Buck O’Neil Bridge Demolition

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IBC 25-03

KEYWORDS: Bridge Demolition, Tied Arch, Plate Girder, Non-Redundant Through Plate Girder, Precut Analysis, Explosive Demolition, Dynamic Response, Segmental Girder Removal, Falsework Tower, Trestle Bridge

ABSTRACT: The Buck O’Neil Bridge over the Missouri River in Kansas City consisted of multiple bridge types. The main river unit consisted of three signature tied arch spans that were removed using explosives. The northern end of the northern arch was positioned above a levee wall that was to remain undamaged. A falsework tower was constructed to support the arch structure to remain and the arch was strengthened to resist demands experienced during the explosive event.

INTRODUCTION

The Buck O’Neil Bridge connects downtown Kansas City with North Kansas City. The bridge carries Hwy 169 over the Missouri River between Jackson and Clay County, Missouri. The bridge is a vital link for the adjacent communities and the local commuters. This signature bridge had been a prominent structure in the downtown Kansas City skyline since its construction in the 1950’s.

The bridge required replacement due to various structural deficiencies, but the bridge replacement was also intended to improve traffic flow into and out of the city. To quote MoDOT, “While safe, the bridge, which opened in 1956, is nearing the end of its projected lifespan. Even though the bridge underwent a short-term rehabilitation to extend its service life in 2018, a new bridge is needed to provide a crossing that will support continued use well into the future.”

The bridge consisted of multiple unique units of various bridge types. South of the river there were 3 approach units and there was also an approach unit to the north. For brevity, a discussion of the approach units north of the river is not included. An arial view of the southern approach and main span units can be seen in Figure 1.

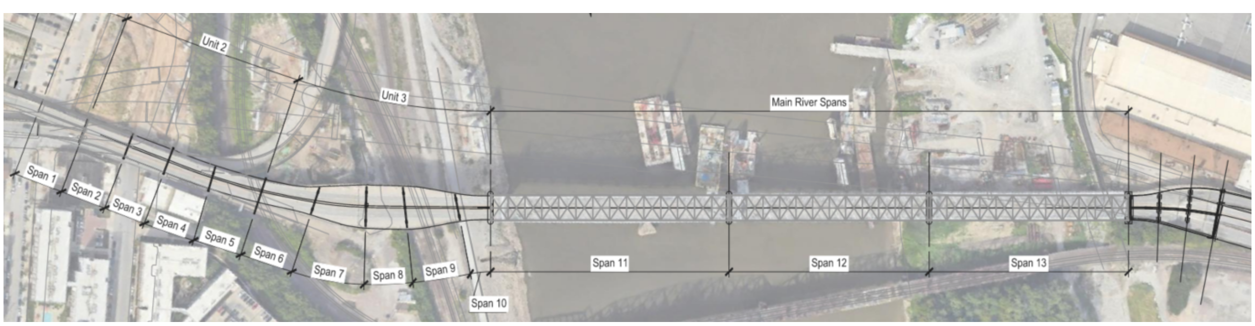

Figure 1 - Bridge Plan View

The furthest unit to the south (Unit 1) consisted of a non-redundant through plate girder span being braced by the cast-in-place deck. Bridge Units 2 & 3 can both be considered a more typical approach unit which consisted of continuous steel plate girders with connecting diaphragms for support of stringers. The main river spans (Unit 4) consisted of three unique signature tied arch spans with span lengths varying from 450' to 540' (see Figure 2).

Figure 2 - Main River Span Unit 4

A joint venture between Massman Construction Company & Clarkson Construction Company was awarded the project which included the demolition of the existing Buck O'Neil bridge and the construction of the new Buck O'Neil Bridge for a total contract price of $220M.

DEMOLITION SEQUENCE

The demolition sequence was influenced by four major factors.

- The new bridge tied into the same streets as the existing bridge so a portion of both the north and south ends of the existing bridge had to be removed before the new bridge could be completed.
- North bound traffic had to be maintained as emergency services for the airport came from south of the river.
- South bound traffic closure was allowed but only for up to 616 days.
- Work over the railroads would be slow and could bottleneck the entire project.

With these factors in mind, the project team chose to work in three phases. Phase 1 closed south bound traffic. The south bound Unit 1 and the south bound north approach Unit 5 were then demolished. This allowed the new south bound approaches to tie into the existing road network. Once complete, north bound traffic was diverted onto the new south bound bridge.

This allowed for the start of Phase 2, where the northern and southern ends of the existing bridge were demolished to allow for the completion of the new north bound bridge. This included the demolition of north bound Unit 1 & Unit 5 (south bound removed in Phase 1), Spans 2 & 3 (portion of Unit 2), and the main truss Spans 12 & 13 (portion of Unit 4). Following completion of Phase 2 demolition, construction of the new north bound bridge could continue. Once all new construction was complete, north bound traffic was switched over onto the north bound bridge and south bound traffic was opened on the south bound bridge.

Then Phase 3 of demotion was to remove the "island" remaining from Span 4 to Span 11. This includes the north end of Unit 2, all of Unit 3, and the south end of Unit 4. This portion of the demolition proved to be challenging, as there are multiple spans that are above and / or adjacent to two major railroad lines. The BNSF and Union Pacific railroads travel through this area south of the Missouri River. Phase 3 demolition was not on the critical path for construction of the new bridge.

Specifics on the various components of the three phases of demolition are to follow. But in general, following deck removal the approach unit superstructure was piecewise dismantled using cranes, and each of the three arch spans were removed using explosives. The engineered solution and execution of the demolition plan is the focus of this paper.

DECK REMOVAL OPERATION – UNITS 2 / 3

EQUIPMENT – The existing concrete deck was removed using (2) 56,000-lb John Deere 225 excavators positioned on the existing bridge deck.

Over the existing railroad and while above miscellaneous adjacent structures, the existing deck was saw cut into manageable sized panels and removed with a slab crab attachment. The remainder of the deck was removed using a hydraulic hammer attachment.

DECK EVALUATION – The reinforced concrete deck is assumed to span between the stringers and girder supporting elements.

The slab crab attachment was assumed to weigh ±2,000 lbs and with a supported deck panel weight of ±6,800 lbs, the combined lifted load was then ±8,800 lbs. This lifted weight controlled over the hydraulic hammer weight of ±3,800 lbs.

Based upon the noted lifted weight and provided specifications for the excavator, the track bearing pressures were determined for the condition when working over the front and for working over the side. The front-loaded condition results in a trapezoidal or triangular pressure distribution with equal loading between the two tracks. The side loaded condition results in a uniform pressure distribution with unequal loading between tracks. The controlling track bearing pressure was used in the evaluation.

It was determined that the deck was adequate for support of the excavator for general movements, but was not adequate for support of the excavator during active deck removal operations. Thus, while actively hammering or using the slab crab, the excavator tracks were specified to be positioned directly above the girder / stringer supporting members. In this manner the flexural demand on the deck was eliminated.

STRINGER EVALUATION – The stringer span exceeded the length of the excavator track, thus they were evaluated to support the full load from a single excavator track. The tributary dead load from the existing concrete deck along with the stringers self-weight was also applied. The stringers were continuous over the supporting diaphragm members (as seen in Figure 3). Controlling span lengths were used to determine beam demands. The framing plan consisted of multiple stringer sizes that were also considered.

Excavator positioning and movement was restricted such that multiple excavators could not load a given stringer or girder line at any time.

DIAPHRAGM EVALUATION – The diaphragm members (visible in Figure 3) act to support the stringers and transfer load to the supporting girders. The diaphragms are conservatively assumed to act as simple spans and the presence of the bracket members located below the diaphragm are disregarded.

Diaphragms have been evaluated to support the full load from a single excavator track and associated dead load from the existing concrete deck and structural steel weight.

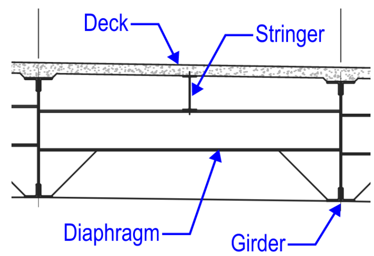

Figure 3 – Unit 2 / 3 Partial Cross Section

GIRDER EVALUATION – The girders directly support the diaphragms (thus indirectly support the stringers) and concrete deck and span between the supporting substructure. Each girder was unique, thus a structural model for each was created in Lusas to establish the demand during demolition operations.

The girders were built-up shapes composed of flange angles, flange cover plates, and web plates. There were various configurations of cross-sectional components along the member length (as detailed in the original design plans) that were accounted for both in the model and capacity calculations.

The dead load assigned to the structural model included the girder self-weight, in addition to the weight of the stringer, diaphragms, deck, barrier, & rail. The ever-changing deck loading along with the excavator weight was applied to represent the actual deck removal operation. The excavator loading was appropriately applied adjacent to the leading edge of deck removal.

The girders were modeled as independent straight beam elements, as the minor kinks in the bridge horizontal alignment were considered negligible. Any load sharing between girder lines (with force distribution through the diaphragms) was conservatively ignored.

For model verification, the deflection results from the structural model were compared to the original plan data to ensure there was good agreement.

Many stages of the deck removal operation were modeled and controlling demands at each unique cross section were compared to girder capacity.

DECK REMOVAL OPERATION – UNIT 4 (MAIN RIVER SPANS)

For discussion of the demolition equipment used, the deck evaluation, and the stringer evaluation, refer to the previous Unit 2 / 3 deck removal operation discussion. The evaluation was similar and is also applicable here.

The bridge deck was removed utilizing the hammer attachment. When removing deck over the river a debris catch barge system was positioned below to catch the falling concrete.

FLOORBEAM EVALUATION – The main arch span cross section framing is as shown in Figure 4. The floorbeam supports the stringers / deck and spans between the hanger / tie girder supports.

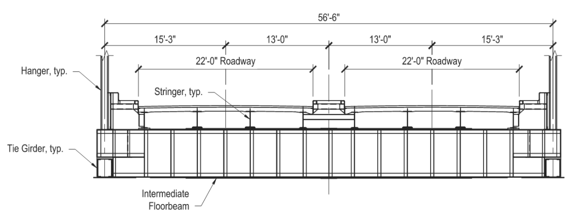

Figure 4 - Unit 4 Cross Section

The floorbeams were built up sections consisting of web plates and flange angles and there were also flange cover plates over a portion of their span.

The floorbeams were assumed to act as simple spans. They were evaluated to support the full load from two excavators along with the associated dead load from the existing concrete deck, stringers, and its own self weight.

The excavators were positioned within the limits of the existing roadway to maximize bending and shear demands.

ARCH EVALUATION – There were 3 spans, but 2 spans were identical. Thus only 2 structural models were required for evaluation. Refer to the following Main Span Arch Explosive Demolition section for further description.

The truss built-up cross section varied by member location, but an example is provided with Figure 6. The truss member material was either Carbon Structural Steel with Fy = 33 ksi or Low Allow Steel with Fy = 50ksi as indicated in the design plans.

The dead loads assigned to the structural model included the main truss self-weight, laterals, floorbeams, floorsystem, sidewalk, and deck.

Prior to defining the multi-staged deck removal operation, model verification was performed. The original design plans provide anticipated truss deflections and member forces for total dead load. The deflection and member force results from the structural model were compared to the original plan data to ensure there was good agreement. This comparison led to the inclusion of a dead load details factor to better match the as-built dead load condition.

During deck removal evaluation, the weight of both excavators was applied to the bridge appropriately positioned adjacent to the free edge of the deck removal line. Multiple stages of the deck removal process were considered to ensure the controlling demand to all arch components were captured.

Windward and leeward direction wind loads were assigned to each truss member. The applied full design wind load was conservatively considered during each stage of the deck removal operation.

For demolition operations, an operating level allowable stress is appropriate and was considered in the evaluation.

MAIN SPAN ARCH EXPLOSIVE DEMOLITION

The southern most span (Span 11) was a 540' span. The center (Span 12) and northern most spans (Span 13) were both 450' spans. Each of the three were tied arch spans, and the arch consisted of a unique truss style framing system as can be seen with the original design plans in Figure 5. Each of the three arch spans were to be demolished with the use of explosives.

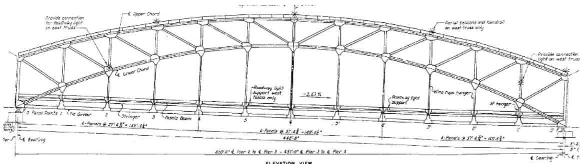

Figure 5 - Truss Elevation

The locations of explosives were selected to minimize the impact potential to the supporting piers, to minimize the entanglement of the blasted segments, and to ensure the resulting pick weight of each blasted segment was of a manageable size.

As mentioned previously the bridge was constructed in the 1950's. The typical arch truss member consisted of a built-up cross section composed of a combination of plates and angles. A typical cross section is as shown in Figure 6. Here the corner angles are attached to the flange and web plates with connecting rivets.

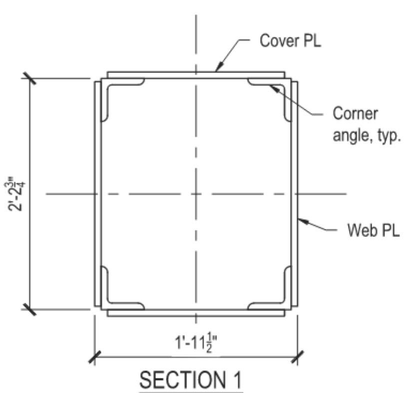

Figure 6 - Typical Section

The explosive charges work best when detonated on straight plates and with minimal plies. Thus, the existing steel sections must be precut or trimmed to provide a suitable section to receive the explosive charges. A typical precut cross section is as shown in Figure 7.

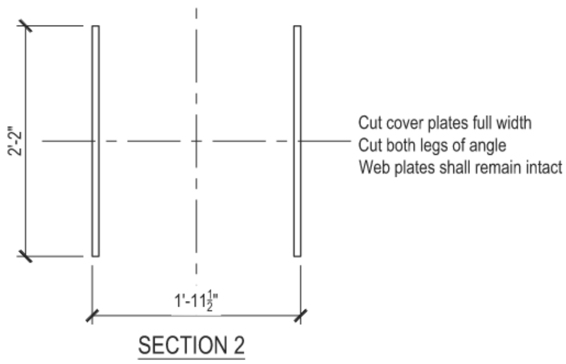

Figure 7 - Typical PreCut Section

The precuts on the main truss members were to be performed following the deck removal operation. In this manner the dead load demand to the precut sections could be minimized. The precuts on the main truss members were to be performed within 2-3 weeks of the explosive event. With this limitation the wind exposure was reduced.

The lateral bracing members that must be precut prior to the explosive blast event were to be precut in a short time duration period immediately prior to the blast during which wind speeds could be reasonably predicted to be below 30mph. This was specified to ensure there would not be a significant wind event following the removal of the lateral wind load resisting system.

In addition to the sectional precuts required at each of the explosive cut lines, less extensive precuts were also required along the upper chord of Span 11 to allow for a quick access rigging connection. Following the explosive event, Span 11 blasted segments were to be retrieved from the river. Through coordination with the coast guard, a 48-hour window was allowed for segment removal from the main channel. Thus, the blasted segments were to be removed in a timely fashion.

Isolated holes were cut into the top chord of the truss to allow for installation of shackles which were to act as connection points for the rigging. For ease of access, the rigging holes were to be precut into the top chord of the truss prior to deck removal. Thus, the reduced cross section at the precut rigging holes had to be evaluated for the controlling demand during the deck removal operation while including the effects of the full design wind loading. The resulting precut rigging cross section can be seen in Figure 8.

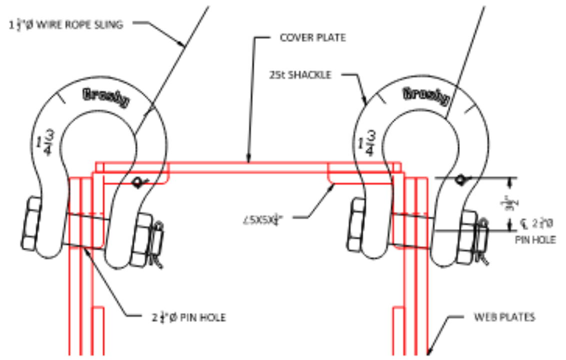

Figure 8 - Rigging PreCut Section

The cranes used for segment removal from the river the following:

- Manitowoc 2250 S3
- Manitowoc 7000 S3 Barge Mounted Ringer
- Manitowoc 999 S3

MAIN ARCH SPAN DYNAMIC RESPONSE

The northern end of the northern arch was positioned directly above a Corps of Engineers levee wall that runs along the north shore of the Missouri River as visible in Figure 9. This levee wall was to remain undamaged following the bridge demolition.


Figure 9 - North Arch

There were multiple options considered for the northern arch span, but ultimately it was decided that the preferred demolition method included the use of explosives (similar to the other two arch spans). The majority of the span (9½ panel points) was to be dropped using explosives, while the portion of the arch positioned above the levee (2½ panel points) was to remain in place following the explosive event. The instantaneous loss of a portion of the span resulted in a dynamic response for the structure to remain in place. The structure remaining following the explosive event can be seen in Figure 10.


Figure 10 - Remaining Structure

The structural response was evaluated using a transient dynamic analysis in lusas (structural analysis software). This complex analysis was required to more accurately predict the demands during the seconds immediately following the blast. The dynamic response eventually converges on the linear static response after enough time has passed.

The definition for the dynamic analysis required the inclusion of a few additional dynamic material properties assigned to the structural steel. These material properties are a function of the natural frequency and damping ratio of the remaining structure; thus, an accurate representation of the remaining structural stiffness and mass were required.

The four key components considered during the dynamic response evaluation included the following: the temporary bent design, the temporary post design, the Pier 4 bearing uplift restraint design, and the evaluation of the structural steel arch to remain following the explosive event.

TEMPORARY BENT – A temporary falsework tower was constructed to support the arch structure to remain following the explosive blast. As is visible in Figure 10, the temporary falsework support was positioned at the 2nd panel point from the end, plan identified as "T2".

Multiple options were considered, but ultimately a typical reinforced cast-in-place concrete column supported on spread footing was selected as the preferred option (see Figure 11).

Figure 11 - Temporary Falsework Bent

The dynamic response analysis was used to determine the complete range of anticipated demands. The maximum downward and upward reactions anticipated in the time immediately following the explosive event were determined. As time progresses, the support reaction ultimately converges on the at-rest (static) value as shown graphically in Figure 12.

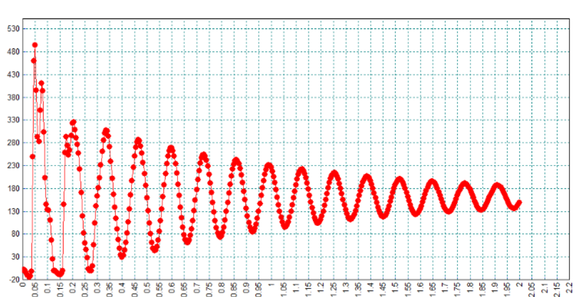

Figure 12 - Reaction Time History

The maximum downward reaction of approximately 500 kip controlled the design of the reinforced concrete column and footing design. But the results of the analysis model also indicated brief moments of uplift at the temporary bent support location. A maximum tensile reaction of 30 was anticipated. XIP wire rope tie down cables were installed to resist the instantaneous tension demand to keep the arch securely positioned on the falsework. With a close examination of Figure 10 and / or Figure 11 the tie down cable system is visible.

TEMPORARY POST – The temporary bent was positioned at the "T2" panel point. The original arch structure supported the tie chord with a cable hanger at this location. In this bridge type the floorsystem was essentially "hung" from the arch. Without modification the original bridge structure did not have an adequate load path for the resulting compressive reaction at the temporary bent during or following the explosive event.

Following complete deck removal and construction of the temporary bent, the tie girder was adequately supported at T2, thus the supporting cable hangers could safely be removed at this location. This allowed adequate clearance to implement the required strengthening measures.

A temporary post was custom fabricated and installed between the T2 (tie girder) and L2 (arch) panel points to provide a mechanism for load transfer between the tie girder and arch for the post blast compressive design load. With close study, the temporary post can be seen in Figure 10.

The temporary post demand range was defined with the dynamic response analysis as 320 kip compression to 200 kip tension.

A HP14x73 column was selected as the main post. The upper connection to the arch and lower connection to the tie girder required custom fabrication to ensure an adequate load path was provided to resist the substantial compressive and tension design loads.

PIER 4 UPLIFT RESTRAINT – The permanent bridge support bearings were not originally designed to prevent uplift. The top half of the bearing casting was directly not connected to the bottom half. Under normal operating conditions, uplift was never expected to occur.

Immediately following the explosive event the permanent bearings at Pier 4 experience uplift. Based upon the results of the dynamic analysis, the uplift reaction was only anticipated to occur for a very small fraction of a second, but this uplift reaction had to be restrained to ensure the arch remained securely supported on the permanent bearing.

The dynamic model results indicated the maximum uplift reaction was approximately 260 kip. To resist the anticipated uplift, a load path was provided via an external restraint system that consisted of a series of plates that were welded directly from the top bearing casting to the bottom bearing casting. A photo of the restraint system installed at the Pier 4 bearing is provided in Figure 13.

Figure 13 - Pier 4 Uplift Restraint

It was determined the anchor bolts into the pier capbeam were adequate for the anticipated uplift demands.

REMAINING ARCH EVALUATION – In addition to the dynamic load time history acting on the temporary structural components previously discussed, the remaining arch structure also experienced a similar wide range of demands that required consideration.

The dynamic response model was defined to provide results at 0.005 second intervals. Thus for the 2 seconds following the explosive event, 400 individual load case results were generated. Each arch member to remain (as visible in Figure 10) was evaluated for each of these 400 load case results generated during the dynamic response.

For reference, the temporary post, bearing uplift, and arch member demand time history plots each resemble that which is shown in Figure 12.

PIECEWISE TRUSS DISMANTLE

Following the explosive event the remaining truss was to be piecewise dismantled. That is individual members were to be lifted out via crane. Each intermediate stage of removal was considered to ensure the remaining structure was adequate for the ever-changing demands and braced conditions.

UNIT 2/3 GIRDER REMOVAL

Unit 2 consisted of 4 spans with 4 continuous girders, and Unit 3 consisted of 5 spans with either 4 or 8 continuous girders. The cross-section width varied at Unit 3 to accommodate a prior toll plaza (that has long since been removed).

All diaphragms, stringers, & girders were piecewise removed with cranes. The girders were removed utilizing multiple different cranes based on access restrictions and estimated girder segment weights. A summary of the various cranes used during removal of Unit 2/3 structural steel is as listed below:

Unit2:
- Grove RT650E
- Grove RT880E
- Mantis GTC-1200
- Mantis GTC-800
- Manitowoc 2250 S3

Unit3:
- Manitowoc 2250 S3
- Manitowoc 4100W S2
- Link Belt 248
- Manitowoc 7000 S3 - Barge Mounted Ringer

The girders were removed in single girder lifts and paired girder lifts with rigging installed through cut "windows" located in the girder web.

Massman Construction self-performed all crane access, crane adequacy, and crane clearance studies to ensure all girders could be removed. Genesis Structures performed all calculations to ensure girders could be safely lifted without concern and that the remaining structure was adequate following each lift. There were multiple occurrences of a single girder remaining in a cross section that required end bracing for stability as illustrated in Figure 14.

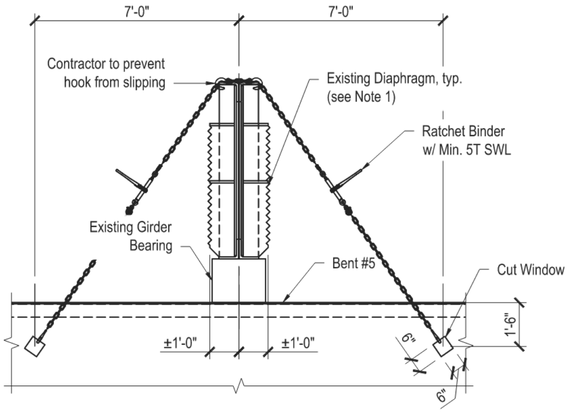

Figure 14 - Single Girder End Bracing

UNIT 1 DEMOLITION

The demolition plan for Unit 1, was to use an excavator with hammer attachment operating on the deck. The demolition of Unit 1 / Span 1 (southernmost unit) presented its own set of challenges. Unit 1 consisted of two unique bridge spans, one span for the southbound roadway and one span for the northbound roadway. The non-redundant through plate girder cross sections at Bent 1, showing both the southbound and northbound roadway, can be seen in Figure 15.

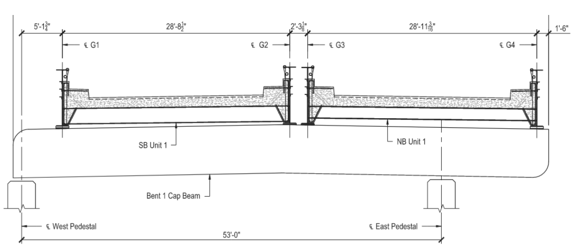

Figure 15 - Unit 1 Cross Section

The original bridge plan details indicated that the design intent was to provide flexural connectivity between the deck and girder. The transverse deck reinforcing was welded to the girder stiffeners to achieve the design intent. In this manner the deck acted to brace the girder.

A lusas structural analysis shell model was created to evaluate the demolition of Unit 1. Multiple stages of deck removal were evaluated. The interface between the deck and girder was represented with a frame connection at each vertical stiffener to represent the welded reinforcing connection.

DECK EVALUATION - The deck was the structural component for transverse load distribution to the steel plate girders. The deck was required to support the excavator during the deck removal operations by spanning between the girders. A photo of the deck removal operation can be seen in Figure 16.

Figure 16 - Unit 1 Deck Removal

The flexural bending moment demand in the deck from a representative sample stage is shown in Figure 17. The reported demand could then be compared directly to a traditional reinforced concrete design capacity on a per foot basis.

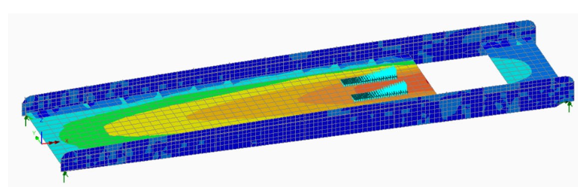

Figure 17 - Unit 1 Deck Stress Contours

GIRDER EVALUATION - The girder cross section constantly changes throughout the span. The flange thickness varies with the introduction / removal of cover plates and the web depth varies (min at Abutment 1 and max at Bent 1). In addition to the ever-changing cross section, the member unbraced length changes throughout the deck removal operation. The deck acts to brace the compression (top) flange, and as this brace is removed the girder is more susceptible to lateral torsional buckling.

With the variability noted above, a detailed eigenvalue buckling study was required to evaluate the girder during the deck removal operations. As deck is removed, the load resisted by the girders is reduced, but the unbraced length is increased, which reduces the girder capacity. Therefore, multiple stages are evaluated to ensure the controlling condition is adequate. The eigenvalue buckling study provides a factor of safety against buckling for the loaded condition.

The final deck removal stage was found to control the girder evaluation. The demand is at a minimum for this condition, but the capacity is also at a minimum. To further study this controlling condition a geometric non-linear model was evaluated where the applied load factor is incrementally increased until the girder system reaches a point of instability. The deflection of a key node near span is plotted against the applied load factor. The applied load factor as this plot is no longer linear is considered to represent the point of instability and / or the factor of safety against buckling. This method is further validation of the eigenvalue results discussed above.

It was determined that a girder end lateral brace was required at each end to provide stability during / following the deck removal operation. The end lateral braces, along with the end floorbeam, were not removed until the crane was rigged up to the girder for removal.

GIRDER REMOVAL – To ensure the girder remains in a level orientation during the lift, a precise center of gravity for the lifted segment had to be determined. This calculation had many variables as noted with the previous section.

The crane hook is then positioned directly above the calculated center of gravity. The southbound roadway was on a straight alignment, but the northbound bridge was curved in plan. To keep a lifted curved girder from rotating (about a longitudinal axis), an ideal spread between lift points was also determined (in addition to the center of gravity). The ideal spread is required to ensure the hook is positioned directly above the center of gravity in the transverse (to the girder) direction.

The lifted segment weight was used to determine the rigging demand and to select the appropriate hardware. An access window was cut into the girder web to allow for the rigging to girder connection. A girder lift operation can be seen in Figure 18.

Figure 18 - Unit 1 Girder Removal

SUBSTRUCTURE REMOVAL

The existing Bent 5 consisted of a built-up steel frame with pinned end connections and straddled an existing roadway and railroad. Without the longitudinal bracing effect that was provided by the supported girders, Bent 5 was not stable. Thus prior to girder removal at Bent 5, temporary stability bracing was installed.

The Bent 5 steel frame was temporarily braced back to the existing roadway below utilizing Dayton Superior Pipe Braces, as visible in Figure 19 below. Once braced, the girders above Bent 5 could safely be removed.

The site constraints we such that Bent 5 could not be removed in a single lift with the crane available. The installed stability bracing then served another purpose as it allowed for the bent to be removed in three segments. This reduced the lifted weight down to a manageable size.

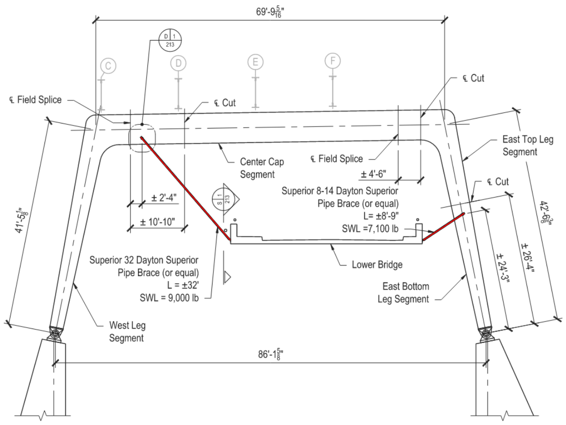

Figure 19 - Bent 5 Brace

SUMMARY

Construction of the new Buck O'Neil Bridge started in June 2021. The closure for southbound traffic started in February 2023 and ended in November of 2024. The demolition of the existing Buck O'Neil Bridge started in June of 2023. The complete demolition of the existing Buck O'Neil Bridge was successfully completed in July of 2025. A status photo showing the new bridge is provided below in Figure 20.

Figure 20 - Current Status

ACKNOWLEDGEMENT

The author would like to recognize the Missouri Department of Transportation, United States Coast Guard, and the United States Army Corps of Engineers for their involvement during the demolition plan review process.