Thin-Walled Structures 116 (2017) 82–90

Contents lists available at ScienceDirect

Thin-Walled Structures

journal homepage: www.elsevier.com/locate/tws

Full length article

Comparative fire behavior of composite girders under flexural and shear loading

M.Z. Naser[1], V.K.R. Kodur*,[2]

Department of Civil and Environmental Engineering, Michigan State University, East Lansing, MI, USA

ARTICLE INFO

Keywords:
Steel girders
Fire resistance tests
Shear
Local buckling
Composite action

ABSTRACT

This paper presents results from an experimental study on the fire behavior of composite steel girders subjected to high shear loading. Four steel–concrete composite girders, comprising of steel girders and concrete slab, were tested under simultaneous structural loading and fire exposure. The first composite girder was subjected to typical flexural loading and fire conditions, while the other three girders were subjected to high shear loading and exposed to fire conditions. The main test variables are level of composite action, type and magnitude of loading. All tested girders failed in less than one hour of fire exposure, however, their failure mode varied significantly. For instance, composite girder subjected to flexural loading failed through flexural yielding of steel girder with large rotation at end supports, while girders subjected to high shear loading failed with no signs of large deflections or rotations at end supports.

1. Introduction

Structural steel members, when exposed to fire, experience loss of sectional capacity and stiffness due to temperature-induced degradation in strength and modulus properties of constituent materials. In addition, these members become vulnerable to different failure modes, i.e. runaway (large) deflections, lateral torsional buckling (LTB), out-of-plane buckling (OPB) and web local buckling (WLB). A review of relevant research indicates that majority of previous experimental, analytical and numerical studies focused on flexural response of steel girders at ambient and fire conditions [1–5].

For instance, Zhao and Kruppa conducted fire tests to study the flexural behavior of fire exposed continuous composite steel beams [4]. The authors reported that such composite beams experienced signs of local buckling at the interior support. However, no further recommendations were discussed, possibly because tested beams were subjected to dominant flexural loading and thus the effect of shear and fire-induced instability could not be isolated.

Aziz et al. [5] conducted fire resistance experiments on three different composite steel girders subjected to a design fire exposure and structural loading. The objective of these experiments was to study the flexural capacity of one hot-rolled steel girder and two built-up (plate girders) subjected to fire. Test variables included: load level, web slenderness, and spacing of stiffeners. Results from these fire tests indicated that typical steel girders can experience failure under standard fire conditions in about 30–35 min. The time to failure and mode of failure in fire exposed steel girders is highly influenced by web slenderness, spacing of stiffeners, and type of fire exposure. Steel girders fail through flexural yielding when web slenderness is around 50; however failure mode changes to web shear buckling when web slenderness in built-up plate girders exceed 100.

Other studies in the literature have pointed out that steel girders are highly susceptible to web shear buckling failure mode under room and fire conditions [6–19]. Although web shear buckling and post-buckling phenomenon in plate girders has been extensively studied at ambient conditions (such as those reported in tests conducted by Wagner [9], White and Barker [10], Yoo and Lee [11], Basler [12–14] and Porter et al. [15]), only few experimental studies have investigated response of built-up plate girders and hot-rolled steel girders subjected to dominant shear force and fire exposure [18,19].

For instance, Tan and Qian [18] conducted an experimental investigation on twelve simply-supported built-up plate girders subjected to predominant shear loading at elevated temperature. The tested girders were subjected to a steady state heating regime of 400, 550, and 670 °C and with different beam restraint stiffnesses. Results from these tests inferred that the ultimate shear capacity in plate girders significantly degrade at elevated temperatures. Vimonsatit et al. [19] also tested a number of small-scale steel beams to evaluate shear effects

* Corresponding author.
E-mail addresses: nasermoh@msu.edu (M.Z. Naser), kodur@egr.msu.edu (V.K.R. Kodur).
[1] Ph.D. Candidate, Civil and Environmental Engineering, Michigan State University
[2] Professor and Chairperson, Civil and Environmental Engineering, Michigan State University

http://dx.doi.org/10.1016/j.tws.2017.03.003
Received 16 September 2015; Received in revised form 17 November 2016; Accepted 4 March 2017
0263-8231/

on fire behavior of steel beams. Theses beams were grouped into five series, three of which were built-up sections, while the rest were hot-rolled beam sections. The test specimens were loaded predominantly in shear loading under steady-state temperature conditions at 400 °C, 550 °C, and 700 °C. Results from these tests indicated that shear capacity of steel beam/girder decreases with rise in temperature and hence the authors concluded that shear limit state need to be considered in evaluating failure of steel beams/girders under fire conditions.

Despite experimental evidence, current fire design provisions for steel girders disregards effect of shear and only accounts for flexural limit state in evaluating failure. This approach of deriving failure in fire-exposed steel girders based on flexural limit state only is valid for most loading scenarios, but may not be representative in certain situations where shear forces are dominant or shear capacity degrades at a rapid pace with fire exposure time. In order to achieve optimum and realistic fire response evaluation, different failure modes are to be accounted for in evaluating fire resistance of steel girders under fire conditions.

One of these failure modes is buckling of thin plates due to instability of the section. As part of stability based design specifications, steel girders are classified as compact, non-compact or slender depending on slenderness of the flange and web plates. This classification defines the stability state of a steel girder to better account for buckling failure mode and arrive at an optimal design of steel girders at room temperature [20,21]. For instance, a compact section does not exhibit any local buckling and can attain full sectional plastic capacity, thus eliminating any localized failure. A non-compact or slender section exhibit either plastic or elastic buckling behavior, respectively, and fail pre-maturely through a localized failure in web or flanges, before attaining full plastic capacity.

The classification of steel girders based on sectional stability is quantified by comparing sectional slenderness of a steel section against a pre-defined slenderness limit (λ). Sectional slenderness represents the geometric feature of web and flange plates in steel section and is computed as D/t_w for the web and $b_f/2\,t_f$ for the flange, where D is the depth of web, t_w is the thickness of the web, b_f is flange width and t_f is flange thickness. Thus, sectional slenderness is a unique (and fixed) value for an individual plate and is independent of the loading conditions, material type or environmental (fire) exposure.

On the other hand, slenderness limit is a numerical value derived based on specific material properties and is a function of ($\sqrt{\frac{E}{f_y}}$), where E and f_y are defined as modulus of elasticity and yield strength of steel. Since material properties degrade with rise in temperature under fire exposure, the value of slenderness limit will also change following temperature-induced degradation of modulus of elasticity and yield strength. Hence, at a certain point in a fire condition (temperature), the temperature-independent value of sectional slenderness can exceed that of the temperature-dependent slenderness limits and cause the web or flange (or both) to buckle. Since the web is generally much more slender than flanges, local buckling in web often initiates before flange local buckling [6–8]. Thus, steel girders are expected to undergo significant degradation in sectional capacity (especially shear capacity) due to degradation of strength properties and onset of temperature-induced web local buckling.

Shear forces can be dominant in girders under certain loading configurations such as concentrated (point) loads acting on a girder, as in the case of transfer girders. The most common example of high shear force situation is when concentrated loads are present close to support regions of girders (especially in girders connected to offset columns in high-rise buildings or large halls/openings) [22]. Another case where shear forces can be significant is in short span beams and beams with reduced cross-sectional area (i.e., coped beams) [1,14]. Further, deep beams and plate girders, a common feature in transfer girders or bridges, can have much lower reserve shear capacity even at ambient conditions, and in such girders shear effects can govern failure under fire conditions.

It is clear that there is lack of data and understanding on the behavior of steel girders subjected to combined effect of high shear forces and fire exposure. To bridge this knowledge gap, this paper presents results from experimental studies on the response of standard hot-rolled composite steel girders subjected to combined effects of high shear loading and fire exposure.

2. Fire resistance tests

To study the effect of high shear loading on the response of composite steel girders subjected to fire conditions, an experimental program consisting of fire resistance tests on four composite steel girders was carried out. The steel girders were tested to failure by subjecting them to combined effects of structural loading and fire exposure.

2.1. Test specimens

The composite steel girders prepared for fire tests, comprised of a steel girder supporting a reinforced concrete slab. The four steel girders, designated as CB1, CB2, CB3 and CB4, were designed according to AISC specifications [21]. The steel girders made of W24×62 section (standard hot rolled section) have flange width, flange thickness, web depth and web thickness of 179, 15, 610 and 11 mm, respectively as shown in Fig. 1. The steel sections were fabricated using A572 Grade 345 steel. The concrete slab, cast along the full length of the steel girder, has a depth and width of 140 and 815 mm, respectively and was made of concrete of compressive strength of 45 MPa.

The composite steel girders were fabricated at the same time, similar design and material batches was used in all girders (as that in CB1) for consistency. Hence, all girders CB2, CB3 and CB4 were designed and fabricated with one 9.5 mm thick stiffener at the supports and one 12.7 mm thick stiffener at the mid-span of the girder (to provide some level of lateral support to the girder). It should be noted that in order for vertical stiffeners to contribute to the shear capacity of the girder, several conditions need to be met. Since,

1. $h/t_w > 1.1\ \sqrt{k_v E/fy}$, for all tested girders; [$h/t_w\ = 55.1 < 1.1\ \sqrt{k_v E/fy} = 59.23$], and also
2. $1 \le\ a/D\ \le 3$, for all tested girders [$a/D\ = 1829/573 = 3.19 > 3.0$]

Stiffeners provided in tested girders CB2, CB3 and CB4 will not contribute to shear capacity of the girders.

The main test variables in these tests are loading type and level of composite action. Composite girder CB1, which represents benchmark specimen, was subjected to flexural loading, while the other composite girders CB2, CB3 and CB4 were subjected to high shear loading. In addition, composite girders CB1 and CB2 were designed to ensure full composite action between the steel girder and the concrete slab, composite girders CB3 and CB4 had two rows of 19 mm diameter shear studs placed at 230 mm to achieve partial (50%) composite action between the steel girder and the concrete slab (see Table 1). The different shear stud arrangements in these girders is shown in Fig. 1a, b and c. Finally, the concrete slab is reinforced with two layers of steel reinforcement of No. 4 rebars (at the top and bottom) as shown in Fig. 1d.

2.2. Test equipment

The fire resistance tests on composite steel girders were carried out at the structural fire testing facility at Michigan State University. This fire test furnace has been specially designed to produce varying conditions of heat scenarios (temperature-time curves) and structural