Structural Response of Post-Fire Exposed Reinforced Concrete Column with Pre-Load

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Abstract. This paper presents the results of experimental and numerical studies of the fundamental response of normal concrete (NSC) columns under fire exposure with pre concentric load. The present study aims to investigate experimentally the behavior of reinforced concrete columns exposed to fire flame with concentric axial load, post-fire behavior under the effect of axial eccentric load. Also, it aims to give a comprehensive account of the fire effects on the ductility and stiffness of these columns. The test results indicated that columns lost about (47.60 - 51.4) % of bearing capacity after exposure to fire at 500 °C with one hour. Moreover, increased decrease in residual bearing capacity significantly with increasing load level applied during fire exposure. Also, Non-linear finite element (FE) analyses of post-fire exposed RC columns with axially load using the ABAQUS computer program is discussed in this paper.

Keywords: FEM, Fire, compressive strength, reinforced concrete columns.

1. Introduction

Fires are one of the most severe environmental conditions structures can be exposed to, so providing appropriate fire safety measures for structural members is one of the major safety requirements in building design [1–5]. The basis for this condition can be attributed to the fact that when other measures fail to contain the fire, structural integrity is the last line of defense for building's occupants and emergency personnel. Fire safety measures for structural members are measured in terms of fire resistance, which is the period during which the structural member shows resistance in terms of structural integrity, stability, and temperature transfer under standard fire conditions. The fire resistance of the structural member usually depends on the structure of the structural member, the materials used in the construction, the density of the load, and the characteristics of the fire itself [6–8]. Concrete generally provides the best fire-fighting properties for any building material. This excellent fire resistance is due to the concrete constituents (i.e. cement and aggregates) [9,10] which, when chemically combined, form a mainly inert material with low thermal conductivity, high heat capacity, and a slower strength degradation with temperature. It is this slow rate of heat transfer and loss of strength that enables concrete to act as an effective fire shield not only between adjacent places but also to protect itself from fire damage [11–13]. The main objective of this paper can be summarized as follows: Investigation experimentally, on the structural response of the reinforced
concrete columns subjected to both fire flame exposure and service axial load at the same time. As well as experimental Study of many factors affected on the eccentric load-carrying capacity of the fire exposed - service loaded R.C. columns, which are Level of service loading through fire exposure. And the eccentricity of the applied load after fire exposure. A numerical study by three-dimensional non-linear finite element utilizing (ABAQUS) computer program, to trace and follow the overall response of the tested columns, and comparing the results with those obtained from experimental work.

2. Materials and Mix Proportions

2.1. Material

A) Cement: Throughout this investigation, Ordinary Portland cement (type1) was utilized for casting concrete of the columns models and samples.

B) Fine Aggregate (Sand) and Coarse Aggregate (Gravel): In this work, sand was utilized as a fine aggregate. It is used for gradation is located within upper and lower limits of the Iraq Standards Region (IQ.S 45/1984) [14]. While rounded, well graded gravel of 14mm maximum size from Badra quarry was used as a coarse aggregate in NSC [15]. The gravel was passed from sieve size (14 mm) to separate the oversize and wash out by water several times, after that, left to dry.

C) Steel Reinforcing bars: Two types of steel deformed bars of Ukrainian organize were utilized in this study (Ø10mm and Ø4mm). The reinforcing bars with diameter 10mm are used as main reinforcement and employed as tension and compression reinforcement. While the bars with diameter 4mm are used as ties.

2.2. Concrete Mix Design

Proportions of the Normal Strength concrete mix are designed with reference to the ACI- 211[16]. After various experimental mixtures, it was found that a mixture of 1:1.9:2.2 (wt) of cement, sand, gravel, respectively. The trail mixture has nominal cylinder compressive strength of 28-days (28.7 MPa). Table 1 shows the mix proportions.

| Materials            | Amount |
|----------------------|--------|
| Cement (kg/m³)       | 391    |
| Sand (kg/m³)         | 763    |
| Gravel (kg/m³)       | 876    |
| w/cement ratio       | 0.49   |
| f c' (28 days) MPa    | 28.7   |

2.3. Specimens Details
All specimens are same in shape and external dimensions. These columns are square in cross section with constant dimensions. The overall length of each column was 1300mm and (140*140) mm cross section. The distance between brackets (middle portion) is 700mm and the dimensions of the bracket are 140*280*300mm as shown in Fig. 1, the objective of making brackets is to apply an eccentric load. All columns are reinforced with four (ϕ 10 mm) deformed steel bars as longitudinal reinforcement (ρ=0.0160) and clear concrete cover (20 mm). Also, the column has steel ties of 4mm diameter with spacing @140mm c/c (or 70 mm). All columns are designed according to (ACI Code 318) [17] specifications. The reinforcement details for columns and brackets are shown in Fig. 2. The current experimental program includes an examination of 8 models of column, 2 control columns, and 6 concrete with normal concrete. The current experimental study consists of examining two groups:

The first group consists of four columns (C1 to C4). The first column (C1) is the control of this group, without (fire exposure, and pre-load). Also, the column (C2) was not pre-load through fire exposure. The column (C3) was exposed to fire with pre-load to (30% Pu). The column (C4) different from (C3) in the applied load (60% Pu). The eccentricities of post repaired load of all columns are (E1). The second group also consists of four columns (C5 to C8). The program of the second group is similar to the first group in all parameters respectively except in eccentricity (E2). Details of all column groups are presented in Table 2.
Table 2. Specifics of tested specimens

| Group No. | Specimen Symbol (Ci) | Fire exposure (F) | Pre-load through fire exposure (Pi) | Eccentricity of post-repaired load (Ei) | Notes |
|-----------|----------------------|-------------------|-----------------------------------|----------------------------------------|-------|
| **Group 1** |                        |                   |                                   |                                        |       |
| C1, E1 | -                    | -                 | 45 mm                             |                                        |       |
| C2, F E1 | F                   | -                 | 45 mm                             |                                        |       |
| C3, F P1 E1 | F               | P1                | 45 mm                             |                                        |       |
| C4, F P2 E1 | F           | P2                | 45 mm                             |                                        |       |
| **Group 2** |                        |                   |                                   |                                        |       |
| C5, E2 | -                    | -                 | 70 mm                             |                                        |       |
| C6, F E2 | F                   | -                 | 70 mm                             |                                        |       |
| C7, F P1 E2 | F               | P1                | 70 mm                             |                                        |       |
| C8, F P2 E2 | F           | P2                | 70 mm                             |                                        |       |

C=concrete column; F =fire exposure; P =concentric pre-load through fire exposure, P1=30% Pu, P2=60% Pu; E1 (e/h=0.33), E2=(e/h=0.50).

2.4. Casting Procedures

In this work, Concrete was mingled using a rotating mixer with a capacity of (0.1 m³). Interior surfaces of cube, cylinder, were cleaned and oiled to avert adhesion with concrete after hardening. Then, each steel cage (for column) is placed horizontally in wooden formwork and fixed in its right location. All molds were filled with concrete in one layer with compaction and vibrated for two minutes by using internal vibrator. Standard procedure was used for compaction of the traditional concrete for the cube and cylinder molds in terms of the number of layers and rod as shown in Fig. 3.

2.5. Fire test

The columns were exposed to fire at an age of more than months after casting. A brick stove having a size of (1400×1400×1100) mm, was used to burning all columns, as shown in Fig. 4. All tests lasted 60 minutes and the maximum temperature reached inside the furnace was, approximately, 500 °C. Digital temperature controller was used to controls the temperature inside the oven by instructing the gas regulator to open and close according to the temperature measured inside the oven by the sensor after installing the degree required by the user. Temperature measurement was performed using type K thermocouples, with 4mm diameter. In order to simulate actual cooling phase of the fire, the cover of the stove was the rise and the specimens were cooled at ambient temperature. For each column, the length exposed to fire was approximately 700 mm. All columns were tested under concentric loads. Columns (C4 and C8) were subjected to a load of 248 kN, which is equal to 60% of the ultimate load. Columns (C3 and C7) were subjected to a load of 124 kN or 30% of the ultimate load, and Columns (C2 and C6) unloaded during exposure to fire. The full details of the stove and equipment with system of pre-loading are shown in Fig. 5. During the test, the column was exposed to fire controlled in such a way that the average temperature in the furnace followed, as closely as possible, the ASTM E119-8810 standard [18] temperature-time curve, as shown in Fig. 6.
Figure 3. Stages of casting operation, a. Assemblage the Molds with Reinforcement Mesh, b. Vibrating the Concrete and c. Samples after Removing the Molds.

Figure 4. Details of the loading frame.

Figure 5. The process of fire exposure with pre-loading.
2.6. Test setup and procedure

All specimens were examined under eccentric loads to fail at 90 day age by use an electro-hydraulic test machine with a maximum range of 2500 kN. Two dial scales of 0.001 mm precision for each deviation and 20 mm capacity were used to measure mid-height lateral and axial displacement for each load increase, as observed in figure 7.

3. Results and Discussion

Results for the columns of each group are compared to each other to determine effect of fire exposure and effects of load applied during fire exposure, and then compare each group with another group to discuss the effect of eccentricity. The test results of the specimen were ultimate load carrying capacity as shown in Table 3, the first crack load, load- lateral deflection relationship, axial deformation, the cracks generated with a load of the columns, and failure mode of column samples. The test results are demonstrates in Table 3.
Table 3. Experimental Results for all tested Columns.

| Group No. | Specimen Symbol (Ci) | Crack Load pc(kN) | Ultimate Load Pu(kN) | Displacement at service Load $\Delta_s$(mm) | Failure Mode |
|-----------|----------------------|-------------------|---------------------|---------------------------------------------|--------------|
|           |                      |                   |                     | Axial | Lateral |                   |
| Group1    | C1, E1               | 90                | 348                 | 3.36 | 3.60    | Tensile failure with crushing of concrete |
|           | C2, F E1             | 78                | 169                 | 2.54 | 3.01    | Ductile Compression Failure with spalling of outer shell |
|           | C3, F P1 E1          | 47                | 154                 | 2.83 | 3.56    | Ductile Compression Failure |
|           | C7, T1 F P2 E1       | 40                | 134                 | 3.71 | 5.09    | Sudden compression Failure with buckling of steel bar |
| Group2    | C9, T1 E1            | 85                | 292                 | 5.54 | 4.05    | Tensile failure with crushing of concrete |
|           | C10, T1 F E2         | 63                | 153                 | 4.46 | 3.12    | Ductile Compression Failure |
|           | C12, T1 F P1 E2      | 43                | 137                 | 4.83 | 4.04    | Tensile Failure with crushing of concrete |
|           | C14, T1 F P2 E2      | 33                | 114                 | 5.77 | 4.51    | Compression Failure with Spalling of the outer Shell |

$\Delta_s$= displacement at service load ($P_s=0.65$ $P_u$) [19].

3.1. Cracking and Ultimate Loads and Failure Modes

Table 3 demonstrates an outline of the test results and the talk of them is displayed in the following sections:

✓ **Control Column C1. E1**

This specimen is made from normal concrete and not exposed to fire, reinforced with (4ϕ 10mm) steel bars as longitudinal reinforcement spacing (ϕ4mm) ties is (140 mm) and the load eccentricity ($e=45$mm and $e/h= 0.33$). The first visible crack is horizontal crack appeared on the tension side at and around mid-height of the column at axial load (90 kN). The failure happened on the compression side suddenly at load (348 kN) with spalling of external shell and buckling the longitudinal reinforcement bars. The axial deformation and the lateral displacement curve are illustrated in Fig. 8.

✓ **Specimen C2 FE1**

This specimen was not loaded during exposed fire. The first visible crack is a horizontal crack appeared at axial load (78kN). The failure happened on the compression side gradually at load (169kN). Compared with control specimen (C1), there is a decrease in the cracking load about (13.33%); also, there was a decrease in the ultimate capacity of about (51.4%), as shown in Fig. 8.

✓ **Specimen C3 FP1 E1**

This specimen was loaded with P1 (0.3Pu) during fire exposure. The first visible crack is a horizontal crack appeared at axial load (47kN). The failure happened on the compression side gradually at load (154kN). Compared with control specimen (C1 and C2), there is a decrease in the cracking load about (47.78%) and (39.74%) respectively; also, there was a decrease in the ultimate capacity of about (55.75%) and (8.88%) respectively, as shown in Fig. 8.

✓ **Specimen C4 FP2 E1**

This specimen was loaded with P2 (0.6Pu) during fire exposure. The first visible crack is a horizontal crack appeared at axial load (40kN). The failure happened on the compression side gradually at load (134kN), followed with buckling of the steel reinforcement. Compared with control specimen (C1, C3 and C5), there is a decrease in the cracking load about (55.56, 48.72 and 14.89) % respectively; also, there is a decrease in the ultimate capacity of about (61.49, 20.71 and 12.99) % respectively, as illustrated in Fig. 8.

✓ **Control Column C5 E2**
This specimen is made from normal concrete and not exposed to fire, reinforced with (4 φ 10mm) steel bars as longitudinal reinforcement and (φ4mm) steel ties of spacing (140 mm) and the load eccentricity (e=70mm and e/h= 0.5). The first visible crack is horizontal crack appeared on the tension side at and around mid-height of the column at axial load (89kN). With increasing of applied load, the cracks propagate (in different positions and more than the specimen (C₁) on the tension side and it begins to merge together. At the ultimate stage, a little increase in the load led to more increase in the displacements than the previous stages and the failure happened on the compression side at load (292 kN), as shown in Fig. 9. Compared with control specimen C₁, there is a decrease in the first crack and ultimate load capacity about (5.56 % and 16.10%) respectively. The load-longitudinal deformation and the load-lateral displacement curve are illustrated in Fig. 8.

✓ Specimen C₆ FE₂

This specimen was not loaded during exposed fire; the first visible crack is a horizontal crack appeared at axial load (63kN). The failure occurred in the inner part at middle of the column on the compression side gradually at load (153kN) as shown in Fig. 9. Compared with specimen C₂ for group1, there is a decrease in the first crack and ultimate load capacity about (19.23% and 9.46%) respectively. Compared with control specimen of this group (C₃), there is a decrease in the cracking load and ultimate capacity about (25.88% and 47.60%) respectively. The load-longitudinal deformation and the load-lateral displacement are presented in Fig. 8.

✓ Specimen C₇ FP1E₂

This specimen was loaded with P₁ (0.3Pu) during fire exposure. In specimen C₇, The first visible crack is a horizontal crack appeared at axial load (43kN). The failure occurred in the inner part of the middle of the column on the compression side at load (137kN), followed with buckling of steel reinforcement, as shown in Fig. 9. Compared with specimen C₃ for group1, there is a decrease in the first crack and ultimate load capacity about (8.51% and 11.03%) respectively. Compared with control specimen of this group (C₅ and C₆), there is a decrease in the cracking load about (49.41 and 31.75) % respectively, and ultimate capacity about (53.08 and 10.46) % respectively. The load-longitudinal deformation and the load-lateral displacement are presented in Fig. 8.

✓ Specimen C₈ FP2E₂

This specimen was loaded with P₂ (0.6Pu) during fire exposure. The first visible crack is a horizontal crack appeared at axial load (33kN). The failure occurred in the inner part of the middle of the column on the compression side gradually at load (114kN) as shown in Fig. 9. Compared with specimen C₄ for group1, there is a decrease in the first crack and ultimate load capacity about (17.50% and 14.92%) respectively. Compared with control specimen of this group (C₅, C₆, and C₇) there is a decrease in the cracking load about (61.18, 47.62 and 23.26) % respectively, and decrease in the ultimate capacity about (60.96, 25.49 and 16.79) % respectively. The load-longitudinal deformation and the load-lateral displacement.

Figure 8. The load-deflection curves for specimens (C₁, C₂, C₃, C₄, C₅, C₆, C₇, and C₈).
3.2 Crack Width and Cracking Pattern

At the early stages of loading, control reinforced concrete columns are free from any cracks. The width of crack was recorded by crack meter and the first crack is known by the natural vision and then recorded the corresponding load. The flexure transverse cracks are usually originated at the tension zone and propagate toward the compression zone for column specimens eccentrically loaded, while for the longitudinal cracks, they initiated at the corbel as a shear crack. The models exposed to the fire were cracked by burning with so called (Hairline cracks) so they are pre-cracked, black cracks are caused by the axially applied load during the burning process while red cracks caused by eccentricity applied load during the test. Table 4 includes maximum crack width for column specimens after fire exposure and at service load for repaired and unrepaired column specimens. From Table 4, it can be seen that crack width values of column specimens without fire is the lowest value of all column, the average value is (0.32 mm), as shown in Fig. 10a. while for column specimens exposure to fire, the average crack width was (0.57 and 0.66) mm, for crack width after fire exposure and crack width at service load respectively, as shown in Fig. 10b. For the column specimen loaded during fire exposure, the average crack width (1.99 and 1.94) mm, for crack width after fire exposure and crack width at service load respectively, as shown in Fig. 10c, this value increased due to the effect of loaded during fire exposure.

### Table 4. Crack Width of the Columns.

| Group No. | Specimen | Symbol (Ci) | maximum Crack width after exposure to fire (mm) | Crack width at service load (mm) | Location of crack |
|-----------|----------|-------------|-----------------------------------------------|---------------------------------|------------------|
| Group1    | C₂₃T₁E₁  | -           | 0.39                                          |                                 | In the middle of the column |
|           | C₅₄T₁F₁E₁ | 0.5         | 0.56                                          |                                 | In the first quarter of the column |
|           | C₅₄T₁F₂P₁E₁ | 0.9         | 1.21                                          |                                 | In the first quarter of the column |
|           | C₅₄T₁F₂P₂E₁ | 3.5         | 2.19                                          |                                 | In the last quarter of the column |
| Group2    | C₅₄T₁E₂  | -           | 0.24                                          |                                 | In the middle of the column |
|           | C₁₀₄T₁F₁E₂ | 0.64        | 0.76                                          |                                 | In the middle of the column |
|           | C₁₂₄T₁F₂P₁E₂ | 1.04        | 1.46                                          |                                 | In the first quarter of the column |
|           | C₁₄₄T₁F₂P₂E₂ | 2.54        | 2.91                                          |                                 | In the first quarter of the column |
3.3. Ductility Index

Ductility is defined as the energy absorbed through materials up to the failure that has been completed [20]. In the current study, ductility ratios are assessed according to the vertical or lateral displacement at maximum load divided by vertical or lateral displacement at the service load (approximately 65% of maximum load) [19]. In the present work, the yield deflection is not clear, so, a similar technique was implemented by using deflection at service load (Ps), the experimental ductility ratios (μ) for reinforced concrete columns are calculated by dividing the axial deformation at ultimate load (Δu) to the axial deformation at service load (Δs). The ductility indices computed from the numerical and experimental analyses are summarized in Table 5. Generally, the results of the Table 5 show the deterioration in ductility ratio in (axial or lateral) displacement with fire exposure and with increase load during fire exposure for column specimens of both groups (1 and 2), as shown in Fig. 11. From these results, residual ductility in axial displacement for columns in group one after fire exposure are (92.16, 73.53 and 65.2) % when columns exposed to fire (without loaded, with pre-load 30% Pu and 60% Pu) respectively. While for columns in group two are (86.29, 75.43 and 70.86) % when columns exposed to fire (without loaded, with pre-load 30% Pu and 60% Pu), respectively. While the experimental residual ductility in lateral displacement for columns in group one after fire exposure are (94.35, 81.36 and 68.93) % when columns exposed to fire (without loaded, with pre-load 30% Pu and 60% Pu) respectively. While for columns in group two are (93.6, 85.47 and 76.16) % when columns exposed to fire (without loaded, with pre-load 30% Pu and 60% Pu), respectively.

Table 5. Numerical and experimental results of the ductility index of Columns.

| Specimen | Service displacement Δs (mm)* | Ultimate deformation Δu (mm) | Ductility index, μ** |
|----------|-------------------------------|-----------------------------|---------------------|
|          | Axial | Lateral | Axial | Lateral | Axial | Lateral |
| C1, E1   | 3.36  | 3.60    | 6.87  | 6.36    | 2.04  | 1.77    |
| C2, F E1 | 2.54  | 3.01    | 4.79  | 5.04    | 1.88  | 1.67    |
| C3, F P1 E1 | 2.83 | 3.56 | 4.25 | 5.12 | 1.50 | 1.44 |
| C4, F P2 E1 | 3.71 | 5.09 | 4.93 | 6.23 | 1.33 | 1.22 |
| C5, E2   | 5.54  | 4.05    | 9.69  | 7       | 1.75  | 1.72    |
| C6, F E2 | 4.46  | 3.12    | 6.75  | 5.03    | 1.51  | 1.61    |
| C7, F P1 E2 | 4.83 | 4.04 | 6.39 | 5.97 | 1.32 | 1.47 |
| C8, F P2 E2 | 5.77 | 4.51 | 7.16 | 5.89 | 1.24 | 1.31 |

*Δs= displacement at service load (Ps = 0.65 Pu) [6].
\[ \mu = \frac{\Delta u}{\Delta s} \]

3.4. Stiffness Parameter

Stiffness is defined as load required for producing unit deformation in the member. The slope of the secant drawn to the load-deflection curve at a load of 0.75 times the ultimate load can be utilized as stiffness criteria [21]; the stiffness result is illustrated in Table 6.

| Specimen | 0.75 Pu (kN) | displacement at 0.75Pu (mm) | Stiffness, \( \kappa \) (kN/mm) |
|----------|-------------|-----------------------------|-------------------------------|
|          |             | Axial | Lateral | Axial | Lateral | Axial | Lateral |
| C1, E1   | 261         | 4.48  | 3.95    | 58.26 | 66.08    |
| C2, F E1 | 126.75      | 2.74  | 4.00    | 46.26 | 31.69    |
| C3, F P1 E1 | 115.5     | 3.14  | 4.67    | 36.78 | 24.73    |
| C4, F P2 E1 | 100.5     | 3.98  | 5.33    | 25.25 | 18.86    |
| C5, F E2 | 219         | 6.95  | 4.82    | 31.51 | 45.44    |
| C6, F E2 | 114.75      | 4.97  | 3.45    | 23.08 | 33.26    |
| C7, F P1 E2 | 102.75   | 5.33  | 4.72    | 19.28 | 21.77    |
| C8, F P2 E2 | 85.5       | 6.12  | 4.94    | 13.97 | 17.31    |

The results, as details in Table 6 and Fig. 11, indicate that, experimental residual stiffness in axial displacement for columns in group one after fire exposure are (79.40, 63.13 and 43.34) % when columns exposed to fire (without loaded, with pre-load 30%Pu and 60%Pu) respectively. While for columns in group two are (73.28, 61.19 and 44.34) % when columns exposed to fire (without loaded, with pre-load 30%Pu and 60%Pu), respectively. While the experimental residual stiffness in lateral displacement for columns in group one after fire exposure are (47.96, 37.42 and 28.54) % when columns exposed to fire (without loaded, with pre-load 30%Pu and 60%Pu), respectively. While for columns in group two are (73.2, 47.91 and 38.09) % and when columns exposed to fire (without loaded, with pre-load 30%Pu and 60%Pu), respectively.

![Figure 11. Ductility of specimens.](image-url)
4. Finite Element Analysis (F.E.A)

In this research, a nonlinear three-dimensional Finite Element Analysis (FEA) was performed to show behavior of columns after a fire using the FEM code ABAQUS/Standard 6.14.

4.1. Material Properties

The element types (C3D8, and T3D2) which provided in the ABAQUS/Standard (6.14.), were utilized to model the concrete, and reinforcing steel bars, respectively. There are several material models to represent concrete, which have been implemented in commercial software used for simulation of concrete structures subjected to loads. For example, the Concrete Damage Plasticity (CDP) model, Concrete Smeared Crack model, and Modified Drucker-Prager/Cap model are the most popular models for modeling concrete material in ABAQUS.

4.2. FE mesh and boundary conditions

Before receiving numerical examination, adequate earlier investigation of the distinctive work densities was performed to decide the best work thickness that gives the necessary exactness relying upon the degree of unpredictability of the examination with the time spent in handling. Nonetheless, a 30 mm component size was picked for the work thickness guaranteeing a decent change between the size of the components and dependability of numerical arrangement. For all sections, endorsed limit conditions for numerical examination were the relocation limitations every which way (X, Y, and Z hub) at the highest point of the segment. While the lower some portion of the segment is controlled to the uprooting on the X and Z hub and free dislodging is accepted toward longitudinal pivot of the segment (Y hub). The stacking was applied as a uniform weight at base of segment as appeared in Figure 13.

5. Theoretical Results and Discussion

In this paper, the finite element modeling is used to represent the column behavior. To verify the precision of the numerical model proposed here, numerical results obtained were compared with those extracted from experimental work. This investigation was directed to assess the ductility and stiffness properties of all columns. This section provides a summary of the overall behavior of columns before and after fire exposure which observed from the numerical and experimental analysis, as shown in Table 7.
Figure 13. Applied load and boundary conditions of modeled column.

| Specimens | Ultimate loads pu (kN) | $P_u$ (FEM) | $P_u$ (EXP) | Service axial displacement $\Delta s_v$ (mm)** | $\Delta s_v$ (FEM) $\Delta s_v$ (EXP) | Service lateral displacement $\Delta s_h$ (mm) | $\Delta s_h$ (FEM) $\Delta s_h$ (EXP) |
|-----------|------------------------|-------------|-------------|---------------------------------------------|----------------------------------------|---------------------------------------------|----------------------------------------|
| C₁, E₁    | FEM 349                | 0.29        | 2.94        | -12.50                                      | 3.12                                   | -13.33                                      |
|           | EXP 348                |             | 3.36        | 2.55                                        | 3.01                                   |
| C₂, F E₁  | FEM 178                | 5.32        | 2.25        | -11.42                                      | 3.01                                   |
|           | EXP 169                |             | 2.54        | 3.16                                        | 3.56                                   |
| C₃, F P₁ E₁ | FEM 165              | 7.14        | 2.46        | -13.07                                      | 3.01                                   |
|           | EXP 154                |             | 2.83        | 3.16                                        | 3.56                                   |
| C₄, F P₂ E₁ | FEM 140              | 4.48        | 3.19        | -14.02                                      | 4.41                                   |
|           | EXP 134                |             | 3.71        | 5.09                                        | 5.09                                   |
| C₅, E₂    | FEM 291                | -0.34       | 4.98        | -10.11                                      | 3.43                                   |
|           | EXP 292                |             | 5.54        | 4.05                                        | 4.05                                   |
| C₆, F E₂  | FEM 164                | 7.19        | 4.15        | -6.95                                       | 2.65                                   |
|           | EXP 153                |             | 4.46        | 3.12                                        | 3.12                                   |
| C₇, F P₁ E₂ | FEM 142              | 3.65        | 4.14        | -14.29                                      | 3.49                                   |
|           | EXP 137                |             | 4.83        | 4.04                                        | 4.04                                   |
| C₈, F P₂ E₂ | FEM 118              | 3.51        | 4.86        | -15.77                                      | 4.22                                   |
|           | EXP 114                |             | 5.77        | 4.51                                        | 4.51                                   |

5.1. Load Carrying Capacity

Test results indicated that all the numerical values of the ultimate load for columns exceed the experimental values by a margin ranging between (0.3–7.2)%, except the control column with eccentricity (70mm) (which is not exposed to fire), the ultimate load decreased by about (0.34)%. While the numerical predicted service axial and lateral displacement is found to be lower by (6.9–15.8) % and (6.4–15.5) %, respectively compared to the experimental values.

Conclusions

The following conclusions can be mentioned within the scope of this study:

1- For reference columns, it is observed that columns exposure to fire has much greater cracking and ultimate load compared to other references in each group, by about (13.34 and 25.88) % and (51.44 and 47.60) % for the specimen with eccentricity 45mm (e/h = 0.33) and 70mm (e/h = 0.50) respectively.

2- Experimental and numerical results indicated that the reduction in residual bearing capacity increases with increase loaded level during the fire exposure. It was found that the specimens pre-loaded during fire exposure (30%Pu) had a change in cracking load compared with the specimens without pre-loading during fire exposure, this change was less from unloaded...
specimens by about (39.74% and 31.75%), for specimens with eccentricity (45mm and 70 mm) respectively. also, the ultimate load was less by about (8.88% and 10.46%) for specimens with eccentricity (45mm and 70 mm) respectively. while for the columns pre-load with (60%Pu) a very obvious change in cracking load was observed about (48.72% and 47.62%), for specimens with eccentricity (45mm and 70 mm) respectively. also, it presented a less ultimate load by about ( 20.71 and 25.49%) for specimens with eccentricity (45mm and 70 mm) respectively.

3- After fire exposure, the load-displacement curves become more flattening which indicates a softer behavior, and the degree of flattening increases with increasing pre-load level during the fire exposure. This is attributed to relative reduction in stiffness of column specimens.

4- Spalling appears significantly during the first 30-min of the fire exposure for the column specimens pre-loaded during fire exposure.

5- With increase in value of the eccentricity of the applied axial loads from 45mm (e/h = 0.33) to 70mm (e/h = 0.33), the flexural effect was increased, so, the cracking and ultimate load were decreased as an average (5.88% and 16.09%) for the control specimen respectively, also the cracking load and ultimate load were decreases in this case:

- Specimen exposed to the fire by about (19.23 and 9.47) % respectively.
- Specimen exposed to fire with (30% Pu) by about (8.51 and 11.04) % respectively, also
- Specimen exposed to fire with (60% Pu) by about (17.5 and 14.93) % respectively.

6- The ductility in (axial or lateral) displacement in columns specimens decreases with exposure to fire with about (7.84 and 13.71) % or (5.65and 6.40) % for the specimen with eccentricity (45 and 70) mm respectively. Ductility ratio in (axial or lateral) displacement decrease with increase load during fire exposure for column specimens about (from 24.57 to 34.80) or (from 14.53 to 31.07) %.

7- The stiffness in (axial or lateral) displacement in columns specimens decreases with exposure to fire with about (20.60 and 26.72) % or (52.04 and 26.80) % for the specimen with eccentricity (45 and 70) mm respectively and crack width also decreases with column exposure to fire. The failure mode was ductile compression. The stiffness in (axial or lateral) displacement decreased with increase load from 100% at ambient temperature to about (from 44.34 to 64.71) % or (from 28.54 to 64.01) %, the crack width also decreases with increase load during fire exposure.

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