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Investigation of differential movement at railroad bridge approaches through geotechnical instrumentation

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Abstract: 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. This differential movement is especially problematic for high speed rail infrastructure as the 'bump' at the transition is accentuated at high speeds. Identification of different factors contributing towards this differential movement, as well as 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. This paper presents the research framework and initial instrumentation details from an ongoing research effort at the University of Illinois at Urbana-Champaign. Three bridge approaches experiencing recurrent geometry problems were instrumented using multidepth deflectometers (MDDs) and strain gages to identify different factors contributing to the development of differential movements.

Key words: Track transition, Differential movement, Bridge approach, Multidepth deflectometer (MDD), Bump at the end of the bridge, Strain gage
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1 Introduction

Track geometry problems at railway transitions are well recognized in the United States as well as Europe. The Association of American Railroads (AAR) reported an annual expenditure of approximately USD 200 million to maintain track transitions (Sasaoka *et al*., 2005; Hyslip *et al*., 2009), whereas more than USD 110 million was spent annually on transition zones in Europe by 1999 (Hyslip *et al*., 2009; ERRI, 1999). Although tunnels, special track work, and highway/rail at-grade crossings are all examples of railway track transitions, the most common transition problems are often encountered at bridge approaches. Due to drastic differences in substructure and loading conditions, the tracks on a bridge deck undergo significantly lower deformations under loading compared to the approach tracks. This sudden change in track deformation behavior at the transition point results in extreme dynamic loading conditions and ultimately leads to rapid deterioration of the track and bridge structural components. These structural damages often manifest themselves as track geometry defects.

Differential movement at bridge approaches often results in the development of a "bump" usually within 15 m from the bridge end (Plotkin and Davis, 2008). A survey of railroads in North America, Australia, and Europe conducted in 2006 (Briaud *et al*., 2006) indicated that approximately 50% of bridge approaches developed a low approach, usually 6 to 102 mm in depth and 1.2 to 15.2 m in length, that adversely affected ride quality. Development of sudden dips adjacent to the bridge deck increases the dynamic impact loads significantly. Koch (2007) reported vertical dynamic loads above twice the static wheel load level for coal gondolas at track transitions. Read and Li (2006) concluded that the bump problem at track transitions was more significant as a train moves from a high-stiffness track to a low-stiffness track. Therefore, the problem of differential movement is more critical at the exit end of a bridge, whereas the sudden track stiffness increase at the bridge entrance leads to rail surface fatigue, tie deterioration and rail seat pad deterioration.

To minimize differential movement at railway transitions used by joint high speed passenger and freight routes in the United States, a new research study sponsored by the Federal Railroad Administration (FRA) has been undertaken at the University of Illinois at Urbana-Champaign in collaboration with several industry research partners. The research project scope involves monitoring and numerical modeling of new and existing track transitions to develop cost-effective designs and providing safe and cost effective solutions to mitigate the problem by eliminating or minimizing differential movements. This paper presents first a summary of previous research studies investigating track transition problems followed by a list of individual research tasks currently being undertaken under the scope of the ongoing research study. Historical track geometry data for three bridge approaches experiencing recurrent differential movement problems are subsequently presented along with identified locations for instrumentation. Instrumentation plans developed for the selected transitions are outlined, and different sensors installed for performance monitoring are introduced. Finally, future project tasks involving instrumentation of other track transitions and numerical modeling of the instrumented field sites are outlined as the proposed research approach for accomplishing the overall objective to develop new design and rehabilitation techniques for improving track transition performance.

2 Previous research findings on differential movement at railroad track transitions

Although the problem of 'bump' development at highway bridge approaches has been extensively studied (Zaman *et al*., 1991; Stark *et al*., 1995; Briaud *et al*., 1997; White *et al*., 2005), only limited research studies have focused on mitigating the differential movement at railway track transitions. These few studies investigating track transition problems have primarily focused on preselected mitigation techniques and presented test section and parametric analyses results on the effectiveness of these remedial measures (Sasaoka *et al*., 2005; Read and Li, 2006; Nicks, 2009). Important findings from some of these past research studies are summarized below along with an emphasis on the effectiveness of different remedial measures adopted.

2.1 Factors contributing to differential movement at track transitions

The relative importance assigned by researchers to different mechanisms contributing to differential movement at track transitions varies from one study to another. From the investigation of four bridge approaches with concrete ballast-deck bridges and concrete ties, Li and Davis (2005) reported inadequate ballast and subballast layer performance to be the primary cause of track geometry degradation. Using settlement rods installed in the test sections, they observed no significant subgrade movements, but instead reported significant track geometry deterioration for a site with cement-stabilized backfill. On the other hand, Selig and Li (1994) identified subgrade stiffness to be the most influential parameter affecting the moduli of ballasted tracks. As track transition problems are often related to the stiffness of the approach track bed, this would indicate that the subgrade layer plays the most significant role in governing the differential movement at track transitions.

In spite of different factors being identified as most critical in affecting the differential movement at track transitions, a general consensus exists regarding the list of plausible mechanisms. Sasaoka and Davis (2005) attributed track transition problems to three primary factors: (1) differential settlement, (2) differences in stiffness characteristics, and (3) discrepancies in track damping properties between adjacent sections. Similarly, Li and Davis (2005) listed (1) track stiffness change, (2) ballast settlement, and (3) geotechnical issues as the major causes of bridge approach problems. Nicks (2009) listed the following ten factors identified by researchers as contributing to the 'bump' development at railway bridge approaches: (1) differential track modulus, (2) quality of approach fill, (3) impact loads, (4) ballast material, (5) drainage, (6) damping, (7) abutment type, (8) bridge joint, (9) traffic considerations, and (10) quality of construction.

Note that although most researchers list "track stiffness difference" as an important factor influencing the differential movement and other track deterioration problems at transitions, Plotkin and Davis (2008) used five different analysis methods to conclude that stiffness differences did not play an important role as far as track behavior and ride quality at track transitions were concerned.

2.2 Remedial measures

Several different remedial measures have been suggested by researchers to mitigate differential movement problems at track transitions. Nicks (2009) divided the remedial measures aimed at mitigating bump development at railway bridges into the following interrelated categories: (1) reduce approach settlement, (2) decrease modulus on bridge deck, (3) increase modulus on approach track, (4) reduce ballast wear and movement, and (5) increase damping on the bridge deck. Kerr and Moroney (1993) concluded that most problems at track transitions arise from rapid changes in the vertical acceleration of wheels and cars in the transition zone. Accordingly, they recommended that all remedial measures should aim to reduce the train vertical acceleration at the transition zones, and can be divided into the following three categories (Kerr and Moroney, 1993): (a) smoothing the track stiffness (often represented as 'k') distribution on the "soft" side of the transition; (b) smoothing the transition by increasing the bending stiffness of the rail-tie structure on the "soft" side, in close vicinity of the transition point; and (c) reducing the vertical stiffness on the "hard" side of the transition.

Remedial measures under category (a) include: use of oversized ties, reduced tie spacing, ballast reinforcement using geogrids, hot-mix-asphalt (HMA) underlayment, and use of approach slabs. On the other hand, the most commonly known method under category (b) was developed by the German Federal Railways (DB) and involves attaching four extra rails (two inside and two outside the running rails) to the cross ties (Kerr and Bathurst, 2001). Finally, the primary approach in category (c) involves the installation of tie pads and/or ballast mats to reduce the track stiffness on the "hard" side of a transition point (e.g., on the bridge deck). Using analytical procedures, Kerr and Moroney (1993) engineered pad stiffness to "match" the track running over the bridge to the approach track. Accordingly, a later study (Kerr and Bathurst, 2001) installed these "matched pads" on three different open-deck bridges near Chester (PA), Catlett (VA), and Philadelphia (PA), USA. Field test results subsequent to the installations indicated significant improvements in track geometry near the bridge abutments.

Sasaoka and Davis (2005) tried different methods to alter the track stiffness and damping characteristics on bridge approaches. Installing ties made of different materials, they reported that plastic ties on a concrete span ballasted-deck bridge effectively reduced the stiffness difference at track transitions. Moreover, using rubber pads underneath concrete ties on the bridge deck, they were able to achieve lower track stiffness on the bridge compared to the approach. Through parametric analyses using the geotrack multi-layered elastic program, they concluded that subgrade improvement in the approach and altering tie pad properties on the bridge deck were the most effective methods to minimize track stiffness differences at bridge approaches. Similarly, from dynamic analyses using NUCARS, they concluded that providing extra dampers on the bridge deck could improve the impact attenuation at the transition by up to 30%. Li and Davis (2005) concluded that remedies intended to strengthen the subgrade were not effective for sites where ballast/subballast layers were primarily responsible for the differential movement. In such cases, mitigation techniques such as rubber pads under the concrete ties, or rubber mats on the concrete bridge deck should be used to reduce the track stiffness and enhance the damping characteristics.

Rose and Anderson (2006) presented asphalt underlayment trackbeds as an effective method for improving the performance of track transitions at tunnels, bridge approaches, special trackwork like crossing diamonds, crossovers and turn-outs, as well as at rail/highway at-grade crossings. Placement of a thicker HMA underlayment adjacent to the bridge and a thinner section close to the existing all-granular trackbed reportedly improved performances of both open-deck and ballast-deck bridges. Rose and Anderson (2006) reported four bridge approaches that were rehabilitated using this technique along a Kentucky mainline with over 50 million gross tons (MGT) annual tonnage and a line speed of 80–96 km/h. Over five years since the renewal of these approaches, no resurfacing was needed to correct track geometry.

Apart from the above listed remedial measures, researchers have also suggested converting open-deck bridges to ballast-deck (Hyslip *et al.*, 2009), and stoneblowing (Chrismer, 1990; McMichael and McNaughton, 2003) as alternatives to mitigate track transition problems. Hyslip *et al.* (2009) proposed chemical grouting as a conceptual solution for bridge transition improvement. Note that most of the research efforts listed above primarily focused on the implementation of pre-selected remedial measures and no study was found focusing on field instrumentation and performance monitoring of track transitions to measure settlements of individual track substructure layers. Accordingly, the current FRA-sponsored research study was undertaken to identify the root caused by quantifying the contributions of individual substructure layers to the differential movement problem, and subsequently to develop design and repair techniques for mitigating the same. The following sections describe different tasks currently being carried out under the scope of the ongoing research effort.

3 Identification of track transitions for instrumentation

The primary objective of this task was to identify existing or new track transitions that experience, or are likely to experience differential movement problems. The objective is to monitor the performance of track transitions under different loading environments, namely, heavy haul freight traffic, high speed passenger traffic, and high speed operation of heavy haul traffic on shared corridors. Care was taken during the selection of candidate transitions to include sites where different mechanisms are likely to contribute towards the differential movement problem.

Preliminary analyses of historical track geometry data for several bridge approaches along the Northeast corridor (NEC) of Amtrak near Chester, Pennsylvania (PA) were presented elsewhere (Tutumluer *et al.*, 2012). Subsequent in-depth analyses of the track geometry data for those bridge approaches resulted in the selection of three different bridge approaches than the ones originally selected. Results from such in-depth analyses of track geometry data are discussed in this section.

3.1 Transitions with excessive ballast movement

A problematic portion of Amtrak's NEC, South of Philadelphia near Chester, Pennsylvania, has 8 to 10 bridges with recurrent differential movement problems and rough ride qualities. The NEC is an existing freight and passenger joint use line with high speed passenger trains operating up to a maximum speed of 240 km/h. This segment of the NEC in Chester, PA comprises four tracks, with Tracks 2 and 3 maintained to carry the high speed Acela express passenger trains at 176 km/h. As the problem of differential movement at bridge approaches becomes magnified under high operating speeds of the Acela passenger trains, the current research study primarily 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. Previous analyses of track geometry data (Tutumluer *et al.*, 2012) resulted in the selection of approaches of two bridges (over Madison and Hinkson streets) as candidate locations for instrumentation and performance monitoring. Later on, more detailed analyses of track performance records of the most problematic Chester, PA track transitions identified three approaches of bridges (over Upland, Madison and Caldwell streets) as the primary locations to instrument with multidepth deflectometers (MDDs) and strain gages under the scope of this project.

Historical track geometry data was obtained from Amtrak spanning the 60-month period from Jan. 2005 to Jan. 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. Moreover, ground penetrating radar (GPR) scanning of the tracks was also conducted to identify significant substructure features that might have influenced the selection of a particular bridge approach for instrumentation over others.

Figs. 1–3 present GPR scans, MCO data, and space curve plots for the three bridge approaches (bridges over Upland, Madison, and Caldwell streets) selected for instrumentation. Although the MCO and space curve data represent the time history of track performance spanning the 60-month period, the GPR scans at all the three bridge approaches were conducted in June 2012. The data presented in Figs. 1–3 correspond to Track 3 at the Upland and Caldwell street locations, and Track 2 at the Madison street location. As shown in Fig. 1, The North approach of Upland street bridge clearly indicated recurrent "bump" problems over the 60-month period. The vertical axis in Fig. 1b represents time in months, whereas the horizontal axis represents distance along the track. Significant deviations in the vertical profile are represented by red color, whereas blue colored patches indicate smooth profiles. As shown in Fig. 1b, the North approach of the Upland street bridge showed recurrent red colored patches representing frequent deviations in the surface profile. Similarly, Fig. 1c shows "bumps" in the space curve at distances of 4.6 and 18.3 m from the North abutment. Accordingly, these two locations have been marked and indicated in Fig. 1 by dashed black lines as the locations for instrumentation.

Fig. 2 shows similar plots for Track 2 near the Madison street bridge. The South abutment of Madison street bridge showed recurrent "bump" problems over the 60-month period represented by the recurrent red colored patches in Fig. 2b. The vertical space curve in Fig. 2c shows significant "bumps" at distances of 3.0 and 18.3 m from the South abutment. Accordingly, these two locations have been marked and indicated in Fig. 2 by dashed black lines as locations for instrumentation.

As shown in Figs. 1 and 2, the entrance sides of the approach (train moving from the approach to the bridge) were selected for instrumentation at both the Upland as well as Madison street bridge locations. Accordingly, while selecting the third bridge approach for instrumentation the objective was to identify an approach with recurrent differential movement problems on the exit side of an approach (train moving from the bridge to the approach). Accordingly, Fig. 3 shows the track geometry data for Track 3 at the Caldwell street bridge location.

As can be seen from Fig. 3c, the space curve shows a significant "dip" at a distance of 24.4 m from the South abutment. To evaluate the uniformity of settlement at either ends of a cross-tie, it was decided to instrument both sides of a single cross-tie at a distance of 24.4 m from the South abutment.

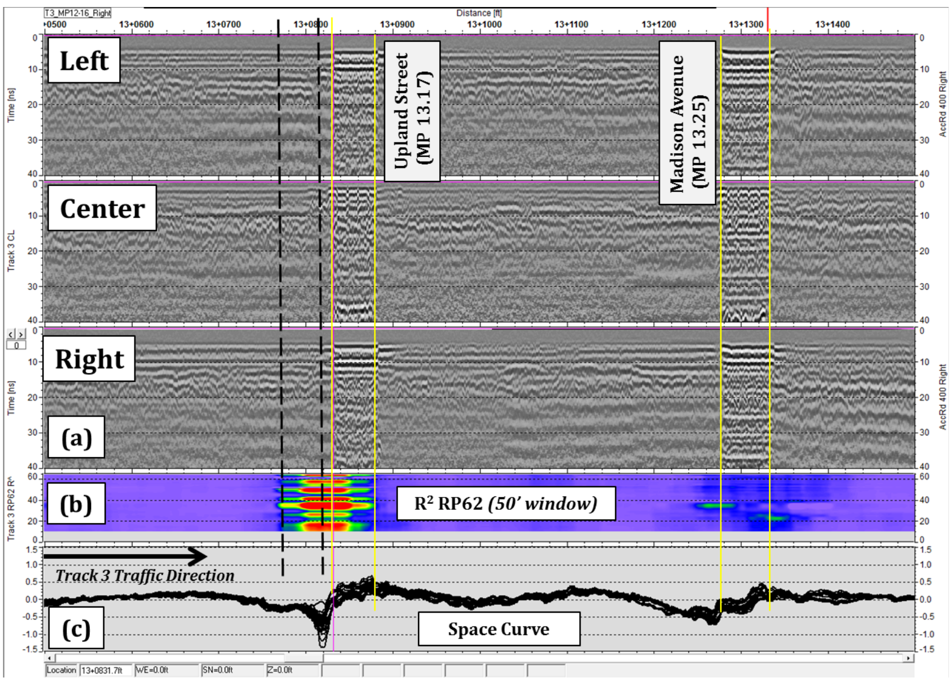

Fig. 1 GPR scans (a), MCO data (b) and space curve (c) for Track 3 at Upland street bridge approach (MDD positions: 4.6 m and 18.3 m from North abutment)

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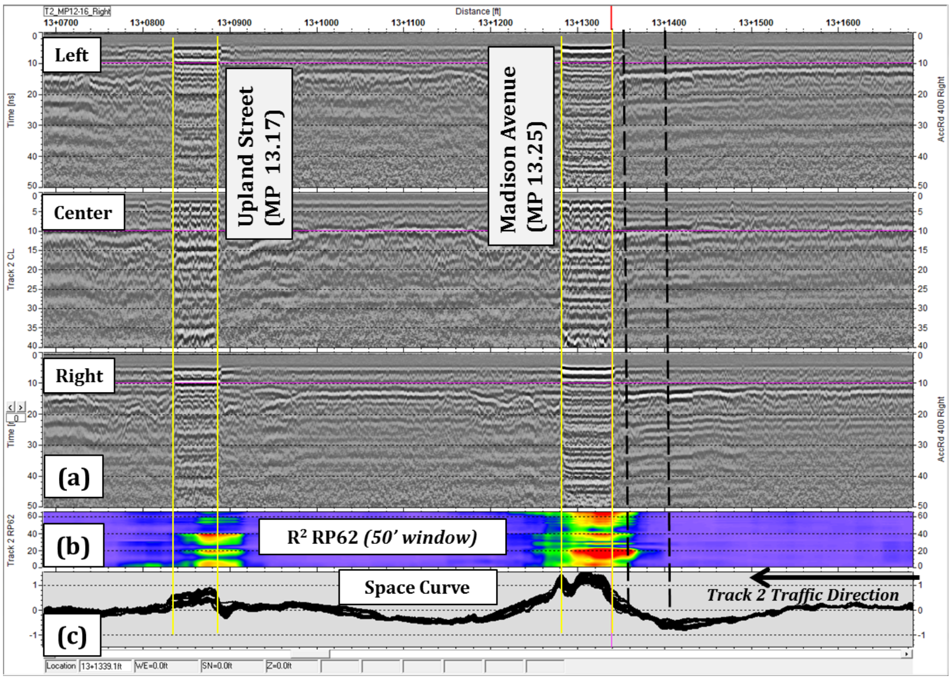

Fig. 2 GPR Scans (a), MCO data (b) and space curve (c) for Track 2 at Madison street bridge approach (MDD positions: 3.0 and 18.3 m from South abutment)

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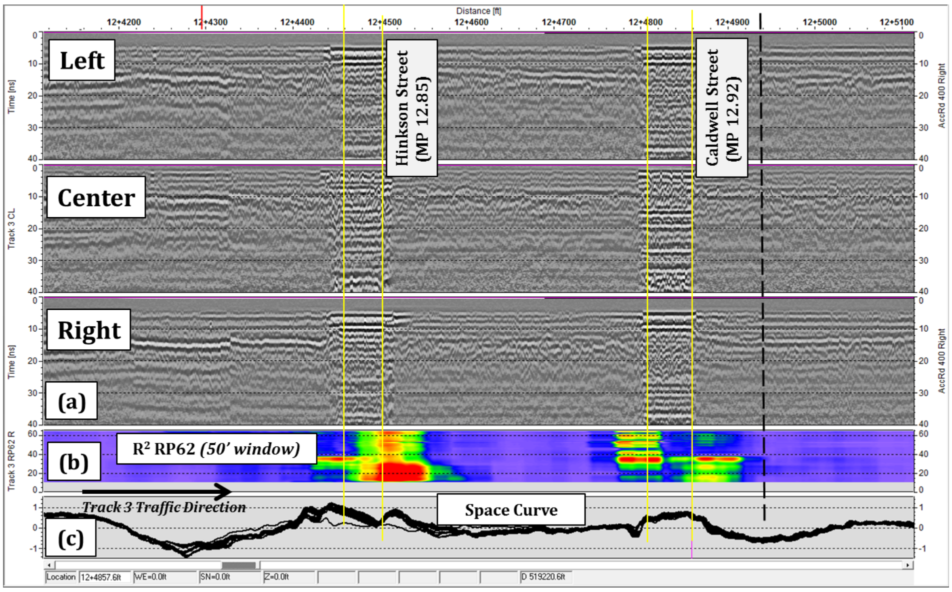

Fig. 3 GPR scans (a), MCO data (b) and space curve (c) for Track 3 at Caldwell street bridge approach (MDD positions: 24.4 m from South abutment)

Note: for interpretation of the references to color in this figure legend, the reader is referred to the web version of this article

4 Instrumentation of selected track transitions

After identification of the track transitions for instrumentation, the next task involved the development of detailed instrumentation plans for each transition. Detailed instrumentation plans for each selected track transition site were developed following the "systematic approach" recommended by Dunnicliff (1993). For identification and quantification of different factors contributing to the differential movement, the primary response parameters of interest in this study are: (1) deformations (both elastic and plastic) of individual track substructure layers, and (2) vertical wheel loads induced by the trains at the instrumented locations. Measurement of vertical wheel loads is particularly important for analyzing the effects of axle load levels and operating speeds on transition performance. Accordingly, the current study selected MDDs and strain gages for measuring individual substructure layer deformations, and train-induced vertical wheel loads, respectively.

4.1 Measuring individual track substructure layer deformations using multidepth deflectometers

MDDs were selected for use in the current study to monitor the elastic and plastic deformations in track substructure layers. The use of MDDs to measure layer deformations in highway pavements was first started in South Africa in the early 1980s (Scullion *et al.*, 1989). Comprising up to six linear voltage differential transducers (LVDTs) installed vertically at preselected depths in a small-diameter hole, an MDD system measures the deformation in each individual pavement/track layer with respect to a fixed anchor buried deep in the ground (DeBeer *et al.*, 1989). Fig. 4 shows the schematic of an MDD module (DeBeer *et al.*, 1989). The clamping nut (Fig. 4) is tightened to displace the steel balls radially, and “clamp” against the inside wall of the borehole at a pre-determined depth. As the layer deforms under loading, these modules register corresponding displacements with respect to the fixed anchor point.

Although several studies in the US have involved the use of MDDs to measure layer deformations in highway and airfield pavements, only two research studies have involved the installation of MDDs to monitor railway track performance (Sussmann and Selig, 1998; Bilow and Li, 2005). Unlike most other sensors used to monitor individual layer performance, MDDs can be installed on an existing pavement and track infrastructure to monitor performance under loading.

Fig. 5 shows the schematic of two MDDs with five modules each installed on either end of a cross-tie. The numbers 1–5 shown in the figure correspond to individual LVDT positions and the five LVDTs are placed at different depths inside the borehole to measure the deflections at (1) bottom of tie, (2) top of ballast layer, (3) top of subballast layer, (4) top of fill, and (5) top of subgrade, respectively. Note that the subgrade layer is conceptually divided into two separate segments: “deformable” and “non-deformable”, respectively. The MDD anchor (A) is fixed in the deep “non-deformable” portion of the subgrade, and movements of individual modules (1 through 5) are measured to quantify the individual layer deformations. Note that the primary assumption associated with an MDD installation is that the anchor point (A) remains stationary and does not move under

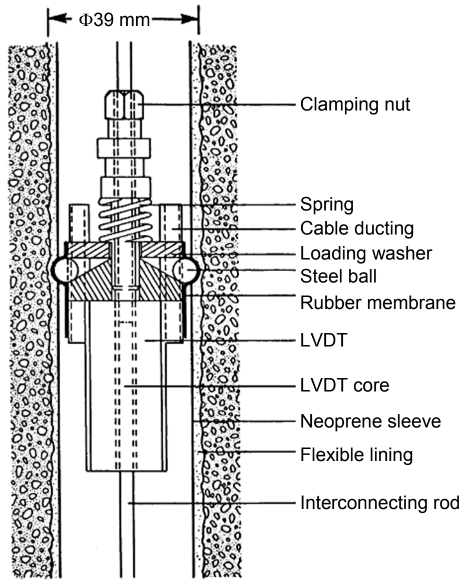

Fig. 4 Schematic of a multidepth delfectometer (MDD) module
LVDT: linear voltage differential transducer

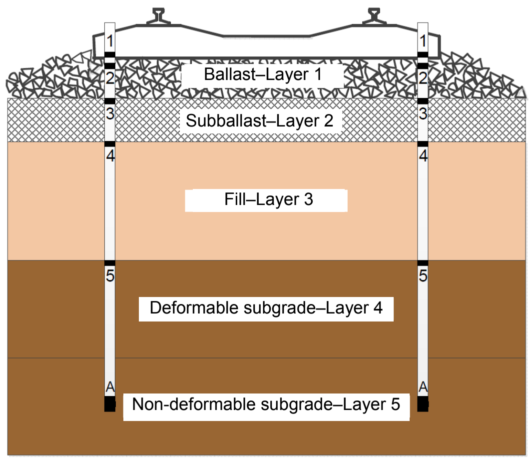

Fig. 5 Schematic of the location of the MDD sensors at layer interfaces (not to scale)

loading. It is therefore critical to ensure that the anchor point is sufficiently deep below the surface and will not be affected by the stresses imposed by train loading. The MDDs installed under the scope of the current study comprised five LVDTs, and were anchored in the subgrade at a depth of 3.0 m below the top of the tie. Note that the depths of individual MDD modules were fixed based on the track substructure layer configuration as determined during the drilling process.

4.1.1 Concept of independent anchoring systems

The current study utilized the concept of "independent anchoring" for measuring individual track substructure layer deformations using MDDs. In the independent anchoring configuration, each MDD module functions as the anchor for the module immediately above it. This concept is schematically represented in Fig. 6. As shown in the figure, the inner core for LVDT No. 5 (attached to MDD module No. 5) is mounted directly to the bottom anchor. Subsequently, the core for LVDT No. 4 (attached to MDD No. 4) is directly mounted on module No. 5. This pattern is repeated for LVDTs 3, 2, and 1. Therefore, except for the bottom-most MDD, all other MDDs have "movable" anchor points. Accordingly, the deflection value measured by each MDD module represents the deflection in that particular layer. For example, the voltage induced in LVDT No. 3 will correspond to the deformation of the 3rd layer (d_3) as shown in Fig. 6. Fig. 7a shows photographs of an assembled MDD module along with its LVDT and inner core, and Fig. 7b shows an MDD module mounted inside the insertion tool ready for installation.

The instrumentation took place in July and August of 2012. The first step of drilling of the MDD holes was carried out in small increments to ensure that the holes were perfectly vertical, and the drill bit did not get stuck in the track substructure. The drill bit was extracted from the hole after every 75–100 mm increments to remove accumulated soil from the bits, and to clean the drilled hole using compressed air. 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. Embankment and subgrade soil properties thus established will be used as inputs for the numerical models developed to predict the transition performance under different rehabilitation measures.

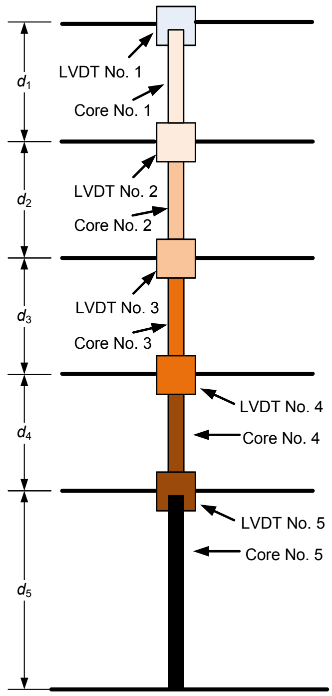

Fig. 6 Schematic showing the concept of MDD installation using the independent anchoring concept (not to scale)

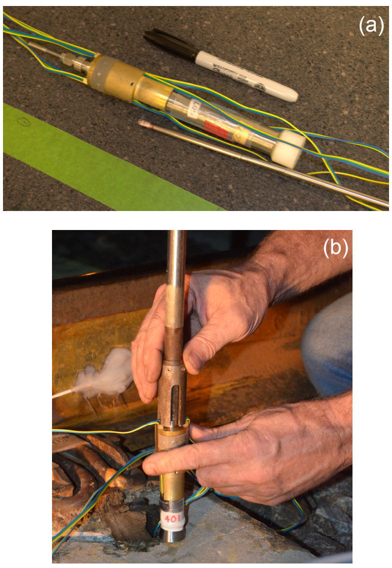

Fig. 7 Photographs of (a) an assembled MDD module along with the LVDT and inner core and (b) MDD module mounted in the insertion tool ready for installation into the drilled hole

4.1.2 Measurement of vertical wheel loads and tie reactions using strain gages

Strain gauges were installed on the rail to measure the vertical wheel load as well as the tie reaction forces. A total of eight strain gauges were installed next to each MDD hole. Fig. 8a shows the picture of a rail section instrumented with "spot-welded" strain gauges. Fig. 8b shows the calibration of strain gages after installation.

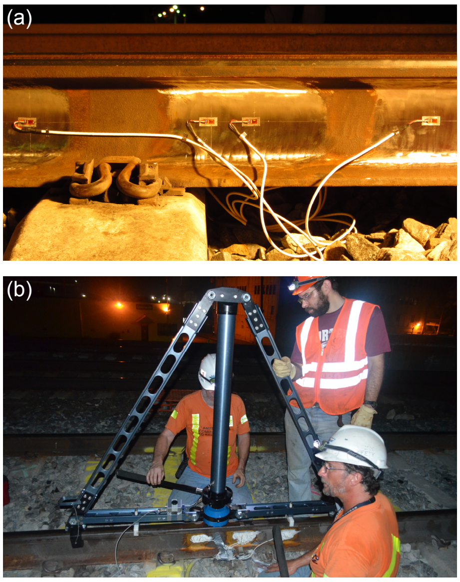

Fig. 8 Photographs of (a) strain gages for vertical wheel load and tie reaction measurements and (b) strain gage calibration on the rails

5 Track substructure layer configurations and locations of MDDs

Table 1 lists the different layer types encountered during drilling of the holes for MDD installation at the three bridge approaches. The reported layer interfaces also correspond to installation depths of the MDD modules with respect to the top of the tie. Note that the top-most MDD (MDD No. 1) was installed inside the tie. As the tie is assumed to move as a rigid body, the depth of the top-most MDD from the top of the tie is immaterial.

6 Preliminary layer deflections under loading

After installation of the MDDs and strain gages, elastic deflections from the MDDs were recorded under train loading. Fig. 9 shows the elastic deflections recorded at the Madison street bridge approach corresponding to the MDD installed at a distance of 3.7 m from the South abutment. Note that although the track geometry records indicated presence of the "dip" 3.0 m from the abutment, site restrictions led to installation of the MDD 3.7 m away from the bridge abutment.

Fig. 9 shows the deflections of individual layers recorded due to train wheel loading. The deflection pulses clearly show the existence of a rest period between the two consecutive car passes. Complete analyses of the track deflection data together with the measured vertical wheel loads from strain gages are

Table 1 Substructure layer profiles for the instrumented bridge approaches

Bridge approach	Location of MDD modules (depths measured from top of the tie)		
	MDD No.	Depth (in)	Layer interface
Upland street bridge (60 ft from North abutment)	1	N/A	Within the tie
	2	19	Clean ballast-fouled ballast
	3	24	Sandy loam
	4	44	Clayey silt
	5	72.375	Top of anchor
Upland street bridge (15 ft from North abutment)	1	N/A	Within tie
	2	19	Clean ballast-fouled ballast
	3	26.5	Sandy loam
	4	63.25	Thin concrete layer
	5	71.75	Top of anchor
Madison street bridge, Track 2 (12 ft from South abutment)	1	N/A	Within the tie
	2	18	Clean ballast-fouled ballast
	3	23.50	Hard pan
	4	46	Gray sandy loam+cinder
	5	73.125	Top of anchor
Madison street bridge, Track 2 (60 ft from South abutment)	1	N/A	Within the tie
	2	20	Clean ballast-fouled ballast
	3	39	Water table
	4	53.375	Fat clay, very wet
	5	67.5	Top of anchor
Caldwell street, Track 3 (80 ft from South abutment, West end of tie)	1	N/A	Within the tie
	2	19	Clean ballast-fouled ballast
	3	25	Brown silty sand
	4	38	Silty clay
	5	74	Top of anchor
Caldwell street, Track 3 (80 ft from South abutment, East end of tie)	1	N/A	Within the tie
	2	19.5	Clean ballast-fouled ballast
	3	27.5	Possible thin HMA layer
	4	36	Clay
	5	73.75	Top of anchor

Note: 1 ft=30.48 cm, 1 in=2.54 cm

currently undergoing, and will be reported in subsequent publications. Moreover, periodic offsets are also being recorded from the MDDs to measure the accumulation of permanent (or plastic) deformation in individual track substructure layers.

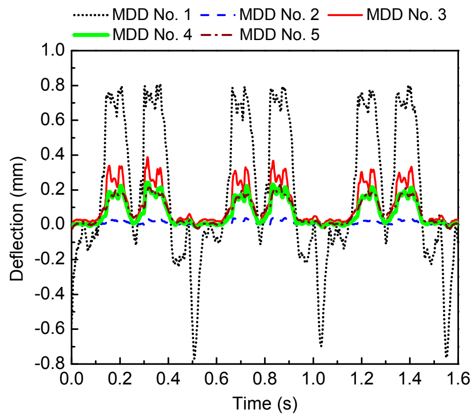

Fig. 9 Preliminary deflection data recorded from the MDDs under the passage of an Acela express train

7 Future research tasks

7.1 Instrumentation of track transitions with potential for excessive fill and/or subgrade movement

Union-Pacific (UP) Railroad is currently reconstructing its tracks from Dwight, Illinois to St. Louis, Missouri in the US as part of the American Recovery and Reinvestment Act (ARRA) High-Speed Intercity Passenger Rail Program (HSIPR) Track 2A Project. This reconstruction involves a major high speed track upgrade that includes improving grade crossings and constructing approximately 38.1 km of new track with nearly 60 bridges. Heavy-haul freight trains along with future high speed passenger traffic on these tracks are likely to induce excessive fill and subgrade settlement in the newly constructed bridge embankments. Two bridge approaches (11 km apart, at mileposts MP 238.47 and MP 231.52) have been identified along this route for instrumentation and performance monitoring to quantify the sources of differential movement. Instrumentation of these bridges is scheduled for the spring and summer of 2013.

7.2 Numerical modeling of instrumented track sections

The final phase of this study will focus on developing calibrated numerical models of the monitored track transitions to predict the performances of different design and/or rehabilitation techniques. This would allow the contributions of the ballast, fill, and subgrade to the total differential movement to be studied in various configurations.

The numerical analyses will use the finite difference program FLAC^{TM} and the finite element program PLAXIS^{TM} (http://www.plaxis.nl/) to simulate the behavior of structures built of soil, rock, or other materials that undergo deformation when their yield limit is exceeded. In addition, this research project will also utilize BLOKS3D discrete element method (DEM) program which has been under development at the University of Illinois to generate a model for the mass of typical railroad ballast. Through modeling different design and rehabilitation techniques implemented at the instrumented track transitions, and using the field data to check the accuracy of the models, this research study will recommend the most efficient alternatives for mitigating differential movement problems at track transitions.

7.3 Selection of remedial measures

Selection of different remedial measures to mitigate the differential movement problem at track transitions will be carried out after careful analyses of the instrumented track response. Depending on the availability of project funds as well as track access, the rehabilitated track transitions will subsequently be instrumented and monitored to evaluate the effectiveness of the remedial measures.

8 Summary

This paper presented the research framework, initial task activities as well as preliminary findings from a recent FRA-sponsored research study (Task Order: FRA BAA-2010-1) in the US aimed at investigating and mitigating the problem of differential movement at railway track transitions. Candidate track transitions identified for instrumentation and performance monitoring were highlighted, and details of instrumentation at three bridge approaches were presented. Finally, future project tasks involving field investigation and numerical modeling of the instrumented track transitions were outlined. Through combination of field instrumentation and numerical modeling of new and existing track transitions, this research project aims to develop design methods to mitigate the differential movement problem at such locations.

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