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Application of polyurethane geocomposites to help maintain track geometry for high-speed ballasted railway tracks*

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Abstract: There are many issues surrounding the performance of critical assets on high-speed ballasted railway lines. At assets like switch & crossings and bridge transitions high track forces can be produced resulting in higher ballast settlements and hence track misalignments. The latter result in higher track forces and hence more settlement, leading to the need for increased track maintenance to ensure comfort and safety. Current technologies for solving issues like ballast movement under high-speed loading regimes are limited. However, a technique that has been well used across the UK and now increasingly overseas to stabilise and reinforce ballasted railway tracks is the application of in-situ polyurethane polymers, termed XiTRACK. This paper discusses how this technique can be used to solve these types of long-standing issues and presents actual polymer application profiles at two typical critical sites, namely a junction and a transition onto concrete slab-track.

Key words: Railways, Polyurethanes, Geocomposites, Modelling, High-speed
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1 Introduction

The need for increases in train speed, axle weight, and track access has resulted in ever greater forces (both in magnitude and number) being imposed on railway tracks. The vast majority of railways across the world are constructed using ballast, a material that develops strong non-linearity under shear forces (Indraratna *et al*., 2005). Stress reversals generate hysteresis and cyclic mobility in the stress-strain loops and hence plastic strains develop resulting in ballast settlement and eventual track misalignments. Tamping of the ballast to restore track geometry is then required, which disturbs the ballast structure generating future geometry issues. Stone blowing can be used to help prevent some settlement occurring by reducing the ballast disturbance, however at critical track assets that are highly loaded, such as switch & crossings, ballast movement is inevitable. Ballast does however have many advantages such as ease of track maintenance, should the track geometry deteriorate, ease of track renewal or line re-routing, drainage properties, energy absorbing properties, and reduced construction costs when compared to concrete slab-track; although a ballast track's life cycle costs are likely to be higher when compared to concrete track. There are many track areas where ballast movement results in track settlement. Of particular concern are assets such as switch & crossings and track transitions. Movement of the ballast can generate large dynamic forces leading to further track settlement and passenger discomfort. Higher line speeds make the need for track maintenance more critical in order to ensure track tolerances and alignments are retained. In addition to these assets it has been reported that the ballast vibration level will increase as

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the train speed increases (Pita *et al*., 2004). The result of this increase in peak particle velocity is to cause the ballast particles to start migrating and hence generating additional track settlement and maintenance. The need to provide track reinforcement is therefore highly desirable. It is common to use planar 2D geogrids in railway applications to try and reduce some of this ballast settlement; however, full-scale laboratory tests suggest that geogrid solutions are limited in their ability to modify the ballast behavior; although some reduction in ballast settlement was observed; the 2D geogrid was unable to reduce the subgrade bearing stress or improve the track stiffness (Brown *et al*., 2007). It is now common to see geogrids directly bonded to geotextiles to form a composite. The benefit of this system is likely to be in its ability to act as a reinforced separator. However, if movement of the ballast is due to ballast vibration then the geogrids would probably have limited effect. Tests on geocells by Kennedy (2011) have indicated that there is a significant difficulty in compacting the material within the cells which can lead to reduced track stiffness. To improve these two areas (i.e., to reduce ballast movement due to vibration and increase the track stiffness), the ballast structure needs to be modified so that it can support tensile loads and transmit shear stresses over a greater area. The discrete nature of the ballast therefore needs to be transformed to enable pavement-like properties, hereby termed geopavement. This can be achieved by reinforcing the ballast in 3D to form a geocomposite (Woodward *et al*., 2005). Reinforcement of the ballast can be achieved using a variety of different polymeric resins, however polyurethane has been found to be particularly beneficial as its rheology can be fully designed and by adding different catalyst levels, the depth of penetration is controlled. Woodward *et al*. (2012a; 2012c) showed that addition of the polyurethane increases the ballast stiffness, strength and resiliency.

The work presented in this paper discusses the issues surrounding transitions and switch & crossings. It then presents equations discussing the dynamic behavior of the track and highlights that the ballast vibration level increases with the train speed. The application of the XiTRACK polyurethane technique to stabilize these issues is then presented.

2 Transitions and switch & crossings

2.1 Transitions

One of the most challenging issues with respect to high-speed is transitions from ballast track to concrete slab-track. Many different types of transition arrangements have been suggested, such as under track concrete transitions and/or long wooden timbers. Fig. 1 shows a typical arrangement whereby long wooden beams situated on top of ballast contained within a concrete channel are used to provide an increase in stiffness prior to the concrete slab-track. However, ballast migration and attrition result in geometry irregularities and hence poor transition performance. Studies on transition behaviour have been reported by Kerr and Moroney (1993), Esveld (2001), Lei and Noda (2002), Lei and Mao (2004), Thompson and Woodward (2004), Li and Davis (2005), Lundqvist and Dahlberg (2005), Banimahd (2008), Li *et al.* (2010), Banimahd *et al*. (2011), Coelho *et al*. (2011), and Zakeri and Ghorbani (2011). At transitions the train wheels experience an abrupt change in track stiffness which results in oscillations being setup in the train suspension system leading to increased dynamic forces. The level of increase in train forces depends on the track stiffness change between the two sides of the transition, the train speed, and the suspension properties of the train. In agreement with other authors, Banimahd (2008) found that the abrupt change in the track stiffness did not in itself lead to significant increases in the wheel forces.

Fig. 1 Concrete channel section transition with long timbers

Settlement of one side of the transition due to track (embankment) consolidation and/or ballast migration did however generate large additional forces;

this can result in track irregularities in the transition zone. In addition, hanging sleepers can develop whereby the base of the sleeper is no-longer in contact with the ballast, resulting in an increase in the dynamic load as the train passes over the transition. In areas of poor subgrade stiffness this can accelerate, generating a self perpetuating mechanism; i.e., the higher the track dynamic loads, the higher the track settlement and hence the higher the dynamic loads. Banimahd *et al.* (2011) showed how the dynamic loads, and hence the induced coach body accelerations, are affected by track geometry issues at the transition. In the case of transitions onto fixed timber deck bridges or concrete slab-track the tamper is unable to lift the rails for a distance of approximately 3 m from the ballast boards due to the track fixity and hence track maintenance in this critical area often does not occur or is at a lower standard than the remainder of the line. If this occurs the transition fault will rapidly develop after track maintenance.

Many different techniques have been suggested to improve the transition performance (Li and Davis, 2005; Li *et al.*, 2010). Often suggestions involving softer rail pads on the stiff side have been recommended to even out track stiffness changes. However, settlement issues in the ballast due to ballast consolidation can still occur (the ballast in the transition zone will still settle as with other plain-line ballast) generating a difference in elevation between the stiff and weak sides, thus generating the conditions necessary for track fault development. Studies have also found that improving the stiffness and strength of the track subgrade have often not improved the transition performance, this is because movement of the ballast still occurs (Li and Davis, 2005). On embankments vibration of the ballast at the ballast boards (i.e., at the transition interface) can result in ballast migrating down the embankments sides generating hanging sleepers and thus transition faults. Generating a fault leads to increases in the stress levels in the adjacent ballast, which generates further plasticity and hence the fault can propagate down the track. Often a track fault around 7-m long can be observed in mid-range train speeds. Li and Davis (2005) reported the results of several different track stabilization methods for transition issues. These included geocell reinforcement, cement-stabilised back fill, and hot-mixed-asphalt layers.

It was found that none of these methods increased the performance of the transition and that most of the transition problems resulted from ballast movement. It is therefore logical to assume that increasing the ballast stiffness and strength using an in-situ reinforcement technique, which can fully stabilize the ballast geo-matrix, would have a very positive effect on the performance of the transition.

2.2 Switch & crossings

Fig. 2 shows a typical swing-diamond illustrating the difficulties in track maintenance at these complex assets. Track maintenance machines require access to the sleepers at the rail supports in order to operate correctly. Typically this is a problem at switch & crossings as the mechanical linkages between the point machine and the switchblades prevent access to two key sleepers. As a consequence, these two sleepers are often voided resulting in damage to the point machine and ultimately expensive train delays. Similarly, the long timbers onto which the motor 'point' machines are fixed may not be maintained at all, at the point where the machine is attached. All of these issues increase the ballast settlement and vibration levels and hence significantly increase the track forces at these critical assets. Often the damage accumulates resulting in failure of the point motor to correctly operate (causing a signal failure), or in a worst case cracking of the rail (perhaps at the nose of the crossing, or at the diamond knuckles) resulting in a rail break.

Fig. 2 Virginia Water Junction UK, showing a typical swing-diamond illustrating the difficulties in track maintenance at these complex assets

3 Ballast track vibration and critical speed

There are many sources of track vibration, such as rail and track irregularities, train suspension systems, and outer-balanced wheels. In addition to these, two dynamic track issues may arise depending on the dynamics of the track and the train itself, as the speed increases. The first issue relates to the critical speed effect (Krylov, 1994; Dietermann and Metrikine, 1996) whereby the train speed V_T approaches that of the track critical velocity (Madshus and Kaynia, 2000; Madshus *et al.*, 2004) given by V_{TCV}, or Rayleigh wave velocity V_R in normal plain line track conditions, i.e., no track stiffening or embankments (Woldringh and New, 1999; Banimahd *et al.*, 2012). The Rayleigh wave velocity can be calculated from

$$V_R = \frac{0.87 + 1.12\nu}{1+\nu} V_S,$$

where V_S is the shear wave velocity and is calculated from

$$V_S = \sqrt{\frac{E}{2\rho(1+\nu)}},$$

where E is the Young's modulus, ρ is the density, and ν is the Poisson's ratio.

The Rayleigh subgrade Mach number M_{RSM} can be defined as

$$M_{RSM} = V_T / V_R.$$

In the case of embankments an additional stiffening effect of the embankment material needs to be taken into consideration, i.e., the track critical velocity maybe higher than the first Rayleigh wave velocity (Krylov, 1994). An additional parameter, called the Rayleigh track Mach number M_{RTM} can therefore be determined based on the track critical velocity rather than the subgrade Rayleigh wave velocity as

$$M_{RTM} = V_T / V_{TCV}.$$

The effect of the train speed being able to achieve the first fundamental 'Rayleigh mode' is that the track displacement can dramatically increase. The definition of these velocities allows a simple Rayleigh wave mitigation index I_{RM} to be calculated as follows:

$$I_{RM} = (M_{RTM} - M_{RSM}) / M_{RSM} \times 100\%,$$

which allows different types of track mitigation strategies to be directly compared.

In addition to critical velocity effects, if the train passing frequencies coincide with a natural frequency of the track system then further dynamic interaction will occur. Speed increases also relate to the train passing frequency (Auersch, 1990; 2012) increasing to that of the sleeper passing frequency f_p given by

$$f_p = V_T / d,$$

where d is the distance between the sleepers. As the train speed increases a Doppler effect in the passing frequency can also occur to give the slightly different frequency f_d over a frequency band:

$$f_d = f_p\left(1 \pm \frac{V_T}{V_S}\right).$$

Ground vibration from this frequency is attenuated rapidly due to the high frequency components being damped out (Auersch, 2012). Frequencies higher and lower can be attributed to many of the different train generated frequencies such as the axle loading frequencies shown in Fig. 3. For single subgrade homogeneous soils, cut-off frequencies can be identified which will stop ground wave transmission for loading frequencies below the ground fundamental frequencies; their values are given by

$$f_n = \frac{V_p(2n-1)}{4H}, \qquad n=1,2,...,$$

where H is the subgrade depth, n is the frequency number, and V_p is the compression wave velocity given by

$$V_p = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}}.$$

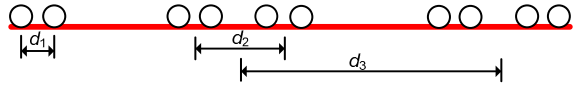

Fig. 3 Influence of bogie distance on train loading frequencies

Ballast and subgrade soils are both non-linear materials, which means that the stiffness varies with many parameters such as the shear strain level. Ballast is a granular material, which means that its stiffness is a function of the confining pressure (Indraratna *et al.*, 2005). Studies at Ledsgard (Madshus and Kaynia, 2000; Paolucci *et al.*, 2003; Takemiya, 2003a; Madshus *et al.*, 2004) showed that non-linear behavior in the geomaterials significantly affected the track behavior. Madshus and Kaynia (2000) commented that shear wave velocities of the ballast reduced from 250 to 150 m/s during train passage. This means that the Rayleigh wave velocity in the embankment and hence the track critical velocity were changing during loading. Fig. 4 shows the effect of the train speed on the ground peak particle velocity (PPV).

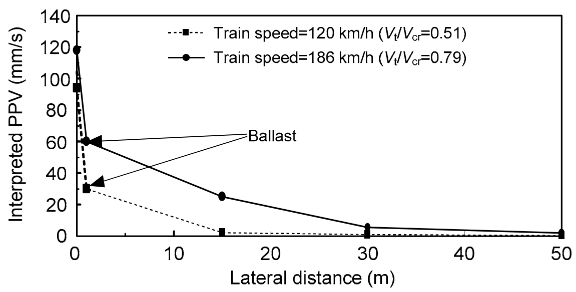

Fig. 4 Variation of peak particle velocity (PPV) with distance (values interpreted from Paolucci *et al.* (2003))
V_t: train speed; V_{cr}: critical velocity

The figure clearly shows that the PPV is increasing as the train speed increases. Values of PPV above 15–18 mm/s have been reported as causing ballast deterioration and loss of compaction (Eisenmann and Rump, 1997; Pita *et al.*, 2004). Recent work by Baeβler *et al.* (2012) has shown that if ballast accelerations exceed 0.7g to 0.8g the ballast will start to decompact, hence a suitable limit is 0.35g. Computer simulations have suggested that once M_{RSM} (or M_{RTM}) exceeds 0.5 the PPV will increase above 18 mm/s as indicated in the measured results in Fig. 4. This suggests that current standards which limit the M_{RSM} value to 0.7 are not conservative with regards to track maintenance, particularly for high-speed trains. Banimahd (2008) performed a series of linear-stiffness computer simulations to estimate the increase in the track PPV with train speed as the Rayleigh wave velocity was approached (M_{RSM}=1.0). Fig. 5 shows the results of these simulations for different subgrade stiffness (ES), ballast stiffness (EB), and ballast depths (BD).

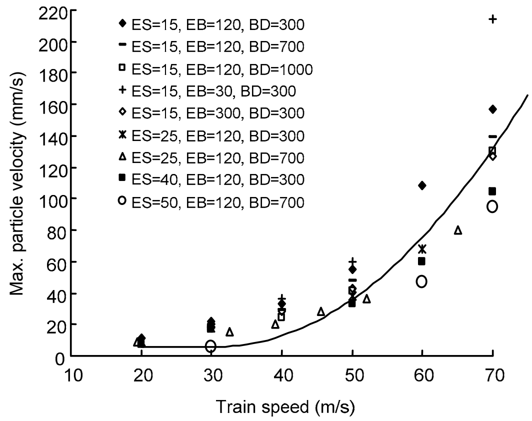

Fig. 5 Simulated increase in track PPV with train speed after Banimahd (2008)
ES: subgrade stiffness; EB: ballast stiffness; BD: ballast depth

If we take conventional track doctrine, i.e., to ensure that the formation stiffness E_{v2}≥120 MPa (the lower layers are less stiff than this) then V_R=511 km/h, which is above current operational speeds (assuming ρ=1800 kg/m^3 and ν=0.5). However if we limit M_{RSM}=0.5 (to prevent high values of PPV) then the train speed limit becomes 256 km/h, which is below some current high-speed ballast lines. This suggests that ballast vibration may be generating higher maintenance levels on some lines at high-speeds. For the Thalys train, Degrande and Schillemans (2001) did not report a strong correlation between the train speed and PPV in the field surrounding the track. Eisenmann and Rump (1997) reported that PPVs greater than 18 mm/s were experienced on ICE lines for train speeds over 200 km/h. Observations therefore suggest that as train speed increases critical velocity, ballast vibration effects will play an important role in track maintenance schedules. Further work in ground vibration can be found in Aubry *et al.* (1994), Takemiya (2003b), Yang *et al.* (2003), Auersch (2005; 2006; 2008), Lombaert *et al.* (2006), Galvín *et al.* (2010), El Kacimi *et al.* (2012), and Woodward *et al.* (2012b). The ability to reinforce the ballast is therefore a key element to improving track quality. Such a technique is the 3D polymer reinforcement of track ballast.

4 3D polyurethane reinforcement

It is considered that any proposed reinforcement technique must satisfy the following criteria in order to provide cost-effective solutions:

1. The treatment must provide 3D elemental type reinforcement across the track.

2. Increases in ballast stiffness and strength should be designable to suit operational and track requirements, i.e., the problem in question.

3. The material must exhibit ductile properties and hence have a tendency to adsorb energy to prevent brittle types of failure.

4. The treated ballast should remain free draining.

5. The technique should require the minimum of permanent-way (p-way) work before installation.

6. The treatment must be relatively quick and return the track to operational use by the end of the possession.

7. Breakdown of the treatment should revert the track back to a normal ballast state (built in fail safe).

8. The treatment should primarily be considered for long-term solutions to significantly reduce future track maintenance.

9. The treatment must satisfy environmental regulations.

10. From a holistic point of view the treatment must be cost-effective.

XiTRACK has been specifically developed to fulfill all of these requirements. It uses tailored visco-elastic polymers that are urethane cross-linked to form an in-situ 3D polymer/ballast geo-matrix. Fig. 6 shows that the polymer is applied to the exposed surface of the ballast where it proceeds to cure as it penetrates into the ballast to form a 3D reinforcing cage down to a specified depth (set by the polymer rheology).

The polymer is applied in-situ by mixing two components through a delivery hose, which contains mixing elements.

Typically the polymer cures within 10 s, forms 50% of its strength within minutes and reaches 90% of its strength within 1 h (absolute rates are dependent on the ambient temperature). The polymer satisfies the requirements of the UK Environment Agency and can typically develop strains in excess of 100% before failure in tension. The benefit of using the polymer as the reinforcing element is the ability to design its rheology, strength, stiffness, damping properties, cure rates etc. The ductility and damping properties of the polymer reinforcing elements make it an ideal material for railway environments where operating conditions can result in suddenly high dynamic loads (e.g., insulated bolted joints, wheel flats). Breakdown of the polymer leads to conventional ballast and drainage is still maintained. Provided the ballast voids are relatively clear the treatment can take place with the minimum of p-way work. It is normal however that the ballast be excavated down to the sleeper bottom level to allow conventional maintenance (e.g., shovel packing and/or stone blowing) to occur should the need arise; this gives the p-way engineer added confidence in the use of the technique. Although apparent adherence to the ballast will occur in many conditions, the primary function of the treatment is to generate polymer reinforcing elements at every level in the ballast matrix, both vertically and horizontally (Fig. 7). Adherence to the ballast is not therefore required for the technique to work and hence it can be used on contaminated ballast, provided the voids are not blocked. Fig. 8 shows the testing of a typical XiTRACK GeoComposite in GRAFT I at Heriot-Watt University (UK).

Fig. 6 Application of the polymer XiTRACK to the ballast surface

Fig. 7 Cross-section of the GeoComposite

Fig. 8 Testing of the XiTRACK Polymer in GRAFT I at Heriot-Watt University (UK)

The GRAFT I facility at Heriot-Watt University consists of a trackbed constructed within a steel tank 1.072 m×3.0 m×1.15 m (width×length×height). The track includes three to five sleeper sections, which replicate one half of a twin block sleeper used in the field, overlain by an I-section with similar stiffness properties as a BS 113 A rail section. Cyclic loading is applied to the track from a 200 t capacity hydraulic loading actuator that can apply realistic loads of both typical passenger and freight traffic. These loads generate realistic stress levels in the ballast and subgrade layers (Kennedy *et al.*, 2012). Track settlement and ballast layer stress levels can be monitored during each test through instrumentation that is connected to a data acquisition system.

The results of the test shown in Fig. 8 are reported by Kennedy (2011). The specimen was cycled for 18.3 million gross tonnes (500000 cycles) at a tangent subgrade modulus of 24.7 MPa. The accumulated plastic deformation was approximately 0.6 mm.

4.1 Example application: Manningtree North Junction UK

Manningtree North Junction is situated on the Colchester to Ipswich line UK. During 2008, track renewal of the points at Manningtree North Junction took place and XiTRACK polymer reinforcement of part of the renewals area was applied. Specifically, the area between CH1355 and the viaduct interface at Bridge No. 228 on the Up Main line was reinforced in the vertical direction only to help strengthen the formation (around 60 m of track). Fig. 9 shows this area.

Trains (including heavy freight) run-off the viaduct bridge onto an old embankment and then pass over an underbridge (Underbridge No. 227) around CH1370 towards Points 1259B. It had been reported that a noticeable deflection occurred between the viaduct and the underbridge. This deflection was thought to be due to movement of the embankment itself and due to sudden changes in track stiffness, leading to larger track forces, vibration, and hence increased formation deflection. The formation comprises sand, clay, and made ground. It is likely that the embankment is of the Victorian type comprising made ground from the local area (assumed mainly to be alluvial clays topped by ash) but very little site investigation data were available.

Fig. 9 Manningtree North Junction UK

The line is accessed by container trains from Felixstowe and by Class 90 hauled passenger stock; axle loads therefore vary between 13 and 25 t. The treatment was based on an assumed multiple-axle secant track stiffness in the plain-line track area of approximately 67 MN/m assuming a standard 300 mm ballast depth and freight loading conditions. The limits of the treatment include the requirement to leave 300 mm below sleeper bottom (BSB) of unreinforced ballast below the switch & crossings and the depth of track over Underbridge 227, estimated to be 400 mm BSB. In addition, the time for renewal was also limited, restricting the additional depth of ballast below 300 mm that could be treated with the XiTRACK polymer.

To accommodate these requirements, the proposed minimum depth of XiTRACK treatment below the unreinforced 300 mm of ballast was specified at 150 mm. This meant that the total minimum depth of ballast to be renewed was 450 mm BSB in the 6 ft (1 ft=0.3048 m) area. The XiTRACK application (Fig. 10) could therefore take place continuously to reduce any disruption to the general track renewal. The normal ladder type structure for timber deck bridge transitions was modified to an enhanced section at Viaduct Bridge No. 228. This allows the track line and level to be reinstated at anytime and allows for ballast reinforcement of the shoulders. To allow 300 mm BSB of unreinforced ballast only the bottom 100 mm of ballast above Underbridge No. 227 was proposed to be reinforced due to the limited depth. A thin chipping layer was recommended between the ballast and the concrete bridge deck to help prevent any pooling of the polymer directly on the concrete underbridge itself.

Fig. 10 Typical installation equipment

The general structure and arrangement of the XiTRACK treatment is shown in Fig. 11. As shown in the figure, the treatment was split into several zones. In addition to the main zones, the cross-section of the track was split into sections, this was to allow an increase in GeoComposite strength directly under the railhead where the developed tensile stresses are higher. Typically in Zone C, the GeoComposite stiffness was likely to be approximately 500 MPa under the railhead.

Zone A: The GeoComposite stiffness was reduced from Underbridge No. 227 towards Points 1259B (towards Colchester, UK). The unreinforced ballast depth was 300 mm and the minimum GeoComposite ballast depth below this was 150 mm.

Underbridge No. 227: The GeoComposite stiffness was kept constant over the underbridge. The unreinforced ballast depth was 300 mm and the minimum GeoComposite ballast depth below this was 100 mm.

Zone B: The GeoComposite stiffness was increased towards Underbridge No. 227 (from Zone C). The unreinforced ballast depth was 300 mm and the minimum GeoComposite ballast depth below this was 150 mm.

Zone C: The GeoComposite stiffness was kept constant over this zone. The unreinforced ballast depth was 300 mm and the minimum GeoComposite ballast depth below this was 150 mm.

Zone D: The GeoComposite stiffness was reduced from Viaduct Bridge No. 228 towards Zone C. In addition, the GeoComposite was used to form an enhanced section to further strengthen the bridge run-off transition.

Sequence of construction: With reference to Fig. 11, the sequence of track work for the XiTRACK treatment of the track was adopted as follows:

1. Remove the spent ballast down to the designed depth.
2. Ensure that the formation is as per Network Rail Standards, including the formation slope (crossfall towards the Cess).
3. Over Underbridge No. 277 add a layer of fine chippings directly over the concrete deck.
4. Over the formation lay a Network Rail approved geogrid (SSLA30). The geogrid must not run over the concrete deck of Underbridge No. 277 and should therefore terminate as per manufacture's standards.
5. Replace the ballast to a depth of 300 mm BSB.
6. Compact the ballast to Network Rail Standards.
7. Apply the XiTRACK polymer (ensuring a separation gap between 50 and 70 mm at the concrete deck/soil interface).
8. Form the GeoComposite enhanced section at Viaduct Bridge No. 228 track interface (two layers of 150 mm pours required).
9. Replace the upper unreinforced ballast and compact to Network Rail Standards.
10. Replace the upper track superstructure including sleepers and rails.
11. Using a tamper reinstate track line and level as per specification.

The treatment area is fully maintainable using all conventional Network Rail equipment. Fig. 12 shows the transition area around the concrete viaduct (Zone D). As discussed one of the benefits of using polyurethane techniques is the ability to vary the ballast stiffness and strength simply by increasing the amount of polymer applied. In addition, if multiple polyol containers are available then the polyurethane polymer itself can be varied to give new properties. In this way the application can be optimized for the track problem at hand; in particular the treatment can be varied on-site if any problems suddenly develop. This occurred at Manningtree North Junction when the existing track ballast was excavated. Within the renewal area World War II tank traps were discovered to prevent foreign military vehicles accessing the viaduct (Fig. 13). These traps were still in place even after several track renewals had occurred since first construction.

Fig. 14 shows the polymer being applied at the site to form the polymer geocomposite across the treatment area and thus form the lower reinforced geopavement. The polymer flow rate was around 16 kg/min. The main pump was electrically driven and was supplied with polymer components from the two IBC transfer pumps. A static mixing head was used passed the main pump.

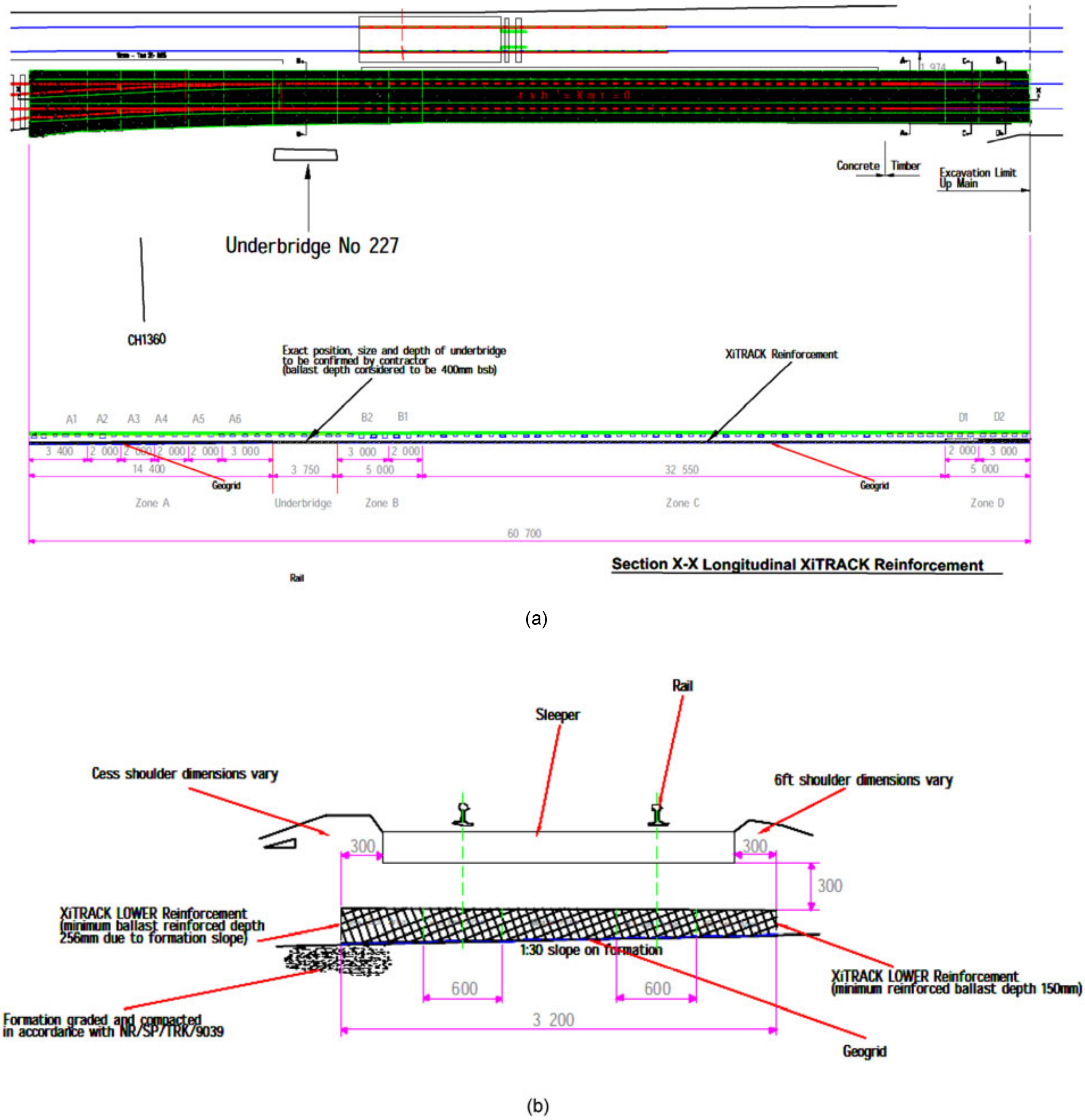

Fig. 11 XiTRACK polyurethane reinforcement diagrams at Manningtree North Junction UK
(a) Longitudinal-section; (b) Cross-section

Fig. 12 Concrete viaduct interface

Fig.13 World War II concrete tank traps

Fig. 14 Application of polymer at the site

The site was treated in June 2008 and to date has performed very well. The lower reinforced layer is therefore acting as a resilient geopavement and thus helping to prevent the development of differential settlement across the site.

4.2 Example application: Falkirk High Tunnel Transitions

Fig. 15a shows the approach to Falkirk High Tunnel at the East Portal (ELR Engineer's Line Reference EGM1), mileage 22.36 (1 mileage=1.609 km). The affected line is the Down Line shown as the left track in the photo. Fig. 15b shows the transition area from the ballast plain line track to the concrete slab -track and clearly shows the application of track spreader bars in order to retain the track and gauge integrity due to failure of the concrete fixings and upper slab.

This damage can clearly be seen in Fig. 15c which shows the upper concrete slab-track in the tunnel near the portal end. This deterioration is considered to be due, in part, to movement of the ballast on the approach to the concrete transition. 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 train speed. The wet bed formation was also evident due to the appearance of 'apparent' mud pumping of the aggregate below the ballast layer. It was initially thought that the change in track stiffness, as trains go from plain line ballasted track, to track with rock at depth, and then over the concrete slab-track transition was generating larger dynamic forces. The effect of the wet beds would be to lower the track stiffness generating increasing vertical movement of the rail, which in turn would increase the vertical force on the concrete pandrol housings. The direction of the force would then change as the train wheel passes over the pandrol housings causing cyclic damage to the concrete; this damage is highlighted in Fig. 15c. In an attempt to reduce the loading on the transition slab the existing approach sleeper spacing on the ballast had been reduced, however site observation indicated that voiding under these sleepers was still occurring. This was confirmed by the track recording vehicle (TRV) traces. The track is used by both freight and local traffic.

Site investigation work by the local contractor was performed in the Cess area in order not to interfere with the operational running of the trains. The proposed sequence of construction for the renewal was:

1. Excavate the ballast and Type 1 granular soils.

2. Add 300 mm of new ballast in layers and compact as per Network Rail Standards.

(a)

(b)

(c)

Fig. 15 Transition at Falkirk High Tunnel
(a) Approach to tunnel; (b) Ballast/slab-track interface; (c) Damage to slab-track due to transition failure

3. Align the upper ballast surface of the final ballast layer as per the track cant.

4. Apply the polymer, before the polymer cures add grit to the surface of the GeoComposite.

5. Replace the ballast in layers for the remainder of the transition and compact to Network Rail Standards (to bring the ballast up to sleeper bottom level).

6. Replace the sleepers and rails.

7. Add the boxing ballast in the crib shoulder areas. In the shoulder areas, the replaced ballast depth (to the lower XiTRACK reinforced zone) was not to exceed 240 mm.

8. Correct track geometry and alignment.

9. Apply the polymer in the Cess and 6 ft shoulders to form the lateral edge beams.

10. Before the polymer cures add red grit to the surface of the polymer.

11. Add the remainder of the boxing ballast to the Cess and 6 ft shoulders.

However, when the ballast was removed (at the time of application), it rapidly became apparent that a concrete transition slab existed in the transition zone, i.e., a lower concrete slab with ballast on top (Fig. 16).

Fig. 16 Discovery of a concrete transition slab at application

Once again, the lack of an adequate site investigation by the local contractor had not identified critical sub-surface features resulting in changes to the polymer application profile at the time of application. It became apparent that one of the main damage mechanisms was ballast attrition over the transition slab, i.e., the influx of water onto the track due to the poor slope drainage system which was generating wet beds and subsequent ballast grinding between the base of the sleeper and the upper section of the lower concrete transition slab. This ballast material was then being pumped to the ballast surface.

The result was the generation of vertical irregularities and hence increased dynamic loads and vibrations. The design was therefore modified on site (Fig. 17) to account for the discovery of the under track transition slab. Fig. 18 shows the application of the polymer over the concrete transition slab.

The polymer was applied in December 2005, and to date has performed very well without any further transition issues being reported.

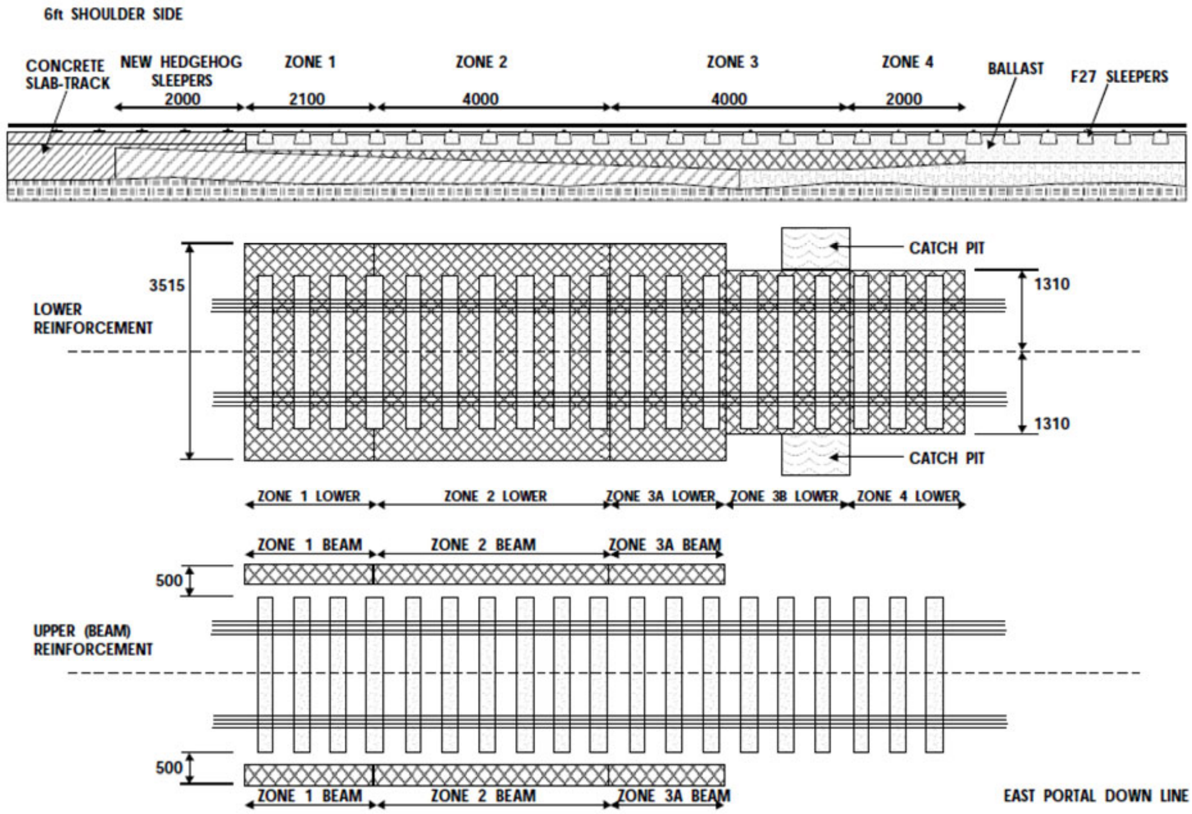

Fig. 17 Modified polymer application profile at Falkirk High Tunnel

Fig. 18 Application of the polymer over the concrete transition slab

5 Conclusions

The development of plasticity in the ballast at critical track assets like switch & crossings and transitions leads to track settlement and hence increases in the dynamic track forces and vibration levels in the ballast. It has also been observed that as train speeds increase the peak particle velocity in the ballast increases, particularly when t he train speed passes 50% of either the Rayleigh wave velocity of the subgrade (defined by the Rayleigh subgrade Mach number) for conventional track structures or the track critical velocity (defined by the Rayleigh track Mach number) in track structures that contain additional structures such as embankments. The increase in the ballast vibration level causes the ballast to migrate and hence can lead to increases in track maintenance. Movement of ballast is therefore not only due to penetration of the ballast into the subballast and/or formation, but also from the ballast vibration level. It has been reported that if the peak particle velocity level exceeds 18 mm/s then ballast destabilization can occur. The ability to stabilize the ballast structure against these types of loading conditions would therefore be very beneficial in reducing ballast settlement and migration.

The application of 3D polyurethane reinforcement provides such stabilization technology. The technique has been applied at many locations in the UK and been found to provide a high degree of track stability, significantly reducing future track maintenance. Although the maximum line speed that the system has been used on is 200 km/h, it is currently being considered for line speeds of 300 km/h. Due to the ability to optimize the ballast stiffness for these speeds the technique has significant potential to stabilize the ballast track structure on high line speeds.

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