The influence of structural response on ballast performance on a high speed railway

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Track for modern ballasted high speed railway lines typically uses continuous welded rail with continuous ballast. Continuous ballast is often specified over features involving discontinuity of track support including structural movement or expansion joints found in long railway bridges. Accelerated degradation of track geometry has been observed at these types of location, resulting in unplanned maintenance. A location was identified on a viaduct of an operational high speed railway, where a reoccurring track defect develops just following a structure expansion joint which was designed for continuous ballast. Trackside monitoring techniques have been used to capture the response of the track in the vicinity of the defect and of the bridge spans on either side of the structure expansion joint under normal operational conditions and to evaluate a typical maintenance process. This gave insight into the performance of the track, demonstrating that the defect was due to voiding and recurring as maintenance was ineffective at filling these voids. Monitoring also provided evidence of bridge behaviour which could have an adverse effect on the ballast over the joint and may be responsible for the original formation of the defect. Evidence from this monitoring has given new insights into the reasons for defect occurrence and recurrence allowing for a more informed approach to specifying maintenance, given the knowledge that there is little that can be done to alter the behaviour of the viaduct structure without major intervention.

Introduction

Many high speed railway track systems use continuous welded rail (CWR) with continuous ballast. Continuous ballast is favoured as it allows automated maintenance, meaning it is often specified where there is a discontinuity in the track support, such as transitions on or off fixed structures or over structural movement/expansion joints. These features are frequently associated with accelerated deterioration of the track. Li and Davis (2005) discuss this problem at transitions onto railway bridges, attributing deterioration to either increased dynamic forces arising from the abrupt change in stiffness or differential settlement between the free track and fixed structure, as observed by (Coelho et al., 2011). Other actions or interactions may be responsible for deterioration at other locations, particularly where the behaviour of a structure or sub-structure could affect the ballast (Dahlberg, 2010). If the track geometry deviates beyond a certain tolerance, that location is defined as having a track defect. Track defects are a significant problem for infrastructure managers as costly unplanned maintenance is required to restore track geometry. Features including discolouration of the ballast and un- or partially-supported sleepers, resulting in excessive track displacements, are symptomatic of track defects and may result in further deterioration (Lundqvist and Dahlberg, 2005). If the causes of defect occurrence and recurrence can be better understood, more effective maintenance may be specified.

This study describes an investigation at a location where a recurring defect was identified just beyond a structural expansion joint for a viaduct on a ballasted high speed railway. This site gave an opportunity to assess the performance of the track and investigate the causes of the track defect with consideration of the influence of bridge and ballast interaction. The results provide an interesting case study on the influence of structural behaviour on ballast performance and they have informed ongoing maintenance and mitigation.

Materials and methods

The Site

The site investigated is located just beyond a structural expansion joint, where rapid recurrence of a track defect has necessitated regular unplanned maintenance. The joint studied is between two triple span decks of a multi-deck viaduct (Figure 1). Each deck is 30 m long with the exception of the middle span of the deck approaching the defect, which is 34 m. At the joint CWR is supported by a 500 mm depth of ballast over a steel spanning plate between decks (Figure 2). The plate is fixed to the deck approaching the defect and free to move on the other. This design is used for all joints within the bridge and defects were observed at other joints between equal length deck sections. The defect at the joint studied was the most severe, however.

Joints are included to accommodate seasonal expansion of the bridge decks which is about 30 mm. Diurnal variation is likely to be small owing to the shading and thermal mass of the structure. Although the annual variation may be significant, the rate of defect recurrence implies a more dynamic process could be affecting the track.

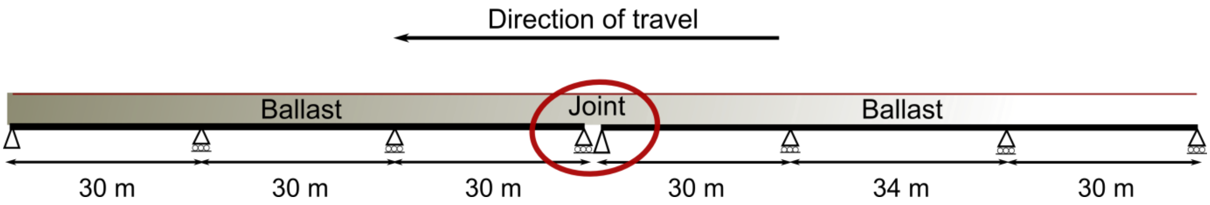

Figure 1 Configuration of bridge decks either side of structural expansion joint studied

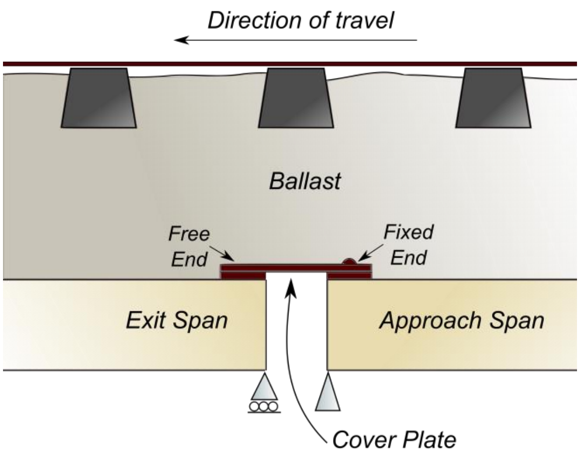

Figure 2 Sketch of expansion joint design.

Bridge behaviour in response to moving trains is generally well understood (Frýba, 1996). The continuous triple span decks and the variation of deck configuration either side of the joint could lead to interesting bridge behaviour. The continuous triple span deck means first three modes of response will occur in a dense frequency region and the extended span in the middle of the approach deck should lower one of these natural frequencies (assuming constant mass per unit length and flexural rigidity). The joint design means that bridge behaviour, particularly at the joint, could affect the ballast. This is of interest as previous investigations into track and bridge interaction have typically focussed on the effects on the rail rather than the ballast (Calcada et al., 2008).

Trackside Monitoring

Trackside monitoring based on methods described by Bowness et al. (2007) have previously been deployed to understand the reasons for track deterioration at certain sites (Coelho et al., 2011, Priest et al., 2012, Le Pen et al., 2014). A monitoring scheme utilising the same technologies: geophone sensors and high speed video recording for digital image correlation (DIC), was deployed at the site (Figure 3). The instruments were positioned on the up line track and the corresponding side of the bridge. The purpose was to capture the response of the track before, within and after the defect zone and of the bridge on either side of the structural expansion joint. This provided evidence of how the track and the bridge were performing and allowed investigation of bridge behaviours that could affect the ballast. Monitoring was also used to asses directly the effectiveness of a maintenance intervention.

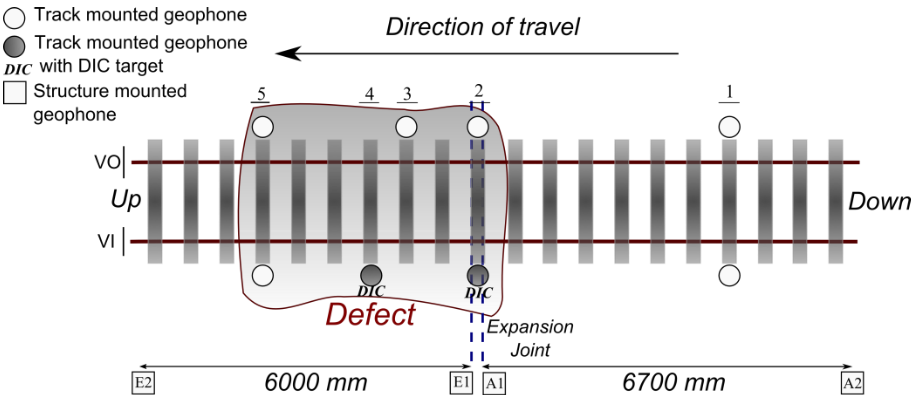

Figure 3 Schematic of instrument positions on the track and bridge.

Results

Data was captured for trains passing on both up and down lines. The geophones measured velocity which was converted into displacement time histories by high- and low-pass filtering then integrating. Displacements were obtained from the video recording by applying cross correlation between recorded video frames, then low-pass filtering. Results are presented for the response to the passage of a 6 car high speed train. Although results are from different trains, they are considered to be characteristic of a normal response. Displacement-time histories are given for the track on the up line only, about three months after the last maintenance (Figure 4 and Figure 5) and for the day before and the day after a more recent maintenance intervention (Figure 6). Displacement time histories are given for the bridge from trains independently passing on both up and down lines (Figure 7) and squared amplitude velocity spectra are presented for the bridge to show the frequencies from excitation and response (Figure 8 and Figure 9) for the same train passage as the first on-track results.

Track Performance

The geophone results from the track had bridge displacement subtracted from them to give relative displacement between the bridge and track. This was done using the bridge measurement and accounting for the relative positions of the instruments, assuming that each span responds in its first mode shape that can be approximated by a half sine wave. The camera used for DIC was fixed to the bridge so no correction to this result was required. DIC was found to be more effective when track displacements became large.

Figure 4 shows the displacement time history for a sleeper ahead of the defect on the approach span. Displacements were small: less than 0.4 mm. Similar results were obtained for a sleeper after the defect zone on the exit span. This magnitude of displacement is expected for the depth and quality of ballast, indicating the track is performing normally before and after the defect zone.

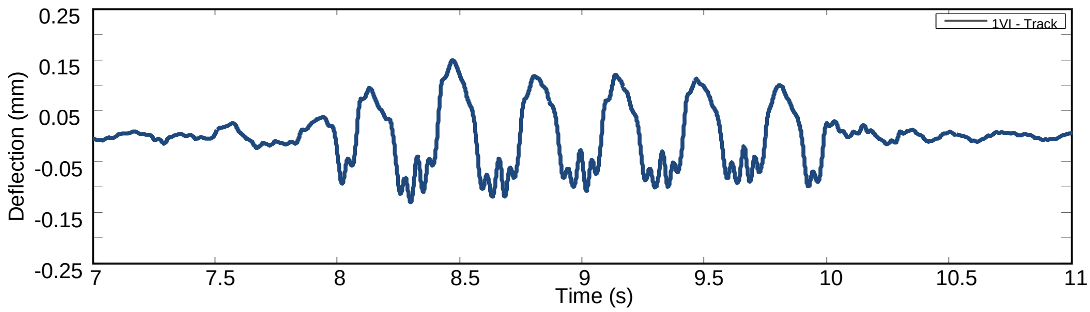

Figure 4 Track displacement time history at location 1VI, obtained from geophone data, filtered between 1.5 and 40 Hz

Figure 5 shows the displacement time history for a sleeper in the defect zone. Displacements were large: greater than 5 mm for this train. Similar results were obtained at another sleeper in the defect zone. These displacements are much larger than would be expected. The track is performing poorly, indicating that sleepers may be voided (un- or partially-supported) in the defect zone. These large displacements were clearly visible at the site as was movement of ballast at the surface.

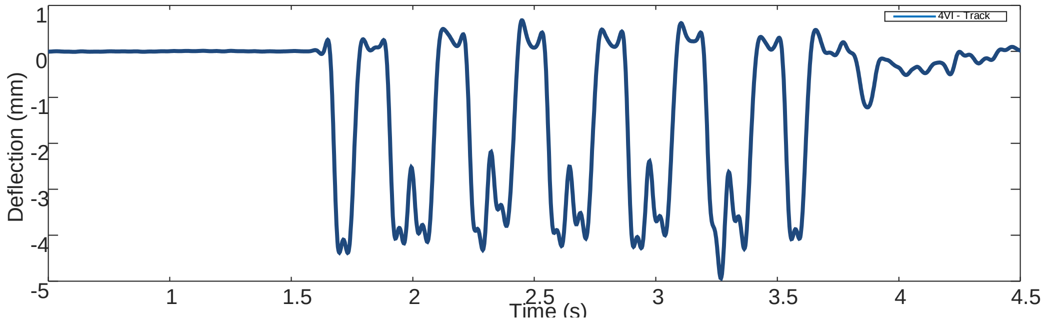

Figure 5 Track displacement time history at 4VI, obtained using DIC, low-pass filtered at 40 Hz.

Maintenance

Figure 6 shows the displacement time history for a sleeper in the defect zone both before and after tamping. Before tamping displacements were large, greater than 5 mm, and these were reduced by the intervention to about 2.5 mm. Although a reduction was observed, displacements were still large compared to the measured performance outside the defect zone. The implication is that the intervention was not effective at fully restoring track performance.

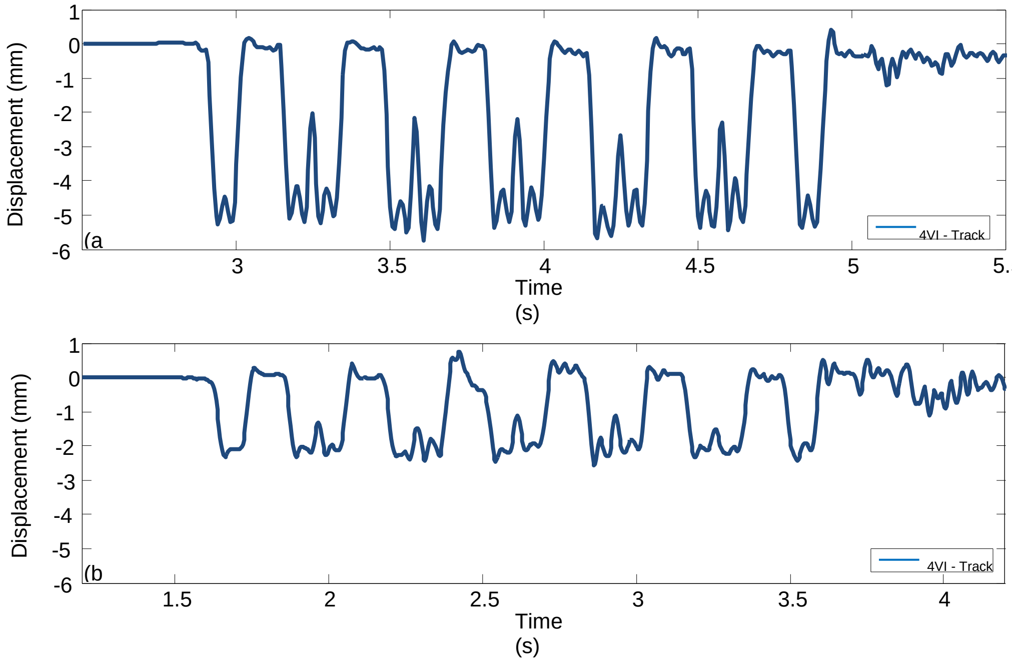

Figure 6 Track displacement data at 4VI (a) before and (b) after tamping, obtained using DIC, low pass filtered at 40 Hz

Bridge Performance

Figure 7 shows the displacement time histories of the bridge spans on either side of the expansion joint from the passage of a train on the up and down lines. For the train on the up line the amplitude of displacement varies from its largest when the train is passing over the instrumented span (about 0.4 mm), decreasing when the train leaves the instrumented span but remains on a continuous deck (about 0.1 mm) and then becomes smaller for the free vibrations when the train has left the deck and bridge. When the bridge was loaded on the down line the measured displacements were smaller whilst the train was on the instrumented spans, but similar once the train had left.

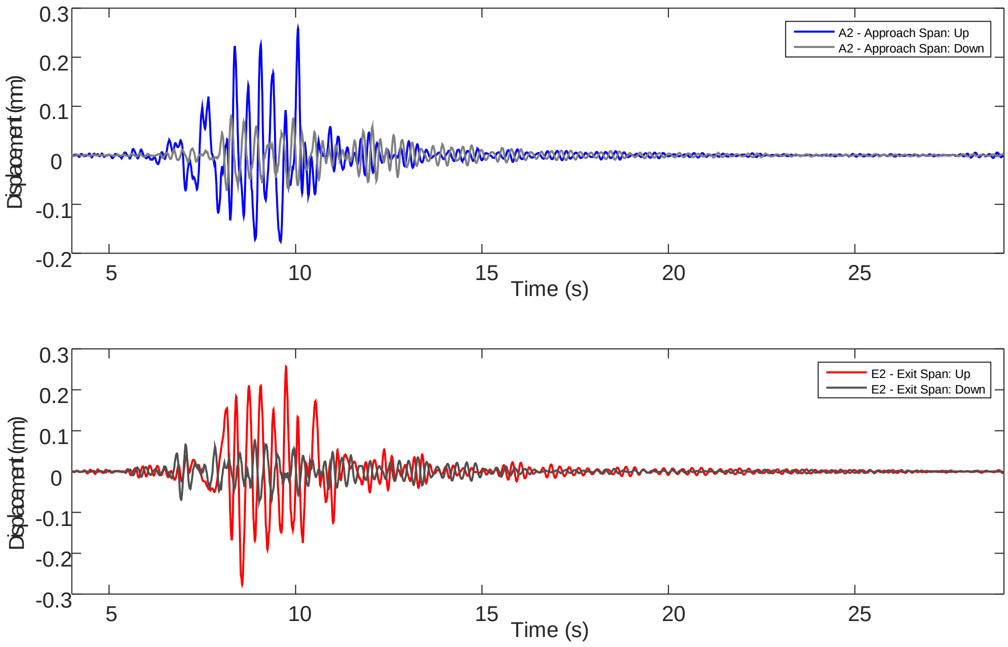

Figure 7 Bridge span displacement time history from a train passing both on up and down lines on (a) the approach span and (b) the exit span, obtained from geophone data, filtered between 1.5 and 40 Hz.

Figure 8 shows the frequency content in the entire velocity signal used to obtain displacement. It shows the frequencies of excitation from a train and of the bridge response. The peaks between about 2.5 and 6.5 Hz are of interest as this suggests the frequencies of excitation and response may be similar. This can be investigated further by computing the frequency spectrum from narrower segments of the time histories (Figure 9), when it is likely that either the frequencies of excitation (solid line for when the train was passing the instrumented span) or of response (dashed line for when the train had left the bridge) will be more significant.

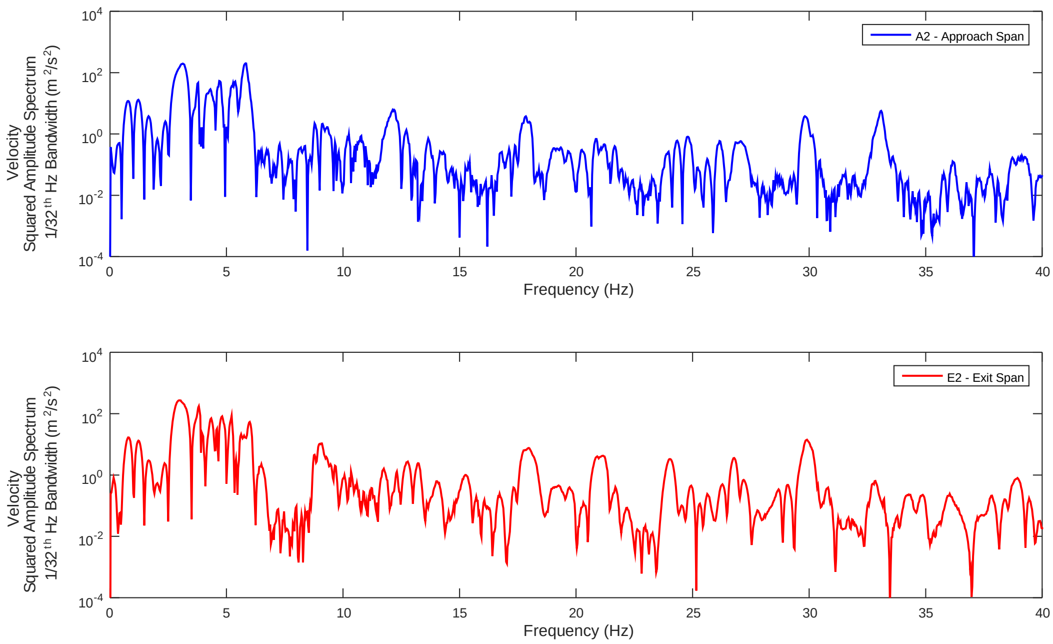

Figure 8 Velocity squared amplitude spectrum with 1/32 Hz bandwidth, from a train on the up line of (a) the approach span and (b) exit span, obtained from full veloctity time history of geophone data.

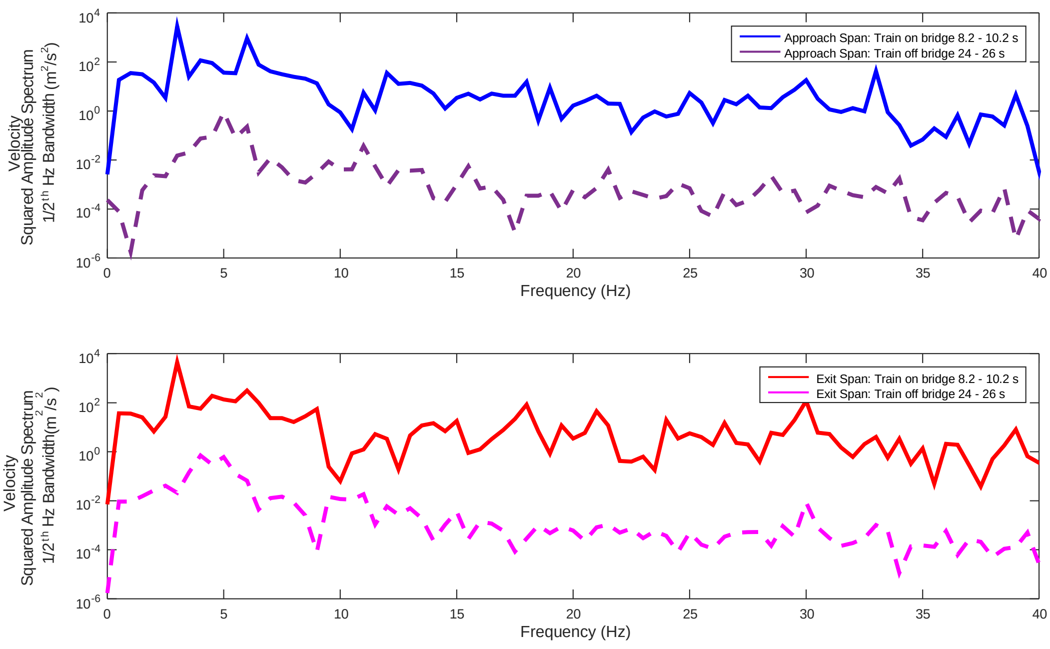

Figure 9 Velocity squared amplitude spectrum with 1/2 Hz bandwidth, from a train on the up line of (a) the approach span and (b) exit span. Obtained from truncated 2 s portion of the veloctity-time history from the geophone data when the train is known to be passing over the span between 8.2 and 10.2 s into the record, and when the train is known to have left bridge between 24 and 26 s into the record.

In Figure 9 for the spectra found for when the train is passing both instrumented spans there is a distinct peak at about 3 Hz which is not present in the spectra for free vibration, suggesting that this is a significant frequency of excitation. For the operational speed of this railway 3 Hz corresponds to a wavelength of about 20 m, a typical car length. On the approach span there are other distinct peaks at 4 and 6 Hz, the 6 Hz peak being more significant. These are present yet less distinct on the exit span. The peaks in the spectra of the free vibration are most significant at 5 and 6 Hz for the approach span and at 4 and 5 Hz for the exit span.

As the significant frequencies of excitation and free response are close together, under steady state conditions dynamic amplification could be significant. As the passage of a train is a transient event, steady state excitation cannot be achieved. However, if the response of the structure to a single action (an axle load) is similar to the frequency at which that action is repeated (e.g. car passing), superposition of the exciting action and response to previous actions could still lead to dynamic amplification, during the loading event (Frýba, 2001, Xia et al., 2006).

Figure 10 shows the measured bridge displacements on either side of the expansion joint over-laid with a measurement taken at a sleeper directly over the expansion joint. This allows the bridge behaviour to be inferred relative to the position of the train. This suggests that the maximum downward displacements of the bridge spans on either side of the joint occur in-phase and the track above the joint is unloaded when this occurs. This process appears to occur close to the car passing frequency.

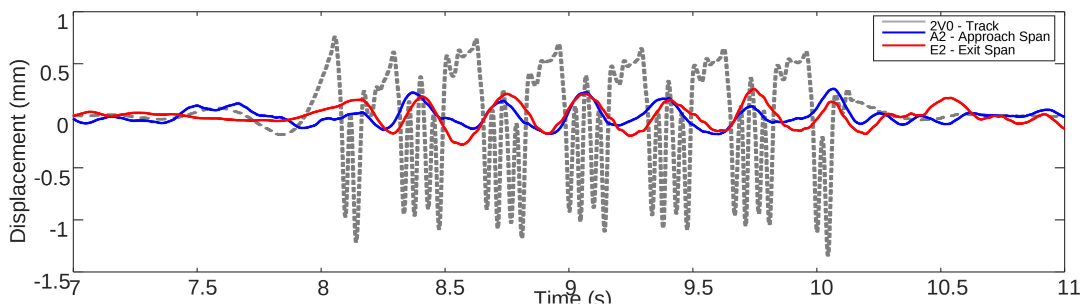

Figure 10 Displacement time history of both bridge spans measured at A2 and E2 and the sleeper over the expansion joint at 2VO, obtained from geophone data filtered between 1.5 and 40 Hz

Discussion

Trackside monitoring has demonstrated that localised performance within the defect zone is poor. Displacements were shown to be large and to vary significantly over just a few bays, due to voiding. A typical intervention was shown to be ineffective at restoring track performance to normal and the defect is thought to reoccur rapidly. This is possibly because the track intervention was unable to restore state where track displacements are normal. These results and observations show that the response depends on the state of the track at the time of measurement. They also suggest that this state may be responsible for or influence the rate of defect reoccurrence, as the ballast is still disturbed by excessive movement after maintenance. However, no behaviour or processes which may have initiated the defect or triggered the cycle of accelerated deterioration could be identified using just the track data.

The bridge behaviour is interesting. The frequencies of excitation (moving train loads) were shown to be close to the frequencies of response (free vibration) meaning dynamic amplification could have a significant effect on the bridge behaviour. The response of the differently configured decks was seen to differ. There is evidence to suggest that three-dimensional effects (e.g. torsion) may be affecting the bridge, particularly during loading. This behaviour may be influencing the severity of the defect at the joint studied, but comparison with another joint is required.

The data also suggest that the maximum displacements of the bridge spans on either side of the joint are in phase, at the car passing frequency. This process will lead to simultaneous deck end rotations, which will cause a cyclic expansion and contraction of the joint. This behaviour has been confirmed using displacement measurements. As the ballast is continuous over the joint the bridge behaviour should cause extension-contraction in the ballast (Figure 11), which could result in mechanisms responsible for or conditions favourable to the formation of track defects. However this behaviour may occur at all the joints on the bridge.

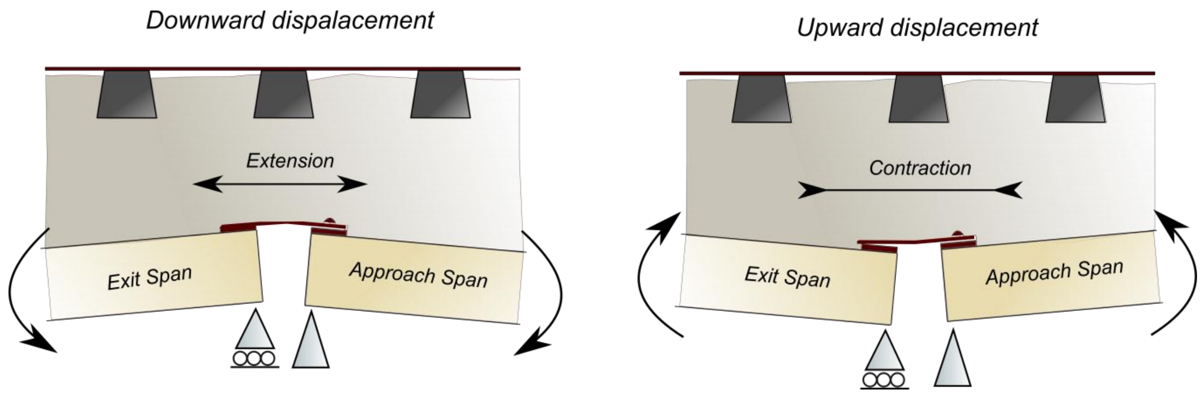

Figure 11 Illustration of extension-contraction behaviour in ballast due to rotation at decks ends. Rotations exaggerated

As the joint design means that the ballast is continuous over the joint this cyclic extension-contraction behaviour could cause localised settlement. If permanent, this may lead to voiding beneath sleepers at this location. Alternatively it could result in a dilative process, which might locally affect structure of the ballast such that it is unable to provide adequate resistant to loads from passing trains or is more susceptible to vibration. Further research into the effects of the bridge behaviour on the ballast is needed to establish in detail how defects may form due to interaction with the bridge.

Both the state of the track and bridge/ballast interaction have implications for maintenance and mitigation. Although the bridge behaviour may have initiated the defect it is likely that the current state of the track influences the rate of defect recurrence. As it is likely to be difficult and costly to alter the bridge behaviour, mitigation which restores and maintains the track in a stable state, removing and preventing voids, may be more effective. Clearly the bridge may affect any mitigation, but this will depend on how the bridge affects the ballast, any permanent effects and the detail of the mitigation. If it does, it may be that the only mitigation that is reasonably practicable is one which the reduces rate of defect recurrence.

Conclusion

Evidence from track-side monitoring suggests that there are two processes responsible for the occurrence and recurrence of the defect studied. The state of the track governs the ongoing performance at the site and, given that normal maintenance was shown to be ineffective in restoring the track to normal performance, it is likely that track in a poor state will continue to deteriorate rapidly. An extension-contraction of the ballast due to bridge and ballast interaction could be responsible for creating conditions which lead to rapid deterioration at a location where the joint design means bridge behaviour can affect the ballast. The bridge behaviour may be exacerbated by an unfavourable excitation and structural response regime occurring under normal operating conditions. Although further work is required to understand how the bridge affects the ballast, the findings have implications for future mitigation.

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