Steel Structures Research Update

# Innovative Steel Deck System for Highway Bridge Applications

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## INTRODUCTION

Ongoing research on innovative steel bridge decks is highlighted. This study, currently under way at the University of Kansas, is led by Dr. William Collins, Associate Professor in the Department of Civil, Environmental, and Architectural Engineering. Dr. Collins's research interests include fatigue and fracture of metallic structures; bridge design, fabrication, construction, and performance; and evaluation and preservation of historic structures. Among Dr. Collins's accolades are the Robert J. Dexter Memorial Award, a Fulbright Scholar Award to conduct fracture mechanics research in Finland, and the AISC Milek Fellowship. The four-year Milek Fellowship is supporting this research on innovative steel deck systems for highway bridge applications—the first Milek Fellowship project to focus on bridges. Selected highlights from the work to date are presented, along with a preview of future research tasks.

## BACKGROUND AND MOTIVATION

Steel decks offer potential benefits but have seen limited use on bridges. Low weight, inherent modularity, and improved durability are among the advantages to using steel bridge decks. High initial costs and challenges with connections and other details are potential barriers to adoption.

Steel bridge decks are typically limited to specific applications. Data from the National Bridge Inventory (NBI) shows that "steel decks are found on less than three percent of the more than 600,000 highway bridges in the United States" (FHWA, 2019). Steel orthotropic decks are most often used on bridges that have self-weight as a major factor in design. These include bridges with a weight constraint, movable bridges, and long-span structures.

Advantages for steel bridge decks include their inherent modularity and their relative light weight. Dr. Collins's conversations with bridge owners reveal that they are interested in deck systems that are easy to fabricate and suited to rapid construction. These discussions align with accelerated bridge construction and other efforts to increase the speed of designing, fabricating, and erecting steel structures (e.g., Mellon et al., 2021; Medlock et al., 2022). Modular construction suits accelerated bridge construction methods, and cost savings can be realized from the reduced construction time. Steel bridge decks are lighter than traditional concrete and precast bridge decks (Mangus, 2005), potentially resulting in reduced superstructure demand for new structures and reduced or eliminated weight restrictions for existing structures. A reduction in weight can also reduce transportation and construction costs.

Steel bridge decks may be a viable option for improved durability as well as performance of bridges in rural areas. Steel decks would not have the issues seen with concrete deck cracking and reinforcing steel corrosion (ASCE, 2017). Meanwhile, the inability to consistently meet material specifications causes issues with concrete in rural areas where mobile batch plants are commonly utilized.

Barriers to adoption of steel bridge decks include fabrication requirements and high initial costs. Past issues with orthotropic bridge deck performance motivated difficult and expensive fabrication requirements (McQuaid and Medlock, 2005). As a result, steel bridge decks are typically more expensive than their conventional cast-in-place and precast bridge decks.

The potential benefits motivate the development of a steel deck system to compete with traditional cast-in-place and precast concrete bridge decks. In addition to cost savings resulting from their light weight and inherent modularity, steel decks could eliminate the need for cross frames and further reduce costs. Steel decks oriented perpendicular to the girder would help with load sharing and potentially replace the cross frames as lateral bracing.

Initial cost, connections, and other details challenge the development of a competitive steel bridge deck system, and any fatigue-prone details must be addressed. Other major considerations include drainage systems, selection of toppings/overlays, barrier rails and their connections to the deck, and panel-to-panel connections. Dr. Collins and his team will address these challenges in their design and evaluation of an innovative, lightweight, modular steel deck system.

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## PROPOSED RESEARCH AND DELIVERABLES

Dr. Collins's research team has a comprehensive plan to develop a competitive steel deck system. The four-phase plan seeks to create an easy-to-fabricate, lightweight, and modular deck system; address potential barriers to implementation; and produce design recommendations and specifications.

An overarching goal for the research is to develop a steel deck system that is competitive with respect to fabrication cost, life-cycle cost, and structural performance. An additional, expected benefit is the deck system's suitability for rapid construction of a variety of bridge spans and configurations. Specific objectives are to:

1. Develop a steel deck system that is competitive with conventional cast-in-place concrete decks for use in highway bridges. The steel deck system should be lightweight, modular, and easily fabricated from readily available rolled sections.
2. Conduct a comparative life-cycle cost analysis for a highway bridge utilizing the steel deck system and a conventional, cast-in-place concrete deck.
3. Address potential barriers to widespread steel deck adoption through development of critical details and experimental evaluation. Details will include panel-to-panel connections, girder-to-deck connections, and barrier rail connections to the deck. Evaluation will include experimental testing of some connections as well as full-scale panel fatigue testing.

The research is organized into four phases, from literature review to final deliverables. In Phase I, the team surveyed existing steel deck options as well as systems proposed as alternatives to concrete decks. Initial analyses on the two most promising all-steel candidates informed the selection of one steel deck system for further development. Phase II will be focused on design and cost comparison of a bridge with an all-steel deck and one with a conventional cast-in-place concrete deck. The life-cycle cost evaluations will include estimates for fabrication, shipping, erection, scheduled inspections, and maintenance. In Phase III, the research team will conduct analytical and experimental evaluations of the proposed steel deck system. Finite element analysis and a hot spot stress (HSS) approach will be used to study fatigue behavior. The team's plans include physical testing of component-scale and full-scale panels, fatigue tests, and proposed panel-to-panel connection details. Additional testing may be conducted on barrier rail-to-deck or other connections. Phase IV will complete the research with design recommendations and a final research report. The team anticipates proposed changes to design codes and specifications. The research may impact the AASHTO *LRFD Bridge Design Specifications* (AASHTO, 2020); the FHWA *Manual for Design, Construction, and Maintenance of Orthotropic Steel Bridge Decks* (FHWA, 2012); and/or the AASHTO/AWS D1.5 *Bridge Welding Code* (AWS, 2010).

## LITERATURE AND EXISTING SYSTEMS REVIEW

The team researched existing bridge deck systems and design criteria. Their literature review summarized the benefits and challenges for existing and proposed deck systems. Additional considerations included bridge components and details such as barriers and their connections to the bridge deck.

### Alternative Bridge Decks

The team explored existing concrete, timber, steel and polymer decks. These alternatives present potential advantages and disadvantages, often related to weight, cost, durability, and performance. All provide a comparison and further inform the development of a competitive steel bridge deck system.

Cast-in-place and precast concrete decks may have issues with weight, durability, and performance. As mentioned previously, concrete decks are heavier than the steel decks being considered, and the cast-in-place decks raise concerns about durability and inconsistencies with concrete mixes in rural areas. Precast concrete decks have advantages of controlled casting and curing off site that can be performed in advance and faster construction times with modular, one-way slab, components that can use conventional steel reinforcement or prestressing. However, the connections between precast panels and panels to girders are cast-in-place, adding time and cost to bridge construction.

Timber decks are lightweight but may see limited use on highway bridges due to applicability, performance, and maintenance. Timber decks are typically used for pedestrian bridges and low-volume, short-span vehicular bridges. Different configurations over time have evolved from large sawn stringers with transverse deck logs; to smaller, closely spaced stringers; to longitudinal decks over transverse spreader beams. Nail-laminated decks were once relatively common, but they presented problems related to proper nail placement, warping and nonuniform bearing of decks on the spreader beams, and the resulting uneven load-sharing and crushing of the wood around nails that adversely affected load transfer. Prefabricated decks were developed, moving from nail-laminated to glue-laminated (glulam) members. A challenge with glulam decks is proper shear transfer between adjacent panels. One solution involves passing high-strength steel rods through holes in the deck panels and post-tensioning the panels together. However, the steel rods must be regularly retensioned due to creep in the wood.

All-steel bridge deck systems such as open grid steel decks and orthotropic decks are good options for temporary structures and bridges needing lightweight decks (Mangus, 2005) but may suffer from poor user experience and high cost. Ride quality and excessive noise are problems noted for open grid decks. Safety concerns include loss of traction on a wet deck surface. Meanwhile, historically poor fatigue performance of orthotropic decks has resulted in stringent design and fabrication requirements, leading to high costs (McQuaid and Medlock, 2005).

Sandwich plate system (SPS) decks were explored but not selected due to cost and durability concerns. The proprietary SPS decks consist of a rigid polyurethane elastomer core and two metal face plates—a steel-fiber reinforced polymer (FRP) composite. In a comparative study, Kennedy et al. (2002) found the performance of SPS decks to be comparable to that of orthotropic steel decks. A benefit for SPS decks is that less welding would be required, presumably resulting in fewer fatigue issues. However, the long-term durability of the SPS decks was in question. It should be noted that all-FRP decks were also explored, given benefits of their light weight and corrosion resistance (O'Connor, 2013). However, these decks raise concerns about long-term durability and degradation of elements exposed to harsh environmental conditions (Kassner, 2004).

Corrugated core steel sandwich panels (CCSSP) present an interesting option with fabrication challenges. The CCSSP decks consist of steel plates with a continuous corrugated core or multiple single-wave channels. CCSSP boast a high stiffness-to-weight ratio (Nilsson, 2017). The panels are fabricated using laser beam welds (LBW) or hybrid-laser arc welds (HLAW) depending on the core configuration. Challenges to CCSSP deck adoption may include obtaining or manufacturing the corrugated core and finding a fabrication facility with the welding capabilities.

## Bridge Decks Selected for Further Study

The team selected HSS sandwich panel and inverted WT decks for further study. These two options aligned best with the objectives for a cost-competitive, lightweight, modular deck system utilizing primarily rolled sections. The potential benefits and challenges are summarized.

Selection of an HSS steel sandwich panel deck was based on performance, production, and speed of construction. Rectangular HSS shapes form the core of a sandwich panel with top and bottom plates (Figure 1). Standard, hot-rolled sections already meet ASTM specifications, thus reducing quality-control concerns, and the use of standard shapes facilitates deck production. The HSS were selected over other shapes because of their relatively high shear resistance and torsional rigidity. The structurally efficient HSS sandwich panel deck is envisioned as a prefabricated, modular system that can be designed with minimal field connections to the girders. Passarelli (2011) proposed a shop-welded, field-bolted, grouted deck to girder connection.

Passarelli (2011) investigated laser beam welds (LBW) and hybrid-laser arc welds (HLAW) for the modular HSS panels and recommended HLAW based on the ability to handle fit-up gaps between components and fatigue performance. However, Pasarelli noted that neither weld process is widely adopted. Also, welding within the closed spaces of the sandwich panel creates challenges for fabrication and inspection.

The inverted WT deck system is another selection based on performance, production, and speed of construction. This system uses inverted WT sections with a steel top plate (Figure 2) and was originally developed by Paterson and Hamadani (2021) for railway bridge applications. As with the HSS deck, the use of standard shapes is expected to facilitate production as a prefabricated, modular system. The WTs are fillet-welded to the top plate. Preliminary models by Paterson and Hamadani demonstrated how the WT deck system could be designed to satisfy current design standards. In contrast to the HSS sandwich system, the inverted WT deck is an open system, eliminating any potential limitations for in-service inspections.

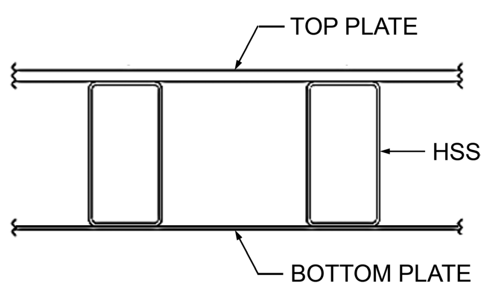


*Fig. 1. HSS sandwich panel deck.*

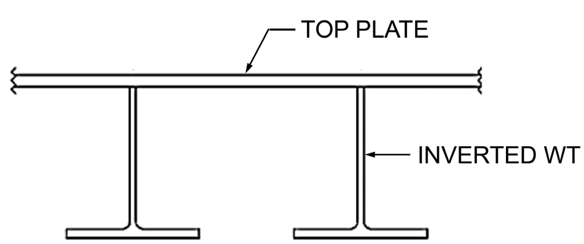


*Fig. 2. Inverted WT deck.*

### Design and Other Considerations

Design criteria and other considerations for construction and in-service performance were gathered by the research team. Deflection limits and maximum allowable fatigue stresses for orthotropic decks were noted (FHWA, 2012). The literature review included types, installation, and maintenance of wearing surfaces. Also considered were types of barriers, their connections to the deck, and performance. Review of these components and their details continues throughout the project, with applicability to the proposed steel deck system.

## DECK SYSTEM DEVELOPMENT AND ANALYSIS

The team analyzed the selected HSS and inverted WT decks to inform the sizing of the plates and members. Preliminary models were developed and parametric studies conducted. Fatigue stress and deflection limits helped to define proper member spacing and other dimensions.

The decks were sized to be modular and consistent initially with previous studies. For modular construction, the preliminary 8-ft-wide panel could be delivered to site on a standard truck. Based on Passarelli (2011), HSS8×4×3⁄16 members were used with a 5⁄8-in.-thick top deck plate and 3⁄16-in.-thick bottom plate. For an inverted WT deck panel with comparable depth and moment of inertia, WT8×20 members and a 5⁄8-in.-thick top plate were used.

The top plate modeling and loading considered truck wheel loads between members. This study used the front axle load of 16 kips for the HS20 design truck, reduced as appropriate for fatigue limit states (AASHTO, 2020). The wheel loads were simply represented as two 8-kip concentrated loads spaced 6 ft apart. The top plate was modeled as a three-span continuous deck with a 20 in. width corresponding to the HS20 wheel area. For this initial evaluation, a single concentrated wheel load was applied to the center of the interior or end span. The HSS or inverted WT members were modeled as simple supports with center-to-center member spacing ranging from 8 in. to 48 in., in increments of 4 in. Maximum top plate deflections and stresses were recorded for both positive and negative moment regions.

HSS and WT member stresses and deflections were evaluated for the truck loading causing transverse moments in the deck system. The team analyzed a single-span, simply supported deck, and two- and three-span continuous decks. The three deck configurations were loaded with one or more truck axles in various configurations, and girder spacing for these analyses ranged from 8 to 14 ft in 2 ft increments. Positive and negative moment values were considered when evaluating maximum stress and deflection demand.

### Top Plate Analyses

Five top plate thicknesses were evaluated along with member spacing. The loading and resulting moment values discussed previously were used to determine the spacing and plate thicknesses needed to satisfy stress and deflection limits (FHWA, 2012). Top plate thicknesses of ½, 5⁄8, ¾, 7⁄8, and 1 in. were used for the elastic stress and deflection calculations.

The resulting stresses and deflections were compared for top plate thickness and member spacing for the HSS and inverted WT decks. Figure 3 shows results for top plate stress (ksi) versus member spacing (in.) for an inverted WT deck configuration. A 33 ksi fatigue stress limit (FHWA, 2012) is marked with a dotted line. Top plates that are 7⁄8 in. or thicker satisfy the limit for all member spacing cases evaluated. Thicknesses of ½, 5⁄8, and ¾ in. exceed the limit for member spacing of 20, 28, and 40 in., respectively. Top plate stresses are slightly lower for HSS decks because of the shorter clear span. For deflections, similar trends were observed; the 7⁄8 in. and thicker top plates again satisfied the limit for all member spacing values.

### Member Analyses

The HSS and inverted WT members were evaluated primarily for fatigue and serviceability. Yielding, local buckling, and lateral-torsional buckling were also considered for the 14 ft girder spacing and original HSS8×4×3⁄16 and WT8×20 members. Members used in both systems were adequate for these limit states. The preliminary stress analysis focused on a single member of the deck system, neglecting load sharing that may occur. For the deflection analyses, additional member sizes were considered at nominal depths ranging from 6 in. to 10 in., but with weights comparable to that of the original WT8×20 (20 lb/ft). All members satisfied the 10 ksi fatigue stress limit and the deflection limit (FHWA, 2012) for the 8 ft girder spacing, and most all members satisfied these limits for the 10 ft girder spacing. At larger member spacing, an increased moment of inertia due to the top and bottom plate thicknesses was needed for the 12 and 14 ft girder spacing. Stresses and deflections in the HSS deck panels were generally lower than those in the WT. This was attributed to the presence of the bottom plate in the HSS deck panel. Figure 4 shows the trends in deflection (normalized to the deflection limit) versus member spacing for a 12 ft girder spacing. As expected, the decks with deeper members satisfy the deflection limit for a wider range of member spacing.

### Panel Weight

Panel weight was also evaluated in a parametric study. Member depth, member spacing, plate thickness, and girder spacing were varied. Nominal member depths again ranged from 6 to 10 in. for six different HSS and six WT sections. Increments of 4 in. were used for member spacing ranging from 8 to 36 in. Girder spacing of 8, 10, 12, and 14 ft were used. For the HSS deck, top plate thickness was ⅝ in. for member spacing up to 20 in. and then ¾ in. for the others. The inverted WT deck used ⅝ in. top plates for member spacing up to 16 in., ¾ in. for 20 to 28 in. spacing, and ⅞ in. for the 32 and 36 in. spacing. The necessity for top plate thickness differences between the systems was the result of the difference between clear spacing and center-to-center spacing for the different member profiles.

The resulting panel weights were compared for HSS, inverted WT, and comparable reinforced concrete deck. Figure 5 shows a sample graph of panel weight (lb/ft) versus member spacing (in.) for 12 ft. girder spacing. The WT panels are generally lighter than the HSS panels. Some exceptions include panels with 6 in. WT sections and member spacing corresponding to a larger top plate for the WT panel. Larger member spacing, with heavier, but fewer members, typically results in a lighter panel. The majority of the panels evaluated are lighter than a comparable reinforced concrete deck—for example, 800 lb/ft for an 8-in.-thick, 8-ft-wide panel.

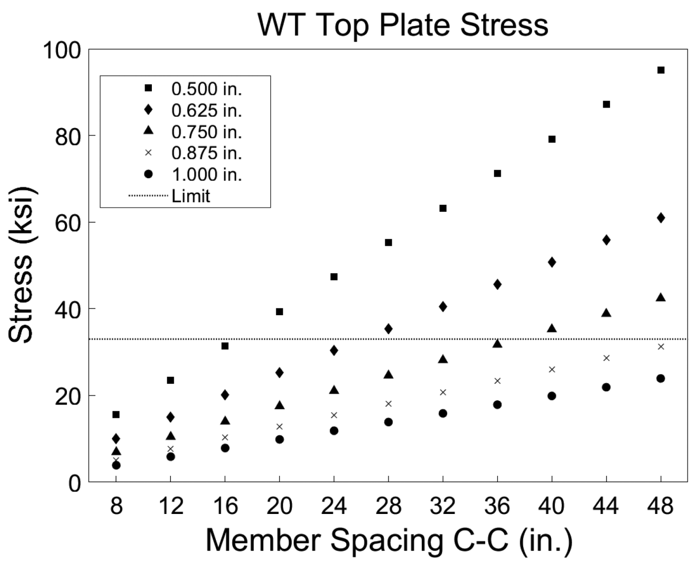


*Fig. 3. Top plate stress (ksi) vs. member spacing (in.) for an inverted WT deck panel.*

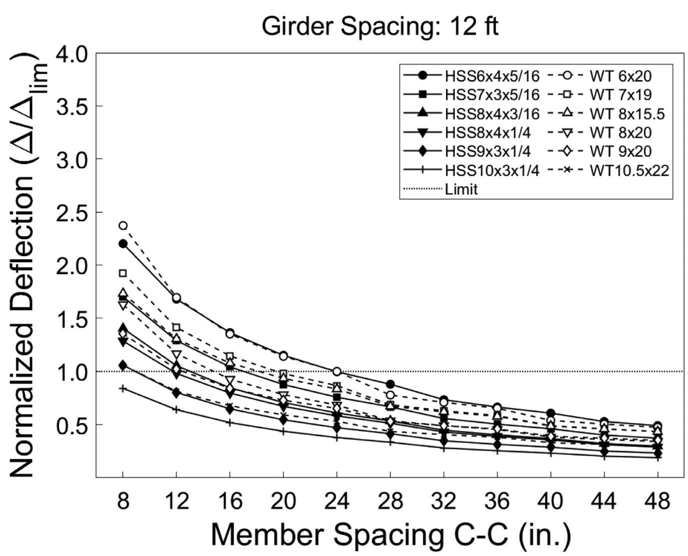


*Fig. 4. Normalized deflection versus member spacing for a 12 ft girder spacing.*

### Evaluation and Selection

The analysis results and practical considerations motivated the choice of the inverted WT deck system. Stress and deflection results were similar for the HSS and inverted WT deck panels. The WT decks tend to be lighter than the HSS panels. The inverted WT is also a more open system with open hot-rolled sections, providing easier access for welding. For these reasons, the research team has moved forward with the inverted WT deck system.

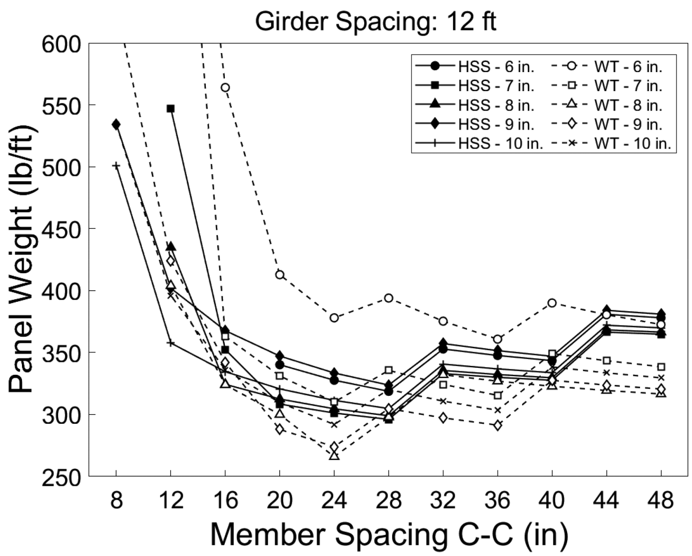


*Fig. 5. Panel weight (lb/ft) vs. member spacing (in.) for 12 ft girder spacing.*

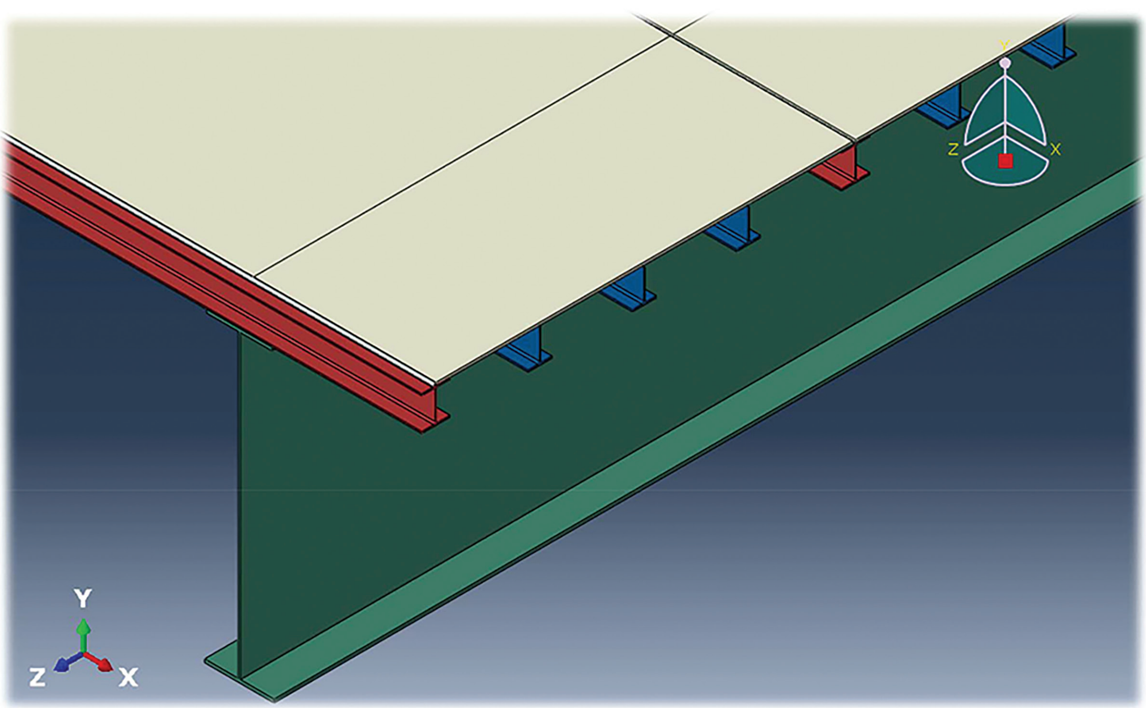


*Fig. 6. Preliminary finite element model of a bridge with an inverted WT deck system.*

## FUTURE WORK

Having addressed the first research objective, the team continues with work on the remaining objectives. The focus will shift from a component level to a system level. Design will be followed by a comparative life-cycle cost analysis of bridges with steel deck and conventional, cast-in-place concrete deck systems. Wearing surfaces, barriers, and connection details will also be addressed. Future work is expected to include finite element (Figure 6), fatigue, and life-cycle cost analyses, as well as physical testing of some connections and full-scale deck panels.

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