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Effect of Fire Flam on Properties of Plain and Reinforced Concrete Beams

تأثير لهب النار على خصائص العتبات الخرسانية المسلحة وغير المسلحة

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Abstract

In the present study, compressive strength, rate of free drying shrinkage of plane concrete and load-deflection behavior of reinforced concrete beam specimens under the effect of fire flame exposure are presented. Plain concrete (150mm) cube specimens and (100*100*500mm) prisms were subjected to fire flame temperatures ranging between (25-550 ^{o}C) at different ages of 30 and 60 days. Two temperature levels of 400 ^{o}C and 550 ^{o}C were chosen with two different exposure durations of 0.5 and 1.0 hour. Cube compressive strength and rate of drying shrinkage were explored after fire flame exposure and compared with the control (unburned) specimens.

Ten rectangular reinforced concrete beam specimens (100*150*1000mm) were cast and subjected to fire flame at temperature levels of 400 ^{o}C and 550 ^{o}C, with two periods of exposure 0.5 and 1.0 hour. These beam specimens were tested in flexure until failure after exposure to fire flame and the load – deflection relationship was recorded and compared with that of the control (unburned) beam specimens.

Based on the results of this research, the compressive strength of concrete was affected adversely by fire flame and the degree of damage increases when the fire temperature and/or period of exposure were raised. For the fire temperatures and periods of exposure investigated, the residual compressive strength ranged between (70-78%) at 400 ^{o}C and (59-65%) at 550 ^{o}C burning temperature.

It was found that the exposure to fire flames increases the rate and intensity of drying shrinkage of free concrete prisms. This increase was found to reach 50% if temperature of fire approaches 550 ^{o}C.

It was noticed that the load-deflection relationship of reinforced concrete beams exposed to fire flame is more leveled representing softer load-deflection behavior than that of the control beams. Also, it was found that both the ultimate resisted load and moment carrying capacity decrease remarkably after fire exposure. This can be attributed to the early cracking and lower modulus of elasticity. Also, it was found that the temperature distribution through the thickness of beam is identical for all the beams which have the same thickness.

الخلاصة

في هذه الدراسة تم عرض تأثير التعرض إلى لهب النار على مقاومة الانضغاط وتطور انفعالات انكماش الجفاف للخرسانة وعلاقة الحمل مع الانحراف للعتبات الخرسانية المسلحة. تم تعريض المكعبات (150ملم) والمواشير (100*100*500ملم) الخرسانية الغير مسلحة إلى لهب النار وبدرجات حرارة تتراوح مابين 25-550 درجة سيليزية وبأعمار 30 و 60 يوما. تم اختيار مستويين من التعرض لدرجة الحرارة هما 400 و 550 درجة سيليزية ولفترتي تعرض نصف ساعة وساعة واحدة لهذه المكعبات والمواشير الخرسانية. ثم جرى فحص مقاومة الانضغاط وكذلك قياس معدل انكماش الجفاف ومقارنة النتائج مع النماذج القياسية التي لم يتم حرقها.

جرى كذلك صب عشرة عتبات خرسانية مسلحة ذات مقطع مستطيل وبأبعاد(100*150*1000مم) (العرض*الارتفاع*الطول) وجرى تعريضها إلى لهب النار بمستويين من درجات الحرارة هما 400 و550 درجة سيليزية ولفترتي تعرض هما نصف ساعة وساعة واحدة. وتم فحصها بالانثناء لحين الفشل وتسجيل علاقة الحمل مع الانحراف ومقارنة النتائج مع النماذج القياسية التي لم يتم حرقها.

اعتمادا على النتائج المستحصلة من هذا البحث وجد إن مقاومة انضغاط الخرسانة تتأثر سلبيا بلهب النار وخاصة عندما تزداد درجة الحرارة وفترة التعرض إلى هذه النار. فقد تم ملاحظة انه للأعمار وفترات التعرض التي جرى دراستها فان مقدار نسبة مقاومة الانضغاط المتبقية بعد الحرق مقارنة بمقاومة الانضغاط قبل الحرق تتراوح مابين (70-78%) في

درجة حرارة حرق 400 سيليزية و(59-65%) في درجة حرارة حرق 550 سيليزية.
كذلك ظهر إن ارتفاع درجة حرارة الحرق يؤدي إلى زيادة سرعة انفعالات انكماش الجفاف للمواشير الخرسانية الغير مقيدة. ووجد إن هذه الزيادة قد تصل إلى 50% أذا وصلت درجة حرارة الحرق إلى 550 درجة سيليزية.
وجد كذلك إن علاقة الحمل مع الانحراف للعتبات الخرسانية المسلحة المعرضة إلى الحرق بلهب النار تكون أكثر استواء مظهرة سلوكا اقل جساءة بالنسبة لهذه العلاقة مقارنة مع النماذج المرجعية (التي لم يتم حرقها). كذلك ظهر ان كل من الحمل الاقصى وقدرة العتب لتحمل العزوم تنخفض بشكل واضح. يمكن إرجاع هذا السلوك إلى التشقق المبكر وانخفاض مقدار معامل المرونة بسبب الحرق بالنار.
كذلك تم تسجيل إن توزيع درجات الحرارة على طول سمك العتبات يتبع توزيعا متشابها في جميع هذه العتبات.

1- Introduction

A deal of information about the properties of concrete and steel after exposure to high temperatures are available. However, information about the effect of direct exposure to fire flames on properties of concrete is limited.
Cracking of concrete is perhaps its major disadvantage which results mainly from its low tensile strength and low tensile strain capacity hence concrete is considered as brittle material and lacks ductility. In general, concrete cracks when there are tensile stresses exceeding in magnitude its tensile strength.

In the structural design of buildings, in addition to the self weight and imposed loads, it is in many instances necessary to design the structure to safely resist exposure to fire. However it is usually necessary to guard against structural collapse due to fire for a given period of time by **Shetty**(1)

2- Research Significance

Elaborate researches on concrete subjected to high temperatures were carried out by **Al-Ausi and Faiyadh**(2)**, Elizzi et al**(3), **Asa'ad**(4) and **Habeeb**(5). The objective of these investigations was to determine the strength and deformation properties of concrete at elevated temperatures and to find out the causes of the changes that the material suffers in consequence of heat. The researchers exposed the concrete and mortar specimens to high temperatures in special furnaces. There are indeed little research work about temperature gradient and exposure time of concrete in direct contact with fire flames. The present work is an attempt to investigate the effect of exposure of concrete to fire flame on shrinkage cracking, load-deflection behavior of reinforced concrete beams and other mechanical properties of concrete.
The main goals of the present study are:
1-Studying the fire effect on cracking tendency and pattern in reinforced concrete beams.
2- Investigating the fire endurance of reinforced concrete beams.
3- Studying the fire flame effect on the immediate deflection of the reinforced concrete beams and comparing the results with control (unburned) beams.
4- Studying the fire effect on essential properties of concrete, such as compressive strength, and drying shrinkage.

3- Literature Review

3-1 Fire Effect on the Mechanical Properties of Concrete

Elizzi et al(3) investigated the influence of different temperatures on the compressive strength and density of concrete. They used (100*100*100mm) cubes subjected for a short duration (one hour) at a special oven to temperatures ranging from 20-600 °**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 small reduction in density up to 300 °**C** which 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 combined water in concrete.

Habeeb(5) investigated the effect of high temperatures on the mechanical properties of high strength concrete (HSC). The specimens were subjected to elevated temperatures ranging between (100-800°C). Five temperature levels of (100,300,500,600 and 800 °C) were chosen with three different exposure durations of 1, 2 and 4 hours without any imposed loads during heating. Residual compressive strength ranged between (90-100%) at 100 °C, (72- 90) at 300 °C, (55-87%) at 500 °C and (22-66%) between 600-800 °C. He also noticed that exposure time more than one hour has a significant effect on residual compressive strength of concrete.

3-2 Shrinkage of Concrete Before and After Burning

Habeeb(5) found from the test results, that the additional shrinkage values due to heating are between (400-800) microstrains, and there is no significant increase in shrinkage values due to the increase of exposure time from 1 hour to 4 hours. Shrinkage values were not more than 10% of that at 1 hour exposure.

Neville(6) reported that at a given workability, which approximately means a specified concrete water content, shrinkage is unaffected by increase of the cement content, or may even decrease, because the water/cement ratio is reduced and the concrete is therefore, better able to resist shrinkage.

Kadhum(7) studied the cracking behavior due to the restrained drying shrinkage of the reinforced concrete slabs before and after exposure to fire. These slabs were externally restrained at (two end, three end and four end restrained). He indicated that the cracks are developed in these slabs during (1.5 hr) period of exposure to fire temperature of (600 °C), and these cracks remain open even after cooling, i.e. the contraction of concrete and steel are insufficient to close the cracks.

Kubba(8) found that the magnitude of drying shrinkage strains of self- compacting concrete (SCC) is higher than that of normal concrete and it depends on the type of filler used.

Al-Abdaly(9) found that drying shrinkage strains are remarkably less in high strength concrete than that in normal strength concrete. She also found that restrained high strength members reveal significant retardation in the commencement of drying shrinkage cracking.

3-3 Fire Effect on Reinforced Concrete Members.

The behavior of reinforced concrete structures exposed to fire depends on the thermal properties, strength and stiffness properties of concrete and steel at elevated temperatures, and on the ability of the structure to redistribute internal forces during the course of the fire studied by **Purkiss**(10)

Asa'ad(4) studied the behavior of reinforced concrete structure subjected to elevated temperatures. Four types of reinforced concrete samples were a dopted. They were composed of singly and doubly reinforced concrete beams having the dimensions of (100*100*1100mm), continuous doubly and singly reinforced beams (100*150*1300mm) and the frame structure is with outer dimensions of (900*750mm). The frame has 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 curing ages of 30 or 90 days and tested in flexure after cooling. The researcher found that both flexure 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 to an increase in magnitude of moment redistribution in continuous beams.

3-4 Temperatures Associated With Fires

The values of temperature associated with fires can be estimated according to the ASTM-E119 standard curve which can be described approximately by the following expression (11).

If $t < 7200$ sec.

$T_f = T_0 + 1/1.8\ [\ 1044 + \tanh\ (0.00023413\ t\ ) - 498.2\ \tanh\ (0.00027044\ t) + 1286\ \tanh\ (0.002475\ t)\ ]$……………………………………………...(1)

If $t \geq 7200$ sec.

$T_f = 927 + 0.011574\ t$ …………………………………... (2)

Where:

t= time in seconds.

T_f= the fire temperature at time t (°**C),** and

T_o= initial temperature (°**C).**

4- Experimental Work

4-1 Introduction

This section describes the materials used in the production of the specimens, mix proportion and the methods of testing.

The specimens were cast, moist cured for 28 days, then dried in the laboratory till the age of testing. They were tested at curing ages of 30 and 60 days before and after exposure to fire flame at temperatures 400 °**C** and 550 °**C** and for two exposure periods 0.5 and 1.0 hour.

4-2 Materials

4-2-1 Cement

The cement used in this study was Ordinary Portland Cement (O.P.C) produced at Kufa Factory. This cement complied with the Iraqi specification No. 5: 1984[(12)]

4-2-2 Fine Aggregate

Al-Akhaider well-graded natural sand was used as fine aggregate. The values of physical and chemical properties of the sand are listed in Table [1]. Its grading conformed to the Iraqi specification No.45: 1984[(13)] Zone (3).

4-2-3 Coarse Aggregate

The gravel used was brought from AL-Nibaee area with a maximum size of 20 mm. the gravel used conforms to the Iraqi specification No.45: 1984[(13)]. The grading and other properties of this coarse aggregate are listed in Table (2).

Table [1]: Properties of the fine aggregate.

Sieve size (mm)	Percent passing%	IQS 45:1984 Limits Zone (3).
9.5	100	100
4.75	96	90-100
2.36	93	85-100
1.18	80	75-100
0.6	54	60-79
0.3	26	12-40
0.15	0	0-10
Properties	Test results	IQS 45:1984 Limits
Sulphate content, SO_3	0.28%	≤ 0.5%
Specific gravity	2.6	-
Absorption (%)	1.7	-

Table [2]: Properties of the coarse aggregate.

Sieve size (mm)	Percent passing%	IQS 45:1984 Limits
37.5	100	100
20	98	95-100
9.5	52	30-60
4.75	2	0-10
Properties	Test results	IQS.45:1984 Limits
Sulphate content, SO_3	0.08	$\leq 0.1\%$
Specific gravity	2.64	-
Absorption (%)	0.7	-

4-2-4 Water

Tap water was used throughout this work for both mixing and curing of concrete.

4-2-5 Reinforcement

Deformed steel bars of 10 mm diameter were used for longitudinal reinforcement and plain wires of diameter 6 mm were used for stirrups.

5- Mix Design and Proportions

The concrete mix was designed according to **BS 5328: Part 2: 1991**(14) to obtain a concrete mix with a (28days) target compressive strength of 40MPa and initial slump of about (60mm).
The concrete proportions of the concrete mix are summarized in Table [3].

Table [3]: Mix proportions

		Weight proportion	**Mix proportion kg/m^3**			
Slump mm	W/C Ratio	Cement:Sand:Gravel (by weight)	Water	Cement	Sand	Gravel
60	0.45	1.0: 1.2: 2.8	195	435	525	1215

6- Testing Fresh and Hardened Concrete

6-1 Slump Test

The workability of the fresh concrete mixes was measured by the slump test before casting these mixes in molds. This test was conducted according to **ASTM C143-1989**(15).

6-2 Compressive Strength Test

Compressive strength test was carried out according to **B.S. 1881: Part 116: 1983**(16). A total number of 50 standard cubes (150 mm) were tested. Each cube was weight before and after heating to determine its density. Each compressive strength value was the average of strength of three cubes.

7- Volume Change Determination of Concrete

7-1 Test Specimens

For the volume change determination plain concrete ,five prisms of [100*100*500mm] were adopted. Stainless steel demec points, multi-positioned were used for measurements. An extensometer, whittemore type, with a gauge length of 200mm and accuracy of (0.002mm/division)

was used to measure strain in the concrete prisms. Figure (1) gives details of the shrinkage test specimen and positions of demec points.
Specimens were cured in water till the age of 28days. The specimens were exposed to drying in laboratory for 32 days (i.e. 28 days curing in water+32 days air dried in laboratory).

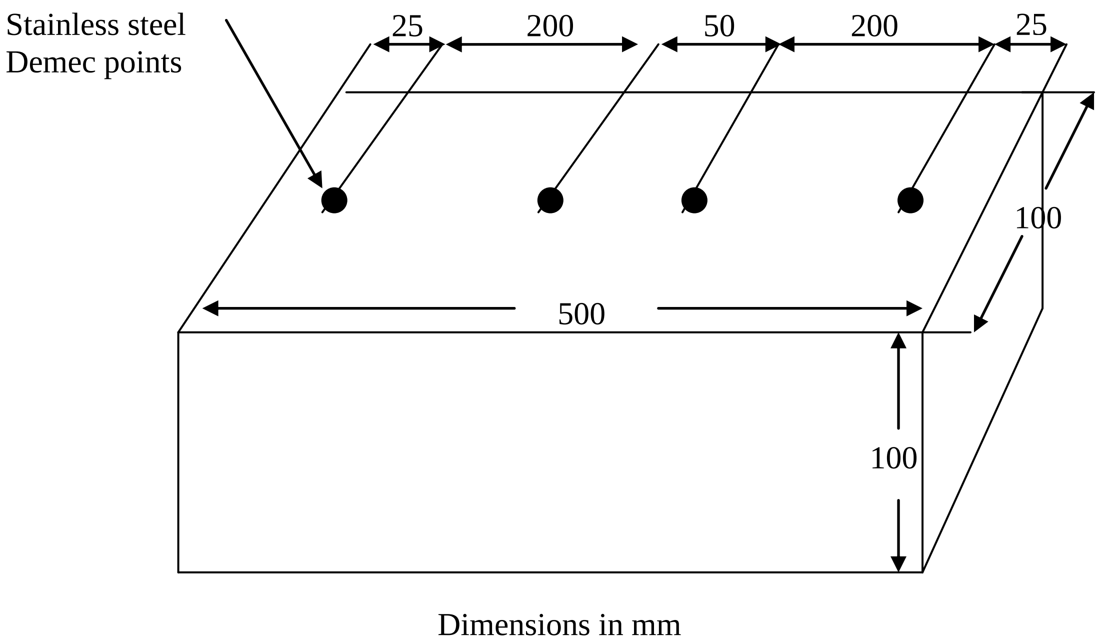

Figure (1) Locations of fixing stainless steel demec points on the concrete drying shrinkage specimen

7-2 Drying Shrinkage Measurement

The drying shrinkage was measured for the concrete prisms after the 28 days in water and the measurement was commenced from the 28 days age till the 60 days age at the dates 29, 32, 37, 45 and 60 days.

7-3 Shrinkage after Burning

Shrinkage of the concrete prisms was measured after exposure to fire flame and cooling in the laboratory to room temperature. The shrinkage prisms were monitored again and the steel demec points were fixed at the gauge lengths as in Figure (1). Shrinkage was measured at the dates of (1,3,7,15 and 30 days) after burning (which was carried out at the 60 days age).

8- Burning and Cooling

The concrete specimens and the reinforced concrete beams were burnt by direct exposure to fire flame from a network of methane burners inside the frame. The dimensions of this burners network are (1500*500mm) (length*width respectively) as shown in Figure (2). The heat radiation of fire flame was intended to simulate the heating condition in an actual fire.
The temperature of the concrete surface was continuously measured by digital thermometers, one of them was positioned in the bottom surface of the beams in contact with the flame, while the other was positioned at the unexposed upper surface of the beam, and by a thermocouple that was inserted in the center of each beam to measure the temperature at the mid-depth (75 mm from the exposed or unexposed surface). The measurement devices are shown in Figure (3).

After burning the concrete specimens and the reinforced concrete beams were cooled by sprinkling with water for about 2 minutes and then stored in laboratory environment for a bout 24 hours before testing.

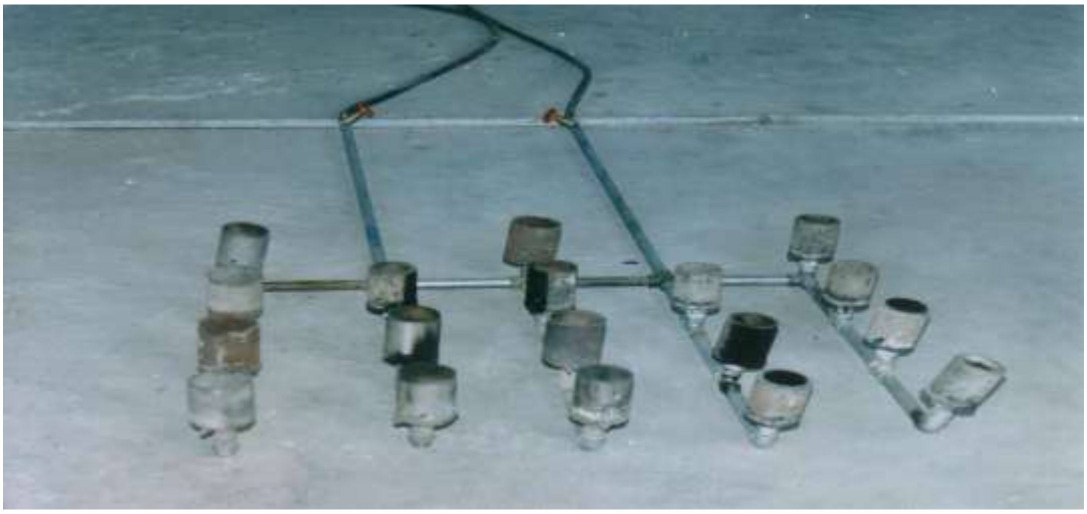

Figure (2) the network of burners

Figure (3) Temperature measurement devices

9- Testing Reinforced Concrete Beams

9-1 Beam Specimens Preparation

The reinforcing bars were cut to the required length, and 90 degree hooks were formed at the ends of each bar dimensioned according to **ACI 318-M-05, ACI 318RM-05**[17]. Stirrups made from 6mm diameter plain wires were provided to prevent the shear failure. The total number of beams cast was ten. Two beams were retained as reference beams for 30 days age. Eight beams were exposed to fire flame with different temperatures, and different periods of exposure. The beams were covered with polyethylene sheets in the laboratory for a bout 24 hours, and then demolded and moist cured in water for 10 days, then left in laboratory to dry till the date of burning

9-2 Beam Specimen Details

The beams were simply supported. All beams were 1000mm length, 150mm height and 100mm width as shown in Figure [4].

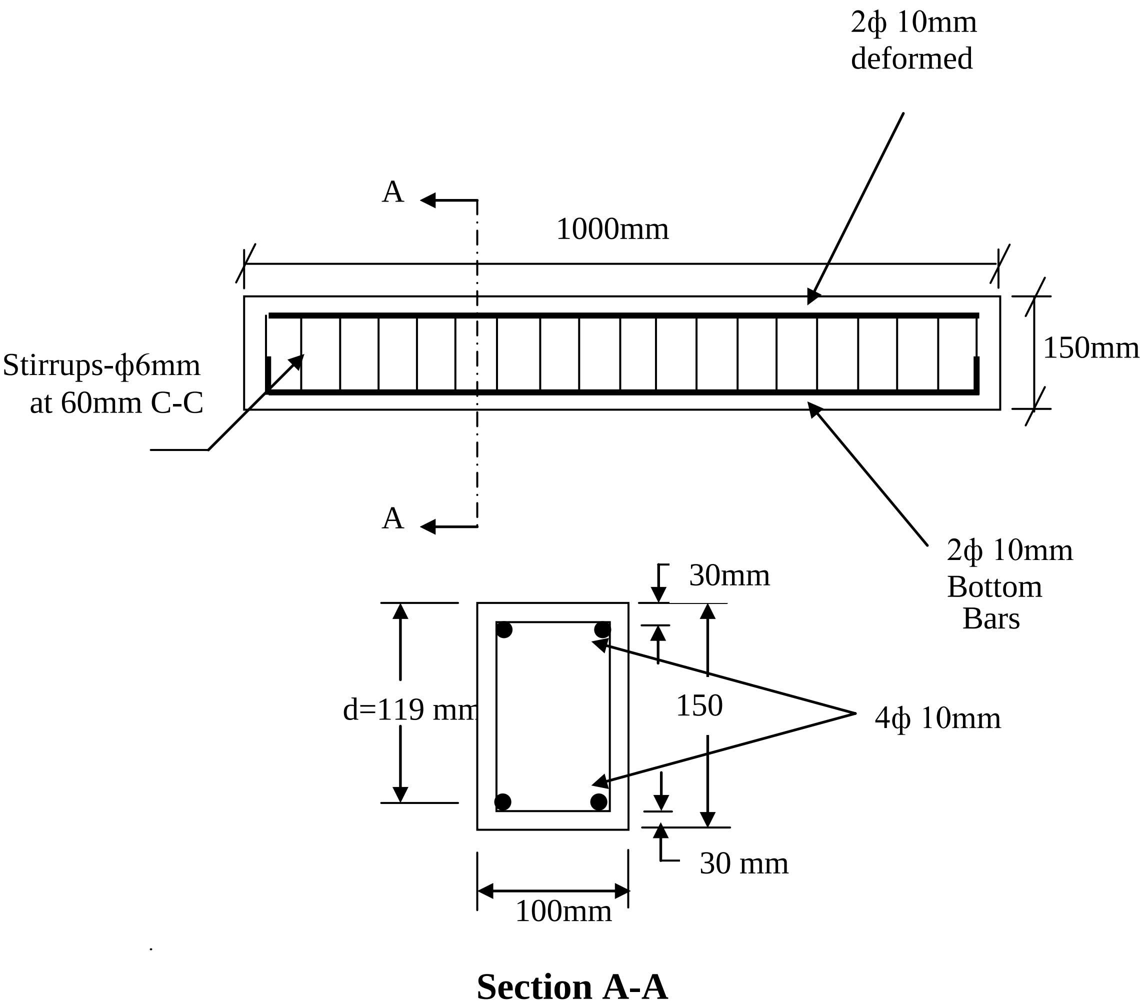

Figure (4): Details of the reinforced concrete beam specimen

10- Deflection Measurement and Test Setup

The mid-span deflection of the reinforced concrete beam specimens was measured for the unburnt and burnt specimens. The deflections were measured by fixing a dial gauge of sensitivity (0.002mm per division) in touch directly under the midspan point of the beam specimen.
Figure (5) shows the test setup of this test

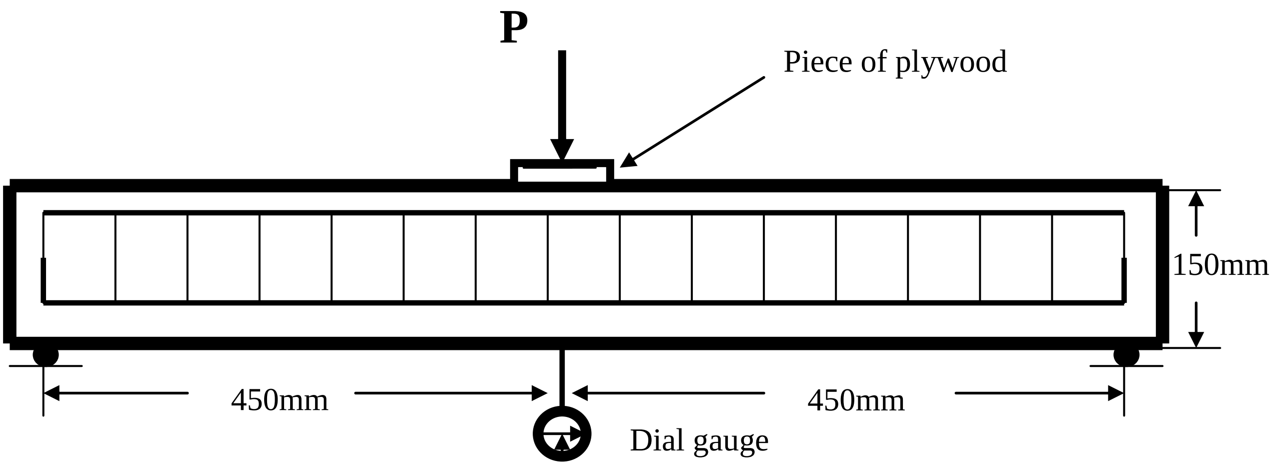

Figure (5): Test setup of the load- deflection test of the reinforced concrete beam specimen

11- Results and Discussion

11-1 Compressive Strength:

Table [4] shows the effect of the exposure to fire flame on compressive strength, while figures (6) and (7) show the relationship between compressive strength and fire flame temperature. It is clear from these figures that there is a reduction in compressive strength after exposure to fire flame, the reduction at 30 days age specimens was more than the reduction at 60 days specimens. This may be attributed to the fact that hydration of cement paste is more complete at later ages.

It was found that at 400 $^{\circ}$**C** the residual compressive strength compared to the original strength before exposure to fire flame was at the range (70-78%). **Al-Ausi and Faiyadh**[2] obtained residual compressive strength (60-71%), while **Umran**[18] obtained (67-82%). This means that these results are close to each other.

Raising the temperature to 550 $^{\circ}$**C** caused concrete strength retaining (59-65%) of its original strength before burning. These results are inline with that obtained by **Al-Ausi and Faiyadh**[2] **and Umran**[18] which were (35-53%) and (52-70%) respectively.

Table (4): Compressive strength test values of concrete cube specimens before and after exposure to fire flame.

Age at exposure (days)	Period of exposure (hour)	Cube Compressive strength (MPa)			fcua/fcub* Ratio *	
		Temperature $^{\circ}$C				
		25 (1)	400 (2)	550 (3)	2/1	3/1
30	0.5	42.4	30.04	24.96	0.73	0.59
	1.0		31.25	23.04	0.73	0.54
60	0.5	44.8	35.15	29.28	0.78	0.65
	1.0		34.31	27.70	0.76	0.62

fcua: cube compressive strength after burning
fcub: cube compressive strength before burning

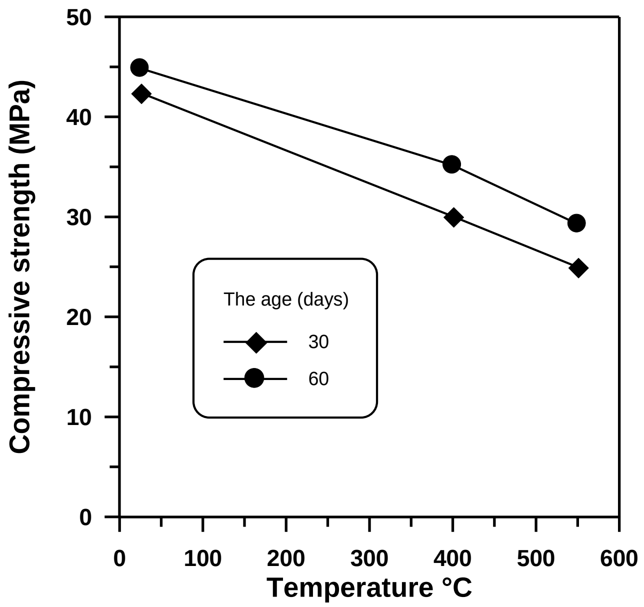

Figure (6): Effect of fire flame on the compressive strength for 0.5 hour period of exposure

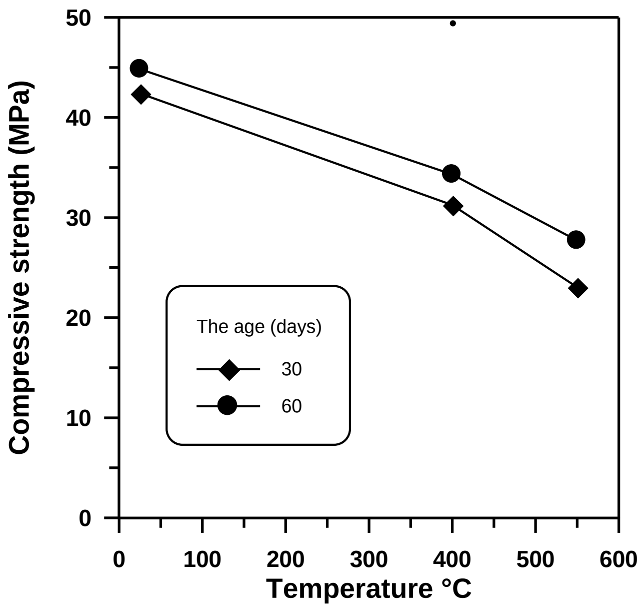

Figure (7): Effect of fire flame on the compressive strength for 1.0 hour period of exposure

11-2 Shrinkage Before and After Exposure to Fire Flame

The values of shrinkage before and after exposure to fire flame are shown in table [5] and plotted in figures [8] to [10] against age. It can be seen from these figures that the shrinkage increases with temperature of burning.

There is no significant increase in shrinkage values due to the increase of exposure period from 0.5 hour to 1 hour.

It can be seen that increasing the temperature of burning from 400 $^{\circ}$**C** to 550 $^{\circ}$**C** appreciably increases shrinkage strains by about (8.5-20%)

Table (5): Test values of shrinkage before and after exposure to fire flame [Prisms 100*100*500mm]

Temperature °C	Period of exposure(hour)	Age (days)	Strain in (millionths)
25 (Before burning)	Nil	1	140
		3	265
		7	315
		15	400
		30	480
		45	555
		60	590
		61	598
		63	605
		67	653
		75	690
		90	690
400 (after burning)	0.5	61	745
		63	865
		67	925
		75	930
		90	930
	1.0	61	750
		63	865
		67	935
		75	950
		90	950
550 (after burning)	0.5	61	760
		63	885
		67	970
		75	1025
		90	1025
	1.0	61	765
		63	885
		67	990
		75	1035
		90	1035

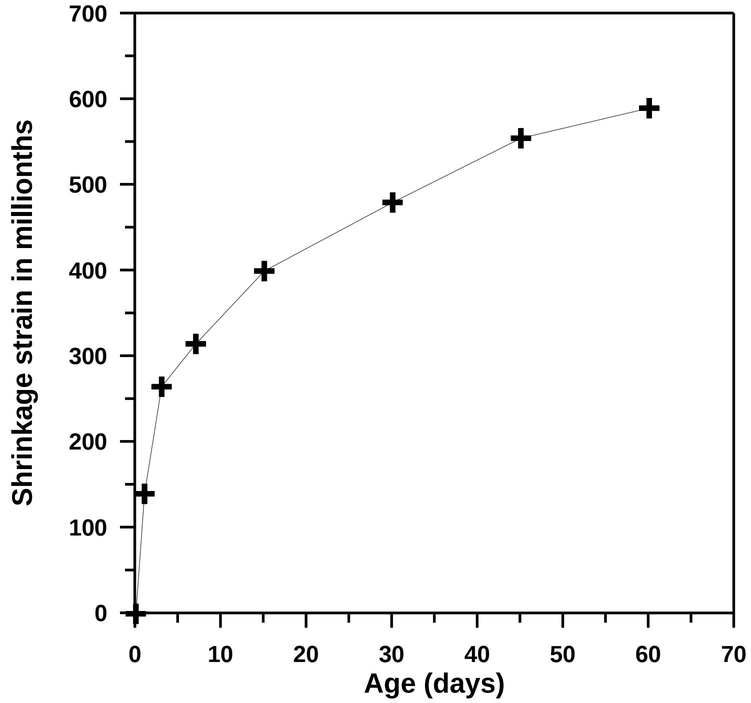

Figure (8): Measured shrinkage strain of concrete free prism (100*100*500) mm with time before exposure to fire

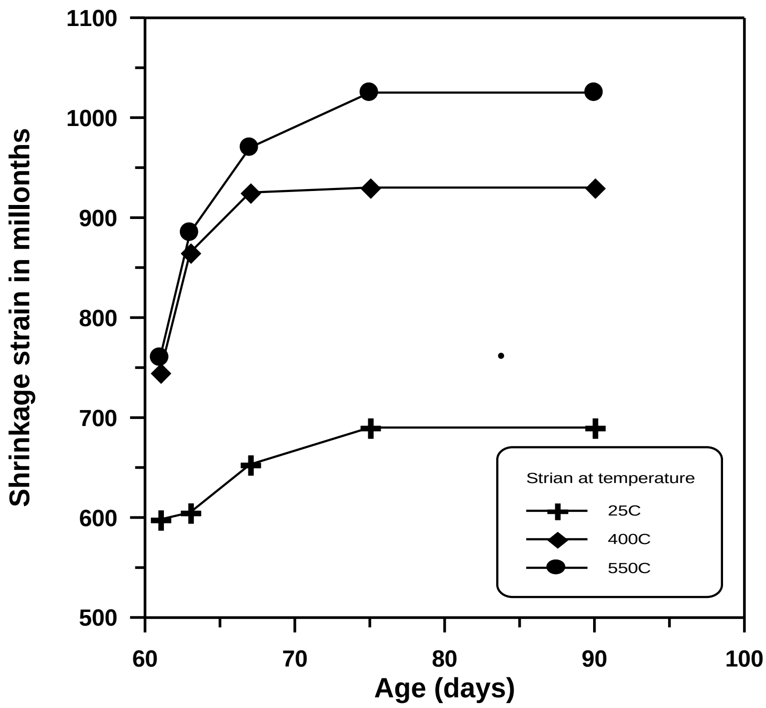

Figure (9): Measured shrinkage strain of the reinforced concrete free prism (100*100*500) mm with time after exposure to fire flame for 0.5 hour.

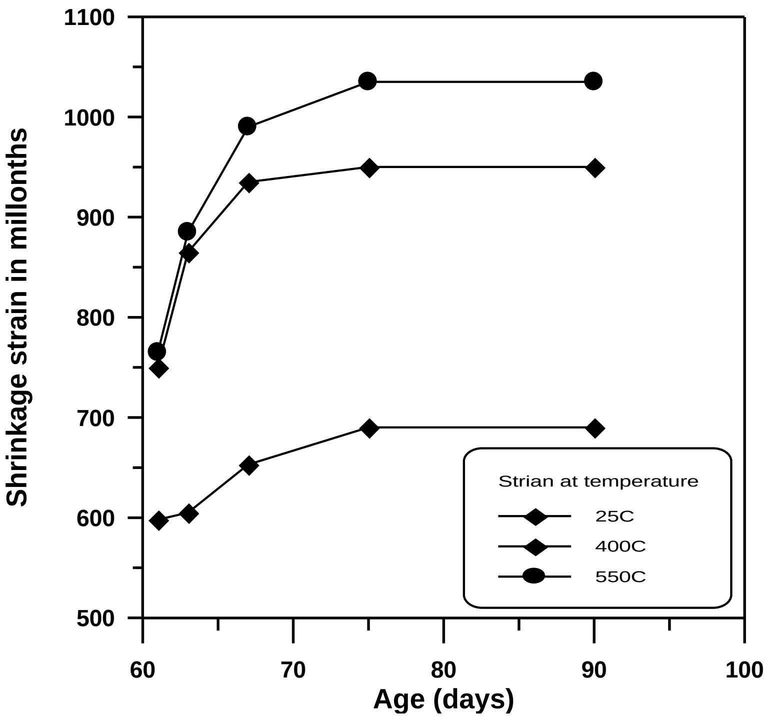

Figure (10): Measured shrinkage strain of the reinforced concrete free prism (100*100*500) with time after exposure to fire flame for 1.0 hour.

11-3 Deflection Before and After Burning:

Progressive concentrated single load applied at mid span was used to determine the resulted immediate deflection of the reinforced concrete simply supported beam specimens. The deflections were recorded at each stage of loading. The load at failure of specimen (yielding of steel reinforcement) was recorded. The test results were summarized in Table [6].

After the beams were subjected to fire flame, two types of cracks were developed; the first was thermal cracks appearing in a random fashion all over the surface. They originated from top or bottom edges and terminated near the mid-depth of the beam. The crack width was about (0.3-0.5mm). The pattern of fine cracks was consistent with the release of moisture being greater in the outer surface than in the interior core resulting in differential shrinkage. The second type of cracks was flexural tensile cracks due to the load applied at the mid-span region.

One–point load on simply supported beams gives only one section of maximum bending stresses, this section occurs under the point load.

It can be indicated from the results in table (6) that the ultimate load capacity of the beams is adversely influenced by the fire flame exposure and this deleterious effect decreases the ultimate load capacity by about 15-37%. Also the maximum deflection at ultimate load increases by about 30% which shows clearly reduced stiffness behavior. This reduction in the stiffness of the reinforced beam specimens subjected to fire flame can be attributed to the following two parameters; the first is the reduction in the moment of inertia of the section (section modulus) I_{eff} due to cracking initiating due to the effect of fire, whereas, the second is the reduction in the modulus of elasticity (E) of concrete and steel.

The ultimate load of the beams (B1-B10) was divided by the load factor (1.6) to calculate the service load. A comparison of the measured and computed midspan deflection of beams at service load is shown in Table (7)

Table (6): The measured ultimate load and mid- span deflection at ultimate load

Beam No	Temperature °C	Experimental Ultimate Load (kN)	ACI Ultimate Load (kN)	Experimental Deflection at failure (mm)
B1	25	41.77	35.2	7.31
B3	400	35.64	33.6	10.32
B5		32.40	33.8	10.01
B7	550	29.51	32.5	10.38
B9		26.40	31.9	10.55
B2	25	42.08	35.4	6.31
B4	400	34.34	34.3	8.23
B6		31.20	34.31	9.00
B8	550	28.64	33.56	9.62
B10		26.78	33.16	10.11

It can be observed from table [7] that the service load and deflection at this service load are remarkably reduced with increasing temperature of burning, this reduction may reach about 50% at 550 °**C** temperature of burning. Theoretical values of ultimate flexural load and deflection at this load were obtained according to the ultimate design method (U.D.M) ACI 318-05
At burning temperature (400-550 °**C**), for period of exposure (0.5-1) hr at age (30) days, the ratio between the experimental and theoretical deflections were in the range from (1.12-0.72).
At burning temperature (400-550 °**C**), for period of exposure (0.5-1) hr at age (60) days, the ratio between the experimental and theoretical deflections were in the range from (1.15-0.45).

Table (7): The measured service load and mid- span deflection at service load

Beam No	Temperature °C	Experimental Service Load (kN)	ACI Service Load (U.D.M) (kN)	Experimental Deflection at Midspan (mm) (1)	Theoretical Deflection at Midspan mm) (2)	Ratio of Exp.def. /Theo.def. (1) / (2)
B1	25	26.11	22.00	1.98	1.14	1.74
B3	400	22.28	21.00	1.72	1.54	1.12
B5		20.25	21.13	1.70	1.58	1.07
B7	550	18.44	20.31	1.49	2.07	0.72
B9		16.50	19.94	1.52	2.05	0.74
B2	25	26.30	22.13	2.10	1.38	1.52
B4	400	21.46	21.44	1.55	1.35	1.15
B6		19.50	21.44	1.43	1.47	0.97
B8	550	17.90	20.98	1.05	2.02	0.52
B10		16.74	20.73	0.95	2.10	0.45

It can be seen from Table (8) that the ultimate load resisted (i.e. bending moment capacity) decreases remarkably with increasing of fire flame temperature and period of exposure. This can be attributed to that fire flame decreases the compressive strength and subsequently tensile strength of the lower surface of the beam which is subjected directly to fire flame. Hence precracking will take place and consequently the effective moment of inertia of the section I_{eff}, will be reduced before loading the beam which decreases the load carrying capacity of that beam.

It can be seen also that the bending moment capacity decreases with increasing of fire flame temperature and period of exposure.

At burning temperature (400-550 °**C**), for period of exposure (0.5-1) at age (30) days, the ultimate design method **ACI 318-M-05, ACI 318RM-05**[17] gave unaccurate results to predict bending moment capacity. The ratio between the measured and ultimate design method values were in the range from (1.06-0.83).

At burning temperature (400-550 °**C**), for period of exposure (0.5-1) at age (60) days, the ratio between the measured and ultimate design method values were in the range from (1.19-0.81).

This finding is justifiable as the splitting tensile strength, flexural strength and modulus of elasticity of concrete subjected to fire flame are more severely affected than compressive strength. Hence, flexural capacity is negatively affected by exposure to fire flame because it depends on the above strengths rather than on compressive strength.

Table (8): Comparison of the flexural test results with that obtained from (U.D.M) ACI 318 Code for beam specimens.

Beam No	Temperature °C	Period of Exposure (hour)	Age (days)	Compressive Strength of Cylinder (MPa)	Ultimate load (kN)	Mu test kN. m	Mu (U.D.M) ACI 318 kN. m	Mu test / Mu (U.D.M)
B1	25	Nil	30	36.04	41.77	9.40	7.92	1.19
B3	400	0.5		25.53	35.64	8.02	7.56	1.06
B5		1		26.38	32.40	7.29	7.60	0.96
B7	550	0.5		21.22	29.51	6.64	7.31	0.91
B9		1		19.58	26.40	5.94	7.18	0.83
B2	25	Nil	60	38.08	42.08	9.47	7.97	1.19
B4	400	0.5		29.88	34.34	7.73	7.74	0.99
B6		1		29.16	31.20	7.02	7.72	0.90
B8	550	0.5		24.89	28.64	6.44	7.53	0.86
B10		1		23.55	26.78	6.03	7.46	0.81

11-4 Surface Condition and Fire Endurance of the Tested Beams:

The aim of design for fire safety should be to limit damage due to fire. The unexposed surface of each tested beam was observed throughout 0.5 and 1.0 hour fire test.

Figure [12] shows the temperature-time curves for the exposed, mid-depth and unexposed surface of the beam. At the beginning the beams are at room temperature, measured to be 25C.

The experimental results clearly indicated that the temperature near the surface to fire is higher and decreases towards the top fiber of the beam thickness. Similar behavior was observed by other investigators **Dhahir** [19] **and Ehm and Von Postel**[20]

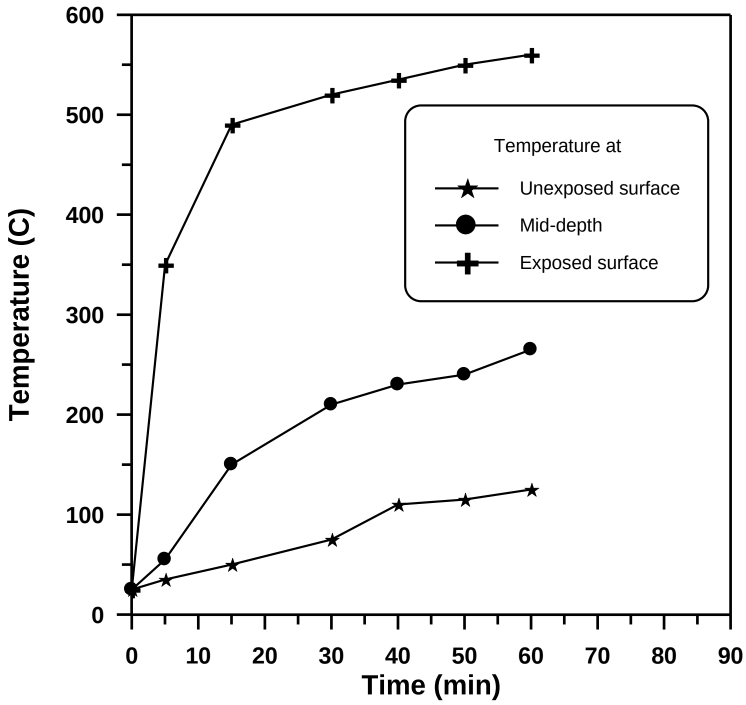

Figure (11) Beam temperature as a function of time at various depths (beam thickness: 150mm)

12- Conclusions

Based on the results obtained from this work, the following conclusions can be with drawn:

1- The residual compressive strength ranged between (70- 78%) at 400 $^{\circ}$C, and (59- 65%) at 550 $^{\circ}$C burning temperature.

2- Large proportion of drop in compressive strength occurs at the first 0.5 hour period of exposure.

3- It was found that the shrinkage values after burning increase with fire temperature increase.

4- The temperature distribution through the thickness of beam was found to be similar for all the beams, which have the same thickness and exposure 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 honeycomb fashion all over the surface. The second cracks originated at mid-span region due to bending stresses from the applied load which are called flexural cracks.

6- It was noticed that the load-deflection relations of specimens exposed to fire flames are more leveled indicating softer load- deflection behavior than that of the control beams. This can be attributed to the early cracks and lower modulus of elasticity.

7- The bending moment capacity decreases with increasing of fire flame temperature and period of exposure.

At burning temperature (400-550 $^{\circ}$C), for a period of fire exposure (0.5-1) hr. at age (30) days, the ultimate design method (ACI 318-M-2005) gave inaccurate results to predict bending moment capacity. The ratio between the measured and ultimate design method values of bending moment capacity is in the range of (1.06-0.83). At burning temperature (400-550 $^{\circ}$C), for a period of fire exposure (0.5-1) hr. at age (60) days, the ratio between the measured and ultimate design method values were in the range of (1.19-0.81).

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