Experimental Study on Fire Flame Exposed of Reinforced Concrete Columns

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ABSRACT

The research deals with the effect fire flame on the behavior 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. It was found that the predicted load carrying capacity of reinforced concrete columns by three codes (ACI-318/08, BS-8110/97 and Canadian/84), was unconservative after burning except the BS Code equation which was found able to predict load capacity after exposure to high fire temperature levels.

الخلاصة

يتناول هذا البحث دراسة تأثير لهب النار المباشر على سلوك الأعمدة 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 النتائج العملية وجدت إن معادلات المدونات (الأمريكي، البريطاني، الكندي) غير قادرة للتنبؤ بمقاومة الحمل الأقصى لنماذج الأعمدة الخرسانية في درجة حرارة (400، 600، 750) درجة مئوية ماعدا معادلة الكود البريطاني وجدت قادرة للتنبؤ بمقاومة الحمل بعد التعرض إلى درجات حرارة عالية للهب النار المباشر ولكلا المجموعتين (30 و40) ميكاباسكال.

INTRODUCTION

Many researchers studied the effect of fire on concrete, reinforced concrete members is concentrated on exposing such members to high temperatures in special ovens. They worked on the strength and deformation properties at elevated temperatures. Such conditions do not represent the effects due to real fires, whereas, subjecting these members to direct fire flame is assumed to simulate the conditions happening in real fires. However, very little work was done on load carrying capacity of reinforced concrete columns exposed to direct fire flame.

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.

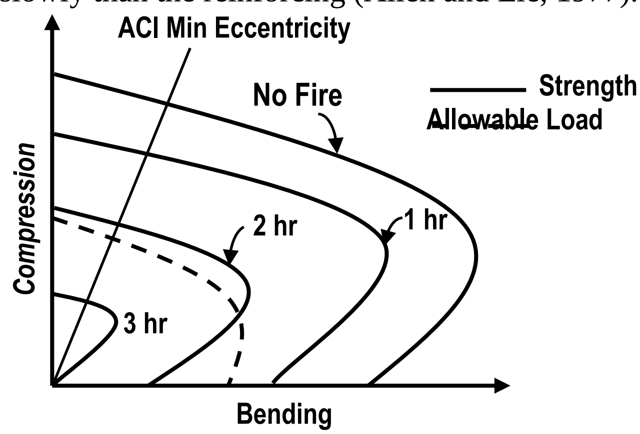
In order to simulate this problem to practical site conditions, reduced scale column models were cast and they were as close as possible to practical circumstances. This research is seeked to cover the limited area of research about this problem. This will guide and facilitate the suggestion of rehabilitation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

LITERATURE REVIEW

Geogali and Tsakiridis, 2005 made a case study of cracking in a concrete building subjected to fire, with particular emphasis on the depths to which cracks penetrate the concrete. It was found that the penetration depth is related to the temperature of the fire, and that generally the cracks extended quite deep into the concrete member. Major damage was confined to the surface near to the fire origin, but the nature of cracking and discoloration of the concrete pointed to the concrete around the reinforcement reaching 700°C. Cracks which extended more than 30mm into the depth of the structure were attributed to a short heating/cooling cycle due to the fire being extinguished.

-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	A	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			600 ºC	A10	30	0	No ties
12				A11	30	0	250
13				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				A20	30	30	No ties
32				A21	30	30	250

33			600 ºC	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	B	1	25 ºC	B10	40	0	No ties
62				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				B20	40	30	No ties

82		**2**	**25 ºC**	**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**
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)

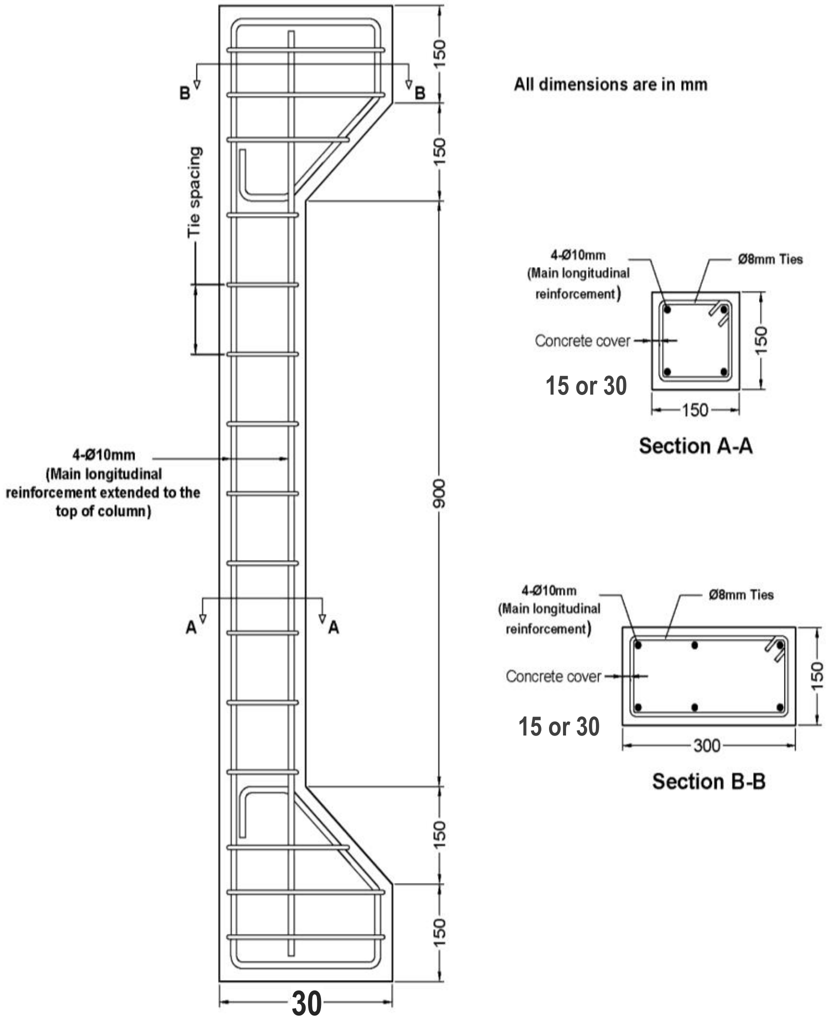


Figure (2): Reinforcement details of reinforced concrete column specimens.

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	$\geq$ 2300
Setting time, Vicat's method		
Initial hrs: min.	1:54	$\geq$ 0: 45
Final hrs: min.	4:25	$\leq$ 10: 00
Compressive strength of 70.7 mm cube, MPa		
3 days	22.5	$\geq$ 15
7 days	31.5	$\geq$ 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	$\leq$ 5.0
SO_3	2.42	$\leq$ 2.8
L.O.I.	2.27	$\leq$ 4.0
I.R.	0.95	$\leq$ 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	$\leq$ 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.

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 (3) shows the setup of axial deformation measurements.

For the column specimens which were subjected to fire flame under loading as shown in Plate (2). 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.

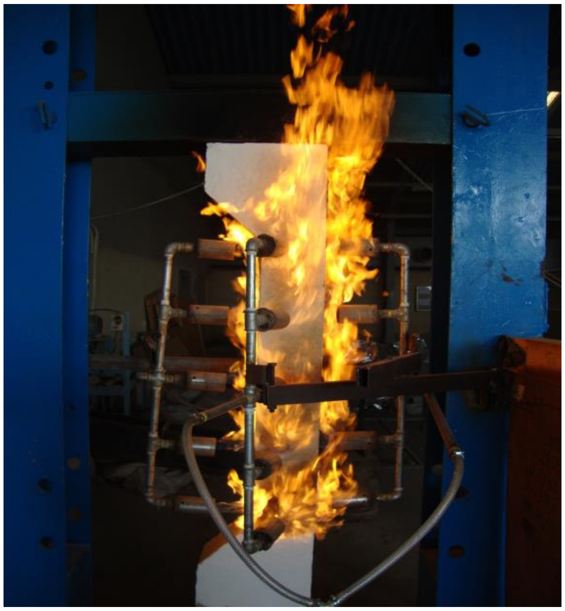


Plate (2): Testing of column specimens under 15% of ultimate load with exposure to fire flame.

Plate (3): The testing measurements of Axial deformation.

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

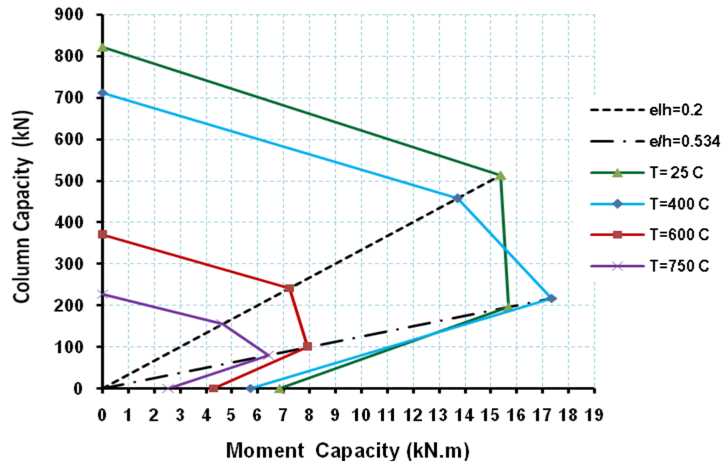


Figure (3): Effect of fire temperature on the interaction diagram of column specimens for series A and duration of burning 1.5 hour.

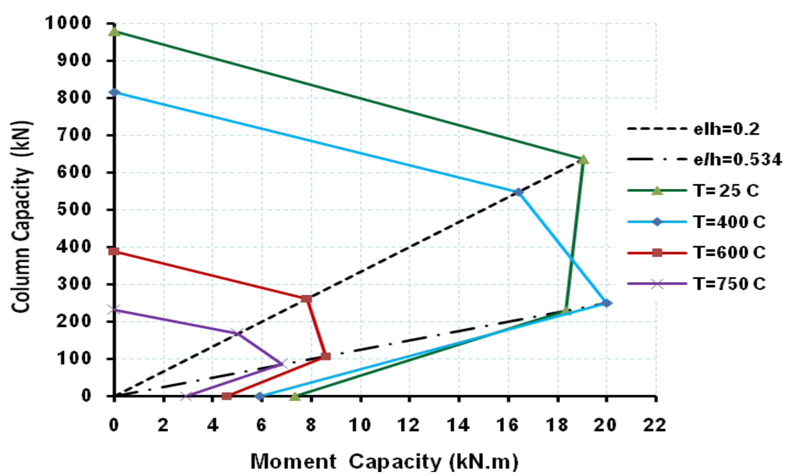


Figure (4): Effect of fire temperature on the interaction diagram of column specimens for series B and duration of burning 1.5 hour.

General Behavior and Verification of ACI-318/08 Building Code Provisions of Axially Loaded Column Specimens

During testing of column specimens to failure, it was noticed that the unconfined concrete column specimens (no lateral ties) exhibit a sudden explosive type of failure at ultimate axial loads after burning for 1.5 hour period of exposure, in the mean time the load carrying capacity dropped immediately.

The general behavior of the confined column specimens by lateral steel ties was more complicated than the unconfined specimens. The tests were completed for confined column specimens after loading to 15% of the ultimate load and exposed to fire for 1.5 hour. Then, the tests were continued until the load development becomes very slow compared with the large excessive deformations. A sharp drop in axial load capacity occurred when the longitudinal steel buckled at one or more locations.

For the unconfined column specimens (no lateral ties) a complete failure destruction occurs promptly by splitting failure with no adequate signs of approaching failure.

Several existing equations are available to predict the axial load capacity of reinforced concrete columns. These equations are selected and used in this study for comparison with the results of the experimental work. These equations are outlined in the Table (9).

Table (9): Summary of formulas for predicting axial load column capacity.

Method	Equation	EQ. NO.
ACI-318M-08 Code	$P_n = 0.85 f_c' \times A_n + f_y \times A_{st}$	**1**
B.S 8110-97 Code	$P_n = 0.4 f_{cu} \times A_n + 0.75 f_y \times A_{st}$	**2**
Canadian Code-1984	$P_n = 0.51 f_c' \times A_n + 0.85 f_y \times A_{st}$	**3**

Where :

A_n = Net concrete area $=A_g-A_{st}$, mm^2

A_{st} = Total area of longitudinal steel reinforcement, mm^2.

The test results were utilized to verify the recommendations and design simplifications of the various Building Codes pertaining to axial load capacity *(P_n)* design. Table (5-10) presents the comparison between the experimental results with (ACI, Canadian and B.S) Codes. To utilize these equations after exposure to fire flame temperatures the relative axial load capacity values **($P_{u\ test}/P_{n\ calculated}$)** were calculated for the column specimens of series A and B. The relationship between fire temperature with residual axial load capacity and ultimate load carrying capacity are illustrated in Figures (5) to (7).

At burning temperature (400°C), for the two series A and B column specimens, the ACI Building Code gave reasonable results to predict axial load capacity, while the (B.S and Canadian) codes gave very conservative values of column capacity. The ratios between the measured and (ACI, B.S and Canadian) predicted values were (1.07, 1.76 and 1.60%) and (1.07, 1.78 and 1.62%) for series A and B respectively.

At burning temperature (600°C), for the two series of column specimens, the ACI Code gave overestimated results to predict axial load capacity, while the B.S and Canadian Building codes gave close results to predict column capacity. The ratios between the measured and (ACI, B.S and

Canadian) predicted values were (0.71, 1.18 and 1.04%) and (0.63, 1.03 and 0.94%) for series A and B respectively.

At burning temperature (750°C), for the two series of column specimens, the ACI Building Code became unable to predict axial load capacity, while the B.S gave well predicted results whereas, the Canadian code gave overestimated results to predict column load capacity. The ratios between the measured and (ACI, B.S and Canadian) predicted values were (0.56, 0.89 and 0.80%) and (0.50, 0.82 and 0.74%) for series A and B respectively.

From the results, it is clear that the predicted ultimate axial load 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. While, B.S-8110 gave results lower than that obtained from experimental results at burning temperatures up to 750°C. The predicted ultimate axial load capacity obtained from Canadian Code provisions is lower than that obtained in the experimental work at burning temperature up to (600°C), while at burning above (600°C) the Canadian Code provisions slightly overestimate ultimate axial load capacity.

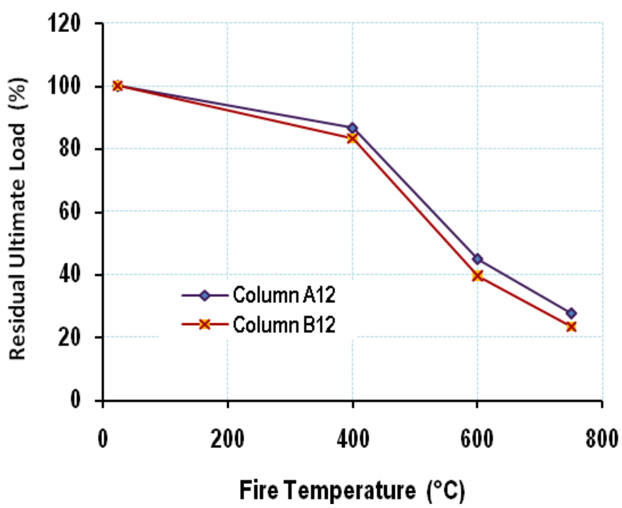


Figure (5): Effect of fire temperature on the residual axial load capacity of column specimens for series A and B.

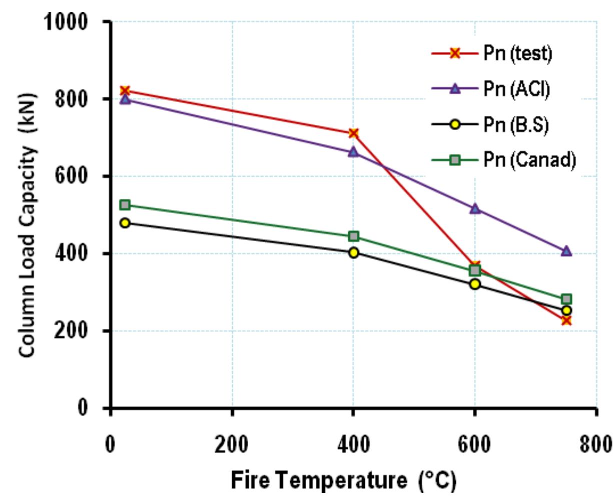


Figure (6): Effect of fire temperature on the axial load capacity of column specimens A_{12}.

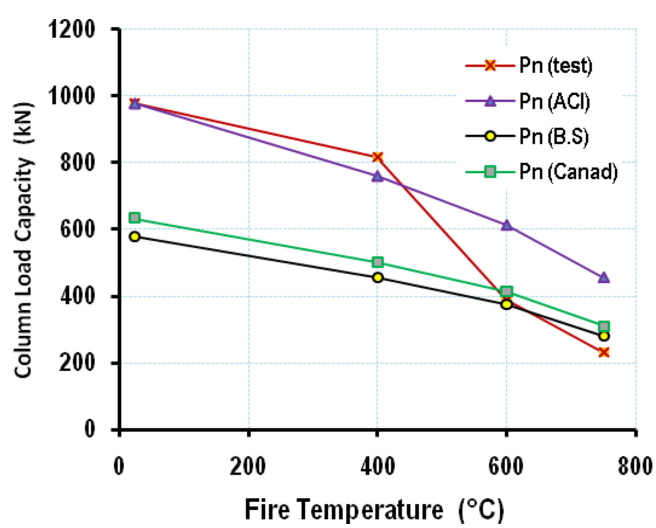


Figure (7): Effect of fire temperature on the axial load capacity of column specimens B_{12}.

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 (8) to (11). 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.

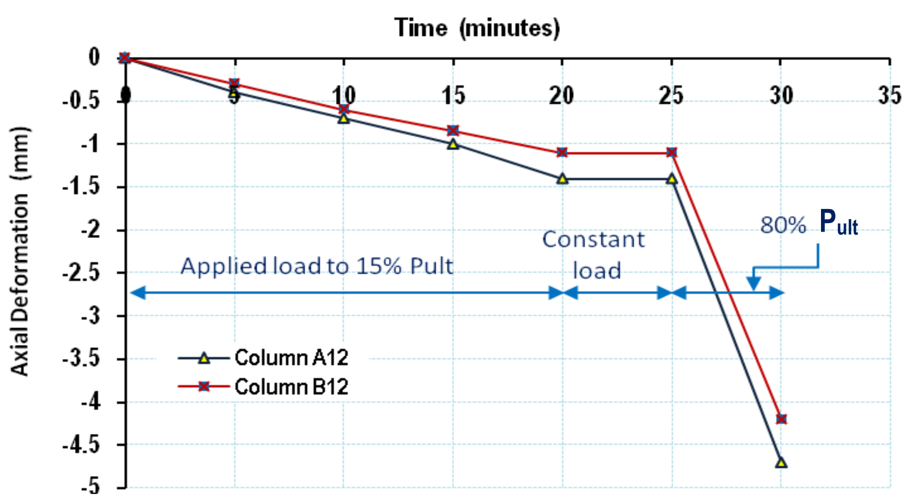


Figure (8): Axial deformation versus time curve at centerline of column specimens A12 and B12 without burning,25 °C.

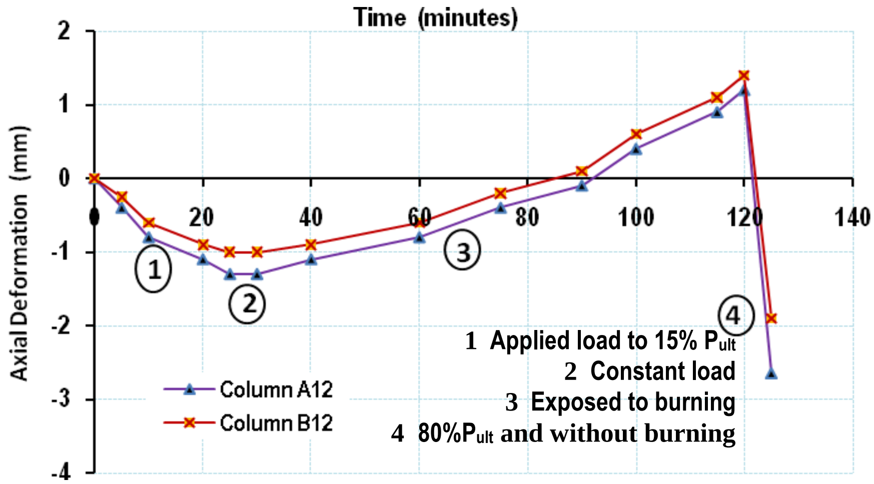


Figure (9): Axial deformation versus time curve at 50mm from the outer edge of column specimens A12 and B12 before, during and after burning at 400 °C.

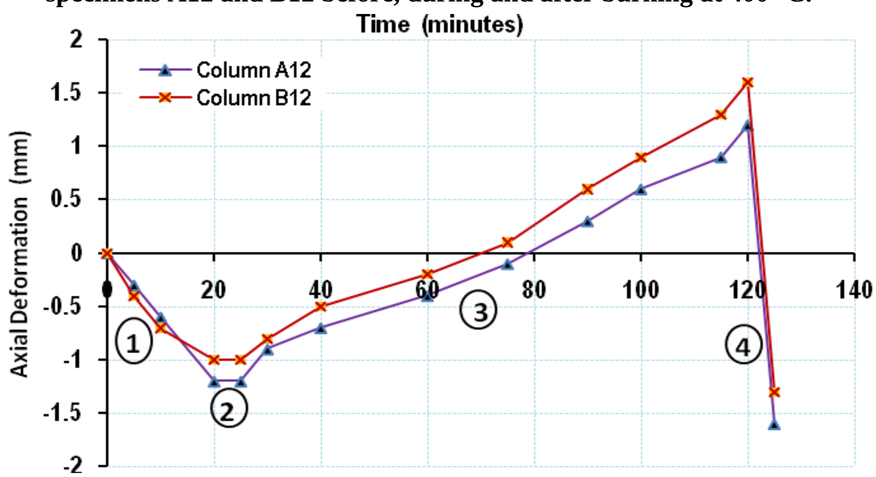


Figure (10): Axial deformation versus time curve at 50mm from the outer edge of column specimens A12 and B12 before, during and after burning at 600 °C.

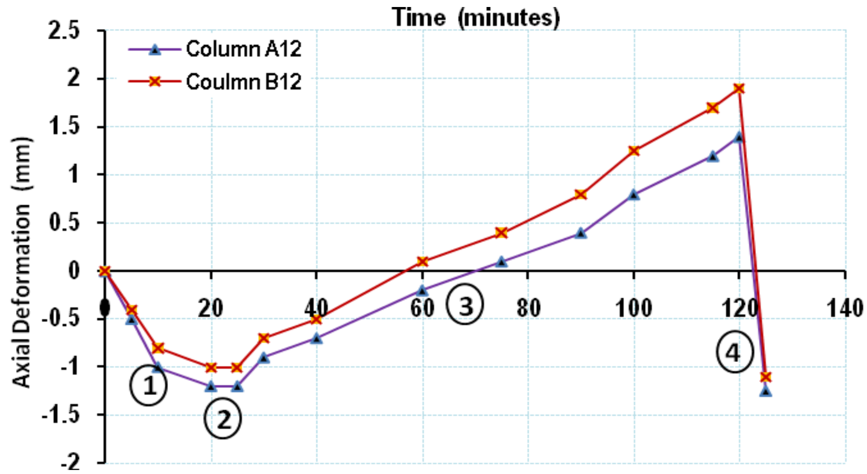


Figure (11): Axial deformation versus time curve at 50mm from the outer edge of column specimens A12 and B12 before, during and after burning at 750 °C.

Crack Pattern and Mode of Failure

The development of cracks and the time at which they appeared and propagated in the reinforced concrete column specimens were detected throughout testing to assess the behavior of the column specimens exposed to fire flame and the control column specimens. The cracks were marked with a blue marking pen, then photographs were taken to the crack pattern. When the load was increased, cracks appeared on the columns loaded at eccentricity 30 and 80mm on the surface from the tension zone towards the compression zone. Further, flexural cracks were formed progressively and widened as the loading increased. However some of short nearly vertical, hairline cracks were detected on the middle third of the columns. For concentric column specimens nearly vertical, hairline cracks appeared at the middle portion of columns. More cracks (mostly vertical) continued to appear on the column faces. Scabbing occurred prior to the column failure due to the crushing of the concrete and subsequent buckling of the main reinforcement at later stage.

Types of failure combined flexural and compression failure for eccentric loaded column specimens and compression failure for concentric loaded column specimens. The columns burned at 400^{o}C, the type of failure for concentric and eccentric loaded specimens stayed without changes. For columns burned at 600 and 750^{o}C, the type of failure also remained constant but scabbing in the concrete cover occurred. This can be attributed to the vapor pressure of the runoff water which exerts internal pressure stresses on the surface layers of concrete which are unconfined by the tie reinforcement resulting in scabbing of these layers. Also, the cracking appeared earlier when the fire flame temperature increased. The typical crack pattern of reinforced concrete columns before and after exposure to fire flame are shown in Plates (4) and (5).

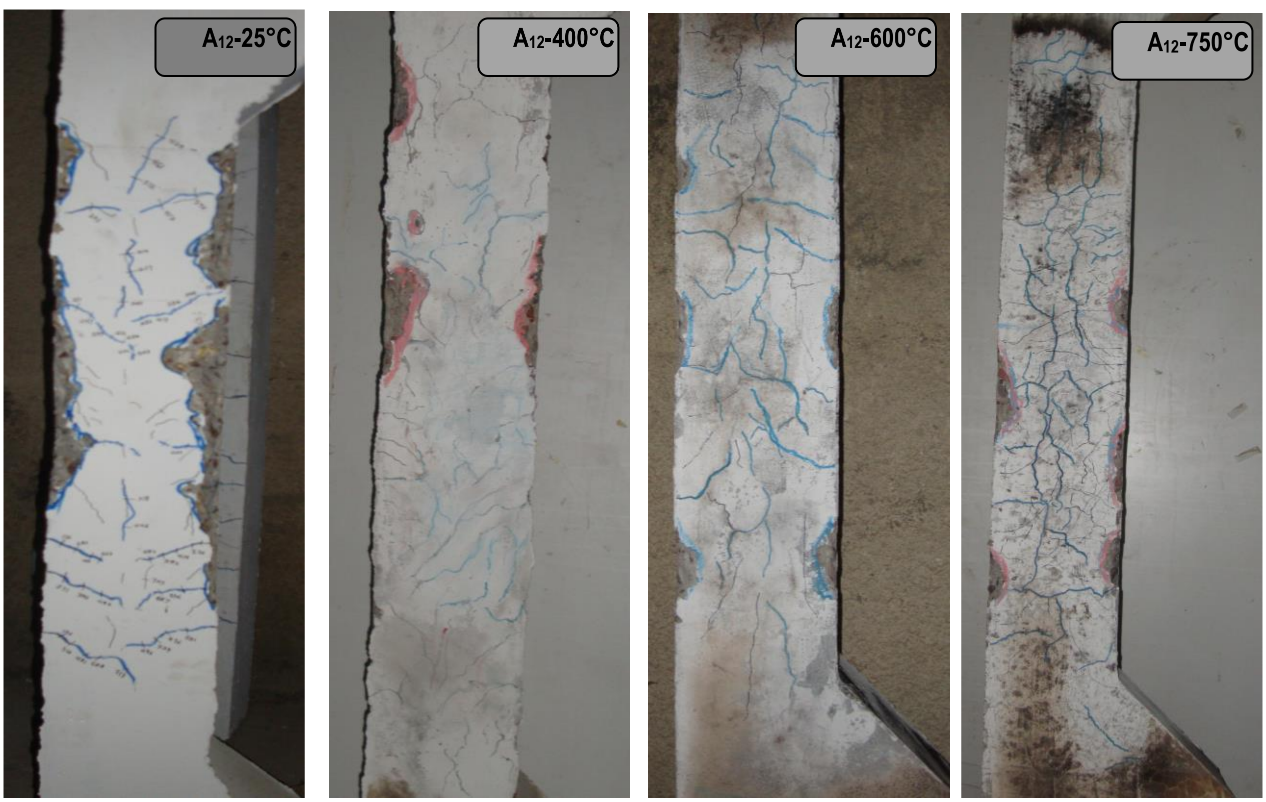


Plate (4): Typical crack pattern of series A column specimens before and after burning and subjected to concentric loading.

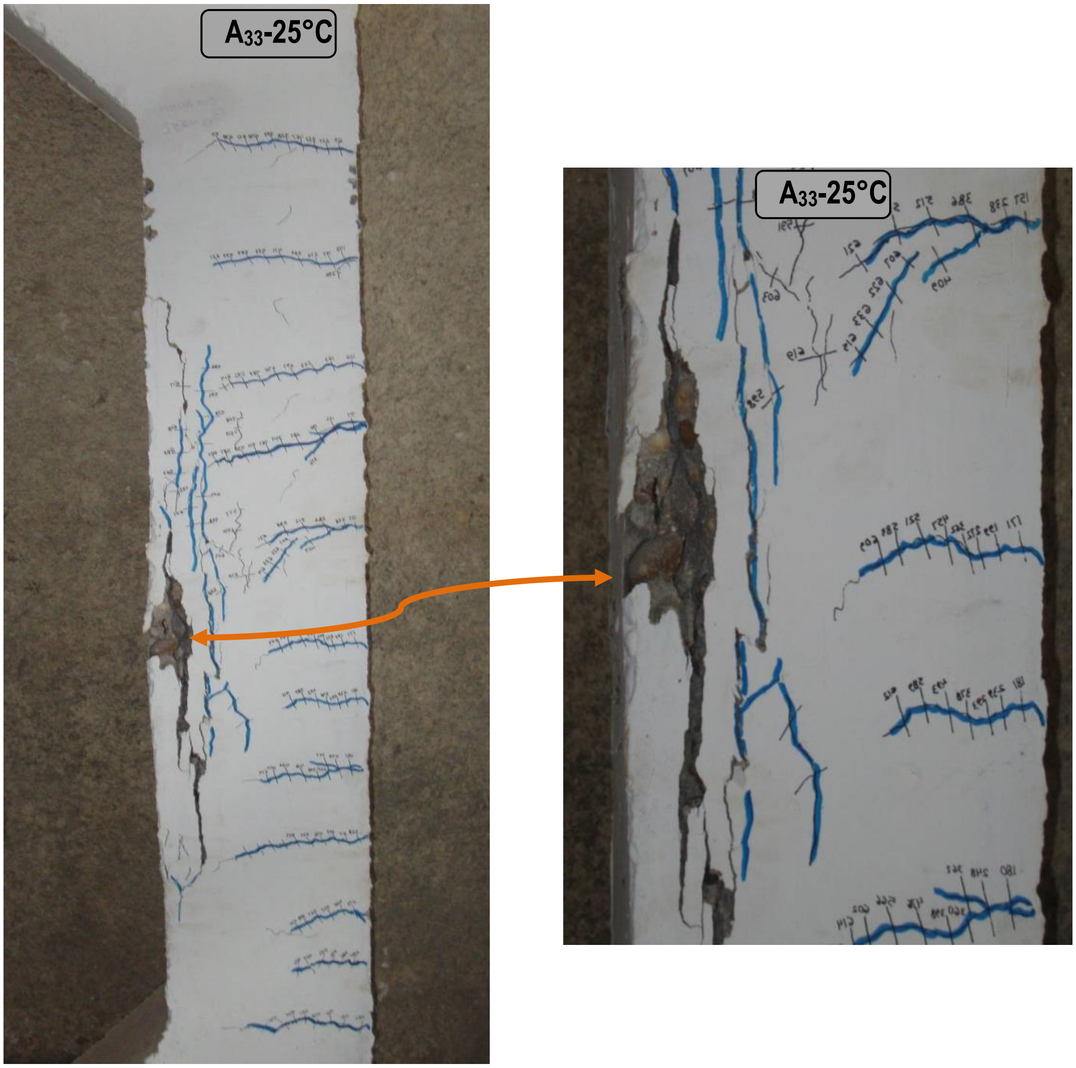


Plate (5): Typical crack pattern of series A column specimens with different tie spacing and subjected to eccentric loading (e=80mm) at failure.

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. The Canadian and B.S Codes predict ultimate load carrying capacity after exposure to (600 and 750 °C) fire flame temperature conservatively.
3. ACI Code gave conservative results to predict ultimate load carrying capacity after exposure to 400 °C fire flame temperature.
4. The experimental results clearly indicate that the crack width in reinforced concrete columns that are subjected to fire flame are higher than the columns that are not burned at identical loads.
5. It was found that the maximum crack width increases with increasing fire temperature and decreasing amount of transverse steel reinforcement for the column specimens.
6. It was noticed that the value of longitudinal crack width is less than flexure transverse crack width for columns with or without burning.

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