Prestressed Steel Girders for Single Span Bridges

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INTRODUCTION

Long single span bridges are becoming more common on today's highway systems. The span lengths of these bridges grow as the widths of the underlying roadways increase to accommodate larger volumes of traffic and the use of intermediate supports in the underlying median are discouraged to allow for future widening of the roadway or mass transit corridors (see Figure 1). Span lengths of 150 to 250 feet are not uncommon and typically require the use of steel plate girders. Since these simple spans have no benefit of continuity at the supports, the required plate sizes tend to be rather large. One method to reduce the size of the plates and thus the overall amount of steel needed for each girder is prestressing. Just as the load carrying ability of a concrete beam is enhanced by the application of prestressing, the required size of a steel girder may be reduced in comparison to its conventional counterpart.

Prestressed steel girders are not a new idea, in fact eastern European designers have used prestressed steel girders for quite some time.[1] The major application in the United States has been in the area of bridge rehabilitation.[a] This paper summarizes the economic advantages of utilizing prestressed steel girders for proposed single span bridges by comparing conventionally designed girders to prestressed girders. The comparative analysis will report differences in material usage and will discuss the effects of the differential cost of construction and the intangible benefits of the prestressed girder alternative.

PRESTRESSED STEEL GIRDERS

Prestressing, or post-tensioning depending on the connotation of time, is the introduction of initial stresses in a member that will directly oppose the stresses induced by the applied loads. This cancellation of stresses allows for a smaller size prestressed member when compared to an identically loaded non-prestressed member. Although many methods exist to prestress a structural member, the procedure that appears best suited to steel plate girders is the use of high strength steel tendons, usually threaded rods. This method for steel plate girders is superior to others involving the use of supporting falsework or girder deflection by jacking since tendons may be placed and tensioned without impacting traffic below the bridge.

By straight-forward analysis, the moment due to prestressing is increased as the distance from the prestress force to the neutral axis is increased ($M = P_e$). This suggests that the prestressing tendons should be placed at or below the bottom of the member for maximum efficiency. One convenient position for the prestressing tendons on steel girders is just above the bottom flange/web plate connection. This location provides room to modify or relocate typical steel details to permit passage of the tendon and clearance to place the hydraulic jack at the end of the girder during construction as shown in Figure 2. Further, this location furnishes a measure of safety for the tendon in the event of a beam strike and does not adversely impact the minimum vertical clearance to the underlying roadway.

The length of the tendon is set a prescribed distance less than the length of the girder. Reference 2 suggests that the tendon terminate approximately 7 percent of the span length away from the support since the effect of prestressing is undesirable at the ends of the girder. This is analogous to the shielding of prestressing strands at the end regions of prestressed concrete beams. At the end of the tendon, the prestress force is transferred to the girder through a transfer bracket assembly. This device usually consists of structural tubing with a bearing plate and is connected both to the web and the bottom flange of the girder.[2] (See Figure 3.)

PROPOSED DESIGN CASES

In order to compare the effect of prestressing on single span steel plate girders, three design cases were developed with span lengths of 150, 200, and 250 feet. The span length was the only variable to maintain a control with which to evaluate the results. The bridge project to be investigated was generated to approximate a typical interstate bridge. The proposed bridge was non-skewed, on a tangent alignment, with an overall width of 43 ft.-1 in. The framing consisted of four steel girders spaced at 11 ft.-6 in. with 4 ft.-3½-in. cantilevers. Figure 4 shows the Typical Section of the proposed bridge.

To achieve results that could be directly compared, the same design and optimization techniques used on the conventional girders were also used on the prestressed girders. These methods included the use of Load Factor Design, Grade 50 Steel, constant flange widths along field sections, and a nominally stiffened web ($^1/_{16}$-in. thinner than unstiffened). Another constraint imposed to obtain design similarity was

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the location of the flange plate transitions which were held for each simple span girder to the 2/10 and 8/10 span point as shown in Figure 5. These techniques, when used in combination, tend toward a general optimization for a steel plate girder design.

The American Association of State Highway and Transportation Officials (AASHTO) Standard Specifications for Highway Bridges, 15th Edition, 1992[3] served as the design criteria. In addition to the applicable AASHTO loads, an allowance for 15 psf and 20 psf was made for a future wearing surface and stay-in-place deck forms, respectively. Live loads were established with the AASHTO HS20-44 vehicle. The prestressing tendons, two per girder, selected for use were 1.75-in. diameter Grade 150 threaded rods. High strength threaded rods were chosen since they have a small value of prestress loss and can be epoxy coated to improve corrosion resistance. Although more than two tendons per girder could be used, one tendon on each side of the web simplified the transfer bracket assembly and would speed construction by eliminating additional jacking operations. Beyond satisfying allowable stress criteria, the girders were also checked against the AASHTO recommended span-to-depth ratio and the maximum allowable live load girder deflection.

DESIGN METHODOLOGY

The design of the conventional girder followed traditional methods. Loads, on a per girder basis, were developed and broken into non-composite dead load, composite dead load, and live load with impact. Girder section properties were generated for girder to slab modular ratios "n" of infinity (non-composite), 3*n (composite), and 1*n (live load). Span-to-depth ratios were checked for compliance of the steel girder depth alone ($D/L > 1/30$) and for the total composite superstructure depth ($D/L > 1/25$).

In every case, the girder section was found to be a braced non-compact section and thus allowable stress values were established. The girder stresses due to applied dead and live loads were calculated and compared against the allowable stresses. Lastly, the midspan live load girder deflection was determined and evaluated against the L/800 criteria (assuming no pedestrians).

This basic design procedure was performed iteratively, incrementing the value of web depth and adjusting the web thickness and size of the flange plates accordingly, until an optimal result was obtained. It was the optimal conventional girder that was used in the final comparison to the prestressed girder.

The method utilized for the prestressed girder directly paralleled the conventional girder method including the iterative optimization approach. The major exception, however, was the inclusion of stresses induced by the prestressing force. It was determined that a one-stage stressing operation was the most advantageous and that the tendons would be tensioned after the deck had become composite. Any advantage to a two-stage scheme was negated by the additional construction time involved in the two-stage jacking process. Since the tendons are stressed after the deck is acting compositely with the girders, the load was considered as a "long-

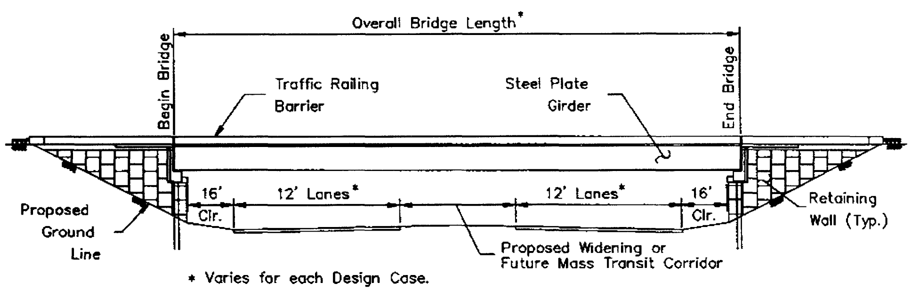

Fig. 1. Bridge elevation.

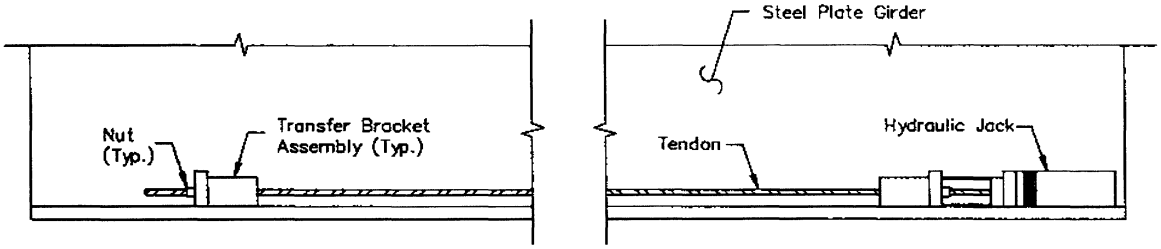

Fig. 2. Tendon tensioning schematic.

Table 1.
Girder Plate Sizes

Span Length	Type	Location	Top Flange	Web	Bottom Flange	Girder Weight	Savings
150 ft.	Conventional	End	1 in. × 12 in.	$\frac{5}{8}$-in. × 72 in.	$1\frac{1}{8}$-in. × 20 in.	48,840 lb.	20.8%
		Middle	1 in. × 20 in.	$\frac{5}{8}$-in. × 72 in.	$1\frac{7}{8}$-in. × 20 in.		
	Prestressed	End	1 in. × 12 in.	$\frac{9}{16}$-in. × 69 in.	$\frac{3}{4}$-in. × 12 in.	38,700 lb.	
		Middle	1 in. × 20 in.	$\frac{9}{16}$-in. × 69 in.	$1\frac{1}{4}$-in. × 20 in.		
200 ft.	Conventional	End	$1\frac{1}{8}$-in. × 14 in.	$\frac{11}{16}$-in. × 99 in.	$1\frac{3}{8}$-in. × 22 in.	93,800 lb.	18.6%
		Middle	$1\frac{3}{8}$-in. × 24 in.	$\frac{11}{16}$-in. × 99 in.	$1\frac{3}{4}$-in. × 30 in.		
	Prestressed	End	$1\frac{1}{8}$-in. × 14 in.	$\frac{5}{8}$-in. × 93 in.	$1\frac{1}{8}$-in. × 14 in.	76,360 lb.	
		Middle	$1\frac{1}{4}$-in. × 24 in.	$\frac{5}{8}$-in. × 93 in.	$1\frac{1}{2}$-in. × 26 in.		
250 ft.	Conventional	End	$1\frac{1}{8}$-in. × 20 in.	$\frac{13}{16}$-in. × 117 in.	2 in. × 20 in.	163,700 lb.	15.3%
		Middle	$1\frac{5}{8}$-in. × 30 in.	$\frac{13}{16}$-in. × 117 in.	$2\frac{1}{4}$-in. × 32 in.		
	Prestressed	End	$1\frac{1}{8}$-in. × 20 in.	$\frac{3}{4}$-in. × 108 in.	$1\frac{1}{4}$-in. × 20 in.	138,650 lb.	
		Middle	$1\frac{5}{8}$-in. × 30 in.	$\frac{3}{4}$-in. × 108 in.	$1\frac{7}{8}$-in. × 30 in.		

term" load and was treated as a composite dead load (3*n) in the stress calculations.

Additional transient loads in the tendons were also investigated. These loads, resulting primarily from live load deflections, were calculated by equations outlined in Reference 2. Modification of these equations were necessary to properly analyze a girder with variable rigidity as employed here. These incremental tendon loads were determined and the initial tendon force was reduced by that amount such that total resultant tendon force would not exceed allowable values.

In the case of live load deflection, the effect of the prestressing is counterproductive. In fact, the prestressed girder inherently experiences more deflection than its non-prestressed counterpart due to reduction in section needed for strength criteria.

The design of the prestressed girder also involves the sizing of the transfer bracket assembly to transfer the prestress force to the girder. Lastly, since the composite slab experiences tensile stresses due to the upward camber of the girder during the prestressing operation, the concrete stresses must be checked to see that allowable values are not exceeded.

SUMMARY

Table 1 summarizes the final selection of girder plate sizes for each of the design cases investigated. It reports the required plate sizes for each girder type and the subsequent material savings associated with the prestressed alternative. This illustrates the marked effect of prestressing on reducing required steel quantities.

The prestressed girder alternatives utilize a shallower, and thus thinner, web than the conventional girder. The top flange plates see little advantage from the prestressing since they are governed mainly by the construction loading conditions and are sized to avoid lateral buckling during the deck pour. The bottom flange plates, however, do see significant reduction in size even when considering that the web depth has also been

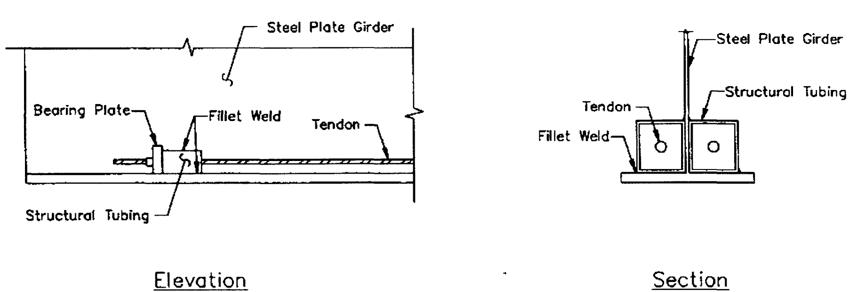

Fig. 3. Transfer bracket assembly.

reduced. Therefore, an average reduction in material of 18 percent was achieved for the prestressed girder alternatives.

CONCLUSIONS

The comparison of a prestressed girder to a conventional girder is not as simple as evaluating the difference in material usage between the two alternatives. Although the prestressed girder does require less steel the effect is partially counterbalanced by an increased cost of constructing that alternative.

The analysis of the three design cases, which varied by span length only, intimated certain trends. As the span length increased the material savings on a per girder basis decreased. This result was expected since additional tendons were not added. Thus, the same prestress force for the longer span cases was acting over a larger area and coupled with an increased section modulus tended to reduce the effectiveness of the prestressing. This could be easily remedied by adding tendons to increase the prestress force.

Although the prestressed girder alternative involves an increased cost of construction when compared with the conventional girders, other benefits of prestressed girders must also be considered. The savings in girder material not only provides an immediate economic impact of less steel cost but also results in a significantly lighter girder. This translates directly into lower loads imparted to the substructure resulting in smaller substructure elements and less foundation cost. Also, the prestressed girder allows the use of a shallower web depth eliminating additional fill for roadway approaches and lessening the amount of steel framing material needed. These savings tend to negate the increased cost of construction associated with the prestressed alternative.

The prestressed steel girder offers the single span bridge designer an economically viable alternative to the conventional steel girder. The concept of prestressing, long applied to concrete beams, is an idea which warrants consideration for steel bridge girders.

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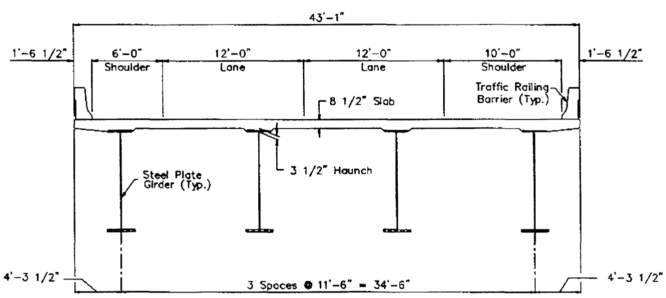

Fig. 4. Typical section.

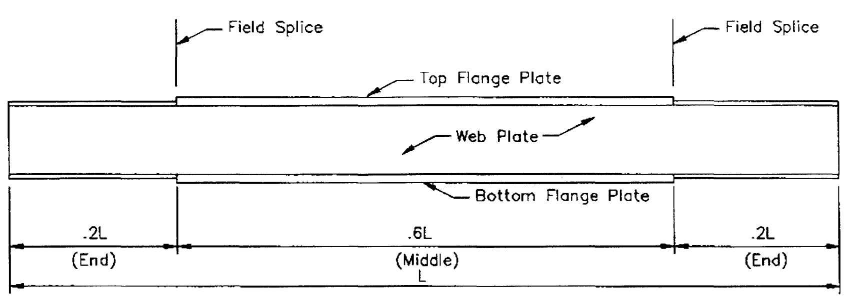

Fig. 5. Girder elevation.