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Tie-Ballast Interaction at Railroad Transitions

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ABSTRACT

This paper presents a conceptual model of tie-ballast interaction for high-speed passenger and freight railroads. The main components of the model are inclusion of a gap between the tie bottom and underlying ballast and a cubic function to model the tie-ballast load-displacement behavior. Field measurements from high-speed passenger and freight railroads were used to develop and verify the model and show tie-ballast gaps exist at every instrumented site. As a result, the main questions for railroad personnel are whether or not: (1) the tie-ballast gap is large enough to cause detrimental load redistribution amongst adjacent ties and (2) will the gap increase with additional traffic causing further load redistribution and gaps to develop under adjacent ties. Substructure load-displacement behavior is also used to estimate track modulus for use in numerical analyses of track transitions to evaluate design and repair options.

Keywords: Tie-ballast interface, tie-ballast interaction, tie-ballast gap, poor support, unsupported, transition zones, track transition, crosstie, ballast

INTRODUCTION

Problematic vertical track movement can occur due to a variety of reasons, such as a rail-fastener gap, tie-ballast gap, ballast rearrangement and/or breakage, and subballast and subgrade displacement. The mechanism(s) causing vertical track movement will vary at each site and a site specific determination is required. A companion paper (*1*) presents non-destructive techniques (accelerometers and video cameras) for identifying the mechanism(s) causing vertical track movement at each site. This paper focuses on the tie-ballast gap and interaction using a site with well- and poorly-supported ties.

A gap between the bottom of a tie and top of ballast is a problem that manifests itself often by increased loading of all track components and can be noted by significant tie movement under passing wheels once the gap is adequately large. The formation of a gap is generally due to uneven track support that can result in local settlement of the tie supporting layers. The main contributor to the formation of a gap is hypothesized to be differential movement between the bridge and approach in transition zones, the presence of loose and/or fouled ballast, and a compressible subgrade. Regardless of the cause, the presence of a tie-ballast gap can result in increased loading and changes in load distribution along the track.

Understanding tie-ballast interaction is important because it determines the distribution of load amongst adjacent ties, which, in turn, defines the stresses and deformations that develop within the track structure. While mathematical models exist to predict track structure stresses and deformations, few field measurements are available to verify the models for a range of field conditions. This paper uses high speed passenger and freight field measurements to develop and confirm a tie load-displacement model for a range of field conditions, including the presence of a gap between the bottom of the tie and top of the ballast. This concept can then be incorporated in numerical modeling to improve remedial measures and bridge design to reduce track differential movements.

PRIOR RESEARCH

Historically rail analysis and design assume intimate contact between the tie and ballast and a linear tie-ballast load-displacement behavior. These assumptions simplify the beam on elastic foundation equations and allow quick calculation of track modulus, maximum rail deflection, rail bending moment, and tie seating load and are used in the AREMA manual (*2*). Assuming a tie spacing of about 20 inches, the percent tie seat load, i.e. percent of axle load carried by a single tie, ranges from about 25% to 45% using the AREMA methodology. Combined laboratory and numerical studies using the software GEOTRACK (*3*) show a tie seat load of about 40% of the wheel load. Most laboratory testing (*4*) and numerical analyses (5-*7*) also impose or assume intimate contact between the tie and ballast. Assuming intimate tie-ballast contact significantly affects the results and probably does not reflect most field conditions because field measurements presented below show that every instrumented tie in this study has a tie-ballast gap. Therefore, the main issue is the size of the gap rather than whether or not a gap exists.

Another important result from passenger and freight field measurements presented herein is the manifestation of non-linear tie load-displacement behavior. This non-linearity occurs from gaps or voids within the track system so an initial displacement is required to mobilize the stiffness of the ballast and subgrade. The load required to close gaps within the track system, i.e., fully mobilize the available substructure resistance, is defined as the tie seating load. At wheel loads below the tie seating load, a non-linear track stiffness occurs while a linear track stiffness or modulus is observed at wheel loads above the tie seating load.

To accommodate this non-linear track behavior, modifications have been made to the beam on elastic foundation theory. For example, Kerr and Shenton (8) suggest track behavior below the tie seating load should display a lower stiffness than above the seating load and use a bi-linear model to capture this behavior. Lu et al. (9) incorporate a cubic expression in the beam on elastic foundation equations to model the initial non-linear load-displacement behavior. Sussmann et al. (10) propose a conceptual model that includes gaps within the track system, i.e., gaps below the rail and tie, and the initial non-linear seating behavior.

While existence of a gap beneath ties has been known, its prominence, magnitude, and effects have not been previously quantified. The non-linear track behavior from tie-ballast gaps has been shown in experimental studies (*11*) and multiple field studies suggest that large measured transient and permanent vertical displacements could be caused by gaps or voids under instrumented ties (*12-15*). Numerical studies (*16,17*) show the existence of a gap redistributes applied wheel load from the tie with a gap to surrounding well supported ties. Most importantly, laboratory ballast box tests by Selig and Waters (*4*) show tie-ballast gaps greater than 1 mm can result in increased ballast settlement during repetitive loading.

This paper uses field measurements to show the existence of a tie-ballast gap and define the non-linear load-displacement behavior of tie-ballast interaction below the tie seating load. The proposed non-linear load-displacement model is verified and the necessary parameters quantified using transient vertical displacements measured using LVDTs installed on high-speed passenger and freight tracks. This allows calibration of field and numerical models to quantify tie support and improve track design and remedial measures for ties with excessive tie-ballast gaps.

This tie-ballast interaction study is part of a larger FRA research project to determine the root cause of differential movement at track transitions and appropriate remedial measures. The main impetus for this research project was a similar project on highway bridge transitions by the second author (*18, 19*), which provided some understanding of load-displacement behavior at bridge transitions and development of new design and remedial measures. This paper summarizes the field measurements, tie load-displacement model, and size of tie-ballast gaps in track transitions.

FIELD INSTRUMENTATION

To investigate the range of causes and solutions for differential movement at track transitions, high-speed passenger and freight track were instrumented to quantify track behavior. The instruments installed are strain gages attached to the rail to measure applied wheel loads and LVDTs to measure the associated transient vertical displacements at different depths. Additional details are in (*20*). The measured wheel loads and transient vertical displacements of passing trains are used to understand track load-displacement behavior, develop the tie-ballast interaction model presented herein, and calibrate dynamic numerical modeling of the instrumented track transitions.

The behavior of high-speed passenger trains was measured along Amtrak's Northeast Corridor (NEC) at six different locations (*20, 21*). This paper focuses on one bridge transition site about 15 ft (4.57 m) from the bridge abutment along with the open track counterpart about 60 ft (18.2 m) from the bridge abutment at the Upland Street Bridge in Chester, Pennsylvania.

These two Upland Street Bridge sites are referred to as Upland (15 ft) and Upland (60 ft) and are investigated herein. The track at these two locations is straight, elevated, and confined by large gravity retaining walls so one-dimensional vertical movement is assumed throughout this investigation. The applied loading consists of high-speed passenger trains that usually operate at 110 mph (177 km/h) over this FRA Class 7 track.

To measure track behavior under freight train operations, one undergrade bridge approach was instrumented at Norfolk Southern's N-Line located at milepost 352.2 between Roanoke, Virginia and Bluefield, West Virginia. The bridge at MP 352.2 is located on a 10-degree curve and on a 1.1% grade with two instrumented sites 13 ft (4 m) and 31 ft (9.5 m) from the bridge abutment. These two instrumentation sites are referred to as MP 352.2 (13 ft.) and MP 352.2 (31 ft.) herein. Track speed is commonly 25 mph (40 km/hr) over this FRA Class 4 track, which can be subjected to empty and loaded cars. This variation in wheel loads facilitated definition of the non-linear tie load-displacement behavior described above.

Transient Vertical Displacements with Depths

To understand substructure behavior, five LVDTs were installed at different depths to measure relative transient vertical displacements with depth. LVDTs were selected because they measure transient vertical displacement of discrete layers, allowing multiple layers within the substructure to be analyzed during train loading.

At the Upland Street Bridge locations LVDT#1 measures vertical displacements from the top of the concrete tie to about 12 inches (0.3 m) into the ballast layer, LVDT #2 measures the subballast displacements, and LVDTs #3 through #5 measure subgrade performance to a depth of 8 ft 3 in (2.5 m). Figure 1 shows these LVDT locations and the substructure profile at Upland (60 ft.). The LVDTs at the Norfolk Southern (NS) sites extend to a depth of 18.3 ft (5.6 m) because it was initially believed that freight trains would induce vertical displacements at depths greater than high speed passenger trains. In this paper only LVDT #1 is investigated because substantial transient and permanent vertical displacements were not measured below LVDT #1 at the NS and Amtrak sites (*22*).

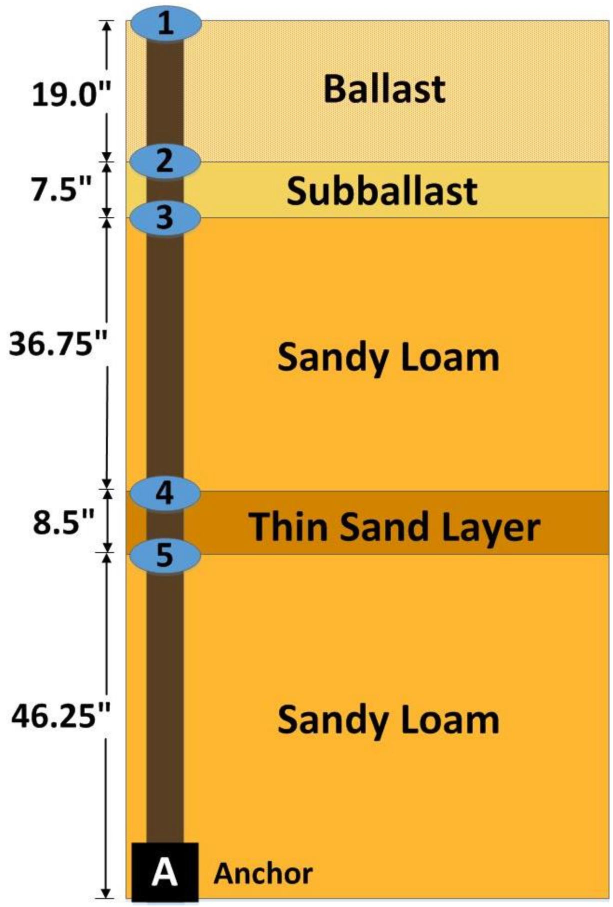
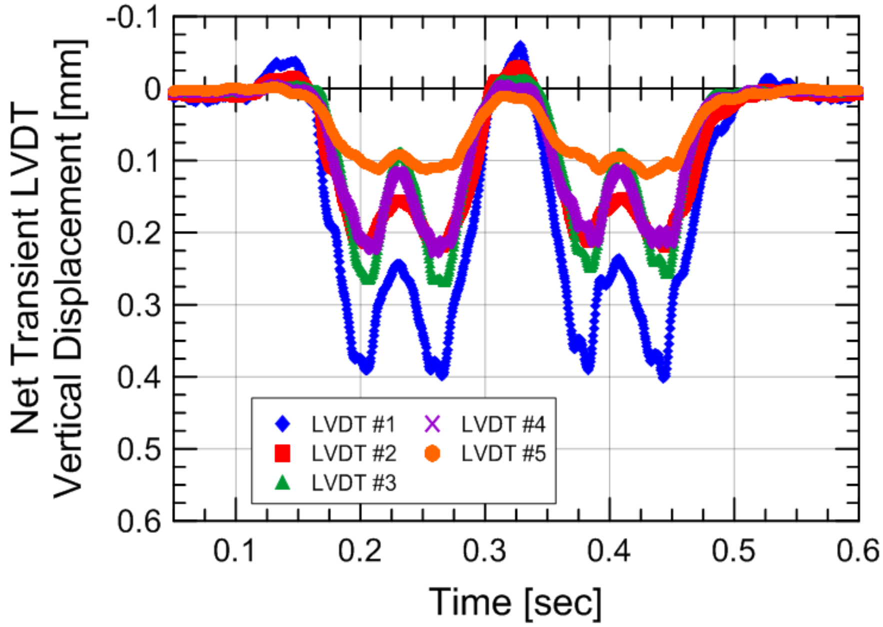


FIGURE 1 Subsurface profile and LVDT locations 60 feet north of Amtrak Upland Street Bridge in Chester, Pennsylvania.

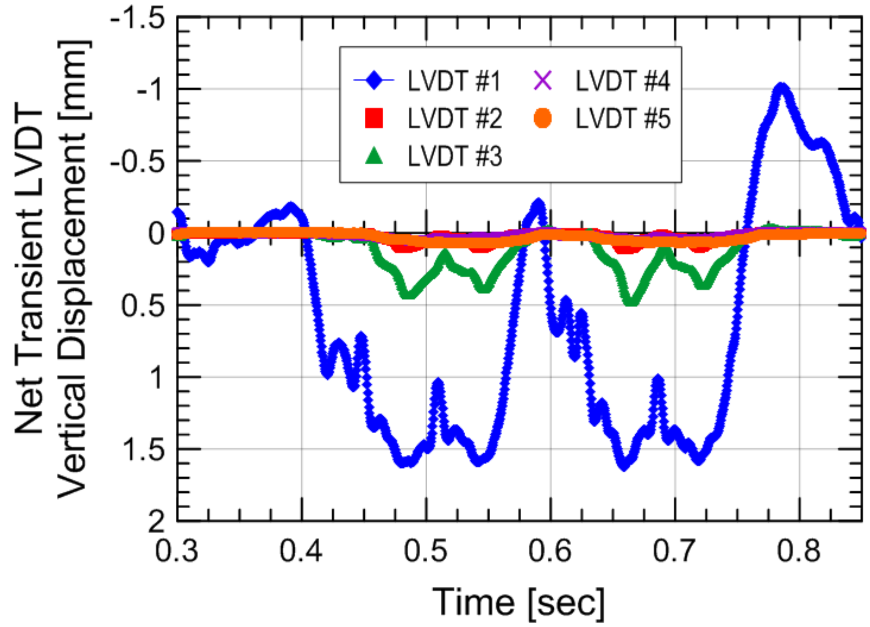
TRANSIENT VERTICAL DISPLACEMENTS

To understand track behavior under transient loading, time histories of measured wheel loads and transient vertical displacement were analyzed for Amtrak high-speed passenger and NS freight trains. Significant differences in substructure behavior were observed for ties experiencing good and poor tie support, leading Stark and Wilk (*22*) to conclude that a relationship exists between tie support and permanent vertical displacements at these two sites. Examples of typical transient vertical displacement responses to a 110 mph (177 km/hr) passing Acela train at Upland Street Bridge on a good (Fig. 2(a)) and poorly (Fig. 2(b)) supported tie are illustrated in Figure 2. The main differences in the measured tie behavior include:

1. The peak transient vertical displacement of LVDT #1 at Upland (60 ft.) is much smaller than Upland (15 ft.), i.e., 0.4 mm (0.015 in) versus 1.5 mm (0.06 in), which is evident by comparing the different vertical axes in Figures 2(a) and 2(b).



(a)



(b)

FIGURE 2 Passenger net transient LVDT vertical displacement behavior response at: (a) Upland (60 ft.) and (b) Upland (15 ft.) on 7 August 2012 in Chester, Pennsylvania.

2. At Upland (60 ft.), all five LVDT vertical displacements begin recording transient displacements at the same time while LVDTs #2 through #5 are delayed after LVDT #1 responds at Upland (15 ft.), which means it takes longer for the tie to transfer load to LVDTs #2 through #5 because of a tie-ballast gap.

3. While the transient vertical displacements in LVDT #1 are smooth at Upland (60 ft.), a more erratic response is observed in LVDT #1 at Upland (15 ft.), which includes a few "bumps" at a vertical displacement of 1 mm (0.04 in).

4. A significant amount of tie rebound is measured at Upland (15 ft.) due to rail bending while there appears to be little to no rebound or rail bending 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 greater displacement to establish contact with the ballast. This greater displacement delays the response of LVDTs #2 through #5 at Upland (15 ft.) because the poorly supported tie takes longer to contact the ballast and transfer load to the ballast. Once the tie establishes contact with the ballast, i.e., closes the tie-ballast gap, the underlying substructure experiences load and displacement, which is measured by LVDTs #2 through #5, to resist the applied wheel load. This implies the measured transient vertical displacements of LVDT #1 may not equate to the physical displacement of the ballast because LVDT #1 measurements includes both closure of a tie-ballast gap and the 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 2(b). This tie movement is manifested by the "dancing tie" behavior observed during field investigations and also reported by Coelho (*7*).

TIE-BALLAST INTERACTION

Field measurements of the track system show most vertical displacements observed at Upland (15 ft) and Upland (60 ft) occur in LVDT #1, which consists of the following three components: (1) closure of any gap between the tie bottom and ballast surface (δ_{gap}), (2) initial non-linear load-displacement behavior of the ballast (δ_{seat}), and (3) displacement of ballast layer to resist the applied loads (δ_{mob}). These three main components of LVDT #1 transient vertical displacement are shown in Figure 3.

The contribution of each component to the total LVDT #1 displacement can be determined by plotting the peak wheel load and corresponding LVDT #1 displacement in a load-displacement diagram (Figure 3). Because the behavior is limited by the range of peak wheel loads, it is desirable to measure trains with a wide range of wheel loads to define the non-linear and linear portions of the ballast load-displacement relationship, which is accomplished herein using data the NS field measurements.

Conceptual Tie-Ballast Model

To aid explanation of the field measurements and tie-ballast interaction, a conceptual model of the tie load-vertical displacement response under high-speed passenger and freight traffic is shown in Figure 3. Formation of a gap between the bottom of the tie and ballast is the key feature of the conceptual model and while a gap may not be detrimental or significant as shown below, it is usually present under most, if not all, ties after passage of a single train due to the initial loose nature of the ballast. For example, newly laid or recently tamped track should have loose ballast that is in intimate contact with the overlying tie as shown in the schematic in Figure 3 for no wheel passes. As the first train loads the track structure, the ballast particles rearrange into a more compact state (δ_{seat}) and displace under the applied load ($\delta_{mobilized}$). Because ballast particles are not elastic, the particles do not return to their initial position after train passage resulting in a gap below the tie as shown in schematic in Figure 3 for greater than or equal to one (1) wheel pass. In other words after the first train passes, the ballast does not elastically rebound to its initial position because of ballast particle rearrangement from the initial loading and the ballast being in a more compact state. After the train passes, the track is supported by ties that settle the least and the rail stiffness then pulls the other ties back up, i.e., tie rebound in Figure 2(b), creating a gap between the tie and ballast as shown in schematic in Figure 3 for greater than or equal to one (1) wheel pass.

The solid line in Figure 3 represents the theoretical tie load-displacement behavior with a gap between the tie bottom and ballast (δ_{gap}). As the tie is loaded, the gap closes and the ballast starts resisting the applied load by mobilizing shear resistance from ballast particle friction and interlocking. Tie displacement during the shear mobilization of the ballast is represented by δ_{seat} and the load to fully mobilize the ballast is defined as the tie seating load. Any tie displacement after seating ($\delta_{mobilized}$) is due to displacement of the ballast and underlying soils to resist the applied wheel load and the tie should displace linearly, based on field measurements to date, with increasing applied load in accordance with the mobilized stiffness (k_{mob}) of the ballast and underlying soils. Because the ballast stiffness is mobilized the corresponding tie displacement is referred to as the mobilized displacement or $\delta_{mobilized}$.

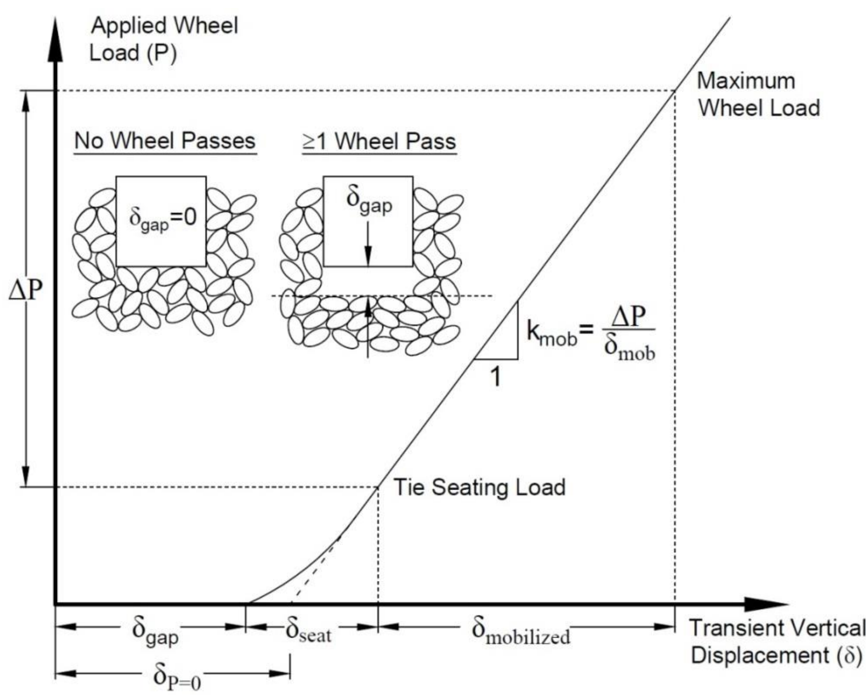


FIGURE 3 Transient vertical displacement behavior of a tie with a gap under applied wheel load.

Similar models using rail displacement (*9, 10*) instead of tie displacement have been proposed. These models show similar behavior, i.e., an initial non-linear region (δ_{seat} in the model proposed herein) followed by a linear region representing full mobilization of ballast stiffness. The main difference is the models by Lu et al. (*9*) and Sussmann et al. (*10*) focus on rail displacement because top-of-rail displacement is used to estimate track modulus, while the model proposed herein focuses on tie displacement because of tie LVDT measurements and the need to develop tie-ballast remedial measures. This means additional vertical displacements could occur between the rail and tie, which is not measured by LVDT #1. These additional vertical displacements are important components and are being measured at future sites but the goal of the proposed model is to isolate the tie-ballast interaction, which is used for numerical modeling of new transition designs and remedial measures.

High-Speed Passenger Behavior

To validate the conceptual model in Figure 3, peak wheel loads and corresponding peak transient vertical displacements from LVDT#1 were measured and plotted in the load-displacement diagram shown in Figure 4 for Upland (15 ft) and Upland (60 ft). This allows separation of the three components of transient vertical displacement shown in Figure 3, i.e., δ_{gap}, δ_{seat}, and $\delta_{mobilized}$, to be illustrated. Separating the tie-ballast gap (δ_{gap}) and seating displacement (δ_{seat}) components is challenging because measurements below the tie seating load were not obtained at the Upland Street bridge site because of the uniformity in load of passenger trains. To overcome this lack of data, the tie-ballast gap is estimated by extrapolating the linear portion, ballast stiffness relationship or k_{mob}, to the unloaded condition (P=0). This estimated “gap” is represented as $\delta_{P=0}$. The non-linear ballast displacement can then be calculated by subtracting the estimated tie-ballast gap ($\delta_{P=0}$) from the transient LVDT #1 displacement.

To determine the estimated tie-ballast gap ($\delta_{P=0}$), the peak wheel load and corresponding peak transient LVDT #1 vertical displacement is recorded for each passing wheel. The following linear mathematical relationship can be fitted to the field data to represent the transient LVDT #1 vertical displacement behavior:

$$\delta_{LVDT\#1} = \delta_{P=0} + \frac{P}{k_{mob}} \tag{1}$$

with $\delta_{LVDT\#1}$ representing the transient vertical displacement of LVDT #1, $\delta_{P=0}$ representing the estimated tie-ballast gap, P equaling the wheel load, and k_{mob} representing the mobilized ballast stiffness as shown in Figure 3. The tie-ballast gap and mobilized ballast stiffness parameters are useful when comparing the stiffness and tie-ballast gaps from different sites and are imperative for remedial measures and numerical modeling.

Figure 4 illustrates the difference in tie behavior for a good (Upland 60 ft.) and poorly supported (Upland 15 ft) tie. While the mobilized ballast stiffness (k_{mob}) is similar for both sites, i.e., about 850 kN/mm (4,850 kips/in), the tie-ballast gap ($\delta_{P=0}$) is much greater at Upland (15 ft.), i.e., 1.42 mm (0.056 in), than at Upland (60 ft.), i.e., 0.26 mm (0.01 in). The larger tie-ballast gap at Upland (15 ft) causes greater load distribution amongst adjacent ties (*16*) which causes overstressing of adjacent ties and an increase in their tie-ballast gaps. The presence of a tie-ballast gap can also amplify applied loads from the momentum of the moving tie impacting the ballast (*1*) using Newton's Second Law which states applied force (F) equals mass (m) times acceleration (a). It is anticipated the increased momentum and impact loads have resulted in greater measured permanent vertical displacements at the Upland (15 ft) location with 14.2 mm/yr (0.06 in/yr) than at Upland (60 ft) with 0.98 mm/yr (0.04 in/yr).

Accounting for the presence of a tie-ballast gap ($\delta_{P=0}$) also increases the calculated mobilized ballast stiffness during numerical analyses, which is important to develop agreement between the ballast stiffness values developed from the LVDT measurements and those obtained from field seismic wave testing. For example, if the stiffness was calculated assuming the tie load-deflection relationship started at the origin ($\delta_{P=0}$ = 0), the stiffness values and estimated modulus would be significantly less than k_{mob} obtained from the method presented above because the load is divided by the total displacement ($\delta_{P=0}$ and $\delta_{mobilized}$) instead of just the ballast displacement ($\delta_{mobilized}$). This usually results in erroneous values from inverse analyses of ballast moduli using field measured wheel load and transient vertical displacements with available software, e.g. GEOTRACK (3), that do not account for a tie-ballast gap.

Freight Train Behavior

LVDT #1 load-displacement relationships for the Amtrak high-speed passenger NEC sites show significant variations in tie support conditions but not wheel loads (Figure 4). Because the range in peak wheel loads is limited for the NEC sites, i.e., primarily only passenger trains cross the instrumentation sites, only the linear portion, i.e., k_{mob}, of the proposed conceptual tie displacement model (Figure 3) is defined because the loads exceed the seating load. This means the tie-ballast gap ($\delta_{P=0}$) could only be estimated by extrapolating the linear "best-fit" line to zero wheel load (P=0) which overestimates the actual tie-ballast gap (δ_{gap}). This interpretation was initially justified because it was anticipated that the non-linear seating displacement (δ_{seat}) is small due to the underlying ballast being compacted due to prior trains passing. However, the NS LVDT data indicates the non-linear seating displacement (δ_{seat}) can be significant as shown below.

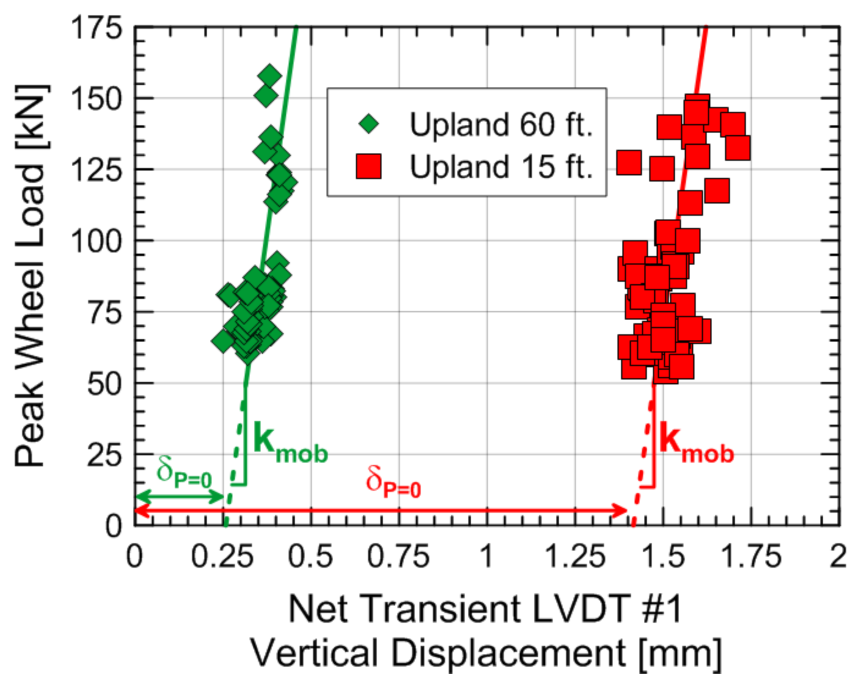


FIGURE 4 Comparison of transient LVDT #1 vertical displacement behavior at Upland (15 ft.) and Upland (60 ft.) on 26 January 2013 in Chester, Pennsylvania.

A benefit from instrumenting the NS N-Line near Bluefield, West Virginia is the wide range of recorded peak wheel loads because some cars were loaded and unloaded. This means the non-linear portion of the load-displacement relationship below the seating load could be discerned. To illustrate the non-linear load-vertical displacement behavior of LVDT #1, data from two different freight trains measured at MP 352.2 (13 ft.) and MP 352.2 (31 ft.) are compared in Figure 5. The measured train at MP 352.2 (13 ft.) consists of both loaded and unloaded cars while the measured train at MP 352.2 (31 ft.) only consists of loaded cars. Therefore, the non-linear region below the tie seating load of 80 kN (18 kips) is only captured at MP 352.2 (13 ft.) because of the wide range of measured wheel loads. MP 352.2 (13 ft.) and (31 ft.) exhibit similar behavior to Upland Street (15 and 60 ft.) at Amtrak (Figure 4) with the bridge transition zone location (MP 352.2 (13 ft.)) displaying poor tie support while good tie support is observed at the open track location (MP 352.2 (31 ft.)). The tie-ballast gap ($\delta_{P=0}$) at MP 352.2 (13 ft) is about 3.5 mm (0.138 in) and only about 0.5 mm (0.02 in) at MP 352.2 (31 ft) as shown in Figure 5.

With non-linear response below the seating load measured at MP 352.2 (13 ft.), the three displacement parameters in the conceptual model presented in Figure 3, i.e., δ_{gap}, δ_{seat}, and δ_{mob}, can be determined. The lowest recorded freight peak wheel load is 20 kN (4.5 kips) so the non-linear relationship still must be extrapolated from 20 kN (4.5 kips) to zero wheel load (P=0) but the curvature of the non-linear portion is sufficiently defined to minimize error (Figure 5). Two non-linear relationships, bi-linear and cubic, are used to approximate the non-linear NS data in Figure 6 for numerical analyses and future sites.

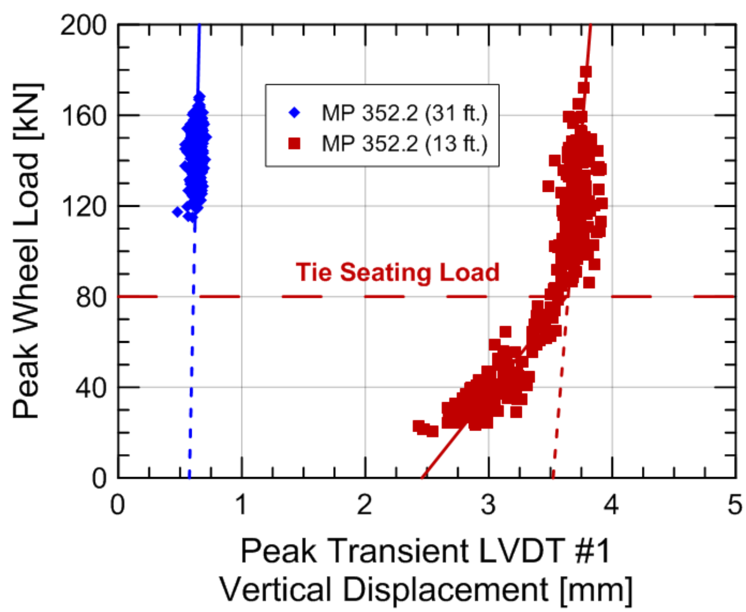


FIGURE 5 Comparison of transient LVDT #1 vertical displacement behavior about 10 m (31 ft) from MP 352.2 (31 ft) and 4.5 m (13 ft) from MP 352.2 (13 ft) bridges on NS freight track on 2 November 2013.

The equations for a bi-linear representation of the NS tie load-displacement behavior in Figure 6 are as follows for displacements above and below the tie seating load in Figure 3, i.e., P_{seat}:

$$\delta_{LVDT\#1} = \delta_{P=0} + \frac{P}{k_{mob}} \qquad if\ P \geq P_{seat} \tag{2}$$

$$\delta_{LVDT\#1} = \delta_{gap} + \frac{P}{k_{seat}} \qquad if\ P < P_{seat} \tag{3}$$

where the linear portion above the seating load is identical to the Amtrak data (Equation 1), i.e., k_{mob}, is ballast stiffness under the tie. Below the seating load, δ_{gap} represents the tie-ballast gap and k_{seat} represents the nonlinear stiffness of the ballast.

The equation for a cubic representation of the data in Figure 6 is as follows:

$$\delta_{LVDT\#1} = a_1P^3 + a_2P^2 + a_3P + a_4 \tag{4}$$

where a_1, a_2, a_3, and a_4 are best fit parameters. The last best fit parameter (a_4) represents the tie-ballast gap (δ_{gap}). The seating displacement, δ_{seat}, is calculated by subtracting the tie-ballast gap from LVDT #1 displacement at the seating load.

$$\delta_{seat} = \delta_{LVDT\#1} * (P_{seat}) - \delta_{gap} \tag{5}$$

Figure 6 shows that both models fit the data reasonably well within the range of measured peak wheel loads: 20 < P < 160 kN (4.5 < P < 36 kips). However, the cubic model provides a better representation at peak wheel loads less than 40 kN (9 kips) than the bi-linear model, which is in agreement with a previous study (*7*). The greater non-linearity of the cubic relation also results in a smaller estimate of δ_{gap} than the bi-linear model.

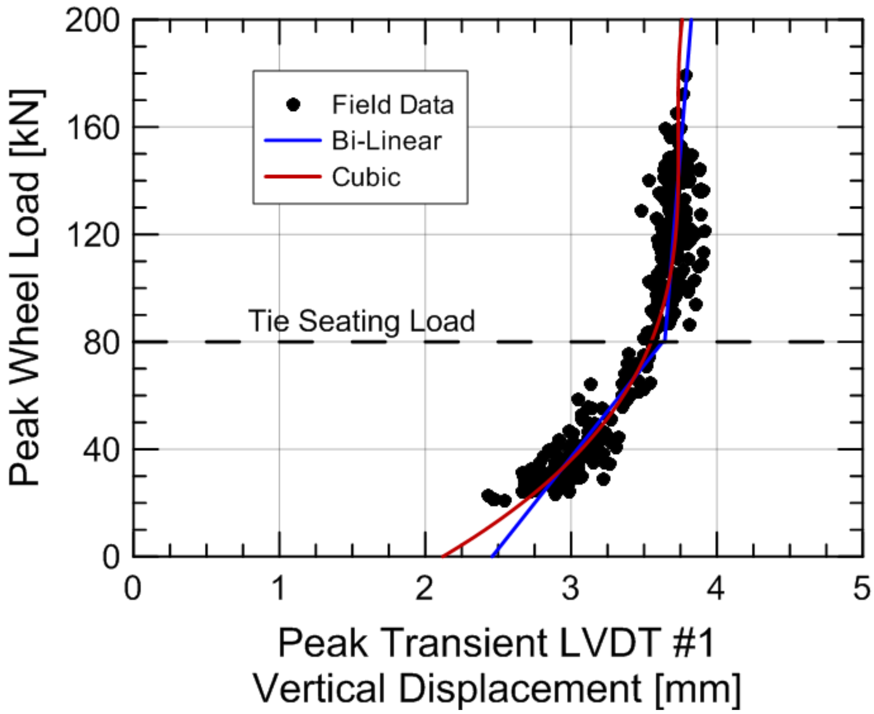


FIGURE 6 Mathematical representation of transient LVDT #1 vertical displacement behavior at MP 352.2 (13 ft) on 2 November 2013.

Table 1 displays the values of $\delta_{P=0}$, δ_{gap}, and δ_{seat} using the bi-linear and cubic models. The value of $\delta_{P=0}$ is included in Table 1 for comparison purposes and was determined using the field data and Equation (1). The range of seating displacement (δ_{seat}) of 1.2 to 1.5 mm (0.047 to 0.06 in) is significant because it means the estimated tie-ballast gap ($\delta_{P=0}$) is overestimated by 40% to 70% using $\delta_{P=0}$ because it does not account for δ_{seat}. This means that remedial measures have to fill a smaller void than indicated by $\delta_{P=0}$. In other words, if grout, tie pads, stone blowing, ballast pushing, or an overlift are used to raise the track by $\delta_{P=0}$, the track will likely be raised too high because δ_{seat} is a significant portion of $\delta_{P=0}$.

TABLE 1 Values of estimated tie-ballast gaps using field data from 4.5 m (13 ft) at MP 352.2 bridge on 2 November 2013

Fitting Model	$\delta_{P=0}$ [mm] (in)	δ_{gap} [mm] (in)	δ_{seat} [mm] (in)
Bi-Linear	3.5 (0.136)	2.4 (0.0949)	1.2 (0.0469)
Cubic	3.5 (0.136)	2.0 (0.0791)	1.5 (0.0598)

REMEDIAL MEASURES

The tie-ballast interaction model can be used to gain a better understanding of which potential remedial measures can reduce permanent vertical displacements at bridge approaches. One significant cause of the permanent vertical displacement is the impact loads resulting from the rapid change of wheel elevation as the train passes from the softer approach onto the stiffer and lower displacement bridge. This is amplified when accounting for the differential permanent vertical displacement between the approach and bridge (*5*). This differential permanent vertical displacement between the approach and bridge yields tie-ballast gaps because the rail, ballast, and subgrade in the approach undergo significant transient displacements while the rail on the abutment does not which results in impact and larger applied loads.

Where the tie-ballast gap is problematic, remedial measures should leave the underlying ballast in the approach compacted while adding material between the tie and ballast to keep the loaded top-of-rail (TOR) elevation, i.e. wheel elevation, smooth as the track passes from the open track to the approach fill and then the bridge abutment and deck. Therefore, assuming the displacement of the rail and fasteners on the approach and bridge deck are equal and the displacement of the bridge deck is negligible, a remediated track with no gap ($\delta_{gap} = 0$) should have a transient ballast displacement ($\delta_{seat} + \delta_{mob}$) resulting in equal loaded TOR elevations on the approach and bridge. In other words, the unloaded TOR elevation at the approach should be greater than the bridge by ($\delta_{gap} + \delta_{seat} + \delta_{mob}$) so the loaded TOR elevation in the approach and on the bridge deck is the same. This can be accomplished using a string-level to set the height of the overlift at ($\delta_{gap} + \delta_{seat} + \delta_{mob}$). If the subgrade is also undergoing transient displacement, the overlift should be increased to include the subgrade transient displacement.

For ties between the bridge approach and open track, the remedial measure should only eliminate δ_{gap} at the open track so the loaded TOR elevation is constant because the ballast at open track will still undergo transient displacement of $\delta_{seat} + \delta_{mob}$. Therefore, knowing the magnitude of $\delta_{gap,}$ δ_{seat}, and δ_{mob} is important to remove abrupt changes in the loaded TOR elevation in railway transitions during repair operation. Similar measures can be used in situations where rail-fastener gaps are problematic by including this gap in the repair process.

SUMMARY

This paper presents a conceptual model for applied tie load-vertical displacement for passenger and freight track that is validated with field measurements at multiple locations. The field measurements show existence of a gap between the tie bottom and ballast and an initial non-linear ballast seating behavior followed by a linear ballast load-displacement behavior. The tie-ballast gap is ubiquitous but the height varies considerably within a track section and between different sites. For the passenger and freight sites measured, transient gap heights range from 0.25 mm (0.01 in) to above 6.0 mm (0.24 in). Field measurements indicate tie-ballast gaps greater than 1 mm usually result in greater permanent vertical displacement, tie and ballast degradation, and track geometry problems (*3*, *21*). Numerical modeling shows gaps greater than 1 mm can result in load redistributing to adjacent ties (*16*). This implies that existence of large gaps (>1 mm) influences how load is distributed from the tie to ballast and implies that current tie seat loads of about 40% of the wheel load are not applicable for ties that have a tie-ballast gap.

NS freight traffic yielded a wide range of applied wheel loads which allowed definition of the non-linear portion of the tie load-vertical displacement relationship below the tie seating load. This non-linear behavior is important for calculating tie-ballast gaps because assuming a linear model ($\delta_{P=0}$) can over estimate tie-ballast gaps by 40 to 70%. Therefore, repair methods only need to fill the physical tie-ballast gap (δ_{gap}) and not the transient vertical displacements associated with the non-linear portion of the load-displacement relationship ($\delta_{P=0}$) outside of the transition zone. A cubic equation provides the best approximation of tie load-displacement behavior based on field measurements and can be used to predict the magnitude of vertical displacement, i.e., δ_{gap}, that has to be remediated at non-instrumented sites.

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