Influence of the tie-ballast interface on transition zone performance

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ABSTRACT: Many locations of railroad track consist of abrupt changes in track stiffness and underlying substructure called transition zones. These regions historically experience greater track settlement than the surrounding track requiring more frequent track maintenance from railroad companies. This paper emphasizes how discontinuities at the tie-ballast interface can be indicative of future track geometry problems and further deteriorate transition zone locations. This is supported by non-invasive field instrumentation using video cameras to measure tie displacement, accelerometers to measure tie acceleration, and three-dimensional dynamic numerical modeling. Transition zones with reoccurring track geometry defects often display unsupported ties and field instrumentation and numerical modeling suggest this condition can further increase tie loading and ballast deterioration. Potential solutions to avoid discontinuities at the tie-ballast interface are balancing transient and permanent displacements between the bridge and transition zone, providing a cushioning layer at the tie-ballast interface, and ensuring good substructure support and ballast condition underneath the transition zone ties. Examples of these transition zones are included in the paper.

1 INTRODUCTION

Railroad track transitions are track locations that experience a rapid change in track structure. This often refers to bridge transition zones but can also include asphalt crossings, culverts, and transitions from ballasted to unballasted track. Track transitions are a common topic of study because they often experience accelerated track geometry deterioration and represent an expensive maintenance location for railroads (Li & Davis, 2005; Mishra et al., 2012; Stark & Wilk, 2016).

Multiple studies have investigated root causes of accelerated settlement in the transition approach, i.e. about 3 to 6-m from bridge abutment (Kerr & Bathurst, 2001; Li & Davis, 2005; Plotkin & Davis, 2008; Coelho et al., 2011). Results suggest that while the specific causes of deterioration is site dependent , the three general root causes are: (1) lack of track settlement on the bridge, (2) increased dynamic loads in the approach, and (3) reduced-performance substructure conditions in the approach. Reduced-performance conditions are defined as ballast or subgrade conditions that result in a reduced stiffness or increased settlement rate than what is anticipated from a compacted substructure in the open track, i.e. track location with no adjacent track structure. Examples of reduced-performance ballast conditions are ballast degradation, fouling, and increased moisture. Examples of reduced-performance subgrade conditions are inadequate compaction due to the abutment or increased moisture. Typically, a combination of all three mechanical root causes and construction issues play a role in the accelerated settlement but the interaction and magnitude of each factor can vary between sites.

A common chain of events that lead to transition zone track geometry deterioration is described below. After resurfacing or being put into service, the ballast and subgrade in the approach will compact and densify from repeated train loading, resulting in some magnitude of differential settlement between the bridge and approach. This magnitude depends on the initial ballast and subgrade density and characteristics (Indraratna et al., 2012). The differential top-of-rail (TOR) elevation at the approach-bridge interface produces rail-tie or tie-ballast gaps as the first few ties “hang” from the stiff rail that is supported by the higher elevated bridge. These gaps can redistribute the wheel loads throughout the track system, produce impacts as the tie establishes contact with the ballast, and promote ballast deterioration from tie-ballast abrasion due to unrestricted tie movement. The increased loads and ballast breakdown in the approach can produce a negative feedback loop that requires frequent resurfacing to maintain track geometry.

This paper presents a general overview of a recently completed study investigating the root causes of the differential movement at transition zones and the benefits of various mitigation techniques (Wilk, 2017). This paper briefly covers data collected from long-term instrumentation (Section 2), short-term instrumentation (Section 3), and numerical modelling (Section 4) along with some recommendations of design, remedial, and resurfacing techniques (Section 5). Both high-speed passenger and heavy haul freight are covered but causes and remedial techniques are similar for both cases.

2 LONG-TERM INSTRUMENTATION

This section presents the results of a long-term instrumentation project to monitor a bridge approach transition zone experiencing historical track geometry problems (Mishra et al., 2012; Stark and Wilk, 2016; Wilk, 2017). The site is located at the Upland Street bridge approach on Amtrak's high-speed passenger line (177 km/hr / 110 mph) with instrumentation located 4.5 m (15 ft) and 18.3 m (60 ft) away from the bridge abutment, representing bridge approach and open track locations, respectively. To keep notation with previous publications (Mishra et al., 2012; Stark and Wilk, 2016), the terms Upland (15 ft.) and Upland (60 ft.) will be used herein. The instrumentation consists of strain gauges attached to the rail to measure wheel loads and strings of LVDTs installed at depth to measure the permanent and transient displacement of individual substructure layers, e.g. ballast, subballast, and three subgrade layers. On two trips, an accelerometer was attached to the Upland (15 ft.) tie to measure tie acceleration.

The project objective was to identify the depth of movement, determine root causes of differential movement, and propose and test solutions to prevent and mitigate differential movement. This section presents a summary of the results from the instrumentation.

2.1 *Permanent Ballast Displacements*

The permanent displacements after 446 days of monitoring of the uppermost LVDT (LVDT #1) are displayed in Figure 1. LVDT #1 measures the displacement from the top of the concrete tie to the bottom of the ballast layer and is considered a reasonable estimate of ballast settlement. The majority of overall track settlement occurred in LVDT #1 and not the subballast (LVDT #2) or subgrade (LVDT #3 through #5) so the ballast layer is considered the region of interest for this particular site (Stark and Wilk, 2016). The instrumented site has been in service for almost a century so it is expected that the subgrade has fully compacted, explaining the minimal subgrade movement. This behaviour may not be true if the transition zone is on a recently constructed fill or subgrade that has not previously experienced train loading (Moale et al., 2016).

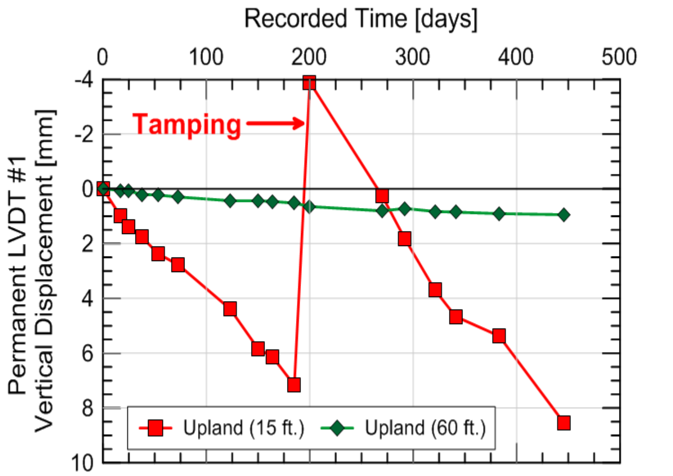


Figure 1. Permanent LVDT #1 vertical displacements of Upland (15 ft.) and Upland (60 ft.)

Comparisons between the approach and open track locations show greater ballast settlement at Upland (15 ft.) with 14 mm/yr than Upland (60 ft.) with 1 mm/yr. The trends show a consistent Upland (15 ft.) settlement rate while the Upland (60 ft.) appears to show decreased settlement with time. This suggests that Upland (60 ft.) has reached a near "equilibrium state" while Upland (15 ft.) has not.

2.2 *Transient Ballast Displacements*

Transient displacements, i.e. displacements from a passing train, were recorded five times during the period of data collection. The results from LVDT #1, measuring the top of concrete tie to bottom of ballast layer, showed non-linear behaviour as the tie must close any tie-ballast gap prior to transferring the load to the ballast. Plotting the peak wheel loads and corresponding LVDT #1 displacements for a passing train allows for an estimation of the tie-ballast gap height ($\delta_{P=0}$) to be calculated. The load-displacement curve of Upland (15 ft.) is displayed in Figure 2.

Once the two LVDT #1 components of tie-ballast gap height ($\delta_{P=0}$) and ballast displacement components (δ_{mob}), i.e. displacement of frictionally mobilized ballast, were separated, the tie-ballast gap (LVDT #1), ballast displacement (LVDT #1), and subgrade displacement (LVDTs #2 through #5) magnitudes could be compared with time and location. Figure 3 compares the transient displacement components at Upland (15 ft.) and Upland (60 ft.). The results show the majority of variation occurs within the tie-ballast gap component and the gap height at Upland (15 ft.) appears to gradually increase with time. This suggests an influence from the tie-ballast gap as there was little to no evidence of a relation between permanent displacements and ballast or subgrade stiffness (Wilk, 2017).

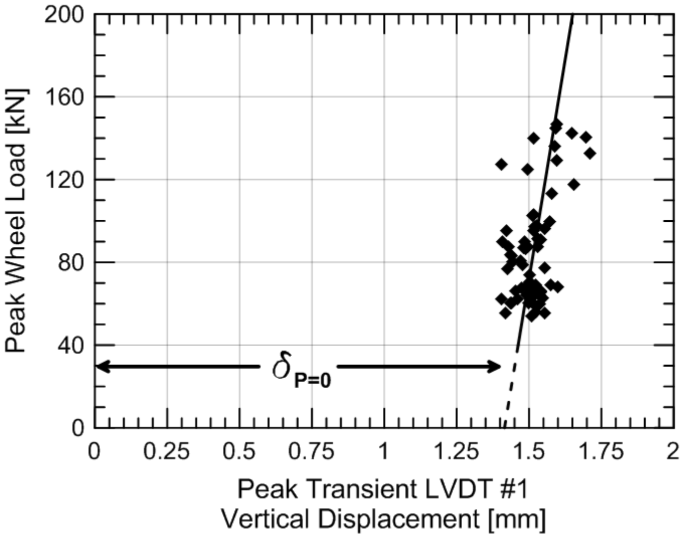


Figure 2. Typical load-displacement curve of Upland (15 ft.) displaying tie-ballast gap

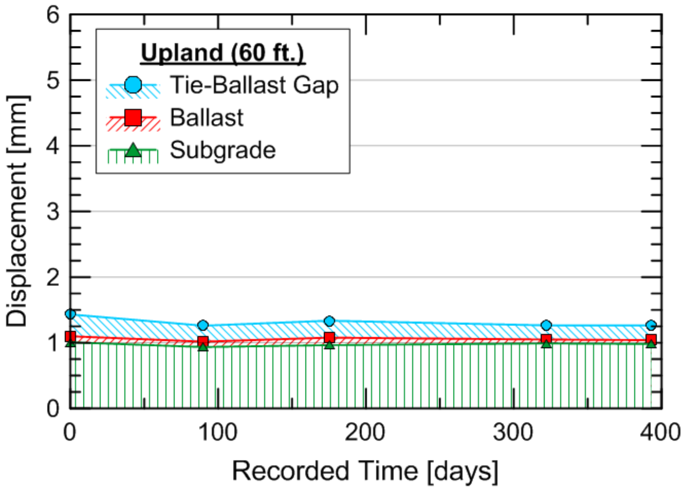
The development of tie-ballast gaps at transition zone locations is anticipated as the rail and connected ties will hang from the track that is supported by the rigid open-deck bridge and open track. However, the influence of tie-ballast gaps on accelerated ballast settlement is still relatively unclear. For example, laboratory testing by Selig & Waters (1994) showed ballast settlement rates can increase up to five times with the inclusion of a 1 to 4 mm gap as opposed to a tie that continually in contact with the ballast. The additional movement from the poor contact could increase ballast abrasion and breakage.

Impacts from the tie-ballast contact and uneven distribution of loads may also increase and concentrated load on particular ties. These two mechanisms are explored in the subsequent sections. An additional factor producing accelerated settlement at Upland (15 ft.) is degraded and moist ballast from ballast breakdown, fine infiltration, and blocked drainage. All factors likely played an interconnected role but deeper look into each is required for better standing of the transition zone system deterioration.

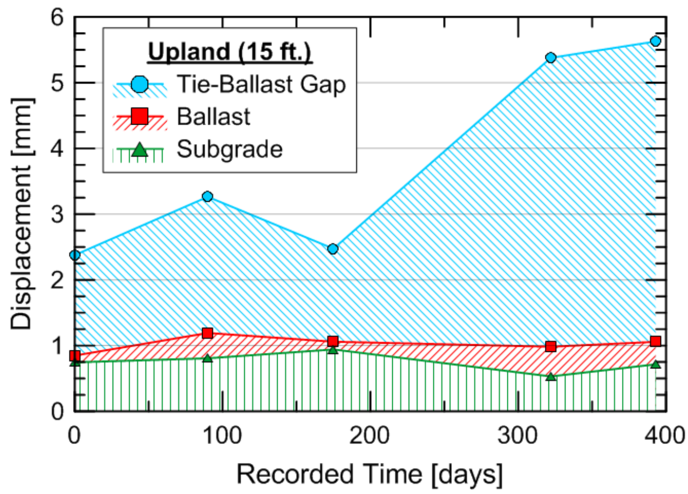
2.3 *Tie Accelerations*

Accelerometers were attached to the Upland (15 ft.) tie on the last two days of transient data collection and Upland (60 ft.) on the last day of transient data collection to measure tie accelerations. The benefits of using accelerometers is they can measure impacts and vibrations occurring within the track, giving additional insight into track behaviour and loading. At the Upland (15 ft.) sites, the accelerometer measured an acceleration spike at the moment the tie contacted the ballast from every wheel pass (Wilk et al., 2016).This shows that some impact load can be present during tie-ballast contact, which may increase the load being distributed to the ballast or damage, degrade, and abrade the ballast.

Figure 4 plots the average peak tie acceleration of each passing wheel from impact/loading measured at



(a)



(b)

Figure 3. Transient displacement components for (a) Upland (60 ft.) and (b) Upland (15 ft.)

Upland (15 ft.) and Upland (60 ft.) with various tie-ballast gap heights. The results show an increase in tie acceleration with increasing tie-ballast gap height. It must be emphasized that no trend is proposed because this relation will be site specific and dependent on train velocity and support conditions of surrounding ties. However, this does suggest tie-ballast closure can play a detrimental role in transition zone performance.

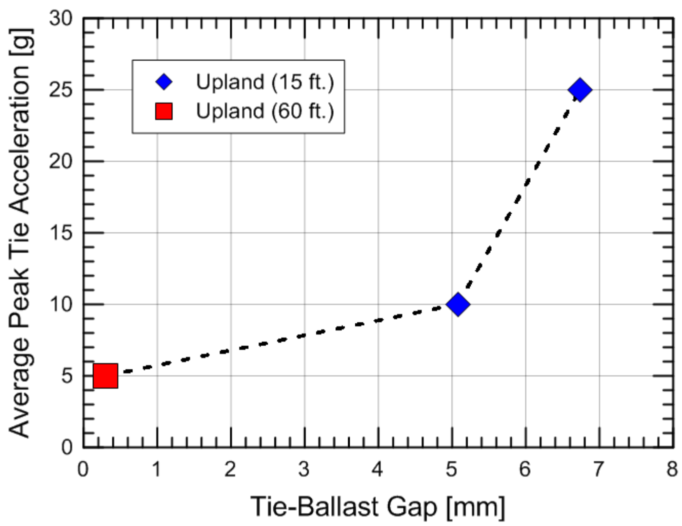


Figure 4. Relation between tie-ballast gap and peak tie acceleration at Upland Street Bridge.

3 SHORT-TERM INSTRUMENTATION

A second system of measurement used for the study is short-term instrumentation. Opposed to the long-term instrumentation introduced in the previous section, the objective of the short-term instrumentation is to evaluate track performance while eliminating invasive measurement techniques and minimizing setup time and track fouling.

The short-term instrumentation setup consists of two high-speed video cameras to measure rail and tie displacements and eight accelerometers to measure track vibrations and impacts along with estimating tie displacements along the track. This setup is used at eight different transition zone locations in the United States, with varying performance. The first three setups involved well-performing transition zones with no known track geometry maintenance since installation. The next three setups involved transition zones requiring reoccurring track geometry maintenance. The last two sites involving recently renovated transition zones with track geometries that have not yet required maintenance, but more time is required before any conclusion can be made.

Table 1 displays the eight sites along with various site attributes. For example, the well-performing track locations, to date, all had ballasted-deck bridges, confining wing walls, and either HMA or geoweb underlayment. The one exception is Site #7 which solely uses under-tie pads (UTPs) in the approach. The three sites requiring maintenance did not have any significant transition designs. Additionally, Site #8 includes four different approaches at a single bridge.

The goal of the short-term instrumentation is to monitor the amount of movements and vibrations in the track, the variation along the track, and identify regions of particular interest. This may include the impact or free-body vibration of an unsupported tie or the impact from a rail joint. Accelerometers are typically evenly spaced along the bridge, approach, and open track while high-speed video cameras usually consists of an approach and open track location.

General results from the short-term instrumentation are as follows:

- Well-performing sites had minimal variation in track response along the track and tie accelerations were typically below 5g. This is attributed to the even load distribution and lack of relative movement between track components, i.e. good tie support.
- Sites with historical track geometry problems had variation in response with distance and time along with higher accelerations from impacts and loading vibrations. This variation in response is believed to be caused from uneven load distribution and impacts due to relative movement between track components. These higher and uneven tie acceleration magnitudes can indicate the potential of increased loads in the track.
- Besides track support, train velocity appears to be the most influential factor with tie acceleration magnitudes. For Site 5, if impacts are excluded, i.e. train loading vibrations only, the trend appears to be about 0.06 to 0.09g/km/hr while about 0.25 g/km/hr if impacts are included. These values will likely vary from site-to-site and more information is required before general conclusions can be made.

To show the first two bullets, the average tie acceleration response between Site #2 and Site #6 is compared in Figure 5a. While train velocity and other factors prevent a true comparison, the difference in behaviour is apparent. The well-performing Site #2 shows consistent tie accelerations below 5g except for the locations of a welded rail joint, which

Table 1. List of measured sites along with basic attributes and transition designs

Site	Geometry Problems	Train Velocity [km/hr]	MGT	Tie Type	Bridge Deck*	Wing Wall [m]	Approach Modification*	Ballast Condition
1	No	40	7	Timber	Ballasted	8.2	HMA	Clean
2	No	40	70	Concrete	Ballasted	7.3	HMA	Clean
3	No	40	70	Concrete	Ballasted	5.2 (1 side)	HMA	Clean
4	Yes	16	?	Timber	Open	1.8	N/A	Unknown
5	Yes	177	?	Concrete	Open	N/A	N/A	Fouled
6	Yes	100	60	Timber	Open	N/A	N/A	Fouled
7	N/A	177	?	Concrete	Open	N/A	UTP	Clean
8a	N/A	40	15	Timber	Ballasted	2.7	Geoweb	Clean
8b	N/A	40	15	Timber	Ballasted	2.1	HMA	Clean
8c	N/A	40	15	Timber	Ballasted	N/A	Geoweb	Clean
8d	N/A	40	15	Timber	Ballasted	2.1	Soil Grout	Clean

* HMA represents hot-mixed asphalt and UTP represents under-tie pad

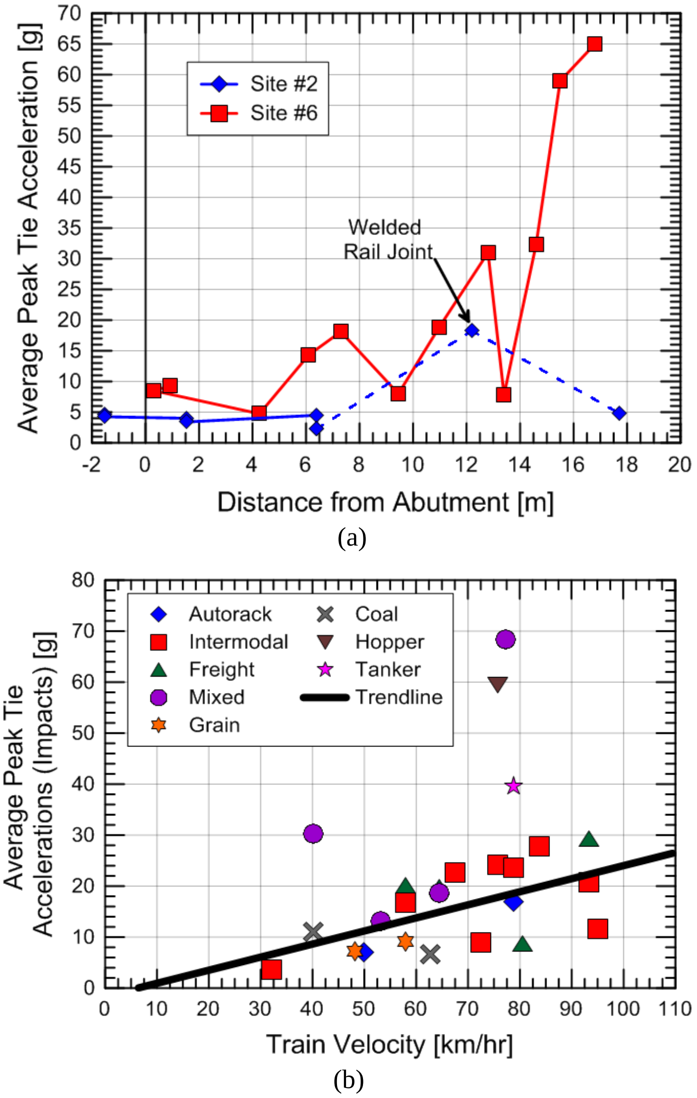


Figure 5. (a) Comparison of average response at Site #2 and Site #6 and (b) relation between average peak tie acceleration and train velocity at Site #6.

displays values of about 18g. Site #6, which displays historical track geometry issues, has varying tie acceleration magnitudes with distance. The low tie acceleration values near the approach are due to load distribution from rail-tie gaps and then impacts and uneven loads are expected further from the abutment.

Figure 5b shows the change in average tie acceleration (average for all eight accelerometer locations) with increasing train velocity for various types of trains. This trend is expected to be different for each site, especially at locations with impacts.

4 3D DYNAMIC NUMERICAL MODELING

A progressive settlement analysis is implemented to simulate the change in loading environment during the settlement of a transition zone. The analysis uses an iterative procedure that predicts loading, displacement, and settlement at 0.4 MGT increments. The settlement model used is based from Sato (1997) and modified by Dahlberg (2001) and outputs ballast settlement as only a function of tie load. The equation is shown in Equation 1:

$$y = 5.87E^{-9} * (P - 25)^4 \quad (1)$$

where y is tie displacement in mm and P is tie load in kN. The load at each tie from the front and back wheels of the passing train truck are used to calculate the settlement under each tie and the geometry is updated. This procedure is repeated until 28 MGT is reached.

The numerical model incorporates the entire track system including the secondary suspension system of a train truck, the rail, concrete ties, and the substructure. The train truck passes from the open track onto a timber tie open-deck bridge. The tie loads of the first ten ties from the abutment are measured allowed the ballast settlement to be calculated in each iteration.. The tie and ballast are modeled as separate entities and discontinuities between the two surfaces are allowed. All elements are modeled as homogenous, isotropic, linear elastic materials. In open track, the rail and substructure stiffness along with tie spacing causes each tie to receive about 40% of the peak wheel load. The ratio of maximum tie load / static wheel load is defined as Maximum Normalized Tie Load and is the assumed load distribution for when all ties are in intimate contact. Increases and decreases of tie load are normalized by this 40% value (see Fig. 6).

The progressive settlement analysis is simulated at 0.4 MGT increments (20,000 wheel passes) for a total of 28 MGT (1.4 million wheel passes). The results show the ballast settlement in the bridge approach settles in manner that evenly distributes the wheel load amongst all the underlying ties, therefore minimizing the tie load. This occurs because if a particular tie experiences greater load than the surrounding ties, it will also experience greater ballast settlement. Then, the ties with greater settlement will receive less load in the next iteration as wheel load gets redistributed from ties with less support, i.e. greater settlement, to ties with better support, i.e. less settlement. This process produces a ballast settlement profile that minimizes tie loads and keeps the approach in a state of equilibrium.

However, this analysis assumes the ballast is perfectly homogenous and does not change properties over space and time. Physically, this is rarely true as ballast properties vary both spatially and temporally. To introduce the effect of heterogeneity, the ballast surface calculated at 28 MGT is randomly varied by ±0.5 mm under each tie and five sensitivity analyses (SA) were conducted. The results in Figure 6 show small deviations from the original ballast surface profile could increase tie loads up to 80%. This suggests that heterogeneities in the ballast or subgrade

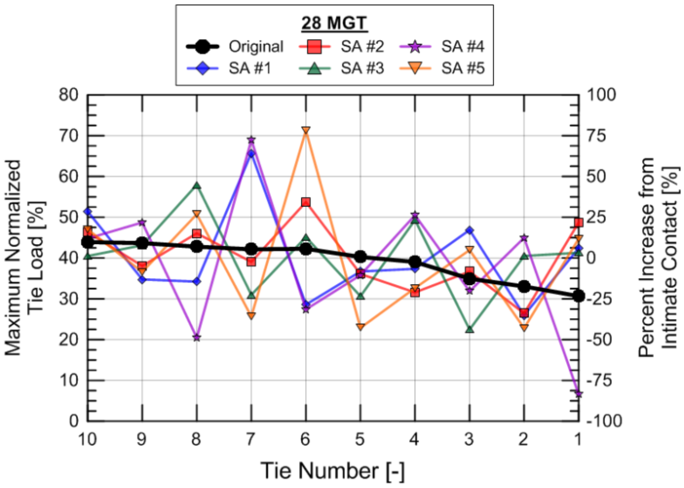


Figure 6. Normalized Tie Load Sensitivity Analysis.

can concentrate and increase tie loads in the approach.

5 REMEDIAL SOLUTIONS

The previous two sections presented field and numerical data that indicates poor tie support, i.e. hanging ties, or local differential settlement can play a detrimental role in transition zone performance. Once a transition zone is put into service and experiences loading, the approach will settle from ballast densification and possible subgrade compaction as opposed to the minimal settlement of the track on a fixed structure such as an open-deck bridge. The differential rail elevation will produce tie-ballast or rail-tie gaps because the stiff rail causes the tie to hang over locations of greater ballast settlement. Then, increased relative movement between track components can increase tie loads and ballast damage. The degraded ballast is then expected to settle at higher rates than its non-degraded counterparts, especially if wet due to blocked drainage from the abutment This can produce a negative feedback loop in which remediation is difficult without replacement of the entire ballast structure.

Other factors that can expedite this process is rapid settlement immediately after tamping as the ballast densifies, gradual subgrade settlement as subgrade compacts or hydrocompresses, degraded and fouled ballast, and blocked drainage.

5.1 *Design and Remedial Techniques*

Design or remedial techniques to prevent the differential settlement from occurring typically requires balancing the entire transition zone system. This means that the stiffness and settlement between the bridge and approach should ideally be equal and differential settlements within the approach should be minimal. Additionally, the transition zone must be recognized as a system in which all components play an important role and disregarding a single component, e.g. ballast or subgrade layer, can initiate the negative feedback loop. This helps prevent increased dynamic loads and prevents the formation of tie-ballast gaps.

The two typical methods of achieving balance are to soften the track on the bridge and better support the approach. Softening the track on the bridge can include the conversion of open-deck bridges to ballasted-deck bridges or using under-tie pads and/or ballast mats. Better supporting the approach can involve using concrete wing walls to better confine the approach, hot-mixed asphalt/geoweb/geogrid underlayment, and subgrade stiffening solutions. Typically, combinations involving three or more solutions have consistently produced transition zones with minimal required maintenance (Stark et al., 2016). The ballasted-bridge deck, confining wing walls, and HMA combination from Site #2 in Table 1 is shown in Figure 7a.

A second potential method is to directly mitigate the negative effects of the tie-ballast interface by installing under-tie pads (UTPs). UTPs are essentially a thin rubber pad that is connected to the bottom of the tie that serves as a cushioning layer. Anticipated benefits of UTP include: (1) a reduction in approach ballast and tie degradation by better distributing the load to the ballast, (2) increased vibration damping, and (3) reduced contact stress between the tie and individual ballast particles. Therefore, UTPs should reduce the ballast pressure and abrasion resulting in a reduction of ballast settlement. Photographs of UTPs installed on concrete ties are displayed in Figure 7b.

(a)


(b)

Figure 7. Photographs of (a) bridge approach with ballasted-deck bridge, confining wing walls, and HMA underlayment and (b) under-tie pads.

5.2 *Resurfacing Techniques*

Another potential underappreciated technique for better maintaining track geometry at transitions is improving resurfacing techniques. Current methods of resurfacing typically involve either automated or pneumatic tamping. Automated tamping raises the rail elevation essentially by loosening the ballast underneath the tie. Pneumatic tamping raises the rail elevation by pneumatically pushing new ballast underneath the rail seat. Both methods tend to disturb and loosen the ballast and result in rapid track settlement that reverts back to its original elevation and resurfacing is required within a year (Stark et al., 2015).

Automated tamping is widely used because of its speed and mechanization and is desirable when resurfacing long stretches of track in short amounts of time with minimal labor. However, a major drawback in the automated tamping procedure is that the ballast is disturbed from its post-compaction equilibrium state and loosened, forcing the ballast to repeat the compaction/post-compaction cycle after each resurfacing event. In addition the current tamping technique degrades and breaks down the ballast every resurfacing event, producing degraded ballast that will settle at a quicker rate (Selig & Waters, 1994).

While replacement of tamping on a wide-scale it is not anticipated because of its speed and cost-effectiveness, recommendations on how to improve resurfacing techniques at specialized location such as transition zones that typically experience track geometry problems are introduced below. Some of these solutions are new and not tested but are potentially viable because they address some of the key factors causing ballast settlement immediately after resurfacing.

Spot tamping bridge approaches two weeks after resurfacing events could potentially extend the service life of the track between maintenance cycles. The purpose of spot tamping is to identify local regions within the approach that has experienced settlement since resurfacing and to re-tamp those specific areas. This can address the weak spots of the newly tamped approach, such as the bridge-approach interface. New pneumatically tamping devices that better distributes the ballast underneath the tie without breaking the ballast could also be developed. This could be accomplished by using different tamping heads and vibration frequencies.

Stoneblowing is an alternative resurfacing method that has been developed and implemented in the United Kingdom and parts of Europe (McMichael, 1991). One of the main benefits of stoneblowing is that it leaves the ballast in its post-compaction state and adds additional stone material to fill the tie-ballast gap. The goal of this procedure is to reduce the ballast compaction stage after resurfacing. An additional benefit of stoneblowing is the reduction in ballast degradation with each resurfacing cycle. Recently, innovative ideas of combining the benefits of stoneblowing and UTPs by blowing stone mixed with rubber pellets have been tested in the laboratory and has shown to further reduce the breakdown and settlement of the ballast (Sol-Sanchez et al., 2016).

One potential drawback from stoneblowing is the stones can still degrade, breakdown, and even fall within the gaps of the underlying ballast. An alternative idea of stoneblowing is installing rubber or plastic shims, defined as hanging tie shims (HTS), underneath the tie during resurfacing. The primary obstacle is getting the shim fully underneath the tie as the underlying angular ballast particles will catch the shim as it slides underneath the tie. Therefore, this technology is considered a work-in-progress.

6 SUMMARY

This paper presents a general overview of a recently completed study investigating the root causes of the differential movement at transition zones and the benefits of various mitigation techniques. Long-term monitoring, short-term monitoring, and numerical modelling were used to investigate these problems. A summary of findings are below:

- The three general root causes are: (1) lack of track settlement on the bridge, (2) increased dynamic loads in the approach, and (3) reduced-performance substructure conditions in the approach.
- The development of tie-ballast gaps in the approach can redistribute the loading throughout the track system, produce impacts as the tie establishes contact with the ballast during loading, and promote ballast deterioration from tie-ballast abrasion due to unrestricted tie movement.
- Design and remedial techniques should attempt to balance the approach and bridge by using a combination of bridge softening and approach supporting techniques.
- Under-tie pads (UTPs) can directly mitigate against the negative effects of the tie-ballast interface. Anticipated benefits are (1) a reduction in approach ballast and tie degradation by better distributing the load to the ballast, (2) increased vibration damping, and (3) reduced contact stress between the tie and individual ballast particles.
- Improvements in resurfacing techniques could extend the geometry life between resurfacing events. This can include new methods of pneumatic tamping, stoneblowing, or the insertion of shims underneath the tie.

7 ACKNOWLEDGEMENTS

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