U.S. TRUNK
HIGHWAY 53
GIRDER DESIGN

ROBERT A. MAGLIOLA,
PE, SE

BIOGRAPHY
Bob Magliola PE, SE, Vice President, manages Parsons’ Chicago structural engineering practice. He received his BSCE and MSCE degrees from Purdue University, and has thirty-five years of experience in structural bridge design and project management.

Bob is a Parsons Certified Project Manager and is experienced in managing large multidiscipline staffs on combined bridge and road transportation projects. He was the project manager for the $80M US 52 tied-arch bridge over the Mississippi River at Savanna, IL, and he was the project manager for the $170M US 20 tied-arch bridge over the Mississippi River at Dubuque, IA.

Bob is a designated subject matter expert for steel bridges within Parsons. Bob was engineer of record for the superstructure design for the 480-foot main span steel I-girder Highway 53 Bridge over Roucheau mine and the 365-foot main span steel tub girder Highway 52 Bridge over the Mississippi River.

SUMMARY
Parsons provided the Minnesota Department of Transportation (MnDOT) the design for a new bridge and roadway between the iron mining cities of Virginia and Eveleth, MN. The existing Highway 53 was on a 1960 easement that expired in 2017 necessitating MnDOT to relocate the highway to allow mining operations to continue.

To meet the project deadline the bridge was designed and built under an accelerated schedule utilizing the Construction Manager/General Contractor (CMGC) delivery method.

Parsons is the Engineer of record for the new 1,130-foot three span steel plate girder bridge, with a 480 ft long main span, 180 feet above the floor of the Roucheau mine.

Overall project services include project management, preliminary and final bridge design and plans, shop drawing review, bridge load rating and roadway design services for the relocation of Highway 53.
U.S. TRUNK HIGHWAY 53 GIRDERS DESIGN

Project Overview

Parsons provided the Minnesota Department of Transportation (MnDOT) the design for a new bridge and roadway between the iron mining cities of Virginia and Eveleth, MN. The existing Highway 53 was on a 1960 easement that expired in 2017 necessitating MnDOT to relocate the highway to allow mining operations to continue. Overall project services included project management, preliminary and final bridge design and plans, shop drawing review, bridge load rating and roadway design services. To meet the project deadline the bridge was designed and built under an accelerated schedule utilizing the Construction Manager/General Contractor (CMGC) delivery method.

Parsons was the Engineer of Record for the new 1,125-foot three span steel plate girder bridge, with a 480 ft long main span, 180 feet above the floor of the Rouchleau mine. Project challenges included an accelerated design schedule to accommodate the easement agreement, near-vertical rock face walls, mine waste rubble to depths of 120 feet below the pit floor, and a lake in the mine which serves as the drinking water supply for Virginia, MN, Fig. 1.

Figure 1: Highway 53 Bridge Elevation

Beginning in late February 2015, Parsons worked collaboratively with MnDOT and the CMGC to validate the structure type and assess design decisions for potential snags which could affect delivery schedule. While many design decisions have both pro and con benefits, MnDOT’s overriding priority was speed of construction and reducing construction risk, which as a side benefit often resulted in the least cost. The design also had to be cognizant of the contractor’s desire for year-round construction through the northern Minnesota winter and the challenges of constructing in a 200-foot deep open-pit mine.

Parsons integrated best-industry practices by including Tensor Engineering as a design team member to provide draft shop drawings as part of the bid package to minimize bid risks, facilitate mill orders, and ultimately expedite fabrication. Within 52 days of notice-to-proceed, Parsons delivered the complete plans for the 5,000 tons of superstructure steel.

MnDOT also signed contracts with Kiewit Corp. to be CMGC, with HNTB to provide an independent design check, and an independent cost estimator (ICE). Parsons hired Dan Brown and Assoc. for geotechnical and deep-foundation engineering.

The CMGC services that Kiewit provided gave assurance to all that Parsons’ design decisions were practical and constructible by a range of contractors. Nothing in the design could be dependent on a contractor’s unique capabilities that would limit competition come bid time. Kiewit also provided ongoing cost estimating, construction risk analysis and construction schedule impact analysis as the design progressed.

A feature of the CMGC delivery method was that Parsons, MnDOT and Kiewit were collocated for the
design phase. The staff from the three organizations came together and worked as a team without walls. This facilitated the rapid pace of design where MnDOT provided continual over-the-shoulder reviews of Parsons’ design and Kiewit provided their construction expertise to guide the design to facilitate the most expedient construction method.

The magnitude of the spans and difficult construction conditions required careful evaluation of seemingly simple decisions that had to made before design could begin. These included:

- Girders' depth
- Girders' piece length and weight
- Steel grade
- Web depth taper rate
- Web stiffeners
- Superstructure cross section
- Analysis methodology

Some of these decisions affected cost, others impacted fabrication, and erection schedules. With the CMGC process in place, Parsons, Kiewit, and MnDOT evaluated and rapidly made these critical early decisions. This paper discusses the factors that figured into making those decisions.

Note, a discussion of the overall project been published by Huston [1]. The design for the piers and substructure is not discussed in this paper. This has been discussed by Graham. [2].

Girders’ Depth

Girder web depth was evaluated for fabrication and shipping and MnDOT’s preference to avoid a longitudinal web splice if possible. Based on a survey of freight haulers, a girder with a 14'-6” web depth could be shipped laying down without undue constraints. Fabricators were also asked for their shop capabilities to handle deep girders, without resorting to a longitudinal splice. The fabrication shop capabilities as of 2015 are given in Figure 2. From these surveys and since a preliminary girder design showed that a girder with a 14'-6” web depth was workable for both strength and deflection cases, the web depth at the piers was set to 14'-6”.

<table>
<thead>
<tr>
<th>Fabricator</th>
<th>Girder Depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>ISC</td>
<td>25 ft. max.</td>
</tr>
<tr>
<td>Veritas</td>
<td>20.5 ft. max.</td>
</tr>
<tr>
<td>Canam Group</td>
<td>14.5 ft. web is within capability *</td>
</tr>
<tr>
<td>Stupp</td>
<td>14.5 ft. max.</td>
</tr>
<tr>
<td>AFCO</td>
<td>14.5 ft. web is within capability *</td>
</tr>
</tbody>
</table>

*Not the limit on maximum girder depth.

Figure 2: Girder Depth Capability of Fabricators

Girders’ Piece Length

The freight haulers and fabricators were also surveyed to set the preferred maximum girder piece length. The fabricator responses to the survey are given below.

ISC. Girders 170-feet long can be fabricated.

Veritas. 135 feet is a common maximum length and should pose no issues. Optimal length for shipping is 120 feet without premiums. We have fabricated up to 150 feet without difficulty.

Canam Group. Girders 145-feet long can be fabricated.

Stupp. 135 feet is fine. We prefer pieces around 118-feet long because those travel the easiest through our plant; however, we can fabricate and ship girders around 165-feet long but those are expensive. I’d say that when the pieces get around 140 feet is when the transportation costs really jump.

AFCO. 135 feet is a manageable length for fabrication provided the cross-section has sufficient stiffness for handling. Somewhere between 115 feet to 125 feet might be slightly more economical to ship. We have built and shipped as long as 160 feet. While that length is not particularly bad, it is pushing the practical limit.

Alberici. Recommend against going longer than 135-feet.
De Long’s Inc. 135 feet is good for a maximum length. Longer pieces can be fabricated though.

135 feet was set for the maximum piece length, which was a confirmation of already existing typical norms.

Girder Piece Weight

The maximum girder piece weight was the segment over the pier. This weighed in at 96 tons. The maximum piece weight was also set by a survey of fabricators and freight haulers. The fabricator responses are based on capacity to handle material in the shop. Survey responses are given in Figure 3. Responses to the survey from freight haulers varied widely due to differences in permit requirements from state to state. In general, 20 tons can be readily hauled with a normal truck and is considered a routine load. Specialized transport equipment is required at 50 tons and above. At 75 tons and above special permits are required.

The investigation of practical maximums for girder depth, length, and weight led to the following parameters being set as maximum values that must be adhered to by the design.

- Depth: 15’-2” total depth (web + flanges)
- Length: 135 feet maximum
- Weight: 100 ton maximum

The greatest single pieces to fabricate and ship were the pier segments. These were 15’-2” deep, 120-foot long and 96 tons. Figure 4 shows delivery of one of these pier segments

<table>
<thead>
<tr>
<th>Fabricator</th>
<th>Girder Segment Wt.</th>
</tr>
</thead>
<tbody>
<tr>
<td>ISC</td>
<td>200-ton crane limit.</td>
</tr>
<tr>
<td>Veritas</td>
<td>100-ton crane limit.</td>
</tr>
<tr>
<td>Canam Group</td>
<td>100 tons</td>
</tr>
<tr>
<td>Stupp</td>
<td>120-ton crane limit.</td>
</tr>
<tr>
<td>AFCO</td>
<td>100 tons</td>
</tr>
<tr>
<td>De Long’s Inc</td>
<td>Prefer &lt; 50 tons*</td>
</tr>
<tr>
<td>Alberici</td>
<td>Prefer &lt; 60 tons*</td>
</tr>
</tbody>
</table>

*Not necessarily the maximum that can be fabricated.

Figure 3: Segment Wt. Capability of Fabricators

Figure 4: Delivery of Pier Segment
Steel Grade

Weathering steel is MnDOT’s preferred steel type, but the selection of 50W versus HPS 70W grade is evaluated on a case-by-case basis. During 2015, the year design was underway, 50W and HPS70W was available in the thicknesses given in Figures 5 and 6 from the listed plate mills.

<table>
<thead>
<tr>
<th>Steel Grade</th>
<th>2015</th>
<th>2019</th>
</tr>
</thead>
<tbody>
<tr>
<td>A709 50W</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arcelor Mital</td>
<td>4”</td>
<td>4”</td>
</tr>
<tr>
<td>Nucor</td>
<td>2.5”</td>
<td>4”</td>
</tr>
<tr>
<td>SSAB</td>
<td>3”</td>
<td>3”</td>
</tr>
</tbody>
</table>

*Figure 5: Thickness Availability of 50W*

The availability of HPS 70W above two-inch thickness from only one mill was considered a price and schedule risk. Live load deflection also factored into steel grade selection. The maximum 14’-6” web depth is relatively shallow for a 480-foot long span. Thus, there was disincentive to reduce flange thickness, which would result in a more flexible girder. Cost was also considered. Using HPS 70W in select tension flanges reduced total structural steel weight by two percent, but reduced steel cost by only one-half percent. Given these three factors, grade 50W steel was selected for all girder flanges and web. Figures 5 and 6 also give thickness availability by grade and mill as of 2019. With two mills now producing HPS 70W up to four-inches thick, the decision to exclude HPS 70W from the design may have turned out differently were the investigation being performed today.

Web Taper

The girder is haunched at the piers. Web depth at the pier is 14’-6”. This tapers to 9’-4” in the first and second spans. In the third span the web depth is 7’-9”. A linear taper was chosen versus a more rapid 2nd-order (a.k.a. parabolic taper) to aid in meeting live load deflection criteria. Note, negative moment rapidly drops off and if one’s design is only constrained by strength; a 2nd-order taper would be workable. Using the linear taper, the girder was somewhat stiffer which aided to reduce live load deflection.

Longitudinally Stiffened Web

Both longitudinally stiffened and unstiffened webs were investigated for the girder segment over the pier. Unstiffened, the 14’-6” web required a 1-3/16-inch thickness to satisfy the D/tw < 150 requirement. (AASHTO 6.10.2.1.1-1/3). With a longitudinal stiffener, the proportion limit increases to 300. In final design, a 3/4-inch thick web with longitudinal stiffener was selected. The reasoning was twofold. First the stiffened web saved 500,000 pounds of steel and an estimated $325,000 after deducting the cost for the longitudinal stiffener. Given that this is only 5% of the total girder weight this alone is not an overly compelling argument for longitudinally stiffening the web. The second reason for going with the longitudinal stiffener was the reduction in the pier-segment weight. Stiffened, the pier segment weighs 96 tons, which is below the 100-ton target piece weight. Unstiffened the pier segment weighs 112 tons due to the extra web thickness.

Other design alternatives do exist and were evaluated. A 1-3/16-inch web thickness could have been used and the pier segment length could have been reduced from 120 feet to 100 feet. Shortening the segment length by 20 feet would get the unstiffened pier segment to under 100 tons. But the girder field splices would be moving to a higher moment-demand location. In this instance the benefits of a longitudinally stiffened web slightly outweighed the unstiffened web.

When evaluating the cost of installing a longitudinal stiffener, one must include the cost of interrupting each of the cross-frame connection plates. (Longitudinal stiffeners would be located on the
opposite side of the web from the shear stiffeners.) To improve fatigue performance, the longitudinal stiffener is run continuous and the transverse cross frame connection plates interrupted. The longitudinal stiffener is connected to the web with a fillet weld but is terminated with a CJP groove weld and radiused as shown in Figure 7 [4]. The longitudinal stiffener does not need to be continuous across web splice plates or at supports with multiple bearing stiffeners.

**Live Load Analysis**

There are three underlying facets to calculations for live load shears, moments and deflections.

1. Notional live load model
2. Load distribution
3. Owner specified load cases

**Notional Live Load Model**

AASHTO’s notional live load model is everything that is included in AASHTO 3.6 Live Loads [3] and AASHTO 3.4 Load Factors and Combinations [3]. These code sections have several nuances and require careful reading for the notional live load model to be correctly applied. Particularly, the load application rules for computing positive-moment-region girder forces, negative-moment-region girder forces, and deflections are all different. Dynamic load allowance and multiple presence factors for shear and moment are also different than for fatigue.

AASHTO’s notional live load model along with all of its nuances was developed to envelope the structural response due to the wide variety of actual truck loads on the highway system and calibrated against weigh-in-motion data. While AASHTO’s notional live load was not initially developed with long-span bridges in mind, Nowak [5] has shown that the notional model is equally applicable to long span bridges.

**Load Distribution**

For straight girder bridges with normal supports or supports that are only moderately skewed, the engineer can opt to use the approximate methods of analysis allowed by AASHTO 4.6.2 [3] or perform a refined analysis per AASHTO 4.6.3 [3]. For the Highway 53 bridge, Parsons used a line girder analysis to enable quick cycling of the design. Parsons’ checked the final design with a LARSA finite element model (FEM). HNTB provided an independent check of the final design and used a CSiBridge FEM.

In both company’s FEM models for determining vertical beam shears and moments, the interior cross frames were omitted. This allowed interior cross frames to be designed as secondary members, i.e. cross frames were not designed for gravity load. Lateral load distribution in the FEM was afforded only by the bridge deck.

**Dead Loads.** In the line-girder model the weights of deck-slab, cross frames and lateral bracing are equally distributed laterally to all girders and applied to the non-composite steel section. The weight of wearing surface and barriers was also equally distributed laterally to all girders and applied to the long-term composite section. All three models (the line girder and two FEMs) yielded results that were in close agreement.

**Live Loads.** AASHTO 4.6.2 [3] includes a series of formulas that may be used to estimate the lateral distribution of live load to girders. This eliminates the need to perform a FEM analysis to determine this distribution. The live load distribution factors (LLDF) are different for interior girders, exterior girders, and moment, shear and deflection. The LLDF formula for moment in an interior girder is given in Figure 8.

![Figure 7: Longitudinal Stiffener Termination [4]](image-url)
0.075 + \left( \frac{S}{9.5} \right)^{0.6} \left( \frac{S}{L} \right)^{0.2} \left( \frac{K_s}{12.0 L t_i^3} \right)^{0.1}

**Figure 8:** LLDF for Interior Girder Moment [4]

Yet this AASHTO formula is limited in its range of applicability by the constraints given in Figure 9.

\begin{align*}
3.5 & \leq S \leq 16.0 \\
4.5 & \leq t_s \leq 12.0 \\
20 & \leq L \leq 240 \\
N_b & \geq 4 \\
10,000 & \leq K_s \leq 7,000,000
\end{align*}

**Figure 9:** Constraints on Range of Applicability [4]

The TH 53 bridge exceeds both the upper limit on L (span length) and $K_s$ (longitudinal stiffness parameter). Span lengths are 375, 480 and 270 and $K_s$ ranges from 16,000,000 at midspans to 65,000,000 at piers. Thus, LLDFs were determined by FEM. With the LLDFs determined, Parsons was able to use line girder analysis to quickly cycle through analysis/design iterations. HNTB’s independent check only needed to evaluate the final girder design. Thus, HNTB extracted live load envelopes directly from their FEM analysis.

It is informative however to compare the FEM-determined LLDFs against AASHTO LLDFs calculated using the structure’s span lengths and actual $K_s$ values even though they exceeded the upper limits permitted by the AASHTO equations.

**Table 1:** LLDFs for Highway 53 Bridge

<table>
<thead>
<tr>
<th>Span/ Pier</th>
<th>Two Plus Lanes</th>
<th>Fatigue</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>FEM</td>
<td>AASHTO</td>
</tr>
<tr>
<td><strong>Moment LLDF, Positive Moment Region</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Span 1</td>
<td>0.782</td>
<td>0.788</td>
</tr>
<tr>
<td>Span 2</td>
<td>0.770</td>
<td>0.737</td>
</tr>
<tr>
<td>Span 3</td>
<td>0.776</td>
<td>0.787</td>
</tr>
<tr>
<td><strong>Moment LLDF, Negative Moment Region</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pier 1</td>
<td>0.852</td>
<td>0.875</td>
</tr>
<tr>
<td>Pier 2</td>
<td>0.857</td>
<td>0.861</td>
</tr>
<tr>
<td><strong>Shear LLDF</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pier 1</td>
<td>0.917</td>
<td>0.918</td>
</tr>
<tr>
<td>Pier 2</td>
<td>0.933</td>
<td>0.918</td>
</tr>
</tbody>
</table>

* Calculated by lever rule

**Figure 10:** Exterior Girder LLDFs

Exterior girder LLDFs are given in Figure 10 and Interior girder LLDFs are given in Figure 11. In the figures, the “Two Plus Lanes” columns compare LLDFs due to two or more loaded lanes adjusted for multiple presence factor. The “Fatigue” columns consider only one loaded lane and uses a multiple presence factor of 1.0.

It is noted that the distribution factors computed by the FEM analysis and AASHTO equations are generally in good agreement, especially since the AASHTO equations out of necessity are enveloping a wide range of structure configurations. The inference from this one comparison is that the AASHTO equations are applicable beyond their constraint limits. The worst alignment between the FEM and AASHTO LLDFs are the fatigue (i.e. one-lane) factors for exterior girders. The AASHTO procedure for exterior girders loaded by one lane is to use the lever rule, which is independent of span length. Span length independence is counterintuitive because as the aspect ratio of span to bridge width increases, one would expect the distributed live load to spread out to more girders and the LLDF to decrease.

Nevertheless, the fatigue LLDFs were practically a moot point for the Highway 53 bridge. Due to the high dead load to live load ratio, the fatigue load cases were far from governing.
Owner-Specified Load Cases
MnDOT [6] has two owner-specified load cases in addition to the AASHTO load cases.

MnDOT Load Case STR IV
MnDOT and AASHTO strength IV load factors are given by the following two equations.

\[
\text{MnDOT STR IV} = 1.40DC + 1.50DW + 1.45LL \\
\text{AASHTO STR IV} = 1.50DC + 1.50DW + 0.0LL
\]

The MnDOT STR IV factors are the result of research performed for the AASHTO T-5 committee by Modjeski and Masters. The load case was voted on by the T-5 committee to replace the current AASHTO STR IV, but it did not pass. A discussion on this is provided in AASHTO C3.4.1 [3]. MnDOT, however, did adopt the Modjeski and Masters equation for their own practice.

For the Highway 53 Bridge, MnDOT STR IV does govern the design by about 3% versus AASHTO STR I in high negative moment regions. In the negative moment regions the total unfactored dead load moments were about 3.3 times the total unfactored live load moment. AASHTO STR IV did not govern.

MnDOT Negative Moment Load Case
AASHTO 3.6.1.3 [3] applies 90% load of two trucks and 90% of the lane load for calculating negative live load moments between points of contraflexure. Whereas MnDOT’s practice is to apply 110% of these loads for long-span bridges. MnDOT adopted the increased live loading for negative moments because when switching to LRFD, MnDOT determined that relatively long span bridges designed using the LRFD load model did not rate for standard Minnesota permit loads.

Multi-Girder Framing
Two framing systems were investigated. The first being deep girders (DG) spaced at 24 feet with intermediate sub-stringers, Figure 12. The second being parallel girders (PG) spaced at 12 feet, Figure 13. The DG design had a maximum web depth in the range of 20 ft to 24 ft. The PG design had a maximum web depth of 14’-6”.

Figure 12: Framing of Deep Girders with Intermediate Sub-Stringers

Figure 13: Framing of Multiple Parallel Girders

Comparing the two framing options, the DG design had several notable strikes against it:

- The need for a longitudinal web splice was disliked by MnDOT.
- The deep section was harder to handle in the shop.
- The cross frames were gravity-load carrying members which required larger bolted connections.
- Only ISC and Veritas indicated they would submit bids for the DG design.
- Maintenance inspections would be more difficult.
- Erection was more difficult.
- Except for a potential steel weight savings, there were no other positives for selecting the DG framing over the PG framing.

The two framing option were evaluated by Parsons, Kiewit and MnDOT and all came to the same conclusion to use the PG framing option.
Conclusions

The Highway 53 Bridge had a major span, a difficult construction site and need for an accelerated construction schedule. MnDOT procured the design through a CMGC process with Parsons as the designer and Kiewit as CMGC. The collaborative design process provided by the CMGC was key to rapidly making key design decisions for:

- Girder depth
- Girder piece length and weight
- Steel grade
- Web depth taper rate
- Web stiffeners
- Superstructure cross section
- Analysis methodology

The benefit was that the project was able to evaluate the pros and cons of each of these decisions from the view of the designer, contractor and owner. And allowed design to maintain a rapid design pace that delivered the steel design in only 52 days from concept to final plans.

*Figure 14: Highway 53 Bridge Under Construction, 2016*

References

6. *LRFD Bridge Design Manual*, Minnesota Department of Transportation, 2019