Sequenced Removal of the Non-Redundant I-30 Bridge

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ABSTRACT: The I-30 bridge over the Arkansas River in Little Rock, Arkansas, consisted of a reinforced concrete deck supported by steel stringers and floor beams that were in-turn supported by two non-redundant plate girders. The multi-phase project required various demolition activities and engineered components that included pier brackets, strongbacks for temporary support of the discontinuous girders, custom lifting lugs, longitudinal bearing lock-up devices, false-work towers, and girder-flange lateral torsional buckling restraints.

INTRODUCTION

The I-30 Bridge connects Little Rock and North Little Rock and spans the Arkansas River in Pulaski County, Arkansas. The bridge is a critical link between the adjacent communities. The original bridge prior to replacement can be seen in Figure 1.

30 Crossing construction, Little Rock, Arkansas, September 25, 2020 [illegible] [illegible]

Figure 1 - Original Bridge

The original I-30 Bridge was constructed in the late 1950s. The main river crossing was highlighted by eight spans with span lengths ranging from 160′ to 210′. The main spans superstructure consisted of a reinforced concrete deck supported by steel stringers and floor beams that were in-turn supported by two non-redundant plate girders. The engineered solution and execution of the demolition plan for the main river crossing is the focus of this paper.

The existing I-30 Bridge necessitated replacement for various reasons. There have been both increasing vehicular demands and navigational demands. Additional travel lanes and shoulder width were required. The width of navigable channel for river traffic also required improvement. There were also structural deficiencies identified in past inspections. Addressing these considerations pointed to the necessity of replacement.

As another reason for replacement, the owner wanted a separation of local traffic and through traffic. With the new bridge, there are designated lanes to provide this separation. This separation is intended to provide better traffic flow for all travelers in the area.

In addition to the eight main river spans, there were also 15 approach spans to the south, 8 approach span to the north, and various on / off ramp bridge structures that were also demolished as part of this project. For brevity the approach span demolition will not be focused on in this paper.

DECK REMOVAL OPERATION

The bridge demolition operations generally worked from the top down. Thus, the first key component for removal was the bridge deck.

The reinforced concrete deck was removed with two excavators operating simultaneously on the bridge deck. Each excavator was equipped with an impact hammer to chip away at the concrete deck for removal.

When performing deck removal operations in a span above land, the deck debris was allowed to fall to ground below and was cleaned up later. When performing deck removal operations in a span above water, multiple catch barges were positioned on the river below the deck to catch the debris as seen in Figure 2. The catch barges minimized the amount of deck debris falling in the river, which sped up the clean up process.

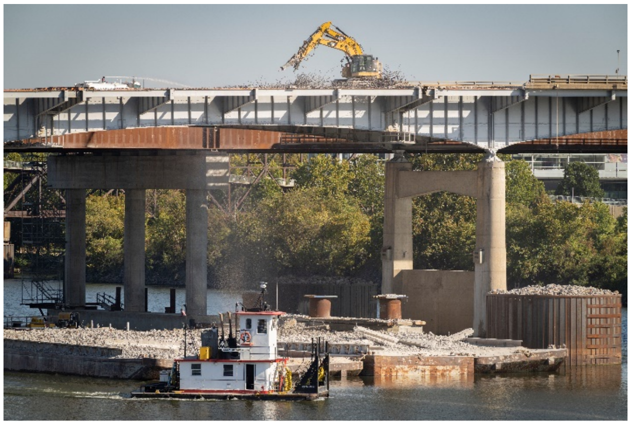


Figure 2 - Catch Barge

The two excavators hammering on the deck during deck removal is a non-typical bridge loading. The effects of the deck removal operation were considered for all structural bridge components. The typical structural elements considered in the evaluation are visible in Figure 3. In this photo the deck has already been removed.

All structural components, including the deck itself, but also the stringers, floorbeams, and non-redundant plate girders can also be seen with the typical cross section for Spans 17 - 19 (others similar) in Figure 4.

Figure 3 - Structural Components

EQUIPMENT - The selected excavator to perform the deck removal operations was a CAT 325F with H115s impact hammer. With the hammer attachment weight included, this excavator weighed approximately 60,000 lbs.

The loading applied to the structure from the excavator was a key consideration in the evaluation of the existing bridge. While working over the front, the excavator track loading is considered to be trapezoidal / triangular and while working over the side is considered to be rectangular. The loading diagram along the length of the track is dependent upon several considerations such as base machine weight, attachment weight, pick weight, and operating radius. Also required to define the track loading, the center of gravity of the excavator is established indirectly through information provided by the manufacturer. When governed by the tipping capacity of the machine, the provided allowable pick weight at a given radius is used to back into the machine center of gravity.

Following definition of the track loading diagram, an impact factor was applied to the excavator for use on the existing bridge. For general movement operations, an impact factor of 5% was included and for the hammering operation, an impact factor of 25% was included. This impact factor was considered to cover any additional loading inherent to the violent nature of the impact hammering operation.

To aid in deck debris removal, an additional piece of equipment was allowed to approach the lead excavators. Specifically, a CAT 966 Loader (with an approximate weight of 50,000 lbs) was evaluated to be within 10' of the lead excavator.

DECK EVALUATION - The deck was a typically reinforced cast-in-place concrete deck with reinforcement running in both directions in the top and bottom of the slab.

As the deck was constructed in the late 1950’s, the material grades were consistent with those of the time-period. The deck was specified to have a compressive strength of 3 ksi. The deck reinforcement was specified to have a design stress of 20 ksi, which is consistent with grade 40 rebar. With material grades less than today’s standards, the deck flexural capacity was an obvious structural component of interest. The deck capacity was established following a strength level reinforced concrete design. The factored demand followed guidance provided by The Manual for Bridge Evaluation based upon an operating level evaluation.

The deck is considered to span between the longitudinal structural elements (girders & stringers). As can be seen in Figure 4 there are five stringer lines between the two girders and there are three / four stringer lines outside the girders on the overhang. The width of overhang beyond the girder lines varies along the length of the bridge.

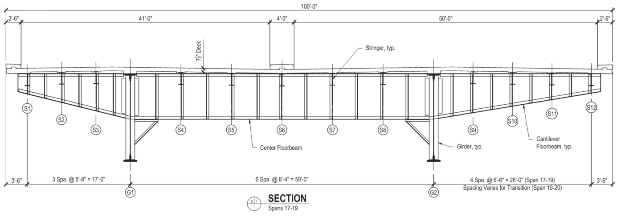
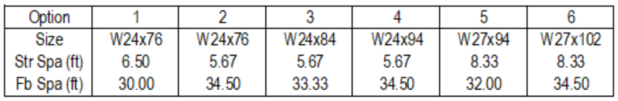


Figure 4 - Cross Section

The demand considers the deck self-weight, but is mainly governed by the demand applied by the excavator. The excavator track loading was positioned to maximize the flexural demand to the deck. The evaluation indicated that the excavator could freely walk between stringer / girder lines. But with the increased impact considered during the hammering operations, it was determined the excavator tracks must be positioned over the stringers / girder lines to reduce the deck flexural demands to an allowable level. Prior to deck removal operations, high visibility paint was used to clearly mark the stringer / girder centerlines to ensure operators could keep the excavator tracks above the supporting elements.

STRINGER EVALUATION - Another structural component of interest during the deck removal analysis were the stringers. The stringers support the deck and span between floorbeams and were to provide direct support for an excavator track. There were multiple stringer size (various wide flange sections) and span length options for consideration as seen in Figure 5.



Option	1	2	3	4	5	6
Size	W24x76	W24x76	W24x84	W24x94	W27x94	W27x102
Str Spa (ft)	6.50	5.67	5.67	5.67	8.33	8.33
Fb Spa (ft)	30.00	34.50	33.33	34.50	32.00	34.50

Figure 5 - Stringer Options

The excavator was conservatively assumed to be in a side loaded condition with a single track in-line with a stringer to maximize the stringer demand. The weight of the excavator was assumed to be operating at midspan for the flexural evaluation and was assumed to be operating at endspan for the shear evaluation. For the flexural evaluation, dead loads were assumed to be resisted by the non-composite section and the excavator was assumed to be resisted by the composite section. For shear evaluation, all loads were assumed to be resisted by the stringer web. Both the flexural and shear demands assume the stringers to act as simple spans supported at the floorbeams.

A material yield strength of 33 ksi was assumed for structural carbon steel. An operating level allowable stress of 0.75fy for flexure and 0.45fy for shear was assumed.

FLOORBEAM EVALUATION – The floorbeams support the stringers and transfer load directly to the girders. The floorbeams are evaluated for direct support of the excavator(s) during deck removal operations. The unique cross section (see Figure 4) provides multiple floorbeam conditions for evaluation. The center floorbeam and cantilever floorbeams are evaluated differently as described below.

The center floorbeam is indicative of the region of floorbeam between the two girders. For this condition, two excavators are assumed to be fully supported by a single floorbeam. The longitudinal transfer of load along the stringers to adjacent floorbeams is conservatively ignored. For the flexural evaluation the excavators were assumed to be centered between the girders (while keeping tracks above the stringers) and for the shear evaluation the two excavators were assumed to be shifted to one side of the center floorbeam span.

The cantilever floorbeam is indicative of the region of floorbeam outside of the girders. For this condition, only a single excavator is assumed to operate. Similar to the center floorbeam evaluation, the full weight of an excavator was assumed supported by a single floorbeam (any potential longitudinal distribution of load was conservatively ignored). The excavator was assumed to have the outer track positioned over the 2nd to outermost stringer. This was the furthest point that was required for the excavator to reach the outer limits of the deck during removal operations.

It was unclear from the design plans whether there was a shear transfer mechanism between the top flange of the floorbeam and the concrete deck. For both the flexural and shear evaluations, all loads (dead & live) were assumed to be resisted by the non-composite section. It was confirmed during the demolition operation that this assumption was appropriate.

There were multiple floorbeam sizes consisting of a built-up section of various angle sizes as flanges with constant web dimension of 7/16″ x 90″. The girder spacing was constant, but the cantilever floorbeam dimension varied along the length of the bridge. The floorbeam spacing, thus tributary dead load, also varied by span. All options were considered in the evaluation.

The steel yield strength and allowable stress for the floorbeams were assumed the same as the stringers (as previously discussed).

GIRDER EVALUATION - The two built-up plate girders that span between piers represent the main structural component for this bridge. There are eight girder spans with lengths that vary from 160′ to 210′. By definition, the girders are non-redundant, as with failure of a single girder the bridge would collapse.

The girders section properties are constantly varied along the length of the bridge by means of cover plates and a tapered web depth. There are combinations of up to 4 cover plates with total combined thickness of 3½″ outboard of the flange angles. The web thickness is consistently ¾″ thick, but the depth varies from a minimum of 10′ at midspan to a maximum of 18′ at the interior piers. The flange angles are held constant throughout, other than the slight upsize in the southeast region where the overhang width increases. This ever-changing cross section was considered in the girder evaluation.

The girders were evaluated for the deck removal operations. The deck was assumed to be removed incrementally along the length of the bridge with the excavators. Each girder was assumed to support a single excavator as two excavators were allowed to work at the leading edge of deck demolition.

The eight main river spans have a combined total bridge length of 1581′ and the deck width varied from 91′ to 100′. With a thickness of 7½″, this equates to approximately 4100 yd3 of concrete for removal. Even with two fully equipped excavators the deck removal process was expected to take some time.

In an attempt to reduce the total time duration required for demolition, it was beneficial to begin the girder removal prior to complete deck removal from all eight main river spans. But during the initial demolition planning, the exact timing of deck removal vs girder removal was uncertain. For this reason, the deck was assumed to be removed to specific locations for multiple girder removed scenarios. By evaluating multiple options (as described below), this provided greater flexibility for the actual timing of demolition operations. In all scenarios, it was assumed that the deck was removed from the North end at Bent 25 and progressed towards the South end at Bent 17.

<u>Begin Scenario</u> - The deck was assumed to be removed along the entire length of the bridge prior to any girders being removed.

<u>Pick 1 Scenario</u> - The deck was assumed removed to at least Pier 23 prior to the removal of Pick 1 (far North end segment). For this scenario, the girders in the Pick 1 region were removed and the deck / excavators were incrementally removed / moved from Pier 23 to Pier 17.

<u>Pick 2 Scenario</u> - The deck was assumed removed to at least Pier 21 prior to the removal of Pick 2 (2nd segment from far North end). This evaluation considers the Pick 1 & 2 girders removed and the deck / excavators were incrementally removed from Pier 21 to Pier 17.

<u>Pick 3 to Pick 20 Scenario</u> - Prior to Pick 3 the deck was assumed to be completely removed. For Pick 3 to Pick 20 the structural steel (girder, floorbeam, stringer, and lateral bracing) is incrementally

removed and the girders are evaluated for each scenario.

Flange Brace – As mentioned previously, the typical girder cross section includes cover plates for the top and bottom flange, but there are sections without any cover plates. The original design took advantage of the dead load contraflexure points, i.e. the region with minimal flexural demand. The lateral torsional buckling capacity in these regions without cover plates greatly reduced the girder allowable stress.

During deck removal, there were multiple girder locations identified that required additional strength to overcome the bending demands. The six identified locations were all located in negative moment regions, where the bottom flange was in compression. There were multiple options considered to improve the evaluation, but ultimately it was decided to install a bottom flange brace (as visible in Figure 6) at these locations to reduce the unbraced length of the compression flange and increase the girder lateral torsional buckling capacity.

Figure 6 - Pipe Brace

To act effectively by preventing the lateral displacement of the compression flange, the girder flange brace was required to resist 2% of the compressive force in the flange. It was conservatively assumed that the flange demand was equivalent to the max allowable demand in the flange. As mentioned above, the girder flange that required a brace did not have any cover plates and only consisted of 2 flange angles.

A Dayton Superior pipe brace with 25,000 lb capacity was selected as the flange brace. The brace was welded between a floorbeam bottom flange and the girder bottom flange (at midspan between floorbeams). This effectively cut the girder unbraced length in half, which provided the lateral torsional buckling capacity increase needed to exceed the demand during the deck removal operation.

GIRDER REMOVAL OPERATION

Following removal of the deck, the structural steel was then planned for removal. The portions of the bridge structure above land were removed with a 275 ton capacity Manitowoc 999 Crane. The portions of the bridge above water were removed with a 600 ton capacity barge mounted Manitowoc 4600 Ringer crane. The location of the girder dictated the picking radius required and associated pick size that was allowed.

Through many iterations and complete evaluations of both the existing bridge and the necessary various temporary works components, a girder removal plan was established. With the selected plan, there were 20 stages of girder removal.

The selected girder removal plan considered the temporary condition of the bridge following each completed stage of removal. Various items for consideration included transverse and longitudinal wind loading, changes to the girder unbraced length as floorbeams and lateral bracing members were removed, and the temporary longitudinal, transverse, and vertical supports requirements.

During the girder removal operation there were various temporary works required to execute the selected sequence of structural steel removal.

PIER BRACKET - As portions of the bridge were cut and removed, temporary support was required for stability of the remaining structure during intermediate stages of the demolition. Temporary support was provided with a pier bracket affixed to the existing piers as seen in Figure 7.

Pier brackets were required at Piers 18 – 24. Thus pier brackets were required at 7 total locations, but only 3 complete pier brackets were fabricated. As a pier bracket was no longer needed at a given location, it was simply disassembled and moved to the next pier based upon the selected girder removal sequence.

A girder removal sequence and the specified location for girder cut lines were selected to keep pier brackets out of the main navigational channel between Pier 20 & 21.

Figure 7 - Pier Bracket

The service level design reaction at the pier bracket was established as 200 kip per side / girder.

Primary Members – The main frame structural components of the pier bracket frame consisted of a top horizontal member, a vertical member (adjacent to the existing pier), and a diagonal member. Each of these main frame members were composed of two HP14x73 Grade 50 beams welded together along the full length of the top and bottom flange. As can be seen in Figure 7, a pier bracket frame was positioned directly below each girder.

When the pier bracket was engaged, the weight of the supported superstructure segment above was transferred through the pier bracket to the existing pier. This load transfer resulted in a tension demand in the top horizontal member, a bending and shear demand applied to the vertical member, and compression in the diagonal member.

The diagonal member compression capacity took advantage of the composite action of the double HP beam. The effective length of the column was improved by increasing the weak axis radius of gyration. The vertical member flexural capacity also benefited from this composite action, with the elimination of the lateral torsional buckling limit state.

In addition to the pier bracket member design components just discussed, there were also various other necessary components for the temporary pier bracket support system at the existing piers including the PT Beam, corbel and pipe stands.

PT Beam – The upper support reaction of the pier bracket was provided by clamping the frame to the existing pier. This clamping force was provided through use of 1¼″ diameter 150 ksi threaded bars post-tensioned (PT) to 130 kip each and W24x117 beams as can be seen in Figure 8. A clamping force was provided to ensure that when the maximum load was applied, the vertical member would stay securely in contact with the existing pier.

Figure 8 - PT Beam

If the pier bracket upper support reaction was provided in line with the top horizontal member, the vertical member shear and bending demand would have been eliminated. But the relation of the pier bracket to the top of the existing pier resulted in a support location that was positioned 4′ below the top horizontal member. The resulting flexural demand on the vertical member necessitated the addition of cover plates near this point of support to increase the flexural capacity.

Corbel - The corbel was a cast-in-place concrete pedestal on the face of an existing pier that provided the vertical support for the pier bracket self-weight and applied reaction from the superstructure above. The corbel also provided resistance for all lateral wind loads acting on the supported girder. The corbel can be seen in Figure 9.

To support the self-weight of the corbel during casting, rebar was attached to the face of the existing pier with drill and epoxy methods. Specifically, Hilti HIT-RE 500 V3 adhesive was used to bond the #5 Grade 60 mild reinforcing for temporary support of the corbel. Once the corbel concrete had cured and was being considered for support of the vertical loads from the pier bracket, this mild reinforcing was neglected. It was determined that a much more substantial means would be required to provide ample resistance for the vertical support demands.

Figure 9 - Corbel

Multiple options were considered, but ultimately a system of post-tensioned bars acting on the corbel served as the primary means of resistance for the bracket reaction. There were 4 - 1¼″ diameter 150 ksi threaded bars stressed to 150 kip each. 1 ¾″ diameter holes were drilled through the full width of the existing pier. A nut and base plate were provided on the back side of the pier to anchor the high strength bar. This effectively eliminated the concrete breakout failure limit state that proved to be an issue when partial depth reinforcing & epoxy options were considered.

Pipe Stands - The elevation difference at the point of support between the top of the pier bracket and the bottom of the girder was accounted for with the introduction of a pipe stand as seen in Figure 10. This pipe stand was required to resist the vertical and lateral demand acting on the pier bracket.

Pipe stands were designed to support the jack stands at the end of the pier brackets. The pipe stands were affixed to the top of the pier bracket and varied in height according to pier location. A typical connection detail at the top of the pipe included a cap plate with cruciform shaped stiffener section below that could be inserted into the top of the pipe stand.

Apart from the height of the pipe stands, a typical design was used for the pipe cross-section, cap plate, and the cruciform shaped stiffener. Demand to the pipe stand was evaluated for the maximum gravity and lateral loading at the pier brackets.

Figure 10 - Pipe Stand

A LUSAS finite element model was used to evaluate stresses in the pipe stand elements. The pipe and associated plates were modeled as shell elements. The cap plate and stiffeners are designed with grade 50 steel and the pipe was assumed to be a minimum of grade 36.

Uplift restraints were included on either side of the pipe stands, although no uplift was anticipated at any of the demolition stages. The lugs and associated rigging were designed to resist 30 kips of uplift at each girder.

A detailed geometry study was performed to determine the anticipated shim heights above the jack stands at the top of the pipe stands. A minimum shim height of 3 in. was used to determine the maximum height of pipe stand at each pier location. As pier brackets and pipe stands were intended to be reused at multiple pier locations throughout the demolition process, a standard pipe stand height of 3′-9″ was specified. For locations with larger gaps between the top of pier bracket and bottom of girder, stub column sections were designed to fill in the empty space and minimize the required shim heights.

Existing Piers Evaluation - With the pier bracket self-weight and eccentric support of the partially demolished superstructure, a flexural bending demand was imposed upon the existing piers. An evaluation of the existing piers was required to confirm adequacy for these temporary bending demands. This flexural bending demand was considered coincident with a reduced axial load as the deck and portions of the bridge structural steel have been removed.

Two pier sections were evaluated and were considered to control over the other pier locations. The maximum demand to the bracket / pier during girder demolition was associated with Pier 22. The evaluation for Pier 22 controlled over the evaluations of Piers 19-23. Pier 24 was also evaluated as it contained fewer and smaller longitudinal reinforcing bars but also experienced less demand as the pier bracket reaction was not as severe.

It was conservatively assumed that two longitudinal pier reinforcing bars in the proximity of the corbels were damaged during the P-T bar installation. These bars were neglected in the development of the pier column capacity curve. The cross-section of the pier considers the plan specified 1:48 taper 30' down from the top of pier. To determine capacity of the piers, spColumn evaluations were completed.

STRONG BACK - After the required pier brackets were securely in place, a midspan segment could then be prepped for removal. The strong back system was installed at each end of a midspan girder segment to allow for a complete section cut of the girder prior to crane support. This allowed for the timely girder cutting process to occur without support of the crane. The strongback system can be seen in Figure 11.

Figure 11 - Strong Back

At each girder end, the system consisted of double strongback beams (2-HP14x89) and a hanger system to support the span segments being cut. The hanger system consisted of an upper and lower double channel assembly to receive the 1¼″ diameter 150 ksi threaded bars. At the front location, the bars were stressed to 100 kip each, thus 200 kip total. The rear location does not see any tensile load, thus bars were only stressed to 20kip each simply to keep the strongback beams securely in position.

FALSEWORK TOWER - The structural steel in the southernmost span was removed during the final stage of superstructure demolition process. This portion of the bridge was above land and was removed with a Manitowoc 999 crawler crane. The size of this crane dictated much smaller girder segments for removal as compared to the much larger barge mounted crane.

A pier bracket was installed to the south side of Pier 18. With only the pier bracket installed, as the initial girder segment was removed adjacent to Cell #17 (as shown in Figure 12), the remaining structure would not have been stable. The center of gravity of the remaining structure was beyond the support provided by the pier bracket. This unstable condition was avoided with the introduction of a falsework tower (in addition to the pier bracket).

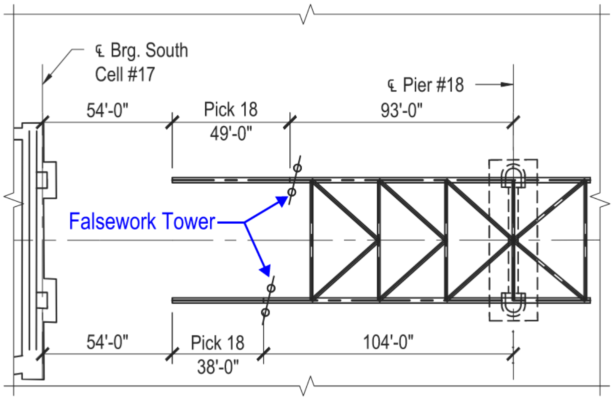


Figure 12 - Falsework Tower Layout

A pair of isolated falsework towers were used to support the girders near midspan during demolition. The falsework tower at each girder consisted of a header beam (Double W24x117) below each girder spanning between two piles (36″ diameter with ½″ wall thickness). Figure 13 shows the falsework towers in use following the removal of the Pick 18 (ref Figure 12).

The falsework tower location was immediately adjacent to a proposed location for a retaining wall. In fact, it was the retaining wall that forced the unsymmetric placement of the falsework towers (visible in Figure 12). Through proper planning corrugated metal sleeves inserts were installed prior to construction of the retaining wall. The piles could then be driven within the sleeves to limit the potential settlement concern of the newly constructed retaining wall.

Figure 13 - Falsework Towers

LIFTING EVALUATION - The superstructure was removed in cut sections with a crane. The structural steel had to be evaluated for the various picked conditions.

There were 11 lifted segments or approximately 5 spans removed as paired girder picks (shown highlighted yellow in Figure 14) with the barge mounted crane. And there were 9 segments (thus 18 total individual girder picks) or approximately 3 spans removed as individual girder picks (shown highlighted orange in Figure 14) with the land-based crane.

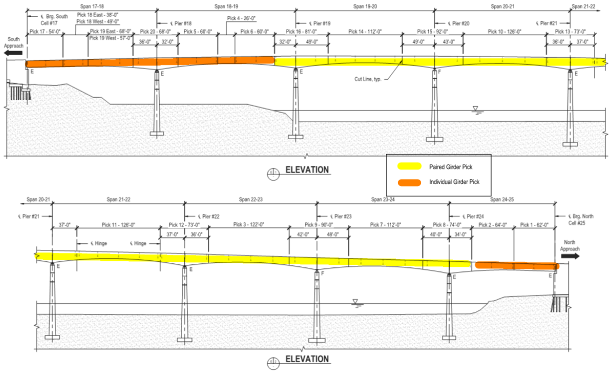


Figure 14 - Girder Pick Condition

Paired Girder Pick - The barge mounted ringer crane lifted large combined sections of the bridge. A paired girder pick included portions of both girders in addition to multiple floorbeams, lateral bracing, and stringers as shown in Figure 15.

Figure 15 - Paired Gider Pick

A transverse spreader was used to ensure there was no transverse component of sling force acting on the cut girder segments. The angle of the inclined slings in the longitudinal direction was limited to 60 deg to limit the axial compression induced into the girder segments.

The paired girder picks were lifted with custom lifting lugs as visible in Figure 16. The existing girder had to be modified at the custom lug locations. The bearing capacity of the web plate was improved with the addition of cheek plates welded to each side of the web. A 4"x28" slot was also carved into the top flange on each side of the web to provide clearance for the lug plates. Large 3½" diameter Grade 110 pins were used both at the interface with the web and the sling.

Figure 16 - Custom Lifting Lug

Individual Girder Pick - The portions of the bridge not accessible by the barge mounted crane, were removed by the land-based crane. The Manitowoc 999 made use of flange grabs to fully support a given girder segment prior to removal. Once fully supported by the crane, the girder segment for removal was then torch cut and lifted out of position as shown in Figure 17.

Figure 17 - Individual Girder Pick

There was a total of 18 individual girder (9 segments) pick operations performed. The individual girder pick size varied from a minimum of approximately 26′ long with weight of 25 kip to a maximum of 68′ long with weight of 159 kip. Depending on the pick weight and geometry either a Crosby NS-35 or a custom 50 Ton flange grab option was used.

The local demand on the top flange was evaluated for the picked condition. The longitudinal stress in the top flange caused from global strong axis bending was combined with the localized transverse bending stress at the flange grab to ensure the top flange was adequate for this condition.

To stay within the barge mounted crane lifting chart, there were many locations where a reduction in overall weight of a paired girder pick was required. To accomplish this, there were multiple floorbeams, stringers, and lateral bracing members removed as individual segments with similar techniques.

LONGITUDINAL BEARING RESTRAINT - Per the original design plans the fixed bearings were at Pier 20 & Pier 23. As sections of the superstructure were removed the longitudinal restraint for the remaining structure was altered. Without a direct load path to a fixed bearing, the existing expansion bearings required modification to provide a longitudinal restraint.

The longitudinal restraint was provided with installation of a bearing bumper system as seen in Figure 18. A restrainer plate was rigidly attached to the top of the pier on each side of the bearing. Each restrainer plate connection was accomplished with 8 – ¾″ diameter A36 threaded rods bonded with HILTI HIT-RE 500 V3 epoxy. At the extreme ends of the restrainer plate were bumper plates that protruded beyond the edge of the bearing and prevented any differential displacement between the bearing and the pier. In this manner, the bearing was locked into the pier and the longitudinal expansion function of the bearing was eliminated.

Figure 18 - Longitudinal Bearing Restraint

The longitudinal restraint system could be installed at any time, but was only engaged with shims wedged between the restrainer plate bumper system and the expansion bearing base plate. By engaging this longitudinal lock up device just prior to needing it, temperature loads could safely be ignored, and only longitudinal wind loads acting on the relevant floorbeams were considered.

FINAL GIRDER PICK BRACE - The last remaining girder segment removed was at Pier 18. At this location the girder was removed as individual segments. To lift as individual girder segments the floorbeams and lateral bracing had to be removed first. Braces were installed to provide lateral wind stability in this temporary condition.

Temporary bracing was required prior to the removal of the existing floorbeam and lateral bracing. A bracing detail similar to the one used to brace girders during deck removal was used for this condition. Two Dayton Superior 33 HD Jumbo Pipe Braces were provided for each girder and were anchored to the existing pier beam.

The photo in Figure 19 shows the first of the final individual girder segments during removal. The temporary bracing providing lateral stability for the final girder segment remaining on the pier / pier bracket is visible.

Figure 19 - Final Girder Brace

The brace bottom connection plate for each of the four braces (2 per girder) was welded to a single common steel base plate that was anchored to the top transverse pier beam. The brace top connection plate was welded to the girder web. With the lateral wind pressure and defined brace geometry a brace force was determined and both connections were evaluated.

Given the fixed length of the braces, the brace connection point to the girder webs would not necessarily be located at a vertical stiffener angle location. Using the resulting loads in each direction and dimension of the brace foot (3.5×6.25"), a LUSAS model was developed to examine the resulting stresses in the girder web.

CONSTRUCTION / DEMOLITION STATUS

Construction began on the new Eastbound I-30 Bridge in October 2020. The new bridge alignment did not conflict with the existing bridge as it was positioned east of the existing I-30 Bridge. The construction of this bridge was completed and was officially opened to traffic in September 2022. It was at this time, that both directions of traffic were switched over to this newly constructed bridge. A photo of the new bridge following removal of the existing bridge superstructure can be seen in Figure 20.

Figure 20 - New EB I-30 Bridge

The existing I-30 Bridge demolition began immediately thereafter also in September 2022. Following the demolition plan discussed herein, the existing I-30 Bridge demolition was completed in June 2023.

Construction of the new Westbound I-30 Bridge began in November 2022. The new Westbound I-30 Bridge has approximately the same alignment as the existing I-30 Bridge did. Even though they share the same basic alignment, the substructure construction could occur simultaneously with the demolition, but the construction of the new superstructure had to trail the demolition. Following the successful demolition of the existing I-30 Bridge, this new Westbound I-30 Bridge is anticipated to be open to traffic in August 2024.

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