# PERFORMANCE OF COMPOSITE CONCRETE BRIDGE DECK UNDER HYDROCARBON POOL FIRE EXPOSURE

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## ABSTRACT

*Precast concrete highway bridges may encounter various extreme load events during their service lives, and the effect of accidental, natural or man-made fire on bridges is one of the least investigated hazard types. To bridge this knowledge gap, a full scale prestressed concrete bridge was tested under a combined hydrocarbon pool fire and simulated AASHTO live load, believed to be the first of its kind in the world. The superstructure of the tested bridge comprised of three Texas standard girders, precast deck panels and cast-in-place deck. The test was conducted for 60 minutes and the fire temperature reached as high as 1131$^{o}$C. It was found that the girder-deck interface was not impacted by the low temperatures at those locations. The precast deck panels sustained significant concrete spalling on fire exposed side, and much higher than that in the cast- in-place deck overhangs, since the former was made of high strength concrete and the latter of normal strength concrete. The fire caused spalling of the entire precast deck panel at some locations, resulting in large decreases in the deck flexural capacity and stiffness. Despite severe spalling on the bottom, the cast in place deck at top showed no signs of crack.*

**Keywords:** Hydrocarbon pool fire, Bridge deck, Precast concrete deck panels, Cast-in-place deck, Composite action, Spalling

## INTRODUCTION

Fire is one of the potential hazards for the integrity and safety of highway bridges. The assumption that fire need not be considered for the design of bridges since bridge fire hazard has a low probability of occurrence may not be appropriate. Voluntary bridge failure surveys of highway departments showed that fire caused more bridge collapse than earthquake[1,2]. Another study found out that, during just the first week of August 2014, nearly 10 tanker truck crashes and fires occurred on the nation's highway system[3]. Bridges are susceptible to fire due to the constant presence of vehicles and the potential for crashed or overturned vehicles to become fuel sources due to their flammable content. Vehicles involved in collision also cause a threat to bridges due to the combustion of their contents, including the onboard hydrocarbon fuel and, increasingly common, hybrid batteries[4]. Other fire causes include arson and wildfire.

Significant investigations have been conducted on the effect of extreme load events, such as earthquake, wind and flood, as compared to fire hazards, even though fire hazards in bridges can cause significant economic and public impacts. The economic losses have a direct cost of repair or reconstruction and indirect costs involving time and energy loss because of traffic congestion and detours[5]. Fire damage to a bridge in a big metropolitan area may lead to a prolonged lane closure which consequently results in significant economic and social impacts. One instance is the collapse of the MacArthur Maze Bridge in Oakland, California. On April 29, 2007, a tanker truck transporting 32.6 $m^3$ of gasoline on I-80/880 highway overturned and caught fire underneath the I-580 expressway. The temperature from the fire reached 1110°C and this led to the strength loss in the steel girders. The bridge collapsed after 22 minutes of fire exposure. The fire caused an estimated $6 million a day total economic impact to the bay area[6], and the bridge repair and retrofitting cost $9 million and took months to complete[7]. The recent collapse of the section of the I-85 bridge in Atlanta, Georgia is another example of the devastating effect of bridge fire. The bridge connects downtown and midtown and used by over 250,000 vehicles daily[8]. The incident caused up to 20% increase in the unit cost per mile for shipping of items and considerable delay to commuters. Rebuilding the bridge cost taxpayers a staggering $16.6 million[9].

While provision for appropriate fire safety measures is a major design requirement for buildings, essentially no structural fire safety provisions exist for bridges. No experimental work has been done to date to study fire hazard on bridges. A limited number of numerical studies have been conducted[1,2,6,10]. Researchers conducted several standard fire tests on columns, beams, and slabs to study their performance at elevated temperature[11,12,13,14]. However, bridge fire is different from standard for the following reasons: (1) difference in heating rate, fire intensity and the duration of fire between hydrocarbon and standard fire. Hydrocarbon fire is characterized by high fire intensity which can reach very high temperatures within the first few seconds of exposure; (2) standard fire test involves uniform heating of structural members which does not happen in real fire scenario; (3) standard fire does not consider parameters that govern hydrocarbon fire behavior such as variation of fuel load and ventilation. (4) real fire has typically three different stages: the growth phase, fully developed phase, and the decay phase. Standard fire does not have the decay phase. The decay or the cooling phase is an important part of fire behavior due to large plastic strains that may develop in structural elements during heating. On cooling, these unrecoverable strains can produce large tensile forces which may consequently lead to failure of connections or other components[15].

The bridge deck primarily supports vehicular loads and distributes them to the supporting girders. The interior bays of the modern bridge deck are typically composite construction and integral with the supporting girders. A popular composite system consists of precast prestressed concrete deck panels supporting a composite cast-in-place deck on top. However, the overhanging part of the deck cantilevering out from the exterior girders are typically made of full-depth cast-in-place (CIP) concrete. Precast prestressed concrete deck panels (Fig. 1) are widely used in the bridge construction industry and have proven to be effective in providing ease of construction and good economy. They are used in approximately 85% of new concrete bridge construction projects in the state of Texas[17]. During a fire event, the fire can occur either on the top of the bridge deck or below the superstructure. From previously documented incidents, it was found out that both have an approximately equal likelihood of happening. However, structural collapse was witnessed only from fires occurring underneath the bridge[1] which could potentially damage the superstructure, bearing pads, bent caps and piers. Researchers conducted studies based on numerical modeling[1,2] to evaluate the performance of bridge decks under the effect of hydrocarbon pool fire, but no experimental work has been done to date.

Fig. 1. Placement of precast prestressed concrete deck panels

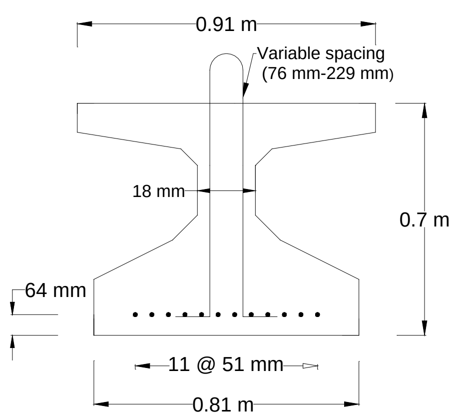
In consideration of the statistics of bridge fire hazards and the substantial increase of petrochemical transport along the nation's vast highway network, adequate research is needed to understand the effect of fire on bridges leading to structural fire design provisions. Little is known on the effect of hydrocarbon pool fire on the various components of concrete bridges, hence, the current investigation was conducted with the aim of understanding the fire performance of bridge deck. Its specific objectives are: (1) Investigate the thermal response of the precast concrete deck panels and cast in place deck; and (2) Evaluate the effect of hydrocarbon fire on the composite action between the girder and the deck.

## TEST BRIDGE DESIGN AND INSTRUMENTATION

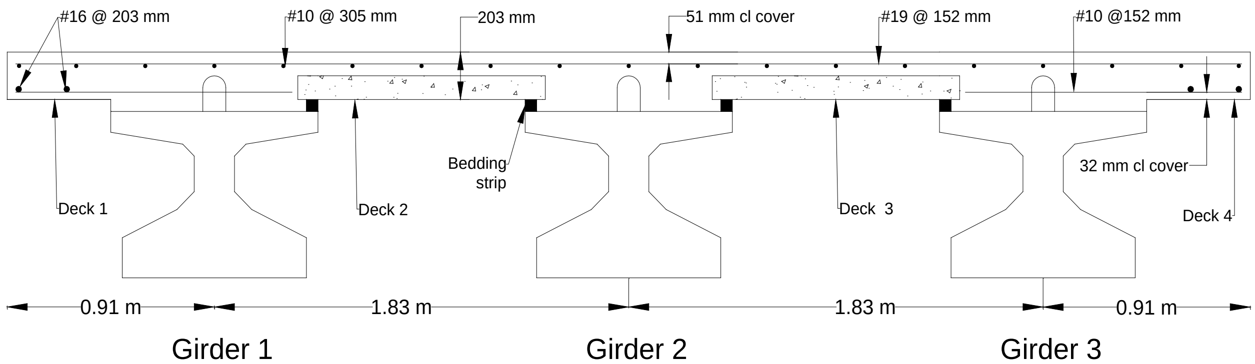
Unlike the standard fire test[16], no specific guideline exists on the size of test specimens and the procedures on how to conduct hydrocarbon pool fire test on bridges. Thus, the current study designed the test bridge members in consideration of various relevant factors.

The superstructure of the test bridge comprised of three Texas standard Tx28 girders[18], precast deck panels and CIP deck. Each girder was 10.06 m long and spaced at 1.83 m on center. A Class H concrete was used for the girders, with minimum release and 28-day compressive strengths of 27.6 and 34.5 MPa, respectively, as specified by the Texas Department of Transportation (TxDOT) Bridge Design Manual[19]. Low relaxation prestressing strands with 12.7 mm diameter and tensile strength of 1862 MPa, and Grade 420 mild steel reinforcement were used. The girders were designed for self-weight, deck dead load and HL-93 live load[20]. Figure 2(a) depicts the cross-section of the girder with simplified reinforcement layout; the detailed drawings may be accessed in the literature[21].

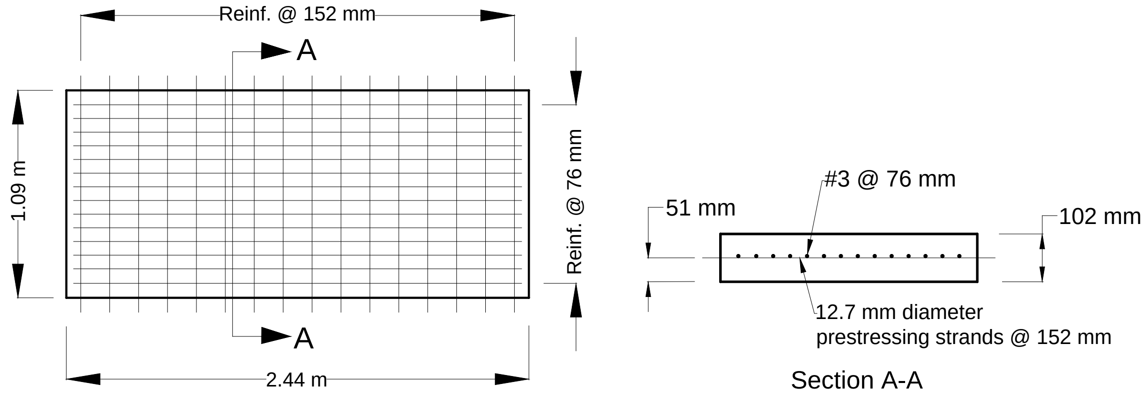
The deck was 9.75 m long, 5.5 m wide, with 0.91 m overhang on both sides of the exterior girders, and 203 mm thick. The concrete clear covers were 51 and 32 mm for the top and bottom reinforcements, respectively. The interior bays of the deck comprised of 102 mm precast prestressed deck panels made from Class H concrete[18] and 102 mm CIP concrete, and the overhangs included 203 mm thick CIP concrete [Fig. 2(b)]. Figure 2(c) shows the simplified precast concrete panel fabrication details as per TxDOT standard drawing[18]. Each panel span 2.44 m in length and 1.09 m in width. Low relaxation prestressing strands with 12.7 mm diameter and tensile strength of 1862 MPa in the transverse direction, and #10 Grade 420 mild steel reinforcement in the longitudinal direction were used. TxDOT Bridge Design Manual[19] specifies that Class S concrete with a 28-day minimum compressive strength of 27.6 MPa should be used for CIP decks. However, for this specific project, a high early strength (HES) concrete mix design (shown in Table 1) with 72- hour compressive strength of 27.6 MPa was used to conform to the tight one month time frame for the fire testing, as described later. Usage of HES helped in achieving the strength earlier than 28 days and allowed early form removal. Grade 420 uncoated reinforcing steel was used in the CIP deck. The deck was designed for self-weight and the simulated HL-93 live load[20].




(a) girder cross section




(b) deck cross-section and girder labeling




(c) precast prestressed deck panel details[18]

Fig. 2. Test bridge details

Table 1. Cast-in-place deck concrete mix design (per 1 $m^3$)

| Components | Weight (kg) |
|---|---|
| Cement | 416 |
| Coarse aggregates | 1090 |
| Fine aggregates: | |
| Concrete sand | 128 |
| Bridgeport sand | 166 |
| Type A MRWR (Mid-range water reducer) | 1.8 |
| Type C NC (Non-chloride accelerating admixture) | 4.7 |
| AEA (Air entraining admixture) | 0.08 |
| Water | 166 |

AASHTO[22] requires highway bridges in urban and rural areas to have a minimum clear height of 4.9 m and 4.3 m, respectively. However, most bridges are in urban areas; hence, a 4.9 m clearance was considered herein. An informal survey conducted by the authors showed that the width and height of most fuel carrying tanker trailers range from 2.4 m to 2.6 m and 3.7 m to 4.1 m, respectively. When these trailers pass under a bridge that satisfies AASHTO[22] minimum clear height requirement, the clearance between the top of the trailers and the bottom of the girders will range from 0.8 m to 1.2 m. During a fire event, the worst damage is expected to occur from the fire originating from the top of the trailers as it is close to the bottom of the bridge. Hence, it is rational and conservative to use supporting members that satisfy the 0.8 m to 1.2 m clearance. Fortunately, there were four discarded Tx28 girders at UT Arlington that were appropriate to be used as supports. Standard Tx28 girders are 0.7 m deep; so, stacking two of them and placing a 64 mm thick elastomeric bearing pad on the top of them provided a 1.5 m clearance between the ground and the bottom of the girder. The pool fire was generated in a 0.3 m deep steel pan which could be filled up to the top. Considering these dimensions, a clearance between the top of the fuel pan to the bottom of the girder of 1.2 m was achieved, as shown in Fig. 3, which is within the desired range.

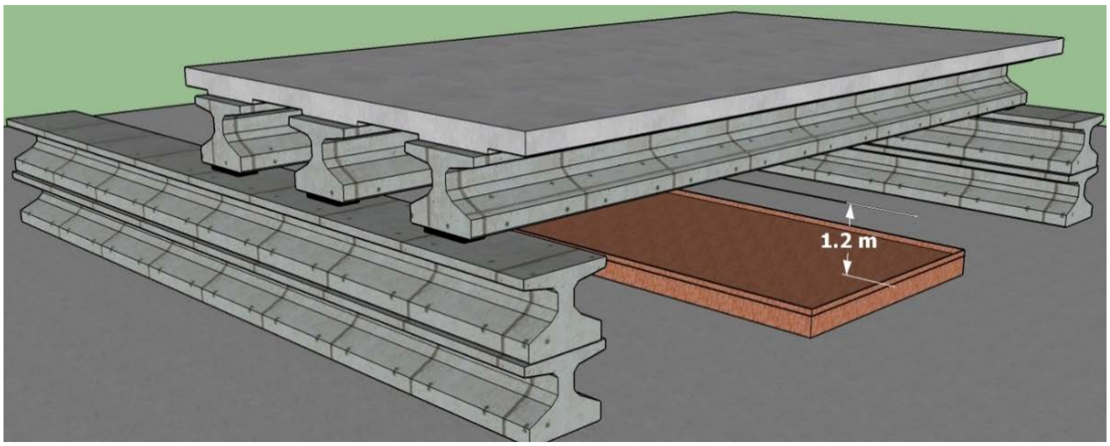



Fig. 3. Test setup

The bridge was instrumented with 32 Type-K thermocouples at different locations to measure temperature progression with time. The thermocouples at the girder-deck interface, designated as 1BT, 2BT, and 3BT, were installed at the mid-span of the bridge (Fig. 4).

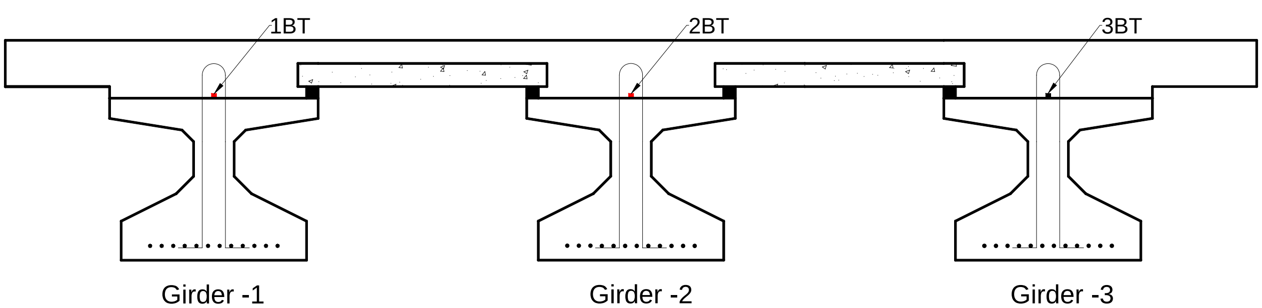



Fig. 4. Thermocouples layout at the mid-span

## TEST BRIDGE CONSTRUCTION AND LIVE LOAD PLACEMENT

The Tx28 girders and the precast deck panels, with concrete mix designs shown in Table 2 and 3, respectively, were fabricated at TxDOT approved precast plants.

Table 2. Girder concrete mix design (per 1 m$^3$)

| Components | Weight (kg) |
|---|---|
| Type III cement | 333 |
| Fly Ash-Class F | 110 |
| Coarse aggregates | 991 |
| Fine aggregates | 833 |
| Water reducing admixture | 2.21 |
| Set retarding admixture | 0.147 |
| Water | 134 |

Table 3. Precast deck panel concrete mix design (per 1 m$^3$)

| Components | Weight (kg) |
|---|---|
| Type III cement | 249 |
| Fly Ash-Class F | 83 |
| Coarse aggregates | 1090 |
| Fine aggregates | 797 |
| Water reducing admixture | 1.44 |
| Retarding and water reducing admixture | 0.22 |
| Water | 127 |

The test girders were set on elastomeric bearing pads placed on the two support girders described previously [Fig. 5(a) and (b)]. The precast deck panels were then placed on the top of the girders after attaching the bedding strips to the edge of the girders [Fig. 5(c) and (d)] followed by forming of the deck [Fig. 5(e)]. The cantilever deck formwork involved placing overhang bracket, designed to support self-weight and the live load from the people working during concrete pouring, at 1.22 m spacing [Fig. 5(f)]. Both the longitudinal and transverse reinforcements were then tied [Fig. 5(g)] and finally the concrete was poured [Fig. 5(h)].

(a) placing of support girders

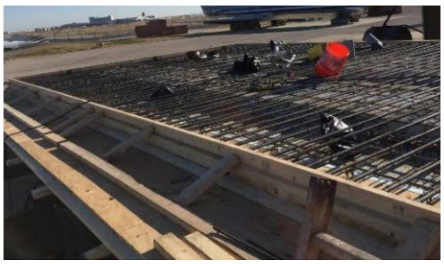
(b) supports and girders after placement

(c) bedding strip installation

(d) placing of precast deck panels

(e) forming of the deck

(f) overhang bracket



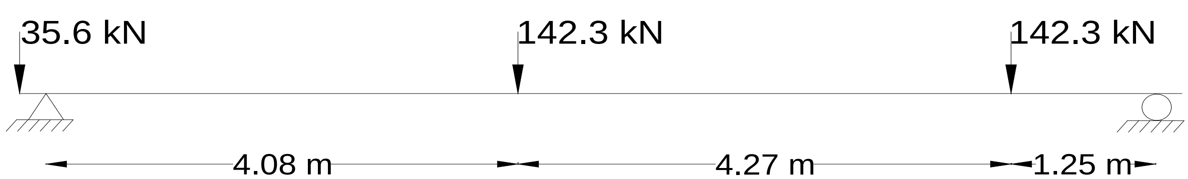
(g) deck reinforcements

(h) concrete pouring

Fig. 5. Test bridge construction

During a fire event, it is a common practice to shut down bridges for any traffic. But if there is a disabled vehicle, the bridge will be subjected to combined live load and fire. For the current study, the fire test was conducted assuming there will be a vehicle on the bridge deck during such event, to investigate the performance under the worst-case scenario. Therefore, the AASHTO LRFD lane loading was omitted.

The total width of the experimental bridge was 5.5 m, which allowed only one traffic lane[20]. A moving load analysis showed that an AASHTO LRFD design truck with the arrangement shown in Fig. 6(a) yielded the maximum moment on the girders. The front axle fell outside the span and did not affect the maximum moment value. The design truck axles were simulated by loading the bridge with blocks of zipper barriers available from TxDOT. Each block weighed approximately 6179 kN and stacking 22 of them at the location of each wheel yielded the approximate AASHTO truck wheel load. The barriers were lined across the width of the bridge, directly in contact with the deck, to provide an approximately equal distribution of load among the three girders per their tributary area, as shown in Fig. 6(b). The AASHTO LRFD tandem load did not govern the design.




(a) arrangement of the design truck load for maximum moment

(b) test bridge loaded with the simulated AASHTO design truck load

Fig. 6. Live load placement

## FIRE TEST

The maximum bending moment in the test bridge occurred at mid-span. Therefore, the fuel pan was placed at this location to investigate the worst effect of the fire on the girder flexural strength.

In actual bridge fires, the fuel volume, fuel type, response time of fire departments, time to extinguish the fire, location of the incident (rural or urban area), total surface area of the fuel spill and many other factors influence the extent of bridge damage. From previously documented incidents[23,2], the fuel volume can be as low as 11.4 $m^3$ and as high as 37.5 $m^3$. The time to extinguish the fire could range between 30 minutes[23] and two hours[24]. Considering these variables, the test bridge was subjected to a fire for one hour generated by 4.32 $m^3$ of aviation training fuel.

One of the challenges of conducting open pool fire experiment is wind because it can influence the uniformity of fire distribution. The authors monitored long-range weather forecast and selected the test date and time at 8 am on a no/minimal wind day.

The pan was filled with water to a depth of 0.25 m first, followed by the fuel and finally the fire was started. The winds were calm at the beginning of the test, as can be seen from the even distribution of flames in Fig. 7(a). But few minutes after the fire initiation, a 5 m/s southeast wind started and disrupted the fire distribution, as seen in Fig. 7(b). The wind persisted until the end of the test. Thus, girders 1 and 2 experienced more fire exposure than girder 3.

(a) even distribution of fire

(b) uneven distribution of fire

Fig. 7. Fire test

## RESULTS AND DISCUSSIONS

The following subsections discuss the response of the test bridge deck to the pool fire which depended on the compressive strength of the concrete at the time of the experiment. Concrete with 20-50 MPa compressive strength is classified as normal strength concrete (NSC), while those in the range of 50 to 120 MPa are classified as high-strength concrete (HSC)[25]. The girders concrete compressive strength at the fire testing age was 75.5 MPa, which can be classified as HSC, while, the cast in place deck was 39.1 MPa, which can be categorized as NSC. Unfortunately, the concrete compressive strength for the precast deck panels at the age when the fire testing was performed was not available. However, a two-day compressive strength of 46 MPa was obtained previously after the decks were cast. Also, the precast deck panels were made of Class H concrete, which is the same class of concrete used for fabricating the girders. Hence, considering the gain in strength with time and the inherent nature of the mix design used, the precast deck panels concrete can be classified as HSC.

### FIRE TEMPERATURE

Figure 8 shows the temperature versus time reading of the fire measured at the mid-span of the bridge. The fire temperature reached a maximum of 1131°C. During the growth phase, the peak temperature was 876°C within 41 secs after the fire initiation. During the decay phase, the temperature plunged by 800°C in three minutes. The fire temperature fluctuated due to the prevalent wind. A temperature difference of 640°C was observed between the readings taken at 2.3 minutes and 3.3 minutes. These erratic thermal fluctuations caused thermal shock which eventually contributed to concrete cracking and spalling.

The temperature versus time for two of the most commonly used standard fire curves, ASTM E119[16] and ISO 834[26], are also shown in Fig. 8. Both standard curves peak to the maximum temperatures at a very slow rate, as compared to the open pool hydrocarbon fire curve. No fluctuations of temperature are included in the standard fire curves, because the tests are performed under controlled environment. The temperatures from the hydrocarbon fire were generally greater than the standard fire curves for most of the 60 minutes duration.

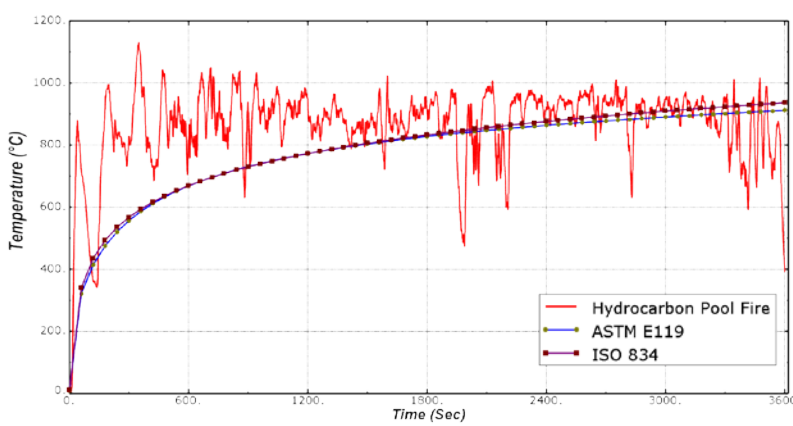



Fig. 8. Time dependent variation for the hydrocarbon pool fire and standard fire temperatures

## COMPOSITE ACTION

Composite action in concrete bridges is achieved when the deck and the girders are combined to act together as one cohesive element that is stronger and stiffer than the individual components. The horizontal shear stress developed at the deck-girder interface is resisted by cohesion, aggregate interlock, and shear-friction developed by the force in the reinforcement crossing the plane of the interface or dowel action. The nominal shear resistance of the interface plane, $V_{ni}$, is given by Eq. (1): [20].

$$V_{ni} = cA_{cv} + \mu(A_{vf} f_y + P_c) \tag{1}$$

Where: $c$ = cohesion factor
$A_{cv}$ = area of concrete engaged in the interface shear transfer
$\mu$ = friction factor
$A_{vf}$ = area of interface shear reinforcement crossing the shear plane within the area $A_{cv}$,
$f_y$ =yield stress of the reinforcement
$P_c$ = permanent net compressive force normal to the shear plane

Among these parameters, the yield strength of the reinforcing dowel bars degrades with temperature, as shown in Fig. 9. The yielding which eventually may lead to failure of the dowel bars may result in slippage of beam to deck connection, resulting in cracking along the beam deck interface and haunch.

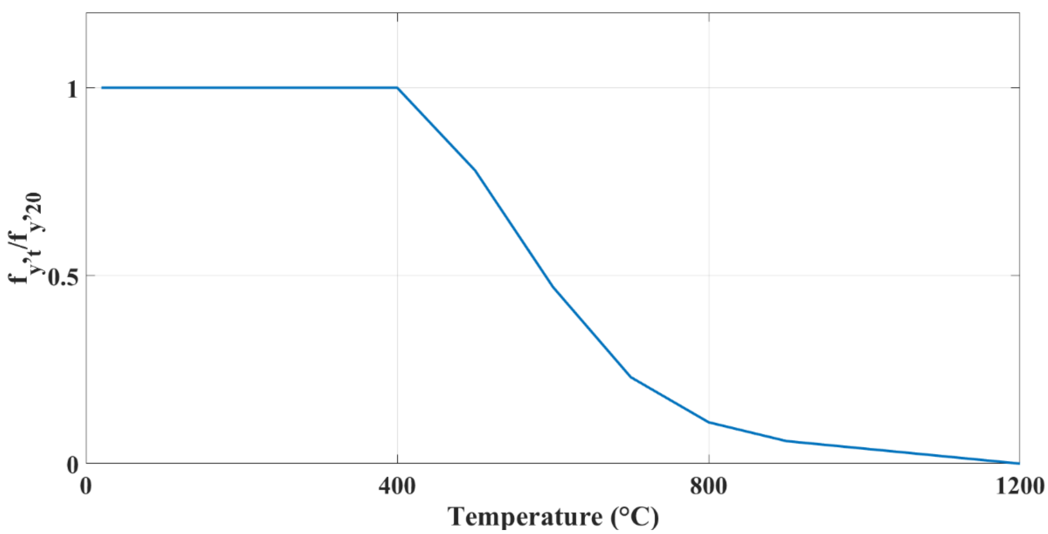
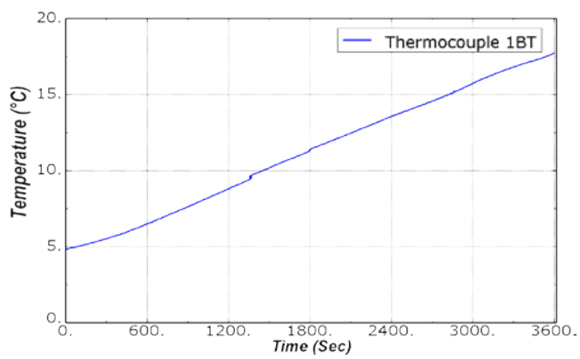
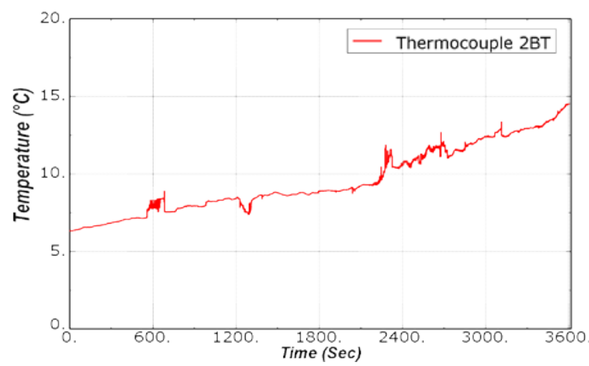



Fig. 9. Variation of normalized yield strength of mild reinforcement with temperature[27]
**$f_{y,20}$ and $f_{y,t}$ are the yield strength at ambient and elevated temperature, respectively.

Figures 10(a) and (b) show the temperature versus time measurement at the deck-girder interface for Girder 1 and 2, respectively. The maximum temperature readings from both thermocouples were minimal, 18°C in the former and 15°C in the latter. This was due to the low thermal conductivity and high thermal capacity of concrete, which played a key role in slowing down the heat penetration to the interface. Among the girders, Girder 2 was provided with a cement-based dry mix fireproofing mortar for wet sprayed application. The detail information on the design and application scheme may be accessed in the literature[21]. As a result, its peak temperature was lower than that in Girder 1. As shown in Fig. 9, there is no loss in the yield strength for mild reinforcement up to a temperature of 400°C. Thus, the measured temperatures at the deck-girder interface did not adversely affect the shear resistance derived from the dowel action of rebars. Visual observation after the fire test revealed melted and charred bedding strips (Fig. 11). However, no cracking or interfacial slippage between the deck and the girder was observed that could affect the composite action. Differential thermal expansion during a fire event may cause distortion in the overall geometry of the deck. However, none was observed in the current investigation.




(a) Girder-1




(b) Girder-2

Fig. 10. Temperature versus time curves at girder-concrete interface

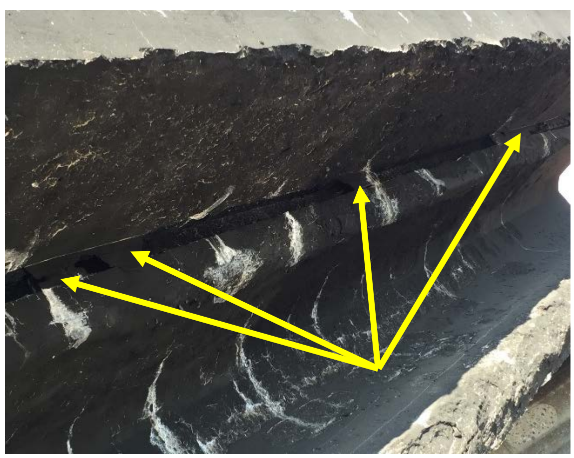


Fig. 11. Charred bedding strip

Following the fire test, the deck was saw cut and each girder was tested in the laboratory under a three-point bending set-up to determine their residual strengths [Fig. 12(a)]. During loading, it was observed that cracks originate from the girder and propagate towards the deck [Fig. 12(b)]. However, no slippage or cracking was observed at the interface between the two until the conclusion of the test. All three girders failed by yielding of the prestressing strands. From the results of the fire and the residual strength test, it can be concluded that the fire did not affect the composite action between the girder and the deck.

(a) Test set-up

(b) crack patterns

Fig. 12. Residual strength test

## SPALLING PERFORMANCE

Spalling of concrete due to high and rapidly increasing fire exposure is characterized by broken pieces of concrete from the member surface. It occurs when the built-up pore pressure during heating exceeds the tensile strength of concrete. Spalling exposes deeper layers of concrete to elevated temperature, consequently increasing the rate of transmission of heat to the inner sections of the member, including the reinforcement. The rise in temperature of reinforcements may result in strength degradation of structural members and finally failure[28]. Inspection of the color of spalled concrete from the deck in the current study revealed that most of the concrete spalled at the early stages of the fire. Concrete shows no discoloration when subjected to a temperature less than 316°C[29].

Figure 13 (a) and (b) shows the bottom of the precast deck panels after the fire exposure. Deck 2 between girders 1 and 2 sustained maximum of 67 and 102 mm deep spalling on approximately 29% and 20% of the fire exposed surfaces, respectively. The spalling was more severe near the central part of the deck span where the positive moment is maximum. The rest of the exposed areas experienced none or smaller than the above-mentioned spalling. The longitudinal mild reinforcements and the transverse prestressing strands in the precast deck panels lost concrete cover for a length of up to 1.98 and 0.7 m, respectively. On the other hand, the fire exposed surface of Deck 3 between girders 2 and 3 experienced 67 and 102 mm deep spalling over 20% and 12% of the exposed surfaces, respectively. In Deck 3, the transverse and longitudinal reinforcements were exposed up to a length of 0.6 m and 1.83 m, respectively. It is apparent that the degree of spalling was more severe in Deck 1. This was due to the more intense fire in this deck resulting from the prevalent wind that pushed the flames towards that deck span. None of the prestressing strands or mild reinforcements was broken.

It may be noted that the most severely spalled section of the deck lost 102 mm deep concrete, equivalent to the entire thickness of the precast deck panels. In essence, the remaining undamaged part of the composite deck consisted of mostly the 102 mm thick CIP and the embedded mild steel rebars in it. The nominal flexural strength for the remaining thickness of the deck, without considering any material strength degradation due to temperature, was calculated herein, using AASHTO strip method[20]. It was found to be 9.2 kN-m, which is a staggering 78% reduction as compared to the 41.6 kN-m flexural capacity of the deck before fire exposure. There was a high probability for the concrete and the reinforcement in the CIP part of this deck to have experienced some strength degradations due to the elevated temperature. So, the actual residual capacity of the remaining deck portion could be even lower than what was calculated above. However, the deck did not experience any collapse because applied bending moments were redistributed due to the continuity of the adjacent spans. Considering the relationship between thickness, moment of inertia and deflection, and disregarding the decrease in self-weight from spalling and the degradation of modulus of elasticity due to the elevated temperature, it could also be inferred that spalling of half of the deck thickness led to 87.5% reduction in the moment of inertia. This may result in increase in the immediate deflection of the deck by about 700% . In addition, concrete cracking and spalling may increase permeability thereby decreasing the resistance to corrosion of reinforcements, consequently compromising the long-term serviceability of the bridge.

(a) Deck 2

(b) Deck 3

Fig. 13. Concrete spalling in precast deck panels

Figure 14 shows the bottom of the CIP overhanging decks after the fire test. Deck 1 overhanging from girder 1 side sustained more severe spalling damage, as compared to Deck 4 overhanging from girder 3. The former lost an average of 38 mm concrete depth along for approximately 7.9 m length of the deck, while the latter did not experience any damage. The nominal flexural capacity of the spalled section of the overhang was 24 kN-m, a 27% reduction from the 33 kN-m flexural capacity of the undamaged deck section before fire exposure.

Despite the severe spalling on the bottom, Deck 1 showed no sign of cracking or distress at the top.

(a) Deck 1

(b) Deck 4

Fig. 14. Overhanging decks after fire exposure

It may be noted that the precast deck panels sustained more severe fire damage than the CIP overhanging decks. This could be attributed to the fact that the former was made of HSC while the latter was from NSC. The degree of concrete spalling in HSC depends on several factors, such as load intensity and type, fire intensity and specimen dimensions[28]. The self-weight of the deck and the simulated HL-93 live load induced positive bending moments on the composite decks 2 and 3 between the girders and negative bending moments on the overhang CIP decks 1 and 4. The presence of tensile bending stresses at the bottom of the composite decks on the fire exposed side and the built-up pore pressure due to fire enhanced the conditions and the degree of concrete spalling. The situation was reversed for the CIP overhanging decks, where the tensile bending stresses occurred at the top of the deck and not at the fire exposed bottom side. This combination allowed less concrete spalling in the CIP deck overhangs. As shown in Fig. 8, hydrocarbon pool fire is characterized by a faster heating rate and more intense fire, as compared to standard fire. The extent of concrete spalling is much higher when HSC members are exposed to high intensity fire. A previous study[28] also revealed that the degree of spalling in HSC increases with increasing size of specimens. This is attributed to the fact that the specimen size is directly proportional to its capacity to store heat energy.

## CONCLUSIONS

The following conclusion may be made based on the results from this study:

1. The current study was groundbreaking in the investigation of a full-scale composite concrete bridge deck under a realistic open-pool fire setting with a representative applied live load. Such fire could be caused through deliberate or accidental reasons. The study

paved the way in understanding the response of concrete bridges components to hydrocarbon pool fire that may eventually lead to developing bridge fire safety provisions.

2. The open pool fire caused a faster temperature change in the test bridge and the bridge deck, as compared to standard fire curves from the literature. The pool fire also showed fluctuations in temperature due to the prevalent wind effect, while the standard fires do not provide for such fluctuations. Actual temperature fluctuations can cause thermal shock leading to concrete spalling. The pool fire temperature was also generally greater than the standard fire specified temperatures.
3. Although the maximum fire temperature in the tested girders was more than 1000°C, the temperature measured at the deck-girder interface was minimal (15 – 18°C) over a 60-minute fire duration. This is not likely to adversely affect the yield strength of the mild steel dowel rebars and the corresponding shear resistance from the dowel action. In addition, no cracking or slippage was observed at the girder-deck interface after the fire and the residual strength test, implying that the composite action between the girders and the deck was not compromised.
4. The precast deck panels between girders sustained extensive concrete spalling, to a depth of 102 mm over 20% of the deck bottom surface, and to a depth of 67 mm over 29% of the deck bottom surface area. Varying intensity of fire due to the prevalent wind caused differential spalling in the two composite decks investigated in this study. The 102-mm spalling essentially means the loss of the entire precast deck panel.
5. The entirely cast-in-place deck overhangs sustained less degree of concrete spalling, as compared to the composite decks. This was attributed to the fact that the former was made of normal strength concrete, while the latter was produced with high strength concrete.
6. Despite the severe spalling on the deck bottoms, the top of all decks did not show any sign of crack or distress. Therefore, it may be assumed that the mild steel rebars in the cast in place parts of the decks were also undamaged.
7. The spalled sections of the precast prestressed concrete decks and the overhanging decks sustained 78% and 27% reduction in their nominal flexural capacity due to the section loss, respectively. The fire effect would result in stiffness losses as well, leading to an almost 700% increase in instantaneous deflections.
8. Extensive concrete cover spalling due to fire may lead to long-term sustainability issues, such as enhanced steel corrosion and durability loss.

## ACKNOWLEDGEMENT

The study was performed under a contract from the Texas Department of Transportation.

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