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Under-Tie Pads to Improve Track Resiliency in Railroad Transition Zones

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ABSTRACT

Under-tie pads were installed under twenty-nine (29) ties within an existing high-speed passenger bridge approach on the Northeast Corridor in an effort to alleviate reoccurring track geometry problems at this transition. Anticipated benefits of under-tie pads include: (1) a reduction in approach ballast and tie degradation by better distributing the load to the ballast, (2) increased vibration damping, and (3) reduced contact stress between the tie and individual ballast particles. After installation, the approach track geometry and behavior was measured using track geometry cars and non-invasive instrumentation, e.g., video cameras and accelerometers, to assess performance. These field measurements show stable track geometry after about one year of traffic.

Keywords: Transition zone, tie support, accelerometers, video cameras, under tie pad, ballast, differential movement, settlement

INTRODUCTION

Railroad transition zones are track locations that display rapid changes in track structure such as, bridge approaches, asphalt crossing, and culverts. The transition zone, i.e., approach track, often experiences increased track structure degradation, settlement, and requires resurfacing to alleviate reoccurring track geometry defects. For example, recent instrumentation of two high-speed passenger transition zones on the Northeast Corridor shows about two (2) and fourteen (14) times higher settlement rate in the approach (4.6 m/15 ft. from bridge abutment) than the corresponding open track (18.3 m/60 ft. from bridge abutment) [1]. The rapid deterioration of transition track geometry and the resulting required maintenance make transition zones an important topic in railroad engineering.

Multiple mechanical factors can increase approach settlement but it is typically attributed to increased loads, changes in rail elevation, and reduced-performance ballast in the approach. Increased vehicle loads can appear from rapid changes in axle elevations due to differential stiffness, settlement, and damping between the approach and bridge [2-4] and increased tie loads are possible from uneven settlement in the approach [5] and the impact from tie-ballast gaps [6.7]. Increased ballast degradation from tie-ballast gaps, increased loads, and repeated tamping is also expected to degrade, weaken, and foul the ballast [8,9] while blocked drainage from bridge abutments can produce wet spots, all of which reduce ballast performance and can lead to increased ballast settlement rates [10].

Recent transition zone solutions have focused on viewing the transition zone as a system and balancing approach and bridge stiffness and settlements [11] and creation of a Balance Sheet to assist engineers with understanding the different sources of transient and permanent displacements [11]. An example involves the installation of a ballasted bridge deck and reducing approach settlement with confining wing walls, hot-mixed asphalt (HMA), geoweb, and/or geogrid underlayment. Variations include further reduction in bridge stiffness by installing rail pads or ballast mats [12] or gradually increasing approach stiffness with a transition wedge [13]. An ideal solution should be effective in reducing differential settlement by balancing the transient and permanent displacements while remaining cost-effective and easy to install.

Under-tie pads (UTPs) were installed on twenty-nine (29) ties immediately before an existing high-speed passenger railway bridge to evaluate their effectiveness at reducing differential settlement between the bridge, approach, and open track. UTPs are relatively inexpensive and easy to install compared to track/tie replacement and the mechanical benefits include reduced tie load from improved distribution of the wheel load to the ties and a “cushion” layer between the tie and ballast to reduce tie and ballast contract stress and degradation. While the UTP “softens” the approach instead of stiffening it, the reduction in ballast degradation and settlement is expected to exceed the negative effects of increased stiffness differential between the bridge and approach. To assess transition performance after installation, periodic track geometry measurements have been made along with one extensive instrumentation effort about a year after UTP installation.

UNDER-TIE PADS

Railroad track geometry problems directly result from the permanent movement of the underlying track substructure. At transition zones, this typically occurs from one or more of the following reasons: (1) densification, lateral migration, and/or degradation of the ballast, (2) infiltration of ballast into the subgrade, and/or (3) continual settlement of an inadequately compacted or consolidated subgrade [8,14]. Previous measurements at this bridge transition zone show the majority of settlement occurred within the ballast layer because of the long service life of the subgrade fill. As a result, a strong correlation was found between ballast settlement and tie-ballast gap height [1,15]. Therefore, reductions in ballast settlement is desirable at this location but mitigating or reducing all three of these factors is necessary for minimizing transition zone maintenance in general [11].

Ballast settlement is a complicated process in which the structural behavior is dependent on multiple mechanisms occurring at the particle level, including: particle densification because of little or no compaction, particle densification, gradation changes from grinding of particle surfaces (Type I degradation), breakage of corners or asperities (Type II degradation), and the splitting or crushing of particles (Type III degradation) [16,17]. Laboratory tests on clean ballast suggest densification and Type II degradation are the primary settlement contributors [16]; however, analysis of in-track ballast typically show high levels of Type II and Type III degradation and an influx of fouling particles, i.e., particles less than 4.75 mm (No 4 sieve), from ballast degradation, infiltration of subgrade particles, or infiltration of particles from the surface [18]. These high degradation levels can occur over time from fatigue, tie-ballast contact and impact loads, and/or tamping [8,9]. Once high levels of ballast degradation or fouling occurs, ballast settlement rates have increased from 100 to 1,000% depending on the level of degradation, degree of fouling, size and plasticity of fouling particles, and levels of moisture in the fouled ballast [10].

One anticipated benefit of UTPs in transition zones is the reduction of ballast degradation and possibly densification. The UTP serves as a resilient layer at the tie-ballast interface that provides a cushion between the tie and ballast and dampens track and tie vibrations. The resiliency of the UTP provides a “cushion” for the ballast and can increase the contact area between the UTP and ballast particles. Studies investigating the contact area between concrete ties without UTPs and ballast show contact areas of only 20% [19], which indicates high particle contact stresses. By increasing the contact area between the UTP and ballast, the ballast particle contact stresses are reduced along with Type II and Type III ballast degradation because these degradation mechanisms result from high stresses within individual ballast particles [20]. An additional benefit of UTPs is they reduce tie degradation, which can result in greater tie-ballast gaps, i.e., additional track movement, and reduced tie performance.

The installation of UTPs in transition zones is a fairly recent development in the U.S. however existing studies in Europe suggest they result in reduced tie and ballast degradation [21]. Class 1 railroads in the United States are beginning to install UTP in transition zones for both concrete and wood ties to reduce ballast and tie degradation as described below.

SITE LOCATION AND INSTRUMENTATION

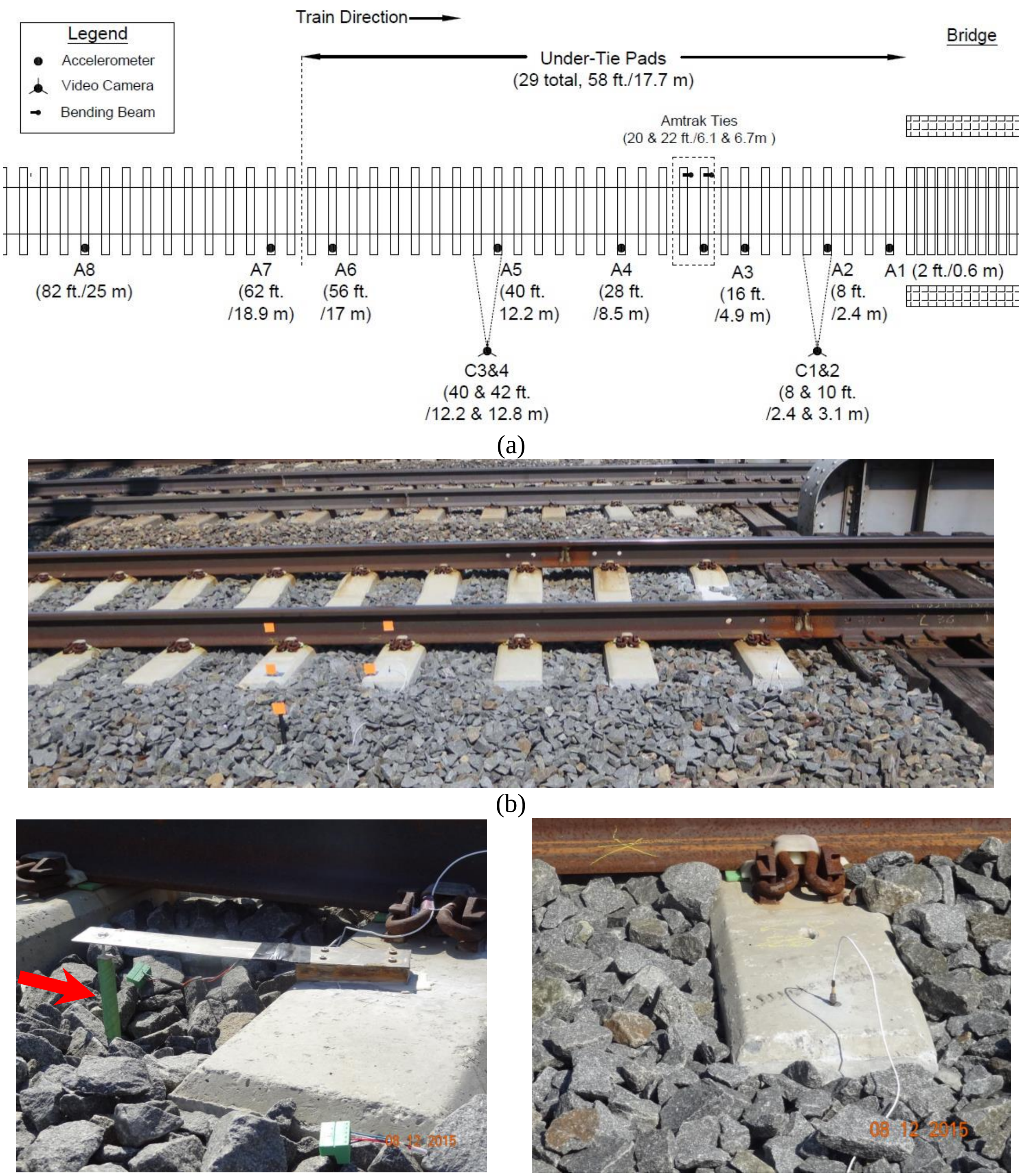
Site Location

On 31 August 2014, twenty-nine (29) UTPs were installed in the Upland Street southern Track #2 bridge approach along the Amtrak Northeast Corridor near Chester, Pennsylvania. This stretch of track consists of four tracks with the middle two tracks (Track #2 and #3) serving high-speed passenger trains while the outside two tracks (Track #1 and #4) serve slower moving commuter trains. The Upland Street Bridge is one of many closely spaced bridges that experience reoccurring track geometry defects [2,4] near Chester, PA and large stretches of unsupported ties in the approach [1,6]. The Upland Street northern Track #3 bridge approach was instrumented in 2012 at a distance of 4.6 m (15 ft) from the bridge and exhibited settlement rates of about 14 mm/year. Additionally, the transient tie displacement, i.e., "elastic" displacement induced by a passing train, is about three times greater in the approach than the open track at 18.3 m (60 ft) from the bridge [1]. While the southern Track #2 approach was not instrumented prior to under-tie pad installation, it was expected to behave similarly. The southern Track #2 approach is also underlain by a layer of hot-mix asphalt (HMA) roughly 0.31 to 0.36 m (12 to 14 inches) below the bottom of the installed UTP-fitted concrete ties. This HMA layer was installed sometime in the 1990's and discovered when the loader excavated the ballast section in preparation for installing the UTP track panel on 31 August 2014. The benefit of the HMA on the UTP performance at this location is not known at this time but should be considered when assessing the effectiveness of the UTPs at this location.

Instrumentation

On 12 August 2015, two instrumentation teams, one from University of Illinois at Urbana-Champaign (UIUC) and the other from Amtrak, installed multiple instruments to evaluate the transient behavior of the southern Track #2 approach of the Upland Street bridge about one year after UTP installation. The instrumentation layout is shown in Figure 1(a). The UIUC team measured tie displacement at Ties #4, #5, #20, and #21 at distances 2.4, 3.1, 12.2, and 12.8 meters (8, 10, 40, and 42 ft.), respectively, from the bridge abutment using two high-speed video cameras. To maintain consistency with previous papers at this location, the distance from the bridge abutment is expressed in Imperial units [1,4]. The cameras measure displacements by tracking 50 mm by 50 mm (2x2 inch) orange targets attached to the concrete ties (see Figure 1(b)) with imaging software developed at the UIUC [22]. The video cameras collect data at 240 frames per second which has been found to be adequate for high-speed passenger traffic by comparing the data with LVDT measurements at the northern Track #3 approach. The Amtrak team measured tie displacements at Ties #10 (6.1 meters/20 feet) and #11 (6.7 meters/22 feet) with an in-house device called a "Bending Beam", which operates similar to a dynamic voidmeter [23]. The Bending Beam is shown in Figure 1(c) and measures the displacement of the moment arm using calibrated strain gauges to +/-10 mm.

Both the UIUC and Amtrak teams also installed accelerometers (Figure 1(d)) to measure tie accelerations and calculate tie displacements using double-integration techniques [22,24]. The UIUC team measured tie accelerations at Ties #1, #4, #8, #14, #20, #28, #31, and #40 at distances 0.6, 2.4, 4.9, 8.5, 12.2, 17.0, 18.9, and 25.0 meters (2, 8, 16, 28, 40, 56, 62, and 82 ft.), respectively, from the bridge abutment using piezo-electric accelerometers. The Amtrak team measured tie accelerations only at Tie #10 at a distance of 6.1 meters (20 ft.) from the bridge abutment also using piezo-electric accelerometers.



(c) (d)

Figure 1: Upland Street northern Track #2 approach with UTPs: (a) diagram showing instrumentation layout, (b) overview photograph of transition with orange targets for video cameras, (c) Bending Beam with reference stake (see arrow) located in crib, and (d) accelerometer attached to a concrete tie.

All three instruments, video cameras, Bending Beams, and double-integrated accelerometers, have their unique advantages and disadvantages. The video cameras directly measure tie displacement at a sampling rate of 240 Hz and camera vibrations were minimized by tracking a stationary stake that is driven into the ballast between Tracks #2 and #1 in front of the orange targets. The camera vibrations are accounted for by subtracting these motions from the measured tie displacements. The Bending Beams directly measure tie displacement at a sampling rate of 10,000 Hz but the Bending Beam is supported on a stake that is placed in the crib. As a result, vertical crib displacement can result in a lower measured displacement than the video cameras or accelerometer measurements. Accelerometers indirectly estimate tie displacement using double-integration techniques at sampling rates of 4,000 Hz (UIUC) and 10,000 Hz (Amtrak). For verification, the video camera and accelerometer results are compared at Ties #4 (8 ft.) and #20 (40 ft.) while the Bending Beam and accelerometer results are compared at Tie #10 (20 ft.). Previous measurements at the Upland Street northern Track #3 bridge approach show tie displacement frequencies range from 5 to 25 Hz, so the sampling rates of all instruments are considered sufficient, which is verified in subsequent sections.

The second transient behavior metric is tie acceleration. Accelerations are measured because accelerations capture the small-displacement high-frequency motion of the tie from vibrations and impacts that are usually not captured by displacement based measurements, i.e., cameras and Bending Beam. Accelerations also provide insight to the loading environment within the track system. Both the UIUC and Amtrak teams used piezo-electric accelerometers, which are designed to capture high-magnitude (500g) and high-frequency (10,000 Hz) accelerations that are often present in railroad track. For example, previous instrumentation of the Upland Street northern Track #3 approach showed tie accelerations up to 30g from the impact between an unsupported tie and the underlying ballast [7]. The UIUC team placed eight (8) accelerometers at the eastern end of concrete ties at various distances from the bridge abutment shown in Figure 1(a). The Amtrak team placed eight (8) accelerometers on a single tie with two of them on the rail seats at a distance of 6.1 m (20 ft.) from the abutment to focus on the vibrational characteristics of this tie. The Amtrak accelerometer closest to the tie edge was used for comparison because it best represents the tie location that was used by the UIUC team. The vibrational characteristics of the ties were not analyzed for this paper but will be discussed in future papers to investigate the damping characteristics of UTPs.

Filtering

A signal processing technique used for the video camera, Bending Beam, and accelerometer records is filtering. The purpose of filtering is to eliminate noise within the signal records or isolate particular frequencies. Multiple types of mathematical filters exist and differ based on the mathematical equation used to smooth the transition from the filtered and non-filtered range. The Butterworth filter is selected for analyses because it is simple to use and sufficiently filters the signal.

The frequency range to be filtered is generally specified by selecting one of three types of filters: low-pass, high-pass, or band-pass. Low-pass filters allow frequencies lower than the frequency cutoff and attenuate higher frequencies. High-pass filters allow frequencies higher than the frequency cutoff and attenuate lower frequencies. Band-pass filters allow frequencies between two frequency cutoffs and attenuate frequencies outside the cutoff range. An example of a low-pass 8^{th}-order Butterworth filter with a frequency cutoff of 500 Hz and its effect on the frequency domain response is displayed in Figure 2. The frequency domain shows the tie predominant frequency responses at 80 Hz, 390 Hz, 740 Hz, and 1,120 Hz. The 80 Hz frequency is attributed to impact between the tie and ballast while the remaining three are attributed to the vibration modes of the concrete tie. The filter largely eliminates the response above 500 Hz and in this case, eliminates the influence of higher order vibration modes.

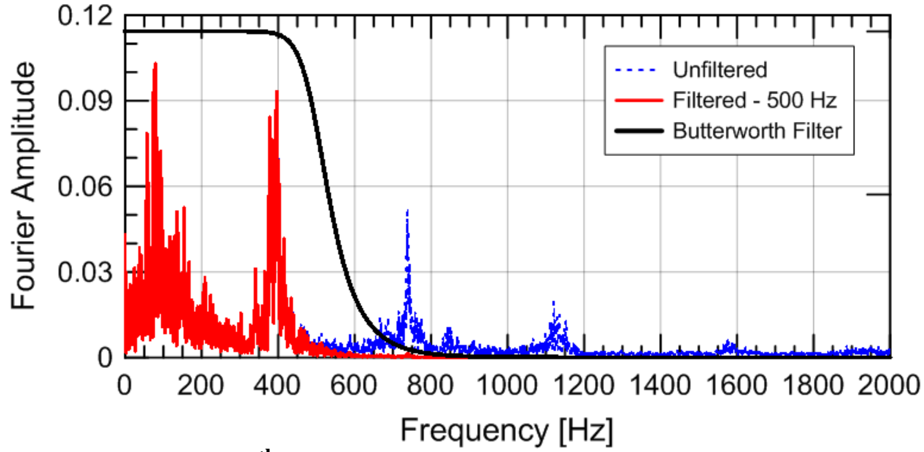


Figure 2: Effect of low-pass 8th-order Butterworth Filter with frequency cutoff of 500 Hz in frequency domain.

FIELD MEASUREMENTS

The measurement results presented below are from a 177 km/hr (110 mph) Acela train traveling north along Track #2 entering the Upland Street Bridge at 12:39 PM on 12 August 2015. Slight variations were observed for different trains but the overall trends are similar so the results of only the 12:39 PM train are presented herein due to space constraints.

Displacement Results

To gain confidence in the instrumentation results, comparisons between the UIUC video cameras and accelerometers and the Amtrak accelerometers and Bending Beam were made for the same tie. The UIUC video camera and accelerometer results from Tie #4 (8 ft.) are compared in Figure 3(a) and (b), respectively. The video camera was filtered using an 8th-order low-pass Butterworth filter with a frequency cutoff at 12 Hz. The thick black line represents the filtered response and the thin red line represents the unfiltered response. A low frequency cutoff was necessary to reduce the influence of the camera vibration and rotation not eliminated by the driven stake because the camera was placed on a steep slope close to Track #1. The accelerometers were filtered using a 3rd-order band-pass Butterworth filter with cutoffs at 2 Hz and 30 Hz. The high-pass cutoff (2 Hz) was necessary to eliminate drift from the double-integration process because of instrumentation noise and this filtering also affects the shape of the response but not the magnitude of the displacement. This means that the peak-to-peak displacement magnitudes will be valid but the shape of the response may not be properly reconstructed. The comparison of the video camera and accelerometer displacements show the magnitudes from the filtered video camera response (4.5 mm/0.18 in) are in agreement with the double-integration displacement (4.1 mm/0.16 in).

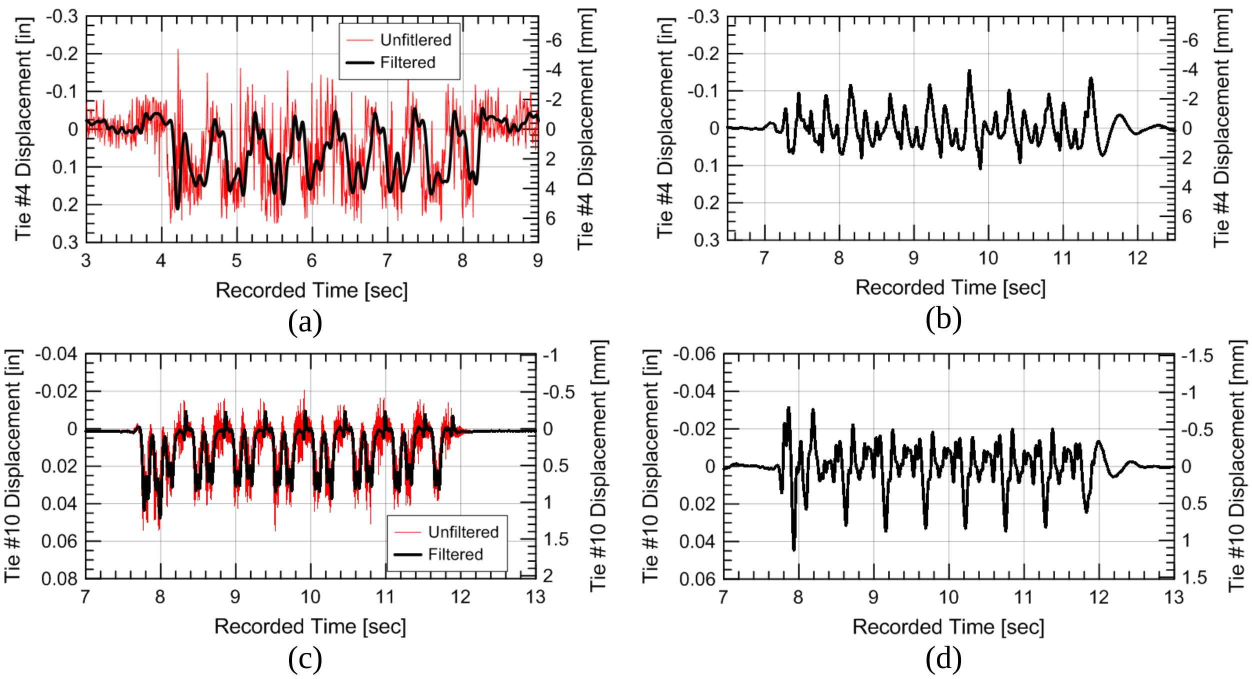
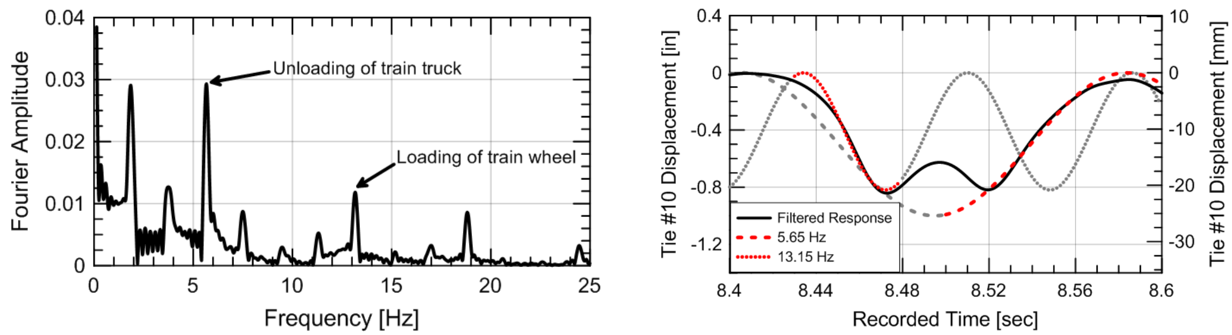


Figure 3: Comparison of Tie #4 (8 ft.) response from UIUC: (a) video camera and (b) accelerometer and comparison with response of Tie #10 (20 ft.) from Amtrak: (c) Bending Beam and (d) accelerometer.

The Bending Beam and accelerometer results at Tie #10 (6.1m/20 ft.) are compared in Figure 3(c) and (d), respectively. The Bending Beam was filtered using a 5th-order low-pass Butterworth filter with a cutoff at 30 Hz while the Amtrak accelerometer was filtered in the same manner as described earlier. The comparison shows a smaller displacement measured by the Bending Beam (0.85 mm/0.03 in) than the accelerometer (1.4 mm/0.06 in). A potential explanation for the difference is vertical displacement of the reference stake within the crib that is not accounted for with the Bending Beam measurements.

To analyze the frequencies of tie displacement, the Bending Beam response was converted to the frequency domain using Fast Fourier Transform (FFT) techniques and displayed in Figure 4(a). The results show tie displacement frequencies ranging from 2 Hz to 25 Hz. By plotting sine waves alongside the time domain response, the influence of the various frequencies is visualized in Figure 4(b). The results indicate that a frequency of 13.15 Hz represents the loading of a single wheel and 5.65 Hz illustrates the unloading of the train truck. The other frequencies also represent physical motion but are not easily visualized; however, most displacement magnitude information appears to be in the 6.65 and 13.15 Hz frequencies. This indicates that 13 Hz is the dominant frequency of train loading at Tie #10 (20 ft.) and all instruments are able to capture this frequency as expected. The 12 Hz cutoff of the video camera captures the peak displacement of each train truck, which is the desired value, but does not capture the entire shape from each passing wheel because of the presence of camera vibrations of about 15 Hz.



(a) (b)

Figure 4: Bending Beam response in: (a) frequency domain and (b) time domain with sine waves at different frequencies.

Accelerometer Results

The accelerometer results at Tie #1 (2 ft.) are shown in Figure 5 at various filtering frequency cutoffs to emphasize the impact of filtering and how to determine the frequency of various acceleration mechanisms in time histories. Figure 5 shows the unfiltered acceleration time history along with the 8th order low-pass Butterworth filter response with frequency cutoffs at 1000, 500, 200, 60, and 30 Hz. This is supplemented with the unfiltered response in the frequency domain (Figure 5(g)). These results indicate the majority of tie acceleration occurs within the 60 to 500 Hz range with the average peak acceleration dropping from 30g (Figure 5(c)) to 5g (Figure 5(e)) when this frequency range is attenuated. This agrees with the frequency domain response in Figure 5(g) that shows the majority of the frequency response occurring within the 60 to 500 Hz range for Tie #1 (2 ft.). This response is attributed to tie loading and appears in the acceleration time history as continual 10 to 20g responses during each wheel pass.

The noticeable frequency responses at 390, 740, and 1,120 Hz are likely due to vibration of the concrete ties with the UTPs attached because the responses are present for all ties measured with UTPs. These responses are not present for the ties that do not have UTPs. For example, Ties #31 (62 ft.) and #40 (82 ft.), which do not have UTPs, do not show significant frequency response greater than about 300 Hz. The large response at 80 Hz is attributed to the impact of the tie contacting/impacting the ballast. This can be observed in Figure 5(d) with a frequency cutoff of 200 Hz time history, where a single downward peak of about -30g occurs during every wheel passage. This behavior is similar to previous Upland Street northern Track #3 measurements, in which the peak acceleration correlated exactly with the time that the tie contacted/impacted the ballast [7]. No other tie except Tie #1 (2 ft.) displays a significant response at 80 Hz, which suggests that Tie #1 (2 ft.) in the southern Track #2 approach is unsupported. This agrees with visual observations of significant tie movement during train passage and large displacements (8.0 mm) from double-integration of Tie #1 accelerations. As a result, Tie #1 is likely experiencing impact during contact with the ballast and additional vibration from the lack of ballast damping even though an UTP is present. The response in Figure 5(f) with a frequency cutoff of 30 Hz isolates the accelerations from tie displacement (~2 to 25 Hz) because this response can be double-integrated to estimate peak displacements.

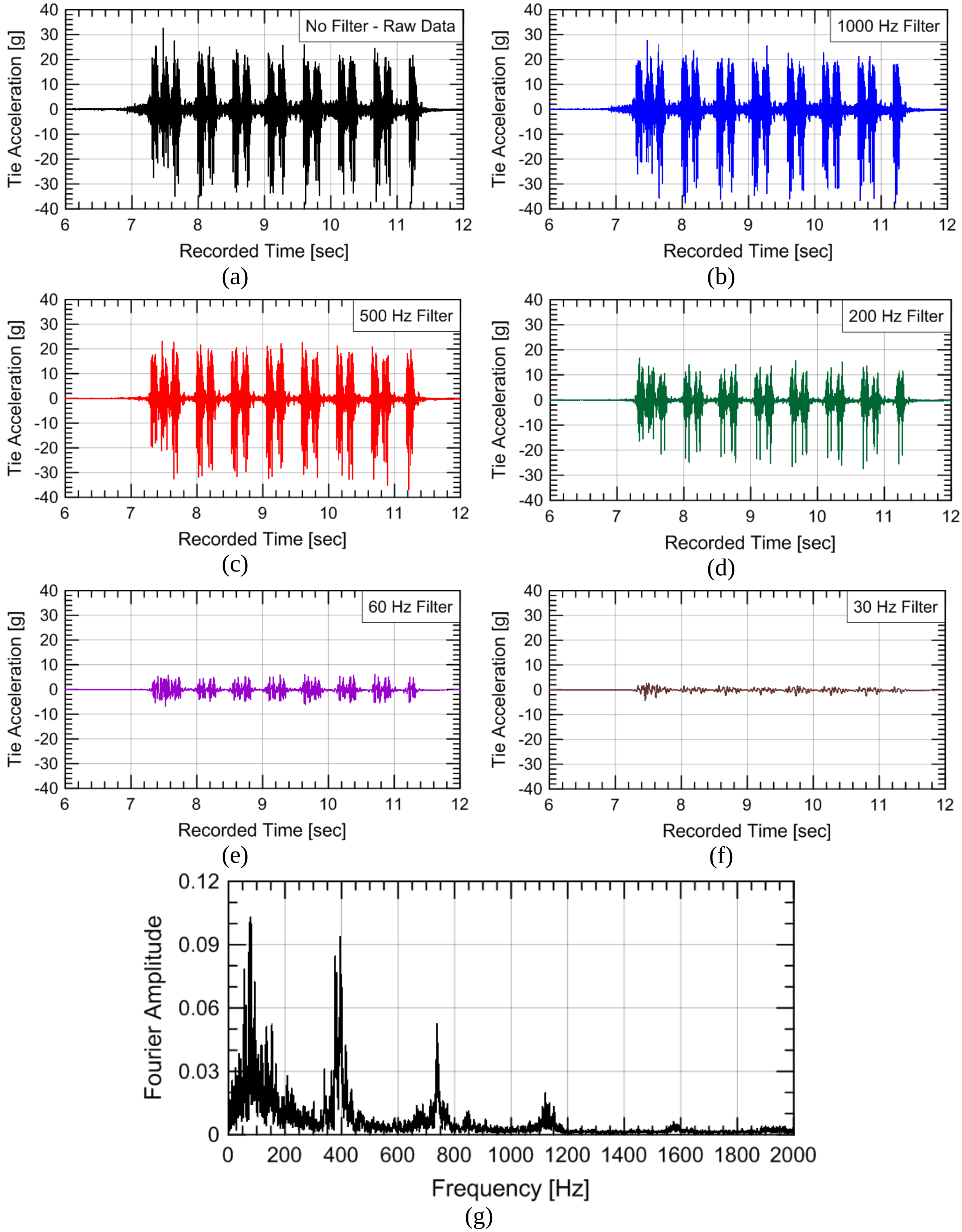


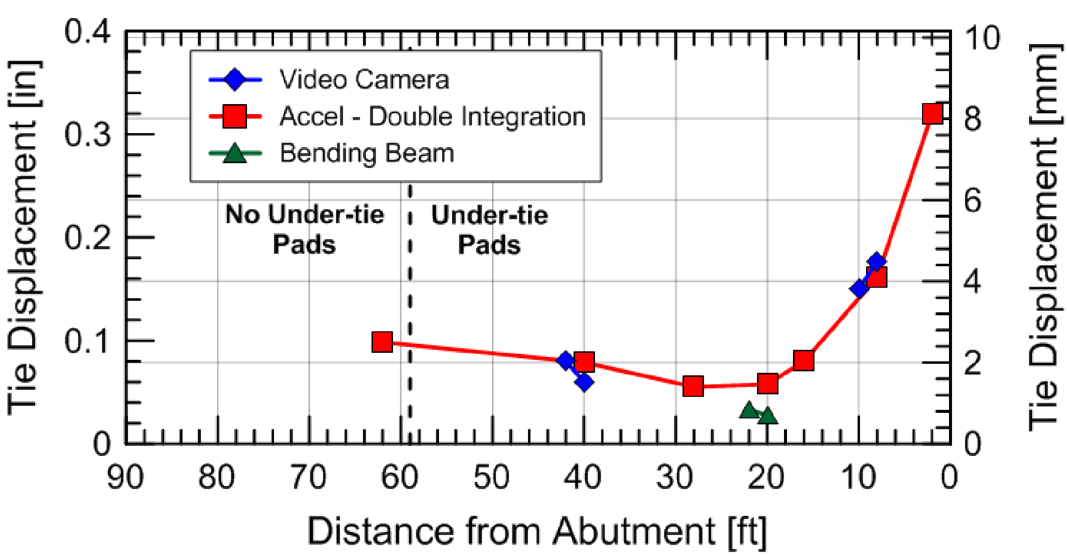
Figure 5: Accelerometer response at Tie #1 (2 ft.) with: (a) no cutoff and frequency cutoffs of: (b) 1,000 Hz, (c) 500 Hz, (d) 200 Hz, (e) 60 Hz, (f) 30 Hz, and (g) corresponding frequency response.

Response with Distance

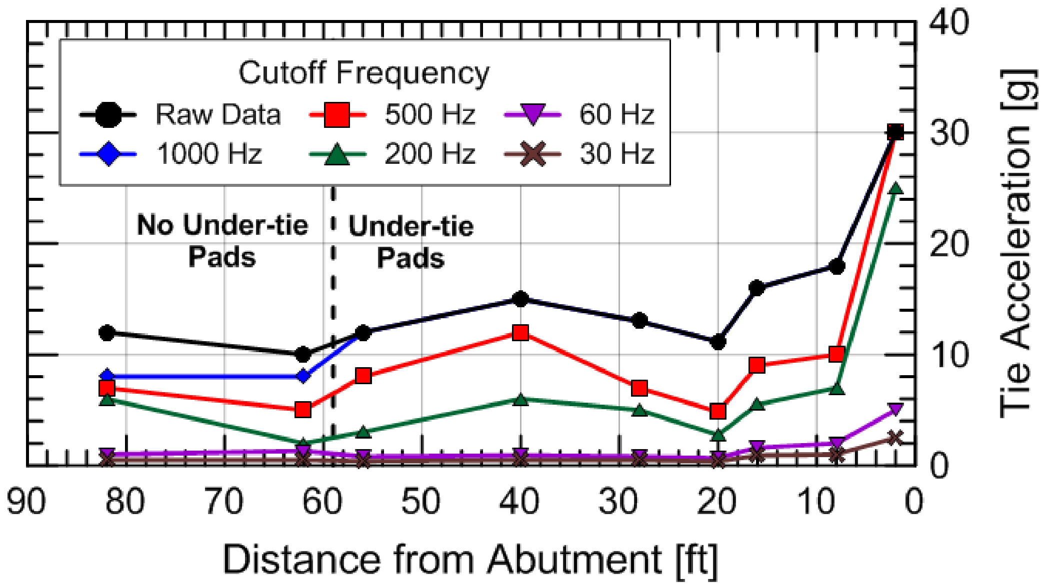
The peak displacement and acceleration values are plotted with distance from the abutment to evaluate changes in behavior along the transition zone for ties with and without UTPs. Figure 6(a) compares the displacements from all available instruments and the displacement magnitudes show general agreement except for the lowest magnitude being from the Bending Beam displacement, which is probably due to vertical displacement of the reference crib ballast (see Figure 2(c)). In general, the results suggest large transient displacements near the bridge approach that decrease to a local minimum at about Tie #10 (20 ft.) and then slightly increase until the open track is reached.

Figure 6(b) shows the average peak accelerations at various frequency cutoffs. The general response agrees with the tie displacements, in which large accelerations are observed near the bridge abutment and the accelerations decrease until about Tie #10 (6.1 m) and then they increase again until the open track is reached. This increase in displacements with distance could be attributed to the “landing zone” for trains that ran southbound on Track #2 instead of the predominant direction of northbound. No significant difference is observed at the transition between ties with UTPs (Ties #1 through #29) and ties without UTPs (Tie #30+) but this is anticipated because all of the instrumented ties in the open track are well supported so the benefits of UTPs are not being manifested.

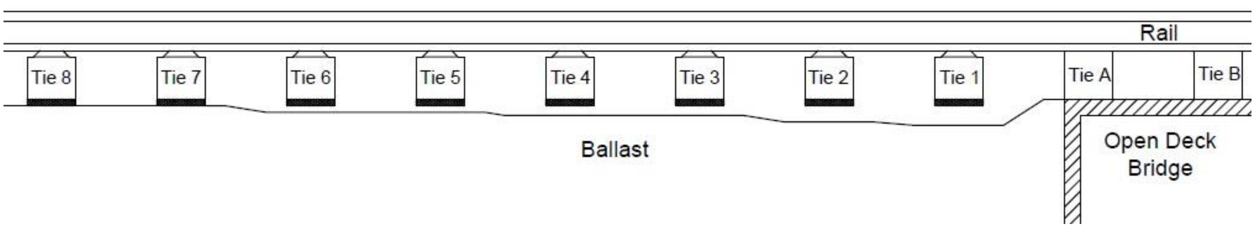
The displacement and acceleration results suggest that the first six to seven ties are unsupported with differing tie-ballast gaps as shown in Figure 6(c). This is in agreement with the impact observed at Tie #1 (2 ft.) in the tie acceleration response that was not observed at any other tie. The unsupported ties were also observed visually during train passage.



(a)



(b)



(c)

Figure 6: Diagrams of: (a) peak displacement and (b) peak acceleration with distance from the abutment and (c) schematic of track support along the transition zone, which shows Ties #1 through #6 unsupported.

Track geometry measurements recorded monthly for this approach suggest little change in track geometry has occurred between the installation of the UTPs and the day of testing on 12 August 2015, i.e., after 346 days. It is not clear whether the six to seven unsupported ties near the abutment have existed since UTP installation, i.e., due to inadequate ballast compaction near the bridge abutment, or whether ballast settlement has slowly been accumulating over the 346 days due to increased vehicle loading at the abutment, poor drainage, ballast degradation, and/or tie damage. Regardless, no track geometry problems have developed at the southern Track #2 transition since UTP installation about one year earlier as opposed to 14 mm/yr of settlement measured at the northern Track #3 transition prior to remediation. Unfortunately, direct comparisons with previous analyses at surrounding sites cannot be made due to the use of different instrumentation.

If track geometry continues to hold, UTPs may show to be an effective remedial measure for reducing differential movement at railroad transition zones. It is hoped future monitoring will confirm this initial observation.

SUMMARY

Under-tie pads (UTPs) were installed under the twenty-nine (29) ties immediately before an existing high-speed passenger bridge to evaluate their effectiveness in reducing dynamic response of the track and differential movement between the bridge, approach, and open track. Track structure measurements about one year after UTP installation show unsupported ties near the bridge abutment but track geometry measurements so far do not show a deterioration of track geometry. This suggests that the unsupported ties near the abutment are likely caused by ballast/subgrade settlement immediately after placement and significant ballast degradation has not occurred since UTP installation as it has in an adjacent approach because the geometry does not appear to be changing with time. It this holds, this suggests the anticipated benefits of reduced loads, vibrations, impacts, and ballast degradation can reduce track geometry deterioration over time. In other words, while UTPs will not result in smaller tie accelerations everything else being equal, but they can prevent conditions that cause higher tie accelerations, tie loads, and track degradation to develop.

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