</td>
<td>LE</td>
<td>42.2</td>
<td>18</td>
<td>No</td>
<td>Yes</td>
<td>BTS</td>
<td>120 - 340</td>
<td>160, 200</td>
</tr>
<tr>
<td>Koch et al., 2018 (Hungary) [85]</td>
<td>3D</td>
<td>FEM</td>
<td>PLAXIS</td>
<td>Dyn.</td>
<td>RB</td>
<td>LE</td>
<td>96</td>
<td>18</td>
<td>-</td>
<td>-</td>
<td>BTS</td>
<td>80, 250</td>
<td>125</td>
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<tr>
<td>Labrado Palomo et al., 2018 (Spain) [86]</td>
<td>3D</td>
<td>FEM</td>
<td>ANSYS</td>
<td>Dyn.</td>
<td>SB</td>
<td>LE</td>
<td>54</td>
<td>36</td>
<td>Yes</td>
<td>Yes</td>
<td>BTS</td>
<td>50 - 300</td>
<td>24</td>
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<tr>
<td>Paixão et al., 2018 (Portugal) [126]</td>
<td>3D</td>
<td>FEM</td>
<td>Pegasus/ MATLAB</td>
<td>Dyn.</td>
<td>EB</td>
<td>NLVE</td>
<td>75</td>
<td>17.4</td>
<td>Yes</td>
<td>Yes</td>
<td>BTS</td>
<td>220</td>
<td>250</td>
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<tr>
<td>Wang and Markine, 2018a (Netherlands) [171]</td>
<td>3D</td>
<td>FEM</td>
<td>LS-DYNA</td>
<td>Dyn.</td>
<td>HL</td>
<td>NLE</td>
<td>120</td>
<td>Varies</td>
<td>-</td>
<td>Yes</td>
<td>BTS</td>
<td>200</td>
<td>142</td>
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<tr>
<td>Wang and Markine, 2018b (Netherlands) [174]</td>
<td>3D</td>
<td>FEM</td>
<td>LS-DYNA</td>
<td>Dyn.</td>
<td>HL</td>
<td>LE</td>
<td>120</td>
<td>Varies</td>
<td>-</td>
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<td>BTS</td>
<td>72 - 288</td>
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<td>Heydari-Noghabi et al., 2017 (Iran) [56]</td>
<td>3D</td>
<td>FEM</td>
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<td>EB</td>
<td>52.2</td>
<td>18</td>
<td>Yes</td>
<td>Yes</td>
<td>BTS</td>
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<td>180 - 250</td>
<td>RD, CF</td>
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<tr>
<td>Sañudo et al., 2017 (Spain) [136]</td>
<td>2D</td>
<td>FEM</td>
<td>ANSYS</td>
<td>Dyn.</td>
<td>-</td>
<td>LE</td>
<td>200</td>
<td>Varies</td>
<td>-</td>
<td>-</td>
<td>BTS</td>
<td>300</td>
<td>VD, BS, TA</td>
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<tr>
<td>Wang et al., 2017 (Netherlands) [175]</td>
<td>3D</td>
<td>FEM</td>
<td>LS-DYNA</td>
<td>Dyn.</td>
<td>HL</td>
<td>LE</td>
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<td>BTS</td>
<td>144</td>
<td>142</td>
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<td>Paixão et al., 2016 (Portugal) [125]</td>
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<td>FEM</td>
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<td>RB</td>
<td>NLE</td>
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<td>Yes</td>
<td>BTS</td>
<td>220</td>
<td>132</td>
<td>VD, CF, TA</td>
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<td>Reference</td>
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<td>Numerical method</td>
<td>Software</td>
<td>Analysis type</td>
<td>Beam type</td>
<td>Foundation type</td>
<td>Model length (m)</td>
<td>Transition length (m)</td>
<td>Model calibration</td>
<td>Model validation</td>
<td>Transition type</td>
<td>Train speed (km/h)</td>
<td>Axle load (kn)</td>
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</tr>
<tr>
<td>Varandas et al., 2016 (Portugal)</td>
<td>3D</td>
<td>FEM</td>
<td>Pegasus</td>
<td>Dynamic</td>
<td>EB</td>
<td>NLE</td>
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<td>Yes</td>
<td>BTS ~ BTCc</td>
<td>130</td>
<td>174</td>
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<tr>
<td>Real et al., 2016 (Spain)</td>
<td>3D</td>
<td>FEM</td>
<td>ANSYS</td>
<td>Static &amp; Dynamic</td>
<td>SB</td>
<td>LE</td>
<td>60</td>
<td>20</td>
<td>Yes</td>
<td>Yes</td>
<td>BTS ~ BsTAS, BsTCD</td>
<td>35 90</td>
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<td>ANSYS</td>
<td>Dynamic</td>
<td>RB</td>
<td>LE</td>
<td>80</td>
<td>7.5</td>
<td>-</td>
<td>-</td>
<td>BTS ~ BsTCD</td>
<td>300 180</td>
<td>VD, TA</td>
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<td>FEM, DEM</td>
<td>GEOTRACK</td>
<td>Dynamic</td>
<td>E</td>
<td>LE</td>
<td>2.22</td>
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<td>Yes</td>
<td>Yes</td>
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<td>Chen, 2013 (UK)</td>
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<td>DEM</td>
<td>PFC(3D)</td>
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<td>-</td>
<td>LE</td>
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<td>25-380</td>
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<td>LE</td>
<td>60</td>
<td>4-10</td>
<td>-</td>
<td>-</td>
<td>BTS ~ BTCd</td>
<td>180-250</td>
<td>170</td>
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<tr>
<td>Gallego et al., 2011 (Spain)</td>
<td>3D</td>
<td>FEM</td>
<td>ANSYS</td>
<td>Static</td>
<td>SB</td>
<td>EP</td>
<td>7.2</td>
<td>4.8</td>
<td>-</td>
<td>-</td>
<td>BTS ~ BTCd</td>
<td>300</td>
<td>180</td>
</tr>
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<td>Coelho et al., 2011 (Netherlands)</td>
<td>3D</td>
<td>FEM</td>
<td>-</td>
<td>Dynamic</td>
<td>RB</td>
<td>20</td>
<td>5.0</td>
<td>-</td>
<td>Yes</td>
<td>Yes</td>
<td>BTS ~ BTCc</td>
<td>96-200</td>
<td>124-193</td>
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<td>Varandas et al., 2011 (Portugal)</td>
<td>1D</td>
<td>-</td>
<td>-</td>
<td>Dynamic</td>
<td>EB</td>
<td>NLE</td>
<td>60</td>
<td>5.0</td>
<td>-</td>
<td>-</td>
<td>BTS ~ BTCc</td>
<td>120-130</td>
<td>108-174</td>
</tr>
<tr>
<td>Galvin et al., 2010 (Spain)</td>
<td>3D</td>
<td>FEM</td>
<td>Q3D</td>
<td>Dynamic</td>
<td>EB</td>
<td>NLE</td>
<td>90</td>
<td>15.4</td>
<td>-</td>
<td>-</td>
<td>BTS ~ BsTCD</td>
<td>298</td>
<td>152</td>
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<tr>
<td>Nicks, 2009 (Texas)</td>
<td>3D</td>
<td>FEM</td>
<td>LS-Dyna</td>
<td>Dynamic</td>
<td>EB</td>
<td>LE</td>
<td>16</td>
<td>1.6-8.4</td>
<td>No</td>
<td>Yes</td>
<td>BTS ~ BTCd, BTS ~ BsTCD</td>
<td>32-161</td>
<td>292</td>
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</table>
Table 1 (continued)

| Reference                                      | Model type | Numerical Method | Software | Analysis type | Beam type | Foundation type | Model length (m) | Transition length (m) | Model calibration | Model validation | Train direction | Mitigation measures | Innovations / Findings                                                                 |
|------------------------------------------------|------------|------------------|----------|---------------|-----------|-----------------|------------------|-----------------------|-------------------|-------------------|------------------|------------------|-------------------------------------|---------------------------------------------------------------------------------------|
| Witt, 2008 (Sweden) [178]                       | 3D         | FEM              | LS-DYNA  | Dyn.          | RB        | LE              | 18               | -                     | -                 | -                 | -                | -                 |                      | Medium strength USPs (with vertical stiffness of 400 kn/mm²) are better at reducing the wheel/rail contact forces. Softer USPs help to reduce the ballast contact forces |
| Namura and Suzuki, 2007 (Japan) [113]           | 3D         | FEM              | -        | Dyn.          | TB        | LE              | 5                | -                     | -                 | -                 | -                | -                 |                      | Ballast settlement increased with the use of longer sleepers (more) and the resilient sleepers (less), length of approach track should be more than 22 m. Resilient sleepers are best at reducing ballast vibration |
| Read and Li, 2006 (Colorado) [133]              | 2D         | FEM              | GEOTRACK | QS            | SB        | LE              | 115              | 0                     | -                 | -                 | -                | -                 |                      | Concrete slab is the best approach, followed by HMA, and then additional rails. Longer and wider sleepers at reduced spacing have an insignificant effect |
| Li and Davis, 2005 (Colorado) [95]              | 3D         | FEM              | NUCARS   | Dyn.          | EB        | LE              | 30               | 30                    | -                 | -                 | -                | -                 |                      | Variations in stiffness lead increase the variations in dynamic load and wheel-rail interaction forces |
| Lei and Mao, 2004 (China) [91]                  | 2D         | FEM              | -        | Dyn.          | SB        | LE              | 231              | 20                    | Yes               | -                 | -                | -                 |                      | Variations in vertical stiffness have no direct effect on wheel/rail dynamic interaction forces. Permanent settlement is the main cause of transition related problems. Suggestions for irregularity angle and length of transition zone |
| Hunt and Winkler, 1997 (UK) [62]                | 2D         | DEM              | -        | Dyn.          | SB        | LE              | 10               | 2.0                   | -                 | -                 | -                | -                 |                      | Rate of track settlement mainly depends on the initial settlement (voids under sleepers) owing to accelerated settlement under impulsive loads |

Numerical Method (FEM: Finite Element Method, DEM: Discrete Element Method)
Analysis Type (St: Static, Dyn.: Dynamic, QS: Quasi-static)
Beam Type (TS: Timoshenko, EB: Euler-Bernoulli, HL: Hughes-Liu, SB: Simple Beam, RB: Rectangular Beam),
Transition Type (BTR: Ballast Track on Rock, BTS: Ballast Track on Soil, BTCd: Ballast Track on Concrete deck, BsTR: Ballast-less Track on Rock, BsTS: Ballast-less Track on Soil, BsTc: Ballast-less Track on Concrete deck, BTCc: Ballast Track on Concrete Culvert, BTCv: Ballast Track on Concrete Viaduct, BsTAS: Ballast-less Track on Asphalt Slab),
Train Direction (SoTSt: Soft to Stiff, STSSt: Stiff to Soft),
Table 2: Summarised outcomes of field measurements on track transition zones

<table>
<thead>
<tr>
<th>Author</th>
<th>Description</th>
<th>Transition type</th>
<th>Transition length (m)</th>
<th>Trainspeed (km/h)</th>
<th>Loading (kN)</th>
<th>Train direction</th>
<th>Track type</th>
<th>Parameters studied</th>
<th>Instrumentation</th>
<th>Outcomes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boler et al., 2019 (USA) [15]</td>
<td>Stone blowing</td>
<td>BTS - BCd</td>
<td>177</td>
<td>150</td>
<td>PT</td>
<td>VID, WL, TA, BS</td>
<td>MDD, SG</td>
<td>60% reduction in transient displacements. Reduction in vertical acceleration and in gaps at sleeper-ballast interface, increased effectiveness and longevity</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Paixão et al., 2018 (Portugal) [126]</td>
<td>WSB+USPs</td>
<td>BTS - BTCv</td>
<td>20</td>
<td>220</td>
<td>250</td>
<td>MT</td>
<td>RD, TA, SD</td>
<td>SG, LU, PSD, Acm, LVDT</td>
<td>Variations in the vertical stiffness of various sections along the transition. USPs with t=7mm reduced the stiffness by 30% of embankment but increases by 22% for UGM, USPs with t=10mm reduced the stiffness by 27% for CBM but remains same for CBM and concrete</td>
<td></td>
</tr>
<tr>
<td>Wang and Markine, 2018 (Netherlands) [114]</td>
<td>Approach slab</td>
<td>BTS - BTCc</td>
<td>4</td>
<td>65-106</td>
<td>76-142</td>
<td>Both</td>
<td>PT</td>
<td>VID, WL, TS</td>
<td>Gp, Acm, VC, SG</td>
<td>Large differences in displacement and increased wheel loads show the presence of hanging sleepers. Non symmetric response and non-uniform distribution of track displacements indicates inefficiency at the transition zone. Pivoting about the culvert generates enhanced impact loading</td>
</tr>
<tr>
<td>Žuda Coelho et al., 2018 (Portugal) [28]</td>
<td>Reinforcement (Geoweb, HMA, Grotul)</td>
<td>BTS - BTCd</td>
<td>15</td>
<td>30</td>
<td>MGT</td>
<td>FT</td>
<td>VID, TA</td>
<td>VC, Acm</td>
<td>Geosynthetic reinforced transitions are less expensive, perform well for freight loads and reduce differential vertical displacements. Geoweb can be used as an alternative to HMA. With Geosynthetic reinforcement stiffness increases (in approaches) and decreases (while used under bridge abutments)</td>
<td></td>
</tr>
<tr>
<td>Wilk et al., 2016 (USA) [177]</td>
<td>Sleeper support effect, BCW, HMA</td>
<td>BTS - BsTCd</td>
<td>8.2</td>
<td>16</td>
<td>40</td>
<td>177</td>
<td>Both</td>
<td>PT, FT, MT</td>
<td>VID, WL, TA</td>
<td>Acm</td>
</tr>
<tr>
<td>Mishra et al., 2014 (Illinois) [107]</td>
<td>Railpads</td>
<td>BTS - BTCd</td>
<td>177-241</td>
<td>135</td>
<td>Both</td>
<td>MT</td>
<td>VID, WL, TA</td>
<td>MDD, SG</td>
<td>Data obtained for track deformation and corresponding loading through field instrumentation is utilised to calibrate a 3D track dynamic model</td>
<td></td>
</tr>
<tr>
<td>Fortunato et al., 2013 (Portugal) [44]</td>
<td>Wedge-shaped approach</td>
<td>BTS - BTCd</td>
<td>20</td>
<td>220</td>
<td>66-125</td>
<td>SoStSt</td>
<td>MT</td>
<td>VID</td>
<td>SG, LU, PSD, Acm, LVDT, IT</td>
<td>Passenger trains at higher speed cause more acceleration amplitude at sleepers than heavy freight at lower speeds. Using wedge-shaped approach leads to a gradual transition of vertical stiffness, and settlement seems to stabilise after one year of construction</td>
</tr>
<tr>
<td>Namura and Suzuki, 2007 (Japan) [113]</td>
<td>Railpads, subgrade stabilization, Auxiliary rails, Glauling resin</td>
<td>BTS - BCd</td>
<td>20</td>
<td>100-300</td>
<td>Both</td>
<td>PT</td>
<td>WL, TA</td>
<td>SG, TLV</td>
<td>Variations in loads occur on ballast track at transition zone. Application of rail pads has no effect on track irregularities. Axle load variation at track transition found which increased with increasing speed. More variations in acceleration while travelling from a ballast track to a slab track</td>
<td></td>
</tr>
<tr>
<td>Li and Davis, 2005 (Colorado) [95]</td>
<td>HMA, geocol, Thick ballast, cement stabilised</td>
<td>BTS - BTCd</td>
<td>30</td>
<td>160</td>
<td>178</td>
<td>Both</td>
<td>FT</td>
<td>RD, VID, VS</td>
<td>SE, TLV</td>
<td>More track geometry degradation at bridge approaches. No improvement in track performance, a very high initial settlement rate of 100 to 180 mm in just six months (80 MGT). Rubber mats reduce track stiffness and increase track damping</td>
</tr>
</tbody>
</table>


**Transition Type**: (BTR: Ballast Track on Rock, BTS: Ballast Track on Soil, BTCd: Ballast Track on Concrete deck, BsTR: Ballast-less Track on Rock, BsTS: Ballast-less Track on Soil, BsTCd: Ballast-less Track on Concrete deck, BTCc: Ballast Track on Concrete Culvert, BTCv: Ballast Track on Concrete Viaduct), **Train Direction** (SoStSt: Soft to Stiff, StStSo: Stiff to Soft), **Track Type** (PT: Passenger Track, FT: Freight Track, MT: Mixed Traffic/Track), **Parameter Studied** (VID: Vertical displacement/deflection, TV: Track Velocity, TA: Track Acceleration, BS: Ballast/subgrade Stresses, WL: Wheel Load, TM: Track Modulus, RD: Rail Deflection, RpF: Railpad Force, CF: Contact Forces, BPA: Ballast Particles Acceleration), **Instrumentation** (MDD: Multiphed Deflectometers, SG: Strain gauges, LU: LASER Units, PSD: Position Sensitive Devices, Acm: Accelerometers, LVDT: Linear Variable Differential Transducer, VGS: Video Gauge System, VC: Video Cameras, Gp: Geophones, IT: Inclinometer Tubes, SE: Survey Equipments, TLV: Track Loading Vehicle)
(a) Track transitions at Berry bridge crossing Tannery road, NSW, Australia

(b) Slab track to ballast track transitions indicating alignment error (adopted from [96])

Figure 1: Rail track transitions due to sudden change in substructural components
Figure 2: Rail track transitions due to the change of superstructural components
Figure 3: Variation in rail deflection, railpad force and track acceleration at track transition (data source: [34])
Figure 4: Comparison of differential settlements measured at various track transitions; (a) slab track to ballast track, (b) bridge crossing, (c) bridge approaches and (d) culvert crossing
Figure 5: Schematic diagram of the development of bump/dip at bridge approaches (modified after [43])
Figure 6: Variation in wheel rail interaction forces at track transition
Figure 7: Summarised track transition problems: causes and effects (inspired by [122])

- Variation in structural properties
- Variation in track stiffness
- Variation in track damping
- Initial track irregularities
- Excessive plastic deformation
- Ballast degradation
- Subgrade failure
- Soil water response
- Increased speed
- Higher loads
- Track derailment
- Sleeper cracking
- Jumping ballast
- Track deterioration
- Low frequency oscillations
- High frequency vibrations
- Enhanced vehicle acceleration
- Vehicle degradation
- Increased maintenance costs
- Train delays
- Frequent maintenance
- Speed restriction
- Passenger discomfort
- Mitigation measures
- Track Transition
- Transition Zone design
- Mitigation measures
- Increased maintenance costs
Figure 8: Track stiffness variation from a ballasted track to a slab track at transition zones
Figure 9: Abrupt variation in track modulus/stiffness at various track transitions; (a) at bridge crossings, (b) soft to stiff track transition
Figure 10: Variation in track stiffness for various track types along the railway track (adopted from [30])
Figure 11: Variation in measured ballast stresses at various track transitions
Figure 12: Peak displacement and stiffness distribution at transition zone (data source: [169])
Figure 13: Mass and spring-dashpot models for ballast track to slab track transition
Figure 14: Two layers mass spring-dashpot model for track transition
Figure 15: Rail deflection along the transition zone (a) for 180 kN vehicle load and various speeds, (b) for 200 km/h speed and various vehicle loads (adopted from [56]).
Figure 16: Selected large-scale ballast testing equipment at the University of Wollongong Australia
Figure 17: Summarised important factors for transition zone design considerations

- **External factors**
  - Traffic loads (suspended, semi/non-suspended)
  - Train speed
  - Train movement direction
  - Water influence
  - Atmospheric temperature

- **Geotechnical factors**
  - Foundation type
  - Soil reinforcement (geogrid, geotextile, piles)
  - Treatment of materials
  - Settlement layers

- **Structural factors**
  - Stiffness variation
  - Damping variation
  - Track-structure interaction
  - Typology
  - Lateral movements

- **Track factors**
  - Rail irregularities
  - Railpads, USPs, slab mats
  - Type of sleepers
  - Ballast characteristics
  - Slab track components

- **Limiting factors**
  - Permissible deformations
  - Allowable differential settlement
  - Acceptable noise and vibration
  - Tolerable vehicle accelerations
10. References


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