Development of a shallow press-brake-formed steel tub girder for short-span bridge applications

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Abstract This paper is focused on the development of modular shallow trapezoidal boxes fabricated from cold-bent structural steel plate using standard mill plate widths and thicknesses. This concept was developed by a technical working group within the Steel Market Development Institute's (a business unit of the American Iron and Steel Institute) Short Span Steel Bridge Alliance (SSSBA), led by the current authors. This working group consists of all stakeholders in the steel bridge industry, including mills, fabricators, service centers, industry trade organizations, universities, and bridge owners. The goal was to develop innovative and economical modular solutions for the short-span steel bridge market. The proposed system meets the needs of current industry trends of accelerated bridge construction, while offering an economical solution. This paper will provide an overview of experimental testing currently being conducted and further parametric analysis and design studies focused on the refinement of cross-section dimensions.

1 Introduction

The Short Span Steel Bridge Alliance (SSSBA) is a group of bridge and culvert industry leaders (including steel manufacturers, fabricators, service centers, coaters, researchers, and representatives of related associations and government organizations) who have joined together to provide educational information on the design and construction of short span steel bridges in installations up to 140 feet in length. From within the SSSBA technical working group, a modular, shallow steel press-brake-formed tub girder was developed [1], as shown in Figure 1.

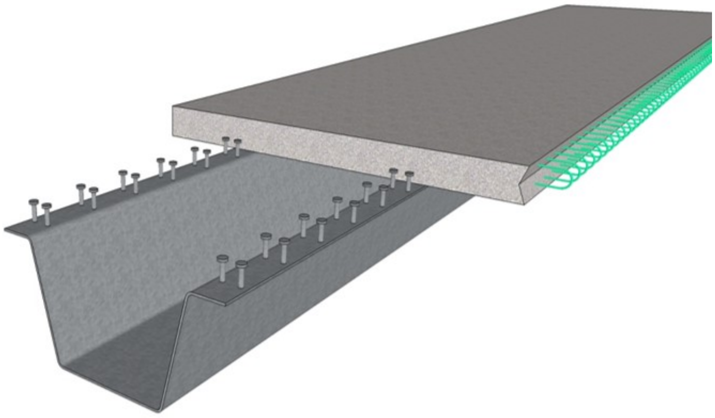

Fig. 1 Modular press-brake-formed tub girder

This new technology consists of cold-bending standard mill plate width and thicknesses to form a trapezoidal box girder. The steel plate can either be weathering steel or galvanized steel, each an economical option. Once the plate has been press-brake formed, shear studs are then welded to the top flanges. A reinforced concrete deck is then cast on the girder in the fabrication shop and allowed to cure, becoming a composite modular unit. The composite tub girder is then shipped to the bridge site, allowing for accelerated construction and reducing traffic interruptions.

A key economic factor with this newly developed system is utilizing a press brake to form a girder from a standard-width plate, as opposed to cutting and welding plates together to form a conventional tub girder. By employing the proposed system, the costs associated with cutting and welding of steel plates are eliminated. Furthermore, no cross-frames are needed (as the deck is cast compositely in the fabrication shop, providing continuous lateral support before the girder is erected), which again reduces the overall bridge system cost. Finally, due to the methods of fabrication, increased quality control would be gained from employing this system as the entire composite girder unit is shop-fabricated.

2 Proposed System Design

Accelerated bridge design and construction are areas of high importance to bridge owners due to the potential for fast, efficient, and economical bridge solutions. As a result, the bridge industry is quickly moving towards prefabricated bridges as the preferred method of bridge construction due to the increased quality control associated with prefabrication and the speed at which prefabricated bridges can be erected (resulting in reduced traffic interruptions, improved construction zone safety, decreased environmental disruptions, and improved life cycle costs).

In October of 2011, a retreat with key steel industry stakeholders was held in Chicago, IL with the intent to develop innovative and economical modular solutions for the short-span steel bridge market. Several solutions were developed from this meeting with a focus on press-brake-formed tub girders. The development of the press-brake-formed tub girder was originated at this retreat by a technical working group within the Short Span Steel Bridge Alliance (SSSBA), led by the authors.

Utilizing a standard plate width, tub girders are fabricated using a large capacity press brake. Plates are aligned in the press brake, and cold bent to achieve target bend radii. Figure 2 shows a large capacity press brake being used to form one of the girders used for experimental testing.

Fig. 2 Forming of a press-brake-formed tub girder (bending of the specimen's top flange)

Current AASHTO provisions are not specifically applicable to the design of cold-bent tub girders [2]. Therefore, preliminary specimen design was completed in two stages.

- First, a spreadsheet was developed to compute the section properties of any configuration of tub girder.
- Next, design iterations were performed based on conservative estimates of press-brake-formed tub girder capacity (limiting the capacity of the composite girders to their yield moment) to assess their validity for the short-

span bridge market. For this effort, two different plate thicknesses were evaluated (7/16” and 1/2”) and three different standard mill plate widths were evaluated (72”, 84” and 96”). These girders are termed 1 through 18 in the Appendix.

- For each variation, a design study was performed by investigating different variations of the girder dimensions in order to obtain an optimum girder configuration. For this study, the slope of the webs was kept at a constant 1:4 slope, and the inside bend radii of the girders was kept at a constant value of five times the respective plate thickness.

Figure 3 presents the results of these studies on an 84” wide standard mill plate. From these plots, it is clear that, for each plate, an optimum depth is seen at the point of maximum yield moment. As shown, the resulting optimum section using an 84” wide plate was found to have a total girder depth of 23 inches. This chosen depth is indicated by the vertical line present in each plot.

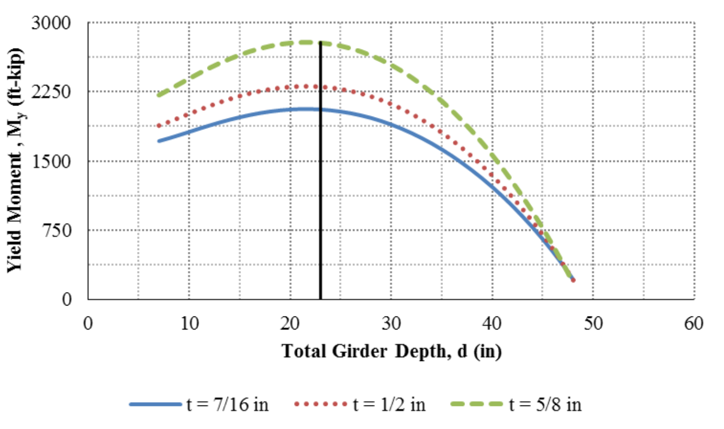

Fig. 3 Preliminary design comparisons for an 84-inch standard mill plate

3 Experimental testing and analytical methods

In order to verify the performance and capacity of this newly-developed modular tub girder, physical flexural testing of two representative specimens of the proposed system was conducted at the Major Units Laboratory at West Virginia University. Testing was conducted on single tub girder specimens with cast-in-place concrete decks. During flexural testing, vertical deflections were measured with RDP Electrosense LVDTs. In addition, a series of uniaxial gages (Micro Measurements CEA-06-250UN-350) and rectangular rosettes (Micro Measurements CEA-06-

125UR-350) were employed to measure longitudinal and shear strains during testing.

The steel employed for the specimens was an 84” × 7/16” × 480” plate utilizing ASTM A709 Gr. 50 steel. Utilizing the full standard plate width, the tub girders were fabricated using a large capacity press brake. The dimensions for the steel girder were taken from data listed in the Appendix. To develop composite action between the deck and the girder, rows of four 7/8” diameter shear studs, longitudinally spaced at 12” were welded to the top flange. In addition, to resist bearing forces, bearing plates 3/4” in thickness were welded to the girders at support locations. The concrete deck utilized was 60” wide by 6” thick. The deck was cast with normal-weight concrete; results from six subsequent concrete cylinder tests yielded an average compressive strength of 4.1 ksi.

Flexural testing was conducted on simply-supported composite press-brake-formed tub girder specimens in three-point bending. The specimens were tested using a 330 kip MTS servo-hydraulic actuator (see Figure 4).

Fig. 4 Experimental test setup

In addition to physical testing of the composite press-brake-formed tub girders, assessments were conducted using refined three-dimensional finite element modeling and strain-compatibility methods.

Modeling was conducted using Abaqus/CAE [3]. Element selection for these models included general purpose shells with reduced integration and hourglass control. The tub girder and concrete deck were modeled using four-noded shell elements. The bearing plates, due to their geometry, were modeled with both four-noded and three-noded shell elements. In order to simulate composite action between the concrete deck and the press-brake-formed tub girder, node to node multipoint constraints were used, which restrict the degree of freedom between the selected nodes. Boundary conditions in the finite element model simulated a simply-supported beam (hinge-roller) condition. The steel girders in this study, comprised of HPS 345 steel, were modeled using a tri-linear elastic-plastic constitutive law including strain hardening effects as found in [4]. Concrete was modeled utilizing a concrete damaged plasticity model available in Abaqus which assumes that the two main failure mechanisms are tensile cracking and compressive crushing of the concrete. The Comitè Europèen du Bèton concrete model was chosen to represent the compressive concrete properties used in the analyses in this work. Previous research by Barth and Wu [5] has shown that the CEB model successfully captures the compressive behavior of the type of decks studied in this work. Figure 5 illustrates a finite element mesh of a press-brake-formed tub girder composite specimen.

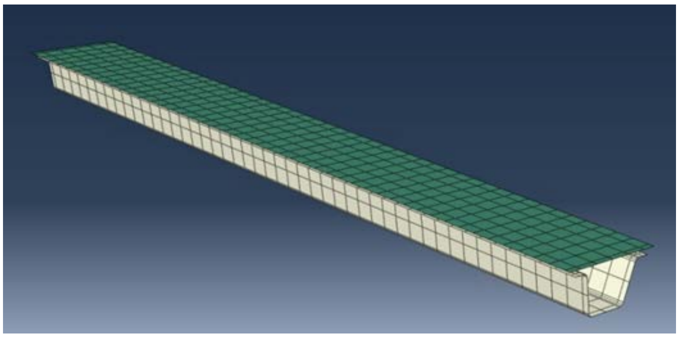

Fig. 5 Finite element mesh of composite girder

To assess the flexural capacity of press-brake-formed tub girders, a strain compatibility based analysis procedure was developed. First, a ratio of M_{DL}/M_y (ratio of initial dead load to the yield moment) is assumed to allow for the dead load effects to be accounted for in strain-compatibility analyses. Next, by assuming a concrete strain of 0.003 at crushing and a linear strain distribution, the ultimate capacity of a typical composite press-brake-formed tub girder in positive flexure can be predicted. The strain compatibility procedure is as follows:

- To begin the analysis, an assumed value to the depth of the neutral axis was chosen.
- Using an assumed linear strain distribution and superimposing the strains induced by dead load effects on the non-composite steel girder, the final strain profile was determined.
- The depth of the neutral axis was then iterated such that a net sum of zero force in the cross-section was obtained. Concrete in compression was assumed to reach a stress equal to 0.85 times the compressive strength, whereas concrete in tension was assumed ineffective. The stresses in steel elements were assumed to be the minimum of the modulus of elasticity multiplied the strain or the yield stress.
- The stresses along the cross-section were then integrated to obtain the nominal capacity of the cross-section.

Figure 6 illustrates a comparison of the data obtained from flexural testing of the two aforementioned specimens and the results from finite element analysis. As shown, finite element modeling is shown to quite accurately capture the behavior of the proposed system. In addition, the strain compatibility analysis proves quite well in predicting the ultimate capacity of the composite press-brake-formed tub girder specimen (indicated by the peak load shown).

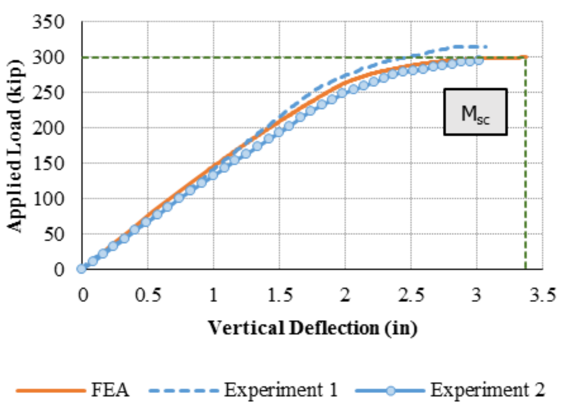

Fig. 6 Comparison of experimental and analytical results

4 Feasibility and economic assessments

In order to assess the feasibility of the proposed system, design evaluations were performed in accordance with AASHTO Specifications on a representative parametric matrix of 18 girders: 6 standard mill plate widths ranging from 60” to 120” in 12” increments and three plate thicknesses: 7/16”, 1/2”, and 5/8”.

DC loads (i.e. dead loads of structural components and nonstructural attachments) were assumed to consist of the self-weight of the girder and the concrete deck. For all of the girders assessed, the width of the girder was kept constant at 7.5 feet; this was selected as the maximum width that a modular unit could employ for feasible shipping. An integral wearing surface of 0.25 inches was assumed and applied in addition to the structural thickness of the concrete deck, which was assumed to be 8 inches. To account for the weight of shear studs, diaphragms, and other miscellaneous details, an additional 5% of the steel girder weight was applied as a distributed load. An additional load of 50 lb/ft was assumed to account for loads associated with steel guardrail systems. DW loads, or the loads of the future wearing surface, were assumed to consist of a 25 psf load applied over the 7.5 foot width of the concrete deck. LL loads (i.e. vehicular live loads) consisted of the AASHTO

HL-93 live load model. Dynamic load allowance (i.e. IM factors) was taken as 1.33, in accordance with AASHTO Specifications.

Resistance for the girders in flexure was computed according to AASHTO Specifications as well as using the aforementioned strain-compatibility approach. Resistance for the girders in shear was also computed according to AASHTO Specifications; the elements resisting shear were conservatively assumed to consist only of the flat portions of the inclined webs. All steel material for this assessment was assumed to have a yield stress, F_y = 50 ksi; all concrete was assumed to normal-weight with a compressive strength, f_c' = 4 ksi and a modular ratio, n = 8.

The feasibility assessment was conducted for each girder at the Strength I limit state (for moment and shear), the Service II limit state (for moment) and for live load deflection for each of the parametric girders. The results are comprehensively documented in [1]. Figure 7 shows a sample comparison at the Strength I limit state. Utilizing linear interpolation, this girder would be viable spans up to 61.44 feet according to AASHTO Specifications and 63.32 feet according to aforementioned strain-compatibility approach. Figure 8 illustrates the maximum applicable span lengths for each of the girders in the parametric matrix. It should be noted that for all of the girders in the matrix (except for the PL 84” × 7/16”), the Strength I limit state for moment governs the design.

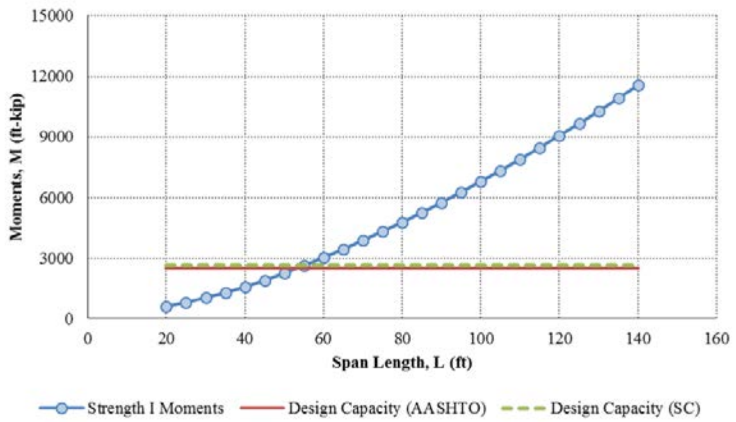

Fig. 7 Strength I moment comparisons (PL 96” × 1/2”)

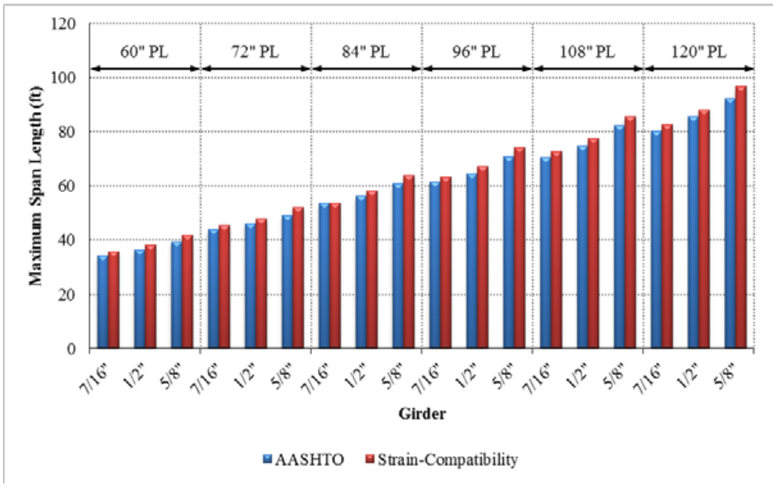

Fig. 8 Maximum applicable span lengths for proposed system

In order to assess the economic competitiveness of the proposed system, the selected standard girders were compared against traditional solutions for short-span highway bridges. Standardized steel bridge solutions were taken from eSPAN140, a complimentary web-based design tool developed by the Short Span Steel Bridge Alliance (SSSBA). There are four major sets of bridge designs in this work: "limited depth" rolled beam sections, "lightest weight" rolled beam sections, homogeneous plate girder sections and hybrid plate girder sections. From these optimized rolled girder designs, limited suites of rolled steel girder sections were selected to investigate the efficiency of using stockpiled girder sections for short span steel bridges. Also, the benefits of stockpiling common steel plate sizes are investigated in the design of steel plate girders. The designs have been made available and complimentary to engineers, and can be found at http://www.espan140.com/.

Standardized prestressed concrete solutions, such as AASHTO standard girders (i.e. Type 3, Type 4, etc.), and Bulb Tees have been available to bridge engineers for decadesStandard girders from the Idaho Transportation Department's Bridge Design LRFD Manual [6] were employed in this section. Since these girders were designed according to assumptions quite similar to those made for the design of eSPAN140's girders as well as those made for the feasibility assessments of the proposed system, these girders proved ideal for economic comparison.

Using the weights of traditional girders from previous sections, economic comparisons were made with the proposed system, and are shown in Figure 7.10. For this plot, the "S" curves refer to typical steel solutions, the "C" curves refer to typical concrete solutions, and "PBFTG" refers to the proposed modular press-brake-formed tub girder. As shown, the proposed system falls within the range expected for traditional steel and concrete girder solutions, thereby displaying its economic viability and competitiveness in the short-span bridge market.

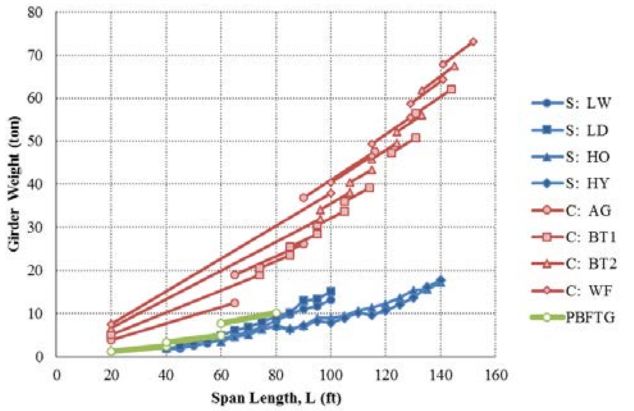

Fig. 9 Economic assessment of proposed system

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