Railway Track Transition Dynamics and Reinforcement Using Polyurethane GeoComposites

P. Woodward, O. Laghrouche and A. El-Kacimi
Institute for Infrastructure & Environment, Heriot-Watt University, Edinburgh, EH14 4AS, UK
E-mail: P.K.Woodward@hw.ac.uk

ABSTRACT: The change in railway track stiffness from ballasted track to a fixed track structure, such as concrete slab-track or a fixed bridge deck, can cause significant track geometry issues, particularly for high-speed and heavy haul. The change in track stiffness generates additional track forces at the ballast interface in an area that can be very difficult to maintain, due to the tamper's inability to lift the track on the fixed geometry side. In this paper the transition problem is investigated using DART3D, a 3-dimensional finite element program that can simulate the train-track coupling behaviour over the transition. The application of a 3-dimensional polyurethane reinforcement technique is then presented as a designable means to control the ballast migration behaviour in the transition zone to reduce dynamic effects from problems like *hanging sleepers*. The paper then discusses the impact of using this type of ballast reinforcement through numerical simulation. Application of the technique at Tottenham Hale Junction UK is presented to illustrate the application of the system to real track transitions.

1. INTRODUCTION

It is well known that the high train-track forces generated at problem sites like turnouts, bridge transitions and poor ground result in higher ballast settlements and hence track misalignments. The track misalignments then result in higher track forces leading to an increasing rate of geometry deterioration. This cycle leads to high track maintenance, often at track assets that are difficult to maintain such as transitions. Current technologies applied at these sites are often based around planar 2-dimensional reinforcement, such as geogrids. A technique that has been used at many sites across the UK and now outside of the UK (e.g. Italy and Hong Kong) is based on the use of polyurethanes to stabilise and reinforce the ballast to a predetermined depth forming an interconnected 3-dimensional GeoComposite. This GeoComposite has a high degree of strength and resiliency and therefore can have a very positive impact on the track behaviour. Reviews of transition behaviour can be found in Galvin *et al.* (2010), Banimahd *et al.* (2011), Coelho *et al*. (2011), Shan *et al*. (2013) and Ang and Dai (2013). Figure 1 shows a typical fault developed in the transition zone at the run-on and run-off interface of a fixed track on a bridge.

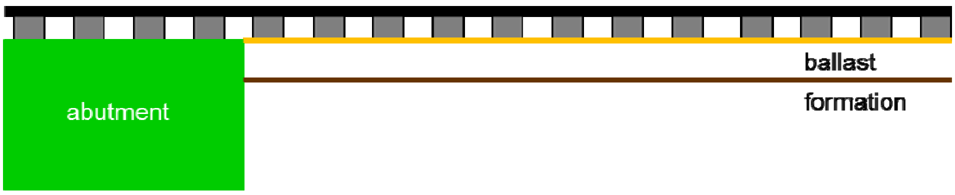
Figure 1 Typical transition fault (indicated) at the bridge interface

The dip in the track around the bridge interface (within the transition zones) can clearly be seen. This causes the train to 'snake' vertically as it passes over the transition significantly increasing the track forces and causing a high vertical acceleration to be induced into the train body. In effect the train wheels need to climb over the difference in the track elevation, brought about due to the differences in track stiffness and vertical settlement of the track in the transition zones (Figure 2). The acceleration level induced into the train system is controlled by the train speed and the rate of climb of the wheels (i.e. the elevation difference, whether from the track stiffness change or the track settlement, or both). Banimahd (2008) showed the effect of the train speed on the induced acceleration. Transition issues can therefore be addressed by ensuring that the induced acceleration is below a given criterion – often stated as a maximum induced acceleration level. Track stiffness changes have been examined by Li and Davis (2005) and Li *et al*. (2011). In many cases they found that trying to increase the track stiffness in the transition zone still resulted in the development of a track fault. A summary of the main issues surrounding transitions are listed below:

1.1 Track stiffness change

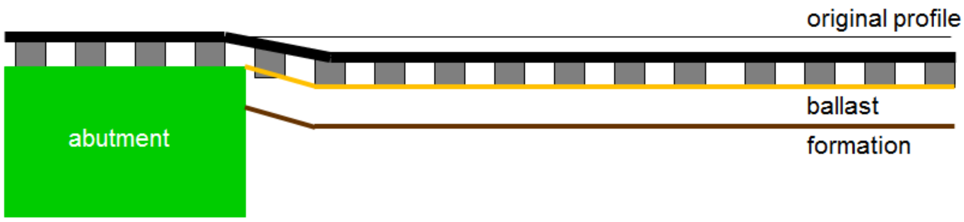
At the transition the train wheels experience a rapid change in track deflection (i.e. stiffness) from the plain-track to the rigid track (Figure 3a). This excites the train body and suspension components which in turn leads to dynamic amplification of the forces and increased stress on the track structure. The level of dynamic amplification depends on the magnitude of the deflection difference between the two structures and the train characteristics. Train body oscillations can affect track geometry further down the track producing so called 'cyclic top'.

Figure 2 Hanging sleepers at bridge interface (from floating to fixed track geometry)



(a) Stiffness change from ‘floating’ to ‘fixed’ track – *the stiffness change*

(b) Development of *hanging sleepers* in the transition zone



(c) Embankment *consolidation* leading to a larger elevation difference

Figure 3 Illustration of the typical issues generated at railway transitions

1.2 Hanging sleepers- *transition fault*

The developing track fault leads to hanging sleepers in the ballast (either through ballast movement and/or formation overstressing). This excites the next train still further generating higher track forces and settlements, hence the situation is self-perpetuating. Correction of the track fault is difficult, often due to the fixed nature of the track on the bridge which prevents the track in the transition zone from being tamped. In Figure 2, and illustratively in Figure 3a, a typical bridge interface problem where a void has developed at the transition leading to hanging sleepers is shown. This fault gradually propagating along the transition length; typically the length of track affected is around 7m from the ballast boards, although this obviously depends upon the speed and track conditions at a particular site.

The appearance of hanging sleepers can be observed at many transitions, particularly for the first few sleepers from the ballast boards (Kerr and Moroney 1993, Thompson and Woodward 2004 and Lundqvist and Dahlberg 2005). The increase in the vertical acceleration force can therefore be the result of embankment movement or from movement of the ballast (Zakeri and Ghorbani 2011). As discussed above, the resulting difference in height between the floating and fixed track means that the train wheels have to climb a higher elevation (up to 16mm of voiding with respect to the abutment under the first transition sleeper has been measured by the first Author). Since the stiff side is fixed, the rail must be unclipped in order to use a tamper to correct the track geometry on the floating side (however no tamper run-out on the track geometry would be possible due to the unclipped rail).

If the track fault is not corrected it can propagate along the track length and hence damage the track structure, e.g. the fixed geometry concrete slab-track. Solving the ballast voiding issue in the area where the track cannot be easily maintained will therefore contribute significantly to improving the performance of the transition. In addition if the track stiffness can be increased in this area then a further added benefit can be achieved.

1.3 Embankment consolidation

Embankment consolidation can arise over many years resulting in track settlement along the entire embankment track as well as within the transition area. In the case of bridge transitions, the abutments are unlikely to settle by the same amount (they may even be piled) then an elevation difference between the floating track (embankment) and the fixed track (the bridge abutments) may result in large induced forces due to a sudden change in vertical track geometry (i.e. a sudden rise in the rail elevation as shown in Figure 3c). This type of settlement is difficult to control (particularly for older structures founded on soft clays) and hence a common method is simply to ‘over tamp’ creating a surplus of track ballast which reduces as the embankment continues to consolidate.

Figure 4 shows a typical TRV (Track Recording Vehicle) trace for a measurement train passing over a bridge transition. The large peaks in the trace at locations 100 and 180 (the scale is in metres) represent the run-on and run-off locations where hanging sleepers are present. Their effect is clearly visible giving poor track geometry and hence the need for ballast maintenance to restore track geometry. This paper primarily concentrates on the solution of this particular transition issue as it forms the majority of transition problems.

2. TRADITIONAL SOLUTION METHODS USED IN UK

There have been a large number of different solutions proposed for improving the performance of transitions. Several of these variants are illustrated below.

2.1 Additional rails

Figure 5 shows the application of additional rails which pass over the transition from the floating to fixed side. The intention is to provide an increase in track stiffness however voiding underneath the transition sleepers has still been observed leading to track geometry faults.

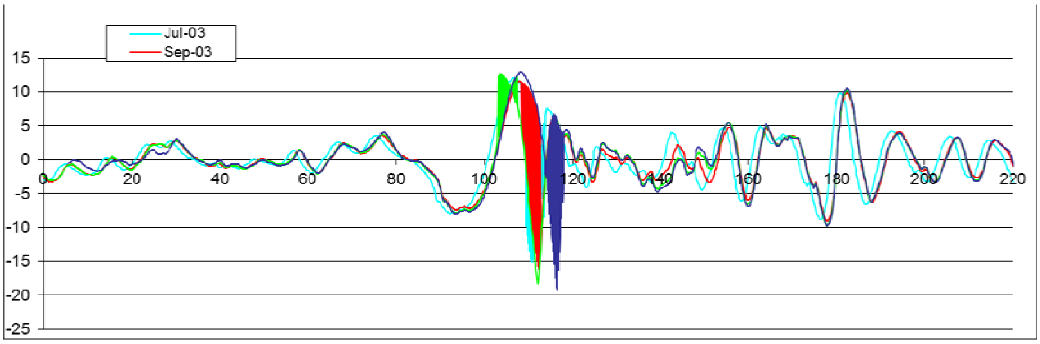


Figure 4 TRV trace over a typical fixed deck bridge showing large track geometry faults at the transitions (vertical axis Left Top in mm; horizontal axis track distance in metres)

Figure 5 Application of additional rails in the transition zone

2.2 Transition slabs

Figure 6 shows the approach to concrete slab-track at Falkirk High Tunnel in the UK. The affected line is the Down Line shown as the left track in the photograph.

Figure 6 Concrete slab-track transition (transition slab is underneath the transition sleepers)

Geometry deterioration is due to movement of the ballast on the approach to the slab-track as the transition sleepers lie over a tapered concrete transition slab. On the Down Line this movement is exacerbated by the presence of wet beds due to infiltration of water onto the track and by the line speed of 152 km/h. The high attrition rate of the ballast combined with wet bed formation generate ballast voids underneath the sleepers (hanging sleepers) above the transition slab, but particularly at the concrete slab-track interface. This voiding generates high uplift forces on the transition slab (here in the tunnel) causing deterioration of the rail fixing and hence slab-track integrity. Figure 7 shows this slab-track deterioration within the tunnel at Falkirk High Tunnel prior to application of the polyurethane reinforcement system presented in this paper. Stretcher bars are being used to hold the track gauge.

Figure 7 Deterioration of the concrete slab-track due to the development of hanging sleepers in the transition zone

Figure 8 shows the application of wooden timbers as transition beams.

The wooden timbers are used to provide a ‘semi-rigid’ transition interface between the fixed and floating track. The rails are directly fixed to the wooden beams which generate significant issues during maintenance interventions as the beams have to be lifted in order to manually tamp the bedding ballast underneath; this is extremely difficult due to the beam depth and weight. This type of transition arrangement is therefore very difficult to maintain and many issues during its use have been observed, such as Grove Hill tunnel (Woodward *et al.*, 2011a). There have been many other types of transition solutions proposed/tried including:

- Oversized sleepers in the transition zone
- Use of Hot Mixed Asphalt (HMA) sub-layers
- Formation improvement using Stone Columns
- Formation improvement using Constant Modulus Columns (CMCs)
- Angled concrete slabs and/or deepened granular layers
- Concrete piling
- Reduced stiffness pads on the fixed side

In many situations the application of the above techniques has not resulted in improvement of the transition performance (Li and Davis, 2005 and Li *et al.*, 2011). Regardless of which solution is adopted, if voiding continues to develop in the ballast underneath the transition sleepers (or transition beams) then the transition performance will still be poor, i.e. the transition is not just about solving a change in stiffness – it must also be about preventing hanging sleepers. The effect of hanging sleepers is now investigated through numerical modelling of the transition geo-dynamics.

Figure 8 Application of long-timber transition beams

3. MODELLING GEO-DYNAMIC INTERACTION IN TRANSITION ZONES

3.1 Numerical modelling

In order to exam the effects of a transition fault due to settlement of either the ballast and/or the formation, the results of a three-dimensional finite element analysis are presented. The finite element program used is called DART3D (Dynamic Analysis of Railway Track 3D). The program is fully coupled, i.e. the geo-dynamic behaviour of the railway track and its subgrade is fully coupled to the vertical deflection of the train. The program has been used to look at track geo-dynamics (Banimahd *et al.*, 2012) including the development of Ground Mach Cones (El-Kacimi *et al.*, 20011 and 2012). It uses a time domain approach to model non-linear effects in the track response and Lysmer and Kuhlmeyer (1969) boundaries to prevent stress-wave reflection. Figure 9 shows that the train is modelled using either a quarter train or full train approximation (Banimahd, 2008 and Banimahd *et al*., 2011).

For the quarter train model used in this paper the equations representing the system components (Lei and Noda, 2002, Lei and Mao, 2003 and El-Kacimi *et al.*, 2012) are,

$$\begin{pmatrix} k_c & -k_c & 0 \\ -k_c & k_b + k_c & -k_b \\ 0 & -k_b & k_b \end{pmatrix}\begin{pmatrix} w_c \\ w_b \\ w_w \end{pmatrix} + \begin{pmatrix} c_c & -c_c & 0 \\ -c_c & c_b + c_c & -c_b \\ 0 & -c_b & c_b \end{pmatrix}\begin{pmatrix} \dot{w}_c \\ \dot{w}_b \\ \dot{w}_w \end{pmatrix}$$

$$+\begin{pmatrix} \overline{m}_c & 0 & 0 \\ 0 & \overline{m}_b & 0 \\ 0 & 0 & m_w \end{pmatrix}\begin{pmatrix} \ddot{w}_c \\ \ddot{w}_b \\ \ddot{w}_w \end{pmatrix} = \begin{pmatrix} \overline{m}_c g \\ \overline{m}_b g \\ m_w g + F_{wr} \end{pmatrix}$$

Where w_c, w_b and w_w are the vertical displacements of the car body, bogie and wheel, m_w is the wheel mass, $\overline{m}_c$ and $\overline{m}_b$ are representations of the car body and bogie masses ($\overline{m}_c = m_c / 8$ and $\overline{m}_b = m_b / 4$). k_b and k_c are the primary and secondary suspension stiffness and c_b and c_c the corresponding damping coefficients. The train suspension system modelled in this paper is the X2000 high-speed; details about this train can be found in (Madshus and Kaynia, 2000). The properties of the suspension systems used are shown in Table 1.

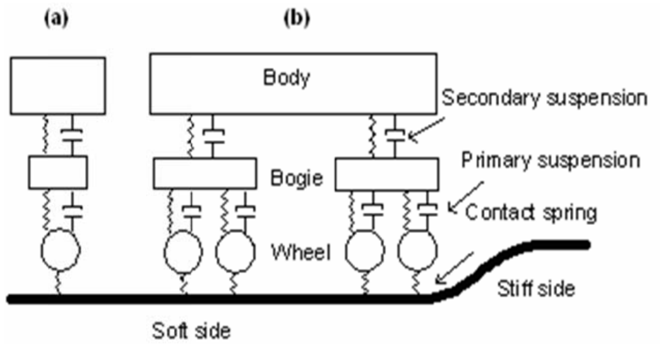


Figure 9 Train modelling used in analysis (a) Quarter train model (b) Full train model

Table 1 Train suspension parameters used in analysis

Suspension Property	
Axle Load (kN)	181
Primary Suspension	
Stiffness (MN/m)	3.28
Damping (kNs/m)	90
Secondary Suspension	
Stiffness (MN/m)	1.31
Damping (kNs/m)	30

3.2 Effect of transition fault

Presently it is not possible to measure the track deflection and hence track stiffness for a moving train in the UK. Track stiffness is normally measured using a Falling Weight Deflectometer (FWD). The FWD uses an impulse load on the sleeper (a weight is dropped onto the centre of the sleeper) in order to estimate the track stiffness by measuring the displaced bowl through geo-phones (the velocity record is integrated to estimate the bowl displacements). The rails must however first be disconnected and all train operations suspended. The stiffness is therefore an estimate of the formation stiffness and not the overall track stiffness (i.e. it does not include the stiffness of the rail and pad). In this paper the effect of a transition fault is simulated for a typical transition onto concrete slab-track. In particular, the analysis assumes that the track has an existing concrete transition slab at depth leading up to the concrete track itself. This transition is a typical arrangement when traversing from floating track (ballasted track) to fixed track (the concrete slab-track). For example it is very similar to the transition arrangement at Falkirk High Tunnel shown in Figure 6. The speed in the computer simulation is higher than the tunnel to illustrate the effect of the transition fault on the train behaviour.

The properties used in the analysis for the track are shown in Table 2 The rail used is a typical UK 113lb and the sleepers are assumed to be concrete with a length of 2.4m (141 sleepers used in the analysis). An explicit time step of 8.0e-6 is used for 156,000 increments (representing 1.2s).

Table 2 Track parameters used in analysis

Ballast	
Young's Modulus (MPa)	124
Poisson's Ratio	0.4
Rayleigh Damping (%)	5
Density (kg/m^3)	1800
Clay Subgrade	
Young's Modulus (MPa)	60
Poisson's Ratio	0.45
Rayleigh Damping (%)	5
Concrete	
Young's Modulus (GPa)	20
Poisson's Ratio	0.2
Rayleigh Damping (%)	0
Density (kg/m^3)	2400

The total number of elements used in the analysis is 42,450 20-noded brick elements and 424 beam-column elements for the rail. The subgrade Rayleigh wave velocity is 102 m/s and hence the modelled train speed of 70 m/s is around 70 % of the critical speed. Figure 10 shows the meshed used in the analysis (the concrete elements assumed are indicated by the red elements and the rails are omitted for clarity). The mesh dimensions are 100 m long by 25 m wide and 15.5 m deep. The concrete transition slab is 10 m long and 300 mm thick and is placed 300 mm below the sleeper bottom at the base of the ballast layer. The properties for the slab are also presented in Table 2. For this paper no additional track irregularities (e.g. rail corrugation) are modelled for simplicity. The concrete slab-track simulated is similar to a Rheda 2000 system (Esveld, 2001) and starts 50 m into the mesh and extends to the end of the mesh.

Three finite element simulations are performed termed: RUN1; RUN2; and RUN3.

- RUN 1: no transition fault above the transition slab
- RUN 2: 7 m long transition fault at a peak of 4 mm above the transition slab
- RUN 3: the 7 m long transition fault above the transition slab stabilised by the polyurethane polymer

Figure 11a shows the track response for the front two wheels (axles) over the transition. The presence of the simulated 4mm track void (track fault) can clearly be seen as a large dip in the train-track time history (RUN 2). Even though the polymer stiffness in RUN 3 is approximately four times higher than that of the *in-situ* ballast (RUN 1) its overall effect is not significant in-terms of the overall track deflection. This is because of the high stiffness of the lower concrete transition slab which reduces the stiffness effect afforded by the polymer over the 7 m long GeoComposite reinforcement. In this application, the primary purpose of the applying the polymer is to stop the ballast attrition and voiding. In lower stiffness soils where no lower concrete slab is applied, the effect of the increase in stiffness of the GeoComposite is far greater in terms of reducing the track deflection.

In Figure 11b the increase in the transient wheel / rail interaction force as the train passes over the transition fault is clearly observed. The increase in this force causes higher track accelerations which in turn increases the magnitude of the track fault and hence increases the interaction force. A self-perpetuating mechanism is developing which feeds the increasing magnitude of the track fault and hence the transition performance can rapidly deteriorate unless the fault is corrected. The increase in train carriage accelerations can be seen in Figure 11c. As the wheels pass over the transition fault the induced train body accelerations increase to 0.1g and then gradually decay as the wheels traverse the concrete slab-track (this peak acceleration would be higher than allowable limits set by SNCF for example). Figure 11d shows the deflection time history of a typical sleeper in the transition zone. The high magnitude of sleeper deflection due to the fault is clearly observed and the overall effect on the rail is highlighted in Figure 11e in-terms of the rail acceleration time history. The high acceleration values clearly demonstrate the increased stresses being placed on the track in this zone.

The numerical analysis proves that allowing a track fault to develop in the transition zone can have a very detrimental effect on the track structure, leading to high induced track forces and accelerations. The large sleeper deflection, which occurs over a relatively short track length, induces high accelerations in the train body which can register as poor track geometry and hence a poor quality ride for the passengers. Experience at Falkirk High Tunnel (Figure 6) and the numerical results presented indicate that preventing the ballast from voiding and forming a fault, will significantly improve the transition performance. The application of a polyurethane reinforcement technique that can achieve this is now presented.

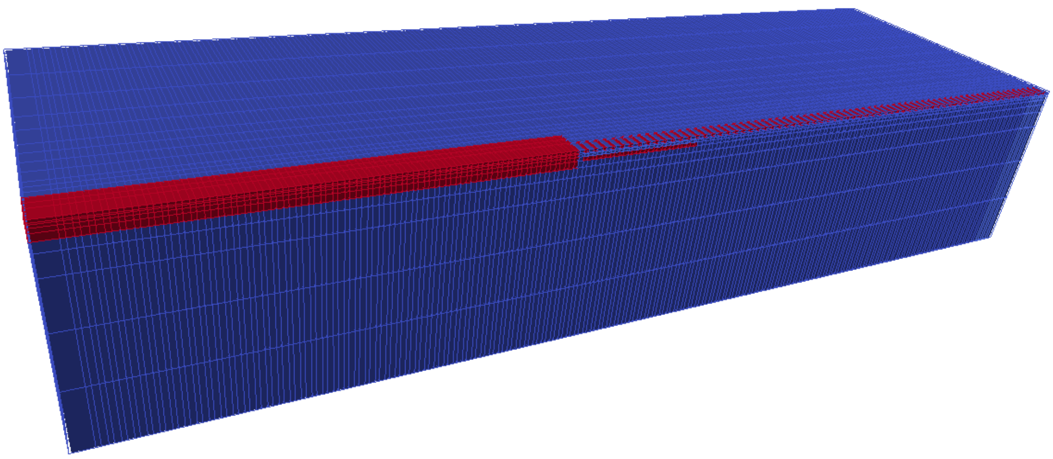
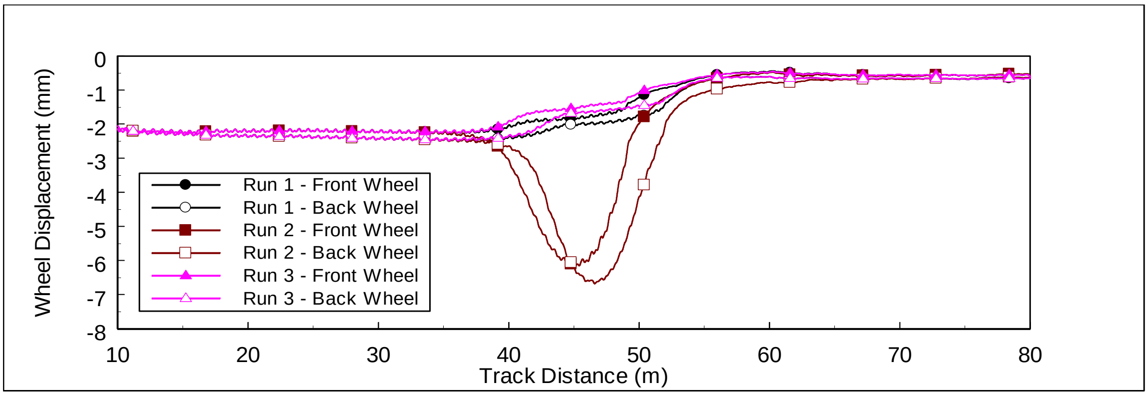
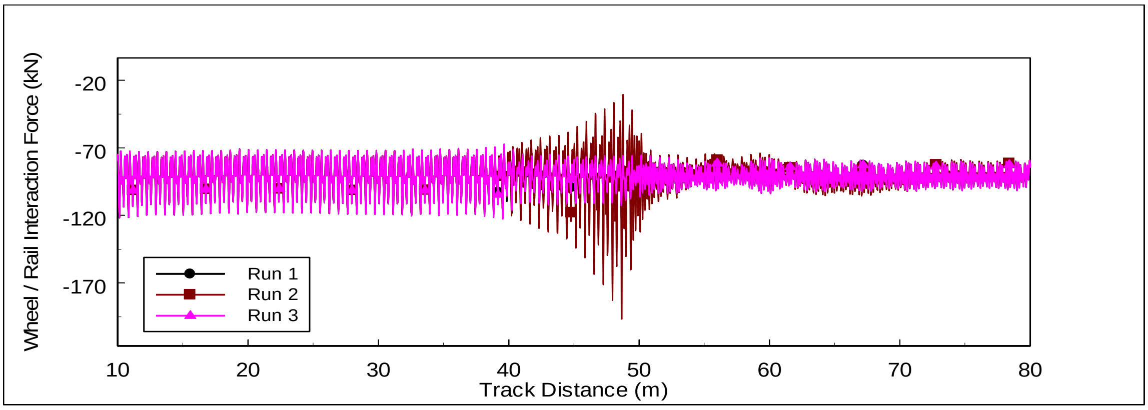


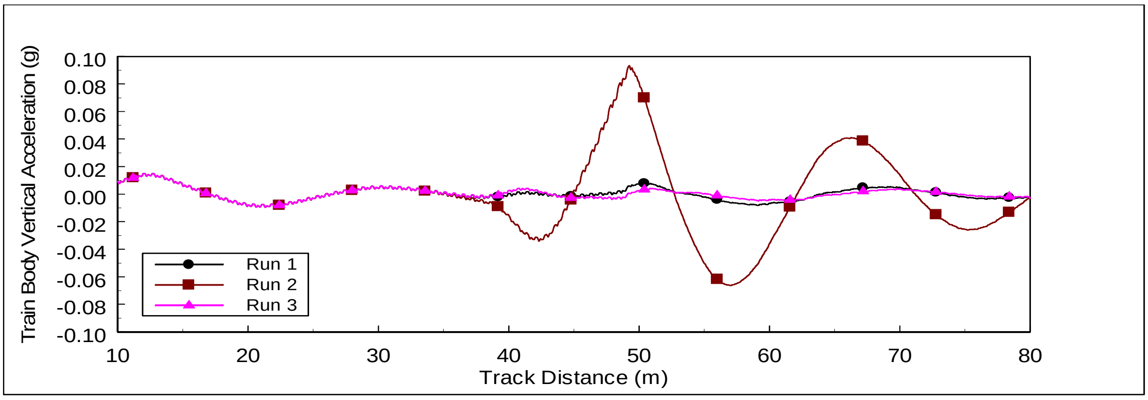
Figure 10 Finite element mesh used in the analysis (distribution of concrete elements shown – rails omitted for clarity)



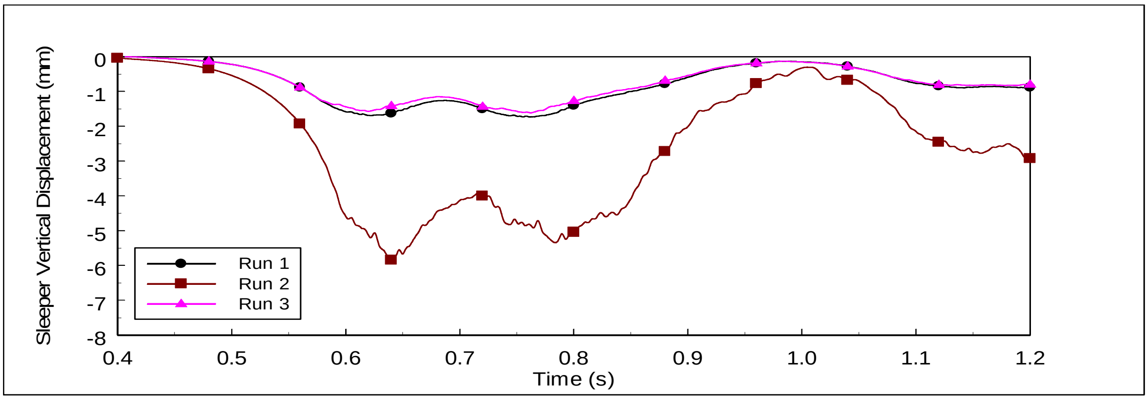
(a) Displacement response of the front two wheels (front bogie) over the transition



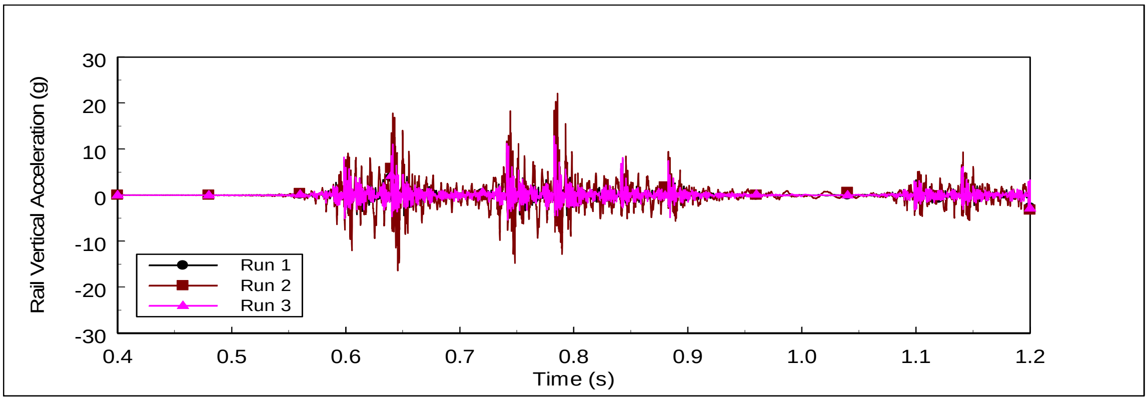
(b) Wheel / rail interaction force for the front wheel over the transition



(c) Induced vertical train body acceleration over the transition



(d) Response of a typical sleeper in the transition zone



(e) Induced accelerations for the rail within the transition zone

Figure 11 Typical response of the train and track for RUN 1,2,3 at a train speed V_T=70 m/s for voided and non-voided transitions

4. IN-SITU POLYURETHANE REINFORCEMENT

From the numerical analysis it is clear that reinforcing the ballast above the transition slab, using visco elastic polymers, will have a very positive effect on improving the performance of the transition (Thompson and Woodward, 2004). Reducing ballast attrition and migration will not only improve the long-term ride comfort level for the passengers, but also improve the longevity of the track components by reducing the level of induced track stresses and forces. The polyurethane polymer used in railway applications comprises two components: a polyol and an isocyanate. These two chemicals are combined in the presence of a catalyst to form the reacted cross-linked polyurethane (Woodward *et al.*, 2005). The gel time is typically 10 seconds and the polymer cures to form around 50% of its stiffness and strength within minutes and around 90% within one hour. Application can either be within possession periods, or during operational use. The polymer is delivered to site in IBCs (Independent Bulk Containers) and transfer pumps, lowered into the IBCs, pump the two components to the primary metered pump where they are then forced along the application lines to the static mixing head. The static mixing head forces the two components to react and hence form the polyurethane. The technique, generally called the XiTRACK technique, has been applied throughout the UK to many different types of long-standing track issues such as transitions (Woodward *et al*., 2005), clearance issues, such as tunnels and station environments (Woodward *et al.*, 2011a) and switch and crossings (Woodward *et al*., 2011b).

The significant improvement in settlement performance of the system compared to unreinforced ballast, tested under laboratory conditions using the GRAFT I facility has been reported by Kennedy (2011) and Kennedy et al. (2013). Application in low temperature requires the use of heated lines and the main pump can either be electrically driven or pneumatically driven (Figure 12). Once formed at the mixing head the polyurethane is simply poured onto the ballast whereby it penetrates to the required depth (set by the catalyst level) to form the GeoComposite. The benefit of using a polyurethane is the ability to design the GeoComposite to the required stiffness, longevity or strength. The application process is shown in Figure 13 and by controlling the amount and type of polyurethane (i.e. the polymer strength and stiffness characteristics) a high degree of properties can be formed. Typically unconfined uniaxial compression strengths can range from between 1 to 14 MPa depending on the particular polymer and quantity used.

The track is placed into its preferred configuration and then stabilised by the polyurethane. The polymer application can be applied either to the lower ballast, to form a lower reinforced GeoComposite slab, or at the surface to form a *ladder* structure (i.e. applied around the sleeper once the track geometry has been corrected). Figure 13 shows that the polymer is simply poured, i.e. it is not sprayed or injected. By knowing the rate at which the polymer is poured the amount applied can be easily controlled by simply measuring the pouring time. Experimental data on the performance of the system at full-scale can be found in Woodward *et al.* (2011a) Kennedy (2011) and Kennedy *et al.* (2013). Data measured from track installations can be found in Woodward *et al*. (2007 and 2011b).

Figure 12 Typical pumping unit (pneumatic variant shown)

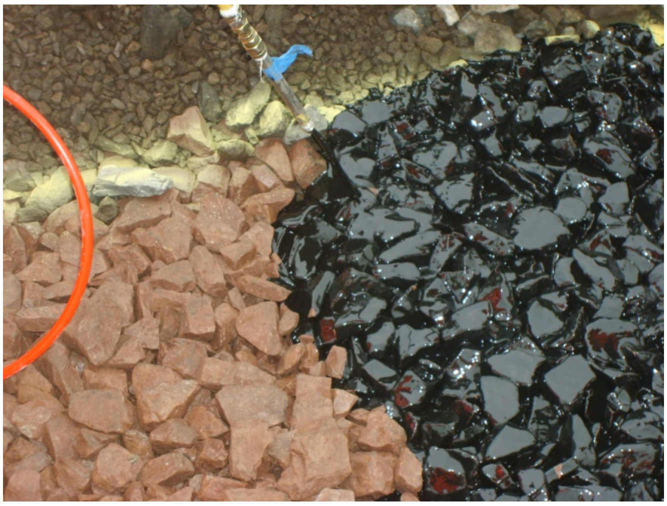


Figure 13 Polymer application to the ballast surface

Figure 14 shows atypical performance of the GeoComposite obtained during an unconfined uniaxial compression test at 12°C. The high degree of ductility within the GeoComposite can be seen during the loading and unloading phase (unloading was due to load removal and not due to strain softening). The GeoComposite is still able to carry load once the stress-strain graph has levelled out as many of the polymer strands (referred to as *runnels*) have not yet failed even though the carry capacity has reached its peak in unconfined conditions. This means that the load can be removed and then reapplied up to the failure line (this can be done many times as shown by Woodward *et al*., 2011a). Of course the in-service stress state would be well below failure, but the material response highlights the ductility and resiliency within the system. For the case considered of a GeoComposite slab above the lower concrete transition slab, the data presented in Woodward *et al.* (2011a) is re-plotted in Figure 15 in terms of the GeoComposite permanent settlement versus load cycles for a contact vertical cyclic stress of 2 MPa; as tested in a Losenhausen compression testing machine (shown in Figure 15).

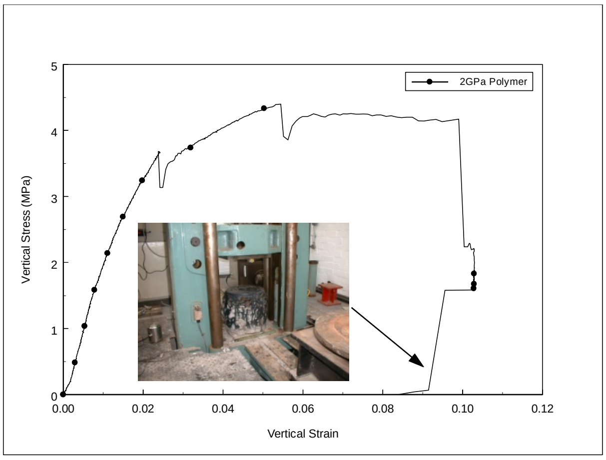


Figure 14 Typical GeoComposite unconfined compression test for a polymer stiffness of 2 GPa (rate=6 mm/min)

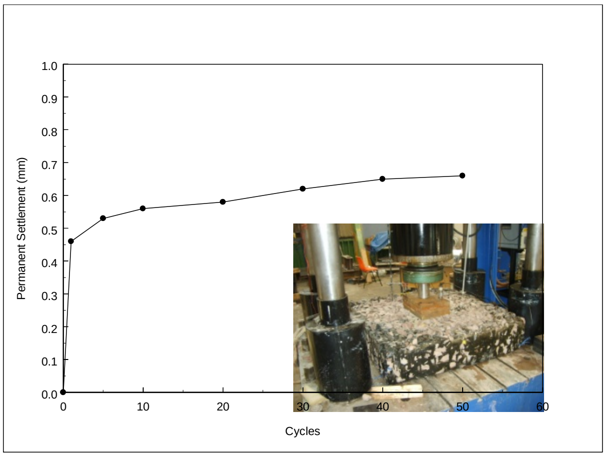


Figure 15 Permanent settlement with load cycles of atypical GeoComposite over a solid base (rate=3 Hz)

The developing resiliency of the GeoComposite with load cycles can clearly be seen; there was not a significant change in track stiffness with loading rates applied Kennedy (2011). The first loading cycle settlement is thought to be due to the very low compaction applied to the ballast prior to polymer pouring. Under normal conditions the cyclic stress of 2 MPa is above the likely contact stress level in the ballast over the concrete transition slab. The test therefore shows that the GeoComposite would prevent ballast voiding and hence provide the solution analysed in RUN 3. If contact stresses were greater than 2 MPa the polymer loading applied to the ballast could simply be increased to improve strength.

5. SITE APPLICATION OF POLYURETHANE REINFORCEMENT

Tottenham Hale is located in the East Anglia region of Network Rail's South East Territory in the UK. The junction provides a spur from the main London to Cambridge railway line to the North London Line just north of Tottenham Hale station. As such it carries regular high speed trains to Stansted airport, and also heavy freight to/from the main line towards the North London Line. This heavy freight approaches from a very narrow radius curve and as such has tended to force the junction laterally out of alignment. The combination of this with the presence of the switch tips across the threshold of a bridge abutment and the combination of forces provided the maintainer with a difficult alignment problem that required regular maintenance. Figure 16 shows a longitudinal view of the bridge transitions at Tottenham South Junction (Tottenham Hale side).

Figure 16 View of Tottenham Hale Junction Showing Bridge Transitions

The problems associated with these types of bridge transitions and how polyurethanes can be used to reduce track maintenance, have been discussed by Thompson and Woodward (2004).

At Tottenham South Junction two additional problems occur, namely the two point motor machines located on the bridge and the length of the ballasted track that exists on the bridge itself to support these point machines. Figure 17 shows that on the Down Main this ballasted bridge track extends approximately 2m onto the bridge deck and on the Up Main it extends approximately 3.3m onto the bridge deck. This represents a form of lower transition slab (hereby called the lower deck transition slab). From then onwards, fixed timbers support the track across the bridge in order to provide the required track fixity condition.

The Track Recording Vehicle (TRV) readings indicated poor track geometry over these transition areas. The depth of ballast on the bridge deck sections for both the Down Main and the Up Main is approximately 250 mm below sleeper bottom (Figure 18). In order to improve the track geometry and reduce the maintenance required the ballast above the lower deck transition slab and in the transition area of the ballast was reinforcement using the polymer technique.

5.1 XiTRACK Polyurethane Procedure

In order to determine the necessary design for the transitions the following steps were taken during the installation process:

1) Site inspection performed in March 2005
2) Application solution proposed in June 2005
3) Polymer installed August 2005

The transition was instrumented with both displacement and acceleration sensors in May 2005, however the results of the data acquisition were inconclusive due to the presence of the switch and crossing, which generated a significant level of noise within the measurement system.

Figure 17 Bridge Transitions Showing Current Location of Point Machines on the Bridge Deck

Figure 18 Extension of bridge deck between fixed timbers and floating ballast sections (*lower deck transition slab*)

5.2 Polyurethane solution

A continuous GeoComposite structure was formed under the track to provide a high degree of track reinforcement in August 2005. The minimum required depth of the ballast at renewal for the off-bridge transitions was 400mm. The lower 300mm of ballast was reinforced with polyurethane, the upper 100mm was left to allow any maintenance required after treatment; this was sufficient for the tampering tines. Figure 19 shows the application of the polymer at the site.

The design was essentially split into three parts: the off-bridge transitions (i.e. the ballasted track which lies over the soil formation), the lower deck transition slab and the treatment of the lateral beams to provide lateral restraint.

5.2.2 Part 1: Off-bridge Transitions

The off-bridge transitions started at the ballast boards (these were skewed) and extend into the junction. The nested joints were not treated, only the two transitions zones. As shown in Figure 20 for the Down line, the length of the transition zones (from the Down and Up centre lines) were Z1=2.0 m and Z2=4.25 m. These zones were offset due to the bridge skew. Since the extent of the reinforcement zones covered the majority of the transition area, these areas are connected together through the GeoComposite treatment to form a continuous raft. This improves track performance since the reinforcement makes the track behave as a linked geopavement over the reduced transition length. The 100 mm top ballast was added prior to polymer curing to ensure that it bedded into the polymer. The polymer used for the off-bridge transitions was designed to flow down to a depth of approximately 300 mm.

Figure 19 Application of the polyurethane along the bridge transition

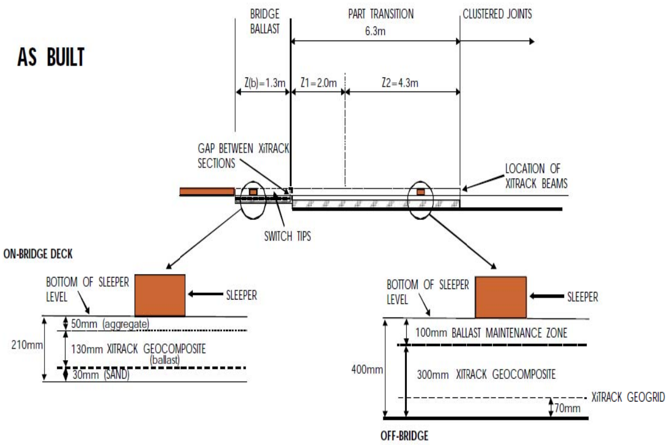


Figure 20 Application profile for the polymer

5.2.3 Part 2: On-bridge Transitions

For the on-bridge transitions the depth of ballast available was limited to approximately 250 mm. The polymer used in this area was different to that used on the normal track due to this limited depth. In addition, the upper and lower ‘fine’ aggregate zones were included within the bridge depth to allow maintenance to occur and to reduce ballast contact stresses on the steel deck. The polymer used for the on-bridge transitions was designed to flow to a depth of approximately 200mm. The bridge system and the track system polymers therefore differed only in their penetration depths (set by the polyol component, which will be different in each case).

The sequence of events for the bridge treatment was as follows:

1) A 40mm thick layer of fine aggregate was placed onto the bridge deck and compacted. The use of a resilient pad between this aggregate and the bridge deck was not approved by the track maintainer.
2) A 160 mm thick layer of ballast was added and compacted. It was then treated with the polymer.

3) A 50mm thick layer of stone-blower aggregate (directly after and before curing of the polymer) was placed in the crib and sleeper bottom areas (not in the shoulder areas; since ballast was added before polymer curing due to subsequent beam formation).

5.2.4 Part 3: Upper Lateral Beams

Lateral beams were included where possible. The beam polymer was applied after the lower reinforcement zones were completed and the track had been correctly realigned by the tamping machine (Figure 21).

Figure 21 Tamping over the GeoComposite to reinstate the track geometry after lower polymer treatment and before formation of the upper lateral GeoComposite beams

The remaining polymer beams were formed in two stages:

1) Stage 1, the lower ballast depth treated in the shoulder areas did not exceed the sleeper depth mid-point.
2) Stage 2, ballast was added up to the top of the sleeper in the shoulder areas and treated with the polymer. While the polymer was still curing red grit was applied to form a slip-resistant walking surface.

The formation of the upper lateral GeoComposite beams to prevent lateral movement of the point machines is shown in Figure 22. The increased capacity of lateral beams to provide track support has been described by Woodward *et al.* (2011a and b).

Figure 22 Formation of the upper GeoComposite beams to prevent lateral movement of the sleepers

Since a maintenance layer above the transition GeoComposite was left untreated (as requested by the infrastructure owner to allow future tamping) some movement of the ballast above the geopavement layer is possible, i.e. a complete *ladder* structure was not formed for this layer. The ladder structure (Kennedy *et al*., 2013) can be used to capture the track geometry when the track has been put into the correct track alignment (i.e. after the tamping cycle shown in Figure 21). At Tottenham Hale Junction a further application of the polymer could therefore have been applied to stabilise the upper 100mm maintenance zone (that was not treated) to provide a full depth solution. This additional treatment would then generate complete stability at the bridge interface, as modelled in the numerical analysis shown in Figure 11. The formation of the ladder structure is shown in Figure 23, here for track treatment at Worplesdon UK (Woodward *et al.*, 2005). In this type of application the ballast is removed down to the base of the sleeper and the polymer applied in the crib and shoulder areas only; this confines the unreinforced ballast directly under the sleeper and hence 'captures' the track geometry (Kennedy *et al.,* 2013).

Figure 23 Application of the polymer to form a ladder structure at Worplesdon UK

5.2.5 After polymer treatment

The treatment at Tottenham Hale was used to reduce ballast movement through polymer reinforcement. If changes in track geometry occur (e.g. movement of the formation or the upper unreinforced ballast) then tamping, stone blowing and/or shovel packing can be used in the off-bridge treated areas. However, shovel packing would need to be used to correct any misalignments within the on-bridge (steel deck) treatment areas. It should be noted that due to the unreinforced ballast above the XiTRACK geopavement some maintenance should be expected in the long-term. As far as the Authors are aware performance has been very good to date.

6. CONCLUSIONS

In this paper the application of 3-dimensional finite element techniques to examine the behaviour of a typical floating track to fixed concrete slab-track, located over a concrete transition slab, have been used. The results of the analysis show how the development of a track fault in the ballast above the transition slab can have a significant effect on the train-track interaction mechanism. In particular the fault leads to a high level of coach body accelerations and high induced wheel-rail interaction forces. The fault also significantly increases the sleeper deflection and the induced rail accelerations, placing a high degree of additional stress on the upper track components. Stabilisation and reinforcement of the ballast above the transition slab was then simulated and the significant improvement in track response highlighted. Ballast reinforcement can be achieved using in-situ polyurethane polymers applied at the ballast surface. Stress-strain behaviour of the formed GeoComposite during unconfined uniaxial compression tests illustrated the high degree of hysteresis in the material response.

This hysteresis is highly desirable in railway environments as it provides a high degree of ductility, particularly during impact loading such as wheel flats. The use of this reinforcement technique, called XITRACK, at Tottenham Hale Junction transitions was presented to demonstrate the applicability of the system to real railway track transitions. The ease of application and the typical installation equipment required was highlighted. The ability of the technique to solve both the lateral and vertical track ballast migration issues was discussed together with how a typical methodology can be adopted for polymer pouring.

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