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Evaluating tie support at railway bridge transitions

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Abstract

This paper compares the behavior of three different railway bridge transition zones to illustrate how poor tie support affects track performance. The three bridge transitions consist of a high-speed passenger line, a freight line, and a spur track. All bridge transitions were instrumented with accelerometers that allow tie support and track performance to be non-invasively evaluated by analyzing the measured acceleration magnitudes and vibration frequencies in the frequency domain. The results show tracks with good tie support display tie accelerations below 5 *g* and small vertical displacements during train loading whereas approaches with poor tie support display accelerations generally greater than 5 *g*. These results are used to evaluate other transition zones and identify problematic track locations that require repair procedures to retain acceptable track geometry.

Keywords

Tie–ballast gap, poor support, unsupported, transition zones, accelerometers, tie vibration, cross-tie, ballast, railroad track-bed

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Introduction

Reoccurring track geometry problems, especially at transition zones, often require maintenance by railroads and highway departments.[1–5] Although advances in the measurement of track geometry with track geometry cars and vehicle/track interaction (V/TI) systems provide a quick and efficient method to identify track geometry problems, these technologies do not determine the underlying track structure problem(s) that caused the poor track geometry. This makes it difficult to select the appropriate remedial measures to address the track structure problem based on only geometry car and V/TI data.

Instrumentation of two bridge approaches on Amtrak's Northeast Corridor (NEC) near Chester, Pennsylvania[6–10] with linear variable differential transformers (LVDTs) showed a strong relationship between reoccurring differential track settlement and tie–ballast gaps at the instrumented locations.[8,9] The development of tie–ballast gaps at or near bridge approaches is attributed to the inherent problem of the approach track lying on deformable earthen materials whereas the bridge deck is essentially a non-deformable man-made structure. The passing train transiently displaces the approach substructure whereas the bridge deck remains essentially rigid, resulting in transient and permanent settlement of the substructure of the approach track, e.g. ballast. The subsequent permanent settlement of the ballast results in the rail and connected ties in the approach being cantilevered from the bridge deck after unloading. Once developed, tie–ballast gaps increase applied loads on the ballast from the momentum of the moving tie contacting the ballast, and redistribution of the load from poorly supported ties to better supported ties.[11] This increase in applied loading further increases the permanent vertical displacement of the ballast and substructure and creates a progressive degradation of the approach area.

Additional increased applied loads in the transition zone occur from rapid changes in wheel elevation as the front axle of a wheelset accelerates upwards when it hits the bridge abutment causing the back axle to accelerate downwards and this increases the dynamic wheel load in the transition zone.[5] This is important because increasing dynamic wheel loads from a train entering a bridge enlarge the "dip" or "bump" typically observed at bridge transition zones.[5]

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This paper investigates non-invasive measurement techniques, e.g. accelerometers, to monitor tie support and transient track performance at bridge transition zones. Three sites were selected for this study to compare the behavior of well- and poorly supported bridge transition zones for a wide range of track types. These include the following bridge approaches: a high-speed passenger bridge approach (Class 7 track), a freight site (Class 3 track) and a spur track (Class 1 track). Instrumenting and comparing both well- and poorly supported bridge approaches resulted in the development of a criterion for how railway bridge transition zones ideally should perform and allow evaluation of other track transitions. The desired outcome of this and future investigations is to improve the understanding of the movements and forces generated in poorly supported track and identify effective track design and remedial measures. These non-invasive measuring techniques can also be used in the evaluation of other track structure defects, e.g. fouled ballast, etc., but these applications are outside the scope of this paper.

Instrumentation

Strain Gages and LVDT Strings

In a previous project to investigate the root cause of differential movement at bridge transitions, an instrumentation system consisting of strain gages and LVDT strings, i.e. five LVDTs embedded at various substructure depths, were installed at six high-speed passenger bridge transitions in the USA.[6,7] The strain gages were installed at 45° along the neutral axis of the rail to measure wheel loads and the LVDT strings measured the relative displacement of various substructure layers at different depths. The LVDT strings were installed near the edge of the instrumented tie with the top LVDT attached to the top of the concrete tie and the other LVDTs embedded at various depths directly below the tie. An example LVDT string is shown in Figure 1 and full details of the strain gage and LVDT instrumentation is described in other papers.[6,7] This instrumentation system is highly effective in observing the permanent and transient behavior of multiple substructure layers, however, they are expensive, require track fouling, are time-consuming to install (~1 month installation), and are invasive to the rail and tie.

Tie Accelerometers

Analysis of the data obtained using the LVDT instrumentation showed that the majority of the observed permanent vertical displacement occurred within the ballast layer for all six sites and that there is a strong relationship between the magnitude of the vertical displacement of the permanent ballast and the height or magnitude of the tie–ballast gap.[8,9] This suggests that

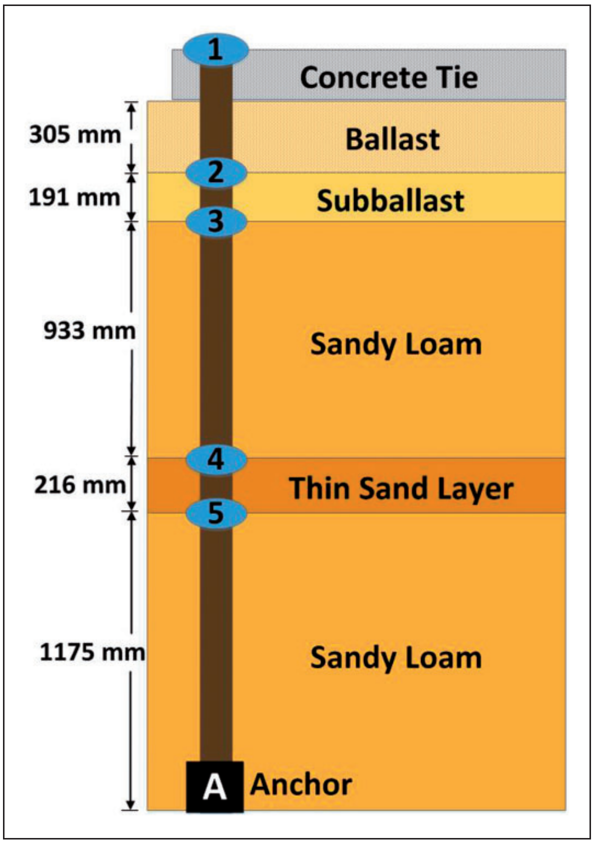

Figure 1. Subsurface profile and LVDT locations 4.6 m (15 ft) north of Upland Street Bridge in Chester, Pennsylvania.

poor tie support is an indicator of reoccurring permanent vertical differential displacements;[8,9] thus, alternative non-invasive methods were sought to evaluate the effects of poor tie support on bridge transition behavior, in order to understand how track behaves once this progressive degradation process has begun. After considering a wide variety of instrumentation techniques, accelerometers were selected for data collection and track assessment as they provide an inexpensive, easy, non-invasive, durable, and reusable means to evaluate tie and track behavior by measuring tie acceleration time histories. The accelerometers are only 13 mm long (half an inch), weigh less than 3 g (0.1 ounces), and are connected to a concrete or timber tie with a drop of superglue. This results in a quick and non-invasive monitoring system that does not interfere with train operations, which makes accelerometers suitable for short-term monitoring, i.e. a single train pass or day, as well as long-term monitoring during wet and inclement weather conditions since weather-resistant accelerometers also are available.

Acceleration time histories are beneficial because they provide insight to the dynamic tie movements or what can be considered the "heartbeat" of the track. Tie accelerations can be produced from at least seven factors:

- wheel–rail impacts;
- wheel–rail vibrations such as braking;

- rail–tie impacts;
- tie loading;
- track and tie vibrations;
- tie–ballast impact; and
- tie displacement due to train loading.

Each factor tends to have its own unique signature and can typically be identified by analyzing the measured tie accelerations in both the time and frequency domains. Well-supported track will typically display tie accelerations from only tie loading but can also show wheel–rail impacts and wheel–rail vibrations because those tie accelerations are associated with the train vehicle. The remaining four factors are typically indicative of poorly supported track, i.e. rail–tie impact, track and tie vibrations, tie–ballast impact, and tie displacement as they directly relate to track support. As poor tie support is the focus of this paper, the factors: track and tie vibrations, tie–ballast impact, and tie displacement due to train loading, are emphasized in this paper as well as wheel–rail impacts, which usually occurs because of transient or permanent displacements in the approach.

The transfer of load from the rail to the tie produces the most basic tie acceleration signature and is noticeable at well-supported track. The acceleration signature usually involves the gradual increase in tie acceleration until a maximum value is obtained followed by a gradual decrease in tie acceleration as the wheel or wheelset passes. Maximum accelerations range typically from 1 to 5 g for well-supported track; however, it can be much higher for poorly supported track (10 g to over 100 g). The dominant frequencies of the tie typically range from 50 to 300 Hz, but are sometimes difficult to isolate because of the multiple sources of tie acceleration and coupling of vibration modes.

Similar to tie loading, a second common source of tie acceleration is tie and track vibration. All deformable materials exhibit unique bending/vibration modes and multiple laboratory investigations have identified the first three vibration modes for concrete ties to be about 100–150 Hz, 330 Hz and 630 Hz.[12–15] These distinct vibration modes can be detected by monitoring the resulting tie vibration of an isolated unsupported tie after each wheel loading.[16] This behavior is similar to how a bell "rings" after being struck, it is due to unsupported ties not being damped and constrained by the underlying and nearby ballast. If a group of ties are unsupported, and additionally the track is vibrating, these vibration modes become less distinct due to additional vibration of the track. The authors have not measured and are unaware of distinct vibration modes for timber ties.

In addition to tie loading and vibration, tie accelerations from an unsupported moving tie contacting the ballast can amplify the force being applied to the ballast, this is a result of Newton's Second Law that states applied force (F) equals the mass (m) times acceleration (a). These impacts often involve sharp peaks in tie accelerations at varying frequencies (~50–300 Hz) depending on train speed and track compliance and are indicative of poorly supported track.

Accelerations from tie displacement are the last factor to be investigated in this paper and typically involve low accelerations at low frequencies. Low tie accelerations are produced because tie displacement involves frequencies within the 0 to 15 Hz range. As acceleration and displacement have a second-order polynomial relationship with respect to time (acceleration $\approx$ displacement/(time squared)), this means that the time required to displace a tie even large distances (~50 mm) is long enough to keep the magnitudes of the tie accelerations low compared with the sudden and large accelerations produced from impacts and low-displacement vibrations. Therefore, these signatures are difficult to analyze unless double-integration techniques of the time history are used.

Several external factors affect tie acceleration signatures, including but not limited to: train weight, speed, bouncing, and possibly wheelset dimensions, tie spacing and type, and track curvature. For well-supported track, where load transfer from the wheel to substructure is smooth, these external factors do not seem to have a significant influence, due to the low magnitudes of the tie displacement. Based on several sites investigated by the authors (three are presented in this paper), well-supported track consistently produces tie accelerations from tie loading at or below 5 g for a variety of train weights, lengths and speed.

However, external factors are expected to significantly influence the accelerations for poorly supported track, due to the larger tie movement. For example, a higher train weight or speed will either displace the tie at a quicker rate or apply greater force, which results in higher tie accelerations. Different types of wheelsets may also affect how the tie is loaded and therefore its acceleration magnitudes. For this reason, accelerometers are considered suitable for qualitative measurements of tie support and can give additional insight into how these external factors affect track movement and loading. In future site investigations, high-speed video cameras will be included to measure rail and tie displacement for a more quantitative analysis.

Bridge Location and Instrumentation

Site 1: Poorly Supported High-Speed Passenger Bridge

The first instrumented bridge transition zone is a NEC high-speed passenger open deck timber bridge over Upland Street near Chester, Pennsylvania. The bridge transition is a straight, elevated approach consisting of concrete ties with timber ties on the bridge deck. The approach is confined by a large gravity wall along one side of the track and abutment. Primarily, Acela high-speed passenger trains pass over the transition zone with a velocity of up to 177 km/h (110 mph).

The Upland Street site was initially instrumented with strain gages and LVDT strings and later instrumented with eight accelerometers on 1 July 2014 to non-invasively evaluate tie support and compare with the strain gage and LVDT equipment already installed at the bridge transition.

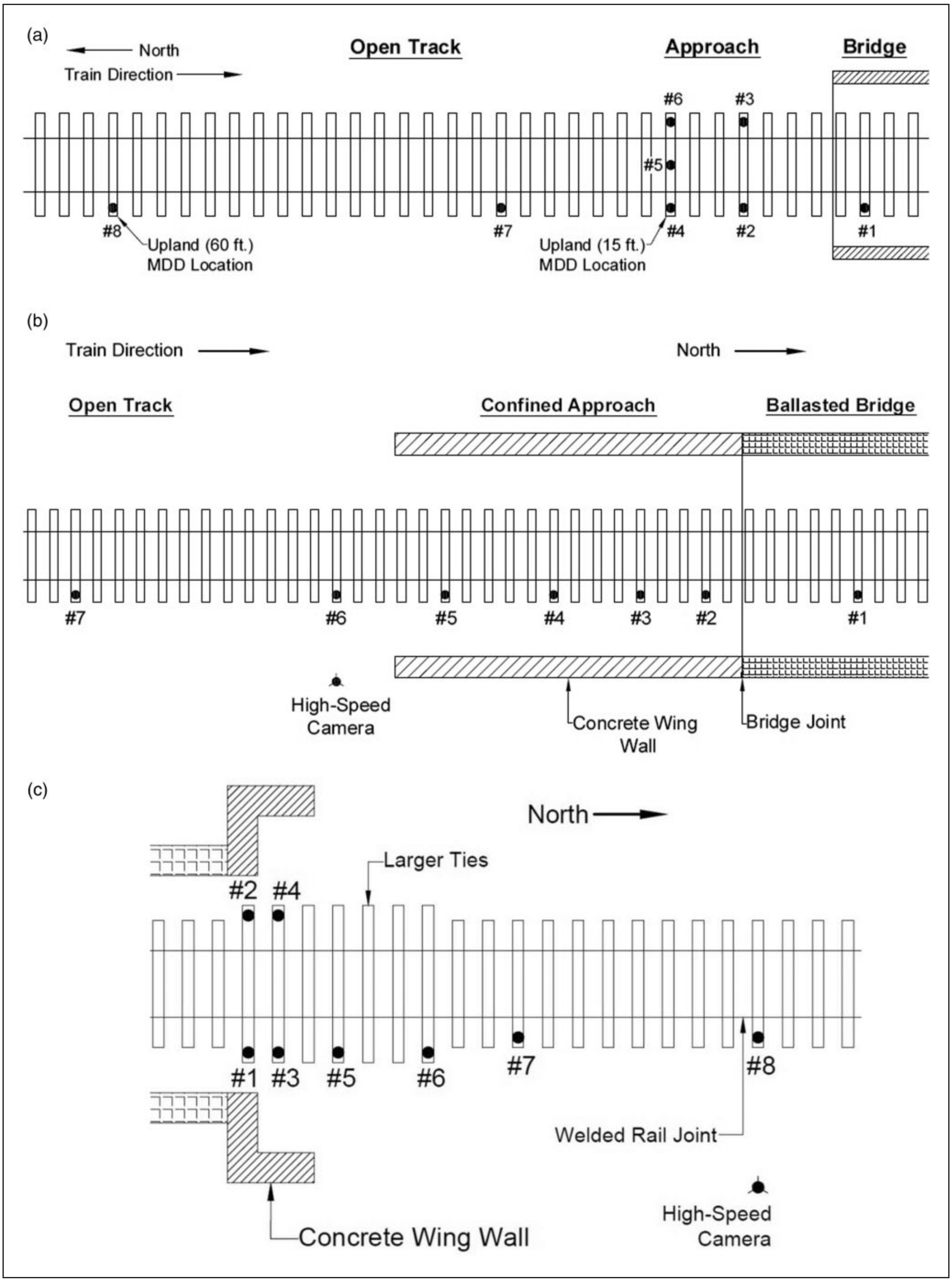

Figure 2. Instrumentation locations of: (a) accelerometers and LVDTs at Site 1 (Upland Street Bridge approach) near Chester, Pennsylvania; (b) accelerometers at Site 2; and (c) accelerometers at Site 3.

The instrumentation layout for the Upland Street Bridge is shown in Figure 2(a). This paper only analyzes Accelerometers 4 and 8, which were installed at the two LVDT string locations. These locations were chosen to compare LVDT and accelerometer results by recording the same passing train with both instruments. The transition zone LVDT and Accelerometer 4 were located 4.57 m (15 ft) from the bridge abutment, whereas the open track LVDT and Accelerometer 8 were located 18.2 m (60 ft) from the bridge abutment. These sites are referred here onwards as Upland (15 ft) and Upland (60 ft).

Site 2: Well-Supported Freight Bridge

The second instrumented bridge transition is on a freight line consisting of a ballasted concrete deck bridge, timber ties, a 150 mm (6 inch) hot-mixed-asphalt (HMA) layer underneath a 300 mm (12 inch) thick layer of ballast in the approach, and concrete wing walls perpendicular to the bridge abutment and extending approximately 16 ties (8.2 m/27 ft) from the abutment. These features are important because they improve ballast confinement, reduce tie support problems, and keep the track tight, i.e. small transient displacements, which limit the differential transient and permanent displacements between the bridge and approach. Specifically, the ballasted bridge deck reduces the stiffness or load-displacement difference between the approach and bridge deck, the HMA supports or stabilizes the ballast layer[17], and the concrete wing walls add confinement to the subgrade layers. Both empty and loaded freight trains pass over the bridge moving at approximately 40 km/h (25 mph) and the track is considered Class 3 for operations. During train passage, the ties did not visually move and no track geometry problems have arisen since the bridge was placed in service in 2009 or over about 5 years.

The site was instrumented with seven accelerometers on 12 June 2014 and Figure 2(b) shows the layout of the accelerometers at this site. Figure 3 shows a photograph of Accelerometer 3 installed on a timber tie in the bridge approach. Due to space constraints, only the results of Accelerometers 3 and 7 are presented to display the difference between the bridge transition and open track for a well-supported transition site. Accelerometer 3 is located 2.1 m (7 ft) from the bridge abutment and Accelerometer 7 is located 15.4 m (51 ft) from the bridge abutment. These two sites are referred to as Site 2 (7 ft) and Site 2 (51 ft) here onwards.

Figure 3. Photograph of Accelerometer 3 installed on a timber tie at Site 2.

Site 3: Poorly Supported Freight Bridge

The third instrumented bridge transition zone is on spur track consisting of an open deck timber bridge, timber ties, and short concrete confining walls perpendicular to the bridge deck. The concrete walls extend only two ties from the bridge abutment instead of 16 ties as at Site 2. Freight trains pass over the terminal bridge at a maximum 10 mph (Class 1 track), however, permanent displacement has occurred over time with a noticeable "dip" in the bridge entrance and exit, which has required frequent remediation.

Eight accelerometers were installed at Site 3 on 29 July 2014 and the accelerometer layout is shown in Figure 2(c). Only the results of Accelerometers 5 and 6 are discussed here onwards to illustrate the behavior difference between poor and good tie support at the same site. Accelerometers 7 and 8 were not used because they were installed on a split tie and near a welded rail joint, respectively, which increased the acceleration response and are not considered representative of the good tie support. Accelerometer 5 is located 2.1 m (7.0 ft) from the bridge abutment and Accelerometer 6 is located 3.8 m (12 ft) from the bridge abutment so these two sites are referred to as Site 3 (7 ft) and Site 3 (12 ft) here onwards.

Behavior of Site 1: Poorly Supported High-Speed Passenger Bridge Transition

The Upland Street Bridge along Amtrak's NEC has experienced reoccurring track geometry problems that continued during the strain gage and LVDT monitoring period.[6] During the 17-month monitoring period of the two LVDT sites, the majority of permanent vertical displacements were located within the ballast layer (LVDT 1) and the average rate of permanent vertical ballast displacement at Upland (15 ft) was 15 mm per year whereas only 0.8 mm per year was measured at Upland (60 ft).[9] This verifies prior observations by track geometry cars of differences in permanent vertical displacement between the transition zone and open track.[6] This also resulted in Amtrak tamping the transition zone 8 months into the monitoring period.

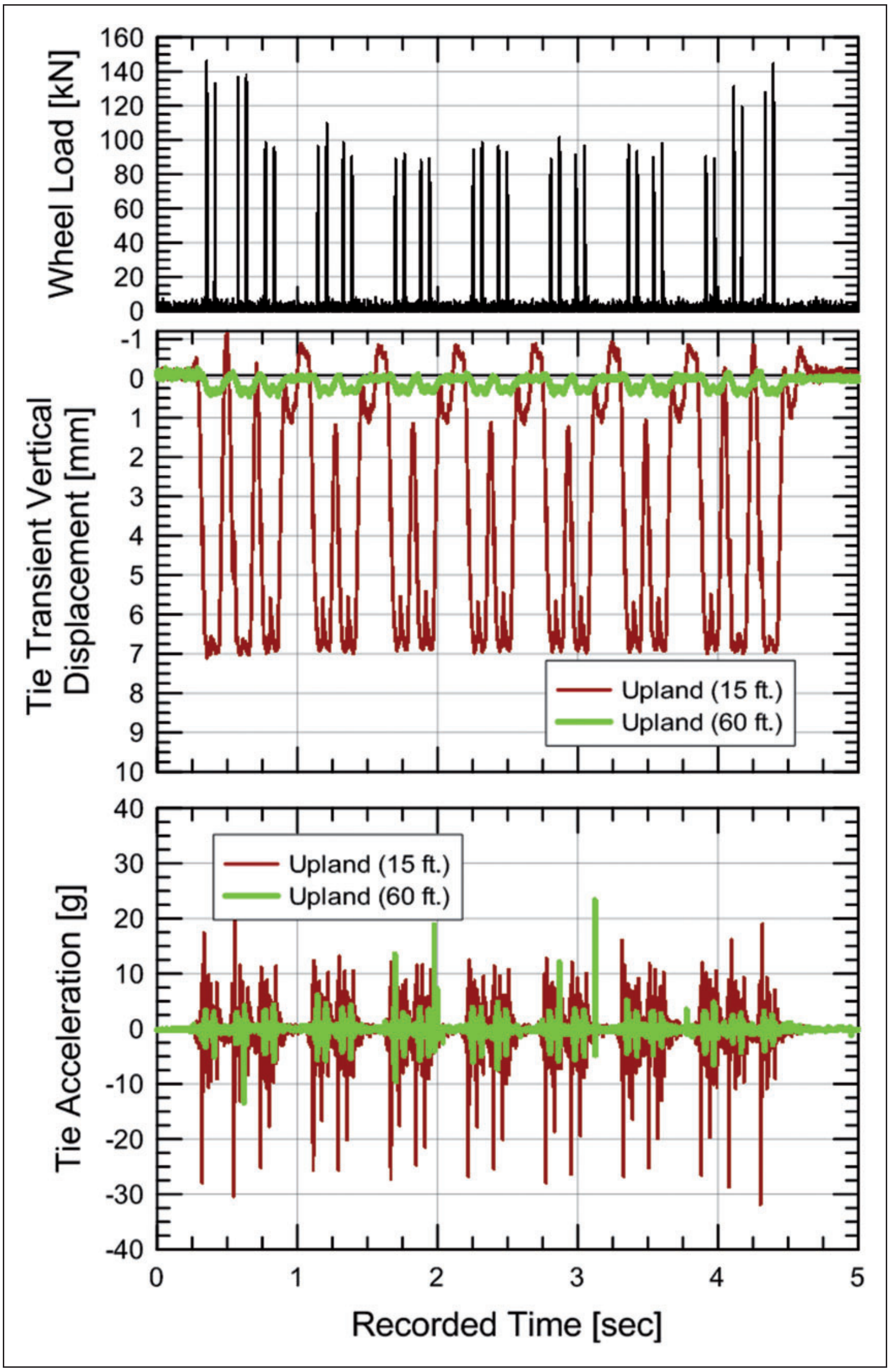

Figure 4. Recorded time histories of the wheel load, tie transient vertical displacements, and tie accelerations at Upland (15 ft) and Upland (60 ft) on 1 July 2014.

Ballast/LVDT 1 Response

To investigate the causes of greater permanent vertical displacements at Upland (15 ft) than Upland (60 ft), the transient track response was analyzed. Transient behavior is important because the negative effects of each passing train can accumulate into noticeable permanent track geometry problems. Therefore, identifying and remediating problems within the transient timescale can prevent long-term track structure and track geometry problems.

The most apparent transient behavior difference between the two Upland Street sites is the vertical displacement magnitudes within LVDT 1. This LVDT string measures the transient vertical

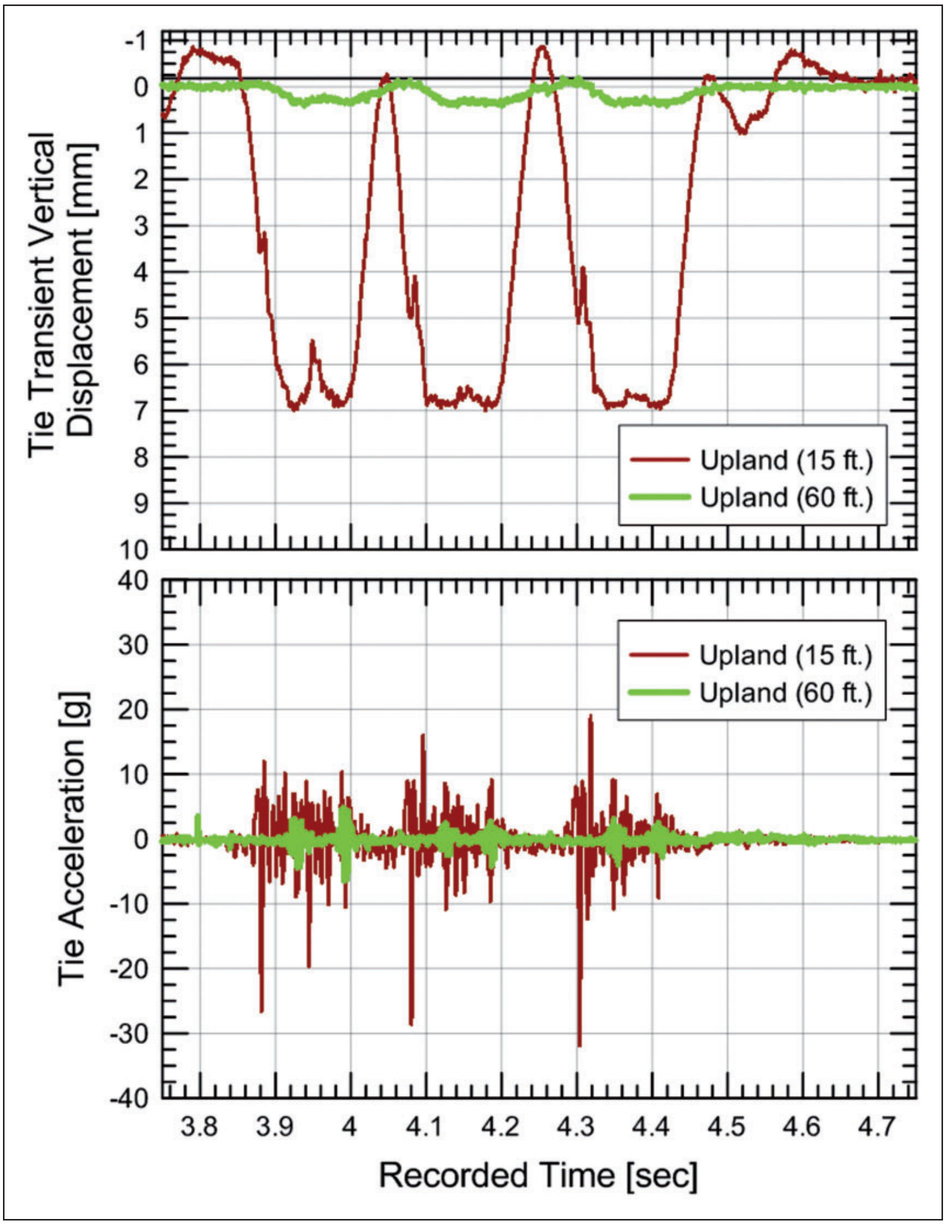

Figure 5. Tie transient vertical displacement and tie acceleration response from a passing Acela Power Car at Upland (15 ft) and Upland (60 ft) on 1 July 2014.

displacements from the top of the concrete tie to the bottom of the ballast layer (0.3 m in depth). This means that the LVDT 1 measurements include both the closure of the tie–ballast gap and the displacement of the ballast underneath the tie. For simplicity, this paper will here onwards reference the LVDT 1 displacement as the vertical tie displacement because LVDT 1 is fixed to the tie so the displacements measured by LVDT 1 correspond to tie movement.

Figure 4 displays the wheel load, tie transient vertical displacement, and tie acceleration time histories of Upland (15 ft) and Upland (60 ft) resulting from the same Amtrak passenger train. The Upland (60 ft) time histories were shifted so that the peak wheel loads matched the Upland (15 ft) time history. Upland (15 ft) shows peak vertical tie displacements of about 7.0 mm whereas Upland (60 ft) displays peak tie displacements of only about 0.4 mm. The significantly larger peak vertical tie displacement (17 times larger) at Upland (15 ft) suggests a tie–ballast gap is present at that location, as this difference in tie displacement magnitude cannot be explained by variation of ballast stiffness. The tie accelerations are discussed in the next section.

Figure 5 shows a more detailed view of the last three wheelsets of the passenger train (3.75 to 4.75 s). At about 3.38, 4.08 and 4.30 s, a sharp change in tie displacement occurs and this is attributed to the tie contacting the ballast, this is based on similar behavior being observed at poorly supported ties in the Netherlands.[18] Additionally, the Upland

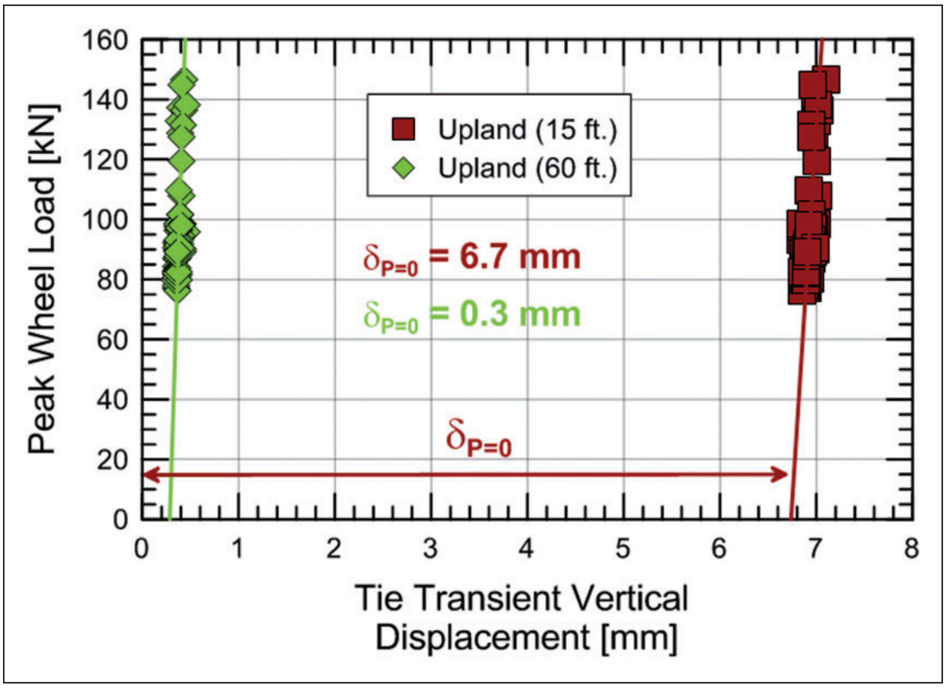

Figure 6. Tie load–displacement behavior at Upland (15 ft) and Upland (60 ft) on 1 July 2014.

(15 ft) tie rebounds (positive vertical displacement) after the passing wheel set. These indicate the tie at Upland (15 ft) is poorly supported and this behavior is usually manifested by a "dancing tie."

To estimate the tie–ballast gap height, the peak wheel load and tie displacement of each wheel was used to develop a load–displacement diagram for Upland (15 ft) and Upland (60 ft). To mathematically describe the load–displacement behavior, the following two parameters were incorporated in the load–displacement model presented by Wilk et al.:[8]

- mobilized stiffness of the ballast (k_{mob});
- the tie–ballast gap ($\delta_{P=0}$)

$$\delta_{LVDT\_1}(P) = \delta_{P=0} + \frac{P}{k_{mob}} \tag{1}$$

where P is the wheel load. Figure 6 shows the load–displacement behavior of the concrete tie at both Upland (15 ft) and (60 ft) locations for the same train on 1 July 2014. The mobilized stiffness of the ballast (k_{mob}) is the slope of the load–displacement trend lines and is about the same for both locations. However, Upland (15 ft) shows a larger estimated tie–ballast gap ($\delta_{P=0}$). The tie–ballast gap is estimated by extrapolating the ballast stiffness to the zero load condition ($P=0$).[8]

The load–displacement measurements in Figure 6 show a significant difference in the estimated tie–ballast gap ($\delta_{P=0}$) with values of only 0.29 mm at Upland (60 ft) and 6.74 mm at Upland (15 ft). In reality, the tie–ballast interaction displays nonlinear behavior below the seating load,[8,19,20] i.e. load at which the ballast becomes fully mobilized and displays linear behavior, so the actual tie–ballast gap (δ_{gap}) at Upland (15 ft) is expected to be smaller at about 5 mm, as in agreement with Figures 4 and 5. The ballast particles likely rearrange after each loading so the actual tie–ballast gap (δ_{gap}) will vary after each wheel pass.

Accelerometer Response

The acceleration time histories from the same Amtrak passenger train as presented in the previous section is displayed in Figures 4 and 5. To eliminate high-frequency movement, the time histories were passed through a low-band Butterworth filter at 500 Hz. As with the LVDT displacements, a significant difference in tie acceleration response is observed. Upland (15 ft) displays much greater consistent peak accelerations of about 30 g whereas Upland (60 ft) shows consistent peak accelerations of less than or equal to 5g.

The large downward accelerations (~30 g) of Upland (15 ft) typically appear directly before the passing of each wheel. By comparing the transient vertical displacement of the tie and its acceleration time histories in Figure 5, the acceleration peaks for the tie occur at recorded times of 3.38, 3.95, 4.08 and 4.30 s, which correspond to the sharp change in displacement observed when the tie is suspected of contacting the ballast. The only exception is the second wheel of the Acela power car wheelsets (wheel loads of about 140 kN), in which the tie remains in full mobilized contact with the ballast (see Figure 5). This suggests the large downward accelerations (~30 g) are

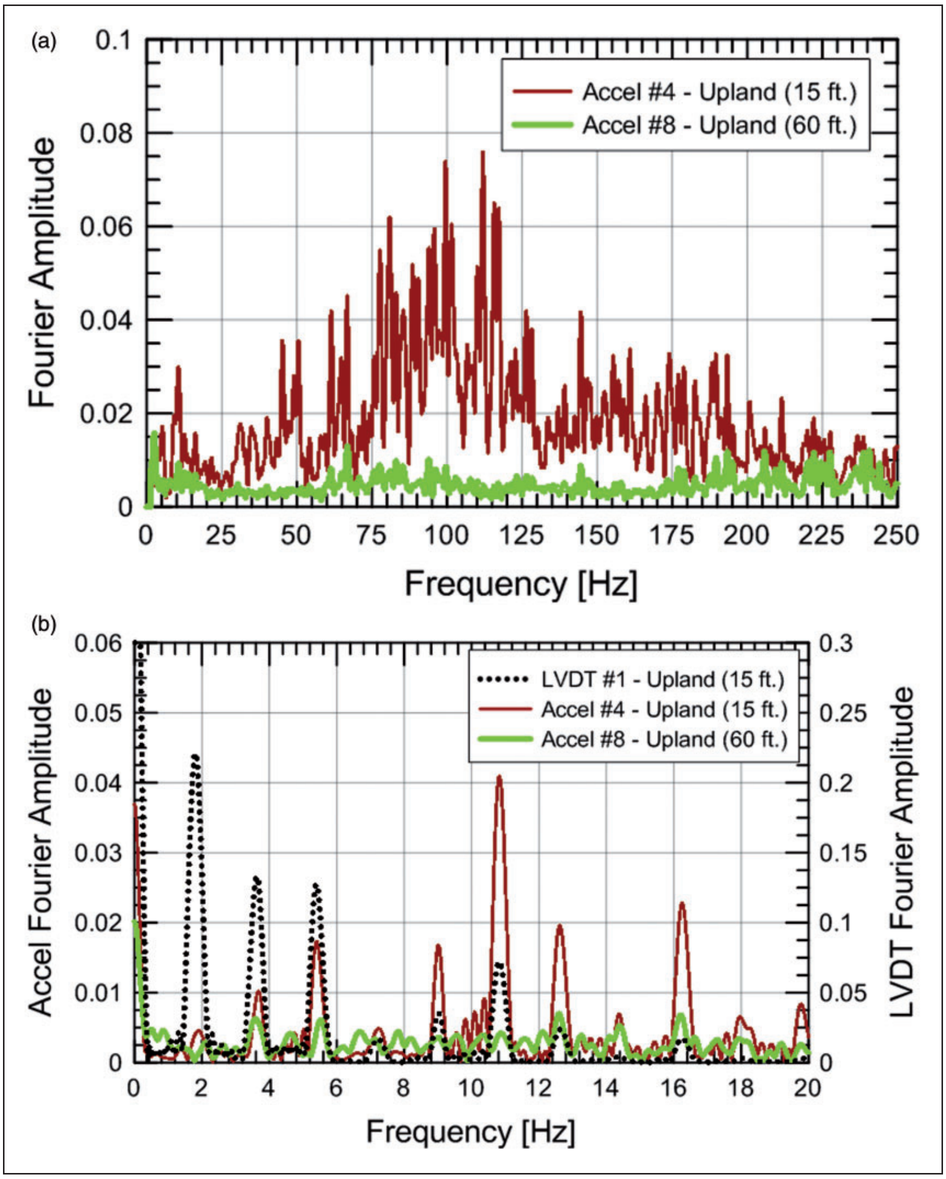

Figure 7. (a) Measured tie acceleration time histories for Upland (15 ft) and Upland (60 ft) and (b) recorded LVDT 1 and measured tie acceleration time histories for Upland (15 ft) and Upland (60 ft) in Figure 4 converted to the frequency domain for a passing train on 1 July 2014.

Table 1. Values of the permanent vertical displacement rates, peak transient displacement, estimated tie–ballast gap values ($\delta_{P=0}$) and peak accelerations for Upland (15 ft.) and Upland (60 ft).

Site location	Rate of permanent vertical displacement (mm/year)	Peak transient displacement (mm)	$\delta_{P=0}$ (mm)	Peak tie acceleration (g)
Upland (15 ft)	15	7.0	6.7	30
Upland (60 ft)	0.8	0.4	0.3	~5

produced from the tie establishing contact with the ballast and possibly amplifying the tie–ballast load from impact, a consequence of Newton's Second Law. The remaining movement (~10 g) is likely due to load transfer and vibrations of the track and tie. The tie accelerations at Upland (60 ft) show distinct responses (~5 g) from the loading of each passing wheel. A few isolated high-frequency peaks can be observed in Figure 5(a) and these are likely from wheel or data anomalies.

Figure 7(a) and (b) compares the acceleration time histories in Figure 4 in the frequency domain using

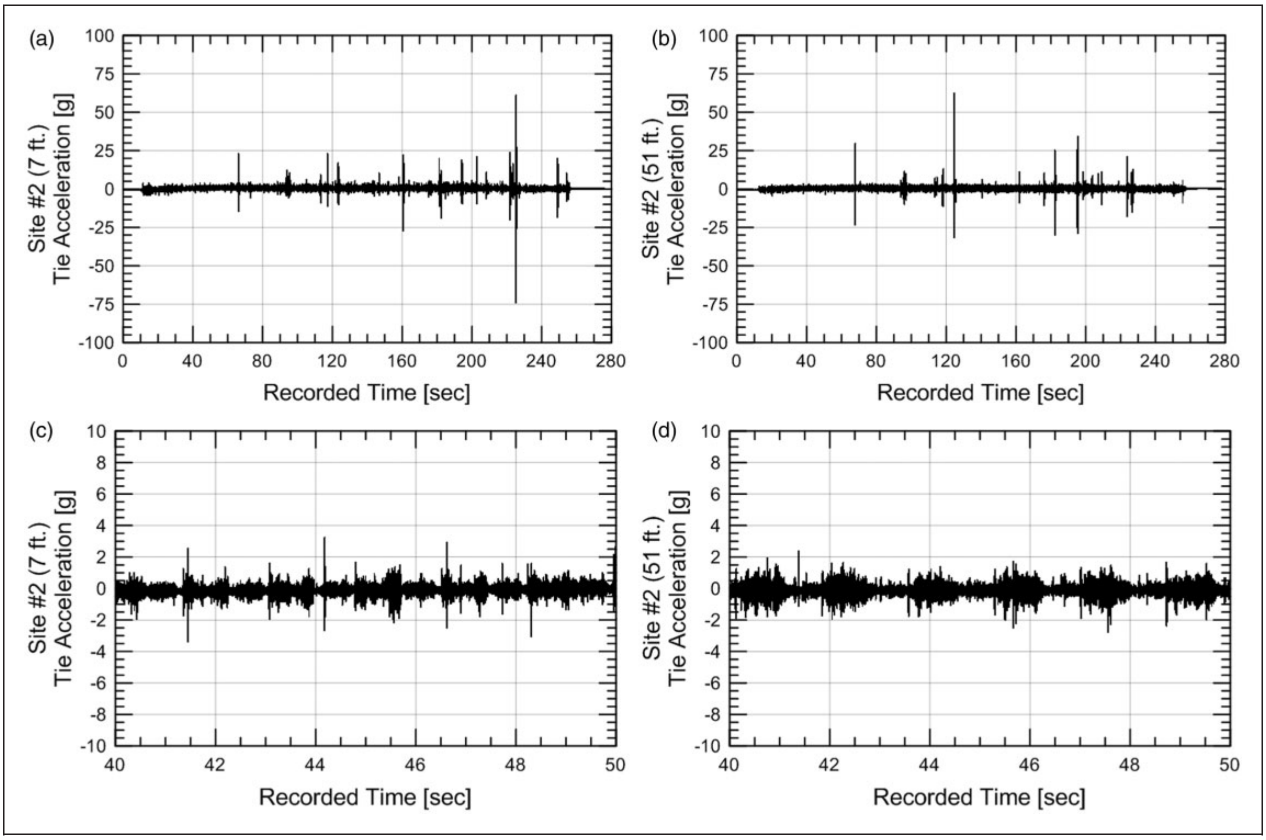

Figure 8. Measured tie acceleration time histories for Site 2 (7 ft) and Site 2 (51 ft) for: (a) and (b) the entire passing unloaded freight train; and (c) and (d) 10 s of the passing unloaded freight train on 12 June 2014.

fast Fourier transform techniques. Figure 7(a) shows a range from 0 to 250 Hz that emphasizes the larger Fourier amplitudes from train loading and track and tie vibrations at Upland (15 ft). Sites with poor tie support will experience greater magnitudes of the tie acceleration and offer less damping and resistance to tie and track vibrations than well-supported sites, and they will display larger Fourier amplitudes within that frequency range.

Figure 7(b) shows a frequency range from 0 to 20 Hz to emphasize the influence from tie displacement and compares the tie's displacement response measured by LVDT 1 at Upland (15 ft), the tie's accelerometer response at Upland (15 ft), and the tie's acceleration response at Upland (60 ft). Three dominant frequencies appear in the LVDT 1 response: 1.8, 3.6 and 5.4 Hz. These frequencies roughly associate with unloading between the wheelsets, loading from a wheelset, and loading from an individual wheel. The Upland (15 ft) tie accelerations show dominant frequencies at identical frequencies, however, it has its largest value at 5.4 Hz, which implies a double-integrated displacement from wheel loading could be obtained. The Upland (60 ft) tie acceleration shows little response, which agrees with the low measured displacement values.

Table 1 summarizes the permanent vertical displacement rate, peak transient displacements, estimated tie–ballast gap ($\delta_{P=0}$), and peak tie acceleration values for Upland (15 ft) and Upland (60 ft). These results show significant differences in track behavior between the transition zone and open track due to the presence of tie–ballast gaps within the transition zones. This poor tie support, identified by both LVDTs and accelerometers by larger tie displacements and tie accelerations, can lead to additional permanent vertical displacements from impact loading and load redistribution. The low consistent peak accelerations ($<5\,g$) at Upland (60 ft) is indicative of well-supported track with smooth load transfer because of a small tie–ballast gap.

Behavior of Site 2: Well-Supported Freight Bridge Transition

In contrast with Site 1, Site 2 compares the transition zone and open track behavior of a well-supported freight bridge transition. This bridge transition handles freight trains with velocities of about 40 km/h (25 mph) and has not required track geometry maintenance since being placed in service in 2009. This site provides insight into the ideal transition zone response.

Figures 8 and 9 compare the transition zone (Site 2, 7 ft) and open track (Site 2, 51 ft) tie acceleration responses from passing unloaded and loaded freight

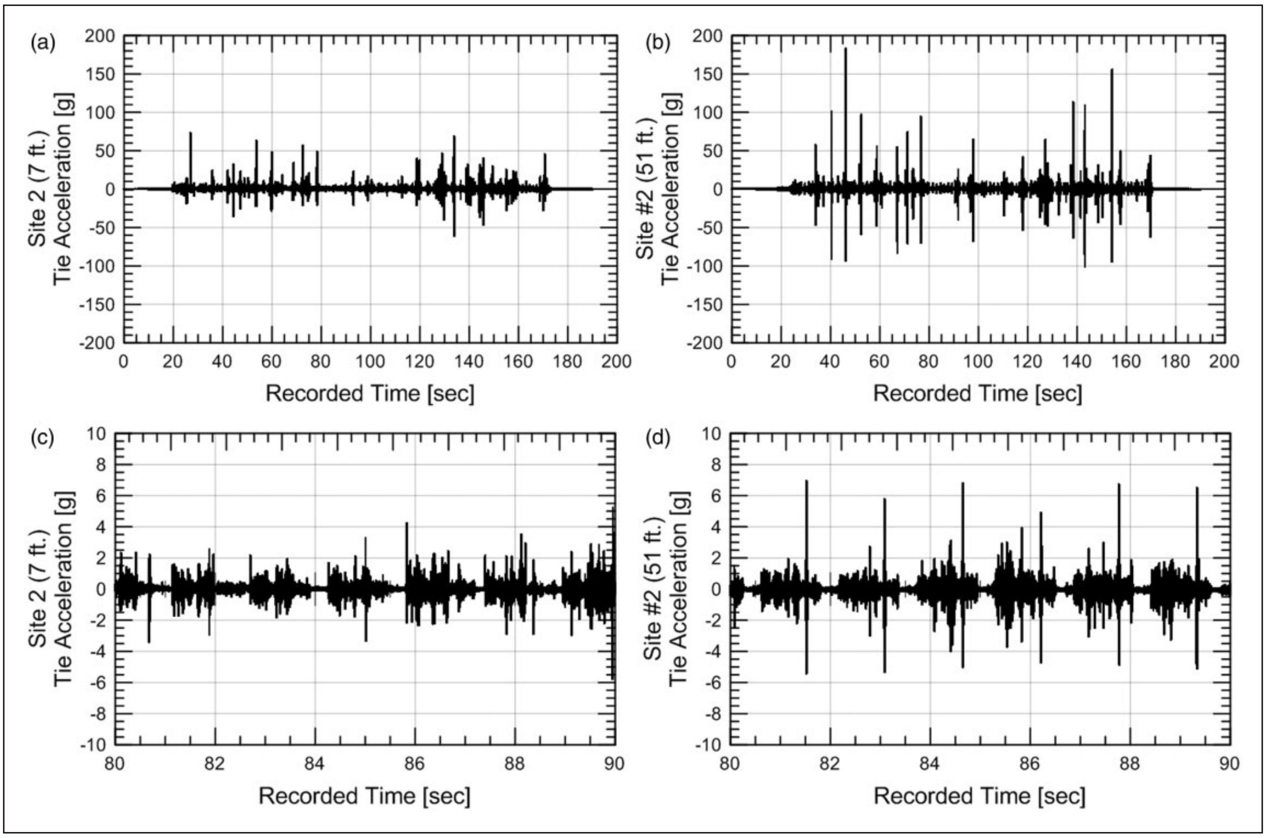

Figure 9. Measured tie acceleration time histories for Site 2 (7 ft) and Site 2 (51 ft) for: (a) and (b) the entire passing loaded freight train; and (c) and (d) 10 s of the passing loaded freight train on 12 June 2014.

trains on 12 June 2014, respectively. Figure 8(a) and (b) displays the entire recorded time history with tie acceleration responses typically below 1 g to 2 g apart from a few large spikes that reach magnitudes of about 75 g. These spikes are attributed to wheel–rail impacts, such as wheel flats or other wheel defects; due to their random and inconsistent nature they are not considered within the track structure analysis because they are train vehicle issues. Figure 8(c) and (d) display 10 s of the time history without any wheel–rail defects and the tie acceleration response from the loading of each passing wheelset is clearly illustrated by a gradual increase then decrease of tie acceleration, with peak values typically ranging from only 1 g to 2 g.

The entire tie acceleration time histories of a loaded freight train in Figure 9(a) and (b) show significantly more wheel flats or defects with tie acceleration spikes reaching 190 g. The existence of these wheel flats was audibly verified by a loud rhythmic "clacking" as the flat repeatedly contacted the rail. Emphasizing a section of the recorded time history that does not include wheel flats (Figure 9(c) and 9(d)), the peak tie accelerations from the loading of the passing wheelsets are about 2 g to 4 g. Consistent spikes ($\sim$6 g) are observed at Site 2 (51 ft) and are likely due to a rail defect or some movement between the rail and tie plate or tie plate and tie, as the spike occurs within the passing wheelset and not before as observed at Upland (15 ft).

Both the transition zone and open track sites measure tie acceleration magnitudes from tie loading consistently below 5 g for both unloaded and loaded freight trains, which suggests the track behavior in the transition zone is similar to the open track and it is well supported. This is verified by the five other accelerometers that displayed similar behavior and with visual monitoring of track displacements where little, if any, track displacement was observed at any location within the bridge transition or open track. Analyzing the time histories in the frequency domain also showed no significant differences between Site 2 (7 ft) and Site 2 (51 ft). This behavior likely arises because of the following reasons.

1. The low levels of the transient vertical displacements in the open track, transition zone and bridge do not result in additional loading within the transition zone.
2. The ballasted bridge deck, HMA and concrete wing walls limit permanent vertical displacement within the transition zone, which prevents formation of a "dip" that results in increased loading in the transition zone.[5]

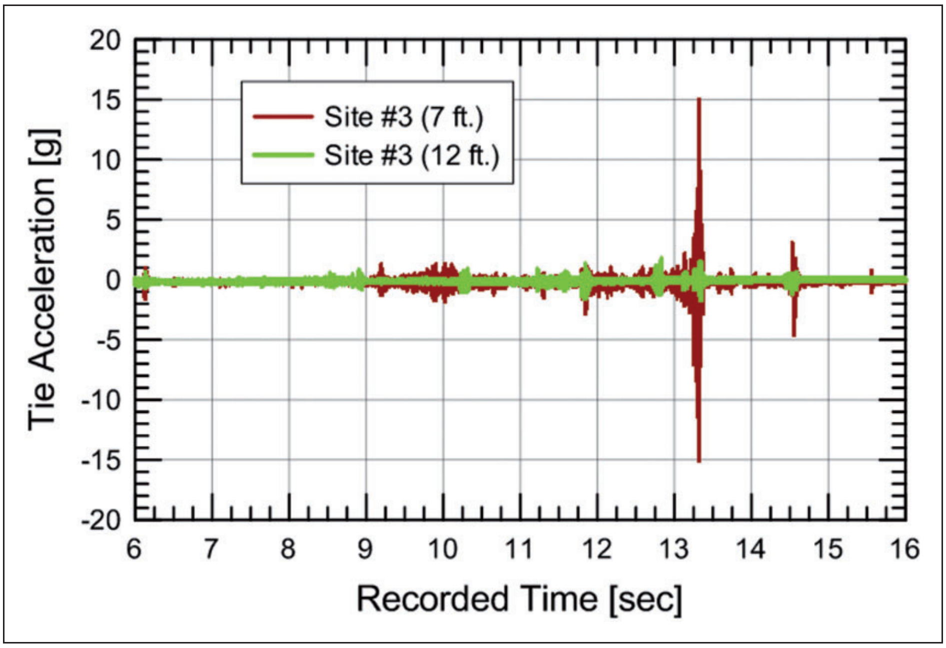

Figure 10. Measured tie acceleration time histories for Site 3 (7 ft) and Site 3 (12 ft) for a passing locomotive on 29 July 2014.

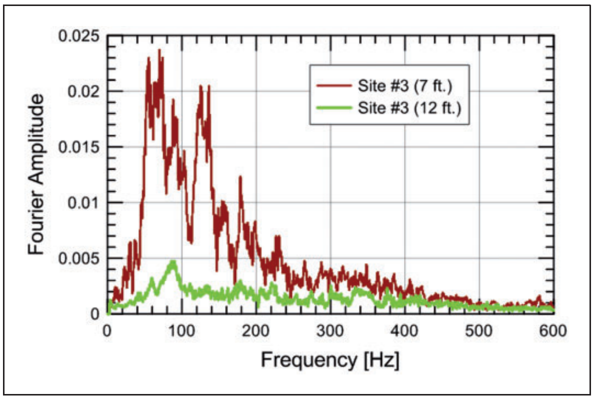

Figure 11. Measured tie acceleration time histories for Site 3 (7 ft) and Site 3 (12 ft) converted to the frequency domain for a passing train engine on 29 July 2014.

Behavior of Site 3: Poorly Supported Freight Bridge Transition Zones

The third instrumented site is a poorly supported spur track bridge transition and consists of freight trains entering and exiting into a loading terminal at 16 km/h (10 mph). The transition zone experiences a reoccurring permanent vertical displacement leaving a "dip" about 1.5 to 2.5 m (5 to 8 ft) wide from the bridge abutment that must be frequently remediated to maintain a suitable track geometry. Visual inspection and video records show poorly and possibly even completely unsupported tie behavior 0.6 to 3.0 m (2 to 10 ft) from the bridge abutment and better tie support 3.7 m (12 ft) and further from the bridge abutment.

On 29 July 2014, five bridge approach and five bridge exit measurements of a passing locomotive were collected at Site 3 at various speeds. The bridge approach measurements involved a single locomotive moving South onto the bridge and then reversing direction and moving off the bridge for the bridge exit measurement. A typical tie acceleration response for the transition zone location (Site 3, 7 ft) and (Site 3 12 ft) is displayed in Figure 10. The locomotive had a velocity of 16 km/h (10 mile/h) and was entering the bridge. Due to the low train velocity and tie loading, tie accelerations less than 1 g were measured and are difficult to discern, however, the key feature is the spike in tie acceleration ($\sim$15 g) at Site 3 (7 ft) at about 13.2 s into the recorded time history. This spike in the tie acceleration was observed at Accelerometers 3, 4 and 5, with Accelerometers 3 and 4 being installed at opposite ends of the tie located 0.6 m (2 ft) from the bridge abutment in the transition zone. As the locomotive wheel spacing of a single wheelset is about 2.5 m (6 to 8 ft), the spike appears when the first wheel of the second wheelset passes the bridge abutment. Therefore, this spike is probably caused by a sudden increase in dynamic wheel load. Due to the differential track stiffness and settlement of the ballast and substructure in the transition zone, the locomotive's front wheel will experience a sudden upward acceleration when hitting the bridge abutment causing the back wheel of the wheelset to accelerate downward and increase the dynamic wheel loads on the rail. This increase in dynamic wheel load has been numerically simulated by modeling the passing of a single wheelset onto a bridge approach.[5] It is not clear why the suspected spike does not appear during passage of the first wheelset, however, it may be caused by an interaction between the primary and secondary suspension systems of the locomotive.

Despite the low tie acceleration magnitudes from tie loading, significant differences are observed when this data is converted to the frequency domain (Figure 11). From a comparison with the bridge exit results, where the spike in the tie acceleration was not observed, it can be concluded that track and tie vibrations produce dominant frequencies between 50 and 100 Hz, whereas the spike at 13.2 s in Figure 10 produces frequencies of about 140 to 160 Hz. A significant difference is observed between the poorly supported and better-supported tie locations within the range of 50 to 100 Hz, suggesting the ballast surrounding the better-supported ties is more effective in damping and limiting these frequencies.

Discussion of Results

The presented data can be used as a benchmark to evaluate tie support conditions and the performance of railway track. For a variety of train types (passenger and freight), train loads (unloaded and loaded), durations, and train speeds (16, 40, and 177 km/h or 10, 25, 110 mph), the peak tie accelerations of well-supported track consistently measure at or below 5 *g*. Although increasing train load and speed should increase tie accelerations, the results suggest that peak tie accelerations will likely not exceed 10 *g*, even in high-speed, high-loading conditions in well-supported track. This is attributed to the smooth load transfer between all track components, e.g. rail, tie plates, tie and ballast, which limits displacements, vibrations and sudden movements within the track system, even when higher speeds and loads are applied.

For poorly supported track, tie accelerations consistently exceed 5 *g* to 10 *g* and include mechanisms, such as impact loads, between the tie and ballast and/or wheel and rail. Although it cannot be assessed from the presented data, it is suspected that tie accelerations are dependent on internal factors, such as rail–tie and tie–ballast gap heights, rail and wheel defects, and external factors such as train weight, speed, "bouncing", and wheelset types, tie spacing, and track curvature.

The results obtained using the field instrumentation show that accelerometers installed on railway ties are capable of providing information about whether the track is well supported or not, however, it does not necessarily provide quantitative information about the magnitude of poor support, this is due to the point that the accelerometers do not directly measure tie displacement, rather it must be estimated using double-integration techniques. However, evaluating the acceleration time histories in both the time and frequency domains, especially if coupled with video camera or LVDT data, can provide valuable insight into the location of track movement and its effect on track structure loading. This can help identify and diagnose common problems within track that experiences frequent track settlement or geometry problems. Accelerometers can also provide insight to the effectiveness of various bridge transition zone designs and track remediation techniques.

Summary

This paper describes the use of non-invasive techniques, e.g. accelerometers, to evaluate and compare track structure behavior for well- and poorly supported bridge transitions. The main results obtained in this study are as follows.

1. LVDT and accelerometer data from Site 1 show that accelerometers are capable of qualitatively identifying poorly supported ties, due to poorly supported ties experiencing large tie displacement, track and tie vibrations, and impact between the tie and ballast, all of which contribute to larger tie accelerations. These movements typically occur within the frequency range of 50–300 Hz and analyzing the time history in the frequency domain shows greater Fourier amplitudes within this frequency range.
2. From the instrumented sites that experience low amounts of permanent substructure settlement and are considered well supported, tie accelerations from tie loading are consistently at or below 5 *g*, due to the lack of tie movement. Tie accelerations below 5 *g* typically imply smooth load transfer from the rail to the subgrade, whereas tie accelerations above 10 *g* typically imply tie or track movement, which can amplify loads through impacts and accelerate ballast degradation.
3. Tie accelerations of well-supported track are consistently below 5 *g* for a variety of train types, loadings and speeds, due to increases in load and speed not generating significantly greater tie movement at well-supported ties. The threshold of 5 *g* may be exceeded with high-load, high-speed trains on well-supported ties; however, the data suggests this increase will not be significant due to minimal tie movement.
4. By analyzing tie acceleration signatures in the time and frequency domains, various types of impacts and vibrations can be discerned, as each type of impact and vibration tends to exhibit a unique signature. This aids in identifying poorly supported track, assessing how the track is behaving, and can give insight into the forces that the track components are actually experiencing. These acceleration magnitudes and frequencies are suspected to be dependent on tie–ballast and rail–tie gap heights and the external factors previously listed.

5. Results from well-supported bridge transitions (see Site 2) suggests that bridge transition designs that limit differential transient displacements between the bridge, approach, and open track are vital to preventing reoccurring track geometry problems. Additionally, bridge transitions with small permanent settlements experience similar track behavior between the transition and open track.

This study shows that tie accelerometers are inexpensive, non-invasive and easily installed devices for diagnosing and measuring track behavior. Tie accelerations can be produced from a variety of sources, e.g. tie loading, wheel flats, braking, rail–tie and tie–ballast impact, etc., which can make interpretation difficult, however, it can provide a wide range of information on the health of the entire track system. One main limitation of the tie accelerometers is that they are not capable of directly measuring the height of a tie–ballast gap; however, it can be estimated from double-integration of the acceleration time history. Accelerometers can be easily supplemented with video cameras, lasers and/or LVDTs to directly measure tie–ballast and rail–tie gaps.

If accelerometers are to be installed at transition zones, the authors recommend using at least eight accelerometers with the accelerometers being located on the bridge, along the transition zone, and in open track to effectively compare track behavior at different locations. It is generally helpful to observe the passage of a train prior to accelerometer installation to assess track behavior and determine the tie locations that will provide the most desired or best information. This can include locations of greater rail or tie movement, opposite ends of the same tie, middle of a tie to investigate center-binding, and nearby joints or stiffness transitions.

The authors are continuing to non-invasively monitor various types of railway track to expand the current database and improve tie acceleration interpretation. High-speed video cameras are included in all of the instrumentation setups for a quantitative and visual assessment of rail and tie movement. This instrumentation can be used to diagnose poorly performing track or evaluate the effectiveness of new track or transition designs along with remedial measures.

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