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Modeling parameters for predicting the postbuckling shear strength of steel plate girders

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ABSTRACT

Bridge fires are becoming an increasing concern, and for steel plate girder bridges in particular, web shear buckling is one of the failure mechanisms that can make it necessary to replace the girder after the fire is extinguished. The objective of this study is to evaluate the web shear buckling response of two experimental plate girder specimens subject to fire conditions, and also to determine how complex computational models must be to accurately characterize the web shear buckling response of steel plate girders subjected to fire. Three parameters are evaluated: boundary conditions representing the flange, representation of thermal gradients, and composite action with the slab. To meet this objective, finite element models with varying parameters are compared to each other and to experimental results. Results show that the presence of a composite slab significantly increases the shear capacity of the plate girder. The presence of thermal gradients makes finite element modeling of the flange more sensitive to the results compared to a uniform temperature distribution. Modeling the girder with a uniform temperature equal to the temperature of the web leads to similar results as modeling with thermal gradients.

1. Introduction

Fires pose a significant risk to highway infrastructure, particularly steel plate girder bridges [1]. The lack of fire protection (active or passive) means that steel members may be directly exposed to elevated temperatures in the event of a fire occurring beneath or adjacent to a bridge. Additionally, since these steel plate girder bridges typically have high web slenderness ratios, they are prone to web shear buckling failures at elevated temperatures [2,3].

Several notable fires have occurred throughout the United States that have caused steel girder bridges to collapse or be severely damaged [1]. The collapse of two spans of the I-80 east to I-580 east flyover within the MacArthur Maze freeway complex in Oakland, California due to a tanker truck fire underscores the severity of these fire scenarios. The aftermath of this collapse that occurred on April 29, 2007 is shown in Fig. 1(a). Replacing the bridge cost $9 million, while the economic impacts to the San Francisco Bay area were estimated to be $6 million per day of the bridge closure (26 days total) [1]. Fig. 1(b) shows a tanker fire within the I-81/Route 322 interchange in Harrisburg, Pennsylvania on May 9, 2013. While the steel bridge did not collapse, extensive damage prompted its demolition. Fig. 1(c) shows extensive deformations in bridge steel plate girders due to a tanker truck fire at the I-65/I-59/I-20 interchange in Birmingham, Alabama on January 5th, 2002. Similar to the tanker truck fire shown in Fig. 1(b), the bridge did not collapse but the damage was extensive and the structure was demolished and rebuilt. Web shear buckling was observed in the bridges shown in Fig. 1(b) and (c), which motivated studies to explore how this failure mechanism contributes to the fire performance of steel plate girder bridges [2,4].

Most finite element analyses and experiments that have studied web shear buckling at elevated temperatures were conducted at steady state, uniform temperatures [2,5,6,7]. These studies have been important steps in understanding how postbuckling shear strength develops at higher temperatures, but in real fire scenarios thermal gradients would be expected to develop in the steel plate girders due to uneven heating of the structure, environmental conditions such as wind, and the varying nature of the fire itself [8].

A recent study by Peris-Sayol et al. [9,10] numerically re-created a complete bridge fire scenario from the development of the fire loading to its effects on the bridge itself. Computational fluid dynamics (CFD) models numerically characterized the fire loading from a tanker truck crashing adjacent to a steel plate girder highway bridge. Using the results from their CFD model, the authors were then able to use finite

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Fig. 1. (a) MacArthur Maze fire in Oakland, CA [1], (b) I-81/Rt 322 interchange fire in Harrisburg, PA [21], and (c) malfunction junction fire in Birmingham, AL (photo courtesy of Alabama Department of Transportation).

element modeling to characterize the structural fire performance of the non-composite bridge subjected to the fire scenario.

Michigan State University in partnership with Princeton University conducted experiments on three different composite steel girders subjected to a design fire loading. The girders that were tested experienced thermal gradients since the experiments were done in transient temperature conditions (i.e., a time–temperature curve was used). The objective of these experiments was to study the flexural and shear capacity of steel plate girders subjected to fire. Relevant details of this experiment are discussed in the next section and additional details may also be found in [4]. The work presented in this current paper builds on this experimental work by focusing on web shear buckling.

The objective of this study is to (1) evaluate the web shear buckling response of two experimental specimens under fire conditions and (2) determine how complex computational models must be to accurately characterize the web shear buckling response of steel plate girders subjected to fire. Three parameters are evaluated: boundary conditions representing the flange, representation of thermal gradients, and composite action with the slab. To meet this objective, finite element models with varying parameters are compared to each other and to experimental results. This work is novel and significant since, as discussed previously, bridge fires are becoming an increasing concern, especially for steel plate girder bridges, where web shear buckling can lead to significant damage and lead to demolition of the bridge. Further, this is the first study of shear buckling of web plates under thermal gradients and composite action with a slab.

2. Experiments

The authors conducted experiments on three steel girders at Michigan State University [4]. This section of the paper expands upon the work discussed in [4] by presenting a specific and deeper analysis of the web shear buckling response of two girders that were tested. The experimental setup and methodology are discussed; of the three girders tested, two of them were observed to have experienced web shear buckling as a result of the combined mechanical and thermal applied loading. The third girder experienced a flexural failure due to the combined mechanical and thermal loading and, since web shear buckling was not observed for this girder, it is not the focus of this particular discussion. A full treatment of the flexural response of steel girders to combined mechanical and thermal loading can be found in [4,11].

2.1. Setup

Three steel girders were tested in three separate tests at Michigan State University, labeled G1, G2, and G3. These girders were tested to failure under combined mechanical and high temperature loadings. Girder G1 was a hot rolled W24 × 62 section, while girders G2 and G3 were built-up plate girders designed according to the 2012 AASHTO LRFD Bridge Design Specifications [12]. All three specimens were fabricated with A572 Gr 50 steel, a high strength, low-alloy steel that is common for highway bridge construction. The following discussion will focus specifically on girders G2 and G3 since their slenderness ratio, measured as the depth of the web between flanges (D) divided by the web thickness (t_w), equals 122, thus making these girders susceptible to web shear buckling.

Table 1 lists the physical dimensions and loading parameters of girders G2 and G3. Both girders were constructed with a 0.140 m thick concrete slab, a 0.831 m effective width, and were designed to achieve full composite action. This concrete slab was an integral part of the furnace setup because it also served as the top lid of the furnace, sealing the chamber so that the furnace temperature could be regulated. In addition, it allowed the test girders to be exposed to a three-sided fire situation, which resembles an actual bridge fire scenario. The effective width was equal to twice the distance between the girder centerline and the furnace wall. From Table 1, the applied load value (V^{Exp}), which was held constant, was selected such that both girders G2 and G3 had the same V^{Exp}/V_u^{Design} value, where V_u^{Design} equals the shear capacity of the specimen at ambient temperature.

Fig. 2 shows the placement of girder G3 in the furnace before the combined mechanical and fire loading test started. The actuator shown in Fig. 2 was positioned at mid-span for both girders G2 and G3. Additional details of this experimental setup can be found in [4].

The test specimens were first mechanically loaded by gradually increasing the hydraulic pressure in the actuator. Once the target load, V^{Exp}, was reached, a 30 min hold time was maintained to allow the actuator loading to stabilize. Following this 30 min hold, the fire loading following the ASTM E119 temperature versus temperature fire curve was applied while V^{Exp} was maintained constant. The girders were considered to have failed when either the mid-span vertical deflection was recorded to have exceeded $L/30$ (L equals span length) or the girders could no longer sustain the applied load, V^{Exp}.

Girder G3 failed due to web shear buckling under the combined V^{Exp} load and ASTM E119 time–temperature fire curve, while girder G2 failed

Table 1
Physical dimensions and loading parameters for girders G2 and G3.

	Girder G2	Girder G3	Description
a (mm)	587.4	881.1	Distance between transverse stiffeners
D (mm)	587.4	587.4	Clear depth of web plate between flanges
t_w (mm)	4.8	4.8	Web thickness
L (mm)	3658	3658	Span between supports
a/D	1.00	1.5	Web panel aspect ratio
D/t_w	122	122	Web slenderness ratio
b_f (mm)	177.8	177.8	Flange width
t_f (mm)	12.7	12.7	Flange thickness
V_{cr} (kN)	336	239	Elastic shear buckling strength[a]
V_u (kN)	480	399	Ultimate shear buckling strength[a]
V^{Exp} (kN)	538	448	Applied load
V^{Exp}/V_u	0.56	0.56	
Failure limit state	Flexural[b] + shear[c]	Shear[c]	

[a] Calculated from 2012 AASHTO LRFD Bridge Design Specifications [12].
[b] Flexural failures imply failure due to yielding shear buckling.
[c] Shear failures imply failure due to web shear buckling.