Structural Behavior of Steel

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1.0 INTRODUCTION

The behavior of steel structures is an intricate and fascinating topic. This module is intended to serve as a guide to the AASHTO (2010) Load and Resistance Factor Design (LRFD) Specifications, 5th Edition with 2010 Interims, and their representation of the behavior of steel bridge systems and members. The module focuses on the structural form and function of bridge systems and members, with emphasis on strength limit states. Where relevant, recent advances in the AISC (2010) *Specification for Structural Steel Buildings* as well as findings from research developments are discussed in addition to the AASHTO provisions.

Selection of the most cost effective bridge structural systems and members is of course dependent on many factors, far beyond the fundamental behavior, which affect the overall material, fabrication, shipping, construction and maintenance costs for a given bridge. The companion Steel Bridge Design Handbook modules address these considerations. Also, steel bridge behavior is tied inextricably to the physical loadings or actions that the structure must resist, the corresponding load models implemented by the AASHTO Specifications to represent these actions, and the analysis of the structural systems to predict the overall responses and the individual component requirements. The Steel Bridge Design Handbook modules titled Loads and Load Combinations addresses the AASHTO (2010) load models and the module titled Sytructural Analysis discusses methods of analysis. Service and fatigue limit states, redundancy and fracture control, and constructability are addressed in separate modules. In addition, the design of cross-frames and diaphragms and their connections, girder splices, bearings, decks and substructure units are addressed separately.

Many of the words of J.A.L. Wadell (1916), a famous engineer and teacher of the early 20th century, are still very relevant to the design of bridge structures today. Principle V in Wadell's Chapter XV on "First Principles of Designing," which he refers to as "the most important (chapter) in the book," reads:

> "There are No Bridge Specifications Yet Written, and there Probably Never Will be Any, which will Enable an Engineer to Make a Complete Design for an Important Bridge without Using His Judgment to Settle Many Points which the Specifications Do Not thoroughly Cover… the science of bridge-designing is such a profound and intricate one that it is absolutely impossible in any specification to cover the entire field and to make rules governing the scientific proportioning of all parts of all structures.
>
> The author, however, has done his best in Chapter LXXVIII of this treatise to render the last statement incorrect."

Certainly, the AASHTO LRFD Specifications (2010) have also done their best in this regard. Nevertheless, there are numerous areas where a broad understanding of the fundamental behavior of structures is key to the proper interpretation, application, and where necessary, extension of the AASHTO provisions. This module aims to aid the Engineer in reviewing and understanding the essential principles of steel system and member strength behavior and design.

2.0 BEHAVIOR AND STRUCTURE TYPES

There are many ways to classify steel highway bridges. Classification of bridge systems in terms of maximum achievable span lengths is possibly the most relevant pertaining to fundamentals of the structural behavior. Steel highway bridges range from minor structures spanning only a few feet over creeks or streams to major technical achievements with spans larger than 4000 feet that define the geographic regions in which they are located. For bridges with spans ranging up to about 400 feet, stringer systems are very common. These types of structures are very important since they constitute the majority of the highway bridges within the nation's transportation system. These types of bridges are discussed first, followed by other systems that are viable at longer span lengths. (Note that here and throughout this module, the section number of the module is not included in the citation of any equations, when the citation is located in the same section as the reference.)

2.1 Rolled I-Section Stringer Systems

Steel bridge spans smaller than about 100 feet are often achieved most economically using rolled I-section members. For the shortest spans, the efficiency of the structural system tends to play a minor role in the overall cost and competitiveness relative to other attributes pertaining to simplicity, standardization, and speed of design, fabrication, delivery and construction. AISI (2000) has developed short-span bridge plans and software that address these considerations. For the shortest spans, the primary structural members in these types of bridges are typically simple-span rolled I-beams. Both composite and noncomposite deck systems are common in these types of bridges.

Spans longer than 100 feet start to push the technical limits of rolled I-beam stringer systems. Flexibility of the structure, vibration and motion perception tend to become more significant considerations in simple-span I-beam systems as the ratio of the span length, L, to the total structural section depth, D_{total}, exceeds roughly $L/D_{total} = 25$. These limits may be extended by establishing continuity between structural elements (i.e., making the I-beams composite with a concrete deck), use of continuous spans or simple-spans for dead load that are subsequently made continuous for live load (Talbot 2005), making the I-beams integral with the substructure at piers (Wasserman 1997), or use of rigid-frame bridges in which major structural elements of the superstructure and a portion of the substructure are steel I-sections (Heins and Firmage 1979). In addition, other modifications to the structural system are possible such as the use of cover plates within negative moment regions and/or longitudinal post-tensioning (Troitsky 1990, Xanthakos 1994). However, these modifications have only a minor influence on the structure stiffness and dynamic characteristics, and their cost may often outweigh their benefits. Generally, one can achieve the largest overall stiffness for a minimum constant depth by using composite continuous spans with integral piers, and by applying AASHTO (2010) Appendix B6 to allow for minor inelastic redistribution of the interior pier moments. Nevertheless, L/D_{total} values larger than about 35 are exceedingly difficult to achieve in stringer-type systems by any of the above measures. Also, as discussed in Stringer Bridge module, where the depths are not limited due to clearance restrictions, etc., often the greatest economy can be achieved by using sections that are deeper than suggested by the above maximum L/D_{total} limits.

2.2 General I-Section Stringer Systems

2.2.1 Overview

Welded plate I-girders become an attractive option at span lengths within the upper range applicable for rolled I-beams. Furthermore, depending on the costs of welded I-section fabrication versus the production costs of rolled I-shapes, welded I-section members can be cost-effective at smaller span lengths. Figure 1 shows a typical composite rolled I-beam or welded I-girder bridge cross-section. In this system, the I-sections are spaced such that the deck spans between them. The I-section members are referred to generally as girders in the following discussions.

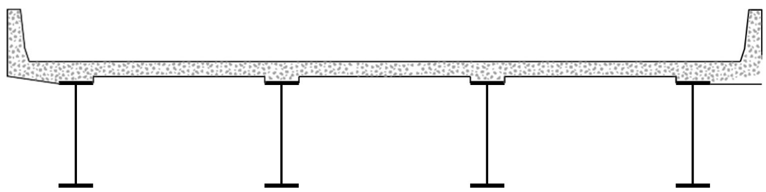

Figure 1 Typical composite rolled I-beam or welded I-girder bridge cross-section.

Cast-in-place composite concrete slabs may be designed in straight bridges of these types, without skew or with small skew, using the AASHTO (2010) Article 9.7.2 Empirical Design rules. AASHTO (2010) does not explicitly restrict the use of empirical design to straight bridges with small skew; however, it does require additional reinforcing in the end zones if the skew exceeds 25 degrees. Additional considerations may be prudent in some cases with horizontally curved bridges. The slab empirical design rules account for beneficial arching action in transferring loads to the girders, and are allowed for cast-in-place slabs up to approximately 13.5 feet spacing between the girders, *S*, or a maximum ratio of the girder spacing to the slab thickness of $S/t_s = 18$, among other requirements.

Precast decks and a number of other deck systems also are capable of spanning a large *S* with relatively small t_s. Wider girder spacing potentially eliminates one or more extra girder lines and the corresponding cross-frames and bearings, and also tends to give a more efficient structural system. This is because the live loads are positioned to produce the maximum response in each girder, but they do not generally produce the maximum effects in all the girders at a given bridge cross-section simultaneously. With wider girder spacing, the sum of the girder resistances in a given bridge cross-section will tend to be closer to the total required live load capacity for the various positions of the live load. Trade-offs associated with wider girder spacing include increases in deck thickness, and reinforcing and forming costs (which are typically offset by reduced labor costs). Also, future staged redecking considerations may influence how many girders may be removed from the cross-section. Cross-frame forces tend to be larger with wider girder spacing, due to larger differential live loads and live load deflections of the girders, as well as larger stiffness of the cross-frames relative to the slab. Interestingly, the design efficiency associated with wider spacing in ordinary stringer bridges is no longer impacted by the approximation of slab shear lag effects by effective width rules. This consideration is discussed below.

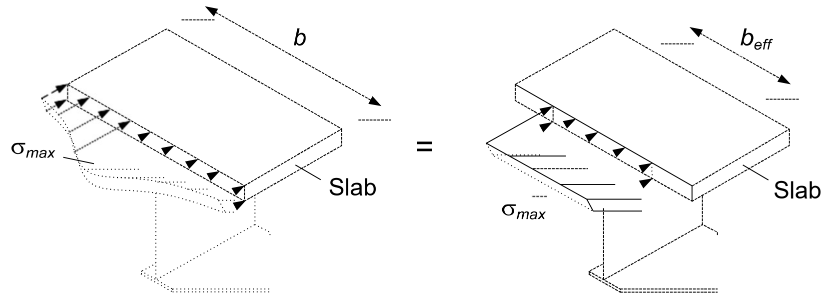

Figure 2 Effect of Shear lag.

Prior AASHTO Specification provisions, such as AASHTO (2004) Article 4.6.2.6.1, restricted the effective width of the slab, b_{eff}, to a maximum of $12t_s + b_f/2$ or ¼ of the effective span length for interior girders, along with a comparable limit for exterior girders, to account for the effect of shear lag in the slab (illustrated in Figure 2). Basically, slab flexural stresses arising from major-axis bending of the girders are developed by shear stresses in the plane of the slab. This can be seen by making a longitudinal cut to isolate a portion of the slab from the rest of a given girder and drawing a free-body diagram. Shear deformations associated with these shear stresses tend to reduce the magnitude of the flexural stresses at the slab locations farther from the girder webs. The prior slab effective width rules limited the slab contribution to the composite section in some situations with wider girder spacing.

The 2008 interims of Article 4.6.2.6.1, which are retained in AASHTO (2010), replaced the above traditional rules simply with the use of the full tributary width perpendicular to the axis of the member. The new provisions are applicable to all concrete deck slabs in composite or monolithic construction, except that separate slab effective width requirements are retained for segmental concrete box and single-cell cast-in-place box beams, orthotropic steel decks, and for transverse floor beams and integral bent caps. In negative moment regions, the new simplified rule is based on the use of the idealized fully-cracked section for cross-section level resistance calculations under both service and strength loading conditions. However, for the structural analysis, AASHTO (2010) Article 6.10.1.5 states that the concrete deck is to be assumed fully effective (uncracked) over the entire bridge length for structural analysis, using specified concrete modular ratios for short- and long-term loadings applied to the composite bridge. These changes in Article 4.6.2.6.1 are based on extensive studies by Chen et al. (2005) and others. Chen's studies indicate that the use of a slab effective width b_e equal to the full tributary width has a negligible influence on the design calculations relative to the physical bridge response in the above permitted situations. The research by Chen et al. (2005) demonstrated that there is no significant relationship between the slab effective width and the slab thickness within the practical ranges of the deck proportions in ordinary stringer bridge systems, as implied by previous Specifications.

Similar to rolled I-beam bridges, the structural efficiency of welded I-girder bridges can be improved substantially by establishing continuity between the various components or sub-systems. Also, welded I-girder cross-section proportions are typically changed at field splice locations or at the limits of available plate lengths. The use of cross-section transitions at other locations may or may not be cost effective depending on the specifics of the bridge and the economics of welding and inspecting a splice compared to the cost of extending a thicker plate. The Steel Bridge Design Handbook module titled Stringer Bridges discusses these considerations in detail. Variable web depth members with haunches over the interior supports may be used in continuous spans, thus following the shape of the elastic moment envelopes more closely. Hall (1992) indicates that haunched composite girders are usually advantageous for spans in excess of 250 feet, when the depth is limited in a portion of the span, or when a decrease in the positive moments reduces critical fatigue stresses. Current rules of thumb place this limit at 350 to 400 feet (Pfeifer 2006). Up to these span lengths, AASHTO (2010) Appendix B6 can be used to design the lightest straight I-girder bridges with limited skews using prismatic sections between the field splice locations.

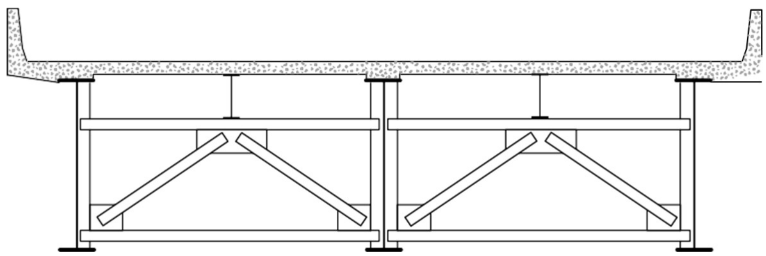

Figure 3 Typical composite I-girder substringer system.

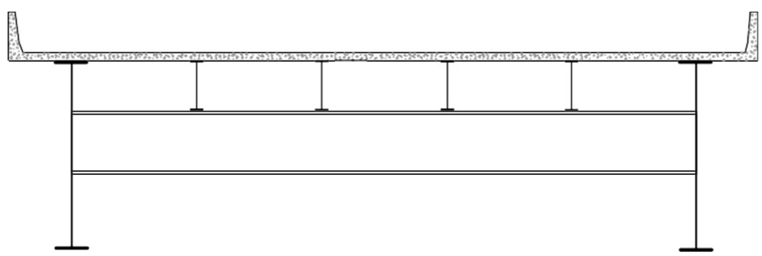

Figure 4 Two-girder system with floor beams and stringers.

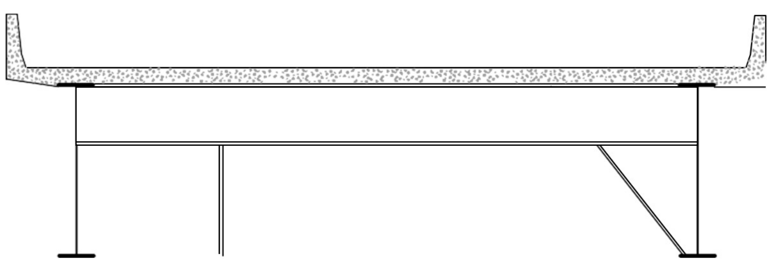

Figure 5 Two-girder system with cross-girders.

At spans exceeding about 250 feet, the I-girders in a bridge cross-section such as the one shown in Figure 1 need to be spaced relatively close together compared to their depths for the deck to span efficiently between the girders while keeping its thickness low to minimize the dead weight. In these cases, it may be attractive to use a girder-substringer system such as in Figure 3. In this type of system, shallower rolled I-section substringers are framed over the top of the cross-

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