DESIGNING THE RIGHT BRIDGE:

SIMPLE PRINCIPLES LEAD TO ECONOMICAL BRIDGE DESIGNS

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BIOGRAPHY

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He is active in the Pittsburgh Section of ASCE, where he has served on the structures group and the Board of Directors. He is also a member of ASHE; ABCD; and the Executive Committee for the International Bridge Conference, which is sponsored annually by the Engineers' Society of Western Pennsylvania. He has presented at numerous conferences including the International Bridge Conference, the Western Bridge Engineers' Seminar, the FHWA Steel Bridge Conference and the Ohio Transportation Engineering Conference.

SUMMARY

There is both science and art in bridge design. As computer-related tools have improved, the art of bridge design is in danger of being buried under a sea of numbers. Bridge engineers need to re-focus on basic design decisions and to reconsider the current applicability of some common rules of thumb.

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INTRODUCTION

Economical plate girder design combines both science and art. The "*science*" of steel girder bridge design has not changed significantly in the recent past. The basic design equation is still $\sigma = Mc/I$. However, many misconceptions exist in the bridge design community about steel girder design. These misconceptions result from changes in the steel fabrication industry, advances in materials, lessons learned from undesirable details used over the years, and some common assumptions about economical steel design. It is critical for bridge designers to stay abreast of the latest technology, but even more important is the need to adjust perceptions about economical steel design as industry changes occur. Communication of these changes between designers, fabricators and erectors has historically been poor to non-existent. The "*art*" of girder design occurs when the designer begins to process and prioritize the decisions that must be made regarding various detail issues discussed herein.

Over the past 50 years, prestressed concrete bridges have eroded the market share of steel bridges in the shorter span ranges. This gradual progression has resulted in a shift in perceptions regarding the competitiveness of steel in the shorter span ranges. As a result, faulty assumptions are frequently made when steel structures are compared to concrete structures. Steel structures with much longer spans and fewer piers are often compared to short span concrete structures, generally with unfavorable results.

Span Arrangement – The Most Important Design Decision

The most important aspect of designing an economical steel bridge is to select the optimum span lengths. It has been demonstrated in the preliminary designs of numerous structures that shorter span steel structures in the 120' to 150' span range are often competitive with concrete structures using similar span lengths. This is especially true for unpainted weathering steel designs. In most span ranges where steel girders are feasible, shorter spans will be more cost-effective than longer spans unless the foundation conditions dictate expensive piers (such as river piers in deep water or extremely tall piers) or unless there is a hard constraint requiring long spans (such as maintaining a navigation clearance or spanning a wide freeway). Three projects are offered as evidence to support this thesis.

The Clifford Hollow Bridge in Hardy County, WV, illustrates this principle on a longer span structure than is common for most bridges. HDR designed both steel and segmental concrete alternates for a 1522' bridge carrying four lanes of traffic on the new Route 33 approximately 280' above the valley floor. Three plate girder options, a continuous deck truss and a deck arch were studied in the preliminary design phase. The truss and arch alternates were much more costly than any of the girder options, demonstrating that, without a hard constraint requiring the longer span lengths, shorter spans are more economical. Three composite plate girder options were studied: 4-, 5- and 6-span continuous girders. The piers for this bridge were expensive because of their extreme height (up to 260'). However, as the span lengths were increased, the weight of steel increased significantly to meet the greater demands. Thus, the increase in superstructure cost more than offset the decrease in substructure cost (approximately $500,000 per pier) as piers were eliminated. A comparison of the three composite plate girder options studied is shown in Table 1.

Table 1 – Clifford Hollow Bridge Cost

	4-Span Steel 335' – 2 @ 423' – 335'	5-Span Steel 264' – 3 @ 331' – 264'	6-Span Steel 210' – 4 @ 276' – 210'
Steel Weight (lb)	7,760,800	5,576,800	4,730,600
Superstructure Cost	$10,631,000	$ 8,202,300	$ 7,343,200
Substructure Cost	$ 2,967,400	$ 3,529,100	$ 3,992,000
Total Cost	$13,598,400	$11,731,400	$11,335,200

The bridge carrying SR 6220 approximately 90' above old SR 0220 in Centre County, PA, also demonstrates that increasing span lengths does not always lead to the most economical steel bridge. Both prestressed concrete and steel girder options were studied in preliminary design. The initial steel design studied was a balanced four-span continuous bridge, which was compared to a six-span concrete bridge made continuous for live load. These designs were very competitive, with only 2.5% separating them. A six-span steel bridge with span lengths very similar to those used for the concrete alternate was then studied and found to be cheaper than the 4-span steel alternate by 7.5%, as demonstrated in Table 2.

Table 2 – SR 6220 over SR 0220

	6-Span Concrete 2 @ 122' – 2 @ 126' – 2 @ 122'	4-Span Steel 161' – 201' – 210' – 168'	6-Span Steel 2 @ 114' – 2 @ 134' – 2 @ 122'
Superstructure Cost	$ 2,184,600	$ 2,528,700	$ 1,998,500
Substructure Cost	$ 1,802,100	$ 1,555,100	$ 1,802,100
Total Cost	$ 3,986,700	$ 4,083,800	$ 3,800,600

The costs shown in Tables 1 and 2 are based on preliminary design studies for those projects, and thus the relative accuracy of the cost estimates could be questioned. However, a Midwest DOT project provided bid evidence that can be extrapolated to demonstrate the competitiveness of shorter-span steel designs. Dual designs were bid on this project, and a bid analysis yielded interesting results. The concrete alternate was a five-span structure with spans of 75' – 3 @ 113' – 106'. The steel alternate was a three-span structure with spans of 162' – 205' – 162'. Table 3 summarizes the bid prices from six bidders and, on the surface, appears to support the assumption that concrete designs provide the most cost-effective bridges for short spans. However, a more careful analysis of the data leads to a different conclusion. Table 4 summarizes the superstructure bids, and Table 5 the substructure bids. As expected, the steel superstructure is more expensive than the concrete, and the substructure for the steel alternate is cheaper than that for the concrete alternate.

Table 3 – Total Bid Comparison

Rank	Bidder	Material	Bid Price	Index
1	A	Concrete	$ 5,005,700	1.000
2	B	Concrete	$ 5,040,200	1.007
3	C	Concrete	$ 5,182,500	1.035
4	***D***	***Steel***	***$ 5,323,100***	***1.063***
5	E	Concrete	$ 5,752,100	1.149
6	***F***	***Steel***	***$ 6,117,400***	***1.222***

Table 4 – Superstructure Cost

Rank	Bidder	Material	Bid Price	Index
1	B	Concrete	$ 2,998,800	1.000
2	A	Concrete	$ 2,999,500	1.001
3	C	Concrete	$ 3,090,900	1.031
4	E	Concrete	$ 3,222,000	1.074
5	***D***	***Steel***	***$ 3,742,300***	***1.248***
6	***F***	***Steel***	***$ 3,899,400***	***1.300***

Table 5 – Substructure Cost

Rank	Bidder	Material	Bid Price	Index
1	***D***	***Steel***	***$ 1,580,800***	***1.000***
2	A	Concrete	$ 2,006,200	1.269
3	B	Concrete	$ 2,041,400	1.291
4	C	Concrete	$ 2,091,700	1.323
5	***F***	***Steel***	***$ 2,217,900***	***1.403***
6	E	Concrete	$ 2,530,100	1.601

Since the DOT viewed this project as a litmus test to help determine whether steel bridges could be competitive for shorter span bridges, the steel industry was following the project closely. The NSBA performed a post mortem design study on this bridge after the bid results were made public. A review of the information from Tables 3, 4 and 5 led to the conclusion that, based on the actual bid prices, a short-span steel bridge using the same span arrangement as the concrete alternate reduced the quantity of steel by almost 1,200,000 pounds when compared to the as-designed steel alternate. Based on bid unit prices, the short-span steel alternate would have been at least $175,000 less expensive than the concrete alternate had it been taken to final design.

Girder Framing

Choosing the proper girder spacing is an important aspect in developing an economical design. The balance between girder spacing and overhang width can significantly impact economy. It is ideal to space the girders so that the moments are balanced between the interior and exterior girders, permitting duplicate designs that will result in the greatest fabrication economies. It has been shown that when a refined analysis is used, the moments are reasonably balanced when the overhang width is between 30 and 35 percent of the girder spacing.

In years past, many owners preferred girders spaced at 8 to 9 feet on centers to provide increased redundancy and to facilitate the use of removable deck forms. As composite girder design has allowed designers to account for the deck stiffness in their designs, and as the use of stay-in-place deck forms has become more common, wider girder spacings have gained greater acceptance in the bridge community. In general, girder spacings in the 11' to 14' range prove to be economical, although stay-in-place forms are available that span girder spacings as wide as 15'.

Crossframe spacing is another important aspect of developing an economical framing plan. The AASHTO Standard Specification suggests that a maximum crossframe spacing of 25' is appropriate, whereas the AASHTO LRFD code does not dictate a maximum crossframe spacing. Rather, the designer is directed to design to the crossframe spacing chosen. However, crossframe spacings significantly greater than 25' are not recommended due to the reduction in stability during erection and future redecking of the structure. Some designers are proponents of longer crossframe spacings and installing temporary crossframes to facilitate deck placement. Temporary crossframes are usually required to be removed after deck placement, which usually more than offsets the savings in crossframe material achieved by carrying only temporary loads in the frames.

While crossframe spacings between 30 and 35 feet may be achievable from the standpoint of stability during erection, the impact of wind on the overall framing system needs to be considered. Extending the crossframe spacings in excess of 30 feet needs to be considered carefully when assessing erection. Eliminating too many crossframes can have a detrimental effect on stability of the framing system during girder erection. While the normal design wind speed of 100 miles per hour is generally considered to be excessive during construction, a wind speed of approximately 70 mph provides a reasonable balance between cost and the risk of excessive lateral stresses and deflections during construction. It is likely that the web thickness may need to be increased in the positive moment regions to avoid web bend buckling during concrete placement.

For curved girders, it is generally advisable to decrease the crossframe spacing from the 25 foot maximum in order to reduce the lateral flange bending stresses. Crossframes for curved girders and skewed girders must resist much higher differential deflections than occur in tangent square bridges, and generally require heavier members. For girder radii less than 2000 feet, a maximum crossframe spacing of 20 feet is advisable. As the radius decreases, the crossframe spacing should also be reduced. This will keep the crossframe loads manageable and, more importantly, will help the erector maintain proper relative position between girders during erection. A practical minimum crossframe spacing has been about 12 feet, but only when the girder radius falls significantly below 500 feet.

Field splice location can also impact the economy of a girder design. It is ideal to locate the bolted field splices near the dead load inflection points for continuous girders. This will result in the smallest splice possible by code, minimizing the number of bolts in the splice, which in turn minimizes the erection time. However, when long-span girders are involved, it is often more important to locate the splices to limit the length and weight of shipping pieces. This saves more money in the long run than locating the splices at the inflection points.

Material Selection

Material selection is another critical aspect of economical girder design. The most commonly used bridge steel is ASTM A709, Grade 50 or 50W material. It is desirable to use unpainted weathering steel whenever possible. The National Steel Bridge Alliance (NSBA) publication “Uncoated Weathering Steel Bridges”, (Volume 1, Chapter 9 of the Highway Structures Design Handbook) summarizes the appropriate use of unpainted weathering steel in bridges. When unpainted bridges were first used, a savings of between five and ten cents per pound could be realized, based on the savings in painting cost, which was partially offset by the increase in material cost for weathering steel. Recently the material cost premium for weathering steel has fallen and the cost of painting has increased to the point that painted steel designs have been bid as much as 20 to 25 cents per pound higher than similar unpainted designs. The maintenance costs of unpainted bridges are also much lower than those for painted designs. Some agencies require future painting costs to be included for cost comparisons between painted steel bridges and concrete bridges, which further underscores the advantage of using unpainted weathering steel where appropriate.

High Performance Steel (HPS) was developed in the 1990s to improve the economy of steel as a construction material. HPS can be obtained in 50 and 70 ksi grades and exhibits fracture toughness that, at a minimum, exceeds the AASHTO Zone 3 requirements for Grade 70W. The corrosion resistance of HPS is also improved relative to that of A709 weathering steel by about 10%. The high strengths available in HPS allow constant depth plate girders to be used economically in long-span structures. Recent studies prepared jointly by HDR Engineering, Inc. and the University of Nebraska have shown that hybrid girders using HPS 70W steel can be economical when compared to homogeneous grade 50 designs. Further studies are currently underway to better assess the optimal methods of incorporating HPS into girder designs. The AASHTO LRFD Third Edition now permits the use of hybrid girders for curved girders. The use of hybrid curved girders was not permitted in the earlier Guide Specification.

Girder Proportions

Conventional wisdom has held that the lowest weight girder is the most economical girder. Given the change in the relative costs between labor and materials, this is not always true. The emphasis for economical design has shifted to developing simple, easily fabricated details. Simple details minimize the labor component of the fabrication, and they tend to perform well under repetitive loading, improving the serviceability of the structure.

Girder web design illustrates this approach. Years ago, it was desirable to design girders with fully stiffened webs (web designed as thin as possible while providing the transverse stiffeners necessary to develop the required shear capacity). Partially stiffened or unstiffened girder webs now tend to be more economical because the fabrication cost associated with additional transverse stiffeners exceeds the material cost for the thicker webs. Fewer stiffeners also reduce the number of fatigue-prone details, minimizing future inspection and maintenance efforts for the girders.

The use of longitudinal web stiffeners becomes a design consideration for long-span girders. Longitudinally stiffened girders do not become economical until the web depth exceeds 10 feet. Longitudinal stiffeners increase fabrication cost because of the details required at the stiffener termination points and at the intersection with the crossframe connection plates. Longitudinal stiffeners are typically attached to the opposite side of the web from the transverse stiffeners, which requires that the girders be turned over during fabrication, increasing the fabrication cost.

There are several helpful rules of thumb for sizing girder flanges. The first, suggested by NSBA, recommends that the minimum flange width in any field section be L/85 to permit reasonable shipping and handling of the girder piece. If L/85 is not met, maintaining girder stability during handling or erection may be difficult. When designing continuous girders, the flange plate widths in the negative moment regions are generally set somewhere between 2 and 8 inches wider than the adjacent positive moment flanges in order to limit plate thicknesses. For curved girders, it is generally advisable to use flanges 2 to 4 inches wider than would be used for tangent girders of the same span length. This additional width significantly increases the stiffness of the girders to resist lateral flange bending resulting from curvature.

Maintaining minimum flange thicknesses is also important to the overall stability of the girder. It is desirable to meet the basic flange b'/t ratios even though the AASHTO code permits larger b'/t ratios in areas of low stress, such as the top flange of a composite girder in the positive moment regions. The additional thickness increases the stiffness for shipping and handling of the section. The flanges will require less heat straightening to meet required tolerances for straightness (see Figures 1 and 2). The extra top flange material also reduces bending stresses, which helps avoid web bend-buckling during deck placement, prior to hardening of the concrete. As a general rule, flanges less than ¾" thick x 12" wide should not be used.

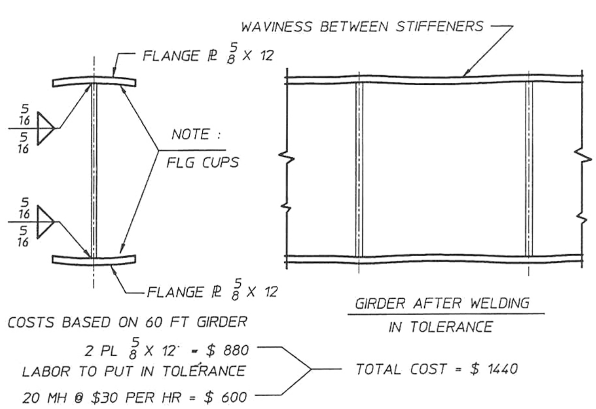

Fig. 1 – Distortion with Thin Flanges

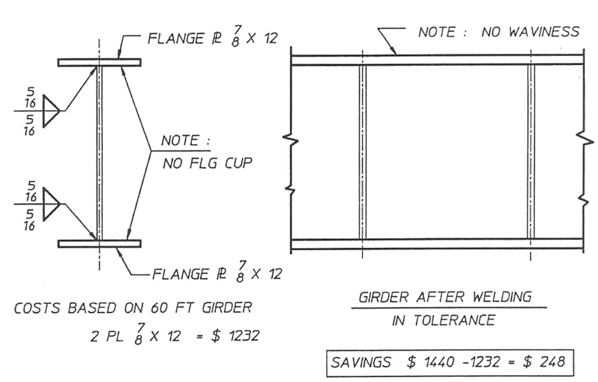

Fig. 2 – Distortion Eliminated

Using an appropriate number of welded flange transitions is critical to achieving an economical design. Determining the optimum number and location of flange splices has always been widely variable because each fabrication shop has its own criteria for the economy of welded flange transitions. However, AISC developed rules of thumb for determining economy of welded flange transitions that are relatively easy for designers to use. The rules of thumb, as noted below, were first published in the United States Steel Highway Structures Design Handbook, Volume 1. The equations compute a material weight savings required to justify the welded transition.

36 ksi material: Weight Savings = $\{300 + 25 \text{ x } A_{\text{smaller flange}}\}$

50 ksi material: Weight Savings = $0.85 \text{ x } \{300 + 25 \text{ x } A_{\text{smaller flange}}\}$

Experience has shown that the AISC equations are acceptably conservative. The AASHTO/NSBA Steel Bridge Collaboration has recently developed guidelines that are more conservative in an attempt to better reflect the current balance between material and labor costs for fabrication. The new guidelines are shown in G12.1-2003 "Guidelines for Design for Constructability", which is available in electronic form on the AASHTO/NSBA Steel Bridge Collaboration website (http://www.steelbridge.org/standards.htm).

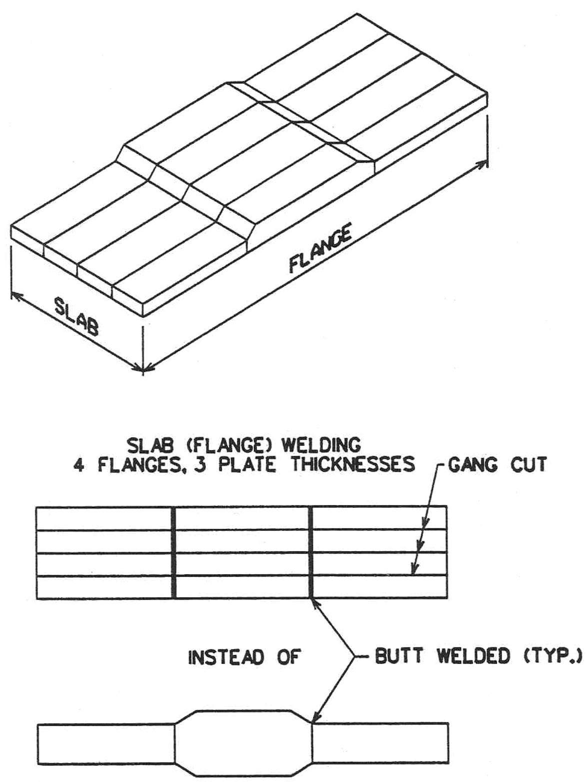

Fig. 3 – Flange Stripping Schematic

One guideline that we have adhered to at HDR is to limit the minimum size of the smaller flange at a welded transition to at least one-half the area of the larger flange. This limits the stress gradient in the girder web in the area of the transition. Experience has shown that if the flange area can be reduced by more than 50% at a welded transition, either the transition should be moved toward the area of peak moment until no more than a 50% reduction is justified, or an additional transition should be considered in the flange.

It is generally accepted that maintaining constant flange widths within each field section is ideal from the standpoint of fabrication. This permits wide plates to be butt spliced together, requiring runoff tabs only at the two edges of the plates. The plates can then be stripped to the desired width for each girder. Figure 3 illustrates this concept. An added advantage to constant width plates is the simplification of deck forming details.

Crossframe Type Selection

Selection of efficient crossframe details can impact the economy during both fabrication and erection of the bridge. Ideally, the aspect ratio of girder spacing to girder depth should dictate the crossframe configuration. The most efficient crossframe generally will have the diagonals at a 45 degree angle or steeper. When the aspect ratio is near unity, X-frames are the most efficient configuration. When the girder spacing exceeds the girder depth, K-frames tend to be more efficient. One-piece crossframes are usually more economical than "knocked down", or multiple piece, crossframes when all aspects of the fabrication and erection are considered. One-piece frames require less field bolting than knocked down frames, decreasing the erection

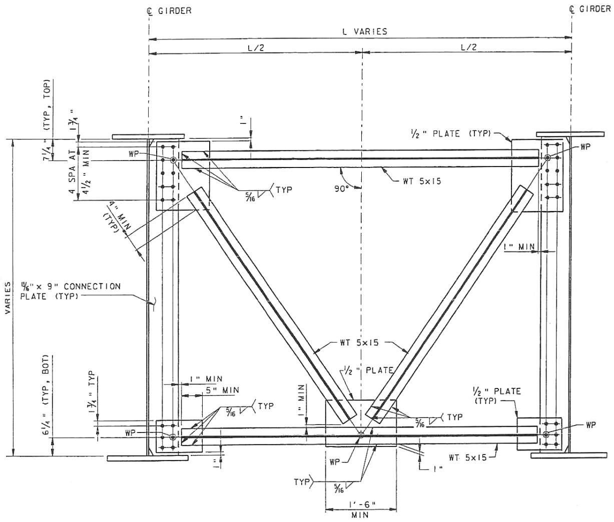

Fig. 4 – One-Piece K-Frame

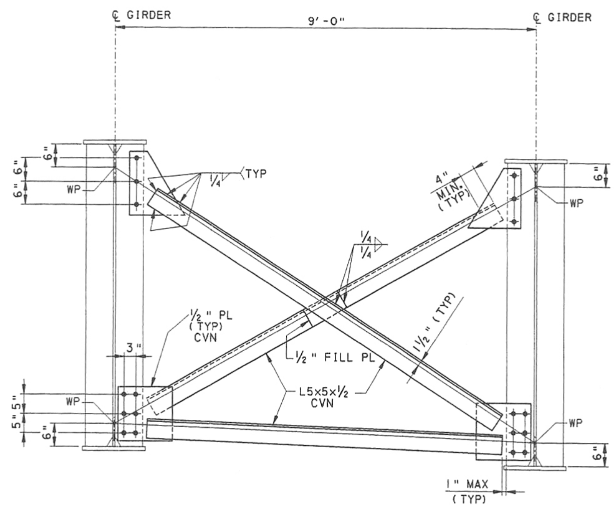

Fig. 5 – One-Piece X-Frame

cost. In addition, the one-piece frames provide an advantage to the erector in forcing the girders into the proper relative position, which is particularly important for curved and skewed structures.

When dealing with one-piece crossframes, there are fabrication advantages to be gained through the use of K-frames rather than X-frames. One-piece frames are generally fabricated by welding the members to gusset plates, which are then field bolted to the crossframe connection plates attached to the girder webs. For K-frames, all the welding can be accomplished from one side without turning the crossframe over during fabrication. When X-frames are used, the diagonals must either be continuous in different planes (requiring the crossframe to be turned over to make the welds for one diagonal) or one diagonal must be spliced where it crosses the other. Both X-frame options increase the fabrication cost.

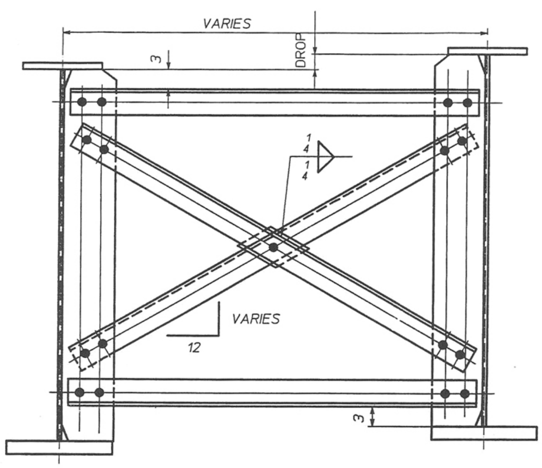

Fig. 6 – Knocked Down Crossframe

Recent input from fabricators will lead to detail changes in the crossframe connections to facilitate economical fabrication. One of the main changes being suggested is that the lines of action for the various crossframe members be detailed to intersect at a bolt hole in the connection, rather than at the girder web as has been done traditionally by designers. This results in an eccentric connection, which can be accommodated in the design. The connection plate-to-flange welds need to be checked for adequacy to carry the eccentric loads back into the girder properly. The advantage of moving the work point is that it becomes much easier for the fabricator to adjust the jigs by which they set the crossframe drops.

Shipping and Erection

Several of the detailing criteria covered previously are important for both economical fabrication and ease of shipping and erection. Careful consideration of field section lengths and weights is important to economical shipping and erection as much as it is to keeping the field splice sizes to a minimum. Adherence to the minimum b'/t ratios for the girder top flanges and minimum flange widths as noted previously facilitate shipping, handling and erection of the girders. For very long girder spans, stability of the total girder span

Fig. 7 – Temporary Top Flange Stiffening Truss

may be questionable prior to completion of the framing. In such a case, a temporary top flange stiffening truss can be attached to stabilize the girder until a second girder line is erected and attached with crossframes to form a stable system.

The use of lateral bracing is a topic of heated debate within the industry. The AASHTO provisions in Section 10.21 of the Standard Specification check the need for lateral bracing in the service condition of the bridge after deck placement. Experience has shown that, in the final condition of the bridge, lateral bracing is rarely, if ever, justified. However, there are times when it is appropriate to provide lateral bracing to assure safe erection. When spans exceed 250', particularly if the structure is very high, some level of lateral bracing is advisable to facilitate safe erection. If lateral bracing is used, it is desirable to minimize the number of bays that include lateral bracing. A single bay down the center of the bridge is preferable. Partial length bracing should also be considered. Often, placing lateral bracing in the first few bays adjacent to the support locations will stiffen the framing adequately to permit safe erection while minimizing the cost of the bracing.

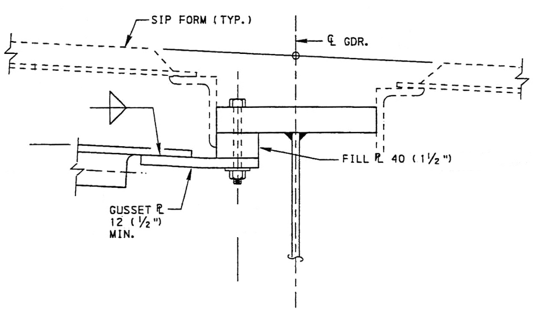

Fig. 8 – Top Flange Lateral Bracing Connection

If lateral bracing is incorporated into the structure, several detail issues need to be addressed. In the past, bottom flange lateral bracing has typically been used. However, as more refined methods of analysis are used, it has been shown that bottom flange lateral bracing can often carry significant loads when included in the models. The bottom lateral system tends to form an effective box girder section with the girders and the deck. However, many agencies will not permit the lateral bracing to be incorporated into the model because they do not want the structural capacity of the bridge to be dependent upon lateral bracing members. Some agencies now prefer top flange lateral bracing because its proximity to the concrete deck significantly reduces its participation in carrying loads in the completed structure. If top flange bracing is used in conjunction with stay-in-place deck forms, fill plates may be required between the top flange and the bracing connection plate to keep the bracing from fouling the form support angles that are attached to the sides of the girder flanges. (See Figure 8.)

Fig. 9 – Pier Brackets

The designer should develop an understanding of common construction methods for structures. For continuous girder structures, pier brackets are often used to support negative moment sections as shown in Figure 9.

As span lengths increase, temporary falsework bents may also be necessary, similar to those shown in Figure 10.

Site access is also a significant consideration when assessing girder details. If the terrain under the bridge is rugged and would make access by crane difficult, or if there are significant environmental constraints placed on the area under the new bridge, launching of the girders may be an economical solution. If it can be determined during the design that launching is necessary, appropriate details can be incorporated into the girder design so that additional engineering during construction is minimized.

Fig. 10 – Temporary Bent **Fig. 11 – I-girder Launch with Cable Tie-Backs**

SUMMARY

It is possible for all bridge engineers to design economical steel girder bridges. The key to success is that designers must stay abreast of developments in the steel industry. This will happen by tracking what other designers are doing in the field of steel girder design. More importantly, it is critical that steel designers stay in regular contact with key fabricators in their geographic area in order to glean from those shops what types of details can be fabricated economically. All bridge engineers have access to the tools necessary to design economical steel girder bridges. Designers must strive to keep abreast of developments within the steel industry. This can be accomplished by tracking what other designers are doing as well as maintaining regular contact with fabricators and erectors in their region.