Rehabilitation of Mohammed Al-Qassim Bridge after Fire Attack Using CFRP Sheets: A Case Study

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ABSTRACT: This paper focuses on a case study of Mohammed Al-Qassim Bridged highway that developed extensive damages in main prestressed concrete girders due to a huge fire attack in two successive spans and the adjacent ramp span. This highway bridge consists of 118 spans composite concrete bridge (i.e., precast prestressed concrete girders and cast-in-situ deck slab) is existing in Baghdad city on the main circular express highway. The bridge is 4012 m long, where each span is 34 m long. The importance of this structure from economic and traffic points of view has made it impossible to think of the total replacement. Accordingly, the possibility of the replacement of the three spans was ruled out due to the tedious nature of the process, the time and cost. The main goal of the study was how to restore the original load capacity of the pretensioned girders using CFRP strengthening technique. To achieve this goal, a strengthening system was proposed to the three defected spans by installing a series of CFRP sheets on the soffits and sides of the main prestressed concrete girders. After strengthening, a load test was carried out to verify the strengthening system. Results of the load test and the numerical analysis proved that the proposed strengthening system improved the stress distribution in all components of the bridge and maintained the original load resistance mechanism provided by the prestressed girders and the deck slab.

1   INTRODUCTION

The deterioration of the existing bridges is at present one of the most important problems in contemporary bridge engineering. It is of technical, economic and social nature and concerns bridge infrastructure in many countries (Radomski, 2002). The deck slab and the pretensioned precast concrete girders of a composite prestressed concrete I-girder bridge serve as an integral part of the structural bridge section, and any damage to any part may seriously affect the structural performance, the load-resistance mechanism, and could have endangered the overall stability of the bridge. Many factors have to be taken into consideration while rehabilitating deteriorated pretensioned prestressed concrete I-girder bridge, such as the overall integrity of the structure, keeping the load transfer mechanism and most importantly, the preservation of the original prestressing effect of the system (Durgesh et al., 2013).

The Mohammed Al-Qassim express highway is a part of the national network that carries an annual average daily traffic of 45,000 to 50,000 vehicles in Baghdad city of Iraq. On this highway the two-individual-way Mohammed Al-Qassim bridge is existing with 118 simply supported spans each of 33.5 m. The total length of the bridge is 4012 m and the width of each way of the bridge is 14.65 m. The cross-sectional dimensions of the bridge are shown in Fig. 1. Eight precast, prestressed tee girders with identical dimensions and reinforcement were used to build the bridge.

The bridge represents a very important highway structure for the Iraqi highway system and traffic officials due to the intense traffic volume which continuously passing over it. After the chaos that swept through Iraq after the war of 2003, the spaces under the bridge and along three adjacent spans were used to store and trade antique furniture. This fact served the main reason behind the huge fire that occurred under three successive spans of the above-mentioned bridge, which developed extensive structural damages in their main prestressed concrete girders due to a huge fire attack for four hours.

It is common practice that structurally damaged prestressed concrete bridge members are taken out of service and replaced. This, however, is not an efficient use of materials and resources since the member can often be repaired in situ (Kasan, 2012). Recently, the emphasis has been placed on repairing these girders, ultimately saving both economic and monetary resources and reducing the length of time in which the structure is out of service for bridge replacement. The importance of the mentioned structure, from an economic and traffic points of view, has led to the exclusion of the idea of the total replacement of the three successive spans. Accordingly, the possibility of the replacement was ruled out because of the tedious nature of the process, the time and the huge money consumption. Accordingly, the decision was made to rehabilitate the defected spans.

Generally speaking, bridge rehabilitation is project specific since no two bridges are alike and all are located in different traffic conditions. Rehabilitation design is diagnostic and the diversity and complexity of the issues make it different from conventional new bridge design. Rehabilitation of bridges is a far more diverse and challenging subject than a new design based merely on code compliance. For the maintenance of an existing bridge, there are fewer alternatives available to the designer than when designing a new bridge (Khan, 2010). For this purpose, an extensive and detailed investigation was carried out to identify all the underlying deficiencies of the three spans and to determine the overall strength of the composite concrete bridge across its various structural elements, such as the deck slab and the I-precast girders. Meanwhile, it is very difficult to accurately assess the strength of these components due to the lack of design drawings and technical design data that destroyed or burned due to the wide vandalism which swept through Iraq after the US occupation in 2003. For structural solutions, complete rehabilitation for removing all deficiencies, or justifying their retention, is necessary. It includes the work required to restore the structural integrity of portions of the original bridge, as well as the installation of a new strengthening system.

The main objective of this study is to propose a practical procedure which assists to restore serviceability and original functionality following distress from severe localized deterioration due to the huge fire attack.
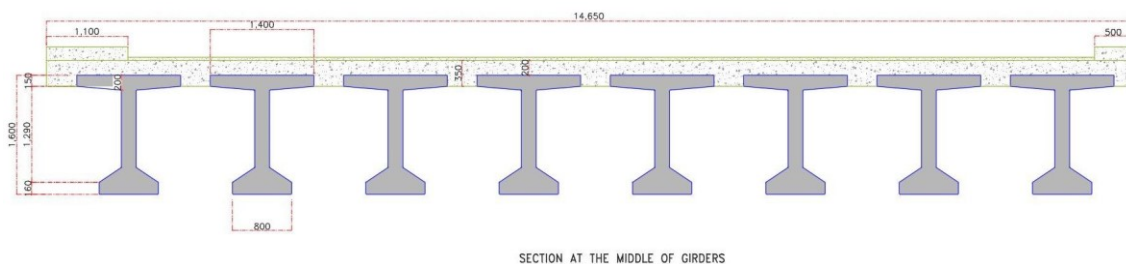

Figure 1. Typical cross-section of Mohammed Al-Qassim bridge

2   VISUAL INSPECTION PHASE OF THE BRIDGE STATUS

The evaluation process for Mohammed Al-Qassim concrete bridge structure for fire damage started with a visual inspection of cracking, discoloration and spalling for the two successive deteriorated and the ramped spans and ended with a load test to evaluate the actual status of the bridge. It was noticed that with heating, a variation in color from normal to pink is often observed in many locations along the span of most of the girders. This fact indicated that the process of a significant loss in the strength of concrete was started. Camber was increased for these spans compared to other unexposed to fire spans; this reflected the deteriorations that happened in concrete. Because the decreasing of the camber of prestressed concrete beams is a sign of increasing losses in prestressing force in contrast, increasing camber indicates that deteriorations could be happened in concrete strength whilst the prestressing force not affected. So, increasing of the elevated temperature had a bad effect on the residual camber, denoting more deterioration has occurred. Most of the I-girders have been suffered from spalling at some reigns in the soffit and edges near to the fire source. Also, surface hairline cracks appeared along the span. It is clear that the spalling depths are deeper at the bottom surface of the I-girders which exceed the stirrup reinforcement cover. Likewise, hairline cracks also occur in deck slab at the steel reinforcement positions and in the bottom surface of I-girders at prestressing steel position. It is worth to mention that due to all these effects the degree of composition between the two different materials (prestressed girders and cast in situ deck slab) could not be specified. Figure 2 shows the deteriorated locations in Mohammed Al-Qassim concrete bridge.

Figure 2. Status of deteriorated girders after fire attack of Mohammed Al-Qassim bridge.

3   LOAD TEST PHASE

Bridge load testing offers a unique opportunity to investigate the behavior of real structures. The main goal of the load test will generally be to demonstrate satisfactory performance under a specified load. This is usually judged by measurement deflections under this load, which may be sustained for a specified period.

A static load test is conducted on bridges and is considered as accepted criteria and useful information concerning testing and deflection measurement. A static and dynamic load-testing program was carried out by Al-Mustansiriya University on Mohammed Al-Qassim bridge to evaluate the residual strength of its components.

Two test vehicles were designed to transfer the ultimate live loads specified by the AASHTO standard specifications (2012). The vehicles are tractor-trailer combination designed specifically for the purpose of testing, weighing 450 kN when fully loaded with concrete blocks. The fully loaded testing vehicle represents the specified AASHTO ultimate live load plus 30 percent impact for an H20-44 truck. These vehicles were driven and positioned at the critical locations on the bridge while the data from the deflection transducers are immediately collected, analyzed, and compared to the theoretical prediction to ensure the safety of the bridge.

According to AASHTO, when investigating maximum relative displacements, the number and position of loaded lanes should be selected to provide the worst differential effect. Therefore, five positions in each of the three loaded lanes were specified for the location of trucks during the testing process. Five I-girders were examined by placing deflection transducers at critical locations along the girders. The load was applied in such a manner that stopping the test can be taken if any untoward distress is observed at any stage. The movement from one position to another was performed only after the deflections under the previous load location have stabilized and all the stipulated observations are completed. Survey process was carried out for the bridge components during loading.

The data collected from the various deflection transducers were used to evaluate the performance of each girder under the applied load.

A maximum measured deflection of 22.52 mm was observed at midspan during the load test. Meanwhile, the AASHTO deflection limit may be considered for concrete vehicular bridges as span/800 which equals to 33500/800 = 41.875 mm. Thus, the measured maximum deflection consisted of 53.8% of the AASHTO deflection limit.

4   RESIDUAL STRENGTH OF CONCRETE AND STEEL AFTER FIRE EXPOSURE

It is important to note that the performance of structural concrete members in fire depends on several factors, mainly, on the change of properties of the composes materials due to fire exposure and the temperature distribution within the composition of the structure. Georgali and Tsakiridis (2005) reported that concrete is a poor conductor of heat, thus can experience serious damages when it exposed to fire. The discovery of the heating history of concrete is significant to define whether the concrete structure exposed to fire and its components remain intact from the structural aspect. Chan et al. (1999) categorized temperatures into three ranges in terms of the effect on concrete strength loss, namely 20-400 °C, 400-800 °C and above 800 °C. In the range, 20-400 °C, the high strength concrete (HSC), unlike the normal strength concrete (NSC), maintained its original strength. While, in the range 400-800 °C, both HSC and NSC lost most of their original strength, especially at temperatures above 600 °C. Above 800 °C, only a small portion of the original strength was left for both HSC and NSC.

In HSC and NSC there was a variation in the pore structure known as a"microstructure coarsening effect" at high temperatures. The change in pore structure is supposed to be one of the reasons behind the loss of concrete strength at a temperature of less than 600 °C. It was noted that the tensile strength has been reduced more significantly than compressive strength at a temperature of 600 °C in HSC, such as NSC.

It is known that concrete conductivity decreases with increasing temperature due to the loss of pore water and the dehydration of cement paste. The surface of concrete exposed for the high temperature will subject to these changes and it results in the creation of an insulating material that acts as a material heat-resistant that reduces the heat entry and in turn acts as a fire wall resistance for reinforcements. This helps the concrete to be excellent material in fire resistance (Riley, 1991). Extreme temperatures can destroy the concrete structure from excessive spalling due to concrete expanding with increasing temperatures, but higher temperatures also lead to more shrinkage of hardened concrete paste. These two movements work in opposite directions that forming micro-cracks at the cooling. This is more complex while longitudinal expansion is restricted, as in prestressed concrete (Britain, 1975). In addition to that, concrete composes of different materials each has different physical properties, the rapid rise in temperature can lead to loss of inter-particle bond which happens due to different expansion coefficients causing spalling. On another hand, the concrete and steel show a similar thermal expansion of temperatures up to 400 °C; however, a significant expansion in steel will happen at higher temperatures compared to concrete and, if temperatures are reached to 700 °C, the steel resistance will be reduced to about 60% (Fletcher et al., 2007) and debonding may happen (Kodur and Bisby, 2005).

Pretensioned and post-tensioned concrete members are more sensitive to damage during the exposure to high temperature than the reinforced concrete members due to the debonding that may happens for the prestressed strands and the deterioration of its surrounding concrete.

Zhang et al. (2014) proposed three empirical equations to evaluate the residual yielding ($f_{py,R}$) and ultimate strengths ($f_{pu,R}$) of prestressing steel strands Grade 270 that exposed to different elevated temperatures (T), depending on the original yielding strength (f_{py}), the original ultimate strength (f_{pu}), and the temperature value (T), where

For temperature $20\,°\mathrm{C} < T < 400\,°\mathrm{C}$

$$f_{py,R} = f_{py} \tag{1}$$

$$f_{pu,R} = f_{pu} \tag{2}$$

For temperature $400\,°\mathrm{C} \leq T \leq 800\,°\mathrm{C}$

$$f_{py,R} = (1.707 - 1.76 \times 10^{-3}\,T)\,f_{py} \tag{3}$$

For temperature $400\,°\mathrm{C} \leq T \leq 700\,°\mathrm{C}$

$$f_{pu,R} = (1.71708 - 1.83 \times 10^{-3}\,T)\,f_{pu} \tag{4}$$

For temperature $700\,°\mathrm{C} < T \leq 800\,°\mathrm{C}$

$$f_{pu,R} = (0.55074 - 1.684 \times 10^{-4}\,T)\,f_{pu} \tag{5}$$

Aslani (2013) proposed an empirical equation to estimate the residual cylinder compressive strength ($f'_{c,R}$) of normal strength concrete with siliceous aggregate that exposed to different elevated temperatures (T), depending on the original cylinder compressive strength (f'_c) and the temperature value (T), where

For temperature $200\,°\mathrm{C} < T \leq 800\,°\mathrm{C}$

$$f'_{c,R} = (1.06 + 0.00025\,T - 2.235 \times 10^{-6}\,T^2 + 8 \times 10^{-10}T^3)\,f'_c \qquad (6)$$

Accordingly, the estimated residual strengths for prestressing strands of original ($f_{py} = 1676$ MPa and $f_{pu} = 1862$ MPa) and the concrete of the I-girders ($f'_c = 42$ MPa) and the deck slab ($f'_c = 30$ MPa) are shown in Table 1.

Table 1. Residual strengths for prestressing steel and concrete

	Temperatures (°C)			
	300	500	700	800
Yielding strength for strand $f_{py,R}$, MPa	1676	1386	796	501
Ultimate strength for strand $f_{pu,R}$, MPa	1862	1493	812	775
Concrete of $f'_c = 42$ MPa				
Cylinder compressive strength $f'_{c,R}$, MPa	40	31	17	10
Concrete of $f'_c = 30$ MPa				
Cylinder compressive strength $f'_{c,R}$, MPa	29	22	12	7

5 ANALYSIS PHASE AND PROPOSED STRENGTHENING SCHEME

It is very difficult to accurately assess the strength of the different structural elements of the bridge due to the lack of design drawings and technical design data. During the analysis phase, there are some aspects that should be considered since they can influence the performance process, namely: (a) for which load, the analysis of the structure or the structural component is performed? For instance, dead load and live load or only live load; (b) Type of dominant load in the critical component? Compression, tension, bending or torsion; and (c) Effect of strength degradation of the structure? e.g. effect of the redistributed internal forces on the structure.

As a result, a numerical analysis using finite elements approach of ETABs software program, as the main modeling program, was used to analyze Mohammed Al-Qassim bridge. In the creation of this model, it was noted that two factors are the main contributors to the accuracy of the structure model. These factors include the material properties and load distribution. Based on these factors, a number of recommendations were proposed to follow for the creation of the model, and they are: (a) For pretensioned I-girders, the transformed moment of inertia for composite members should be used; (b) Load distribution percentages are based on the number of girder lines, construction materials and span length; and (c) Load is distributed longitudinally and transversely.

It should be mentioned that the residual mechanical properties of prestressing strands and concrete after the exposure to fire attack were assessed using equations proposed by Zhang et al. (2014) and Aslani (2013), respectively. In calculations, it was assumed that the three succesive affected spans were exposed to 700 °C temperature for three hours. Accordingly, the residual yielding $f_{py,R}$ and ultimate $f_{pu,R}$ strength for strands were adopted equal to 796 and 812 MPa, respectively. While the residual concrete cylinder compressive strengths $f'_{c,R}$ for affected girders and deck slab were used equal to 17 and 12 MPa, respectively.

It is worth to note that, the analysis process was modeled and simulated the performed load test following the same load pattern and the same five positions in each of the three loaded lanes. The mechanical properties for concrete and the steel strands were adopted in the analysis equal to the residual properties which determined in Table 1 for temperature of 700 $^{\circ}$C. A maximum calculated deflection of 18 mm resulted at the midspan section during the analysis under the applied loading. That means the measured deflection was greater than the calculated by 125%. The main reason behind this difference is the strength degradation of materials of the bridge components, which was not considered in the analytical models.

6   PROPOSED CFRP STRENGTHENING SCHEME FOR MAIN GIRDERS

CFRP 'Wet lay-up' system consists of a primer, CFRP sheet, and the resin was applied for strengthening the main girders of the three successive affected spans to compensate for the loss in flexural and shear strengths that occurred due to the fire attack. The mechanical properties of these materials were provided by the manufacturer data sheet as shown in Table 2. The primer increases the bond between the composite and the concrete substrate and it consists of two parts. The CFRP sheets are high-performance carbon fibers sheets supplied in unidirectional tow sheets of 500 mm width. Fibres thickness was reported to be 0.131 mm and its weight was 230 g/m^3. The resin consists of two parts used to impregnate carbon fibres forming a composite bonded to the primer.

Table 2. Mechanical properties of primer, CFRP sheet, and resin

Material	Nominal elastic modulus, (MPa)		Nominal tensile strength, (MPa)	
	Direct	Flexure	Direct	Flexure
Primer	N/A	3489	33.8	60.6
CFRP sheet	238000	N/A	4300	N/A
Resin	N/A	N/A	17	35

The required area of the CFRP sheets (A_f) used for flexural strengthening the I-girders was determined in such a way that the internal force achieved by the CFRP sheets (T_f) shall be equal to the shortage of internal force of the prestressing strands due to the degradation of the steel strength. While the required area of the CFRP sheets (A_{fv}) used for shear strengthening the I-girders was determined in such a way that the internal force achieved by the CFRP stirrups (T_{fv}) shall be equal to the shortage of internal force of the concrete due to the degradation of the strength of concrete.

Accordingly, two longitudinal strips each of 250 mm width was attached to the soffit of each girder of the suffered spans. Also, the end blocks were covered by CFRP stirrups with a width of 500 mm to wrapping the whole region and extend along the web up to the top flange. Between the end blocks, CFRP stirrups with 100 mm width were distributed at 250 mm c/c along the span of each girder. Each stirrup extended along the web of the girder up to the top flange of the girder. Figure 3 shows the proposed strengthening scheme for main girders.

After removing all the deteriorated concrete the wet layup procedure was implemented for the application of CFRP which involved saturating the CFRP sheets prior to placement on the concrete surface. The CFRP sheet installation considered a bond-critical application which relies completely on the developed bond of the sheet to the concrete surface to transfer the

stresses. With bond-critical applications, proper surface preparation and application is essential to ensure the compatibility of behavior.

Accordingly, pores of the concrete surface should be opened up so the resins mechanically lock into it. The surface preparation started with simply cleaning the concrete to remove any dirt, followed by grinding and water blasting to achieve the required roughness of the surface. Then all defects in the concrete area, which subjected to strengthening, was repaired and all holes were filled with epoxy. Surface preparation was conducted before any application of the CFRP wrap to enhance the adhesiveness between CFRP sheets and the epoxy resin. Consequently, the concrete surface was treated with epoxy resin based primer of 3 mm thick layer to allow the bond to develop deeply into the concrete. This step was immediately followed by placement of the saturated strengthening sheets.

After strengthening, a load test was carried out to verify the strengthening system. Results of the load test and the numerical analysis proved that the proposed strengthening system improved the stress distribution in all components of the bridge and maintained the original load resistance mechanism provided by the prestressed girders and the deck slab.
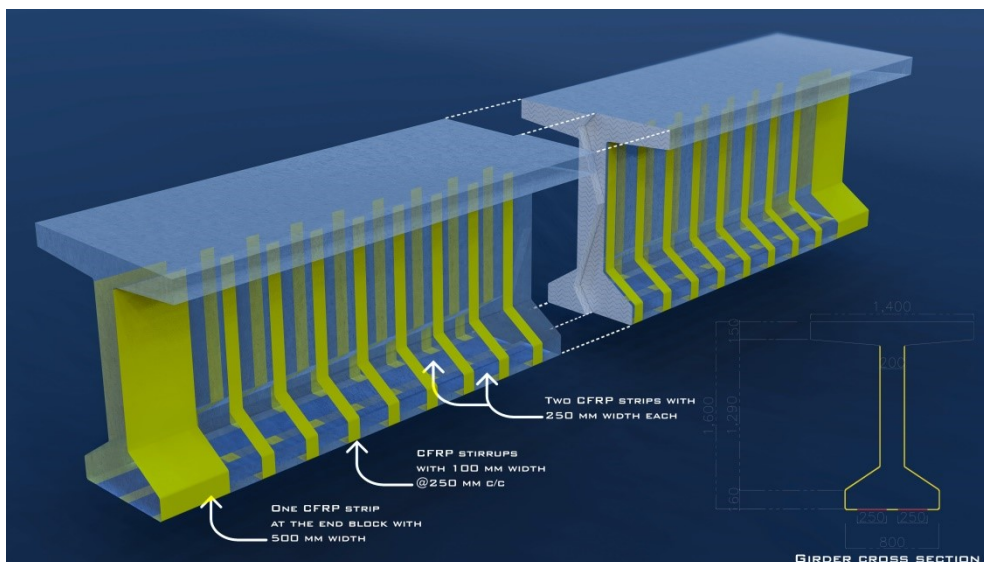

Figure 3. The proposed strengthening scheme for Mohammed Al-Qassim bridge.

7   CONCLUSIONS

A strengthening system was proposed to rehabilitate three defected by fire attack spans of Mohammed Al-Qassim bridge by installing a series of CFRP sheets on the soffits and sides of the main prestressed concrete girders. The required area of the CFRP sheets used for flexural and shear strengthening the I-girders was determined to compensate the degradation of strength of prestressing steel and concrete, respectively.

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