Experimental Studies on the Fire Flame Behavior of Reinforced Concrete Beams with Construction Joints

Ahmed A. Abbood[1,a *] and Majid M. Kharnoob[1,b]

[1]Department of Civil Engineering, University of Baghdad, Baghdad, Iraq

aahmed.abood2001m@coeng.uobaghdad.edu.iq and bdr.majidkharnoob@coeng.uobaghdad.edu.iq

*Corresponding author

Abstract. This study was experimentally investigated experimentally, investigating the effect of such mechanical properties and deflection behavior on the performance of reinforced concrete beams with construction joints exposed to fire flame. These beams were built using a slanted connection at a 45° angle in the middle of the beams. Four beams were for a wide temperature range (25–800°C) for the fire flames used on the concrete beams. Two temperature levels (600°C and 800°C) were chosen, with (1 and 2) hrs. period time, respectively. One beam wasn't burnt (Reference specimen). When put through a fire, flexural strength drastically decreased. The experimental program's results suggested the worst effect on construction joint beam for temperature 800°C with 2 hrs. period time because that heating reduces the bonds between two surfaces in the joint and makes like slip and disconnect between the joint after exposure to load. After 1 and 2 hours in a fire at 600°C, the residual flexural strength was 85% and 72%, respectively, whereas, after 1 and 2 hours in a fire at 800°C, it was 41% and 28%, this is showed that the stiffness of beam its decrease when exposed to fire flam with raising temperature and time and increase the deformation by cracks that appear on the beams. It was noticed that the load-deflection relation to beams exposed to fire flame is flat, representing softer load-deflection behavior than that of the control beam.

Keywords: Construction joint; fire; concrete; beam; cracks; flexural.

1. INTRODUCTION

1.1 Joints in Concrete Structures

Pouring all the concrete for the building's foundation at once is impossible, so joints must be used in the framework. Depending on the temperature fluctuations, joints may be required to relieve tensile or compressive stresses that would otherwise be formed in the structure due to concrete's natural contraction and expansion. Joints of varying types are required in most concrete buildings, and they must be built appropriately and positioned so that the buildings can serve their intended purposes. Joints in a building mustn't interfere with how it works, and it's usually best if the joints blend in with the rest of the building's look [1]. In general, the different kinds of joints may be separated into two basic groups [1]:

(a) Functional joints are positioned to accommodate movement (volume fluctuations) resulting from temperature, shrinkage during setting, expansion, sliding,warping, etc.
(b) Construction joints produced when the construction schedule is disrupted.

1.2 Construction Joints

Construction joints are places where the concrete pouring process stops to make constructing a structure simpler or even feasible, depending on the kind of work, job site circumstances, and the plant's or labor's output capability. This occurs constantly; there is a pause in the building process while massive amounts of concrete are being placed. Joints in structurally sound buildings are not designed toallow motion; they serve as dividing lines between sections of concrete laid in separate pours. Joints used in construction are not to be confused with expansion joints, which are meant to facilitate the unrestricted movement of structural components andare often built for total separation. In most cases, the weakest parts of a building are the places where two or more pieces of construction come together. As such, the current focus should be on perfecting a construction joint that can act as a strong bonding interface between the two types of concrete (cured and uncured). Therefore, joints in concrete buildings should be installed in areas with predicted low shear forces.In order to provide sufficient structural performance and an acceptable aesthetic, the junction's location and size should be established depending on the kind of building [1]. Sometimes, provisions for future extension of a building or a structure are required to be kept. Construction joints are often required at the ends of beams, slabs, tie beams, etc., in such cases for future extension [2].

Reinforced concrete beams' construction joints might be horizontal, vertical, inclined,or key joints, as shown in Figure 1. Assuring adequate shear transmission and flexural continuity is the key concern while constructing a junction. Extending the reinforcement across the junction creates flexural continuity, while dowel action in the reinforcement and shear friction between the old and new concrete create shear transversal [1]. The structure may have areas, such as joints, with less than 100% shear strength. The joints of a building should look like this [1].

To do this, before anything else, the surface of the hardened concrete at the joint should be substantially roughened. The second stage is to remove any rubbish or other materials that may have been embedded in the concrete while the first section of the cast is being completed. Third, it is necessary to properly wet old concrete before pouring the new concrete over previously hardened concrete. Removing any standing water after watering down a concrete surface is crucial.

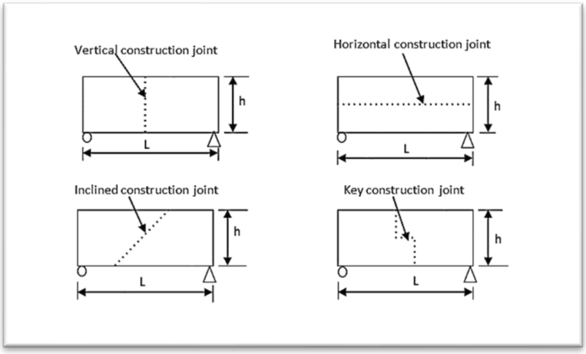

Figure 1: Types of construction joints [1].

1.3 Performance of Structure in Fire

Any building is vulnerable to damage or destruction, which can occur from various causes. Fire is a popular and severe cause of devastation caused by various disasters. Fire catastrophes may severely damage both life and property. Numerous studies have been conducted on fire-damaged buildings. The majority of these examinations are restricted to either the outward condition of the building or the laboratory assessment of structural components. An existing structure cannot be subjected to flexural strength testing or internal inspection. As a result, the structure frequently needs to be completely repaired, which is expensive and causes issues with the service. For this reason, with an understanding of how fire and excessive temperature influence the structure, a correct judgment on the structural integrity of astructure will be able to be made without laboratory testing. As a result, fewer repairswill be required, lowering the cost of fire damage. The average experimental to calculated deflection percentage was 1.06 for the first method and 0.877 for the second method [3].

1.4 Performance of Concrete in Fire

Concrete was formerly assumed to be a non-combustible substance that acted asa thermal barrier to prevent flames from spreading [4]. Concrete conductivity decreases as temperature increases due to pore water loss and cement paste drying. These changes will occur on the surface of heated concrete, producing an insulating material that is heat resistant and reduces heat entry. As a consequence, concrete has excellent fire resistance [5]. Unlike steel, concrete does not regain its structural properties after a fire. After cooling, it typically returns to its original condition, and this is because cement's chemical and physical properties change irreversibly [6,7]. Rapid temperature increases may cause stored water to evaporate and interparticle bonds to loosen, resulting in spalling due to variable expansion coefficients. Extreme temperatures may cause significant spalling in concrete buildings because concrete expands as it warms, but they can also cause hardened concrete paste to shrink. These two processes must be in conflict in order to produce small cracks during cooling. While prestressed concrete limits longitudinal expansion, this is more difficult [8].

Several variables will influence the type of concrete used in high-temperature situations. Considerations for high-temperature concrete include exposure time, rate of temperature increase, initial concrete temperature, the degree of water saturation for concrete, the age of the concrete, the type of aggregate used, the type of cement used, the aggregate/cement ratio, the amount of concrete cover over reinforcing bars or prestressing tendons, and the loading conditions at the time of exposure are all factors to consider. When subjected to high temperatures, concrete's chemical composition, physical structure, and moisture content all change. The aggregates are the first to exhibit indications of deterioration, followed by the cement paste. High temperatures promotethe dehydration of hardened cement paste and the conversion of calcium hydroxide to calcium oxide, resulting in the slow release of chemically bound water into free water.At high temperatures, aggregates lose evaporation water, hydrous aggregates dehydrate, and crystalline transition occurs at a considerable volume expansion temperature [9]. Around 150°C, the concrete characteristics diminished, and the specimens lost some of their initial strength. Although there was no substantial loss of strength between 150 and 300 degrees Celsius, all types of concrete mixes continued to lose compressive strength over 300 degrees Celsius, and the length of heating did not affect this loss [10]. At the same time, [11] length of exposure was shown to significantly affect strength decrease. The concrete loses substantial strength when exposed for over an hour, with the most significant loss occurring between 1 and 2 hours. Not only do aggregates undergo chemical and physical changes between 600 and 900 degrees Celsius, but so does hardened cement paste. The porosity of solid cement paste decreases when it is dried, and water evaporates from the particles [11]. These groups were separated based on the temperature range chosen for the heating exposure (ambient, 400, and 700°C) [12].

1.5 Performance of Steel Bars in Fire

At high temperatures, reinforcing steel's sensitivity is greater than concrete's [13]. Concrete and steel have equivalent thermal expansion up to 400 °C; however, steel expands substantially more than concrete at higher temperatures, and if temperatures near 700 °C, steel bearing capacity is reduced to roughly 20% of that of concrete. Due to the establishment of impermeable regions where moisture can become trapped, the reinforcement can considerably affect water transfer within the heated concrete member. This allows water to flow around the reinforcing bars and raises the pore pressure in particular concrete portions, increasing the possibility of cracking. Furthermore, these retained water zones affect the flow of heat around the reinforcement, tending to reduce the internal temperatures of the concrete [14]. Essentially, the fire behavior of concrete corresponds to the properties of temperature-based components. Because the thermal emission of concrete is relatively low compared to steel, substantial temperature gradients often occur inside exposed concrete members during a fire. The core region may take a long time to heat up due to the significant thermal inertia. As a result, the compressive strength of the concrete is lost at the critical temperature, which is not significantly different from the temperature at which the steel would lose its strength. Based on this, structural efficiency is not reduced until the mass of the material reaches the same temperature [4]. The most common cause of structural collapse is when the reinforcement loses its effective strength due to heating. As a result, most studies show that reinforcements with enough cover will be fire-resistant [15]. The absolute vertical displacement of the external supports, torsional capacity, angle of twist, and first crack occurrences are the variables studied in this research [16].

1.6 Cooling Effect on Concrete Member

When concrete is exposed to high temperatures, the cooling rate significantly influences the residual strength properties. The concrete may be exposed to sudden cooling when a firefighting engine begins impinging water on a burning concrete building. Concrete may be subjected toprogressive cooling, as in the case of chimneys, etc., or even intermittent cooling, as in the case of some firefighting systems. In all of these cases, the concrete is subjectedto varying cooling rates, which undoubtedly affects the residual strength qualities of the concrete. As a result, the investigation of concrete subjected to varying rates ofcooling becomes an important parameter of investigation. The current investigation looks into slow and sudden cooling in concrete that has been exposed to high temperatures [17].

This experimental investigation aims to understand how fire flaming affects the structural behavior of beams with construction joints exposed to fire flame with temperatures (600-800) °C and two periods of 1.0 and 2.0 hrs. Five beam specimens were made and tested for failure under two-point loads. The testing concentrated four beams on the effects of temperatures of 600°C and 800°C over 1.0- and 2.0-hour time periods.

2. EXPERIMENTAL WORK

2.1 Specimens Details

The experimental work of this investigation consists of five beams of normal-strength concrete with the same dimensions and reinforcement. Simply supported beams have dimensions of 200 mm width, 300 mm height, and 2700 mm total beam length, with a distance of 2500 mm between two supports. Two-point loads were applied at a third distance from each support's clear span. The longitudinal reinforcements of beams were 3Φ12 mm at the tension zone and 2Φ10 mm at the compression zone, in addition to Φ10@ 130 mm of shear reinforcement along the beam length. Each beam has a 45° angle construction joint in the middle of its length. There was a 24-hour gap between when the two parts were poured. This was done to determine the temperature range and how long the fire would last. It was decided to limit the amount of time a beam could be exposed to a fire's flame to between 600°C - 800°C, using a range of exposure periods from 1 to 2 hours. For the most part, this addressed all eventualities in tests conducted at elevated temperatures. Then they were put through flexural loads until they broke. The details of all tested beams exposed to fire flame are mentioned in Table 1.

Table 1: Details of the tested beams (Researcher).

Beam specimens' designation	Burning temperature ℃	Period of Exposure (hr.)
C	Without *	----
F1-600	600	1
F2-600	600	2
F1-800	800	1
F2-800	800	2

* Without burning, left at a lab temperature of 25℃.

2.2 Materials

2.2.1 Cement

In this experiment, ordinary Portland cement was utilized. The cement's chemical and physical characteristics are listed in Tables 2 and 3, respectively.According to I.O.S. 5/2019 [18], tests were conducted at the National Center for Building Labs and conducting research.

https://doi.org/10.1051/e3sconf/202342702018

Table 2: Physical properties of cement.

Test		**Test results**	**Iraqi specification (I.O.S. 5/2019) [18].**
Specific surface area. Balain method (m^2/ kg)		365	230 (Min.)
initial setting (Vigat) (hrs: minutes)		1:45	0.45 (minutes-Min.)
Final setting (Vigat) (hr: minutes)		2:50	10 (hr.-Max.)
Compressive strength(N/mm^2)	3 days	18.20	15 (Min.)
	7 days	25.50	23 (Min.)

Table 3: Chemical properties of cement.

Test	**Test results, %**	**Iraqi specification (I.O.S. 5/2019) [18].**
Al_2O_3	4.62	-
SiO_2	20.08	-
Fe_2O_3	3.60	-
CaO	61.61	-
MgO	2.10	5 (Max)
L.S.F	0.95	0.66-1.02
Loss on Ignition, L.O.I	2.50	4 (Max)
Insoluble Residue	0.7	1.5 (Max)
SO_3	2.72	2.5 (Max)
C_3A	6.26	8 (Max)

2.2.2 Coarse Aggregate

Graded crushed gravel of 10mm maximum size. Tests were performed on the grading and physical properties of the coarse aggregate in the KUT technical institute's Elc. and Construction Lab. test, according to IQS 45/1984 [19]. The results are shown in Tables 4 and 5, respectively.

Table 4: Grading of the coarse aggregate.

Sieve size (mm)	**Percentage Passing(%)**	**Percentage passingaccording to Iraqispecifications No. 45/1984 [19].**
14	100.00	100
10	99.72	85-100
5	10.96	0-25
2.36	2.48	0-5

Table 5: Physical properties of the coarse aggregate.

NO	**Properties**	**Test results**	**Iraqi specificationNO.45-1984 [19].**
1	Material passing throughthe sieve (0.075) mm	1.44	3 Max.
2	Sulfate content SO_3, %	0.08	0.1 Max.

2.2.3 Fine Aggregate

Washed red sand with a maximum size of 4.75 mm was used as fine aggregate inall the concrete mixes for the beams. The sieve analysis of the used sand is shown in Table 6, while the physical properties are shown in Table 7. According to the limit of IQS 45/1984 [19].

Table 6: Grading of the fine aggregate (Researcher).

Sieve size (mm)	**Percentage Passing (%)**	**Iraqi specification NO.45-1984Zone 2 [19]**
10	100.00	100
4.75	93.32	90-100
2.36	77.80	75-100
1.18	64.36	55-90
0.6	57.36	35-59
0.3	28.52	8-30
0.15	5.48	0-10

Table 7: Physical properties of the fine aggregate (Researcher).

NO	**Physical Properties**	**Value**	**Iraqi specification NO.45-1984 [19]**
1	Material passing throughthe sieve (0.075) mm	1.76	5 (Max.)
2	Sulfate content SO_3, %	0.21	0.5 (Max.)

2.2.4 water

Tap water was used in all of the mixing and curing processes.

2.2.5 Steel Reinforcement

The steel utilized in this investigation came in two sizes:10 mm and12 mm. Stirrups and top longitudinal reinforcement are made of 10 mm steel bars. The bottom longitudinal reinforcement is made of 12 mm steel bars. The steel bar was tensile tested at the KUT Technical Institute's Electrical and Construction Lab, according to ASTM A615 [20]. The rebar utilized in the testing has a Mass brand, made in Iraq. Tensile strength, yield stress, elongation, and steel bar diameters are summarized in Table 8. The modulus of elasticity Es = 200000 MPa is according to ASTM-A36 [21] for all steel bars.

Table 8: Results and study of steel reinforcement properties (Researcher).

Nominal reinforcingbar diameter (mm)	Actual reinforcing bar diameter (mm)	Area (mm^2)	Tensile strength (N/mm_2)	Yield stress (N/mm^2)	Break. Elong. %
10	9.5	78.54	655.84	506.59	17.15
12	11.75	113.01	639.36	528.42	18.65

2.3 Mechanical Properties of Concrete

Three cube specimens with dimensions of 150 mm were poured and tested undercompression; the average compressive strength is shown in Table 9.

Table 9: Compressive strength of concrete (Researcher).

Specimens No.	Cylinder Compressive Strength (MPa)	Average cube'scompressive strength (MPa)
1	36.545	
2	31.784	
3	36.654	34.99

2.4 Mix Proportions

One mix was created to meet the requirements for fresh characteristics and the compressive strength ceilings used in this study. Thus, several test mixtures were created before the 35 MPa compressive strength was chosen. Details of the mix are given in Table 10.

Table 10: Details of the trial mixes* (Researcher).

Mix Ratio (by weight)	w/c	Mix Proportion (kg/m^3)				Comp. strength, MPa
		Water	Cement	Sand	Gravel	fc' *
1: 1.8: 3.2	0.50	180	360	660	1150	**38**
1: 1.8: 3.2	0.55	198	360	660	1150	**35**
1: 1.4: 3.2	0.50	190	380	535	1215	**40**

***** These values are the average of three control specimens, Cubes of 150mm dimensions.

The mixing of concrete was accomplished with a tilting pan mixer. In all the mixtures, the aggregates and cement were initially mixed dry for approximately 60 seconds, followed by adding water and mixing for another 60 seconds. After mixing, the concrete was poured into three layers of lightly greased molds and crushed with a handle rod vibrator.

2.5 Burning Procedure and Test Setup

To burn one specimen at a time, the furnace was constructed from a 3 mm thick steel plate in the configuration of two L-shapes or beam covers (see Figure 2). From the fire sources, to get to the fire flame (high temperature), the inner clear area was 500 mm in height, 500 mm in width, and 3000 mm in length, which matched the proportions of the concrete beam (nozzle). All 18 of the beam's nozzles fired off at thesame instant.

Figure 2: The furnace.

Four beams were burned by fire at two temperature levels (600 and 800°C) for one and two hours, respectively. The furnace was covered with mud insolation to preserve the high temperature within attaining the desired temperature. The fire was extinguished after this time. The specimen was progressively cooled by keeping it in the air after the furnace case was removed. This study's experimental effort included analyzing several variables such as strain, deflection, and load capacity utilizing measurement equipment. Three 5 mm strain gauges were installed on the reinforcing steel bars of the unburned beam, as shown in Figure 3. One strain gauge was placed in the center of the main reinforcement, another in the center of the top reinforcement, and the last one on the first stirrup. Figure 4 shows the location of the concrete strain gauge of 60 mm, which was placed in the center of the beam on the compression extreme sides. The strain gauges were connected to a data logger, which recorded the strains per second, as illustrated in Figure 5. The beams were tested in a load-control protocol and subjected to monotonic increasing loading until failure. The beams were loaded using a hydraulic jack with a capacity of 1000 kN, as indicated in Figure 6. The applied load was measured using a load cell with a 500 kN loading capacity. Vertical deflection at mid-span was measured using LVDT. The length of LVDT was 150 mm, as shown. Figure 7 presents the test setup utilized in this experiment. Figure 8 shows a thermocouple sensor wire attached to a digital thermometer reading.

A. Top and bottom steel strain B. stirrup steel strain

Figure 3: Locations of the steel strain gauge.

Figure 4: Locations of the concrete strain gauge.

Figure 5: National instruments data logger.

Figure 6: Load cell.

Figure 7: Linear variable differential transformer (LVDT).

Figure 8: Digital thermometer.

3. RESULTS AND DISCUSSIONS

3.1 Effects of Burning on Concrete's Compressive Strength

The results of a study into the effects of fire on compressive strength are shown in Table 11 to 600 °C for one and two hours, respectively. This resulted in a residual compressive strength of (63.5, 59.2) % compared with the cube without burn. Residual compressive strength after exposure to the fire flame was found to be (50.9, 47) % after (1.0,2.0) hours of exposure, respectively, at 800°C.

Table 11: The concrete specimens' compressive strength was tested before and after being exposed to flames.

Age at exposure (days)	Exposure Period (hrs)	Compressive Strength (MPa) Temp. (°C)			Residual of compressive strength (600, 800) °C respectively
		25	600	800	
28	Without Fire	35	----	----	----
60	1.0	40	25.4	20.39	(63.5, 50.9) %
	2.0		23.69	18.81	(59.2, 47) %

3.2 Effects of Fire Flame Burning on the Modulus of Elasticity of Concrete

Table 12 shows a summary of the results of the elasticity modulus tests. These results clearly show that, at the same fire flame temperatures, the reduction values of the elasticity modulus were more significant than those of the compressive modulus. The modulus of elasticity retained between (28 and 25) % of its original value at 600°C. At 800°C, the residual modulus of elasticity was (19 and 16) %. Loss of strength is the consequence of the physicochemical transformation of concrete's ingredients when it burns, which is connected to an increase in the frequency of cracks created due to fire exposure.

Table 12: The concrete specimens' modulus of elasticity of concrete was tested before and after being exposed to flames (Researcher).

Age at exposure (days)	Period of exposure (hours)	Modulus of Elasticity of Concrete (GPa) Temperature (°C)			Residual of Modulus of Elasticity (600, 800) °C respectively
		25	600	800	
28	Without Fire	33	----	----	----
60	1.0	36	10.2	6.7	(28, 19) %
	2.0		9	5.9	(25,16) %

3.3 Effect of Burning on Load-Deflection and Strain in Concrete Results

Beams with construction joints were burned in a fire at 600°C and 800°C, and their load-deflection curves were compared to those of unburned beams. The maximum residual force was used until failure occurred. At each loading step,the beam's midspan deflection was recorded, as was the load at which cracking became apparent (in the control beam) and failure occurred. All the while, the deflection was measured, and its rate of rise was monitored so that an early warning of beam specimen failure could be provided. Table 13 summarized the best results; the results showed that the residual ultimate load capacity for the specimens that burned at fire temperature (600°C) was (84.4 and 71.4%) with 1.0- and 2.0-hour exposure durations, respectively. Accordingly, the flexural ultimate strength of beams decreased by 15.6% and 28.6%, respectively, and at fire temperature (800°C), the residual ultimate flexural capacity was 41.6% and 28.8%, respectively. This suggests that the loss in ultimate strength was 58.6% and 71.2%. With (1.0- and 2.0-) hour exposure periods compared with control specimens. Figure 9 and Figure 10 depict the relationship between load and deflection. These figures show that as the fire temperature rises, the load-bearing capacity declines and the deflection of beam specimens increases. This is explained by the fact that heating decreases beam stiffness, primarily caused by a decrease in the elastic modulus of concrete and a reduction in the effective section due to cracking [5]. The load-deflection relationship of the beam specimens is also shown in these figures to be virtually linearly proportional for both the temperature exposure (600°C and 800°C) and the two exposure periods (1.0 and 2.0 hours). Beam specimens' load versus mid-span deflection relations showed softer behavior at 600°C and 800°C than at lower burning temperatures and with the control specimens. The reduced binding strength between the concrete and the steel reinforcement is to blame for this. This may be explained by the fact that the bond strength between the concrete and the steel reinforcement in the structure is not as strong as it might be. The tensile strength difference of steel reinforcing rebars subjected to high temperatures reveals no detectable loss of tensile strength for all kinds of steel rebars up to 500°C temperature. Steel rebars saw a 31 and 45 percent drop in tensile strength after being exposed to 900 degrees Celsius. At 1000°C, the maximum temperature of exposure, the tensile strength dropped by 38% and 45% [22].

The beam produced two sorts of cracks after being treated to fire ignition. The first was thermal cracks that appeared all over the surface in a honeycomb pattern. They started at the top or bottom edges and finished at about the mid-depth of the beam. Flexural tensile cracks induced by mid-span tension created the second crack. The data presented here show that beam specimens' load-bearing capacity and deflection decrease with increasing fire temperature. The elastic modulus of concrete and the effective section of the beam decreases with temperature, so these two phenomena together explain why heating reduces beam stiffness. The beam specimens' load against mid-span deflection relations demonstrated a softer behavior at temperatures of 600 °C and 800°C and a lower burning temperature when compared to those of the control beam specimens. Figures 11 and 12 depicted the load and strain compere relationship between the control beam and each burn state. Figures 13 to 15 depict the relationship between load and strain for reinforcement steel (unburn beam). It shows the behavior of flexural failure. The load-strain relations on the concrete surface were measured at specific locations (the front side of the beam in the compression zone). Since concrete deteriorates when exposed to fire, it is evident from all of the load-strain curves that the stiffness of a beam declines with increasing burn temperature for the same compressive strength value. Table 14 summarizes the ultimate load and max. Strain in concrete for reference beams and beams exposed to fire flame.

Table 13: The tested beams' first crack load, ultimate load, and maximum deflection results.

Temp. (°C)	Specimen Identification	First Crack Load (kN)	Ultimateload (kN)	Percentage Residual Ultimate Load %	Max Deflectionat Midspan (mm)
25	C-25°C	20	86.4	100	23
600	F1-600°C	Pre-cracking	72.9	84.4	26
	F2-600°C	Pre-cracking	61.7	71.4	28
800	F1-800°C	Pre-cracking	35.9	41.6	32
	F2-800°C	Pre-cracking	24.9	28.8	35

Table 14: Ultimate load and maximum strain in the concrete of tested beams.

Temp. (°C)	Specimen Identification	Ultimateload (kN)	Max. Strain (10^{-6})
25	C-25°C	86.4	2886
600	F1-600°C	72.9	2761
	F2-600°C	61.7	2587
800	F1-800°C	35.9	1997
	F2-800°C	24.9	1786

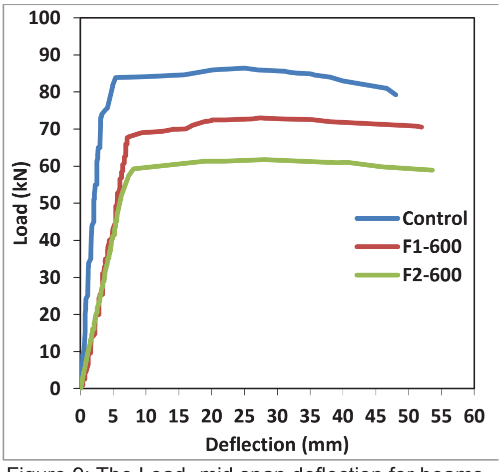

Figure 9: The Load- mid span deflection for beams beams C, C, F1-600 and F2 -600.

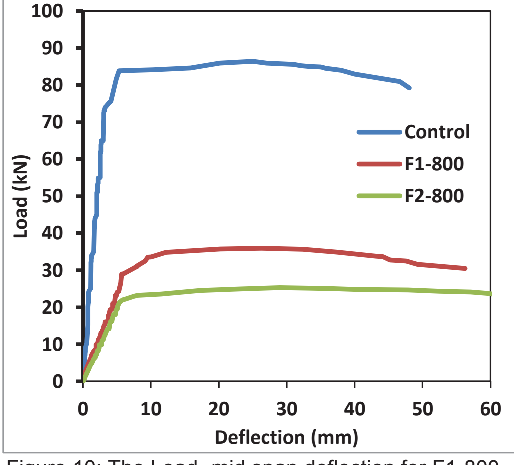

Figure 10: The Load- mid span deflection for F1-800 and F2-800.

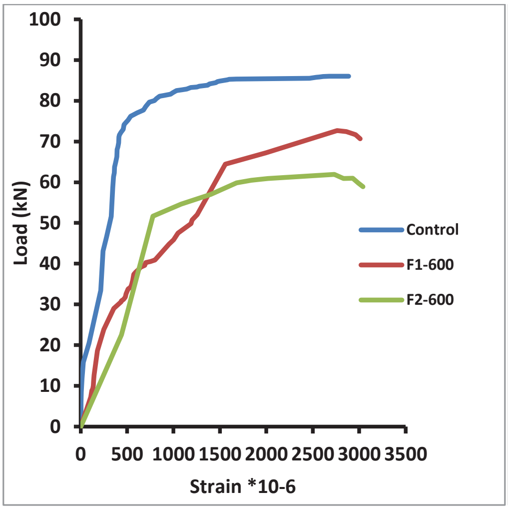

Figure 11: Load-Strain for Concrete Surface of Beams C, F1-600 and F2-600.

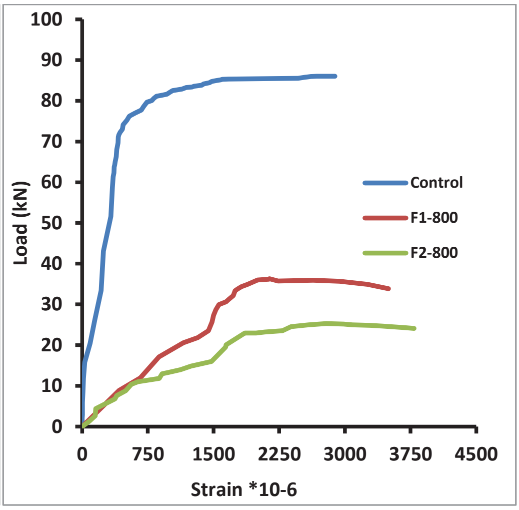

Figure 12: Load-Strain for Concrete Surface C, F1-800, and F2-800.

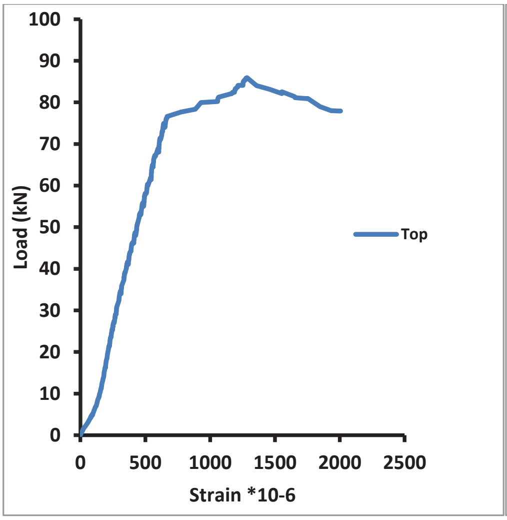

Figure 13: Load-strain for reinforcement steel of unburn beam (top side).

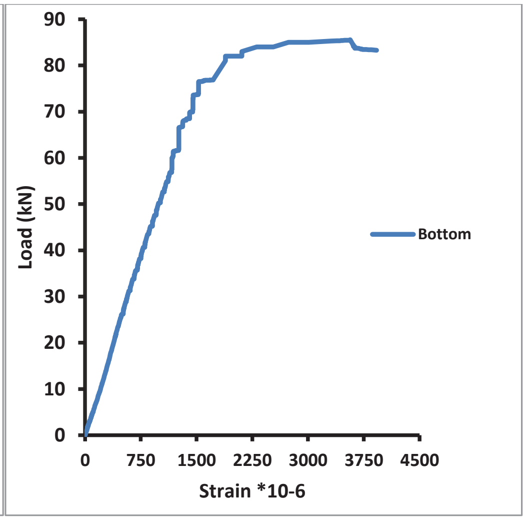

Figure 14: Load-strain for reinforcement steel of beam (bottom side).

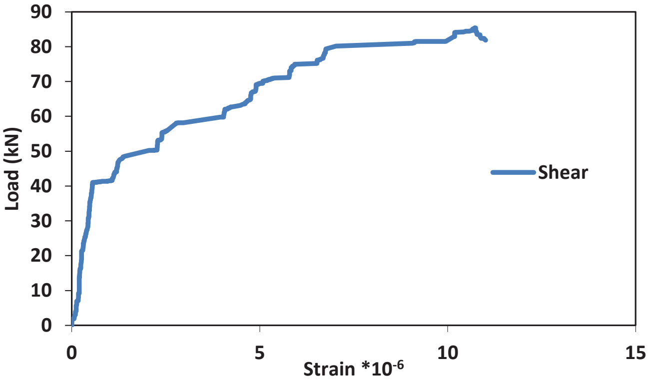

Figure 15: Load-strain for reinforcement steel of unburn beam (stirrups).

3.4 Crack Pattern and Failure Mode

All of the burned beams had a flexural failure. Cracks have appeared on all sides of the beams, which grew from the fire cracks. The cracks were noticed on the specimens' side faces. The crack pattern for the burned specimens is shown in Figures 16 to 20. The control beam C (without burn) failed in flexural mode due to the yielding of the bottom reinforcement, as evidenced by the visible large cracks at the beam mid-zone near the failure load with increasing deformation at the same applied load, as well as the visible permanent deformation after removing the applied load. Initially, there were flexural fractures along the bottom of the beam when the applied load was around 20 kN. Some of these flexural cracks in the intermediate zone of the beam progressed toward the beam's compression zone. Figure 16 depicts the failure of the beam at a total applied load of 86.4 kN. As the applied load rose, new cracks occurred throughout the beam and widened and propagated until the beam failed C.

The beams experienced cracks before the static load was applied due to the burning and cooling processes. Therefore, it was challenging to tell the new cracks apart. Beam F1/F2 -600 burned for one and two hours at a temperature of 600°C. These beams behaved the same way during the initial stages of loading, but as the applied load reached nearly 13 kN for beam F1-600 and 14 kN for beam F2-600, the cracks grew wider and spread, leading to failure of the beams at loadings of 72 and 61 kN, respectively, which illustrated in Figures 17 and 18. Significant deterioration was noticed in specimen F2-800, where cracks emerge and the concrete surface spalls, particularly close to the corners. Consequently, when the beam was tested under static load, it was very difficult to tell when new cracks formed from those caused by burning and cooling. However, when the applied load was 10 and 4 kN for the beams, which reached 800°C for 1 hour and 800°C for 2 hours, respectively, the cracks grew upward, propagated, and became wider until failure occurred at loads of 35.9 and 24.9 kN, respectively according to Figures 19 and 20, for beams burnt at 800°C.

Figure 16: Cracks pattern for specimen (C).

Figure 17: Cracks pattern for the specimen (F1-600).

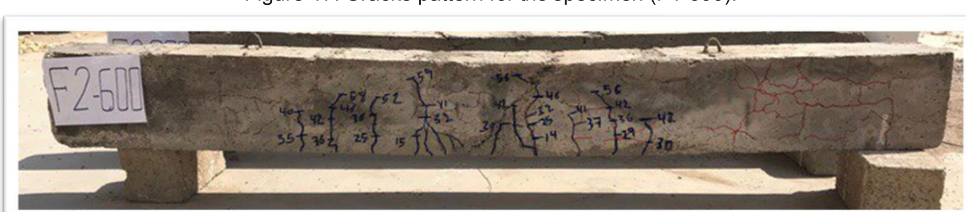

Figure 18: Cracks pattern for the specimen (F2-600).

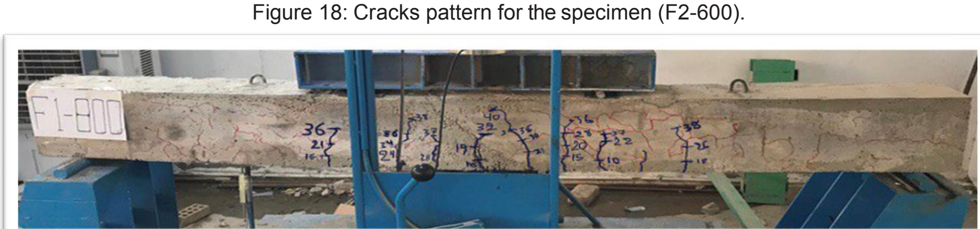

Figure 19: Cracks pattern for the specimen (F1-800).

Figure 20: Cracks pattern for the specimen (F2-800).

4. CONCLUSIONS

- With increasing heat from the fire, the compressive strength decreases. This is the reason for the cracks. It happens when exposure to fire and high temperature increases when increasing the temperature with an increase in the time that decreases compressive strength. As the exposure temperature rises, after one and two hours of exposure to fire temperatures of 600°C, the residual compressive strength was 63.5 and 59.2%, respectively. However, after the same duration of time at 800°C, the values dropped to 50.9 and 47 %, compared to the cube (without fire).
- After one and two hours of exposure to fire temperatures of 600°C, the residual flexural strength was 85% and 72%, respectively. However, after the same time, at 800°C, the values dropped to 41 and 28%, compared to the control beam (without fire). This shows that the stiffness of the beam is affected when exposed to fire flam with different temperatures and times.
- After one and two hours of exposure to fire temperatures of 600°C, the residual modulus of elasticity of concrete was 28% and 25%, respectively. However, after the same time, at 800°C, the values dropped to 19% and 16%, compared to the control beam (without fire). Loss of strength is the consequence of the physicochemical transformation of concrete's ingredients when it burns, which is connected to an increase in the frequency of cracks created due to fire exposure.
- Increasing the temperature by fire flame affects the load–strain behavior on the concrete at the ultimate load. The recorded strain was (2886, 2761, 2587, 1997and 1786) microstrains for the specimens (C, F1-600, F2-600, F1-800, and F2-800) respectively.
- It should be noted that as the fire temperature rises, the deflection increases and the load-bearing ability of beam specimens decreases. This is because heating reduces beam stiffness and increases deformation.
- Two types of cracks formed when the beams were exposed to fire. The first was thermal cracking, which occurred all over the surface in a honeycomb pattern. The second kind of crack, known as flexural cracks, formed in the mid-span area due to bending caused by the applied stress, making burnt beams weak.
- Compared to the reference beam, the load-deflection relations at 600 and 800°C burning temperatures were softer and flatter. The early crack development and decreased elasticity modulus are responsible for this discovery. It should be observed that, compared to the beams not exposed to fire, all of the exposed beams had cracks before the load test started.
- The outcomes of the experimental program showed that the worst effect on the construction joint in the beams was its temperature of 800°C for two hours because that heating reduces the bonds between two surfaces in the joint and makes a slip and disconnect between the joint after being exposed to a load.
- Fire flame temperature impacts concrete's modulus of elasticity more than its compressive strength.

REFERENCES

[1] Abass ZW. Effect of construction joints on performance of reinforced concrete beams. Al-Khwarizmi Engineering Journal. 2012 Feb 29; 8(1):48-64.
[2] Gerges NN, Issa CA, Fawaz S. The effect of construction joints on the flexural bending capacity of singly reinforced beams. Case Studies in Construction Materials. 2016 Dec 1; 5(1):112-123.
[3] Abdulkareem BF, Izzet AF. Residual post fire strength of non-prismatic perforated beams. In IOP Conference Series: Earth and Environmental Science. 2022; 961(1):012002).
[4] Fletcher IA, Welch S, Torero JL, Carvel RO, Usmani A. Behaviour of concrete structures in fire. Thermal science. 2007; 11(2):37-52.
[5] Kadhum MM. Effect of burning by fire flame on the behavior of reinforced concrete beam models. Journal of Babylon University. 2013; 21(5).
[6] Alarcon-Ruiz L, Platret G, Massieu E, Ehrlacher A. The use of thermal analysis in assessing the effect of temperature on a cement paste. Cement and Concrete research. 2005 Mar 1; 35(3):609-13.
[7] Mohaisen SK, Abdulhameed AA, Kharnoob MM. Behavior of reinforced concrete continuous beams under pure torsion. Journal of Engineering. 2016 Dec 1; 22(12):1-5.
[8] BRITAIN, Fire resistance of concrete structures: report of a Joint Committee of the Institution of Structural Engineers and the Concrete Society, 1975, London: Institutionof Structural Engineers.
[9] A Ferhat Bingol and Rustem Gul. Compressive Strength of Light weight Aggregate Concrete Exposed to High Temperatures. Indian Journal of Engineering and Materials Science. 2004 Feb; 11(1): 68-72.

[10] Umran MK. Fire flame exposure effect on some mechanical properties of concrete. Master A Thesis, College Engineering, Civil Department, Babylon University. 2002 Oct.
[11] Bažant ZP, Kaplan MF, Bazant ZP. Concrete at High Temperatures: Material Properties and Mathematical Models. London: Addison-Wesley. 1996.
[12] Abdulkareem BF, Izzat AF. Serviceability of post-fire RC rafters with openings of different sizes and shapes. J Eng. 2022 Jan 1; 28(1):19-32.
[13] Bilow DN, Kamara ME. Fire and concrete structures. In Structures Congress: Crossing Borders. 2008.
[14] Chung JH, Consolazio GR. Numerical modeling of transport phenomena in reinforced concrete exposed to elevated temperatures. Cement and Concrete Research. 2005 Mar 1; 35(3):597-608.
[15] Kadhum, M.M. Behavior of Fire Exposed of Reinforced Concrete Columns. Ph.D. Thesis, College of Engineering, Department of Civil Engineering, Baghdad University, Baghdad, Iraq. 2010.
[16] Kodur VK, Bisby LA. Evaluation of fire endurance of concrete slabs reinforced with fiber-reinforced polymer bars. Journal of structural engineering. 2005 Jan; 131(1):34-43.
[17] Saqier, A.A. Effect of Fire Flame Exposed on Flexural Behavior and Shear Strength of Reinforced NSC and HPC Beams. M.Sc. Thesis, College ofEngineering, Department of Civil Engineering, University of Babylon.1984.
[18] Iraqi Standards (I.O.S 5/2019).
[19] Iraqi Specification, No.45, 1984.
[20] ASTM A615/A615M-16. Standard specification for deformed and plain carbon-steel bars for concrete reinforcement. ASTM Standards. 2016.
[21] ASTM International. ASTM A36/A36M-19 Standard Specification for Carbon Structural Steel. 2019.
[22] Bharathi MU, Kumar CP. Study on fire performance characteristics of steel reinforcement in RCC structures. In IOP Conference Series: Earth and Environmental Science. 2022 Sep 1; 1086(1):012005.