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Strengthening of CWR Track Stability by Utilising State-of-the-Art Track Bedding Optimisation Technologies

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Abstract

Severe vehicle-track dynamic responses with significant amplitude and high track displacement are evidenced being responsible for unwanted misalignment and buckling of the Continuous Welded Rails (CWR) track. In past 2 decades, multiple effective technologies were developed to strengthen track stiffness to mitigate severe vehicle-track dynamic responses, which include under sleeper pads (USP), elastomeric pads, and ballast glue for track stiffness optimization. In this study, these three technologies are investigated through a comparative analysis. The vehicle dynamics, track stability and interactions between vehicle and continuous welded rail (CWR) track are studied experimental and theoretically. The track stiffness is investigated analytically via developing an analytic mathematical model and its influences on vehicle dynamics are identified numerically by using finite element modelling and simulations. For comparison and validation, a series of in-field tests are conducted to assess the effectiveness of these technologies. The advantages and disadvantages of these three technologies are identified as main outcomes of this research.

Keywords: CWR, track, stability, under sleeper pad, ballast glue, optimization.

1 Introduction

The railway track has been significantly strengthened by the Continuous Welded Rails (CWR) since it was widely utilized 70 years ago. However, the CWR tracks eliminated the rail joints and the gaps between the rail sections/modules, track safety issues is arisen from high thermal stress which is introduced by the ambient temperature changing within the year from severely cold winter to hot summer. At some circumstances, if the CWR structure is combined with poorly track maintenance, it will cause catastrophically disasters such as track buckling (usually happened in summertime), rail broken (usually in winter time), and derailment. According to the information from the Australian Transportation Safety Bureau, within the recent decade about 50% of the huge costly train derailments are directly caused byor related to the track buckling.

As it has been acknowledged by the time the CWR concept was firstly developed 8 decades ago, the changing/variation of the neutral temperature (when the rails are fastened at a designed stress-free temperature) predominated the condition of the longitudinal stress within a CWR track module, i.e. the stability of it. The stability of the CWR track is represented as a load- displacement curve, as shown in Figure 1, in which the temperature increase of the rail above the original designed neutral temperature versus the lateral displacement of the track is plotted (Kish, A & Samavedam G 2013, and Maertens, M 2018). Theoretically, when a CWR track module is at the neutral temperature, there are no longitudinal stress existed in the rails. However, as the increasing of the rail temperature, the compressive stress is raised and keep increase until reached a turning point (ΔT_{max}, in Figure 1) where the track lateral deformation cannot linearly change with the rail temperature, the sudden significant lateral deformation will happened, i.e. the CWR misalignment or buckling.

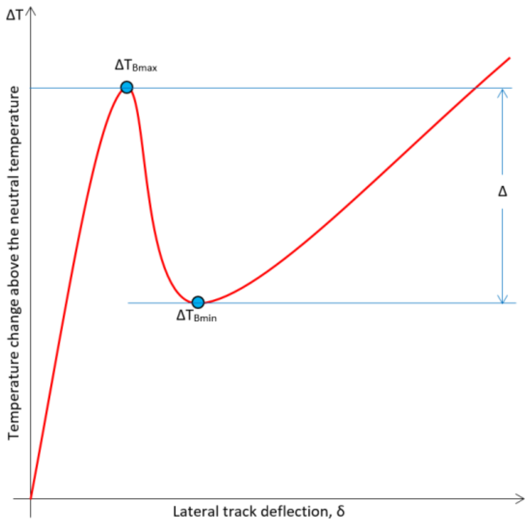


Figure 1: The load-displacement curve for track buckling ofCWR track

Focusing on the post-buckling phase on the load- displacement curve, there are two significant neutral temperature increasing values ΔT_{Bmax} and ΔT_{Bmin}, between them multiple positions of equilibrium are existed. As presented by Kish and Samavedam (2013), when sufficient external energy is supplied (for example, the energy which is introduced by the dynamic train action), the CWR track can jump from the pre-buckling stable configuration to a post- buckling stable configuration at temperatures changing below ΔT_{Bmax}, which the unstable configuration is went through.

As represented in Figure 1, the temperature changing range between ΔT_{Bmin} and ΔT_{Bmax} is named as the buckling regime of CWR tracks. The CWR tracks will buckle at the temperature changing reached ΔT_{Bmax} without any external energy. Below ΔT_{Bmin}, the CWR track will not buckling because it has only one stable equilibrium configuration. However, the track buckling can also happened at ΔT_{Bmin} if sufficient external energy is applied to the track. From theoretical study and in- field measurement are all confirmed that the dynamic train loads are the major resources of the external energy.

Since the late of 1980s, the track engineers and scholars have found a large amount of evidences that not only the significant thermal stress but also the severe track / vehicle dynamic responses are the major contributors for the instability of CWR tracks. Based on the outcomes of the investigations of the mechanism of CWR track buckling/misalignment, it has been concluded that the locations where track under the significant dynamic vehicle loading prone to lost their lateral stability. Since late of 1980s, the railway researchers and engineers have acknowledged that the vehicle dynamic has significant effect to the CWR stability (Zarembski, 1989) & (Kish & Samavedam, 2013). The uplift wave which is initiated by the bogies of the vehicles is prone to decrease the lateral and longitudinal resistance from ballast to CWR track (Figure 2). The efficient track resistance is the “sine qua non” of the stability of CWR track.

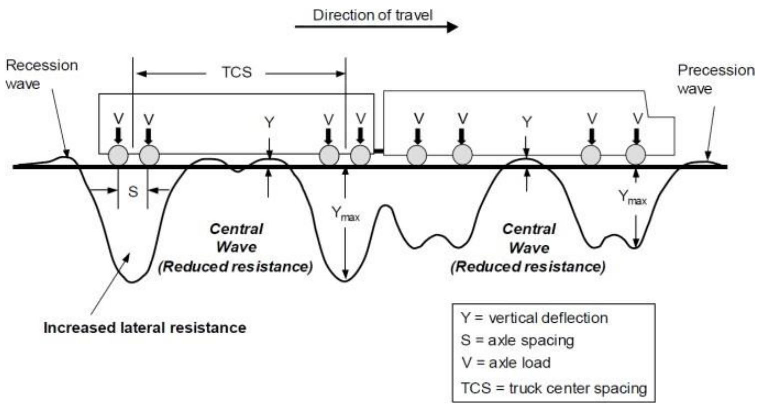


Figure 2: Definition of uplift waves

The amplitude of vehicle dynamic is impacted by:

- Track stiffness;
- Evenness of distribution of track stiffness along the track (i.e. Transition areas); and
- Special track structures (turnout, steel girder bridge, slab track, etc.).

The detailed structural features of the misaligned CWR tracks of a major Australian railway authority from 2008 to 2018 is analysed. Based on this study, for the main line track, the special track components/structures (including the turnout, diamond, catch point, tunnel, large-span culvert and bridge) which are involved within the misaligned CWR modules are counted.

In this study, the track stiffness and its effects on wheel/track dynamic and track stability have been analyzed by structural modelling, in-field tests and numerical study. The innovation is focused on the feasibility study of improve the dynamic stability of turnout in CWR turnout. Research questions are whether and how the new track components and maintenance works can be utilized to mitigate the abrupt changing of track stiffness at the transition area of the open track and turnout.

Research methods including the in-field testing of the state- of-the-art track components and new track strengthening methods, the feasibility study of high elastic rail pad materials were undertaken by finite element analysis. Expected outcomes of this study including if and how the new innovated track components such as under sleeper pads, ballast glue and elastomeric pads can improve the performance and stability of turnout in CWR.

The track stiffness and its major influencing factors are studied by the available and widely-acknowledged track analysis model in Section 2. The concept of track stiffness optimization is provided in the first sub-section of Section 3. In the followed sub-sections, the newly developed track component – under sleeper pad and its performance through a long period in-field testing is presented; An new track strengthening methodology – ballast glue were tested at a track transition zone where the abrupt change of track stiffness are existed.

Hence, utilising special maintenance strategies and technologies to improve the track stiffness and its distribution especially on the weak spots (transition areas) and/or special track structures (turnouts) is the directions of efforts for the stability of CWR track.

2 Track Stiffness and Its Effects on Vehicle Dynamics

Track stiffness may be constant or variant, but generally refers to the constant one. The track stiffness variation is a major influential factor in the wheel-track interaction, and structural vibration of a track (Wang, 2015). It involves both track integral stiffness and track component stiffness. The track integral stiffness is represented by a ratio of the force acting on rail surfaces to the displacement of a corresponding point. Track component stiffness refers to the stiffness of track components (i.e., rail, sleepers, fasteners, ballast bed/sub-ballast layer, and subgrade).

The dynamic responses of each specified track module are predominated by its track stiffness. The track stiffness can be further classified into vertical and lateral stiffness.

2.1 Vertical stiffness of a rail support

The vertical stiffness of the rail support is defined as the force acting on the support top for unit settlement of the rail support (N/m). This concept is widely used in the elastically supported beam model in track mechanics. In s track section, this stiffness can be described as the stiffness summation of a series of the vertical stiffness of the fastenings and sub-rail foundation. It can be expressed by:

$$\frac{1}{k_D} = \frac{1}{k_f} + \frac{1}{\frac{k_S}{2}} + \frac{1}{\frac{k_b}{2}} \qquad (1)$$

Where:
k_D is the stiffness of the rail supports (N/m);
k_f is the vertical stiffness of the fastening (N/m);
k_s is the track bed (ballast layer) stiffness (N/m); and
k_b is the stiffness of the anchor bolts and cover plate of the turnout structure only (N/m).

For the slab track, k_s can be infinite. In turnout area, k_f varies with the length of sleeper/bearer plate. k_s and k_b vary with the length of sleeper/bearer. Specially, within a turnout area, the k_D is changed longitudinally along the turnout.

2.2 Fastening Stiffness

Turnout and open track railway sections have different structures. In a turnout, indirect fixation fastening with a bearerplate (or slide plate) is used, and two elastic layers beneath the rail and steel plate, respectively, are provided. The fastener connects the rail and the bearer plate, and the bolt connects the bearer and the bearer plate, as shown in Figure 3.

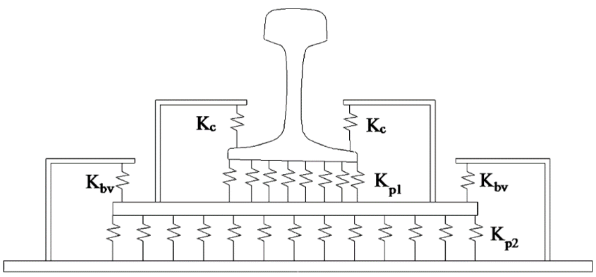


Figure 3: Vertical stiffness of fastening

The vertical stiffness of the fastening consists mainly of the vertical stiffness of the fastener, pads underneath the rail and plate, anchor bolts, and cover plate. It can be calculated by:

$$k_f = \frac{(2k_{bv}+k_{p2})(2k_c+k_{p1})}{2k_c+k_{p1}+2k_{bv}+k_{p2}} \qquad (2)$$

Where:
k_f is the vertical stiffness of the fastening (N/m);

k_c is the vertical stiffness of an individual fastener (the initial fastening force P_C is about 10 kN, and the spring coefficient is about 1 kN/mm);
k_{p1} is the vertical stiffness of the rail pad;
k_{p2} is the vertical stiffness of the plate pad; and
k_{bv} is the vertical stiffness of anchor bolts and cover plate.

2.3 Sub-rail foundation stiffness

In this research, the sub-rail foundation stiffness include: the upper and bottom parts of track bed stiffness (ballast). In the dynamic analysis of a train-track system, the vibration mass of the track bed can be calculated as the total mass of the ballast layer constructed by stress dispersion lines at the bottom of a sleeper. Therefore, the track bed stiffness equals the bearing stiffness of the vibration ballast layer against the sleeper. This ballast layer is divided into an upper part and a lower part in calculations. Track bed stiffness is the combined stiffness of the two parts (upper and bottom parts). The vertical stiffness of the upper part is:

$$k_{s1} = \frac{2(l-b)E_s \tan\phi}{\ln\left(\frac{l \cdot a}{b(l+2h_1 \tan\phi)}\right)} \qquad (3)$$

Where:
l is the length of the sleeper/bearer, which increases gradually along the length of a turnout;
a is the space of two adjacent fastening supports; b is the bottom width of the sleeper/beater;
h_1 is the distance from the intersection point of track bed stress dispersion lines to the sleeper bottom;
E_s is the elasticity modulus of the bed, generally taken as 150 MPa; and
φ is the bed stress dispersion angle, generally taken as 35^o.

The vertical stiffness of thebottom part is expressed as:

$$k_{s2} = \frac{2aE_s \tan\phi}{\ln\left(l+\frac{2H\tan\phi}{l}+2h_1\tan\phi\right)} \qquad (4)$$

Where:
H is the bed thickness, generally designed as 35 cm. If there is ballast, half-thickness of the ballast will be considered.

Thus, the vertical stiffness of the track bed under a long sleeper/bearer can be expressed as (N/m):

$$k_s = \frac{k_{s1}k_{s2}}{k_{s1}+k_{s2}} \qquad (5)$$

3 The Track Bed Optimisation Technologies and the Preliminary Observation from Application

As shown in the above equations, the track bed stiffness is high due to consolidated ballast layer and subgrade with greater load-bearing capacity, and it can be decreased by installing elastic elements with lower levels of bedding modulus.

3.1 The concepts of the track stiffness optimization

Turnouts and open railway sections are different in structure. Track integral stiffness in a turnout is different from that in a section due to the nature of the turnout structure. It is not simply a linear combination of the stiffness of various parts, but may be affected by rail profiles, e.g., the 60kg/m or 53kg/m rails in Australia, length of bearers and pads, ballast type and conditions, and other structural parameters. In addition, there are significant variations of the track stiffness within the short turnout zone. Hence, one research focus is to stabilize the turnout zone under dynamic loads by homogenizing the track stiffness of the turnout zone. Based on the best practices of turnout design and maintenances, the available and effective methods include the installation of under sleeper pads (USP), elastomeric pads, ballast glue, etc.

The under sleeper pads (USP) has been installed on the track transition area, turnout, diamond areas. The elastomeric pads has been trialled on the concrete bearers of turnout. The ballast glue method is applied at the location where has the highest track stiffness difference which is between the non-ballast steel girder bridge and high embankment track.

All of these locations have been tested and monitored for its dynamic response of the passenger and freight trains. The test results are represented as the following. These results have been reviewed and analysed. The potential of the application for use them for stabilization of the turnout zone have been assessed.

3.2 Under sleeper pads

Under sleeper pads (USP) are designed to reduce the track stiffness by providing a layer of rubber material between the sleeper and ballast (Figure 4). With the reduced stiffness and vertical impact, it will prolong the life of the sub-structure (ballast/capping).

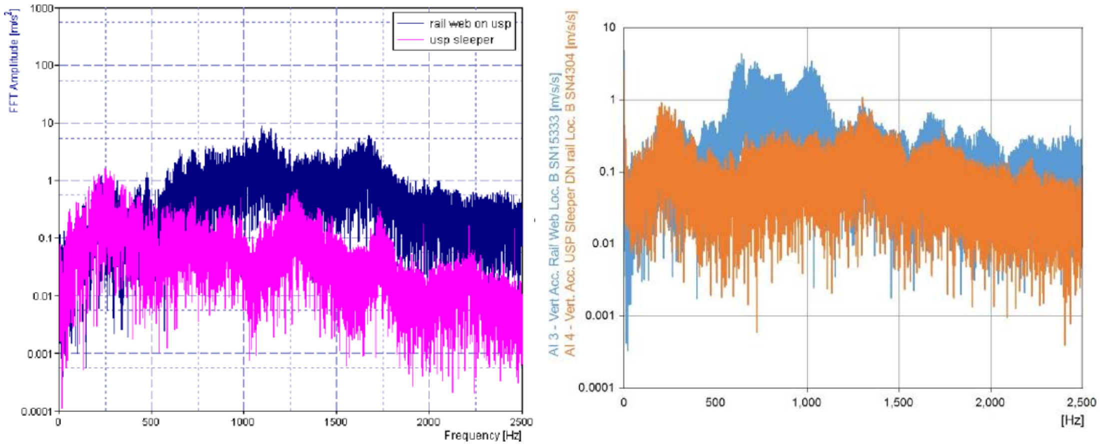
Figure 4: Under sleeper pad located underneath of the concrete sleeper

The USPs (10mm of thickness) were installed as part of a trial on Australian railway network under glued insulated joints in 2011. The goal of this trial is to examine the service performance of the USPs in order to improve the dynamic impact isolation at Glued Insulated Joints (GIJs). The adjacent tracks were also renewed and used as baseline. The vibration measurements were carried out by using the accelerometers and microphone and speed radar which are installed on the track as shown in Figure 5 and 6.

Figure 5: The devices for acceleration measurement at thetest site

Figure 6: The Accelerometers (verts) are installed on the railweb and concrete sleepers at the test site.

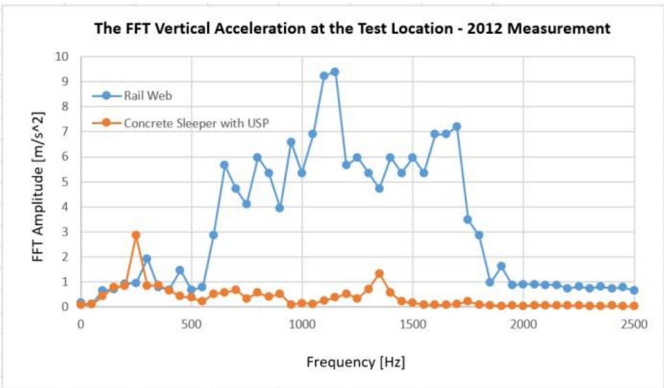
The early inspections happened in 2012 shows promising results (Figure 7, a). After the revisit in 2018, the inspection and measurements undertaken shows that the ballast underneath the USP were maintained and still in good conditions (Figure 7, b).



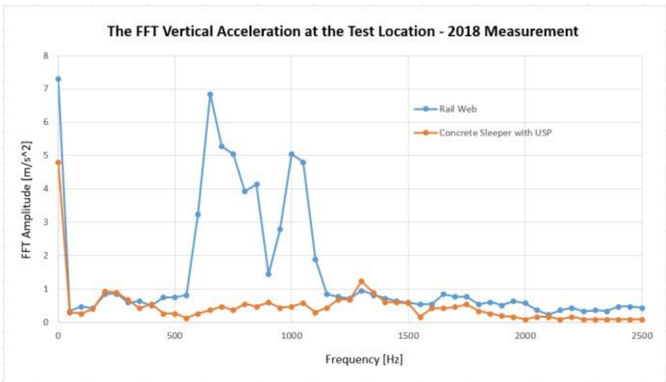
(a) (b)

Figure 8: The FFT vertical acceleration at the test location -2012 measurement (a) vs. 2018 measurement (b).

For detail analysis, the measurement results that represented in Figure 8 are re-arranged by filtered out the maximum Fast Fourier Transform (FFT) amplitude data in a 50 Hz interval. The re-arranged results are presented in Figure 9.



(a)



(b)

Figure 9: The FFT vertical acceleration at the test location -2012 measurement (a) vs. 2018 measurement (b).

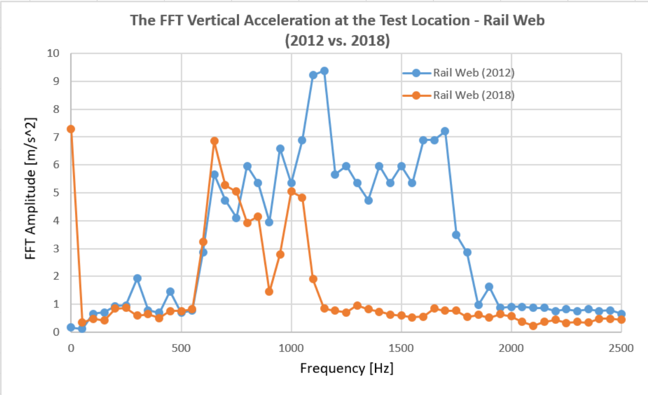


Figure 10: The FFT vertical acceleration at the test location - Rail web (2012 vs. 2018)

Compared with the vertical acceleration which are obtained from the concrete sleepers with USP installed, the results are kept stable, no significant changing can be observed (as shown in the comparison diagram in Figure 11). That can be explained as there are no noticeable either track settlement or severe track geometry defects have been happened in the 6 years' time interval.

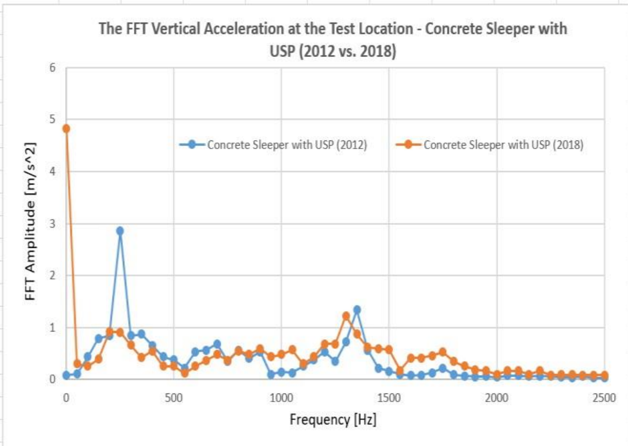


Figure 11: The FFT vertical acceleration at the test location - concrete sleeper with USP (2012 vs. 2018)

From Figure 8 and 9, a significant finding is the acceleration results from concrete sleeper is dramatically lower than the value from rail web, and the situation is even significant for the frequency range of 500 Hz – 2000 Hz in 2012 and 500 Hz - 1200 Hz in 2018. At these frequency range, the acceleration value from rail web can be 5 to 6 times higher than from concrete sleeper with USP. As the rail track at the test locations is featured by 60kg/m head hardened rails assembly with Pandrol "e" clips, and the rail pad that utilized between rail and concrete sleeper is the HDPE pad. The HDPE pad has almost "0" impact attenuation (the impact attenuation is 0%-5%). Hence, the significant deduction of acceleration is mainly contributed by the USP that glued on the underneath surface of the concrete sleeper.

In addition, in the low frequency spectrum (< 500 Hz), the results from rail web and concrete sleeper (with the USP installed) of 2012 and 2018 have shown no significant difference and very close. The average value is rail web records are higher than concrete sleeper records. Similar situation is also been observed in the higher frequency spectrum (2000 Hz - 2500 Hz of 2012 measurement results and 1200 Hz – 2500 Hz of 2018 measurement results).

Another interesting finding is that after 6 years' installation of USP, the high acceleration results (> 3 FFT amplitude (m/s^2)) on rail web also has shown a "narrowed" frequency range (changed from 500 Hz – 2000 Hz in 2012 to 500 Hz – 1200 Hzin 2018) and lower values. This is an evidence for rail top surface is trending towards stable

Studying the Figure 10 (b), a pick value of vertical accelerationis observed in the frequency spectrum 500 Hz – 1000Hz and noobvious decrease or become evened compare with the results on higher frequency spectrum. This pick value is caused by the"dip" of the glued insulated joint.

From the in-field measurement results, there are strong evidences for that the USP strengthened track is kept in the stable condition after 6 years of mixed freight (25t axle load) and passenger traffic.

3.3 Ballast glue

Sub-rail foundation stiffness at the open track / special structure transition area can be upgraded by ballast glue. To evaluate thein-situ performance and effectiveness of a bridge end improvement using ballast glue/bond, ballast glue was trailed in a bridge-embankment transition zone. It involved the condition investigation and survey evaluation of the bridge-embankment transition zone, and the investigation of available comparable geometry data. This study will only review the bridge end performance before and just after the improvementprogram was implemented.

The transition from ballasted track forms to non-ballasted trackforms or structures often causes a high rate of trackdeterioration due to the dynamic impact excitation by theabrupt change of track stiffness. This results in accelerated rates of track geometry and component degradation, high maintenance need, poor ride comfort, and high fatigue stress threshold for passing rolling stocks. End of the bridge was instrumented with accelerometers to measure vibration responses. The bridge is a transom-top on the steel girders. The track configuration consists of 60kg rails, standard guard rails, Pandrol fastening system,low profile medium duty concrete sleepers, ballast bed and formation. Both ends of the steel girder bridge were resurfacedby tamping machine.

During a possession, the bridge ends were lifted andadjusted to the alignment by the tamping machine. Then, the ballast glue/bond material was applied to both bridge ends. TheMC-ballastbond 70 was used. In addition, from the technical specification it is stated that the ballast-bond material has a structural design life of 50 years. The stiffness ramping, which defines the area to glue the ballast, was designed as shown in Figure 12. The ballast glue was sprayed over the full shoulder for 6 sleeper-spacings' distance, and then only half shoulder for the next adjacent 6 sleeper-spacings, next to this only area of the sleeper length (2.5m in width of track) to be ballast glued along the track for 6 sleeper-spacings, and the work finished by the last 6 sleeper-spacings by only glue the track area of ½ sleeper length.

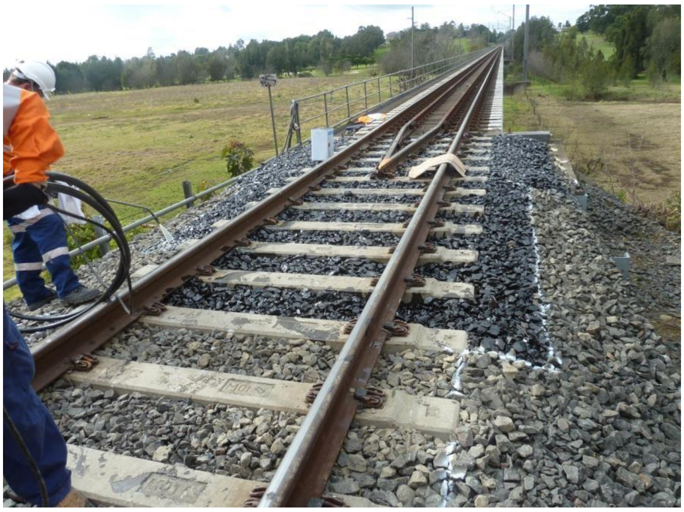


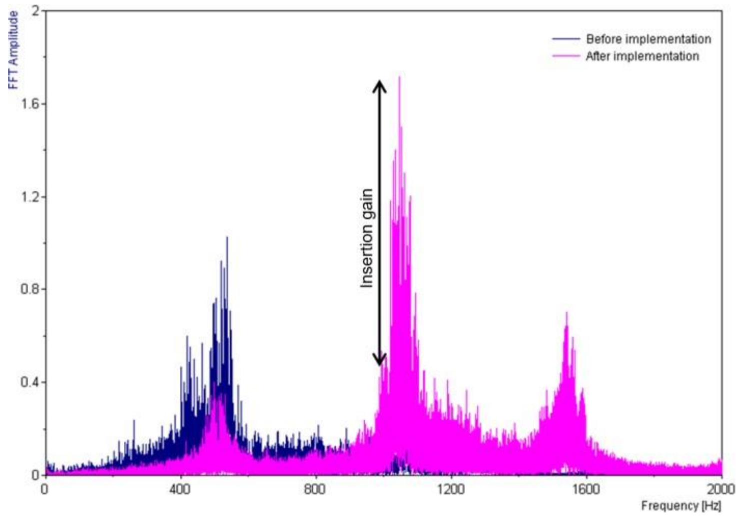
Figure 12: Ballast glue work at the end of the bridge

Post the inspection the vibration measurements were performed to examine the track condition after ballast gluing. It was observed that the glue did not seepage much through the voids of ballast aggregates, even though the ballast had recently been lifted and tamped. The glue is varied about 250-300mm deep from the top surface of ballast, resulting in about 70 - 100mm underneath the sleeper soffit (250-180 = 70mm).

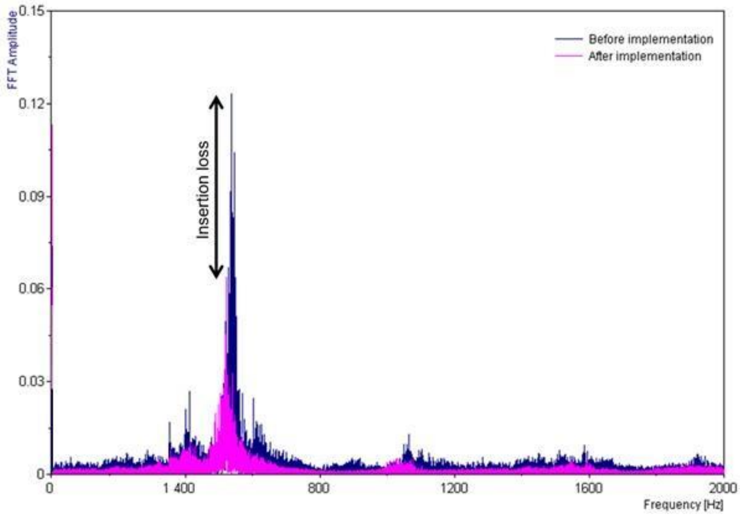
Vibration measurements were carried out at two stages, initial condition (pre-ballast glue), and after condition (post ballast glue). The tests will allow benchmarking the dynamic performance of the bridge ends and evaluation of the effectiveness of the ballast glue method.

The comparison of averaging peak vibration magnitudes at the bridge end shown that the ballast glue changes the dynamic track characteristics. The maximum amplitude of vibration of rail at the bridge end tends to significantly increase due to ballast adhesion, whilst the vibration amplitude of sleeper at the bridge end surprisingly remains at the same level. It is noted that the rail vibration levels at the bridge end are in a similar range with those over the bridge. This shows that the track stiffness at the bridge end increases significantly, which consequently increases the wheel-rail dynamic interaction.

It is noticeable that the sleeper is damped by the ballast cohesion. The vibration suppression level is very high as the dynamic effect of larger lumped mass attached to sleeper is pronounced. Also, the duration of sleeper's vibration spectra is shortened due to ballast glue (Figure 13).



(a) Rail's peak vibration at bridge end



(b) Sleeper's peak vibration at bridge end

Figure 13: Comparative frequency responses of bridge end

Based on the vibration test measurements, it is evident that the ballast glue can help suppress the vibration of sleepers and ballast at the bridge approaches in a short term. Track inspection vehicle data shows that the dynamic rail deflection deviation is improved by about 30% in a short term. The linear regression prediction of track settlements shows that the ballast glue will strengthen the bridge ends and reduce the level of track settlement overtime.

4 Conclusions

There is no doubt that an adequate and evenly distributed track stiffness within the track and turnout zone (turnout structure and the adjacent open CWR tracks on its two ends and on both of the main and turnout divergent directions) can significantly contribute to the stability of track modules and turnouts in CWR. Furthermore, a smooth transition of stiffness within the turnout zone is also required for the riding quality of the trains.

From the theoretical analysis in this paper, it can be found that the higher track bed stiffness which resulted from the consolidated ballast and subgrades with greater load-bearing capacity can be adjusted by installing elastic elements with lower levels of bedding modulus. The currant available state- of-the-art technologies and products that support the concept including: The under sleeper pads (USP) for the track transition area, turnout, diamond areas. The ballast glue method for the location where has the highest track stiffness difference which is between the non-ballast steel girder bridge and high embankment track.

Based on the vibration test measurements, it is evident that the ballast glue can help suppress the vibration of sleepers and ballast at the bridge approaches in a short term.

The visual inspection and vibration performance evaluation show that the USP has a potential to improve dynamic performance at the locations of GIJs and turnouts.

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