Bridge and ballast interaction at a viaduct structural expansion joint on a high speed railway

Ponts et ballast interaction à un joint de dilatation du viaduc sur une ligne à grande vitesse

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ABSTRACT Sections of railway line associated with variations in track stiffness often have an increased maintenance requirement owing to the relatively rapid deterioration of track geometry. Variations in stiffness could arise from changes in ground conditions, transitions from free track onto a fixed structure and other localised features. Increased dynamic loads may be responsible for track degradation at sites with these characteristics. This is especially relevant to future high speed rail infrastructure, where increased operating speeds have potential to increase dynamic loading on the track significantly. At present, there is little direct physical evidence linking increased dynamic loading to accelerated geometry deterioration, and monitoring of problem sites is needed to investigate their real behaviour. This paper presents some findings from an ongoing study of a ballasted viaduct on a high speed railway, at which ballast migration has been found to occur in the vicinity of a structural expansion joint leading to the occurrence of unsupported sleepers.. The response of the structure and the track to the passage of high speed trains has been monitored using geophones. Using the resulting data supported by appropriate theoretical modelling, mechanisms for the enhanced track degradation are proposed. The results of the study provide insights into the interaction between trains, the track, the ballast and the structure, and can be used to inform the cost-effective maintenance and design of existing and future high speed rail infrastructure.

RÉSUMÉ Les parties des rails qui sont liées aux variations de rigidité de la voie ferrée ont souvent une exigence de maintenance plus élevées en raison de la détérioration rapide de la géométrie de la voie ferrée. Les variations de rigidité pourraient être le résultat de changements dans les conditions du sol, des transitions d'une voie ferrée à une structure fixée et d'autres fonctionnalités locales. Les charges dynamiques plus élevées peuvent être responsables de la dégradation de la voie sur les sites qui présentent ces caractéristiques. Ce fait serait particulièrement pertinent pour l'infrastructure ferroviaire rapide à l'avenir, dont les vitesses de fonctionnement augmentés ont le potentiel d'augmenter la charge dynamique sur la voie, d'une manière significative. Actuellement, il n'y a pas beaucoup de preuves concrètes liant le chargement dynamique accrue à la détérioration accélérée de la géométrie, et la surveillance des sites de problèmes est nécessaire pour étudier leur fonctionnement. Cet article présente une étude d'un viaduc lesté sur une ligne à grande vitesse, où la migration des ballasts a été trouvé à se produire, entrainant des traverses non supportées, qui développent dans le voisinage d'un joint de dilatation de structure. La réaction de la structure et la voie pour le passage des trains à grande vitesse ont été suivis en utilisant l'aide de géophones. En utilisant ces données soutenus par les modèles théoriques appropriés, les mécanismes de dégradation de la voie sont identifiés. Les résultats de l'étude fournissent un aperçu de l'interaction entre les trains, le ballast, le rail et la structure et peuvent être utilisés pour influencer l'entretien rentable et la conception des lignes à grande vitesse existants et à l'avenir.

1 INTRODCUTION AND BACKGROUND

Track defects, generally qualified as deviations in track geometry, require maintenance to ensure that safe track geometry is maintained. Defects can, in some cases, be attributed to locally enhanced track degradation mechanisms that lead to repeated occurrence of the defect and require additional frequent and costly maintenance.

Locally increased dynamic forces are a common driver for the formation of track defects (Dahlberg, 2010). Increased forces could arise from a variation in support stiffness (either natural or man-made changes in support conditions), curving forces and track-ground or track-structure interactions. However, it is not straightforward to find direct physical evidence linking increased dynamic forces to the mechanisms responsible for observed track degradation.

This paper is concerned with the influence of an under-bridge on the ballast. Although the dynamic behaviour of bridge structures under railway loading is well understood (Frýba, 1996, Yang et al. 2004) investigation into track-bridge interaction is less well reported and has typically been concerned with the effects on the rails (Calcada et al. 2008) rather than the ballast.

This paper describes a site where the dynamic behaviour of the bridge is thought to be affecting the ballast, presenting analyses of results of in-situ monitoring and discussing possible causes of track degradation identifiable from the data.

2 MATERIALS AND METHODS

2.1 The site

The site described in this paper is located on a ballasted viaduct for a high speed railway just beyond a structural expansion joint (Figure 1) in the normal direction of travel. The continual recurrence of a track defect at this location has necessitated frequent unplanned maintenance to ensure acceptable track geometry. A monitoring scheme was implemented to capture the response of the track before, within and after the occurrence of the defect and that of the bridge spans on either side of the structural expansion joint.

Figure 1. Photograph of the defect investigated.

The bridge comprised five continuous deck sections, fourteen support piers with CWR across its entire length. The joint under investigation is situated between two of these sections. Each section has three spans typically 30 m long, with the exception of the middle span of the section approaching the defect, which is 34 m long. The joint is of simple design: the track consists of continuous welded rail (CWR) on concrete sleepers over a continuous 500 mm deep ballast layer supported by a steel plate spanning between the decks. This plate is fixed to the deck approaching the defect and is free to move on the other (Figure 2). Defects were apparent at several joints although none was as severe as the defect presented in this paper.

A function of the joint is to accommodate the seasonal expansion of the decks; a witness mark on the structure suggests a range of movement of about 30 mm. Diurnal variation is likely to be small owing to the high thermal mass and shading of the structure. The frequency of defect recurrence suggests a more dynamic process is involved, hence interest in the bridge response to rail traffic rather than thermal effects.

Other joint solutions exist, including designs where the ballast is physically separated between decks which are typically used in longer span bridges where thermal expansion is more significant. These designs require rail expansion joints, which are associated with increased rail wear. The continuous ballast joint is advantageous for automated maintenance under normal operating conditions and for reduced wear on the rail (Ramberger, 2002, UIC, 2001).

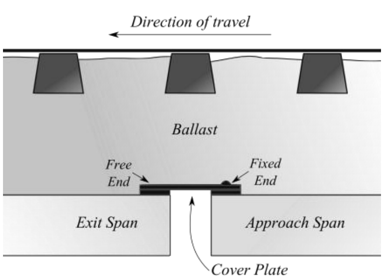

Figure 2. Design of the expansion joint.

2.2 Monitoring

Earlier studies report the use of in-situ monitoring to provide evidence for the causes of track defects at specific sites. Priest et al. (2013) investigated the influence of enhanced lateral forces and Coelho et al. (2011) considered the effects of permanent settlement of track laid between a fixed structure and embankment. Both studies provide insight into behaviour that may have affected the track but also highlight how difficult it can be to identify conclusively a mechanism responsible for track degradation given the number of factors potentially influencing a site.

Monitoring was carried out using geophone sensors as well as high speed video recording track mounted targets for digital image correlation (DIC), described in detail by Bowness et al. (2007). The geophones give an output voltage proportional to velocity above a threshold frequency of about 1 Hz. All data were high- and low-pass filtered with a frequency domain calibration applied to obtain velocities. The signal was subsequently integrated to give displacement.

DIC allows the displacement to be determined via cross-correlation of pixels between frames calibrated to a known length within the frame. Low-pass filtering is used to remove high frequency noise as described by Le Pen et al. (2014).

Figure 3 shows the transducer positions (sleeper number and rail VI or VO). Geophones were used on both the track and the bridge. The camera was fixed to the bridge to capture the movement of the track relative to the bridge. Eight geophones were positioned on the track and five on the bridge. DIC targets were fixed to geophone brackets within the defect zone and at the expansion joint.

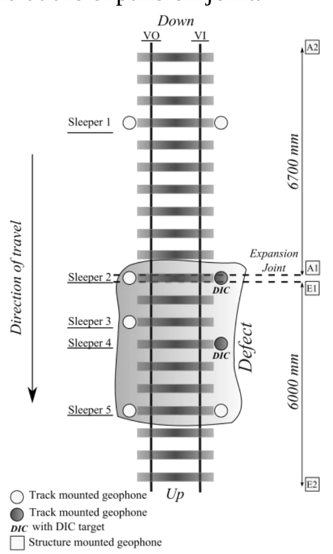

Figure 3. Monitoring schematic.

Data were collected on two occasions, five days after a maintenance intervention (localised tamping) and three months later. The monitoring techniques described above were used to capture the response of the track and the bridge during the passage of high speed trains. The purpose was to provide evidence about the state and performance of the track and bridge which may affect the ballast.

3 RESULTS

Figures 4 to 7 show the displacement-time histories for the instrumented sleepers during the passage of the front ten cars of a symmetrically configured 20 car high speed train. Figure 8 shows the entire train. The geophone data were filtered using a 5th order high pass Butterworth filter with a 2 Hz cut-off and by a 7th order low pass Butterworth filter with a 40 Hz cut-off.

Sleepers 1 and 5 were corrected for bridge movement, to give the displacement of the track relative to the bridge. This has been obtained by subtracting the amplitude of bridge displacements from the total displacement measured at the sleeper. The bridge displacements (Figure 8) have been scaled for span position assuming that the span vibrates in its first mode shape which can be approximated by a half sine wave.

3.1 Before the defect

Figure 4 shows the displacement associated with the ballasted track only. Displacements are small: less than 0.4 mm which is of the expected order of magnitude for a ballasted high speed railway line on a relatively rigid concrete support. There is a regular pattern of displacement associated with the bogies and axles. These results indicate that the track is performing in a normal manner ahead of the defect.

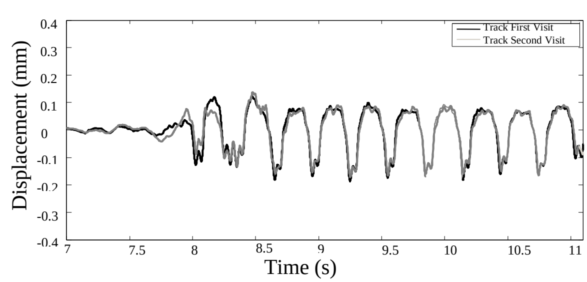

Figure 4. Dynamic track displacement ahead of defect at 1VI. Geophone data filtered 2-40 Hz.

3.2 *Within the defect*

Owing to the large sleeper movements DIC was found to be more suitable for measuring sleeper movements within the defect zone. Figures 5 and 6 present the response of the track over the expansion joint and in the middle of the defect. These results show that the track displacements were in excess of 8 mm at the time of the second measurement. This indicates that the ballast was voided and the track performing poorly.

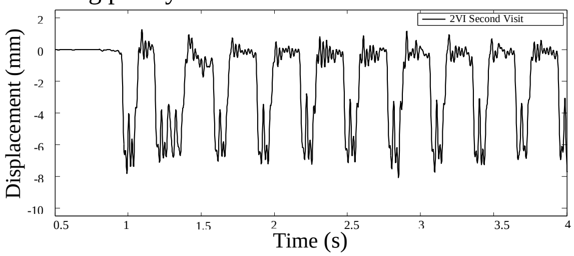

Figure 5. Dynamic track displacement within defect at 2VI: DIC data low pass filtered at 40 Hz.

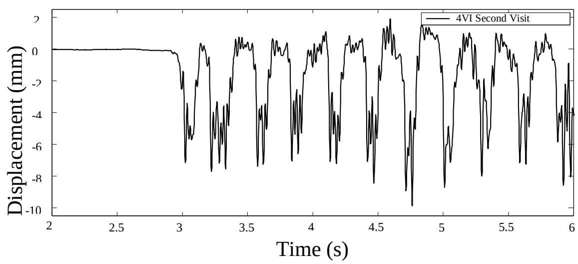

Figure 6. Dynamic track displacement within defect at 4VI: DIC data low pass filtered at 40 Hz.

3.3 *After the defect*

After the defect, the track appeared to perform well, (Figure 7). The pattern of displacement was not as smooth as it was ahead of the defect (Figure 4) but displacements were small (less than 0.4 mm).

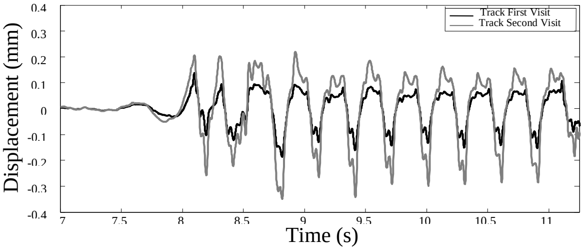

Figure 7. Dynamic track displacement after defect at 5VI. Geophone data filtered 2-40 Hz.

3.4 *Bridge Response*

Figure 8 shows the dynamic displacements measured for the bridge spans 6.7 m ahead of and 6.0 m beyond the expansion joint, which were unchanged between visits. Figure 9 shows Fourier transforms of bridge displacement. The peaks in the figure can be used to identify which frequencies were of significance in the bridge response. The peaks between 3 and 4 Hz correspond to the forced excitation associated with bogie passing. Analysis of the time history suggests the peaks seen between 4.5 and 5 Hz were associated with the free vibrations after the train had left the bridge. The differences in frequency content between the two decks may be attributed to the 34 m span in the approach deck which will lower the natural frequencies of this deck.

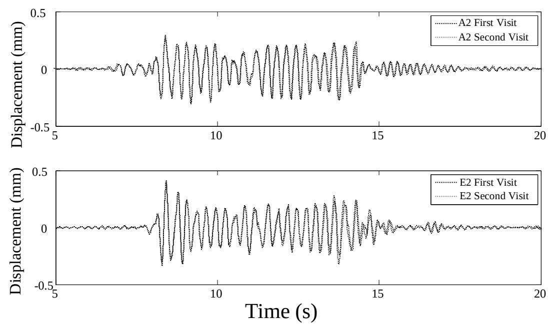

Figure 8. Dynamic bridge displacements at one fifth span. Top: Approach span at A2. Bottom: Exit span at E2. Geophone data filtered 2-40 Hz.

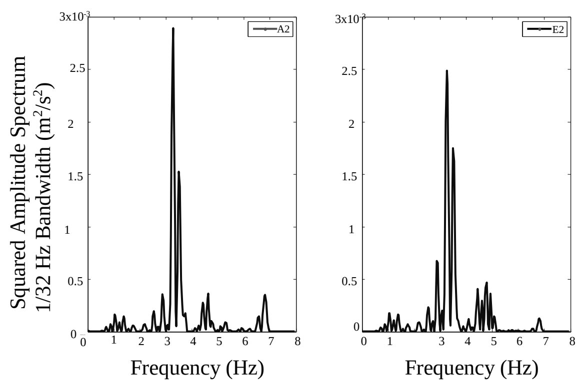

Figure 9. Unfiltered frequency content from bridge geophones. Left: Approach span at A2. Right: Exit span at E2.

4 IMPLICATIONS

The results indicate that the track was performing normally both ahead of and after the defect. Deflections were small and the pattern of response was regular. Within the defect zone the track was performing very poorly. Measurements indicated that certain sleepers within the defect zone were deflecting by over 8 mm indicating that these sleepers were hanging (voided).

The frequency content of the measured bridge response showed that the forced excitation from bogie passing was close to the frequencies of free vibration of the spans, meaning that dynamic amplification could be significant.

4.1 Ballast structure interaction

The behaviour of the bridge on either side of the structural expansion joint is thought to be responsible for the initial occurrence of the track defect and its continuing reoccurrence. The time history in Figure 10 shows the response of the bridge span either side of the expansion joint overlain by the response of the track at the expansion joint. This gives insight into the behaviour of the bridge and track relative to the train.

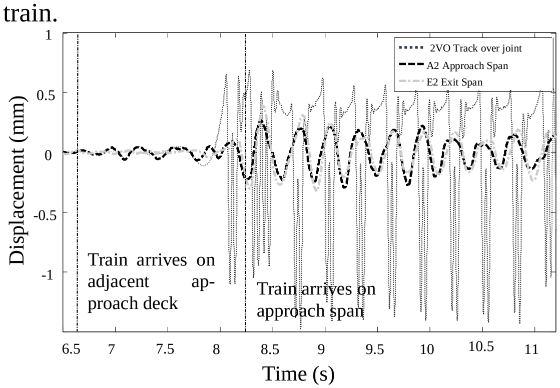

Figure 10. Dynamic bridge displacements, approach at A2 and exit at E2 span overlain by track displacement over expansion joint at 2VO. Geophone data filtered 2-40 Hz.

Figure 10 shows that the maximum vertical (down-ward) displacement of the bridge spans on either side of the joint occurred simultaneously, when the track over the expansion joint was unloaded. This is illustrated schematically in Figure 11. The deck ends rotate in opposite senses, resulting in a variation in the width of the expansion gap as the train passes. The effect of this behaviour on the ballast may provide an insight into mechanisms initiating the defect.

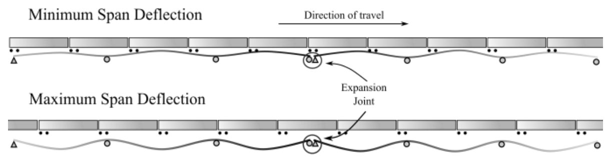

Figure 11. Illustration of how the dynamics span displacements varies with the passage of the train on the bridge. Vertical scale exaggerated for clarity.

4.2 Mechanisms

Using the available data, the following possible causes of the defect are presented.

Given that the ballast is continuous over the joint, any change in the geometry of the joint will affect the ballast. Assuming the total volume of ballast remains constant, any relative rotation of the deck ends would cause a settlement of the ballast (Figure 12). This could cause a gap to develop between the ballast and sleeper, with the sleepers being suspended by the rails.

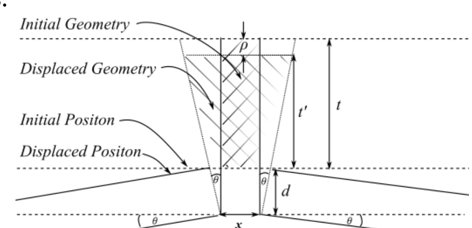

Figure 12. Mechanism of ballast settlement, assuming constant ballast volume.

Assuming the total volume is constant between the initial and displaced positions indicated in Figure 12, the ballast settlement ρ maybe estimated by equating the volumes associated with the initial and displaced geometries to obtain

$$\rho = \frac{\theta t(t+2d)}{2\theta(t+d)+x} \qquad (1)$$

where it is assumed that both deck ends rotate by θ, t and t' are the original and displaced depths of ballast and d is the depth of the bridge deck to the bearings.

Using equation (1) with the measured deck end rotations θ = 6x10^{-5} radians, a deck depth of $d = 2.8$ m, a ballast depth t = 0.5 m and an initial joint width x = 0.2 m, the calculated settlement is 0.9 mm. If a proportion of this settlement is permanent, the sleeper over the joint could become voided over a number of expansion-contraction cycles.

An alternative approach would be to consider the effect of the observed extensional behaviour on the stress state within the ballast. An initial analysis of a moving load 2D finite element model of the bridge and track, calibrated to the site measurements, suggests that the stresses in the ballast over the unloaded joint could reduce at the time of maximum displacement. This may locally affect the ballast strength, behaviour, and mode of degradation (Lackenby et al., 2007).

5 CONCLUSION

Field monitoring of a track defect associated with a structural expansion joint at a viaduct on a high speed railway line has provided evidence of how the track behaves before, within and after the defect. The data suggest that the defect is highly localised and the spatial variation in performance is quite dramatic: sleeper displacements varied from less than 0.5 mm to more than 8 mm over a length of track corresponding to only a few sleeper bays, indicating poor performance within the defect zone.

Capturing the bridge behaviour on either side of the structure expansion joint has given insights into how the behaviour of the bridge is affecting the track. The data show that the response of the bridge affected the total movement at rail level and also that the first frequency of free vibration of the bridge span is close to the bogie passing frequency meaning that dynamic amplification could be significant.

The data suggest an in-phase displacement of the bridge spans on either side of the joint, leading to a cyclic extension-contraction of the ballast over the expansion joint. It is likely that this behaviour is responsible for the formation of the defect. Further exploration of the effects of the bridge behaviour on the continuous ballast is needed to confirm the exact detail of the mechanisms involved.

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REFERENCES

Bowness, D., Lock, A. C., Powrie, W., Priest, J. A. & Richards, D. J. 2007. Monitoring the dynamic displacements of railway track. *Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit*. **221**. 13-22.

Calcada;, R., Delgado;, R., Campos E Matos, A., Goicolea, J. M. & Gabaldon, F. 2008. *Track-Bridge Interaction on High-Speed Railways.* CRC Press, London.

Coelho, B., Hölscher, P., Priest, J., Powrie, W. & Barends, F. 2011. An Assessment of Transition Zone Performance. *Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit.*, **225**. 129-139.

Dahlberg, T. 2010. Railway track stiffness variations–consequences and countermeasures. *International Journal of Civil Engineering.* **8.** 1-11.

Frýba, L. 1996. *Dynamics of railway bridges.* Thomas Telford. London.

International Union of Railways (UIC). 2001. *UIC Code 774-3*. Paris.

LACKENBY, J., INDRARATNA, B., MCDOWELL, G. R. & CHRISTIE, D. 2007. Effect of confining pressure on ballast degradation and deformation under cyclic triaxial loading. Géotechnique, 57, 527-536.Le Pen, L. Watson, G. Powrie, W. Yeo, G. Weston, P. Roberts, C. 2014. The behaviour of railway level crossings: Insights through field monitoring. *Transportion Geotechnics* http://dx.doi.org/10.1016/j.trgeo.2014.05.002

Priest, J. A., Powrie, W., Le Pen, L., Mak, P. & Burstow, M. C. 2013.The effect of enhanced curving forces on the behaviour of canted ballasted track. *Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit*. **227**. 229-244.

Ramberger, G. 2002. *Structural bearings and expansion joints for bridge.* Zurich. International Association for Bridge and Structural Engineering.

Yang, Y.B. Yau, J.D. Wu, Y.S. 2004. *Vehicle-bridge Interaction Dynamics: With Applications to High-speed Railways.* World scientific publishing. Singapore.