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RESEARCH PAPER

Evaluation of Full-Scale Concrete Frames Exposed to Natural Fires at Early Ages.

Muhammad Ismaiel Omer[1*], Dilshad Kakasoor Jaf [2]

[1&2]Department of Civil, College of Engineering, Salahaddin University-Erbil, Kurdistan Region, Iraq

A B S T R A C T:

This article presents the evaluation of full scale reinforced concrete frames subjected to natural fire at early age. The test program consisted of constructing three large frames of reinforced concrete as well as revealing them to the natural fire by shooting their formwork when the age of concrete achieves three and five days. The evaluation of reinforced concrete frames was done by the load test method as described in the American Concrete Institute (ACI), namely the 24 hrs load test method, which is evaluation criteria that have been in use for several decades. For each frame, the structural evaluation based on deflection criteria is discussed. Results showed that the frame exposed early to natural fire was generally more affected than the other frame, as its midspan deflection was increased to about 109% if compared to frame not exposed to fire.

KEY WORDS: Reinforced concrete structures, Natural fire, Early ages, ACI load test methods
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1. INTRODUCTION

Concrete is the composite material widely used in Iraq's structural system and worldwide (Bikhiet et al., 2014). The common material used in Iraq for molding formwork of reinforced concrete structures is timber. Timber is a famous material that burns rapidly, particularly in the summer season, when oiled. Therefore, in recent years, in some developing countries numerous fire accidents occurred in reinforced concrete buildings during the construction stage, due to problems with the construction procedure and on-site management (Lu et al., 2019). The concrete in the mold usually reaches an early age in these conditions (i.e., the "young"), and the inner chemical composition of early-age concrete differs from that in the carrier due to unfinished early-age hydration.

The literature contains overall evaluations of concrete performance at high temperatures (Khoury, 1992, Schneider, 1988, Xiao and König, 2004). An extensive study has revealed that concrete is a multipurpose material and, if properly designed, can be fundamentally fire-resistant. The reaction to a natural fire of the concrete material and structure relies on the kind of fire, which can differ significantly (Khoury et al., 2007).

Available studies have been established either using "time-temperature curve" methods sometimes called "standard fire curves" such as those mentioned in ISO 834 (ISO 834, 2014) and ASTM E119 (ASTM E119, 2000) (based on experimental observations) or using "natural fire"

* **Corresponding Author:**
Muhammad Ismaiel Omer
E-mail: muhammad.omer@su.edu.krd
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methods (Behnam and Ronagh, 2013). The studies showed that the behavior of structural elements under natural fire is completely different from that observed during the standard fire tests. The standard fire curve is not representative of a real fire in a real building (Li et al., 2015). However, fires in compartments are simulated using natural fire where both heating and cooling phases are taken into consideration, and they provide a realistic illustration of a fire event (Behnam, 2018).

Bisby et al. (2013) results have been reviewed for non-standard large-scale fire test structures. They noted that non-standard fire tests conducted worldwide over the last three decades detected different weaknesses in our knowledge of natural fires; in most cases, standard furnace tests could not identify these weaknesses.

Fire causes heat to flow into the concrete structure. The temperature in the concrete mass will rise, thereby causing thermal expansion of the constituents, evaporation of moisture, the buildup of pore pressure, and the degradation of mechanical properties (Khoury et al., 2007). The deterioration of concrete strength due to short-term exposure to elevated temperature (fire) has attracted attention in the last decades. The nature of the fire, types of structure, and loading system are reasons that the modes of concrete failure under fire exposure will be varied (Bikhiet et al., 2014).

When concrete exposed to high temperatures, the chemical composition and physical structure change considerably, resulting in a significant reduction of the mechanical properties, such as strength, modulus of elasticity, and volume stability. These changes are related to differential thermal expansions between the aggregate and cement paste associated with dehydration of the cement paste due to the decomposition of the calcium silicate hydrate (C-S-H) (Kirchhof et al., 2015). However, when the temperature reaches about 300 °C, some of the combined water from C–S–H and chloraluminite hydrates and the interlayer (C–S–H) water will evaporate. Calcium hydroxide $Ca(OH)_2$, which is one of the essential compounds in cement paste, dissociates at around 530 °C, thereby resulting in the shrinkage of concrete (Chen et al., 2009).

While severe fire can cause significant damage to reinforced concrete structures, reinforced concrete structures failure is rarely caused by fire damage. If a fire accident takes place during the construction stage, then the residual strength of younger reinforced concrete structural elements should be assessed by the engineers so that the safety and reparability of the structure after the fire evaluated (Lu et al., 2019).

An important issue involving researchers and engineers in many countries is the evaluation of existing buildings (Pucinotti, 2015). In-situ load testing is a method widely used for evaluating the strength of the existing structure (ElBatanouny et al., 2015). The U.S. is a century-old tradition that the in-situ load testing of concrete structures where one of the 1890s ' oldest cases with excellent documentation. The most direct evidence of the exclusive and novel performance of construction materials and methods was on-site load testing in the early days. The American Concrete Institute ACI started standardizing load testing processes of concrete structure in 1920. The ACI addresses in-situ load testing in two standards: a) ACI 318 chapter 27 “Building Code Requirements for Structural Concrete” (ACI 318-19, 2019), and b) ACI 437 “Code Requirements for Load Testing of Existing Concrete Structures” (ACI 437.2-13, 2013). The test criteria for passing the stress test focused on maximum deflection under continuous load in combination with deflection recovery following removal of the test load (Galati et al., 2008).

In technical literature, there is almost no relevant study available on assessment of early ages of reinforced concrete frames exposed to natural fire. Thus, an investigational study was performed on the strength assessment of early-age concrete by in situ load test after exposed high-temperature. The consequence of removing the formwork was identified by firing the wood on reinforced concrete frames at an early age.

2. MATERIALS AND METHODS

2.1 Frame Dimensions and Details

The test program consisted of casting three frames of reinforced concrete, which has the same dimensions. These dimensions are shown in Figure 1; the column cross-section was 300×300 mm; the beam width was 300 mm, 300 mm dropped below the slab; and the slab thickness was 120 mm. The span of the frame in both directions between the center of the columns was

4000 mm, and the height of the column was 3000 mm. The test frame was supported on four single column footings built specifically for this purpose.

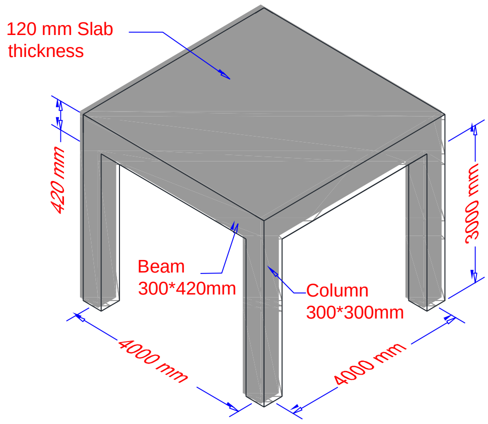

Figure 1: Dimensions of frame structure

The casting of reinforced concrete frames was performed by ready mixed concrete, which was provided by Lafarge Company (a special company for supplying high quality ready mixed concrete). Furthermore, the casting of the test frame occurred in two stages. The footings were cast first on the strong-floor. The columns, beams, and slab of the frame were cast in one stage during the pour of concrete in the structure. Care was taken during mixing and placing operations to ensure consistent concrete properties and consolidation. A sufficient number of control specimens were collected to monitor strength development in the test frame, particularly for determining the strength of concrete in the frame at the time of the load test.

The reinforcement details of the frame are shown in Figure 2 and Figure 3. Table 1 shows the detail of the full-scale reinforced concrete frames and variables. The primary variable used in this investigation is the age of concrete when subjected to fire in order to understand the effect of concrete age on reinforced concrete frame deformation, as presented in Table 1.

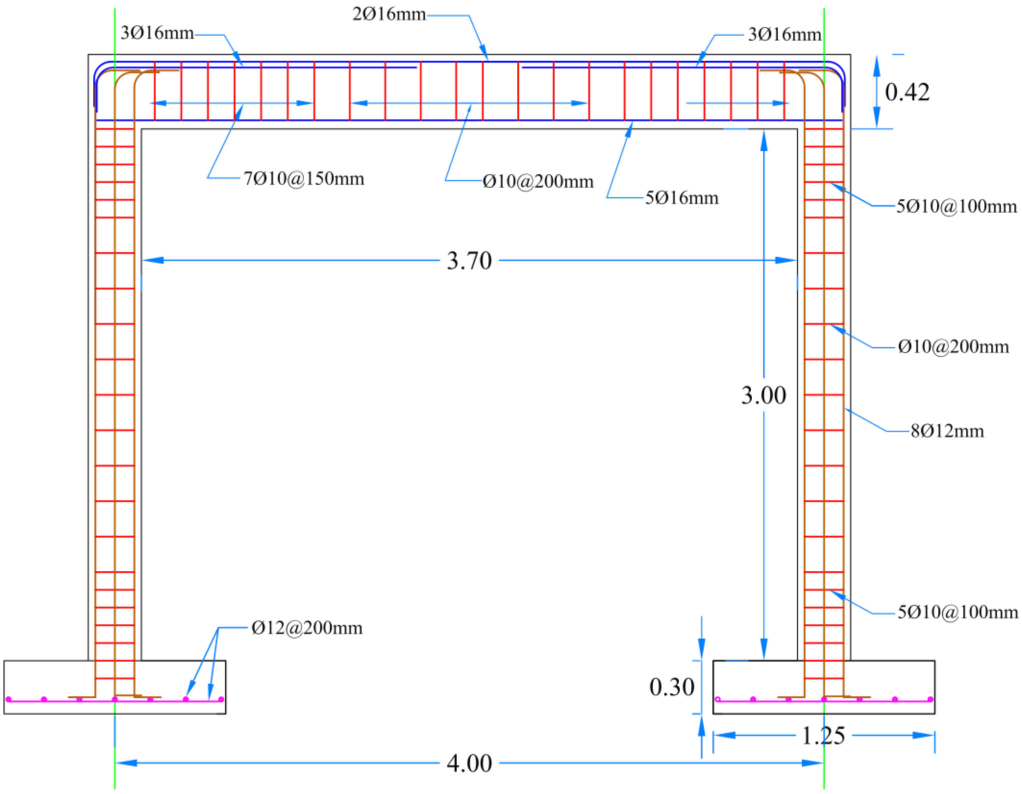

Figure 2: Reinforcement details of column, beam and footing

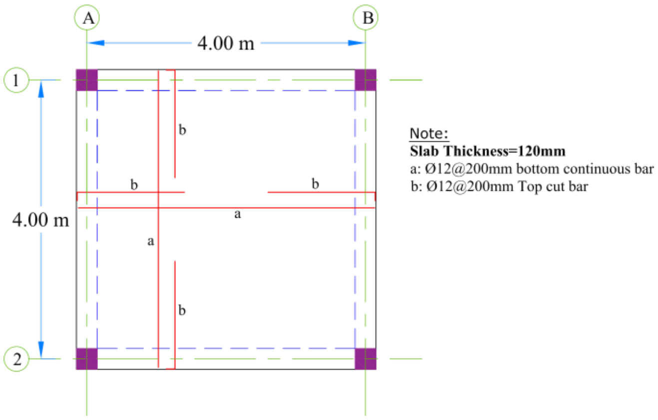

Figure 3: Reinforcement details of slab

Table (1) Frames with variables

Frame No.	f_{cu} (MPa)	Exposed to fire	Age at fire exposure (Days)
FF3	33.5	yes	3
FF5	41.9	yes	5
FN	38.3	no	Nil

2.2 Materials

Three reinforced concrete frames were constructed using ordinary Portland cement / Lafarge OPC, water, fine aggregate, crushed aggregated gravel was locally available used as coarse aggregate with a maximum size of 19 mm. The mix proportion used in fabricating reinforced concrete frames is presented in Table 2.

Table (2) Mix proportion

Specified concrete strength f_{cu} (MPa)	W/C (%)	Cement (kg/m^3)	Water (kg/m^3)	Fine Aggregate (kg/m^3)	Coarse Aggregate (kg/m^3)
35	0.51	356	182	701	1131

2.3 Time–Temperature Curve

In any fire test, it should follow one kind of the time-temperature curve. Furthermore, there are two types of time-temperature curves; standards and natural. However in the current paper we prefer the stance taken by Harmathy and Lie who rightly noted that “it always must be borne in mind that in a strict sense standard fire endurance (testing) is not a measure of the actual performance of an element in fire, and, furthermore, that it is not even a perfect measure for comparison” (Harmathy and Lie, 1970).

After creating a wooden mold for the concrete frame, then cast with concrete. The concrete curing process began and continued until the concrete age was three days for FF3 and five days for FF5. Then the wooden mold of the reinforced concrete frame was burned, as shown in Figure 4. The temperature of the inside concrete was increased after the wooden mold was burned. The measuring temperature inside the concrete was done by inserting k-type thermocouples into the mold before casting, as shown in Figure 5. A total of 11 number of embedded thermocouples were used and secured in place before the casting of concrete. The location of the thermocouples is shown in Figure 6.

Figure 4: Reinforced concrete frame exposed to fire

Figure 5: Insertion of a thermocouple

The average time-temperature curve, as shown in Figure 7, was drawn after all recorded temperatures were collected. Thus, with time, the

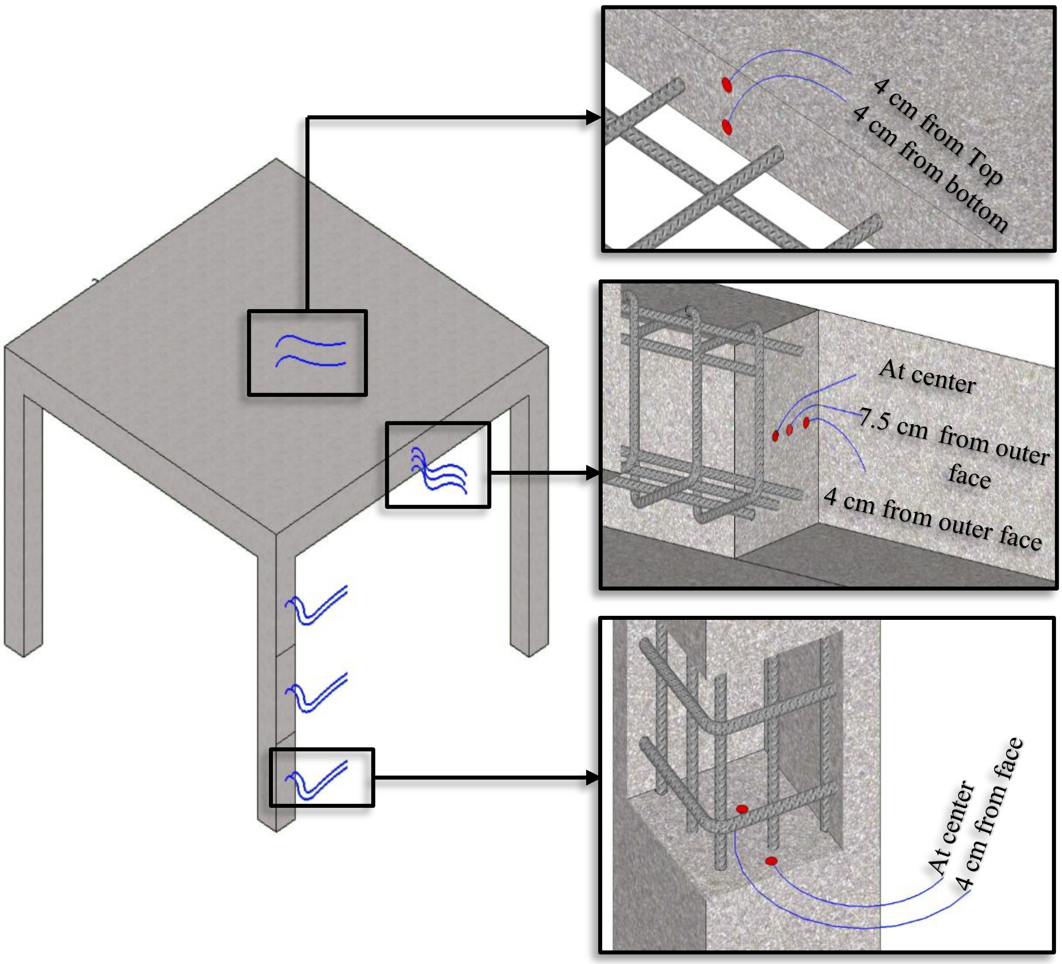

Figure 6: Location of thermocouples

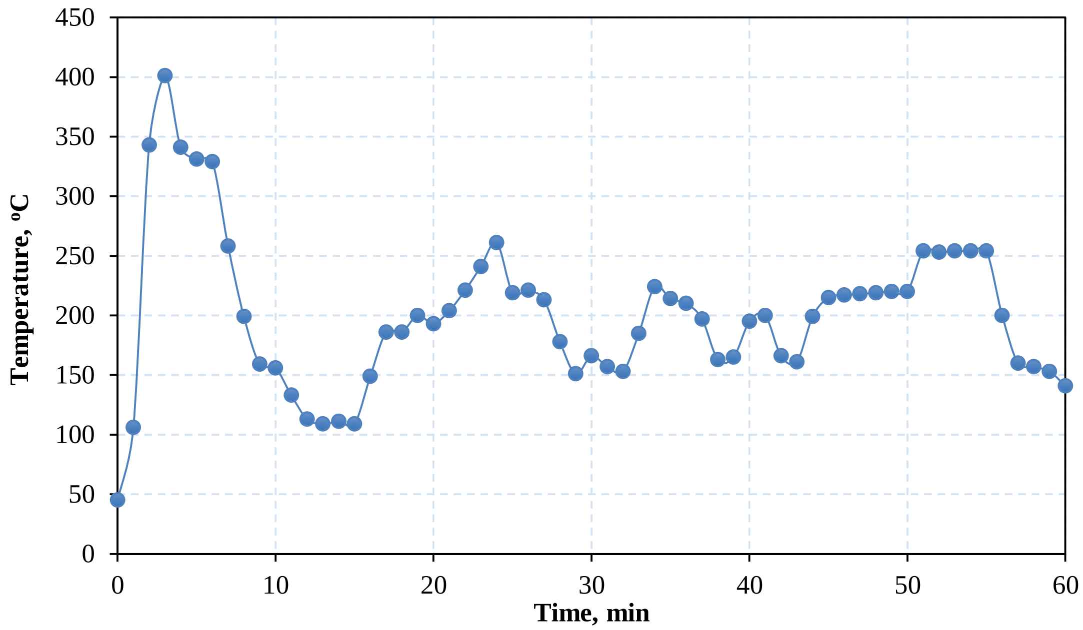

Figure 7: Average time-temperature relationship

temperature was increased. Flashover occurred between four and five minutes, after which, the fire advanced to its fully developed stage. Generally, the temperature rose significantly in the first 10 minutes and reached 400 °C due to the fire in the ignition step. The temperature then dropped to an average of 200 °C until the firing process was completed. The temperature variation inside the elements of the reinforced concrete frame is shown in Figures 8, 9, 10, 11 and 12. After the fire was burned all the wooden formwork of frame, then the frame was cooled in the air without using water.

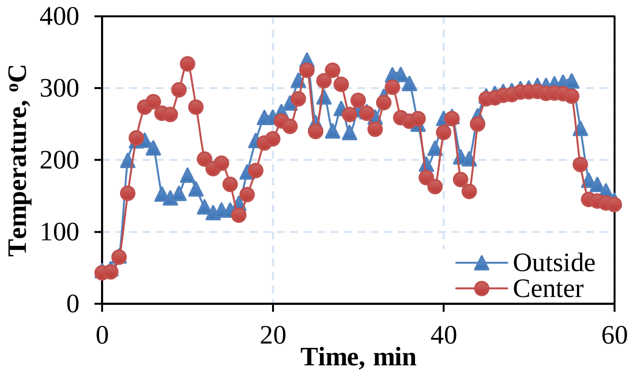

Figure 8: Time-temperature relationship of column at bottom

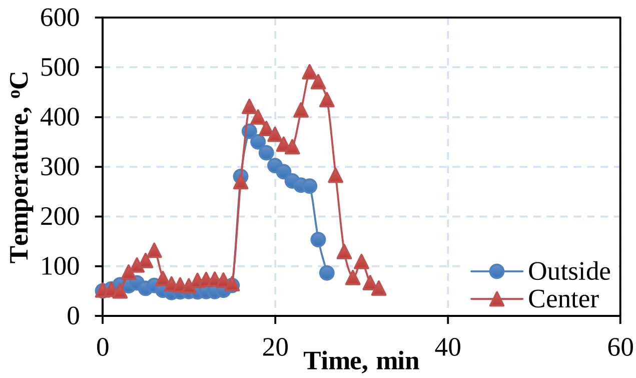

Figure 9: Time-temperature relationship of column at mid height

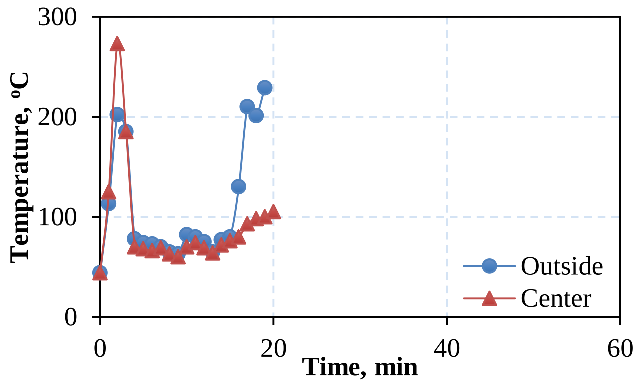

Figure 10: Time-temperature relationship of column at top

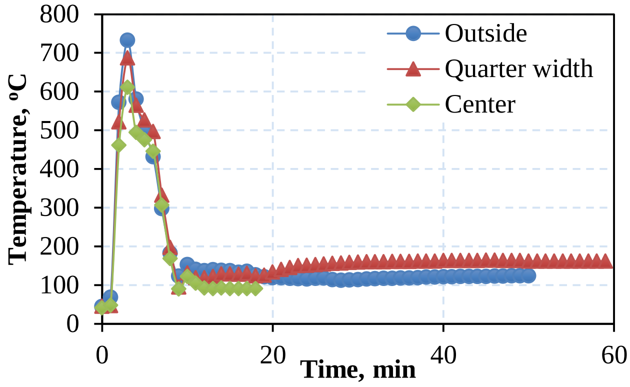

Figure 11: Time-temperature relationship of beam

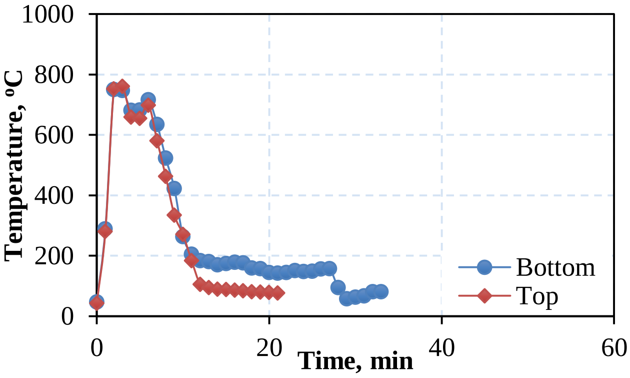

Figure 12: Time-temperature relationship of slab

2.4 Description of the Load Test

In particular, load testing was performed in a download method by uniformly distributing a load of sandbags with known weight over the entire area of the slab to check the deflection of the slab.

The design loads were simulated utilizing sandbags, and the weight of each sandbag was equal to 25 kg in order to be easy to installing and removing it on the structure. To distribute the sandbags on the slab equally, for this purpose, the slab divided into strips of 1m width to set the required load on one m^2. The sandbags were distributed precisely like a real load. These sandbags give the perfect response of the slab system and allow using lighter equipment at a low cost. The load test of the slabs was done in phases, as presented in the following sections.

Deflection measurements were taken in three different locations in the slab and the beams by installing dial gauges, while two locations were selected for column displacement in order monitoring the displacement during the load test, Deflection and displacement measurements were taken by 100-millimeter dial gauge which fixed on stands and set below the slab & beams and beside of column. The first dial gauges were installed at the center point of the slab, and the second and third dial gauge was mounted in the middle of the two beams, while the fourth and fifth dial gauges were installed beside the column in two directions and at mid-height as shown in Figure 13.

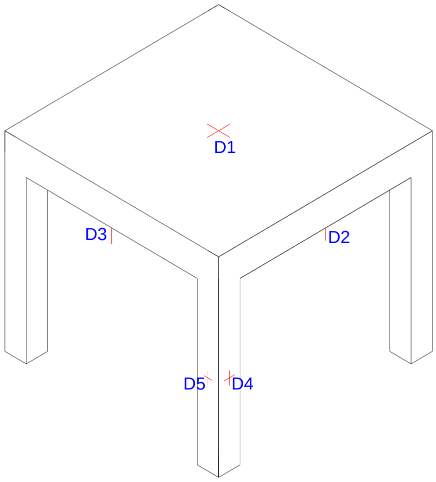

Figure 13: Location of installed dial gauges

2.5 Testing Procedure

The next section shows the conceptual steps followed in order to:

- Determine the value of the total test load magnitude during a preparatory phase.
- Obtain continuous structural assessment during the load test performance.

2.5.1 Protocols of Load Test

Both the ACI 318-19 chapter 27 (ACI 318-19, 2019) and ACI 437.2-13 (ACI 437.2-13, 2013) stated that the age of concrete structure should be higher than 56 days also stated that the 24 hrs load test consists in the monotonic loading of the structure up to the designed load level followed by a phase in which the load is sustained for a time period of at least 24 hrs. The two variables are considered for the principal evaluation, and they are:

•Dead load effect such as self-weight of the slab and superimposed load.
•Live load effect.

Thus, as suggested by ACI 318 (ACI 318-19, 2019) and ACI 437.2 (ACI 437.2-13, 2013), the total load (weight) applied to the slab can be calculated as follows.

Test Load Magnitude (TLM) to be applied is the larger of:

$TLM = 1.3\,(Dw + Ds) - D$ …..……….…….(1)

or $TLM = (1.0\,Dw + 1.1Ds + 1.6\,L + 0.5\,(Lr \text{ or } S \text{ or } R)) - D$ ………………………………. ……(2)

where:

L: Live load = 250 kg / m^2

Ds: Superimposed dead load= 230 kg/m^2

Dw: Self-weight dead load= 288 kg/m^2

Total Dead Load (D) = (Dw + Ds) = 518 kg/m^2

$TLM = ((1.3\,(Dw + Ds)) - D) = 1.3\times(518)-518$

$= 155$ kg/m^2

or $TLM = (1.0\,Dw + 1.1Ds + 1.6\,L + 0.5\,(Lr \text{ or } S \text{ or } R)) - D$

$= (1.0\times288 + 1.1\times230 + 1.6\times250)-518$

$= 423$ kg/m^2

$TLM = 423$ kg/m^2

2.5.2 Load Configuration

The load was applied uniformly throughout the area to the slab. Before installing sandbags, it should apply the superimposed dead load. For this purpose, concrete blocks were used as a superimposed dead load, and the load was 230 kg/m^2. The intensity of the load applied as determined in four layers of the sandbag, and a total load of these four layers is equal to the exactly calculated test load, which is distributed uniformly. In each layer of one square meter contains four sandbags (4×25kg=100 kg/m^2/layer).

The ACI requirements and standards for the structural performance must be considered and limited by two variables that are the maximum deflection and residual deflection. The limits of maximum deflection and the residual deflection are as follows:

$\Delta l \leq lt / 180$ ………………………...…….. (3)

$\Delta r \leq \Delta l / 4$ ……………………………..... (4)

Where:

Δl: the maximum measured deflection during the test.

Δr: residual deflection measured after the 24 hrs recovery period following complete removal of the load after the load test.

lt: span of slab on the short side taken as the smaller of the distance between the center of supports or clear distance between supports plus thickness h of the member.

If the maximum deflection *Δl* measured during the test is less than 1.27mm or deflection as a fraction of span length, *lt* is less than *lt/2000*; the residual deflection requirements are given by equation (4) could be ignored.

2.5.3 Load Testing Procedure

1. At first, the concrete blocks were located on the slab as a superimposed dead load.
2. After 24 hrs of installing the concrete blocks, the dial gauges (No. 1 to No. 5) were installed on to the center of the slab and center of two beams and mid-height of the column in two directions are located as shown in Figure 13.
3. The magnetic base was used to install dial gages, as shown in Figure 14.
4. All the initial readings were recorded before the testing.
5. The load (sandbags) were added step by step from 0%, 25%, 50%, 75%, and 100% of the maximum test load, and each load step is held for 2-3 minutes.
6. After each of the loading steps records, all dial gauges reading except for the maximum test load (100%) that has to maintain 24 hrs, as shown in Figure 15.
7. After 24 hrs, the test load should be removed then record all dial gauges reading.

3.RESULTS AND DISCUSSIONS:

3.1 Visual Observations

The visual observation was done for the fire affected reinforced concrete frames. Spalling, cracking, and discoloration status of some components of the structure have recorded.

Figure 14: Installation of dial gage by using the magnetic base

Figure 14: Applying full load by using sandbags

3.1.1 Spalling

Spalling is the violent or non-violent breaking off of layers or pieces of concrete from the surface of a structural element when it is exposed to high and rapidly rising temperatures, as experienced in fires with heating rates typically 20–30 ^{o}C/min. Spalling can be grouped into four categories: (a)aggregate spalling; (b) explosive spalling; (c) surface spalling; (d) corner/sloughing-off spalling. The first three occur during the first 20–30 min into a fire and are influenced by the heating rate, while the fourth occurs after 30–60 min of fire and is influenced by the maximum temperature. Surface and explosive spalling are violent, while corner/sloughing-off spalling is non-violent. It could also be argued that surface spalling is a subset of explosive spalling, which is the most serious, and hence most researched, a form of spalling (Khoury, 2000).

Spalling begins to occur when the concrete reaches an elevated temperature of 250 °C. From 250-420 °C, some spalling occurs. After reaching 300 °C, concrete starts to lose its strength. Within 550-600 °C, cement-based materials experience creep and lose their load-bearing capacity (Iffat and Bose, 2016). As shown in Figure 16, despite the observed surface spalling at the bottom of the slab, the test frames did not suffer any significant spalling (loss of big chunks) of concrete or loss of cover (exposing reinforcement) at any location in the test frame.

Figure 16: Spalling and change color occurred in slab

3.1.2 Cracking

When reinforced concrete is subjected to high temperatures due to fire, the losses in compressive strength of concrete and the reduction in its stiffness are related to gradual deterioration of the hardened cement paste and the destruction of the bond between the cement paste and the aggregates. Also, thermal expansion causes internal cracking and spalling of concrete, as well as the debonding of the reinforcement bars (Riad et al., 2017).

No deterioration in the strength of the burned frames after load testing was observed. Despite the observed inelastic excursions, the test frames did not suffer any significant structural damage and noticeable cracking.

3.1.3 Discoloration

The color change of concrete is a significant indication of the effect of fire. Color change provides a perfect visual guide to estimating the temperature range to which concrete has been exposed at various depths during the fire. However, the change in color is not directly related to a change in mechanical properties, but the occurrence of a color change indicates a temperature range where the mechanical properties may start to decrease. Concrete may change color from its typical grey to pink or red between 300 – 600 °C, whitish-grey between 600 – 900 °C and buff between 900 – 1000 °C (Short et al., 2001).

After exposing the reinforced concrete frame to the natural fire by burning its wooden formwork, the inner face color of the frame was black due to wood smoke, but no major change in the color of concrete itself was observed as shown in Figure 16.

3.2 Fire Effect on Compressive Strength

The compressive strength of concrete cubes is plotted in Figure 17 as a function of the age of concrete. Before burning the wooden formwork of reinforced concrete frame, the concrete control specimens were put on the iron table, which was under the concrete frame. After burning, the concrete specimens were left in the laboratory environment without curing until the compressive strength test day. However, the test of the compressive strength of mature concrete cubes was done in the laboratory in which concrete cubes were curried totally until the age reached a specified day without being burned.

When the temperature is 150 °C, the evaporation of free water in the concrete specimens increases capillary pressure, which generates pore and capillary cracks inside the specimens, and then reduces concrete compressive strength (Chen et al., 2009). When the specimens are exposed to 350 °C, the free water in the specimens is subjected to continuous evaporation. Although the bound water is released from the cementitious substance and enhanced the bonding action (Khoury et al., 2002, Guo and Li, 1993), the continuous development of pores and capillary cracks finally results in a further decrease in concrete's strength.

The significant difference in deformation, the self-expansion, and breakage significantly reduce compressive strength (Liu et al., 2005). Especially in the early phase of curing, concrete has a lower strength and relatively less dense microstructure, which is easy to crack and intensified the decrease in strength under high temperatures.

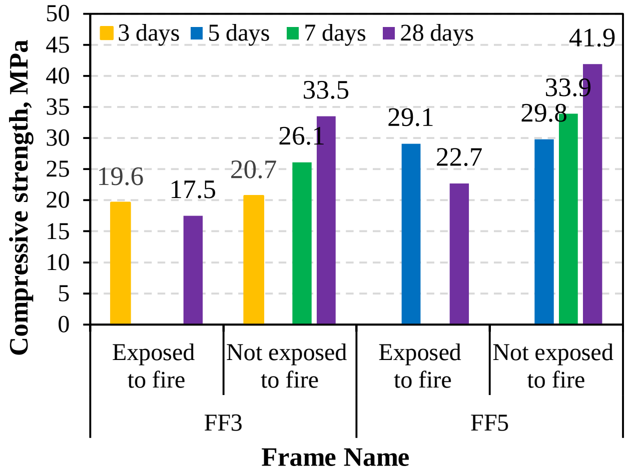

Figure 17: Effect of fire on compressive strength of concrete cubes

As shown in Table 3, the concrete cubes of frame FF3 exposed to high temperatures on the 3rd day have low strength. After being exposed to high temperature, the internal water rapidly evaporates, and many pores and microcracks form, the bonding between paste matrix and aggregate is broke. Nevertheless, the concrete cubes of frame FF5 exposed to high temperatures on the 5th day have a higher hydration degree than that on the 3rd day, but its strength is still lower, because high temperatures break the internal structure of concrete subjected to high temperature on the 5th day, and the hydration in subsequent rest that can offset the strength loss is weaker than the strength loss in the concrete exposed to high temperature on the 3rd day. Therefore, the reduction strength ratio of concrete with initial curing of 5 days is 2.4 and is lower than that of concrete with an initial curing of 3 days is 5.3. However, the reduction in compressive strength of concrete cubes of both frames FF3 and FF5 at age 28 days was 47.8% and 45.8% respectively because when concrete cubes were exposed to fires the curing process of concrete cubes were stopped from age 3 days and 5 days and the hydration process also stopped.

Table (3) Compressive strength of concrete cubes

Age (Days)	FF3 Exposed to Fire	FF3 Not Exposed to Fire	FF3 Reduction Due to Fire %	FF5 Exposed to Fire	FF5 Not Exposed to Fire	FF5 Reduction Due to Fire %
3	19.6	20.7	5.3	-	-	-
5	-	-	-	29.1	29.8	2.4
7	-	26.1	-	-	33.9	-
28	17.5	33.5	47.8	22.7	41.9	45.8

(Compressive strength, MPa; Frame Name)

3.3 Slab Deflection

The main effect of firing the formwork of the concrete frame at an early age is on the slab element because of the slab element wider and more exposed to fire than beams and columns. After firing the formwork of reinforced concrete frame, the slab becomes not having formwork support; therefore, the slab loaded by its self-weight at an early age. In order to find the effect of firing of it is formwork, the load test was done by sandbags, the results of the slab deflection of three reinforced concrete frames shown in Table 4 and Figure 18. According to ACI 437 & ACI 318, the slab deflection after 24 hrs of loading should be less than 21.22 mm while the slab deflection of frame FF3, FF5, FN were 2.4, 1.93 and 1.15 mm respectively and satisfied the ACI limitation. Besides, the three frames satisfied the ACI 437 and ACI 318 limitation of residual slab deflection after 24 hrs of unloading.

As shown in Table 4, the slab deflection of frame FF3 and frame FF5 after 24 hrs of loading was higher by 109% and 68%, respectively, if compared to slab deflection of frame FN, which not exposed to fire. However, the residual slab deflection of frame FF3 and frame FF5 after 24 hrs of unloading was higher by 175% and 33%, respectively, if compared to slab deflection of frame FN.

The deflection expression is a function of load, span, and end conditions divided by the flexural rigidity (EI) (modulus of elasticity of concrete and moment of inertia of the member) (Darwin et al., 2016). The relation between deflection and the modulus of elasticity is inverse and as known the modulus of elasticity of concrete is depending on the compressive strength of concrete; therefore, the increase in slab deflection of frame FF3 and FF5 is due to decrease in compressive strength of concrete after exposure to fire as shown in Table 3.

Table (4) Slab Deflection

Loading stage	Deflection, mm		
	Frame Name		
	FF3	FF5	FN
Initial	0	0	0
1/4 loading	0.1	0.18	0.16
1/2 loading	0.25	0.31	0.37
3/4 loading	0.74	0.53	0.51
Full loading	1.8	0.95	0.59
After 24 hrs loading	2.4	1.93	1.15
After 24 hrs unloading	0.33	0.16	0.12

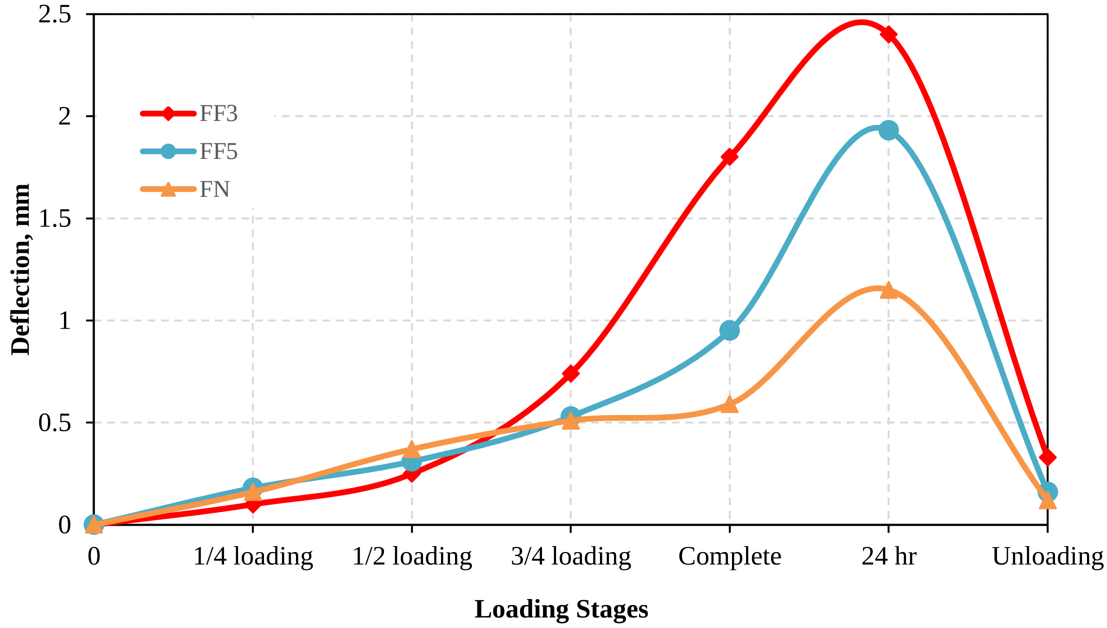

Figure 18: Slab Deflection Curve

3.4 Beam Deflection

Beams in reinforced concrete frames are essential elements that carry loads from the slab. When a building structure sustains a fire accident, a beam with three surfaces and a slab with one surface exposed to fire are the most common working conditions (Guo and Shi, 2011).

The average mid-span deflections of the beam elements, which were loaded by sandbags, are listed in Table 5 and shown in Figure 19. The deflections were recorded at each stage of the load test at mid-span of the beam. As shown in Figure 19, it can be noted that the increase in the midspan deflection of frame FF3 when compared to frame FF5 due to earlier exposed to fire. This increase in midspan can be attributed to the fact that burning causes a reduction in beam stiffness, which is mainly due to the reduction in the modulus of elasticity of concrete. Furthermore, it can be noted from Figure 17 the midspan deflection of beams of two frames FF3 and FF5 for all stages of load test are near to each other and far from frame FN.

The midspan of beam deflection of frame FF3 and frame FF5 after 24 hrs of loading was higher by 71% and 55%, respectively, if compared to beam deflection of frame FN, which not exposed to fire. Also, the residual beam deflection of frame FF3 and frame FF5 after 24 hrs of unloading was higher by 50% and 38%, respectively, if compared to beam deflection of frame FN, which not exposed to fire.

Table (5) Beam Deflection (Average)

Loading stage	Deflection, mm		
	Frame Name		
	FF3	FF5	FN
Initial	0	0	0
1/4 loading	0.035	0.025	0.015
1/2 loading	0.065	0.06	0.03
3/4 loading	0.13	0.115	0.09
Full loading	0.19	0.165	0.1
After 24 hrs loading	0.325	0.295	0.19
After 24 hrs unloading	0.15	0.12	0.025

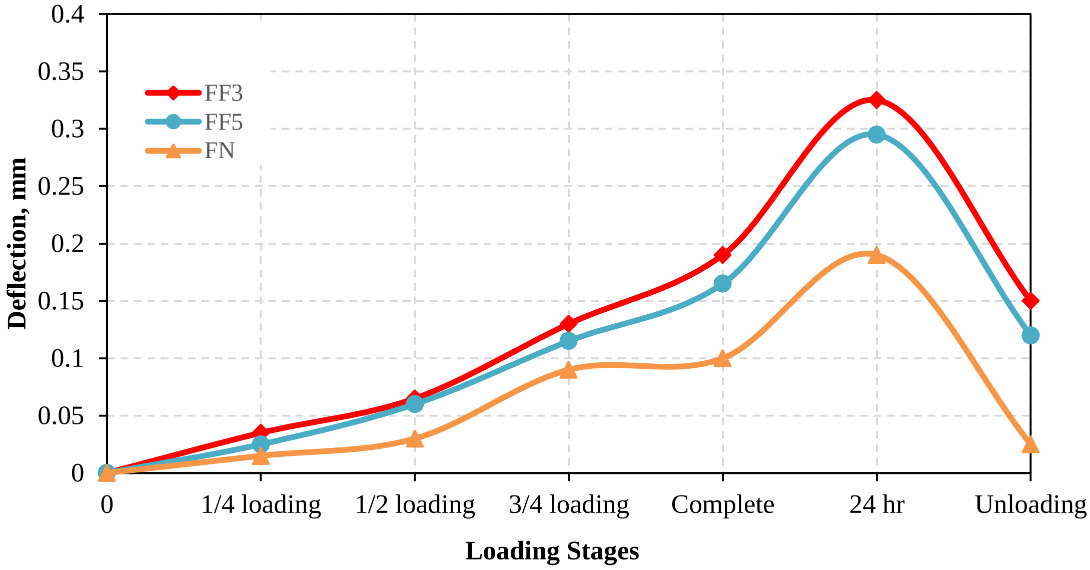

Figure 19: Beam Deflection Curve (Average)

3.5 Column Deflection

Reinforced concrete columns are essential elements because these are primary load-bearing members, and a column could be crucial for the stability of the entire structure. Usually, reinforced concrete columns simultaneously bear the actions of the bending moment and axial compression. When a building sustains a fire accident, four, three, two, or one surface of the rectangular cross-section of an eccentric compressive member may be exposed to high temperature, depending on its position (Guo and Shi, 2011). However, in our experimental investigation, the columns exposed to fire by two faces.

The column displaced at the mid-height of the column into the outside (buckled) after loading. As shown in Table 6 and Figure 20, the column displacement of all frames was near to each other at loading stages, but after 24 hrs of loading, the displacement of FF3 significantly higher than frame FF5 and FN. The Column displacement of frame FF3 and frame FF5 after 24 hrs of loading was higher by 97% and 49%, respectively, if compared to column displacement of frame FN, which not exposed to fire. However, the residual column displacement of frames FF3 and FF5 after 24 hrs of unloading was higher by 113% and 41%, respectively, if compared to column displacement of frame FN, which not exposed to fire.

Table (6) Column Displacement (Average)

Loading stage	**Displacement, mm**		
	Frame name		
	FF3	**FF5**	**FN**
Initial	0	0	0
1/4 loading	0.015	0.025	0.005
1/2 loading	0.045	0.07	0.03
3/4 loading	0.12	0.11	0.105
Full loading	0.26	0.19	0.12
After 24 hrs loading	0.77	0.58	0.39
After 24 hrs unloading	0.17	0.113	0.08

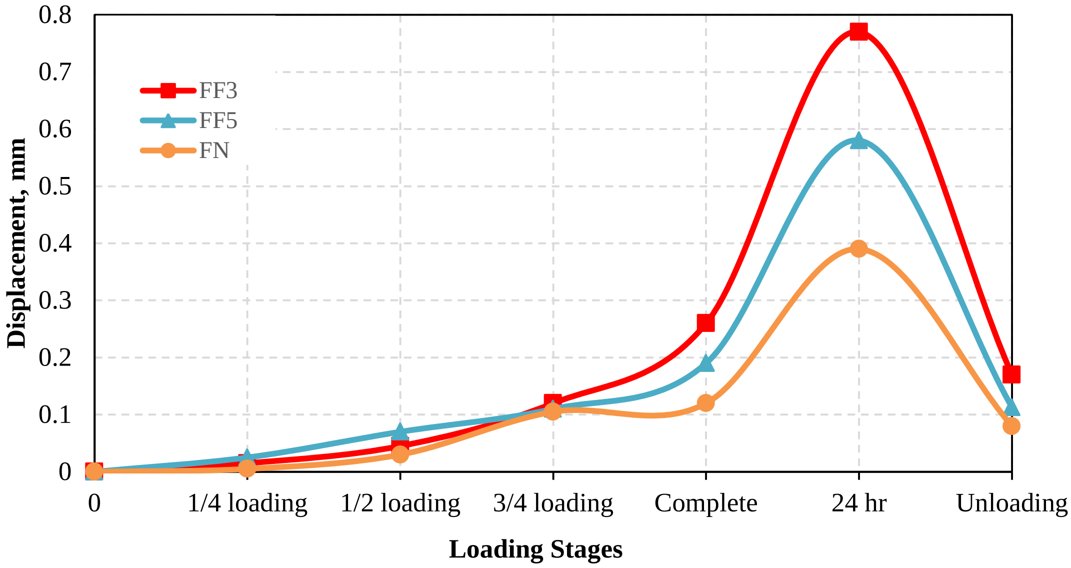

Figure 20: Column Displacement Curve (Average)

4. CONCLUSION:

The following conclusions were drawn based on the observation and analysis of the test results. Using the ACI load test, the concrete quality of early-age structural elements was evaluated under natural fire:

1-The application of the load test does not require special loading devices (hydraulic cylinders, actuators, or others) as the load can be applied using sandbags, However, the structure remains inaccessible for at least 48 hrs during the application of the load test.

2-In general, the frame that was exposed early to natural fire (FF3) was more affected than the frame that was later exposed (FF5), and its slab deflection increased to approximately 109% and 68% respectively compared to the frame that was not exposed to fire (FN), due to earlier firing wooden support of the slab.

3-After comparing the result of the load test of all reinforced concrete structures with limits in ACI criteria, it can be concluded that all reinforced concrete frame verified the ACI limitation.

4-It can be concluded that if a fire accident occurred in any reinforced concrete structure at an early age due to any reason, the structure did not significantly affect by the fire because the time was short and fire exposure was not high.

Conflict of Interest

Authors declare that there is no conflict of interest

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