Evaluating Fire Resistance of Steel Girders in Bridges

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Abstract: In current practice, no special measures are applied for enhancing structural fire safety of steel bridge girders. Further, there is very limited information and research data in the literature on the fire resistance of structural members in bridges. In this paper, the fire response of a steel bridge girder under different conditions is evaluated using the FEM computer program ANSYS. In the analysis, the critical factors that influence fire resistance, namely, fire scenario, fire insulation, and composite action arising from steel-concrete interaction, are accounted for. Results from numerical studies show that the composite action arising from steel-girder–concrete-slab interaction significantly enhances the structural performance (and fire resistance) of a steel bridge girder under fire conditions. Other significant factors that influence fire resistance of steel bridge girders are fire insulation and type of fire scenario. DOI: 10.1061/(ASCE)BE.1943-5592.0000412. © 2013 American Society of Civil Engineers.

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Introduction

Fire is one of the most severe environmental hazards to which the built infrastructure may be subjected during its lifetime. In recent decades, due to rapid development of urban ground-transportation systems, as well as increasing transportation of hazardous materials (e.g., flammable liquids, spontaneously combustible materials, poisonous substances, etc.), bridge fires have become a growing concern (Kodur et al. 2010). Whereas the perception may be that it is very unlikely that a bridge will collapse under fire, a recent U.S.-wide survey by the New York State Department of Transportation (NYDoT) has shown that nearly three times more bridges have collapsed in the 1990–2005 period due to fire than due to earthquakes (M. Garlock, personal communication, 2008). In some cases, bridge fires might lead to the collapse of structural members, leading to major traffic delays, detours, and costly repairs. The following fire incidents illustrate the magnitude of fire problem in bridges.

On July 15, 2009, a tanker truck carrying 13,000 gal of flammable liquid was involved in an accident with another truck under the 9-mi road overpass over the I-75 expressway near Hazel Park, Michigan. This bridge consisted of 10 hot-rolled steel girders in a 24-m span that supported a RC slab. The intense heat from the fire reached about 1,100°C, and this heat led to weakening of the steel girders and resulted in collapse of the overpass, as illustrated in Fig. 1. This collapse occurred in about 20 min after the start of the fire. It took about 105 min for the firefighters to extinguish the fire. This accident caused millions of dollars of damage. Preliminary recommendations called for rebuilding the entire 9-mi bridge over I-75, and the freeway below also needed some level of repair. It took several weeks of detours on I-75 and several months for repair of the bridge (National Steel Bridge Alliance 2010).

Another example of fire-induced bridge collapse is the one that occurred on the MacArthur Maze I-80/880 interchange in Oakland, California, on April 29, 2007. A tanker truck carrying 8,600 gal of gasoline overturned underneath the I-580 expressway, which consisted of six plate girders supporting a RC roadway. The firefighters responded to the spot within 14 min, but the intense fire from the accident resulted in temperatures reaching 1,100°C. This intense heat lead to loss of strength in the steel girders, and ultimately, the connections at the supports gave way, leading to collapse of two I-580 spans. The failure occurred at 22 min into the fire. Preliminary analysis revealed that the failure was due to overpressuring of connections under high-temperature effects. This incident cost $9 million to repair the bridge, and the retrofitting took months to finish (Astaneh-Asl et al. 2009).

The response of bridge structural members under fire can be different from that of buildings for the following reasons:

1. Fire source: The common source of ignition in bridges is the crashing of gasoline tanker trucks and burning of gasoline in the vicinity of the bridge. However, the common source of ignition in buildings is the burning of combustion materials (mostly wood and plastic-based products) in the compartment.

2. Fire ventilation: Most building fires are enclosed (compartment fires) and are limited by the amount of ventilation. However, bridge fires are generally under open-air conditions and have unlimited access to ventilation (oxygen).

3. Fire severity: Bridge fires can be much more intense than building fires and are representative of hydrocarbon fires because the fire source is generally gasoline.

4. Fire protection: In contrast to buildings, which are provided with active fire-protection systems such as sprinklers and passive fire protection such as fire insulation, no special fire-protection measures are provided in bridges.

5. Failure limit state: Bridge girders are much deeper than beams in buildings and may have quite slender webs. As a result,
shear failure due to web buckling is likely to dominate in bridge girders, whereas flexural failure is the dominant failure mechanism in beams.

6. Connections: Bridge girders are typically supported through bearing of the bottom flange. In contrast, the connections in buildings are made through the web and/or the flange. These variations in support conditions have an effect on the resulting fire resistance (Paya-Zaforteza and Garlock 2010).

A review of literature indicates that there is a lack of information on the fire performance of bridge girders, and this is mainly due to the fact that little attention has been paid to structural fire safety in bridges. The limited studies on the fire resistance of bridges are those reported by Dotreppe et al. (2006), Kodur et al. (2010), and Paya-Zaforteza and Garlock (2010). These studies clearly indicate that bridge fires are a significant problem and typically result from the crashing of vehicles in the vicinity of the bridge. The time to failure is typically less than 30 min, and thus very little time is available for firefighters to respond. Further, there is lack of data on the fire behavior of bridge girders as well as on the main factors that influence fire resistance. To overcome some of these drawbacks, a research project on the fire performance of steel bridge girders was carried out using the FEM computer program ANSYS multiphysics, 11.0 SP1. This program is capable of handling coupled and uncoupled thermomechanical problems. For the analysis, a simply supported steel bridge girder was selected. This bridge girder is generally comprised of different structural components, namely, girder, RC slab, and intermediate diaphragms. Two sets of discretization models were developed for undertaking thermal and mechanical analyses. The thermal-analysis results are applied as a thermal-body load on the structural model uniformly along the girder span.

FEM Model

To illustrate the response of a steel girder exposed to fire, a numerical study was carried out using the FEM computer program ANSYS (ANSYS multiphysics, 11.0 SP1). This program is capable of handling coupled and uncoupled thermomechanical problems. For the analysis, a simply supported steel bridge girder was selected. This bridge girder is generally comprised of different structural components, namely, girder, RC slab, and intermediate diaphragms. Two sets of discretization models were developed for undertaking thermal and mechanical analyses.

Thermal properties of steel and concrete, namely, thermal conductivity, specific heat, and thermal expansion, vary with

Discretization for Thermal Analysis

The heat-transfer analysis of the selected composite steel–concrete girder was carried out for two cases, namely, with and without stiffeners. For the discretization of the girder, slab, and stiffeners, SOLID70 elements were used. SOLID70 is a three-dimensional (3D) element with 3D thermal conduction capability and has eight nodes with a single degree of freedom, namely, temperature, at each node. This element is applicable to 3D steady-state or transient thermal analysis. The external surface areas of the SOLID70 elements that are exposed to fire, except the top surface of the slab, were used to simulate the surface effects of convection and radiation that occur from the ambient air to the steel girder. The discretization adopted for the thermal model is shown in Fig. 2.

The girder-slab assembly segment AB shown in Fig. 2(b) was meshed with SOLID70 elements. Both heat-convection and radiation loads were applied at the exposed surface areas of the solid element. Convection coefficients of $\alpha_c = 50 \text{ W/(m}^2\text{C)}$ and $\alpha_c = 35 \text{ W/(m}^2\text{C)}$ were used in the thermal analysis under hydrocarbon and external fire, respectively, and this is based on Eurocode 1 [European Committee for Standardization (CEN) 2002] recommendations. Depending on the exposure boundaries, different values of effective emissivity factor were used (CEN 2002). An effective emissivity factor of 0.7 was used for the bottom and side surfaces of the bottom flange of the girder. For the side surfaces of the web, an emissivity factor of 0.5 was used, whereas a factor of 0.3 was used for the top flange and bottom of the slab. This variation in emissivity factor is to reflect the fact that the web, top flange, and slab will experience slightly less radiation due to the effect of the larger girder depth of the girder section. A Stefan-Boltzmann radiation constant of $5.67 \times 10^{-8} \text{ W/(m}^2\text{C)}$ was applied in the thermal analysis.

The thermal properties of steel and concrete, namely, thermal conductivity, specific heat, and thermal expansion, vary with

![Fig. 1. Fire-induced girder collapse in Hazel Park overpass in Michigan (photograph from Zapletal 2009)](image)

![Fig. 2. Three-dimensional discretization of girder cross section for thermal analysis: (a) typical girder in a bridge; (b) 3D mesh of segment AB)](image)
temperature. In the analysis, the high-temperature properties are assumed to follow according to Eurocode 2 (CEN 2004) and Eurocode 3 (CEN 2005) provisions. The temperatures $T$ obtained via FEM analysis were averaged at every time step of fire exposure by taking the arithmetic mean of the temperatures at several points for each component (i.e., flange, web, or slab portion) of the steel girder–concrete slab composite section, as shown in Fig. 3.

Discretization for Structural Analysis

For structural analysis, the bridge girder was modeled with two elements, namely, element SHELL181 for the bottom flange, web, top flange, and stiffeners and element SOLID65 for the concrete slab. SHELL181 has four nodes with six degrees of freedom per node, three translations in $x$-, $y$-, and $z$-directions, and three rotations about the $x$-, $y$-, and $z$-axes. This element can capture buckling of flange and web as well as lateral torsional buckling of the member and therefore is well suited for large-rotation, large-strain, and nonlinear problems. SOLID65 has eight nodes with three degrees of freedom, namely, three translations in the $x$-, $y$-, and $z$-directions. This element can be used for 3D modeling of solids with or without reinforcement and is capable of accounting for cracking of concrete in tension, crushing of concrete in compression, creep, and large strains. The output from the thermal analysis (temperatures) was applied as a thermal-body load on the structural model to evaluate the mechanical response of a steel-concrete composite girder. The 3D structural model and the meshing adopted in the analysis are shown in Fig. 4(a).

To account for composite action between the concrete slab and the top flange of the steel girder, node-to-node interaction was discretized in the structural model as shown in Fig. 4(b). The same nodes are shared between the solid elements of the concrete slab and the shell elements of the top flange of the steel girder. To discretize the boundary condition in the structural FEM model, the support conditions of the bridge girder were applied on multiline nodes at the lower face of the bottom flange, as shown in Fig. 4(c). This boundary condition reflects a practical scenario, reduces the stress concentration at the boundary-condition nodes, and improves the convergence of the FEM solution. Also, to consider the continuity effect of the concrete slab, the slab was restrained from lateral movement [see Fig. 4(c)].

Material Properties

The progression of temperatures in the steel section under fire exposure depends on the fire scenario and the thermal properties of constituent materials, namely, thermal conductivity, specific heat, and thermal expansion, which vary as a function of temperature. The mechanical properties of steel and concrete that are critical for...
fire-resistance evaluation are stress-strain relationships and modulus of elasticity, which also vary with temperature. The temperature-dependent thermal and mechanical properties of steel and concrete are assumed to follow the Eurocode 2 (CEN 2004) and Eurocode 3 (CEN 2005) provisions. The thermal conductivity, specific heat, thermal expansion, and stress-strain curves of structural steel and concrete used in the analysis are given in Figs. 5–8. The fire insulation applied on the steel girder is CAFCO 300, and this insulation has a specified thermal conductivity of 0.078 W/(m °C) and a density of 240 kg/m³ at room temperature (Isolatik International 2008). CAFCO 300 is typically used for interior applications, but the thermal properties (i.e., thermal conductivity and specific heat) of this type of insulation used in exterior applications are quite similar to those of the insulation used in interior applications. The high-temperature thermal properties of the insulation (both thermal conductivity and specific heat) were assumed to follow the recommendations of Bentz and Prasad (2007). The temperature-dependent thermal conductivity and specific-heat value of the insulation used in the analysis is plotted in Fig. 9 as a function of temperature.

Model Validation

There is lack of fire test data on the responses of bridge girders under fire conditions. Therefore, validation of the ANSYS model we developed was carried out by selecting a steel-beam–concrete-slab assembly tested by British Steel Corporation (British Steel Technical and Swinden Laboratories 1989). This beam-slab assembly, typical of that in buildings, was tested under ISO 834 (ISO 1975) fire exposure. The validation process included comparison of both thermal and structural response predictions from the analysis with that reported in the fire test. The steel beam was not insulated. The beam-slab assembly, together with thermocouple layout, used in the test for measuring temperatures is shown in Fig. 10. The analysis was carried out with the mesh discretization and high-temperature properties discussed earlier. The assembly was exposed to ISO 834 fire exposure as in the fire test.

Fig. 11 shows a comparison of predicted steel temperatures by the FEM model with those measured in the fire test. It can be seen that the top flange of the beam experienced much lower temperatures compared with the bottom flange, and this is due to the effect of the concrete slab, which dissipates the temperature in the top flange because of the lower thermal conductivity and higher thermal capacity of concrete compared with steel. The web temperatures are slightly higher than those in the bottom flange, and this is due to the fact that the thickness of the web is much less than that of the flanges.

Overall, the predicted temperatures from the analysis compare well with the measured data from the test. The slight difference can be attributed to variation in the heat-transfer parameters, such as emissivity and convection coefficients, used in the analysis compared with the actual values in the test (furnace).

A comparison of midspan deflections predicted by the ANSYS model and those measured in the test is shown in Fig. 12. It can be seen that the midspan deflection gradually increases with time in the early stage of the fire (up to 10 min). These initial deflections are

![Graphs showing thermal properties as a function of temperature](image-url)
mainly due to high temperature gradients that develop between the top and bottom flanges of the steel section and the slight reduction in elastic modulus of steel resulting from increased temperatures in the girder. After 10 min, the rate of deflection increases slightly due to spread of plasticity, which results in faster strength and stiffness degradation of steel at high temperatures. At about 21 min when the bottom flange and web temperatures are about 600°C, the midspan deflection increases rapidly due to the effect of high-temperature creep, and failure of the girder occurs at 23 min through the formation of a plastic hinge at the midspan section.

Fig. 6. Variation in mechanical properties as a function of temperature for carbon steel: (a) yield strength and elastic-modulus reduction factor; (b) stress-strain relationship for Grade 50 steel

Fig. 7. Variation in thermal properties as a function of temperature for normal-weight concrete with siliceous aggregate: (a) thermal conductivity; (b) specific heat; (c) thermal expansion
Overall, predictions from the ANSYS model match well with the reported test data. The slight variation in deflections can be attributed to minor variations in idealization adopted in the analysis, such as different length that is exposed to fire in the test compared with that assumed in analysis. Whereas the entire span (4.5 m) was exposed to fire in the FEM model, only 4 m of the span assembly was exposed to fire during the test, and the support regions were outside the furnace, as reported by the British Steel Technical and Swinden Laboratories (1989). It can be seen that the ANSYS model can predict the time to failure with good acceptability. For instance, the predicted failure time was 22.5 min compared with 23 min in the test considering deflection limit state as the governing failure criterion.

Case Study

Selection of Bridge Girder

To evaluate the response of a typical bridge girder under fire conditions, a simply supported steel bridge girder was selected for analysis from the literature (Paya-Zaforteza and Garlock 2010). The steel bridge consisted of five hot-rolled steel girders of W33 × 141 supporting a RC slab 200 mm thick. The steel girder is assumed to be in full composite action with the slab and to be laterally supported by transverse diaphragms at the midspan and at the both ends to prevent lateral movement, as shown in Fig. 13. The bridge girder is 12.2 m in span length and has two expansion joints at its ends with a width of 36 mm. The girders are fabricated from Grade 50 steel (yield strength 350 MPa), whereas the concrete used in the slab has a compressive strength of 30 MPa.

Varied Parameters

The validated FEM model was applied to evaluate the fire resistance of the bridge girder under five different cases in which the fire scenario, fire protection, and composite action arising from steel-concrete interaction are variables. Table 1 shows the test parameters and summary of results from the analysis. The analysis was carried out under two fire scenarios, namely, hydrocarbon and external fire.
To study the effect of the composite action arising from steel girder–concrete slab interaction, two cases were considered. In Case 1, composite action from the slab was neglected, and the strength analysis corresponds to that from a plain girder only. In Case 2, the effect of the slab was included by considering full composite action between the steel girder and the concrete slab. In Case 3, Case 2 was replicated under an external fire scenario to evaluate the effect of the fire scenario on the fire resistance of the girder. External fire, which is less severe than hydrocarbon fire, can be a possible fire scenario in bridges (CEN 2002). The time-temperature curves representing hydrocarbon and external fires are shown in Fig. 14. The effect of fire insulation on girders was...
Table 1. Summary of Test Parameters and Results from Case Study

<table>
<thead>
<tr>
<th>Case</th>
<th>Parameter</th>
<th>Fire scenario</th>
<th>Time to failure (min)</th>
<th>Maximum midspan deflection (mm)</th>
<th>Maximum horizontal displacement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>No composite action</td>
<td>Hydrocarbon fire</td>
<td>12</td>
<td>-326.6</td>
<td>167.7</td>
</tr>
<tr>
<td>Case 2</td>
<td>Full composite action</td>
<td>Hydrocarbon fire</td>
<td>21</td>
<td>-164.6</td>
<td>110.2</td>
</tr>
<tr>
<td>Case 3</td>
<td>Fire scenario</td>
<td>Hydrocarbon fire</td>
<td>No failure</td>
<td>-91.7</td>
<td>113.4</td>
</tr>
<tr>
<td>Case 4</td>
<td>Fire insulation (12.5 mm)</td>
<td>Hydrocarbon fire</td>
<td>61</td>
<td>-191.6</td>
<td>112.1</td>
</tr>
<tr>
<td>Case 5</td>
<td>Fire insulation (25 mm)</td>
<td>Hydrocarbon fire</td>
<td>107</td>
<td>-185.2</td>
<td>119</td>
</tr>
</tbody>
</table>

Fig. 13. Elevation and transverse section of the bridge girder: (a) elevation; (b) section near the supports

Fig. 14. Time-temperature curve for typical fire scenario
considered in Cases 4 and 5 using two insulation thicknesses, namely, 12.5 and 25 mm.

The fire-resistance analysis on the bridge girder was carried out under an applied loading consisting of dead load plus 30% live load. The self-weight of a girder section (2.0 kN/m) and that contributed by the tributary area of the concrete slab and wearing surface of the deck (22.5 kN/m) were considered in the dead load according to AASHTO provisions (AASHTO 2009). For the live load, a uniformly distributed load (9.3 kN/m) representing 0.3 times the live load was applied.

Results and Discussion

Results from ANSYS thermal analysis for Case 2 are plotted in Fig. 15, which shows the temperature distribution of the steel-concrete composite girder as a function of time for the case of hydrocarbon fire exposure. It can be seen in Fig. 15 that the top-flange temperature is much lower than that of the bottom flange. This is mainly due to the insulating effect of the concrete slab, which dissipates heat from the top flange to the slab. Also, the temperatures in the web are slightly higher than those of the bottom flange, and this is because the web is much more slender (lower thickness) than the flanges, and this produces a rapid rise in web temperatures.

The development of thermal gradients across the girder-slab cross section is plotted in Fig. 16 for Case 2 (noninsulated girder) and Case 4 (insulated girder) of the analysis. The thermal gradient is computed as the temperature difference between the middepth of the slab and the middepth of the web. At 20 min, the thermal gradients is 950°C in Case 2 versus 500°C in Case 4. The significant thermal gradient that develop along the depth of the cross section in Case 2 results from higher temperatures in the bottom flange compared with the concrete-slab temperature. However in Case 4, the presence of insulation minimizes the development of significant thermal gradients until about 60 min, at which point it reaches about 875°C. Therefore, the insulated bridge girder in Case 4 survived longer time (61 min) under fire compared with the uninsulated girder in Case 2 (21 min). Thermal gradient is also influenced by the type of fire scenario. This can be seen by comparing resulting temperature gradients from Case 2 and Case 3. For Case 2 (under a hydrocarbon fire scenario), the gradient is 945°C at 60 min, whereas in Case 3 (under an external fire scenario), the gradient is only 530°C at 60 min. This is attributed to the fact that an external fire is less severe (lower fire temperatures) compared with a hydrocarbon fire (see Fig. 14). In general, higher thermal gradients produce higher thermal strain at the bottom of the steel girder (and in the web) compared with that in the concrete slab. Thus, a significant curvature (thermal bowing) develops in the girder, resulting in higher thermal stresses even in a statically determinate girder (unrestrained girder). The developed curvature at the initial stage of fire exposure is independent of applied load because this curvature results mostly from the thermal-gradient effect. Therefore, the curvature resulting from the thermal gradients alone contributes to the deflection at the early stage of fire exposure. Once steel temperatures exceed 400°C, girder deflections increase significantly due to degradation of the mechanical properties of steel.

As shown in Fig. 16, the temperatures across the web is uniform. Results from thermal analysis show that temperature distribution

**Fig. 15.** Temperatures progression in a bridge girder subjected to hydrocarbon fire exposure (Case 2)

**Fig. 16.** Thermal gradient along the depth of a bridge girder section for Cases 2 and 4: (a) cross section; (b) Case 2; (c) Case 4
across the bearing stiffeners is quite similar to that in the web. This is so because of the similarity in the slenderness of stiffeners (depth and thickness) compared with that of the web. Therefore, the same temperatures were applied to the web and the bearing stiffeners during the structural analysis.

The structural response of a typical bridge girder is illustrated in Fig. 17, where midspan deflection of the girder is plotted as a function of fire exposure time. These deflection curves are plotted for the five cases considered in the analysis. The general trend of the deflection progression can be grouped to different stages. At the early stage of the fire, midspan deflection increases linearly up to first yielding, which depends on the temperature progression in the girder cross section. Therefore, the time at which yielding occurs is different in different cases. The deflections start to increase with progression of fire exposure time due to the spread of plasticity and deterioration in strength and stiffness properties of steel and concrete at high temperatures. Toward the final stages of fire exposure, the midspan deflection, in all cases except Case 3, increases rapidly due to high-temperature creep.

The failure mode in Case 1 (with no composite action) was by yielding of the bottom flange (flexural) because there is no slab effect in this case. For Cases 2, 4, and 5, where full composite action exists with the slab, failure of the girder is through yielding of the web near the supports. This can be attributed to the fact that the flexural capacity of the girder is enhanced as a result of composite action arising from steel-concrete interaction; however, shear capacity does not get enhanced significantly. In fact, the web looses strength at a faster rate than the flanges due to the faster raise in temperature in the slender web compared with the flanges. In Case 3, the girder survives burnout conditions, and this is due to the less severe fire temperatures resulting from an external fire, where the maximum fire temperature reaches only 680°C.

A summary of the analysis results, including midspan deflection and failure time, for five cases is presented in Table 1. The structural response of the girder under fire conditions (and thus its fire resistance) is enhanced when the composite action is accounted for in the analysis. The time to failure increases from 12 min in Case 1, with no composite interaction, to 21 min in Case 2 by considering the fully composite interaction between the concrete slab and steel girder. Both cases, Case 1 and Case 2, were analyzed under a hydrocarbon fire scenario, and the same level of thermal gradients developed in both cases. Also in Case 1, deflections progress at a rapid rate, and the maximum midspan deflection at failure was 327 mm compared with 165 mm in Case 2. This is so because of the absence of a concrete slab effect and the rapid degradation stiffness of steel at high temperatures. The significant enhancement of structural performance (and related fire resistance) in Case 2 can be attributed to the contribution of the concrete slab to the flexural capacity of the girder assembly under fire. Further, the flexural capacity of the assembly is governed by the strength of steel section and concrete slab at the early stage. But, with the faster degradation of strength properties of steel at high temperature, the steel section mainly governed the capacity of the girder. The concrete (in the slab) remained adequate to resist the forces from the steel due to slower loss of strength properties in concrete and also lower temperatures in the slab. As a result, concrete helps to sustain the applied load at later stages under fire. Therefore, the bridge girder with composite action survived for a longer time than the bridge girder with no composite action.

The effect of fire scenarios on fire performance of the bridge girder can be illustrated by comparing results from Case 2 and Case 3, as tabulated in Table 1. In Case 3, the bridge girder survived burnout conditions under an external fire scenario compared with failure in 20 min under a hydrocarbon fire exposure in Case 2. This is attributed to the fact that the external fire is less severe than the hydrocarbon fire. For instance, the maximum fire temperature attained in a hydrocarbon fire is about 1,100°C compared with 680°C in the case of an external fire. Also, the heating rate is much higher in a hydrocarbon fire than in an external fire producing higher thermal gradients in the section. This differentiation in peak temperature and heating rate between these two fire scenarios lead to slower deterioration in strength and stiffness properties in Case 3 compared with Case 2. As a result, the bridge girder sustained the applied load for the entire fire duration under external fire exposure. The steady-state deflection in Case 3 after 30 min is due to the fact that the girder reached thermal equilibrium (680°C across the girder), as indicated in the structural-analysis results under external fire exposure. At the same time, though, the moment capacity of the girder is still higher than the applied moment. Therefore, no failure resulted in Case 3.

The effect of fire insulation on fire resistance can be gauged by looking at results from Cases 2, 4, and 5 in Table 1. In Cases 4 and 5, the girder is insulated with 12.5 and 25 mm of fire insulation, whose thermal properties are as discussed earlier and presented in Fig. 9. The midspan deflection in these three cases is compared in Fig. 17 as a function of time. It can be seen that fire insulation has a significant influence on failure time of the bridge girder. The insulation effect minimized the temperature rise in the steel girder, which slowed down deterioration of the strength and stiffness properties of steel. Furthermore, using insulation decreased the thermal gradient across the girder section significantly, as shown in Fig. 16. This extended the failure time of girder to 61 min in Case 4 and 107 min in Case 5 compared with 21 min in Case 2, where the girder is uninsulated.

**Conclusions**

A nonlinear FEM analysis was applied to evaluate the response of bridge girders under fire conditions. Based on the results of analysis, the following conclusions can be drawn:

1. Fire can be a significant hazard in steel-girder bridges under some scenarios. Currently, there is limited information on the fire performance of steel bridges.
2. The fire behavior of bridge girders can be significantly different from that of beams in buildings due to different fire, loading, geometry, and sectional characteristics. Therefore, the available fire-resistance information from building structures cannot be applied directly to bridge girders.
3. ANSYS can be applied successfully to model the fire response of bridge girders. The thermal response can be simulated using SOLID70 elements, whereas structural response can be simulated using SHELL181 and SOLID65 elements.

4. Composite action arising from steel-girder–concrete-slab interaction significantly enhances the fire resistance of bridge girders. Thus composite action is to be accounted for properly in the evaluation of the fire resistance of bridge girders.

5. The type of fire exposure and the presence of insulation have a significant influence on the resulting fire resistance of bridge girders.

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References

AASHTO. (2009). LRFD bridge design specifications, AASHTO, Washington, DC.


