NURail Project
NURail2016-UKY-R11-combined

The final report for NURail project: **NURail2016-UKY-R11** consists of three distinct documents.

- Section 1, 22 pages, titled "Bridge Transition Trackbed Behavior Modifications Using Hand Tamping Techniques", written by Dr. Timothy D. Stark, Stephen T. Wilk, Dr. Jerry Rose and William Moorhead.
- Section 2, 26 pages, titled "Well-Performing Railway Bridge Transitions – Design and Performance", written by Dr. Timothy D. Stark, Stephen T. Wilk and Dr. Jerry Rose.
- Section 3, 22 pages, titled "Vertical Transient Track Displacement Measurements Using Non-Invasive Techniques", written by Stephen T. Wilk, Dr. Timothy D. Stark and Dr. Jerry Rose.

These were completed under grant number: DTRT13-G-UTC52.

National University Rail Center - NURail
US DOT OST-R Tier 1 University Transportation Center

NURail Project ID: 2016-UKY-R11

Bridge Transition Trackbed Behavior Modifications Using Hand Tamping Techniques

By

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

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

William Moorhead
Principal Engineer
TRAMMCO, LLC
trammco@mindspring.com

March 10, 2016

Grant Number: DTRT13-G-UTC52

DISCLAIMER

Funding for this research was provided by the NURail Center, University of Illinois at Urbana - Champaign under Grant No. DTRT12-G-UTC18 of the U.S. Department of Transportation, Office of the Assistant Secretary for Research & Technology (OST-R), University Transportation Centers Program. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the information presented herein. This document is disseminated under the sponsorship of the U.S. Department of Transportation's University Transportation Centers Program, in the interest of information exchange. The U.S. Government assumes no liability for the contents or use thereof.

TECHNICAL SUMMARY

Title

Bridge Transition Trackbed Behavior Modifications Using Hand Tamping Techniques

Introduction

A common maintenance technique to correct track geometry at bridge transitions is hand tamping. This report presents a non-invasive track monitoring system involving high-speed video cameras that evaluates the change in track behavior before and after hand or pneumatic tamping at a bridge transition zone experiencing reoccurring track geometry deviations. The track monitoring shows significant permanent vertical displacement (settlement) in the transition zone during the first few wheel passes after tamping (~0.6 inches) and a return to the pre-tamping transient behavior after about four train passes. This implies that significant differential settlement occurs between the transition zone and bridge abutment immediately after the first passing train which can result in increased dynamic loads in the transition zone and further deteriorate the transition zone geometry. Methods to reduce the initial settlement are discussed.

Approach and Methodology

To investigate how actual transition zone track behavior after tamping compares with the conceptual cycle a non-invasive monitoring system using high-speed video cameras was developed to detect transient rail and tie movement and evaluate overall track performance. This monitoring system is used to compare the transient track behavior at a particular tie before and after tamping and evaluates the amount of settlement that occurs immediately after tamping, i.e.,"compaction" phase.

This report emphasizes the results of the high-speed video cameras and how they can be used to identify track system gaps and evaluate track performance by measuring the rail and the transient time histories. High-speed video cameras were selected because of their mobility, the only contact with the track system is placing removable targets on the rail and tie, and the recorded video provides a visual account of track movement. The cameras typically record at a sampling frequency of 240 frames per second and are capable up to 1000 frames per second which is sufficient to capture and quantify the track movement even for high-speed passenger trains.

Findings

This report reviews the ballast compaction cycle and presents field-measured rail and tie displacement values immediately after the hand tamping of a bridge transition zone. The main findings include:

- Before hand tamping, the bridge transition zone experienced multiple rail-tie and tie-ballast gaps because of the differential ballast settlement between the bridge, transition zone, and open track.

- After hand tamping, the first train pass immediately compacted the ballast resulting in 0.45 inches (11 mm) of rail settlement and 0.7 inches (18 mm) of tie settlement. This occurred because not enough ballast particles were holding up the rail and ties to the specified elevation and therefore immediately "pushed out" during the first train loading.

- The ballast seemed to reach an equilibrium condition after about 4 trains and resulted in 0.5 inches of rail settlement and 0.8 inches of tie settlement with a 0.3 inch gap between the rail and tie. The transient behavior of the tie was very similar to the pre-tamping conditions.

- An "overlift" was used during hand tamping to account for the initial settlement. While it reduced the severity of track geometry deviations after the first few train passes, the significant transient movement, i.e., rail-tie gaps, will still generate increased dynamic loads and accelerate ballast degradation in the transition zone.

- Because of the large initial ballast settlements, emphasizing better ballast compaction and density during tamping of high-maintenance regions such as bridge transition zones may reduce this initial settlement resulting in the track geometry holding for longer time periods and hopefully increases tamping cycles for railroad companies.

Conclusions

The field-measured rail and tie displacement time histories at a bridge transition zone illustrates the significant ballast settlement that can occur immediately after tamping, i.e., "compaction" settlement phase, which caused the track to nearly return to its original transient behavior after a few train passes. This rapid settlement is detrimental to the transition zone because the differential substructure settlement results in the development of rail-tie and tie-ballast gaps as the rail remains supported and cantilevering from the bridge deck. The existence of rail-tie and tie-ballast gaps causes the redistribution of load and impacts which can result in higher local dynamic loads and accelerates the geometric deterioration in the transition zone section

Recommendations

While the problem of differential settlement at transition zones can never be completely eliminated because of the inevitable and sometimes random nature of ballast settlement, there have been many suggestions to reduce track degradation. Many of these options, however,

involve new track design or installing new track components and require significant effort by railroad companies.

Another option would be attempting to limit the amount of initial ballast settlement, i.e., "compaction" phase, by ensuring the ballast is better compacted underneath the tie during tamping. If not, the first passing train will compact the ballast at the expense of track performance. If the settlement in this "compaction" phase is reduced, this will likely result in adequate level of track geometry being maintained for longer periods of time and require less frequent tamping. The additional time and expense to compact the ballast underneath the tie can hopefully extend the tamping cycle by a few weeks or months and may eventually prove cost-effective.

Other possible limitations with the current method of tamping at high-maintenance locations is how new ballast is placed in the crib and squeezes the existing ballast underneath the tie. This implies that the degraded and fouled ballast, which tends to settle at a more rapid rate than clean ballast, will be continually reused directly under the tie and result in further ballast degradation. If tamping methods instead pushed the ballast from one side instead of squeezing from both sides, this "ballast rotation" could extend the life and effectiveness of the ballast.

Publications

Stark, TD, Wilk, ST, Rose, JG, Moorhead, W. Effect of Hand Tamping on Transition Trackbed Behavior, Proceedings of the 2015 American Railway Engineering and Maintenance of Way Association Annual Conference, Minneapolis, MN, October 2015, 10 pages.

Primary Contact

Principal Investigator
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
(217) 333-7394
swilk2@illinois.edu

Other Faculty and Students Involved

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

William Moorhead
Principal Engineer
TRAMMCO, LLC
(757) 356-0616
trammco@mindspring.com

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

TABLE OF CONTENTS

LIST OF FIGURES

SECTION 1 INTRODUCTION

Railroad ballast plays an important role in the track structure and provides four key functions: (1) distributing the axle loads to the subgrade, (2) restraining the track laterally, longitudinally, and vertically, (3) providing adequate drainage, and (4) maintaining proper track cross level, surface, and alinement [1]. As a granular material, the ballast layer eventually settles from repeated loading resulting in track geometry deviations which can increase the dynamic loads and further deteriorate the track geometry [2]. This inevitable process forces railroad companies to resurface the track frequently to maintain desirable track geometric features and allow the ballast to perform the above mentioned functions as intended.

The most common resurfacing technique used in the United States is tamping which essentially involves raising the track to the desired elevation and squeezing the crib ballast from both sides of the tie to fill the space underneath the tie. Tamping has proven to be an efficient and cost-effective resurfacing technique but is effective for only a temporary period of time before resurfacing is required again due to the natural settlement of the ballast from particle rearrangement and breakage [2]. This is especially true in track with abrupt changes in stiffness such as bridge transition zones where differential stiffness, settlement, and damping can result in significantly higher dynamic loads [3-5] which accelerates track degradation and represents a significant maintenance cost for railroad companies in the United States [6].

This report (1) investigates the conceptual ballast behavior during a tamping cycle in a high-maintenance track location such as a bridge transition zone and illustrates certain portions of this cycle with field data and (2) discusses some limitations of the current tamping process along with a few potential ideas to improve tamping in high-maintenance regions where implementing different techniques may prove cost-effective.

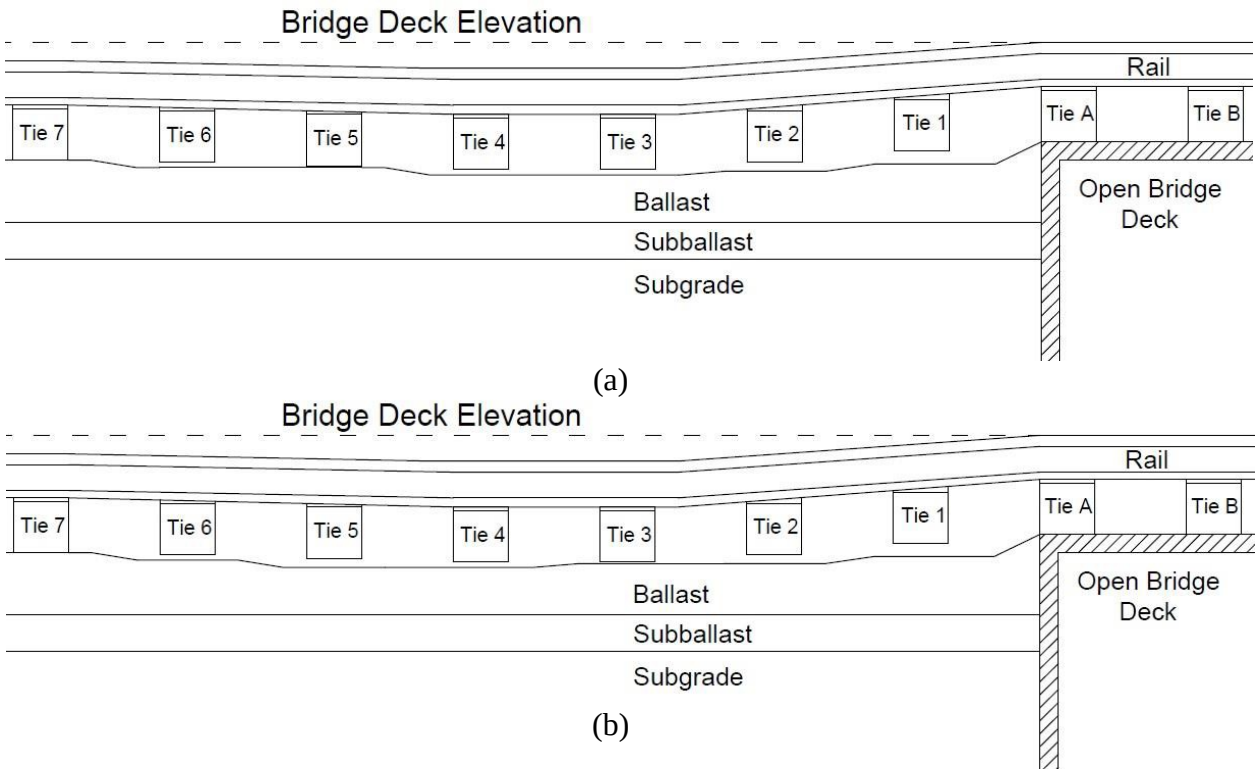
SECTION 2 TAMPING CYCLE AND BALLAST COMPACTION

The primary reason for tamping and resurfacing is the inevitable ballast settlement from particle rearrangement and breakage during repeated axle loadings. While ballast settlement is a continuous process, it has been often split into two distinct phases: (1) the “compaction” phase and (2) “post- compaction” phase [7,8].

The first “compaction” phase occurs immediately after tamping and can involve significant settlement from particle rearrangement as the ballast re-densifies into a more compact state where the magnitude of settlement is typically inversely related to the initial density. The loose ballast condition immediately after tamping is historically well-known from observations of lower track modulus [9] and this initial settlement has been included in many settlement models [8,10]. The second “post-compaction” phase involves a linear or decreasing relationship between settlement and loading and is caused by several mechanisms, including: continued densification of the ballast, infiltration of the ballast in the subballast or subgrade, volume reduction from particle breakdown and abrasion, and lateral or longitudinal movement of the ballast particles [8]. This settlement eventually reduces the ability of the ballast to properly distribute train axle loads and restrain track movement, requiring the track to be resurfaced. If this resurfacing is not performed and the track geometry further deteriorates, the track geometry problem can evolve into a safety issue.

While ballast settlement is undesirable anywhere in railroad track, it is especially problematic at locations where the potential for settlement varies considerably across a short distance such as bridge transition zones or culverts. This is because the rail and ties naturally rest upon the underlying layer, e.g. ballast or bridge deck, but local differential settlement and the high bending stiffness of the rail causes the rail and tie to be supported at the regions that experience the least amount of settlement and hang over regions that experience the greatest amount of settlement, i.e. hanging ties. At bridge transition zones, this leads to the “dip” where the rail is supported from the bridge deck and also out farther in the open track, i.e. 15 or 20 feet out, and results in either rail-tie or tie-ballast gaps in the transition zone. Two examples of tie-ballast gaps in transition zones are displayed in Figure 1 where the rail is supported at the bridge deck (Tie A) and Tie 7 in both situations.

The existence of hanging ties leads to a situation where the train load will not be evenly carried by all ties, e.g. 30 to 50% of the axle load taken by the underlying tie [11], but redistributed which increases the load on surrounding better supported ties [12]. Bridge transition zones with track system gaps are complicated situations because of the variation in track support and rail elevation between the bridge, transition zone, and open track. Therefore, while the load distribution is difficult to quantify, the train load is not expected to be evenly distributed throughout the transition zone but instead concentrated on the few well- or better supported ties in the transition zone. For example, in Figure 1, the ties expected to receive the concentrated dynamic load would be Tie 5 in Figure 1(a) and Tie 3 in Figure 1(b). This concentrated dynamic loading, from uneven ballast settlement in transition zones, will overload the ties and underlying ballast and can lead to the accelerated degradation of the transition zone track. Additionally, this uneven loading will also result in concentrated loading of several end ties on the bridge deck, resulting in crushing and short tie life.



(a)

(b)

Figure 1: Two Example Diagrams of Bridge Transition Zones with Tie-Ballast Gaps.

SECTION 3 INSTRUMENTATION

To investigate how actual transition zone track behavior after tamping compares with the conceptual cycle mentioned in the previous section, a non-invasive monitoring system using high-speed video cameras was developed to detect transient rail and tie movement and evaluate overall track performance. This monitoring system is used to compare the transient track behavior at a particular tie before and after tamping and evaluates the amount of settlement that occurs immediately after tamping, i.e. “compaction” phase.

This report emphasizes the results of the high-speed video cameras and how they can be used to identify track system gaps and evaluate track performance by measuring the rail and tie transient time histories. High-speed video cameras (Figure 2) were selected because of their mobility, the only contact with the track system is placing removable targets on the rail and tie, and the recorded video provides a visual account of track movement. The cameras typically record at a sampling frequency of 240 frames per second and are capable up to 1000 frames per second which is sufficient to capture and quantify the track movement even for high-speed passenger trains.

Figure 2: Photograph of the High-Speed Video Camera.

SECTION 4 INSTRUMENTATION SITE

The instrumented site is a double transition surrounded by an open deck bridge to the north and an asphalt crossing to the south that has developed noticeable rail-tie and tie-ballast gaps from ballast settlement. The site was instrumented on 21 October 2014 and 22 October 2014 with a high-speed video camera that measured the rail and tie displacements of two different tie locations. On the morning of the second day (22 October 2014), the bridge transition zone was hand tamped using a hand tamper. This allowed for the pre- and post-tamping behavior to be measured with a high-speed video camera.

The instrumented site involves the south end of the railway bridge transition zone displayed in Figure 3. The traffic is considered Class 4 for track operations and consists of mixed freight, loaded coal, and intermodal trains passing from 30 to 60 mph. The bridge is roughly a 50 ft. steel open deck bridge constructed in 1923 with few bridge or transition zone design features to minimize differential displacements. The first six ties are longer at 10 feet in length instead of the standard 8.5 foot ties and a timber support connected to the bridge abutment was placed under the first two ties in attempt to limit track settlement but the timber support has become tilted and likely does not receive any load. Nearly every tie in the southern transition zone contains either a rail-tie and/or tie-ballast gap and ballast fouling is prevalent in and around all of the ties. The rail-tie gaps are found within 16 ft. (5 m) of the bridge abutment and tie-ballast gaps are mainly observed 16 ft. (5 m) or more from the abutment. The rail-tie gaps develop from the upward reaction force of the rail after unloading which pulls the spikes from the tie. Because the rail-tie gaps are found within 16 ft. (5 m) of the bridge abutment, this suggests that the upward reaction force is greater within the 16 foot zone than outside it.

Additionally, an asphalt road crossing exists about 70 ft. (20 m) south of the bridge abutment, which results in two transitions in the instrumented region, i.e., a transition from an asphalt crossing to open-track to a steel open deck bridge, and likely blocks drainage. This means that both north and southbound trains experience “galloping” or “bouncing” when passing over the nearby transition zones because of the multiple abrupt changes in track displacements, modulus, and geometry [13].

The two ties of focus in this paper are located 14 ft. (4.3 m) and 20 ft. (6 m) from the bridge abutment because they display different behavior. The first tie (14 ft.) from the bridge abutment displays a rail-tie gap and the second tie (20 ft.) from the bridge abutment displays a tie-ballast gap. A high-speed video camera measures the rail and tie displacements of the east end of both ties.

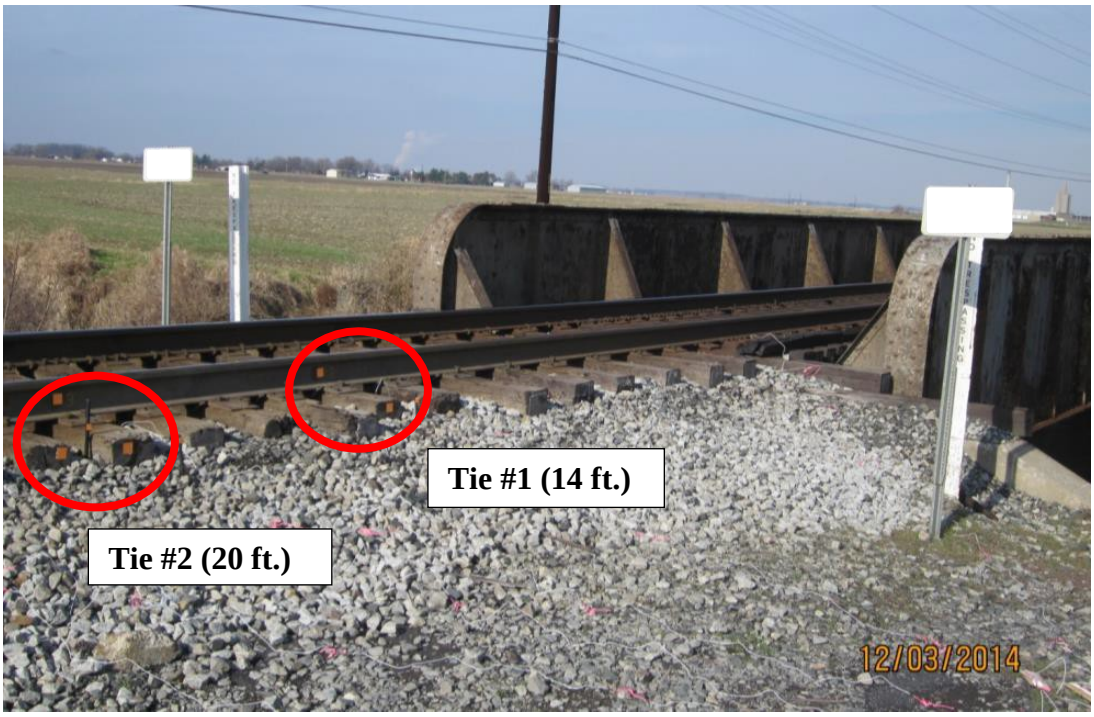


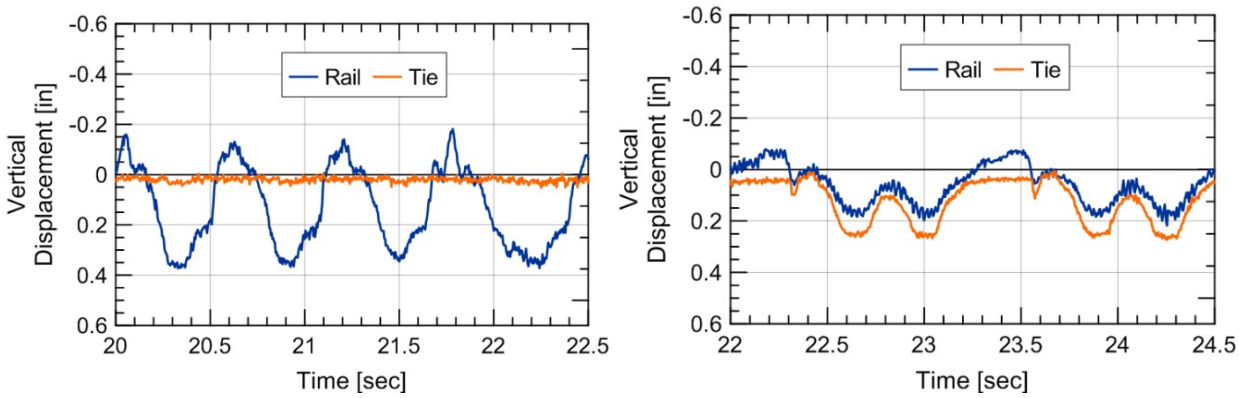
Figure 3: Photograph of the South End Bridge Transition

SECTION 5 MEASURED TRACK BEHAVIOR

5.1 Pre-Tamping

Figure 4 compares the pre-tamping rail and tie displacements measured with a high-speed video camera for: (a) Tie #1 (14 ft.) and (b) Tie #2 (20 ft.). The train measured at Tie #1 (14 ft.) consists of a northbound (approach) intermodal train moving at a velocity of 58 mph while the train measured at Tie #2 (20 ft.) consists of a southbound (exit) loaded autorack train moving at a velocity of 49 mph. Train direction did not seem to affect the tie behavior at Tie #1 (14 ft.) and Tie #2 (20 ft.) but did have noticeable effects to accelerometers further from the bridge abutment (36 to 51 ft.). The likely explanation is the train "gallops" or "bounces" after exiting the bridge or asphalt crossing.

Only two seconds of the time histories are shown to better illustrate the differences in track behavior. The rail-tie gap location (Tie #1, 14 ft.) only shows significant peak displacement of the rail (0.4 in/~10 mm) while the peak displacement of the tie is insignificant (0.05 in/~1.25 mm). This is expected because the rail-tie gap limits the amount of loading the rail applies to the tie and is expected to redistribute to surrounding ties. The tie-ballast gap location (Tie #2, 20 ft.) shows peak rail displacement of 0.2 inches (5.0 mm) and tie displacements of 0.25 inches (6.4 mm). It is typically expected that the rail displaces more than the tie but due to a potential center bound tie condition, the end of the tie bends when loaded resulting in greater displacement at the end of the tie. Additionally, the rail-tie gap location (Tie #1, 14 ft.) experiences more rail displacement than the tie-ballast gap location (Tie #2, 20 ft.), which is expected because the rail-tie gap location (Tie #1) is closer to the bridge abutment and likely experienced greater substructure settlements.



(a) (b)

Figure 4: Video Camera Measured Rail and Tie Displacements at: (a) Tie #1 (14 ft.) and (b) Tie #2 (20 ft.) for a Passing Freight Train on 21 October 2014

5.2 Post-Tamping: 1st Train

On the morning of the second day, the transition zone was resurfaced using a hand tamper. The rail was lifted about 7/8 of an inch at Tie #1 (14 ft.) and was resurfaced with an overlift, e.g. rail elevation in the transition zone was slightly higher than on the bridge, with the hope that the ballast would eventually equalize so the rail elevation in the transition zone would end up being about the same as the elevation of the bridge deck. After tamping, the rail and tie displacements at Tie #1 (14 ft.) from the first passing train, a northbound (approach) loaded autorack train moving at a velocity of 25 mph, was measured and the first 18 seconds (13 train trucks) are displayed in Figure 5. Tie #2 (20 ft.) was not able to be measured because of a maintenance vehicle blocking the view.

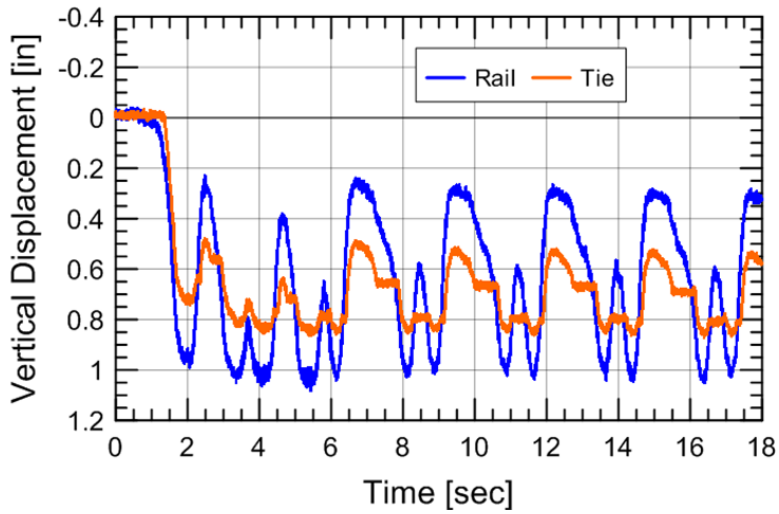


Figure 5: Video Camera Measured Rail and Tie Displacements at Tie #1 (14 ft.) for a Passing Freight Train immediately after Handing Tamping on 22 October 2014.

Figure 5 illustrates that the loading from the first train truck results in significant settlement of the rail and tie. The tie settles about 0.5 inches (13 mm) after the first truck and eventually reaches 0.7 inches (18 mm) by the end of the train. The rail shows about 0.45 inches (11 mm) of settlement by the end of the train. The reason the tie settles more than the rail is that a gap develops between the rail and tie because the upward reaction force of the rail after unloading pulls the spikes from the ties. As more trains pass and the ballast settles further, this repeated upward reaction force will continue to pull the spike from the tie and increase the rail-tie gap.

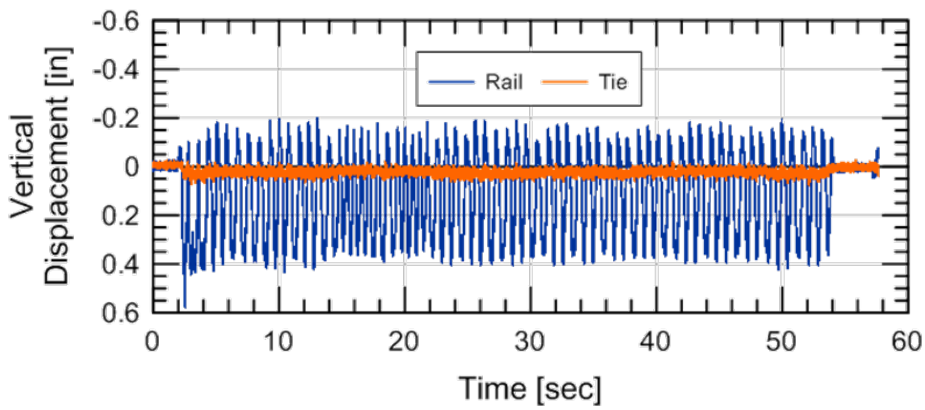
The initial truck loading produced about 0.7 to 0.8 inches (18 to 20 mm) of ballast settlement and video of the train shows ballast particles pushed out from underneath the tie. One explanation is not enough ballast particles were supporting the tie and rail to the specified elevation and therefore were not able to withstand the entire train loading. This caused the few ballast particles to either be pushed into the underlying ballast, pushed outside of the tie, or suffered from particle breakage. The presence of ballast fouling may have facilitated this process. The magnitude of ballast settlement may also be related to the pre-tamping ballast condition because ballast after tamping is observed to have a “memory” of its pre- tamping state [2]. Either way, the first train immediately compacted and re-densified the ballast and resulted in about 0.8 or 0.9 inches (20 to

23 mm) of substructure settlement which can then cause the concentration of train loads and reinstitute the track deterioration process.

5/3 Post-Tamping: 6th Train

The first train produced the greatest amount of track settlement and the track seemed to reach equilibrium after about three to four trains. To evaluate how the track behaved after reaching equilibrium, the rail and tie displacement time histories from the sixth train after tamping is measured at Tie #1 (14 ft.) and displayed in Figure 6(b). This train is a southbound (exit off the bridge) loaded train moving at a velocity of 33 mph. By comparing the (a) pre-tamping and (b) post-tamping states, it is clear that post- tamping transient behavior quickly returns to its pre-tamping behavior after a few passing trains. Due to the continual upward force of the rail pulling the spike from the ties, the rail-tie gap reaches about 0.3 inches (8 mm) and will likely eventually increase to the 0.4 inch (10 mm) gap that existed pre-tamping.

(a)



(b)

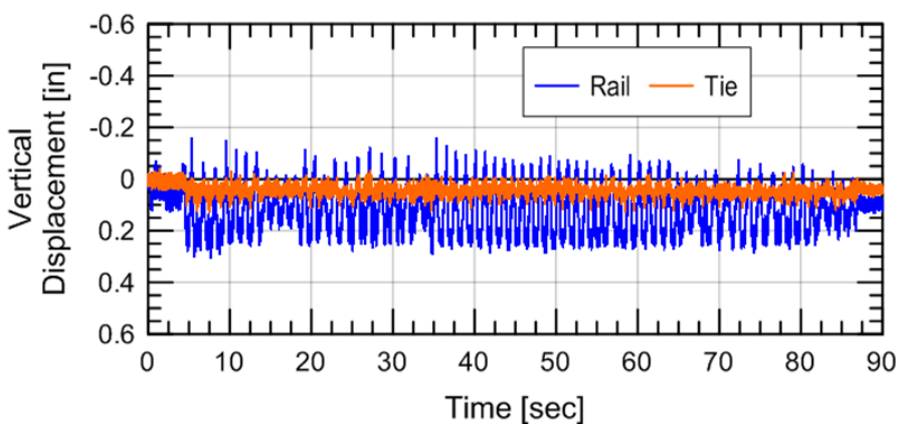


Figure 6: Video Camera Measured Rail and Tie Displacements at Tie #1 (14 ft.) (a) before tamping on 21 October 2014 and (b) the 6th train after tamping on 22 October 2014.

The cumulative rail and tie settlement is estimated to be around 0.55 inches and 0.8 inches, respectively, and no noticeable changes in behavior were observed at Tie #2 (14 ft.) for the remainder of the day.

SECTION 6 DISCUSSION

The in-track-measured rail and tie displacement time histories at a bridge transition zone illustrates the significant ballast settlement that can occur immediately after tamping, i.e. “compaction” settlement phase, which caused the track to nearly return to its original transient behavior after a few train passes. As mentioned earlier, this rapid settlement is detrimental to the transition zone because the differential substructure settlement results in the development of rail-tie and tie-ballast gaps as the rail remains supported and cantilevering from the bridge deck. The existence of rail-tie and tie-ballast gaps causes the redistribution of load and impacts which can result in higher local dynamic loads and accelerates the geometric deterioration in the transition zone section.

While the problem of differential settlement at transition zones can never be completely eliminated because of the inevitable and sometimes random nature of ballast settlement, there have been many suggestions to reduce track degradation [3,14,15]. Many of these options, however, involve new track design or installing new track components and require significant effort by railroad companies.

Another option would be attempting to limit the amount of initial ballast settlement, i.e. “compaction” phase, by ensuring the ballast is better compacted underneath the tie during tamping. If not, the first passing train will compact the ballast at the expense of track performance. If the settlement in this “compaction” phase is reduced, this will likely result in the track geometry being maintained for longer periods of time and require less frequent tamping. The additional time and money needed to compact the ballast underneath the tie can hopefully extend the tamping cycle by a few weeks or months and may eventually prove cost-effective.

Other possible limitations with the current method of tamping at high-maintenance locations is how new ballast is placed in the crib and squeezes the existing ballast underneath the tie. This implies that the degraded and fouled ballast, which tends to settle at a quicker rate than clean ballast [16], will be continually reused directly under the tie and result in further ballast degradation. If tamping methods instead pushed the ballast from one side instead of squeezing from both sides, this “ballast rotation” could extend the life and effectiveness of the ballast.

SECTION 7 SUMMARY AND REMARKS

This report reviews the ballast compaction cycle and presents field-measured rail and tie displacement values immediately after the hand tamping of a bridge transition zone. The main findings include:

- Before hand tamping, the bridge transition zone experienced multiple rail-tie and tie-ballast gaps because of the differential ballast settlement between the bridge, transition zone, and open track.

- After hand tamping, the first train pass immediately compacted the ballast resulting in 0.45 inches (11mm) of rail settlement and 0.7 inches (18 mm) of tie settlement. This occurred because not enough ballast particles were holding up the rail and ties to the specified elevation and therefore immediately "pushed out" during the first train loading.

- The ballast seemed to reach an equilibrium condition after about 4 trains and resulted in 0.5 inches of rail settlement and 0.8 inches of tie settlement with a 0.3 inch gap between the rail and tie. The transient behavior of the tie was very similar to the pre-tamping conditions.

- An "overlift" was used during hand tamping to account for the initial settlement. While it reduced the severity of track geometry deviations after the first few train passes, the significant transient movement, i.e. rail-tie gaps, will still generate increased dynamic loads and accelerate ballast degradation in the transition zone.

- Because of the large initial ballast settlements, emphasizing better ballast compaction and density during tamping of high-maintenance regions such as bridge transition zones may reduce this initial settlement resulting in the track geometry holding for longer time periods and hopefully increases tamping cycles for railroad companies.

ACKNOWLEDGMENTS

The authors acknowledge the funding provided by a Federal Railroad Administration (FRA) Research Grant for the project titled: “Seismic Testing for Track Substructure (Ballast and Subgrade) Assessment” under the direction of Hugh B. Thompson II and Theodore R. Sussmann, Jr. where the video cameras were first tested. The authors also acknowledge the FRA Broad Agency Announcement funding for the “Differential Movement at Railway Transitions” research project and the project supervision and direction provided by Cameron Stuart. Lastly, the authors thank University of Kentucky students Macy Purcell, John Magner, and Jordon Haney for help with instrumentation and University of Illinois at Urbana- Champaign graduate students Arthur Tseng and Yang Jiang for help with the video camera analysis. This research was supported by the National University Rail (NURail) Center a US DOT OST-R Tier I University Transportation Center.

REFERENCES

1. Track Safety Standards Part 213, Subpart A to F, Class of track 1 – 5. Office of Safety, FRA, U.S. Department of Transportation, 2014.

2. Selig ET and Waters JM. (1994). Track Geotechnology and Substructure Management. London: Thomas Telford. 1994.

3. Li, D., and Davis, D. (2005). Transition of Railway Bridge Approaches. Journal of Geotechnical and Geoenviornmental Engineering, ASCE, November, 2005. 131(11): pp. 1392–1398.

4. Nicks J. (2009). "The Bump at the End of the Railway Bridge". PhD Thesis, Texas A&M University, College Station, TX.

5. Stark TD, Wilk ST, and Sussmann TR. (2015). Evaluation of Tie Support at Transition Zones. Transportation Research Record: Journal of the Transportation Research Board, No.15-2902, Transportation Research Board of the National Academies, Washington, D.C. 2015.

6. Sasaoka C, Davis D, Koch K, Reiff RP, and GeMeiner W. (2005). Implementing track transition solutions. Technology Digest TD-05-001, Transportation Technology Center Inc., Pueblo, CO. 2005.

7. Jeffs T and Marich S. (1987). Ballast Characteristics in the Laboratory. In Proceedings of Conference on Railway Engineering. Perth, Western Australia, September 14th-16th, 1987.

8. Dahlberg, T. (2001). Some railroad settlement models – a critical review. Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit. Vol. 215. pp. 289 300. 2001.

9. Hay, WW. (1982). Railroad Engineering. New York: John Wiley and Sons. 1982

10. Sato, Y. (1995). Japanese Studies on Deterioration of Ballasted Track. Vehicle System Dynamics: International Journal of Vehicle Mechanics and Mobility. 24:sup1, 197-208. 1995.

11. American Railway Engineering and Maintenance-of-Way Association. (2012). AREMA Manual for Railway Engineering, Landover, Maryland. 2012

12. Lundqvist A, and Dahlberg T. (2005). Load Impact on Railway Track due to Unsupported Sleepers. Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit. 2005. Vol. 219, pp. 67 – 77

13. Sasaoka C and Davis D. (2005). Implementing Track Transition Solutions for Heavy Axle Load Service. In Proceedings, AREMA 2005 Annual Conference, Chicago, IL. 2005.

14. Kerr, A.D., and Moorney, B.E. (1993). Track transition problems and remedies. Bulletin 742, American Railway Engineering Association, Landover, MD., 267-298

15. Mishra D, Tutumluer E, Stark TD, Hyslip JP, Chrismer SM, and Tomas M. (2012). Investigation of differential movement at railroad bridge approaches through geotechnical instrumentation. J Zhejiang Univ-SciA (Appl Phys & Eng). 2012. 12(11), 814-824

16. Han X and Selig ET. (1997). Effects of Fouling on Ballast Settlement. In Proceedings of 6th International Heavy Haul Railway Conference. Cape Town, South Africa, April 6-10, 1997.

National University Rail Center - NURail
US DOT OST-R Tier 1 University Transportation Center

NURail Project ID: 2016-UKY-R11

Well-Performing Railway Bridge Transitions – Design and Performance

By

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

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

March 10, 2016

Grant Number: DTRT13-G-UTC52

DISCLAIMER

Funding for this research was provided by the NURail Center, University of Illinois at Urbana - Champaign under Grant No. DTRT12-G-UTC18 of the U.S. Department of Transportation, Office of the Assistant Secretary for Research & Technology (OST-R), University Transportation Centers Program. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the information presented herein. This document is disseminated under the sponsorship of the U.S. Department of Transportation's University Transportation Centers Program, in the interest of information exchange. The U.S. Government assumes no liability for the contents or use thereof.

TECHNICAL SUMMARY

Title

Well-Performing Railway Bridge Transitions – Design and Performance

Introduction

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

Approach and Methodology

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

Findings

Differential movement at higher-speed railway transition zones represents a safety and maintenance issue because of the continual upgrade to heavier, longer, and faster trains. To reduce differential movement and the need for frequent track resurfacing, a variety of transition zone designs and remedial measures have been proposed. These typically involve: (1) increasing and smoothing track stiffness in the approach and/or (2) lowering the track stiffness of the bridge deck because differential stiffness and settlement usually causes undesirable transition zone performance. Potential solutions in the approach include: additional rails to increase track stiffness, increased tie lengths, decreased tie spacing, under-tie pads (UTPs), abutment wing

walls, hot-mixed asphalt (HMA) underlayment, concrete approach slabs, geoweb or geocells, and soil stabilization. Bridge deck solutions include: a ballasted bridge deck, rail and tie pads, and ballast mats. While a few of these solutions have shown promising results, none are all-encompassing and typically only work on a site-specific basis.

Conclusions

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

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

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

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

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

Recommendations

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

processes that are usually present. In summary, focusing on reducing and balancing bridge stiffness and ballast settlement is recommended.

Publications

Stark, TD, Wilk, ST, Rose, JD. Design and Performance of Well-Performing Railway Transitions, Paper TRB 16-5926, Transportation Research Board Annual Meeting, Washington, DC, January 2016, 17 pages.

Primary Contact

Principal Investigators
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

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/

TABLE OF CONTENTS

Page

LIST OF FIGURES

SECTION 1 INTRODUCTION

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

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

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

SECTION 2 CAUSES OF INCREASED AND DIFFERENTIAL SETTLEMENT

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

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

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

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

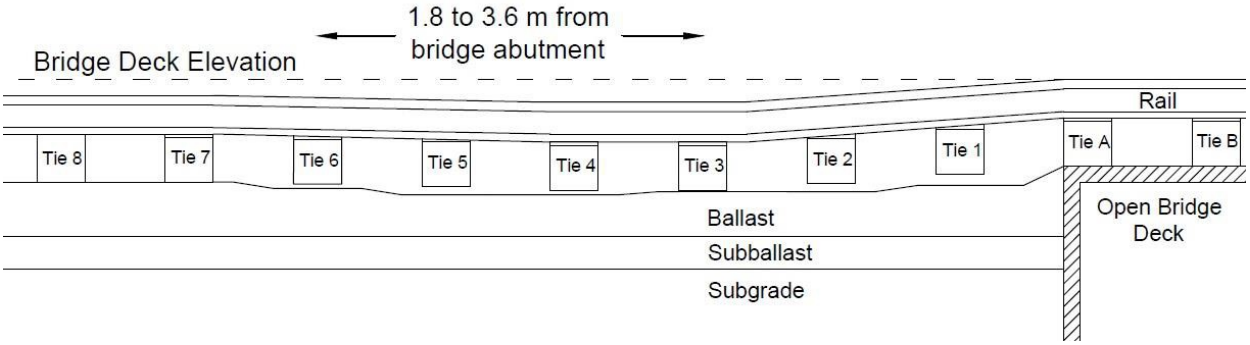


Figure 1 Example diagram of bridge transition zone with various approach tie-ballast gaps

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

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

SECTION 3 TRANSITION BALANCE SHEET

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

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

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

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

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

SECTION 4 WELL-PERFORMING TRANSITIONS

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

SECTION 5 NON-INVASIVE INSTRUMENTATION

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

Figure 3. Miniature accelerometer attached to concrete tie.

Tie acceleration time histories can be informative of track performance because any impact or movement of the tie is recorded by the accelerometer. Ideal track conditions, defined as "well-performing" herein, typically consist of track experiencing vertical displacements of only 1 to 2 mm, a smooth and evenly distributed load path from the wheel to the ballast, and minimal track geometry maintenance. In these cases, the tie is expected to only accelerate from the loading of the passing wheels and typically produce tie accelerations of less than 5g [20]. Non-ideal track conditions, defined as "poorly-performing" in the paper, typically consist of track experiencing vertical displacements greater than 2 mm, movement and impacts from the closing of gaps in the track system, and recurring track geometry maintenance. In addition to tie accelerations from the loading of the passing wheels, the tie can also accelerate from impacts in the track superstructure, tie-ballast impacts, tie vibrations, and tie displacements [21]. These additional factors can produce tie accelerations ranging from 10 to 100g and these values are highly dependent on train type, loading, and speed [20]. Impacts and vibrations from the vehicle, e.g.

wheel flats and braking, are also recorded but are not considered in the track analysis because these factors are vehicle issues, not track issues.

5.1 Site #1

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

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

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

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

material wet-of- optimum or using a granular fill with a vegetative soil cover to avoid erosion of the granular fill.

Figure 4 West End of Site #1 Bridge Transition Zone.

Approach and Bridge Displacement	Potenti al Transie		Potential Permane	
Rail compression	✓	✓		
Rail-tie gap				
Tie pad/plate	✓	✓		
Tie displacement	✓	✓		
Tie-ballast gap				
Ballast displacement	✓	✓	✓	✓
Subballast displacement				
Subgrade displacement				
Lateral displacement				

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

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

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

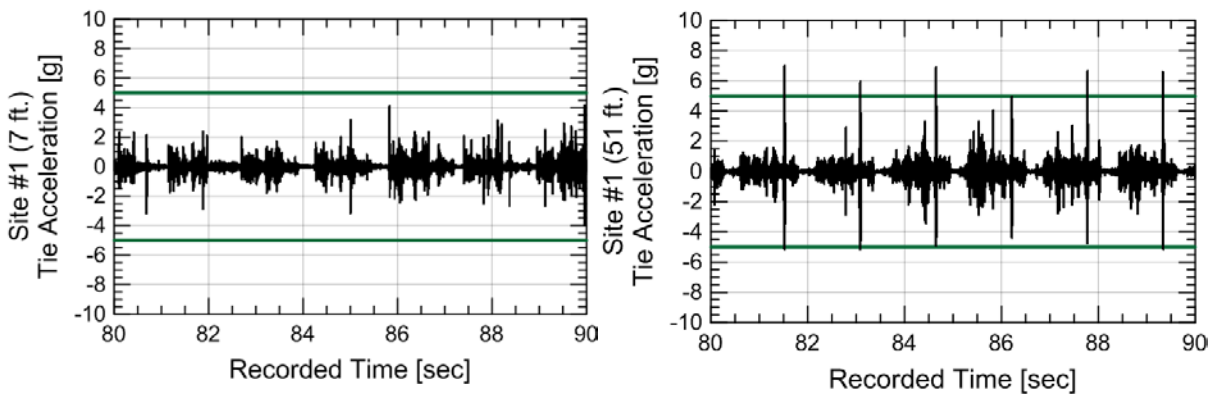


Figure 6 Measured tie accelerations at (a) Site #1 (7 ft.) and (b) Site #1 (51 ft.) for a passing freight train on 12 June 2014.

5.2 Site #2

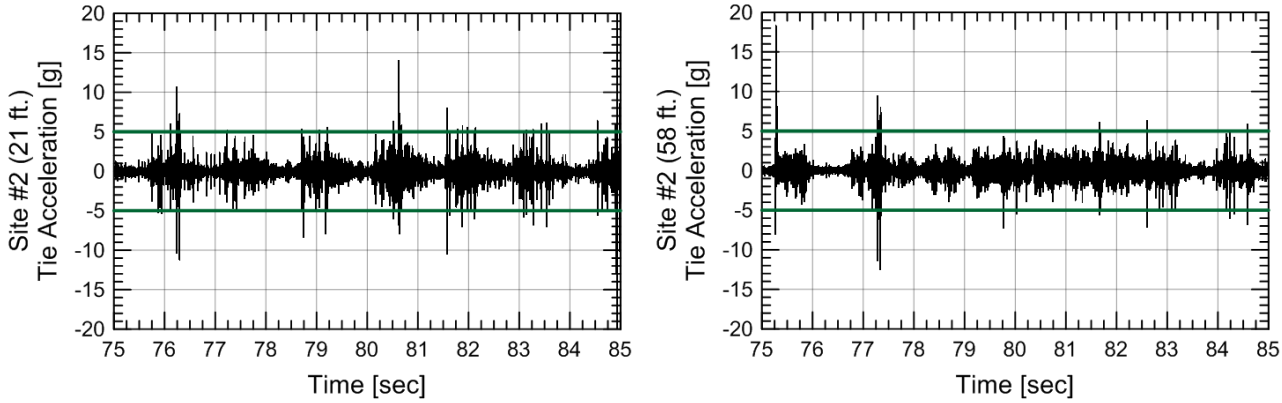
The second instrumentation site involves a similar freight bridge transition (Figure 7) that supports unloaded and loaded freight trains moving at velocities of 40 km/hr (25 mph), an annual traffic at about 70 MGT (10 times more than Site #1), and the transition also has required minimal track maintenance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

layer that extends for only 150 m (500 ft.) under 300 mm (12 in) of ballast, and (3) 7.3 meter (24 ft) long concrete wing walls perpendicular to the bridge abutment.

Figure 7 West End of Site #2 Bridge Transition Zone

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

Figure 8 displays ten (10) seconds of the 165-second long acceleration time histories at Site #2 (21 ft.) and Site #2 (58 ft.). The measured accelerations range from 2 to 3g which suggests the track behavior in the transition zone is also similar to the open track. The results of the other six accelerometers show similar results except for an accelerometer located near a welded rail joint (12 m/40 ft. from the bridge abutment), which resulted in tie accelerations



(a) (b)

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

of about 15g. The good track performance was also evident by little if any track displacement being noticed at any location during train passage. The greater acceleration at the welded rail joint, while located in the open track, also suggests that welded rail joints should not be used in the approach track whenever possible because it can lead to increased dynamic loads, increased track displacement, and formation of gaps that accelerate track degradation.

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

SECTION 6 ALTERNATIVE DESIGNS

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

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

SECTION 7 BALLAST SETTLEMENT

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

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

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

SECTION 8 SUMMARY

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

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

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

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

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

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

ACKNOWLEDGMENTS

The authors acknowledge the funding provided by a Federal Railroad Administration (FRA) Research Grant for the project titled: “Seismic Testing for Track Substructure (Ballast and Subgrade) Assessment” under the direction of Hugh B. Thompson II and Theodore R. Sussmann, Jr. of the Volpe Transportation Center in Massachusetts. The authors also acknowledge the FRA Broad Agency Announcement funding for the “Differential Movement at Railway Transitions” research project and the project supervision and direction provided by Cameron Stuart. The authors are grateful to the National University Rail (NURail) Center, Gerald Gupton and Tom Garrett of P&L Railroad, Ed Sparks of CSX, and University of Kentucky graduate student Macy Purcell for their assistance. This research was supported by the National University Rail (NURail) Center a US DOT OST-R Tier 1 University Transportation Center.

REFERENCES

1. Sasaoka, C., D. Davis, K. Koch, R.P. Reiff, and W. GeMeiner. Implementing track transition solutions. Technology Digest TD-05-001, Transportation Technology Center Inc., Pueblo, CO. 2005.

2. Kerr, A.D., and B.E. Moroney. Track transition problems and remedies. Bulletin 742, American Railway Engineering Association, Landover, MD., 267-298. 1993.

3. Li, D., and D. Davis, D. Transition of Railway Bridge Approaches. Journal of Geotechnical and Geoenviornmental Engineering, ASCE, November, 2005. 131(11): pp. 1392–1398.

4. Read, D. and D. Li. Design of Track Transitions. Research Results Digest 79, Transit Cooperative Research Program, Transportation Research Board, National Academies, October 2006, 37 p.

5. Li, D., D. Otter, and G. Carr. Railway bridge approaches under heavy axle load traffic: problems, causes, and remedies. Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit. 2010. Vol. 224, pp. 383 – 390

6. Paixão, A., E. Fortunato, and R. Calçada. Design and construction of backfills for railway track transition zones. Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit. 2015. Vol 229(1), pp. 58-70

7. Nicks, J. "The Bump at the End of the Railway Bridge". PhD Thesis, Texas A&M University, College Station, TX. 2009.

8. Coehlo, B., P. Hölscher, J. Priest, W. Powrie, and F. Barends. An assessment of transition zone performance. Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit. 2011. Vol 225, pp. 129-139

9. Mishra, D., E. Tutumluer, T.D. Stark, J.P. Hyslip, S.M. Chismer, and M. Tomas. Investigation of Differential Movement at Railroad Bridge Approaches through Geotechnical Instrumentation. Journal of Zhejiang University-Science A, 2012. 13(11):814-824

10. Wang, H., V.L. Markine, I.Y. Shevtsov, and R. Dollevoet. Analysis of the Dynamic Behavior of a Railway Track in Transition Zones with Differential Settlement. Proceedings of the 2015 Joint Rail Conference. March 23-26, 2015, San Jose, California.

11. Wilk S.T., T.D Stark, and T.R. Sussmann. Tie-Ballast Interaction at Railroad Transitions. Proceedings of Transportation Research Board 94th Annual Conference, Washington D.C., January 2015.

12. Stark T.D., S.T. Wilk, H.B. Thompson II, and T.R. Sussmann, Effect of Unsupported Ties at Transition Zones. Proceedings of Railway Engineering-2015 Conference, June 30th – July 1st. Edinburgh, Scotland.

13. Stark T.D., S.T. Wilk, and T.R. Sussmann. Evaluation of Tie Support at Transition Zones. Transportation Research Record: Journal of the Transportation Research Board, No.15-2902, Transportation Research Board of the National Academies, Washington, D.C. 2015.

14. Li. D, and L. Maal. Heavy Axle Load Revenue Service Bridge Approach Problems and Remedies. Proceedings of the 2015 Joint Rail Conference. March 23-26, 2015, San Jose, California.

15. Light, T.J., C.L Ho, and R.S. Lambert. Effect of Moisture on the Engineering Characteristics of Fouled Ballast. Proceedings of Transportation Research Board 93th Annual Conference, Washington D.C., 2014.

16. Han, X. and E.T. Selig ET. Effects of Fouling on Ballast Settlement. In Proceedings of 6th International Heavy Haul Railway Conference. Cape Town, South Africa, April 6-10, 1997.

17. Qian, Y., E. Tutumluer, Y.M.A Hashash, and J. Ghaboussi. Effects of ballast degradation on permanent deformation behavior from large-scale triaxial tests. Proceedings of the 2014 Joint Rail Conference. April 2 – 4. Colorado Springs, CO, 2014.

18. Chen, R., J. Chen, X. Zhao, X. Bian, and Y Chen. Cumulative settlement of track subgrade in high-speed railway under varying water levels. International Journal of Rail Transportation. 2014. Vol. 2, No. 4, pp. 205-220

19. Qian, Y., E. Tutumluer, Y.M.A. Hashash, J. Ghaboussi, and D. Davis. Ballast Settlement Ramp to Mitigate Differential Settlement in a Bridge Transition Zones. Transportation Research Record: Journal of the Transportation Research Board, Transportation Research Board of the National Academies, Washington, D.C. 2015.

20. Wilk, S.T., T.D. Stark, and J.G. Rose. Evaluating tie support at railway bridge transitions. Proc. IMechE. Part F: J. Rail Rapid Transp. 2015. Vol 0, pp. 1-15.

21. Taherinezhad, J., M. Sofi, P.A. Mendis, and T. Ngo. A Review of Behavior of Prestressed Concrete Sleepers. Special Issue: Electronic Journal of Structural Engineering. 2013. 13(1).

22. Rose, J. and H. Lees. Long-Term Assessment of Asphalt Trackbed Component Materials' Properties and Performance. Proceedings of the American Railway Engineering and Maintenance-of-Way Association 2008 Annual Conference, Salt Lake City, UT, September, 2008. 28 pages

23. Anderson, J.S. and J.G. Rose. In-Situ Test Measurement Techniques Within Railway Track Structures. Proceedings of the 2008 Joint Rail Conference. April, 2008, Wilmington DE.

24. Rose, J.G., T.D Stark, S.T. Wilk, and M. Purcell. Design and Monitoring of Well-Performing Bridge Transitions. Proceedings of the 2015 Joint Rail Conference. March 23-26, 2015, San Jose, California.

25. Selig, E.T., V. DelloRusso, K.J. Laine. Sources and Causes of Ballast Fouling. Report No. R-805, Association of American Railroads, Technical Center, Chicago. 1992.

26. Douglas, S.C. Ballast Quality and Breakdown during Tamping. Proceedings of 2013 American Railway Engineering and Maintenance-of-Way Association Conference. Indianapolis, IN.

27. Tennakoon, N., B. Indraratna, C. Rujikiatkamjorn, and S. Nimbalkar. Assessment of ballast fouling and its implications on track drainage. In: G.A. Narsilio, A Arulrajah, and J. Kodirkara (Eds.), 11th Australia – New Zealand Conference on Geomechanics: Ground Engineering in a Changing World. pp. 421 – 426. 2012.

28. Sussmann, T.R., H.B. Thompson, T.D. Stark, S.T. Wilk, and C.L. Ho. Use of Seismic Surface Wave Testing to Assess Track Substructure Conditions. Proceedings of Railway Engineering-2015 Conference, June 30th – July 1st. Edinburgh, Scotland.

29. McHenry, M.T., M. Brown, J.L. LoPresti, J. Rose, R. Souleyrette. The Use of Matrix Based Tactile Surface Sensors to Assess the Fine Scale Ballast-Tie Interface Pressure Distribution in Railroad Track. Proceedings of Transportation Research Board 94th Annual Conference, Washington D.C., January 2015.

30. Schnedier, P., R. Bolmsvik, and J.C.O. Nielsen. In situ performance of a ballasted railway track with under sleeper pads. Proceedings of the Institution of Mechanical Engineers, Part F: Journal of Rail and Rapid Transit. 2011. Vol. 225, pp. 299 – 309.

National University Rail Center - NURail
US DOT OST-R Tier 1 University Transportation Center

NURail Project ID: 2016-UKY-R11

Vertical Transient Track Displacement Measurements Using Non-Invasive Techniques

By

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

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

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

March 10, 2016

Grant Number: DTRT13-G-UTC52

DISCLAIMER

Funding for this research was provided by the NURail Center, University of Illinois at Urbana - Champaign under Grant No. DTRT12-G-UTC18 of the U.S. Department of Transportation, Office of the Assistant Secretary for Research & Technology (OST-R), University Transportation Centers Program. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the information presented herein. This document is disseminated under the sponsorship of the U.S. Department of Transportation's University Transportation Centers Program, in the interest of information exchange. The U.S. Government assumes no liability for the contents or use thereof.

TECHNICAL SUMMARY

Title

Vertical Transient Track Displacement Measurements Using Non-Invasive Techniques

Introduction

This report describes a non-invasive monitoring system that measures rail and tie displacements using high-speed video cameras and double-integration of acceleration time histories. While accelerometers are also used for high-frequency analysis, e.g. impact loads, vibrations, and applied loadings, this report focuses on how these two systems can be used to measure transient track displacement time histories. The benefit of accelerometers for measuring impact and vibration are described elsewhere.

Approach and Methodology

Two systems for non-invasively measuring transient rail and tie displacements using high-speed video cameras and accelerometers were utilized. The purpose of selecting these instruments is to develop a non-invasive instrumentation system that can monitor track performance under a range of environmental conditions. High-speed video cameras have many advantages such as tracking multiple rail and tie locations with a single camera, not needing to be base-isolated, and providing a visual account of the loaded track. Three cameras are used to illustrate rapid changes in rail displacement along short sections of track. Accelerometers are capable of measuring tie displacements and have many advantages such as being able to function if the optical view is blocked, e.g., rain, snow covering, or center of tie, and to measure lateral displacements.

Findings

This report describes the use of high-speed video cameras and accelerometers to non-invasively measure transient rail and tie displacements. These instruments combine to create a non-invasive, instrumentation system that can monitor track performance under a range of environmental conditions. Some of the main findings of this study are:

- High-speed video cameras and accelerometers are capable of measuring multiple rail and tie locations to analyze differing support conditions along the track or transition, which results in uneven load distribution, increased dynamic loads, and progressive track degradation.

- Comparable transient rail and tie displacements can be obtained using high-speed video cameras and double-integration of acceleration time histories if distinct tie frequencies above 0.75 Hz can be obtained. This generally limits the piezo-electric accelerometers to train velocities and tie displacements greater than 40 km/hr (25 mph) and 3 mm (0.10 in). For situations out of this range, geophones or DC accelerometers are probably better suited.

Conclusions

In summary, the high-speed video cameras sufficiently measure rail and tie displacement time histories. The results also show that a 30 fps video camera at Tie #1 (9 ft. from bridge abutment) and Tie #2 (11 ft.) is also able to capture full displacement time histories but it is not recommended that this slow frame rates be used if high-speed video cameras are available because high-speed video cameras (240 fps) can capture individual impact events that a 30 fps video camera cannot capture.

While the focus of this report is transient track displacement measurements, the goal of the accelerometer instrumentation is to non-invasively measure track displacements, support, and impacts so the high-frequency measuring piezo-electric accelerometers were selected. However, the procedure for piezo-electric and DC accelerometers are identical with the only difference being the frequency cutoff used for filtering.

Recommendations

Measuring the transient vertical displacements of railroad track is useful for track assessment because greater track displacements typically correlate to low values of track modulus and greater substructure settlement. Increased track displacements and loads can also accelerate track geometry problems and component degradation. Multiple tools are available to measure different aspects of vertical track displacement. For example, track geometry cars are beneficial for quickly measuring train axle displacement and accelerations along track. If the track behavior at a single location is desired, stationary measurements using Linear Variable Differential Transducers (LVDTs) can measure rail and tie displacement time histories of a passing train. For example, LVDTs can be attached to the bottom of the rail to monitor rail displacement or installed with depth using a borehole to measure tie and substructure displacements.

Publication

Wilk, ST, Stark, TD, Rose, JD. Non-Invasive Techniques for Measuring Vertical Transient Track Displacements, Paper TRB 3636, Transportation Research Board Annual Meeting, Washington, DC, January 2016, 18 pages.

Primary Contact

Principal Investigators

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

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

Other Faculty and Students Involved

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/

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

SECTION 1 INTRODUCTION

Measuring the transient vertical displacements of railroad track is useful for track assessment because greater track displacements typically correlate to low values of track modulus and greater substructure settlement [1,2]. Increased track displacements and loads can also accelerate track geometry problems and component degradation. Multiple tools are available to measure different aspects of vertical track displacement. For example, track geometry cars are beneficial for quickly measuring train axle Displacements and accelerations along track. If the track behavior at a single location is desired, stationary measurements using Linear Variable Differential Transducers (LVDTs) can measure rail and tie displacement time histories of a passing train. For example, LVDTs can be attached to the bottom of the rail to monitor rail displacement [3] or installed with depth using a borehole to measure tie and substructure displacements [4].

Optical techniques, typically lasers or high-speed video cameras, are becoming increasing popular for stationary measurements because they directly and non-invasively measure rail and tie displacement. For example, laser measurements are highly accurate and can accommodate sampling rates of over 1,000 Hz [*5*]. Lasers have been used to measure open track tie displacements in Brazil and transition zone rail displacements in Portugal [*5,6*]. The primary disadvantages of lasers are safety concerns of a laser near traffic and only a single location can be measured with each individual laser, therefore requiring a stable base. Alternatively, high-speed video cameras can be used for stationary measurements and can monitor multiple locations without laser related safety concerns. Vertical displacements can be derived from video camera recordings using Particle Image Velocimetry (PIV) or Direct Image Correlation (DIC) to track targets attached to the rail and ties [*3,7,8*]. High-speed video cameras are capable of measuring multiple targets in a single shot, do not require a completely stable foundation, and provide a visual account of the moving track. The disadvantages include more complicated image processing and typically lower accuracy or sampling rates than lasers. Advances in both laser and high-speed video technology in the past decade have made both of these methods more practical to use and analyze.

The integration of velocity and acceleration time histories offer an indirect method of non-invasively measuring tie displacements. Velocity measuring geophones have been used by the University of Birmingham to measure tie displacements with successful results [*7-9*] and accelerometers have been used to measure both tie and substructure displacements [*9-11*]. Accelerometers, as opposed to geophones, provide the additional benefit of measuring high-frequency tie movement, which allows for the evaluation of track support and wheel-rail, rail-tie, and tie-ballast interaction and impacts to be investigated [*12,13*]. This gives accelerometers a dual-benefit while geophones are typically limited to estimating tie displacements after single integration. However, the double-integration process to estimate displacements from acceleration time histories is less stable than single-integration of a velocity time history [*10,11*].

This report describes a non-invasive monitoring system that measures rail and tie displacements using high-speed video cameras and double-integration of acceleration time histories. While accelerometers are also used for high-frequency analysis, e.g. impact loads, vibrations, and applied loadings, this report focuses on how these two systems can be used to measure transient

track displacement time histories. The benefit of accelerometers for measuring impact and vibration are described elsewhere [*12-14*].

SECTION 2 INSTRUMENTATION

2.1 High-Speed Video Cameras

Consumer high-speed video cameras (Figure 1a) were selected to directly and non-invasively measure transient rail and tie displacements. The high-speed cameras are capable of measuring two rail and tie locations in a single shot and typically record at a frequency of 240 frames per second (fps). Non-high-speed video cameras with capabilities of 30 fps are also used to test its effectiveness for railroad applications. The literature indicates that camera monitoring initially used 30 fps [*7*] but higher values of 100 fps [*3*] to 500 fps [*8*] are now typical because of advancements in camera technology.

A MATLAB code was developed herein that tracks the movement of the targets attached to the rail and tie. An orange target color is used because it is distinct from common background colors and can be isolated during post-processing (Figure 1b). The code locates the targets by creating a binary image in which all pixels with the pre-selected orange color is converted to white while all non-orange pixels are converted to black. Secondly, the code calculates the centroid of each target, i.e. white pixels, in each frame and produces a time history by tracking the centroids during the course of the video. The influence of ground vibrations are minimized by also tracking a target attached to a 0.5 m (18 inch) stake that is driven into the ballast shoulder about 0.3 m (1 ft.) from the tie edge and subtracting the stake time history from the rail and tie time histories. This method reduces setup and image processing time compared to established PIV, DIC, and lasers methods (*6-8*) but sacrifices accuracy if low displacement values (<0.25 mm) are desired. The video cameras are capable of tracking both transient vertical and longitudinal displacements but only vertical results are presented herein.

(a) (b)

Figure 1. Photographs of (a) consumer high-speed video cameras and (b) orange targets attached to rail, timber tie, and stake locations.

2.2 Accelerometers

A second non-invasive tie displacement measurement tool is the double-integration of railroad tie acceleration time histories. This is an indirect measurement of transient displacements but has the advantage of measuring tie locations when the optical view is blocked and when many tie locations are required because umbers of sensors used is only limited by the Data Acquisition (DAQ) System. Sampling rates of 8,000 Hz are typically used, which is high enough to capture the desired accelerations [*13*]. While uniaxial accelerometers (vertical direction only) are discussed herein, tri-axial accelerometers are also available if longitudinal and lateral displacements are desired.

The accelerometers are 13 mm long (one-half inch), weigh less than grams (0.1 ounces), and are bonded to the concrete or timber tie with superglue (see Figure 2). The accelerometers do not interfere with train operations and can be set up in 20 to 30 minutes, making them 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 also available.

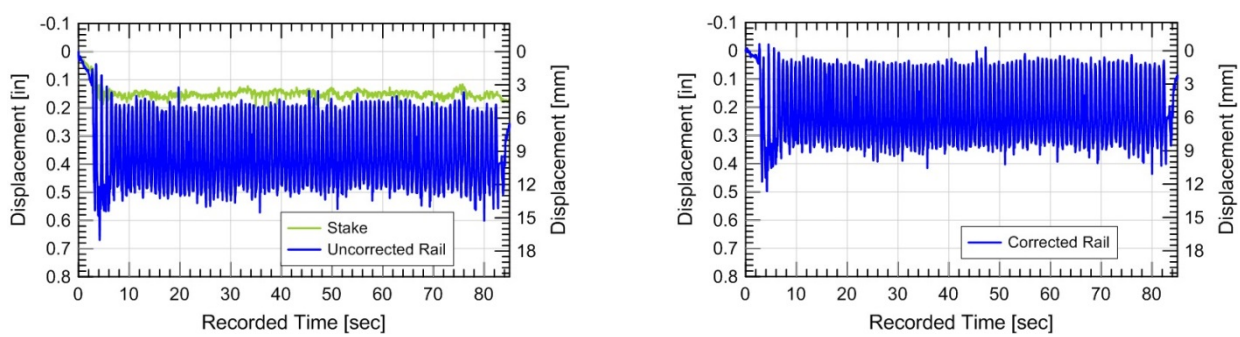
Figure 2. Photograph of accelerometer attached to a timber tie.

SECTION 3 DISPLACEMENT MEASUREMENTS – CAMERAS

To assess the practicality of measuring rail and tie displacements with consumer high-speed video cameras, the cameras monitored the south-end of an open deck bridge transition zone. The train traffic is considered Class 4 for operations and consists of empty, mixed, and loaded freight trains passing at velocities of 48 to 96 km/hr (30 to 60 mph) and accumulating 60 MGT annually. The spacing of the timber ties is 0.5 m (19.5 inch) and the bridge is a 15 m (50 ft.) steel open deck bridge. Because of increased settlement in the southern transition zone, nearly every tie has either a rail-tie and/or tie-ballast gap. Ballast fouling is also prevalent in and around all of the ties based on visual inspection. The rail-tie gaps were found within 4.6 m (15 ft.) of the bridge abutment and tie-ballast gaps were 4.6 m (15 ft.) or greater from the bridge abutment.

Six rail and tie locations over a span of 3.4 m (11 ft.) were measured on 10 June 2015 to investigate the rapidly changing track behavior with two consumer high-speed video cameras (240 fps) and a single non-high-speed video camera (30 fps). The non-high-speed video camera measured the east rail and tie locations 2.7 m (9 ft.) and 3.4 m (11 ft.) from the bridge abutment while the two high-speed video cameras measured the east rail and tie locations 4.3 m (14 ft.) ,4.9 m (16 ft.) , 5.5 m (18 ft.), and 6.1 m (20 ft.) from the bridge abutment. These ties will be referred to as Tie #1 (9 ft.), Tie #2 (11 ft.), Tie #3 (14 ft.), Tie #4 (16 ft.), Tie #5 (18 ft.), and Tie #6 (20 ft.) herein.

To eliminate ground vibration effects on the cameras, a subtraction method is used. This method involves subtracting the stake displacement time history from the rail and tie time histories. The stake is placed about 0.3 m (1 ft.) into the ballast shoulder and should experience minimal vertical displacement from passing trains. Sample results of raw rail, stake, and corrected transient vertical rail displacement time histories are displayed in Figure 3. To ensure the stake is far enough in the ballast shoulder, the stake time history can be checked for evidence of recurring vertical displacements. Small displacements (<0.05 mm) are observed in Figure 3(a) but it is not apparent if the displacements are from surface displacement or vibrations from the ground and/or wind.



(a) (b)

Figure 3. Typical results of: (a) raw vertical rail and stake displacement time histories and (b) corrected vertical rail displacement time history.

Figure 4 shows the corrected vertical rail and tie displacement time histories of an empty freight train at 76 km/hr (47 mph). The results show consistent rail and tie behavior except for the first

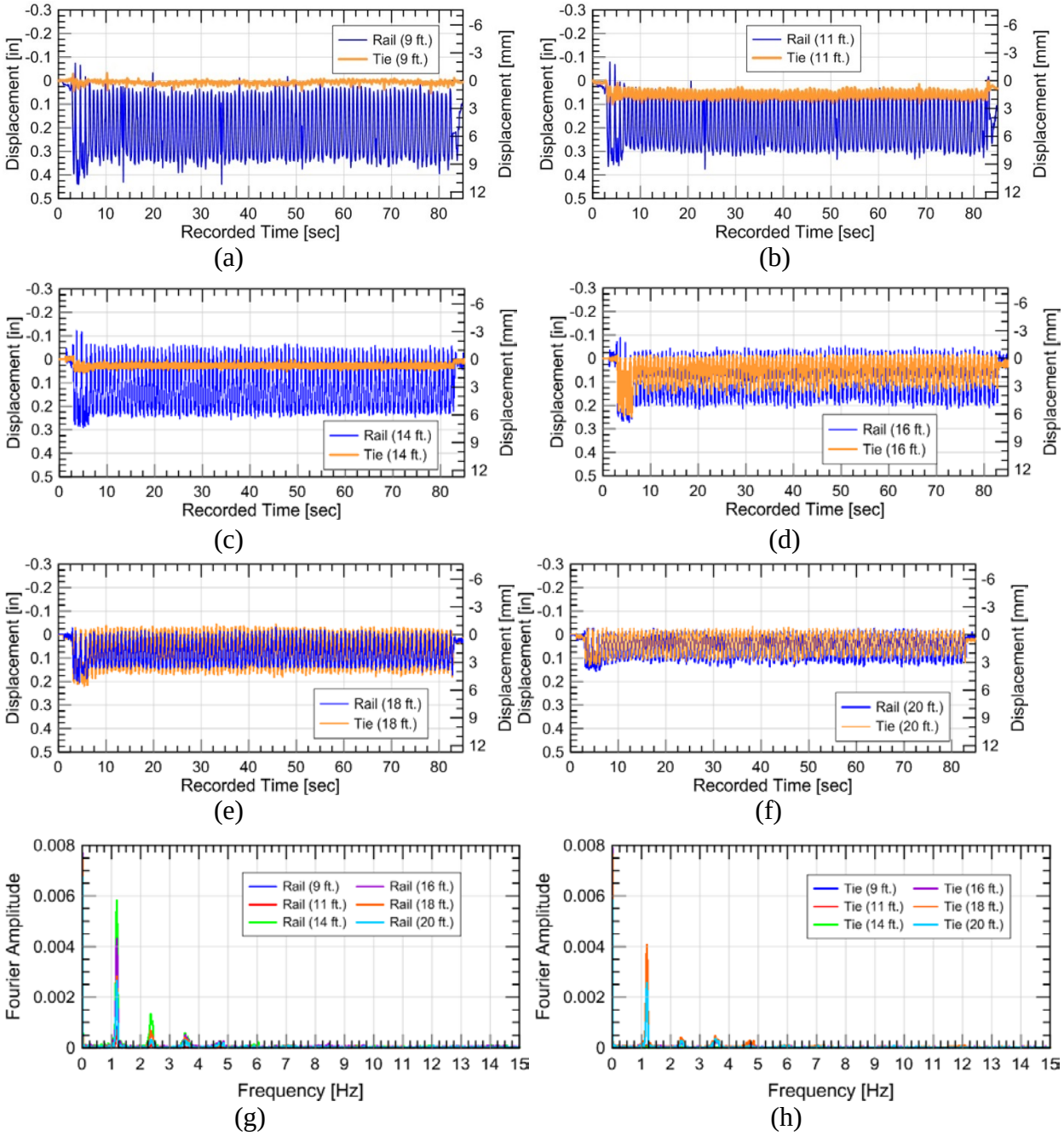


Figure 4. Vertical rail and tie displacement time histories of: (a) Tie #1 (9 ft. from bridge), (b) Tie #2 (11 ft.), (c) Tie #3 (14 ft.), (d) Tie #4 (16 ft.), (e) Tie #5 (18 ft.), and (f) Tie #6 (20 ft.) for an empty freight train passing at 76 km/hr (47 mph).

few heavy locomotive axles. The average peak rail and tie displacements at each location from the empty cars with estimated wheel loads of 35 kN (8 kips) are also displayed in Table 1. The results show significant variation in track behavior with the track locations closest to the bridge abutment displaying peak rail displacements of about 9.0 mm (0.35 inches) and tie displacements of about 0.5 mm (0.02 inches) while rail and tie displacements of 3.0 mm (0.12 inch) are observed farther away from the bridge abutment. The rapid decrease in rail displacement as the

train moves farther from the abutment (6.0 mm in 3.4 m or 0.23 inches in 11 ft) translates to a rail slope of roughly 1:500 and can result in increased loading within the transition zone [*15*]. Additionally, a change in tie behavior is observed 4.6 m (15 ft.) from the bridge abutment where the track switches from having rail-tie gaps to tie-ballast gaps. This switch implies the upward reaction force when two wheels are surrounding the tie is great enough within 4.6 m (15 ft.) to partially pull out the tie spikes. Rail-tie gaps, tie-ballast gaps, and track modulus can still be estimated using these measured displacements [*16*].

Table 1. Peak vertical rail and tie displacements for an empty freight train passing at 76 km/hr (47 mph).

	Rail		**Tie**	
	[mm]	**[in]**	**[mm**	**[in]**
Tie #1 (9 ft.)	9.0	0.35	0.5	0.02
Tie #2 (11 ft.)	8.0	0.31	2.0	0.08
Tie #3 (14 ft.)	6.0	0.24	1.0	0.04
Tie #4 (16 ft.)	5.0	0.20	3.0	0.12
Tie #5 (18 ft.)	3.5	0.14	4.0	0.16
Tie #6 (20 ft.)	3.0	0.12	2.5	0.1

Figure 4(g) and (h) present the rail and tie time histories in the frequency domain. The results show a dominant frequency of 1.2 Hz with the majority of information ranging from 1 to 5 Hz. Ideally, the sampling rate should be ten times the highest desired frequency, 5 Hz in this particular case, meaning the 240 fps recording is sufficient. The 30 fps recording does capture peak displacements because the majority of peak displacement information have frequencies below 3 Hz but frame rates above 50 would be ideal. This is relevant as 30 fps is typically the frame rate of consumer video cameras and high-speed cameras are not always available and may not be practical for long-term monitoring.

In summary, the high-speed video cameras sufficiently measure rail and tie displacement time histories. The results also show that a 30 fps video camera at Tie #1 (9 ft. from bridge abutment) and Tie #2 (11 ft.) is also able to capture full displacement time histories but it is not recommended that this slow frame rates be used if high-speed video cameras are available because high-speed video cameras (240 fps) can capture individual impact events that a 30 fps video camera cannot capture.

SECTION 4 DOUBLE INTEGRATION OF ACCELERATION

4.1 Theory

A second non-invasive tool for measuring transient tie displacements is double-integration of acceleration time histories because of the relationship between acceleration and displacement. Analyzing tie acceleration from accelerometers is more complicated than tie displacement time histories from video cameras because railroad track is a coupled multi-layer system and railroad tie acceleration is influenced by the motion and impacts from the wheel, rail, fastening system, ties, ballast, and subgrade. Some examples include: (1) wheel-rail impacts, (2) wheel-rail vibrations such as braking, (3) rail-tie impacts, (4) tie loading, (5) track and tie vibrations, (6) tie- ballast impact, and (7) tie displacement from train loading. Each factor tends to produce a unique acceleration signature and can often be identified by analyzing the acceleration record in both the time and frequency domains [*12-14*]. All of these influences except factor (7), i.e., tie displacement from train loading, produce high-accelerations (5 to 500g), high-frequency motion (>100 Hz), and low vertical displacements (<0.1 mm). This means factors (1) through (6) dominate the acceleration time histories but have negligible influence on the double-integrated displacement time histories. Therefore, these factors are filtered out before double-integration is performed as discussed below. Factor (7) is tie displacement from train loading which is a low-acceleration (<1g) and low-frequency (<5 Hz) motion, which controls the magnitude of the transient vertical displacement obtained from the double integration process. This low-acceleration (<1g) and low-frequency (<5 Hz) motion is difficult to identify by reviewing the acceleration time history in the time domain but can be easily identified in the frequency domain because of its low-frequency signature. As a result, the acceleration time histories are first converted to the frequency domain and then filtered to remove the high-frequency motions before double-integration is performed to calculate the displacement time history as discussed below.

A complication arising from the double integration procedure is the inherent noise within the accelerometer and track systems. It is important to remove the noise because it results in unrealistic displacements during double integration. Noise exists at all frequencies but is more prevalent at low- and high-frequencies. This range depends on the accelerometer system and external track factors but the primary source of noise in the acceleration time histories is from the accelerometers themselves. There are two main types of accelerometers, piezo-electric and Direct Current (DC), and each has its own frequency range of noise. Piezo-electric accelerometers typically allow for a wide range of acceleration magnitudes and frequencies, e.g., +/- 500g and 0.7 to 20,000 Hz, and are therefore beneficial for measuring both tie displacements and impacts. DC accelerometers typically have a restricted magnitude (+/- 50g) and frequency range (0 to 500 Hz) but are designed to have limited noise at low-frequency motion and are more expensive. In general, piezo-electric accelerometers are better suited for measuring the wide range of railroad acceleration and loadings while DC accelerometers, similar to geophones, are better suited if double-integration is the primary purpose of the instrumentation.

To remove the inherent noise and higher frequency motion in acceleration time histories, signal filtering must be performed [*10,11*]. Signal filtering essentially removes frequencies of

a specified range from a time history by multiplying any frequency outside the range by zero (0) and any frequency inside the range by unity (1). Filters can generally be described by the mathematical filter and frequency range the filter passes. Multiple types of mathematical filters exist and differ based on the mathematical equation used to smooth the transition from the filtered and non-filtered range but the commonly used Butterworth Filter is used herein because it is simple to use and sufficiently filters the signal. The frequency range can be specified by selecting one of three types of filters: low-pass, high-pass, or band-pass. Low-pass filters allow frequencies lower than the frequency cutoff and attenuate higher frequencies. High-pass filters allow frequencies higher than the frequency cutoff and attenuate lower frequencies. Band-pass filters allow frequencies between two frequency cutoffs and attenuate frequencies outside the cutoff range. To demonstrate how low-frequency noise is removed from an acceleration time history in the frequency domain, Figure 5 shows the effect of a high-pass Butterworth filter with a cutoff frequency of 0.75 Hz.

For the double-integration procedure to be successful, the tie displacement signature must either be of a different frequency than the noise so the noise can be removed by filtering or the signature must produce a significantly greater magnitude signal to overpower the noise. Because tie displacement frequency, e.g., the number of times the tie moves up and down in a second (Hz), is a function mainly of train speed, the double integration procedure is restricted by train speed. For example, trains moving between 40 to 80 km/hr (25 and 50 mph) typically produce tie displacement frequencies between 1 and 5 Hz. Slower trains produce lower tie displacement frequencies and make it difficult to isolate and filter out the noise component.

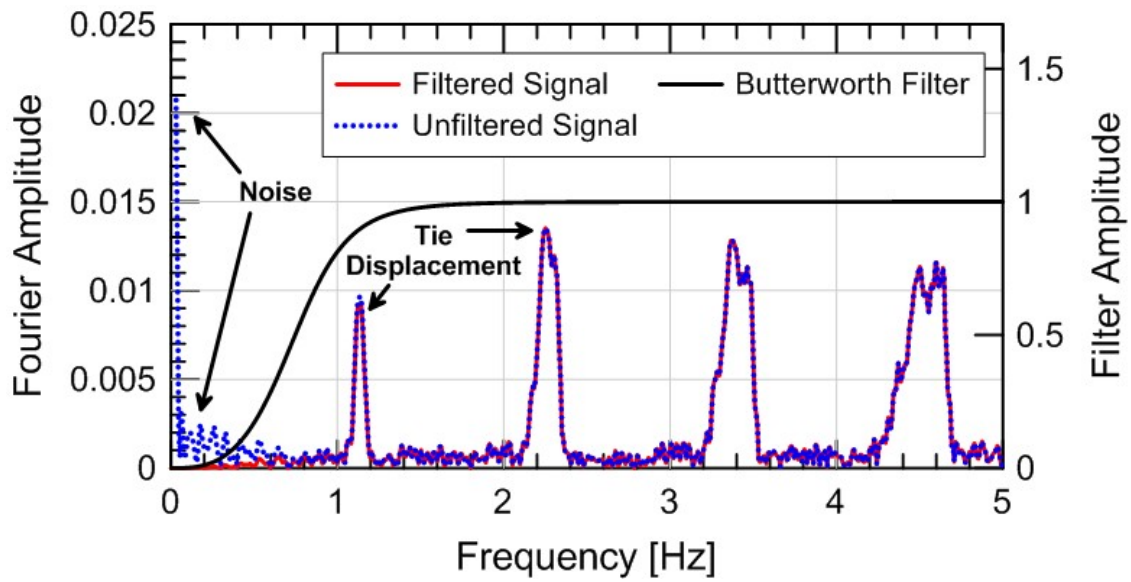


Figure 5. High-pass Butterworth filter with a cutoff frequency of 0.75 Hz applied to a tie acceleration time history presented in frequency domain.

4.2 Analysis Procedure

While the focus of this report is transient track displacement measurements, the goal of the accelerometer instrumentation is to non-invasively measure track displacements,

support, and impacts so the high-frequency measuring piezo-electric accelerometers were selected. However, the procedure for piezo-electric and DC accelerometers are identical with the only difference being the frequency cutoff used for filtering. The double-integration procedure includes the following steps:

1 Passing the acceleration time history through a band-pass Butterworth filter (0.75 to 50 Hz) to eliminate low-frequency noise and high-frequency motions. The lower frequency cutoff is set by the accelerometer while the upper limit is more arbitrarily set because high-frequency tie accelerations (>50 Hz) have negligible effect on the calculated tie displacements. An upper limit value of 50 Hz was selected because it removes the high-frequency motion and generally isolates the tie displacement component.
2 Integrating the acceleration time history using the trapezoidal method to obtain a velocity time history;
3 Passing the velocity time history through a high-pass Butterworth filter (0.75 Hz) to remove residual noise from the integration process;
4 Integrating the velocity time history using trapezoidal method to obtain a displacement time history.

This procedure is illustrated in Figure 6 which shows the unfiltered and filtered tie acceleration time histories, the integrated tie velocity time history, and double-integrated tie displacement time history. The significant reduction in acceleration magnitudes (~10g to ~1g) when the raw acceleration time history is passed through the band-pass filter reinforces the prevalence of high-acceleration magnitude and high-frequency motion in track. The acceleration spikes greater than 30g are inconsistent and likely from passing wheel flats while the consistent 10 to 15g accelerations are likely from impact loads within the track system during wheel loading and therefore are not associated with vertical tie displacements. The velocity and displacement time histories show consistent values which is expected from a passing train and can be used to estimate peak-to-peak tie velocity and displacements. This means the difference between the minimum and maximum tie displacements from the double integration procedure is the difference between the minimum and maximum tie displacement from the high-speed video camera. The exact displacement signature from video cameras will not be replicated by accelerometers because some signal information is filtered from the acceleration time history, which is explained in more detail below.

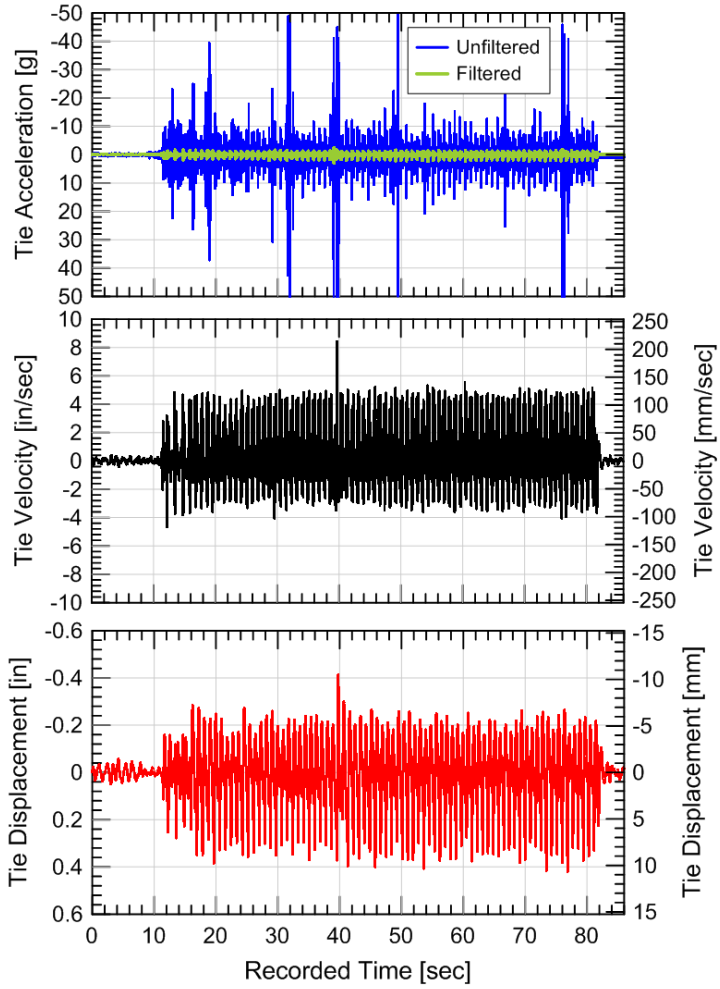


Figure 6. Comparison of unfiltered and filtered acceleration (top), integrated velocity (middle), and double-integrated displacement (bottom) time histories.

SECTION 5 COMPARISON OF TRANSIENT DISPLACEMENTS

Tie displacement time histories from a high-speed video camera and double-integrated accelerometer time histories are compared using the procedures explained above. Figure 7 compares the high-speed video camera and double-integrated displacement time histories for a loaded coal train moving at a velocity of 63 km/hr (39 mph). Figures 7(a) and 7(b) compare the full time histories while Figures 7(c) and 7(d) display only ten seconds of this time history to facilitate comparison of the displacement signatures. The peak-to-peak tie displacement values of the high-speed video camera (12.75 to 14 mm or 0.5 to 0.55 inches) and accelerometers (12.75 to 15.25 mm or 0.5 to 0.6 inches) are comparable. A key difference in the time histories is the high-speed video camera data (Figures 7a and 7c) show the tie moving downward (downward displacement is positive) from the origin while the double-integrated time history is more "symmetric" about the origin. The lack of "symmetry" in the video camera data is caused by the existence of low-frequency movements (<0.05 Hz) that are filtered out in the accelerometer signal (Figures 5 and 8). The 10-second data in Figures 7(c) and 7(d) shows the exact signature is not matched well but this is expected when comparing filtered and double-integrated data from piezo-electric accelerometers.

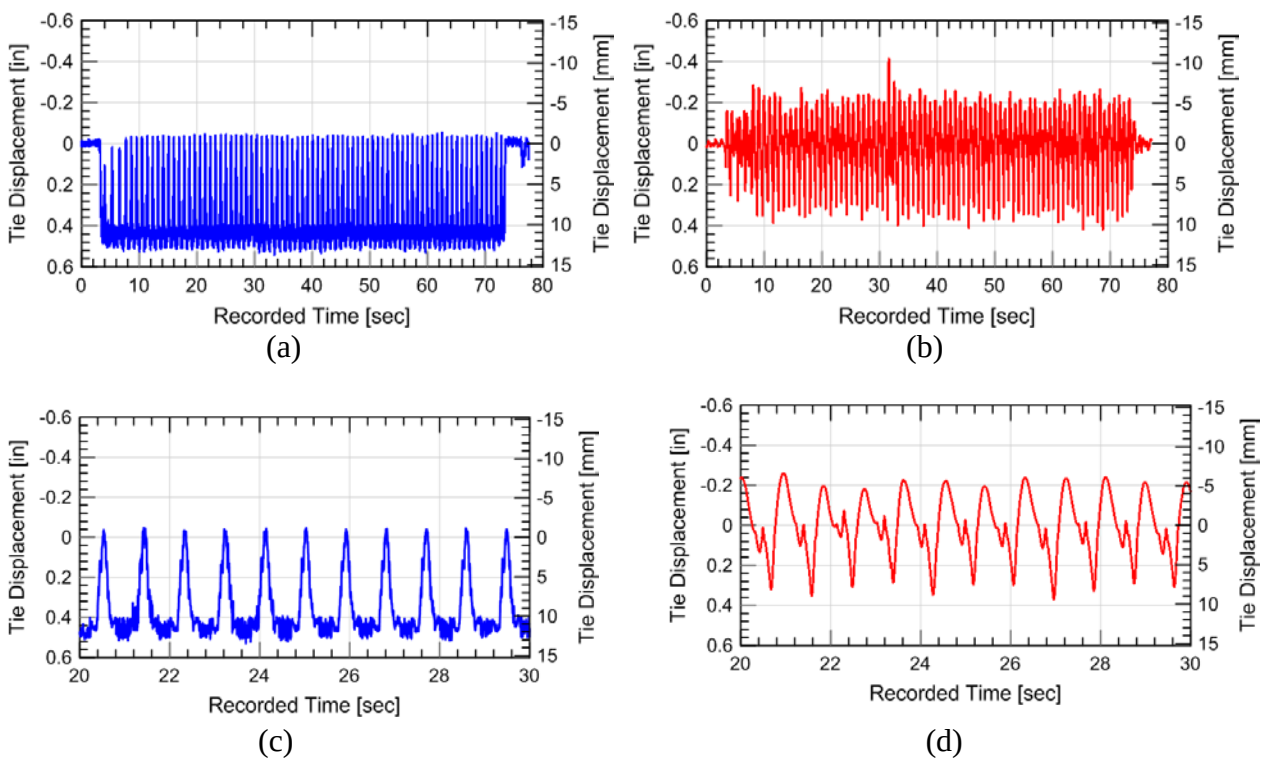


Figure 7. Tie displacement time histories of a passing coal train with a velocity of 63 km/hr (39 mph): (a) entire train with high-speed video camera, (b) entire train with double-integrated accelerometer, (c) 10-seconds of high-speed video camera and (d) 10-seconds double-integrated accelerometer.

Figure 8 compares the high-speed video camera and filtered accelerometer time histories in the frequency domain and shows the amplitudes are in agreement for a range of tie frequencies (1.15, 2.3, 3.45, and 4.45 Hz). The magnitude of the Fourier Amplitudes differ in Figure 8, which is in agreement with the difference in displacement signatures in Figures 7(c) and (d). A

second difference is the high-magnitude, low-frequency (<0.05 Hz) movement in the high-speed video camera signal, which makes the video time history lose its “symmetry” about the x-axis but does not affect the peak-to-peak displacement values. This low-frequency motion was filtered out in the acceleration time history because it shares the same frequency as the noise from the accelerometer system.

Figure 8. High-speed video camera and filtered accelerometer time histories in the frequency domain for a passing coal train with a velocity of 63 km/hr (39 mph).

Table 2 compares the high-speed video camera and double-integrated accelerometer results of six recorded trains. The results show acceptable matches for Trains 2, 3, 5, and 6 and detailed results for Train 5 are shown in Figures 7 and 8. Trains 1 and 4 have poor matches because the dominant frequency of tie movement is lower than the 0.75 Hz filter cutoff used in the double-integration procedure. Train 1 slowed from an initial velocity of 80 km/hr (50 mph) to 40 km/hr (25 mph) at the end of recording and the unusually low dominant frequency of Train 4 is postulated to be from the wheel and truck spacing. An important observation from Table 2 is the while the train velocity and dominant frequency are related but they do not perfectly correspond. From the authors’ experience, train velocities of 40 to 48 km/hr (25 to 30 mph) typically correspond to a dominant frequency of about 1.0 Hz but this is not always the case. Lastly, distinct tie frequencies were observed for all six recorded trains (see Figure 8) but if distinct tie frequencies are not apparent, the double-integration process will not be successful because the displacement components will be lost in the noise from the accelerometer. From the author’s experience, this can be an issue for ties with low displacements (<3 mm or 0.1 in). In these cases, geophones or DC accelerometers may be more suitable instruments.

Table 2. Comparison of peak tie displacements between high-speed video camera and double-integrated accelerometer data.

Train	Train Velocity	Direction	Type	Dominant	Camera	Accelerometer Range
1*	50 to 25	North	Mixed	1.0 to 0.5	10.0 – 11.5	5.0 – 12.7
2	59	North	Mixed	1.25	11.5 – 12.7	11.5 – 14.0
3	36	South	Loaded	0.95	12.7 – 15.3	10.0 – 15.3
4	31	South	Mixed	0.47	11.5 – 12.7	6.5 – 8.0
5	39	South	Loaded	1.15	12.7 – 14.0	12.7 – 15.3
6	30	South	Loaded	0.9	11.5 – 12.7	10.0 – 14.0

*Train 1 slowed from 50 mph to 25 mph during recording

SECTION 6 SUMMARY

This report describes the use of high-speed video cameras and accelerometers to non-invasively measure transient rail and tie displacements. These instruments combine to create a non-invasive, instrumentation system that can monitor track performance under a range of environmental conditions. Some of the main findings of this study are:

- High-speed video cameras and accelerometers are capable of measuring multiple rail and tie locations to analyze differing support conditions along the track or transition, which results in uneven load distribution, increased dynamic loads, and progressive track degradation.
- Comparable transient rail and tie displacements can be obtained using high-speed video cameras and double-integration of acceleration time histories if distinct tie frequencies above 0.75 Hz can be obtained. This generally limits the piezo-electric accelerometers to train velocities and tie displacements greater than 40 km/hr (25 mph) and 3 mm (0.10 in). For situations out of this range, geophones or DC accelerometers are probably better suited.

ACKNOWLEDGMENTS

The authors acknowledge the funding provided by a Federal Railroad Administration (FRA) Research Grant for the project titled: "Seismic Testing for Track Substructure (Ballast and Subgrade) Assessment" and the excellent support and insights of project director Hugh B. Thompson II. The authors also acknowledge the FRA Broad Agency Announcement (BAA) funding for the "Differential Movement at Railway Transitions" research project and the project supervision and direction provided by Cameron Stuart. The authors are grateful to Mike Tomas from Amtrak and Damon Smith from CSX for their assistance and insights during this project. Finally the authors want to thank the many University of Illinois at Urbana-Champaign students, including Yang Jiang, Arthur Tseng, and Lardner Chaves, for their excellent assistance during this project. This research was supported by the National University Rail (NURail) Center a US DOT OST-R Tier 1 University Transportation Center.

REFERENCES

1. Carr, G. "Dynamic Response of Railroad Track Induced by High Speed Trains and Vertical Stiffness Transitions with Proposed Method of Measurement". MS thesis. Tufts University, Meford, Mass., 1999.
2. Sussmann, T.R., Ruel, M., and Chrismer, S.M. (2012).Source of Ballast Fouling and Influence Considerations for Condition Assessment Criteria. *Transportation Research Record: Journal of the Transportation Research Board, Transportation Research Board of the National Academies*, No. 2289. Washington, D.C., 2012. Pp. 87-94.
3. Murray, C.A., W.A. Take, and N.A. Hoult. Measurement of vertical and longitudinal rail displacements using digital image correlation. *Can. Geotech. J.* 52:141-155. 2015
4. Mishra, D., E. Tutumluer, H. Boler, J.P. Hyslip, and T.R. Sussmann. Instrumentation and Performance Monitoring of Railroad Track Transitions using Multidepth Deflectometers and Strain Gauges. *Transportation Research Record: Journal of the Transportation Research Board*, No.14-5405, Transportation Research Board of the National Academies, Washington, D.C., 2014.
5. Pinto, N., C.A. Ribeiro, J. Gabriel, and R. Calçada. Dynamic monitoring of railway track displacement using an optical system. *Proc. IMechE. Part F: J. Rail Rapid Transp.* 2015. 1Vol. 229(3) Pp. 280-290.
6. Silva, F.C.M., M.R.S. Logistica, C.E.L. Paiva, and P.R. Aguiar. Evaluation of Track / Ballast Behavior under Increased Axle Load: Measuring Deflection on Track. *IHHA Specialist Technical Session (STS).* Kiruna, Sweden. 2007.
7. Priest, J.A., W. Powrie, L. Yang, P.J. Grabe, and C.R.I. Clayton. Measurements of transient ground movements below a ballasted railway line. *Géotechnique*. 2010. Vol. 60. 24 No. 9. Pp. 667-677.
8. Le Pen, L., G. Watson, W. Powrie, G. Yeo, P. Weston, and C. Roberts. The behavior of railway level crossings: Insights through field monitoring. *Transportation Geotechnics*. Vol. 1. 2014. Pp. 201-213
9. Coehlo B., P. Hölscher, J. Priest J, W. Powrie, and F. Barends. An assessment of transition zone performance. *Proc. IMechE. Part F: J. Rail Rapid Transp*. 2011. Vol 225, 30 pp. 129-139.
10. Coehlo B., P. Hölscher, and F. Barends. Enhancement of double integration procedure through spectral subtraction. *Soil Dynamics and Earthquake Engineering*. 2011. Vol 31. Pp. 716-722.
11. Lamas-Lopez, F. V. Alves-Fernandes, Y.J. Cui, , S.C. D'Aguiar, N. Calon, J. Canou, J. Dupla, A.M. Tang, and A. Robinet. Assessment of the Double Integration Method using Accelerometers Data for Conventional Railway Platforms. *Proc: 9th International Conference on Engineering Computational Technology.* Stirlingshire, Scotland. Pp. 1 – 38.
12. Stark, T.D., S.T. Wilk, and T.R. Sussmann. Evaluation of Tie Support at Transition Zones. *Transportation Research Record: Journal of the Transportation Research Board. Transportation Research Board of the National Academies*, Washington, D.C. 2015. 13.
13. Wilk, S.T., T.D. Stark, and J.G. Rose. Evaluating tie support at railway bridge transitions. *Proc. IMechE. Part F: J. Rail Rapid Transp*. 2015. Vol 0, pp. 1-15 14.
14. Rose J.G., T.D. Stark, S.T. Wilk, and M. Purcell. Design and Monitoring of Well-Performing Bridge Transitions. *Proc: ASME 2015 Joint Rail Conference (JRC2015)*, March 23-26, 2015. San Jose, California, USA

15. Nicks, J., “The Bump at the End of the Railway Bridge”. PhD Thesis, Texas A&M University, College Station, TX, 2009.
16. Stark, T.D. and S.T. Wilk. Root cause of differential movement at bridge transition zones. *Proc. IMechE. Part F: J. Rail Rapid Transp*. 2