CASE STUDY OF LONG SPAN STEEL BRIDGE STABILITY DURING DESIGN AND CONSTRUCTION

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BIOGRAPHY

Kevin Sear has been a structural engineer for over 42 years, the first 10 years designing vertical structures and the remainder designing highway structures. He graduated from Villanova University with a BS degree in 1975 and has been a registered PE in Pennsylvania since 1979.

He joined Urban Engineers in 1982, and then joined Greiner Engineering in 1992 which became URS in 1997 then became AECOM in 2013.

Kevin has been both a Project and a Department Manager and currently is one of several Technical Quality Leaders for Highway Structures in the Greater Pennsylvania Area.

His involvement in the case study structure included constructability, stability and management of the construction phase structural engineering services.

Sr. Bridge Erection Engineer, Susan Steele has been with High Steel Structures, LLC since 2008, and prior to that with High Concrete Group since 2001. Susan was a General Contractor for 11 years focusing on commercial and residential rehabilitation. She graduated from Penn State University with a BS degree in Structural Design and Construction Engineering Technology and a Masters Degree in Engineering Science.

Her involvement with the case study was erection analysis and engineer-in-charge of the steel construction.

SUMMARY

The SR0903 Bridge over the Lehigh River in Jim Thorpe, PA is a new, 961' long, four span, welded steel plate, multiple girder structure completed in 2016. It replaced a 614' long, five span, riveted steel, stringer-floorbeam-girder structure located about 1,000' downstream and built in 1949. The structure was deemed functionally obsolete and structurally deficient.

The new bridge location eliminated an awkward jog in SR0903 through a residential community. However the topography and an alignment skewed to the river significantly increased the bridge length. In addition, the presence of a historic, abandoned canal lock required a 335' long span to avoid impacts to the lock and the river.

Structures with span lengths over 200' long are susceptible to instability and overstress due to wind and gravity loads. This condition can occur during girder erection, after girders are fully erected, during deck concrete placement, or once the bridge is put into service.

Analysis is required to evaluate the need for temporary supports or holding cranes during erection, and to determine if permanent lateral bracing is required.

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Keywords

Stability, analysis, overstress, deflection, bracing, diaphragm, cross frames, shoring tower, holding crane

Abstract

Replacement of a bridge over the Lehigh River in Jim Thorpe, PA required long spans (335 ft maximum) and deep steel plate girders (10 ft webs) to avoid impacts to two active railroads, a rail yard, a historic canal lock and the Lehigh River. As the deck is about 100 ft above the river and valley below, stability of the large steel girders during erection, prior to concrete deck placement and during deck placement was a primary concern of the designers and constructors.

During the design phase, AECOM performed a structural wind load analysis on the completed steel framing prior to concrete deck placement. While the maximum unbraced lateral deflection of over 15 inches appeared excessive, it was well under the L/150 limit permitted by PennDOT Standard BD-620M. However, the combined self-weight plus wind service load flange tip stress was several times greater than the allowable lateral torsional stress at locations within each of the four spans. The structure was stiffened by adding lateral cross bracing in all spans between the bottom flanges of two interior girders to transfer the wind load back to the substructure units.

During the design phase, AECOM performed a structural gravity load analysis on the steel framing during concrete deck placement. Given the extensive 961 ft length of continuous deck, it was not practical to place the over 1,700 cubic yards of concrete in one operation. A staged placement was developed that kept non-composite stresses in the girder flanges within allowable limits. The sequence progressed from Abutment 1 to Abutment 2 placing concrete in the positive moment zones, one at a time, and then progressed from Abutment 2 back to Abutment 1 placing concrete in the negative moment zones, again one at a time.

During the construction phase, High Steel performed structural analyses on models on each of the erection stages to evaluate strength and stability of the girders. Given the numerous features to be avoided and the height of the girders above ground, the High Steel opted to erect the girders without temporary shoring towers. This required multiple cranes, sometimes as many as five working simultaneously, and complicated the sequential structural analysis performed to verify girder stability under self-weight through the erection sequence. Erection stability was verified for all construction stages; these including a 1.5x safety factor for all girders erected over railroad property.

Providing stability for steel plate girders during erection, prior to deck placement, and during deck placement can often be a controlling condition for long span steel girder design. Construction analysis that is specific to the structure type, the site conditions and construction sequence is critical, as the girders themselves possess only a portion of their final, composite strength and stiffness during the various stages of construction. This is especially important for deep and long span girders. This paper will benefit both bridge designers and constructors; highlighting key design and construction considerations associated with complex and long span steel girder erection.

Bridge Details

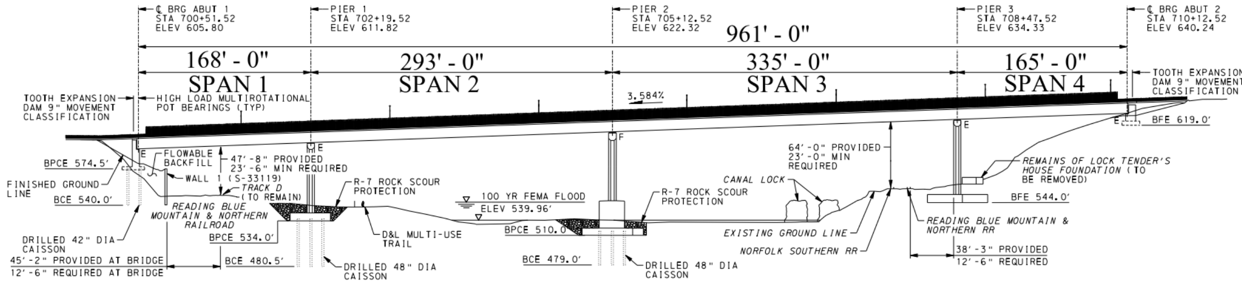

Figure 1 – Bridge elevation.

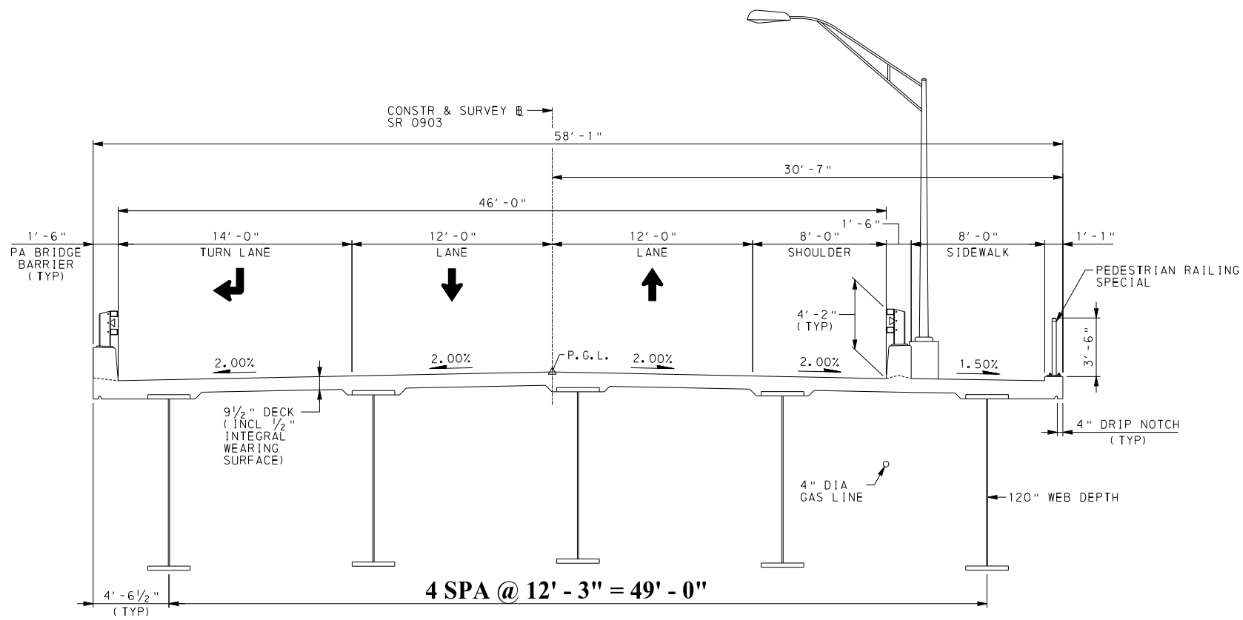

Figure 2 – Bridge typical section.

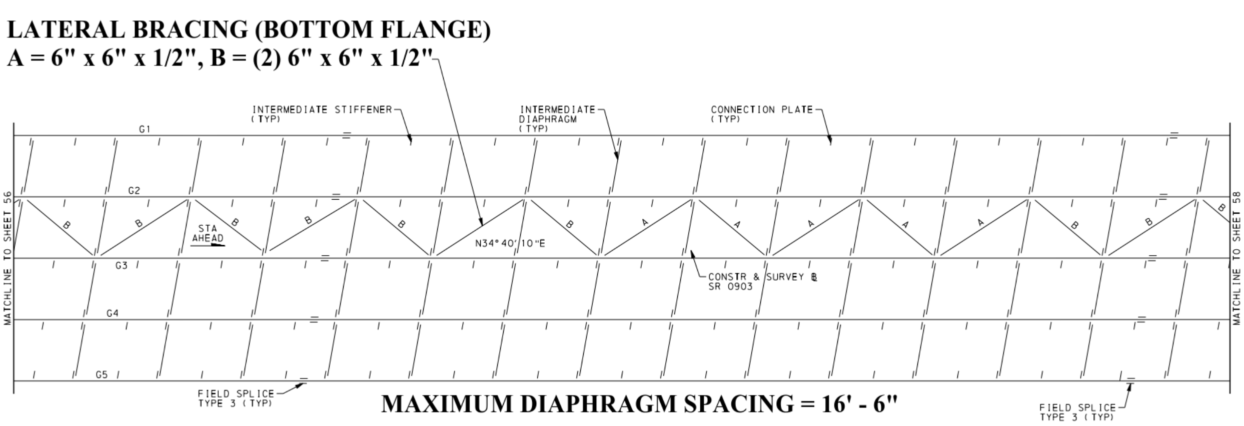

Figure 3 – Bridge framing plan.

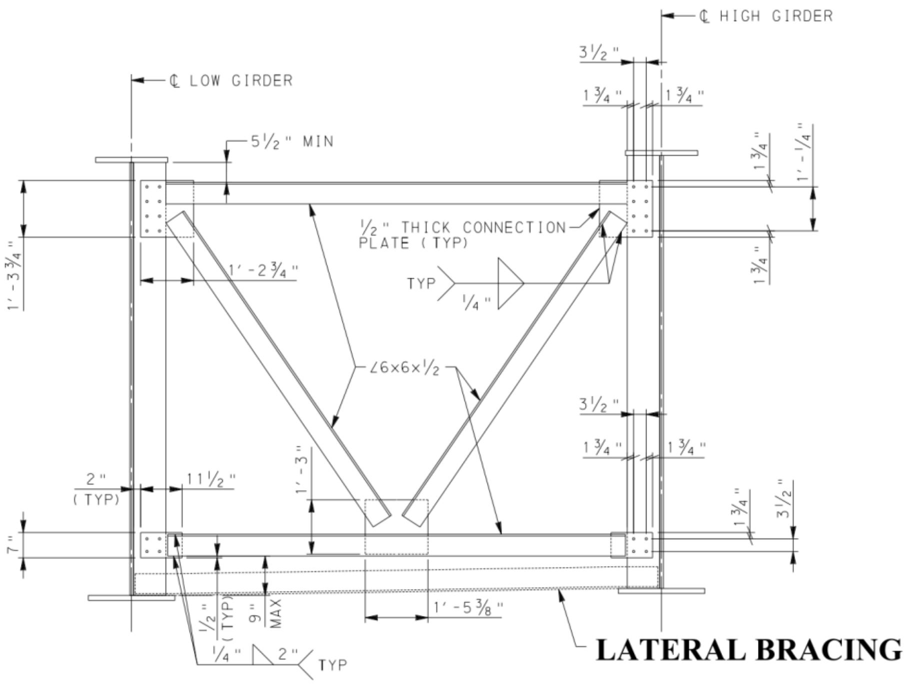

Figure 4 – Intermediate diaphragm detail.

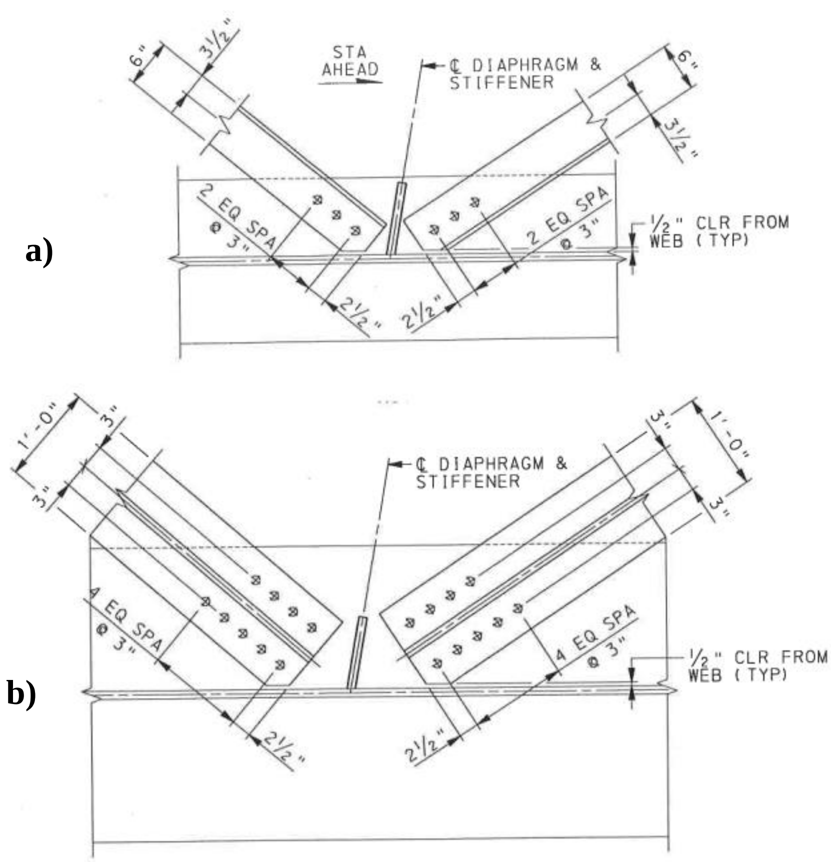

Figure 5 – Lateral bracing details.

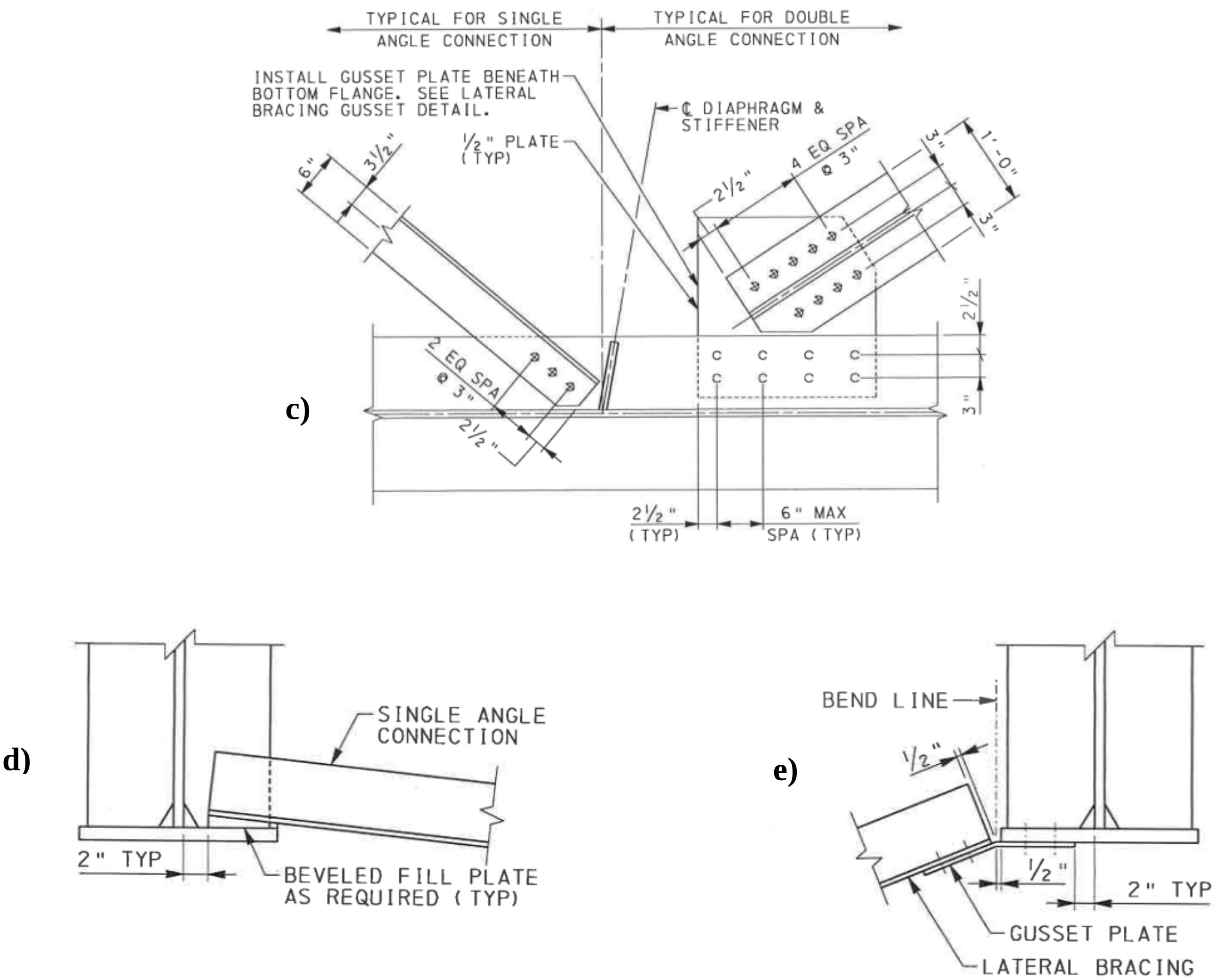

Figure 5 – Lateral bracing details (Cont'd).

Introduction

Stability is the absence of undesirable deflections or rotations. Strength is the absence of undesirable member stress.

The goal of economical structural designs is to determine a system where stability is provided using members that require the least amount of material above that needed to meet strength requirements.

Over the life of a steel girder bridge there are four distinct stages where stability and strength of the superstructure must be evaluated.

The first stage is during staged erection when only portions of the girders are in place, supported by substructures, temporary shoring or cranes.

The second stage is after the girders have been erected and diaphragms installed but before the deck has been formed.

The third stage is while the deck is being placed but before composite action with the girders has been achieved.

The fourth stage is after composite action is achieved and the bridge is open for traffic.

Each stage has its own loading and member conditions.

Typically the erector is responsible for the first stage and the designer is responsible for the remaining three stages.

This paper discusses how each stage of the case study bridge was evaluated for stability and what measures were needed to ensure the structure met client and industry standards.

To illustrate how stability becomes less complex as a structure progresses through the Stages, we will begin with the last stage and work backwards. Note that the girders in the case study bridge are straight so forces which result from curvature (v-load, centrifugal, etc) are not present.

Stage 4 – Completed Structure

Once the deck has hardened, the steel bridge girders are composite with the deck and stability of the top flange is provided. Web stability is provided by the intermediate stiffeners and diaphragm connection plates. Overall stability is provided by the entire system of deck, girders and diaphragms. This leaves stability of the bottom flange to be evaluated.

The client/owner of the bridge mandated maximum diaphragm spacing of 25 ft would have resulted in reasonably small stresses in the bottom flange due to wind loads. However, that diaphragm spacing was too large to sufficiently brace the bottom flange so that a large portion of the flange yield stress could be utilized while supporting all dead loads (including future wearing surface) and all vehicular live loads (at inventory and operating levels). The solution was to REDUCE THE SPACING FOR DIAPHRAGMS NEAR THE PIERS, where the bottom flange was in compression, to about 16.5 ft.

Stage 3 – Deck Placement

Prior to hardening of the deck the steel bridge girders are NOT composite and must resist loads by themselves. These loads are less than those during Stage 4 (no superimposed dead weight or vehicles) but do include temporary construction material and equipment. The major difference in stability from Stage 4 is that the girder top flange is now braced by the diaphragms rather than the hardened deck.

Similar to Stage 4, spacing the diaphragms at 25 ft would have resulted in reasonably small stresses in both the top and bottom flanges due to winds loads. Also similar to Stage 4, that diaphragm spacing was too large to sufficiently brace the top flange so that most of the yield strength could be utilized.

The solution was to REDUCE THE SPACING FOR THE REMAINING DIAPHRAGMS to about 16.5 ft. This solution allowed for about 90% of the yield stress to be utilized and simplified fabrication.

Stage 2 – Framing Complete

This stage is the same as Stage 3 but instead of the girder flanges spanning between the diaphragms to resist wind loads, the flanges are assumed to span between the piers with no stability provided by the diaphragms. The client for the case study bridge (PennDOT) has a design standard (BD-620M) which mandates how this lateral stability analysis is to be performed, including wind pressures and allowable deflections. With a girder spacing to depth ratio less than 2 (See Figure 2), the design wind pressure was the lowest required for a given height above ground.

For spans in excess of 300 ft, BD-620M requires lateral bracing between the girders regardless of analysis results. Between 200 ft and 300 ft spans the standard requires lateral bracing only if the lateral deflection acceptance criterion of L/150 is exceeded. However if no lateral bracing is provided, the girders must be designed for the combined dead load, construction live load and wind load.

Span 2 of the case study bridge at 293 ft and Span 3 at 335 ft (See Figure 1) meant that lateral bracing was required. However, AECOM performed an initial analysis without lateral bracing just to see how susceptible the bridge was to instability during Stage 2. The results indicated significant overstress of the top flange in all spans and of the bottom flange in all but Span 4. The results also indicated that the maximum deflection in the top and bottom flanges was L/215 and L/263. Not surprisingly this occurred in Span 3. However the criterion in BD-620M was a maximum of L/150 which both flanges did not exceed.

One level of lateral bracing is sufficient to provide stability as the diaphragms can transfer the wind forces from the unbraced flange to the braced flange. Typically the flange with the greater width is selected for the lateral bracing to avoid gusset plates by attaching the bracing members directly to the flange. However, on the case study bridge the top and bottom flange widths are the same at each section. It was decided to provide lateral bracing of the bottom flange to avoid potential conflicts with the stay-in-place deck forms which extend below the top flange (See Figure 4).

Since Span 1 at 168 ft and Span 4 at 165 ft (See Figure 1) were well under the 300 ft limit in BD-620M, consideration was given to providing lateral bracing in Span 2 and 3 only. It was thought that the bracing in those spans would be enough to eliminate the fully unbraced flange overstress in Span 1 and 4. This idea was discarded after discussing the potential for stress concentration in the girder at the interfaces between braced and unbraced spans.

AECOM performed an analysis with lateral bracing between the bottom flanges in one bay between interior girders (See Figure 3) in all four spans. As expected, the lateral deflections reduced significantly to about 10% of the unbraced values. Now modeled as a horizontal truss, the flange bending stress changed to a much smaller axial stress and the unbraced length changed from the span length to the diaphragm spacing which eliminated all unbraced flange overstresses.

The braced analysis provided design axial loads for the diagonal bracing members, for the diaphragm bottom chords which acted as verticals in the horizontal truss, and for the wind load reactions that needed to be transferred to the piers and abutments by the end and intermediate diaphragms.

Single and double angles were utilized depending on the magnitude of the axial load. Connection details for the diagonal bracing members to the bottom flange were developed (See Figure 5). Where space permitted, the details indicated a direct connection between diagonal and flange with beveled fill plate to accommodate any slope on the diagonal. Otherwise, a bent gusset plate was indicated. However, the fabricator decided to use bent gusset plates at all of the diagonal to flange connections.

Stage 1 – Staged Erection

The original conceptual steel erection plan included shoring towers at Span 1, Span 3, and Span 4. Early on in the erection review, concerns arose regarding the stability of a shoring tower that would be 120 ft tall and need to be supported on driven steel piles. The preliminary modeling of the tower included a 4 inch eccentricity from plumb (L/360) which was less than the requirement from The AASHTO Guide Design for Bridge Temporary Works (Ref. 1). Significant uplift was compounded by wind loading on the tower and the 10’-6” deep girders. This was a serious concern, as the deep river valley was subject to significant gust effects. Figure 6 shows the shoring tower support at Span 3.

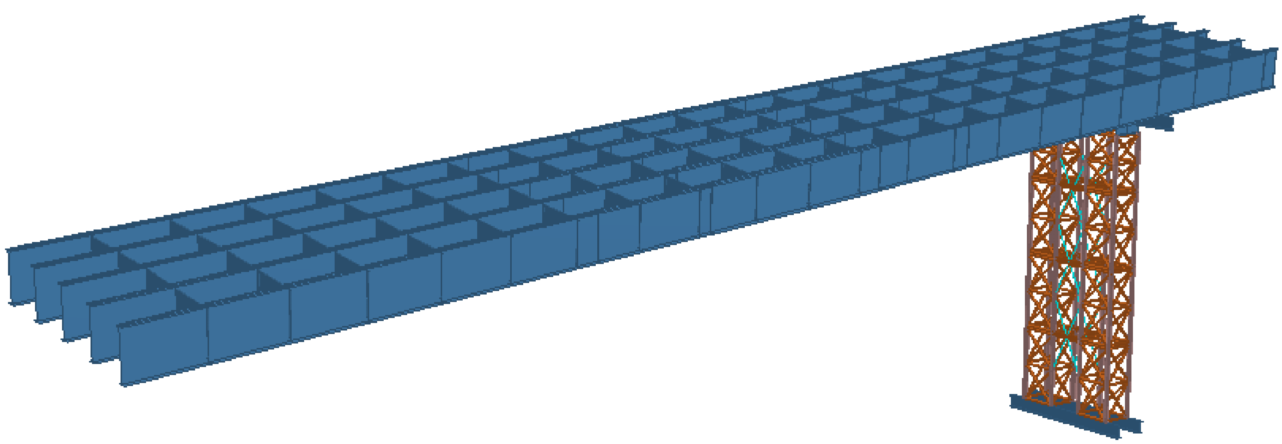

Figure 6: Model of shoring tower on Span 3.

The concept was abandoned after the splice location had to be changed to comply with 150% crane capacity requirement for the girders erected over railroad property. The revised splice location created a 100 ft cantilever on the shoring tower. The tower could not be relocated due to the adjacent boundary of the historic canal.

Bridge Modeling

A 3D model of the steel superstructure was created in Bentley's STAAD.pro computer software (Ref. 2). The intermediate and pier cross frames were modeled as idealized components to add the correct overall stiffness. Figure 7 shows the staged construction of the closure Span 3. The closure span length was 335 ft.

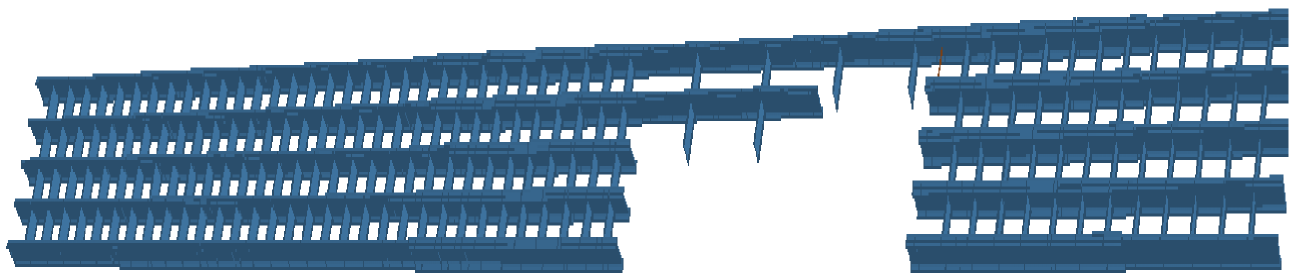

Figure 7: Model of closure Span 3.

The model was checked using UT Bridge Analysis software from the University of Texas at Austin (Ref. 3). The bridge in partially erected states had a stress interaction of 1.0 (LRFD). UT Bridge is a finite element analysis software. The eigenvalue of Span 3 was 1.7 confirming the holding analysis that stability of the bridge during construction with indicated cross frames were at the limit of stability. Figure 8 shows the UT bridge analysis of closure span.

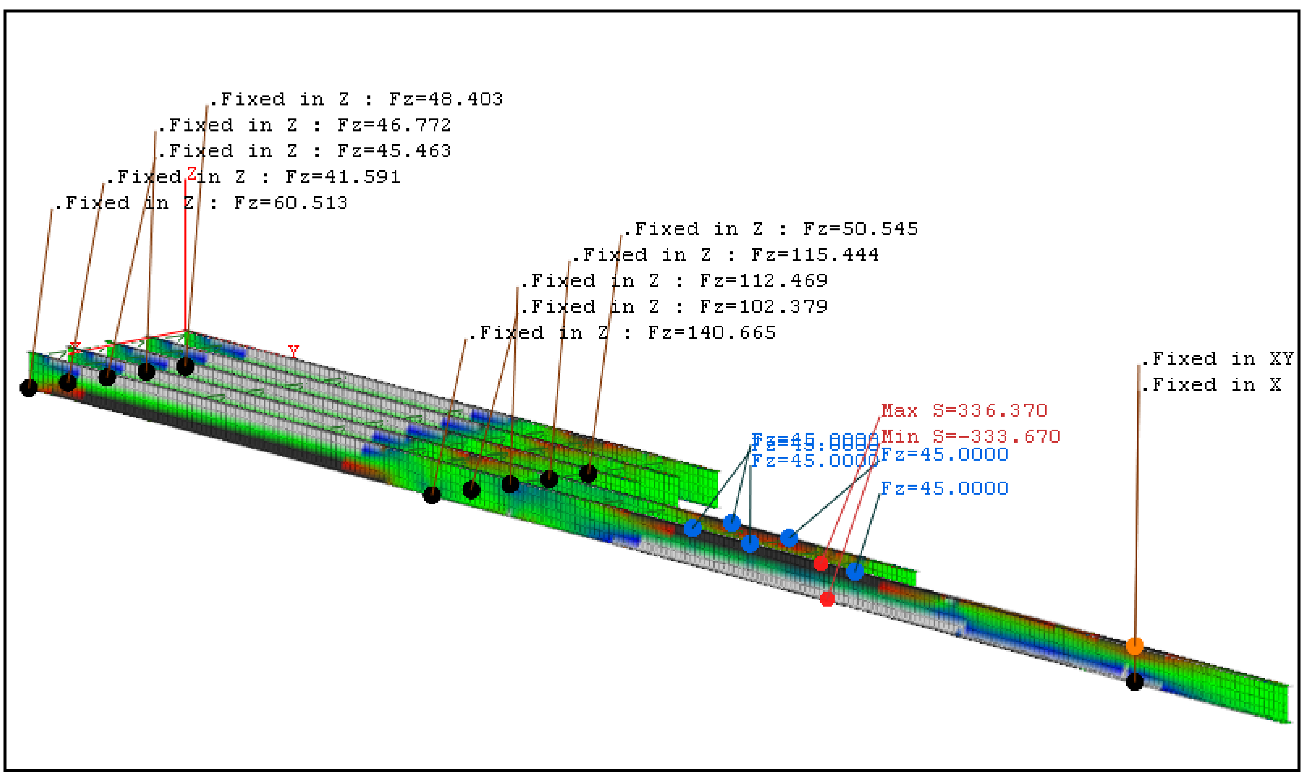

Figure 8: UT Bridge model of closure span.

Holding cranes were modeled as compression springs to check deflections at the specified capacity. Typically, the holding crane required about 10,000 lbs of additional capacity to pull up on the girder line to keep the web vertical and maintain the splice at the correct survey elevation.

Site Considerations

The maximum girder length that could be trucked onto the temporary causeway was 131 ft. Between the critical lift component over the railroad and the site constraints, a total of three supplemental field splices were designed using the PADOT SP LRFD software program. Figure 9 shows site access limits for trucking deliveries. Figure 10 shows the as-designed schematic girder elevation along with the proposed, re-designed girder elevation

Figure 9: Site access constraints with maximum possible girder length.

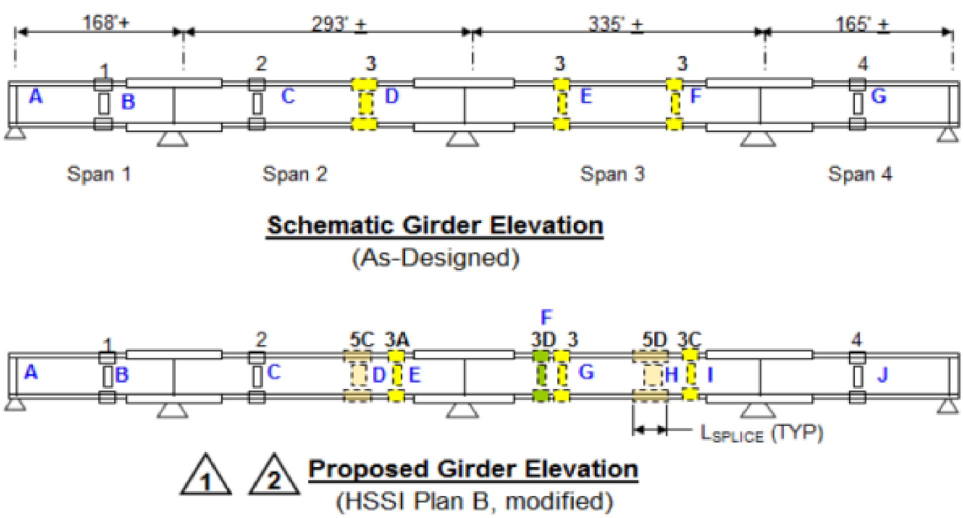

Figure 10: Supplemental field splices required by girder length limits at site.

The bridge erection was designed using AASHTO LRFD Bridge Design Specifications (Ref. 4) as well as PennDOT Design Manual, Part 4, (Ref. 5). Stability of the partially erected spans was based on self-weight of the steel members, wind loading and construction live load effects. The construction live load effects during erection of the superstructure were limited to splice cages, weight of tools and Ironworkers.

Erection Plan

A plan was conceived to use multiple cranes to erect the spans, air-splicing the steel girders. The first girder is erected and set down on the bearing. The next girder is spliced into the end of the first girder with the splice being bolted at least 50% before the second girder is a set down on the Pier bearing. The girders are aligned on the center of bearing survey marks. While both cranes are holding the first erected girder line, a third crane hooks onto the girder line to pull up on this girder keeping it vertical and controlling the induced moment and braced length for girder stress analysis. The cranes that erected the first girder line can now release their hold.

While crane number three is still holding, a fourth crane places the required cross frames for global stability between line 1 and line 2.

Web layover became a problem during the erection of Span 2 which was 293 ft. Due to uncertainty of manually pulling the girders into alignment to bolt cross frames, three cranes were used hold the two girder lines. This required the cranes to hook and re-hook to limit vertical and lateral deflections. The fourth crane placed the indicated cross frames. Cross frames were bolted 100% to limit lateral deflections. Figure 6 shows multiple cranes erecting Span 3 over the Norfolk and Southern Railroad.

Figure 11: Span 3 over Reading Blue Mountain and Norfolk Southern Railroad.

Controlling Bridge Geometry

Bridge geometry of long span steel multi-girder skewed bridges can be controlled using the following methods;

1. Tie-downs to brace the bottom flange to prevent transverse movement.
2. Bearing locks (rods and washers) to hold the longitudinal position.
3. Placement of angles and cables to restrain movement of bearings onto which the girders have been tack welded into position.
4. Bolting of cross frames 100% to limit lateral deflections.
5. Placement of indicated cross frames at the end of spans (upstation and downstation) to increase the stiffness of the span to emulate pinned column behavior.
6. Longitudinal jacking at abutments to adjust for load shifting and drift during erection on bearings that are not fully engaged.

Figures 12 and 13 show transverse tie-down restraining and longitudinal jacking, respectively.

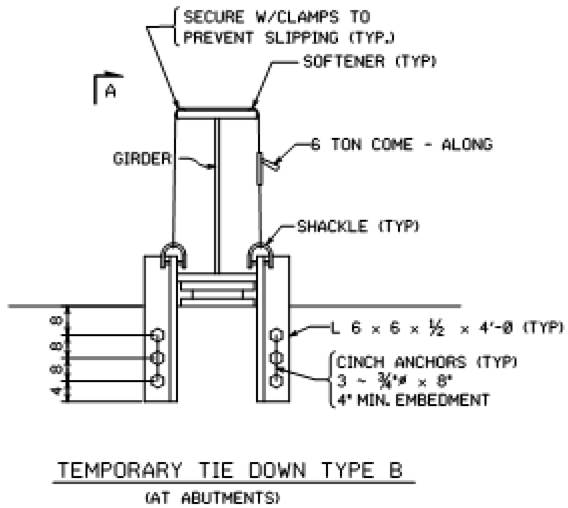

Figure 12: Transverse girder restraint.

Figure 13: Longitudinal jacking of girder at abutment.

During the phased erection plan, bolts were snug-tightened. After the steel superstructure was complete, the final bolts were torqued. After each span was torqued, the bearings were welded into final position.

Anchor bolts were not grouted-in-place until after the superstructure was completed. Often the anchor bolts will not be grouted in place until after the deck pour is completed (creating additional movement of the bearings). The anchor bolts were placed in 6 inch diameter cored holes to allow for movement. If required, some additional jacking was completed to maintain the concentric bearing and alignment of the Polytetrafluoroethylene (PTFE) plates before finally being grouted in position.

Staging Conclusions

Erecting with multiple cranes provided the erector the ability to shorten the construction schedule, as well as respond to site conditions that were not planned. The capacity to move crane locations for trucking delivery, site congestion and vegetation made it possible to maintain the erection schedule without delays. Multiple cranes were advantageous to control bridge geometry and assist with fit-up of the long span steel bridge.

Summary

Stability of long span steel bridges during and after erection depends upon the designer considering how the girders behave under composite and non-composite conditions using code prescribed design forces. Numerous previous examples of failures illustrate the risk of ignoring one or more of the stages that a steel girder undergoes from erection to the end of its service life.

References

1. AASHTO Guide Design Specifications for Bridge Temporary (1995), with 2008 Interim Revisions
2. STAAD.pro. Bentley Systems, Inc. Corporate Headquarters, 685 Stockton Drive, Exton, PA 19341 USA
3. UT Bridge Version 1.5-1, University of Texas at Austin. Program developed for the Texas Department of Transportation as part of a funded research project (TxDOT 0-5574)
4. AASHTO LRFD Bridge Design Specifications, 2010
5. PennDOT Design Manual, Part 4, May 2012 Edition.