Transportation Infrastructure Durability Center AT THE UNIVERSITY OF MAINE

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Detailed Finite Element (FE) Model of Devon Railroad Bridge East Abutment Span Under Typical Operation

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Introduction

Most railroad bridges in the United States were built in the late 19th and early 20th century using outdated design codes and technology. Although those bridges still operate under periodic inspections and enforced rating plans, they often exhibit an unusual dynamic response due to wear and tear owing to their old age.

Figure 1: Devon Bridge span 7th, deck (left), and elevation (right)

Objectives

With the approaching life expectance of most bridges in the United States, it is essential to establish a methodology to evaluate the structural condition of existing bridges using cost-effective techniques. Therefore, there is a need to understand the bridge's behavior better. This study uses a different passenger vehicle to investigate long-span open-deck railroad truss bridge structural response using a computer model.

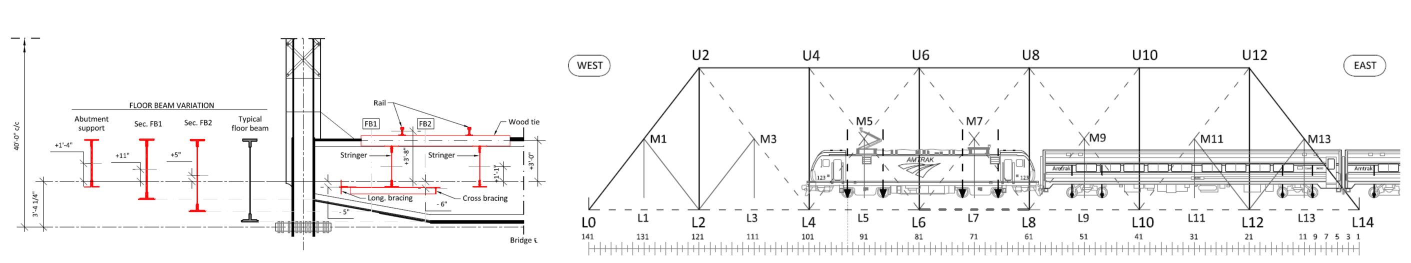

Figure 2: Devon Bridge Model assumptions, deck (left), and elevation (right)

Devon Bridge – Span 7th

Devon bridge is located over the lower Housatonic River between Milford and Stratford, Connecticut. The bridge is a part of the Northeast corridor, the busiest passenger line in the nation. It operates primarily under regular passenger trains such as Metro-North M8, Amtrak Acela, and Amtrak Regional. The FE model was created using ANSYS Workbench® to replicate the span 7 of Devon bridge, an open-deck Baltimore truss with 66.32 m (217'-7") of span length. A series of triangular step forces with a specific spacing has been used to represent different types of vehicles.

Methodology

The 3D bride span model was created using 483 members in wire elements, with 33 different cross-sections, such as eyebars, stringers, floor beams, and diagonals. Figure 3 shows the mesh of the half-span of the bridge, the wire elements member on the left, and the 3D render view on the right.

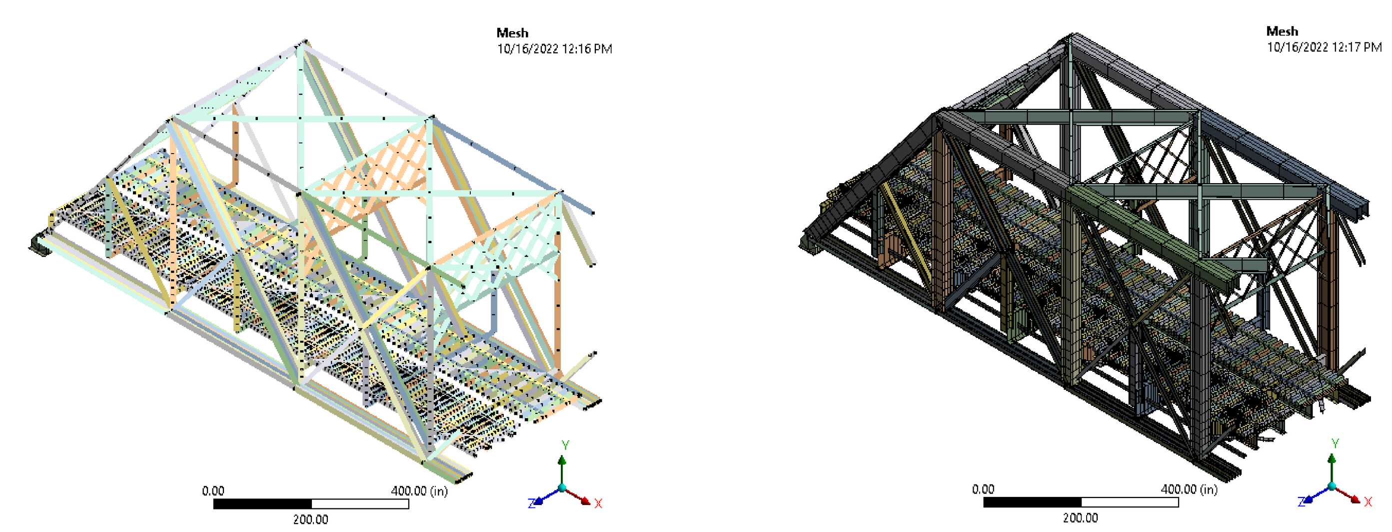

Figure 3: FEM 3D Model meshed half section, Wire elements model (left), Render model (right)

Special attention has been given to eyebar details during the modeling phase (figure 4 left). The eyebar was modeled based on the original drawing's specifications (figure 4), such as the cross-section and the spacing:

- All eyebar member was assigned to trusses under tension-only condition.
- The FE model did not account for the tear and wear of the eyelets of the eyebars.

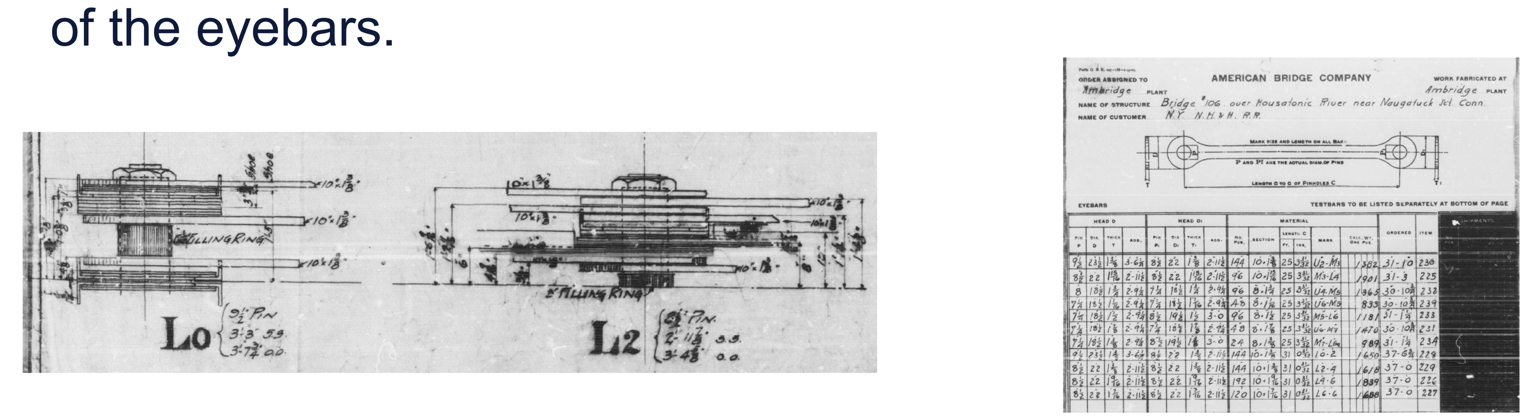

Figure 4: Eyebar detail original drawings, node detail (left) and eyebar list (right)

The FE structural analysis represents and analyses the bridge's static and dynamic response using the most common train composition, the Metro-North Railroad M8, the Amtrak Regional, and the Amtrak Acela. Figure 5 shows the triangle step force model used to represent the vehicle axle load and spacing.

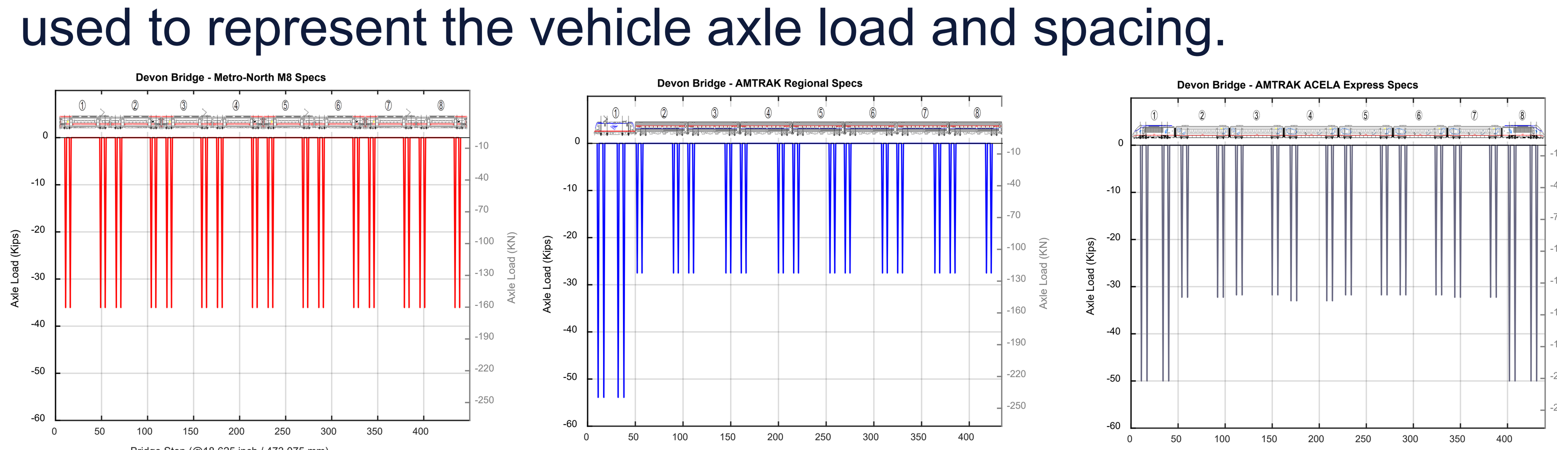

Figure 5: Axle load in triangle step load: MTNR M8 specifications (left), Amtrak Regional specifications (middle), and Amtrak Acela (right)

Results and Discussion

The data collected during the field test using a laser Doppler vibrometer (LDV) was converted to displacement time variation and compared with FE results from static and transient models. Figure 6 shows the comparison of vertical displacement results from *Train 3*, *Train 5*, and *Train 7*, all traveling east to west, using FE model static and transient analysis with field LDV test data. *Train 3* is typical Amtrak Regional train composition, and vertical displacement was recorded from node L12 (figure 2 right). *Train 5* is a regular Amtrak Acela train, and *Train 7* is an eight-couches Metro-North Railroad M8 train composition. The vertical displacement from both trains was recorded from the midspan of the floor beam on member L10 (figure 2 right).

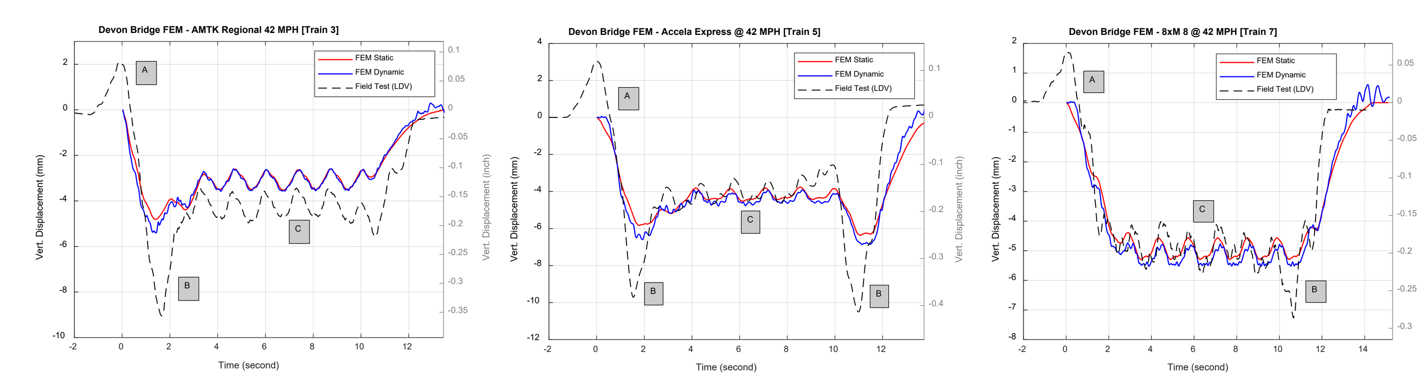

Figure 6: Vertical displacement FEM static (red), dynamic (blue), and LDV field test (dashed), Train 3 (left), Train 5 (middle), Train 5 (right)

Based on the comparison of the FE model results and the field test data, below are the key observations:

A. The correlation of the field test data with the FE analysis has shown that the uplift occurs when the train is outside the bridge span.

B. The maximum magnitude from the field test and FE modal have shown a significant difference; this behavior still needs to be fully understood and will be investigated.

C. The typical operation frequency from the field test data and FE model has shown agreement in magnitude and behavior.

Conclusions

This presentation will help to enhance railroad bridge safety by understanding eyebar bridges' dynamic behavior and the benefits of a detailed FE model for railroad bridge structural analyses.

Acknowledgments

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