REPAIR, STRENGTHENING, AND RE-USE OF STEEL GIRDER BRIDGES: TWO CASE STUDIES

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BIOGRAPHY

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SUMMARY

Bridge repairs and strengthening techniques require careful consideration of the behavior of the structure and load paths through the repairs and construction sequencing. This presentation will discuss two case studies that required unique solutions to repair, strengthen, modify, and extend the service life of the steel superstructure, and the challenges associated with these unique solutions.

The first case study is a three-span continuous steel plate girder bridge that originally had span lengths of 59.0′-91.25′-88’.5′, with all supports skewed at 13.5 degrees. To accommodate a new lane arrangement underneath the bridge, the first interior pier was relocated nearly 10 feet closer to the rear abutment, increasing the center span length. The girders were strengthened in the center span positive moment region and in the end span for negative moment, and concrete counterweights were added in the end span. Phased construction was necessary to accomplish these repairs, as one lane of traffic was maintained on the bridge.

The second case study focuses on repairs necessitated by a full-depth fracture of a fascia beam due to repetitive vehicle impacts. The bridge is a curved four-span continuous chorded steel beam bridge, with supports skewed between 30 and 45 degrees, and spans of 64′-91′-84′-59′. Repair work included the replacement of the damaged beam section and connecting cross frames while providing temporary support and removing a portion of the concrete deck slab, and raising the entire superstructure. A three-dimensional LARSA finite element model was used to determine member forces, temporary support locations and reactions, and screed elevations necessary for partial deck replacement.

REPAIR, STRENGTHENING, AND RE-USE OF STEEL GIRDER BRIDGES: TWO CASE STUDIES

Introduction

As our infrastructure continues to age, the repair and strengthening of existing bridges has become more commonplace, with some repairs requiring unique engineering solutions. Bridge repairs and strengthening techniques require careful consideration of the behavior of the structure and load paths through the repairs and construction sequencing. This paper will discuss two such case studies that required unique solutions to repair, strengthen, and modify the steel superstructure, and their associated challenges. The first case study is the Lincoln Avenue Bridge over I-71 in Hamilton County, Ohio, and the second case study is the Hawthorn Parkway Bridge over US 422 in Cuyahoga County, Ohio.

The first case study is a three-span continuous steel plate girder bridge that originally had span lengths of 59′-0″ - 91′-3″ - 88′-6″, with all supports skewed at 13.5 degrees. To accommodate a new lane arrangement beneath the bridge, the first interior pier was relocated nearly 10′ closer to the rear abutment, increasing the center span length. The girders were strengthened in the center span positive moment region and in the end span for negative moment. Concrete counterweights were added in the end span to counteract live load uplift. Phased construction was necessary to accomplish these repairs, as one lane of traffic in each direction was maintained on the bridge throughout construction.

The second case study focuses on repairs necessitated by a full-depth fracture of a fascia beam due to repetitive vehicle impacts. The bridge is a curved four-span continuous chorded steel beam bridge, with all supports skewed 40 degrees with respect to the reference chord, and spans of 64′-0″ - 91′-0″ - 84′-0″ -59′-0″. Repair work included the replacement of the damaged beam section, connecting cross-frames while providing temporary support, removing a portion of the concrete deck slab, and raising the entire superstructure. A three-dimensional LARSA finite element model was used to determine member forces, temporary support locations and reactions, and screed elevations necessary for partial deck replacement.

Lincoln Avenue Bridge Strengthening and Re-use

The Lincoln Avenue Bridge is a straight three-span continuous steel plate I-girder bridge originally constructed in 1969 with span lengths of 59′-0″ - 91′-4″ - 88′-6″, and all supports skewed at 13.5 degrees. Figure 1 shows a general elevation view of the existing bridge prior to any new work. The bridge crosses over I-71 in the northeast section of the City of Cincinnati, Ohio. The bridge deck has a total width of 60′-4″, which includes four lanes of vehicular traffic and 6′-10″ sidewalks on each side of the bridge. The non-composite deck is supported by seven steel I-girders spaced at 9′-2″.

The work on the Lincoln Avenue Bridge was part of a larger design-build project that included the widening of the I-71 corridor below Lincoln Avenue along with additional bridge replacements and rehabilitations along the corridor. The overall project included work on 12 different bridges.

Below the Lincoln Avenue Bridge, a new ramp lane was cut into the existing slope below Span 1 and the lane arrangement was widened below Span 2. A soil nail wall was installed in front of the Rear (West) Abutment to allow the new ramp lane below Span 1. To accommodate the new I-71 mainline lane arrangement beneath the bridge, the first interior pier was relocated nearly 10′ closer to the Rear Abutment, increasing the center span (Span 2) length, and resulting in a new span arrangement of 49′-3″ - 101′-0″ - 88′-6″. It can be seen that an already poor span arrangement was made worse by the span reconfiguration, such that the first end span is less than 50% of the center span length. Figure 2 shows a general elevation view of the new span arrangement and I-71 lanes below.

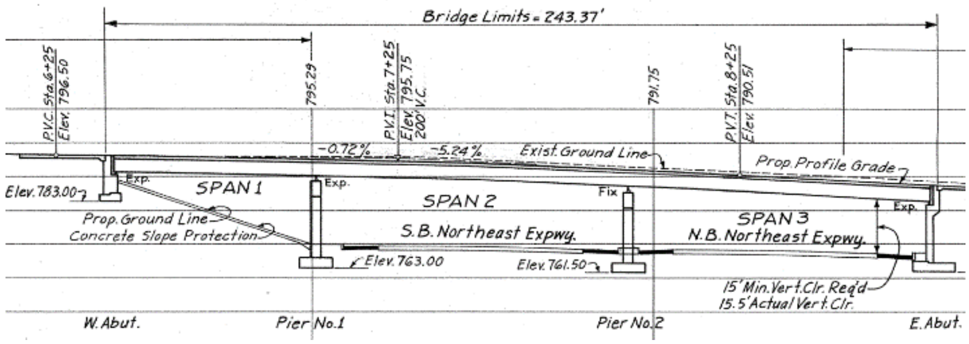


Figure 1: General Elevation View of the Existing Lincoln Avenue Bridge.

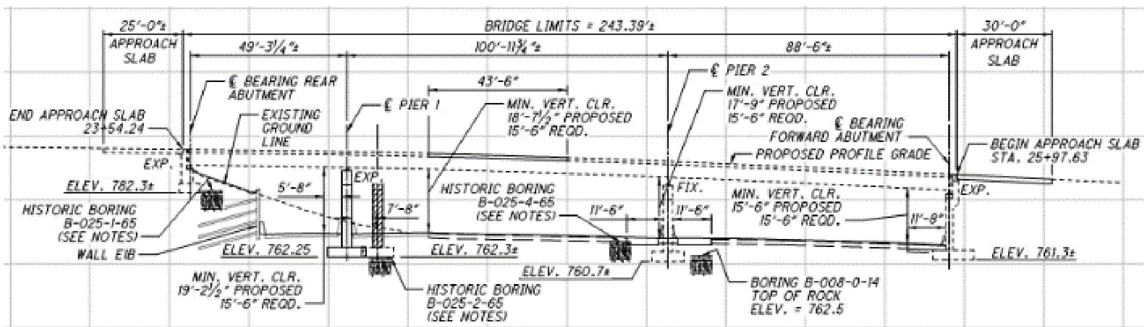


Figure 2: General Elevation View of the Proposed Lincoln Avenue Bridge.

Additionally, the bridge had an unacceptable vertical clearance at the Forward (East) Abutment. To limit the amount of approach work, the bridge was raised non-uniformly along the length of the bridge. At the Forward Abutment, the bridge was raised 7.625 inches, while there was no raise at the Rear Abutment, and proportional raises at new Pier 1 and existing Pier 2.

All repairs and strengthening had to occur while maintaining at least one lane of traffic in each direction, with limited full closures allowed for jacking operations. Therefore, any deck removal and replacement to access the girders for strengthening had to be done using phased construction.

Lastly, the repairs, strengthening, and new pier were designed in accordance with the AASHTO Standard Specifications, 17th Edition (1) and the Ohio Department of Transportation Bridge Design Manual, 2004 edition (2). This follows typical Ohio DOT policy for the design of repairs and strengthening of structures designed prior to use of the AASHTO LRFD Bridge Design Specifications (3).

Pier 1 Relocation

The new span arrangement to accommodate the widened I-71 corridor resulted in the need for a new Pier 1 to be reconstructed nearly 10′ closer to the Rear Abutment. The new pier is similar to existing pier in that it is a 4-column cap and column pier, and utilizes individual spread footings on rock for each column. Each new footing is 9′-6″ by 9′-6″ in plan. Due to the close proximity of the new pier with the existing, two of the existing column spread footings were in conflict with the new pier spread footings, with a maximum overlap of nearly 1′-6″. The new pier is aligned on a slightly different angle from the existing pier due to the new I-71 alignment, and thus only two footings were in conflict. Through analysis of the existing bridge in combination with recent geotechnical information, it was determined that the existing foundations could be partially removed to accommodate the new spread footings, and was thus the method used to allow construction of the new pier. Figure 3 shows the new pier constructed next to the existing pier.

Figure 3: New Pier 1 Constructed to the West (left) of the Existing Pier 1.

Girder Strengthening

The change in span arrangement resulted in required strengthening repairs to all girders in the superstructure. The new span arrangement causes a shift in the major-axis bending moment diagram. This behavior is generally illustrated in Figure 4. Of course, due to the increased center span length by approximately 10′, the maximum positive moment in the final condition is substantially increased over the existing design major-axis bending moment. The maximum negative major-axis bending moments do not substantially increase, however the location moves because of the Pier 1 relocation. The negative moment area in Span 1 changes, with more of the span subjected to negative moment. The increase in maximum positive moment in Span 2 and the shift of the negative moment area into more of Span 1 resulted in girder strengthening repairs in these areas. Conversely the existing girders in Span 3 required no additional strengthening.

The project design-build documents were written such that the contractor only had to replace the entire bridge deck if they had to remove and replace the concrete deck from more than one span of the bridge. If a repair strategy required only removing and replacing the concrete deck in one span, a significant cost savings would be realized since the entire deck did not need to be replaced. Therefore, the contractor and design team developed a cost-effective strengthening strategy that only required the removal of a portion of deck in the center span. Figure 5 shows a girder elevation with the repair locations identified.

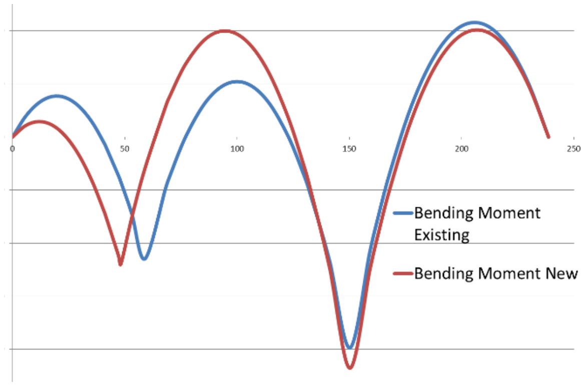


Figure 4: General Shift in Major-Axis Bending Moment Diagram Caused by New Span Arrangement.

The Span 1 repair was unique in that it was developed so the deck did not need to be removed. The shift in the moment diagram resulted in stresses that exceeded the resistance of the existing flanges at the flange transition point now 8′ away from Pier 1. This flange transition was 18′ from Pier 1 in the existing configuration. Two strengthening plates were added to the bottom flange transition location to adequately transfer new bottom flange forces resulting from increased live load, and dead load effects caused by relocating Pier 1. Because the bottom flange could be in reversal in this area, the repair plates were bolted to the existing flanges. A WT 4x29 section was added to each side of the web at the flange transition location, and they were located near the top of the web. These WT members help to increase the moment of inertia at this location, thus reducing the applied stresses in the top flange such that they were less than the flange resistance. The length of the repair assemblies were designed to adequately transfer forces from the existing girder, into the repair assemblies, and then back into the existing girder. The use of these WT's in combination with the bottom flange repair made it unnecessary to access the top flange by removing the concrete deck.

Also note, new bearing stiffeners were required at the new Pier 1 support. Figure 6 shows the Span 1 repair assemblies installed on a fascia girder prior to installation of the new bearing stiffeners.

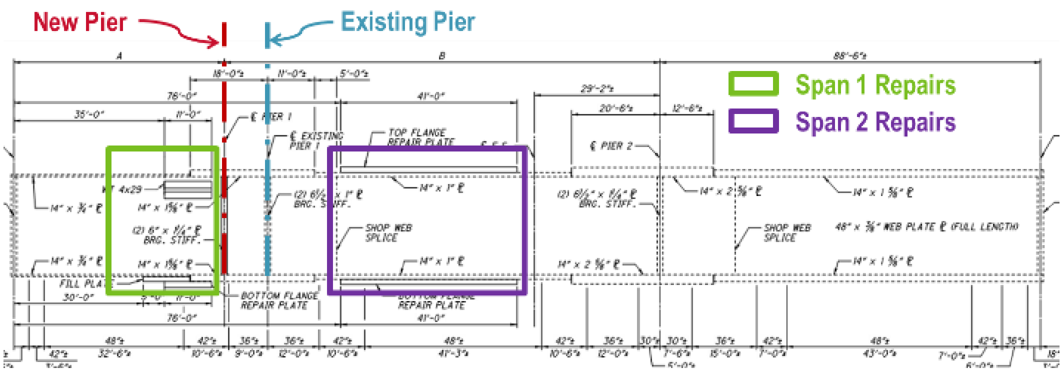


Figure 5: Girder Elevation With Repair Locations Noted.

Figure 6: Span 1 Repairs Installed in Fascia Girder.

The Span 2 repair in the center span utilized more conventional girder strengthening details. The repairs were required because of the substantial increase in the live load moments, and additional dead load moment caused by the relocation of Pier 1. Strengthening plates were added to the top and bottom flanges, with each measuring 41′ in length and nearly centered in the new Span 2 length. The bottom repair plate was completely bolted to the existing bottom flange. The bottom repair plate was the same width as the existing bottom flange. The top repair plate was welded to the top flange except at the ends which were bolted to provide a means for adequately transferring forces back into the existing top flange. The ends of the top flange repair plate were also subjected to a limited amount of moment reversal and thus subjected to tensile forces. Welding to the member locations subjected to tension was not permitted. The top flange repair plate was not as wide as the existing top flange. One-inch of clearance was provided on each side of the repair plate so that it could be welded to the existing top flange from above.

The Span 2 repairs did require removal and replacement of the deck concrete above the repair area so that the top flange could be accessed. This deck removal and replacement occurred in two phases, maintaining two lanes of traffic on the bridge at all times. The first phase was used to make repairs on the 4 girders on the south side of the bridge (Figure 7), and the second phase was used for the remaining 3 girders (Figure 8). Mechanical connectors were used in the concrete deck at the phased construction to eliminate the need to lap the transverse steel reinforcement at this construction joint.

Figure 7: Phase 1 of the Span 2 Repairs with Deck Removed.

Figure 8: Phase 2 of the Span 2 Repairs with Deck Removed.

The bolting patterns used to connect the repair assemblies to the existing girders allowed for some fabrication flexibility. The existing girders had stiffened webs and many transverse stiffeners along the length of the bridge, which was typical practice for bridges of this vintage. It was found that during design, the exact transverse stiffener cross-frame connection plate locations did not match those on the original design plans. Therefore, the proposed bolt patterns allowed for a minimum 3 inches and maximum 6 inches bolt spacing near each transverse stiffener and cross-frame connection plate. This detail was shown on the design plans, as shown in Figure 9 with notes provided regarding the minimum and maximum permissible bolt spacing.

The contractor performed a field scan and survey of the steel superstructure to determine the exact location of the stiffeners. The detailer used this survey data to accurately locate the stiffeners and the subsequent bolts patterns for the repair assemblies. This process saved design schedule time, as the plan production and the approval process associated with this project was not hindered by waiting for exact measurements.

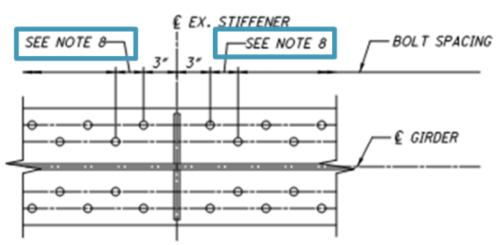


Figure 9: Bolt Pattern Detail at Intermediate Transverse Stiffeners and Cross-frame Connection Plates.

Careful consideration had to be given to the construction and loading sequences to determine the appropriate stresses in the repair plates and the existing girders. For example, the detailed staged construction analysis of the superstructure considered the removal and replacement of the deck in the center span, the installation of the counterweights, the installation of the repair assemblies, the relocation of the support provided by the Pier, and the final live loading. The finite element program LARSA was used for the construction staging analysis. It was important and necessary to recognize that the repair assemblies only participate in response to dead loads associated with the deck removal and replacement and the relocation of the support provided by Pier 1, and the live load applied upon completion of all repairs.

Counterweights

The less than ideal new span arrangement resulted in net uplift at the Rear Abutment. Live load placement in Span 2 would cause uplift reactions that could overcome the downward dead load reactions, using factored load combinations. There was no net uplift considering unfactored load combinations. The net uplift was resolved with the placement of counterweights in Span 1. As shown in the plan view in Figure 10, the counterweights were located as close as possible to the Rear Abutment.

Two different types of counterweights were required. A 5′ (Length) x 7′ (Width) x 2′-6″ (Height) was used in all bays except for the northern exterior bay. A utility duct was to remain in place in this exterior bay. Therefore, two concrete counterweights were used, with each having dimensions of 7′ (Length) x 2′ (Width) x 2′-6″ (Height). The counterweights were formed and cast in place since the Span 1 deck remained in place. A steel framing system had to be installed to support the counterweights. This included new connection plates on the girders if the existing stiffeners could not be used, W6x12 support beams spanning between the girders, steel grating, and ½″ steel plates.

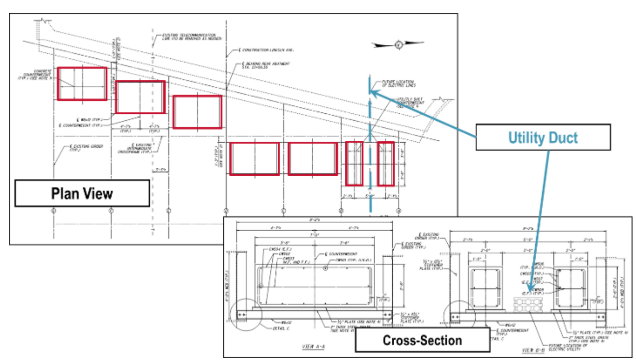


Figure 10: Counterweights Provided Near the Rear Abutment.

Increasing Vertical Clearance

The vertical clearance at the Forward Abutment had to be increased by 7.625 inches. In lieu of raising the entire bridge a uniform 7.625 inches, it was determined the bridge could be raised in a non-uniform fashion without adversely affecting the vertical profile of the roadway. At the Forward Abutment, the bridge was raised 7.625 inches, while there was 0 inch raise at the Rear Abutment, and proportional raises at Pier 1 and Pier 2. While not typical, the non-uniform raise was a cost-effective solution as it limited the amount of approach and abutment work to just one end of the bridge.

Careful consideration had to be given to the non-uniform raise because of the skewed supports. If the supports would have been skewed at a greater angle or if the bridge was wider, the amount of raise at each bearing along a support would have been different due to the bridge pivoting about a line parallel with the Rear Abutment (See Figure 11). It was found through exhaustive geometric calculations that the difference in elevation change between the fascia girders at the same support was less than 1/16″. Therefore, it was assumed the raise at each support was uniform for each bearing. It was also found that the non-uniform raise, and subsequent rotation at the Rear Abutment did not adversely affect the existing expansion joint. New jacking stiffener details and jacking reactions for the raising of the bridge were provided on the design plans.

At new Pier 1, elastomeric bearings replaced the existing steel rocker bearings. The exact height of these bearing was computed, however to allow for adjustment in the field due to the raising of the bridge, a maximum steel shim stack of 1/2″ was permitted. The existing fixed steel bearings at Pier 2 were reused. However, these bearings were retrofitted with a new steel masonry plate that would sit on the pier, and a steel shim plate (or stack) that is located in between the new masonry steel plate and the existing masonry plate of the reused bearing. The new masonry plate and steel shims were made wider than the existing bearing in a pyramid arrangement so they could be welded from above. The new steel masonry plate required new anchor bolts to be placed into the existing pier cap. In order to not damage the reinforcement in the pier cap, the reinforcement location was to be verified in the field through the use of a pachometer, prior to fabrication of the new fixed bearing shim plates.

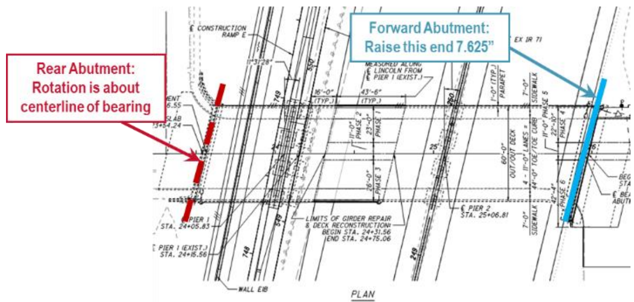


Figure 11: Plan View of Lincoln Avenue Bridge and Bridge Raising.

At the Forward Abutment, the existing steel expansion bearing were simply reused, as the abutment bridge seat and backwall were rebuilt to accommodate the raising of the bridge at this location. Similar to Pier 1, a maximum steel shim plate stack of 1/2″ was allowed to compensate for any elevation differences realized in the field.

Raising the bridge at the forward abutment resulted in reconstructing the abutment backwall, expansion joint, and approach slab. This work was also done in two phases to maintain traffic on the bridge similar to the Span 2 repair discussed previously.

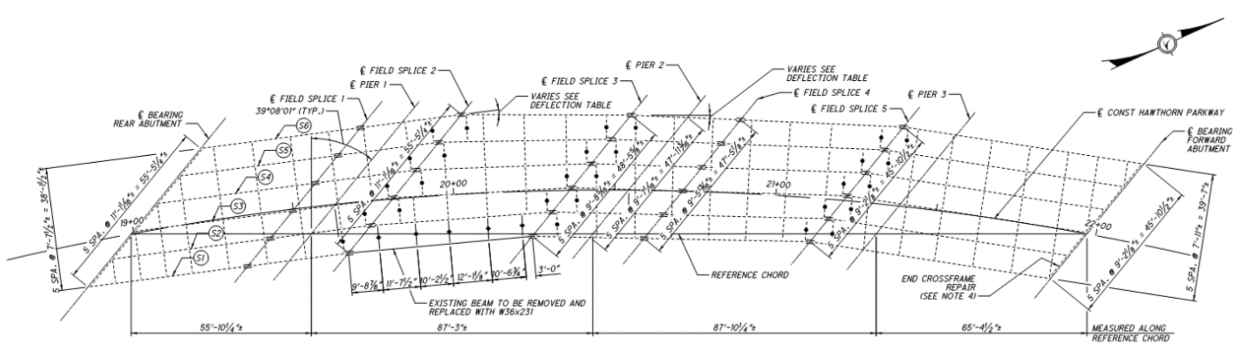


Figure 12: Framing Plan of the Hawthorn Parkway Bridge

Hawthorn Parkway Bridge Repair

The Hawthorn Parkway Bridge is a curved four-span continuous chorded steel beam bridge, with all supports skewed at 40 degrees with respect to the reference chord, and with spans of 64′-0″ - 91′-0″ - 84′-0″ -59′-0″. A framing plan of the bridge is shown in Figure 12. The non-composite concrete deck is supported by 6 wide flange beams with variable spacing along the length of the bridge. The bridge carries Hawthorn Parkway over US 422 in the City of Solon, Ohio, and is located within the Cleveland Metroparks. The bridge carries two lanes of vehicular traffic, as well as, a bridle path used for horseback riding.

On August 7, 2014, a full-depth crack in the fascia beam was discovered during a routine annual bridge inspection being performed by the Ohio Department of Transportation (ODOT) District 12 (See Figure 13). The bridge was subsequently closed to vehicular traffic, and an emergency temporary web splice plate was added to the bridge. The bridge remained closed to traffic until a permanent repair was made.

Figure 13: Full-depth Crack in Fascia Beam.

Furthermore, visible damage could be seen on the bottom flange of the fascia and adjacent interior beams where vehicles have impacted the bridge. The fascia beam had been heat straightened once in 1987 and again in 2005. Through a General Engineering Services contract with ODOT District 12, HDR was engaged to investigate repair solutions and subsequently develop construction plans for the repair. In addition to developing repairs for the cracked beam, the project also included general maintenance items that needed repair or replacement as well. The repairs were designed in accordance with the AASHTO Standard Specifications, 17th Edition (1) and the Ohio Department of Transportation Bridge Design Manual, 2004 edition (2).

Repair Strategy

Several repair strategies were considered for the repair of the Hawthorn Parkway Bridge. The repair not only needed to repair the damaged section, but increase the vertical clearance over US 422 which was determined to be deficient at only 14′-4″ where the preferred minimum vertical clearance is 15′-6″. The chosen repair strategy included:

- the replacement of the fascia beam from field splice 2 to field splice 3 with a same depth section, removing the damaged beam section
- the raising the structure 4” at each support to improve vertical clearance
- the consideration of future plans by ODOT D12 that would lower US 422 as part of a future resurfacing project.

Since the beam section was being replaced between two existing field splices, the existing bolt holes were used on the two beam sections that remained in place. The contract plans noted that the existing field splice plates were to be used as templates for drilling the holes on the new splices plates and the new beam section, eliminating potential fit up issues with the beam sections that remained in place. A flange splice detail is shown in Figure 14.

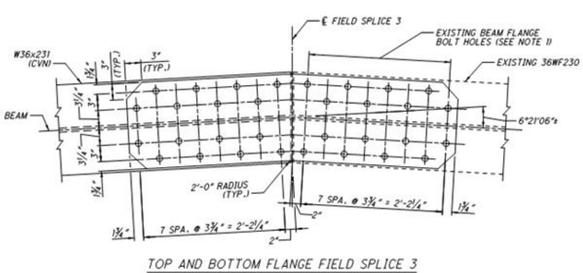


Figure 14: Flange Splice Plates for Field Splice 3.

A detailed Beam Removal and Replacement Procedure was provided on the contract plans. The detailed procedure was necessary to accurately, and safely, transfer forces in the bridge system while the beam and deck sections were removed and replaced.

A 7′-4″ by 65′-8″ portion of deck above the beam field section was to be removed and replaced, as shown in Figure 15. Temporary support locations and jacking forces to ensure proper beam fit up during installation were provided on the contract plans as well.

Figure 15: Deck Removed, Existing Beam Still in Place.

It was necessary to install the new beam section and provide bracing to prevent lateral torsional buckling of the beam during the partial deck replacement. To accomplish this, only the top and bottom strut of the cross-frames attaching the new beam to the adjacent beam were initially installed and connected at both ends, prior to placing the deck. To allow the new beam to deflect as a result of the wet concrete, vertically slotted holes were provided on the connection stiffener on the girder. This would allow for the struts to act similar to "lean-on" bracing, but also allow the new beam to vertically deflect as concrete was placed (See Figure 16).

Upon completion of the deck placement, the diagonals could be connected, and then all members field welded to the connection stiffeners. Note that a new connection plate on the existing adjacent girder was also required. Existing cross-frame members utilized the ODOT standard detail of welding the member directly to the web. This type of connection does not allow the movement needed to accomplish this repair, and thus the connection stiffeners were needed.

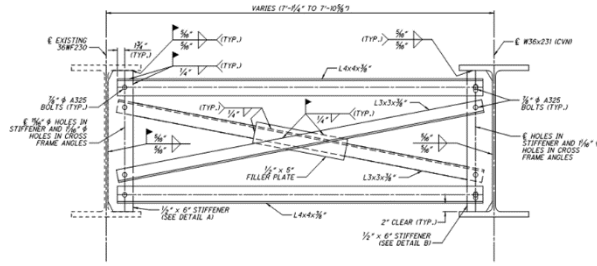


Figure 16: Typical New Cross-frame and Connection Stiffeners.

Increasing Vertical Clearance

As stated previously, the superstructure was raised 4 inches at each support. Two 2-inch steel shim plates were installed underneath each bearing in a tiered arrangement. The use of the shim plate stack allowed for the raising of the superstructure, without making height modifications to the beam seats.

The bearing shim plates at the fixed pier had to have anchor bolts located so that they did not conflict with the steel reinforcement in the pier cap. Existing plans were used to ideally locate the reinforcement. However, it was noted in the contract plans that the pier cap reinforcement location was to be verified in the field through the use of a pachometer, prior to fabrication of the new fixed bearing shim plates to ensure there were no conflicts with the anchor bolt locations.

The existing sliding plate expansion joints at each end had to be replaced since the bridge was being raised. The new detail utilized portions of the existing joint armor on the deck side, in combination with a new strip steel assembly, as shown in Figure 17. The existing L8x6x1 angle on the side of the existing deck was modified by cutting the horizontal leg, and then welding the steel retainer for the new strip seal gland. A portion of the abutment backwall was to be removed and replaced due to raising the bridge, so access was provided to make the necessary modifications. See Figure 18 for a comparison of the existing and new expansion joint.

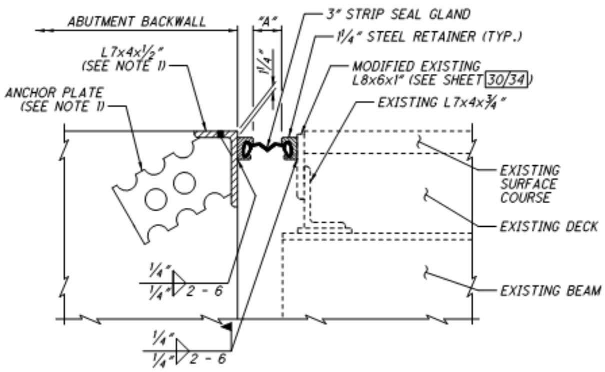


Figure 17: Expansion Joint Replacement Detail.

Figure 18: Expansion Joint Replacement (existing at left, new at right).

New approach slabs and modifications to the abutment wingwalls and backwalls were also required. Additional concrete repairs were made at the abutment backwall and at select beam seat and bearing pedestals. To better convey these repairs to contractors bidding on the project, inspection and field photos were provided on the contract plans along with the recommended detailed repairs.

Structural Analysis for the Beam Repair

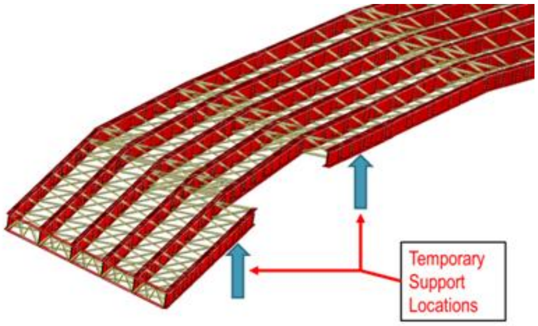
Bridge repairs and strengthening techniques require careful consideration of the behavior of the structure and load paths through the repairs and construction sequencing, especially when removing and replacing a portion of the bridge. Therefore, the design of the repair employed a 3D finite element model using LARSA (See Figure 19). The 3D FEM allowed for the accurate modeling of the superstructure elements and to develop the construction sequence for the partial removal of the deck, beam, and cross-frames.

Figure 19: 3D Finite Element Model, deck removed for clarity.

The 3D FEM was needed to be able to remove individual portions of the bridge and check stresses at each stage of the construction sequencing. Because of the curvature (dog-leg framing) and skew, a line girder or grid analysis would not have accurately captured the behavior of the superstructure as elements were removed and replaced. The 3D FEM also allowed for better prediction of the system deflections needed for camber and screed calculations, and to determine the temporary support and jacking locations that were to be shown in the contract plans. Figure 20 shows the placement of the temporary supports in the 3D FEM as well as in the field.

Aesthetic Improvements

Aesthetic improvements benefited those crossing the bridge via vehicle, as well as those who use the bridge for recreation purposes within the Cleveland Metroparks. The existing and deteriorated fence was removed from the bridge. The existing fence attached to the outside of the bridge parapet. A new fence was installed that attached to the top side of the bridge parapet. The parapets and deck overhangs on each side of the bridge were sealed to provide a better looking finish. The bridle path riding surface on the bridge was also replaced. Figure 21 shows the upper surface of the bridge before and after the repairs.



a) 3D FEM and Temporary Support Locations

b) Field Photo Showing Temporary Support Locations

Figure 20: 3D Finite Element Model, and Field Photo Showing Placement of Temporary Support.

Also, several improvements were made just beyond the bridge limits and along the bridle path. This included replacing damaged guardrail with new guardrail along the edge of the roadway, and replacing deteriorated portions of bridle path fencing as needed at both ends of the bridge.

a) Before Rehabilitation

b) After Rehabilitation

Figure 21: Overall Bridge Rehabilitation Comparison.

Summary

This paper presented two case studies that required unique solutions to repair, strengthen, and modify the steel superstructure, and the challenges associated with these unique solutions. The Lincoln Avenue Bridge over I-71 required strengthening of the steel girders due to the relocation of an interior Pier and subsequent change in span arrangement. The construction and loading sequences had to be carefully considered to determine the appropriate stresses in the repair plates and the existing girders. The bridge was also raised non-uniformly in order to provide necessary vertical clearance at the forward abutment, requiring consideration of the effects caused by the skew and bridge width on the amount of raise at each individual bearing. The repair assemblies were developed to give the contractor maximum flexibility in order to account for field conditions. The Lincoln Avenue Bridge provides a perfect example of how a steel bridge can be retrofitted and reused for a change in span arrangement.

The second case study considered the Hawthorn Parkway Bridge over US 422. Repairs to this bridge were necessitated by a full-depth crack in the fascia beam. It was clear that the bottom flange of the fascia beam had been damaged due to repetitive impacts by over-height vehicles. The existing beam was removed and replaced, requiring new field splices and specialized cross-frames that provided necessary stability bracing, but also allowed vertical deflection of the new beam as it was loaded with wet concrete. Since the bridge was curved and skewed, a 3D FEM was used to accurately capture the behavior of the bridge during the construction sequence associated with the partial removal and replacement of the deck, beam, and cross-frames.

These two projects illustrate how existing steel bridges can be repaired, strengthened, and/or altered to accommodate a modified condition, without requiring full superstructure replacement. The re-use of these structures saved construction time and costs as compared to full-replacement, while also extending the service life of these structures.

Acknowledgments

The authors would like to acknowledge Kokosing Construction Company as the prime contractor for the I-71 Design Build Project for the Lincoln Avenue Bridge, and ODOT District 8 as the owner of the bridge. The authors would also like to thank ODOT District 12 for the opportunity to develop the repair strategies for the Hawthorn Parkway Bridge. Union Industrial Contractors was the prime contractor for the Hawthorn Parkway Bridge repairs.

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