SETTING THE STAGE FOR SUCCESSFUL CONSTRUCTION RESEQUENCING OF A MAJOR RIVER BRIDGE

SHANE R. BEABES

WILLIAM F. ALKO

RAGHU KRISHNASWAMY

BIOGRAPHY

Shane R. Beabes, PE, is a Senior Technical Leader and an Associate Vice President at AECOM. He has a Bachelor of Science in Civil Engineering from The Pennsylvania State University and a Master of Engineering with a structural emphasis from The University of Maryland. Mr. Beabes is the Chair for the AASHTO/NSBA Steel Bridge Collaboration Committee, Task Group 13 – Analysis of Steel Bridges, as well as a member of Task Group 12 – Design for Economy & Constructability and Task Group 10 – Erection of Steel Bridges. He is also a member of the International Bridge Conference Executive Committee.

William F. Alko, PE, serves as a Technical Leader in the Structures Department where he manages project design efforts and acts as a technical resource for all structural design in the AECOM Pittsburgh office. He earned his Bachelor's degree at the University of Pittsburgh, and is a member of the American Society of Civil Engineers (ASCE), and the Association for Bridge Construction and Design (ABCD) Pittsburgh Chapter. Mr. Alko has over 27 years of experience in the design and analysis of steel and concrete bridges and transportation related structures. He is a Registered Professional Engineer in the Commonwealth of Pennsylvania and the State of Ohio.

Raghu Krishnaswamy, PE is currently managing the Western Pennsylvania Structures Department based in Pittsburgh. Mr. Krishnaswamy has a Bachelor of Engineering in Civil Engineering from India, Master of Science with Structural Engineering focus from The Pennsylvania State University, and a Master of Business Administration from The Carnegie Mellon University. He is a member of multiple national and local professional societies. He has 11 years of experience designing steel bridges and transportation-related structures.

SUMMARY

The 11th Street Bridge was part of an overall design-build-to-budget project let by the District Department of Transportation, in Washington, DC. The Bridge is a 915-foot long, five-span, continuous, seven girder steel bridge over the Anacostia River. The framing plan consists of curved, tangent, horizontally kinked and splayed I-girders with K-type cross frames.

The bridge was analyzed using a 2-D grid/grillage model and detailed for the originally proposed full-deck width construction. However, after the structural steel was fabricated and substantially erected, the sequence of construction was changed from a single stage to three stages of construction in order to accommodate renewed project objectives. The revised sequence required a rigorous re-analysis to preserve 1,663 tons of new fabricated structural steel.

SETTING THE STAGE FOR SUCCESSFUL CONSTRUCTION RESEQUENCING OF A MAJOR RIVER BRIDGE

Introduction

The 11th Street Bridge was one of three main river crossings for an overall design-build-to-budget project let by the District Department of Transportation, Washington, D.C., USA. The project, awarded based on a $260M best-value, design-build procurement, included extensive ramp reconfigurations on both sides of the Anacostia River to reconnect improved sections of the Anacostia and Southeast Freeways (Figure 1). The project goals included reducing congestion, improving vehicle and pedestrian mobility, and providing alternate evacuation routes out of the District. Collectively, the project served as a critical element in revitalizing the Anacostia neighborhood and supporting planned development as part of the Anacostia Waterfront Initiative. AECOM was a major partner on the project and the engineer for the 11th Street Bridge.

Figure 1. Aerial View of Project Interchange (Substantially Complete).

Bridge Overview

The new 11th Street Bridge replaced the existing 11th Street Bridge and separated the local vehicular and pedestrian traffic from the interstate through-movements (Figure 2).

Figure 2. New 11th Street Bridge Separates Local from Interstate Traffic.

The new bridge is a 916-foot long, 5-span continuous (169.75' – 170.00' – 234.00' – 171.25' – 170.50'), steel haunched-girder bridge over the Anacostia River. The bridge was constructed of seven plate girders fabricated from M270 Grade 50W steel with linear haunches transitioning to the piers. The haunches were required to meet the project-specific aesthetic criteria (Figure 3).

Figure 3. New 11th Street Bridge.

The bridge typical section accommodated four 11-foot lanes of traffic and a 17-foot wide multi-use pedestrian and bicycle sidewalk (Figure 4).

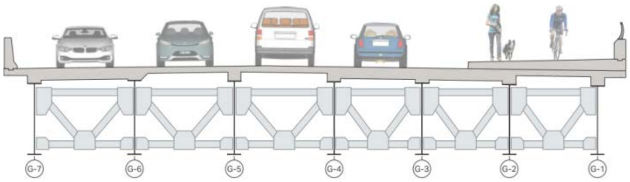

Figure 4. Ultimate Bridge Typical Section.

Sequence of Construction

In accordance with AASHTO LRFD (1), staged construction refers to the situation in which the superstructure is built in separate longitudinal units separated by a construction joint.

The originally proposed sequence of construction for the 11th Street Bridge required all seven girders to be erected and the deck placed the full-width of the ultimate typical section. The sidewalk and bridge barriers would then be placed and all four lanes of traffic shifted to the new bridge.

However, during construction of the bridge, the contractor re-sequenced the maintenance-of-traffic scheme for the project to meet renewed project objectives, requiring the new 11th Street Bridge to be opened to traffic early. This change resulted in a revised sequence of construction for the bridge, but more importantly, introduced the need for a multi-staged sequence of construction significantly different than the originally proposed single-stage sequence of construction for which the bridge was designed.

The Challenge

The challenge resulting from the change in the sequence of construction was multi-faceted. Of largest consequence was the effect on the bridge – originally analyzed, designed and detailed for single-stage construction, the bridge now had to accommodate a multi-stage sequence. This had cascading effects on both the design and fit-up of the structural steel and deck, as well as the loading on the bearings and substructure.

Typically, staged construction is not a significant issue. However, when 1663 tons of steel – including 1446 tons of fabricated I-girders and 217 tons of cross frames – had already been designed, detailed, and nearly substantially erected for single-stage construction, and **THEN** staged construction is introduced, **there becomes a significant challenge**.

Effect on Superstructure Design

The bridge alignment includes curved, tangent, kinked, and splayed girders. In accordance with AASHTO LRFD and industry practice, a 2-D grid/grillage analysis was implemented in the original analysis and design. The framing arrangement resulted in both the girders and cross frames being primary load-carrying members jointly resisting the dead load, superimposed dead load and live loads applied to the bridge.

With a proposed change in the sequence of construction, including when the various loading components were to be applied to the bridge, the resulting forces in the girders and cross frames were subject to change. The resultant forces in the individual primary load carrying members could be lower or higher than they were initially designed for, the latter of which would become a significant concern in the overall strength and serviceability assessment.

Effect on Superstructure Fit-Up

The predicted deflections from the original 2-D grid were used to develop the girder camber profiles, as well as the cross frame drops for steel fabrication. The proposed change in the sequence of construction not only affected the girder forces, but also the predicted deflections. The differences from the as-detailed model to the new staged model would need to be considered for completion of the remaining steel fit-up. Since the girder web profiles were already set, there was a risk in not only achieving steel fit-up, but also deck fit-up between the stages – with one of the larger risks being the achievement of the structural thickness of the deck. The revised sequence substantially altered the relative position of the girders; some girders would be positioned lower than originally anticipated while other girders would be positioned higher – each having their own consequence. If the position is too low, the effects of the additional load from the concrete haunches may further affect the deflections; if the position is too high, the girders could infringe into the structural thickness of the deck.

Effect on Bearings and Substructure

The re-sequencing of the bridge would result in a re-distribution in the forces throughout the steel framing, as well as the reactions. The change in the predicted reactions, and the girder rotations had the potential to affect the high-load, multi-rotational (HLMR) bearings already designed, detailed, fabricated, and erected.

Similarly, the redistribution of loads had a potential effect of changing support reactions at the pile bent piers and abutments. Although the abutments had the highest potential to accommodate the revised loading sequence, the pile bent piers provided a large risk, since 66-inch diameter, prestressed cylinder piles were already installed to the required end-bearing capacity.

With a thorough understanding of the multi-faceted challenges resulting from the proposed revised construction sequence, the solution required ingenuity in attempting to retain the structural steel already substantially erected, as well as preserving the bearings and substructures.

The Solution

The solution required beginning with the end in mind. The overall objective was to re-sequence the bridge superstructure construction with little to no change in the structural steel, bearings, or substructure, and to do it expeditiously – as this was a design-build project, and construction was moving full-steam ahead.

Integral to the solution was the understanding that a change in the staged sequence of construction would affect the predicted forces and deflections in the structural steel framing system. With this in mind, the design team vetted numerous options and models to determine which combination would require the minimal modifications to the bridge components, since any change at the time, could become a large cost on a project already procured at a fixed-price.

The ingenuity that ensued and the attention to the details set the stage for a successful resequencing of the construction of a major river bridge.

An Additional Twist

The bridge framing plan included nine field sections and eight bolted field splices (FS) located near the dead load inflection points. The field splices were numbered sequentially from FS1 to FS8. Girder lines (G) were numbered from G1 to G7 (Figure 5).

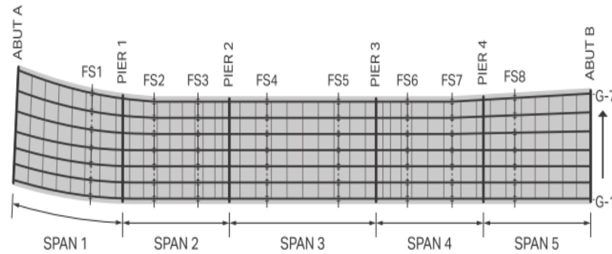

Figure 5. Framing Plan.

Starting in Span 1, the girders were curved, then kinked at FS2, then tangent, and then kinked and splayed at FS7. The girder spacing varied from 10 feet to nearly 13 feet to accommodate the flared geometry at each end of the bridge. The girder web depth linearly varied from 76 inches to 108 inches within the negative moment regions, transitioning from the bolted field splices to the piers. The cross frames were K-frames with a top chord. The members were shop welded to the gusset plates and field bolted to the girder connection plates. The cross frames varied in spacing from 19 feet to 25 feet; were contiguous between bays; and were oriented perpendicular to the girders. The substructure units were oriented at 90 degrees to the construction baseline.

At the time the change in sequence of work was proposed, all but two girder field sections were erected; the field sections along girder lines G1 and G2 from FS1 to Abutment A were not installed, leaving G1 and G2 to cantilever into Span 1 (Figure 6).

Figure 6: Cantilevered Ends of G1 and G2.

The twist – the contractor wanted to place the deck full-width of the bridge from Abutment B to Pier 1, and partial width measured from the outside edge adjacent to G7 to G3 from Pier 1 to Abutment A, and then place traffic on the bridge. This would allow traffic to be removed from the existing bridge, which would then allow the existing bridge to be removed, allow the remaining field sections of the new bridge to be placed, and remaining deck to be completed. It sounded simple, but the challenge was in the details.

As G1 and G2 were cantilevered from Pier 1 and were connected by cross frames to G3 from Pier 1 to Abutment B, the loading due to the proposed deck placement was predicted to cause substantial forces in the girders and cross frames adjacent to the discontinuity. In essence, the adjacent framing would deflect down under the deck load and pull the unloaded cantilevers of G1 and G2 down with it while the continuity of G1 and G2 in the ahead-spans fought against the movement.

Numerous scenarios were explored to maintain the connectivity of all girders and cross frames already erected while placing the deck, however none were successful at mitigating the forces – particularly in the cross frames – without substantial changes to the as-fabricated and erected steel.

Setting the Stage for Successful Resequencing

After exhaustive efforts to explore maintaining the already connected girders and cross frames while the deck was placed, the decision was made to introduce a full-length, longitudinal, deck closure-pour between Girder lines G2 and G3. This would allow the girder system G1 – G2 to be disconnected from the girder system G3 – G7 and act independently while the deck was placed on G3 – G7, and traffic was shifted onto the new bridge.

This decision culminated in a 3-stage sequence of construction (Figure 7).

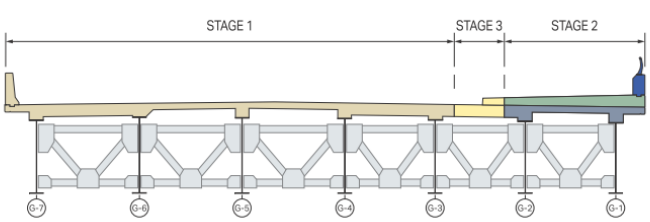

Figure 7. Proposed Sequence of Construction.

Revised Sequence of Construction

Stage 1

The revised sequence of construction allowed both traffic and pedestrians to be relocated from the existing bridge to the new bridge in Stage 1, accommodating the contractor's revised maintenance of traffic scheme for the project (Figure 8).

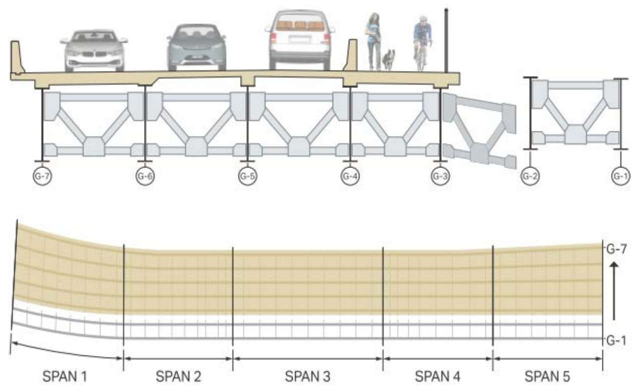

Figure 8. Stage 1 Construction.

Since the revised sequence of construction now required the previously installed cross frames between G2 and G3 to be disconnected, the bolts were removed from G2 and the cross frames were swung out of the way by loosening the bolts on G3. This avoided any conflict when Stage 1 deflected under the placement of the deck, and also eliminated the need to reinstall the cross frames when overhead access would no longer be available once the subsequent stages were completed.

Following the shift of traffic to the new bridge, the demolition of the existing bridge was completed (Figure 9). Subsequently, the remaining field sections for G1 and G2 were placed completing the respective girder lines.

Figure 9. Removal of Existing Bridge.

Stage 2

Stage 2 was critical to the success of resequencing the bridge. Since the girders and cross frames were detailed for the as-built cambers and drops, additional measures were warranted to achieve fit-up of the cross frames in the closure bay between G2 and G3, as well as the deck.

From an overall perspective, Stage 2 included the placement of the deck, portions of the sidewalk, and portions of the pedestrian barrier over G1 and G2 (Figure 10). However, this alone would not achieve the relative fit-up of the cross frames and deck. Therefore, strategic loading was implemented to achieve the necessary geometry.

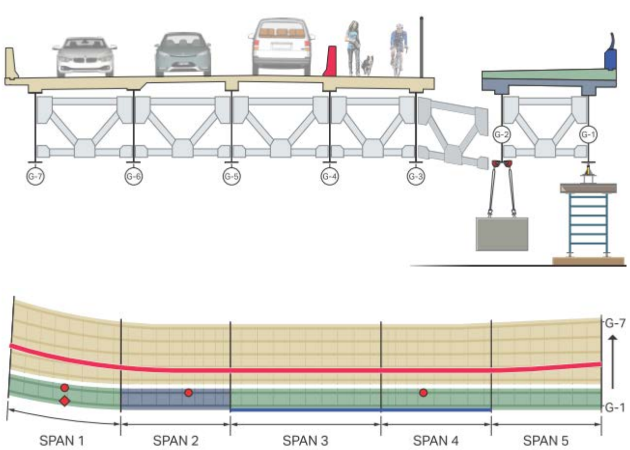

Figure 10. Stage 2 Construction.

Strategic Loading in Stage 2

The strategic loading included the sequencing of the permanent dead load on the bridge supplemented by temporary loads. The permanent loads were sequenced longitudinally in specified spans to achieve the desired deflection of the steel framing. The temporary loads were placed longitudinally and transversely within the steel grid framing to further tune the deflections of Stage 2 to achieve final fit-up of the cross frames and deck in the closure bay (G2-G3).

As part of Stage 2, the cross frames between G1 and G2 were fully connected such that the girders behaved as a 2-girder system. The system was evaluated for both strength and global performance to address stability concerns associated with two-girder systems. The cross frames in the closure bay remained disconnected and were temporarily supported by only G3.

Prior to the deck placement, temporary loads acting both upward and downward were applied to G1 and G2. The principle of continuity was leveraged in the strategic placement of the loads to assist in controlling the deflected shape of the 2-girder system. This was done by placing loads within specific spans and on specific girders to obtain the deflected shape needed in both the primary loaded and remote spans of the bridge. An illustration in Figure 11 shows the changes in deflection throughout the length of the structure as the external loading is increased in Span 1. The thin yellow line denotes deflection before load application and the thick green line green line denotes the deflection after load application.

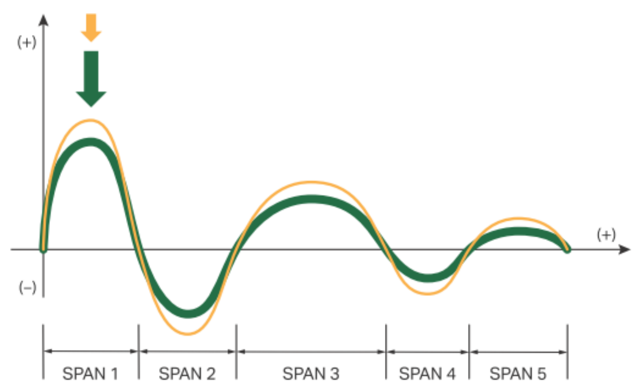

Figure 11. Leveraging the Principles of Continuity.

To achieve the required deflected shape of the 2-girder system, temporary loads were applied as shown in Figure 12 where L is the length of the particular span within which the load was placed.

Girder	Span	Location	Weight	Direction
G1	1	0.50L	15 Tons	Up
G2	1	0.50L	10 Tons	Down
G2	2	0.60L	5 Tons	Down
G2	4	0.40L	5 Tons	Down

Figure 12. Stage 2 Temporary Loads

In order to provide the required downward loads, the contractor constructed concrete block weights and supported them from the bottom flange of the girders using beam clamps (Figures 13 & 14).

Figure 13. 5-Ton Concrete Block Weight.

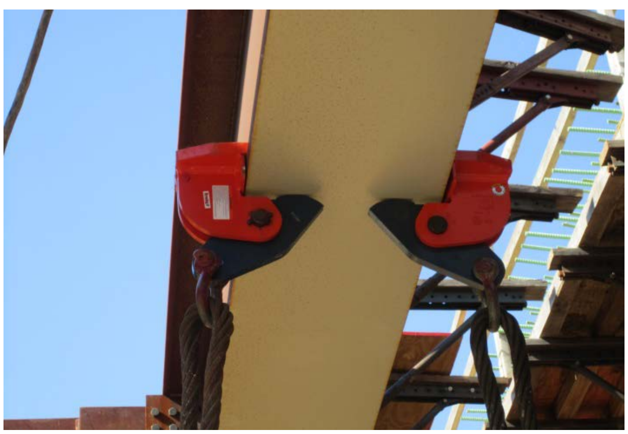

Figure 14. Beam Clamp Supporting Block Weight.

The upward and downward loads on G1 and G2 at the mid-span of Span 1 were required to apply a counteracting force-couple to the 2-girder section to overcome the twist due to curvature in Span 1 and the absence of the connecting cross frames in the closure bay. Without the external couple, the 2-girder system would twist towards the outside of the curve. To accomplish this, the contractor erected a jacking frame and installed a jack to push up on G1 with a steady 15-ton load while simultaneously pulling down on G2 with a 10-ton concrete block load (Figure 15).

Figure 15. Jack and Block Weight Counteracting Two-Girder System from Rotating (arrows showing outside of curve, direction of girder rotation, and counteracting couple from temporary loads).

Once the temporary loads were installed, the deck was placed in Stage 2. This deflected the 2-girder system down into closer relative position for subsequent fit-up. However, additional measures were still required and were facilitated by the strategic placement of the sidewalk and pedestrian barrier. To further tune the deflected shape of the girder system, the sidewalk and pedestrian barrier were only placed in specific spans to again leverage the concept of continuity. The strategic placement of the loading is shown in Figure 10 - with green representing the sidewalk, blue representing the pedestrian barrier, and red representing temporary loads.

After the sidewalk and barriers were placed, the contractor was able to proceed to Stage 3.

Stage 3

The primary objective for Stage 3 was to achieve final alignment for connection of the cross frames and deck in the closure bay between G2 and G3. For this to be achieved, additional temporary loads were required on both the 5-grider system constructed in Stage 1 and the 2-girder system constructed in Stage 2. To accommodate the loading on the 5-girder system, traffic on the bridge was required to be temporarily reduced from three lanes to two lanes (Figure 16).

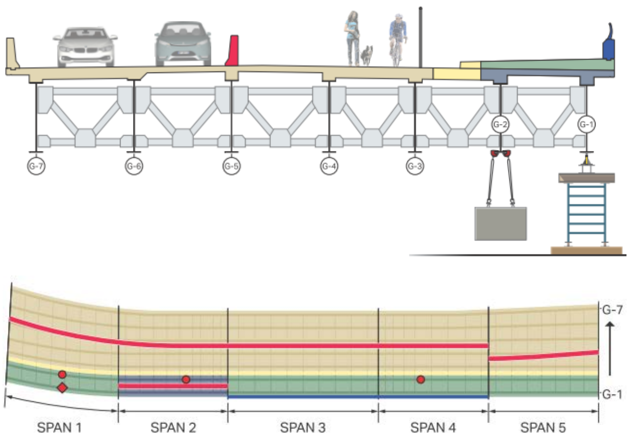

Figure 16. Stage 3 Construction.

Strategic Loading in Stage 3

For Stage 3, additional loading was required in strategic locations to bring the two stages in final relative alignment for connection of the cross frames and deck closure pour. The additional loading included the use of single and double face concrete barriers placed in specified locations both longitudinally and laterally on the girder systems.

The single face temporary traffic barrier was used on the 5-girder system and placed using similar reasoning employed in Stage 2 to leverage the effects of continuity, however, because the girders and cross frames were acting as a system, the influence surface of the grid system was utilized to tune the deflections in the 5-girder system. The temporary traffic barrier was located adjacent to G3 in Spans 1, 2, 3, and 4, but remained in place adjacent to G4 in Span 5. Additionally, a double-face traffic barrier equivalent to approximately 600 lbs/ft was placed on the 2-girder system in Span 2 only (Figure 16).

When the temporary loading was in place, the 2-girder and 5-girder systems were in relative position to connect the cross frames in the closure bay between G2 and G3 (Figure 17).

Figure 17. Typical Cross Frame in Closure Bay.

Following the installation of the cross frames, the deck closure pour was placed in a single continuous operation from abutment-to-abutment achieving both the finished slope and structural thickness of the deck (Figure 18).

Figure 18. Longitudinal Deck Closure Pour (note construction joints).

All temporary loadings were removed from the bridge and the remaining sections of sidewalk and pedestrian barrier were placed (Figure 19).

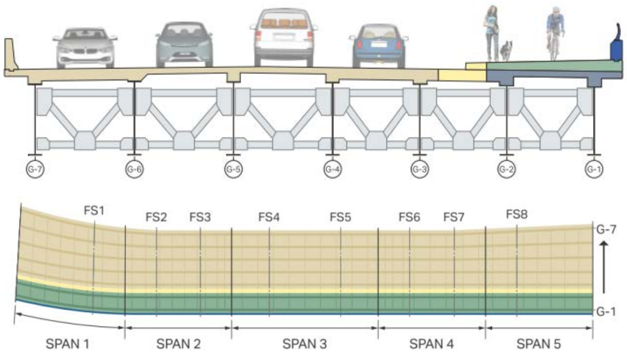

Figure 19. Completion of Sidewalk and Pedestrian Barrier.

Analytical Model & Results

For successful resequencing and fit-up of the structural steel and deck, an analytical model commensurate with the complexity of the sequence of construction was critical – not only for the predicted forces for strength and serviceability requirements, but also the predicted deflections and behavior of the system at each stage of construction.

Although the sequencing and strategic loading was developed to promote overall fit-up and minimize changes in the structural steel, some changes were still required to address the redistribution of forces and deflections in the structural system.

Modeling and Analysis

For both the original and re-sequenced construction, a 2D grid/grillage model was used in the analysis. The 2D grid model was used to analyze the re-sequenced bridge as it afforded efficiencies in a fast-track schedule while also affording what was considered to be a reasonable approach based on the complexity of the bridge and the sequence of construction. The modeling included consideration of the sequence of loading, the magnitude of loading, and the stiffness and bracing conditions of the girder system during each stage.

The analysis recognized both the timing and application of the temporary loads including the resulting force effects after all of the temporary loads were removed. Additionally, the analysis recognized that the cross frames in the closure bay would be installed as zero-load members and would only pick-up load as additional loading was applied to the bridge.

Due to the complexity of the staging, nine models were used with the results, which were in the elastic range, superimposed to obtain the forces and deflections in the system at critical points in the sequencing of the bridge.

Girders

Stresses

The cumulative girder stresses were evaluated at each stage of construction, as well as for the strength and service limit states in the final condition with live load effects. As required by AASHTO provisions, the factored resistance was equal to the yield strength of steel or less than the yield strength where other structural responses controlled. All of the checks were satisfied for the interim and final conditions.

The change in sequence of construction caused the loads to be redistributed in a manner different from that of the original sequence. This resulted in increased loads and stresses at several locations. All but three locations on two girders had adequate factored resistance to satisfy the new loading demand; at these locations the bottom flange factored compressive stress for the strength limit state was greater than the buckling resistance but was lower than the yield strength. To mitigate this issue, lean-on bracing was implemented to brace the compression (bottom) flange of the affected girder to those of the adjacent girders (Figure 20) which would increase the buckling resistance of the affected girder.

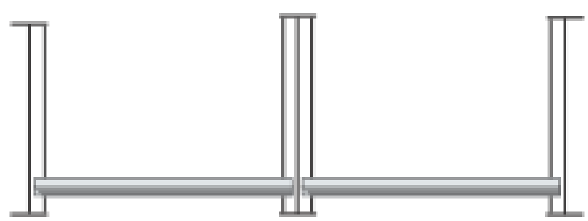

Figure 20. Lean-On Bracing.

The design of the bracing was based on the theory that lateral buckling of a girder at a braced location cannot occur unless at that location, all of the braced girders buckle – Yura et. al. 1992 (2); Galambos 1998 (3). The affected girder gains buckling strength by leaning on to the adjacent girders. As outlined in the AISC Specifications (4), the bracing members were designed to satisfy the stiffness requirements in addition to a compressive force equal to 2% of the factored compressive force in the girder sections. The addition of the lean-on bracing brought all of the girders into code compliance.

Deflections

When comparing the estimated final girder deflections from the original single-stage sequence to those in the 3-stage sequence, the deflected shapes differed up to 2 inches – some girders deflected more (+), others less (-) (Figure 21). As the girders were already fabricated to the original deflections, the differences in predicted deflections were accommodated in the concrete deck haunches to achieve the required finished deck elevations without impacting the structural deck thickness.

Span	1	2	3	4	5
Deflection	+7/8	+2	+1 1/4	-5/8	-1

Figure 21. Comparison of Maximum Girder Deflections Between Single and 3-Stage Construction.

Cross Frames

The cross frames were a K-type configuration with a top chord. The top chord was used to provide a pseudo-box shape between the girders to promote improved torsional resistance for the curved and kinked girders prior to the placement of the deck.

The cross frame members were single angles. The angles were welded to the gusset plates, and the gusset plates were bolted to the girder connection plates (Figure 22).

Figure 22. Typical Cross Frame Connection (shown prior to bolt installation).

Cross Frame Fit-Up

Prior to the analyses of the resequenced construction, the maximum differential deflection for fit-up of the cross frame bolted connections was established as 3/8 inch for the 1-1/8 inch holes in the gusset and connection plates. This limit was based on the contractor's experience in the erection of steel girder bridges and the ability to use full-size drift pins to bring the connections into alignment for bolt installation.

The analyses indicated that out of 46 cross frame locations in the closure bay that the predicted differential deflections (Δ) between G2 and G3, after completion of Stages 1 and 2, were as follows –

34 locations $\Delta \leq 3/8$ inch;

4 locations 3/8 inch $< \Delta \leq$ 1/2 inch; and,

8 locations 1/2 inch $< \Delta \leq$ 7/8 inch.

The strategic loading brought all but 12 locations along a nearly 1,000-foot long bridge into relative fit-up for the bolted connections to be installed.

The strategy for the 12 locations where fit-up could not be achieved included:

1. Air-arc removal of the welded angles; bolting the original gusset plate to the connection plate, and re-welding the angles; or,
2. Air-arc removal of the welded angles, replacing the existing gusset plate with a blank gusset plate, re-welding the angles and match-drilling the blank gusset plate to the connection plate.

To accommodate either option, the bolted connections were analyzed to determine that the resulting loads on the eccentrically loaded bolt groups were satisfactory. Ultimately, only 3 out of the 12 locations required retrofit – which correlated to the areas where the largest difference in differential deflection was predicted. The contractor elected to install blank gusset plates at these locations.

Cross Frame Strength

The original design approach for the cross frame evaluation grouped the cross frames into four general types that were then evaluated for maximum force envelope effects. The cross frame types were based on their location within the framing plan which included abutments, piers, constant-depth girder webs, and variable-depth girder webs within the haunched areas.

The first-tier analysis of the four types of cross frames using the maximum force envelope effects from the staging sequence resulted in 13 cross frames being overstressed. Therefore, it was necessary to perform a more rigorous second-tier analysis of the cross frames to determine where staged construction loads on specific cross frame members or cross frame connections exceeded the original design capacity. An extensive evaluation of the cross frame loads was performed to minimize the number of locations in non-compliance. As part of the analysis, the original design capacity of the select components and connections was revisited and refined in order to further eliminate as many non-conforming locations as possible. The evaluation included checks of the cross frame members, gusset plates, bolted connections, and welded connections.

At the conclusion of the second-tier analysis, there were several locations where non-conformance in the members and connections remained. Therefore, a third-tier analysis was conducted using the following strategies:

1. Use actual steel properties obtained from the material certifications versus code specified minimums,
2. Use A490 high strength bolts to replace the original A325 high strength bolts,
3. Strengthen the welded connections with additional field welds; and,
4. Obtain actual weld lengths versus the minimum lengths shown on the design plans.

Using the actual steel properties obtained from material certifications resulted in all cross frame members and gusset plates to be in compliance with code provisions. The evaluation of the bolted and welded connections follows.

Bolted Connections

At all of the cross frame locations, the original design for the cross frame bolted connections required a 1-inch diameter A325 bolt with threads excluded from the shear plane and a Class B contact surface. In order to mitigate the non-conformance of the connections due to the increased forces from the revised sequence of construction, the bolt type and diameter were changed in the individual connections at six cross frame locations. At these locations, 1-inch diameter A490 bolts or 1-1/8-inch diameter A490 bolts were used.

Welded Connections

Considering the actual geometry and spatial relationship between the various cross frame members and gusset plates, it was deemed likely that longer welds were provided in the actual connections during fabrication than the minimum weld lengths required on the plans. Therefore, field measurements were obtained by the contractor at specified cross frame locations. In order to expedite the data collection, template diagrams were developed and provided to the contractor for use as a guide in obtaining the field measurements. At each location, photo documentation of each measured welded connection was also obtained. These tools were implemented to minimize the potential for miscommunication, since the cross frames were erected and access over the river was difficult.

Considering the actual as-fabricated length of the welds, the smallest weld size was determined for retrofit that would mitigate the effects of the staged construction forces. Eight cross frame locations (some locations being the same as the locations where the bolts were altered) required a larger weld size than the original design. These welds were increased in size using field welding.

Bearings & Substructure

Through the entire process of evaluating the staged sequence, the effects were assessed on the bearings and substructure. For the bearings, the high-load, multi-rotational (HLMR) bearings were assessed for both interim and final reactions and rotations. The as-designed and installed bearings were determined to be satisfactory.

Similarly, the redistributed loads on the abutments and piers were assessed for both interim and final conditions. The stub abutments on H-piles located behind MSE walls, as well as the pier caps and large-diameter, prestressed, cylinder piles were determined to be satisfactory.

Therefore, no changes were warranted to the bearings and substructure that were already constructed.

Conclusion

In a complex bridge where a higher order analysis is performed using a 2D or 3D model, the girders and cross frames are recognized as primary load carrying members. As such, the sequence of construction will affect the loads and predicted deflections of the structural framing system. In this case, a 2D model was used to originally analyze and design the primary members for the forces and deflections used in the design and detailing of the bridge. When the as-detailed sequence of construction was changed, it required a re-analysis to assess all member forces in girders, cross frames and associated connections. This included an assessment of the steel details such as the bolted field splices, bearing stiffeners, and bearings, as well as an assessment of the supporting piers and abutments due to change in the girder reactions.

It is important that the model the designer chooses to assess the bridge is commensurate with the complexity of bridge. This will assist with best predicting the anticipated response of the structure.

In the case of the 11th Street Bridge, the attention to detail and the level of analysis yielded a successful outcome – the as-fabricated and erected bridge was re-analyzed and the model results closely correlated to the performance in the field. This resulted in minimal changes to the bridge to accommodate the revised sequence of construction.

Additionally, when assessing the challenges introduced by modifications during construction, many tools in the designer's "toolbox" should be leveraged. The 11th Street Bridge Project is a complex example where numerous such tools within the designer's toolbox were utilized for the successful completion of the project, ensuring that the original design intent was achieved despite changes that occurred during construction.

References

(1) *AASHTO LRFD Bridge Design Specification*s, 4th Edition, with Interim Revisions through 2008, Association of State Highway Transportation Officials, Washington, DC, 2008.

(2) Yura, J.A., B. Phillips, S. Raju, and S. Webb. Bracing of Steel Beams in Bridges, *Research Report 1239-4F*, Center for Transportation Research, University of Texas, Austin, 1992.

(3) Galambos, T.V. *Guide to Stability Design Criteria for Metal Structures*, 5th Edition, John Wiley & Sons, Inc., New York, 1998.

(4) *Manual for Steel Construction*, 3rd Edition, American Institute for Steel Construction, 2001.