The 2019 World Congress on
Advances in Structural Engineering and Mechanics (ASEM19)
Jeju Island, Korea, September 17 - 21, 2019

Fire response of horizontally curved continuous composite bridge girders

*Gang Zhang[1)], Chaojie Song [2)], Qiao Huang [3)], Weifa Yao [4)] and Shuanhai He [5)]

[1), 2), 5)] *School of Highway, Chang'an University, Xi'an 710064, China*
[3), 4)] *School of Transportation, Southeast University, Nanjing 210096, China*
[1)] *zhangg_2004@126.com*

ABSTRACT

Fire severely threats the structural safety in a bridge; however, no provisions are specified in current codes and standards. Horizontally curved steel-concrete composite bridge girders, widely used in bridge construction at interchange, have significant bending-torsion coupled effect even under transversely symmetric loads, which makes the fire response significantly different from that of horizontally straight composite bridge girders. To bridge this gap, a 3-D nonlinear finite element (FE) model, established using computer program ANSYS and validated through experimental data, is utilized to trace the fire response of a typically horizontally curved three-span continuous composite bridge girder. The bridge girder in single span is exposed to localized fire individually, i.e., fire exposure on the first span or the second span. Subsequently, the predicted results including deflection and web buckling, are used to quantitatively evaluate fire resistance of horizontally curved composite bridge girders. Results from analysis results show that the outer steel girder fails earlier than the inner steel girder when the bridge girder is subjected to transversely symmetric fire and structural loading. The bridge girder at the end of localized fire exposure length along the span close to the hogging moment zone is vulnerable to failure. Deflection and rate of deflection based failure criterion cannot be applicable to evaluate fire resistance of horizontally curved continuous composite bridge girders.

1. INTRODUCTION

Steel-concrete composite bridge girders make full use of tensile properties of steel and compressive performance of concrete, and thus possess many advantages, e.g., small section size, light weight (strong crossing ability), good seismic performance, rapid construction and good ductility. Therefore, steel-concrete composite bridge girders are widely used in interchange bridge construction using continuous system (Hasan et al.

[1)] Professor
[2)] Ph.D. student
[3)] Professor
[4)] Ph.D.
[5)] Professor

2017). However, steel members used in composite bridge girders exhibit lower fire resistance as compared to similar structure members in concrete bridge girders. This can be attributed to its high thermal conductivity, low specific heat, and rapid degradation of strength and modulus at elevated temperatures (Aziz et al. 2015).

Bridge fires are commonly caused by overturning or crashing of tankers and violent burning of petrochemicals in the vicinity of a bridge, which are more severe than building fires (Gong and Agrawal 2015). These fires are characterized by rapid heating rates and high fire intensities, which temperature can reach 1200 ℃ within the first few minutes of fire exposure (Naser and Kodur 2015). Following these fires, traffic on fire damaged bridge is usually hard to detour, and thus has a negative impact on the traffic quality in the region (Paya-Zaforteza and Garlock 2012). These fire hazards usually cause enormously direct (repair and reconstruction costs) and indirect (traffic delays) economic losses (Kodur and Naser 2019). Although the probability of fire broke out on bridges is much lower than that in buildings, bridge fires may be more devastating due to lack of fire protection measures (active and passive fire protection) and difficult firefighting rescue (Naser and Kodur 2015). Furthermore, steel-concrete composite bridge girders, under high intense fire, are more susceptible to failure due to their slenderness as compared to similar structural member in buildings, especially in hydrocarbon fire (Glassman et al. 2016). While provision of fire resistance to structural members in buildings is a mandatory design requirement, no special requirements are provided for enhancing fire safety of bridges as per current standards and codes (Hu et al. 2018, Peris-Sayol et al. 2016).

In past few decades, several researchers have made limited studies on structural behavior of composite girders under fire exposure by performing FE simulation or experiments (Aimin et al. 2013, Alos-Moya et al. 2017, Aziz et al. 2015, Beneberu and Yazdani 2018, Kodur et al. 2013, Paya-Zaforteza and Garlock 2012, Zhang et al. 2017, Zhang et al. 2019). These studies are concentrated on web slenderness, stiffener spacing, composite action, axial restraint, material types, live load, structural shape and fire scenarios. Further, these selected girders are commonly fabricated using single I-shaped steel girder with simply supported conditions. A realistic composite bridge girder is much complex in structural system, sectional configuration and supporting conditions. Information from the previous literature cannot reflect a realistic bridge girder suffering from a fire scenario in practice. In addition, there is very limited research on performance of curved bridge girders under fire exposure. A horizontally curved composite bridge girder exhibits significant bending-torsion coupling effects even under transversely symmetric loads, which makes its fire response significantly different from that of a straight composite bridge girder.

Within this general context, considering that curved bridge is quite commonly used, this paper delves into the fire response of horizontally curved continuous composite bridge girders.

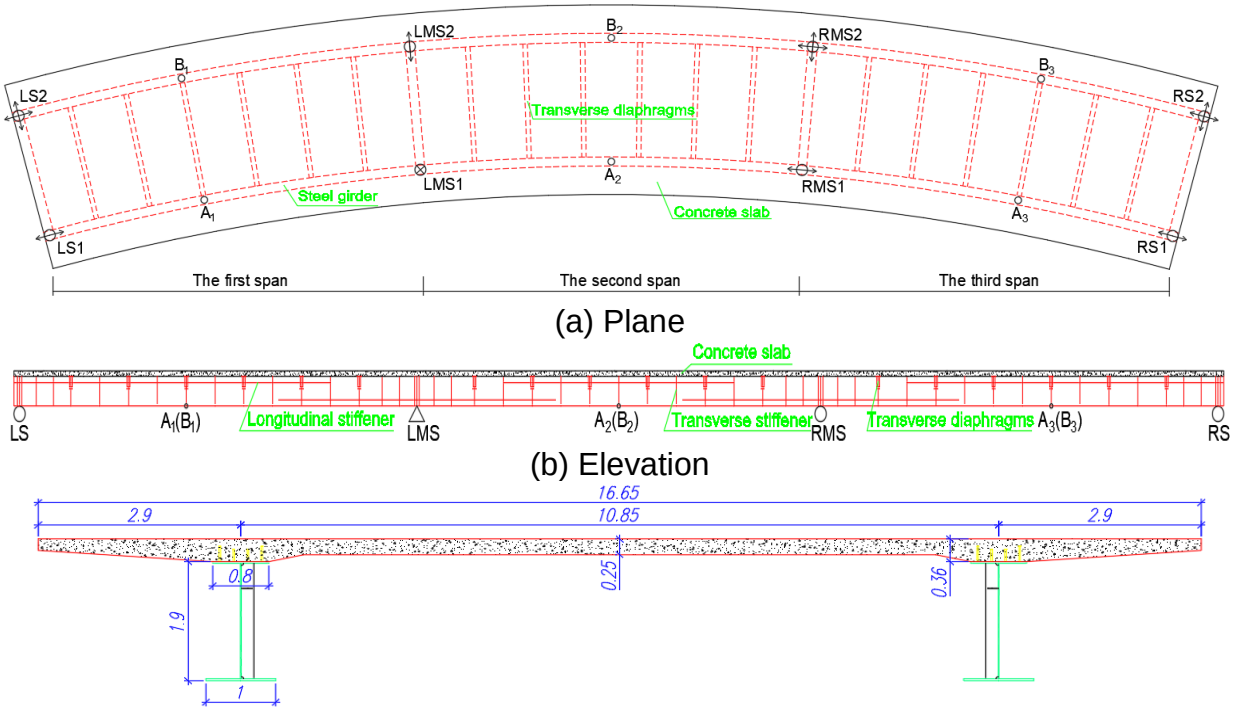
2. PROTOTYPE AND ANALYZED PARAMETERS

2.1 Prototype

A typically horizontally curved three-span continuous steel-concrete composite bridge girder, comprising of twin I-shaped steel girder supporting a concrete slab, is

selected to carry out fire resistance analysis. The bridge spans 34.95 m length in both two side spans, 35 m length in middle span, and the total length is 104.9 m. The width and height of the composite bridge girder are 16.65 and 2.26 m, respectively. Transverse diaphragms, placed between two main girders, with spacing of 5 m along the span length, are designed to enhance lateral stability. Thickness of concrete slab between two main girders is 0.25 m and that on the top of steel girders is 0.36 m. Transverse prestressing strands embedded in concrete slabs is utilized to improve its structural transverse stiffness.

The concrete slab is made of normal-strength concrete using siliceous aggregates with cube strength of 50 MPa and density of 2500 kg/m^3 at room temperature. Steel with yield strength of 345 MPa and density of 7850 kg/m^3 is used for fabrication of the girder. Details of this composite bridge girder is shown in Fig. 1.



(a) Plane

(b) Elevation

(c) Cross section (units: m)

Fig. 1 Details of horizontally curved continuous composite bridge girders

2.2 Parameters of study

To investigate the fire response of horizontally curved continuous composite bridge girder, the first span and the second span are subjected to localized fire and transversely symmetric structural loading, respectively, as shown in Fig. 2. All sides of the composite girder assembly except the top surface of the concrete slab are exposed to fire.

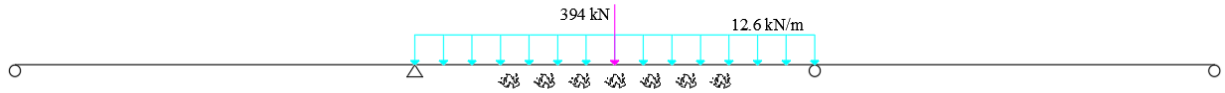
The applied structural loads, consisting of self-weight of the composite bridge girder, pavement and live load are illustrated as follows:

1) The self-weight of the composite bridge girder is applied automatically through the designated dimensions with density of steel and concrete.

2) The weight of pavement, equivalent to a surface load of 2.4 kN/m^2, is applied on the top of concrete slab.

3) Vehicular loads (live load), comprised of a concentrated force and uniformly distributed load, are distributed as per the worst-case along the girder span, as shown in Fig. 2. In which a concentrated force (394 kN) and uniformly distributed load (12.6 kN/m), representing 0.3 times of live load is applied on the composite bridge girder. Also, transverse vehicular loads are laid symmetrically about the central line of the bridge deck, as shown in Fig. 3.

(a) Case 1



(b) Case 2

Fig. 2 Load distribution and fire scenarios

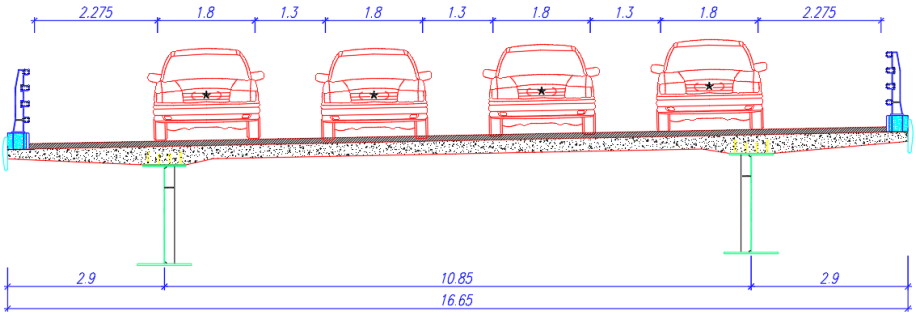


Fig. 3 Lateral distributions of vehicular loads

3. NUMERICAL ANALYSIS METHOD

The numerical model is developed using the finite element computer program ANSYS. Different structural components, including steel girder, concrete slab, prestressing strands and stiffeners, are taken into consideration to establish the numerical model. The model incorporating temperature-dependent material properties and geometric nonlinearity is used to perform fire resistance analysis of the composite bridge girder.

3.1 Discretization

A complex FE model with solid and shell elements (see Fig. 4), instead of a simpler model with beam element, is used to capture local buckling that might control the fire response of the composite bridge girder. Two sets of discretization models are established to undertake thermal and structural analysis. For thermal analysis, steel girder, concrete slab and prestressing strands are discretized with three types of elements in the computer program ANSYS namely, “SHELL57”, “SOLID70” and “LINK33” element (ANSYS 2018). Transient nodal temperatures with respect to fire exposure time are calculated using thermal properties and heat transfer conditions.

Nodal temperatures generated from thermal analysis, together with temperature-dependent mechanical material properties, are applied on the structural

model to obtain fire response of the composite bridge girder. The structural model is established through structural elements namely, “SHELL181”, “SOLID65” and “LINK180” element (ANSYS 2018).

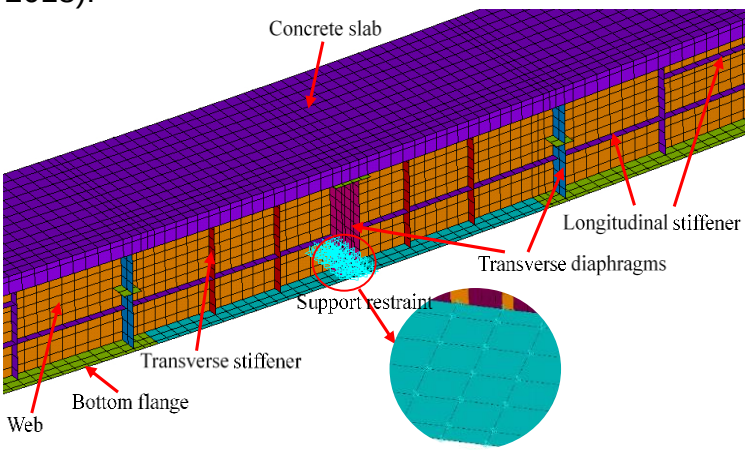


Fig. 4 Discretization of composite bridge girder

3.2 Material properties

The properties of constituent materials play a crucial role in determining the fire response of a bridge girder. Thus, relevant material properties are incorporated into each stage of the analysis. The progression of temperature within girder section depends on the fire intensity and thermal properties namely, density, thermal conductivity, and specific heat. These required material properties, including thermal expansion, elastic modulus and stress-strain relations, are assumed to comply with Eurocode (2014), which are as input data to fulfill structural analysis.

Plastic damage model of concrete is defined using Willam and Warnke model. The required parameters are open, close crack shear transfer coefficient and tensile strength. The open and close crack shear transfer coefficient of concrete is 0.2 and 0.7, respectively (Kodur and Shakya, 2017).

3.3 Boundary conditions

A nominal fire curve, namely hydrocarbon fire, is used to perform the transient heat transfer analysis. The external surface of the element “SHELL57” and “SOLID70” is used to simulate heat transfer from fire zone to the exposed surface of the girder through heat radiation and convection. Convection coefficient is assumed to be 50 $W/m^2{\cdot}K$ at request of Eurocode (2002). Different values of effective emissivity factors are applied on the exposed surface due to different radiation distance from fire source to the external surface in the girder. An emissivity factor of 0.7 is used for bottom flange, 0.5 is used for web and 0.3 is used for top flange and bottom of concrete slab (Zhang et al. 2019). Stefan-Boltzmann constant value of $5.67{\times}10^{-8}$ $W/(m^2{\cdot}K^4)$ is used for the thermal analysis.

For structural analysis, same nodes partake between solids elements of concrete slab and shell elements of top flange of the steel girder to account for composite action between the steel girder and the concrete slab.

According to the drawing in the selected composite bridge girder (see Fig. 1), all degree of freedoms (DOFs) at left middle support (LMS) 1 is fully fixed except the

rotation about the horizontal axis along transverse direction. At the other supports of the inner steel girder, the tangential DOF and the same rotation are free, while all other DOFs are constrained. The support DOFs of the outer steel girder is the same as that of the inner steel girder at the corresponding position, except for the radial DOF is free.

To account for the realistic boundary conditions in the structural model, the support conditions of the composite bridge girder are simulated using multi-line nodes at lower face of bottom flange, as shown in Fig. 4. These boundary conditions reflect the actual constraint scenarios in a practically horizontally curved continuous composite bridge girder.

3.4 Failure criteria

The horizontally curved composite bridge girder is deemed to reach failure state as long as any one of following criteria occurs.

(1) The deflection exceeds L/30, i.e. side span deflection exceeds 1135 mm and middle span deflection exceeds 1153 mm (Aziz et al. 2015).

(2) The rate of deflection calculated over 1 min intervals, exceeds $L^2/9000d$, where L is the clear span length and d is the depth of the girder (BS476-20 2014). Thus, the rate of deflection of side span exceeds 57 mm/min and that of middle span exceeds 59 mm/min in this paper.

(3) The rapid increase of the web out-of-plane displacement (web buckling) occurs.

3.5 Model validation

The above discussed numerical model is validated successfully through experimental data generated from a composite box bridge girder during fire test (Zhang et al, 2019).

4. RESULTS AND DISCUSSION

Data generated form FE analysis is utilized to trace the thermal and structural response of composite bridge girder under fire conditions.

4.1 Thermal analysis

The temperature evolution with in girder section is plotted as a function of fire exposure time, as shown in Fig. 5. Temperatures within concrete slab are significantly lower than that in steel girder due to lower thermal conductivity in concrete.

Temperature in web is higher than that in bottom flange resulting from thinner thickness in web. Top flange as compared to bottom flange has a lower temperature, and this can be attributed to the fact that much heat in top flange dissipate in to the adjacent concrete slab. Thus, web is prone to early buckling than flange. Also, the thinnest stiffener exhibits the highest temperature among all components in steel girder throughout entire fire duration. Therefore, temperature in steel girder is greatly affected by thickness in each portion.

High temperature in the steel girder and low temperature in the concrete slab lead to significant thermal gradients along the girder depth. This temperature difference

increases with fire exposure time, thus causing significant downwards thermal curvature along the girder span.

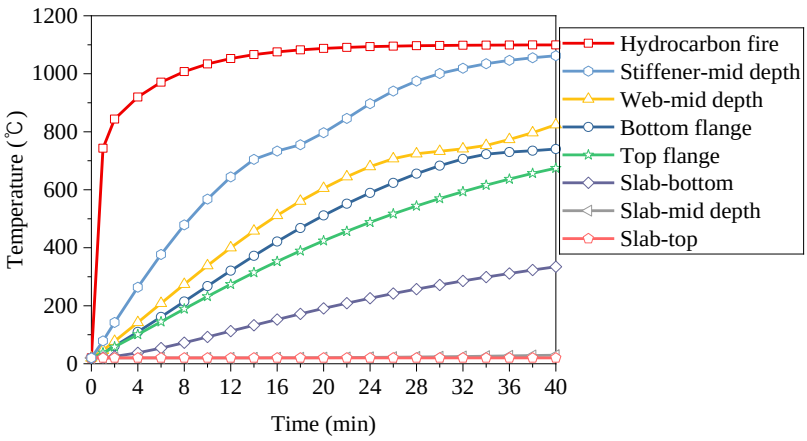


Fig. 5 Temperature evolution with fire exposure time

4.2 Structural response

The structural response of horizontally curved continuous composite bridge girder is analyzed through tracing mid-span deflection and web out-of-plane displacement. These structural responses are utilized to quantitatively evaluate fire resistance of composite bridge girder.

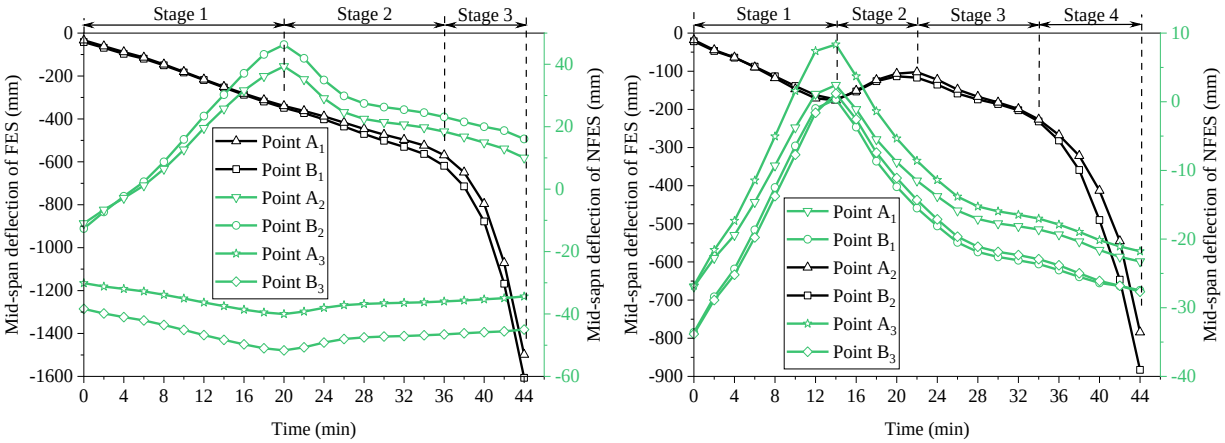
4.2.1 Mid-span deflection

Comparison of the mid-span deflection (points A_1-B_3 in Fig. 1) progression between fire exposed span (FES) and non-fire exposed span (NFES) is shown in Fig. 6. The negative value indicates downward deflection, while the positive value indicates upward deflection. The mid-span deflection of FES generally increases with fire exposure time. However, mid-span deflection in case 2 is much smaller than that in case 1 because the two side spans restrain its deformation. The mid-span deflection in outer steel girder is larger than that in inner steel girder when the bridge girder is subjected to transversely symmetric fire and structural loading.

For case 1, the general trend of mid-span deflection progression of the first span can be grouped into three stages, namely stage 1, 2 and 3. In stage 1, the mid-span deflection of the first span increases linearly mainly due to significant thermal gradients (see Fig. 5) and degradation of stiffness. The mid-span deflection in the second span deflects upwards due to the compatibility of deformation in the continuous composite bridge girder. In stage 2, the mid-span deflection in the first span increases mainly due to degradation of the strength and stiffness properties of the steel. In this stage, plastic hinge 1 begins to form at the end of localized fire exposure length along the span close to the hogging moment zone. This plastic hinge leads to the upward deflection in the second span turn downward. In the final stage of fire exposure, plastic hinge 1 expands further and plastic hinge 2 appears at mid-span in the first span. This variation results in the rapid increase of deflection.

For case 2, the general trend of mid-span deflection progression in the second span can be grouped into four stages, namely stage 1, 2, 3 and 4. In stage 1, the deflection behavior is identical with that in case 1. In stage 2, the second span start to

deflect upward, and this due to the presence of plastic hinge 1 (web buckling at the end of localized fire exposure length along the span close to the hogging moment zone). In stage 3, flexural stiffness of the second span decreases significantly due to degradation of the strength and stiffness properties of the steel. This lead to the mid-span deflection in the second span deflects downward again. In stage 4, similar to stage 3 in case 1, the plastic hinge 2 appears at mid-span in the second span, and thus the mid-span deflection in the second span increases rapidly.



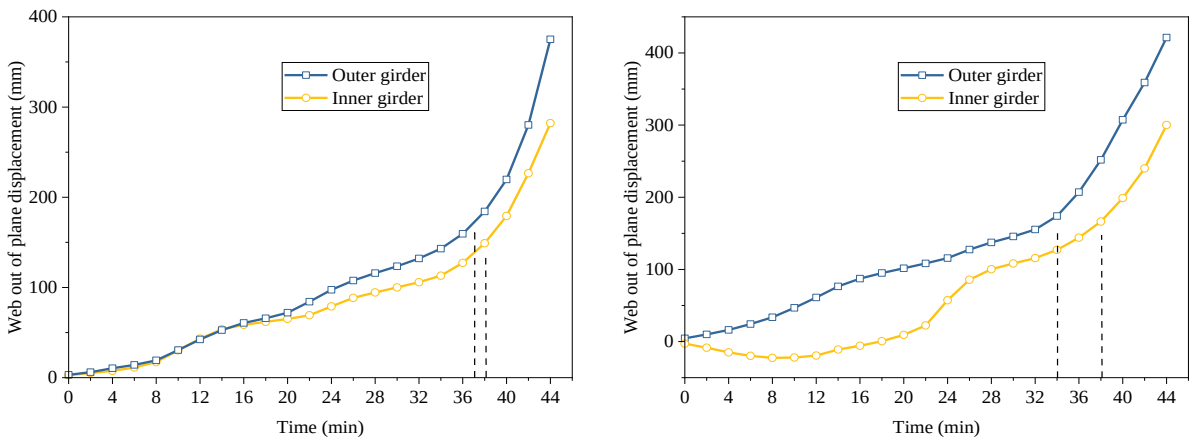
(a) Case 1 (b) Case 2

Fig. 6 Comparison of FES and NFES deflection variation with time

4.2.2 Web out-of-plane displacement

Web out-of-plane displacement (web buckling) at the end of localized fire exposure length along the span close to the hogging moment zone is analyzed, as shown in Fig. 7. The web out-of-plane displacement in the outer girder is larger than that in the inner girder even under transversely symmetric fire and structural loading. Web buckling in case 2 is more severe than that in case 1.

The web out-of-plane displacement increases linearly with fire exposure time from fire ignition. The web out-of-plane displacement increases rapidly after 37 min in case 1 and 34 min in case 2. This variation indicates the failure in web.



(a) Case 1 (b) Case 2

Fig. 7 Variation of web out-of-plane displacement as a function of time

4.2.3 Fire resistance

A summary results of fire resistance computed through numerical analysis on horizontally curved continuous composite bridge girder is presented in Table 1. The curved shape causes the failure time in the two steel girders slightly different. The outer girder generally fails earlier than the inner girder. The failure of the composite bridge girder occurs at 37 min and 34 min in case 1 and 2, respectively. Therefore, web buckling based failure criterion is recommended to determine the failure of horizontally curved continuous composite bridge girders. Although the mid-span deflection of FES in case 1 is larger than that in case 2, fire resistance is higher than that in case 2 based on web buckling failure criterion.

Table. 1 Fire resistance based on failure criteria

Case		Fire resistance (min)		
		Deflection	Rate of deflection	Web buckling
Case 1	Outer girder	42	38	37
	Inner girder	43	39	38
Case 2	Outer girder	N.F.	39	34
	Inner girder	N.F.	40	38

Note: N.F. = no failure.

5. CONCLUSIONS

In this paper, a numerical method is used to delve into fire response of horizontally curved composite bridge girders. The following conclusions can be drawn from the numerical analysis:

(1) Proposed numerical analysis method is capable of predicting fire response of horizontally curved continuous composite bridge girders during simultaneous application of fire and structural loading.

(2) The outer steel girder fails earlier than the inner steel girder, even under transversely symmetric fire and structural loading.

(3) A plastic hinge is firstly formed at the end of localized fire exposure length along the span close to the hogging moment zone. Then, another plastic hinge appears at the mid-span of the fire exposed span.

(4) Web buckling is a more reliable indicator evaluating failure of horizontally curved composite bridge girders under fire conditions.

REFERENCES

Aimin, Y., Yuli, D., and Litang, G. (2013). “Behavior of unbonded prestressed continuous concrete slabs with the middle and edge span subjected to fire in sequence”, *Fire Saf. J.*, **56**, 20-29.

Alos-Moya, J., Paya-Zaforteza, I., Hospitaler, A., and Rinaudo, P. (2017), “Valencia bridge fire tests: Experimental study of a composite bridge under fire”, *J. Constr. Steel. Res.*, **138**, 538-554.

ANSYS (2018), ANSYS metaphysics (Version 19.0), ANSYS Inc., Canonsburg, USA.

Aziz, E.M., Kodur, V.K., Glassman, J.D. and Garlock, M.E.M. (2015), "Behavior of steel bridge girders under fire conditions", *J. Constr. Steel. Res.*, **106**, 11-22.

Beneberu, E., and Yazdani, N. (2018), "Performance of CFRP-Strengthened Concrete Bridge Girders under Combined Live Load and Hydrocarbon Fire", *J. Bridge Eng.*, **23**(7), 04018042.

BS476-20 (2014), Fire tests on building materials and structures. Part 20: Method for determination of the fire resistance of elements of construction, BSI; London, UK.

Eurocode (2002), Actions on structures. Part 1.2: General actions - Actions on structures exposed to fire, European Committee for Standardization; Brussels, Belgium.

Eurocode (2014), Design of composite steel and concrete structures. Part 1.2: General rules - Structural fire design, European Committee for Standardization; Brussels, Belgium.

Glassman, J.D., Garlock, M.E.M., Aziz, E.M., and Kodur, V.K. (2016), "Modeling parameters for predicting the postbuckling shear strength of steel plate girders", *J. Constr. Steel. Res.*, **121**, 136-143.

Gong, X. and Agrawal, A.K. (2015), "Numerical simulation of fire damage to a long-span truss bridge", *J. Bridge Eng.*, **20**(10), 04014109.

Hasan, Q.A., Badaruzzaman, W.W., Al-Zand, A.W., and Mutalib, A.A. (2017), "The state of the art of steel and steel concrete composite straight plate girder bridges", *Thin-Walled Struct.*, **119**, 988-1020.

Hu, J., Usmani, A., Sanad, A., and Carvel, R. (2018), "Fire resistance of composite steel & concrete highway bridges", *J. Constr. Steel. Res.*, **148**, 707-719.

Kodur, V., Aziz, E. and Dwaikat, M. (2013), "Evaluating fire resistance of steel girders in bridges", *J. Bridge Eng.*, **18**(7), 633-643.

Kodur, V.K.R., and Naser, M.Z. (2019), "Designing steel bridges for fire safety", *J. Constr. Steel. Res.*, **156**, 46-53.

Kodur, V.K.R., and Shakya, A.M. (2017), "Factors governing the shear response of prestressed concrete hollowcore slabs under fire conditions", *Fire Saf. J.*, **88**, 67-88.

Naser, M.Z., and Kodur, V.K.R. (2015), "A probabilistic assessment for classification of bridges against fire hazard", *Fire Saf. J.*, **76**, 65-73.

Paya-Zaforteza, I. and Garlock, M.E.M. (2012), "A numerical investigation on the fire response of a steel girder bridge", *J. Constr. Steel. Res.*, **75**, 93-103.

Peris-Sayol, G., Paya-Zaforteza, I., Balasch-Parisi, S., and Alós-Moya, J. (2016), "Detailed analysis of the causes of bridge fires and their associated damage levels", J. Perform. Constr. Facil., **31**(3), 04016108.

Zhang, G., Kodur, V., Hou, W., and He, S. (2017), "Evaluating fire resistance of prestressed concrete bridge girders", *Struct. Eng. Mech.*, **62**(6), 663-674.

Zhang, G., Kodur, V., Yao, W., and Huang, Q. (2019), "Behavior of composite box bridge girders under localized fire exposure conditions", *Struct. Eng. Mech.*, **69**(2), 193-204.