11TH STREET BRIDGE - ANALYSIS USING TWO-DIMENSIONAL GRID MODEL TO ACCOMMODATE THREE-STAGE SEQUENCE OF CONSTRUCTION

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ABSTRACT

The 11th Street Bridge is a 916-ft long, five-span continuous, seven-girder, steel bridge over the Anacostia River located in Washington, DC. The steel framing plan consists of curved, tangent, horizontally kinked and splayed I-girders with K-type cross frames. The bridge was analyzed using a two-dimensional (2-D) grid model for the originally proposed full-deck width construction. However, after the structural steel was fabricated and substantially erected, the sequence of construction was changed to three stages. The revised sequence included a five-girder system with deck, a two-girder system with deck, and a longitudinal deck closure pour. The re-sequencing of the bridge construction required a reanalysis to check strength and serviceability requirements for the redistribution of loads in the structural steel. Critical to the success was accommodating the revised deflections and relative fit-up of the girders and deck in the closure bay. A rigorous 2-D grid analysis was performed indicating the girders and cross frames would require modifications to accommodate strength provisions from the redistributed forces within the system. The analysis also indicated that predicted deflections were appreciably different than the as-detailed girder cambers and cross frame drops, and additional measures would be required to achieve relative fit-up of the cross frames and deck in the closure bay. Additional analyses were performed to develop a successful sequence of construction for the girders and cross frames, strategically applying both permanent and temporary dead loads. The bridge was constructed achieving fit-up of the deck and cross frames with limited changes in the structural steel.

Keywords: bridge, steel I-girders, curved I-girders, 2-D grid analysis, staged construction, predicted deflections, camber, cross frame, fit-up, load rating, design-build
INTRODUCTION

Bridge design is conventionally accomplished through the application of design codes and industry practice to produce plans for a structure allowing for typical fabrication and construction methods. Many steel bridge structures with unique characteristics are designed using refined analysis methods rather than conventional line girder analysis. There are some cases where staged construction is employed and the staging results in a different distribution of forces throughout the structure framing system in comparison to a single stage construction. It is important to recognize this redistribution of forces and predicted deflections as it will affect the design, fabrication of the girders and cross frames, and fit-up during construction. Steel structures with a combination of both complex framing and staged construction can require extensive analysis and special attention to detailing and fabrication. The rigorous analysis and evaluation of the 11th Street Bridge staged construction provided the basis for the development of a sequence of construction comprising straightforward solutions applied to a complex bridge, resulting in the successful construction of the structure.

PROJECT

The 11th Street Bridge is part of an overall design-build-to-budget project let by the District Department of Transportation (DDOT) in Washington, DC, USA. The project, awarded based on a USD $260M best-value, design-build procurement, also included extensive ramp reconfigurations on both sides of the river to reconnect the new parallel I-295 bridges to improved sections of the Anacostia and Southeast Freeways (Figure 1).

FIGURE 1 Project aerial view (substantially complete) - 11th Street Bridge on left.
BRIDGE

Overview
The 11th Street Bridge is a 916-ft long, five-span continuous, fabricated steel I-girder bridge (Figure 2). The span lengths from Span 1 to Span 5 are 169.75 ft, 170.00 ft, 234.00 ft, 171.25 ft, and 170.50 ft, respectively, with the longest span located over the navigable channel. The welded plate girder web thickness varies from \(\frac{5}{8}\) in. to \(\frac{3}{4}\) in., and web depth varies from 76 in. within the positive moment regions to 108 in. within the negative moment regions. The web thickness transitions occur at the bolted field splices, and the depth transitions with a linear varying haunch in the vicinity of the piers. The webs are partially stiffened with transverse intermediate plate stiffeners. The top and bottom flange widths vary from 18 in. to 24 in. and the thicknesses vary from 1 in. to 2-\(\frac{1}{2}\) in.

FIGURE 2 11th Street Bridge.

The cross frames are inverted K-frames with a top chord, and the individual members are single-angle sections with sizes varying from 6 x 6 x \(\frac{5}{8}\) to 8 x 8 x \(\frac{3}{4}\). The members are shop welded to the gusset plates and field bolted to the connection plates (Figure 3). The cross frames vary in spacing from 19 ft to 25 ft and are detailed in four groups to accommodate the abutment, intermediate (constant and variable depth), and pier locations. The cross frame locations are contiguous between bays and are perpendicular to the girders.

The bearings are high load multi-rotational (HLMR) bearings using a combination of non-guided expansion, guided expansion, and fixed bearing types. Due to the horizontal alignment of the bridge, the bearings are oriented based on a reference chord originating at the point of estimated zero movement.

The abutments are concrete, cast-in-place caps supported on steel H-piles behind
mechanically stabilized earth (MSE) walls. The piers are cap-and-column bents with a single row of 66 in. diameter cylindrical pre-stressed piles with a cast-in-place concrete cap. The abutments are set perpendicular to a local tangent on the roadway baseline and the piers are set perpendicular to the baseline.

FIGURE 3 Representative cross frame.

Framing Plan
The bridge framing plan is established to economically accommodate the horizontal roadway alignment which is comprised of a curve on the approach to Span 1 and a flared transition-lane in Span 5 for an off-ramp at the end of the bridge. Each girder line of the framing plan includes nine field sections and eight bolted field splices (FS) located near the dead load inflection points. The field splices are numbered sequentially from FS1 to FS8. There are seven girders (G) which are numbered from right to left looking spans ahead (Figure 4).

FIGURE 4 Bridge framing plan.

Beginning in Span 1, G1 to G7 are curved with radii of approximately 1,585 ft which terminates at FS2 with a horizontal kink. The average deflection angle is approximately 1-
degree and 30-seconds from the local tangent. All of the girders are tangent from FS2 to FS7.
Beginning at FS7 in Span 5, G1 to G4 remain tangent and G5 to G7 are kinked with an average
deflection angle of 0-degrees and 52-seconds. The substructures are oriented at 90-degrees to the
construction baseline which is parallel G1 to G4.

Typical Section
The bridge has a seven-girder typical section with the girder spacing varying from 10 ft to nearly
13 ft to accommodate the flared geometry at each end of the bridge. The out-to-out width also
varies from approximately 68 ft to 75 ft. The bridge carries four 11 ft lanes of vehicular traffic
and a 17 ft wide multi-use bicycle and pedestrian sidewalk (Figure 5).

FIGURE 5 Bridge typical section.

Materials
All structural steel for the superstructure is AASHTO M270, Grade 50W, with supplemental
requirements for Charpy V-notch toughness for all plates and members with tensile stresses. All
high strength bolts are AASHTO M164 or M253, Type 3. All shear studs are AASHTO M169.
The bridge deck and sidewalk are cast-in-place, sand-lightweight, reinforced concrete with a 28-
day minimum compressive strength of 4,500 psi. The sand-lightweight concrete has a maximum
unit weight of 122 pcf plastic, and 117 pcf 28-DAD (day air dry). The bridge barriers are normal
weight, cast-in-place, reinforced concrete with a 28-day minimum compressive strength of 4,500 psi. All reinforcement in the superstructure is epoxy coated conforming to AASHTO M284,
Grade 60.

DESIGN CRITERIA
The bridge is designed in accordance with the AASHTO LRFD Bridge Design Specifications,
4th Edition with 2008 interim revisions (1). In accordance with AASHTO LRFD, the design
vehicle is the HL-93 notional load and the pedestrian loading is 75 psf. Although the actual lane
width is less than the AASHTO LRFD 12-foot design lane, four 11 ft. lanes are used in the
analysis in accordance with AASHTO LRFD Section 3.6.1.1.1 (1). In further accord with
AASHTO LRFD, the pedestrian load is considered as one loaded lane when determining the
multi-presence factor - Section 3.6.1.1.2 (1). The optional live load deflection criterion in
AASHTO LRFD Section 3.6.1.3.2 (1) is employed limiting deflections to L/1000 due to
pedestrians. The bridge barriers are neglected in contributing to stiffness for deflection and
fatigue checks. Since the bridge is in a low seismic zone, an equivalent static or dynamic
analysis is not employed.


**BRIDGE MODEL**

Due to the curved and kinked geometry of the framing system and the partially curved alignment, a 2-D grid (grillage) model is used to predict the performance of the bridge. The 2-D model is based on elastic small deflection theory.

The curved and kinked (chorded) girders will experience twist (torsion) since the load application will be applied eccentric to the chord between the girders’ supports. This will cause torsional and warping stresses, flange lateral bending stresses, and twisting deformations. The curved roadway alignment will also cause global load shifting resulting in additive loads to the girders on the outside of the curve and relieving loads to girders on the inside of the curve (closer to the center of curvature).

Since the I-girders are an open section with low St. Venant torsional stiffness, the girders primarily carry the torsion by means of warping, with the cross frames and deck providing resistance to such behavior. The normal stresses on the girder include axial stress, major axis bending stress (strong and weak axis), lateral bending stress, and warping normal stress. Similarly, the normal shear stresses on the girder will include vertical and horizontal shear stress, a small amount of St. Venant shear stress, and warping shear stress. To address all of these effects, a generally accepted, commercially available, bridge software program was used to analyze the bridge.

The 2-D grid analysis uses beam elements to model both the girders and cross frames. In curved and splayed bridges, the cross frames are analyzed, designed and detailed as primary structural members. For the cross frames, the equivalent flexural stiffness of the cross frame is used for the beam element to connect the adjacent beam elements representing the girders. The equivalent primary flexural stiffness of the cross frames is discussed in more detail in the AASHTO and National Steel Bridge Alliance (NSBA) Joint Collaboration document G13.1 Guidelines for Steel Girder Bridge Analysis.

In the analysis, the grid model is used to predict the behavior of the system at four different time-stages: 1) girder self-weight, 2) wet concrete on the girder, 3) superimposed dead loads on the composite girder and deck section (e.g. sidewalk, traffic barriers, etc.), and 4) live loads on the composite girder and deck section (i.e. vehicular load and sidewalk load).

In curved, continuous, composite bridges, AASHTO LRFD Section 6.10.10.1 requires that shear connectors be used along the entire length of the girder in both the positive and negative moment regions. By interlocking the deck with the girders along the entire length of the girder, the composite section is able to provide a uniform resistance to torsion, since torsional shear exists in the composite section along the entire length of the bridge - AASHTO LRFD Section C6.10.10.1. Since the relative slip between the deck and girders is prevented with the use of the shear connectors, the model uses the deck in the composite section properties for the stiffness analysis where the relative stiffness of the equivalent beam elements contribute to the distribution of the applied loads through the plane grid. However, for calculation of stresses and strength resistance, the model uses the appropriate girder properties and transformed composite girder and deck section properties for positive moments, and transformed composite girder and deck reinforcement properties for negative moment. The effective flange width as defined in AASHTO LRFD Section 4.6.2.6 is used to compute the composite section properties.

For live load analysis, the influence lines for each girder in the system is developed and an influence surface is generated for determining the effects of the HL-93 truck and lane loading, as well as the pedestrian loading. The influence line ordinates for the girders and influence surfaces consider both the presence and absence of global load shifting due to curvature. The
governing ordinates are then used for creating the live load force effects and envelopes for the
girders and cross frames.

The bearings are modelled as being pinned at each support; that is, the bearings are free
to rotate about the transverse axis and not allowed to translate. Because the bridge is modelled
using a traditional 2-D grid model, this approach is considered to be acceptable. The girders are
supported on five-column bents and abutments, validating the assumption of infinite stiffness in
the vertical direction.

SEQUENCE OF CONSTRUCTION

Original Sequence of Construction
In this paper, staged construction is defined in accordance with AASHTO LRFD Section C6.7.2
\((1)\) as the situation in which the superstructure is built in separate units with a longitudinal joint.
This is to be distinguished and held separate from the longitudinal deck placement sequence.

The originally proposed sequence of construction for the 11th Street Bridge required that
all seven girders were erected and the deck placed the full-width of the proposed bridge typical
section. After the deck was to be placed using a longitudinal deck placement sequence, the
bridge sidewalk and barriers were to be placed. In this sequence, there is no staged construction.
However, during construction, the contractor decided to re-sequence the maintenance of traffic
scheme for the project allowing a portion of the new 11th Street Bridge to be opened to traffic
early.

The change in the sequence of construction resulted in the 11th Street Bridge being
constructed in three stages. The challenge that presented itself was the girders and cross frames
were already fabricated and partially erected based on the original sequence of construction.
Therefore, the bridge had to be reanalyzed not only to check the girders and cross frames for
strength, but just as importantly, check the girders, cross frames, and deck for relative position
and fit-up between the stages of construction. As the AASHTO/NSBA Joint Collaboration
document G12.1 Guidelines for Design for [Constructibility] \((3)\) indicates “If phased
construction is required, the differential deflections between units due to the application of dead
loads at different times can be significant.” The reference to phased construction in the
preceding quote shall be construed as staged construction.

The additional challenge associated with re-sequencing the bridge construction was that
field sections in Span 1 for two girder-lines (G1 and G2) would not be completed until after the
existing 11th Street Bridge was removed, due to the conflict between the existing and proposed
bridges (Figure 6). As the AASHTO/NSBA Joint Collaboration document G13.1 \((2)\) indicates
“...On continuous bridges, girder deflections are influenced by adjacent spans. Just as the
presence of girders in one span reduces the deflections in the adjacent spans, when the girders in
an adjacent span are not present, deflections are greater.”
FIGURE 6 Conflict between existing bridge (left) and proposed bridge (right); notice the discontinuous girder-lines (G1 and G2) on the proposed bridge.

Revised Sequence of Construction

Staged Construction

To accommodate the re-sequencing of the new bridge construction, several options were assessed to conclude that the most viable strategy to minimize structural steel changes was to implement a three-stage sequence of construction (Figure 7). Since most of the girders and cross frames were already erected, the decision was made to disconnect the cross frames between G2 and G3 and introduce a closure pour in the bay between the two girders, creating a five-girder (Stage 1) and two-girder system (Stage 2), separated by a full-length longitudinal deck closure pour (Stage 3). This was necessitated in order to address the high stresses in the cross frames predicted based on the analysis of the system without a closure bay. The stresses were the result of the five-girder system having the deck placed and traffic on it while the two-girder system remained unloaded until the existing bridge could be removed, and the final field sections for girder lines (G1 and G2) from the last field splice (FS1) to the new abutment could be placed.

Stage 1 was completely constructed and open to traffic. In order to aid the fit-up of the cross frames and the deck in Stage 3 construction, permanent and temporary loads were strategically utilized in Stage 1 and Stage 2. Without external loading, the girders adjacent to the closure pour would not be in relative position for the cross frame and deck fit-up.

- The temporary barrier that was needed for the maintenance of traffic in Stage 1 was placed over G4 in Span 5 and over G4 in the other spans.
- Disconnecting the cross frames in the closure bay caused G1 and G2 in Stage 2 to twist more in Span 1 due to the curvature. To correct the twist, an opposing couple of 30 kips
upward force in G1 and 20 kips downward force in G2 was applied using temporary loading. A temporary loading of 20 kips was also applied on Span 2 and Span 4 of G2. These temporary loads were applied to the non-composite girder section.

- The partial-width sidewalk was constructed in Spans 1, 3, 4 and 5.
- The sidewalk barrier was constructed in Spans 4 and 5.
- A temporary load was applied to the Span 2, Stage 2 deck in between G1 and G2.

Followed by this strategic loading, the cross frames between G2 and G3 were in relative position to be connected. At this point, the cross frame members theoretically are resisting no loads. The deck closure pour was placed and 48 hours later, all of the temporary loading in the Stage 2 region was removed. Then, the remaining portions of the sidewalk and sidewalk barrier were constructed.

**Modeling**

In order to evaluate the effect of the new construction sequence on the various bridge components, the analysis considered the sequence of loading, the magnitude of loading, the stiffness of the girders and the system during each stage, and the lateral bracing conditions during each stage. Due to the staged construction, the interior girders, G2 and G3, adjacent to the longitudinal closure bay had to be carefully modeled to address the reduced composite section properties since the effective flange widths were temporarily reduced (2). To assist with mitigating the ultimate fit-up of the cross frames in the closure bay, the limits of the closure pour were to established to provide nearly the same effective flange widths for G2 and G3. This resulted in a deck slab closure pour approximately 4 ft wide. Matching the effective widths assisted not only in the temporary composite section properties, but also in the tributary width of the deck used in computing the dead load applied on each of the adjacent girders.

The analysis of the staged construction sequence used the original 2-D grid model with the necessary adjustments. Due to the complexity of the sequencing of the loads and the ultimate introduction of temporary loadings in the Stage 2 construction, multiple models were required to predict the behavior of the system. Since behavior of the girders and cross frames remained in...
the linear elastic range, the individual model results for forces and deflections were added together (superposition) to predict the behavior during each stage of construction and in the final configuration (1).

Four individual bridge models were used to accomplish the analysis of the proposed sequence of construction. Each model considered the non-composite and composite stiffnesses, structural steel framing, and loading appropriate for the respective stage of work. Model 1 was used to analyze Stage 1 and Stage 2 of the proposed sequence of construction. However, in the model the Stage 1 and Stage 2 framing systems remained structurally isolated from each other. Model 2 was used to identify adjustments to the temporary loading to enable cross frame fit-up in Stage 3. Model 3 was used to analyze the effects of the deck closure pour. Model 4 was used to apply the remainder of the permanent superimposed loads and removal of all temporary loads previously applied for deck and steel fit-up.

In addition to these four models for the staged sequence of construction, three live load models for the as-built condition, one dead load model, and one live load model considering the effects of future widening of the bridge were analyzed. In all, a total of nine bridge models were analyzed and the results appropriately combined using superposition to address the numerous loading conditions.

**Stresses**

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

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 load 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. 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 (4) (5). The affected girder gains buckling strength by leaning on to the adjacent girders. As outlined in the AISC Specifications (6), 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 AASHTO LRFD compliance.

**Predicted Deflections**

**Girders** When comparing the estimated final girder deflections from the original construction sequence to those in the staged construction, Spans 1, 3 and 5 were deflecting ⅞ in., 2 in. and 1-¼ in. more, respectively. Spans 2 and 4 were deflecting ⅝ in. and 1 in. less, respectively. Since the girders were already cambered to the deflections for the original construction sequence, the differences in predicted deflections were accommodated in the concrete deck haunches to achieve the required bridge deck elevations.
Cross Frames Prior to the analyses of the re-sequenced construction, the maximum differential deflection for field fit-up of the cross frames was established as ⅜ in. This limit was based on the contractor’s experience. The staged analyses indicated that out of 46 cross frame locations in the closure bay, the predicted differential deflections between G2 and G3 were less than ⅜ in. at 34 locations, between ⅜ in. and ½ in. at four locations, and between ½ in. and ⅞ in. at eight locations. The plan was to retrofit the previously fabricated and erected cross frames where fit-up was not achievable.

Cross Frame Strength and Fit-up
The original design approach for the cross frame evaluation grouped the cross frames into four general categories that were then evaluated for maximum force envelope effects within each category. The proposed staged sequence of construction resulted in overstresses in 13 cross frames. Based on the results of the staged analysis, it was necessary to perform a more rigorous assessment 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. Also, the original design capacity of these select components and connections was revisited and refined in order to further eliminate as many non-compliant locations as possible. The evaluation included checks of the cross frame members, gusset plates, bolted connections, and welded connections.

At the conclusion of the initial assessment, the results were evaluated and several options were assessed to mitigate the remaining overstresses in the cross frames and connections. The options included: 1) the use of actual material properties obtained from the material certifications versus AASHTO LRFD specified minimums, b) the use of M253 high strength bolts versus the original M164 high strength bolts, c) strengthening the welded connections with additional field welds, and d) obtaining measurements of actual weld lengths versus the minimum lengths shown on the design plans.

Bolts At all of the cross frame locations, the original design for the cross frame bolted connections required a 1-inch diameter M164 bolt with threads excluded from the shear plane and a Class B faying surface. In order to mitigate the 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 M253 bolts or 1-⅛ in. diameter M253 bolts were used.

To introduce further tolerance in the field modification of the bolted connections, an investigation was performed for the cross frame connections to determine the maximum vertical distance the cross frame members could be placed from the as-detailed centroid of the bolt group. The cross frame connections considered in the evaluation occurred on the G2 side of the cross frames in the closure bay.

A summary chart was developed and provided to the contractor for the allowable tolerance of the girder displacements (i.e. member eccentricity from the bolt group centroid) for the cross frames in the closure bay. Based on the analyses, it was determined that generally the allowable tolerance for the bolted connections was 2 in. (up or down). This tolerance was deemed reasonable considering that it was highly likely that a 2-in. displacement would also cause a fit-up issue between the cross frame angles and gusset plates. The fit-up issue of the member to gusset plate was anticipated to present itself with the cross frame member being
located partially or fully off of the as-detailed gusset plate.

To address the risk of the cross frame members not fitting on the as-detailed gusset plate, provisions were established to provide a new gusset plate that would accommodate the existing bolt pattern and the cross frame members, when warranted. This would require air-arc removal of the weld connecting the existing angle to the existing gusset plate, then bolting a new plate in place, and field welding the existing cross frame members to the new plate.

Welds Considering the actual geometry and spatial relationship between the various cross frames members and gusset plates, it was deemed likely that there may have been longer welds provided in the 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 welded connection was also obtained. These tools were implemented to minimize the potential for miscommunication, since all of 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.

Construction

The girder and cross frame fit-up was achieved by strategically using permanent and temporary loads including concrete barriers, concrete-block counterweights, and a hydraulic jack system to bring the structural steel framing of the two stages into relative position for connection (Figure 8). The finished deck slab geometry was achieved by using a closure pour between the two stages of construction.
Drawings were prepared to communicate the new staged construction sequence to the contractor. These drawings included superstructure section views and a narrative which provided a detailed description of the individual steps comprising each stage. Items covered in the narrative included cross frame installation, deck placement, temporary barrier placement, temporary load application, sidewalk placement, permanent barrier placement, and deck closure pour placement. The narrative detailed the sequence of the selective application of the permanent loads and temporary loads, leveraging the effects of the structure continuity to achieve the desired result.

During the staged construction operations, the cross frame bolted connections were readily completed at all but three cross frame locations within the closure bay. These three cross frames were located in Span 2 within the zone where analysis predicted the largest differential deflection. These three locations were addressed with the replacement of the gusset plates.

The design configuration of the lean-on bracing utilized simple struts fabricated with WT sections. Bolted connections were used to attach the bracing to existing as-fabricated transverse web plate stiffeners at one end of the brace and to new field installed connection plates bolted to the girder webs at the other end of the brace. Bolted connections were used to minimize field welding, as well as to improve the fatigue performance of the connection.

An evaluation of the deck was performed to evaluate the interim conditions. Due to the contractor’s means and methods for constructing the closure pour overhangs, several deck reinforcement bar laps were analyzed in sufficient detail to minimize their length in order to accommodate the overhang forming brackets within the closure bay between G2 and G3. This effort included reevaluating the original deck design and performing a detailed analysis within this area to reduce the reinforcement lap lengths to a dimension workable for the contractor.
With the rigorous analyses, the proposed construction sequence was successful – achieving girder relative position for the fit-up of all but three cross frames that were retrofit in the field. The deck closure pour was placed and the final deck geometry was achieved to obtain both the structural depth of the deck and the cross slope geometry.

CONCLUSION

Bridges are becoming more complex in the industry. Bridge spans are becoming longer; skews are becoming sharper, and curvatures are becoming tighter. With the increase in the complexity of the bridges, the models must also increase in complexity to better predict the performance of the bridge - this leads to improved economy in design and improved correlation with actual field performance.

The model chosen for analysis must be carefully considered. The engineer must make an informed decision on what model to use that will reasonably predict the performance of the bridge. NCHRP Report 725 Guidelines for Analysis Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges (7) was developed to assist the designer in this decision. The choice of models can become particularly critical when staged construction exists, whether it is a new bridge built in stages or an existing bridge that is widened. For the best success in predicting the forces and deflections for fit-up, the staged sequence of construction should be recognized in the analysis.

For a bridge that is highly skewed or curved, a 2-D grid/grillage model or 3-D model is typically warranted to reasonably predict the performance of the bridge. For these types of bridges, the girders and cross frames are both load carrying members acting together to resist the applied forces as a system. The manner in which the forces are distributed in the system is dependent on many variables including the staged sequence of construction. Therefore, any changes in the sequence of construction during the design process or in construction need to be assessed.

Beyond just the design process, the industry may need to consider the effects of a staged sequence of construction when load rating bridges. In general practice, routine bridges are typically load rated as though the bridge was constructed in a single stage. However, it may be prudent for load rating analyses to evolve with the design process, since staged construction results in a different set of forces and deflections in the girders and cross frames in comparison to a bridge built in a single stage.

For the 11th Street Bridge, a load rating manual was developed for guidance in establishing the dead load forces in the girders and cross frames, so the baseline model used in the live load rating assessment would reflect the effects of the staged sequence of construction.
REFERENCES