Economical Steel Plate Girder Bridges

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As a service to the bridge design profession, Bethlehem Steel has conducted hundreds of studies of steel plate girder bridges using its Preliminary Bridge Girder Optimization Program. The program is unique because it optimizes plate girder designs on the basis of least *cost*, not least *weight.* An evaluation of many study results leads to a number of guidelines which are offered in this paper. When applied by the bridge designer, these guidelines should result in economical steel plate girder designs for continuous composite bridges with span lengths of up to approximately 200 ft (61 m), representing a majority of the bridge population.

Comparisons of various parameters are made on a cost index basis. These subjects are discussed: load factor vs. working stress design, weathering steel, painted high-strength steel, number of girders in a cross section, optimum web depth and thickness, transverse vs. longitudinal web stiffeners, flanges and flange splices, plus other considerations leading to cost effective steel plate girders for bridges.

In the design of steel plate girder bridges, it used to be sufficient to determine a least weight solution and develop it as a complete set of plans for competitive bidding. That was believed to be the most economical design. However, over the past several years, many improvements have been made in design and analysis methods, materials and construction techniques, obliging the designer to consider an increased number of options. Today's heightened competition due to alternate bidding practices, including contractor sponsored alternates, makes it further incumbent upon all concerned with steel bridge design and construction to always strive for the most cost-effective solution.

ECONOMICAL GUIDELINES

This paper will present some features of economical steel plate girder bridges and offer some guidelines. Application of these guidelines by the bridge designer will, in most cases, result in economical steel plate girder designs for continuous composite bridges with span lengths up to around 200 ft (61 m), representing a majority of the bridge population. Continuity and composite design are assumed because these features combine to save 20% or more in main girder cost compared to simple span non-composite designs. Therefore, only continuous composite designs will be discussed.

PRELIMINARY BRIDGE GIRDER OPTIMIZATION STUDIES

Most of the guidelines are based on the hundreds of studies accomplished with Bethlehem Steel's Preliminary Bridge Girder Optimization Program over more than a decade. Certain material and fabrication costs are built into the program, allowing comparisons to be made on a cost index basis. The cost indices cited are based on the fabricated main girder material only. Other components, such as diaphragms, bearings, deck slab, etc. would have to be evaluated before selecting a final superstructure design. Also, it may be noted that the difference between cost indices sometimes seems small. This is because all comparisons are between designs which are optimum for their respective parameters.

DESIGN METHOD

Within the 200-ft (61-m) span range, Load Factor Design (LFD) produces savings of 5 to 12% compared to Working Stress Design (WSD). For spans longer than 200 ft (61 m), LFD saves from 12 to 20%. This increasing cost advantage of LFD as spans become longer is a function of the increase in the proportion of dead load to live load as spans increase in length. Dead loads can be predicted with greater certainty than live loads. Therefore, with LFD, dead loads carry a lower factor of safety than live loads. LFD is the clear economical choice compared to WSD.

However, some states have used a higher HS live loading for LFD than for WSD, or they have increased the beta factor above the value in the standard AASHTO formula. The cost for using such modified Load Factor Designs is about the same as for Working Stress Designs. The LFD method should not be modified through increases in live loading or beta factors unless such increases represent a decision to upgrade loadings in general without regard to design method. Using greater loadings for LFD than for WSD effectively nullifies the cost advantage of LFD which for equivalent conditions is the clear economical choice over WSD.

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STEEL GRADES

Unpainted Weathering Steel

The most cost-effective choice of steel grade is unpainted ASTM A588 weathering steel used in appropriate environments. The cost advantage of unpainted weathering steel designs compared to painted high-strength steel designs can range from 2 to 11% for the main girders, even though the unit cost of A588 is the highest of the three commonly used bridge steels. Unpainted weathering steel is less expensive, on a first cost basis, than painted designs without consideration of future maintenance painting. These cost differences are based on the use of an average paint system. If a more expensive, higher quality, longer life paint system were specified, the cost advantage of weathering steel would be greater. Therefore, the clear economical choice of steel grade is unpainted A588 weathering steel.

Painted High Strength Steel

In environments which are inappropriate for unpainted weathering steel, the most economical painted designs use high-strength steel. High-strength steel designs are less expensive than ASTM A36 homogeneous designs by approximately 6 to 10% for the main girders. They are less expensive than mixed designs by approximately 3 to 5%. A typical mixed design uses high-strength steel in the negative moment regions and A36 in the positive moment regions. Each field section is homogeneous with respect to yield strength, but the yield strengths differ from field section to field section.

In this discussion, "high-strength steel design" means hybrid or 50 ksi (345 MPa) homogeneous. Hybrid designs usually use A36 steel in the webs and A36 or 50 ksi (345 MPa) steel in the flanges. The 50 ksi (345 MPa) homogeneous designs are ASTM A572 Gr. 50 throughout, except A588 is used where the maximum 2-in. (50.8-mm) thickness of A572 Gr. 50 is exceeded. The switch to A588 for thicknesses over 2 in. (50.8 mm) will not be necessary once AASHTO has adopted the newly revised ASTM A572 Gr. 50 specification which maintains 50 ksi (345 MPa) yield through 4 in. (101.6 mm).

There is usually a slight advantage (1% to 2%) with hybrid designs compared to the 50 ksi (345 MPa) homogeneous designs for span lengths less than 200 ft (61 m). For spans of 200 ft (61 m) or more, 50 ksi (345 MPa) homogeneous designs are slightly favored. Again, these differences seem small from a percentage standpoint, but it should be remembered the solutions being compared are optimum for their respective parameters.

For spans up to approximately 200 ft (61 m) it is suggested that hybrid be considered the standard painted design. However, the clear economical choice overall is unpainted weathering steel.

NUMBER OF GIRDERS IN CROSS SECTION

Substantial savings can be achieved by minimizing the number of girders in a cross section commensurate with the overall economy of the total superstructure system. For example, Fig. 1 represents a structure with 11 girders spaced at 7 ft-6 in. (2.29 m) supporting a wide multilane divided roadway. If, as in Fig. 2, three girders are eliminated and the spacing of the remaining eight girders is increased to 10 ft-8 in. (3.25 m) a savings of approximately 8% to 13% in main girder cost would result. Such savings might be partially offset if the deck thickness had to be increased to accommodate the wider girder spacing. However, the savings would be amplified further through use of fewer diaphragms, bearings etc. There would also be fewer girders to erect.

Therefore, selection of number of girders (girder spacing) is one of the most important influences on the economy of a plate girder bridge design. It is suggested 10 ft (3.05 m) be considered the minimum spacing for economical results.

WEB DESIGNS

Transverse Stiffeners

In all cases, transverse stiffeners should be placed on only one side of the web, except at diaphragm connections of interior girders, where they are needed on both sides.

Definitions

In the following discussion, girder webs having transverse stiffeners only at diaphragm connections are defined as unstiffened. A nominally stiffened web is defined as having a thickness 1/16 in. (1.6 mm) less than the unstiffened web. The thinnest web allowed by AASHTO with a maximum number of transverse stiffeners is defined as fully stiffened.

Web Optimization

The web designs which the Bethlehem program iterates trade off plain material cost vs. the fabrication cost of applying stiffeners. For a typical input, the program investigates girders with a practical range of web depths in specified increments. Within each depth, it iterates the web thickness by 1/16-in. (1.6-mm) increments. Web/stiffener designs are not generated by themselves, but are an integral part of an overall girder design which takes into consideration web depth and thickness, flange sizes, stiffener location and size, material cost and fabrication cost. The optimum overall girder design is assigned a cost index of 1.00.

Web Depth

A plot of the cost index for the optimum girders vs. several web depths (Fig. 3) shows that depth variations either side of

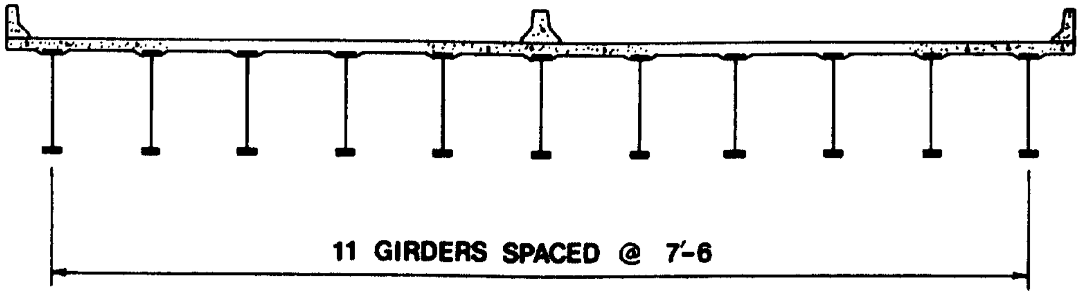

Fig. 1. Note: 1 ft=0.30m

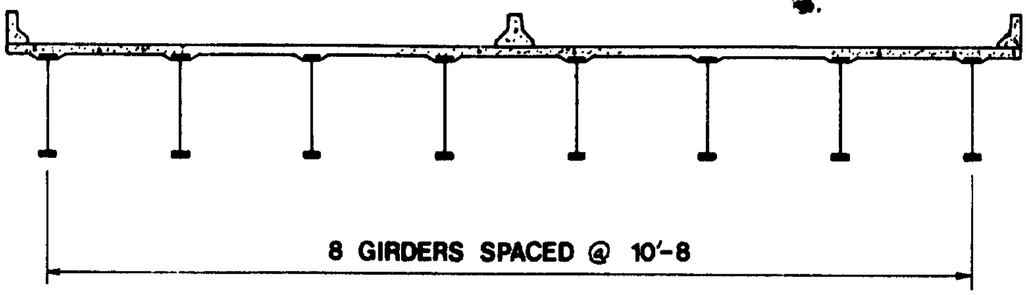

Fig. 2. Note: 1 ft=0.30m

the overall optimum depth are only slightly more costly for both hybrid and homogeneous solutions. This is beneficial in situations where the optimum depth girder cannot be used, such as having to limit structure depth to meet underclearance requirements. For whatever reason, for hybrid or homogeneous designs, web depths may be increased or decreased several inches from the optimum without significant cost penalty.

Web Thickness

Very seldom does the Bethlehem program determine an overall optimum solution having an unstiffened or fully stiffened web. The optimùm usually falls somewhere between. For spans up to approximately 150 ft (45.7 m), the web thickness for the optimum solution is usually 1/16 in. (1.6 mm) thinner than the unstiffened design, requiring nominal stiffening. As span lengths increase, the optimum web thicknesses tend to be thinner than the unstiffened design by increments larger than 1/16 in. (1.6 mm). Sometimes the increment is as large as ¼ in. (6.4 mm).

Figure 4 shows how girder cost varies with web thickness for a given depth. The difference in cost between web thicknesses varied by 1/16-in. (1.6-mm) increments is relatively small as the curves are traced in the direction of the thicker webs from their lowest, or optimum, points. Figure 4 is intended to demonstrate that, for spans up to 200 ft, a designer can select a web thickness 1/16 in. thinner than the unstiffened web and rest assured that he has not incurred undue cost penalty if his selection does not happen to be the actual cost optimum. For purposes of this paper, the only way to determine the actual cost optimum is through use of the Bethlehem program.

The dissimilarity in the shape of the two curves in Fig. 4 is linked to tension field action which is not permitted in the load factor design of hybrid girders. This is why Fig. 4 indicates a thicker web for the optimum hybrid girder than for the optimum homogeneous girder. The thinner optimum web thickness occurs on the homogeneous girder because tension field action may be included when evaluating web strength. This phenomenon is mentioned in the discussion that follows on longitudinal stiffening.

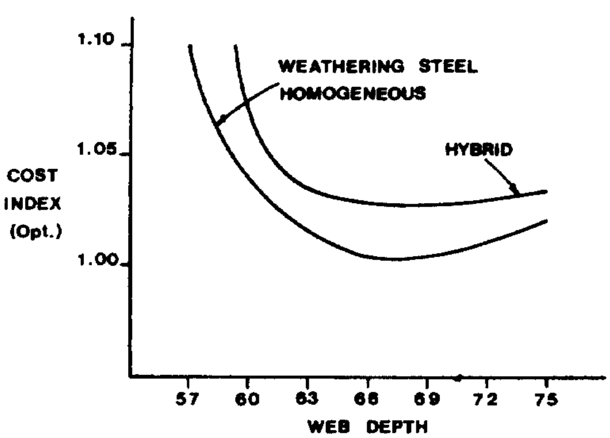

Fig. 3. Note: 1 in. = 25.4mm

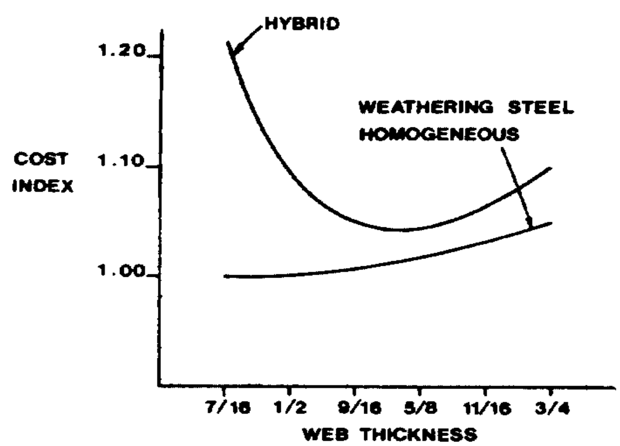

Fig. 4. Note: 1/16 in. = 1.6mm

Some designs use webs of constant thickness for the entire length of the girder. Others use webs whose thickness varies by field section (i.e., the web thickness is constant within a field section, but varies from field section to field section). The difference between constant and variable web thickness designs is small (only 1% or 2%), usually in favor of variable webs. However, as a general rule, variable thickness web designs are suggested.

Longitudinal Stiffening

Longer spans sometimes require deeper webs stiffened both transversely and longitudinally. For convenience, such girders will be referred to as longitudinally stiffened, even though it is understood they are also transversely stiffened. In such cases, the longitudinal stiffeners should always be placed on the opposite side of the web from the transverse stiffeners. This minimizes the number of places where longitudinal and transverse stiffeners intersect, such places occurring only at diaphragm or bearing stiffeners.

None of the Bethlehem studies show that longitudinal stiffening is economically justified in spans of 200 ft (61 m) or less. In fact, longitudinally stiffened designs are not usually competitive with transversely stiffened designs until span lengths approach 300 ft (91.5 m). The optimum longitudinally stiffened girders are deeper and, within the 200-ft (61-m) span range, they weigh 1% to 12% less than their optimum transversely stiffened counterparts, but they cost more. The homogeneous designs cost from 1% to 3% more while hybrid designs cost from 4% to 7% more.

The larger cost differential for hybrid girders occurs because AASHTO prohibits use of tension field action in the Load Factor Design of hybrid girders. The result is an increase in the number of stiffeners for the longitudinally stiffened design. This prohibition also causes the transversely stiffened hybrid design to have a thicker web than the corresponding homogeneous design. This phenomenon is further aggravated by the requirement of Load Factor Design to consider interaction of shear and moment. The specifications as written suggest the designer check for maximum shear and maximum moment as occurring at the same point. It is hoped this provision will be clarified in the near future.

In any case, for reasons of economy, longitudinal stiffeners should not be considered for span lengths less than 300 ft (91.5 m).

FLANGES

A least weight design would require several changes of flange size to make the girder section properties closely approximate the moment curve. However, because of the cost of making flange splices, this is not practical. Bethlehem studies show that, as a rule, the number of plates in the top or bottom flange of field sections up to 130-ft (39.6-m) long should not exceed three (or the number of shop splices should not exceed two). In some cases, especially the top flange in positive moment regions, a single flange plate size should be carried through the full length of the field section. As a general rule, an average of about 700 lbs. (318 kg) of flange material should be saved to justify the introduction of a flange splice.

It generally does not pay to vary flange widths in a field section because of the cost of tapering the ends of the wider flange plate to feather into the edges of the narrower plate. Also, many fabricators purchase wide plates, splice them and then strip the required flange plates. This practice results in constant-width flanges. Therefore, flange widths should be designed as constant for the length of the field section.

HAUNCHES

Another form of least weight design is the haunched plate girder. Some studies have shown that these designs are not competitive with constant depth, or parallel flange, designs in spans of 200 ft (61 m) or less. In fact, in recent bidding, it has been shown that they are not generally economical for spans up to around 400 ft (122 m). One state invited bids on haunched versus parallel flange designs for a four-span continuous structure with maximum span of 178 ft (54.3 m). All five bidders priced the parallel flange design below the haunched design. In the longer span range, another state advertised a haunched design and invited contractor alternates for a river crossing with a maximum span of 420 ft (128 m). All steel bids were based on an alternate parallel flange design. There were no bids on the original haunched design. The bridge is now under construction using parallel flange plate girders. Therefore, it is suggested that haunched designs be considered only when span lengths exceed 400 ft (122 m).

PRECAST DECKS

To make steel plate girder bridges even more economical, some consideration is now being given to designs using precast, prestressed concrete decks which would make it possible to use a greatly reduced number of girders with very wide spacings and overhangs. Figure 5 shows a four-girder cross section featuring 16-ft (4.88-m) girder spacings and 9-ft (2.74-m) overhangs. Comparing this to a conventional multi-girder cross section using a spacing of 10 or 11 ft (3.05 or 3.35 m), the savings in main girder cost is in the range of 25% to 40%. Of course, the savings might be partially offset by additional deck costs, including attachments to produce composite action between the precast panels and the steel girders. Precast panels have been used for deck replacement, but there is little bidding history for new construction. However, on one recently bid project, bids of a conventional-

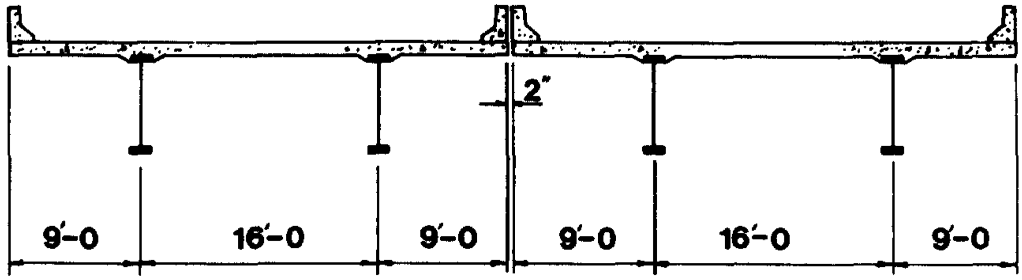

Fig. 5. Note: 1 ft=0.30mm

deck/multi-girder design vs. a precast-deck/two-girder design were within 3%, even though there were site constraints working against the latter. This is encouraging for the future of precast decks and wide-girder spacings in new construction. Certainly this concept deserves serious consideration.

OTHER CONSIDERATIONS

Within the present state of the art there are possibilities for improving economy, such as: (1) omission of bottom lateral bracing as covered by new AASHTO empirical methods, (2) use of elastomeric bearings or pot bearings instead of expensive, custom-fabricated steel rocker bearings and (3) use of composite construction in negative moment regions over the piers. Even if negative moment regions are designed as non-composite, there is a statement of AASHTO allowing inclusion of the whole deck slab in the section properties of negative moment regions when calculating maximum live load deflections. It is possible that steel plate girder designs have been made unnecessarily conservative because this provision was overlooked.

SUMMARY

Most of the guidelines developed in this paper are derived from the many studies accomplished with Bethlehem Steel's Preliminary Bridge Girder Optimization Program. These guidelines apply to spans up to 200 ft (61 m) in length, a majority of the bridge population, although some also apply to longer spans.

1. Load Factor Design (LFD) is more economical than Working Stress Design (WSD). Modifying LFD by imposing higher loads than on WSD nullifies the usual cost advantage of LFD.
2. Unpainted A588 weathering steel is the most economical design. Properly designed in the appropriate environment, weathering steel bridges are more economical than those requiring painting of the whole structure.
3. The most economical painted design is hybrid. Painted 50 ksi (345 MPa) homogeneous designs are a close second.
4. Designs should use the fewest number of girders compatible with deck design and other factors. It is suggested a girder spacing of 10 ft (3.05 m) be considered the minimum for economical results.
5. Transverse stiffeners (except diaphragm connections) should be placed on only one side of the web.
6. Web depth may be varied several inches greater or lesser than the optimum without significant cost penalty.
7. A nominally stiffened web (1/16 in. or 1.6 mm thinner than unstiffened) will be the cost optimum or very close to it.
8. Designs with web thickness which varies by field section are suggested.
9. Longitudinally stiffened designs should not be considered for spans less than 300 ft (91.5 m).
10. Use no more than three plates (two shop splices) in the top or bottom flange of field sections up to 130-ft (39.6-m) long. In some cases, a single-flange plate size should be carried through the full length of the field section.
11. An average of about 700 lbs. (318 kg) of flange material should be saved to justify the introduction of a flange splice.
12. Use constant flange widths within field sections.
13. Haunched girder designs should not be considered for most conventional cross sections until spans exceed 400 ft (122 m).
14. Omit bottom lateral bracing where permitted by AASHTO.
15. Use elastomeric bearings or pot bearings in lieu of custom-fabricated steel bearings.
16. Consider use of composite construction in negative moment regions.

Probably the most influential of these guidelines are use of (1) Load Factor Design, (2) unpainted weathering steel and (3) minimum number of girders in the cross section. These three should always be the first consideration.

Application of all these guidelines in the design office should lead to economical steel plate girder bridges.