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- **Section 1**, 25 pages, titled “Monitoring of Well-Performing Bridge Transition Trackbeds Using Accelerometers”, written by Dr. Jerry G. Rose, Dr. Timothy D. Stark, Stephen T. Wilk and Macy L. Purcell

- **Section 2**, 32 pages, titled “Evaluating the Effects of Variable Tie Support at Railway Bridge Transitions”, written by Stephen T. Wilk, Dr. Timothy D. Stark and Dr. Jerry G. Rose

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Monitoring of Well-Performing Bridge Transition Trackbeds Using Accelerometers

By

Dr. Jerry G. Rose, Ph.D., P.E.
Professor
Civil Engineering
University of Kentucky
jerry.rose@uky.edu

Dr. Timothy D. Stark, Ph.D., D.GE
Professor
Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
tstark@illinois.edu

Stephen T. Wilk, EIT
Graduate Research Assistant
Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
swilk2@illinois.edu

Macy L. Purcell, EIT
Graduate Research Assistant
Civil Engineering
University of Kentucky
macypurcell@uky.edu

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DISCLAIMER

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Title
Monitoring of Well-Performing Bridge Transition Trackbeds Using Accelerometers

Introduction
This report presents a review of railroad trackbed transition designs that have performed well, e.g., ballasted bridge decks, hot-mixed asphalt (HMA) sublayer, and concrete wing walls parallel to the track, to guide future design and maintenance of bridge trackbed transitions.

Approach and Methodology
Non-invasive techniques, e.g., miniature accelerometers attached to ties, were used to measure and evaluate the response of well-performing trackbed transitions under revenue traffic at two railroad bridges. The results of the tests were related to the composition and design of the trackbed support within the transition areas.

Findings
Instrumentation of three bridge transition zones with no history of track geometry problems resulted in the following main findings:

Bridge transition zone design should limit all possible differential transient and permanent displacements between the transition zone and bridge. For these three transitions, this includes using: ballasted bridge deck, HMA underlayment in the transition zones, and concrete wing walls parallel to the track. However, other solutions may accomplish the same results.

Well-performing bridge transitions display tie accelerations below 5 g and similar behavior in the transition zone and open track. In contrast, transition zones experiencing track geometry problems consistently exhibit tie accelerations above 10 g. Similar behavior was observed for both timber and concrete ties along with silty loam and clayey subgrades.

Accelerometers can detect poorly supported ties by identifying the first four vibration modes in the frequency domain.
Conclusions
The results show well-performing trackbed exhibits tie accelerations of 5 g or less with little difference between: (1) the bridge deck, approach embankment, and open track, (2) concrete and wood ties, and (3) clayey or silty subgrades. The measured transient vertical tie displacements are negligible, which verifies the observed good track support. The results from these two sites are being used as a control for comparison with poorly-performing bridge transitions, track defects, e.g., broken ties, rail-fastener gaps, fouled ballast, broken rail, and to verify the success of remedial measures.

Recommendations
Future work will include incorporating high-speed video cameras in the non-invasive monitoring system to measure transient and permanent displacement of the rail and tie. This expands the ability of the monitoring system to quantitatively measure the rail-fastener and tie-ballast gaps which are often prominent at bridge transitions experiencing reoccurring track geometry problems.

Publications

Primary Contact
Principal Investigators
Dr. Timothy D. Stark, Ph.D., D.GE
Professor
Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
(217) 333-7394
tstark@illinois.edu

Stephen T. Wilk, EIT
Graduate Research Assistant
Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
(217) 333-7394
swilk2@illinois.edu
Other Faculty and Students Involved

Dr. Jerry G. Rose, Ph.D., P.E.
Professor
Civil Engineering
University of Kentucky
(859) 257-4278
jerry.rose@uky.edu

Macy L. Purcell, EIT
Graduate Research Assistant
Civil Engineering
University of Kentucky
(270) 589-7755
macypurcell@uky.edu

NURail Center
217-244-4999
nurail@illinois.edu
http://www.nurailcenter.org/
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SECTION 1 INTRODUCTION

Differential movement and track geometry issues at railway bridge transition zones are a reoccurring maintenance issue for railroads [1-10]. This differential movement is largely due to the rapid change from a soft deformable earth substructure to a rigid, essentially non-deformable bridge structure, with the substructure in the transition zone able to settle while little or no settlement occurring on the bridge deck. This can produce gaps within the track system, i.e. rail-fastener and tie-ballast gaps, because the rail and connecting ties in the transition zone will be held up by or cantilevered off the higher elevation bridge deck [10-13]. These tie-ballast gaps can increase the applied loads on the ballast from impact of the moving tie contacting the ballast and load redistribution from the additional rail bending required to close the tie-ballast gap, which causes the rail to distribute some additional load to adjacent and better supported ties [9,14].

In addition, when the passing wheel enters the bridge deck a significant increase in load can occur if the top-of-rail (TOR) elevation in the approach is below the bridge deck. If transient differential movement results in the approach TOR being below the bridge deck TOR, the increased load from the wheel hitting the deck can reach 400% [3]. If the TOR elevation is equal or constant from the transition zone to the bridge deck, the increased load is usually less than 50% because the upward wheel acceleration is relatively small [2-3, 15]. Increased loads from track system gaps and the associated differential settlement causes a greater difference in TOR elevation between the approach and bridge deck, which results in greater loads and more settlements and a continually deteriorating system.

To prevent this continuing differential movement problem at bridge transition zones, sudden changes in wheel elevation must be prevented at both the transient, i.e. track stiffness, and permanent, i.e. settlement, levels. The factors producing these rapid changes in wheel elevation, e.g. tie-ballast gaps, rail-fastener gaps, poor subgrade, etc., are often site specific with multiple factors operating at each site. This implies that a single remedy, e.g., stronger ballast, will not likely solve the problem [1] so the entire track system must be considered.

This report presents the response of two bridge transitions that have not experienced problematic differential movement at the bridge transition since construction along with a non-invasive monitoring system involving accelerometers installed on various crossties that provides an insight to track performance. This in-track data is used to understand the design features resulting in a well-performing bridge transition and illustrates the response of well-performing track transitions for comparison with poorly-performing transitions.
2.1 Philosophy

Development of successful bridge transitions designs has proven difficult for both highway and railway applications [1-10, 16-17]. Specific reasons for differential movement are often site dependent and involve multiple factors which prevent a single design or remedial technique to be consistently successful [1]. However, the main objective of bridge transition design is to minimize differential transient and permanent displacements between open track, transition zone, and bridge deck. The inherent difficulty in this objective is the open and bridge transition zone track lie on deformable earth materials while the bridge deck lies on an essentially non-deformable structure.

Historically, the focus of bridge transition design and remediation involved minimizing the difference in track stiffness between the transition zone and bridge deck [1, 4, 18]. This may involve a “smooth” increase in stiffness along the transition zone or reduction of the stiffness of the bridge using rubber pads or plastic ties [4-5, 8, 18-19]. One shortfall of this philosophy is a “smooth” track stiffness may address the differential transient movement of the track but it does not address the permanent movement of the transition zone substructure while the bridge deck remains essentially fixed [1, 11]. On the other hand, if only the permanent movement is addressed, then impact loads from differential transient movement can initiate a self-perpetuating cycle of track degradation.

Because of the discrete nature of the track system, a list of movements leading to differential transient and permanent movements between the approach and an open deck bridge was developed. While some of these factors are often negligible, the goal is to create a comprehensive list which addresses all possible transition differential movements to understand the system behavior. The approach factors include:

(1) rail displacement,
(2) rail-fastener gap,
(3) fastener displacement,
(4) tie displacement,
(5) tie-ballast gap,
(6) ballast displacement,
(7) subballast displacement,
(8) subgrade displacement, and
(9) vertical displacement of the substructure from lateral movement.

If an open deck bridge is used with this approach, the list of factors contributing to transition differential movements of the bridge deck include:

(1) rail displacement,
(2) rail-fastener gap,
(3) fastener displacement,
(4) tie displacement, and
(5) bridge deck displacement.

This implies differential transient and permanent movement will occur unless the bridge deck transient and permanent displacements are equal to the cumulative displacement of the tie-ballast gap, tie-rail gap, ballast displacement, subballast displacement, subgrade displacement, and vertical displacement of the substructure from lateral movement.

The large number of factors leading to both transient and permanent movement shows the need for multiple design and remedial techniques to be considered for a single transition zone. The difficulty in predicting these displacements and implementing these different stiffnesses helps explain the frequent poor performance of previous design and remediation techniques that have utilized only a single solution, e.g., cemented backfill, HMA, or geocells [1].

In addition to the mechanical properties of bridge transition zones, proper drainage and constructability are also imperative for any good transition design and remediation. While these factors are not specifically addressed in this report, they are inherently considered in the recommendations.

2.2 Site 1 Bridge (West End)

The first monitored bridge transition zone is a ballasted concrete bridge deck, 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, with concrete wings walls perpendicular to the bridge abutment and extending approximately 16 ties (25 feet) from the abutment. The track structure is supported by a compacted earth fill about 23 m (75 feet) high and unsupported on the north side. At first glance, this transition appears to be a good candidate for a poorly-performing bridge transitions because of the large fill height and heavy volume and weight of traffic [16-17] but it is performing extremely well. The reason for the minimal subgrade displacement is because the fill was placed five (5) years before track construction, allowing the fill to consolidate and withstand the high self-weight and train loadings. Figure 1 presents a photograph of the west transition zone.

The bridge design features are important because they have limited the differential transient and permanent movement of the bridge transition zone. The ballast concrete deck bridge adds (5) tie-ballast gap and (6) ballast displacements on the bridge while the HMA limits (6) ballast displacement and the influence of (7) subballast displacement and (8) subgrade displacement in the transition zone [20]. The concrete wing walls parallel to the track also confine the transition zone and limit the (9) vertical displacement of the substructure from lateral movement.
The traffic across the transition consists of both empty and loaded freight trains that pass over the bridge moving at approximately 40 km/hr (25 mph) and the track is considered Class 3 for operations. During train passage, the ties did not visually move much and no track geometry problems have arisen since the bridge was placed in service in 2009.

2.3 Site 2 Bridge (West and East End)

The second and third monitored bridge transition zone are the opposite ends of a bridge within the United States. Similar to the previous example, the west transition zone consists of a ballasted concrete deck bridge, concrete ties, a 150 mm (6 inch) hot-mixed-asphalt (HMA) layer underneath a 300 mm (12 inch) thick layer of ballast in the approach, with concrete wings walls parallel to the track or perpendicular to the bridge abutment and extending approximately 13 ties (24 ft.) from the abutment. A photograph of the transition zone is shown in Figure 2.

The east transition zone only has a single concrete wing wall on the south end extending 9 ties (17 ft.) with just an embankment on the north end. The concrete bridge deck is ballasted and the approach also has an HMA layer underneath the ballast. The subgrade of the east transition zone is silty loam compared to a clayey subgrade for the west transition zone.

The traffic consists of both empty and loaded freight trains that pass over the bridge moving at approximately 40 km/hr (25 mph) and the track is considered Class 3 for operations. During train passage, the ties did not visually move much and no track geometry problems have arisen since the bridge was placed in service in 1998.
Figure 2. West End of Site 2 Bridge Transition Zone
The instrumentation used for these three transition zones consists of eight accelerometers that were placed within the bridge, transition zone, and open track. After considering a wide variety of instrumentation techniques, accelerometers were selected for data collection and track assessment because they provide an inexpensive, quickly installed, non-invasive, durable, and reusable means to non-invasively evaluate track behavior by measuring tie acceleration time histories. 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 drop of superglue or epoxy. This results in a quick and non-invasive monitoring system that does not interfere with train operations. This 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 because weather resistant accelerometers are available. A photograph of an accelerometer is shown in Figure 3.

![Accelerometer Installed on Concrete Tie at Site 2](image)

**Figure 3. Miniature Accelerometer Installed on Concrete Tie at Site 2**

Acceleration time histories are beneficial because they provide insight to the increased loading on the tie bottom and top of ballast especially if a tie-ballast gap is present. Higher tie accelerations result in higher impact forces on the bottom of the tie and top of the ballast because Newton’s Second Law states applied force (F) equals the mass (m) times acceleration (a). The acceleration time history can be converted to the frequency domain to determine the dominant frequencies of the tie deflection-vibration response which gives insight to tie support conditions that can influence different vibration modes [21-24]. While
support conditions were the motivation for using accelerometers, tie accelerations also can be used to investigate the impact of damaged ties, fouled ballast, moisture conditions, wheel-rail impacts, rail and wheel defects, and substructure support on track performance.

While the accelerometers do not quantitatively measure the displacement caused by all of the factors listed above, which would require an expensive and time-consuming instrumentation setup, accelerometers provide a measurement of the track movement as an entirety. Therefore, if the track is moving at some discrete location, e.g. tie-ballast gap or substructure, the accelerometers measure this movement and further analysis can then identify or locate the problematic region.

Future instrumentation includes measuring both rail and tie displacement time histories with high-speed video cameras. This allows for the measurement of both the (2) rail-fastener and (5) tie-ballast gaps and give better insight into the overall track performance.
SECTION 4 INSTRUMENTATION RESULTS

4.1 Site 1 Bridge (West End)

The Site 1 Bridge was instrumented with eight accelerometers on 12 June 2014 to non-invasively evaluate track performance. The eight accelerometers were installed on ties within different regions on the bridge, transition zone, and open track to compare the track behavior at various locations. For this paper, only the results of two accelerometers are presented to display the difference between the bridge transition and open track behavior during passage of an unloaded freight train. Accelerometer #3 is located 2.1 m (7.1 feet) from the bridge abutment and Accelerometer #8 is located 15.4 m (51 feet) from the bridge abutment. These two sites were selected to compare the representative transition zone and open track behavior and will be referenced as Site 1 (7 ft.) and Site 1 (51 ft.) herein.

Figure 4 displays the entire time history of Site 1 (7 ft.) which shows most acceleration magnitudes of only 1 to 2 g but with a few larger acceleration spikes (>50g), which is the response from passing wheel flats. The sudden impact of the wheel defect on the rail can produce large magnitude but short duration tie accelerations. The magnitude and direction of the acceleration can vary depending on multiple factors such as location of impact on the rail, impact of the near or far rail, damping characteristics within the rail, fastener, tie, and ballast, continuity of the rail, fastener, and tie, and finally tie integrity.
Figure 5 displays only 10 out of the 280 seconds of Site 1 (7 ft.) and (51 ft.) to emphasize a few details of the time history. At both sites, the acceleration magnitudes range from 1 to 2g which is representative of good track performance in the transition zone and is similar to open track behavior. The six other accelerometers exhibited similar behavior. The good track performance is verified with visual monitoring of track displacements where almost no track displacement was recorded by video cameras at any location within the bridge transition or open track. Reasons for the good transition zone behavior include the ballasted bridge deck, HMA underlayment, and confinement of the transition materials by concrete wing walls parallel to the track, which limit the differential transient movement between the bridge, approach, and open track.

Figure 5. 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 6 displays both the Site 1 (7 ft.) and Site 1 (51 ft.) acceleration time histories in the frequency domain. As expected, the results are similar, which comports with no significant differences in open track and transition performance being observed.

![Figure 6. Measured Tie Accelerations in Frequency Domain at Site 1 (7 ft.) and Site 1 (51 ft.) for a Passing Freight Train on 12 June 2014](image)

**4.2 Site 2 Bridge (West End)**

The west Site 2 bridge transition zone was instrumented with eight accelerometers on 28 July 2014. The accelerometers were installed on ties within various regions within the bridge, transition zone, and open track to compare track behavior at different locations. Only the results of Accelerometers #5 and #8 are presented for brevity with Accelerometer #5 being located 21 ft. (6.35 m) from the bridge abutment and Accelerometer #8 being located 57 ft. (17.5 m) from the bridge abutment. These two sites will be referenced as Site 2 West (21 ft.) and Site 2 West (58 ft.) herein.

Figure 7 displays ten (10) seconds of the acceleration time histories at both locations. The measured acceleration magnitudes 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 joint which resulted in tie accelerations of about 15g. The good track performance is again verified with visual monitoring of track displacements where almost no track displacement was noticed at any location within the bridge transition or open track.

Figure 8 displays both full time histories in Figure 7 in the frequency domain. The results are similar and no significant differences are observed. The dominant frequency for both ties appears to be around 175 or 180 Hz. This is likely a loading frequency, e.g. frequency of
train loading, or tie vibration influenced from the rail and fasteners because the first vibration mode of concrete ties usually resides within the 100 to 150 Hz range [21-24].

![Image](image1.png)

**Figure 7. Measured Tie Accelerations in Frequency Domain at (a) Site 2 West (21 ft.) and (b) Site 2 West (58 ft.) for a Passing Freight Train on 28 July 2014**

![Image](image2.png)

**Figure 8. Measured Tie Accelerations in Frequency Domain at Site 2 West (21 ft.) and (58 ft.) for a Passing Freight Train on 28 July 2014**

4.3 Site 2 Bridge (East End)

The opposite end of the Site 2 Bridge was also instrumented on 28 July 2014 but with six accelerometers due to limited track time. The accelerometers were installed on ties within the bridge, transition zone, and open track to compare behavior at various locations. The primary difference in bridge design between the west and east transition zones is the east transition zone has a single shorter wing wall (9 ties, 17 ft.) on the south side and just an embankment on the north side of the transition zone. A second difference is the isolated poorly supported tie 1.4 m (5 ft.) from the bridge abutment. During passage of a train, the tie would displace to establish contact with the ballast, which also resulted in an upward displacement of the first tie on the bridge. Accelerometers #3 and #4 were installed on opposite ends of this tie to measure the behavior of an isolated poorly supported tie.
Figure 8 displays the tie acceleration time histories of a passing freight train at 5 ft. (Site 2 East 5 ft.) and 20 ft. (Site 2 East 20 ft.) and Figure 9 displays the same time histories in the frequency domain. While the acceleration magnitudes from the time histories of the two accelerometers are similar (~3 to 5g), the behavior in the frequency domain is significantly different. The supported tie (Site 2 East 20 ft.) shows only a single dominant frequency of vibration of about 110 Hz, which is the first vibration mode of a concrete tie [21-24]. The poorly supported tie (Site 2 East 5 ft.) shows four dominant frequencies of vibration at 110 Hz, 300 Hz, 585 Hz, and 900 Hz, which are the first four vibration modes of concrete ties [21-24]. The additional vibration modes in the poorly supported tie are explained by the lack of damping and confinement from the ballast which allows the concrete tie to freely “ring” during every wheel loading. This also shows how accelerometers can be used to identify poorly supported concrete ties.

Figure 9. Measured Tie Accelerations in Frequency Domain at (a) Site 2 East (5 ft.) and (b) Site 2 East (20 ft.) for a Passing Freight Train on 28 July 2014

Figure 10. Measured Tie Accelerations in Frequency Domain at Site 2 East (5 ft.) and (20 ft.) for a Passing Freight Train on 28 July 2014
SECTION 5 FINDINGS AND RECOMMENDATIONS

The instrumentation of three bridge transition zones with accelerometers shows good transition zone performance due to the inclusion of a ballasted bridge deck, HMA underlayment, and parallel concrete wing walls in the transition zone design. These features help reduce the differential transient and permanent displacement from the bridge transition zone and the bridge deck, which prevents initiation of the self-perpetuating cycle of transition zone degradation.

Both transition zones and open deck bridges experience displacements within the (1) rail, (2) rail-fastener gap, (3) fastener, and (4) tie. However, the ballast bridge deck adds (5) tie-ballast gap and (6) ballast displacement to the bridge displacement while the HMA underlayment and concrete wing walls limit the formation of (2) rail-fastener gaps and (5) tie-ballast gaps and reduce the (6) ballast displacement, (7) subballast displacement, (8) subgrade displacement, and (9) substructure displacement from lateral movement. These three features are not the only solutions that can limit all potential differential movements.

The non-invasive monitoring system verified the good track performance by showing all tie accelerations below 5g. Tie accelerations below 5g indicate well-performing track while values larger than 10g are more common at problematic or poorly-supported locations [25-26]. A primary feature of good transition zone performance is similar track behavior within the open track, transition zone, and bridge. Instrumented transition zones that experience reoccurring track geometry problems tend to have significantly different open track and transition zone behavior with the measured transition zone tie accelerations surpassing 10g, e.g., 40 g [25-26]. Other observations include similar behavior for both timber and concrete ties (Site 1 v. Site 2) and different subgrade material (Site 2 West v. Site 2 East).

Instrumentation of the poorly supported tie at Site 2 East (5 ft.) shows that accelerometers can detect the “ringing” of poorly supported concrete ties by their dominant frequency vibrations in the 2nd, 3rd, and 4th modes of vibration [21-24]. This also shows that a single poorly supported tie does not necessarily lead to track geometry problems even though the load is redistributed to the surrounding adjacent ties [18]. Transition zones experiencing track geometry problems often display poor tie support along a stretch of ties not just a single tie. This region of poor tie support can increase the applied loads to an extent that initiates a self-perpetuating cycle of transition zone degradation.
SECTION 6 SUMMARY AND FUTURE WORK

Instrumentation of three bridge transition zones with no history of track geometry problems has resulted in the following main findings:

- Bridge transition zone design should limit all possible differential transient and permanent displacements between the transition zone and bridge. For these three transitions, this includes using: ballasted bridge deck, HMA underlayment of the ballast, and concrete wing walls parallel to the track. However, other solutions may accomplish the same results.

- Well-performing bridge transitions display tie accelerations below 5g and similar behavior in the transition zone and open track. In contrast, transition zones experiencing track geometry problems consistently exhibit tie accelerations above 10g. Similar behavior was observed for both timber and concrete ties along with silty loam and clayey subgrades.

- Accelerometers can detect poorly supported ties by identifying the first four vibration modes in the frequency domain.

Future work includes incorporating high-speed video cameras in the non-destructive monitoring system to measure transient and permanent displacement of the rail and tie. This expands the ability of the monitoring system to quantitatively measure the rail-fastener and tie-ballast gaps which are often prominent at bridge transitions experiencing reoccurring track geometry problems.
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Evaluating the Effects of Variable Tie Support at Railway Bridge Transitions

By

Stephen T. Wilk, EIT
Graduate Research Assistant
Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
swilk2@illinois.edu

Dr. Timothy D. Stark, Ph.D., D.GE
Professor
Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
tstark@illinois.edu

Dr. Jerry G. Rose, Ph.D., P.E.
Professor
Civil Engineering
University of Kentucky
jerry.rose@uky.edu

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Title
Evaluating the Effects of Variable Tie Support at Railway Bridge Transitions

Introduction
This report 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.

Approach and Methodology
The primary purpose of this research was to use non-invasive measurement techniques, e.g., accelerometers, to monitor tie support and transient track performance at railway 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 report.

Findings
The results obtained using the in-track 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 fact 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.

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.

Conclusions
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 adequacy 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 camera, lasers, and/or LVDTs to directly measure tie-ballast and rail-tie gaps.

Recommendations
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 subsequent 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.

Publications
Primary Contact

**Principal Investigator**
Stephen T. Wilk, EIT
Graduate Research Assistant
Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
(217) 333-7394
swilk2@illinois.edu

Dr. Timothy D. Stark, Ph.D., D.EG
Professor
Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
(217) 333-7394
tstark@illinois.edu

**Other Faculty and Students Involved**

Dr. Jerry G. Rose, Ph.D., P.E.
Professor
Civil Engineering
University of Kentucky
(859) 257-4278
jerry.rose@uky.edu

**NURail Center**
217-244-4999
nurail@illinois.edu
http://www.nurailcenter.org/
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SECTION 1 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].

This report 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 report.
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 450 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.

**Figure 1.** Subsurface Profile and LVDT Locations 4.6 m (15 ft.) North of Upland Street Bridge in Chester, Pennsylvania.
2.2 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 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 & 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 suit-able 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.
SECTION 3 BRIDGE LOCATION AND INSTRUMENTATION

3.1 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.

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.).

3.2 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
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.
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.](image)

### 3.3 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 s 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.
SECTION 4 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.

4.1 Ballast/LVDT 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 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 (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.”
Figure 4. Recorded Time Histories of the Wheel Load, the Transient Vertical Displacements, and Tie Accelerations at Upland (15 ft.) and Upland (60 ft.) on 1 July 2014
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 (kmob);
- the tie–ballast gap ($8P \frac{1}{4} 0$)

\[ \delta_{LVDT} = \frac{8P}{k} \]
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 (kmob) 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 (8P ¼ 0). The tie–ballast gap is estimated by extrapolating the ballast stiffness to the zero load condition (P ¼ 0) [8].

The load–displacement measurements in Figure 6 show a significant difference in the estimated tie–ballast gap (8P ¼ 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 (8gap) 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 (8gap) will vary after each wheel pass.

![Figure 6. Tie Load-Displacement Behavior at Upland (15 ft.) and Upland (60 ft.) on 1 July 2014](image)

### 4.2 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 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 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 (8P ¼ 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.
Figure 7. (a) Measured Tie Acceleration Time Histories for Upland (15 ft.) and Upland (60 ft.) and (b) Recorded LVDT I 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 ($8P \neq 0$) and Peak Accelerations for Upland (15 ft.) and Upland (60 ft.).

<table>
<thead>
<tr>
<th>Site location</th>
<th>Rate of permanent vertical (mm/year)</th>
<th>Peak transient displacement (mm)</th>
<th>$8P \neq 0$</th>
<th>Peak tie acceleration ($g$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upland (15 ft)</td>
<td>15</td>
<td>7.0</td>
<td>6.7</td>
<td>30</td>
</tr>
<tr>
<td>Upland (60 ft)</td>
<td>0.8</td>
<td>0.4</td>
<td>0.3</td>
<td>&quot;&quot;&quot;&quot;5</td>
</tr>
</tbody>
</table>
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 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 accelerations from the loading of the passing wheelsets are about 2 g to 4 g. Consistent spikes ("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 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) 10s of the Passing Unloaded Freight Train on 12 June 2014

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) 10s of the passing Loaded Freight Train on 12 June 2014
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 ("'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.
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
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.
SECTION 8 SUMMARY

This report 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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