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Computational studies of horizontally curved, longitudinally stiffened, plate girder webs in flexure

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ABSTRACT

Summarized herein is a study that explored single span, horizontally curved, plate girders having a yield stress of 50 ksi (345 MPa) to investigate their flexural behavior as a function of the position of a single longitudinal stiffener at various locations along the depth of the web. The studies were conducted using ABAQUS [1] with the girder cross-sections under high vertical bending moment and low shear. As a result of these studies, recommendations are made for positioning longitudinal stiffeners on horizontally curved webs that complement existing criteria for straight plate girders in bending. The study shows that, for the high flexure situations and girder specimens that were examined: (1) the optimal position for longitudinal stiffeners on a horizontally curved web does not appear to differ appreciably from that for a straight web as recommended in the *AASHTO LRFD Bridge Design Specifications* [2]; and (2) horizontal curvature can contribute to enhancing web stability, and, in certain instances, curvature may mitigate the need to use longitudinal stiffeners to help increase cross-section flexural strength.

1. Introduction

Plate girders used in bridges in the United States are usually deep I-beams having relatively thin webs. Thus, web buckling becomes an important factor to consider during their design. When the limit state of web buckling governs the design, transverse and longitudinal stiffeners may be used to increase section strength. The impact of transverse stiffeners on straight plate girder shear strength is well understood to be limited largely to post-buckling response, whereas the impact of longitudinal stiffeners on bending strength is known to influence both pre- and post-buckling behavior of the web. Straight girders have been extensively studied with respect to their stiffener placement, resulting in well-defined recommendations for effectively locating both transverse and longitudinal stiffeners. Stiffener influence on horizontally curved plate girders, whose flexural behavior is complicated by the horizontal curvature, has not been as extensively studied, especially when longitudinal stiffeners are considered. The current study looked at single span, horizontally curved, plate girders using ABAQUS [1] to examine their flexural behavior as a function of longitudinal stiffener position on the web. It was shown that optimal longitudinal stiffener position does not differ much for a horizontally curved girder from that for a straight girder and that horizontal curvature could contribute to web stability and, subsequently, may mitigate the need to place longitudinal stiffeners in certain curved girder designs to enhance their flexural strength.

2. Background

Horizontally curved, steel, plate girder bridges are often utilized where limited site conditions and complicated roadway alignments exist. In many instances, these structures also include relatively long spans that require deep I-girder sections and, as the depth increases, local stability of the girder web becomes a concern under both shear and flexural compression. To ensure that girder capacity is not controlled by these local web instabilities, transverse and longitudinal stiffeners are traditionally used. While the location and influence of transverse and longitudinal stiffeners on shear and flexural strength and stability are well known for straight girders and the effects of transverse stiffeners on horizontally curved girder behavior are reasonably well reported, the influence of longitudinal stiffeners on horizontally curved plate girder flexural response is still not well documented.

Plate girder webs are often reinforced with transverse or longitudinal stiffeners or a combination of the two to prevent or control local buckling. Historically, transverse stiffener design has been governed by tension field theory first published by Wagner [3]. Tension field theory was developed to account for the post-buckling shear strength that transversely stiffened, thin, girder webs demonstrate via the formation of a structure that behaves similarly to a Pratt truss [4]. Based on this

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theory, it was proposed that stiffener functions should be twofold [5]: (1) to preserve the shape of the girder's cross section; and (2) to ensure adequate post-buckling strength. As a consequence, transverse stiffener design is commonly based on: (1) minimum moment of inertia values to guarantee a line of zero deflection when the web buckles and, subsequently, increase its buckling capacity; and (2) minimum area requirements to provide enough strength so the stiffener can sustain axial forces resulting from tension field action after post-buckling [6].

While transverse stiffeners are generally designed with the intent of enhancing plate girder shear resistance, longitudinal stiffeners are generally proportioned to help control lateral deflection of the web in its flexural compression zone and, subsequently, increase plate girder bending strength by increasing bending compressive stresses that the web can carry. They also have been shown to contribute to improving girder flange bending resistance by providing enhanced web restraint to the flanges [7]. In straight plate girders, it was demonstrated that the optimum location of a longitudinal stiffener to increase bending strength was 0.22 times the depth of the web below the compression flange [8]. The same study also found that the optimal location for longitudinal stiffeners to enhance plate girder shear strength was at mid-depth of the web.

The influence of longitudinal stiffeners on straight girder behavior has been examined using finite element analyses by multiple researchers during the last decade [9–12]. The intentions of these studies were largely to: optimize longitudinal stiffener placement and proportioning for various girder sections; validate other analysis techniques to predict critical loads and failure modes; and examine the effects of longitudinal stiffeners on girder fatigue susceptibility. Most research related to the effects of longitudinal stiffeners on straight plate girder post-buckling behavior was completed a number of years ago and different beliefs regarding behavior existed, with one researcher believing that each subpanel in a transversely and longitudinally stiffened plate girder developed its own tension field after buckling [13] while another assumed that only one tension field region occurred between the flanges and transverse stiffeners even in the presence of longitudinal stiffeners [14]. Even though agreement has still not been reached as to a preferred approach for dealing with tension field effects in longitudinally stiffened girder webs, it has been suggested, based on test observations, that tension fields do develop in the entire web [15] and this belief now prevails.

Based on the cited literature and on information provided in the relevant AASHTO LRFD criteria [2], determining bending strength, shear strength and the most effective locations for longitudinal stiffeners in straight plate girders are fairly well defined processes. Since no clear differentiation exists in the AASHTO LRFD criteria between curved and straight webs, these processes may not clearly define when curvature could possibly influence behavior. As a result, this paper summarizes a study where the influence of horizontal curvature on the need for, and location of, longitudinal stiffeners in curved plate girder webs in flexure was examined.

3. Finite element models

ABAQUS Version 6.9 [1] commercial finite element software was used to conduct this investigation and doubly-symmetric, homogeneous girder sections were studied. The ABAQUS general purpose S4R shell element was used to model the plate girder flanges and web and all stiffeners. This element was selected because it was developed using a large strain formulation based on an exact geometric description of large rotation kinematics and it allows for transverse shear deformation. Thus, given that the analyses incorporated material nonlinearities using S4R elements was deemed acceptable.

Model discretization levels were established based on previous research and current design requirements [16] where it was indicated that 10 elements across the flange width and 20 elements through the web depth were sufficient to obtain accurate buckling results. Based on previous work [8], AASHTO LRFD [2] recommends placing longitudinal stiffeners at one-fifth the web depth, D, from the compression flange for straight girders to best enhance their bending strength. Using this information, the current study compared the behavior of webs without longitudinal stiffeners to ones that placed p longitudinal stiffeners at $D/6$, $D/5$, $D/4$, $D/3$, $D/2$ and $4D/5$ from the underside of the top (compression) flange. So that finite element models were compatible with these stiffener locations, the girder cross section was discretized using 45 shell elements through the web depth. The compression zone contained 30 of these elements the tension zone had 15. To keep flange shell element aspect ratios as close as possible to the web shell element aspect ratios, which were approximately 1.0, 14 elements were used across the flange width. Fig. 1 depicts a representative ABAQUS girder specimen model isometric view looking from near the center of curvature.

A tri-linear stress–strain curve was used to represent the steel behavior with Young's modulus, E, equaling 29,000 ksi (200 GPa) and Poisson's ratio, ν , equaling 0.3. The yield stress was set to 50 ksi (345 MPa) and its ultimate strength was set to 65 ksi (450 MPa). As discussed in Section 4, point loads were applied to the models to achieve desired behavior.

3.1. Geometric imperfections

Given that the models focused on design limit states that could be governed by buckling, geometric imperfections were included to replicate actual conditions. Two types of geometric imperfections, as defined by the American Welding Society's (AWS) *Bridge Welding Code* [17], were considered. The first was out-of-flatness of the web, which has limiting values based on the least web panel dimension, d, or the web depth, D, of $d/67$ or $D/150$ for stiffened and unstiffened webs, respectively. A half sine wave was used along the entire span of the girder and oriented as shown in plan in Fig. 2(a) to incorporate this imperfection. Assuming this imperfection spanned the entire girder length, rather than between transverse stiffener locations as AWS discusses, produced lower critical loads to initiate any web instabilities and, as a result, was a conservative approach.

The second geometric imperfection related to tilt of the compression flange relative to a horizontal plane [17]. For this study, tilt at the tip of the compression flange was taken as the maximum of 1% of the total flange width, b_f, or 1/4 in. (6.4 mm). To conservatively model this imperfection, the tip of the compression flange toward the center of curvature was tilted downward and the opposite tip was tilted upward by the maximum of the values above as shown in Fig. 2(b). This imperfection was conservatively assumed to occur along the length of the web panel of interest.

3.2. Residual stresses

The finite element models also incorporated residual stresses using a method recommended from previous research [16]. This method used the European Convention for Constructional Steelwork [18] model to estimate the residual stress pattern caused by flame cutting and welding as shown in Fig. 3. For this model, residual stresses were set equal to yield in tension over small widths, c_{fc}, at the flange tips and over $2c_2$ at the flange-web junction, near assumed heat affected zones. The ECCS approach determines the width over which the yield stress in tension is reached and residual, constant compressive stresses are then found via equilibrium. To accomplish this in a finite element model given that the calculated ECCS residual stress width was less than that of one element, resulting tension residual stresses were set to a value less than yield so that equilibrium would be maintained as shown in Fig. 3 [16]. Three modification factors were required to accomplish creation of the resulting residual stress pattern: a tension stress modification factor for the flange tips, δ_1; a tension stress modification factor for the flange-web junction, δ_2; and a compression stress modification factor found via equilibrium, γ_1. Web residual stresses were