Effect of Burning on Load Carrying Capacity of Reinforced Concrete Columns

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Absract

In this investigation, the effect of burning by fire flame on the behavior and load carrying capacity of reinforced concrete columns.

The experimental program consisted of casting and testing of 120 column specimens were divided into two series with target compressive strength (30 and 40 MPa) and named series A and B respectively. Each series was divided into three main groups axially concentric loaded, 30mm and 80mm eccentrically loaded.

Results indicate remarkable reduction in load carrying capacity after exposure to fire flame. The residual load capacity was (86-87%), (83-85%) for concentric loaded specimens for the two series A and B respectively, while it was (88-89%), (85-87%) and (110-112%), (108-110%) for column specimens loaded at eccentricity of (30 and 80mm) respectively at fire temperature (400oC), whereas the residual load capacity at fire temperature (600oC) was (43-46%), (39-41%) for concentrically loaded specimens and (46-48%), (37-43%) and (46-52%), (45-47%) for columns loaded at eccentricity of (30 and 80mm) respectively, whereas the residual load capacity at fire temperature (750oC) was (26-29%), (24-27%) for concentrically loaded specimens and (28-32%), (23-28%) and (40-42%), (34-38%) for columns loaded at eccentricity of (30 and 80mm) respectively.

Load-deflection curves indicate deleterious response to the fire exposure. Also, it was noticed that the maximum crack width increases with increasing fire temperature and amount of spacing between lateral steel ties.

الخلاصة

في هذا البحث تم دراسة تأثير الحريق بواسطة لهب النار المباشر على سلوك وسعة التحمل القصوى للأعمدة الخرسانية المسلحة. تضمن الجزء العملي تهيئة وفحص 120 أنموذجا لأعمدة خرسانية مسلحة مقسمة إلى مجموعتين بمقاومة انضغاط (30، 40) ميكاباسكال، وكل مجموعة قسمت إلى ثلاث مجموعات رئيسيه محورية التحميل أو لامحورية التحميل بلا تمركزية 30 و 80 ملم.

لقد أشرت النتائج انخفاضا ملحوظا في مقاومة الحمل بعد التعرض إلى لهب النار، حيث كانت مقاومة الحمل المتبقية هي (86-87%) و (83-85%) للأعمدة المحملة محوريا، بينما كانت (88-89%)، (85-87%) و (110-112%)، (108-110%) لنماذج الأعمدة المحملة لامحوريا وبلا تمركزية 30 و 80 ملم على التوالي عند التعرض إلى درجة حرارة (400) درجة مئوية، بينما مقاومة الحمل المتبقية وبعد الحرق إلى (600) درجة مئوية هي (43-46%) و (39-41%) للنماذج المحملة محوريا و (46-48%)، (37-43%) و (46-52%)، (45-47%) لنماذج الأعمدة المحملة لامحوريا وبلا تمركزية 30 و 80 ملم على التوالي، بينما مقاومة الحمل المتبقية وبعد الحرق إلى (750) درجة مئوية هي (26-29%) و (24-27%) لنماذج محملة محوريا و (28-32%)، (23-28%) و (40-42%)، (34-38%) لنماذج الأعمدة المحملة لامحوريا وبلا تمركزية 30 و 80 ملم على التوالي.

إما منحنيات الحمل-الانحراف فقد كانت الاستجابة سلبية مع درجة التعرض للنار. كذلك يمكن ملاحظة إن عرض الشق الأقصى يزداد بزيادة درجة حرارة النار وزيادة المسافة بين أطواق التسليح العرضية.

Introduction

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

Concrete columns are considered to be important structural elements in reinforced concrete structures because they support the structure and transfer the loads to the supports or foundation, so any failure or damage occurs in the column may cause a partial or complete failure of the structure by perhaps chain action *(Sakai and Sheikh, 1989)*.

Research Significant

It was found that the literature lackes investigating the effect of exposing reinforced concrete columns to direct fire flame which needs a considerable attention to find the extent of damage which may occur in these important compression and flexural load bearing members.

In order to simulate this problem to practical site conditions, reduced scale column models were cast and they were as close as possible to practical circumstances. This research is seeked to cover the limited area of research about this problem. This will guide and facilitate the suggestion of rehabilitation of such members exposed to fires under loading of different degrees.

The current research proposes a reinforced concrete column model which resemble the simulation of the state of stress which the reinforced concrete columns are subjected to during fire in laboratory.

Simulation of real fires in laboratory using a set of methane burners which subjecting the column specimens to real fire flame.

Literature Review

-The Effect of Fire on Reinforced Concrete Columns

In recent years a number of notable fires have occurred during construction of concrete-framed buildings, when formwork and false work has caught fire, see Plate (1). Fortunately, even after a sever fire, reinforced concrete structures are generally capable of being repaired rather than demolished *(Ingham and Tarada, 2007)*.

Plate (1): The concrete frame of a ten-storey building that was fire-damaged during construction, (Ingham and Tarada, 2007).

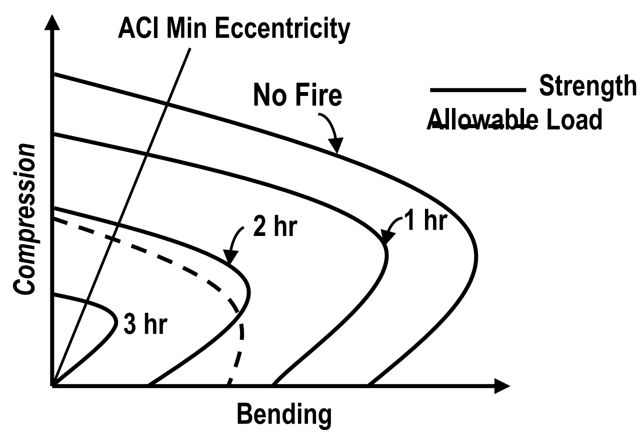
After a fire, an appraisal is normally required as soon as the building can be safely entered and generally before the removal of debris. To ensure safety, temporary false work may be required to secure individual members and stabilize the structure as a whole.

Jau and Huang, tested 22 reinforced concrete corner columns under high temperature of 270cm long were manufactured. The cross-section was (30×45cm). The parameters considered were the cover thickness, concrete strength, steel ratio, eccentricity, and fire duration. They concluded that the factors affecting the initiation of cracks in the sequence of importance are fire duration, concrete cover thickness, steel ratio, concrete strength, and eccentricity. However, the existence and characteristics of surface crack does not directly relate to the strength reduction.

Kodur et al., 2005 investigated the behavior of fibre reinforced polymer (FRP) wrapped (confined) reinforced concrete columns under exposure to the standard fire. Three full-scale reinforced concrete columns, two of these columns were circular and the third column was square in cross-section. The circular columns were 400 mm in diameter, while the square column was 406 mm in width. All three columns were 3810 mm long. The longitudinal reinforcement in the circular columns was comprised of eight 19.5mm diameter bars, with 40mm clear cover to the spiral reinforcement and 10mm for lateral reinforcement. The square column had four 25mm diameter longitudinal reinforcing bars with 40mm cover to the ties, and 10mm diameter for ties spaced at 406mm. These specimens were heated in furnace chamber by 32 propane gas burners, arranged in eight columns containing four burners each. The test results showed that the FRP materials used as externally bonded reinforcement for concrete structures were sensitive to the effects of elevated temperatures. They also noticed that the providing proper fire insulation a (5 and 4) hours fire endurance rating can be achieved for loaded circular and square reinforced concrete columns strengthened with FRP wraps respectively.

-Effect of Fire on Load Eccentricity of Reinforced Concrete Columns

The fire resistance of reinforced concrete columns is the time it takes for strength to be reduced to the level of the applied load. Figure (1) shows strength interaction curves during fire for a typical column, fire resistance being of the order of 2.0 hours. The fire resistance in this figure is greater in compression than in bending because the concrete core heats more slowly than the reinforcing *(Allen and Lie, 1977)*.



Figure(1): Typical column interaction curves (Allen and Lie, 1977).

Jae-Hoon and Hyeok-Soo, 2000, verified the basic design rules of high strength concrete columns. A total of 32 column specimens were tested to investigate structural behavior and strength of eccentrically loaded reinforced concrete tied columns. The main variables included in this test program were concrete compressive strength, amount of steel, and load eccentricity. In this work, concrete compressive strength varied from 34.9 to 93.2 MPa, and the longitudinal steel ratios ranged between 1.13% and 5.51%. Test results of column sectional strength were compared with the result of analysis by using the American Concrete Institute (Building Code requirements for structural concrete) rectangular stress block, trapezoidal stress block, and modified rectangular stress block. Axial force-moment-curvature analysis was also performed for predicting axial load-moment strength and compared with test results. It was found that the ACI (318-95) code rectangular stress block provides overestimated column strengths for the lightly reinforced high strength column specimens.

Experimental Work

Reinforced Concrete Column Specimens

The column specimens were divided into two series A and B with two target compressive strengths (30 and 40) MPa respectively. The specimens of each series were tested by applying compressive axial loads and divided to three groups depending on the way of load application. The specimens of the first group were concentrically loaded, whereas, the specimens of the second and third group were eccentrically loaded by eccentricity of (30mm) and (80mm) respectively. The details of the reinforced concrete column specimens are shown in Table (3-1). Each column is identified by three symbols. The first is a letter that refers to concrete strength series. The letters (A) and (B) refer to target compressive strength (30 and 40 MPa) respectively. The second symbol is a number that refers to the eccentricity of applied load; (1) refers to axially concentrically loaded columns, while (2) and (3) refer to the columns that were loaded at eccentricities of 30 and 80 mm respectively. The third symbol refers to the tie spacing; (0) for no ties, (1), (2) and (3) for 250, 150 and 50 mm tie spacing, respectively.

After greasing the moulds of the column specimens, reinforcement bars were held carefully in their position inside these moulds. In order to get a cover, small pieces of steel were placed at sides of the column reinforcement.

The reinforcement used is deformed steel bars of Ø8 mm and Ø10 mm respectively. Figure (2) shows the details of the reinforcement of column specimens.

Table (1): Summary of column test specimens.

Number	Series	Group	Temperature Stage (ºC)	Column No.	Concrete Compressive Strength (MPa)	Eccentricity of Applied Load (mm)	Spacing of Lateral Ties (mm)
1		1	25 ºC	A10	30	0	No ties
2				A11	30	0	250
3				A12	30	0	150
4				A13	30	0	50
5				A12*	30	0	150
6			400 ºC	A10	30	0	No ties
7				A11	30	0	250
8				A12	30	0	150
9				A13	30	0	50
10				A12*	30	0	150
11				A10	30	0	No ties
12				A11	30	0	250

Table (1): Continued.

13	A		600 ºC	A12	30	0	150
14				A13	30	0	50
15				A12*	30	0	150
16			750 ºC	A10	30	0	No ties
17				A11	30	0	250
18				A12	30	0	150
19				A13	30	0	50
20				A12*	30	0	150
21		2	25 ºC	A20	30	30	No ties
22				A21	30	30	250
23				A22	30	30	150
24				A23	30	30	50
25				A22*	30	30	150
26			400 ºC	A20	30	30	No ties
27				A21	30	30	250
28				A22	30	30	150
29				A23	30	30	50
30				A22*	30	30	150
31			600 ºC	A20	30	30	No ties
32				A21	30	30	250
33				A22	30	30	150
34				A23	30	30	50
35				A22*	30	30	150
36			750 ºC	A20	30	30	No ties
37				A21	30	30	250
38				A22	30	30	150
39				A23	30	30	50
40				A22*	30	30	150
41		3	25 ºC	A30	30	80	No ties
42				A31	30	80	250
43				A32	30	80	150
44				A33	30	80	50
45				A32*	30	80	150
46			400 ºC	A30	30	80	No ties
47				A31	30	80	250
48				A32	30	80	150
49				A33	30	80	50
50				A32*	30	80	150
51			600 ºC	A30	30	80	No ties
52				A31	30	80	250
53				A32	30	80	150
54				A33	30	80	50
55				A32*	30	80	150
56			750 ºC	A30	30	80	No ties
57				A31	30	80	250
58				A32	30	80	150
59				A33	30	80	50
60				A32*	30	80	150
61				B10	40	0	No ties

Table (1): Continued.

62	B	1	25 ºC	B11	40	0	250
63				B12	40	0	150
64				B13	40	0	50
65				B12*	40	0	150
66			400 ºC	B10	40	0	No ties
67				B11	40	0	250
68				B12	40	0	150
69				B13	40	0	50
70				B12*	40	0	150
71			600 ºC	B10	40	0	No ties
72				B11	40	0	250
73				B12	40	0	150
74				B13	40	0	50
75				B12*	40	0	150
76			750 ºC	B10	40	0	No ties
77				B11	40	0	250
78				B12	40	0	150
79				B13	40	0	50
80				B12*	40	0	150
81		2	25 ºC	B20	40	30	No ties
82				B21	40	30	250
83				B22	40	30	150
84				B23	40	30	50
85				B22*	40	30	150
86			400 ºC	B20	40	30	No ties
87				B21	40	30	250
88				B22	40	30	150
89				B23	40	30	50
90				B22*	40	30	150
91			600 ºC	B20	40	30	No ties
92				B21	40	30	250
93				B22	40	30	150
94				B23	40	30	50
95				B22*	40	30	150
96			750 ºC	B20	40	30	No ties
97				B21	40	30	250
98				B22	40	30	150
99				B23	40	30	50
100				B22*	40	30	150
101		3	25 ºC	B30	40	80	No ties
102				B31	40	80	250
103				B32	40	80	150
104				B33	40	80	50
105				B32*	40	80	150
106			400 ºC	B30	40	80	No ties
107				B31	40	80	250
108				B32	40	80	150
109				B33	40	80	50
110				B32*	40	80	150

Table (1): Continued.

111			600 ºC	B30	40	80	No ties
112				B31	40	80	250
113				B32	40	80	150
114				B33	40	80	50
115				B32*	40	80	150
116			750 ºC	B30	40	80	No ties
117				B31	40	80	250
118				B32	40	80	150
119				B34	40	80	50
120				B32*	40	80	150

Concrete cover=15mm, except (* =30mm)

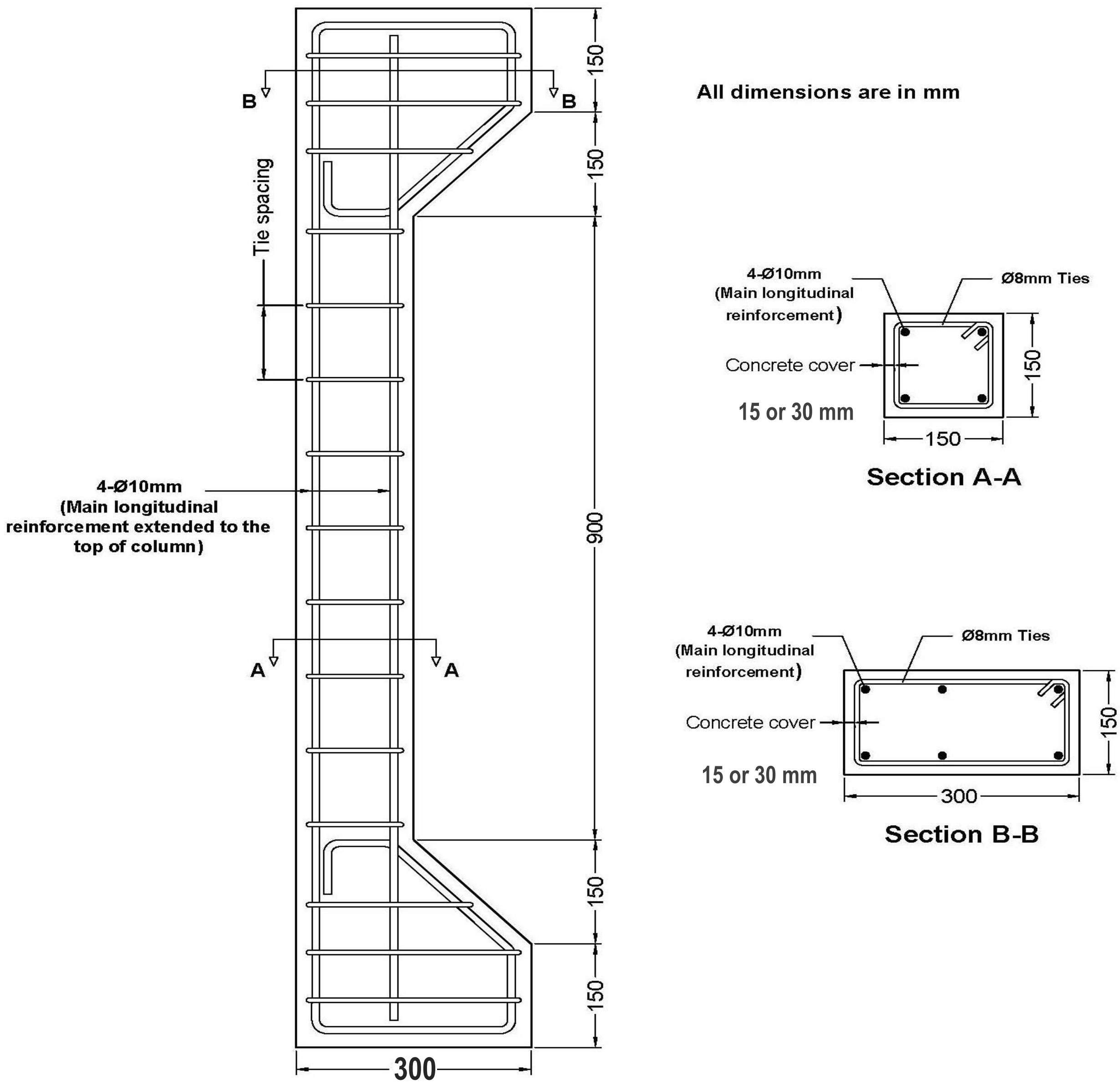


Figure (2): Reinforcement details of reinforced concrete column specimens.

Materials and Mixes

Introduction

The properties of materials used in any structure are of considerable importance (Neville, 1995, and ACI Committee 211, 1997). The properties of materials used in the current study are presented in this chapter. Standard tests according to the American Society for Testing and Materials (ASTM) and Iraqi specifications IQS were conducted to determine the properties of materials.

Cement

Tasluga-Bazian Ordinary Portland cement (O.P.C) (ASTM Type I) manufactured in Iraq was used for concrete mixes throughout the present work. The cement was properly stored in a dry place to avoid the exposure to the atmosphere. This cement complied with the Iraqi specification (IQS, No.5:1984). Testing of cement was conducted in the laboratories of Consultant Engineering Bureau in Babylon University. The physical properties and chemical analysis of the cement used are given in Tables (2) and (3). Also, the compounds of cement calculated according to Bogue equations are listed in Table (3).

Table (2): Physical properties of the cement.

Physical Properties	Test results	IQS (No.5: 1984) Limits
Fineness, Blaine, cm^2/gm	3105	≥ 2300
Setting time, Vicat's method		
Initial hrs: min.	1:54	≥ 0: 45
Final hrs: min.	4:25	≤ 10: 00
Compressive strength of 70.7 mm cube, MPa		
3 days	22.5	≥ 15
7 days	31.5	≥ 23

Table (3): Chemical composition of the cement.

Oxide	Percentage (%)	IQS (No.5: 1984) Limits
CaO	61.49	-------
SiO_2	21.18	-------
Fe_2O_3	3.68	-------
Al_2O_3	5.16	-------
MgO	2.35	≤ 5.0
SO_3	2.42	≤ 2.8
L.O.I.	2.27	≤ 4.0
I.R.	0.95	≤ 1.5
Compound composition	**Percentage (%)**	**IQS (No.5: 1984) Limits**
C_3S	41.59	--------
C_2S	29.59	--------
C_3A	7.45	--------
C_4AF	11.20	--------
L.S.F.	0.81	0.66-1.02

Fine Aggregate

Well-graded natural sand from Al-Akhaidher region in Iraq was used for concrete mixes. The fine aggregate was sieved at sieve size (9.5 mm) to separate the aggregate particles of diameter greater than 9.5 mm. The sand was then washed and cleaned with water several times, then it was spread out and left to dry in air, after which it was ready for use. The physical and chemical properties of the sand are listed in Table (4). Its grading conformed to the Iraqi specification(IQS, No.45:1984), Zone(3).

Table (4): Properties of fine aggregate.

Sieve size (mm)	Percentage passing (%)	IQS (No.45: 1984) Limits, Zone 3
9.5	100	100
4.75	94	90-100
2.36	93	85-100
1.18	81	75-100
0.6	62	60-79
0.3	27	12-40
0.15	0	0-10
Properties	**Test results**	**IQS (No.45 : 1984) Limits**
Sulphate content, SO_3 (%)	0.28	≤ 0.5
Specific gravity	2.60	-----
Absorption (%)	1.6	-----

Coarse Aggregate

The gravel used was brought from Al-Nibaii area in Iraq with a maximum size of (20 mm). The gravel was sieved at sieve size of (20 mm). The gravel was washed and cleaned by water several times, later it was speared out and left in air to dry before use. The gravel used conforms to the Iraqi specification (IQS, No.45:1984). The grading and other properties of this type of aggregate are shown in Table (5).

Table (5): Properties of coarse aggregate.

Sieve size (mm)	Percentage passing (%)	IQS (No.45 : 1984) Limits size 20-5mm
37.5	100	100
20	100	95-100
9.5	53	30-60
4.75	5	0-10
Properties	**Test results**	**IQS (No.45 : 1984) Limits**
Sulphate content, SO_3 (%)	0.08	≤ 0.1
Specific gravity	2.64	-----
Absorption (%)	0.8	-----

Mixing Water

Ordinary clean tap water was used throughout this work for both making and curing of specimens.

Reinforcing Steel Bars

Deformed steel bars of diameters (Ø8 mm) and (Ø10 mm) were used as reinforcement. Their mechanical properties were obtained from a digital computer complementary with the testing machine. Table (6) gives the results of testing three 1000 mm long samples from each size of bars (8 and 10 mm).

Table (6): Strength properties of the used steel reinforcement.

Approximate Diameter (mm)	Measured Diameter (mm)	Area (mm^2)	Yield stress *Fy* (MPa)	Ultimate Strength *Fu* (MPa)	** Modulus of Elasticity (GPa)	Uses
10	10.01	78.69	585	745	200	Main Reinforcement
8	8.00	50.26	523.5	694.4	200	Ties

** Assumed

Mix Design and Proportions

Two target compressive strengths of 30 and 40 MPa were denoted as series A and B respectively. 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 (7).

Table (7): Mix Proportions.

		Mix Proportion kg/m^3				
Series	W/c ratio	Water	Cement	Sand	Gravel	Slump (mm)
A	0.52	205	394	717	1024	80
B	0.45	193	429	733	1024	60

Reinforced Concrete Columns Testing Procedure

The column specimens were tested using a load cell of maximum capacity of (150 Tons) at the age of (60 days). The load was applied through a bearing plate for the axially loaded columns, and through a cylindrical roller to simulate line load, attached to the top of bearing plates. The load was applied in small increments and the readings were taken every 10.0 kN load until failure occurrs. For each increment, the load was kept constant until the required measurements were recorded. Cracks were detected and drawn on the faces of the test column specimens. The reading of the lateral deflections versus loads were recorded simultaneously for each load increment. Testing continued until the reinforced concrete column shows a drop in load capacity with increasing deformation. The axial deformation of the columns was measured using vertical dial gauges having a minimum graduation of 0.001 mm and a maximum needle length of 50 mm mounted at the bottom face of the specimens. Plate (2) shows the setup of axial deformation measurements.

For the column specimens which were subjected to fire flame under loading as shown in Plate (3). The specified (target) fire temperature was reached by mounting the fire subjecting burners by a sliding arm to control the fire distance to the surface of the column specimens, and also by monitoring the fire intensity through controlling the methane gas pressure in the burners. The temperature was measured by the digital thermometer and infrared rays thermometer continuously till reaching the specified (target) fire temperature. Then, the sliding arm and gas pressure were kept at this position along the period of burning (1.5 hour). The lateral deflection of the column specimens exposed to fire are resulting from loading to 15% of ultimate load before burning, loading 15%and applied fire flame, and loading after burning until failure. While, for column specimens without burning the lateral deflection is resulting from applied load only.

Plate (2): The testing measurements of Axial deformation.

Plate (3): Testing of column specimens under 15% of ultimate load with exposure to fire flame.

RESULTS AND DISCUSSION

Effect of Burning on the Load Carrying Capacity of Columns With Different Eccentricities

The test results of the ultimate load of the reference column specimens and those exposed to fire flame are summarized in Table (8) for both series A and B. It is obvious that the values of ultimate load capacity, decreased for all column specimens when exposed to fire flame.

At fire temperature (400°C), for series A and B and for the column specimens eccentrically loaded at e=30mm, the residual ultimate loads were 89% and 86% for 1.5 hour burning exposure respectively, while the residual ultimate loads were (110-112%), (108-110%) respectively for columns loaded at eccentricity of 80mm. This can be explained by the low reduction percentages in tensile and compressive strength of the concrete member at this low fire temperature exposure. On the other hand, the expansion happening due to this fire exposure can cause an increase in the axial compressive stress affecting on the eccentric (e=80mm) loaded column specimens, which raise the moment capacity of the column and consequently the load capacity.

At fire temperature (600°C), for series A and B and the column specimens eccentrically loaded at e=30mm, the residual ultimate loads were (46-48%), (37-43%) respectively for 1.5 hour burning exposure, while the residual ultimate loads were (46-52%), (45-47%) respectively for columns loaded at eccentricity of 80mm.

At fire temperature (750°C), for series A and B and the column specimens loaded at eccentricity of 30mm, the residual ultimate loads were (28-31%), (23-28%) respectively for 1.5 hour burning exposure, while the residual ultimate loads were (40-42%), (34-38%) respectively for columns loaded at eccentricity of 80mm.

It is obvious from the results that the percentage of residual ultimate load decreases at eccentricity of 30mm for series A and B and for all exposure temperatures of (400, 600, 750 °C). The reduction in ultimate load carrying capacity of the column specimens can be attributed to the fact that the fire flame subjected to the concrete columns causes evaporation of free moisture in the concrete. With the continual exposure to burning, the temperature inside the column increases and the strength of concrete decreases. In certain cases, the pressure generated by conversion of moisture within columns may be too high for the surface layer of concrete to resist, and it may spall causing a reduction in concrete section and compressive strength. This reduction in the concrete compressive strength and yield strength of steel is causing a reduction in load carrying capacity of the column specimens.

Figures (3) and (4) present the effect of fire flame temperature on the interaction diagram for the column specimens for series A and B respectively. It can be noticed that the balanced failure point of the unburnt column specimen is located at the eccentric loaded specimens of (e=30mm). As for the fire burnt column specimens at (400, 600 and 750°C) the balanced failure position extends to the eccentric loaded specimens of (e=80mm). This indicates that fire exposure to column specimens decreases the capacity of these specimens to resist concentric loads more that to resist eccentric loads. This can be attributed to the sharp reduction in concrete compressive strength after burning at temperature of 750°C.

Table (8): Test results of ultimate load and moment capacity for reference columns and columns exposed to fire flame for series A and B.

Specimens Identification	Temperature Level (°C)	Ultimate Load (kN)	Eccentricity to Width Ratio of Column (e/h)	Ultimate Flexure Load (kN)	Moment Capacity (kN.m)
A_{12}	25	822	0	---	---
A_{12}	400	712	0	---	---
A_{12}	600	369	0	---	---
A_{12}	750	227	0	---	---
A_{12}	25	---	---	34.23	6.84
A_{12}	400	---	---	28.62	5.72
A_{12}	600	---	---	21.41	4.28
A_{12}	750	---	---	12.80	2.56
A_{22}	25	513	0.20	---	15.39
A_{22}	400	457	0.20	---	13.71
A_{22}	600	241	0.20	---	7.23
A_{22}	750	155	0.20	---	4.65
A_{32}	25	196	0.534	---	15.68
A_{32}	400	217	0.534	---	17.36
A_{32}	600	99	0.534	---	7.92
A_{32}	750	80	0.534	---	6.40
B_{12}	25	978	0	---	---
B_{12}	400	816	0	---	---
B_{12}	600	388	0	---	---
B_{12}	750	231	0	---	---
B_{12}	25	---	---	36.60	7.32
B_{12}	400	---	---	29.51	5.90
B_{12}	600	---	---	22.80	4.56
B_{12}	750	---	---	14.40	2.88
B_{22}	25	635	0.20	---	19.05
B_{22}	400	547	0.20	---	16.41
B_{22}	600	261	0.20	---	7.83
B_{22}	750	167	0.20	---	5.01
B_{32}	25	229	0.534	---	18.32
B_{32}	400	250	0.534	---	20.0
B_{32}	600	107	0.534	---	8.56
B_{32}	750	85	0.534	---	6.80

--- No Values

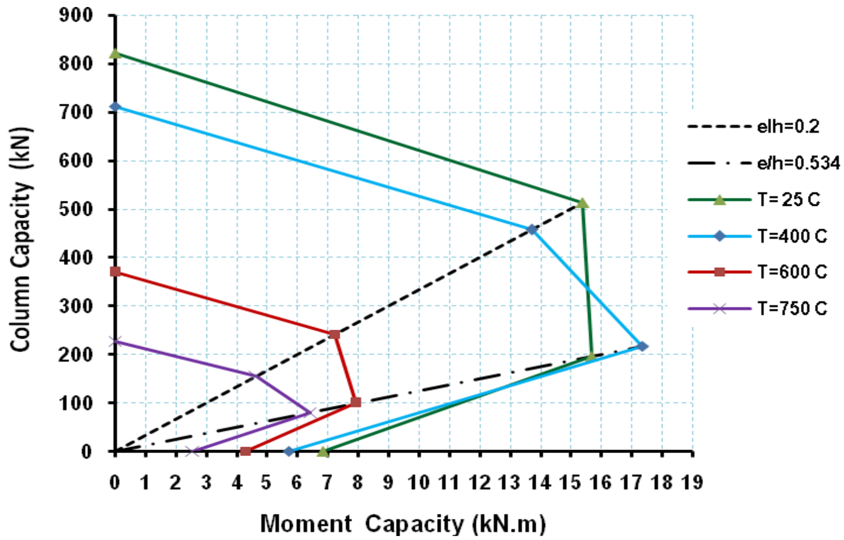


Figure (3): Effect of fire temperature on the interaction diagram of column specimens for series A and duration of burning 1.5 hour.

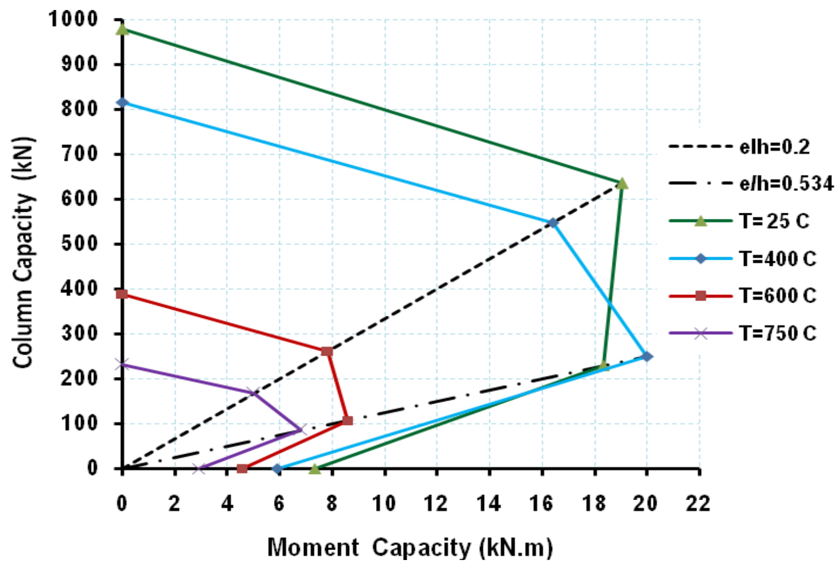


Figure (4): Effect of fire temperature on the interaction diagram of column specimens for series B and duration of burning 1.5 hour.

Effect of Burning on Load Versus Deflection Results

The load versus midheight lateral deflection relationship of reinforced concrete column specimens loaded at eccentricity of (30 and 80mm) for series A and B are presented in Figures (5) to (8).

Deflection of these column 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 column specimens.

From these Figures, it can be seen that the increase in the fire temperature has a significant effect on midheight lateral deflection of column specimens for series A and B. In addition, it can be noted that the increase in the fire temperature decreases the load carrying capacity and increases lateral deflection in column specimens. This can be attributed to the fact that heating causes a reduction in column 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, which means that load-deflection curves for series A are more sensitive to high temperatures compared with series B. These Figures reveal that the load-deflection relation of the column specimens is almost linearly proportional for the two eccentricities (30 and 80mm) and for temperature exposure (600°C and 750°C).

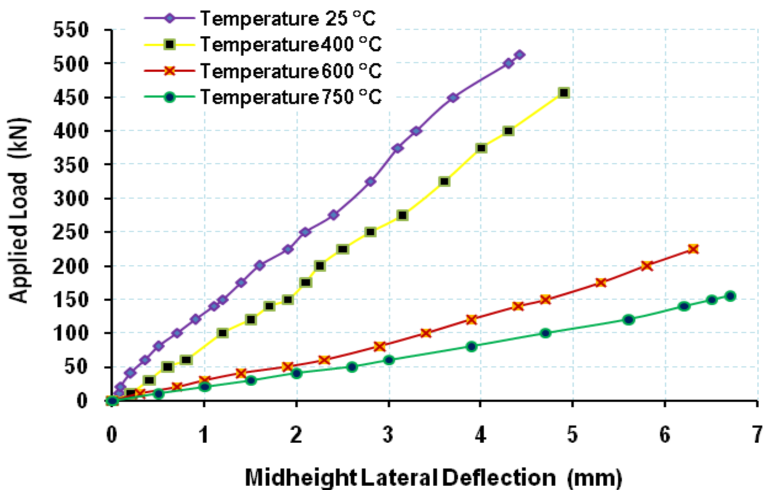


Figure (5): Load versus midheight lateral deflection curve of column specimen A22 at eccentricity (e=30mm).

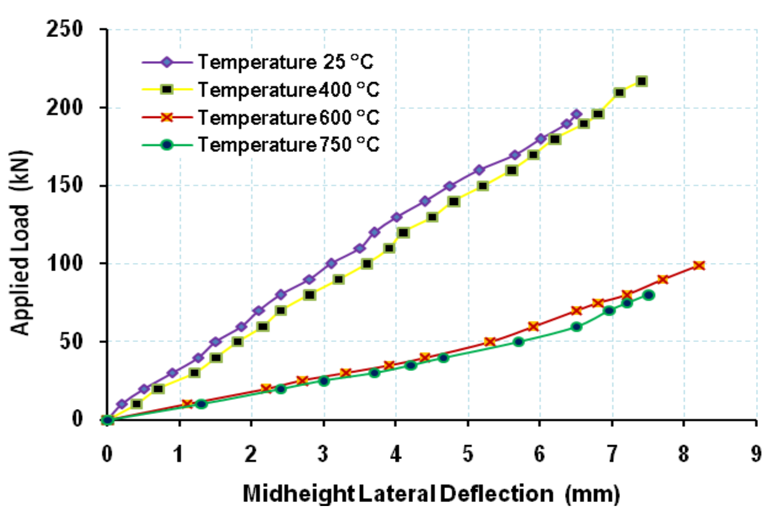


Figure (6): Load versus midheight lateral deflection curve of column specimen A32 at eccentricity (e=80mm).

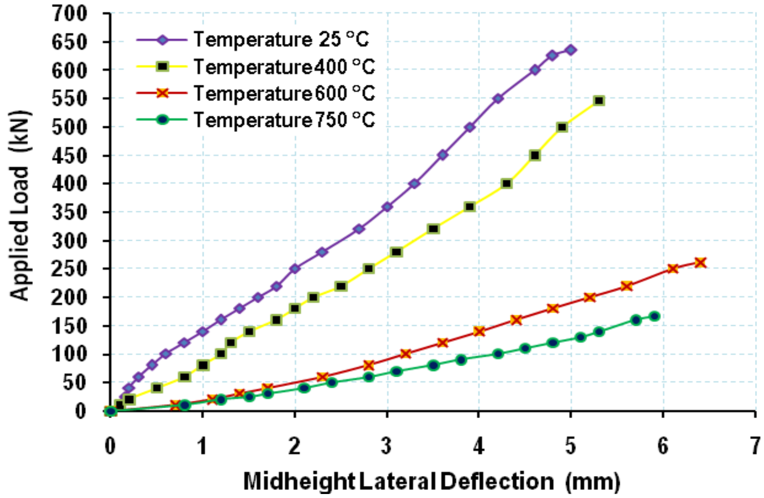


Figure (7): Load versus midheight lateral deflection curve of column specimen B22 at eccentricity (e=30mm).

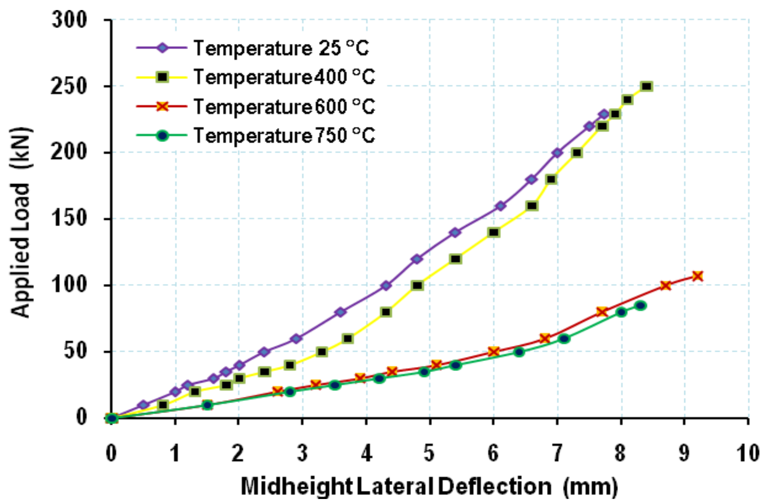


Figure (8): Load versus midheight lateral deflection curve of column specimen B32 at eccentricity (e=80mm).

Axial Deformation of Column Specimens

Effect of burning by fire flame on the characteristics of axial deformation of the column specimens is presented in Figures (9) to (12). Two dial gauges were fixed at distance 50mm from the nearest edges of these columns to measure the deformation at these edges due to burning. Also, another one was fixed at the centerline of column specimens to measure the axial deformation due to concentric load. The positive and negative axial deformation values indicate expansion and contraction respectively of the concentric column specimens. It can be seen from these Figures that the deformation of series B is significantly lower than that of series A for the same burning temperature.

There is a significant contraction in the column specimens of series A leading to a gradual (ductile) failure. While, it was found that there is a lower contraction in the column specimens of series B. From these Figures, it can be seen that the concentric column specimens exhibited a small contraction when loaded to 15% of the ultimate load during 25minutes then remain constant for the later period of this applied load, then the columns exhibit a sudden increase in contraction which was identified as failure after applied residual load. While, for column specimens exposed to fire flame temperature (400, 600, and 750°C) also exhibited a small contraction when loaded to 15% of the ultimate load during 25 minutes then remain constant for the later period of this load, thereon noticed a large elongation at the left and right edge of column specimens when exposed to burning, then a sudden contraction was observed when applying the residual load until 95% of ultimate load.

Finally, these Figures present the time versus the vertical displacements in y-direction (expansion or contraction) for column specimens at different stages during burning and loading.

The data of axial deformation was recorded up to 95% of the ultimate load, because the final axial deformation at ultimate load can not be measured due to the immediate type of failure of concrete column specimens.

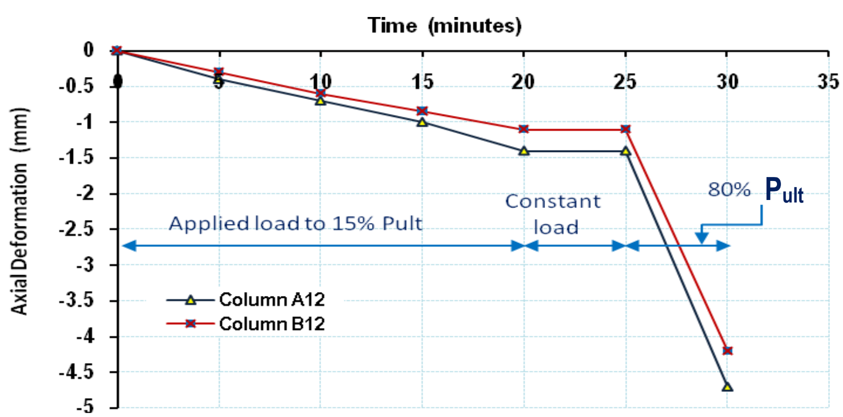


Figure (9): Axial deformation versus time curve at centerline of column specimens A12 and B12 without burning,25 °C.

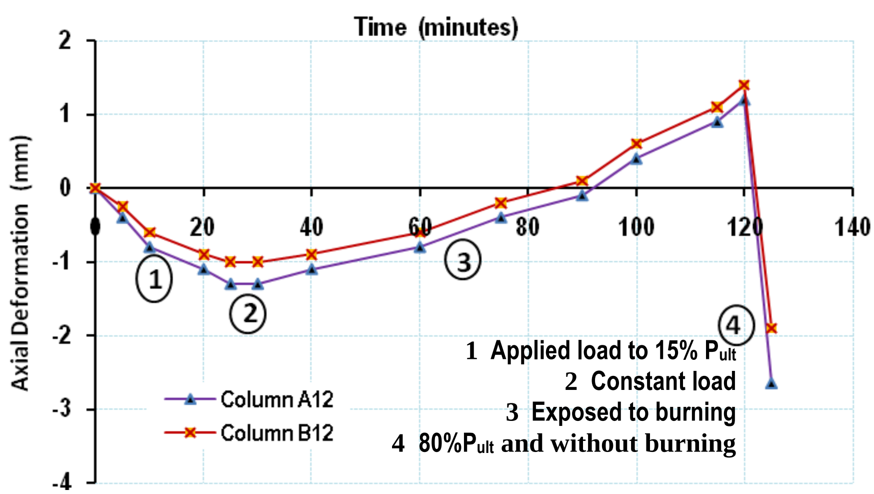


Figure (10): Axial deformation versus time curve at 50mm from the outer edge of column specimens A12 and B12 before, during and after burning at 400 °C.

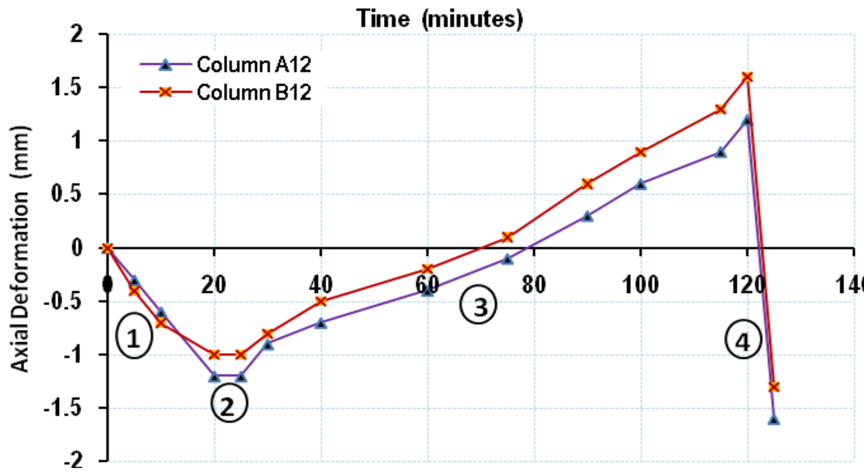


Figure (11): Axial deformation versus time curve at 50mm from the outer edge of column specimens A12 and B12 before, during and after burning at 600 °C.

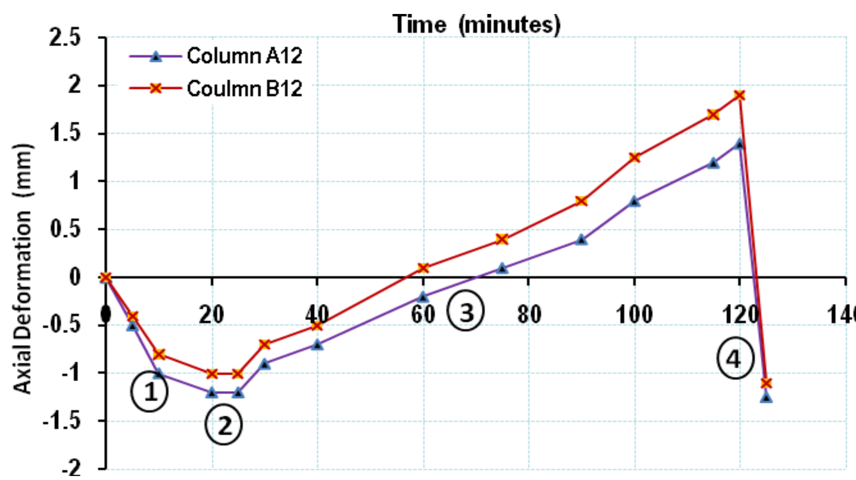


Figure (12): Axial deformation versus time curve at 50mm from the outer edge of column specimens A12 and B12 before, during and after burning at 750 °C.

COCLUSIONS

1. It was found that the ultimate load capacity of column specimens series A and B decreases significantly when subjected to burning by fire flame.
2. In this study, it is noticed that the load-midheight lateral deflection relation of column specimens exposed to fire flame temperature around 750^{o}C are flatter and reveals softer stiffness response than that of the control column specimens. This behavior can be attributed to the continual decrease in specimens stiffness with the increase cracking due to fire flame exposure.
3. The experimental results clearly indicate that the concentric column specimens exhibited a small contraction when loaded to 15% of the ultimate load during 25minutes then remain constant for the later period of this applied load, then the columns exhibit a sudden increase in contraction which was identified as failure after applied residual load.
4. It was found that the column specimens exposed to fire flame temperature (400, 600, and 750°C) also exhibited a small contraction when loaded to 15% of the ultimate

load during 25 minutes then remain constant for the later period of this load, thereon noticed a large elongation at the left and right edge of column specimens when exposed to burning, then a sudden contraction was observed when applying the residual load until 95% of ultimate load.

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