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Root cause of differential movement at bridge transition zones

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Abstract

The results of a Federal Railroad Administration research project into the factors that contribute to differential displacements at railroad track transitions are presented in this paper. Data from instrumented high-speed passenger (Amtrak) sites suggest that poorly supported ties increase the loads applied on the underlying ballast and can accelerate differential displacements. Poorly supported ties amplify the tie–ballast interaction, which eventually results in large permanent vertical displacements at those locations. This paper presents the location and depth at which permanent vertical displacements are occurring, the "root cause" of these permanent differential vertical displacements, and design and remedial measures that focus on reducing poorly supported ties in transition zones.

Keywords

Transition zone, tie support, tie–ballast gap, differential movement, unsupported tie

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Introduction

In upcoming decades, railroads in the United States are expected to increase freight capacity while expanding high-speed passenger traffic on dedicated or shared lines. This continuing growth in freight capacity means increases in tonnage, axle loads, speeds and train frequencies, which can accentuate existing permanent displacements at track transitions. In shared corridors these permanent displacements can be problematic, due to the sensitivity of high-speed passenger traffic to differential displacements. Therefore, track transitions are an important topic for future shared corridors as recurrent differential displacements are often observed at these transitions.

Track geometry problems already represent a large maintenance issue at railway transitions, costing railroad companies approximately $200,000,000 a year according to the Association of American Railroads[1] and this amount will likely increase. One item of major concern is the occurrence of differential displacements within bridge transition zones, or the region connecting the stiff bridge with the typically softer open track, in the form of a "bump" or "dip". This geometry issue tends to amplify loads, which can degrade and damage the surrounding ballast, ties, fasteners and rail. Successfully addressing track geometry problems at railway transition zones can lower maintenance costs, minimize the number of slow orders issued due to safety concerns, and are imperative for the continual upgrade of track use in the United States.

One reason why the geometry problem at transition zones has not been alleviated is that the mechanism(s) causing differential displacements have not been identified and a suitable remedial or design measure has not been developed to mitigate the problem. Most commonly, these displacements are attributed to the significant change in stiffness as the train passes over the abutment, which increases dynamic loading within the transition region.[2,3] Other factors possibly contributing to differential displacement, such as the natural settlement of the ballast and earth substructure, are also addressed.[2–4] A significant change in stiffness is typically considered to be the primary cause of differential displacements; thus, the majority of past research on bridge transitions has focused on reducing or smoothing the stiffness difference between the open track, transition zone and bridge deck.[2,5]

Despite all of the possible stiffness-related solutions, few field studies have tested the benefits of these remedial measures. One field study near Marysville, Kansas, compared the permanent displacements of a control site with three different transitions treated with either HMA, geocells or

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reinforced soil.[4] Despite the remedial action, the subsequent permanent displacements at the remediated sites were greater than at the control site. The explanation for the lack of success of the remedial measures is that the track modulus of the bridge remained greater than the approach by a factor of two, meaning an abrupt and significant stiffness difference still existed at the bridge approach after the repair.

Due to continued uncertainty of the cause of differential displacements at railway transitions, the Federal Railroad Administration (FRA) sponsored a research project to investigate and minimize differential displacement at railway transitions used by high-speed passenger trains in the United States. The project's scope included: field instrumentation and numerical modeling of new and existing track transitions in order to develop cost-effective designs; and remedial measures to mitigate the problem. The main objectives of this project were:

1. identify the depth and location of permanent vertical displacements;
2. identify the ''root cause'' of the permanent vertical displacements;
3. perform field and numerical studies to test remedial measures that address the ''root cause'' to prevent future permanent displacement issues.

This paper addresses the first two objectives, meanwhile, the remedial measures are being field tested.

Instrumentation

Because the depth and ''root cause'' of permanent vertical displacements at high-speed passenger transition zones was unknown at the start of the project, an instrumentation program to measure the behavior of the track system at both open track and bridge approach sites was undertaken.[6,7] This program included measuring wheel and tie loads with strain gages, and the measurement of permanent and transient vertical displacements as a function of depth using linear variable differential transformers (LVDTs) connected together to form a multi-depth deflectometer (MDD).[8] By comparing the load and displacement differences between and within the open track and bridge approach sites, the ''root cause'' of the observed permanent vertical displacements could be determined and is described herein.

Instrumentation location

As differential displacements at transition zones are an issue for high-speed passenger rail lines, Amtrak's high-speed northeast corridor (NEC) near Chester, Pennsylvania was selected for instrumentation. Instrumentation was installed at three bridge approaches (Upland Street, Madison Street and Caldwell Street) and at their corresponding open track sites that historically have experienced recurrent rail vertical profile deviations, so that the factors causing the track geometry problems could be identified. Bridge approach sites were selected over exits because vertical profile measurements showed greater deviations at the approaches.[6] The west end of the concrete ties were instrumented 15 and 12 feet (4.6 and 3.7 m) from the Upland Street and Madison Street bridge abutments, respectively, and the corresponding open track sites were also instrumented on the west end of the ties located 60 feet (18.3 m) from the Upland and Madison Street bridge abutments. At the third bridge site (Caldwell Street), both ends of a single tie were instrumented 80 feet (24.4 m) from the bridge abutment to compare behavior differences at opposite ends of a single tie. These six sites will be referred to as Upland (15 ft.), Upland (60 ft.), Madison (12 ft.), Madison (60 ft.), Caldwell (East), and Caldwell (West) herein. A plan view of the three Amtrak instrumentation sites is displayed in Figure 1. This figure also shows the location of damaged or missing ties at these three locations. The track at these locations is straight, elevated and confined by large gravity walls, thus one-dimensional vertical displacements were assumed throughout the analysis. Although all six instrumented sites are located within the bridge approach or open track, the closely spaced bridges may result in wheel bouncing from the previous bridge exit. The applied loading consisted of high-speed passenger trains that operate at a fixed 110 mile/h (177 km/h) over this FRA Class 7 track.

Wheel load and tie reaction

Wheel loads were measured to provide insight into track loading and to assess the potential impact of the applied loads on the track structure, ballast and subgrade. Dynamic loads are a major concern for track performance and understanding the loading and tie support conditions is necessary to interpret the measured transient and permanent displacement behavior measured using LVDTs and validate associated numerical models.

Eight strain gages were installed at 45° along the neutral axis of the rail to measure the vertical wheel loads and tie reaction at each instrumented site (see Figure 2). The strain gages measure the shear strain along the neutral axis of the rail and are related to the dynamic vertical load of a passing wheel. A static load frame was used for calibration. The tie reaction measures the force experienced by the instrumented tie and was calculated by subtracting the load measured by the strain gages above the tie from the load measured by the strain gages above the crib as shown below

$$\textit{Tie Reaction} = \textit{Load above crib} - \textit{Load above tie} \tag{1}$$

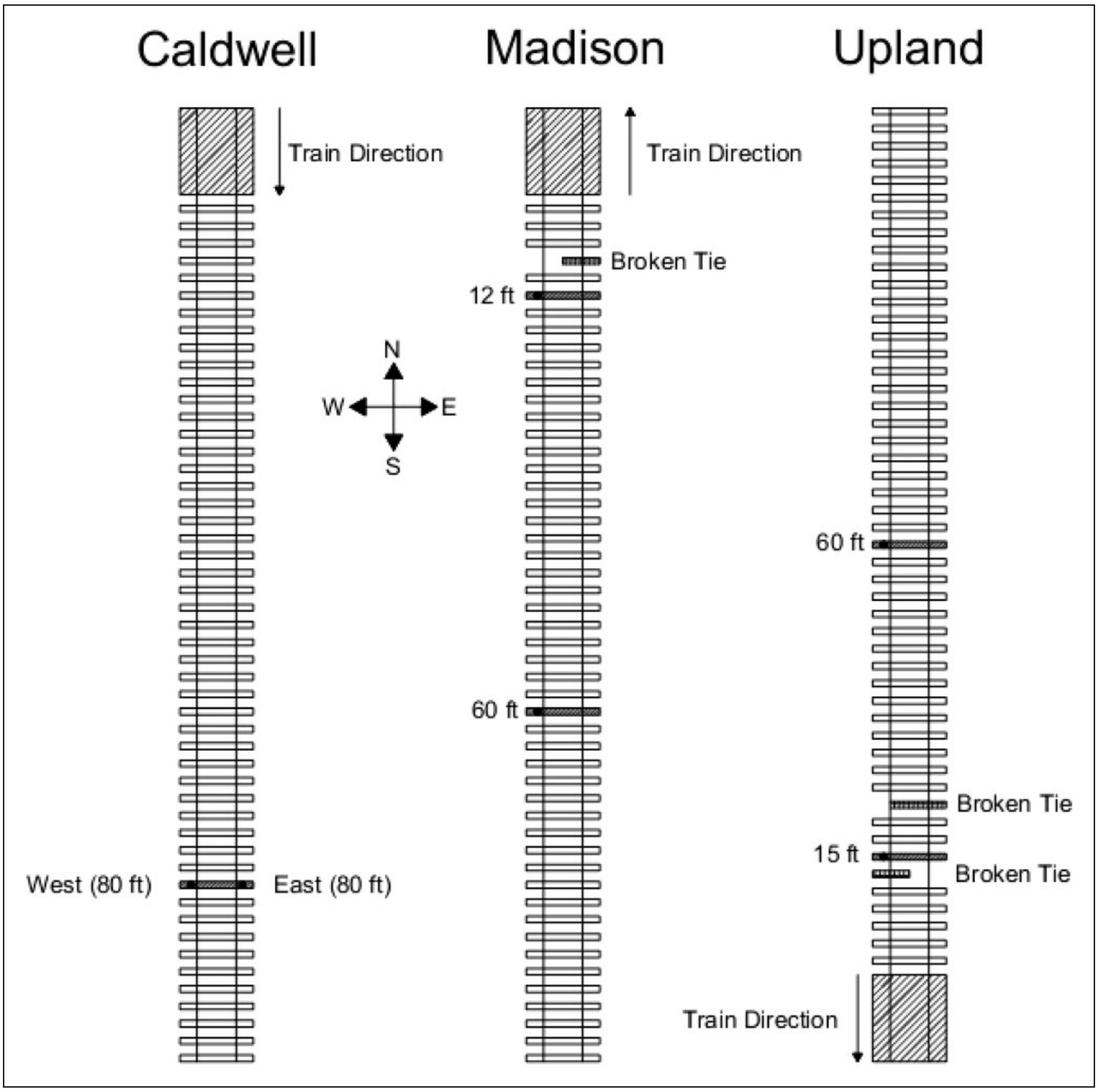

Figure 1. Plan view of the locations of the instruments deployed in Chester, Pennsylvania.

For normalization purposes, the percentage of wheel load carried by the tie was calculated by dividing the tie reaction by the wheel load (load above crib). This is defined as the tie load ratio as shown below

$$Tie\ Load\ Ratio = \frac{Tie\ Reaction}{Wheel\ Load} = \frac{Load\ above\ crib - Load\ above\ tie}{Load\ above\ crib} \quad (2)$$

The physical meaning of the tie load ratio is explained using the following scenarios. In the extreme case of a perfectly rigid foundation, i.e. no displacement of the tie and underlying ballast, no bending or shear strain will develop within the rail above the tie. This means the "load above the tie" will be measured as zero because the rail does not bend, thus, the tie reaction will equal the wheel load (load above the crib). The tie load ratio is calculated as 100%, implying 100% of the wheel load is carried by the tie. At the other extreme, a hanging or unsupported tie produces equal rail shear strains as at the crib, resulting in a calculated tie reaction of zero. Therefore, the tie load ratio, or percent wheel load carried by the tie, will also equal zero. Table 1 summarizes the ranges.

As the tie and substructure are not perfectly rigid, the rail above the tie will deflect and develop shear strains (bends), however, not to the extent seen above the crib. Tie load ratio values of about 40% are often associated with good tie–ballast support[9] whereas values below 30% imply poor tie support. However, it is possible that a poorly supported tie later establishes contact with underlying layers even after significant rail bending. Impact forces (impulses) at the tie–ballast interface associated with a change in the momentum of the moving tie are not measured with the strain gages; however, they may exist even

Figure 2. Photograph showing the strain gage locations in the crib area and above the tie for the vertical wheel load and tie reaction calculations. The strain gages are covered by a watertight gray epoxy for weather protection.

Table 1. Representative tie reaction and tie load ratio values for a range of conditions where *P* represents the wheel load.

Condition	Tie reaction	Tie load ratio (%)
Pinned	*P*	100
Hanging	0	0
Good support	0.4*P*	40
Poor support	<0.3*P*	<30

though they were not measured. Later instrumentation with accelerometers attached to the top of the ties illustrated that ties with poor support experienced greater tie accelerations than well-supported ties.[10]

Displacements as a function of depth

The current study selected LVDTs to measure the vertical displacements as a function of depth as it allows the recording of the vertical displacements of various layers within the substructure. The LVDTs were located at five different depths at each bridge site and were able to measure both transient and permanent vertical displacements at each LVDT depth.[11] Strings of LVDTs, or MDDs, have been used previously to investigate track substructure behavior.[8,12,13]

All six LVDT strings reached depths of about 2.51 m (8 feet 3 inches) with LVDT 1 measuring from the top of the concrete tie 0.3 m (11 to 13 inches) into the ballast layer, LVDT 2 measuring the sub-ballast response, and LVDTs 3 to 5 measuring the subgrade response. Figure 3 shows the LVDT locations for the substructure profile at Upland (60 ft.). The fixed datum point at a depth of 2.51 m was selected because it is the maximum depth obtainable with the available instrumentation and it was assumed that little-to-no transient or permanent displacement occurred below this depth.

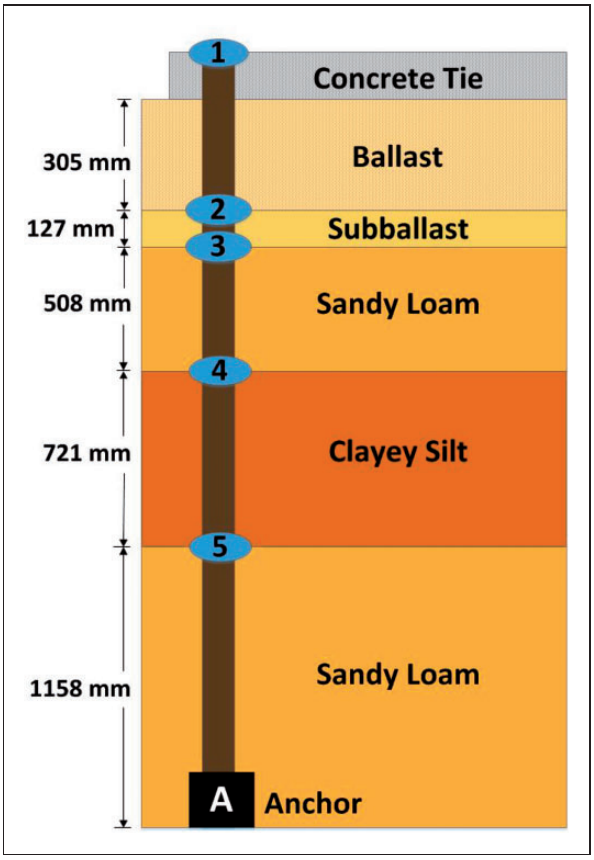

Figure 3. Subsurface profile and LVDT locations 60 feet north of Upland Street Bridge.

Permanent vertical displacements

The current state of knowledge about differential movement at transition zones is that permanent vertical displacements occur; however, the location or layer, e.g. ballast, sub-ballast and/or subgrade, in which the displacement is occurring, is not well understood. This makes developing and implementing appropriate remedial measures a significant challenge; it would be ineffective to stabilize the subgrade if all of the movement is occurring in the ballast or vice versa. Therefore, one of the primary objectives of instrumenting the six Amtrak bridge sites with LVDTs was to locate the depth at which the majority of permanent vertical displacements were occurring. Identifying the layer experiencing the majority of the permanent vertical displacement, e.g. ballast, sub-ballast and/or subgrade, would allow appropriate remedial measures to be implemented. This objective was accomplished by periodically measuring the relative transient and permanent vertical displacements of each LVDT at the Amtrak instrumentation sites.

Directly after installation, the LVDTs were suspected to be moving, a result of the corrugated tube and foam around the tubing becoming compressed due to the weight of the overlying LVDTs, connecting rod and/or tubing, and becoming engaged in the ballast and underlying materials. This meant that it took about 2 months for the LVDTs and supporting casing to become fully engaged to the physical substructure, thus, these initial permanent vertical displacements are also included in the shifting of the LVDT in relation to physical subgrade.

In response, the permanent vertical displacements of the six Amtrak sites were analyzed between 28 September 2012 and 1 April 2013, a total of 185 days. This timeframe was selected because factors resulting in additional movements, e.g. tamping or settlement of the tube, other than the permanent vertical displacements of the substructure did not occur at any of the six sites. The results show that the majority of permanent vertical displacement occurs in LVDT 1, the region consisting of the region between the top of the concrete tie and the bottom of the ballast layer, for all six instrumented locations. As an example, the net permanent vertical displacements as a function of time for the five LVDTs installed at Upland (15 ft.) are displayed in Figure 4. The net permanent vertical displacement is defined as the amount of measured permanent vertical displacement of a particular LVDT. The net permanent vertical displacement of LVDT 1 shows an approximately linear increase of about 14.1 mm/year, whereas LVDTs 2 to 5 do not show any significant permanent vertical displacement with time. The only exception is LVDT 2 at 122 days; it displayed a temporary settlement that rebounded. It is possible that the LVDT "slipped" due to wet weather conditions and returned to its original position after drying. This measurement only includes the movement of the sub-ballast so did not affect the movement of LVDT 1.

The results of the other five instrumented sites are similar in that the permanent vertical displacement of

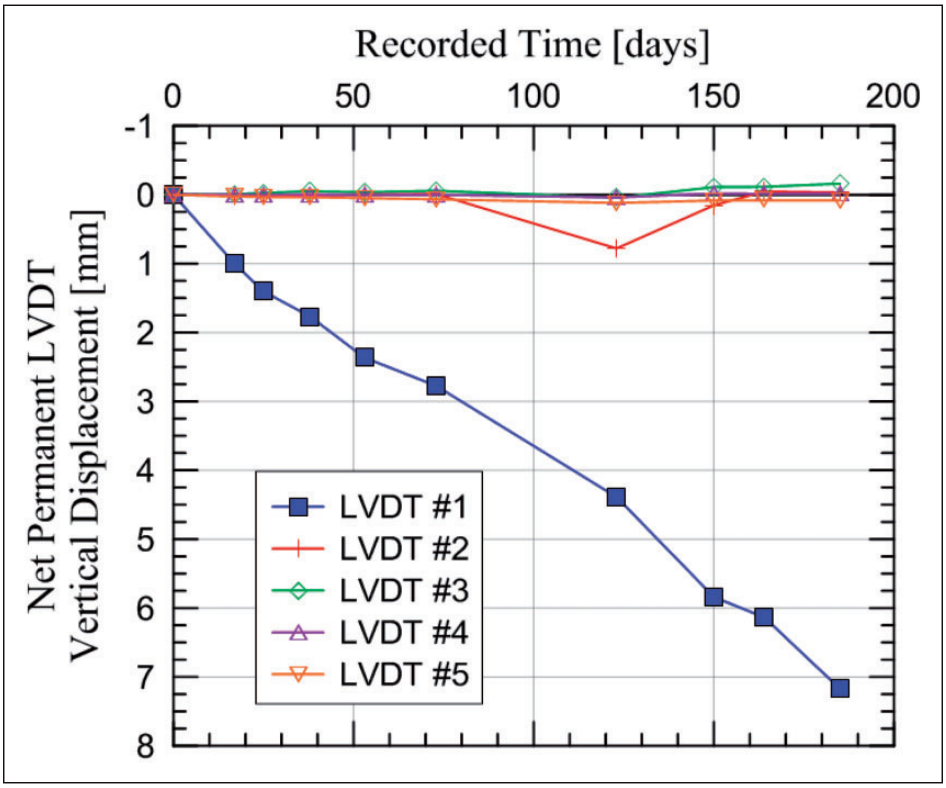

Figure 4. Net permanent vertical displacement at Upland (15 ft.) with the data collected in the first 2 months being omitted.

Table 2. Permanent vertical displacements of each LVDT at the six MDD locations between 27 September 2012 and 1 April 2013 (note: negative values indicate heave and positive values indicate settlement).

	Caldwell Street		Madison Street		Upland Street	
LVDT	East (mm)	West (mm)	12 ft. (mm)	60 ft. (mm)	15 ft. (mm)	60 ft. (mm)
1	1.61	1.61	3.58	2.02	7.17	0.52
2	0.15	0.59	0.12	–0.09	–0.03	0.27
3	0.04	–0.01	0.35	–0.11	–0.16	0.13
4	0.32	0.04	–0.08	0.02	–0.02	0.03
5	0.17	0.04	0.04	0.23	0.08	0.15

LVDT 1 is significantly greater than those for LVDTs 2 to 5. This means the majority of the permanent vertical displacement at these sites is occurring within the region between the top of the concrete tie and the bottom of the ballast layer, which is consistent with previous track settlement studies.[14] This suggests that the sub-ballast and subgrade is experiencing minimal consolidation or settlement from repeated train loadings, thus, remedial measures should focus on the ballast or tie–ballast interface in the transition zone.

The measured permanent vertical displacements of LVDT 1 for all six instrumented sites are listed in Table 2, which shows the greatest permanent vertical displacement occurs at the two bridge approach sites, i.e. Upland (15 ft.) with 7.17 mm and Madison (12 ft.) with 3.58 mm. This verifies that bridge approach sites experience greater permanent displacements than do open track sites. However, transient vertical displacements are required to determine why Upland (15 ft.) experiences a larger permanent vertical displacement than does Madison (12 ft.) and why Upland (60 ft.) experiences the smallest permanent vertical displacement (0.52 mm) of all six instrumentation sites.

Track behavior under transient loading

To understand the behavior of the track when it experiences transient loading, time histories of the wheel loads, tie reactions and transient vertical displacement were collected for multiple passing trains at each site. An example response from a 110 mile/h Acela train passing over the Upland (60 ft.) site is presented in Figure 5. The figure shows data for the passage of eight cars. The first four peaks (~120 kN) represent the initial Acela power car. The next 24 peaks (~80 kN) represent the lighter passenger cars that are followed by another Acela power car. For each passing wheel, all five LVDTs displace but show different response patterns and peaks, with the largest peaks being associated with the Acela power cars. The Acela power cars produce LVDT displacements of about 0.4, 0.22, 0.45, 0.35 and 0.2 mm for LVDTs 1, 2, 3, 4 and 5, respectively. The passenger cars produce LVDT displacements of about 0.375, 0.2, 0.275, 0.2 and 0.1 mm for LVDTs 1, 2, 3, 4 and 5, respectively. The low values of the peak vertical displacement of LVDT 5 suggest that only small amounts of vertical displacement occur below the anchor at a depth of 2.51 m.

The rest of this section presents a general overview of the transient behavior of the track system along with differences in behavior of transition and open track sites by comparing Upland (60 ft.) and Upland (15 ft.). These two sites were selected because they represent sites with the lowest and highest amount of permanent vertical displacement measured by LVDT 1. A thorough analysis of the transient data is important because poorly performing transient behavior of a track leads to larger permanent vertical displacements, thus, identifying and remediating poor performance at the transient stage can prevent track geometry issues over time.

Tie reaction

One difference in track system behavior between Upland (60 ft.) and Upland (15 ft.) is the tie reaction, which is shown in Figure 6 for passage of the same Acela power car (four wheels) on 7 August 2012. At Upland (60 ft.), the four tie reaction load peaks are equal to about 40% of the peak wheel load. The four corresponding wheel loads are 84, 86, 79 and 77 kN giving wheel load ratios of 39, 44, 44 and 46%. This indicates good tie support and is in agreement with 40% of the peak wheel load being supported by the underlying tie.[9] At Upland (15 ft.), the four tie reaction load peaks are not readily apparent and all of the values of the tie reactions are less than 30%, which indicates poor tie support.

The low values for the tie load ratios suggest a few scenarios are possibly occurring. First, the rail or tie at Upland (15 ft.) could be hanging, resulting in no contact and therefore no load transfer to the ballast underlying the instrumented tie. In this case, the wheel load is transferred to adjacent ties, which increases the load on adjacent ties. Second, a passing train could cause significant bending of the rail above the tie, however, contact between the tie and ballast is still established. In this case, the bending of the rail over the poorly supported tie will distribute load to adjacent ties, however, the instrumented tie may still transfer a significant load to the underlying ballast. Because the strain gages only measure rail bending, the transfer of the loading of the instrumented tie to the underlying ballast is not known in unsupported conditions.

The tie reaction data suggests that Upland (60 ft.) is experiencing typical or desired load distribution (40% of wheel load) due to good tie support whereas an undesired load distribution is occurring at Upland (15 ft.) due to poor tie support. This undesired load

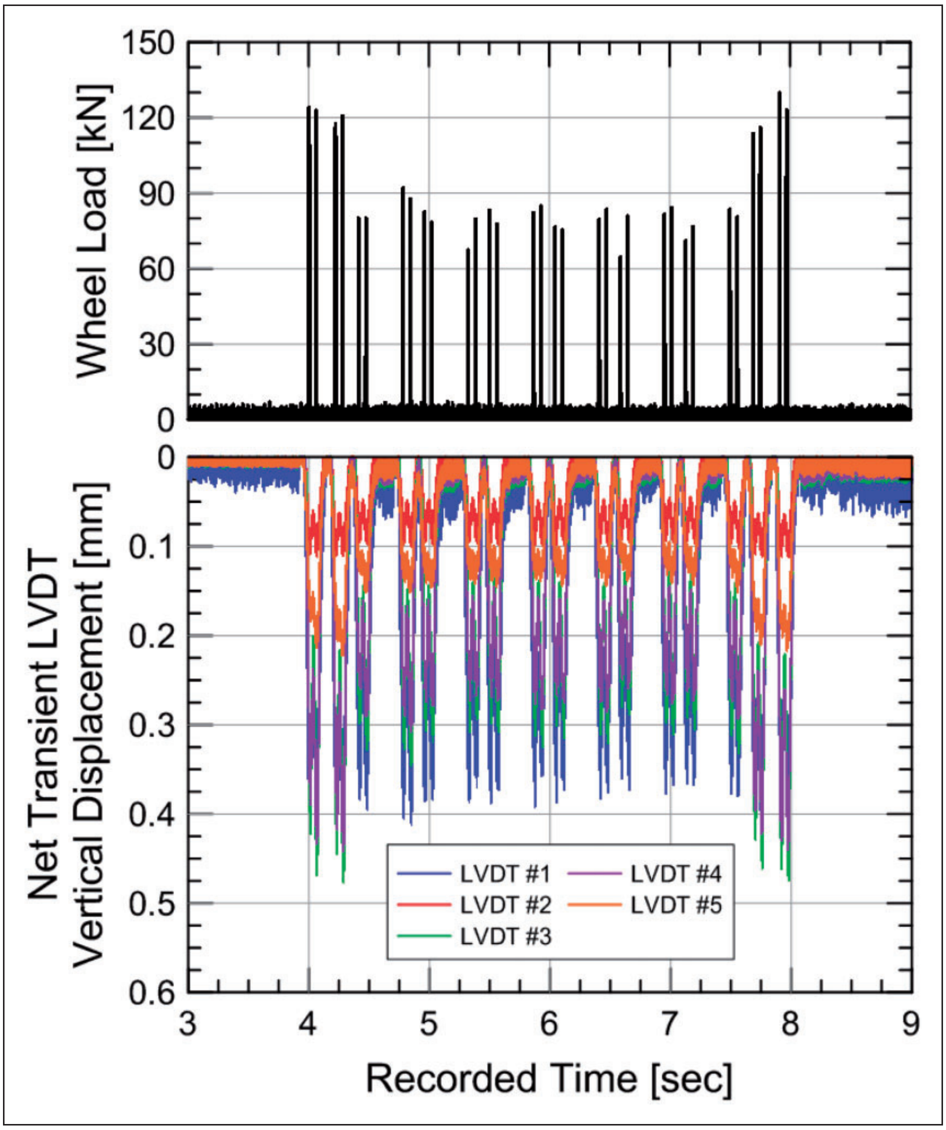

Figure 5. Measured wheel loads and their corresponding net transient vertical displacements at Upland (60 ft.) measured on 7 August 2013 at 11:18 am.

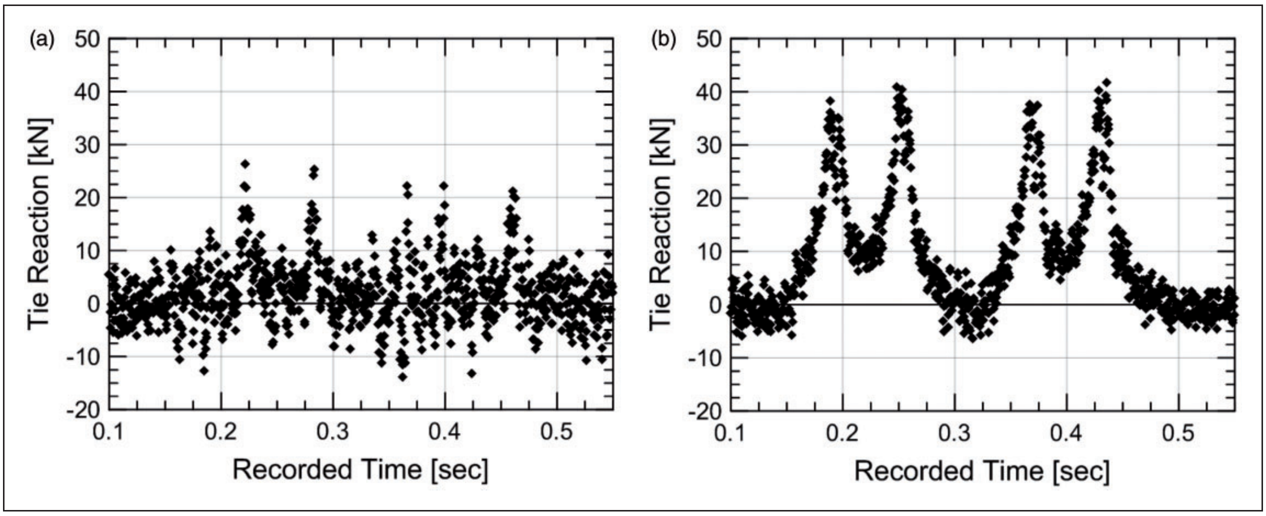

Figure 6. Tie reaction loading at (a) Upland (60 ft.) and (b) Upland (15 ft.) measured on 7 August 2012 at 10:17 am showing good and bad tie support.

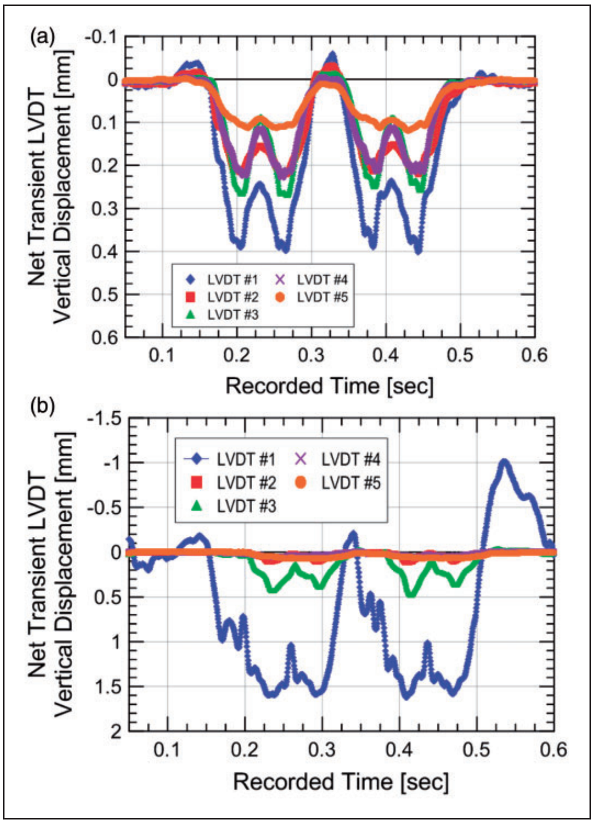

Figure 7. Net transient vertical displacement behavior in response to the Acela power car measured on 7 August 2012 by the LVDTs at (a) Upland (60 ft.) and (b) Upland (15 ft.).

distribution will result in increased loads to adjacent ties, which can damage track components and accelerate permanent displacement of track section and/or ballast.

Transient vertical displacements of a tie

A second significant behavioral difference between transition and open track sites is the measured transient vertical displacements. An example of the different response from four passing wheels from the same Acela power car at Upland (15 ft.) and Upland (60 ft.) is illustrated in Figure 7. The following list summarizes some of the differences in the response of Upland (15 ft.) and Upland (60 ft.).

1. The peak transient vertical displacement of LVDT 1 at Upland (60 ft.) is smaller than Upland (15 ft.), i.e. 0.4 mm versus 1.5 mm, which is evident by comparing the different vertical axes.
2. At Upland (60 ft.), all five LVDTs begin recording vertical displacements at the same time whereas the measurements of the vertical displacements by LVDTs 2 to 5 at Upland (15 ft.) are delayed after LVDT 1 responds.
3. The vertical displacements in LVDT 1 are smooth at Upland (60 ft.) whereas a more erratic response is observed in LVDT 1 at Upland (15 ft.), which indicates ‘‘dancing tie’’ behavior.
4. A significant amount of rebound at the tie is measured at Upland (15 ft.) after wheel passage whereas there appears to be little or no tie rebound at Upland (60 ft.).

These four observations are indicators of poor tie support at Upland (15 ft.) and good tie support at Upland (60 ft.). If the concrete tie is in poor contact with the underlying ballast, LVDT 1 experiences a larger displacement in order to establish contact with the ballast. This displacement delays the responses of LVDTs 2 to 5 at Upland (15 ft.) because the poorly supported tie takes longer to contact the ballast and transfer the applied load. Once the tie establishes contact with the ballast, i.e. it closes the tie–ballast gap, the underlying ballast and sub-ballast experience load and displacement, which is measured by LVDTs 2 to 5, to resist the applied wheel load. This implies that the measured transient vertical displacements of LVDT 1 may not equate to the physical displacement of the ballast, due to the LVDT 1 measurement including both closure of the tie–ballast gap and the tie displacement required for the ballast seating load.

Also poor tie–ballast support gives the tie more freedom to move, explaining the erratic behavior and rebound observed at Upland (15 ft.) in Figure 7(b). This tie movement is manifested by the ‘‘dancing tie’’ behavior observed during field observations. The existence of poor tie support at transition zones experiencing recurrent track geometry issues is not uncommon and the increased tie displacement and subsequent rebound has also been reported at instrumented sites in Europe.[15]

Load–displacement response at LVDT 1

The previous two sections showed qualitative differences between good and poorly supported ties using tie reaction and transient vertical displacement time histories. This section quantifies tie support by illustrating how the height of the tie–ballast gap can be estimated. This is accomplished by recording the peak wheel load and peak transient displacement of LVDT 1 for each measured passing wheel (see Figure 8) in a load–displacement diagram. By fitting a linear best-fit line to the data using the least square method, the load–displacement behavior of LVDT 1 can be quantified using the following expression

$$\delta_{\mathrm{LVDT\_1}}(P) = \delta_{P=0} + \frac{P}{k_{\mathrm{mob}}} \tag{3}$$

where $\delta_{\mathrm{LVDT\_1}}$ is the displacement of LVDT 1 as a function of wheel load (P), $\delta_{P=0}$ equals the displacement at a zero load condition, and k_{mob} represents the stiffness of the moving ballast. By assuming that the ballast is compact and creates full shear resistance by

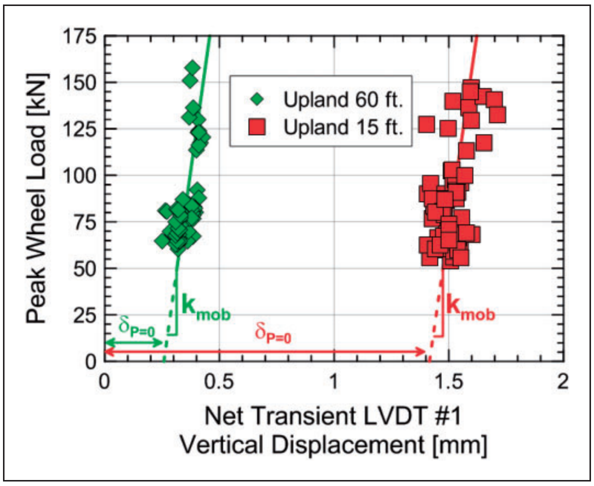

Figure 8. Comparison of transient vertical displacements and peak wheel loads at Upland (15 ft.) and Upland (60 ft.) measured by LVDT 1 on 26 January 2013.

particle interlocking upon contact with the tie, the tie–ballast gap is represented by $\delta_{P=0}$. Therefore, the parameters of the tie–ballast gap ($\delta_{P=0}$) and stiffness of the moving ballast (k_{mob}) are used to describe the response of LVDT 1.

Figure 8 illustrates the tie–ballast gap, i.e. $\delta_{P=0}$, using open track (60 feet from the bridge) and transition (15 feet from the bridge) data from the Upland Street Bridge site. The solid lines represent the best-fit response interpolated between measured wheel loads ($R^2 = 0.3603$ for Upland (60 ft.) and $R^2 = 0.2274$ for Upland (15 ft.)). The dotted lines show the linear extrapolation to the unloaded condition. Upland (60 ft.) shows a small tie–ballast gap (~0.25 mm), which resulted in the smallest permanent vertical displacement of all six instrumented sites. Conversely, Upland (15 ft.) shows a larger tie–ballast gap (~1.42 mm), which resulted in the largest permanent vertical displacement of the six sites. This validates and quantifies the results in the previous section and shows significantly different behavior at Upland (60 ft.) and Upland (15 ft.) as a result of varying tie support. Figure 8 also shows that the ballast exhibits a similar stiffness at Upland (60 ft.) and Upland (15 ft.) after the tie–ballast gap has closed due to the ballast being compacted by the tie.

Table 3 compares the tie–ballast gap ($\delta_{P=0}$) and stiffness of the moving ballast (k_{mob}) parameters for all six instrumented sites. A wide range of values is observed, especially for ballast stiffness values back-calculated using the measured wheel loads and vertical displacements. This is likely due to different fouling and drainage conditions along with different load distributions on the underlying ballast and adjacent ties.

As the measured data suggest that the tie support may be correlated to track geometry problems along transition zones, then the calculated tie–ballast gap ($\delta_{P=0}$) is related to permanent vertical displacements.

Table 3. Values of estimated tie–ballast gap, stiffness of moving ballast and Young's modulus at all six instrumented sites for 26 January 2013.

Instrumented site	Caldwell Street		Madison Street		Upland Street	
	East	West	12 ft.	60 ft.	15 ft.	60 ft.
$\delta_{P=0}$ (mm)	0.78	0.76	1.61	0.62	1.42	0.26
k_{mob} (kN/mm)	530	192	1322	410	848	876

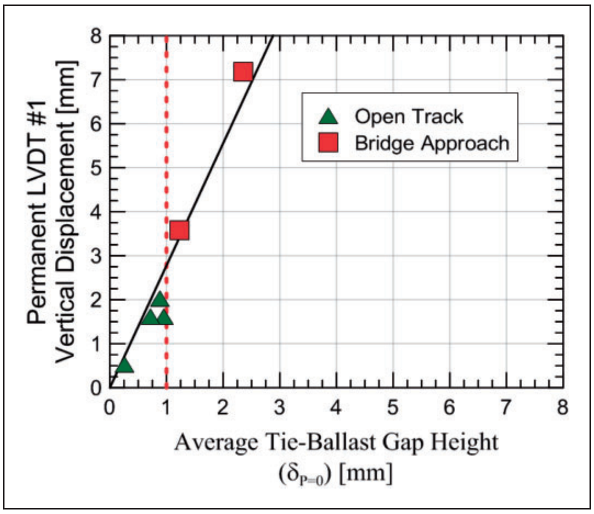

Figure 9. Correlation between average tie–ballast gap height and net permanent displacement measured using LVDT 1.

Figure 9 shows a strong correlation between the average tie–ballast gap ($\delta_{P=0}$) obtained during four different data recordings (August 2012, November 2013, January 2013 and June 2013) and the accumulated permanent vertical displacement measured over the same time period ($R^2 = 0.9352$). The response appears to be linear and strongly matches the data. This implies that the tie–ballast gap may be the "root cause" of the permanent vertical displacements observed at the Amtrak sites and remedial measures should aim to decrease the tie–ballast gap or prevent gap formation altogether. Although it may be unrealistic to eliminate the tie–ballast gap, this analysis suggests that tie–ballast gaps ($\delta_{P=0}$) greater than 1 mm can cause load redistribution to adjacent ties and amplify loads at the subject tie as suggested by Selig and Waters.[14]

Root cause of transition differential displacements

Root cause

The data from the six Amtrak instrumentation sites suggest that the primary factor causing the observed permanent vertical displacements is the presence of a tie–ballast gap. Tie–ballast gaps can result in increased permanent vertical displacements due to a

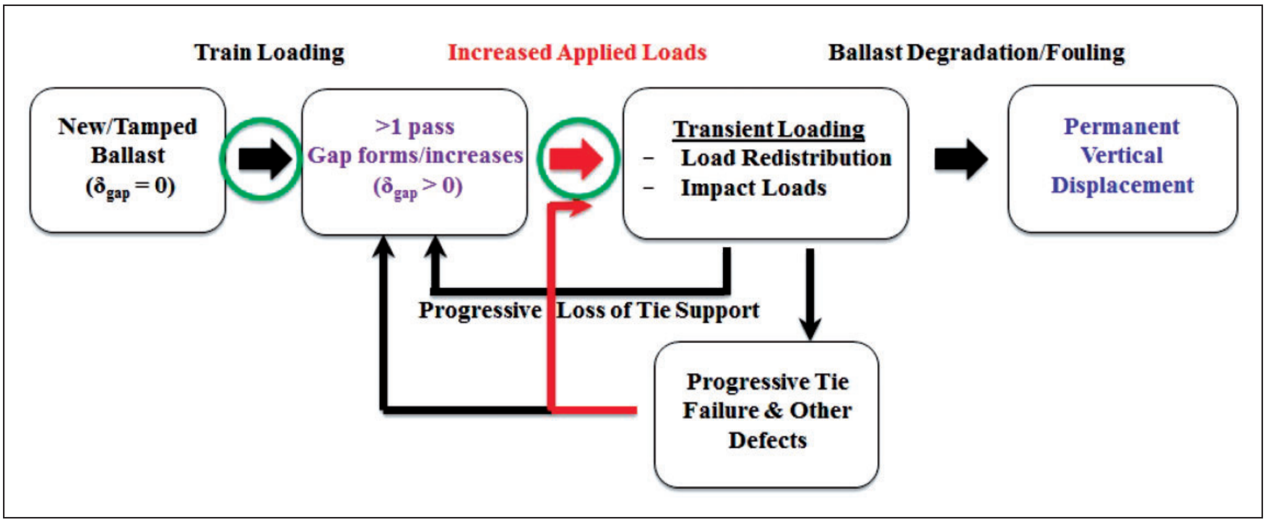

Figure 10. The chain of events that results in the creation of permanent vertical displacements.

gap being able to significantly alter how a load is distributed to the ties and ballast. The existence of a gap results in unfavorable load distributions to the underlying ties and impact loads from the momentum of a moving tie contacting the ballast. This load amplification can accelerate tie damage and ballast degradation from particle crushing and fouling.

From the measured data, tie–ballast gaps greater than 1 mm appear to separate sites experiencing good tie support and acceptable permanent vertical displacements from sites experiencing poor tie support and permanent vertical displacements that require recurrent tamping and resurfacing or some other remedial measure. Although this value appears small and almost negligible, experimental ballast box testing has shown significantly greater permanent vertical displacements when a gap greater than 1 mm is present.[14] Also, numerical studies have shown that significant load redistribution occurs with gap heights of 1 mm.[16,17]

In addition to the analysis of the measured field data, multiple field visits were made to visually assess track quality. Site observations showed multiple track irregularities, such as damaged ties and rail height differences, near the instrumented ties experiencing large permanent vertical displacements along with fouled ballast and mud pumping. A literature review also showed that abrupt changes in track stiffness at bridge abutments can produce impact loads within the transition zone. [2] Although these factors are not measured by the installed instrumentation, they will affect track behavior by increasing the loads applied to the track system. Therefore, other factors in addition to the tie–ballast gap contribute to the "root cause" of the permanent vertical displacements of ballast near instrumented bridge approaches such as increased applied loads on the ballast, inadequate drainage, wet fouled ballast, and damaged ties in certain locations.

Chain of events

With the primary "root cause" identified as being applied loads from multiple factors, namely the existence of a tie–ballast gap, the probable chain of events was investigated and the results presented in Figure 10. Presenting the chain of events as a flow chart allows for the conceptualization of the entire process along with identification of one or multiple locations in the chain where a suitable repair or intervention could be introduced to slow development of additional permanent vertical displacements. Two instances of potential times of intervention are illustrated with the green circles and they are explained in the following section.

After new track is placed or tamping occurs, the ballast initially begins in a loose state and quickly compacts under the tie due to high wheel loads from the first passing train, this results in permanent substructure displacements.[14,15,18] This permanent displacement of the substructure varies along the track due to uneven ballast compaction, abrupt changes in subgrade, fouling and man-made structures. After the first wheel passes, the stiff rail "hangs" or "cantilevers" from the track regions experiencing the least amount of permanent substructure displacement and pulls the tie back up at regions with larger permanent substructure displacements, which produces a gap between the bottom of the tie and top of the ballast. This situation is most extreme at bridge transition zones where the man-made structure is essentially rigid and will not permanently displace, unlike the earth material substructure in the nearby transition zones. As the permanent substructure displacement increases, the rail will eventually "dip" and develop the noticed rail profile deviations common at railway bridge transition zones. A conceptual diagram of the final state is illustrated in Figure 11.

Many of the field observations made during this instrumentation setup also agree with a previously

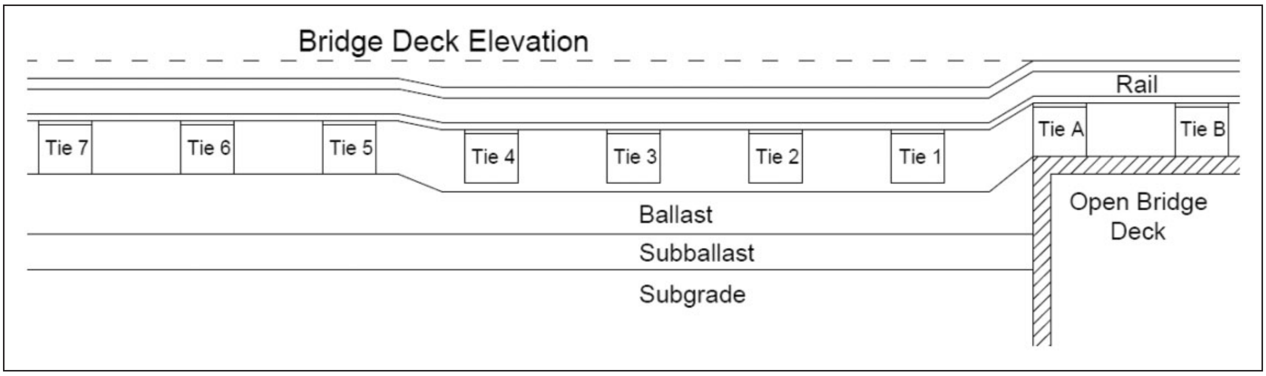

Figure 11. A railway bridge transition zone, with tie–ballast gaps, permanent substructure displacements and rail profile deviations.

instrumented culvert transition zone in the Netherlands.[15] At that site, it was observed that:

- significant track settlement was observed immediately after tamping;
- the transitions zone site experienced significantly larger tie displacements than the culvert and free (open) track sites, due to the existence of tie–ballast gaps;
- four transition zone ties were shown to rebound and contact the ballast (seat) in unison.

The first two observations agree with the instrumentation results presented in this paper along with the importance of tie–ballast gaps and the third suggests the entire track section was poorly supported and being supported by ties in the free (open) track and above the culvert (see Figure 11).

Once a tie–ballast gap develops, load redistributions and impact loads from subsequent wheel and train loadings further increase the gap height and potentially damage the instrumented and surrounding ties. The load redistribution mechanism has been demonstrated with numerical simulations of a typical open track section and the instrumented Netherlands site. Each model resulted in increased applied loads of surrounding ties because of the existence of poorly supported ties.[17,19] The increased gap height and damaged ties then result in greater load redistribution and impact loads, further increasing the poorly supported behavior and spreading the damage. These cyclic processes are defined as "progressive loss of tie support" or "progressive tie failure". As the applied loads increases due to the increase in the tie–ballast gap, the ballast continues to compress, degrade and foul, causing additional permanent vertical displacements.

When the permanent vertical displacements exceed a particular threshold, railroads tamp to return the rail to its original elevation. With the previously compacted ballast now in a loose state due to tamping, the chain of events described above repeats itself because the ballast quickly compacts again under the first train passage.[14,15] Tamping will be required in another few months to re-level the rail. As a result, more long-term remedial options are being sought so railroads do not have to return to tamp the area.

Potential designs and remedial action

The results of the instrumentation and data analysis suggest that permanent vertical displacements within the transition zone can be reduced by preventing or limiting a tie–ballast gap greater than 1 mm from developing. This can be accomplished in two different ways.

1. Designing a transition zone so the conditions that create tie–ballast gaps do not develop, e.g. differential transient and permanent displacements.
2. Remediating the track so the ballast remains compact and filling the tie–ballast gap with material.

Referring to the flow chart in Figure 10, remediation 1 prevents the gap from initially developing whereas remediation 2 fills the gap before the applied loads become amplified enough to spread the tie–ballast gaps to surrounding ties and cause further permanent vertical displacements in the ballast layer.

Numerous design and remedial measures for transition zones have been suggested, they typically consist of stiffening or smoothing the stiffness difference between the open track, bridge approach and bridge[4–6,20] or reducing the stiffness of the bridge.[5,21–23] This philosophy attempts to minimize the differential transient displacements between the transition zone and bridge. These fixes can be successful but are difficult to implement as the bridge stiffness often remains significantly greater than the approach even after implementation[4,23] and once the approach substructure begins to permanently displace, the differential transient and permanent displacements between the approach and bridge will restart the deterioration process.

As a result, new track designs should focus on minimizing both the differential transient and permanent displacement between the approach and bridge. For example, in an open bridge deck scenario the transition zone experiences additional transient displacement with tie–ballast gaps, ballast displacement, sub-ballast displacement, subgrade displacement, and any lateral substructure displacement. The transition zone also experiences additional permanent displacement from the ballast, sub-ballast, subgrade materials and lateral displacements. For example, remediating a ballasted deck bridge will add transient and permanent ballast displacements to the bridge but the transition zone will still experience additional transient displacements from tie–ballast gaps, sub-ballast, subgrade and lateral displacements along with permanent displacements from sub-ballast and subgrade settlements. Therefore, all these factors should be considered to reduce both the transient and permanent differential displacement between the transition zone and bridge. This likely involves using multiple design features some of which are described in the literature.[4–6,20–23]

For remediation of track geometry, tamping is a commonly used technique that involves raising the ballast; this loosens the ballast underlying the tie. The first train pass after tamping compacts the loosened ballast and creates a tie–ballast gap.[24] Ideally, remediation techniques that address the tie–ballast gap should keep the underlying ballast compact and add new material underneath the tie. Stone-blowing accomplishes this and comparisons show that stone-blowing holds the track geometry longer than tamping[24], however, the small rocks ''blown'' under the tie may eventually be pushed or migrated into the voids or matrix of the larger ballast particles. Installation of a rigid shim or pad underneath the tie instead of tamping or stone-blowing may be beneficial, it will fill the gap and essentially act as an extension of the tie and will not foul the ballast. Additional remedial methods include attempting to reduce the stress applied to the ballast with larger ties[21,25] or reduce the tie spacing. Although these solutions help by reducing the applied stress, they do not address the development of a tie–ballast gap and/or a rail–tie gap.

Summary and future work

Transition zones represent a challenge to the upgrading of railroad track for use by high-speed passenger trains due to recurring track geometry problems that create safety issues and amplified loads can accelerate track deterioration in the transition. As part of an FRA-sponsored research project on minimizing differential movement at railway transitions for high-speed passenger routes, six Amtrak NEC sites near Chester, Pennsylvania were instrumented with strain gages and LVDTs to determine the ''root cause'' of observed differential displacements and develop cost-effective designs and remedial measures to mitigate the problem.

Based on the data and analysis presented herein, the following observations can be made about differential movement at high-speed passenger transition zones.

1. The majority of the permanent vertical displacements at these locations occurred in the ballast layer.
2. The ''root cause'' of the permanent vertical displacements at instrumented bridge approaches was determined to be an increase in the load applied to the ballast from the existence of a tie–ballast gap resulting from ballast compaction and settlement. The gap redistributes wheel load and amplifies contact forces on impact, leading to increased ballast compaction and degradation. Stiffness differences between the soft approach and stiff bridge, in addition to damaged ties and other track system defects, also contribute to increasing the load applied on the ballast.
3. Poor tie support can be detected by analyzing the tie reaction time histories, transient vertical displacement time histories, and by developing a relationship between peak wheel load and transient displacement using a load–displacement plot.
4. New railway transition zone design should focus on minimizing both the transient and permanent displacements between the transition zone and bridge deck. This likely involves using multiple design features and not a single fix.
5. Remedial measures should involve closing the tie–ballast gap while maintaining the compacted density of the underlying ballast. This can be accomplished in a variety ways that are being field tested.

By identifying the ''root cause'' of the differential movement at these six instrumentation locations, various remedial measures can be tested to help mitigate the recurrent track geometry problem at other railroad transitions. Successfully addressing this geometry issue can reduce future maintenance costs and slow orders and help the expansion of high-speed rail in the United States.

Future research will be focused on: measurement of track system behavior at other high-speed passenger and freight sites, numerical modeling of the influence of poorly supported ties and the effect on transition zones, and development of a non-destructive instrumentation system to quickly and non-invasively quantify the tie support.

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