Use of Multidepth Deflectometers and Strain Gauges to Investigate the Differential Movement at Railway Bridge Approaches

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Abstract

This paper presents findings from an ongoing research study at the University of Illinois focusing on the instrumentation and performance monitoring of railroad bridge approaches using multidepth deflectometers and strain gauges. Different sensors installed at the selected approaches are introduced, and details of the instrumentation activity are explained. Example track settlement data acquired over time is presented to compare the contributions of different substructure layers to the permanent deformation accumulation. Similarly, transient track deformation data gathered for train loading is presented to highlight the contributions of individual track substructure layers towards the total transient deformations. The ballast layers appear to be the primary source of accumulation for both permanent as well as transient deformations.

Keywords: track transitions, differential movement, multidepth deflectometers, strain gauge, instrumentation, ballast settlement.

1 Introduction

Railway transitions experience differential movements due to differences in track system stiffness, track damping characteristics, foundation type, ballast settlement from fouling and/or degradation, as well as fill and subgrade settlement. Identification of different factors contributing towards this differential movement and development of design and maintenance strategies to mitigate the problem is imperative for the safe and economical operation of both freight and passenger rail networks. One of the primary examples of differential movement at track transitions involves the development of vertical differential settlements or “bumps” between the railroad bridge decks and approaches. Due to the sudden change in track profile, railway track transitions are often exposed to magnified dynamic loads as a train...
passes over them. Such magnified load levels ultimately result in rapid degradation in track geometry and profile, requiring frequent maintenance and resurfacing.

The differential movement at track transitions can be equally problematic for high-speed passenger trains, as well as slow-moving freight trains. Accordingly, it presents a critical problem for shared corridors carrying both freight and high-speed lines. Transitions along shared corridors need to be maintained to satisfy the high ride quality requirements associated with high-speed trains. Additionally, these transitions also need to withstand the heavy loads imposed by slow-moving freight trains without undergoing excessive deformations. With the current impetus for development of high speed lines and the challenges associated with shared corridors for operation of passenger trains at increased speeds, preventing and mitigating the differential movement at bridge approaches and other track transitions has become more significant. Although several research studies have focused on the mitigation of this type of differential movement problem [1, 2, 3, 4], no significant research study has focused on quantifying the contributions of different factors towards the “bump” development.

The United States Federal Railroad Administration (FRA) is currently sponsoring a research study aimed at the identification and quantification of different factors contributing to the problem of differential movement at track transitions, and development of design and rehabilitation measures to mitigate this problem. The study, currently undertaken through collaborative efforts of researchers from both academia and industry, has involved the instrumentation and performance monitoring of problematic bridge approaches experiencing recurrent differential movement problems. Three problematic bridge approaches along Amtrak’s Northeast Corridor (NEC) near Chester, Pennsylvania were first monitored in July, 2012 using Multidepth Deflectometers (MDDs) and strain gauges. Moreover, two problematic bridge approaches along Norfolk Southern’s N-Line mainline near Ingleside, West Virginia were subsequently instrumented in October of 2013. This paper presents details of the instrumentation procedure along with important observations from performance monitoring of the Amtrak bridge approaches over the past one year period.

2 Site selection and transition characteristics

2.1 Amtrak NEC bridges

A problematic portion of Amtrak’s NEC is located south of Philadelphia near Chester, Pennsylvania. This portion has 8 to 10 closely-spaced bridges with recurring differential movement problems at the bridge/embankment interfaces that are causing ride quality issues. The NEC is primarily a high speed railway with occasional freight traffic. The high-speed passenger trains operate up to a maximum speed of 241 km/h (150 mph). This segment of the NEC comprises four tracks with Tracks 2 and 3 maintained for high-speed Acela express passenger trains traveling at 177 km/h (110 mph) in this region. The problem of differential movement at bridge
approaches becomes magnified under these higher operating speeds of the Acela passenger trains, compared to slower speeds of freight trains, therefore, the current research study focused on the performances of Tracks 2 and 3. The predominant direction of traffic along Track 2 is Northbound whereas Track 3 predominantly carries southbound traffic.

Historical track geometry data was obtained from Amtrak spanning the 60-month period from January 2005 to January 2010. Vertical profiles of Tracks 2 and 3 were analyzed using a 62-ft. (1 ft. = 30.48 cm) mid-chord offset (MCO) and associated data from the vertical space curve. Ground penetrating radar (GPR) scanning of the tracks was also conducted to identify substructure features that might suggest a particular bridge approach for instrumentation over others. The detailed approach adopted for selecting the bridge approaches along the Amtrak NEC for instrumentation and performance monitoring have been reported elsewhere [5].

2.2 Norfolk Southern East Megasite Bridges

The two Norfolk Southern (NS) undergrade bridge approaches instrumented in this project are located at mileposts (MP) 352.2 and 352.8 on the “N-line” mainline between Roanoke, Virginia and Bluefield, West Virginia. The bridge at MP 352.2 is located on a 10-degree curve and on a 1.1% grade, whereas the bridge at MP 352.8 is located on a 9.7-degree compound curve on a 0.9% grade. Track speed in the region is 40 km/h (25 mph) as loaded trains move downhill from west to east with full dynamic brake and often with air brakes applied. Figure 1 shows a photo of one of the bridge approaches instrumented during this effort. The custom-designed drill rig used during this instrumentation effort can also be seen in the picture.

Figure 1: Photograph showing one of the instrumented bridge approaches along the NS N-line and the custom-designed drill rig used in instrumentation
This section of the track is subjected to heavy axle load train operation with an annual tonnage of approximately 55 MGT (55 million gross tons or 49.5 million gross tonnes). Historical evidence indicates that these open-deck bridges and their approaches have experienced track geometry degradation (both in the vertical and horizontal directions), and therefore have required frequent surfacing work. As part of the East Mega Site research program funded by the Association of American Railroads (AAR) and the FRA, engineers from the Transportation Technology Center, Inc. (TTCl) and NS conducted investigations between 2006 and 2008 to determine the causes of the track geometry deterioration problem. The investigation included site inspection, vertical track modulus tests using TTCl’s Track Loading Vehicle (TLV), subgrade strength testing using TLV-equipped Cone Penetrometer Test (CPT), gage restraint testing using FRA’s Gage Restraint Measurement System (GRMS) test vehicle, standard test borings (SPT and sampling), and shear wave modulus measurements from down-hole seismic testing.

Longitudinal track substructure profiles developed based on the investigation results showed the approaches of both bridges to have significantly thick clay and/or silt layers in the track substructure immediately adjacent to the abutments. These clay and/or silt layers could undergo permanent deformation under the slow-moving heavy-haul traffic, leading to differential movements immediately adjacent to the abutment. The bridge at MP 352.2 was modified from an open-deck structure to a ballast-deck in the fall of 2007 to remediate the recurrent track geometry problem.

Track geometry data for these two bridges were analyzed for the time period between August 2005 and October 2012 to identify the rate and severity of track geometry deterioration. The analyses indicated that converting the bridge at MP 352.2 from open deck to ballast deck in the fall of 2007 initially helped reduce the vertical surface roughness, but since January 2010 the vertical surface roughness has been increasing. Assuming that there is only minor differential settlement between the ballast on the approach and the ballast on the ballast-deck bridge, some other factors appear to be contributing to the observed differential movement. Accordingly, the current instrumentation effort was required to further identify and investigate different factors affecting the track profile deterioration.

3 Instrumentation approach

The primary instrumentation types used in this effort were MDDs and strain gauges. First developed in South Africa in late 1980s [6,7], the MDD technology comprises the installation of five or six linear variable differential transformers (LVDTs) vertically at preselected depths in a small-diameter hole (typically 45 mm diameter) to measure the deformation of individual substructure layers with respect to a fixed anchor buried deep in the ground. Typically installed in 3-m. deep holes, MDDs can be used to record both the permanent (plastic) as well as transient (elastic) deformations at different depths within the track substructure. Figure 2 presents a schematic showing a railroad track instrumented using MDDs. LVDT modules are mounted within the drilled hole at positions corresponding to different interfaces.
between track substructure layers. Movement of the substructure layers, with time as well as under traffic, can be measured through the corresponding voltages induced in the individual LVDT modules. Drilling the MDD hole through a crosstie enables the mounting of the topmost LVDT at the tie-ballast interface to accurately monitor deformations within the ballast layer. More details on the principle of operation of MDDs can be found elsewhere [5, 8, 9].

Figure 2: Schematic of track substructure profile with MDD modules at individual layer interfaces

Note that the two different instrumentation efforts (Amtrak and NS) carried out in this project utilized two different depths of MDD installation. As the instrumented bridges along Amtrak’s NEC have been in service for more than 100 years, it was presumed that embankment fill and subgrade materials have undergone all possible consolidation, and therefore the movements (both permanent as well as transient) of deep substructure layers were negligible in magnitude. As the two tracks instrumented at these bridge approaches primarily carry passenger trains (relatively light weight), the stress waves are not likely to penetrate very deep into the track substructure layers. Accordingly, this assumption of insignificant movement with the deep substructure layers was justified. The MDDs installed at the Amtrak bridge approaches were accordingly anchored at a depth of 3 meters. On the other hand, previous geotechnical investigation at the two NS bridge approaches had shown the presence of thick clay and/or silt layers as deep as 5.5 m (18 ft.) from the track
surface. Moreover, as these lines primarily carry slow-moving freight traffic, the induced stresses are likely to affect the deep substructure layers. Accordingly, it was important to ensure that the MDD anchor was placed sufficiently below the track surface in a zone of insignificant movement. The MDD installation tools were therefore modified to accommodate the drilling and instrumentation of 5.5-m (18-ft.) deep holes. Note that the manufacturing and modification of all tools used in this project were carried out at the University of Illinois Civil Engineering workshop under the guidance of Mr. Michael Tomas of Amtrak.

Beside the MDDs, strain gauges were mounted on the rail to measure vertical wheel loads applied during the passage of a train, as well as to monitor the support conditions underneath the instrumented crossties. Dual-element 350 Ohm shear gauges mounted on a stainless shim were welded on the rail at the neutral for this purpose. Specific dimensions of the rail sections were used to identify and mark the rail neutral axis in the field. Before installation, the strain gauges were pre-harnessed for vertical load measurements and made ready for mounting on the rail with connection to a signal cable. A calibration frame was used after installation of the strain gauges to correlate applied vertical load levels to the voltages induced in the strain gauges by the use of a Wheatstone bridge circuit. Figure 3 shows photographs of different stages during the installation of strain gauges.

![Figure 3: Photos showing different steps during the installation of strain gauges](image-url)
4 Drilling and MDD installation

The instrumentation of Amtrak NEC bridge approaches took place in July-August of 2012, whereas the instrumentation at the NS bridges was carried out between 15 October and 1 November of 2013. A special drill-rig was manufactured at the University of Illinois to carry out the drilling process. The drill rig comprised a tripod base, and was mounted on a wooden-triangular base which in turn was clamped to the rails. The rig was mounted with a mechanical winch to facilitate the movement of the hammer drill along two vertical shafts. Steel extensions were manufactured for the drill bits to enable the drilling up to a depth of 5.5-m. The drilling was carried out in small increments of 75-100 mm, and the bit was repeatedly extracted from the hole to remove accumulated soil from the bits, and to clean the drilled hole using compressed air as well as a high-capacity vacuum cleaner. Drilling in such small increments ensured that the substructure layer boundaries could be identified up to a resolution of approximately 25 mm. Layer boundaries were marked upon noticing significant differences in the material type being removed from the drilled hole. Soil samples were collected from different depths during the drilling process for subsequent testing and characterization in the laboratory. Figure 4 shows photographs of different stages during the drilling of MDD holes. After achieving the desired depths, each MDD hole was “lined” using a flexible tube, and individual MDD modules were installed at pre-determined depths. A custom-designed tool was used to install the individual MDD modules, and the voltages recorded by each LVDT were monitored throughout the installation process.

A total of six MDD strings (two at each instrumented approach) were installed at the Amtrak NEC bridge approaches, whereas four MDD “strings,” were installed at the NS bridge approaches. Each MDD “string” comprised five or six LVDT modules depending on the depth of the hole. The instrumented tracks at Amtrak NEC comprised concrete crossties placed at center-to-center spacing of 610 mm, whereas the NS tracks comprised wood crossties placed at 483-mm intervals.

The primary challenge during drilling of the MDD hole involved drilling through the ballast layer, and preventing the loose ballast particles surrounding the hole from falling into the hole. This was accomplished by using expandable polyurethane foam that penetrated into the ballast layer, and created a “stabilized” zone that could subsequently be drilled using a coring drill bit. Photographs of different steps involved in the stabilization of ballast during drilling are presented in Figure 5. Figures 5c and 5d clearly show the expansion of the polyurethane foam within the drilled hole. Covering the top of the hole during this process ensures the permeation of the urethane foam into the ballast layer, which in turn bonds the ballast particles together resulting in a cohesive mass. Drilling of the hole can be continued through this cohesive mass without risking the collapse of individual ballast particles. Figure 6 shows an example boring log created during the drilling process showing the soil types encountered at different depths. This information is subsequently used to fix the positions of individual LVDT modules inside the drilled hole.
Figure 4: Photos showing different stages of drilling for MDD installation
Figure 5: Photos showing different stages of ballast stabilization during drilling

(a) (b) (c) (d)

Figure 6: Sample boring logs created during the drilling process
5 Monitoring track settlement with time

The settlements with time of individual substructure layers at the instrumented Amtrak approaches have been monitored at 1-2 week intervals by collecting data from the offset positions of individual LVDT modules with respect to the initial (zero) position. This task was performed in coordination with Amtrak as it was not possible for the research team members to visit the site at such frequent intervals. Figure 7a shows the settlement records of individual track substructure layers with time for one of the Amtrak bridge approaches (Caldwell street; 24.3 m from the South abutment; west end of the tie). Individual lines in Figure 7a correspond to the “offset” voltages recorded from individual LVDT modules for the first eleven months (327 days) after instrumentation. Figure 7b shows the substructure layer profile for that particular location determined during the drilling. As clearly evident from the figure, LVDT 1 registers significantly higher settlements compared to the other four LVDTs. LVDT 1 was located inside the tie, and represents the deflection within the ballast layer. Accordingly, the data presented in Figure 7a clearly indicates that the track settlement with time at the Caldwell street location was primarily taking place within the ballast layer. Results from the other ties instrumented in the current study, not presented for brevity, also showed similar trends.

![Layer settlement records with time](image1)
![LVDT modules installed](image2)

Figure 7: MDD instrumentation and performance of Caldwell St. Bridge approaches

6 Transient response under train loading

The transient deformations of individual substructure layers are being monitored at the instrumented bridge approaches to quantify the contributions of individual substructure layers to the total deformations. This requires connecting a laptop computer to the signal conditioner amplifier and acquiring data from the individual channels as a train passes over the instrumented ties. The data acquisition is...
accomplished using a National Instruments Analog/Digital signal converter connected to a laptop computer running the CMS continuous data acquisition software. Figure 8a shows an example of the transient deformation time history recorded at one of the instrumented NS bridge approaches under the passage of a freight train. The data in Figure 8 corresponds to a 1711-m long freight train with 88 cars (49 loaded; 39 empty) with a total gross weight of 6729 metric tons. Clearly, LVDT 1, mounted within the tie, registers the highest transient deformations. Peaks corresponding to the passage of each wheel over the instrumented tie are quite distinguishable. Moreover, it is important to note that all the substructure layers registered finite amounts of transient deformation under train loading. This indicates the propagation of stress waves generated by the passing train all the way down to the bottom LVDT module. Figure 8b shows the vertical wheel load time history recorded by the strain gauges for the same train. The transient deformation response within each substructure layer is essential in evaluating the long term performances of the instrumented bridge approaches under loading.

Figure 8: (a) Transient response of individual LVDTs at one of the instrumented bridge approaches under train loading;
(b) Vertical wheel load time history recorded by the strain gauges for the same train
7 Current and future tasks

Performances of the instrumented Amtrak and NS bridge approaches will continue to be monitored till the end of 2014. Based on the initial significant findings from the instrumentation, the research team is currently working with Amtrak to explore the application of selected remedial measures to prevent excessive movement and reorientation of ballast particles, thus reducing the severity of differential movement, and increasing the time gap between successive track maintenance activities. Several different numerical modeling methods, namely: the discrete element method (DEM), finite element method, and finite difference method, are being used to model the instrumented bridge approaches, and identify different mechanisms contributing to the differential settlement. The developed numerical models will help evaluate the effectiveness of different remedial measures in mitigating the differential movement at existing bridge approaches. Finally, new design alternatives will also be proposed to prevent differential movement at newly constructed bridge approaches.

8 Summary and conclusions

This paper presented findings from an ongoing research study at the University of Illinois aimed at investigating different factors contributing to the problem of differential movement at railway track transitions. Five different problematic bridge approaches have thus far been instrumented along Amtrak’s Northeast Corridor and Norfolk Southern’s N-line main line. Multidepth deflectometers (MDDs) have been used to successfully monitor the permanent (plastic) as well as transient deformations of individual track substructure layers. Analyses of the track settlement (or permanent deformation) data at the instrumented Amtrak sites so far established the ballast layer to be a primary source of differential movement. All installed LVDT modules registered certain amounts of deformation under train loading, corresponded to the passage of individual wheel sets over the instrumented crossties. Strain gauges mounted on the rail are being monitored to correlate the applied vertical wheel loads to corresponding deflections recorded by individual LVDTs.

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