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Elevated temperature evaluation of an existing steel web shear buckling analytical model

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ABSTRACT

Steel plate girders with slender webs are particularly susceptible to severe damage when subjected to high temperatures due to fire. Using nonlinear finite element (FE) models, this study examines the buckling strength of steel plate girder webs subject to fire temperatures. The models were validated with experimental results presented by other researchers, and the validation study resulted in recommendations for appropriate FE representations of material properties and boundary conditions. The elastic shear buckling stress (τ_{cr}) and ultimate shear buckling stress (τ_u) was then studied for web plates with various span-to-depth (a/D) ratios and a range of temperatures representing fire conditions. The results of this parametric study were compared to predictions given by the Basler–Thürlimann (BT) closed-form solution, which was originally developed to predict τ_u at ambient temperature. Various representations of the elevated temperature stress, at the time of τ_u, were used in the BT solution and compared to the FE results. It was found that the BT solution provides adequate predictions of τ_u at elevated temperatures with appropriate substitutions for the yield stress.

1. Introduction

Structural members composed of steel plates are potentially susceptible to web shear buckling, depending on the slenderness ratio, D/t_w, where D is the depth and t_w is the thickness of the web plate. Typically, web shear buckling is of particular concern for deep structures such as plate girder bridges and buildings with deep plate girders used for long spans or to transfer columns. For both structures, the accuracy of existing analytical tools for calculating postbuckling shear strength at elevated temperatures must be studied to determine the vulnerability of these structural systems to web shear buckling during a fire.

In the example of bridge structures, historical events show that fires pose a significant hazard to highway bridges. Data collected by the New York Department of Transportation (NYDOT) from voluntary submissions of 18 US states found that of the total recorded bridge failures up to and including the year 2011, 53 were due to fire compared to 18 due to earthquakes (seismically active states like California participated in the study) [1,2]. The primary cause of bridge fires is vehicular accidents occurring beneath or adjacent to the bridge. The most devastating of these fires are caused by accidents involving tanker trucks, whose large volume of combustible fuel can cause severe damage or collapse of nearby highway bridges (Fig. 1). Known as liquid pool fires, they can result in steel temperatures exceeding 1000 °C [3]. Should any fuel leak from a damaged fuel tank and spread across the roadway, this would only lead to an increase in the energy output of the liquid pool fire and a larger fire load on the structure [4].

This hazard to highway bridges in particular is only compounded by the lack of fire design guidelines and post-fire strength assessment schemes [1,5]. The National Fire Protection Association (NFPA) requires the consideration of high temperature loading for bridge design. NFPA 502: Standard for Road Tunnels, Bridges, and Other Limited Access Highways requires a water standpipe system for situations where the distance to a water supply exceeds 122 m (Section 6.5), and also requires critical structural elements to protect from collision and high temperature loading (Section 6.5) [6]. Despite these requirements, the engineer receives no guidance regarding how to design a bridge that withstands high temperature loading, nor how to assess the post-fire strength of a bridge that has been damaged.

In contrast to bridges, building fires are fueled by paper, draperies, and home or office furnishings [7]. Further, buildings have combinations of active and passive fire resistance. Fire sprinklers (active) activate automatically in the event of a fire, while thermal insulation (passive) shields steel members from excessive heating. Fire is often a "secondary" event where the primary initiating event may be an earthquake, blast, or impact. This primary event may render the active and passive fire protection inoperable. Therefore, for important structural elements, such as a deep girder supporting many loads, a fundamental understanding of their response under fire conditions is important.

A literature review of relevant research has shown extensive experimental and FE studies of web shear buckling at ambient temperatures

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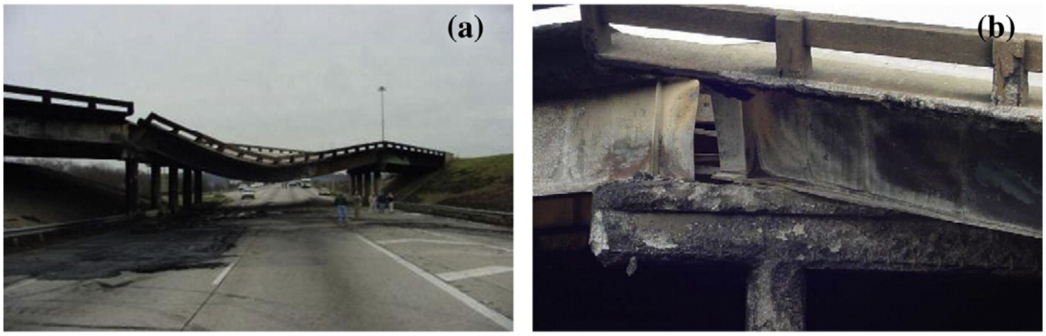

Fig. 1. I-20/I-59/I-65 interchange in Birmingham, AL after a fire: (a) severe deflections in the steel girders, and (b) web shear buckling observed near the bent cap. Photos courtesy of the Alabama DOT.

(summary provided in [8]). Elevated temperature experimental and FE studies were conducted at Nanyang Technological University in Singapore on small-scale plate girders, the results of which are discussed in Section 3 [9,10]. This paper differs from previous research since the web plates studied are deeper than that of previous research and are representative of bridges in service. Further, this paper examines the effects of material properties at elevated temperatures on the development of tensile stresses within the web at ultimate shear buckling.

The objective of this current study is to determine if an existing closed-form solution (developed for ambient temperature) for determining the ultimate shear buckling stress, τ_u, of a steel web plate is applicable at elevated temperatures. To accomplish this, τ_u values from the closed-form solution were compared with those from finite element (FE) analysis using the software Abaqus [11] for temperatures between 20 °C and 1100 °C. These FE models were based on the 1982 Standard Plans prepared by the Federal Highway Administration (FHWA) [12]. The FE models were validated with experimental data published by other researchers. This work will benefit engineers in evaluating the strength of steel plate girders in fire conditions.

2. Background

2.1. Elastic shear buckling at ambient temperature

To calculate the elastic critical buckling stress, τ_{cr}, of a rectangular plate subjected to pure shear loading, the following equation can be used [13]:

$$\tau_{cr} = k\frac{\pi^2 E}{12(1-\nu^2)\left({}^{D}/_{t_w}\right)^2} \quad (1)$$

where E is Young's modulus, ν is Poisson's ratio, D is the depth of the plate, t_w is the plate thickness, and k is the elastic shear buckling coefficient. The value of k is a function of the span-to-depth ratio (a/D) of the plate and the boundary conditions supplied to its edges. When transverse stiffeners are used, a represents the centerline spacing between the stiffeners. D/t_w is the slenderness ratio and indicates how susceptible the girder is to web shear buckling. The elastic critical shear buckling load, V_{cr}, is calculated by multiplying Eq. (1) by Dt_w.

For a plate that is simply supported on all four edges, the elastic shear buckling coefficient, k_{ss}, is calculated as [13–15]:

$$k_{ss} = 4.00 + \frac{5.34}{\left({}^{a}/_{D}\right)^2} \text{for}\, a/D < 1 \quad (2a)$$

$$k_{ss} = 5.34 + \frac{4.00}{\left({}^{a}/_{D}\right)^2} \text{ for } a/D \geq 1 \quad (2b)$$

For a plate that is simply supported on two opposing sides and fixed on the remaining two sides, the elastic shear buckling coefficient, k_{sf}, is calculated as [13–15]:

$$k_{sf} = \frac{5.34}{\left({}^{a}/_{D}\right)^2} + \frac{2.31}{\left({}^{a}/_{D}\right)} - 3.44 + 8.39\left({}^{a}/_{D}\right) \text{ for } a/D < 1 \quad (3a)$$

$$k_{sf} = 8.98 + \frac{5.61}{\left({}^{a}/_{D}\right)^2} - \frac{1.99}{\left({}^{a}/_{D}\right)^3} \text{for } a/D \geq 1 \quad (3b)$$

Transverse stiffeners are typically designed to provide simple support to the web and are idealized as such [15]. The web–flange juncture realistically offers support to the web plate that exists somewhere between a simple and fixed support. Various authors have elected to idealize this web–flange juncture as simply supported, fixed, or half of full fixity [8]. In the 1990s, finite element investigations were used to develop a more robust means of characterizing the edge support at the web–flange juncture by interpolating the value for the k coefficient between the calculated values of k_{ss} and k_{sf} depending on the flange-thickness-to-web-thickness ratio, t_f/t_w [14,15]. Thus, the value of k may be computed as:

$$k = k_{ss} + \frac{4}{5}\left(k_{sf} - k_{ss}\right)\left[1 - \frac{2}{3}\left(2 - \frac{t_f}{t_w}\right)\right] \text{for } ½ \leq t_f/t_w < 2 \quad (4a)$$

$$k = k_{ss} + \frac{4}{5}\left(k_{sf} - k_{ss}\right) \text{ for } t_f/t_w \geq 2 \quad (4b)$$

2.2. Ultimate shear buckling at ambient temperature

Various theories have been developed to compute the ultimate shear buckling stress, τ_u, of plate girder webs by accounting for the postbuckling strength reserve for thin, rectangular plates loaded in shear [8]. The theories discussed in [8] are based on the fundamental assumption formulated through Wagner's 1931 published work, which states that compressive stresses in the direction perpendicular to the observed diagonal tension field do not increase once the elastic critical buckling strength has been reached [16,17]. By accepting this assumption, the subsequent tension field theories based the postbuckling strength of the web on the additional amount of tension that can be developed in the diagonal tension field.

The tension field theory proposed by Basler, which serves as the basis of the *AASHTO LRFD Bridge Design Specifications* [18] and is notable for balancing ease of use with accuracy [19], was selected as the analytical model against which to compare the finite element results to be discussed later. Basler's work published in the early 1960s offered the first postbuckling strength theory for steel plate girder webs [8,20]. In developing his model, Basler assumed that the flanges were too flexible