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Behavior of steel bridge girders under fire conditions

Esam M. Aziz [a,b,1], Venkatesh K. Kodur [a,*], Jonathan D. Glassman [c], Maria E. Moreyra Garlock [c]

[a] Department of Civil and Environmental Engineering, Michigan State University, USA
[b] College of Engineering, University of Sulaimani, Sulaymaniyah, Iraq
[c] Department of Civil and Environmental Engineering, Princeton University, USA

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ABSTRACT

This paper presents results from experimental and numerical studies on the fire performance of typical steel girders used in bridges. As part of experimental studies three steel–concrete composite girders were tested under simultaneous loading and fire exposure. Test variables included: load level, web slenderness, and spacing of stiffeners. Results from fire tests indicate 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 bridge girders fail through flexural yielding when web slenderness is around 50; however failure mode changes to web shear buckling when web slenderness in girders exceed 100. Data from fire tests is utilized to validate a finite element based numerical model for tracing the response of steel bridge girders exposed to fire. Results from numerical analysis show that the proposed finite element model is capable of tracing the response of steel bridge girders under simultaneous loading and fire conditions.

1. Introduction

Steel is widely used in structural members of bridges due to number of advantages steel offers over other construction materials. These advantages include high strength, ductility, ease of fabrication, and speed of construction. A major drawback of steel construction is that steel structural members possess low fire resistance due high thermal conductivity and low specific heat of steel, as well as faster degradation of strength with temperature [1]. As a result, steel structural members can lose load carrying capacity (strength and stiffness) at a rapid pace under fire conditions. Therefore, steel structures are to be provided with appropriate fire protection measures to maintain structural stability and integrity in the event of a fire. While structural members in buildings are required to have adequate fire resistance under fire conditions, no specific fire resistance provisions are specified for bridges in AASHTO and other standards [2,3].

The response of structural members in bridges under fire conditions can be significantly different than those in buildings due to different fire scenarios, load level, support conditions, and sectional characteristics that are present in bridges [4]. Therefore, fire provisions used for building elements might not be directly applicable to bridge elements. The effect of various factors, such as connection details, fire protection, restraint and loading conditions, on fire resistance have been studied for building elements [5–9]. However, limited research has been carried out on fire resistance of structural members commonly used in bridges.

Information from fire-induced bridge collapses, such as the I-75 expressway near Hazel Park, Michigan in 2009, and the MacArthur Maze I-80/880 interchange in Oakland, California in 2007, clearly indicates that bridge fires can pose a significant problem [10]. The time to failure of steel bridge girders in these incidents was less than 30 min [11,12], which gives very little time for firefighters to respond. Currently, there is lack of data on the governing failure limit states in steel bridge girders exposed to fire [13,14].

In the last four decades, extensive experimental work has been conducted on steel bridge girders at ambient (temperature) conditions. These studies focused on web buckling, composite action, post-buckling shear capacity, and shear failure mechanism in steel bridge girders [15–17]. However, there is very little data on the behavior of steel bridge girders under fire conditions. Most of the reported fire experiments are on the response of hot rolled steel beams typical of building applications. The only experimental study on the fire response of steel plate girders is from the work reported by Vimonsatit et al. [18], on small-scale steel girders of span 1.6 m, and web thickness of 2 mm. These test specimens had no concrete slab and were loaded predominantly in shear under a steady-state fire exposure. Failure in bending mode was prevented by stiffening the flanges. The tested girders, as well as some of the conditions used in fire tests, do not represent realistic conditions encountered in practice. Indeed, the authors of this study recommended further detailed experiments to trace the fire response of steel girders.

* Corresponding author at: 428 S. Shaw Lane, EB 3580, Michigan State University, East Lansing, MI 48824-1226, USA. Tel: +1 517 353 9813; fax: +1 517 432 1827.
E-mail addresses: azizesam@msu.edu (E.M. Aziz), kodur@egr.msu.edu (V.K. Kodur), jdglassm@princeton.edu (J.D. Glassman), mgarlock@princeton.edu (M.E. Moreyra Garlock).
[1] Tel.: +1 517 802 1226; fax: +1 517 432 1827.

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A detailed literature review further reveals that there is a lack of understanding on the behavior of steel bridge girders exposed to fire [4,13,19,20]. To develop such an understanding, experimental and numerical studies on the fire performance of steel bridge girders are being carried out as part of a collaborate research project between Michigan State University and Princeton University. This paper presents results from experimental and numerical studies on the behavior of steel bridge girders under fire conditions.

2. Fire resistance tests

The experimental program consisted of fire resistance tests on three steel bridge girders. The girders were tested to failure by subjecting them to combined structural loading and fire exposure.

2.1. Test specimens

The tested bridge girders comprised of a steel section (hot rolled or built-up) supporting a reinforced concrete slab. The steel girders, designated as G1, G2, and G3, were designed according to AASHTO specifications [2]. The first test girder (G1) was a hot rolled section of W24x62 [21], while the other two test girders (G2 and G3) were built-up plate girders. The main variable in these test specimens were web slenderness and spacing of stiffeners. The web slenderness, defined as D/t_w ratio (where D is the web depth and t_w is the web thickness), of girder G1 was 52, while in girders G2 and G3 it was 123. Girders G2 and G3 were stiffened with traverse stiffeners and the aspect ratio defined as a/D ratio (where a is the stiffener spacing), in G2 was 1.0 and in G3 it was 1.5. All other dimensions for G2 and G3 were kept the same. Bearing stiffeners were provided at both the supports and at the location of the load point at mid-span. Longitudinal and traverse sections of girders G1, G2, and G3 are shown in Fig. 1, while sectional dimensions are summarized in Table 1.

The steel girders were fabricated using A572 Grade 50 steel, which is a high strength, low-alloy steel commonly used in highway bridge applications. All of the steel girders were designed to achieve full composite action with a 140 mm thick concrete slab. For this purpose two rows of 19 mm diameter shear studs were provided to ensure full composite action between the steel girder and the concrete slab as shown in Fig. 2. The concrete slab is reinforced with a layer of tension steel rebars at the bottom and a layer of wire mesh at the top.

2.2. Test equipment

Fire resistance tests were carried out using a specially built fire testing furnace at the Civil Infrastructure Laboratory at Michigan State University. The furnace has been specially designed to accommodate varying conditions of temperature, loading, support conditions and heat transfer to which a structural member is exposed during a fire incident. The test furnace and the steel girder placement within the furnace are shown in Fig. 3(a). This furnace and the actuator set-up allow simultaneous application of both thermal and structural loading on test specimens to simulate conditions experienced by a real structural member during a fire event.

The test furnace consists of a steel framework supported by four steel columns, with the furnace chamber inside the framework. The heating chamber of the furnace is 2.44 m wide, 3.05 m long, and 1.78 m high, and this produces a maximum heat power of 2.5 MW. Six natural gas burners located within the furnace to provide the thermal energy, while six type-K chromel–alumel thermocouples distributed throughout the test chamber to monitor the furnace temperature during a fire test. During the course of the fire test, the gas supply is manually adjusted such that the furnace temperatures follow a predetermined time–temperature curve of a fire, which can be either a standard fire or a design fire. To facilitate visual observation of test specimen during the fire test, two small view ports are provided on either side of the furnace wall. Four vertical pressure actuators are provided to apply loading on the test specimens.

2.3. Instrumentation

The steel girders were instrumented with thermocouples, strain gauges, and displacement transducers to monitor thermal and mechanical response during fire resistance tests. Cross sectional temperatures were measured using Type-K (0.91 mm thick) chromel–alumel thermocouples placed at mid-span and quarter-span sections along the girder length. At each section, thermocouples were installed on the test girders at bottom flange, web, top flange, stiffeners, shear studs, and at different depths of concrete slab to measure temperature progression during the fire test. High-temperature strain gages were attached to the flanges (top and bottom), shear studs, and the web of the girders to directly monitor progression of strain at these locations. Thermocouple locations on steel girders G1, G2, and G3 are illustrated in Fig. 1.

To measure mid-span deflection and axial displacement of the girders, as well as out-of-plane displacement of the web panel, vertically and horizontally oriented linear variable displacement transducers (LVDTs) were attached at various distinct locations on each girder. The mid-span deflection was measured through two LVDTs that were attached to the top surface of the concrete slab beneath the loading actuator. To measure out-of-plane displacement of the web, a well-insulated stiff threaded steel rod was attached to the center of the web panel and extended horizontally parallel to the concrete slab wing. The steel rod extends vertically to pass through the furnace lid for which an opening was made in the lid. The steel frame that carries the LVDT was installed on top surface of the concrete slab (outside the furnace zone). This is to ensure vertical movement of the steel frame (LVDT) during the deflection of the steel girder. A schematic of the set-up that is used to measure the web out-of plane displacement is shown in Fig. 3(b). The measured out-of-plane displacement of the web is for the web panel that is adjacent to the mid-span of the steel girder. To measure progression of axial displacement during the fire test, an additional LVDT was placed on the bottom flange location at one end of the steel girders. In addition, furnace temperatures during the fire test were continuously monitored using six thermocouples distributed spatially inside the furnace.

Data through the above instrumentation network was recorded at five second intervals through central data acquisition system. Visual observations were also made at five minute intervals to record significant changes (such as local buckling, spalling, etc.) throughout the duration of the test and also after the tests were terminated.

2.4. Test condition and procedure

The three steel girders tested under fire exposure have different flexural and shear capacities resulting from variations in sectional geometry of each girder (web slenderness, flange thickness, and stiffener spacing). Therefore, the girders were subjected to different load levels; evaluated as a percentage of shear and/or flexural capacity of the girder at room temperature. The flexural capacity, shear capacity, fire scenario, and the load level for the three steel girders are shown in Table 1. Girder G1 was subjected to a single point load of 691 kN at mid-span, which is equivalent to 40% of its room temperature flexural capacity and to 27% of its shear capacity. Girders G2 and G3 were subjected to a single point load at mid-span representing 40% and 33% of their flexural capacities, respectively, which equates to 56% of their shear capacities. During fire tests all three steel girders were exposed to ASTM E119 fire from three sides, with slightly higher heating rate in the first 5 min of fire exposure.

During each fire test, one girder was tested by subjecting it to combined thermal and structural loading. The instrumented steel–concrete composite girder assembly was placed inside the furnace as shown in Fig. 3. A length of 3.0 m of the mid portion of the girder was inside the