STRUCTURAL BEHAVIOR AND FATIGUE REHABILITATION OF THE I-64 STEEL DELTA FRAMES OVER KERRS CREEK AND MAURY RIVER IN VIRGINIA

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BIOGRAPHY

Loai El-Gazairly has more than 38 years of design and management experience in transportation projects. He participated in the design and construction of landmark bridge projects both nationally and internationally. Loai is knowledgeable with the current design standards of the AASHTO LRFD and participated in developing the Bridge Detailing Manual for the Georgia Department of Transportation (GDOT) and the Bridge Design Manual for the West Virginia Department of Transportation (WVDOT). He served as the President of the American Society of Civil Engineers (ASCE) in Richmond, VA and chaired the Structural Technical Group (STG) of the ASCE in Atlanta, GA. Loai is a registered Professional Engineer (P.E.) in several states in the USA that include GA, FL, SC, VA, NJ, PA and Ohio.

Rex L. Pearce, P.E. has 32 years of professional engineering experience with transportation projects including bridges, highways, tunnels and airfields in VA, MD, PA, NJ, NY and CT. He is the Staunton District Bridge Engineer for the Virginia Department of Transportation Staunton District, holds a BSCE from Virginia Tech and Professional Engineering licensing in Virginia and Pennsylvania.

SUMMARY

The twin delta frame bridges carrying I-64 over the Maury River within the Virginia Department of Transportation's (VDOT) Staunton District have experienced fatigue cracking problems that caused structural deterioration and a deficiency in the bridges' inventory ratings. Analytical investigation by researchers at Virginia Polytechnic Institute and State University (Virginia Tech) showed that the bridges could be retrofitted to achieve essentially infinite fatigue life. A fatigue retrofit approach, recommended by VA Tech, has been implemented and a 3-D finite element computer model was developed to examine the stress levels within the structures and its global stability through the retrofit process. In addition, the bridges' structural behavior was monitored through a structural health monitoring program aimed at correlating the actual response with that of the analytical model. The paper discusses the comprehensive structural retrofit program designed to control the fatigue cracks to attain infinite fatigue life. Currently, the bridge rehabilitation has been completed and the retrofitted cracks and the areas of potential cracking are being scrutinized through stringent bridge inspection and maintenance programs. The 2017 and 2018 bridge inspection reports show successful implementation of the retrofits with no observed fatigue cracks at the existing connections.

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Introduction

The twin delta frame bridges carrying I-64 over the Maury River in Virginia Department of Transportation's (VDOT) Staunton District were constructed in 1974.

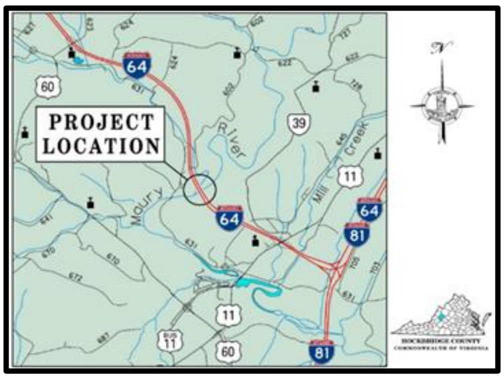

Figure 1: Bridge Location

The two parallel bridges are multi-span structures located approximately 2.7 miles west of I-81 near Lexington, Virginia (Figure 1). The twin bridges were completed using steel frame superstructures; each consists of three delta frames supported by concrete wall piers and stub abutments. Each superstructure contains four spans (182'-7", 240'-0", 240'-0" and 182'-7") for a total length of 845'-2". The East-Bound Bridge (EBB) and the West-Bound Bridge (WBB) have the same superstructure layout and on a tangential horizontal alignment with a straight, positive longitudinal gradient of 5% with no skew (Figures 2 and 3).

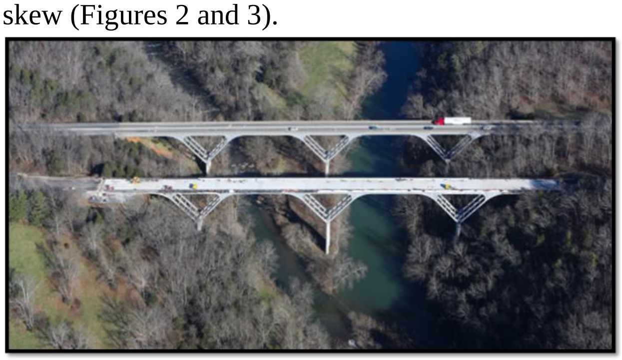

Figure 2: Bridge Aerial View

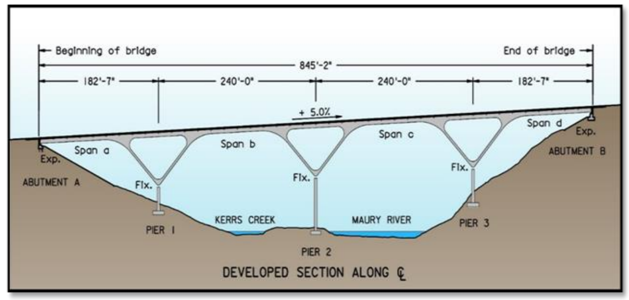

Figure 3: Bridge Layout

The superstructure for each bridge consists of a 43'-0" wide reinforced concrete deck (out-to-out) with an original minimum thickness of 9 ¾". The roadway width is 39'-0" from curb-to-curb with a 2'-0" concrete parapet/railing combination on each side. The concrete deck is supported by three steel frame delta girders on 16'-6" spacing with 5'-0" overhangs (Figure 4).

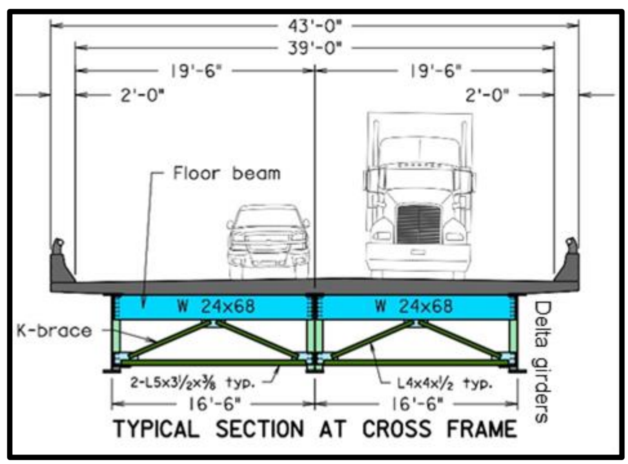

Figure 4: Transverse Section at Cross-frames

The delta-girder web depth is a minimum of 72", deeper at the haunches of the delta frames, which are approximately 75' from the top of the deck to the top of the pier caps. The structure is supported by expansion bearings at the abutments and fixed, high-profile steel shoe bearings at the piers.

The three delta girder lines are braced with cross frames spaced at 20' and composed of steel floorbeams just below the deck and lower vertical K-bracing framing into the floorbeams. Lower lateral cross bracing frames between interior and exterior girders at each cross frame line are spaced in a 40 feet repeating pattern. The bridge structure is founded on three reinforced concrete solid wall piers ranging in height from approximately 32' to 71' and founded on rock.

Despite a relatively low Average Daily Traffic (ADT), the original design introduced several fatigue-prone conditions resulting in a series of cracks in the superstructure steel elements.

In 1991, a large crack was discovered in the EBB south fascia girder web, near the bottom flange, at the cross-frame K-brace and lower lateral bracing connections in the vicinity of Pier (2). The crack was immediately repaired using a bolted web splice connection (Figure 5).

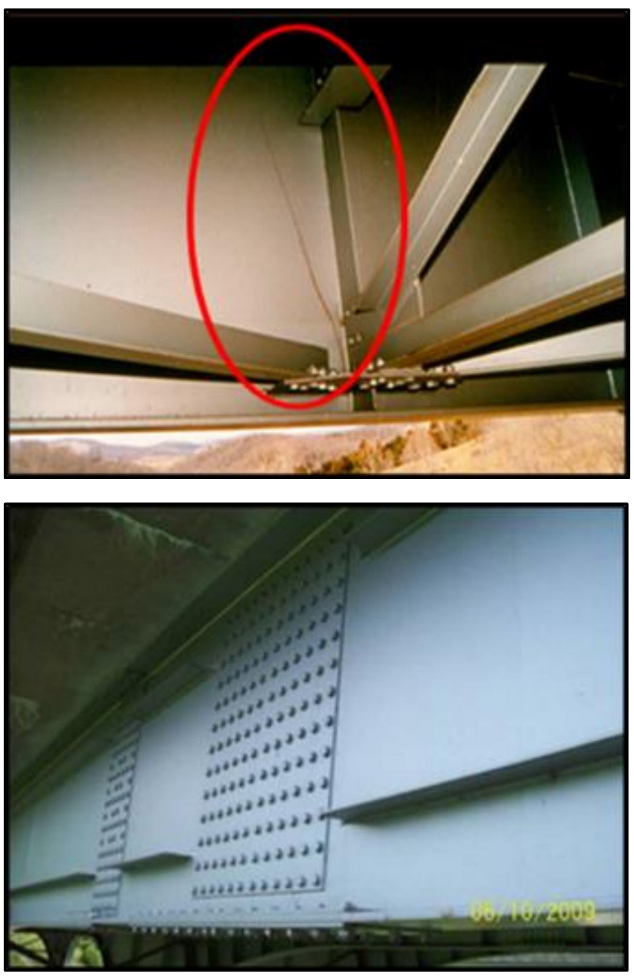

Figure 5: 1991 Web Crack and Bolted Retrofit

Later the same year, three additional cracks were discovered in the interior girder of the WBB bridge. The cracks occurred in the web gap between the connection plates for the cross-frame K-brace and the gusset plate connecting the lower lateral bracing (Figure 6).

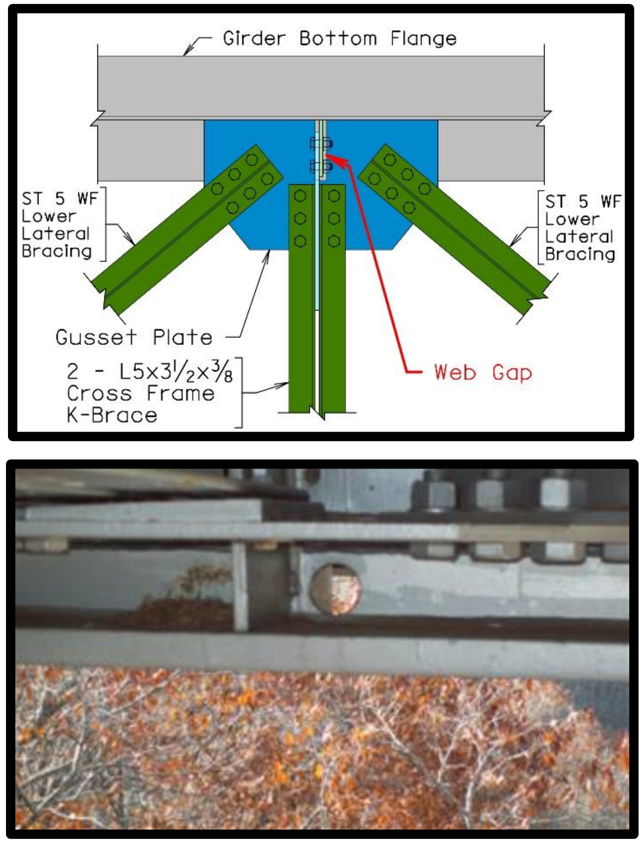

Figure 6: Crack at Web Gap Between Connection Plate of Cross-Frame K-Brace and Gusset Plate Connecting Lateral Lower Bracing

In 1994, researchers at Virginia Polytechnic Institute and State University (Virginia Tech) classified the cracks as fatigue cracks produced by repetitive out-of-plane bending (distortion-induced fatigue) acting on the girder webs and caused by the horizontal component of the forces generated in the cross frames. Later, the cracks were reclassified as constraint-induced fractures based on the 2004 American Association of State Highway and Transportation Officials (AASHTO) guidelines on the separation between transverse welds in the gusset plate cutout around the transverse connector plate.

In order to eliminate the potential for additional cracks, a decrease in the restraint at the connection between the girder web and the vertical/lateral bracing systems was introduced by VDOT. This was achieved by partially disconnecting the lower lateral bracing members (by the removal of all but one or two bolts) from the bracing connections while leaving the lower cross frame and the lateral bracing connections at the haunch "knuckles" areas connected to provide global lateral stability for the structures.

In 2009, a new pattern of cracks appeared at the cross frame floorbeam connections to the delta girder web near the girder top flange. The cracks were discovered along the total length of the bridge. Studies at Virginia Tech suggested that the cracks resulted from local-distortion/constraint-induced fatigue conditions (Figure 7).

Figure 7: 2009 Cracks at Floorbeams and Delta Girders

Consequently, the bridge required another round of retrofitting that focused on improving the fatigue response of the structures. Virginia Tech proposed that the bridge could attain infinite fatigue life by: a) introducing bi-directional composite behavior between the deck and the floorbeams by removing the existing deck, installing shear connectors at the top flange of the transverse floorbeams and reconstructing a new, 2-way composite deck and; b) modifying the fatigue-prone connection details to attain a higher (essentially infinite) fatigue capacity.

The connection retrofits were to be implemented in the positive and negative bending moment regions as shown in Figure 8.

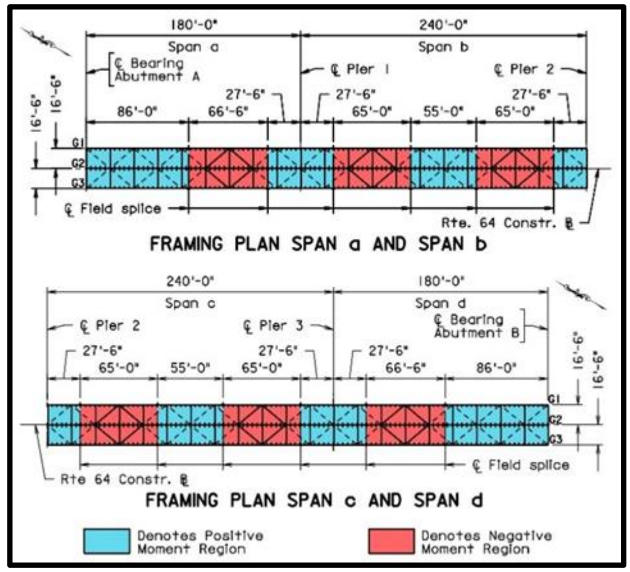

Figure 8: Lateral Bracing Showing Regions of Positive and Negative moments

In the positive moment regions, the horizontal lower lateral bracing members and the gusset plates located within the tension areas of the delta frame web-plate were to be removed after replacing the deck. Then the previously disconnected lower cross frame K-brace connections were to be reconnected and the transverse connector plates at the bottom flange of the girders were to be restored with a 5/16" fillet weld along both sides. Finally, the floorbeam copes were to be ground smooth to a uniform radius to prevent gouges and sharp edges thus increasing the fatigue resistance of the connection to at least Category (B). The cope radius was recommended to be between ½" and 1" for copes with no evidence of fatigue cracking while a 2" diameter drilled hole was to be provided at the crack tip for copes with pre-existing fatigue cracks to prevent any crack propagation. The perimeter of the hole was to be ground smooth to reduce any surface stress concentration.

In the negative moment regions, the retrofit retained the cross frame K-braces and the lower lateral system attached to the girders to provide compression flange bracing. Gusset plates attached to web areas in compression were to be connected to the transverse plates at the top flange of the girders with a 5/16" fillet weld along both sides. For any observed fatigue cracks in the web of the girders, holes were to be drilled at the crack tips to prevent further propagation.

These retrofit recommendations comprehensively produce an essentially infinite fatigue-life for the structure. The VDOT Staunton District Bridge Section retained Whitman, Requardt, and Associates, LLP (WRA) to prepare the structural plans for the retrofit details recommended by Virginia Tech. In addition, WRA was to design the new superstructure deck and to replace the existing stub abutments with a Virginia Style Abutment, thus shifting the deck joints beyond the integral backwall reducing maintenance issues at the bearings. WRA was also to develop a 3-D computer model of the structural retrofits investigating stability and internal stresses developed during the retrofit process.

Analysis and Computer Modeling

WRA initiated the bridges' structural analyses by developing a comprehensive 3-D computer model using LARSA-4D. The model included the previous structures' retrofits and established the as-built baseline of the element nodal stresses to compare with those developed during the subsequent construction stages (Figure 9).

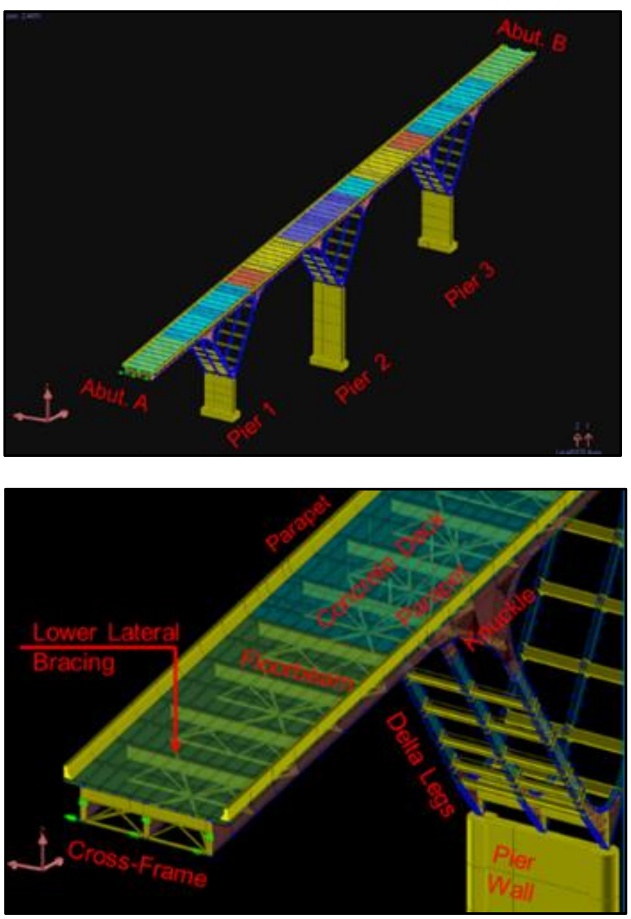

Figure 9: 3-D Finite Element LARSA-4D Model

Additionally, the model included the recommended deck reconstruction sequence – beginning with the removal of a longitudinal 120' section in the middle of the bridge. The presence of the shear studs at the top flange of the transverse floor beams were introduced (incrementally modelling 2-way slab action) followed by the replacement of the removed deck section with lightweight concrete. Once the constructed part of the deck was cured, the anticipated two-way composite action was established between the girders and the transverse cross frame floorbeams. The process was repeated until the entire deck was replaced as shown in Figure 10.

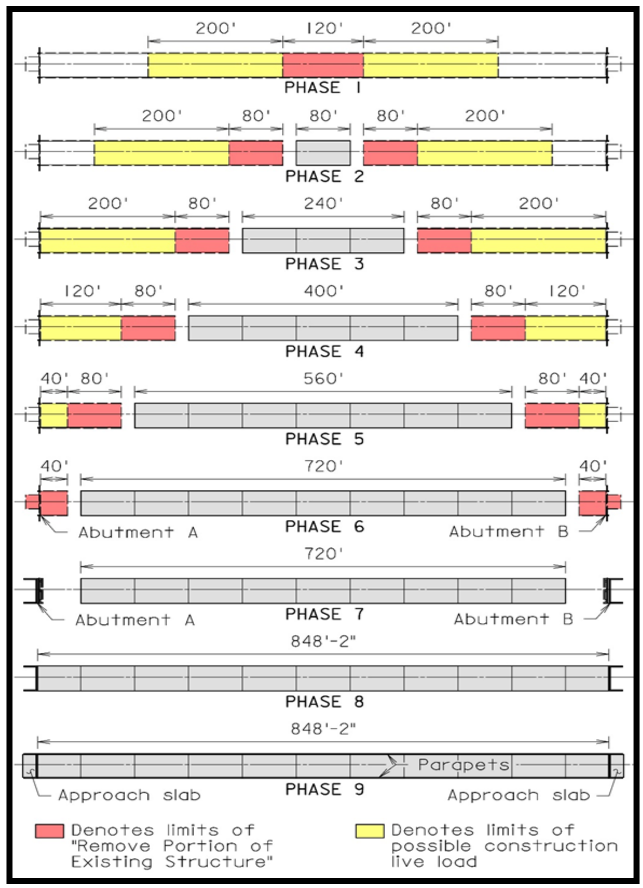

Figure 10: Layout of Deck Demolition and Construction Phases

Nodal element stresses were monitored in all proposed construction stages and the structure model reflected no stresses in excess of those established by the as-built condition threshold or by the AASHTO Standard Specification for Highway

Bridges for the Allowable Stress Design (ASD). The structural element strength capacities were checked against the required force demands and found to be sufficient. Furthermore, the elements' nodal maximum and minimum average principal stresses were compared against those established by the Von Mises yield criteria and confirmed a safe structure response during the retrofitting and the construction sequencing (Figure 11).

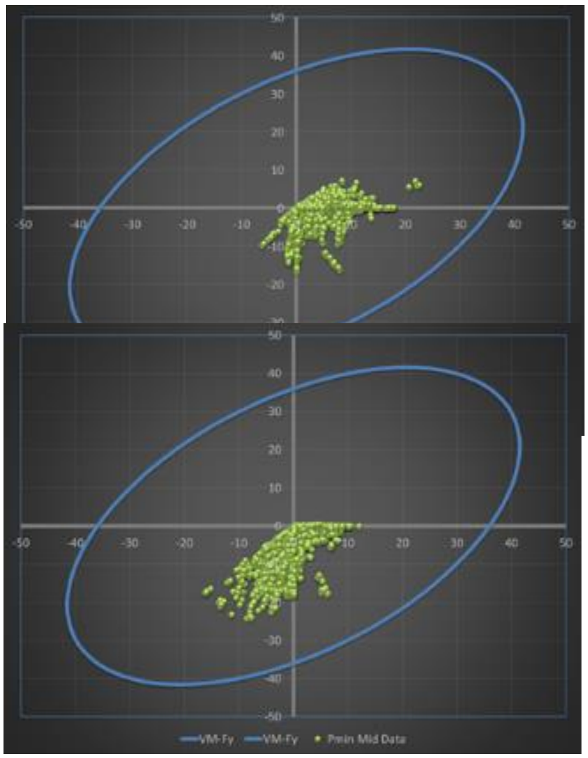

Figure 11: Max/Min Nodal Principal Stresses vs. Von Mises Criteria

Field Retrofit Implementation

VDOT Staunton Bridge District Section contracted the structural retrofit rehabilitation under a design-bid-build process. Simultaneously, VDOT received a Federal Highway Administration (FHWA) Tiger Grant to fund the construction phase of the contract.

Prior to deck removal, the Contractor began by re-establishing the cross-frame K-brace lower connections and lower lateral bracing connections thus restoring its full bracing capacity. The deck removal proceeded from the longitudinal-center toward the abutments in strict accordance with the design plans and the construction load allowance (Figure 12).

Figure 12: Deck Staged Construction

The shear studs were installed on all the transverse floorbeams to produce fully composite two-way deck action (Figure 13).

Figure 13: Providing two-way Composite Action

The new deck configuration necessitated a thickness increase which increased the dead load and was offset by the use of lightweight concrete and the variation was minimal. The design of the new deck provided adequate reinforcement capable of resisting both the longitudinal and transverse moments and minimized the potential for the formation of any shrinkage cracks within the deck. Once the deck replacement was complete and the permanent lateral bracing provided by the deck restored, the lower lateral bracing and associated gusset plates were removed in the positive moment regions per retrofit plan.

Coincident with the deck replacement and the cross frame-floorbeam integration, the structural steel retrofits were performed at all fatigue-prone areas associated with the cross frame and the lateral bracing-to-girder connections. The retrofits outlined in the structural plans were performed at both the positive and negative moment regions.

In the positive moment regions, the repairs included; a) fatigue repair type (1) that was comprised of the removal of the gusset plate, horizontal lateral bracing and improvement of the fatigue detail to Category (C) (Figure 14);

Figure 14: Fatigue Repair (type 1)

and b) fatigue repair type (2) was to provide better cope details at the floorbeam level by smooth grinding the cope and introducing a 2"-diameter drill to prevent potential cracks propagation (Figure 15);

Figure 15: Fatigue Repair (type 2)

and c) fatigue repair type (3) involved the increase in the system rigidity by reconnecting the loose vertical K-bracing system and welding the bottom flange girders to the vertical stiffeners on both sides of the girder (Figure 16).

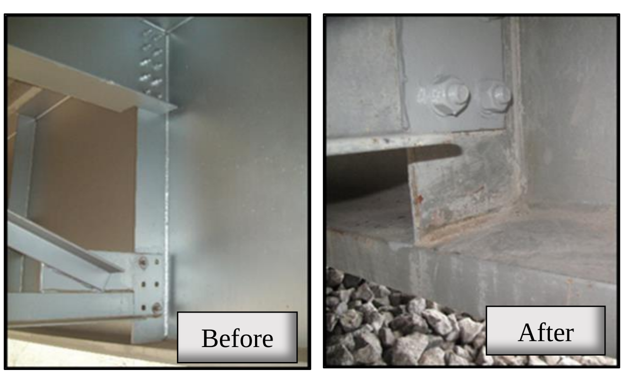

Figure 16: Fatigue Repair (type 3)

In the negative moment regions, the repairs included: a) fatigue repair type (4) which was to increase the stiffness connectivity between the floorbeams and the longitudinal girders by providing a 5/16" weld on both sides of the transverse stiffeners (Figure 17);

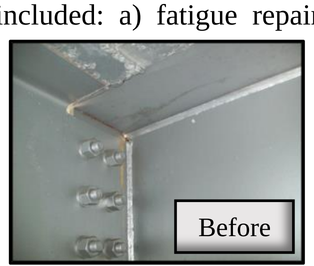

Figure 17: Fatigue Repair (type 4)

and b) fatigue repair type (5) that was for the diagonal fatigue cracks developed at the floorbeam levels due to poor coping details and roughness. To cease the progress of the crack, a 2"-diameter stop drill and a ½" steel plate bolted to the web of the floorbeams were introduced to confine the cracks and to produce extra strength capacity at the connection (Figure 18).

Figure 18: Fatigue Repair (type 5) at Floorbeam Coping

Lastly, fatigue repair type (6) was designed to stop the propagation of the existing fatigue cracks along the web/flange weld or within the girders' web. This detail was the most challenging to implement due to limited access for portable magnetic drills. A drilled hole, usually 2" in diameter, was introduced to intersect the crack path and to prevent future deterioration (Figure 19).

Figure 19: Fatigue Repair (type 6) at Web/Flange Weld

Meanwhile, the VDOT Staunton District Bridge Section directed WRA to extend the jointless deck superstructure and introduce a Virginia Abutment configuration to locate the expansion joint beyond

the bridge seats. The originally designed superstructure continuity delivered an 845' jointless deck system over the piers but required large-movement compression seals at the abutment backwalls. These original seals failed repeatedly and caused embankment erosion and abutment undermining. The project restoration incorporated the modification of the existing stub abutment into a Virginia Abutment producing a truly jointless bridge design (Figure 20).

Figure 20: Newly Virginia Alternate Abutment

Health Monitoring System

A Thermoelastic Stress Analysis (TSA) system designed by the Virginia Transportation Research Council (VTRC) was installed to measure the full field stress at the floorbeam-to-girder connection at the interior of the fascia girder on the south side of the EBB before retrofitting. This system operated on the fact that whenever matter expands or contracts due to external forces, there is a corresponding change in temperature that is directly related to the sum of the principal stresses developed into the material based on the following relationship:

$$\Delta T = (\alpha T / \rho C_{\varepsilon}) \, \Delta \, (\sigma_1 + \sigma_2) \quad \text{.......................} \quad (1)$$

Where:

ΔT = Change in temperature

T = Bulk absolute temperature of the material

α = Linear coefficient of thermal expansion

ρ = Martial density

C_{ε} = Specific heat at constant strain

σ_1 & σ_2 = Principal stresses

In order to measure the small changes in surface temperature associated with localized stresses at a fatigue prone detail, the system was designed to respond to, and capture, a high stress range that is above 5 ksi. The system was set to capture random loading events that exceeded a preset threshold, while a sensor (strain gage, displacement sensor or accelerometer) was attached to the connection to capture these events. A separate Infra-Red (IR) camera head was positioned within two feet of the connection and connected to the field computer. Initially and before retrofitting, the system was deployed to measure the full field stress at the floorbeam to girder connection at the interior of the fascia girder on the south side of the Eastbound Bridge (EBB). The data was collected for thirteen heavy trucks and over a period of two hours. The results showed a significant stress concentration at the termination of the fillet weld between the floorbeam connection plate and the girder web. The stress intensity indicated a stress range well above 20 ksi. After retrofitting the stresses were far less than those previously recorded, an indication of a properly retrofitted connection directed toward improved fatigue life (Figure 21).

Figure 21: Thermoelastic Stress Analysis (TSA) monitoring system layout

Bridge Load Rating

Post-retrofitting, the structures were load rated per AASHTO Manual for Bridge Evaluation (2010), along with the latest guidelines from the VDOT Structure and Bridge Division (2016). As the structures were designed in accordance with the 1965 (with 1966-1967 interims) AASHO Standard Specifications for Highway Bridges, only the Load Factor Rating (LFR) analysis was required. The load rating was performed using the three-dimensional finite element computer model developed earlier using the LARSA-4-D software. The model included the Design, Legal and Permit Loads per AASHTO specifications. Results showed passing Rating Factors (RF) for both Operating and Inventory levels. The results were a considerable improvement compared to those of the original structure confirming better response due to retrofit. In addition, the structures were rated for an umbrella of Special and Permit Superloads, specified by VDOT Load Rating Program Manager, to establish a threshold for the heavy vehicles allowed to use the bridges.

VDOT continued condition inspection with increased frequency to ensure a successful retrofit after the bridges were opened to traffic in 2017. Reports confirmed that the original fatigue cracks were confined, and no additional cracks have formed. The bridge inspection reports in 2017 and 2018 showed successful implementation of the retrofits with no observed fatigue cracks at the existing connections. Upon completion of the retrofit, the superstructure condition ratings were upgraded from General Condition Rating (GCR) 4 to GCR 5. After 2 years of successful performance, the superstructure condition ratings have been restored to GCR 6.

Conclusion

The early development of fatigue cracks in the superstructure of the delta frame bridges that carry I-64 over the Maury River indicated a long-term potential problem within the structure that could affect its inventory rating and structural performance. The VDOT Staunton District Bridge Section addressed the potential implication of these cracks through rigorous bridge inspection and retrofitting programs.

Collaborative efforts by VDOT, Virginia Tech, VTRC and WRA investigated the source of the structural steel cracks and developed successful retrofitting methodologies. In lieu of complete structure replacement, this retrofit rehabilitation was implemented for better bridge performance without compromising its structural integrity or public safety. The goal was to achieve a status of infinite fatigue life for the structure and to extend its service life. A reliable restoration solution based on analytical studies that included a comprehensive 3-D finite element model with construction stage analysis was implemented to improve the fatigue-prone connection details and to attain higher fatigue capacity.

The performance of the fatigue prone connections under normal traffic conditions was closely monitored through: a) continuous and comprehensive bridge inspection programs; and b) newly developed technology based on thermoelastic stress analysis in which the differential temperature developed from the live load application was directly related to the corresponding generated stresses within the material. The bridge inspection reports after the completion of the bridge retrofitting program in 2017 and 2018 showed successful implementation of the retrofits with no observed fatigue cracks at any connections. Additionally, low stresses developed at the retrofitted connections and the undetected responses reported through the attached strain gauges indicated a successful retrofit.

Ultimately, the implementation of the structural retrofit has increased the fatigue life, mitigating the formation of additional fatigue cracks, and extending the service life of the bridges (Figure 22). The condition of the structures will continue to be monitored through the rigorous VDOT bridge safety inspection program.

Figure 22: Complete Retrofitted Structures

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