BEHAVIOUR OF FIRE EXPOSED REINFORCED CONCRETE COLUMNS

Assist. Prof. Dr. Nada M. Fawzi
University of Baghdad

Prof. Dr. Mahdi S. Essa
University of Babylon

Dr. Mohammed M. Kadhum
University of Babylon

ABSTRACT

This research is devoted to investigate the behaviour and load carrying capacity of reinforced concrete columns exposed to fire flame.

The experimental program consisted of casting and testing of 128 column specimens divided into two series A and B with target compressive strength (30 and 40 MPa) and named series A and B respectively. Each series was divided into three main groups loaded eccentrically with eccentricities 30mm and 80mm.

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. The BS Code equation was found to predict load capacity after exposure to high fire temperature levels better than other codes.

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.

تصرف الأعمدة الخرسانية المسلحة المعرضة للنار

الخلاصة

إن الغرض من هذا البحث هو التحري عن سلوك وسعة التحمل القصوى للأعمدة الخرسانية المسلحة والمعرضة إلى لهب النار المباشر. تضمن الجزء العملي تهيئة وفحص 128 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.

النتائج العملية أعطت مؤشرا بأن 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.

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

Concrete columns are considered to be an 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).

High temperatures due to fire have a significant effect on the strength and deformation characteristics of various structural components, such as columns, beams, slabs, shear walls, etc. Therefore, a good understanding of the structural behavior and response of reinforced concrete exposed to fire is important towards the saving of human lives and avoiding costly damage of structures (Book Ng et al.,1990). Human safety is one of the considerations in the design of residential, public and industrial buildings**.**

RESEARCH SIGIFICANCE

There are indeed little research about temperature gradient and exposure time of concrete in direct contact with fire flames.
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 reinforced concrete columns are subjected to during fire in laboratory.

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

LITERATURE REVIEW

-The Effect of Fire on Reinforced Concrete Columns

Kodur et al., 2005 investigated the behaviour of fibre reinforced polymer (FRP) wrapped (confined) reinforced concrete columns under exposure to 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 providing proper fire insulation (5 and 4) hours fire endurance rating can be achieved for loaded circular and square reinforced concrete columns strengthened with FRP wraps respectively.

Wu and Li, 2008 studied the behavior of four concentrically loaded L-shaped reinforced concrete columns subjected to ISO834 fire. All columns have a nominal height of 2340mm and only the center portion of 1650mm was exposed to fire on all sides. Each column was reinforced with 12Ø10mm as the longitudinal reinforcement, and Ø6mm were used for ties, which are spaced at 100mm within the central fire exposed portion. They investigated the effect of axial restraint on columns during both expanding and contracting phases. The fire tests, which consist of four combinations of two levels of axial load with two degrees of axial restraint, were conducted at the fire laboratory of South China University of Technology. The boundary conditions of the columns were considered as fixed-fixed for all tests. Moreover, the finite element program, SAFIR, was employed to conduct a numerical analysis of the tests. The authors concluded that columns subjected to same load ratio, the axial restraint ratio seems to have little effect on the development of column axial load during contracting and cooling phases. They also found that the maximum additional axial forces induced in axially restrained columns with axial restraint ratios of 0.0578 and 0.0875 are, respectively, around 34% and 53% of the design load for the load ratio of 0.25. For the load ratio of 0.35, the additional forces were 26% and 30% of the design load.

-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).

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 (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.

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).

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).

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).

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).

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).

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 positions of the visual cracks in the concrete and the loads at which these cracks were formed, were recorded. 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.

For the column specimens which were subjected to fire flame under loading as shown in Plate (1). 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 resulted from applied load only.

RESULTS AND DISCUSSION

Comparison Between Residual Compressive Strength Results and Recommended Design Curves (CEB & CEN)

In the present study some of the test results of residual compressive strength were found to lie between CEB and CEN curves, while other results diverged from the CEB curve and converged to the CEN curve especially at 400˚C of fire exposure. At 600˚C and 750˚C, the test results were found to be near the CEN curve only. From the figure (3), it can be concluded that the test results of the current study have better agreement with CEN design curve than with CEB curve.

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 (4) to (7).

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).

General Behavior and Verification of Building Code Provisions of Axially Loaded Column Specimens

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 (8).
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.

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.

COCLUSIONS

1. For the studied temperature range in this study, the compressive strength-reduction curve, recommended by the Euro codes CEN (1993, 1994) is in better agreement with the test results rather than CEB (1991) strength-reduction curve.
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 Canadian/84 and B.S-8110/97 Codes predict ultimate load carrying capacity after exposure to (600 and 750 ^{o}C) fire flame temperature conservatively.
4. ACI Code give conservative results to predict ultimate load carrying capacity after exposure to 400^{o}C fire flame temperature.

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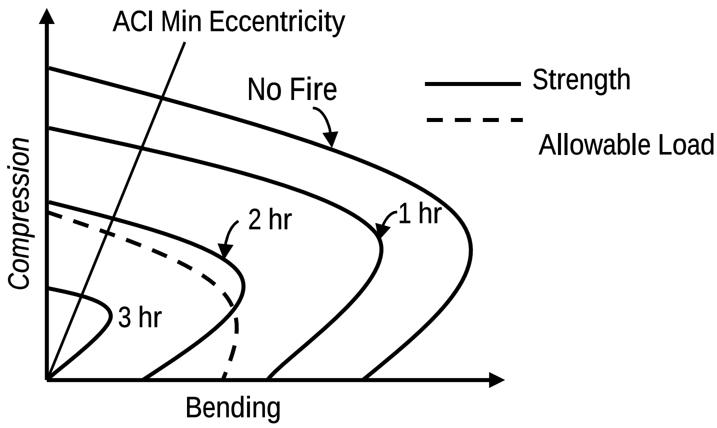

Figure(1): Typical column interaction curves (*Allen and Lie, 1977*).

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			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	A	1		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			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				A20	30	30	No ties

37			750 ºC	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			25 ºC	B10	40	0	No ties
62				B11	40	0	250
63				B12	40	0	150
64				B13	40	0	50

65	B	1		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				B20	40	30	No ties

92			600 ºC	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)

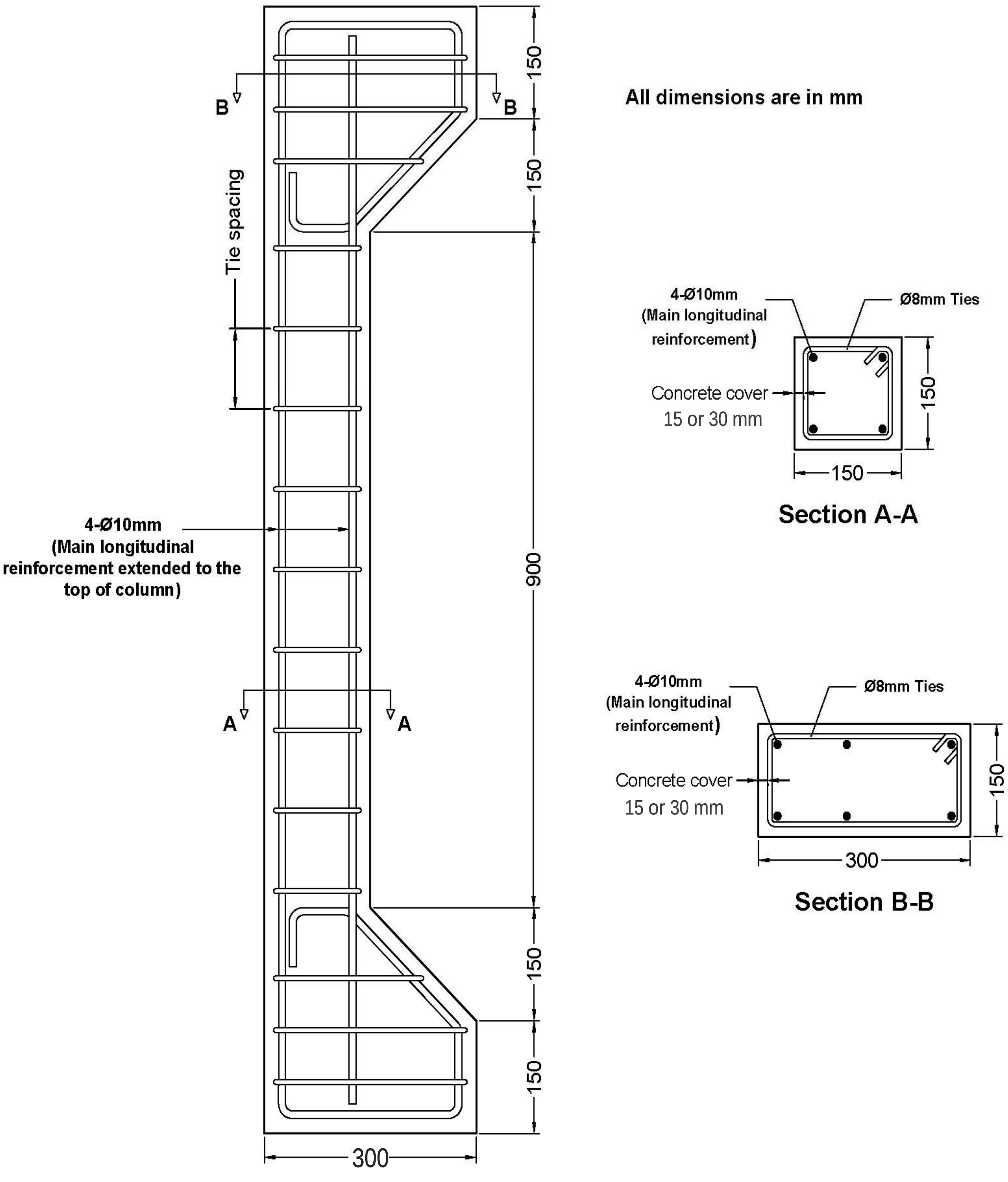

Figure (2): Reinforcement details of reinforced concrete column specimens.

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

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	-----

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	-----

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

Approximate Diameter (mm)	Measured Diameter (mm)	Area (mm²)	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

*Testing of steel bars was carried out in Strength of Materials laboratory at the College of Materials Engineering / University of Babylon.

** Assumed

Table (7): Mix Proportions.

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

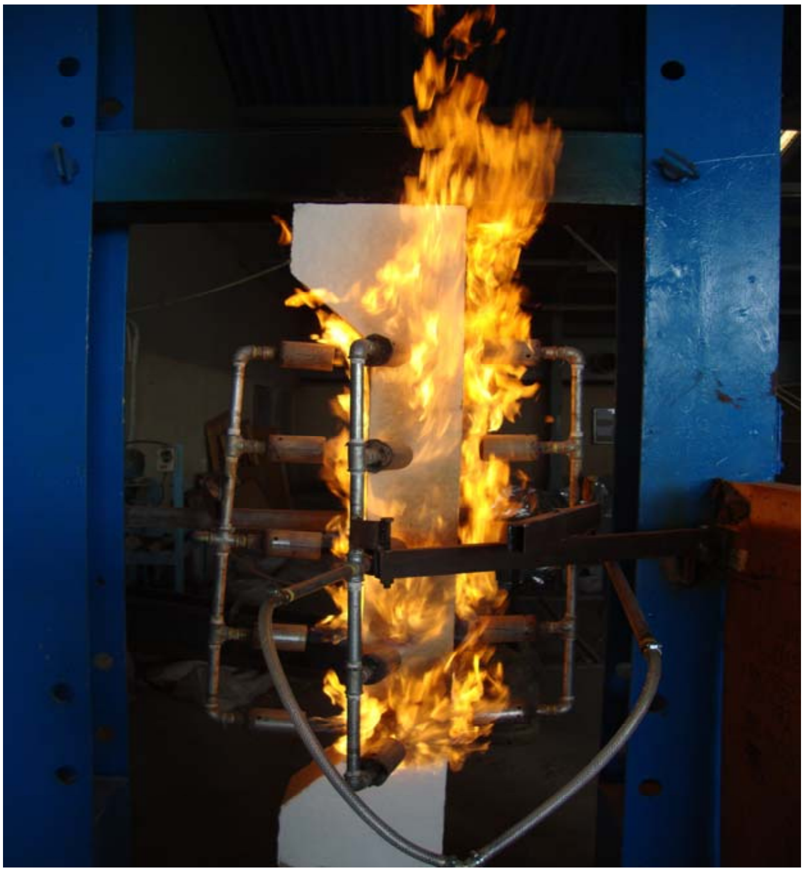

Plate (1): Testing of column specimens under 15% of ultimate load with exposure to fire flame.

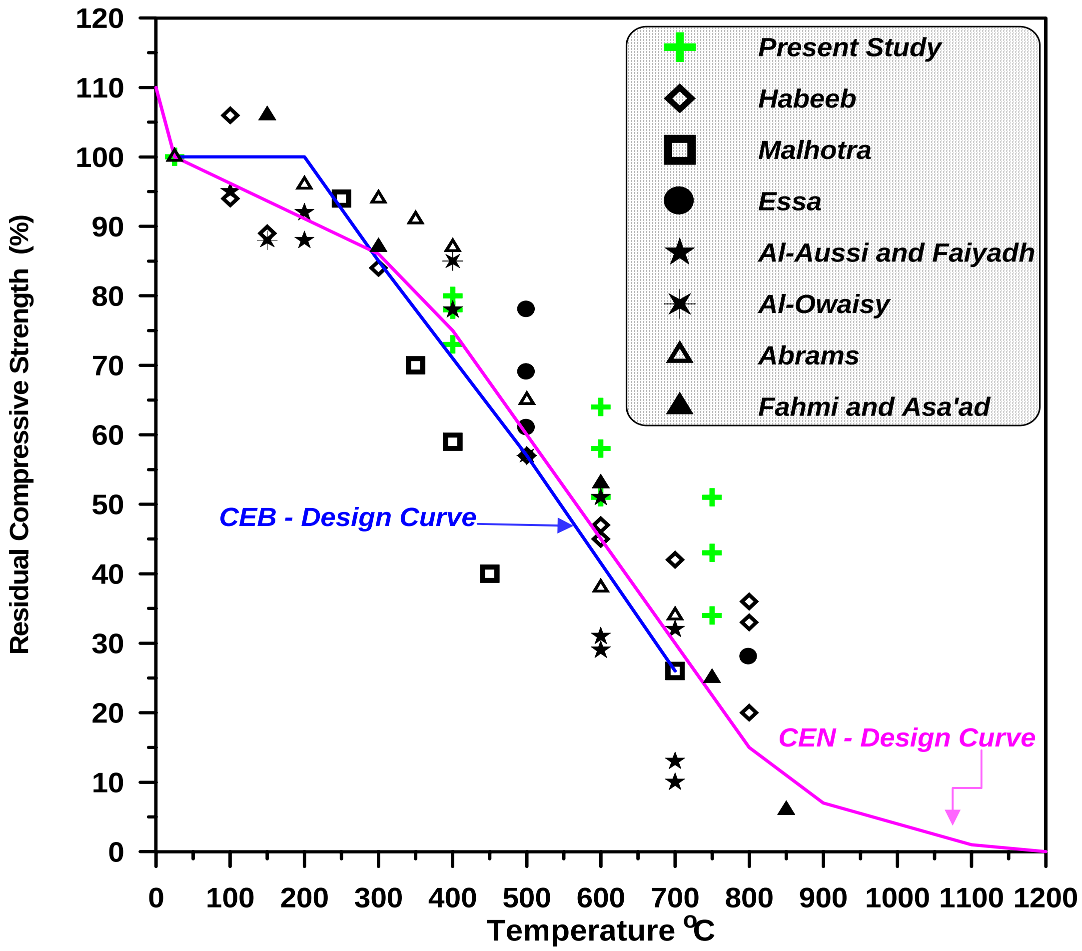

Figure (3): Comparison of residual compressive strength results and the recommended design curves of CEB and CEN.

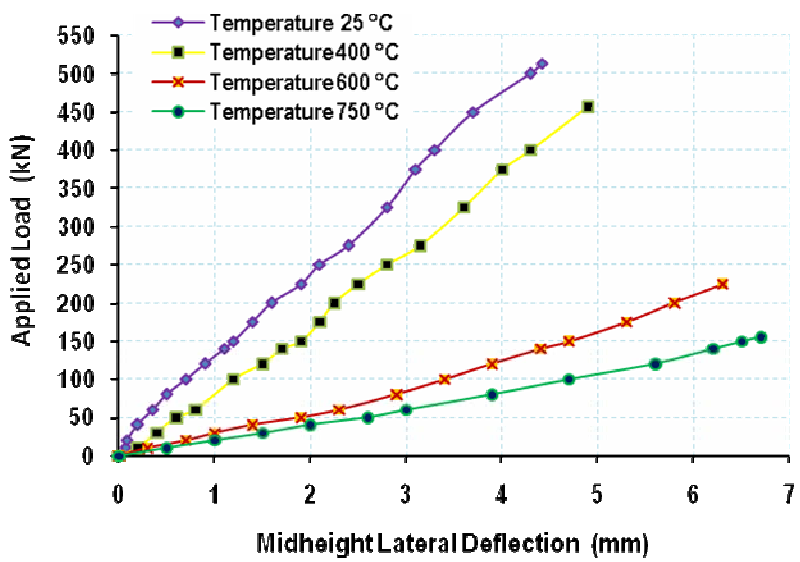

Figure (4): Load versus midheight lateral deflection curve of column specimen A22 at eccentricity (e=30mm).

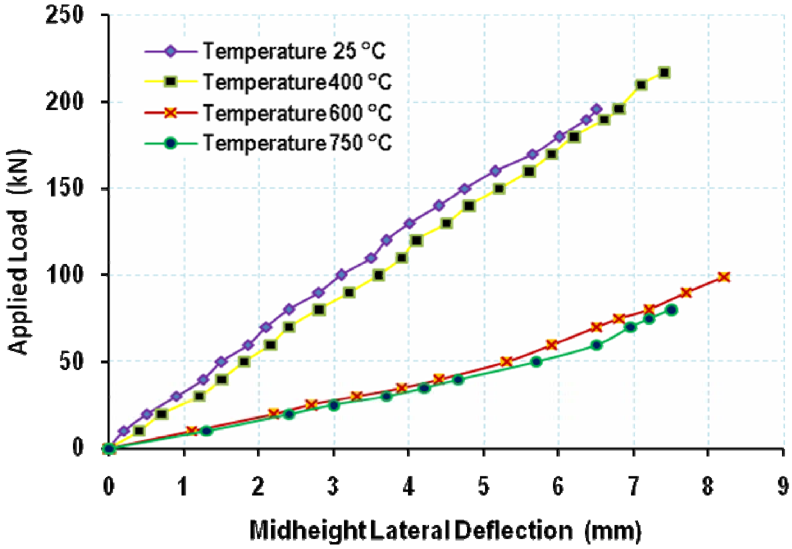

Figure (5): Load versus midheight lateral deflection curve of column specimen A_{32} at eccentricity (e=80mm).

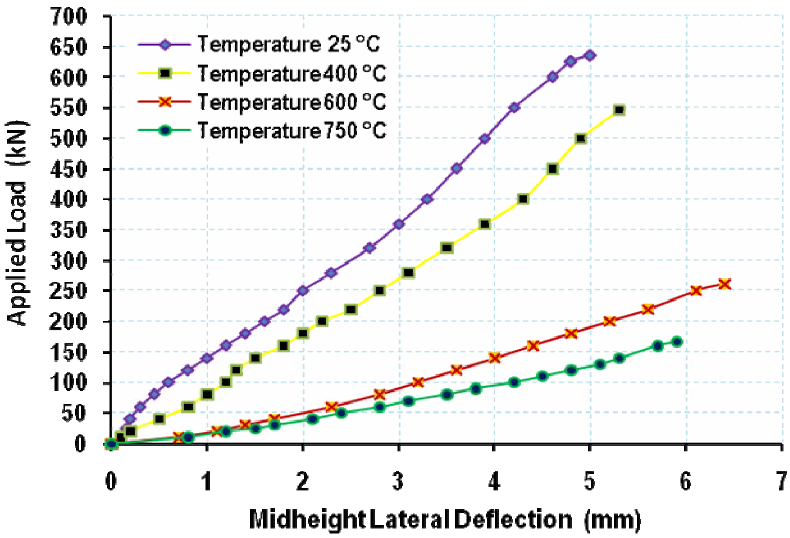

Figure (6): Load versus midheight lateral deflection curve of column specimen B_{22} at eccentricity (e=30mm).

Table (8): 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 :

An = Net concrete area =Ag-Ast, mm2

Ast = Total area of longitudinal steel reinforcement, mm2.

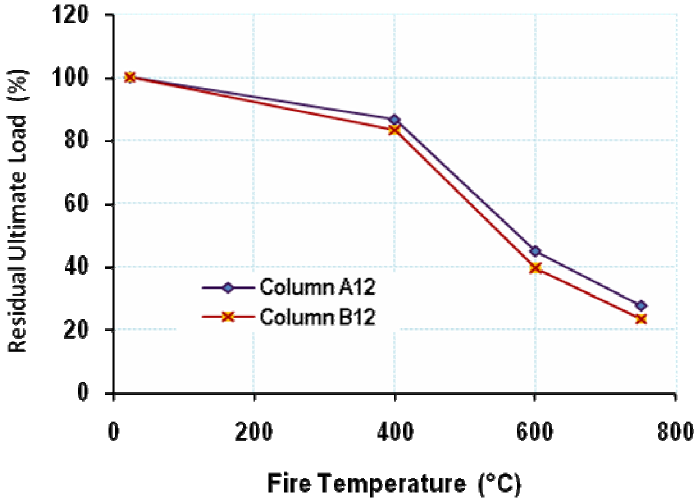

Figure (8): Effect of fire temperature on the residual axial load capacity of column specimens for series A and B.

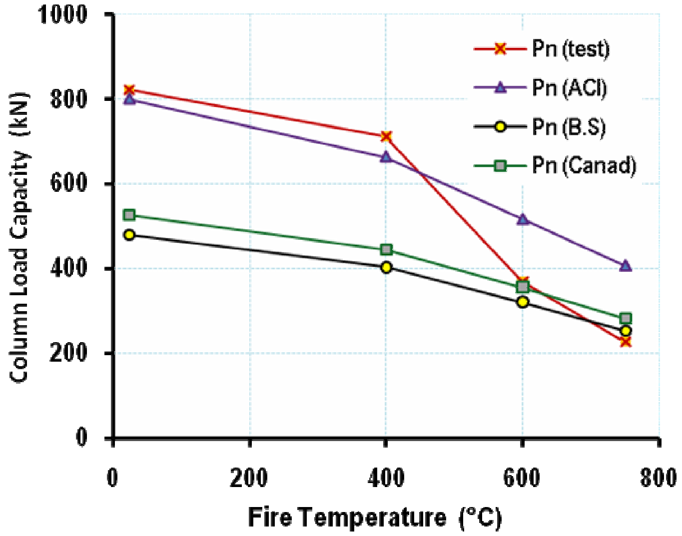

Figure (9): Effect of fire temperature on the axial load capacity of column specimens A12.

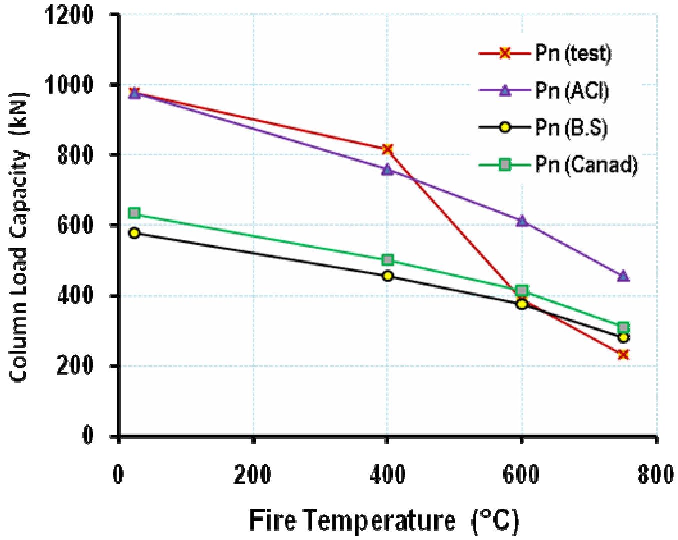

Figure (10): Effect of fire temperature on the axial load capacity of column specimens B_{12}.