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ORIGINAL ARTICLE

Behavior of reinforced concrete short columns exposed to fire

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Abstract Fire could dramatically reduce strength of reinforced concrete columns. The objective of this work is to study columns exposed to fire under axial load and to evaluate reduction in column compressive capacity after fire. The first part of this research is experimental investigation of fifteen-column specimens (15 × 15 × 100) cm exposed except one specimen to (600 °C) fire. The second part is a theoretical analysis performed using three-dimensional nonlinear finite element program. The main studied parameters were concrete strength, fire duration, level of applied loads, longitudinal reinforcement yield strength, percentage of longitudinal reinforcement, and bar diameters.

Comparison between experimental results and theoretical analysis indicated that for columns not exposed to fire, the first crack appeared at 80% of column failure load while the first crack occurred at 50% of column failure load for columns exposed to fire. Columns with the same reinforcement percentage but with smaller bar diameters gained less lateral strain and smaller vertical displacement than columns with bigger bar diameters. Using high-grade steel as main reinforcement showed failure load higher by 55% than that of column reinforced by mild steel. Cooling column by jet water resulted in 17% reduction in failure load than columns cooling gradually in room temperature.

1. Introduction

Reinforced concrete is the common material used in structural system in Egypt and all-over the world. Thus, the behavior of these structures and their failure modes are extensively studied. The degradation of concrete strength due to short-term exposure to elevated temperature (fire) has attracted attention in the last decades. The behavior of concrete exposed to fire depends on its mix composition and determined by complex interactions during heating process. The modes of concrete failure under fire exposure vary according to the nature of fire, loading system, and types of structure. Moreover, the failure could happen due to different reasons such as a reduction of

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bending or tensile strength, loss of shear or torsional strength, loss of compressive strength, and more.

In the past decade, several experimental and theoretical studies have been carried out on the degradation of column concrete strength due to the short term exposure to fire [1–7]. These studies of columns exposed to fire have indicated the following observations:

i. Surface cracking in concrete occurs at nearly 300 °C with a deeper cracking at 540 °C. Spalling occurs followed by breaking off thin concrete cover at corner and edges.
ii. Concrete begins to lose about 30% of its compressive strength when heated up to 300 °C and loses about 70% of its compressive strength when heated up to 600 °C.
iii. Concrete modulus of elasticity reaches 60% of its original value at 300 °C and reaches 15% of its original value at 600 °C.
iv. Concrete stiffness decreases with the increase in temperature and the reduction in stiffness is accompanied with a reduction in the concrete strength with the increase in the concrete strains.
v. Vertical cracks clearly appear and then crushing of concrete accompanied by crackle sound with a local buckling of the longitudinal reinforcement occurs.
vi. Columns with large longitudinal bars diameters lead to fire resistance appreciably smaller than columns with smaller bar diameters and the increase in concrete cover has a positive effect on the columns fire resistance.

Studies showed that there is an excellent correlation between column models and prototype (similar modes of failure and cracking patterns), which means that there is no need to study true effect on a full-scale model, and time scale factor for models and prototype can be used [3,4].

Design building codes require some provisions for structural fire-resistance to ensure building integrity for a certain period under fire conditions. Such provisions allow safe evacuation of occupants and access for firefighters. Egyptian design building code ECCS-2007 [8], recommend minimum column dimension not less than 25 cm. The minimum thickness of the concrete covers varied between 25 and 35 mm according to both ECCS-2007 [8] and ACI building code [9] for fire resistance periods (1.0–3.0) hours to protect the main longitudinal reinforcement. However, the behavior of buildings after fire, whether it is worthy to repair it or not, is another point of interest that needs more investigation.

This research is aimed at investigating the effect of fire on the behavior of axially loaded reinforced concrete columns subjected to fire and to estimate the percentage loss of column compressive strength under the effect of the following parameters:

1. Concrete characteristic strength.
2. Fire duration.
3. Applied loads on columns during fire.
4. Diameters of the main steel reinforcement.
5. Percentage of the main steel reinforcement.
6. Cooling manner of column after exposed to fire.
7. Grade of the longitudinal steel reinforcement.

To achieve these objectives, mathematical models in conjunction with laboratory experiments are used to simulate the behavior of columns exposed to fire. Analysis was performed to examine the influence of these different parameters on the column strength. The results of these analyses are presented and discussed hereinafter.

2. Experimental investigation

The experimental program consisted of fifteen reinforced concrete column specimens (C_1 to C_{15}) having $(15 \times 15 \times 100)$ cm in cross section [representing one third scale model] with 0.6% percentage of stirrups (∅6 mm @ 10 cm). All specimens except the reference column C_1 were subjected to 600 °C constant temperature fire and were divided into seven groups as follows:

Group 1: consisted of four specimens (C_3, C_4, C_5, C_{14}), to study the effect of concrete characteristic strength.
Group 2: consisted of four specimens (C_1, C_{10}, C_{12}, C_{13}), the aim of this group is to study the effect of fire period.
Group 3: consisted of three specimens (C_{11}, C_{13}, C_{15}), to study the effect of the applied load on column during fire.
Group 4: consisted of two specimens (C_7, C_8), to study the effect of different grade of the main longitudinal reinforcement.
Group 5: consisted of two specimens (C_2, C_9), to study the effect of the diameters of the main longitudinal reinforcement.
Group 6: consisted of two specimens (C_{13}, C_6), to study the effect of column cooling manner after exposed to fire.
Group 7: consisted of three specimens (C_7, C_9, C_{14}), to study the effect of the percentage of the main longitudinal reinforcement.

Table 1 summarizes the tested specimens for different groups.

2.1. Concrete mix design

Four mix designs for one-meter cube of concrete representing four series were used in this study to classify a various concrete characteristic strengths and the mixes were used in manufacturing of column specimens. For each mix, six cubes were cast and tested under compression to evaluate the target strength of the mix. The average strength values for each mix and corresponding concrete strength are as shown in Table 2.

2.2. Testing procedures

All column specimens, except the reference one, were exposed to fire first, and then tested under compression until failure. The furnace is a steel structure made of metals and consist of three main parts, loading frame, firing cage, and isolation caps as shown in Fig. 1, the column placed in the furnace manually by using a lever crane.

Columns were exposed to a 600 °C constant fire temperature at Building Research Center laboratory. After exposed to fire, columns were tested using a hydraulic loading machine of 500-ton capacity and 0.5-ton accuracy at Concrete research laboratory – Cairo University. The load was

Table 1 Parameters of Columns Testing Specimens.

Group No.	Parameters	Column No.				Fire period, min.	Loads on column at fire	% Of main reinf.
1	Concrete strength (kg/cm^2)	C_3 375	C_4 425	C_5 500	C_{14} 300	15	10–16 (ton)	1.40%
2	Fire period (minute)	C_1 0	C_{10} 10	C_{12} 15	C_{13} 20	0:20	10 (ton)	1.40%
3	Applied loads on column	C_{11} 20 ton	C_{13} 10 ton	C_{15} 15 ton		20	10–20 (ton)	1.40%
4	Grade of main steel	C_7 St 24/35		C_8 St 36/52		15	10 (ton)	1.40%
5	Diameters of main steel	C_2 5∅ 12 + 3∅10		C_9 4∅16		15	10 (ton)	3.1%, 3.6%
6	Cooling manner	C_{13} Room temp.		C_6 Water jet		20	10 (ton)	1.40%
7	Percentage of main steel	C_7 2.0%	C_9 3.6%	C_{14} 1.40%		15	10 (ton)	1.40%: 3.6%

Table 2 Concrete mix design for 1 m^3 concrete.

Material	First mix	Second mix	Third mix	Fourth mix
Fine aggregate kg	630 kg	620 kg	600 kg	580 kg
Coarse aggregate kg	1180 kg	1150 kg	1120 kg	1080 kg
Cement kg	300 kg	350 kg	450 kg	500 kg
Water lit	160 Lit.	160 Lit.	165 Lit.	165 Lit.
Admixture (addicreate B_2)	6 kg	7 kg	9 kg	10 kg
Average concrete cube strength kg/cm^2	300	375	425	500

Figure 1 Loading frame, firing cage and column during test.

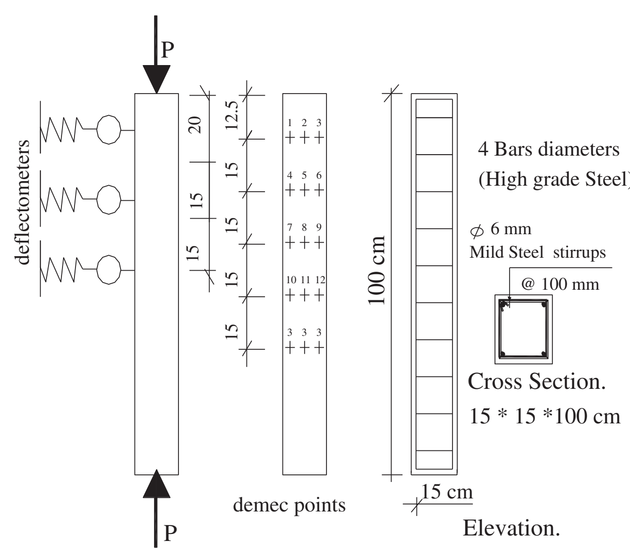

Figure 2 Details of tested columns.

controlled manually through testing machine and the strains were measured using a mechanical strain gage every 5.0-ton intervals. The axial and the lateral deformation of the specimens were recorded using deflectometers dial gages. The positions of demec points for strain reading and dial gages for axial and lateral deformations were arranged as shown in Fig. 2. The top and bottom ends of column specimens were confined with bolted steel boxes made from 10 mm thickness steel plates as shown in Fig. 1. For the used furnace, the time–temperature relation was determined and compared with results of standard time-fire test.

2.3. Behavior of specimens during fire

All tested specimens (C_2 to C_{15}) were exposed to fixed 600 °C fire temperature except the reference column (C_1) as mentioned before and it was observed that:

- For all the tested specimens after nearly 30% of fire period, the concrete cover began to crack and parts of it started to spall of with noticeable cracking sound. The hydraulic jack readings as the load is applied to the column during fire indicated an increase in value gradually up to 25% high with increasing fire period. This may be due to expansion of column due to fire temperature.
- After column is completely exposed to fire period, the concrete cover showed random cracks at column surface.
- Column color immediately after fire was close to red, and then after cooling, the column color began to change to gray and then to black. These observations on columns are compatible with conclusions of some tests by Gernay and Dimla [10] as mentioned before.

2.4. Behavior of specimens during test

Fig. 3(a–c) shows cracking patterns for tested specimens, also the failure loads for the entire tested columns are as shown in Table 3, from which the following observations are made:

- Columns not exposed to fire (reference case column C_1) showed that the first crack appears at load level nearly 80% of column failure load.
- Column exposed to fire showed that the first crack started at load level about 50% of column failure load. This is due to the decrease in column stiffness because of fire and hair cracks appearing on surface.
- Cracking patterns of tested columns were generally vertical cracks, with sometimes-slight inclination.
- Columns' ultimate axial deformations increased while columns' ultimate failure load decreased after the columns are exposed to fire. This could be attributed to the relative reduction in stiffness of columns when exposed to fire with respect to columns not exposed to fire.
- Generally, the position of maximum column lateral deformations along column height occurred at the mid- height of column.
- As fire progressed, the vertical cracks widened, then crushing of concrete cover occurred accompanied with a large explosion with a local buckling of the longitudinal reinforcement as shown in column C_5 (Fig. 3c).

3. Finite element analysis

A non-linear finite element package ASSEMBLY by Emara [11] was used to investigate the behavior of the tested reinforced concrete columns exposed to fire. The program is used to simulate lab experimental results. A comparison between experimental results and numerical ones is presented hereinafter.

The fifteen column specimens were analyzed under equal static incremental loads from zero loads up to failure. The increment of loads was set to 2.0 ton. Since the outputs of the numerical program are huge data, a computer program is used to rearrange the output and to analyze the output results. The program was used to determine the values of maximum load, vertical, and lateral displacements and the maximum lateral strain for all the analyzed columns.

Each column specimens was divided into finite element model. These elements were eight nodded twenty-four degrees of freedom solid (brick) elements used to simulate the concrete, two nodes vertical truss element to simulate longitudinal reinforcement, and two nodes horizontal truss elements to simulate stirrups. The steel stress–strain relationship was modeled as a tri-linear with a perfect bond or bond slip taken into consideration as shown in Figs. 4a, 4b. Boundary conditions for the numerical analysis were chosen as a free translation, and restrained rotations in all joints to simulate the test specimens in the experimental condition, except for the base joints, which were fully restrained in both rotations and translation.

The column load is applied in increments equal to (2.0 t) with initial load equals to zero for all column specimens. The mechanical properties of concrete and steel, measured from the experimental work, were implemented as an input data in the finite element program.

Unfortunately, the used finite element program cannot take into consideration the variation of fire temperature gradient through column cross-section, fire duration, and fire temperature. Consequently, the effect of fire on column could not be simulated directly by the used finite element program. To overcome this disadvantage, the temperature was assumed to be constant all over the column height and the cross-section and was set equals to 600 °C (fire temperature). The reduction in the concrete strength, steel yield strength, steel and concrete Young's modulus due to fire was calculated from

(a) (b) (c)

Figure 3 (a–c) show cracking patterns for the tested specimens.

Table 3 Failure Loads for the Tested Column Specimens.

Col. No.	Concrete strength (kg/cm^2)	Failure load (ton)	Fire period* min.	Applied load at fire (ton)	Cooling manner	$P_{reference}$** (ton)
C_1	300	70.5	0	–	–	78.8
C_2	300	70	15	10	Room temp.	92.6
C3	375	58	15	12	Room temp.	95.7
C4	425	64	15	14	Room temp.	107
C5	500	65.5	15	16	Room temp.	123
C6	300	44	20	10	By water jet	78.8
C7	300	70	15	10	Room temp.	75
C8	300	38	15	10	Room temp.	78.8
C9	300	73	15	10	Room temp.	96.5
C10	300	60	10	10	Room temp.	78.8
C11	300	50	20	20	Room temp.	78.8
C12	300	59.5	15	10	Room temp.	78.8
C13	300	53	20	10	Room temp.	78.8
C14	300	56.5	15	10	Room temp.	78.8
C15	300	51.5	20	15	Room temp.	78.8

* The listed fire period for the prototype (square of scale model 1/9) i.e. (10-min model equivalent 1.5-h prototype) (15-min model equivalent 2.25-h prototype) (20-min model equivalent 3.0-h prototype).
** $P_{reference}$ calculated failure load for column not exposed to fire.

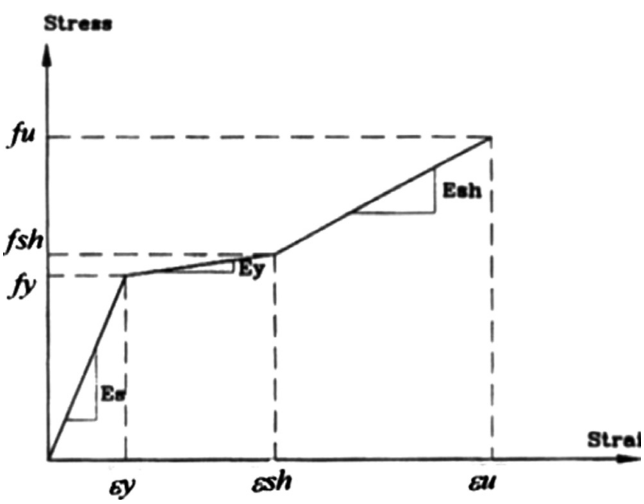

Figure 4a Tri-linear stress–strain curve.

Dotrepp.1999 [3] and is used as an input data for the finite element program.

Comparison between experimental and numerical results is presented in Table 4. In this table, P_{exp} gives the measured failure load obtained from the experiments, P_{th} is the theoretical failure load obtained from the finite element program, and P_{ref} is the predicted ultimate failure load.

The ultimate failure load (P_{ref}) represents the failure load for the column under no fire conditions. P_{ref} calculated for C_1 is used as a reference case for other columns. Consequently, P_{ref} for other specimens' columns is calculated as P_{ref} value of C_1, taking into consideration the effect of different column's concrete characteristic strength and percentage of reinforcement for each column. P_{ref} calculated for all columns.

($P_{ref} = A_c f_{cu} + A_s f_y$) is as given in Table 4.

3.1. Columns general behavior and sensitivity analysis

Analysis of columns exposed to fire and subjected to concentric static loads revealed the following behavior:

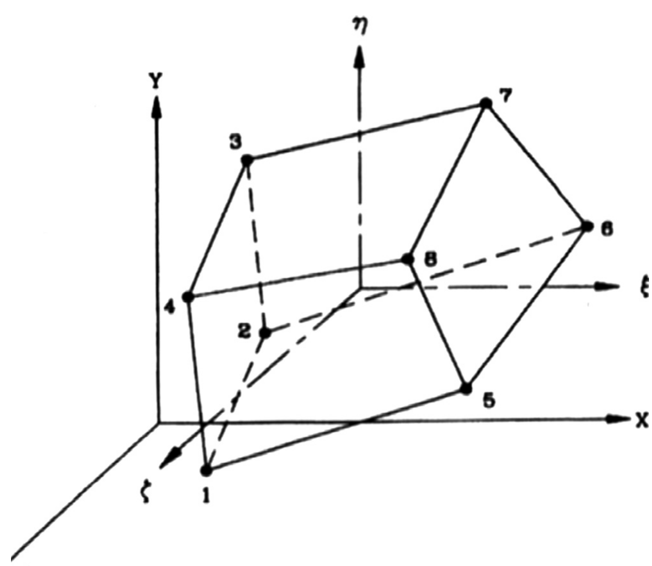

Figure 4b Concrete brick element.

- Columns' ultimate axial deformations increase while columns' ultimate failure loads decrease after exposed to fire. This could be attributed to the relative reduction in stiffness for columns exposure to fire with respect to columns not exposed to fire.
- The maximum lateral deformation along column axis occurred mainly at the mid height of the column from the beginning of loading until failure.
- The failure load for columns exposed to fire is in the range between (65% and 80%) of the failure load for columns not exposed to fire. This result is in agreement with those mentioned before [3] and may be attributed to the reduction in concrete strength for columns when exposed to fire than columns not exposed to fire.
- The concrete modulus of elasticity for columns exposed to fire is nearly (50%) of that for columns not exposed to fire, which is very close to the result discussed before [3].

Table 4 Comparison between experimental and theoretical column failure loads.

Col. No.	Failure load experimental P_{exp} (ton)	Failure load theoretical P_{th}.(ton)	Failure load calculated P_{ref} (ton)	% Of loss in column strength	Modulus of elasticity E t/cm^2
C_1	70.5	76	78.8	10.5	250
C_2	70	72	92.6	24.4	220
C_3	58	72	95.7	40.0	200
C_4	64	78	107	41.0	180
C_5	65.5	84	123	47.0	130
C_6	44	55	78.8	44.2	230
C_7	70	72	75	6.0	220
C_8	38	50	78.8	52.0	150
C_9	73	74	96.5	24.4	160
C_{10}	60	62	78.8	23.8	230
C_{11}	50	62	78.8	36.5	160
C_{12}	59.5	62	78.8	24.5	150
C_{13}	53	62	78.8	32.0	140
C_{14}	56.5	62	78.8	28.0	190
C_{15}	51.5	62	78.8	34.6	150

P_{ref} – calculated failure load of column not exposed to fire. $P_{ref} = (A_c f_{cu} + A_s f_y)$.
P_{th} – failure loads by finite element analysis.
P_{exp} – failure loads of column from experimental result.

In the following, a sensitivity analysis for different parameters that may affect column behavior is presented. This analysis, conducted using both experimental and theoretical results, is summarized as follow:

3.1.1. Effect of the concrete characteristic strength

The column specimens (C_{14}, C_3, C_4, and C_5) are selected to address the effect of concrete characteristic strength (f_{cu}). For these columns, the same column cross-section, materials properties, fire temperature (600 °C), and fire duration are considered constant. The concrete characteristic strength f_{cu} for columns C_{14}, C_3, C_4, and C_5 is 300, 375, 425, and 500 kg/cm^2, respectively. The analytical results showed the following behavior:

- The increase in concrete characteristic strength increased the column failure load by nearly (25%) as concrete characteristic strength increased from (300 to 500 kg/cm^2), as shown in Fig. 5a.
- Analysis of column specimens (C_{14}, C_3, C_4, and C_5) showed nearly linear behavior for strain, and stress up to 50% of column failure loads with a constant concrete Young's modulus for column then the behavior was non-linear. Also both experimental and theoretical analysis, at the same load level, indicated that as concrete characteristic strength increased, the column lateral strains decreased.
- It was found up to 40% of column failure loads, the column vertical displacement has negligible values. At 70% of column failure load, the displacement for column C_{14} (f_{cu} = 300 kg/cm^2) is twice that for column C_5 (f_{cu} = 500 kg/cm^2). This means that column stiffness of C_{14} is nearly half stiffness of column of C_5.
- Columns' vertical displacement obtained from experimental results showed much higher values of columns' displacement compared with theoretical analysis. This result indicated that the finite element analysis gives stiffer behavior as compared to the experiment one as shown in Fig. 5b.
- Experimental results showed that concrete modulus of elasticity of column specimens is reduced by 30% as concrete characteristic strength increased from (300 kg/cm^2 to 500 kg/cm^2).

3.1.2. Effect of fire duration

Column specimens (C_1, C_{10}, C_{12}, and C_{13}) have different fire duration equals to (0, 10, 15, 20 min.) respectively which equivalent to (0, 90, 135, and 180 min) respectively in the prototype scale. The results revealed the following:

- Both experimental and theoretical analysis indicated that for column sample as the period of fire increased from 10 min to 15 or 20 min, the strain of column increased by 45% and 55%, respectively. This result could be attributed to the fact that more yield in longitudinal steel occurred with increasing the fire duration, as shown in Fig. 6a
- Theoretical analysis showed a linear strain behavior from zero loads up to 70% the failure load then changed to a non-linear behavior until failure.
- Both experimental and theoretical analysis indicated that up to 60% the failure load, the strain measured from experimental data is nearly twice the strains predicted from finite element analysis, but the maximum lateral strain at failure of column is nearly the same for both experimental and theoretical results.
- Both experimental and theoretical analysis showed that as the period of fire increases, the column stiffness decreases.
- Both experimental and theoretical analysis showed that with increase in the period of fire (10, 15, 20 min), the corresponding column failure load compared with C_1 decreased by nearly (12%, 15%, 25%) respectively, as shown in Fig. 6b

  The percentage loss in column strength with fire duration can be represented with best fitting using the following equation:

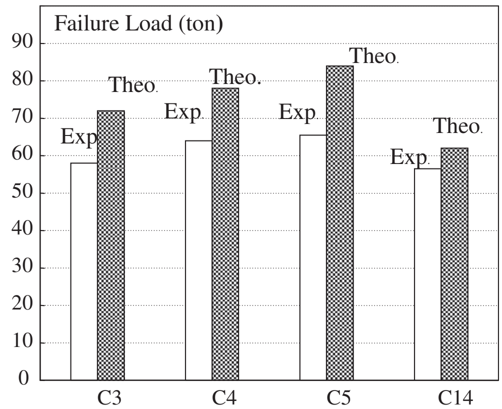

Figure 5a Theoretical and experimental failure load for (column with different concrete strength).

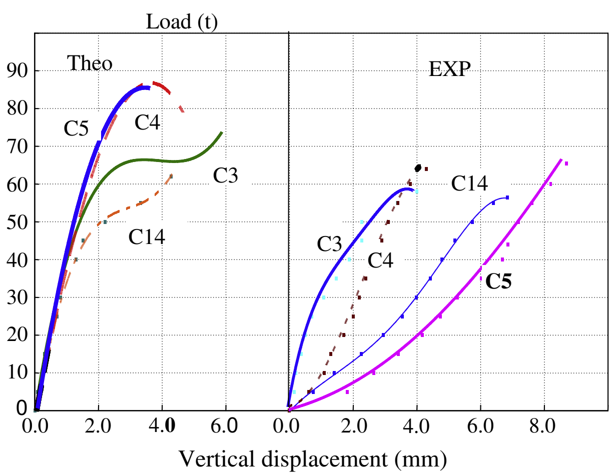

Figure 5b Experimental and theoretical Load-Vertical displacement (Col. C3, C4, C5, C14).

$$Y = 0.105X^3 - 2.8X^2 + 28.9X - 12.6$$

where Y = percentage of loose in column strength. X = fire duration in hours.

- The theoretical analysis gave higher values for strain, column failure loads, and vertical displacement than the experimental results.
- Both experimental and theoretical analysis showed that as the period of fire increased, the column vertical displacement increased as shown in Fig. 6b.

3.1.3. Effect of the applied load during fire on column behavior

The column specimens (C_{11}, C_{13}, and C_{15}) differ in the applied vertical load during fire, which was (20 ton, 10 ton, and 15 ton), respectively, the behavior for columns when exposed to fire is summarized as follows:

- Experimental test and theoretical analysis showed that, as applied load during fire increased from (10 to 20 ton), the ultimate failure load decreased by 3–6% and the column vertical displacement decreased by 25–50%, as shown in Figs. 7a, 7b.

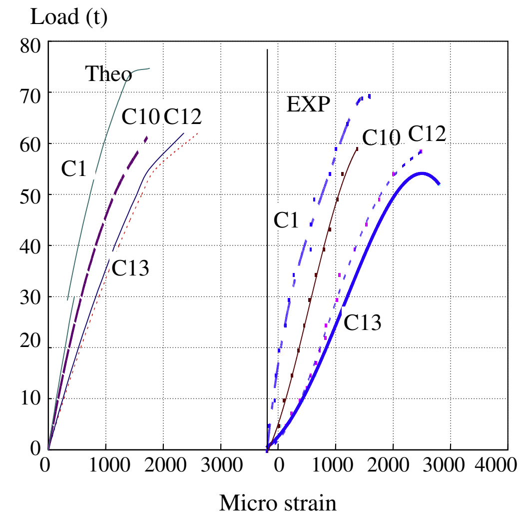

Figure 6a Concrete experimental and theoretical load–strain relationship.

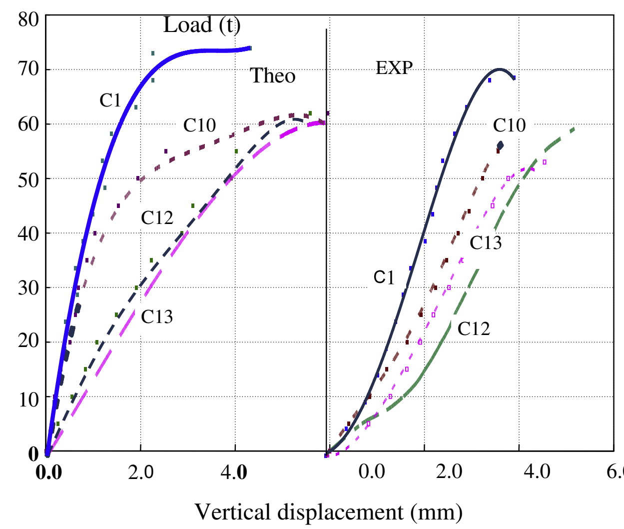

Figure 6b Experimental and theoretical load–vertical displacement relationship.

- Theoretical analysis showed a linear strain and stress behavior with nearly the same values of strain from zero loads up to 50% of column failure load. In addition, these columns have nearly the same concrete modulus of elasticity. After that, the stress–strain behavior changed to non-linear.
- Both experimental and theoretical analysis showed that, at the same load levels, the column, which has a bigger applied load during fire, has a smaller strain.
- The concrete modulus of elasticity increased with the increase in the value of applied load on column during fire as shown in Table 4
- Loss in strength for columns (C_{11}, C_{13}, and C_{15}) with respect to column C_1 (not exposed to fire) is about 32–36.5%, as given in Table 4.

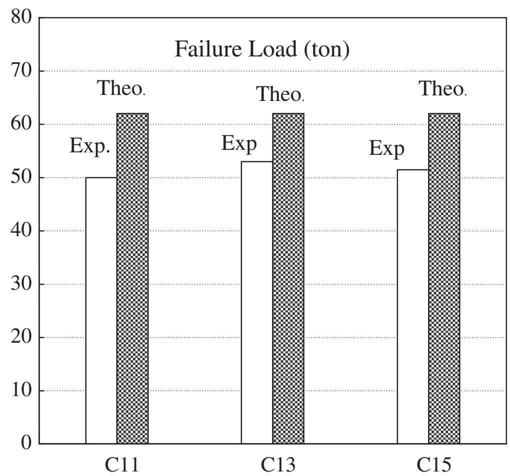

Figure 7a Theoretical and Experimental failure loads for column with different load during fire.

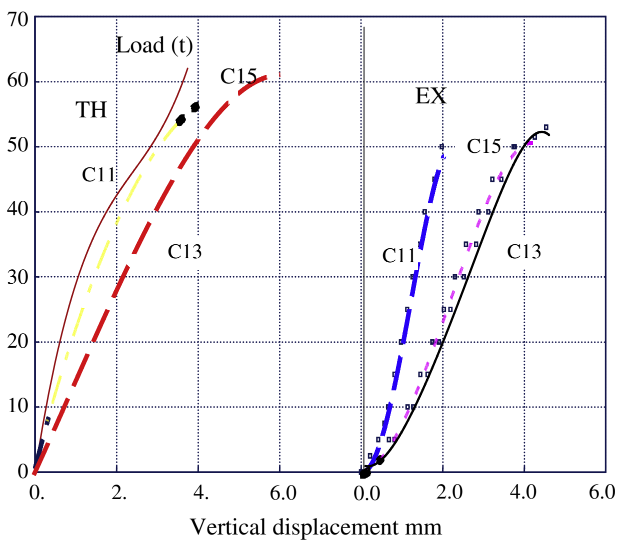

Figure 7b Experimental and theoretical load–vertical displacement relationship (Col. C11, C13, C15).

3.1.4. *Effect of the diameter of the longitudinal reinforcement*

The column specimens (C_2 and C_9) have nearly the same percentage of vertical reinforcement but with different diameters of the main vertical reinforcement steel. C_2 is reinforced by ($5\varnothing12 + 3\varnothing10$), while C_9 is reinforced by ($4\varnothing16$). The behaviors for these specimens are as follows:

- Both experimental and theoretical analysis showed that C_9 (reinforced with bigger diameters) gives a higher failure load by 5% than C_2 (reinforced with smaller diameters).
- Theoretical analysis of columns (C_2, C_9) showed linear strain and stress behavior, with nearly the same values of strains and stresses up to about 40% of the failure load. The concrete modulus of elasticity also had nearly the same values up to 40% of the failure loads and then the behavior changed to non-linear.
- Experimental results indicated that the lateral strains for larger bar diameter in column C_9 are higher than lateral strains for column C_2 from zero loads up to failure loads. This means that column reinforced with a smaller bar diameters produced a higher stiffness than that reinforced with a larger diameters. Fig. 8a
- At all load values, it was noticed that theoretical column vertical displacement for column C_9 which has ($4\varnothing16$) as longitudinal reinforcement was twice the value for column C_2 which has ($5\varnothing12 + 3\varnothing10$) as longitudinal reinforcement. This result implies that the stiffness of column C_2 is higher than that for column C_9 as shown in Fig. 8b.
- Column vertical displacement obtained from the experimental results showed that C_2, and C_9 have nearly the same vertical displacement up to 70% of column failure load. Then, column C_2 showed a little more displacements than column, C_9 until failure loads.
- Up to 70% of failure loads, experimental results indicated higher values of vertical column displacement than the theoretical results.

3.1.5. *Effect of percentage of the longitudinal reinforcement*

Column specimens, C_7, C_9, and C_{14}, have percentage 2.0%, 3.6% and 1.4%, respectively for the main vertical reinforcement. The analysis of results for columns, which were exposed to 15-min fire, showed the following:

- Both experimental and theoretical analysis indicated that, as the percentage of longitudinal main reinforcement increased, the column failure load increased.
- Considering column C_{14} (1.4% longitudinal reinforcement) as a reference case, the experimental results showed an increases in column failure load by (23%, 29%), as the percentage of longitudinal reinforcement increased to 2.0% and 3.6% for columns C_7 and C_9 respectively. Theoretical analysis showed an increase in column failure load by 16% and 19% for columns C_7 and C_9, respectively, with respect to column C_{14}.

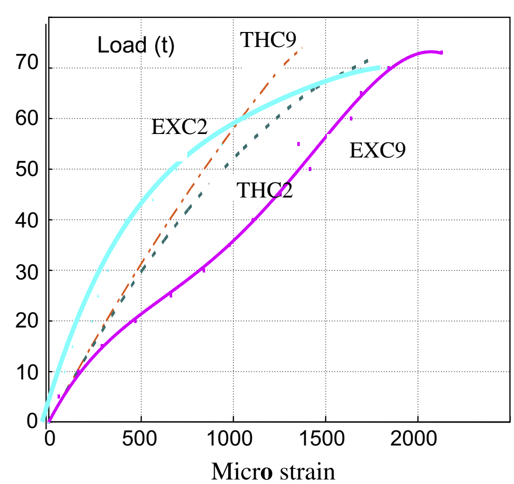

Figure 8a Concrete experimental and Theoretical for load–strain.

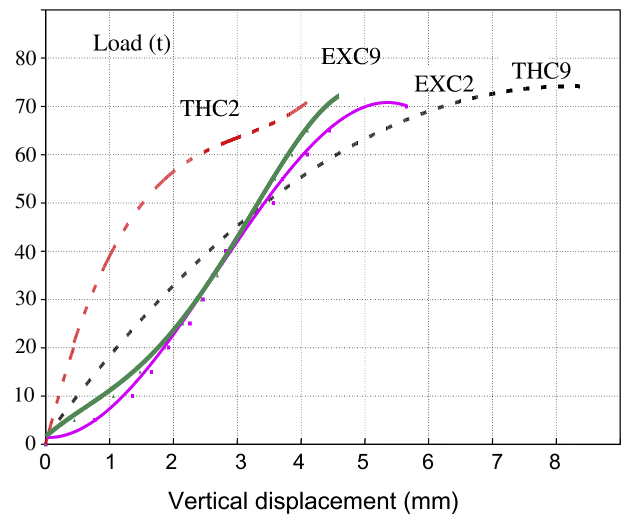

Figure 8b Experimental and theoretical load–vertical displacement.

- Theoretical analysis for columns, C_7, C_9, and C_{14}, indicated a linear strain behavior, and linear stress behavior with nearly the same value of strain up to 30% of failure loads, with nearly the same concrete modulus of e1asticity, and then the behavior changed to a non-linear behavior.
- At the same load level (from zero load up to the failure load), theoretical analysis indicated that, as the percentage of the main longitudinal reinforcement increased, the lateral strain decreased. The strain in columns C_7 and C_9 is reduced by 34% and 40%, respectively, with respect to C_{14} as the percentage of longitudinal reinforcement increased to 2.0% and 3.6%, respectively.
- The experimental results indicated that up to 40% of the failure load, the lateral strain was nearly the same value, with nearly same concrete modulus of elasticity, and then the behavior changed to non-linear behavior. Fig. 9a
- Theoretical analysis for columns, C_7, C_9, and $C_{14,}$ showed that as the percentage of main longitudinal reinforcement increased from 1.4% for C_{14} to 3.6% for C_9 the concrete modulus of elasticity increased by 40%.
- Both experimental and theoretical analysis indicated nearly the same maximum value of lateral strains, which are 1650, 1750, and 2130 for column C_{14}, C_7, and C_9, respectively, as shown in Fig. 9b.
- Theoretical analysis gives higher values of vertical displacement than experimental ones, except for C_{14} where experimental results gave higher values than theoretical analysis as shown in Fig. 9b.

3.1.6. Effect of cooling manner

Column specimens (C_6 and C_{13}) have two different methods of cooling after fire. Column C_6 is cooled with water jet, while Column C_{13} is cooled in the room temperature. The behavior for these columns is as follows:

- Columns C_6 and C_{13} showed a linear strain behavior from zero loads up to 80% of the failure load with nearly the same concrete modulus of elasticity. After that, a non-linear behavior was obtained.

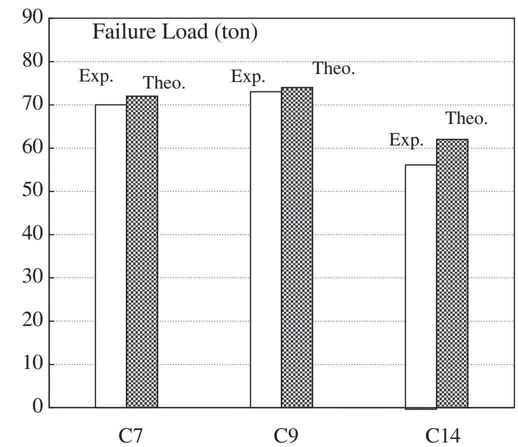

Figure 9a Theoretical and experimental failure load for column with different% vertical RFT.

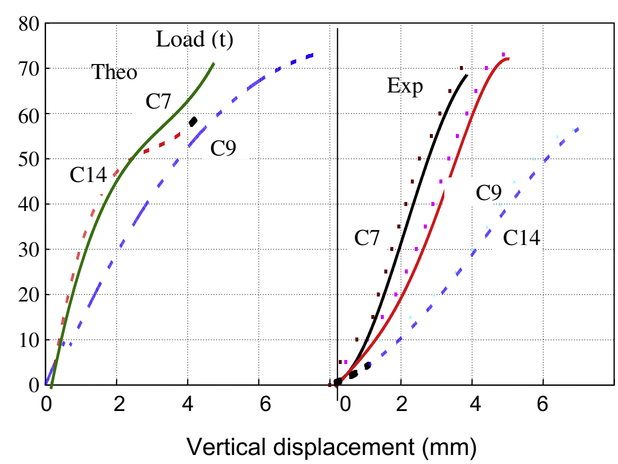

Figure 9b Comparison between experimental and theoretical load–vertical displacement.

- The way of "cooling" had a major effect, cooling column with water gave 38% loss in column strength with respect to column C_6 cooled in room temperature, also showed a smaller failure load by nearly 17% less than column failure load cooled in the room temperature as shown in Fig. 10a.
- Cooling column with water showed less concrete strains (i.e. higher stiffness) than cooling column in the room temperature. In addition, cooled column after fire exposure in the room temperature indicated more ductile failure than cooling column by water jet, as shown in Fig. l0b especially that cooling column by water jet caused a sudden shock for column concrete cover and caused noticeable cracks with a rapid reduction in column strength than cooling column in the room temperature. Fig. 10b

3.1.7. Effect of grade of longitudinal reinforcement

Columns C_7 and C_8 were selected to investigate the effect of reinforcement yield stress. Two different yield stress of main vertical reinforcement high-grade steel (St 36/52), and normal mild steel (St 24/35) are used for columns C_7 and C_8, respectively. The comparisons between columns are as follows:

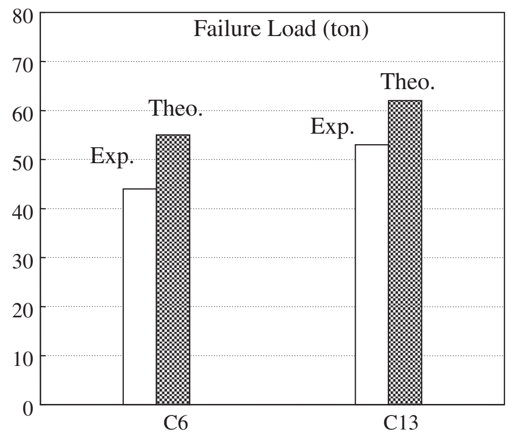

Figure 10a Theoretical and experimental failure load for (column with different cooling).

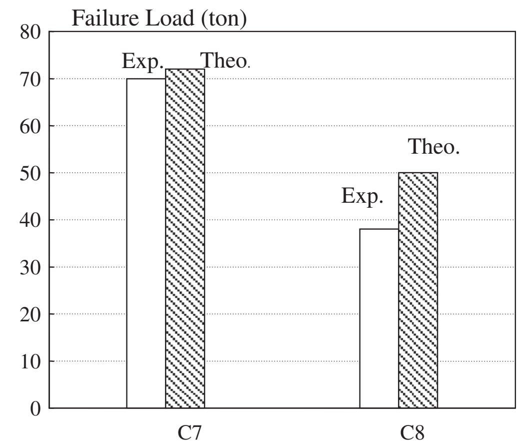

Figure 11a Theoretical and Experimental failure load for (column with different vertical RFT).

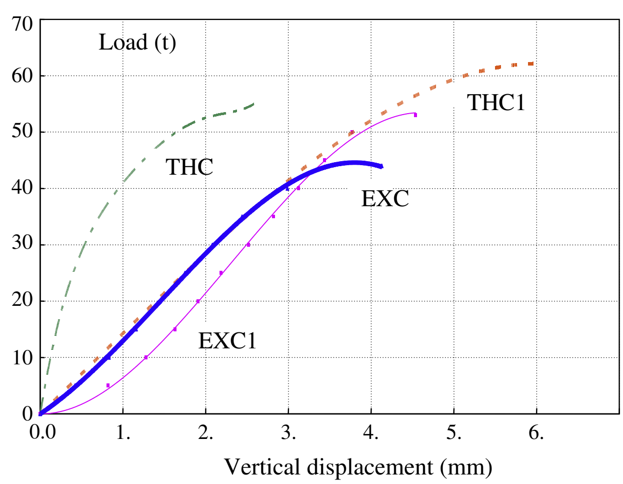

Figure 10b experimental and theoretical load–vertical displacement.

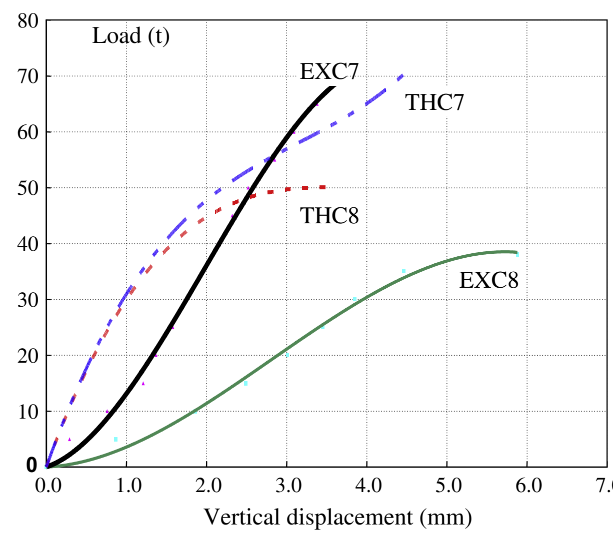

Figure 11b Experimental and theoretical load–vertical displacement.

- Both experimental and theoretical analysis showed that column with high-grade steel (St 36/52) had a failure load higher by about 55% than that for column reinforced with normal mild steel (St 24/35), as shown in Fig. 11a.
- From zero load up to failure, the vertical displacement for column C_8 reinforced with mild steel (St 24/35) was twice that for column C_7 reinforced with high-grade steel (St 36/52), which means that columns with high-grade steel (St 36/52) have more stiffness than column reinforced with mild steel (St 24/35), when exposed to fire, as shown in Fig. 11b.
- Both experimental and theoretical analysis showed an agreement in the stress–strain relationship with nearly the same values of concrete modulus of elasticity with theoretical analysis giving more stiff results.
- Using mild steel (St 24/35) as a main longitudinal reinforcement in columns showed a reduction in the concrete modulus of elasticity by 30% less than using high grade steel (St 36/52) as main longitudinal reinforcement.
- At the same load level, column reinforced with mild steel (St 24/35) has a bigger strain nearly 35% more than column reinforced with high grade steel (St 36/52).

4. Conclusions

Based on the results of the parametric study introduced in this paper using both experimental and theoretical analysis for columns exposed to fire under axial loads, the conclusions can be drawn;

- Ultimate failure loads for columns, which were exposed to fire, are smaller than columns, which were not exposed to fire by (20–40%).
- Columns not exposed to fire showed first crack load at nearly 80% of column failure load, while columns exposed to fire showed first crack load at about 50% of column failure load.
- As fire progresses, cracks widen and crushing of concrete cover occurs accompanied with a large explosion with some local buckling of longitudinal reinforcement, especially for columns with a high concrete characteristic strength.

- Concrete modulus of elasticity for columns exposed to fire is approximately (50–70%) of that for columns not exposed to fire.
- Theoretical analysis for column specimens showed a linear stress–strain behavior, up to nearly 50% of column failure loads, after which the behavior was non-linear.
- The finite element analysis results were in good agreement with the experimental results. Columns reinforced with a bigger bar diameters showed higher lateral strains, and vertical displacements than columns reinforced with smaller bar diameters if the percentage of reinforcement is the same.
- Columns reinforced with 2.0% and 3.6% longitudinal reinforcement showed an increase in column failure load by 23% and 29%, respectively, with respect to column reinforced by 1.4% longitudinal reinforcement.
- Columns reinforcement with high-grade steel (St 36/52) indicated a higher failure load, and vertical displacement were nearly 55% larger than columns reinforced with mild steel (St 24/35).
- Cooling columns with water jet resulted in smaller column failure load by about 17% than that for columns cooled in the room temperature.

  The percentage loss in the column strength with the duration of fire can be represented with best fitting using the following equation:

$$Y = 0.105X^3 - 2.8X^2 + 28.9X - 12.6$$

where Y = percentage of loose in column strength. X = fire duration in hours.

References

[1] H. Hosny, Elmagd Abo, Fire of Reinforced Concrete Structures, Dar El Nasher For Egyptian University, 1994.

[2] C.R. Cruz, Elastic properties of concrete at high temperature, J. PCA Res. Develop. Lab. 8 (1) (1996) 37–45.

[3] J.C. Dotreppe et al, Experimental research on the determination of the main parameters affecting the behavior of reinforced concrete columns under fire conditions, Mag. Concr. Res. 149 (1997) 117–127.

[4] Ng Ah Book, M.S. Mirza, T.T. Lie, Response of direct models of reinforced concrete columns subjected to fire, ACI Struct. J. 87 (3) (1990) 313–325.

[5] Wei-Ming Lin, T.D. Lin, A.J. Durrani, Microstructure of fire damaged concrete, ACI Mater. J. 93 (3) (1996) 199–205.

[6] Bikhiet M. Mohamed, Behavior of Reinforced Concrete Columns Exposed to fire Master of Science, Cairo University, Giza, Egypt, 2004.

[7] ACI Committee 216 R,81, Fire Resistance and Fire Protection of Structure, American Concrete Institute, 1987.

[8] ECCS-2007, Egyptian Code for Design and Construction of Reinforced Concrete Structure.

[9] ACI 318, International Building Code, ACI, 2005, American Concrete Institute, 2005.

[10] T. Gernay, M.S. Dimla, Structural behavior of concrete columns under natural fires including cooling down phase, in: International Conference on Recent Advances in Nonlinear Models – Structural Concrete Applications, 2011.

[11] M.B. Emara, Nonlinear Analysis of RC Slab-Beam Column Subassemblies Under Earthquake Loads, Ph.D. Thesis, Department of Civil Engineering, Helwan University, Cairo, Egypt, 1989.