# Design and Performance of Well-Performing Railway Transitions

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## ABSTRACT

This paper presents a review of railroad track transition behavior, causes of undesirable transition performance, and designs that exhibit desirable transition performance based on field measurements. The first focus of the paper is reviewing common factors that lead to transition geometry deviations. This involves the inherent problem of a train passing from an earthen and ballasted approach to a nearly rigid bridge structure. The differential movement between the earthen approach and bridge usually results in increased dynamic loads. To avoid these increased dynamic loads, all transient and permanent displacements between the approach and bridge deck should be balanced by reducing ballast and subgrade settlements in the approach and decreasing the stiffness of the bridge. Two well-performing bridge transitions were monitored using non-invasive accelerometers to illustrate design techniques that can balance transition differential movements and thus reduce dynamic loads. Other design techniques and ballast remedial measures are discussed because of their relevance to reducing ballast settlement in the approach.


**Keywords:** Transition zone, tie support, accelerometers, under-tie pad, ballast settlement

## INTRODUCTION

Differential movement at higher-speed railway transition zones represents a safety and maintenance issue because of the continual upgrade to heavier, longer, and faster trains. In 2005, the Association of American Railroads estimated the annual maintenance cost for transition zones to be about $200 million (*1*) and this value will have likely increased.

To reduce differential movement and the need for frequent track resurfacing, a variety of transition zone designs and remedial measures have been proposed (*2*-*6*). These typically involve: (1) increasing and smoothing track stiffness in the approach and/or (2) lowering the track stiffness of the bridge deck because differential stiffness and settlement usually causes undesirable transition zone performance. Potential solutions in the approach include: additional rails to increase track stiffness, increased tie lengths, decreased tie spacing, under-tie pads (UTPs), abutment wing walls, hot-mixed asphalt (HMA) underlayment, concrete approach slabs, geoweb or geocells, and soil stabilization. Bridge deck solutions include: a ballasted bridge deck, rail and tie pads, and ballast mats. While a few of these solutions have shown promising results, none are all-encompassing and typically only work on a site-specific basis.

This paper: (1) reviews the multiple causes of differential settlement in transition zones, (2) describes non-invasive field measurements to quantify transient and permanent transition zone performance, (3) illustrates two successful bridge transition zone designs, and (4) discusses potential design and remedial measures.

## CAUSES OF INCREASED AND DIFFERENTIAL SETTLEMENT

Multiple studies have investigated the potential causes of increased settlement at transition zones (*2*,*3*,*7*-*9*). Besides the consensus that the causes are typically site specific, increased dynamic loading within the transition zone is often viewed as the primary mechanical factor that results in increased settlement in the transition zone. Inadequate drainage, ballast degradation, and inadequate subgrade and/or ballast compaction also contribute to magnitude of settlement.

Many mechanisms can increase the dynamic loads in the transition zone and some commonly mentioned factors include: (1) rapid changes in axle elevation, (2) uneven load distribution, (3) impact loads from moving ties contacting the ballast, and (4) high-stiffness and low-damping of the bridge. To optimize transition design, it is important to identify which factors contribute significantly to increased dynamic loads and then focus on reducing the influence of these factors, which result in smaller differential movement in the transition.

The rapid change in axle elevation at the abutment factor is frequently cited as the cause of increased dynamic loads (*7*,*10*) and results from differential stiffness and settlement between the approach and bridge deck. In both cases, the lower track stiffness and/or greater track settlement in the transition zone cause the front axle of a truck to accelerate upwards when it hits the bridge abutment. The rapid upward acceleration of the front axle results in an increased loading on the bridge abutment. In addition, the coupling of the front and back axles causes the back axle to be pushed downward, which increases the loading 1.8 to 3.2 meters (6 to 12 ft.) from the abutment. A distance of 1.8 to 3.2 m (6 to 12 feet) from the bridge abutment corresponds to the distance of the back axle from the abutment and is speculated to produce the “dip” often observed 1.8 to 3.2 m (6 to 12 feet) (see Figure 1) from the bridge abutment. Numerical models isolating the effect of differential stiffness between the bridge and approach, i.e. no ballast settlement, show increased dynamic loads of less than 20% while increased dynamic loads of greater than 100% have been calculated when the approach is assumed to settle uniformly (*7,10*). This suggests that differential settlement is more detrimental than differential stiffness but both should be avoided if possible.

The second and third factors resulting in increased applied dynamic loads (uneven load distribution and impact loads from moving ties contacting the ballast) result from tie-ballast gaps developing within the approach near the abutment (*11*). Tie-ballast gaps, i.e. hanging ties, develop because the ballast and/or earthen materials in the approach substructure settle while the bridge deck height remains essentially constant and rigid over time because it is on deep foundations. This results in the rail and ties hanging or cantilevering from bridge deck while tie-ballast gaps of varying height develop in the approach (Figure 1). Due to the existence and variation of tie-ballast gaps along the approach, the wheel load redistributes and concentrates on particular ties (*12*). The load applied to the ballast also can increase when the moving ties impact the ballast because of Newton's Second Law that states the applied force (F) equals mass (m) multiplied by tie acceleration (a). Accelerometers attached to concrete ties in transition zones show increased accelerations during contact with the ballast, which supports the explanation of increased applied dynamic loads due to ballast impacts (*13*).

The fourth factor resulting in increased applied dynamic loads (high-stiffness and low-damping of the bridge) results from the bridge being founded on deep foundations especially when concrete ties are used on the bridge deck. This results in a stiff structure with little damping of the resulting vibrations. The effects of bridge stiffness and damping have been investigated by other authors (*2,4,14*).

Additional factors that can increase settlement within the approach are poor drainage, ballast degradation, and undesirable construction and maintenance practices (*15-17*). Excess water, typically coupled with ballast fouling, can result in lower stiffness (*15*), increased settlement (*16,17*), and development of excess pore-water pressures within the ballast during train passage (*18*), all of which accelerate track geometry deterioration. Undesirable construction and maintenance practices include: inadequate geotechnical characterization, inadequate compaction, non-uniform soil, narrow embankment widths, steep side slopes, and inadequate ballast tamping (*4*).

## TRANSITION BALANCE SHEET

To reduce the dynamic loads in the transition zone, transient and permanent displacements in the approach and bridge must be balanced. In other words, an ideal transition will have a constant rail elevation between the approach and bridge during train passage so the wheel does not bump into the abutment. This eliminates increased dynamic loads from rapid changes of axle elevation when the wheel hits the abutment and load redistribution.

To illustrate the potential sources of detrimental differential transient and permanent displacements between the approach and bridge, a "Transition Balance Sheet" is developed and presented in Figure 2. The Balance Sheet lists the many sources of potential transient and permanent displacements in the Approach (A) and Bridge (B) including: (1) rail compression, (2) rail-tie gap, (3) tie pad/plate displacement, (4) tie displacement, (5) tie-ballast gap, (6) ballast displacement, (7) subballast displacement, (8) subgrade displacement, and (9) lateral displacement. Figure 2 displays a worst-case scenario of an open-deck bridge in which the approach may experience transient and permanent substructure displacements while the bridge does not. The check marks represent a "potential detrimental displacement" that will be problematic if it is not balanced by the bridge. In this particular case, the "potential detrimental displacement" applies for the entire substructure along with gaps that develop in the track system because of differential permanent substructure displacements.

Many previously proposed designs to reduce differential settlement have attempted to increase approach track stiffness, reduce bridge stiffness, or reduce approach ballast settlement by focusing on a single track component (*2-6,19*). This may be successful if displacements are balanced but often the stiffness and settlement differences are too great for a successful design to only focus on a single track component or even focusing on only the bridge or approach. This is why the entire track should be viewed as a system in which all of the potential transient and permanent displacements in Figure 2 are addressed because once differential transient and/or permanent displacement develops, the dynamic loads will increase and accelerate track degradation.

## WELL-PERFORMING TRANSITIONS

A significant amount of previous research on transition design involves implementing a solution and then monitoring its performance over time to assess its effectiveness (*3,14*). This method is beneficial because it shows a quantitative "before" and "after" comparison and the influence of a particular solution but it is costly and does not always produce beneficial results, especially if the solution solely focuses on a single track component. Because of the large costs involved with a new transition design or remedial measure, the authors decided to investigate bridge transitions that already perform well instead of installing a solution and hoping it performs well. To accomplish this objective, two well-performing bridge transitions were instrumented. Future instrumentation will hopefully involve other well-performing designs with different design attributes to add the database.

### Non-Invasive Instrumentation

The instrumentation used for these two transition sites consists of eight miniature accelerometers that were placed on the bridge, approach zone, and open track. The accelerometers are only 13 mm long (one half inch), weigh less than 3 grams (0.1 ounces), and are connected to the tie with a small amount of superglue. This results in a non-invasive monitoring system that can be set up in less than 30 minutes and does not interfere with train operations. This makes accelerometers suitable for short-term monitoring, i.e., a single train pass or day long monitoring, as well as long-term monitoring during wet and inclement weather conditions because weather resistant accelerometers are available. A photograph of an accelerometer attached to one of the concrete ties is shown in Figure 3.

Tie acceleration time histories can be informative of track performance because any impact or movement of the tie is recorded by the accelerometer. Ideal track conditions, defined as "well-performing" herein, typically consist of track experiencing vertical displacements of only 1 to 2 mm, a smooth and evenly distributed load path from the wheel to the ballast, and minimal track geometry maintenance. In these cases, the tie is expected to only accelerate from the loading of the passing wheels and typically produce tie accelerations of less than 5g (*20*). Non-ideal track conditions, defined as "poorly-performing" in the paper, typically consist of track experiencing vertical displacements greater than 2 mm, movement and impacts from the closing of gaps in the track system, and recurring track geometry maintenance. In addition to tie accelerations from the loading of the passing wheels, the tie can also accelerate from impacts in the track superstructure, tie-ballast impacts, tie vibrations, and tie displacements (*21*). These additional factors can produce tie accelerations ranging from 10 to 100g and these values are highly dependent on train type, loading, and speed (*20*). Impacts and vibrations from the vehicle, e.g. wheel flats and braking, are also recorded but are not considered in the track analysis because these factors are vehicle issues, not track issues.

**Site #1**

The first instrumentation site consists of a freight bridge transition zone with velocities of about 40 km/hr (25 mph), annual traffic of about 7 MGT, and minimal track geometry maintenance since being placed in service in 2009 (~6 years of service). The site is shown in Figure 4 and involves the west bridge approach and is built on a 23 m (75 ft) compacted fill embankment. The track has timber ties, supports both loaded and unloaded freight trains, and is considered Class III for operations (maximum train velocity of 65 km/hr / 40 mph). Despite its allowable 40 km/hr (25 mph) speed, the operating speed at Site #1 is only about 40 km/hr (25 mph) because the train is near its destination.

To avoid differential movement and the subsequent increase in dynamic loads, the bridge transition was designed with the following four major features: (1) ballasted concrete bridge deck, (2) a 150 mm (6 in) thick hot-mixed asphalt (HMA) layer that extends for 600 m (2,000 ft.) from the abutment that is overlain by a 300 mm (12 in) thick ballast layer on the approach embankment, (3) 8 m (27 ft) long concrete wing walls that are perpendicular to the bridge abutment (see Figure 4), and (4) wetting and hydrocompression of the approach fill for five (5) years prior to track construction. These features are important and help balance the approach and bridge displacements because: (1) the ballasted bridge deck reduces the load-displacement differences between the approach and bridge deck by increasing track displacement and settlement on the bridge, (2) the HMA layer creates a higher ballast modulus, spreads the train loads over the approach fill, confines the ballast laterally, and provides an infiltration barrier between the ballast and subgrade to reduce softening of the approach fill, all of which reduce settlement in the approach (*22*,*23*), (3) perpendicular concrete wing walls provide confinement to the ballast and subgrade which reduces vertical and lateral ballast settlements in the approach, and (4) waiting five (5) years for the 23 m (75 ft) fill to experience infiltration and hydrocompression reduces future approach fill (subgrade) settlement due to train and environmental loadings.

Accounting for these design features in the “Transition Balance Sheet” results in an acceptable balance of the transient and permanent displacements between the approach and bridge. The ballasted bridge deck and ballast confinement in the approach balances the ballast displacements while the hydrocompressed fill minimizes subballast and/or subgrade displacements. The lack of differential permanent displacement between the bridge and approach reduces the formation of rail-tie and/or tie-ballast gaps in the approach.

Approach fill hydrocompression is detrimental to transition zone performance because it lowers the rails and creates a “dip” at the bridge abutment. As a result, new ballast is periodically added to compensate for the fill compression and maintain track geometry. It is ideal if the approach fill is constructed off-line and has enough time to experience infiltration and hydrocompression before track construction but this construction delay is rarely practical. Alternative methods to avoid fill settlement involve compacting/placing the fill material wet-of-optimum or using a granular fill with a vegetative soil cover to avoid erosion of the granular fill.

Using non-invasive accelerometers, the track on the bridge, transition zone, and open track were monitored at Site #1. The results of two accelerometers are presented to display the difference between the bridge transition and open track behavior during passage of a 40 km/hr (25 mph) loaded freight train. The two accelerometers are located 2.1 m (7 ft) and 15.4 m (51 ft) from the bridge abutment. These two sites will be referenced as Site #1 (7 ft.) and Site #1 (51 ft.) herein and represent the transition and open track responses, respectively.

Figure 6 displays only 10 of the 280 seconds of the Site #1 (7 ft.) and (51 ft.) acceleration time history of a passing freight train to emphasize a few important observations. At both locations, the peak acceleration magnitude is about 5g, which is representative of good track performance in the transition and open track. The six other accelerometers behaved similarly with tie accelerations of about 5g for loaded freight trains and tie displacements of about 1.0 mm (0.04 inches) (*22*). The low values of acceleration (~5g) and tie displacement (~1.0 mm) indicate good track support when compared to poorly supported track where tie accelerations can range from 10 to 100g and tie displacements can reach 10 mm (0.4 in) or greater (*13*). The lack of discernable difference between the seven selected ties at varying locations from the bridge (0 to 8 meters) and open track (greater than 8 meters), suggests the four design/construction features used for this bridge significantly reduced differential track displacements and prevented development of increased applied dynamic loads in the transition zone.

**Site #2**

The second instrumentation site involves a similar freight bridge transition (Figure 7) that supports unloaded and loaded freight trains moving at velocities of 40 km/hr (25 mph), an annual traffic at about 70 MGT (10 times more than Site #1), and the transition also has required minimal track maintenance since construction in 1998 (~17 years). A few notable differences between Site #1 and Site #2 is that Site #2 has accumulated over 1,000 more MGT than Site #1, longer service life of 9 years, uses concrete ties instead of timber ties, and minimal fill was placed before construction so there is a small depth of compacted fill below the track system. Figure 7 shows three of the four design techniques used for Site #2 that also were used for Site #1, i.e., (1) a ballasted concrete bridge deck, (2) a 150 mm (6 in) HMA layer that extends for only 150 m (500 ft.) under 300 mm (12 in) of ballast, and (3) 7.3 meter (24 ft) long concrete wing walls perpendicular to the bridge abutment.

The Site #2 bridge transition zone was also instrumented with eight accelerometers to capture the bridge, approach, and open track behavior. Due to space constraints, only the results of accelerometers located 6.4 m (21 ft.) and 17.5 m (57 ft.) from the bridge abutment are discussed. These two sites will be referenced as Site #2 (21 ft.) and Site #2 (58 ft.) herein and represent the transition and open track responses, respectively.

Figure 8 displays ten (10) seconds of the 165-second long acceleration time histories at Site #2 (21 ft.) and Site #2 (58 ft.). The measured accelerations range from 2 to 3g which suggests the track behavior in the transition zone is also similar to the open track. The results of the other six accelerometers show similar results except for an accelerometer located near a welded rail joint (12 m/40 ft. from the bridge abutment), which resulted in tie accelerations of about 15g. The good track performance was also evident by little if any track displacement being noticed at any location during train passage. The greater acceleration at the welded rail joint, while located in the open track, also suggests that welded rail joints should not be used in the approach track whenever possible because it can lead to increased dynamic loads, increased track displacement, and formation of gaps that accelerate track degradation.

The performance of Sites #1 and #2 show a strong correlation between small transient and permanent displacements and minimal need for track resurfacing. This relation is expected because the small magnitudes of transient displacement in the approach imply a smooth load transfer between the rail, tie pads, ties, and ballast which prevents the initiation of increased dynamic loads. The lack of dynamic loads prevents the track degradation process from continuing and allows the track to maintain track geometry for extended periods of time, i.e., 6 and 17 years, respectively, for Sites #1 and #2.

## ALTERNATIVE DESIGNS

The design techniques presented in the previous section appear to have balanced the transient and permanent displacements between the approach and bridge as suggested by the Transition Balance Sheet because the track geometry and tie accelerations do not show increased dynamic loads at Sites #1 and #2 (*24*). However, these design and construction techniques are not the only available techniques for balancing the displacements between the approach and bridge and may not be the most cost-effective solution for every situation.

Having the ability to choose from a wide range of transition zone designs is beneficial because it allows for cost-effective and site-specific solutions (*2,3,4,6,14*). For example, the use of wedge-shaped backfills in a transition zone in Portugal has shown promising results by incrementally increasing the track stiffness to match the bridge (*6*). Conversely, some ballasted-deck bridges in the United States have also installed rail pads and/or ballast mats to further decrease bridge stiffness and increase bridge displacements (*14*). Under-tie pads in the approach have been attempted in Europe and are being explored in the United States as well.

## BALLAST SETTLEMENT

An ideal transition should eliminate all differential transient and permanent movements but accomplishing this difficult and probably not cost-effective. In these cases, focusing on decreasing the stiffness of the bridge and reducing settlement within the approach ballast layer appear to be the most effective alternatives for reducing the increased applied dynamic loads in the transition because these two sources of displacement contribute the greatest differential movements in the Transition Balance Sheet. As a result, this section reviews some factors that cause ballast degradation and settlement along with suggestions for increasing ballast life to reduce ballast displacements.

One of the main components of ballast settlement is fouling of the ballast due to breakdown of ballast particles and infiltration of fines from external sources. Ballast particles can breakdown from repeated train loadings and mechanical tamping (*25,26*) so a strong rock, e.g., basalt or granite, should be used for ballast. Fouling can also occur due to fine infiltration from the subgrade, train cars, degraded ties, and wind-blown sediment (*25*) and this change in gradation changes the strength, stiffness, and drainage properties of the ballast (*16,17,27*). Laboratory and field testing of fouled ballast show increased settlement and decreased stiffness, i.e. modulus, when fouled ballast is wetted (*28*). This suggests solely focusing on eliminating differences in track stiffness may be beneficial until ballast degradation/fouling starts to occur. At that point, the approach track stiffness and settlement start to change and can initiate the track degradation process because of the factors mentioned in previous sections. Efforts to clean and properly drain the ballast can prevent the negative effects of fouling and confining the transition zone with concrete wing walls perpendicular to the abutment can reduce settlement.

Stiff ties and large ballast particles also can cause the tie to unevenly distribute load to the ballast. Field measurements of the tie-ballast stress distribution performed by McHenry et al.

(*29*) show a 10 to 20% average contact area for new ballast and about 30 to 40% for highly degraded ballast. This low contact area for new ballast results in higher local stresses acting on the ballast particles, which can accelerate the ballast degradation process. These high local stresses can be reduced by decreasing the stiffness of the tie using alternative tie material, e.g., timber, or under-tie pads (UTPs). While UTPs lower track stiffness in the approach and result in greater transient displacements, they can provide beneficial effects by reducing stress concentrations and distributing the load along a single tie. Therefore, UTPs can lower local stresses on the ballast, reduce ballast breakdown, and reduce ballast settlement (*30*). To account for the greater transient displacements with UTPs, a slight overlift in the approach may be necessary to minimize increased dynamic loads from rapid changes in axle elevation and/or use of UTPs on the bridge to balance the displacements.

## SUMMARY

Successfully designing and remediating transition zones are difficult tasks because of the multiple factors that can lead to increased applied dynamic loads and track differential movement at the transition. This paper summarizes a few causes of increased dynamic loads in transition zones, presents two examples of successful bridge transition design, and discusses causes of ballast degradation over time and its effect on transition zone performance. A summary of the main findings are:

- Transition zone degradation is often attributed to increased applied dynamic loads due to: (1) rapid changes in axle elevation, (2) load redistribution, (3) impact loads, and (4) high stiffness and low damping of the bridge. Increased ballast settlement from wet, fouled ballast is also a contributing factor.

- To avoid increased dynamic loads, transition design should balance transient and permanent track displacements between the bridge approach and abutment.

- Two bridge transition zones that have performed successfully show the use of a ballasted bridge deck, HMA ballast underlay, and concrete wing walls that extend perpendicular to the bridge abutment can minimize differential moment between the bridge, approach fill, and open track. The ballasted bridge deck decreases bridge stiffness and allows greater transient and permanent displacements on the bridge to balance the approach displacements. The HMA underlay helps distribute stresses in the approach, confine the ballast, and prevent infiltration between the ballast and subgrade. The perpendicular concrete wing walls help confine the ballast and reduce ballast settlement in the approach.

- Constructing approach fills well in advance of bridge construction allows the fill to undergo infiltration and hydrocompression, which removes fill settlement prior to track construction. However this delay in constructing the track system is usually not practical for railroads. Other alternatives include placing the approach fill material wet-of-optimum or using a granular fill with a vegetative soil cover to prevent erosion of the granular fill.

- Solutions such as smoothing track stiffness between the approach and bridge may not be effective for bridge transitions because: (1) other factors, such as, differential settlement and load redistribution, can increase applied dynamic loads to a greater degree than differences in track stiffness, (2) track stiffness is largely influenced by construction and maintenance practices not design, and (3) ballast and track degradation will occur with time causing changes in track and ballast stiffness. This makes it difficult to develop an all-encompassing solution that is flexible for the range of field conditions, construction and maintenance practices, and ballast degradation processes that are usually present. In summary, focusing on reducing and balancing bridge stiffness and ballast settlement is recommended.

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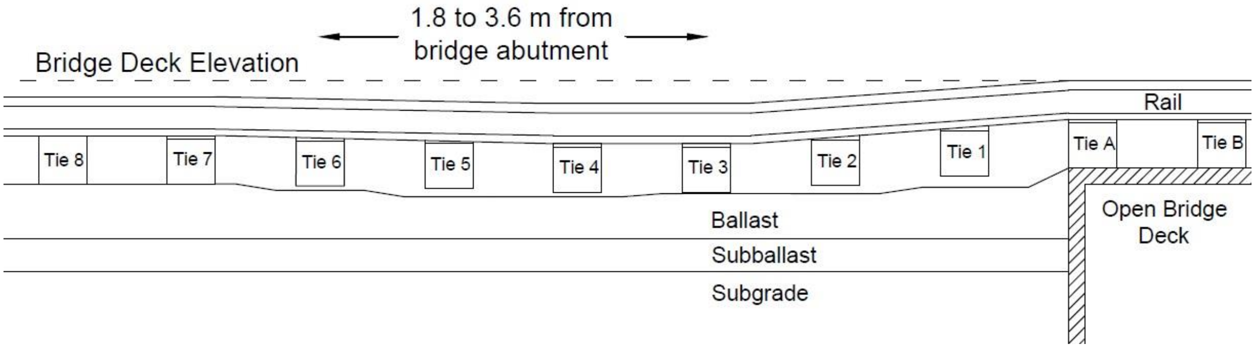


**FIGURE 1 Example diagram of bridge transition zone with various approach tie-ballast gaps**

| Approach and Bridge Displacement Component | Potential Transient A | Potential Transient B | Potential Permanent A | Potential Permanent B |
|---|---|---|---|---|
| Rail compression | ✓ | ✓ | | |
| Rail-tie gap | ✓ | | | |
| Tie pad/plate displacement | ✓ | ✓ | | |
| Tie displacement | ✓ | ✓ | | |
| Tie-ballast gap | ✓ | | | |
| Ballast displacement | ✓ | | ✓ | ✓ |
| Subballast displacement | ✓ | | ✓ | |
| Subgrade displacement | ✓ | | ✓ | |
| Lateral displacement | ✓ | | ✓ | |

**FIGURE 2 Transition Displacement Balance Sheet for an open-deck bridge to compare Approach (A) and Bridge (B) transient and permanent displacements to aid bridge design and remedial measures.**

**FIGURE 3 Miniature accelerometer attached to concrete tie.**


**FIGURE 4 West End of Site #1 Bridge Transition Zone.**

| Approach and Bridge Displacement Component | Potential Transient A | Potential Transient B | Potential Permanent A | Potential Permanent B |
|---|---|---|---|---|
| Rail compression | ✓ | ✓ | | |
| Rail-tie gap | | | | |
| Tie pad/plate displacement | ✓ | ✓ | | |
| Tie displacement | ✓ | ✓ | | |
| Tie-ballast gap | | | | |
| Ballast displacement | ✓ | ✓ | ✓ | ✓ |
| Subballast displacement | | | | |
| Subgrade displacement | | | | |
| Lateral displacement | | | | |

**FIGURE 5 Transition Balance Sheet to compare Approach (A) and Bridge (B) transient and permanent displacements for Site #1.**

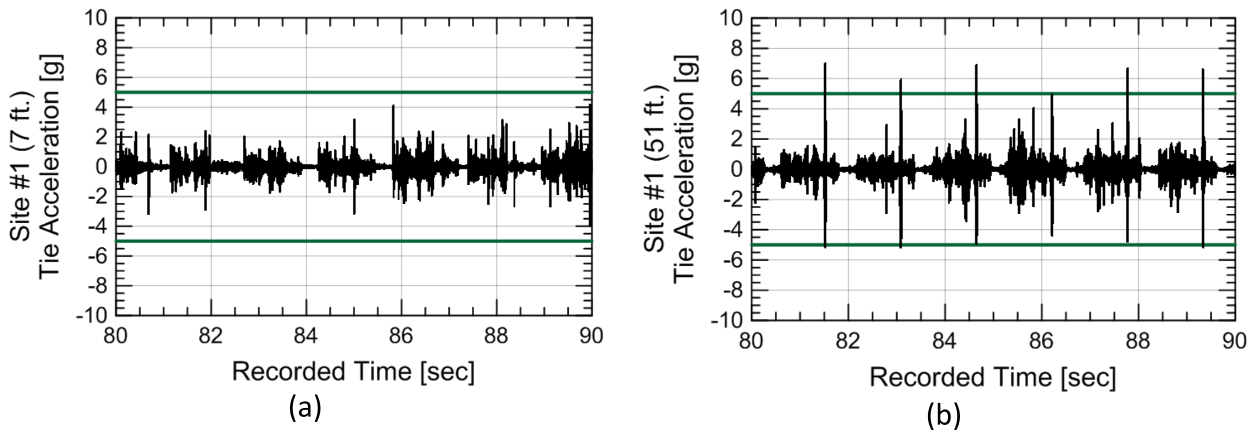


**FIGURE 6 Measured tie accelerations at (a) Site #1 (7 ft.) and (b) Site #1 (51 ft.) for a passing freight train on 12 June 2014.**


**FIGURE 7 West End of Site #2 Bridge Transition Zone.**

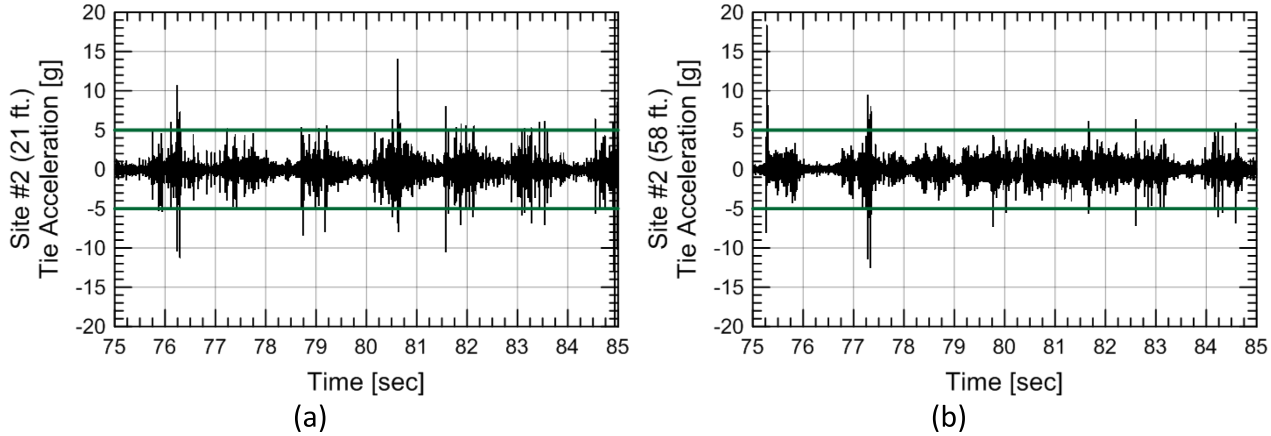


**FIGURE 8 Measured tie accelerations at (a) Site #2 (21 ft.) and (b) Site #2 (58 ft.) for a passing freight train on 28 July 2014.**