# IL-170 OVER THE ILLINOIS RIVER: LONG-SPAN STEEL PLATE GIRDERS ACCOMMODATE TIGHT GEOMETRY

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## BIOGRAPHY

Christopher L. Stine, PE, SE is a Structural Engineer at AECOM in Chicago, IL and holds both a B.S. and M.S. from Southern Illinois University Carbondale. Chris has 10 years of experience in bridge design, construction, & management, including work with several complex structures. Project experience includes the East/West Wacker Drive Reconstruction Project (Chicago IL), George Washington Carver Tied Arch Bridge (Des Moines, IA), I-355 Des Plaines River Bridge (Lemont, IL), SH130 Project (Austin, TX), SH161 Project (Dallas, TX), I-70 Tri-Level Interchange (East St. Louis, IL), and 41st/43rd Street Pedestrian Bridges (Chicago, IL). Chris is currently licensed as a structural professional engineer in Illinois, Iowa, Texas, and Wisconsin.

## SUMMARY

The existing structure carrying IL-170 over the Illinois River was a 4-span Pennsylvania thru-truss found to be deficient both structurally and functionally. The proposed bridge uses long-span plate girders to maintain 360-ft horizontal navigational clearance, while improving the vertical navigational clearance. As a result of the span-to-width and span-to-depth ratios used and support limitations in river, unique solutions were required to ensure global stability of the structure during girder erection as well as pouring of the decks. Based on this experience, many recommendations are made to preclude the need for additional temporary lateral bracing during construction for long-span plate girders with large span-to-width and/or span-to-depth ratios.

# IL-170 OVER THE ILLINOIS RIVER: LONG-SPAN STEEL PLATE GIRDERS ACCOMMODATE TIGHT GEOMETRY

## Introduction

The IL-170 Bridge is located just south of the village of Seneca, IL about 65-miles southwest of Chicago. The existing structure carrying IL-170 over the Illinois River was a four-span, Pennsylvania through-truss made up of three 200-ft spans and a 363-ft mainspan (see Figure 1). Existing approach spans consisted of 60-ft simple-span, stringer spans of five spans at the north approach and four spans at the south approach. The bridge was constructed in 1932 by the Wisconsin Bridge & Iron Company and rehabilitated in 1986. Rehabilitation included replacing the cast-in-place deck with precast concrete panels and asphalt overlay, replacing finger joints with neoprene expansion joints, replacing steel rockers with elastomeric bearings, replacing/repairing many structural steel elements, and miscellaneous concrete repairs to the substructure.

**Figure 1: Existing IL-170 Bridge over the Illinois River (Seneca, IL)**

AECOM was hired by the Illinois Department of Transportation (IDOT) for Phase I & II services in 2003. Despite recent rehabilitation efforts, the trusses and superstructure were only found to be in fair condition, while the substructure was found to be in poor condition. Although posting the bridge was not necessary, the advanced deterioration of the substructure suggested that the bridge should be replaced (see Figure 2). However the deciding factor that dictated replacing the structure was not based on its structural condition, but rather on the fact that it was functionally deficient with a roadway width of just 22.9-ft (see Figure 3). This width made it dangerous for opposing traffic to pass especially with semi-trucks and farm equipment that used the structure in this rural area. Truss repairs were required in 1996 after being struck by a truck. In addition, pedestrians including children at a nearby school used the bridge despite its lack of sidewalks.

**Figure 2: Existing substructure deterioration**

**Figure 3: Functionally deficient bridge width**

# Planning

The central challenge on this project was designing a long-span bridge to accommodate limited geometry. The first major decision was whether to detour traffic, use stage construction, or build a parallel structure. The last nearby structure to be built across the Illinois River was the Shippingsport Bridge in La Salle, IL, which was demolished in 2001 and reopened in 2003. Detouring IL-351 traffic for two years was allowed since IL-251 and Interstate I-39 bridges were located only a mile upstream and downstream, respectively. For the case of the IL-170 Bridge in Seneca, IL, the nearest structures were the IL-47 Bridge in Morris, IL located eleven miles upstream and the Main Street Bridge in Marseilles, IL located six miles downstream. Due to these distances and the fact that IL-170 was an emergency evacuation route for the LaSalle County Nuclear Power Station located five miles southwest of Seneca, detouring IL-170 traffic was not an option. Like Shippingsport, stage construction was also not an option since the existing bridge was a truss bridge.

With the new structure required to be built alongside the old one, the second major decision was where to locate the new IL-170 alignment. Since there was an at-grade intersection on the south end of the project, four alignments were studied with two alignments east of the existing alignment and two to the west of it. For each pair of alignments beside of existing IL-170, one had an at-grade intersection with DuPont Road while the other had a grade separation with DuPont Road passing beneath the proposed IL-170 alignment. At-grade options required raising DuPont Road 12 to14-ft, forced all DuPont Road traffic to cross IL-170, eliminated access to the boat yard to the southeast, and required replacing a local bridge to the southwest. Despite its cost savings, the at-grade options were eliminated due to safety and impact to local businesses. Western options required embankments that eliminated the front yards of several houses and required the removal of one house to the southwest. Thus western options were eliminated due to impacts to residents.

With the new alignment established just east of the existing one (see Figure 4) and a new grade separation at DuPont Road on the south end of the project, the third major decision was how wide to built the bridge. Based on the current ADT of 5,500 vehicles per day (2007), future ADT of 7,700 vehicles per day (2027), and the need for a turn lane at DuPont Road, traffic studies indicated the need for either a 4-lane bridge or a 2-lane bridge with a flared width to accommodate a dedicated turn lane at the DuPont Road intersection. Since the 2-lane bridge was estimated to be much less expensive, this alternative was ultimately selected. Thus, AECOM proposed a 48’-10” wide bridge with two 12-ft lanes, two 6-ft shoulders to accommodate farm equipment, a 9-ft sidewalk to accommodate bicycles, two 1’-7” F-shaped parapets, and an 8-in rail. However, it was felt that this width was not economically justified, so shoulders were reduced to 3-ft and the sidewalk to 5-ft, which produced a 38’-10” wide bridge in the non-flared Units 1 and 2 (see Figure 5).

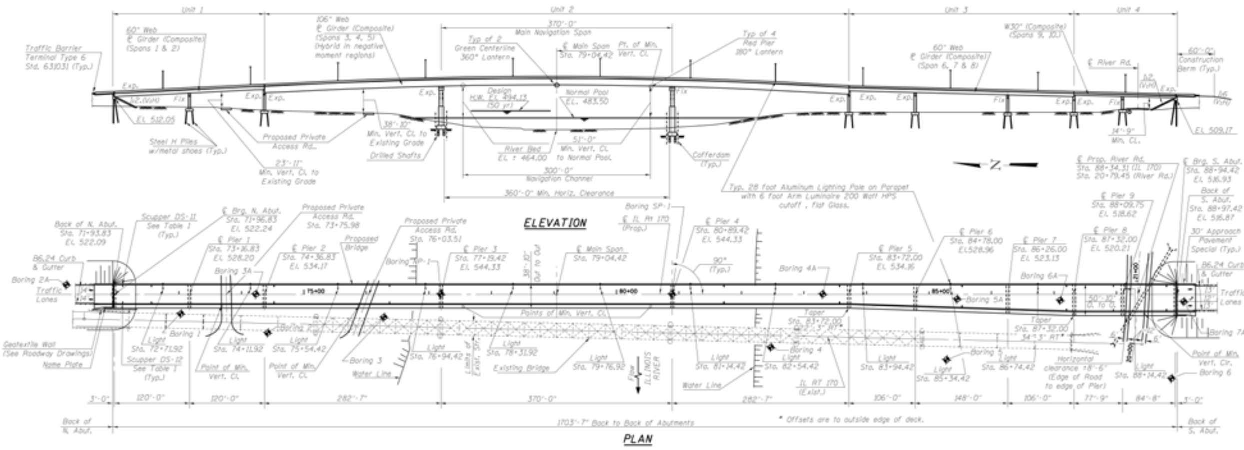


**Figure 4: Plan & Elevation of Proposed IL-170 Bridge**

With the bridge width set at 38’-10”, the fourth major decision was to choose suitable span arrangements. For the mainspan the horizontal navigational clearance was determined by the U.S. Coast Guard (USCG). While normally given online by the USCG, no horizontal clearances were available for the Illinois River. The USCG determined the navigational channel was 300-ft, but required a minimum horizontal clearance of 360-ft along this section of the Illinois River in order to match the existing clearance of nearby bridges. The existing 363-ft mainspan was therefore increased to 370-ft in order to provide sufficient width for the proposed river piers, which would be wider than the current piers that were not designed for barge impact. The endspans flanking the 370-ft mainspan in Unit 2 were originally 290-ft but were decreased to 282.5-ft in order to remove the flare from Unit 2 and limit the 360-ft turn lane transition completely within Unit 3. The resulting 282.5-ft / 370-ft / 282.5-ft span arrangement produced an overall length of 935-ft for Unit 2.

With the mainspan set at 370-ft, the fifth major decision was to establish the minimum vertical clearance. Similar to the horizontal clearance, the vertical navigational clearance was also determined by the USCG, while the elevation it was measured from was determined by the U.S. Army Corp of Engineers (USACE). According to USCG data shown online, the minimum vertical navigational clearance on the Illinois River between Joliet and Starved Rock is 47-ft, which explains the clearance of 47.8-ft on the existing structure. After contacting the USCG, they confirmed in July 2003 that 47-ft was indeed required but indicated that 50-ft was preferred. It must be noted that 50-ft is the minimum vertical navigational clearance of the next downstream section of the Illinois River from Starved Rock to Peoria according to the online USCG data. After subsequently contacting the USACE and finding out that the “Flat Pool Elevation” was EL 482.78, AECOM proceeded with establishing the vertical profile based on a 50-ft clearance above that elevation.

With the clearances established, the sixth major decision was to ascertain the most suitable structure type. While about half of the existing bridges across the Illinois River are steel truss bridges built in the 1930s, steel truss bridges began to fall out of favor in the 1960s to the more economical steel plate girder bridge. AECOM’s Bridge Type Study identified two options to economically span 370-ft across the Illinois River which were steel plate girder and concrete segmental. Steel truss bridges were not considered due to their non-redundant, fraction-critical nature and their high fabrication cost, while arch and cable-stayed bridges were not considered due to the required span length not being long enough to justify their significant cost. A steel plate girder option with a 2003 estimate of $16.6M was chosen over the concrete segmental option with a 2003 estimate of $20.3M due to its cost savings as well as IDOT’s preference of steel plate girders. The low bid of $20.9M was below the final estimated cost due to the competitive bidding climate in 2008.

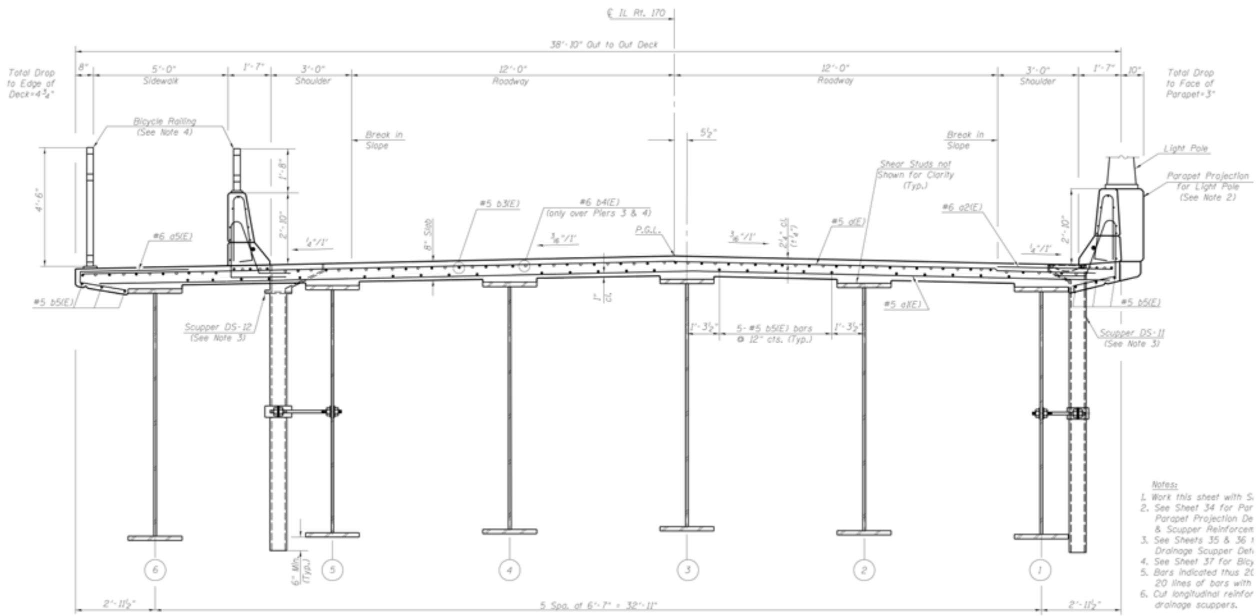


**Figure 5: Proposed Cross-Section for Units 1 & 2**

With steel plate girders selected for the bridge type, the final major decision was to determine the number, strength, and depth of plate girders. While AASHTO LFD does not specify minimum number of girders, AASHTO LRFD 4.6.2.2.1 (2) requires four girders to prevent non-redundant, fracture-critical conditions. However IDOT's more stringent requirements control here. According to IDOT Bridge Manual 2.3.8 (4):

> *Stage traffic over deck-girder superstructures shall be supported by at least three girders. New deck-girder superstructures which may not be initially staged should consider the number and arrangement of girders in order to provide at least three girders for possible future staging. This requirement may be waived if traffic can be detoured during future reconstruction or if approval is obtained from the IDOT Bureau of Bridges and Structures.*

The classic, conservative interpretation of this requirement is that typical bridges which must utilize stage construction during future redecking operations must possess at least six girders in the final cross-section. Alternatively, this requirement could also be interpreted as requiring a minimum of just five girders in the final cross-section if the centermost girder is assumed to be applicable to each stage during the redecking. Since IDOT follows the first interpretation, six girders were chosen despite the ability to use four or five. As far as plate girder material, Grade 50W weathering steel was chosen over Grade 50 painted steel since the structure was located in a rural area with emphasis on minimizing construction and maintenance costs. Use of high performance steel (HPS 70W) also seemed like a natural choice since it is most economically used on long-span bridges, is compatible with 50W steel, is only slightly more expensive than 50W steel, and was used on the last two bridges built over the Illinois River (i.e. Shippingsport and Morris Bridges).

For discussion of the plate girder depths, both AASHTO LFD (1) and AASHTO LRFD (2) bridge design specifications are referenced, although only AASHTO LFD was utilized for preliminary and final design. According to AASHTO LFD 10.5, the span-to-depth ratio for simple-span, noncomposite girders is L/25, whereas the span-to-depth ratio for the much stiffer simple-span, composite girders is L/30 (see Figure 6). For continuous spans the distance between dead load contraflexure points can be used instead of the span. If this distance is approximated as 80% of the span, the span-to-depth ratio for continuous, noncomposite girders becomes L/31.25, while the span-to-depth ratio for continuous, composite girders becomes L/37.5. This is similar to AASHTO LRFD Table 2.5.2.6.3-1 which limits the span-to-depth ratio to 0.033L (L/30) for continuous, noncomposite girders and 0.027L (L/37) for continuous, composite girders (see Figure 7). For the continuous, composite 370-ft span used here, a girder depth of 10-ft (120-in) was initially chosen.

**10.5 DEPTH RATIOS**

**10.5.1** For beams or girders, the ratio of depth to length of span preferably should not be less than $1/25$.

**10.5.2** For composite girders, the ratio of the overall depth of girder (concrete slab plus steel girder) to the length of span preferably should not be less than $1/25$, and the ratio of depth of steel girder alone to length of span preferably should not be less than $1/30$.

**10.5.3** For trusses the ratio of depth to length of span preferably should not be less than $1/10$.

**10.5.4** For continuous span depth ratios the span length shall be considered as the distance between the dead load points of contraflexure.

**10.5.5** The foregoing requirements as they relate to beam or girder bridges may be exceeded at the discretion of the designer.*

**Figure 6: Maximum Span-to-Depth Ratios per AASHTO LFD 10.5 (1)**

| Superstructure | | Minimum Depth (Including Deck)<br>When variable depth members are used, values may be adjusted to account for changes in relative stiffness of positive and negative moment sections | |
|---|---|---|---|
| Material | Type | Simple Spans | Continuous Spans |
| Steel | Overall Depth of Composite I-Beam | $0.040L$ | $0.032L$ |
| | Depth of I-Beam Portion of Composite I-Beam | $0.033L$ | $0.027L$ |
| | Trusses | $0.100L$ | $0.100L$ |

**Figure 7: Maximum Span-to-Depth Ratios per AASHTO LRFD Table 2.5.2.6.3-1 (2)**

# Final Design

In November 2003, the USCG informed AECOM that they now required a navigational clearance of 51-ft (an additional 12-in of vertical clearance) after studying other recently built bridges over the Illinois River such as the Shippingsport Bridge, which possessed a vertical navigational clearance of 15.45-m (50.7-ft). According to online USCG data, Shippingsport required 50-ft since it was located between Starved Rock and Peoria, while Seneca really only required 47-ft since it was located between Joliet and Starved Rock. Finding this out after finalizing the alignment, vertical profiles, and stage construction was disappointing. While increasing the vertical clearance usually necessitates increasing the vertical profile by that amount, this would have meant lengthening the 1,700-ft bridge and totally revising the DuPont Road Interchange. Since budget and time constraints prevented such rework, it was decided to just deduct the additional foot from the superstructure depth. Consequences of the decision would have to be examined in girder design.

After completing the various civil studies (drainage, environmental, hydraulic, right-of-way, traffic, etc.), work on the preliminary girder design and the Type, Size, and Location (TS&L) plans began in July 2005. As part of this preliminary girder design, AECOM requested that National Steel Bridge Alliance (NSBA) investigate the economic feasibility of using high performance steel (HPS 70W) for the IL-170 structure. As requested, they compared six girders using Grade 50W steel versus six girders using both Grade 50W and HPS 70W steel. In addition, they did homogeneous and hybrid girder designs using just four girders. NSBA used their in-house line-girder analysis software SIMON to design the girders per AASHTO LFD. As is typically found, use of HPS 70W steel was only economical for flanges in negative moment regions. The cost of each of the four options normalized against the least expensive option for a range of unit costs ($0.75/lb - $1.25/lb) that includes fabrication and delivery, but not erection, is given below (see Figure 8).

As can be seen in Figure 8, both 4-girder options provided sizeable cost savings over the 6-girder options, regardless of the unit price of steel. This is in-line with other girder optimization studies which show that eliminating girderlines is the best way to decrease weight and cost. However, AECOM only asked NSBA to check the 6-girder options since IDOT requires at least that many girders within the final cross-section. Among 4-girder options, Option B2 with HPS 70W steel was slightly more economical especially as unit prices increased. On the other hand for 6-girder options, Option A1 with all 50W steel was slightly more economical especially as unit prices decreases. As can be observed from each pair of options in Figure 8, HPS 70W steel is not always economical, but it always becomes more economical as unit prices increase. Because the price of steel was escalating and it was desirable to keep the flange thicknesses less than 3-in, it was decided to go ahead and use HPS 70W steel for the negative moment flanges of the 6-girder option.

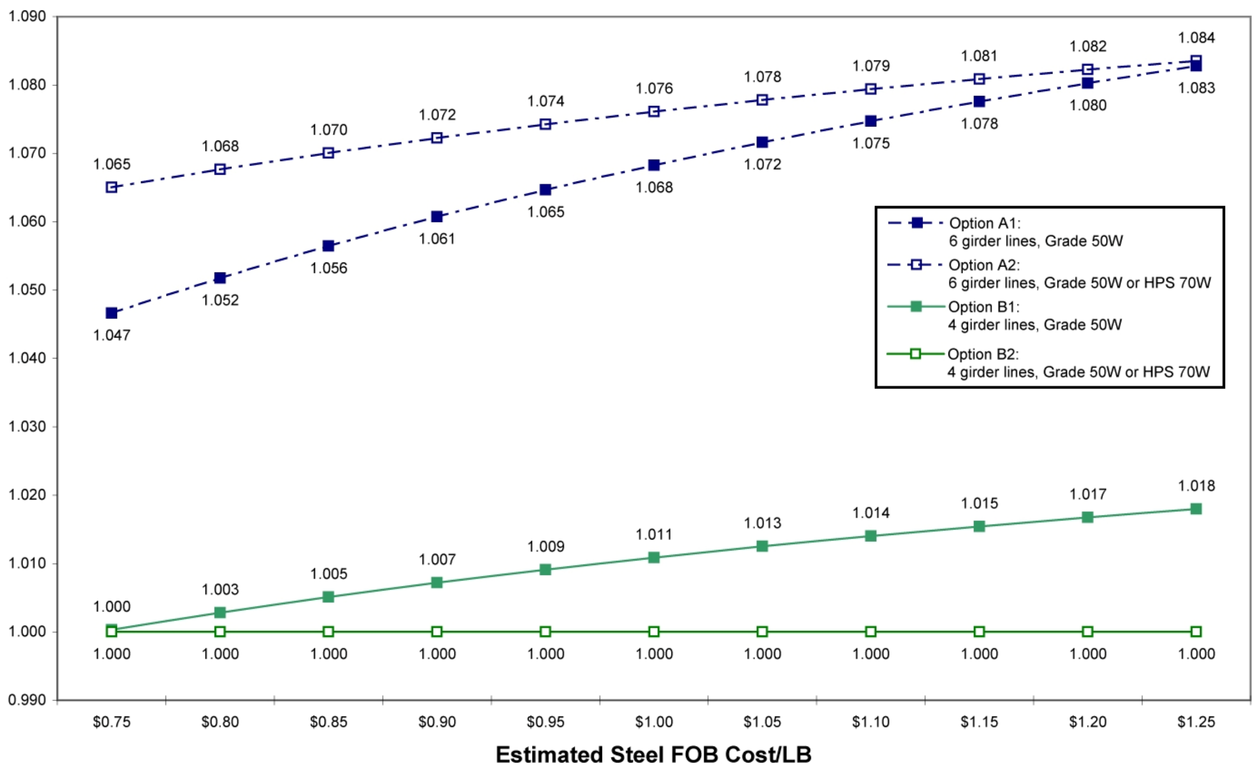


**Figure 8: NSBA Girder Optimization Study (July 2005)**

After submitting the TS&L in October 2005, IDOT provided preliminary review comments in June 2006. IDOT also contacted the USCG in June 2006 and questioned the range of vertical navigational clearance. According to IDOT, other nearby bridges over the Illinois River just had to provide the minimum vertical clearance over the 300-ft navigational channel, instead of the entire 360-ft minimum horizontal clearance. The USCG concurred in June 2006 and the range of applicability for the minimum vertical clearance was reduced to the 300-ft navigational channel which helped recover about 6.5-in of the web depth lost earlier. Unfortunately, a third USCG revision in September 2006 required that the minimum vertical clearance be measured from a "Normal Pool Elevation" of EL 483.50 rather than a "Flat Pool Elevation" of EL 482.78. Similar to the vertical clearance increase, the only option was to deduct this extra clearance from the web. Together the three revisions resulted in a net reduction of 14-in from the girder web depth as given below:

*Vertical Navigational Clearance Increase: - 12-in*
*Min. Vert. Clr. moved 30-ft away from Pier: + 6.5-in*
*EL Change from Flat Pool to Normal Pool: - 8.5-in*
*Net Reduction in Plate Girder Web Depth: - 14-in*

This reduced the initial 120-in web depth to only 106-in and increased the initial span-to-depth ratio from L/37 to L/42 within the 370-ft mainspan, which exceeds the maximum span-to-depth ratio for continuous, composite girders (see Figures 6 & 7). Fortunately, the maximum span-to-depth ratios are not mandatory. The final horizontal and vertical navigational clearances for the mainspan are shown below (see Figure 9).

In addition to the final span-to-depth ratios being large, the resulting span-to-width ratios were also large. If the 38'-10" deck width is used, the span-to-width ratio in the mainspan is L/9.5 for the final conditions. If the 32'-11" steel framing plan between exterior girders is used, the span-to-width ratio in the mainspan is about L/11.25 during construction. While AASHTO has no explicit guidelines for span-to-width ratios, these values are double or triple what is typically encountered with traditional grade separation structures. The large span-to-width and span-to-depth ratios would later prove to be troublesome for global stability.

The final TS&L was submitted and approved in November 2006 with design beginning in February 2007. According to an important Federal Highway Administration (FHWA) memorandum dated June 28, 2000, "all new bridges on which States initiate preliminary engineering after October 1, 2007, shall be designed by AASHTO LRFD Bridge Design Specification". Because the IL-170 Bridge project preceded that date, the superstructure design was done in accordance with AASHTO LFD, which was applicable at that time. It should be noted that some future concerns that emerged would have been handled by AASHTO LRFD. Since the structure was neither curved nor skewed, a line-girder analysis was determined to be acceptable with girders designed utilizing the MERLIN DASH girder analysis software. Girders were designed to be composite in positive moment regions only and utilized an 8-in concrete deck, recently adopted by IDOT. Live load was HS-20 truck with live load deflections limited to L/1000 due to the presence of pedestrians.

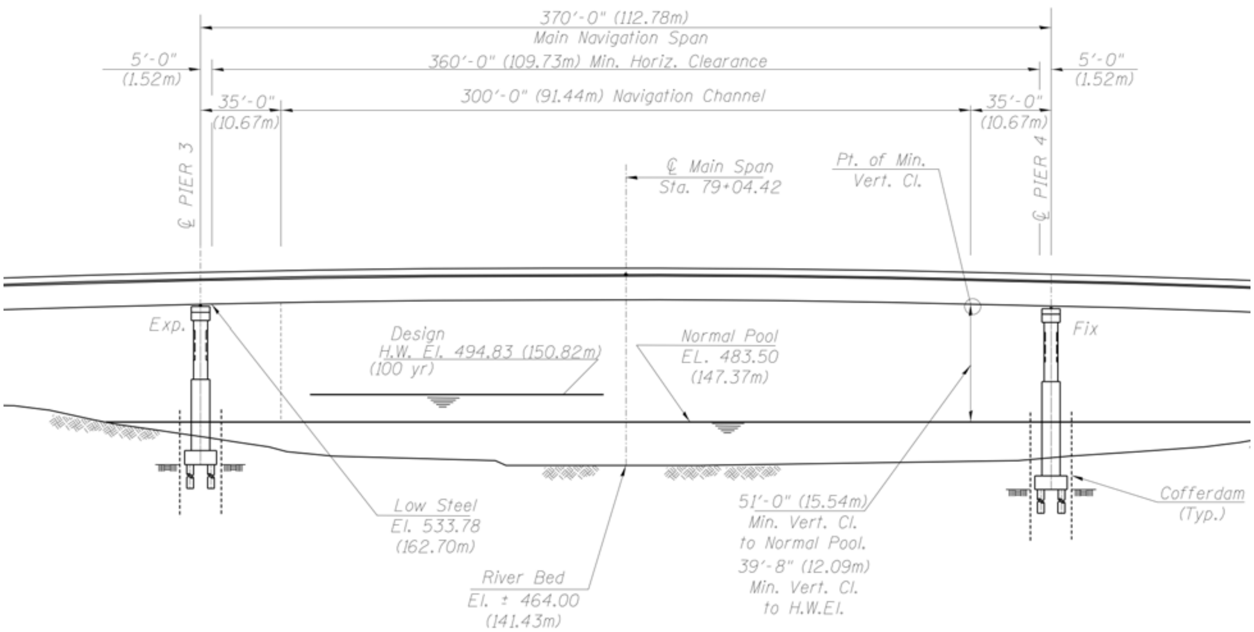


**Figure 9: Final Horizontal & Vertical Navigational Clearances**

As a result of limiting both approach grades to 5% for ADA compliance and spanning over DuPont Road, the bridge was required to be 1,700-ft long and was divided into four units based on spans (see Figure 4). Unit 1 on the north end consisted of two 120-ft spans. Unit 2 over the Illinois River consisted of a 370-ft mainspan with 282.5-ft endpsans. The flared Unit 3 consisted of a 148-ft mainspan with 106-ft endspans. Unit 4 on the south end consisted of two 80-ft spans. The bridge width was 38'-10" within Units 1 and 2 (see Figure 5), transitioned from 38'-10" to 50'-10" for the turn lane in Unit 3, and was 50'-10" in Unit 4. As stated earlier, Unit 2 had six 106-in deep plate girders. Units 1 and 3 used six 60-in deep plate girders, but in Unit 3 the six girders were flared and introduced a seventh girder through the use of a header beam. While the 60-in girders could have been 48-in deep, they were deepened to better match the 106" girders. Unit 4 used eight W30x173 wide-flange beams to minimize the superstructure depth above DuPont Road.

Due to the long spans utilized in Unit 2, eleven girder segments with ten bolted field splices were required to keep splices away from high bending moments and limit all girder segments lengths to less than 135-ft, according to IDOT Bridge Manual 3.3.21 (4). Girder dimension changes were limited to splice locations.

Using the MERLIN DASH girder analysis software, numerous plate girder configurations were analyzed. The final plate girders selected employed a 3/4-in thick web and 22-in wide flanges throughout the Unit 2. Flange thickness for the 282.5-ft end spans varied from 1-in at the end, to 1 3/8-in at the positive moment region and positive/negative moment transition region, to 2.5-in at the negative moment region over piers. The 370-ft mainspan used the same flange thickness in the negative moment region and transition region, but used 2-in flanges for the center 110-ft segment at midspan in order to meet L/1000 deflection criteria. All flanges used Grade 50W steel except top and bottom flanges over the pier which used HPS 70W steel.

Bracing for Units 1, 2, and 3 consisted of cross-frames, while bracing for Unit 4 consisted of diaphragms. Standard IDOT bracing was used except interior cross-frames in Unit 2 (see Figure 10), which included a top flange strut in addition to standard diagonals and bottom flange struts to provide extra lateral stability. According to Fasick et.al. (3), cross-frames with top flange struts should be used when web depths exceed half the girder spacing. Since K-style cross-frames of various web depths are used to illustrate that point, it seems that diagonals with angles more than 45-degrees become ineffective and require top flange struts. Hence, for X-style cross-frames, top flange struts should be used when web depths exceed girder spacing. Since the 8’-10” web depth exceeded the 6’-7” girder spacing, all cross-frames included top flange struts. Lateral bracing was not used due to its high cost and fatigue related concerns. It should also be noted that neither the Shippingsport Bridge nor the Morris Bridges possessed any top flange struts or lateral bracing.

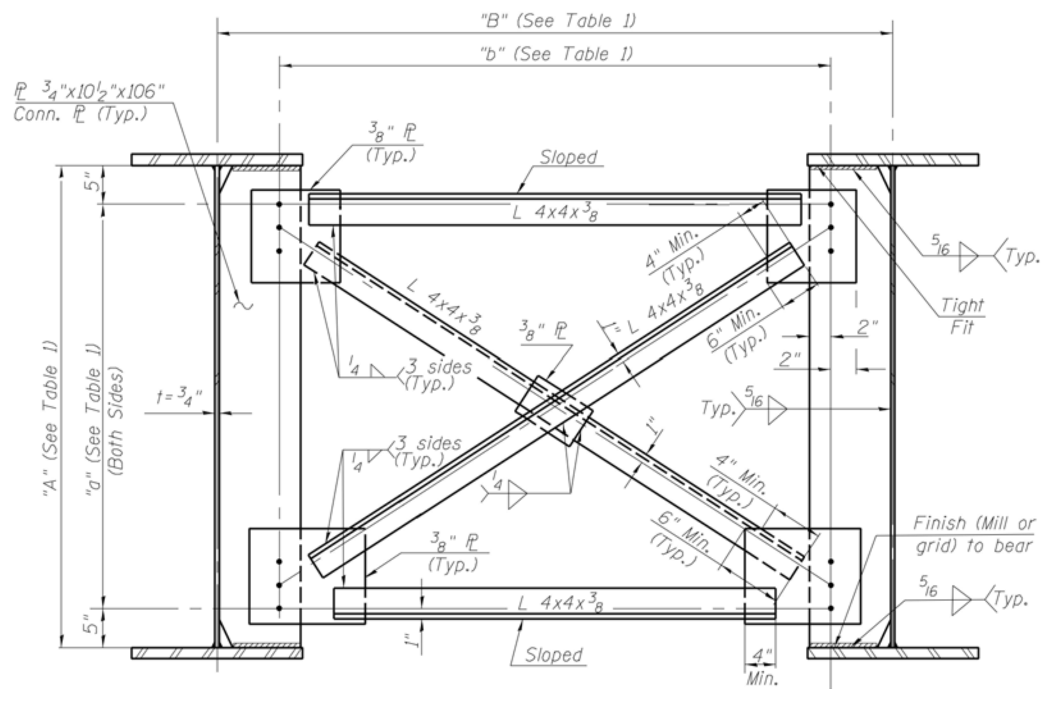


**Figure 10: Typical Interior Cross-Frame for Unit 2**

# Construction

In March 2009, the Contractor contacted IDOT (who then contacted AECOM) about some concerns they had involving the global stability of the structure during upcoming steel erection and concrete deck pours. Concern was based on a comparison of the lateral-torsional buckling (LTB) capacity of the girder system (without a hardened concrete deck) in the endspans of the Seneca Bridge versus that of the Morris Bridge. While AASHTO code checks local LTB of single girders, it does not check global LTB of girder systems. Based on a global stability paper by Yura et.al. (8), the Contractor estimated the torsional system stiffness (Cw) by multiplying the strong-axis moment-of-inertia (Ix) by the spacing between exterior girders (Stot). The span length (L), web depth (D), number of girders (N), girder spacing (S), total girder spacing (Stot), vertical and lateral girder stiffness (Ix & Iy), and vertical, lateral, and torsional system stiffness (Ixx, Iyy, & Cw) for these two structures and Shippingsport are provided below for comparison (see Tables 1 & 2).

As can be seen in Figure 11, the torsional system stiffness for the endspans of the Morris Bridge is nearly 75% greater than that of the Seneca Bridge as a result of the vertical moment-of-inertia being 130% more. The Morris Bridge is a 5-span bridge of 91-m (298.57-ft) endspans, 110-m (360.91-ft) intermediate spans, and a 123-m (403.56-ft) mainspan. The longer mainspan coupled with 45% larger girder spacing resulted in the girders for the Morris Bridge being 16-in deeper and much stiffer than those for the Seneca Bridge. The weaker torsional system stiffness of the Seneca Bridge is primarily a result of having more girders at tighter girder spacing with smaller moments-of-inertia, despite being similarly adequate for vertical loads. This is even more obvious when comparing endspans of the Seneca Bridge with the Shippingsport Bridge which is nearly identical to the Seneca Bridge except for the 12% larger distance between exterior girders. This results in a 27% increase in torsional system stiffness over the Seneca Bridge as this term is squared.

As can be seen in Figure 12, the torsional system stiffness for the intermediate spans of the Morris Bridge was only 12% more than the Seneca Bridge mainspan as a result of the vertical stiffness being 50% more, while spacing between exterior girders was 13% less. As for Shippingsport, the torsional system stiffness of its mainspan was 20% smaller than the Seneca Bridge due to its vertical stiffness being about 35% less. However, the main concern from comparisons with Shippingsport was its lack of lateral system stiffness, which was around 50% of that for Seneca's endspans and only around 25% of that for Seneca's mainspan. During our review, AECOM became aware that the Contractor erecting the Seneca Bridge had erected the Shippingsport Bridge only seven years earlier, without the use of either top flange struts or lateral bracing. Therefore, while some of the concerns regarding the global stability of the structure appear to have merit, it cannot be said that this was unique to the Seneca Bridge compared to similar bridges erected in the past.

| IL River Bridge | Span | Web | Girder No. & Spa. | | | Girder Stiffness | | System Stiffness | | |
|---|---|---|---|---|---|---|---|---|---|---|
| (State Route) | L (ft) | D (in) | N | S (ft) | Stot (ft) | Ix (in^4) | Iy (in^4) | Ixx (in^4) | Iyy (in^4) | Cw (in^6) |
| Seneca Bridge (IL-170) | 370.00 | 106.0 | 6 | 6.58 | 32.92 | 331,076 | 3,594 | 1,986,455 | 21,564 | 5.17E+10 |
| Morris Bridge (IL-47) | 360.91 | 122.0 | 4 | 9.51 | 28.54 | 493,673 | 6,291 | 1,974,690 | 25,166 | 5.79E+10 |
| Shippingsport (IL-351) | 370.75 | 106.3 | 6 | 7.35 | 36.75 | 211,725 | 966 | 1,270,348 | 5,796 | 4.12E+10 |
| Seneca Bridge (IL-170) | 100% | 100% | 100% | 100% | 100% | 100% | 100% | 100% | 100% | 100% |
| Morris Bridge (IL-47) | 98% | 115% | 67% | 145% | 87% | 149% | 175% | 99% | 117% | 112% |
| Shippingsport (IL-351) | 100% | 100% | 100% | 112% | 112% | 64% | 27% | 64% | 27% | 80% |

**Table 1: Girder & System Stiffness for Endspans of Various Illinois River Bridges**

| IL River Bridge | Span | Web | Girder No. & Spa. | | | Girder Stiffness | | System Stiffness | | |
|---|---|---|---|---|---|---|---|---|---|---|
| (State Route) | L (ft) | D (in) | N | S (ft) | Stot (ft) | Ix (in^4) | Iy (in^4) | Ixx (in^4) | Iyy (in^4) | Cw (in^6) |
| Seneca Bridge (IL-170) | 282.58 | 106.0 | 6 | 6.58 | 32.92 | 223,089 | 2,152 | 1,338,532 | 12,913 | 3.48E+10 |
| Morris Bridge (IL-47) | 298.57 | 122.0 | 4 | 9.51 | 28.54 | 512,006 | 6,731 | 2,048,024 | 26,923 | 6.01E+10 |
| Shippingsport (IL-351) | 300.21 | 106.3 | 6 | 7.35 | 36.75 | 228,019 | 1,094 | 1,368,117 | 6,565 | 4.43E+10 |
| Seneca Bridge (IL-170) | 100% | 100% | 100% | 100% | 100% | 100% | 100% | 100% | 100% | 100% |
| Morris Bridge (IL-47) | 106% | 115% | 67% | 145% | 87% | 230% | 313% | 153% | 209% | 173% |
| Shippingsport (IL-351) | 106% | 100% | 100% | 112% | 112% | 102% | 51% | 102% | 51% | 127% |

**Table 2: Girder & System Stiffness for Mainspans of Various Illinois River Bridges**

Due to the concerns raised by the Contractor, it was collectively determined that a global stability analysis should be performed, which AECOM then carried out based on another global stability paper by Yura (6). While it was felt that the girders were indeed adequate for strength during the deck pours or a wind event, we were concerned about their strength during a deck pour, if simultaneously coupled with modest winds. Another concern was that of lateral deflections. According to our LARSA finite element analysis (FEA), lateral deflections of about 5-ft in the 370-ft mainspan could occur for a design wind of 100-mph (50-psf), unless the design wind was reduced to account for the low probability of it occurring during construction. While lateral deflection criteria could not be found, the resulting L/75 deflections were deemed excessive.

Rather than defend the structure based on its adequate global strength, it was decided to exercise care and modify the structure in order to improve its global stiffness, with any added global strength being a bonus.

One option to improve the performance of the structure during the deck pours was to decrease the size of the deck pours in the three positive moment regions. This was found to be impractical because it required adjusting the cambers and screeds which were already set as a result of the girders having been fabricated, and it did not reduce lateral deflections from wind. Another option was adding permanent lateral bracing. While bolting lateral bracing directly to top or bottom flanges is standard detailing practice in some states (see PennDOT Standard BD-620M (5)), conventional IDOT cross-frame details require lateral bracing to be bolted to WT-shape support brackets which are bolted to the bottom of the web near the bottom flange. Since the already fabricated cross-frames prohibited adding support brackets around the connection plates and girder capacity (both strength and fatigue) would be greatly reduced using a bolted flange connection, permanent lateral bracing was also eliminated, leaving temporary lateral bracing as the only viable option.

Temporary lateral bracing consisted of pairs of rebar welded to both sides of ¾-in plates which were then welded to top flanges in an X-pattern (see Figure 11). This limited lateral deflections to 6-in under wind. Bracing width was limited to the three girder bays between Girders 3 and 6 (19'-9") since the cross-slope, with the crown above Girder 3, prevented utilizing all five girder bays between Girders 1 and 6 (32'-11"). The most critical aspect however was ensuring that the live load stress range ($\Delta\sigma LL$) in the top flange did not exceed the allowable stress range (Fsr) of 4.5-ksi (Category E) per AASHTO LFD Table 10.3.1A (1). For bracing cast in positive moment deck pours, max $\Delta\sigma LL$ = 1.9-ksi < Fsr = 4.5-ksi (E) which was okay. For bracing welded to the HPS 70W steel over the piers, max $\Delta\sigma LL$ = 2.8-ksi < Fsr = 4.5-ksi (E) such that removal was optional. For bracing in moment transition regions, max $\Delta\sigma LL$ = 5.9-ksi > Fsr = 4.5-ksi (E) such that bracing had to be removed by grinding off top flange welds after positive moment pours had set.

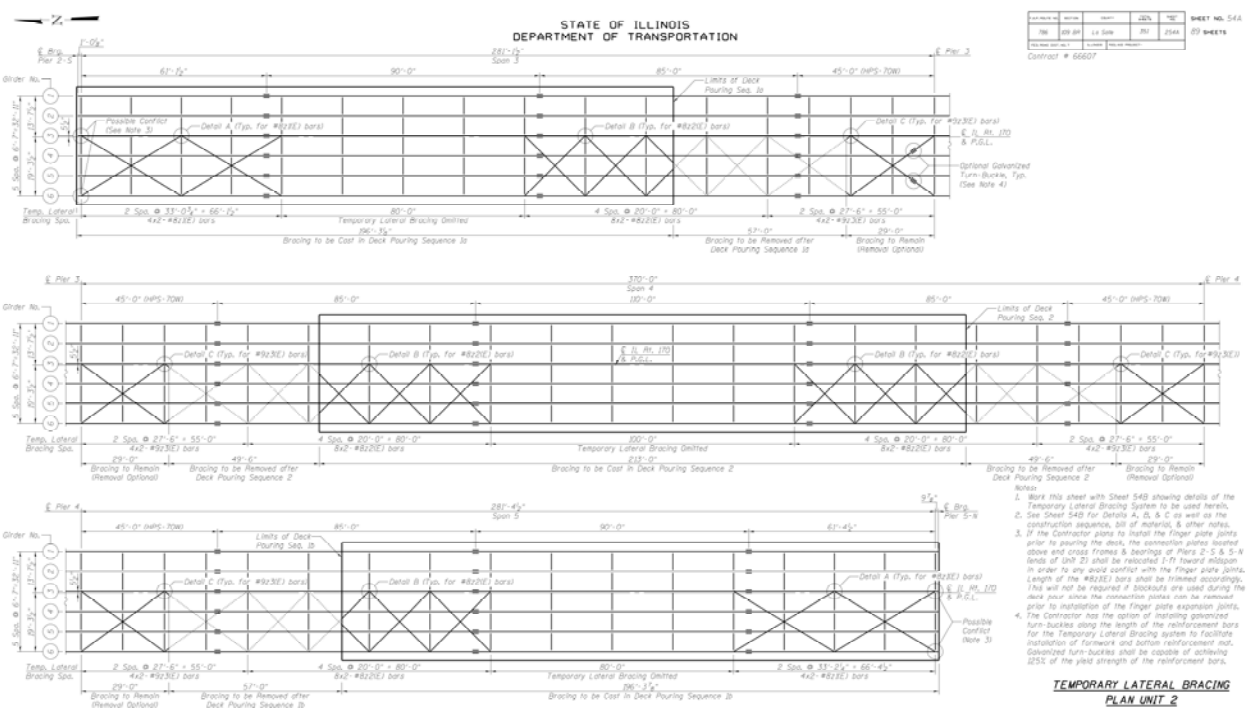


**Figure 11: Temporary Lateral Bracing for 3-Spans of Unit 2**

While most engineers believe that they are responsible only for final design, IDOT's interpretation is that the Engineer becomes responsible after the steel is erected (not just after the concrete deck has hardened). After some debate about using the same temporary lateral bracing for both girder erection and deck pours, IDOT decided to divide the responsibility according to their interpretation above such that the Contractor would be responsible during the erection, while the Engineer would be responsible during the deck pours.

The Contractor's erection scheme was limited by the fact that no permanent lateral bracing was provided, and the USCG required that the navigational channel not be closed to barge traffic for more than two days which prevented the use of typical external bracing systems such as temporary shoring towers in the river. In order to overcome these limitations, the Contractor utilized an unconventional external bracing system that used the adjacent, existing IL-170 truss bridge to laterally brace the new girders at they were erected.

Although bracing to adjacent structures is not a new concept, it is seldom utilized for a number of reasons. Fasick et.al. (3) warns that this bracing method often does not provide true lateral stability and can induce lateral deflections in the erected girders, when either the existing or proposed structure deflects vertically. However, the Contractor resolved the first issue by bracing both the top and bottom flanges of the girders and resolved the second one by providing hinged connections at the existing truss to permit large rotation, threaded connections at erected girder flanges to permit pipe lengths to be adjusted for lateral deflections, and notes instructing the Contractor to allow the girders to deflect for self-weight before attaching braces. External bracing consisted of two 8-in pipes at the midspan and quarter-spans of the three spans in Unit 2. Upper bracing was removed prior to erection of the 110-ft gate segments at midspan, while lower bracing was removed prior to the first deck pour due to the large deflections at those stages (see Figures 12 & 13).

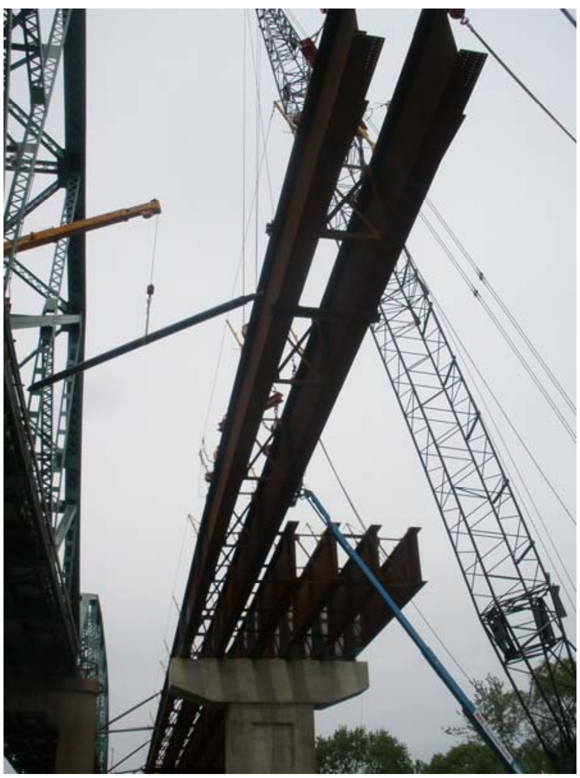

**Figure 12: Cantilevered Erection of 2-Girders with External Pipe Bracing to Existing Truss**

**Figure 13: Drop-in Erection of Gate Segment with External Pipe Bracing to Existing Truss**

# Conclusions & Recommendations

The central challenge on this project was designing a long-span bridge to accommodate limited geometry such as narrow deck width and shallow girder depth (relative to span length) as a result of decisions made to minimize the cost of the structure and maintain the vertical profile after increases in vertical clearance. As a result of pushing the envelope with both the span-to-width and span-to-depth ratios on this long-span structure coupled with support limitations in the river during construction, unique solutions were required to ensure global lateral and torsional stability of the structure during the erection and pouring of the decks. In the end, measures taken to improve the stability of the structure prior to hardening of the deck were put to the test as multiple storms with tornadic winds battered the structure during the structural steel erection. Based on this experience, recommendations are made to preclude the need for extra lateral bracing during construction for long-span plate girders, especially those with large span-to-width or span-to-depth ratios.

The first recommendation for long-span plate girders is choosing geometry that increases global stability. According to Yura (6), Yura & Widianto (7), and Yura et.al. (8) global stability can be increased by either increasing the distance between exterior girders or increasing the girder's strong-axis moment-of- inertia. While the roadway engineer and/or client will usually dictate the number of lanes and width of shoulders, the structural engineer can advocate for as wide a bridge as possible. In this particular case, it would have been possible to have a 4-lane structure using the same minimum number of girders required by the client. As for increasing girder size, this should be accompanied by fewer girders to keep the design economical. In most states with number of girders governed by AASHTO LRFD 4.6.2.2.1 (2) this will not be an issue. Therefore, consider using the minimum number of girders allowed by the client or discuss the advantages of utilizing fewer girders, with larger moments-of-inertia, with clients that require more than four girders.

The second recommendation for long-span plate girders is choosing bracing that decreases lateral bending stresses and deflections in the top flange during construction, even if not necessary for the final condition. In addition to adding top flange struts to standard DOT cross-frames for deep girders per Fasick et.al. (3), consider adding lateral bracing to either top or bottom flange, especially if shoring towers cannot be used. According to IDOT Bridge Manual 3.3.19 (4), the need for lateral bracing shall be investigated according to AASHTO LFD 10.21 (1) or AASHTO LRFD 6.7.5 (2). While the LRFD code suggests lateral bracing be considered for spans greater than 200-ft, the LFD code (applicable at the time) provides little guidance. More prescriptive guidelines are given by other DOT's. According to PennDOT Standard BD-620M (5), lateral bracing should be avoided for spans less than 200-ft but is mandatory for spans greater than 300-ft. Therefore, consider using lateral bracing in spans over 200-ft, unless cost or fatigue issues prohibit its use.

The third recommendation for long-span plate girders is treating them as complex bridge superstructures, similar to those that are curved or highly skewed. Thus, long-span plate girders should be analyzed using either a 2D grid analysis or 3D finite element analysis (FEA) rather than just a simple line-girder analysis. According to IDOT Bridge Manual 3.3.9 (4), girders are considered curved when the arc span divided by the girder radius is more than 0.06-rad (3.44-deg). Likewise, according to IDOT Bridge Manual 3.3.5 (4), girders are considered highly skewed when the skew is more than 45-deg. Although IDOT has no criteria for distinguishing long-span plate girders, they may consider them as girders with spans more than 250-ft, since that is the span at which they require cambers and screeds to be adjusted for deck pouring sequence. According to this definition, the 370-ft span used here would definitely be qualified as a long-span girder and should have used a more sophisticated girder analysis to capture some of the global instability effects.

Another suggestion for the analysis of long-span plate girders is applying AASHTO LRFD 4.6.2.2.2d (2), which computes a distribution factor (DF) for exterior girders assuming the bridge rotates as a rigid body. According to IDOT Bridge Manual 3.3.1 (4), this procedure gives unrealistic DF's and should not be used although it is one way to reasonably increase girder stiffness for global stability and is thus recommended. One final suggestion is not using HPS 70W steel when span-to-width and/or span-to-depth ratios are large or when live load deflections govern since additional stiffness (not strength) is needed for these situations. All the above recommendations are especially important if shoring towers cannot be used during erection.

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| | |
|---|---|
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| Geotechnical Sub-consultant: | Wang Engineering |
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| Structural Steel Fabricator: | Industrial Steel Construction, Inc. |
| Contractor's Structural Engr: | WHKS & Company |
| Demolition Sub-contractor: | Spirtas Wrecking Co. & Dykon Explosive Demolition |


## References

(1) AASHTO (2006). *Standard Specifications for Highway Bridges, 17th Edition.* American Association of State Highway & Transportation Officials, Washington, DC.

(2) AASHTO (2007). *AASHTO LRFD Bridge Design Specifications, 4th Edition.* American Association of State Highway & Transportation Officials, Washington, DC.

(3) Fasick, J. G., Leech, T.G., and Carnevale, P.F. (2005). "The New James Rumsey Bridge: Long-Span Steel Girder Bridge Erection over Potomac River." *Proc. of the World Steel Bridge Symposium, 2005, Orlando, FL.* American Institute of Steel Construction, National Steel Bridge Alliance, Chicago, IL.

(4) IDOT (2006). *Bridge Manual.* Illinois Department of Transportation, Bureau of Bridges & Structures, Springfield, IL.

(5) PennDOT (2010). *Bridge Design Standard BD-620M: Steel Girder Bridges Lateral Bracing Criteria.* Pennsylvania Department of Transportation, Bureau of Design, Harrisburg, PA.

(6) Yura, J. A. (2001). "Fundamentals of Beam Bracing." *Engineering Journal, First Quarter, 2001, pp. 11-26.* American Institute of Steel Construction, Chicago, IL.

(7) Yura, J. A. and Widianto (2005). "Lateral Buckling & Bracing of Beams: A Re-Evaluation after the Marcy Bridge Collapse." *Proc. of the Structural Stability Research Council, 2005, Montreal, QC, pp.277-294.* Structural Stability Research Council, Rolla, MO.

(8) Yura, J., Helwig, T., Herman, R., and Zhou, C. (2008). "Global Lateral Buckling of I-Shaped Girder Systems." *Journal of Structural Engineering, September 2008, pp. 1-8.* American Society of Civil Engineers, Reston, VA.