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DESIGN AND MONITORING OF WELL-PERFORMING BRIDGE TRANSITIONS

Dr. Jerry G. Rose, Ph.D., P.E.
Professor of Civil and Environmental Engineering
University of Kentucky
Lexington, Kentucky

Dr. Timothy D. Stark, Ph.D., P.E., D.GE
Professor of Civil and Environmental Engineering
University of Illinois at Urbana-Champaign
Urbana, Illinois 61801

Stephen T. Wilk
Graduate Research Assistant
University of Illinois at Urbana-Champaign
Urbana, Illinois 61801

Macy Purcell
Undergraduate Research Assistant
University of Kentucky
Lexington, Kentucky

ABSTRACT

This paper presents a review of railroad track 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 transitions instead of installing a concept and monitoring its performance. Using non-invasive monitoring techniques, e.g., miniature accelerometers, the performance of the two railroad track transitions has been measured and evaluated. The results show well-performing track exhibits tie accelerations of 5g 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.

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 paper 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 field 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.

BRIDGE TRANSITION DESIGN

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 paper, they are inherently considered in the recommendations.

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.

FIGURE 1 – WEST END OF SITE 1 BRIDGE 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.

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.

FIGURE 2 – WEST END OFSITE 2 BRIDGE TRANSITION ZONE

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.

NON-INVASIVE INSTRUMENTATION

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.

FIGURE 3 – MINATURE 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.

INSTRUMENTATION RESULTS

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.

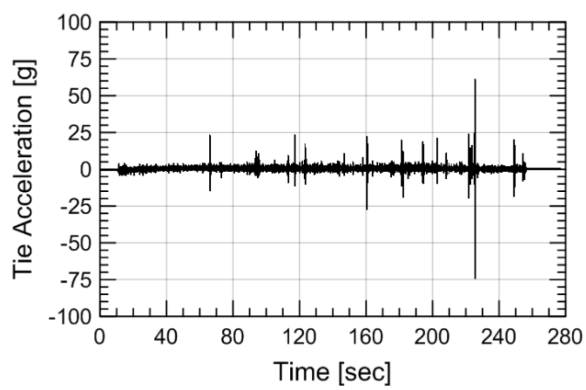
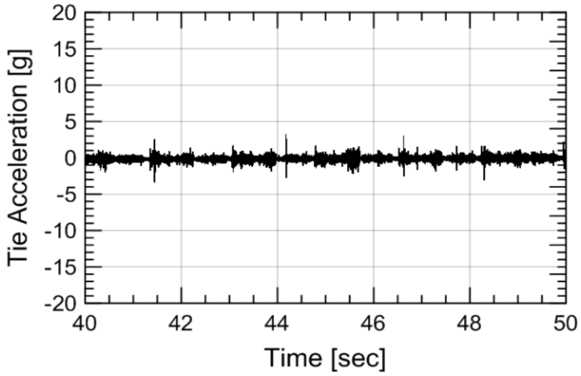
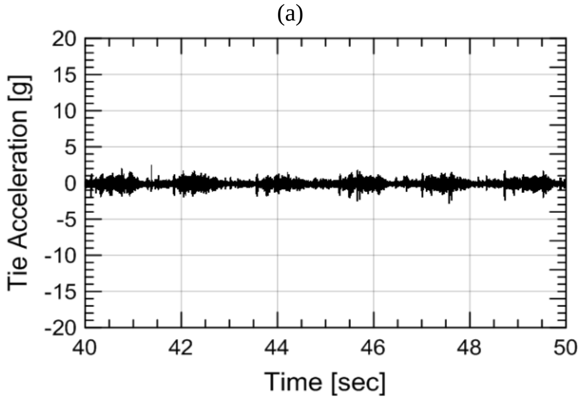


FIGURE 4 – MEASURED TIE ACCELERATIONS AT SITE 1 (7 FT.) FOR A PASSING FREIGHT TRAIN ON 12 JUNE 2014

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.



(a)



(b)

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.

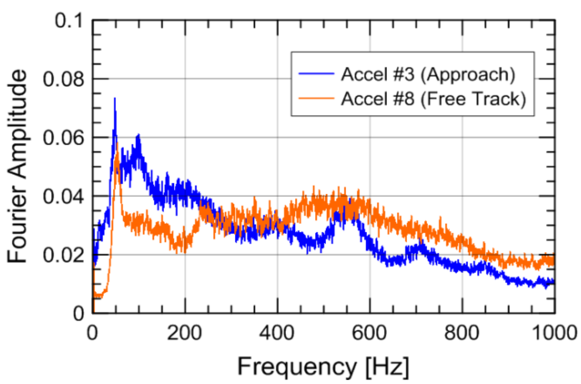
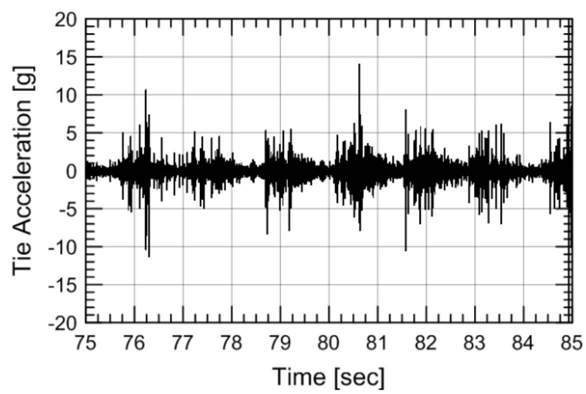


FIGURE 6– MEASURED TIE ACCELERATIONS IN FREQUENCY DOMAIN AT SITE 1 (7 FT.) AND SITE 1 (51 FT.) FOR A PASSING FREIGHT TRAIN ON 12 JUNE 2014

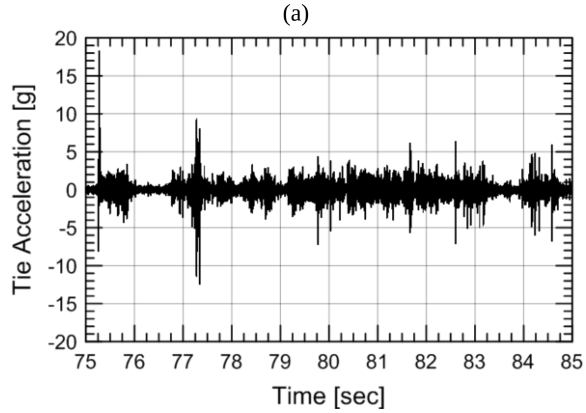
Site 2 Bridge (West End)

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.



(a)



(b)

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

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].

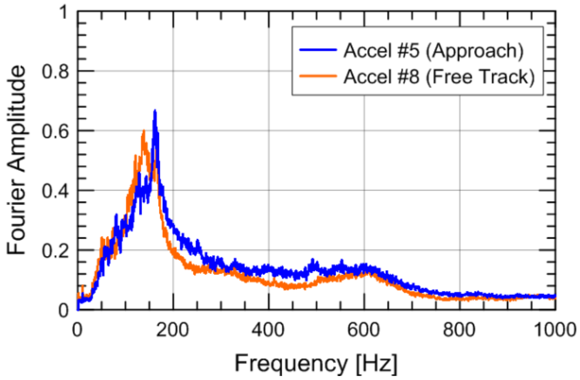


FIGURE 8– MEASURED TIE ACCELERATIONS IN FREQUENCY DOMAIN AT SITE 2 WEST (21 FT.) AND (58 FT.) FOR A PASSING FREIGHT TRAIN ON 28 JULY 2014

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.

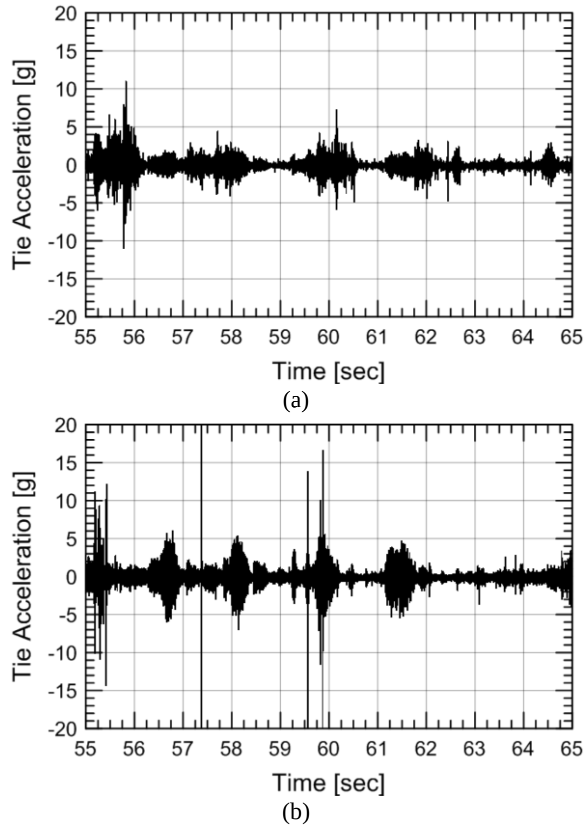


FIGURE 9– MEASURED TIE ACCELERATIONS IN FREQUENCY DOMAIN AT (A) SITE 2 EAST (5 FT.) AND (B) SITE 2 EAST (20 FT.) FOR A PASSING FREIGHT TRAIN ON 28 JULY 2014

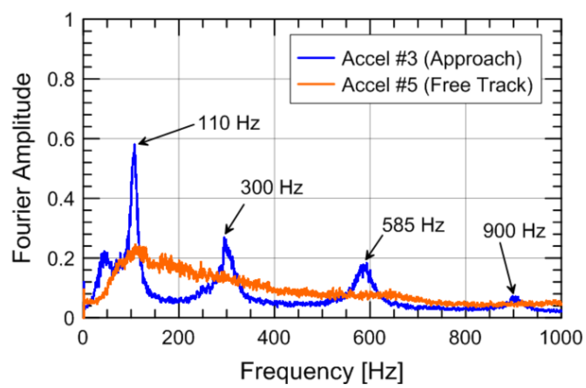


FIGURE 10– MEASURED TIE ACCELERATIONS IN FREQUENCY DOMAIN AT SITE 2 EAST (5 FT.) AND (20 FT.) FOR A PASSING FREIGHT TRAIN ON 28 JULY 2014

FINDINGS AND RECOMENDATIONS

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.

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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