Effect of Burning by Fire Flame on the Behavior of Reinforced Concrete Beam Models

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Abstract

This research is devoted to investigate the behavior of reinforced concrete beams loaded and exposed to fire flame. In this study, some mechanical properties and deflection behavior of rectangular reinforced concrete beam specimens under the effect of burning is presented. The concrete specimens and beams were subjected to fire flame temperatures ranging from (25-800 °C) at different ages of 28, 60 and 90 days, three temperature levels of 400, 600 and 800 °C where chosen for exposure duration of 1.0 and 2.0 hours.

Load-deflection curves indicate changing response with the fire exposure. Also, it was found that the shrinkage values increase with temperature increase.

Results indicate remarkable reduction in flexural strength after exposure to fire flame. The residual flexural strength is (88 and 81%) with 1.0 and 2.0 hour exposure periods respectively at fire temperature (400ºC), while the residual at fire temperature (600ºC) is (66 and 53%) with 1.0 and 2.0 hour exposure periods respectively, while the residual at fire temperature (800ºC) is (45 and 36%) with 1.0 and 2.0 hour exposure periods respectively.

It is found that the ACI 318/ 2008-Code equations are safe to predict bending moment capacity for fire temperature (400ºC), while at burning (600ºC) gave overestimated results to predict moment capacity, but become unable to predict moment capacity at fire temperature (800ºC).

الخلاصة

إن الغرض من هذا البحث هو التحري عن سلوك العتبات الخرسانية المسلحة المحملة والمعرضة إلى لهب النار المباشر. في هذا البحث تم دراسة تأثير لهب النار المباشر على بعض الخواص الميكانيكية وسلوكية العلاقة بين الحمل والانحراف.تم تعريض النماذج الخرسانية للنار بدرجات حرارة تراوحت بين (25–800) درجة مئوية وبأعمار مختلفة تراوحت (28 , 60 , 90) يوما في مستويات حرارة (400، 600، 800) درجة مئوية في فترتين من التعرض (1 و 2,0) ساعة .

إما منحنيات الحمل–الانحراف كانت الاستجابة متغايرة مع درجة التعرض للنار. كما لوحظ إن قيم انكماش الجفاف تتزايد بازدياد الحرارة.

لقد أشرت النتائج انخفاض ملحوظ في مقاومة عزم الانحناء بعد التعرض إلى الحرق، حيث كانت مقاومة عزم الانحناء المتبقية هي (88 and 81%) بفترات تعرض (1، 2) ساعة على التوالي بعد التعرض إلى درجة حرارة (400) درجة مئوية، بينما كانت (66 and 53%) بفترات تعرض (1، 2) ساعة على التوالي بعد التعرض إلى درجة حرارة (600) درجة مئوية، و كانت (45 and 36%) بفترات تعرض (1، 2) ساعة على التوالي بعد التعرض إلى درجة حرارة (800) درجة مئوية على التوالي.

النتائج العملية وجدت إن معادلات الكود الأمريكي (ACI 318/2008) أمينة لتنبؤ سعة عزم الانحناء لحد درجة حرارة (400) درجة مئوية، بينما في درجة حرارة (600) درجة مئوية تعطي قيم أعلى من القيم المختبرية في حين تصبح قاصرة للتنبؤ في درجة حرارة (800) درجة مئوية.

- Introduction

One of the problems confronting buildings is the exposure to high temperatures, hence the buildings should be provided with sufficient structural fire resistance to withstand such circumstances, or at least give occupants time to escape before strength and, or stability failure ensue.
Kadhum (2010).

Fire endurance periods are determined usually by physical tests conducted according to the provisions of (ASTM E119-01). Under this standard, the fire endurance of a member or assembly is determined by the time required to reach any of the following three end points:

1- The passage or propagation of flame to the unexposed surface of the test assembly;
2- A temperature rise of 163°C at a single point or 121°C as an average on the unexposed surface of the test assembly; and
3- Failure to carry the applied design load or structural collapse.

The main goals of this study are as follows:

1. Experimental study the fire flame effect on the flexural behavior strength of reinforced concrete beam specimens and comparing the results with control specimens.
2. Searching the fire flame effect on some mechanical properties of concrete, such as a compressive strength, modulus of elasticity and drying shrinkage before and after exposure to fire flame.
3. Investigation the fire endurance of reinforced concrete beam specimens.
4. Studying the fire flame effect on the immediate deflection of reinforced concrete beams and comparing the results with control beams.
5. Experimental investigation of the fire effect on the cracking tendency and pattern in reinforced concrete beams before and after exposure to fire flame.
6. Studying the fire flame effect on the shrinkage cracking of the specimens before and after exposure to fire flame.

- Fire Effect on the Mechanical Properties of Concrete

Elizzi et al. in (1987) investigated the influence of different temperature on the compressive strength and density of concrete. They used (100×100×100 mm) cubes heated for a short duration (one hour) to temperature ranging from (20-100)°C and the ages of concrete at heating were (14, 28, 90 days). The test results showed that the compressive strength decreased 10% from the original strength up to 400°C and, at 600°C the strength reduction was 50% from the original. They noticed that there was a large strength reduction when heated to temperatures above 400°C. They also mentioned that the small reduction in density up to 300°C was a result to the loss of the free water from concrete specimens. At temperature above 300°C, large reduction in density took place because of loss of the reduction water in concrete.

Umran (2002) investigated the fire exposure effect on some mechanical properties of concrete. The specimens were subjected to fire flame ranging between (25-700)°C. Three temperature levels of (400,500, and 700)°C were chosen with four different exposure duration of 0.5, 1.0, 1.5, and 2.0 hours without any imposed loads during heating. The specimens were heated and cooled under the same regime and tested after exposure to fire flame at ages (30, 60, and 90 days). Compressive strength of 150mm cubes and flexural strength of (100×100×400mm) prisms were measured. Ultrasonic pulse velocity (U.P.V) and dynamic modulus of elasticity (E_d) were tested. Also, he found that the residual compressive strength ranged between(70-85%) at 400°C, (59-78 %) at 500°C and (43-62 %) at 700°C. The flexural strength was found to be more sensitive to fire flame exposure than the compressive strength. The residual flexural strength was in the range of (67-78%) at 400°C, (40-67 %) at 500°C and (20-45 %) at 700°C. He also found that the ultrasonic pulse velocity (U.P.V) and dynamic modulus of elasticity (E_d) were more sensitive to fire flame than compressive strength. He also noticed that exposure time after one hour has a significant effect on residual compressive strength of concrete.

Saqier in (2008) investigated the effect of fire flame exposure on properties of normal and high performance concrete. The specimens were subjected to fire flame ranging between (25-700˚C). Two temperature levels of (400 and 700 ˚C) were chosen with two different exposure duration of (1,1.5) hours without any imposed loads during heating. The specimens were heated and cooled either by air or water. He found that :

1- The percentage residual compressive strength for NSC and HPC ranged between (77-84%) and (51-73%) at 400˚C, (29-65%) and (40-50%) at 700˚C respectively.

2- The splitting tensile strength is more sensitive to fire flame exposure than the compressive strength. The percentage residual for HPC at (400˚C and 700˚C) is about (61-78%) and (37-40%) respectively, while the percentage residual for NSC at (400˚C and 700˚C) is about (50-80%) and (41-51%) respectively.

3- The flexural strength is more sensitive to fire flame temperatures than compressive strength and splitting tensile strength. The percentage residual flexural strength for HPC after exposure to (400˚C and 700˚C) is about (34-40%) and (13-21%) respectively, while the percentage residual flexural strength for NSC at (400˚C and 700˚C) is about (37-42%) and (16-27%) respectively.

- Shrinkage of Concrete Before and After Burning

Neville (1995) reported that the given workability, which approximately means a concrete water content, shrinkage is unaffected by an increase in the cement content, or may even decrease, because the water/cement ratio is reduced and the concrete is therefore, better able to resist shrinkage.

Habeeb in (2000) found from the test results, that the additional shrinkages values due to heating are between (400-800)°C micro strains, and there is no significant increase in shrinkage values due to the increase of exposure time from 1.0 to 4.0 hours. Shrinkage values were not more than 10% of that at 1.0 hour exposure.

- Effect of Burning on Reinforced Concrete Beams

The behavior of reinforced concrete structure exposed to fire depends on the thermal properties of steel and concrete, strength and stiffness properties of the concrete and steel at elevated temperatures, and on the ability of the structure to redistribute internal forces during the course of the fire Purkiss (1984).

Asa´ad in (1987) studied the behavior of structural reinforced concrete beam specimens subjected to elevated temperature. Four type of reinforced concrete samples were used. Singly and doubly reinforced concrete beams having the dimension of (100×100×1100 mm) were used. The double reinforced with both tension and compression steel, while the singly reinforced beams with tension steel only. Continuous beams (100×150×1300mm) and structure frame with outer dimensions of (900×150×1300 mm) and structure frame with outer dimensions of (900×750mm) were used. The frame had a cross-section of (100×150mm) for the beam and (100×100mm) for the column. The specimens were subjected to temperatures of (150, 300, 600, 750 and 900 °C) at the ages of 30 and 90 days and tested the flexure after cooling. The research found that both flexural strength and stiffness decreased with the temperature increase. He also noticed that the use of top reinforcement had limited this decrease. Moreover, he observed that the increase in temperature led an increase in magnitude of moment redistribution in continues beams.

Experimental Work

- Introduction

The experimental work was carried out to decide upon the temperature range and duration of burning. It was decided to limit maximum exposure to fire flame to about 400°C, 600°C and 800°C with two different exposure durations of 1.0 and 2.0 hours which covered the range of situation in the majority of elevated temperature test. The beam specimens were loaded to 35% of the ultimate load before burning. These loads were held constant during the exposure to fire, then they were tested to failure under flexural loads.

- Materials and Mixture Properties

In this investigation, the cement used is Ordinary Portland Cement (O.P.C) produced at Kufa factory. This cement complied with the Iraqi specification No.5 (1984). The physical properties and chemical composition are presented in Table (1). The gravel used was brought from Al-Nibaii area with a maximum size 19mm and the fine aggregate Al-Akhaider well graded natural silica sand was used. Deformed steel bars of 10mm diameter were used for longitudinal reinforcement and plain bars of 6mm were used for stirrups.

The concrete mix was designed according to American mix design method *(ACI 211.1-91)* specification. The proportions of the concrete mix are summarized in Table (2).

Table (1): A- physical properties of cement

Physical properties	Test results	IQS (No.5: 1984) Limits
Fineness, Blaine, cm^2/gm	3220	≥ 2300
Setting time , Vicat´s method		
Initial hrs : min	1 : 45	≥ 1 : 0
Final hrs : min	3 : 75	≤ 10 : 0
Compressive strength of 70.7 mm cube,	24.2	
MPa	28.5	≥ 15
3 days		≥ 23
7 days		

B- Chemical Properties of the Cement

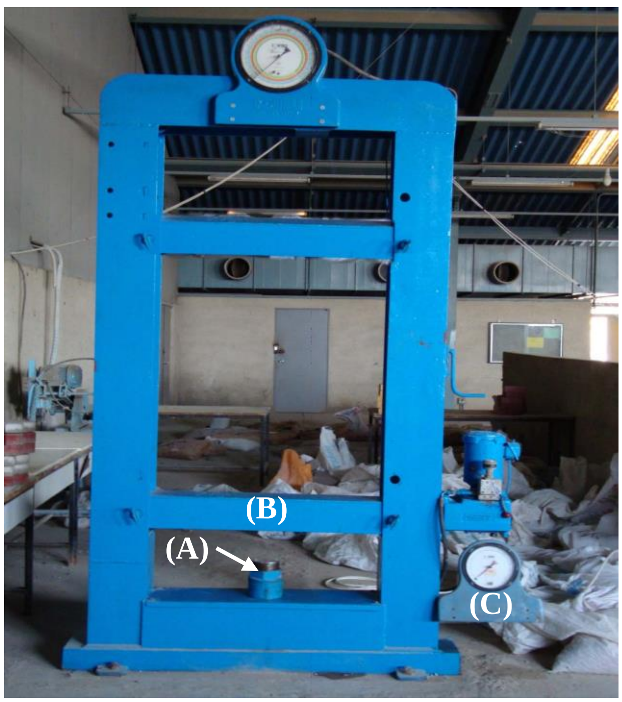
Oxide	Percentage (%)	IQS (No.5: 1984) Limits
CaO	59.50	
SiO_2	21.30	
Fe_2O_3	3.30	
Al_2O_3	6.20	
MgO	4.24	≤ 5
SO_3	2.12	≤ 2.8
Free lime	0.73	
L.O.I	1.58	≤ 4.0
I.R	0.67	≤ 1.5

Table (2): Mix Properties.

Slump mm	W/C Ratio	Weight Proportion	Mix Properties kg/m³			
		Cement : Sand : Gravel	Water	Cement	Sand	Gravel
60	0.45	1.0 : 1.6 : 2.4	196	435	696	1043

- Loading Testing Machine

In this research work the loading machine shown in Plate (1) was manufacture to test the polystyrene concrete ferrocement plate specimens. The loading machine was calibrated in Central Organization for Standardization and Quality Control before accomplishment of the experimental test. The details of the steel frame section W12×79 used in the testing machine is given in Table (3).



(A) The load cell unit with 1500kN capacity
(B) Moveable solid beam
(C) A load dial gauge unit

Plate (1): Flexural strength testing machine.

Table (3): Details of the steel frame section W12×79 used in the testing machine.

Designation	Area A	Depth d	flange		web thickness tw	Nominal Weight Per (m)
			Width bf	Thickness tf		
	(mm2)	(mm)	(mm)	(mm)	(mm)	(Kg.)
W12×79	(13612.8)	(311.15)	(305)	(17.02)	(10.92)	(107.146)

- Reinforced Concrete Beam Specimens

The test included 16 beam specimens. Four beams were retained as reference beams for 60 days of age, twelve were exposed to fire flame with different temperature, different periods of exposed. The beams were covered with polyethylene sheet in the laboratory for about 24 hours, and then demolded for curing in water for 28 days. The beams were simply supported. Figure (1) shows the details of reinforcement for beams. The details of the reinforced concrete beam specimens are shown in Table (4).

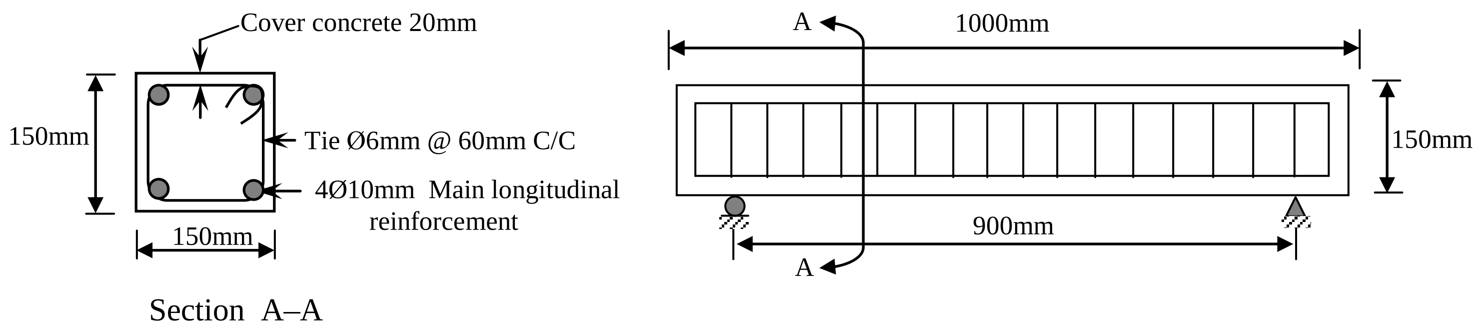


Figure (1): Dimensions reinforcement details of reinforced concrete beam specimens

Table (4): Summary of beam test specimens.

Number of Specimen	Temperature Stage °C	Period of Exposure (hr)	Beam Specimen Designation
4	25	----	BR-25°C
2	400	1.0	BA-1-1.0hr
2		2.0	BA-2-2.0hr
2	600	1.0	BB-1-1.0hr
2		2.0	BB-2-2.0hr
2	800	1.0	BC-1-1.0hr
2		2.0	BC-2-2.0hr

- Test Setup

After 60 days from casting, all of the beam specimens were transported to the hydraulic testing machine and then burned at temperature of (400, 600 and 800 ^{o}C) for 1.0 and 2.0 hours exposure period. Thereon, the beam specimens were tested in flexure by two point load.

Beam specimens were tested as simply supported beams over 900mm span in 1500 kN capacity hydraulic machine. Each beam specimen was supported and loaded by rollers. Forces were distributed through steel bearing plate 150mm in length to cover the entire Beam width. Figure (2) shows a schematic diagram for loading arrangement.

Each specimen was loaded directly at the top face with two equal concentrated loads. Before loading, initial reading of deflection dial gauge was obtained. Deflection was measured at mid span by a dial gauge as shown in Figure (2).

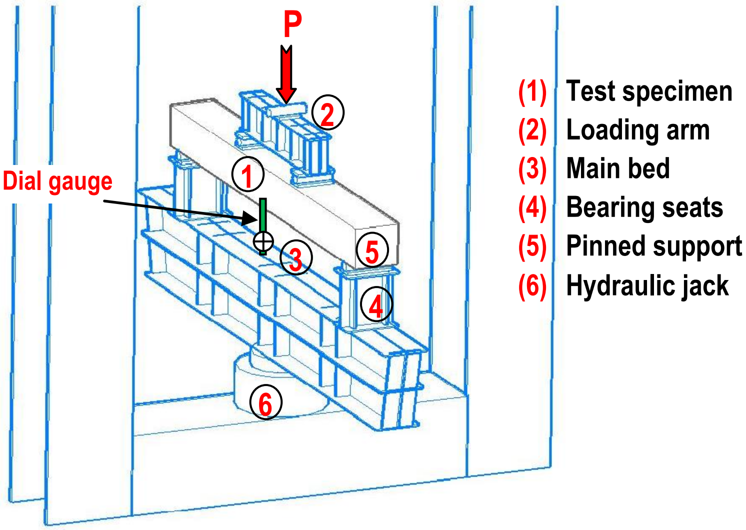


Figure (2): Schematic diagram showing the testing rig of flexure test.

- Volume Change of Concrete
- Test Specimens

For the volume changes tests, prisms, specimens of (100×100×500mm) were used in this work, multi position gauge was used for measurement. An extensometer while more type, with an accuracy of (0.002mm /division) was used to measure strain in the panel prisms.

- Test Procedure

Shrinkage tests were performed according to ASTM C157 : 93 setting gauge plugs : the gauge length was selected to be 200mm, the stainless steel in the Figure (3). Specimens were cured in water at age of 28 days. The specimens were exposed to drying in laboratory for another 28 days (i.e. 28 days curing in water and 28 days air dried in laboratory).

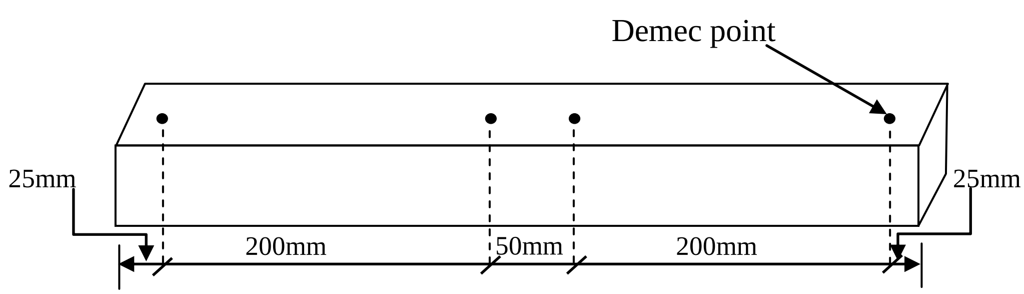


Figure (3): Location of Stainless Steel Demec Points.

- Drying Shrinkage

The drying shrinkage was monitored for the concrete after the 28 days in water and the measurements were done from 28 days to 60 days age at ages of 29, 32, 37, 45 and 60 days.

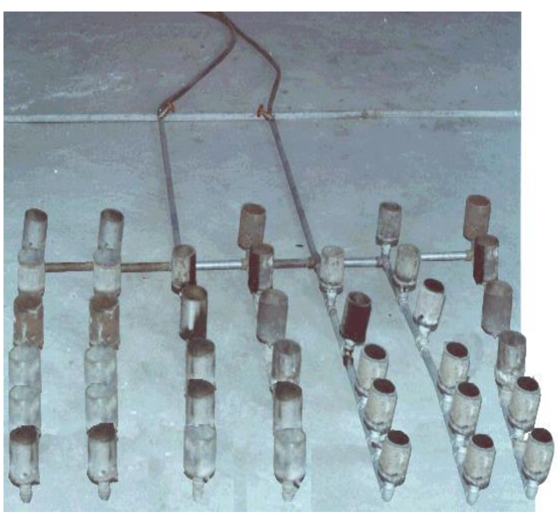
- Shrinkage After Burning

Shrinkage of specimens were monitored after exposure to fire flame and cooling to room temperature. Shrinkage was measured after the specimens were cooled to room temperature, at ages of (1, 3, 7, 15 and 30 days) after burning.

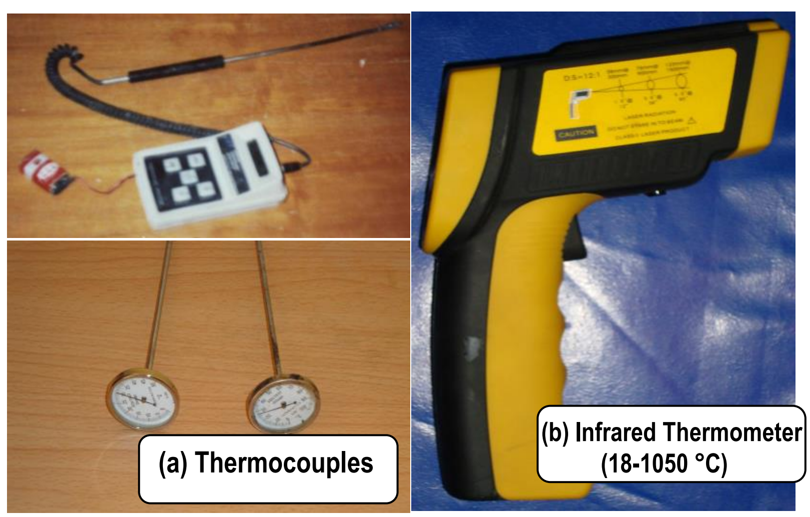
- Burning and Cooling

The concrete specimens and the reinforced concrete beams were burnt with direct fire flame from a net work of methane burners inside the frame. The dimensions of this burner net are (1500×1500mm) (length × width) respectively as shown in Plate (2). The bars of flame were intended to simulate the heating condition in a actual fire. When the target was reached, the temperature continuously measured by digital thermometers, one of them was positioned in the bottom surface of the beams in the contact with the flame, while the other was positioned at the unexposed upper surface of the beam, and by thermocouple that was inserted in the near center of each beam to measure the temperature at the mid-depth (75 mm from the exposed or unexposed surface). In addition, the temperature of concrete and steel reinforcement was measured at different depths by applying Infrared ray thermometer away about 2 meters from the concrete exposed to fire.

The measurement devices are shown in Plate (3). After burning the concrete specimens and the reinforced concrete beams were quenched immediately in water for 2.0 hours and then stored in laboratory environment about 20 hours also before testing.



Plate(2): The work of net methane burners.



Plate(3): Temperature measurements devices.

- Moulds preparation

All the beam test specimens used in this investigation were cast in steel moulds to give a beam specimen had cross-section of (150×150 mm) and a total length of 1000mm. Each one of these steel moulds consisted of a steel plate of (1000×150×5 mm) enclosed by four steel angles section of (40×40×5 mm). Every angle was fixed with the steel plate by three bolts as shown in Figure (4). These moulds were cleaned and their internal surfaces were greased to prevent adhesion with concrete after hardening.

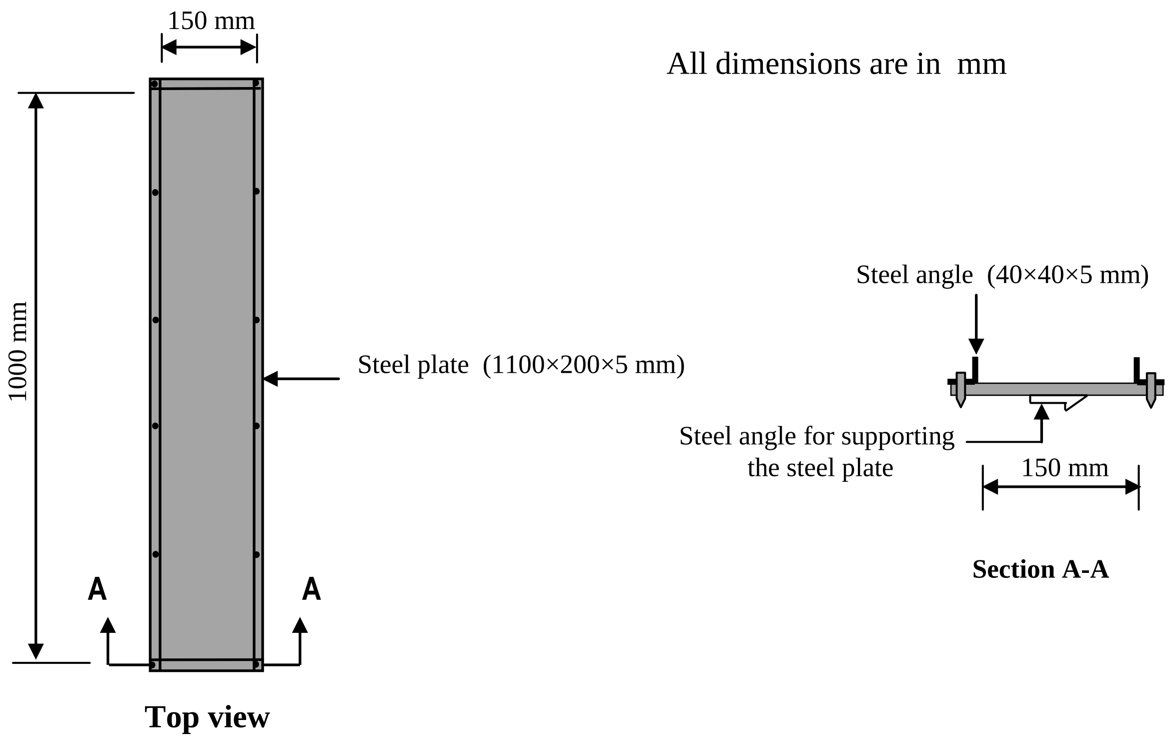


Figure (4): Specification and details of the mould of column specimens.

Results and Discussion

- Effect of Burning by Fire Flame on the Compressive Strength of Concrete

Table (5) show the effect of the exposure to fire flame on compressive strength, while Figures (5 and 6) show the relation between compressive strengths and fire flame temperature. It is clear from these figures that the residual compressive strength after exposure to fire flame the reduction at 30 days age was more than the reduction at 60 and 90 days. It can be seen from these table and figures that the compressive strength behaved as the following:

At 400°C, the residual compressive strength compared to the original strength before exposure to fire flame were (60-71%). These results are similar to that obtained by Al-Ausi and Faiydh (1985), Umran (2002), Chih-hung (2005) and Kadhum (2010) was (60-71%), (67-82%), (59-73%) and (70-77 %) respectively.

At 600°C, the residual compressive strength compared to the original strength before exposure to fire flame was ranged from (51-57%) these results confirmed that of Habeeb (2000), Karim (2005), Saqier (2008) and Kadhum (2010).

At 800°C, it was found that the residual compressive strength after exposure to fire flame ranged from (37-47%).

Table (5): Test values of compressive strength of concrete specimens before and after exposure to fire flame.

Age at Exposure (days)	Period of Exposure (hours)	Compressive Strength (MPa) Temperature (°C)				Rtios F_{ca} / F_{cb}		
		25(1)	400(2)	600(3)	800(4)	(2/1)	(3/1)	(4/1)
28	1.0	38.50	27.34	20.80	16.94	0.71	0.54	0.44
	2.0		23.10	19.64	14.25	0.60	0.51	0.37
60	1.0	41.45	29.02	23.21	19.48	0.70	0.55	0.47
	2.0		25.28	22.38	17.82	0.61	0.54	0.43
90	1.0	44.20	29.62	23.87	20.77	0.67	0.57	0.47
	2.0		27.40	24.31	18.56	0.62	0.55	0.42

F_{ca}= compressive strength (cube) after exposure to fire flame.
F_{cb}= compressive strength (cube) before exposure to fire flame.

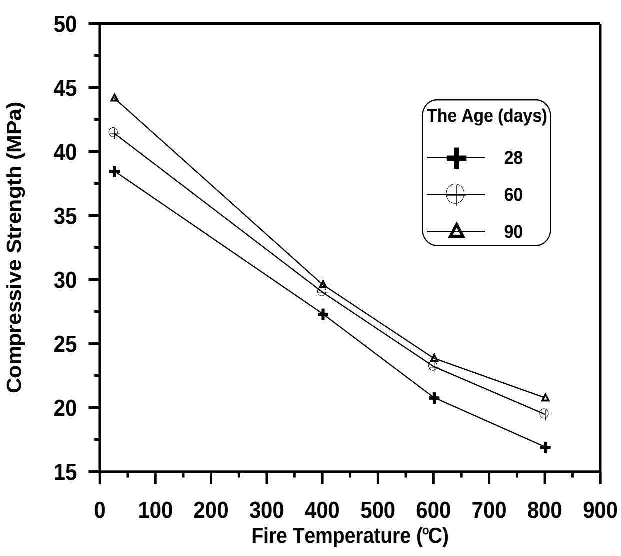


Figure (5): The effect of fire flame on the compressive strength at 1.0 hour period of exposure.

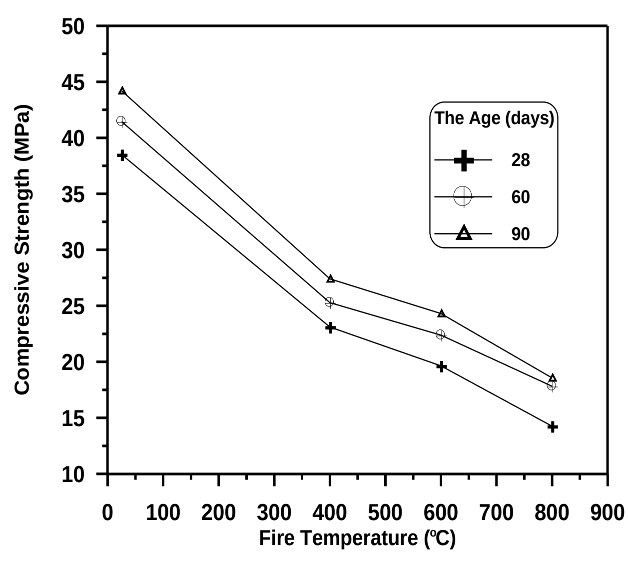


Figure (6): The effect of fire flame on the compressive strength at 2.0 hour period of exposure.

- Effect of Burning by Fire Flame on the Modulus of Elasticity of
- Concrete

Test results of the modulus of elasticity are summarized in Table (6). Figures (7 and 8) illustrates the relationship between the residual modulus of elasticity and fire flame temperatures. From these results, it can be seen that the reduction values of concrete modulus of elasticity were more significant than that of the compressive at identical fire flame temperatures.

At 400°C, there was a significant reduction in the concrete modulus of elasticity due to effect of fire flame. The residual value of modulus of elasticity was ranged from (52-69%).

At 600°C, the f residual value of modulus of elasticity was (27-30%). These results conformed that of Umran (2002), Karim (2005) and Kadhum (2010).

At 800°C, the residual value of modulus of elasticity was (18-21%). The reduction in modulus of elasticity of concrete can be attributed to the increase in the amount of cracks formation due to exposure to fire and the physico-chemical transformation in concrete constituenents during burning will yield strength loss.

Table (6): Test values of modulus of elasticity of concrete before and after exposure to fire flame.

Age at exposure (days)	Period of exposure (hours)	Modulus of Elasticity of Concrete (GPa)				Rtios Mca / Mcb		
		Temperature (°C)						
		25(1)	400(2)	600(3)	800(4)	(2/1)	(3/1)	(4/1)
28	1.0	32.5	21.5	9.8	6.8	0.66	0.30	0.21
	2.0		16.9	9.4	6.2	0.52	0.29	0.19
60	1.0	36.6	24.9	11.0	7.3	0.68	0.30	0.20
	2.0		23.1	9.9	6.6	0.63	0.27	0.18
90	1.0	38.4	28.0	11.9	8.0	0.69	0.30	0.20
	2.0		26.5	10.4	6.9	0.64	0.27	0.18

M_{ca}= modulus of elasticity of concrete after exposure to fire flame.
M_{cb}= modulus of elasticity of concrete before exposure to fire flame.

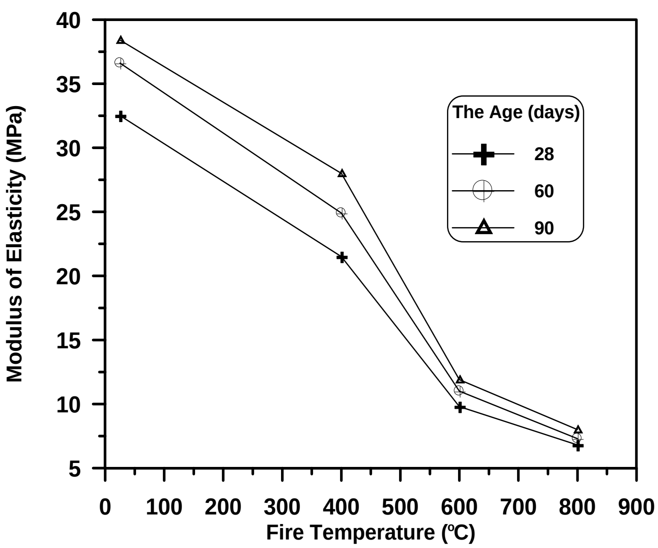


Figure (7): The effect of fire flame on the modulus of elasticity of concrete at 1.0 hour period of exposure.

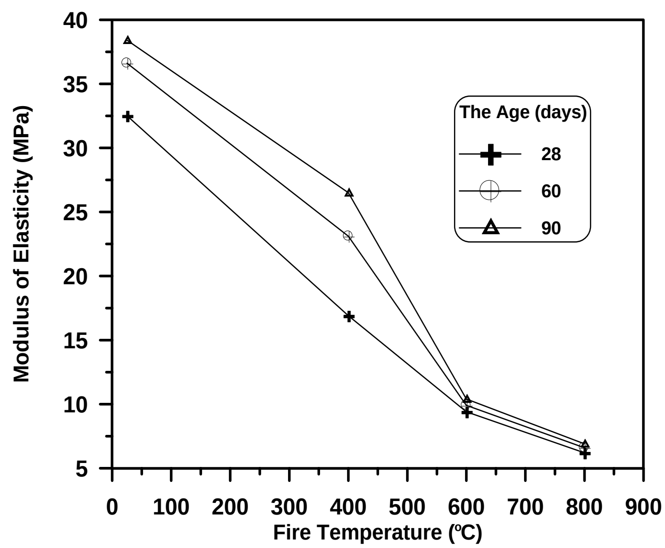


Figure (8): The effect of fire flame on the modulus of elasticity of concrete at 2.0 hour period of exposure.

- Shrinkage Before and After Burning

The values of shrinkage before and after exposure to fire flame are shown in Table (7) and plotted in Figures (9 and 10) against age. It can be seen from these figures that the shrinkage increase with temperature.

There is no significant increase in shrinkage values due to the increase of exposure time from 1.0 to 2.0 hours, shrinkage values were not more than 13% of that at 2.0 hours exposure.

Table (7): Test values of shrinkage before and after exposure to fire flam (prisms100×100×500mm).

Temperature (°C)	Period exposure(hour)	Age (days)	Strain in (millionths)
25 before burning	——	0	0
		1	155
		3	285
		7	325
		15	400
		30	490
		45	565
		60	590
400 after burning	1.0	61	165
		63	280
		67	330
		75	350
		90	350
	2.0	61	165
		63	285
		67	340
		75	365
		90	365
600 after burning	1.0	61	175
		63	295
		67	380
		75	425
		90	425
	2.0	61	185
		63	295
		67	410
		75	440
		90	440
800 after burning	1.0	61	180
		63	310
		67	415
		75	445
		90	445
	2.0	61	185
		63	315
		67	420
		75	450
		90	450

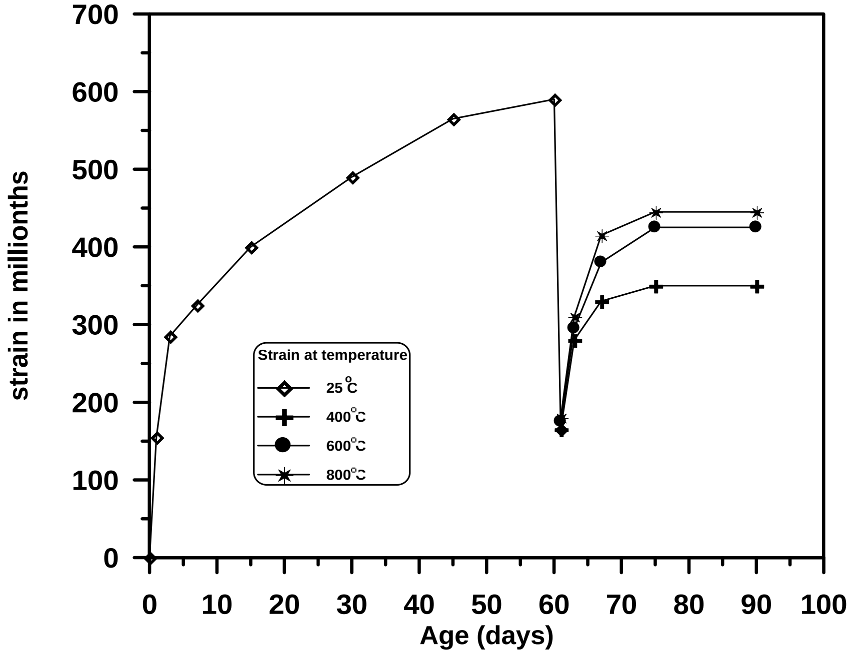


Figure (9): Relation between strain and age of concrete before burning and at 1.0 hour period of exposure.

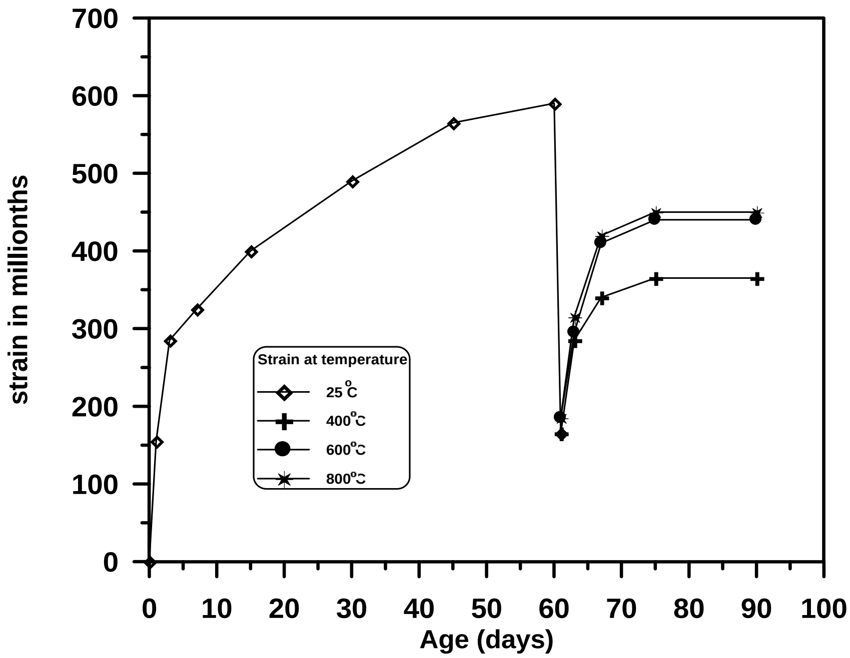


Figure (10): Relation between strain and age of concrete before burning and at 2.0 hour period of exposure.

- Effect of Burning on Load Versus Deflection Results

The mid-span deflection of the beam specimens which were loaded and exposed to fire flame at the same time was measured during this process. Each beam specimen was loaded to 35% of the ultimate load before burning for a period of 30 minute; then exposed to fire flame temperatures of (400°C, 600°C and 800°C) thereon, the residual ultimate load was applied until failure. The deflection were recorded at each stage of loading at mid span of beam, the load at the first visible crack and at failure were recorded. Deflection of these beam specimens, which occurred immediately when they were loaded and subjected to fire flame, this deflection is called immediate deflection or instantaneous deflection. Deflection measurement was taken continually during the test and the rate of increase in deflection was controlled to provide warning of impending collapse of the beam specimens.

The best results were summarized in Table (8) and the relation between the load and deflection were illustrated in Figures (11 and 12). After the beam was subjected to fire flam, two types of cracks developed, the first was thermal cracks appeared in a honeycomb fashion all over the surface. They originated from top or bottoms edges and terminated near the mid-depth of the beam. The crack width was (1.65mm). The second crack were flexural tensile cracks due to loading developed in the mid-span region.

From these Figures, it can be noted that the increase in the fire temperature decreases the load carrying capacity and increases deflection in beam specimens. This can be attributed to the fact that heating causes a reduction in beam stiffness, which is essentially due to the reduction in the modulus of elasticity of concrete and the reduction in the effective section due to cracking. In addition, these Figures reveal that the load-deflection relation of the beam specimens is almost linearly proportional for the two period of exposure (1.0 and 2.0 hours) and for temperature exposure (600°C and 800°C).

At temperature of 25°C and 400°C, the general load versus mid-span deflection behavior of beam specimens was similar. While, for burning temperature 600°C and 800°C, the load versus mid-span deflection relations of beam specimens were indicating softer behavior in general compared with those of the control beam specimens and lower burning temperature. This can be attributed to the weaker bond strength between the concrete and steel reinforcement.

Table (8): Test results of the first crack load, ultimate load and maximum deflection for reference beams and beams exposed to fire flame.

Temperature °C	Specimen Identification	First Crack Load (kN)	Ultimate Load (kN)	Percentage Residual Ultimate Load %	Max Deflection at Mid-span (mm)
25	BR-25°C	9.8	32.4	100	3.45
400	BA-400°C-1.0hr	Precracking	28.5	87.9	5.42
	BA-400°C-2.0hr	Precracking	26.2	80.8	6.72
600	BB-600°C-1.0hr	Precracking	21.4	66.0	7.20
	BB-600°C-2.0hr	Precracking	17.1	52.8	9.20
800	BC-800°C-1.0hr	Precracking	14.6	45.0	7.48
	BC-800°C-2.0hr	Precracking	11.5	35.5	10.25

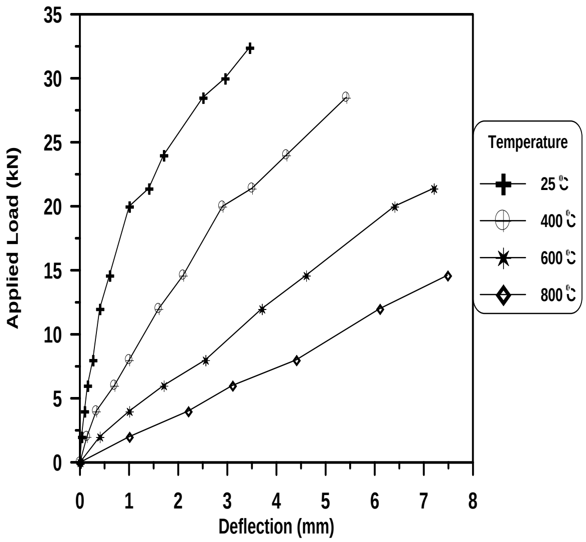


Figure (11): Load versus deflection curve of beam specimens before and after exposure to fire flame at 1.0 hour.

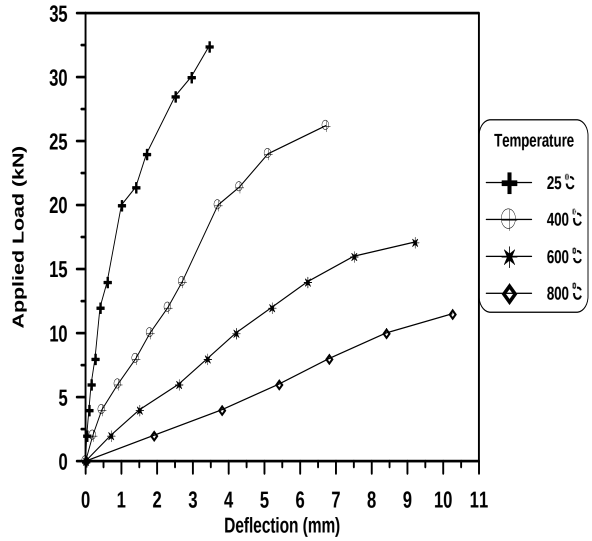


Figure (12): Load versus deflection curve of beam specimens before and after exposure to fire flame at 2.0 hour.

- Effect of Burning on the Steel Reinforcement Bars

The effect of burning on the properties of steel reinforcement bars is summarized in Table (9). At temperature of (400°C), both burning and subsequent cooling did not affect the mechanical properties of the steel reinforcement bars, but this effect was observed at burning temperature of 600°C and 800°C.

The percentages of residual yield tensile stress and ultimate tensile stress were (90.6%, 78.8% and 89.8% ,81.4%) at temperature (600 and 800°C) respectively. The modulus of elasticity was not affected by burning and cooling at all levels of temperature. Similar behavior was also recorded by other investigators Harmathy and Stanzak (1980) and Umran (2002).

Table (9): Effect of burning on the properties of steel bars.

Exposure Temperature (°C)	Yield Tensile Stress N/mm^2	Residual Yield Tensile Stress %	Ultimate Tensile Stress N/mm^2	Residual Ultimate Tensile Stress %	Modulus of Elasticity Es (Gpa)	Residual Es %
25	345	100	480	100	205	100
400	345	100	480	100	205	100
600	315	90.6	438	89.8	205	100
800	260	78.8	414	81.4	205	100

- Effect of Burning on the Flexural Capacity of Reinforced Concrete Beams

The reinforced concrete beam specimens are designed to fail in flexure (yielding of steel reinforcement) for beam specimens. From the test results shown in Table (10), it is clear that the values of bending moment capacity decrease when the beams were exposed to burning temperature 400ºC, 600ºC and 800ºC.

At burning temperature (400°C), the residual bending moment capacity was (88 and 81%) for 1.0 and 2.0 hour exposure periods respectively.

At burning temperature (600°C), the residual bending moment capacity was (66 and 53%) for 1.0 and 2.0 hour exposure periods respectively.

At burning temperature (800°C), the residual bending moment capacity was (45 and 36%) for 1.0 and 2.0 hour exposure periods respectively.

- General Behavior and Verification of ACI Building Code Provisions

During testing of beam specimens to failure, it was noticed that all beam specimens are failed with the typical flexural failure mode before and after burning (yielding of steel followed by crushing of concrete). These mode failures are coincide with that obtained in the experimental results.

The test results were utilized to verify the recommendations and design simplifications of the ACI Building Code Pertaining to predict bending moment capacity.

These results were summarized in Table (10) and the relationship between fire temperature and residual bending moment capacity are illustrated in Figure (13). The results are compared with design provisions of ACI 318/2008 Code. To utilize the ACI Building Code after exposure to fire flame temperatures the measured and ACI-predicted values **($M_{u\ test}/M_{u\ calculated}$)** were calculated for the beam specimens. The relationship between fire temperature and moment capacity are illustrated in Figures (14 and 15).

At burning temperature (400°C), the ACI Building code gave conservative results to predict bending moment capacity. The ratio between the measured and ACI-predicted values were in the range from (1.06-1.10).

At burning temperature (600°C), the ACI Building code gave overestimated results to predict results to predict bending moment capacity. The ratio between the measured and ACI-predicted values were in the range from (0.86-0.96).

At burning temperature (800°C), the ACI Building code become unable to predict bending moment capacity. The ratio between the measured and ACI-Predicted values were in the range from (0.64-0.76).

From the results, it is clear that the predicted ultimate bending moment capacity obtained from ACI code provisions is lower than that obtained in the experimental work at burning temperature up to (400°C). While, at burning above (400°C)the predicted ultimate axial load capacity obtained from ACI Code provisions is greater than that obtained from the experimental work. This can be attributed to the precracking which happens upon burning and increase in intensity when the fire temperature increases.

Table (10): Comparison of the flexural test results with that obtained from ACI 318-08.

Code for flexure beam specimens.

Specimen Identification	Compressive Strength of cube (MPa)	Yield Tensile Stress (MPa)	Ultimate Load (kN)	Mu (test) (kN.m)	Percentage Residual Moment Capacity (%)	Mu(ACI) (kN.m)	Mu (test) Mu (ACI)
BR-25°C	44.65	345	32.4	4.86	100	4.23	1.15
BA-400°C-1.0hr	31.76	345	28.5	4.28	88	3.87	1.10
BA-400°C-2.0hr	28.20	345	26.2	3.93	81	3.72	1.06
BB-600°C-1.0hr	27.60	315	21.4	3.21	66	3.34	0.96
BB-600°C-2.0hr	25.0	315	17.1	2.57	53	2.97	0.86
BC-800°C-1.0hr	21.30	260	14.6	2.19	45	2.89	0.76
BC-800°C-2.0hr	19.80	260	11.5	1.73	36	2.68	0.64

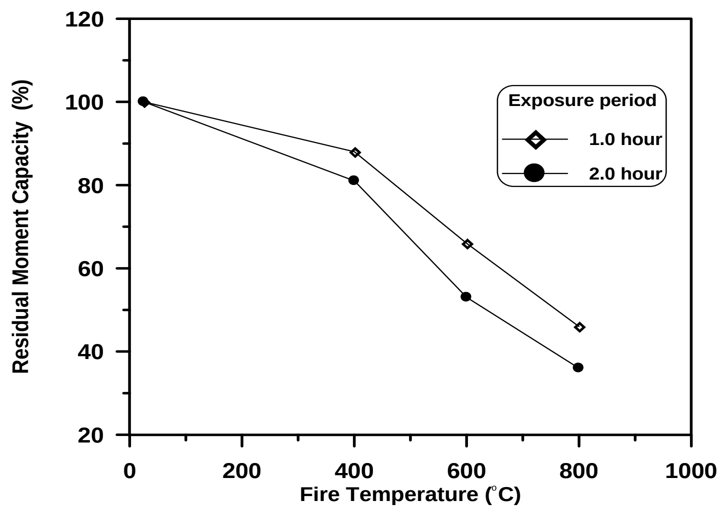


Figure (13): Effect of fire temperature on the residual moment capacity of beam specimens for different exposure period.

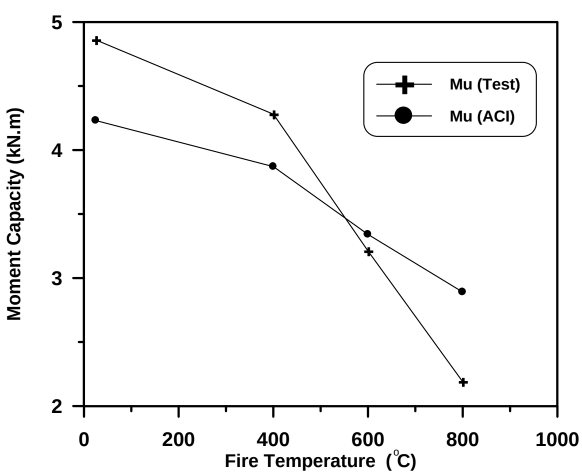


Figure (14): Effect of fire temperature on the moment capacity of beam specimens for 1.0 hour period of exposure.

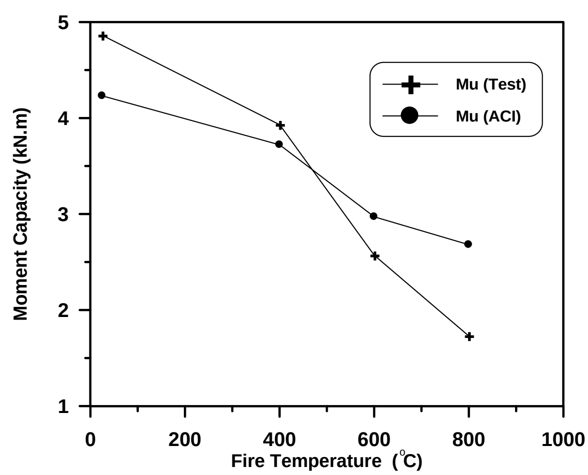


Figure (15): Effect of fire temperature on the moment capacity of beam specimens for 2.0 hour period of exposure.

- Surface Condition and Fire Endurance of Tested Beam

The aim of design for fire safety should be to limit damage due to fire. The fire endurance of a beam is defined as the time to reach failure under exposure to standard fire (ASTM E119-01). At the beginning the beams are at room temperature, measured to be 25ºC.

The temperature of concrete was measured at different depths. This was achieved by Infrared rays thermometer handled at approximately 2 meters from the concrete exposed to fire. Typical temperature-time values, measured during the test, in the exposure surface, steel reinforcement and at various depths in concrete are plotted in Figure (16) for the beam specimen. The experimental results clearly indicate that the temperature near the surface to fire is higher and decreases towards the center of the beam.

Based on the results of this work, it was noticed that the test results agreed with (ASTM E119-01). While, these beam specimens were subjected to fire flame temperatures of (400, 600 and 800 ^{o}C) for (1.0 and 2.0 hour), the fire endurance of all the beam specimens investigated was reached when the inability to carry the applied design flexural load, then these beams were considered failed according to (ASTM E119-01).

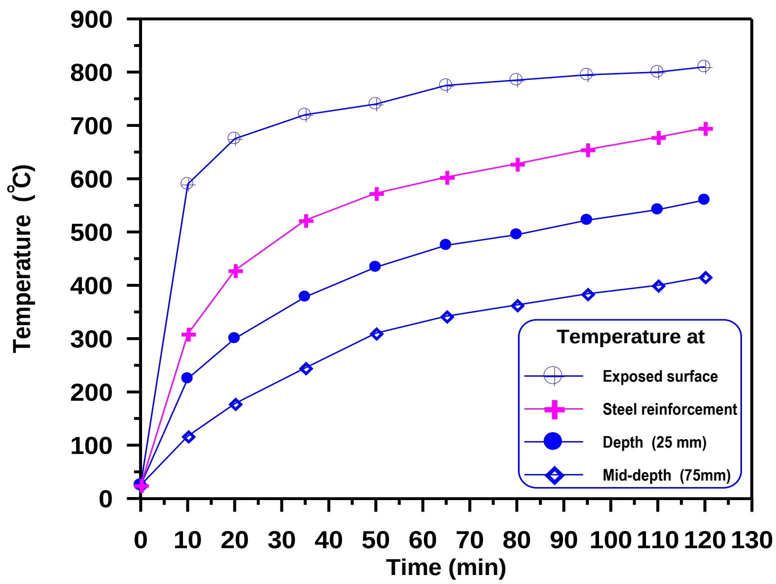


Figure (16): Beam temperature as a function of time at various depths at exposure temperature of 800°C and for 2.0 hour period of exposure.

- Effect of Exposure Fire Flame on The Colour of Concrete

Since the thermal conductivity reduces with the increase in temperature water quenching of the heated concrete samples has not produced uniform cooling. At the temperature of 400°C, there was no apparent visual discoloration occurred in the concrete. The concrete specimens subjected to 600°C and above suffered noticeable color change. The inner section of the concrete bluish in colour. The dark colored area boundary was very distinct and showed no transition zone. Outside this, concrete had maintained its color however, when the fire flame temperature was increased to 800°C the inner color was changed to bluish dark grey.

- Conclusions

Based the results obtained from testing in this work, the following conclusions can be with drown :

1. The residual compressive strength ranged between (60-71%) at 400°C, (51-57%) at 600°C and (37-47 %) at 800°C.
2. Large proportion of drop in compressive strength occurs at the first 1.0 hour period of exposure.
3. Based on the results obtained, it was found that the shrinkage values increase with temperature increase.
4. The temperature distribution through the thickness of beam that was found in this investigation is similar for all the beams which have the same thickness and exposed period to fire flame.
5. After the beams were subjected to fire flame, two types of cracks developed. The first was thermal cracks, which appeared in honey comb fashion all over the surface. The second cracks originated at mid-span region due to bending from the applied load and called flexural cracks.
6. After exposure to fire flame, the cracks in flexural failure speared along the beam until outside the pure bending moment region. Unlike the flexural cracks which formed gradually, the diagonal cracks formed suddenly.
7. It was noticed that the load deflection relations to specimens exposed to fire flame are flat, representing softer load-deflection behavior that of the control beams. This can be attributed to the early cracks and lower modulus of elasticity.
8. At temperature of (400°C), both burning and subsequent cooling did not affect the mechanical properties of steel reinforcement; the effect was observed at 600 and 800 °C. The residual yield tensile stress and residual ultimate stress was (90.6%, 78.8% and 89.8%, 81.4%) respectively.
9. Modulus of elasticity of concrete is the most affected by fire flame temperature rather than compressive strength.
10. Based on the available experimental results in this research work concerning the bending moment capacity for reinforced concrete beam specimens it can be concluded that:

- After exposure to fire temperature (400°**C**), the ACI 318/ 2008-Code can be safely used to predict the residual bending moment capacity from the residual compressive strength after exposure to fire flame.
- After exposure to fire temperature (600ºC), the ACI-Code gave over estimated values to predict bending moment capacity.
- After exposure to fire temperature (800ºC), the ACI-Code become unable to predict bending moment capacity.

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