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Investigation of the Behavior and Properties

of Reinforced Concrete Slabs

Exposed to Fire Flame

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Submitted to the College of Engineering of the
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By

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بسم الله الرحمـــن الرحـــيم

وَأَنزَلَ اللَّهُ عَلَيْكَ الْكِتَابَ وَالْحِكْمَةَ وَعَلَّمَكَ مَا لَمْ تَكُن تَعْلَمُ وَكَانَ فَضْلُ اللَّهِ عَلَيْكَ عَظِيماً *

صدق الله ألعلي العظيم

سورة النساء
الآية(113)

Dedication

To

My Faithful Family

Who Burn the candle at

Both Ends for my

Ambition

The Eyes That Observe

Closely

The Hearts That

Supplicate for Success

Muna

2005

Acknowledgement
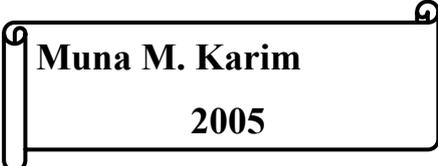
Certification

We certify that this thesis titled **"Investigation of the Behavior and Properties of Reinforced Concrete Slabs Exposed to Fire Flame"** , was prepared by (Muna Mohammed Karim) under our supervision at Babylon University in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

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Date :   /   / 2005   Date :   /   / 2005

Abstract

In the case of exposing a reinforced concrete structure to accidental fire, if this structure remains standing, an assessment of its residual capacity is needed .

This study aims to reveal the effect of fire flame on the behavior of reinforced concrete slab specimens during and after exposure to fire. The experimental work includes casting and testing twenty four reinforced concrete slab specimens with dimensions of (600×600×40mm) divided into two series with target compressive strength of (30 and 38 MPa) designated as series A and B respectively .Each series was reinforced with two steel ratios of (0.00492 and 0.00875) .

Sixteen reinforced concrete slab specimens were subjected to fire flame at the lower surface only to reach temperatures around of (400,500 and 600 $^{\circ}$C) for one hour, then they were cooled gradually to room temperature. After that, they were tested in flexure to failure under uniformly distributed load .Control specimens of concrete(cubes, cylinders and prisms) and steel bars were subjected to the same environmental conditions that were subjected to the slab specimens to determine the changes in compressive strength ,modulus of flexural ,modulus of elasticity and splitting tensile strength of concrete and steel properties .These reinforced slab specimens were tested for weight loss, ultrasonic pulse velocity and rebound number before and after exposure to fire.

Based on the results of this work, it can be concluded that the ultimate load capacity of the reinforced slab specimens decreased with the increase in fire temperatures. The higher decrease occurred in the slab specimens having higher steel ratio and subjected to 600 °C .The residual load capacity of reinforced slab specimens at 600 °C ranges between (84.2-86%) for slab specimens of series (A) which were reinforced with steel

ratios of (0.00875,0.00492) respectively and (83.7-85%) for slab specimens of series (B) with the same steel ratios. The fired slab specimens were found to be capable of resisting the service load but with a decreasing factor of safety. The non-destructive test results showed more reduction in the test results after exposure the slab specimens to fire compared with the control specimens.

Results of the simplified approaches (Resheidat and Beeby methods) for the structural analysis of the reinforced concrete slab specimens subjected to fire flame , showed a good agreement with the experimental results, which conforms the validity of the analytical methods used to predict the structural behavior of the reinforced concrete slab specimens subjected to high temperatures.

Eight reinforced slab specimens were uniformly loaded to two levels of loading and exposed to fire flame. The first level was 40% of the ultimate load of the reinforced slab specimens (22,32,23.2,33.6 kN) and the other was the calculated service load (34.1,47.5,35.5,48.8 kN) . The deflections of these slab specimens were measured .With respect to the increment in deflection of these slab specimens ; the fire resistance durations were found to be (126,117,117 and 105 minutes) for slab specimens which were loaded with 40% of the ultimate load and (110,96,100 and 90 minutes)for slab specimens which were loaded with service load .

Mathematical models for the prediction of the ultimate strength and maximum deflection were proposed in this study. These models use multivariable non- linear regression equations to evaluate good correlation coefficient (0.95) with less difference between the predicted and observed values.

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Notations

Item	Description

As	Area of steel reinforcement (mm^2)
b	Width of the slab specimen (mm)
cn	Depth of the neutral axis from the compressive fiber of the slab specimen (mm)
D	Slab stiffness.
d	Effective depth of the slab specimen (mm)
(E_c)a	Modulus of elasticity of concrete after exposure to fire flame temperature(GPa)
(E_c)b	Modulus of elasticity of concrete before exposure to fire flame temperature(GPa)
E_s	Modulus of elasticity of the steel (GPa)
fy	Yield tensile stress of steel reinforcing bar(MPa)
fu	Ultimate tensile strength of steel reinforcing bar(MPa)
(fc)a	Compressive strength of concrete after exposure to fire (MPa)
(fc)b	Compressive strength of concrete before exposure to fire (MPa)
(fr)a	Flexural strength after exposure to fire (MPa)
(fr)b	Flexural strength before exposure to fire (MPa)
(fsp)a	Splitting tensile strength after exposure to fire (MPa)
(fsp)b	Splitting tensile strength before exposure to fire (MPa)

Notations

Item	Description
h	Depth of the slab specimen(mm)
$\mathbf{I_{cr}}$	Second moment of area of cracked section (mm^4)
L	Clear span (mm)
$\mathbf{M_{cr}}$	Cracked moment, N.mm .
$\mathbf{M_{nt}}$	Nominal ultimate moment, N.mm .
n	Modular ratio (the ratio of modulus of elasticity of the steel to that of concrete at (T°C)) .
$\mathbf{q_{cr}}$	Cracked load, N/mm^2.
$\mathbf{q_u}$	Ultimate load, N/mm^2.
$\mathbf{R_{yt}}$	The ratio of the yield tensile stress of reinforcement steel at (T°C) to that at 25 °C.
$\mathbf{R_{fc}}$	The ratio of the compressive strength of concrete at (T°C) to that at 25 °C.
(RN)a	Rebound number test after exposure to fire
(RN)b	Rebound number test before exposure to fire
$\mathbf{S_1}$	Stress corresponding to a longitudinal strain (0.00005), MPa
$\mathbf{S_2}$	Stress corresponding to 40 % of ultimate load, MPa
(U.P.V)b	Ultrasonic pulse velocity test before exposure to fire, km/sec
(U.P.V)a	Ultrasonic pulse velocity test after exposure to fire, km/sec
wt	% Weight loss of slab specimens.
$\in_2$	longitudinal strain produced by stress S_2
υ	Poisson's ratio .
Δ	Mid – span deflection of slab specimen, mm .

Chapter one

Introduction

1-1 General

One of the problems confronting buildings is the exposure to elevated temperatures that should be provided with sufficient structural fire resistance to give occupants time to escape before strength and, or stability failure ensue.

The most common type of exposing reinforced concrete structural members to elevated temperatures is by accidental fire in buildings, which may be looked at as the worst heating condition of concrete structures. In a short period , may be less than one hour, the temperature of exposed concrete surface is raised up to about 800 $^{\circ}$C[1]. Another type of heat exposure may be found in some industrial installations where concrete is used in places exposed to sustained elevated temperatures as in foundation for blast furnaces and coke batteries, furnaces wall and dampers, industrial chimneys and flues, floors below boilers and kilns and nuclear- reactors[2].

High temperatures due to fire have a significant effect on the strength and deformation characteristics of various structural components, such as columns, beams, slabs, shear wall, etc. Therefore, a good understanding of the structural behavior and response of reinforced concrete exposed to fire is important towards the saving of human lives and avoiding costly damage of structures [3] .

The effect of fire on structural members depends on different factors such as the temperature, nature, and distribution of fire loading, ventilation and compartment size.

One of these structural members is reinforced concrete slabs. This type of structural members may be subjected to high temperatures during fire, which will cause change in properties of its constituents, namely concrete and steel, therefore, in reinforced concrete structural design of buildings it may be necessary to design not only for the dead and live loads, but also for fire resistance[4]. The fire resistance of reinforced concrete slab is expressed in terms of fire endurance as determined by standard fire tests.The fire endurance of the slab is then expressed as the time duration necessary to induce failure. The ASTM E-119 [5]standard method for floor slab fire tests specifies that a fire test will be considered as successful if :

1-The slab does not allow the passage of flames or gasses hot enough to ignite oven-dried cotton, and

2-The rise in temperature of the unexposed surface is less than 120 $^\circ$C(250 $^\circ$F) .For steel reinforcement , these standards specify that steel temperature should not exceed 540 $^\circ$C (1000 $^\circ$F)within the rated fire endurance .

1-2 Research Significance

Many researchers studied the effect of high temperatures on concrete and reinforced concrete members. They worked on the strength and deformation properties at elevated temperatures. However, very little work was done on specimens exposed to direct fire flame.

The main objective of this investigation is to develop fundamental information on the behavior of reinforced concrete slabs after and during the exposure periods to fire flame, predicting the damage by fire and the residual stiffness of structural members.

The main goals of this study are as follow:

1-Studying the fire flame effect on the stiffness and strength of reinforced concrete slabs and comparing the results with control specimens.

2-Investigating the fire endurance of reinforced concrete slab specimens subjected to fire flame without loads.

3-Studying the behavior of fired slab specimens under loading and their fire resistance ratings (durations).

4-Predicting the fire flame effect on the slab specimens by using non-destructive tests to estimate the degree of damage.

5-Studying the fire flame effect on the weight loss of slab specimens.

6-Studying the fire flame effect on some mechanical properties of concrete specimens, such as compressive strength, modulus of rupture (M.O.R), splitting tensile strength and modulus of elasticity.

1-3 Research Layout

The present research includes five chapters:

Chapter two deals with review of previous studies, including the effect of fire flame on the mechanical properties of concrete, steel reinforcement, and reinforced concrete members (beams, frames and slabs)

Chapter three deals with materials considered throughout this investigation, mix proportions, experimental program and tests procedures.

In Chapter four, the results of experimental tests are presented and discussed.

Finally, chapter five is devoted to the main conclusions drawn throughout the study in addition to the recommendations for further studies.

Chapter Two

Literature   Review

2-1 The Effect of High Temperatures on Reinforced Concrete.

2-1-1 Introduction

The behavior of reinforced concrete structures exposed to fire depends on the thermal properties of steel and concrete, strength and stiffness of the concrete and steel at elevated temperatures, and on the ability of structure to redistribute internal forces during the course of the fire [6].

Many studies have been carried out to investigate the effect of high temperature on concrete, steel reinforcement and reinforced concrete members. It is clearly stated that both concrete and steel are affected by exposure to elevated temperatures.

Some of these investigations, which tended to evaluate the effect of exposure of concrete to high temperatures are discussed here.

2-1-2 Effect of High Temperatures on Concrete Properties.

Malhotra [7] in 1956 investigated the effect of high temperatures on the compressive strength of concrete .Tests have been carried out using cylinders of (51mm) diameter by (102mm) height made with ordinary Portland cement, river sand and gravel aggregate , having four water – cement ratios (0.4,0.45,0.5,0.65) . There were three testing conditions when the specimens were heated to a given temperature in an electric furnace. The first group was tested in the hot state, the second group was heated to a

given temperature under a constant stress and tested in the hot state, and the third group was allowed to cool gradually and tested to find the residual strength after cooling .The test results showed that the effect of temperature (up to 600 $^\circ$C) on compressive strength of concrete was independent of water- cement ratio within the range used in mixing.The author found that the reduction in the compressive strength being smaller for lean mixes than for rich mixes . He also observed that the specimens which are under a constant stress during the period of heating exhibit a smaller reduction in strength than the specimens under no stress. Finally the residual strength of heated concrete still shows further reduction in strength on cooling , being approximately 20% less than the corresponding hot strength in temperature range from   200 to 450 $^\circ$C   for 1 : 4.5 and 1 : 6 concrete mix .

Harada et al.[8] in 1972 studied the effect of elevated   temperatures on the concrete properties. The investigated properties were compressive and tensile strengths. Concrete cylinders of (50×100mm) were subjected to a slow rate of heating (1.5 $^\circ$C/min) for one hour exposure duration and cooled gradually in the air and then tested after the exposure to high temperature ranging between (100-800°C). They found that the reduction in compressive strength was about 60% at 400C for all types of concrete. The same reduction occurred in the tensile strength. Other concrete cylinders of (100×200mm) were subjected to long duration exposure. These specimens were heated at slow rate of heating (0.5 $^\circ$C/min) for 72 hours inside the electrical furnace and gradually cooled for 48 hours. They noticed a linear reduction in compressive strength with the increase of temperature. They found that the percentage of residual strength was about (80,75,60%)at temperatures of (100,300 and 450 $^\circ$C) respectively.

Nuri[9] in 1983 studied the effect of high temperatures on some properties of concrete. Concrete cylinders (102×203mm) were used for

compressive and splitting tensile strengths. The specimens were exposed to temperatures in the range of (20-600 $^{\circ}$C) and the periods of exposure of heating were(30,60 and 90 minutes)at different ages of concrete (3,7,28,and 60 days). The test results showed that the compressive and splitting tensile strengths decrease with the increased temperature. He noticed that when concrete is heated at ages (3,7 and 28 days) from (150 – 300 $^{\circ}$C) considerable recovery of concrete compressive strength was accurred,whereas splitting tensile strength was suffered further losses. After 300 $^{\circ}$C exposure, concrete retained from (59-102%) and (44 – 100%) from original compressive and tensile strength respectively. He also observed that after 600 $^{\circ}$C,concrete retained (28 – 64%) ,and (20 –62 %) from original compressive and splitting tensile strengths respectively .

Al-Ausi and Faiyadh[10] in 1985 investigated the effect of heating to high temperatures and cooling in the air and in the water on the compressive strength of concrete using 150mm cubes. The concrete mix was 1: 2: 4, w/c ratio were (0.4, 0.45, and 0.5), the specimens were exposed to different levels of heat up to 700 $^{\circ}$C for different periods of exposure. Specimens were tested hot, others were tested after cooling in the air and the third group was tested after cooling in the water for two hours. Figure (2-1) shows the compressive strength for different condition of testing. They noticed that the compressive strength was not affected significantly after heating to 100 $^{\circ}$C. They found that there was an increase in the compressive strength of air- cooled specimens, which were heated to 200 $^{\circ}$C when compared with hot tested specimens. They noticed further reduction in compressive strength when the specimens were heated beyond 200C, the reduction was 88-91% at 700C.
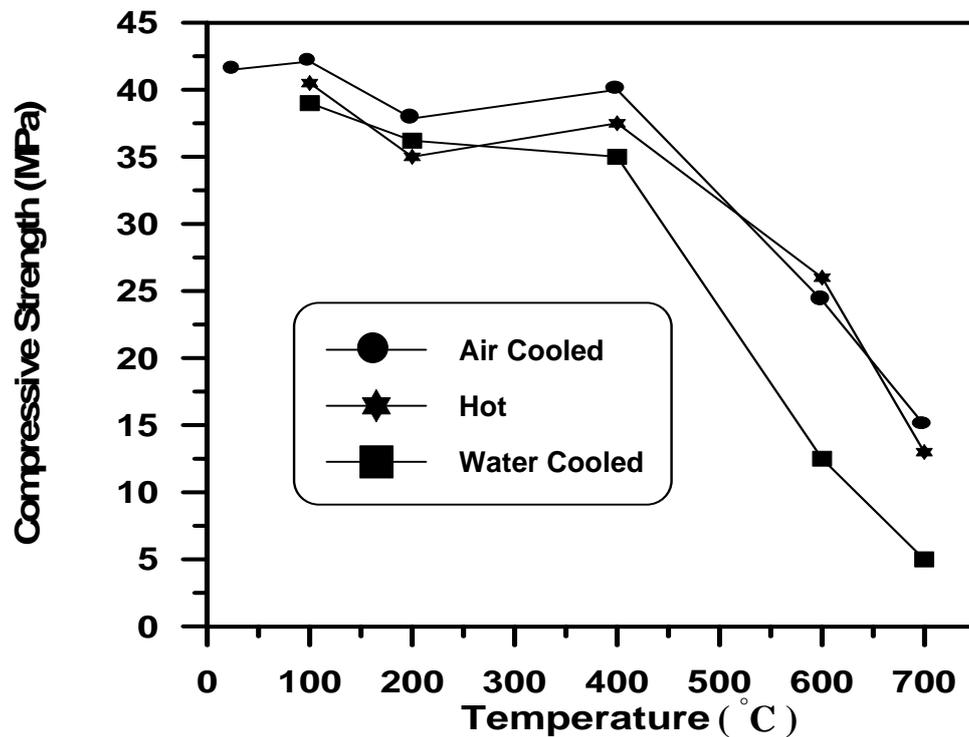
Figure (2-1): Relation between temperature and compressive strength, heated for 90 minutes [10].

Khoury et al [11] in 1985 observed that the change of length or (volume) of cement paste, exposed to elevated temperatures, results from complex processes: an ordinary thermal expansion based on kinetic molecular movements, and hydrothermal volume change associated with the movements of internal moisture. Up to a temperature of 100 $^\circ$C, the paste expands; then it contracts at higher temperatures as it shown in Figure (2-2). The aggregate, depending on the type of their rock origins, had different expansion behaviors during heating. The thermal incompatibility between the aggregates and cement paste should be as small as possible to eliminate deterioration in concrete strength.

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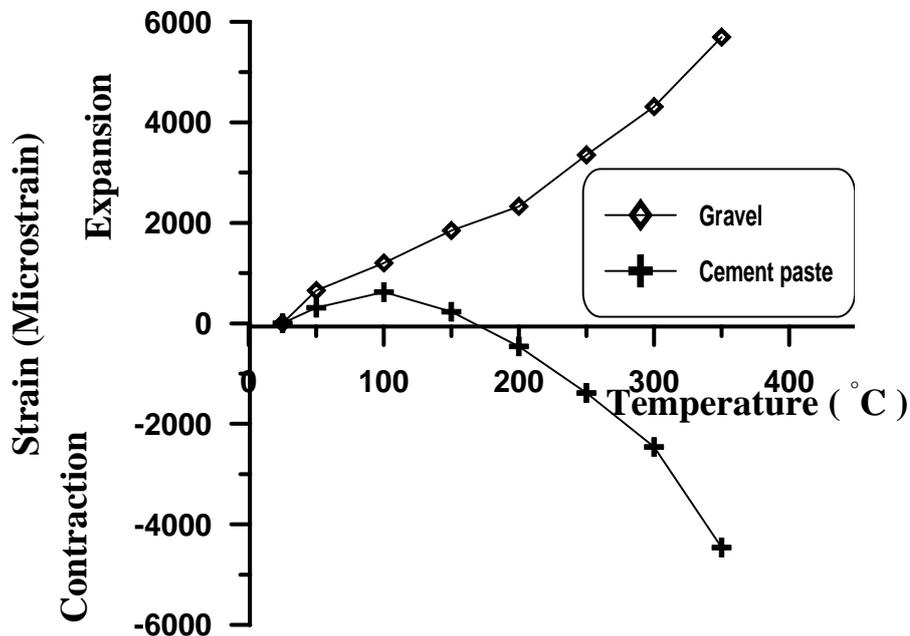
Figure (2-2): Free thermal strain of cement paste and gravel [11]

Mohamedbhai [12] in 1986 studied the effect of various time of exposure and rates of heating and cooling on the residual compressive strength of concrete, the test specimens were 100 mm cubes. The researcher concluded that the exposure time beyond one hour was affected the residual strength significantly and the bulk of strength loss occurred within the first two hours, but the effect was diminished as the level of exposure was increased, Figure (2-3) . The residual strength of concrete after one hour of exposure to(200,400,600 and 800 $^\circ$C)were about (80 ,70 ,60 ,and 30%) respectively of their unheated strength. The corresponding residual strengths after two hours of exposure were about (70, 60, 45 and 25%) . He also showed that the rates of heating and cooling had no effect on the residual strength of concrete heated to 600 $^\circ$C and beyond. At these temperatures, the most of the moisture was removed and the most of the microcracking occurred. However, these variables had a very significant

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effect on residual strength of concrete heated to lower temperatures, this can be seen from Figures (2-4) and (2-5).
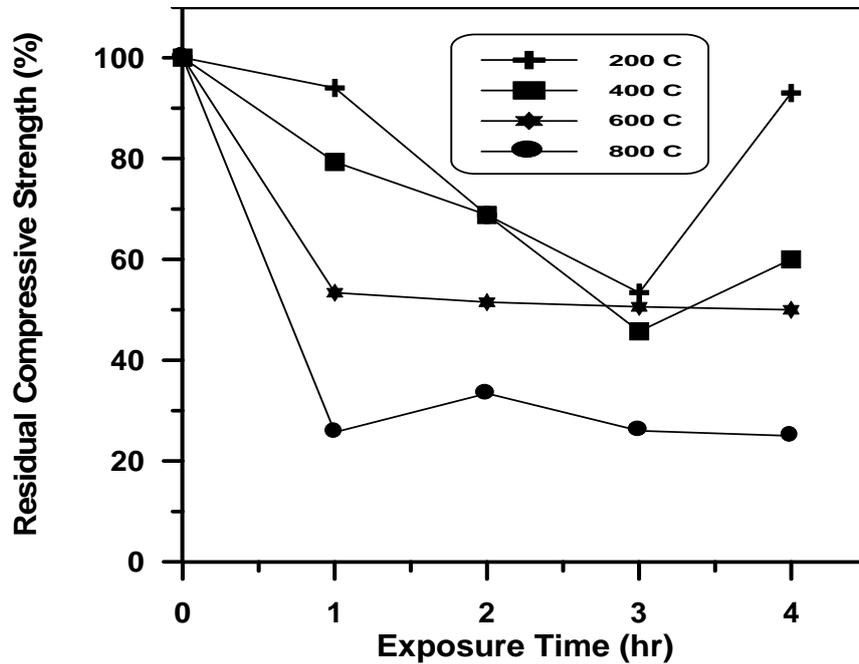
Figure (2-3): Effect of exposure time on residual compressive strength of slowly heated concrete [12] .
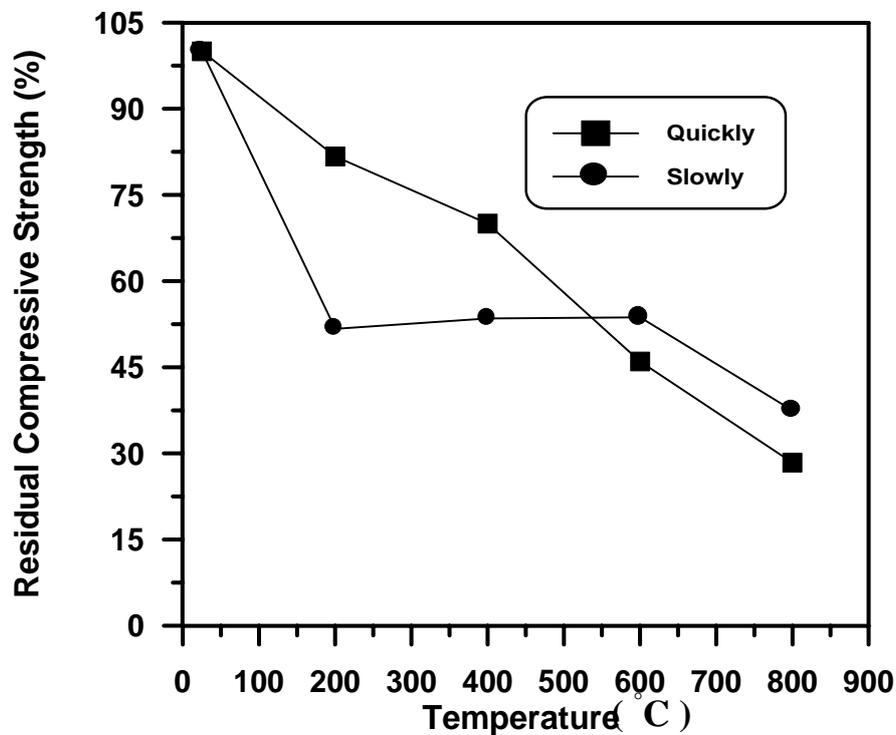
Figure (2-4): Effect of rate of cooling on the residual compressive strength of concrete slowly heating [12] .

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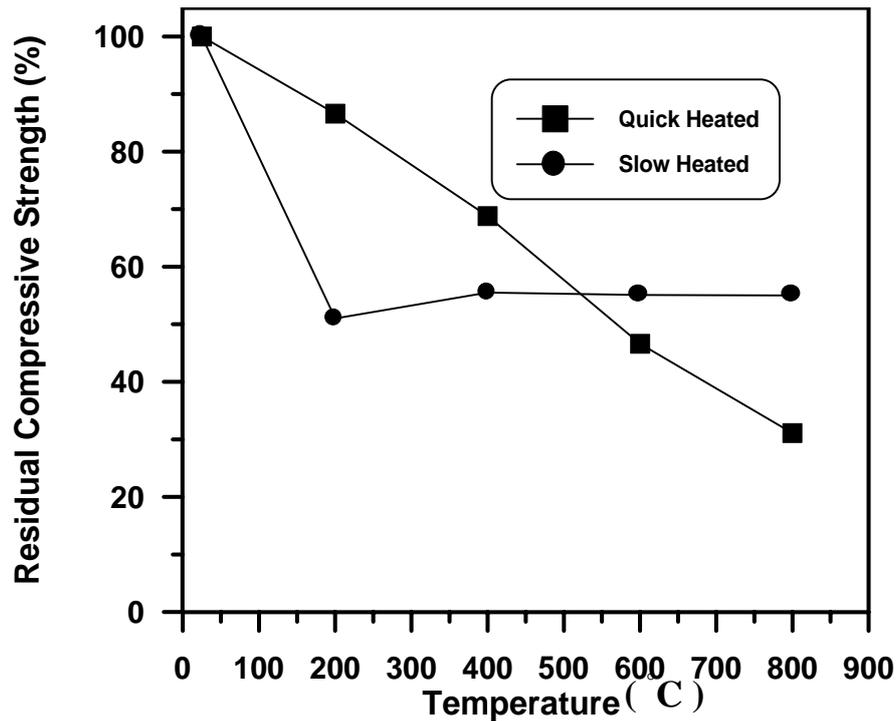
Figure (2-5): Effect of rate of heating on residual compressive strength of slowly cooled concrete [12] .

Essa [13] in 1999   studied the effect of burning by fire flame on some mechanical properties of concrete. The investigated properties were concrete compressive strength and density, Ultrasonic pulse velocity (U.PV) and Schmidt rebound number were tested. The specimens were heated up to two temperature levels 500 $^\circ$C (achieved by subjecting the cubes to direct fire flame from petroleum gas burner) and 800 $^\circ$C (achieved by using an oven). The heating durations were 1 and 2 hours for the specimens exposed to 500 $^\circ$C, while it was 1 hour for the specimens exposed to 800 $^\circ$C. The researcher found that the reduction in compressive strength   ranged between   (23-31%)and 39% at 500 $^\circ$C ,when the periods of burning were 1 and 2 hours respectively. At 800 $^\circ$C, the reduction at 1 hour was 77% from the original strength. He also found that the reduction in density was ranged between (10-14%) and (22-24%) at 500 $^\circ$C, for periods of 1 and 2 hours respectively. The ultrasonic pulse velocity (U.P.V)

and rebound number (R) reduced at 500 $^{\circ}$C to (16-32%) and (39-50%) and to about (11-12%) and (16-21%) for exposure periods 1 and 2 hours receptively. At exposure to 800 $^{\circ}$C for one- hour duration, the reduction in (U.P.V) was 50% while that in rebound number was about 18.5%.

Habeeb [14] in 2000 studied the effect of high temperatures (up to 800 $^{\circ}$C) on some mechanical properties of high strength concrete (HSC). Three design strengths were investigated 40, 60 and 80 MPa. The investigated properties were compressive strength, flexural strength and volume changes. Ultrasonic pulse velocity (U.P.V) and dynamic modulus of elasticity (Ed) were tested also. The specimens were heated slowly to five temperature levels (100, 300, 500, 600 and 800 $^{\circ}$C), and to three exposure periods 1, 2 and 4 hours without any imposed loads during heating . The specimens were then cooled slowly and tested either one day or one month after heating. He concluded that the (HSC) is more sensitive to high temperatures than (NSC).The residual compressive strength ranged between (90 – 106%) at 100C, (72-103%) at 300 °C, (55 – 87%) at 500 $^{\circ}$C and (22-66%) between (600-800 $^{\circ}$C). He also concluded that exposure time beyond one hour had a significant effect on the residual compressive strength of concrete; however, the effect was diminished as the level of temperature increased. Moreover, the compressive strength at age of one month after heating, suffered an additional loss than at age of one day after heating. The flexural strength was found to be more sensitive to high temperature exposure than compressive strength, the residual flexural strength was in the range of (92 – 98%), (52-98%) and (29- 47%) at 100 C, 300 C and 500 C respectively and (2-30%) at 600 –800 $^{\circ}$C .The author also noticed that the (U.P.V) and (Ed) were more sensitive to elevated temperatures exposure than compressive strength.

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Umran [15] in 2002 investigated the effect of fire flame exposure on some mechanical properties of concrete. The specimens were subjected to fire flame .Three temperature levels of (400,500 and 700 $^{\circ}$C) were chosen with four different exposure duration of 0.5 , 1 , 1.5 and 2 hours without any imposed loads during heating. The specimens were heated and cooled under the same regime and tested after exposure to fire flame at ages (30, 60 and 90 days). Compressive strength of 150mm cubes, flexural strength of (100×100×400mm) prisms and static modulus of elasticity of (150×300mm) cylinders were measured. Ultrasonic pulse velocity (U.P.V) and Schmidt rebound number were tested also. The researcher found that the residual compressive strength ranged between (70-85%) at 400 $^{\circ}$C, (59-78%) at 500 $^{\circ}$C and (43-62%) at 700 $^{\circ}$C, The residual flexural strength was in the range (71-79%), (42-58%), and (22-41%) at 400, 500 and 700 $^{\circ}$C respectively. The residual modulus of elasticity ranged between (55-75%) at 400 $^{\circ}$C , (34-51%) at 500 $^{\circ}$C and (16-34%) at 700 $^{\circ}$C. He also found that ultrasonic pulse velocity (U.P.V) and rebound number tests were showed more reduction after burning the specimens compared with control specimens. He also noticed that exposure time after one hour had a significant effect on the residual compressive strength of concrete.

Al – Hayani [16] in 2003 studied the possibility of using local porcelinite rocks for the production of lightweight concrete durable to elevated temperatures. Various types of lightweight aggregate concrete were produced using porcelinite stone as a coarse aggregate with natural sand or fine porcelinite aggregate. The investigated properties were compressive strength, splitting tensile strength, flexural strength, density,dynamic modulus of elasticity, and volume change. The concrete specimens were subjected to six temperature levels of (100, 200, 400, 600,

800, and 1000 $^\circ$C) with exposure duration of 2.0 hour. The results showed that the lightweight aggregate concrete produced was less sensitive to elevated temperatures compared with normal weight concrete .She was concluded that the reduction in mechanical properties throughout heating for lightweight aggregate concrete contanining 50% and 100% fine porcelinite aggregate by volume of natural sand was smaller compared to sanded – lightweight aggregate concrete , the range of difference was between (3-22%) and (6-31%) respectively. She also concluded that the values of drying shrinkage of all types of lightweight aggregate concrete between (78 $\times 10^{-6}$ ─ 111 $\times 10^{-6}$)before heating , additional shrinkage due to exposure to elevated temperatures was ranged between (854 $\times 10^{-6}$ ─ 2820 $\times 10^{-6}$) after exposure to 800 $^\circ$C .

2.1.3 Effect of High Temperatures on Non-Destructive Tests.

2.1.3.1 Ultrasonic Pulse Velocity (U.P.V).

The ultrasonic test is a useful tool for assessing the uniformity of concrete and detecting cracks, voids, or honeycombing. It gives useful information about the size of micro – cracked zone and on crack growth and the interior structure of the concrete element [17,18,19].

Changes in properties occur in concrete as a result of exposure to high temperatures such as in the case of fire. The measurements of pulse velocity are usually indicative of changes in strength; this method has high potential for assessing fire damage [20].

2.1.3.2 Rebound Method.

The surface hardness of concrete is tested by the "Schmidt Rebound hammer". This test estimates the surface hardness by the rebound number,

which can take as a measure of the concrete strength and percentage of voids [21,22] .

A considerable number of research works had dealt with the effect of elevated temperature on the non –destructive tests of concrete.

Logothetis and Economou [23] in 1981 investigated the influence of high temperatures on the properties of concrete using non-destructive methods such as the rebound hammer and pulse velocity. Two series of concrete specimens (20×20×20 cm) cubes were tested; the first series (A) consisted of 32 specimens of different mixes and subjected to different curing conditions. These were subjected to various oven temperatures of (100,200,300,400,550 and 800 °C) for three periods (1,2 and 3 hours). The second series (B) consisted of 30 specimens cast in 3 groups of 10 specimens each. These series were exposed to four-oven temperature (100,200,500, and 800 °C) for a period of 2 hours. They noticed that the pulse velocity had a constant decrease with the increasing temperature. They also found that mixing proportions and heating time were proved to have small influence on the results. They observed that the rebound measurements showed no significant changes in the range up to around 600 °C.

Chung and law [24] in 1983 measured the fire damage of concrete by ultrasonic pulse velocity. The test specimens were (150×150×750 mm) prisms made from concrete of different composition by using granite aggregate. The specimens were heated to different temperatures not exceeding 500 °C at age 28 or 90 days. Two types of cooling, some of them were cooled in the air, others were quenched in the water. The results showed that the reduction in pulse velocity was less than 30 percent corresponding to practically no change in compressive strength when the concrete is air cooled after heating, but when the concrete is quenched, the

same reduction in pulse velocity was accompanied by 20 percent reduction in strength. They were noticed that the reduction in pulse velocity beyond 30 percent was accompanied by reduction in strength at faster rate both under air – cooled and quenched conditions. They observed that ultrasonic testing could be used to assess the fire damage of concrete.

The same authors published another paper [25] in 1985 to assess the fire damage on concrete. The specimens were concrete prisms (150×150×750 mm). The specimens were allowed to dry in the air for about a month and the initial pulse velocity was measured by the surface transmission method. The concrete prisms were burned with kerosene stoves placed underneath. The bare flame was intended to simulate the heating condition in an actual fire. The intensity of the flame was adjusted to raise the specimen temperatures. After the temperature was reached, burning was continued for a short period to ensure a constant surface temperature. After burning, the concrete prisms were allowed to cool to the room temperature. At the temperature of heating in the range between (600-800 °C) with a maximum duration of 175 minutes. They found that the reduction in pulse velocity was between (33-55%) from the initial pulse velocity.

Katwan and Abdul-Hammed[26] in 2000 presented an evaluation study of a fire, which took place at Al- Nahda intersection on the army canal in Baghdad city. The fire resulted from collision of a benzene tanker with tunnel deck slab and continued for more than three hours. They estimated the retained strength of concrete at fire exposed area by using semi destructive tests, such as concrete cores or non-destructive tests such as ultrasonic pulse velocity and Schmidt hammer . It was found that the concrete compressive strength was equivalent to 23 MPa. They found that the actual compressive strength of concrete as obtained from the project

document was more than 35 MPa ; it was estimated that the ratio between strength after and before fire was less than 0.66. They found the attained temperature from BS-8110 part 2 .The corresponding temperature was estimated at 520 $^\circ$C ,which evidently in good agreement with the temperature assessment made from visual inspection. They found that the structure was still capable of sustaining the loads recommended by the specifications of State Organization for Roads and Bridges after carrying out loading test before putting the intersection back into service.

2.1.4. Effect of High Temperatures on Steel Reinforcement

Harmathy and Stanzak[27] in 1980 investigated the effect of elevated temperature on the yield tensile stress and ultimate strength for two different types of reinforcing steel (ASTM-A36,CSAG-40.12),and prestressed steel (ASTM-A421).The bars were heated to a temperature of (25-705C) for two periods (60,90 minutes) and then slowly cooled. The program was designed to provide data on three major parameters (yield stress, ultimate strength and stress-strain relation ship).The results showed that there was no effect on the stress-strain curves below 371 $^\circ$C ,but over this degree there was a little change. They also concluded that the rate of reduction in both yield stress and ultimate strength was little, below 371 $^\circ$C , but the behavior of the second type of steel (G40-12) was to be different, where the yield stress was decreased while ultimate strength was increased. As the temperature was increased more than 371 $^\circ$C, yield stress and ultimate strength were reduced for all specimens.

Holmes and Anchor [28]in 1982 studied the effect of elevated temperatures on the strength and stiffness properties of four reinforcing steels of varying size. The test program was designed to provide data on three major parameters (yield stress , ultimate strength , and elastic

modulus).They found that the normalized results for the yield stress, ultimate strength and elastic modulus for all sizes are as follows: -

1 – There was no significant change in the normalized values below 300°C.

2 – There was 50 % reduction in both the yield stress and ultimate strength was obtained between 520°C and 580°C and between 540°C and 700°C for the elastic modulus.

Edward, and Gamble [29] in 1986 studied the behavior of grade 60 reinforcing bars of diameter (12.7mm) after exposure to fire. The bars were heated to temperatures of (500-800 °C), for about one hour and then were slowly cooled. The bars were tested in tension after they had been cooled. The yield stresses were reduced to a minimum of 73% of the original strength and the ultimate strength was reduced to a minimum of 83% of the as rolled strength. They found that heating and cooling of steel did not change the nature of the stress- strain curves. The modulus of elasticity remained constant at room and elevated temperatures.

2-1-5 Effect of High temperatures on Reinforced Concrete

Gustafferro et al. [30] in 1971 studied the structural behavior of prestressed and reinforced concrete beams exposed to standard fire (ASTM-E119)[6]. The test was included 13 beams with 12.2m length loaded uniformly with design load. The beams were divided in two groups, the first group was pretensioned concrete beams, the second was post-tensioned concrete beams. Each group reinforced by prestressting steel or ordinary deformed steel bars of grade 40 or 60. Both normal and lightweight concretes were used. They found that the beams reinforced with ordinary deformed bars were superior to the prestressed concrete

beams in fire resistance. They also were noticed that the behavior of the beams was not affected significantly by the type of steel strength, but due to the low value of the elastic modulus of the lightweight concrete. Moreover, the initial deflection of the lightweight concrete beams was greater than that of normal weight beams, but after a period of one to two hours of standard fire exposure, the opposite was noticed.

Salse and Lin[31] in 1976 carried out a theoretical study on the effect of elevated temperatures on concrete frames. The investigation program included a theoretical part for the structural design for reinforced concrete members subjected to fire. To determine the properties of concrete and steel reinforcement during fire exposure, the thermal distribution through the member section should be known, therefore, they used a thermal analysis by finite difference method. The authors obtained the following conclusions:

A – The resistance of structural member to fire increased as the restarting force increased.

B – Simple mathematical relationship may be used to find the resistance of sections at the negative and positive moment areas.

Lie and Leir [32] in 1979 studied the effect of different variables on the temperature distributions on reinforced concrete slabs under fire exposure. Specimens of (800 × 900mm) with different thickness of (60, 100 and 150mm) were used: A computer program was used depending on the finite difference method to calculate the temperature distribution in the fire exposed slabs taking into account the effect of time of exposure to elevated temperatures and thermal properties of concrete. They concluded that there is a good agreement between the experimentally and theoretically calculated temperature distributions for all the specimens. They also found

that an increase of moisture in concrete leads to disagreement between the experimental results and those calculated using the computer program. Finally, they observed that the slab thickness affects the temperature distribution significantly.

Lin, Gustaferro and Abrams[33] in 1981 studied the behavior of continuous reinforced concrete beams under fire exposure. Three cases were considered, simply supported beam, one end continuous beam and two-end continuous beam. The simply supported beam represents the case where the cantilever ends are unloaded; similarly, the one end continuous beam represents the case where only one end from the two ends is loaded. The authors were concluded that widely spaced stirrups lead to shear failure under fire exposure, even when this type of failure does not occurred at the ambient temperature under design loads. They also concluded that the simply supported beams under fire exposure fail at the maximum bending moment section (mid span) when the strength at this section decreased to the applied moment value. The decreased flexural strength could be calculated using the decreased concrete and steel strength for a given temperature. Moreover, an early moment redistribution was noticed during the test, resulting an increase in the negative moment values.

Khan and Royles [34] in 1986 carried out a study on the effect of high temperature on the behavior of reinforced concrete beams. They investigated the load-deflection relationship, cracks pattern and bond between steel and concrete. Prismatic concrete beams (960 × 140 × 66 or 107 mm) were used; 8mm plain bars and 16mm torbars were used to reinforce them. The specimens were heated to temperature was ranged from (20 – 800°C) for one-hour exposure duration. They found that the

effect of temperature was insignificant at temperatures ranging from (100 to 200°C), but the strength decreased significantly between (350 to 500°C). Compared with ambient conditions, the flexural strength characteristics weak end by 50% of the original strength. They also noticed that the thermal cracks appeared in a honeycomb fashion all over the surface and they seemed to originate from the top to the bottom edge and terminate near the mid-depth of beam , moreover , they also found that deflection cracks were more prominent in the shear zone of the smaller section beams , whereas for large beams flexural tensile cracking developed in the mid-span region.

Ass`ad[35] in 1987 investigated the effect of elevated temperatures on the behavior of structural reinforced concrete samples. Four types of reinforced concrete samples were used. Singly and doubly reinforced concrete beams having the dimension of (100 ×100×1100mm) were used. The doubly reinforced concrete beams reinforced with both tension and compression steel, while the singly reinforced beams with tension steel only. Continuous beams (100× 150 × 1300mm) and structural frame with outer dimensions of (900 × 750mm) were used. The frame had a cross – section of (100 × 150mm) for the beam and (100 × 100mm) for the column . The specimens were subjected to temperatures of (150, 300, 600, 750 and 900°C) at the ages of 30, 90 days and were tested in flexure after cooling. The researcher was found that both flexural strength and stiffness were decreased with temperature increase. He also found that the use of top reinforcement had limited this decrease. Moreover, he observed that the increase in temperature was led to an increase in the magnitude of moment distribution of continuous beams.

Shirlly et al. [36] in 1988 carried out a study dealing with high strength reinforced concrete slabs and their resistance to fire. The study includes five specimens having dimensions of ($900 \times 900 \times 102mm$) reinforced with 19mm diameter steel bars. Four high strength mixes were used along with one mix with normal strength for comparison. The specimens were subjected to a standard fire, and failure was determined when the average unexposed surface temperature reached 121°C. The results showed that the behavior of the five slab specimens was the same, the fire endurance of the specimens were close, and there was no spalling or falling of the concrete surface due to high temperature exposure. Some cracks were formed over embedded reinforcement bars during the fire tests. However, these cracks appeared in both the high and normal strength concrete specimens.

Hidayat [37] in 1994 carried out experimental and theoretical investigation concerning the behavior of reinforced concrete slabs subjected to high temperatures. The experimental part included fabricating and testing 18 reinforced concrete slab specimens having dimensions of ($600 \times 600 \times 40mm$), with different steel ratios. The specimens were heated and cooled, then were tested to failure under uniformly distributed load. The analytical investigation included thermal analysis using computer program fires $- T_3$ to predict the thermal distribution through the slab section, and a structural analysis of the slab specimens subjected to high temperatures. He observed the following main conclusions:

1. All the heated slabs were found to be capable of resisting the service loads

2. The ultimate flexural strength of the slab specimens was decreased with the   increase of temperature.

3. Good agreement between the thermal analysis and the experimental results were obtained, which confirms the validity   of the method and the mathematical models used.

Al – Owasiy[38] in 2001   investigated the effect of temperatures on bond in reinforced concrete. Single concrete mix 1: 1.5: 3 with two water – cement ratios of (0.45and 0.60). The concrete specimens were heated in electrical furnace for one- hour period of exposure at four temperature levels (150,250,400, and 500 $^\circ$C). The test variables were bond strength by pull – out test, compressive strength and splitting tensile strength. Two types of cooling were used,cooling in the air and in the water. The test results showed that the reduction in compressive and splitting tensile strength for concrete specimens of water – cement ratio of 0.60 and cooled in air were (12, 22, , and 34%) and (14, 28, and 64%) at (150,400 and 500 $^\circ$C) respectively. For concrete of water – cement ratio of 0.45, the reduction in compressive and splitting strengths at (400 and 500 $^\circ$C) were (24, 33%) and (38,63%) respectively. He found that the reduction in bond strength for pull-out with 12mm and 20mm reinforcement bar diameters at (150,250,400, and 500 $^\circ$C) were about (14, 10, 21and 39%)and about (18, 16, 24, and 42%) respectively. The percentage reduction in compressive and bond strengths cooled in water compared to the air cooled specimens at (250, 400, and 500 $^\circ$C) was about (8,10,and 12%) and about (6, 11, and 13%) respectively. He also observed that the splitting tensile strength reduction for water cooled specimens was about (19, 30, 44, and 70%) after exposure to (150, 250, 400, and 500 $^\circ$C) respectively.

Kadhum[39] in 2003 investigated the shrinkage cracking of reinforced concrete slabs exposed to direct fire flame. Four slab models were externally restrained by different end restraints. The chosen dimensions for

the slabs were (2250 ×2250 × 100mm). The reinforced concrete slabs were exposed to drying shrinkage for a period of two months, and then they were exposed to direct fire flame temperature of 600°C for 1.5 hour, without any imposed loads during burning. From the results obtained before and after burning, it was observed that there was a variation in the crack width along the slab length. The author was concluded that the maximum cracks width before burning were (0.24,0.255 and 0.275mm) for two, three and four end- restrained slabs respectively. The maximum cracks width for free, two end, three end and four end restrained slabs during and after burning were (0.25, 0.75, 0.725 and 0.71mm) and (0.07, 0.675, 0.7 and 0.75mm) respectively. He also found that the maximum measured deflection at the mid span of reinforced concrete slabs was recorded during burning was (1.98mm) for free slab, whereas the minimum deflection during burning recorded was (0.85mm) for four end restrained slab. Finally, simplified formulas were modified and suggested for calculating the minimum and maximum crack spacing and maximum crack width at any position along the slab length.

2-2 Temperatures-Associated with Fire.

The values of temperature associated with fires are according to the ASTM-E119 [6] standard curve which can be described approximately by the following expression:

If t < 7200 sec.

$Tf = To + 1/1.8$ [1044 + tanh (0.00023413t) – 498.2 tanh (0.00027044t)

+ 1286 tanh (0.002475t)]. _____(2 − 1)

Where; t: - time in seconds.

Tf: - the fire temperature at t (°C).

To: - initial temperature (°C).

The curve representing this function; known as the "standard time – temperature curve", [6]
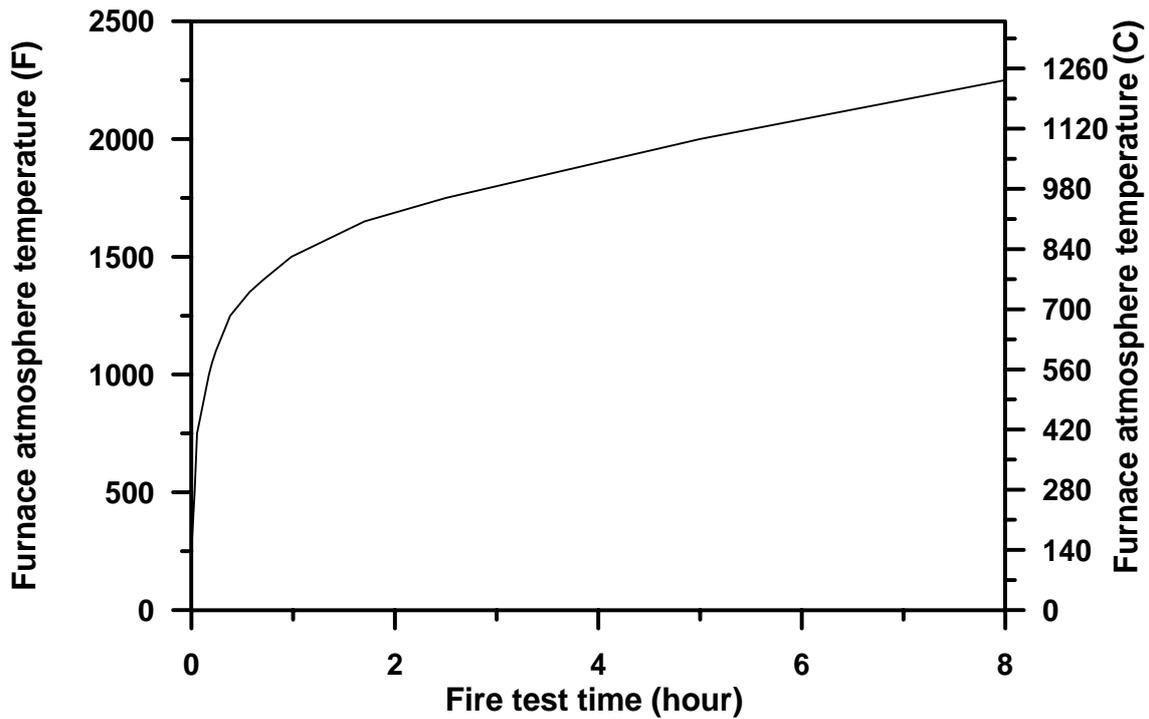
Figure (2-6):-Standard time-temperature relationship of furnace atmosphere (ASTM E 119) [6]

Chapter Three

Experimental Work

3-1 Introduction

The main purpose of the experimental work is to evaluate the effect of fire on the behavior of reinforced concrete slab specimens. An experimental study of the complete response of reinforced concrete slabs subjected to fire and then tested under uniform distributed load is presented. The presence of load in real slab structures was simulated by loading some slab specimens during exposure to fire.

This chapter describes the materials used and their characteristics, mix proportions, preparation and curing of slab specimens, in addition to the testing program.

3-2   Program of the Work.

During this research, the study of the behavior of slabs was based on twenty-four reduced scale reinforced concrete slab specimens caste in laboratory. The chosen dimensions of all the slab specimens were (600×600×40mm) (length× width× thickness respectively). Two concrete mixes with target compressive strengths of (30 and 38 MPa) designated as series A and series B respectively, with two steel ratios of (0.00492, 0.00875) were used. The specimens were cast, water cured for 14 days, air dried in the laboratory for 14 days, then at age of 28 days they were subjected to fire flame. Sixteen slab specimens were exposed to fire

temperature levels of (400,500 and 600 $^{\circ}$C) at the lower surface of slab specimens with exposure duration of one hour, then after 24 hours; they were tested to failure under uniformly distributed load. In addition, these slab specimens were tested for (weight loss, ultrasonic pulse velocity and rebound number) before and after exposure to fire.

Other slab specimens were uniformly loaded to two levels, the first level was 40% of the ultimate load and the other level was the service load. These loads were held constant during the exposure to fire. Tables (3-1-a) and (3-1-b) describe the details of the investigated slab specimens.

Table (3-1-a): Details of reinforced concrete slab specimens.

Series	Slab No.	Exposure Temp. ($^{\circ}$C)	Compressive Strength (MPa)	The Distance Between Bars (mm)	Steel Ratio (ρ)
A	A_{11}*	25	30	80	0.00492
	A_{12}	400		80	0.00492
	A_{13}	500		80	0.00492
	A_{14}	600		80	0.00492
	A_{21}	25	30	45	0.00875
	A_{22}	400		45	0.00875
	A_{23}	500		45	0.00875
	A_{24}	600		45	0.00875
B	B_{11}	25	38	80	0.00492
	B_{12}	400		80	0.00492
	B_{13}	500		80	0.00492
	B_{14}	600		80	0.00492
	B_{21}	25	38	45	0.00875
	B_{22}	400		45	0.00875
	B_{23}	500		45	0.00875
	B_{24}	600		45	0.00875

* The first number refers to steel ratio and the other represents the fire exposure temperature.

Table (3-1-b): Details of reinforced concrete slab specimens

which were loaded and exposed to fire flame .

Series	Slab No.	Exposure Temp. ($^{\circ}$ C)	Applied load (kN)
A	$A_{15}{}^{*}$	600	Service load
	A_{16}	600	40 % of ultimate load
	A_{25}	600	Service load
	A_{26}	600	40 % of ultimate load
B	B_{15}	600	Service load
	B_{16}	600	40 %of ultimate load
	B_{25}	600	Service load
	B_{26}	600	40 %of ultimate load

*The first number refers to steel ratio and the other represents the level of loading .

3-3   Details of the Experimental Work.

3-3-1 Moulds Preparation.

The slab specimens were cast in steel moulds to give a slab specimen with clear dimensions of (600 × 600 × 40mm). Each one of these steel moulds consisted of a steel plate of (700 × 700 × 5mm) enclosed by four steel angles section of (40 ×40 × 5mm) . Every angle was fixed with the steel plate by three bolts as shown in Figure (3-1). These moulds were cleaned and their internal surfaces were greased to prevent adhesion with concrete after hardening.

Top View

Steel plate (700x700x5 mm)

Steel angle (40x40x5 mm)

Steel angle for supporting
the steel plate.

600 mm

700 mm
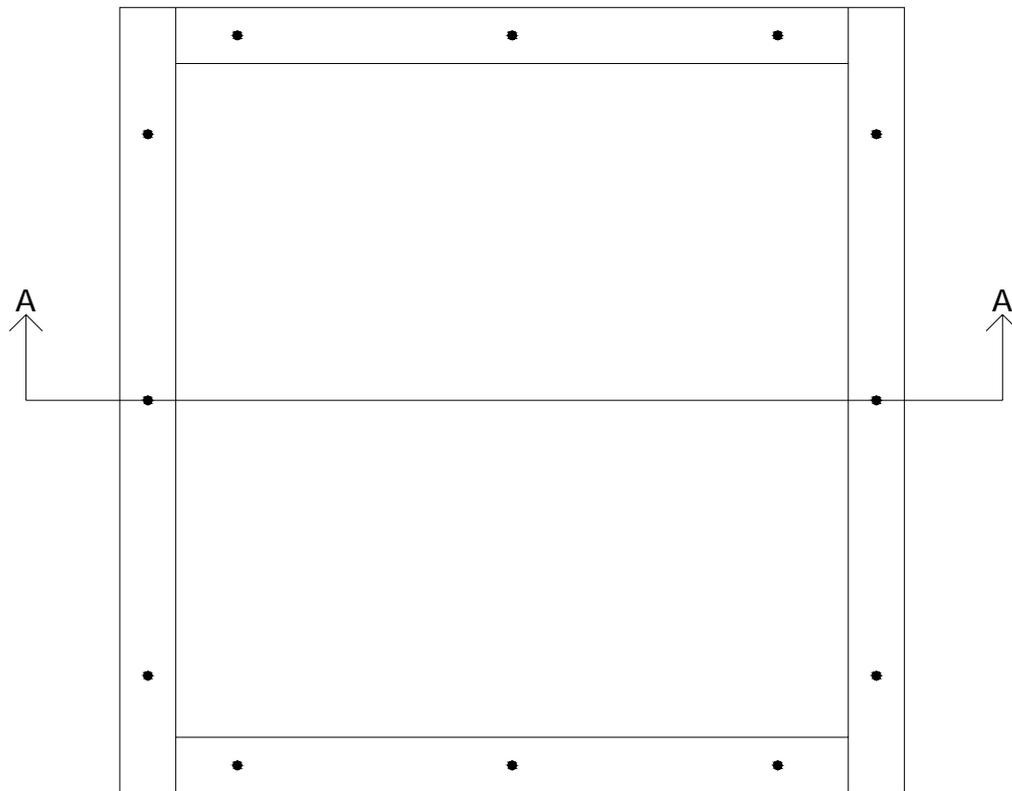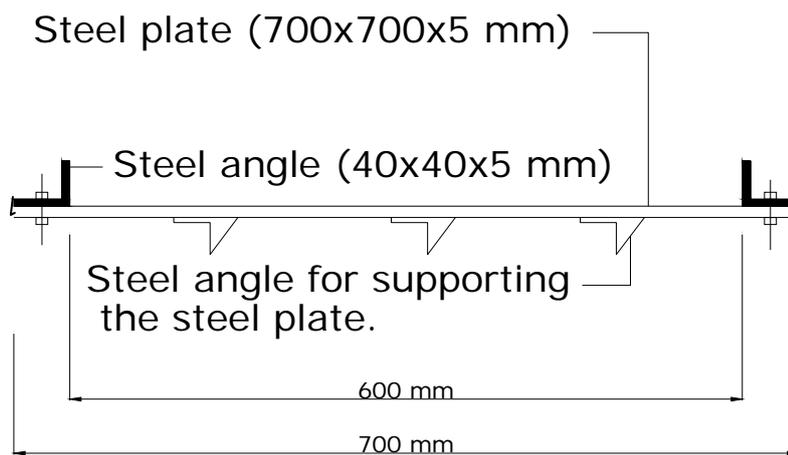
Sec. A-A

Figure (3-1): Details of the mould of slab specimen.

3-3-2 Reinforced Concrete Slab Specimens.

Twenty-four slab specimens of (600×600×40mm) were cast.   These slab specimens were divided into two series A and B with two target compressive strengths (30, 38MPa) , and with two steel ratios of (0.00492 , 0.00875) respectively.

The reinforcement bars were cut to the desired length, then they were uniformly spaced and placed in two directions according to the limited distances of (80 , 45mm) to obtain the desired steel ratios of (0.00492 , 0.00875) respectively .

After greasing the moulds of the slab specimens, reinforcement meshes were held carefully in their position inside these moulds .In order to get a constant clear cover, small pieces of steel of 6 mm diameter were placed under the reinforcement.

The slab specimens were assumed to be simply supported on the four edges. The clear span after supporting was 560mm Figure (3-2) shows the details of reinforcement.
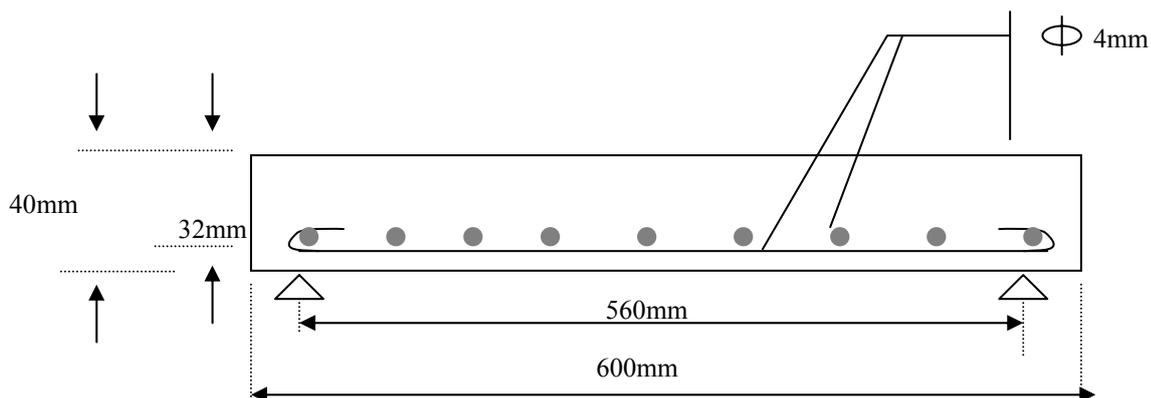
Figure (3-2): Details of reinforcement of the slab specimen.

With each casting process of slab specimen, three concrete cubes of (150×150×150mm), two concrete cylinders of (100×200mm),two concrete cylinders of (150×300), and two concrete prisms of (100×100×400)were

also cast. These specimens were subjected to the same conditions that the reinforced slab specimens were subjected, by subjecting the concrete specimens to direct fire flame from the same network of methane burners .

3-3-3 Materials and Mixes.

3-3-3-1 Cement

Ordinary Portland cement (O.P.C) produced in Lebanon was used throughout this investigation. This cement complied the Iraqi specification No.5/1984[40]. The cement was stored in a dry place to avoid the exposure to the atmosphere. The physical properties and chemical composition are presented in Tables (3-2) and (3-3).

Table (3-2): Physical properties of the cement

Physical properties	Test results	I.Q.S.5:1984[40] limits
Fineness ,Blaine,cm^2/gm	3300	≥2300
Setting time ,Vicat's method Initial hrs: min Final hrs : min	1: 40 3: 20	≥ 00 : 45 ≤ 10 : 00
Compressive strength of 70.0 mm cube , MPa 3 days 7 days	 24 34	 ≥ 15 ≥23

Table (3-3) : Chemical composition of the cement

Oxide	(%)	I.Q.S.5:1984[40] limits
CaO	63.5	………
SiO_2	20.97	……...
Fe_2O_3	3.67	………
Al_2O_3	5.5	………
MgO	0.7	≤ 5.0
SO_3	2.5	≤ 2.8
Free Lime	0.68	-
L.O.I	1.53	≤ 4.0
L.R	-	≤ 1.5
Compound composition	**(%)**	**I.Q.S.5:1984 limits**
C_3S	46.97	………...
C_2S	24.77	………...
C_3S	8.37	……….
C_4AF	11.15	……….
L.S.F	0.9	0.66-1.02

3-3-3-2 Fine Aggregate

Al-Akhaidher well- graded natural sand was used in this investigation.The physical and chemical properties of the sand are listed

in Table (3-4).Its grading conformed to the Iraqi specification No. 45/1984[41] ,Zone 2 .

Table (3-4) : Properties of fine aggregate

Sieve size (mm)	Percent passing	I.Q.S.5:1984[41] limits Zone 2
9.5	100	100
4.75	97	90-100
2.36	82	75-100
1.18	71	55-90
0.6	55	35-59
0.3	27	8-30
0.15	4	0-10
0.075	0.95	5
Properties	**Test results**	**I.Q.S.5:1984[41] limits**
Sulphate content SO$_3$%	0.29	≤0.5
Fineness modulus	2.64	
Specific gravity	2.55	
Absorption	1.5	

3-3-3-3 Coarse Aggregate.

The coarse aggregate was Al-Nibaee gravel with a maximum size of 12.5mm. The coarse aggregate was washed, and then stored in air to dry. The coarse aggregate used conforms to the Iraqi specification No.45/1984. Table (3-5) shows the grading and properties of the coarse aggregate.

Table (3-5) : Properties of coarse aggregate

Sieve size (mm)	Percent Passing	I.Q.S.5:1984[41] limits
37.5	100	-
20	100	100
12.5	100	90-100
9.5	69	50-85
4.75	6.45	0-10
Properties	**Test results**	**I.Q.S.5:1984[41] limits**
Sulphate content , $SO_3\%$	0.09	≤ 0.1
Specific gravity	2.62	-
Bulk density kg/m^3	1694	-
Absorption (%)	0.5	-

3-3-3-4 Water

Tap water was used throughout this work for both mixing and curing of concrete.

3-3-3-5 Steel Reinforcement.

Plain steel wires 4mm in diameter were used as reinforcement to test slab specimens. Three specimens of these wires were tested under tension, and the resulting average stress-strain relationship is plotted in Figure(3-3). This Figure indicates that the yield tensile stress and ultimate tensile strength are (340,480 N/mm$^{2)}$ respectively.
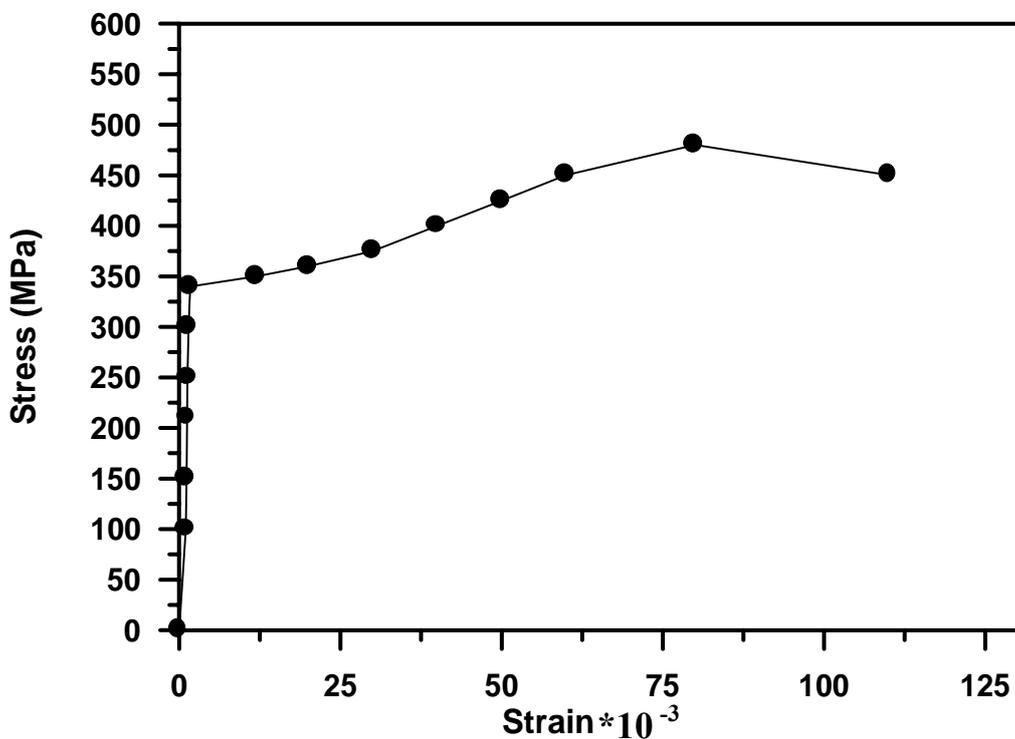
Figure (3-3): Stress- strain curve for 4mm diameter steel wire.

3-4 Mix Design and Proportions

Two target compressive strengths of 30 and 38 MPa were denoted as series A and B respectively. The concrete mixes were designed according to (ACI-211.1-91). The proportions of the concrete mixes are summarized in Table (3-6).

Table (3-6) : Mix proportions and slump of the concrete mixes.

Series	W/C	Mix Proportions (kg/m^3)				Slump(mm)
		Water	Cement	Sand	Gravel	
A	0.55	190	347	810	968	80-100
B	0.48	215	448	684	968	80-100

3-5 Casting Procedure

3-5-1 Mixing of Concrete.

The concrete was mixed in a horizontal drum laboratory mixer, with a capacity of 0.1m^3. The interior surface of mixer was cleaned and moistened before placing the mix contents. The gravel, sand and cement were weighted, and then they were dry mixed for about one minute before the required water was added to the mixer.

The constituents were mixed wet for about two minutes until a homogeneous concrete was obtained.

3-5-2 Slump Test.

The workability of the fresh concrete mixes was measured by the slump test before casting concrete in moulds .Test was conducted according to ASTM C143-89a:1989 [42] specification.

3-5-3 Compaction

After mixing, the concrete mixes were cast into the steel moulds of the slab specimens. Compaction was achieved using a vibrator table till no air bubbles emerged from the surface of the concrete.

After casting, the specimens were leveled by hand trawling, and covered with polyethylene sheet to prevent evaporation of moisture from the fresh concrete. Two days after casting, the specimens were demoulded, marked and immersed in tap water until the age of 14 days. After the water curing, the slab specimens of the two series were painted by white emulsion to ensure clear appearance of crack growth.

Finally, the specimens were exposed to normal weather condition until they complete 28 days which is the age of fire processing and testing .

3-6 Tests of Hardened Concrete

3-6-1 Compressive Strength Test

For the hardened concrete, the compressive strength test was carried out according to B.S 1881: part 116: 1983[43] ,using a machine of (2000kN) maximum capacity as shown in plate (3-1).

The load was applied and gradually increased at a constant rate. Cubes were cast, demoulded and cured in a similar way as for slab specimens. Each compressive strength value was the average compressive strength of three cubes at age of 28 days.

Plate (3-1) : Compressive strength testing machine .

3-6-2 Splitting Tensile Strength Test.

The splitting tensile strength was determined according to procedure outlined   in ASTM C-496[44] specification. The cylinders were cast, demoulded and cured in a similar way as for slabs specimens. Each splitting tensile strength value is the average for two specimens at age of 28 days .

3-6-3 Flexural Strength Test.

Concrete prisms of (100×100×400mm) were cast for measuring flexural strength. The prisms were cast, demoulded and cured in a similar manner as for the slabs specimens. Modulus of rupture test was performed according to ASTM C293-83[45] procedure using midspan-one point loading as shown in plate (3-2). Each value of the modulus of rupture is the average of the test results of two prisms.

Plate (3-2) : Flexural strength testing machine .

3-6-4 Static Modulus of Elasticity Test.

The static modulus of elasticity was determined according to ASTMC-469[46] specification. The test specimens were cast, demoulded and cured as for slab specimens. The top surface of the cylinders was well finished from irregularities and capped with cement paste to avoid any loss of strength.

The load was gradually applied and continuously increased at constant rate until 40% of the ultimate load. The compressometer used has a gauge length of 150 mm and gauge with an accuracy of 0.002mm as shown in plate (3-3). The modulus of elasticity was calculated from the relation:

$$E = \frac{S_2 - S_1}{\epsilon_2 - 0.00005}$$

where:

E= The static elastic modulus,MPa

S_2 :Stress corresponding to 40 % of ultimate load, MPa

S_1: Stress corresponding to a longitudinal strain (0.00005),MPa

$\in_2$: longitudinal strain produced by stress S_2

The recorded results were the average reading of two cylinders at age of 28 days.
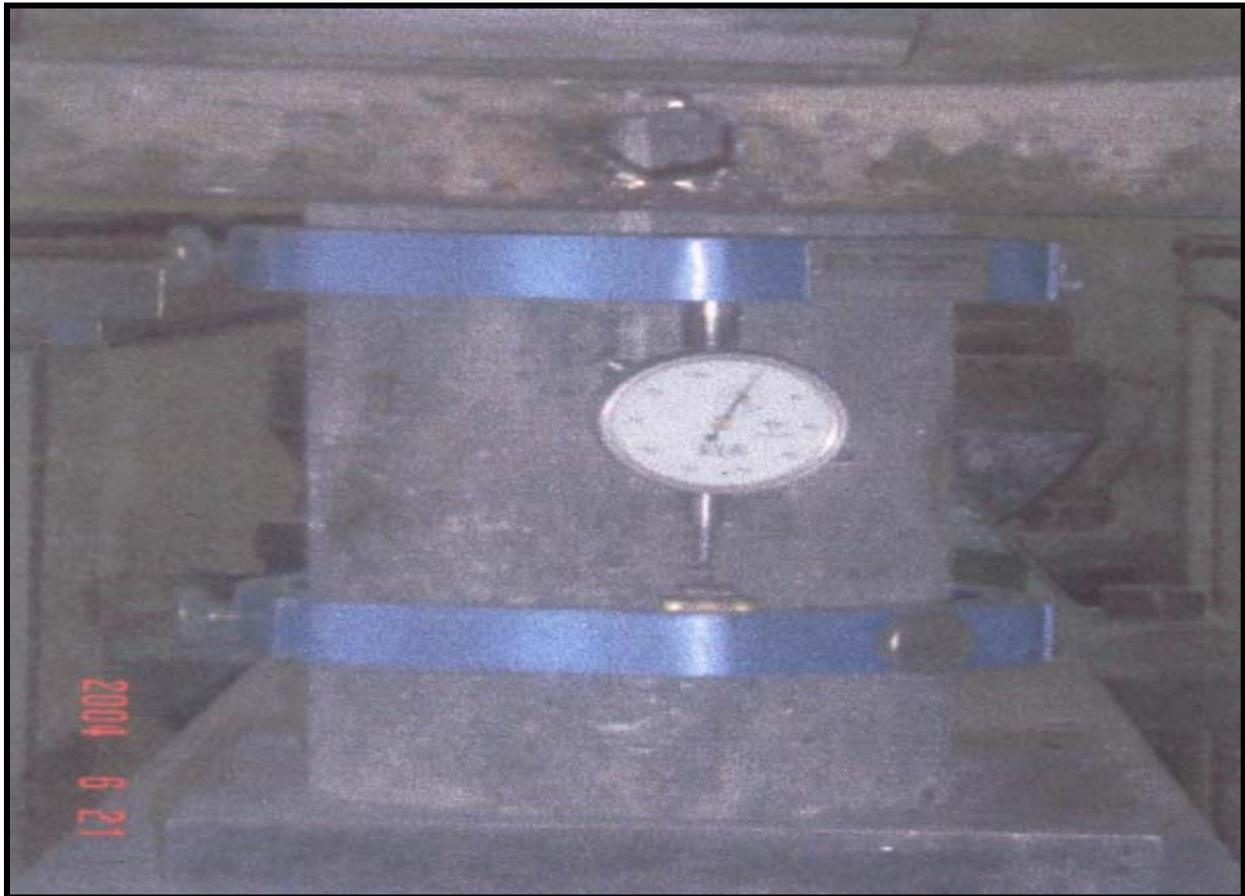
Plate (3-3): Set –up modulus of elasticity test .

3-7 Burning and Cooling

The reinforced concrete slab specimens were subjected to fire flame from a network of methane burners. The fire flame hit the lower face of

the slab specimens. The columns of flame were intended to simulate the heating condition in actual fires.

When the target temperature was reached, the temperature was continuously measured by thermometers, one of them was positioned in the bottom surface of the slab specimen in contact with the flame while the others were positioned at the unexposed upper surface, at the steel reinforcement layer and at the mid depth of the slab specimen to measure the temperature of the exposed surface, unexposed surface, steel layer and mid depth layer respectively ,the thermometers locations were positioned at different depths of the slab specimen as shown in plate (3-4). The measurement devices are shown in plate (3-5).

After burning, the reinforced concrete slab specimens were allowed to cool in the laboratory to the room temperature which is in the range of 25 $^{\circ}$C .

Plate (3-4): Fixing the thermometers locations at slab specimen
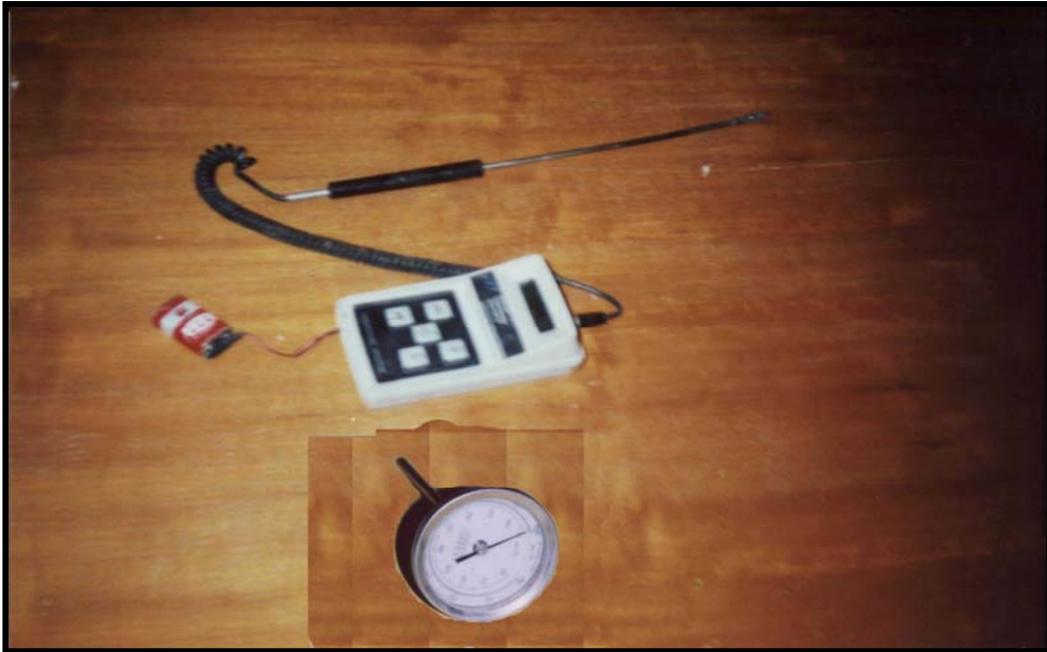
Plate (3-5) : Temperature measurements devices .

3-8   Duration of Burning and Range of Temperatures.

The duration of fire depends on the rate of temperature rise. The total fire duration, including the time of building up of the fire, can vary from about one hour to about one day [47, 48] . For the slab specimens which were exposed to fire and gradually cooled to room temperature, then they were tested, it was decided that the exposure time is one hour at the level of maximum temperature. While for the slab specimens which were subjected to fire under loading, the exposure time to fire was continued until failure .

Although the maximum temperatures reached during fires of buildings are in the order of 1000 to 1200 $^{\circ}$C [12,49] such high temperatures occur only

at the surface of the exposed members. Considering the relatively small size modeling of the specimens tested, it was decided to limit the temperature of the specimens to three temperature levels of (400, 500 and 600 °C).

3-9   Tests of Reinforced Concrete Slab Specimens

3-9-1 Weight Loss Test

This test was carried out for reinforced concrete slab specimens to obtain information about the weight loss due to exposure to fire. The specimens were weighted before and after exposure to fire.

3-9-2 Ultrasonic Pulse Velocity Test (U.P.V)

Ultrasonic Pulse transit times were measured by direct transmission method. This test was carried out according to ASTM C597 [50] .

The velocity of Ultrasonic Pulses transmitted through the reinforced concrete slab specimens was determined before and after burning. Portable ultrasonic concrete tester known as (PUNDIT) 54 KHz was used for this purpose as shown in plate (3-6).

Calibration of the concrete tester was done before testing to check the accuracy of the transit time measurements. This was achieved by the calibration of reference bar.

A thin layer of grease was applied on the surface to act as a couplant and to prevent dissipation of transmitted energy.

The pulse transit path length was measured accurately and the time of its travelling was recorded to the order of 0.1 Msec. The ultrasonic pulse velocity was corrected for the existence of steel reinforcement.
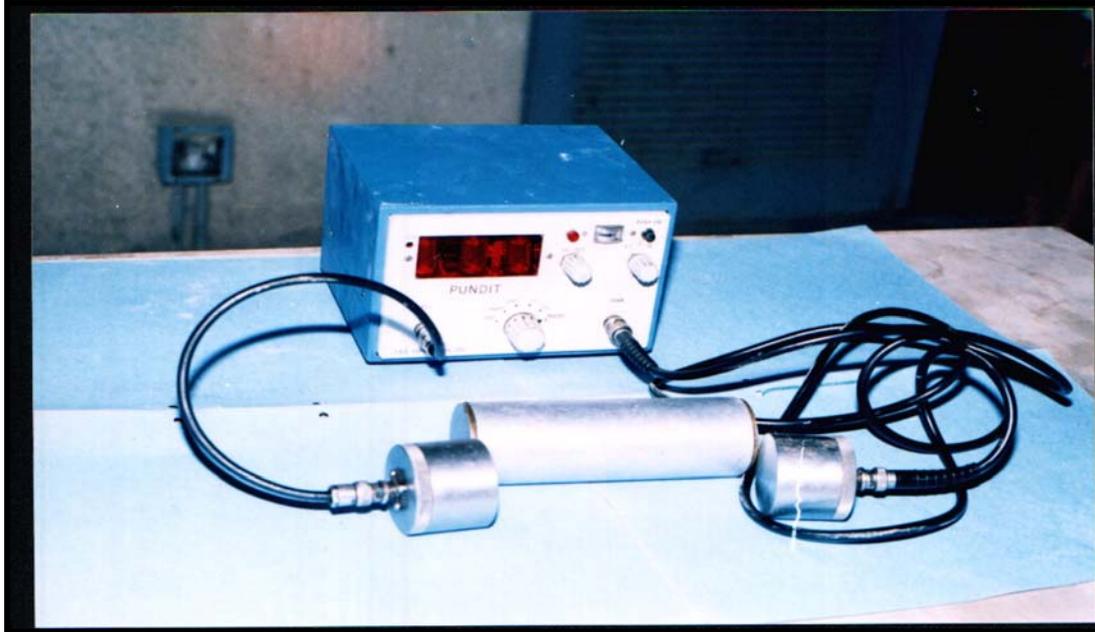
Plate (3-6): Ultrasonic pulse transit time test .

3-9-3 Rebound Hammer Test

Schmidt hammer was used to estimate the surface hardness of the reinforced concrete slab specimens by recording the rebound number, which represent an indication to the concrete strength.

The Schmidt hammer rebound number was recorded at 16 distributed points on the top unexposed face of the slab specimen. The mean rebound number was calculated for each slab specimen before and after burning. Schmidt hammer was used as shown in plate (3-7) . The test method is prescribed by BS-1881: part 202: 1986[51] specifications.
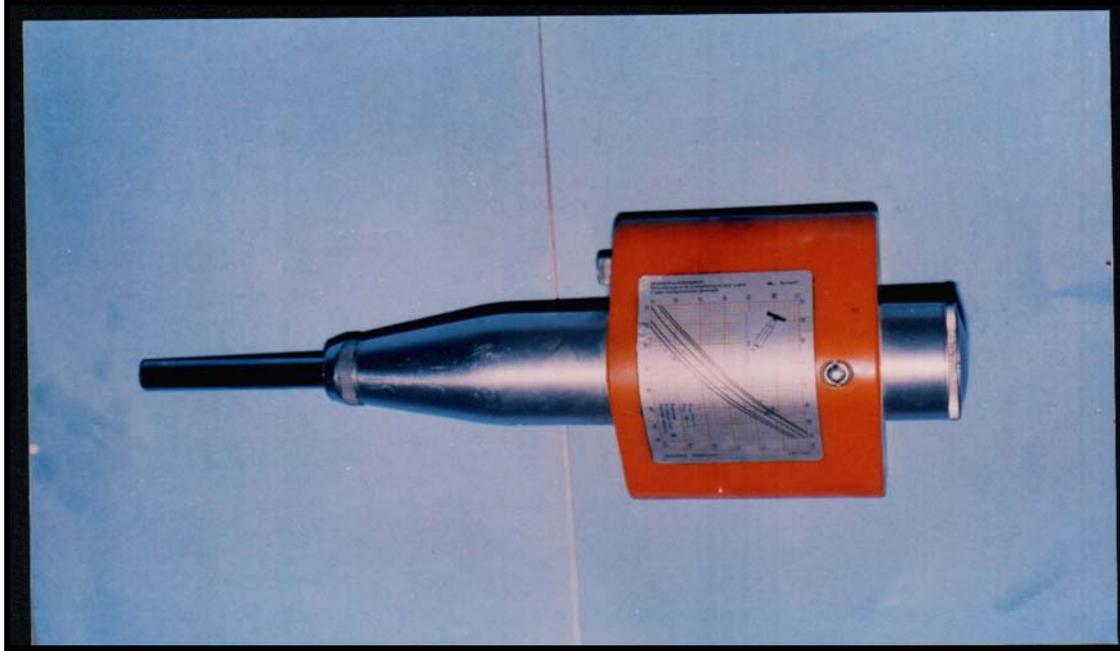
Plate (3-7) : Rebound hammer test .

3-9-4 Uniform Load Test.

3-9-4-1   Test Rigs

All the slab specimens were tested in rigid steel frame with I-section, which was designed as a supporting system. Four 12mm in diameter plain bars were welded on the upper of the square steel base to obtain a simply supported condition for a square shape slab of clear dimension of 560mm. The steel supports were covered by red silicon to furnish a continuous touch between the support and the slab.

A box of dimensions of (560×560×100mm), made of steel plate of thickness 5mm opened from upper and lower square areas and coated by a sheet of Nylon in the inner surfaces, was used to hold the sand to be placed

over the slab as a part of the uniformly distributed load. The sand (100mm) furnishes a good media to distribute the load uniformly coming on the top surface of the sand by the loading base which transmit the load from the hydraulic Jack to the layer of sand.

A load cell with capacity of (70 tons) was used to measure the load applied by a hydraulic Jack which transmitted the load to four points using a loading base which consisted of three steel members with I-section of (120×80mm) and length of 360 mm, two of these members were paralled and the other welded perpendicularly upon them. The parallel steel members were connected to four steel legs of (50mm) diameter by (30mm) height ,these legs were connected by welding to four steel plates of (140×140×5mm) ,then, these plates were fixed over a steel plate of(480×480×5mm) by welding .This steel loading base transmitted the load to the 100 mm layer of sand used between the loading base and the slab specimen as shown in Figure(3-4) This method of loading was adopted by other researchers [37,52,53] also.

Top View

Sec. A-A
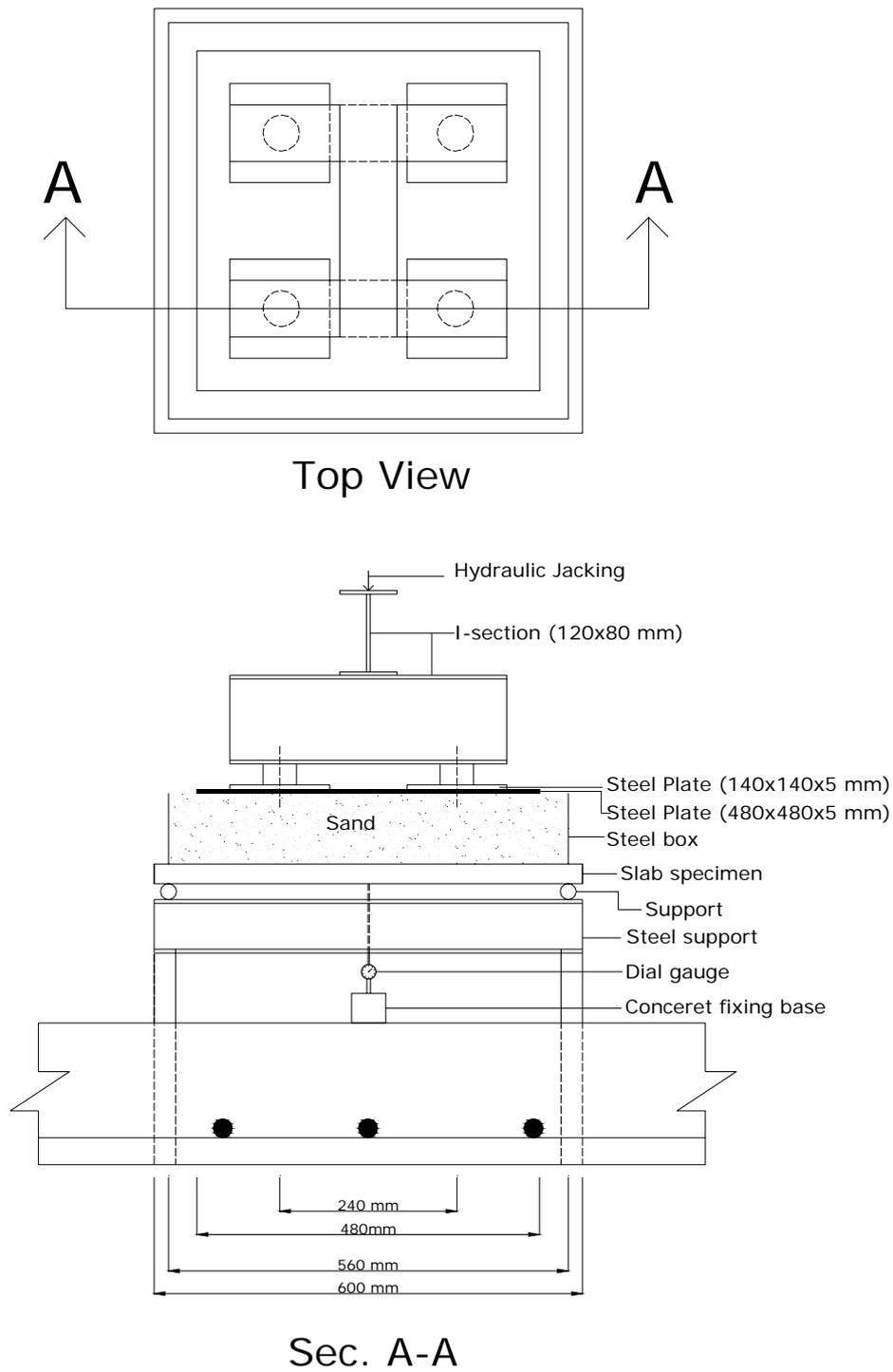
Figure (3-4) : Loading arrangement for uniform load test[37] .

3-9-4-2 Deflection Measurements

Deflections were measured at all load stages using a dial gauge with a capacity of (25.4mm) and accuracy of (0.025mm) at mid span of the slab specimens. The dial gauge was fixed in such a way that it can contact the lower surface of the slab specimens.

For the slab specimens which were subjected to fire flame under loading, the real problem which faced this process was the exposing the dial gauge of deflection to elevated temperature without spoiling it .A protection equipment is especially made for this purpose . This equipment consists of a thin steel cover around the dial gauge; this steel cover is surrounded by a copper pipe of 7.5 mm with a spiral fashion. The surface of the spiral pipe coated with a layer of glass fiber of 8 mm, which is covered by a thin aluminum sheet.

To protect the movable rod of the dial gauge, it is elongated by porcelain rod .This porcelain rod is covered with a steel tube to be fitted with the movable rod as shown in plate (3-8) .

The performance of this equipment depends on the principle of thermal exchange; a water flows in the copper pipe with a suitable discharge that keeps the temperature of dial gauge.

Plate (3-8) : Protection equipment of dial gauge of deflection.

3-9-4-3 Flexure Test of Reinforced Concrete Slab Specimens .

The slab specimens were tested in flexure using a load cell with maximum capacity of (70 tons). Plate (3-9) shows the details of the test set-up .

The first stage of loading is represented by the weights of steel loading base and the layer of sand, under these loads the deflection was recorded .

The test is began by applying load by the hydraulic jack with appropriate increments of load. A load increment of (475 kg) per once was applied on the slab specimen until failure. The reading of the deflections versus loads were recorded simultaneously for each load increment .

For slab specimens were subjected to fire flame under loading as shown in plate (3-10), the deflection of these specimens was recorded every ten minuets during the period of exposure to fire. The deflection of these slab specimens are resulting from loading and applied fire.

Plate (3-9) : Testing of slab specimen under uniform load
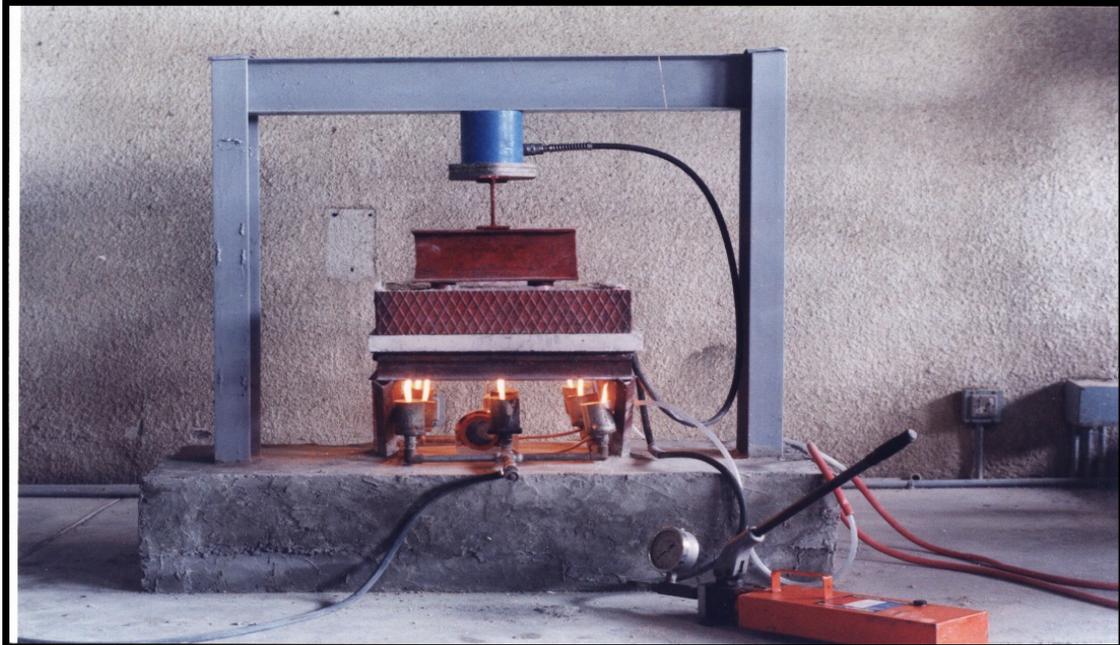
Plate (3-10) : Testing of slab specimen under uniform load

with exposing to fire flame.

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Chapter Four

Results and Discussion

4-1 Introduction

The experimental results that are presented in this chapter show the effect of fire flame on flexural characteristics of reinforced concrete slab specimens such as ultimate load carrying capacity, load- deflection relationship, the cracks detection and deflection due to loading and exposing to fire flame. Also, fire endurance periods of the slab specimens are determined. The test results of weight loss and experimental results obtained from non-destructive tests of slab specimens, and effect of fire flame on some mechanical properties of concrete and reinforcement steel are presented.

4-2 Effect of Fire Flame on the Mechanical Properties of Concrete

Table (4-1) shows the effect of the exposure to fire flame on some mechanical properties of concrete such as compressive strength, modulus of rupture, modulus of elasticity and splitting tensile strength of concrete specimens.

Table (4-1): Effect of fire temperatures on the mechanical properties of

concrete.

Series	Slab No.	Exposure Temp. (°C)	fc⁻ (MPa)	(fc)a/ (fc)b	fr (MPa)	(fr)a/ (fr)b	Ec (GPa)	(E)a/ (E)b	fsp (MPa)	(fsp)a/ (fsp)b
A	A₁₁	25	30.8	1.0	3.14	1.0	26.4	1.0	3.11	1.0
	A₁₂	400	25..8	0.84	2.4	0.77	16.3	0.62	2.36	0.76
	A₁₃	500	22.8	0.74	1.91	0.61	10.9	0.41	1.77	0.57
	A₁₄	600	19.1	0.62	1.57	0.5	6.9	0.26	1.21	0.39
	A₂₁	25	30.6	1.0	3.09	1.0	26.23	1.0	3.08	1.0
	A₂₂	400	26.1	0.85	2.35	0.76	16.8	0.64	2.43	0.79
	A₂₃	500	22.4	0.73	1.95	0.63	11.8	0.45	1.81	0.59
	A₂₄	600	19.6	0.64	1.6	0.52	7.2	0.28	1.26	0.41
B	B₁₁	25	38.4	1.0	3.7	1.0	31.7	1.0	3.64	1.0
	B₁₂	400	31.9	0.83	2.86	0.77	20.3	0.64	2.8	0.77
	B₁₃	500	27.6	0.72	2.2	0.59	13.16	0.42	2.0	0.55
	B₁₄	600	23.0	0.6	1.79	0.48	7.8	0.25	1.38	0.38
	B₂₁	25	38.6	1.0	3.72	1.0	31.73	1.0	3.62	1.0
	B₂₂	400	31.3	0.81	2.9	0.78	19.41	0.61	2.83	0.78
	B₂₃	500	26.5	0.69	2.3	0.62	12.71	0.4	2.1	0.58
	B₂₄	600	22.4	0.58	1.83	0.49	7.1	0.22	1.27	0.35

(fc⁻)a = Compressive strength (cube) after exposure to fire flame.
(fc⁻)b = Compressive strength (cube) before exposure to fire flame.
 (fr)a = Modulus of rupture after exposure to fire flame.
(fr)b = Modulus of rupture before exposure to fire flame.
(Ec)a = Modulus of elasticity after exposure to fire flame .
(Ec) b = Modulus of elasticity before exposure to fire flame.
(fsp)a = Splitting tensile strength after exposure to fire flame.
(fsp)b = Splitting tensile strength before exposure to fire flame

4-2-1 Compressive Strength.

The compressive strength results with the fire temperatures are summarized in Table (4-1) for both series A and B, while Figure (4-1) shows the relation between compressive strength and fire temperatures for both series A and B. It is obvious from the results that compressive strength decreases significantly after exposing to fire as follows:

At temperature around 400 $^\circ$C, the percentages of residual compressive strength were (84-85%) and (81-83%) for series A and B respectively.

The tested specimens showed a further loss in compressive strength at temperature of 500 $^\circ$C where the residual compressive strength ranged between (73-74%) and (69 – 72%) for series A and B respectively.

At temperature around 600°C, the specimens exhibit a further loss of compressive strength.The percentages of residual compressive strength ranged between (62-64%) and (58-60%) for series A and B respectively. These results agreed with that obtained by Essa[13] and Umran[15].The effect of fire flame was seen to be in ranges less than that obtained by Ass' ad[35] and Hidayat[37] . This can be attributed to the effect of fire flame which hits the lower surface of a member is less intensive than the effect of homogeneous heat subjected to the element in electrical ovens.

In addition, it can be observed that the effect of fire flame on the compressive strength approximately is equal for both series A and series B.

The decrease in compressive strength of concrete is attributed to the break down of interfacial bond due to incompatible volume changes between cement paste and aggregate during heating and cooling[54,55] .Also, dehydration of calcium hydroxide occurs when temperature exceeds about 750 $^\circ$F (400°C)

and causes shrinkage of the cement paste . Various quartz like aggregates undergo a crystalline transformation at about 1070 $^\circ$F (577°C)which causes a large expansion and cracking of the concrete[56] .
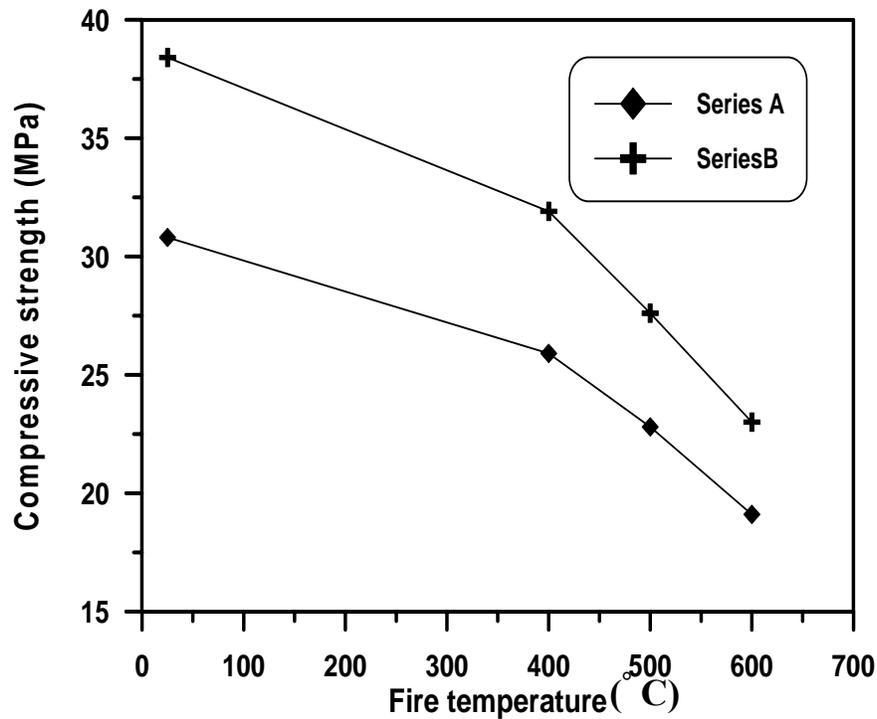
Figure (4- 1): The effect of fire temperature on the compressive strength.

4-2-2 Modulus of Rupture

The values of flexural strength of the specimens in the present investigation are abstracted in Table (4-1). Figure (4-2) shows the relationship between residual flexural strength and fire temperatures for series A and B. It can be seen that the reduction in flexural strength is more pronounced than that in compressive strength.

At temperature around 400°C, the percentages of residual flexural strength were in the the range of (76-77%) and (77-78%) for series A and B respectively.

At temperature around 500°C, the percentages of residual flexural strength were (61-63%) and (59-62%) for series A and B respectively.

At temperature around 600°C, the specimens exhibit a loss of flexural strength, the percentages of residual flexural strength were (50-52%) for series A and (48-49%) for series B .The reduction in flexural strength can be explained by the fact that during exposure to fire temperature , the evaporable water causing triaxial tension within the concrete .When a flexural load is applied the tensile stresses due to the applied load become additive to the triaxial tension resulting in a drop in flexural strength [16].
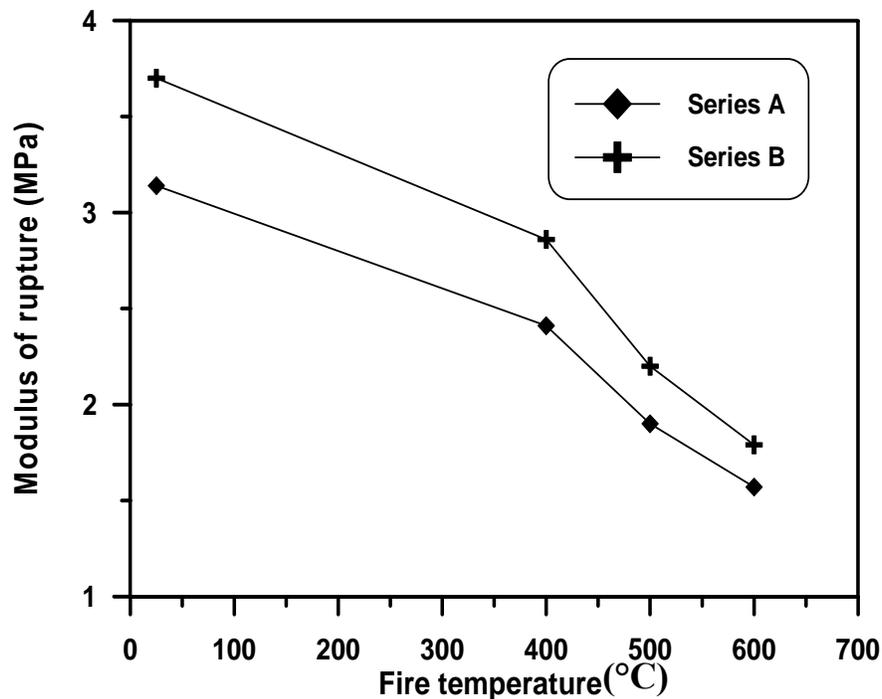
Figure (4- 2): The effect of fire temperature on the modulus of rupture.

4-2-3 Concrete Modulus of Elasticity

Test results of the modulus of elasticity are summarized in Table (4-1).

Figure (4-3) illustrates the relationship between the residual modulus of elasticity and fire flame temperatures for series A and B. From these results, it can be seen that the reduction values of concrete modulus of elasticity were more significant than that of the compressive and flexural strength at identical fire flame temperatures.

At temperature around 400°C, there was a significant reduction in the concrete modulus of elasticity due to effect of fire flame. The percentages of residual value of modulus of elasticity were in the ranges of (62-64%) and (61-64%) for series A and B respectively.

At temperature around 500°C, the percentages of residual value of modulus of elasticity were (41- 45%) and (40 - 42%) for series A and B.

At temperature around 600°C, the percentages of residual value of modulus of elasticity were (26-28%) and (22-25%) for series A and B. The reduction in modulus of elasticity of concrete can be attributed to the increase in the amount of cracks formation due to exposure to fire and the physico - chemical transformation in concrete constituenents during burning will yield strength loss [15, 35, 37].
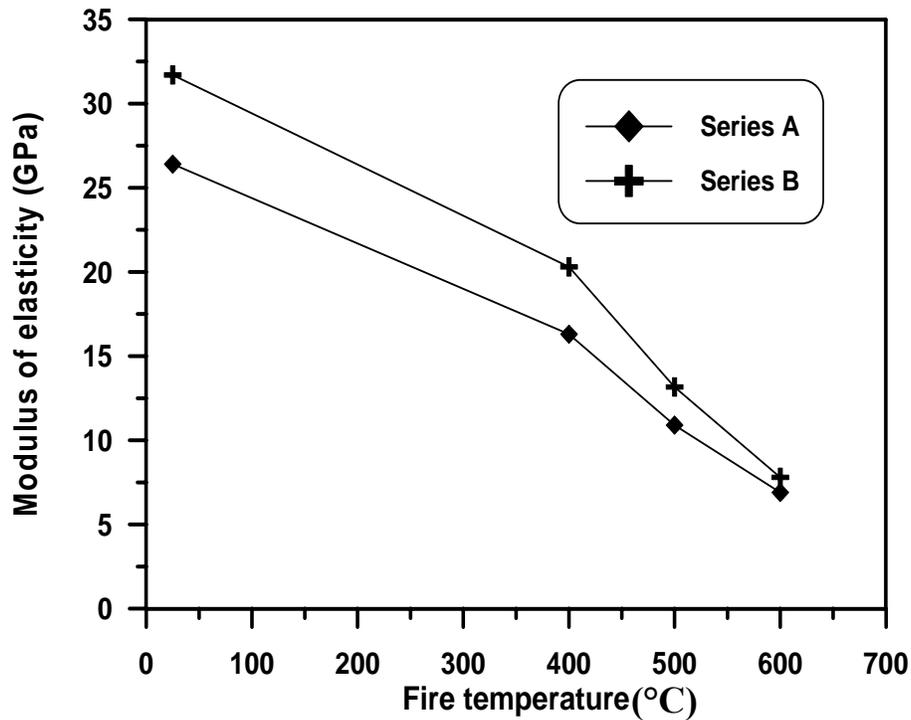
Figure (4-3): The effect of fire temperature on the modulus of elasticity.

4-2-4 Splitting Tensile Strength

The test results of splitting tensile strength are given in Table (4-1). Figure (4-4) shows the relationship between the splitting tensile strengths and fire flame temperatures.

It is clear from the test results that the splitting tensile strength is more sensitive to elevated temperatures than the compressive strength. It can be observed that concrete deteriorates at a faster rate when tested in tension rather than in compression. This observation conforms the results obtained by other investigators [7, 10] . This indicates that severe microcraking insue due to fire.

At temperature around 400°C, the percentages of residual splitting tensile strength were in the ranges of (76- 79 %) and (77-78%) for series A and B respectively.

At temperature around of 500 °C, the percentages of residual splitting tensile strength were in the ranges of (57-59%) and (55-58%) for series A and B respectively.

At 600°C fire temperature, the percentages of residual splitting tensile strength became in the ranges of (39-41%) and (35-38%) for series A and B respectively.

The reduction in splitting tensile strength can be attributed to the formation of tensile stresses during the contraction of hardened cement paste upon cooling ,which , when superimposed onto the already existing tensile stresses formed during heating, would cause further reduction in splitting strength[55] .
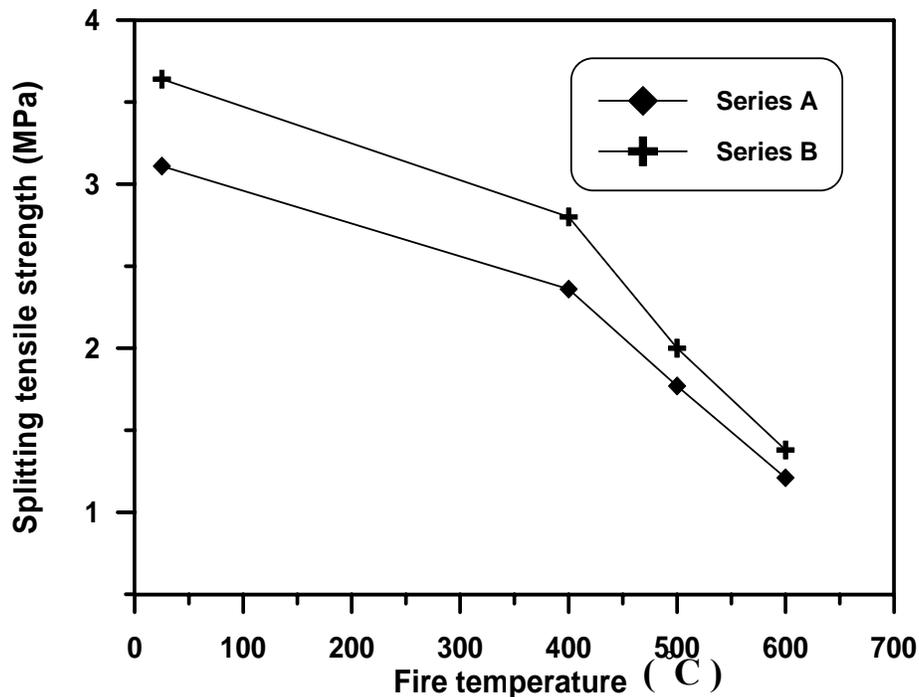
Figure (4-4): The effect of fire temperature on the splitting tensile strength.

4-3 Effect of Fire Flame on the Steel Reinforcement Bars

The effect of fire temperature on the properties of steel reinforcement bars is summarized in Table (4-2). At temperatures of (400 and 500°C), both burning and subsequent cooling did not affect the mechanical properties of the steel reinforcement bars, but this effect was observed at burning temperature of 600°C.

The percentages of residual yield tensile stress and ultimate tensile stress were (89.7% and 91.8%) respectively. The modulus of elasticity was not affected by burning and cooling at all levels of temperature. Similar behavior was also recorded by other investigators [15, 27, 39]

Table (4-2): Effect of fire temperature on the properties of steel bars .

Exposure Temp. (°C)	Yield Tensile Stress N/mm^2	Residual Yield Tensile Stress %	Ultimate Tensile Stress N/mm^2	Residual Ultimate Tensile Stress %	Modulus of Elasticity Es (Gpa)	Residual Es %
25	340	100	480	100	205	100
400	340	100	480	100	205	100
500	340	100	480	100	205	100
600	305	89.7	441	91.8	205	100

4-4 Effect of Fire Flame on the Weight Loss of Reinforced Concrete Slab Specimens.

The weight loss results of the investigated reinforced concrete slab specimens are summarized in Table (4-3). The weight losses are plotted verses fire flame temperatures as shown in Figure (4-5).

For exposure temperature around 400°C, the decrease in weight was (3-3.1%) for series B and (4.8- 4.9%) for series A respectively. This is due to the removal of the capillary and adsorbed water from the cement paste and loss of gypsum water [13].

At temperature around 500°C, the decrease in weight became (4.7- 4.9 %) and (6.0-6.1 %) for series B and A respectively. This is due to the loss of the interlayer calcium silicate hydrate water and some of the chemically combined water from the calcium silicate hydrate and sulfoaluminate hydrates.

At temperature around 600°C, a further reduction took place, which was about (5.4-5.6 %) for series B and (6.5-6.6 %) for A respectively. This is due to the further dehydration of the cement paste due to decomposition of calcium hydroxide [16, 57].

Also it can be seen that slab specimens of series B loses less weight than that of series A , which can be attributed to the less water- cement ratio and less evaporation of adsorbed water on the interfaces of the gel crystals of hardened cement paste , also to less amount of pores and voids in series B.

The weight loss results obtained from tests carried out by Hidayat[37] in addition to the results of the present study are plotted as shown in Figure (4-6) . It can be seen that the weight loss of slab specimens exposed to fire flame from lower surface was less than the weight loss of slab specimens which

were heated in furnace (homogenous heating).This can be attributed to the less evaporation of water during fire flame exposing .

**Table (4- 3): Weight values of slab specimens before
and after exposure to fire flame.**

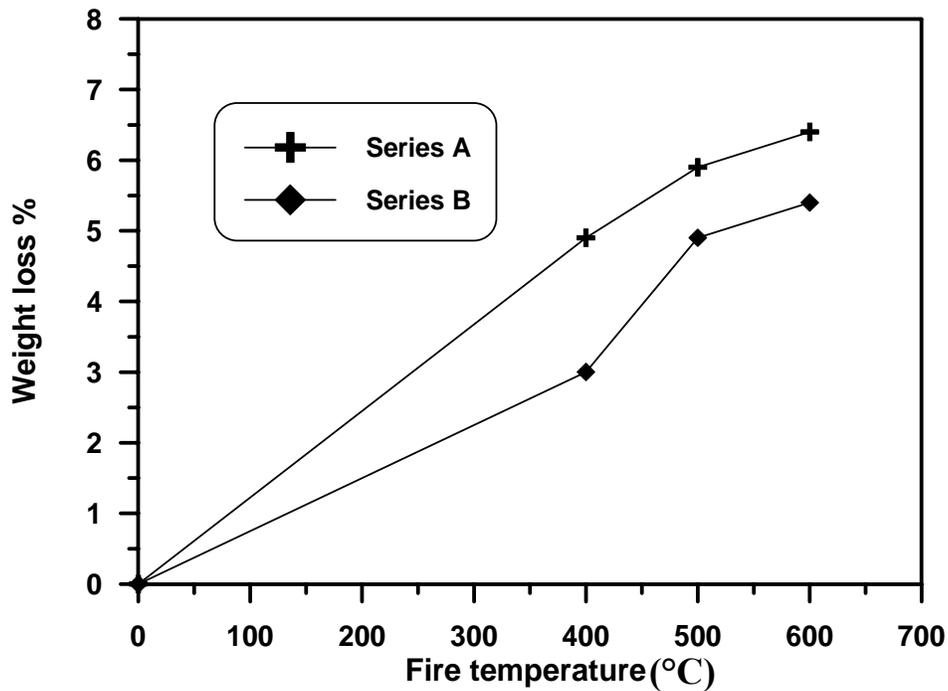| Series	Slab No.	Exposure Temp. ($^\circ$C).	Slab Weight (g)	Weight Loss (%)
A	A_{11}	25	34550	0
	A_{12}	400	32860	4.9
	A_{13}	500	32480	6.0
	A_{14}	600	32300	6.5
	A_{21}	25	35080	0
	A_{22}	400	33400	4.8
	A_{23}	500	32940	6.1
	A_{24}	600	32770	6.6
B	B_{11}	25	35040	0
	B_{12}	400	33950	3.1
	B_{13}	500	33320	4.9
	B_{14}	600	33150	5.4
	B_{21}	25	35480	0
	B_{22}	400	34415	3.0
	B_{23}	500	33810	4.7
	B_{24}	600	33490	5.6

Figure (4-5): The effect of fire temperatures on the % weight loss of reinforced concrete slab specimens
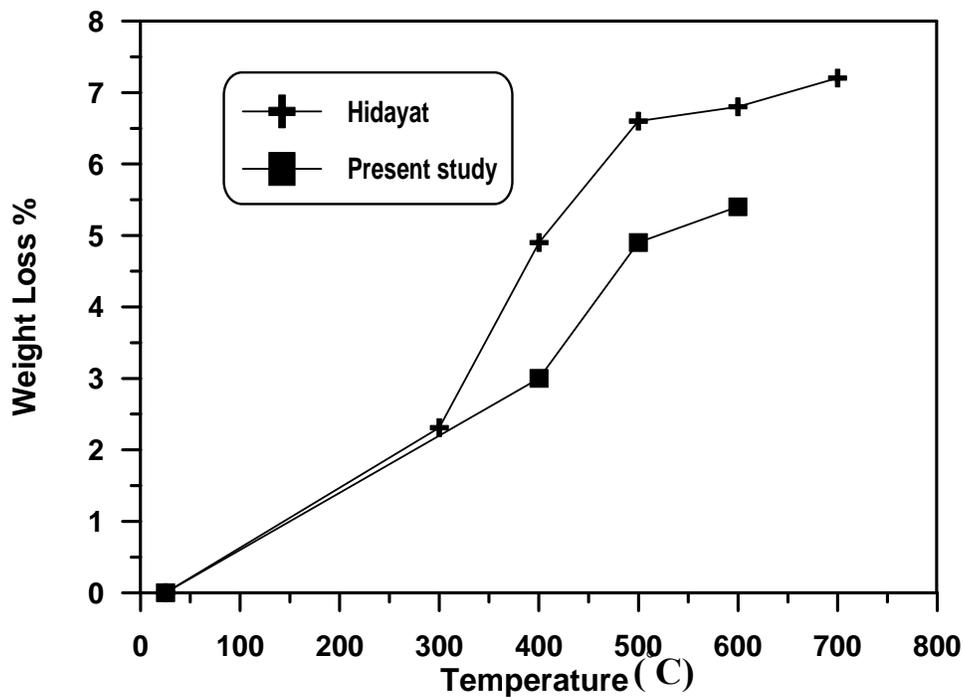

Figure (4- 6): The % weight loss of slab specimens of present study and Hidayat study [37].

4-5 Effect of Fire Temperatures on non-Destructive Tests

4-5-1 Ultrasonic Pulse Velocity Test Results

Ultrasonic pulse velocity of reinforced concrete slab specimens was determined before and after exposure to fire flame, this technique is widely used for assessing the quality of concrete in structures [58]. The measured pulse velocity of concrete can be affected by many factors, including smoothness of concrete surface, temperature of the specimen, moisture conditions, and age of the specimen and presence of steel reinforcement [59].

The ultrasonic pulse velocity results are presented in Table (4-4) for both series A and B. Figure (4-7) shows the relationship between the ultrasonic pulse velocity and the fire flame temperatures. It can seen that the reductions in the ultrasonic pulse velocity after exposure to fire flame were as follows :

At temperature around 400°C, the reduction in (U.P.V) was (23-25%) for series A and (24-26%) for series B.

At temperature around 500°C, the redaction in (U.P.V) was (42-43%)   for series A and (41-44%) for series B.

At temperature around of 600°C, the reduction in   (U.P.V) was (47-48%) for series A and (49-51%) for series B.

The decrease in the ultrasonic pulse velocity is attributed to the formation of microraking, which progressively propagates through the cement mortar, and cement – aggregate interfaces [13]. Similar behavior was observed by other researchers [13, 15].

Table(4- 4):The test results of (U.P.V) and rebound number
of Schmidt hammer of the slab specimens before
and after exposure to fire .

Series	Slab No.	Exposure Temp. $^\circ$C.	U.P.V km/sec	(U.P.V)a/ (U.P.V)b	Rebound Number	$(R_N)a/$ $(R_N)b$
A	A_{11}	25	4.4	1.0	32.0	1.0
	A_{12}	400	3.3	0.75	27.8	0.87
	A_{13}	500	2.5	0.57	25.6	0.80
	A_{14}	600	2.33	0.53	23.7	0.74
	A_{21}	25	4.45	1.0	32.5	1.0
	A_{22}	400	3.45	0.77	28.3	0.88
	A_{23}	500	2.58	0.58	27.0	0.83
	A_{24}	600	2.31	0.52	24.4	0.76
B	B_{11}	25	4.7	1.0	35	1.0
	B_{12}	400	3.5	0.74	30.7	0.88
	B_{13}	500	2.77	0.59	28.4	0.81
	B_{13}	600	2.4	0.51	25.5	0.73
	B_{21}	25	4.9	1.0	35.2	1.0
	B_{22}	400	3.7	0.76	30.3	0.86
	B_{23}	500	2.75	0.56	27.8	0.79
	B_{24}	600	2.42	0.49	25.0	0.71

(U.P.V)a=Ultrasonic pulse velocity after exposure to fire flame .
(U.P.V)b=Ultrasonic pulse velocity before exposure to fire flame.
(R_N)a = Rebound number after exposure to fire flame .
 (R_N)b = Rebound number before exposure to fire flame.
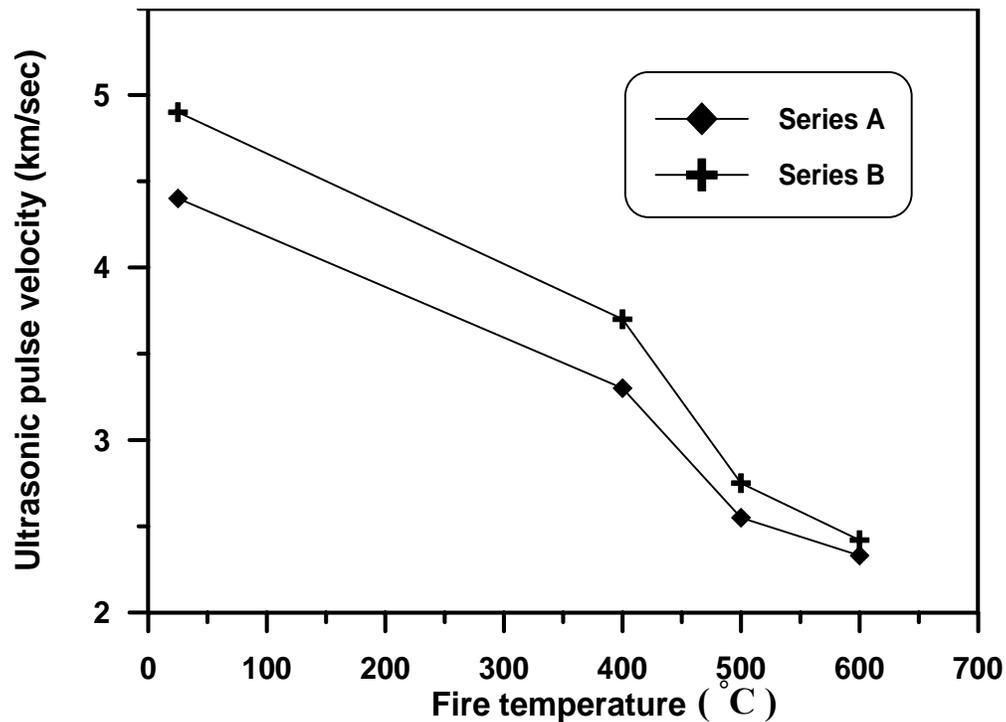
Figure (4- 7): The effect of fire temperature on the ultrasonic pulse velocity of Reinforced concrete slab specimens.

4-5-2 Surface Hardness Results.

Surface hardness of reinforced concrete slab specimens was assessed by the "Schmidt rebound hammer". The readings of the rebound number were recorded before and after subjecting the slab specimens to fire . The Schmidt rebound number could be used as a measurement of the degree of degradation of the concrete surface.

Table (4-4) shows the results of the rebound number for slab specimens of the two series before and after exposure to fire flame

The effect of exposing the specimens to fire flame on the rebound number is shown in Figure (4-8). It can be seen that exposing the slab specimens to fire decrease, the rebound number is as follows:

At temperature around $400°C$, the reduction in rebound number was about(12-13%) for series A (12-14%) for series B.

At temperature around $500°C$, the reduction in the rebound number was about (17-20%) for series A and (19-21%) for series B.

At temperature around $600°C$, the reduction in the rebound number was (24-26%) for series A and (27-29%) for series B. These result agreed with that obtained by other researchers [13, 15]. The reduction in the rebound number can be attributed to the fact that fire decreases the surface hardness of concrete.

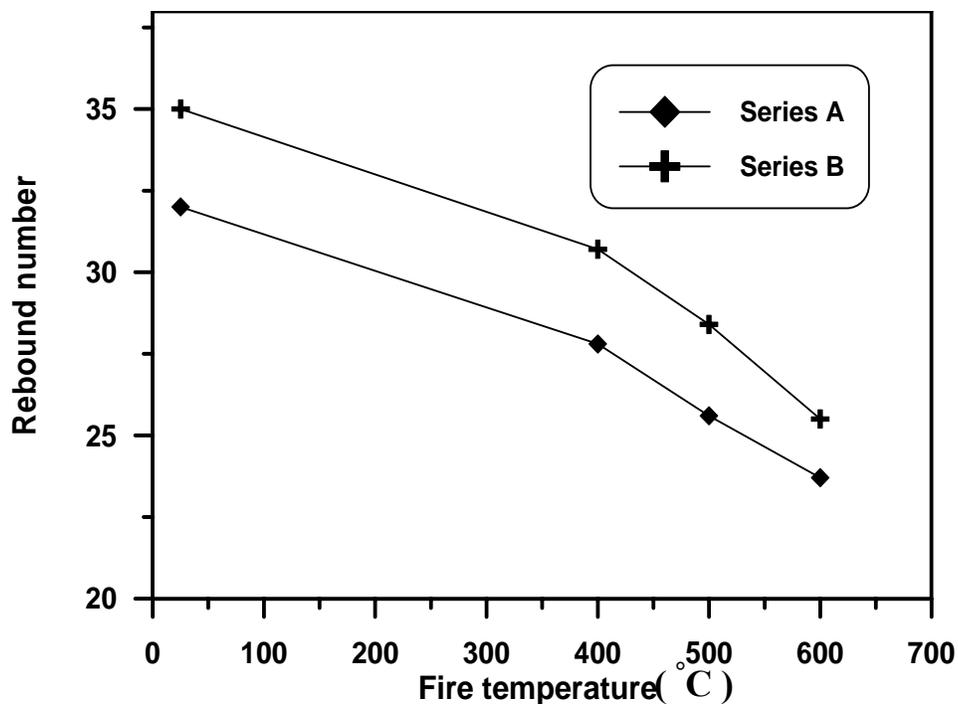**Figure (4- 8): The effect of fire temperature on the rebound number of reinforced concrete slab specimens.**

4-6 Surface Conditions and Fire Endurance of Slab Specimens

The aim of any design for fire safety should be provided to limit the damage resulting from fire .The unexposed surface of the slab specimens, which were exposed to fire and then tested under uniformly distributed load, was observed throughout 1-hr fire exposure. All tested slab specimens developed hairline cracks appearing in honeycomb fashion all over the exposed surface. As exposure to fire flames continued, more cracks appeared and propagated towards the edge of the slab specimens. Similar behavior was observed by other investigators [15, 39].

Figurers (4-9)to (4-20) show the temperature –time curves for the exposed surface , steel reinforcement mid-depth, and unexposed surface for the slab specimen.At the beginning the slab specimens were at the room temperature ,measured to be 25°C .

The experimental results indicate that the temperature near the surface exposed to fire is higher and decreases towards the top surface of the slab. These results agreed with that obtained by other investigators [36, 39].

Figures (4-21) to (4-23) show the unexposed surface temperature of each slab specimen throughout the 1-hour fire test .

It can be seen from these Figures, the behavior of all slab specimens is similar. The slab specimens of series A show slightly lower unexposed surface temperatures than that of series B at the end of 1-hour fire exposure. This is may be attributed to the higher porosity of concrete of this series. On the other hand, the slab specimens reinforced with higher steel ratio show slightly higher unexposed surface temperature than that of slab specimens reinforced

with lower steel ratio at the end of 1-hour exposure. This may belong to the higher concentration of steel reinforcement of slab specimens.

Fire endurance periods are normally determined by physical tests conducted according to the provisions of ASTM E119-88 [6] .Under this standard, the fire endurance of member or assembly is determined by the time required to reach any of following three points [60]:

1- Passage or propagation of flame to the unexposed surface of the assembly.

2-A temperature rise of 163°C at a single point or 121°C as an average unexposed surface of the test assembly , and

3-Inability to carry the applied design load or structural failure.

Fire endurance of the slab specimens was reached when the average temperature rise of the unexposed surface exceeded (121°C)

Table (4-5) lists the fire endurance based on the temperature rise of the unexposed surface for each test slab.

Figure (4- 9): Slab temperature as a function of time at various depths for slab No. A$_{12}$,at exposure temperature of 400 $^{\circ}$C .

Figure (4- 10): Slab temperature as a function of time at various depths for slab No. A$_{22}$, at exposure temperature of 400 $^\circ$C.

Figure (4- 11): Slab temperature as a function of time at various depths for slab no. A$_{13}$, at exposure temperature of 500 $^\circ$C.

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Figure (4- 12): Slab temperature as a function of time at various depths for slab No. A$_{23}$,at exposure temperature of 500 $^{\circ}$C .
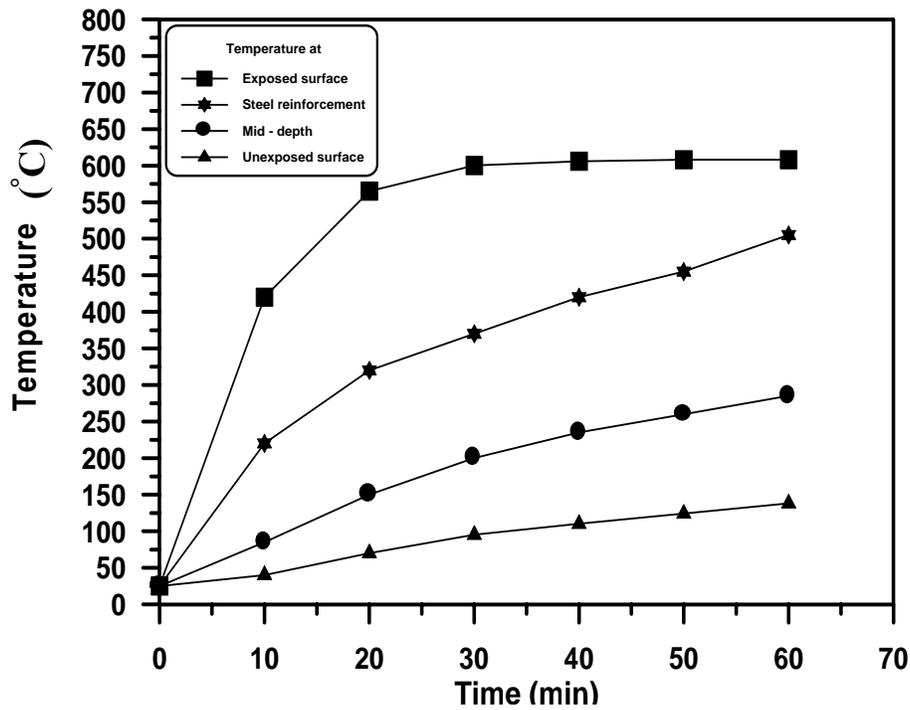
Figure (4-13): Slab temperature as a function of time at various depths for Slab No. A$_{14}$, at exposure temperature of 600 $^{\circ}$C.
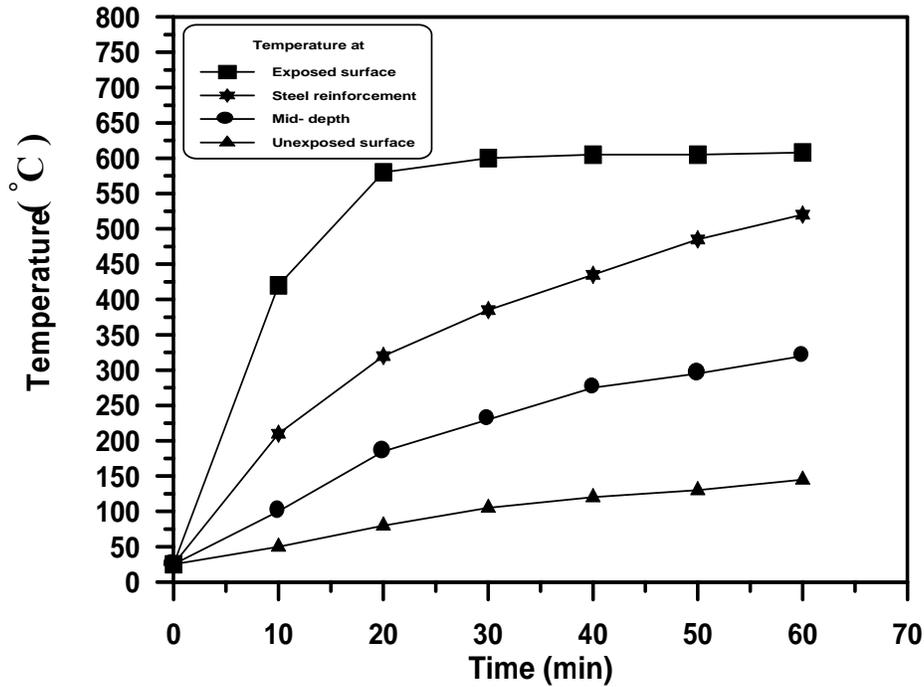
Figure (4-14): Slab temperature as a function of time at various depths for slab No. A₂₄, at exposure temperature of 600 °C.
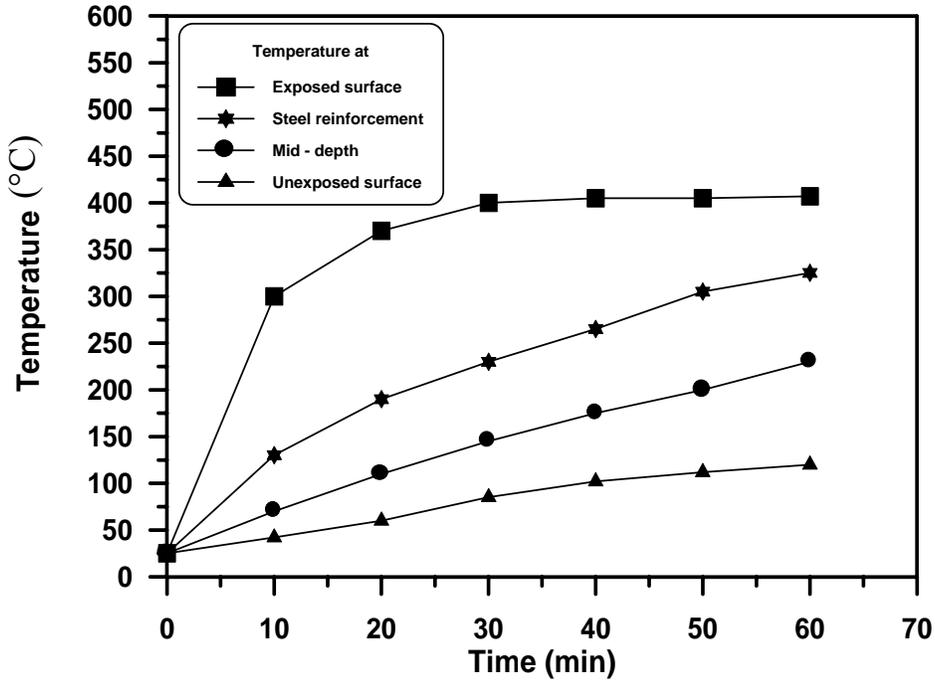
Figure (4-15): Slab temperature as a function of time at various depths for slab No. B₁₂, at exposure temperature of 400 °C.
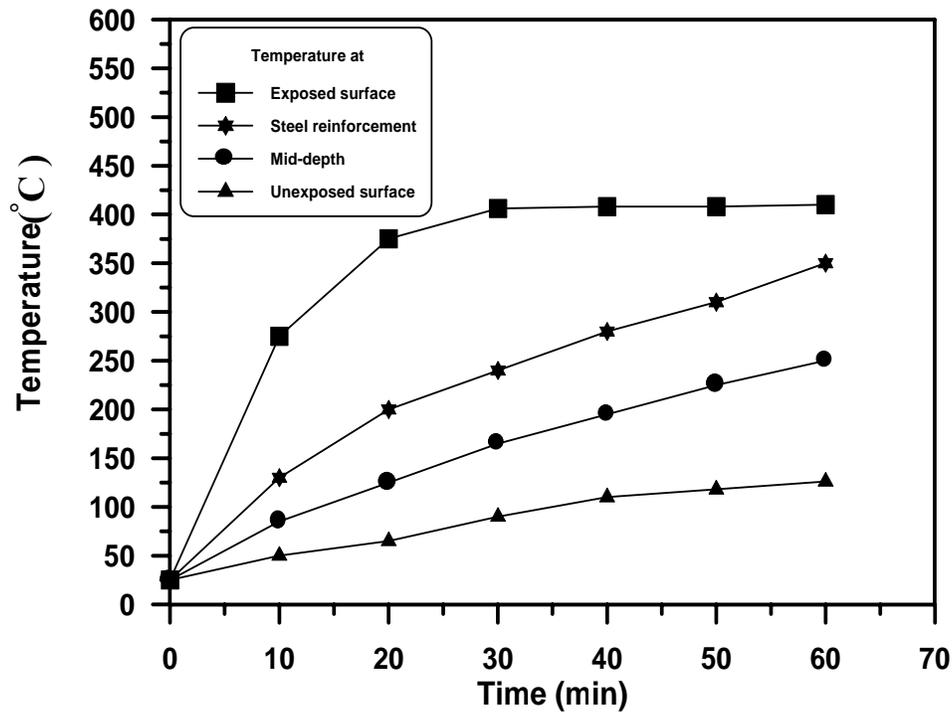
Figure (4- 16): Slab temperature as a function of time at various depths for slab No. B₂₂ ,at exposure temperature of 400 °C .
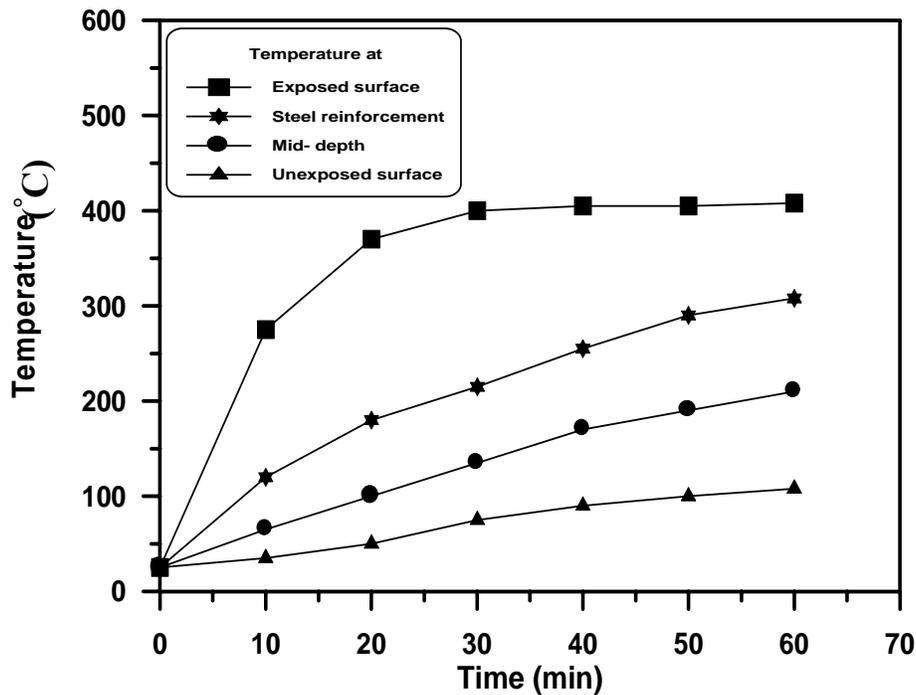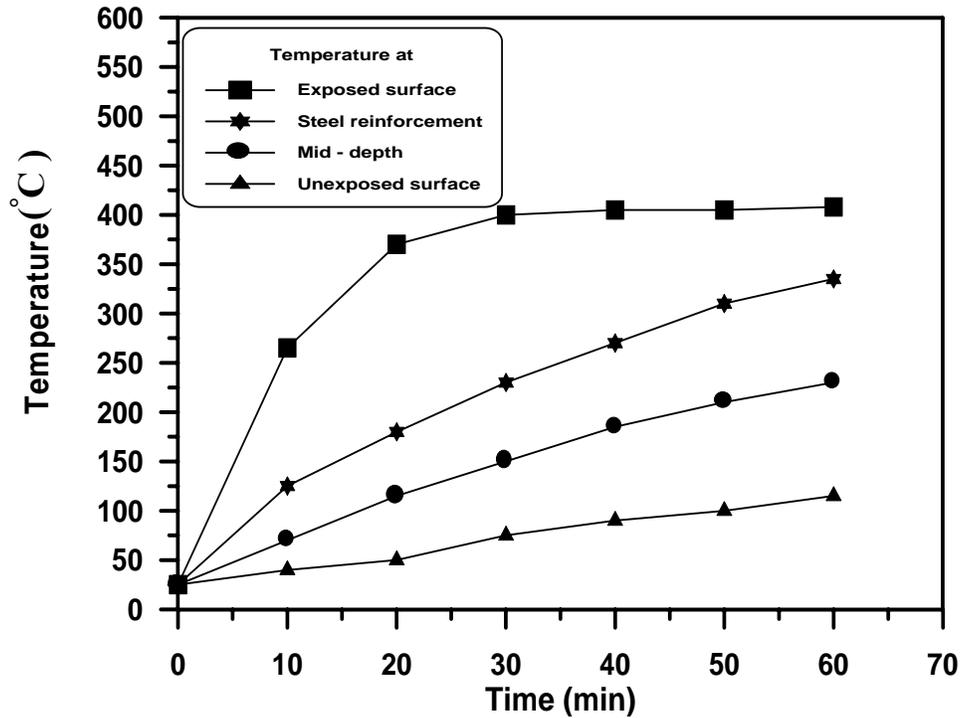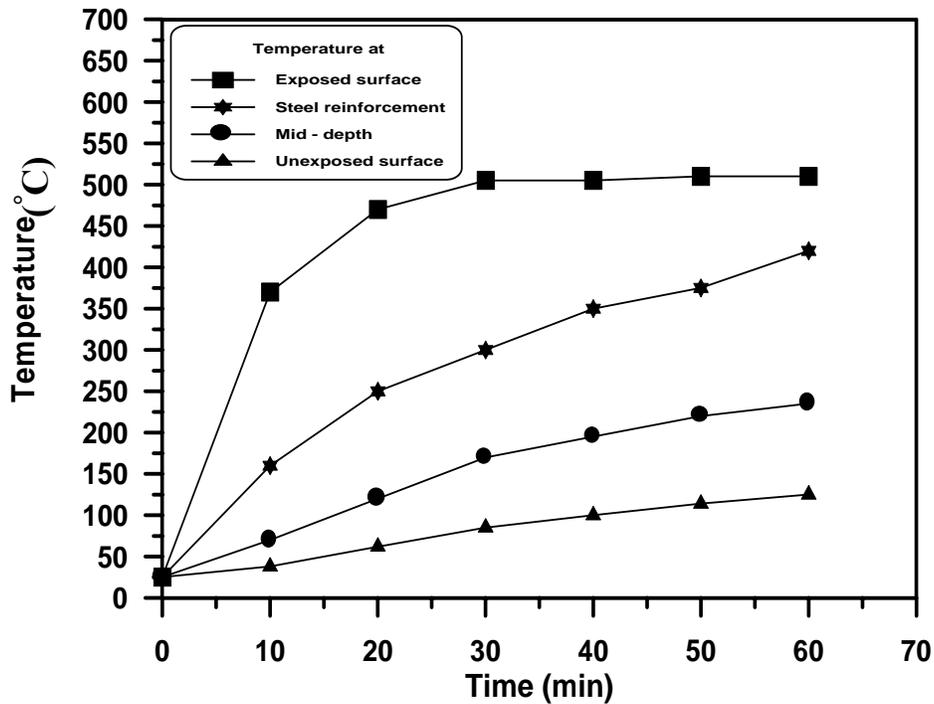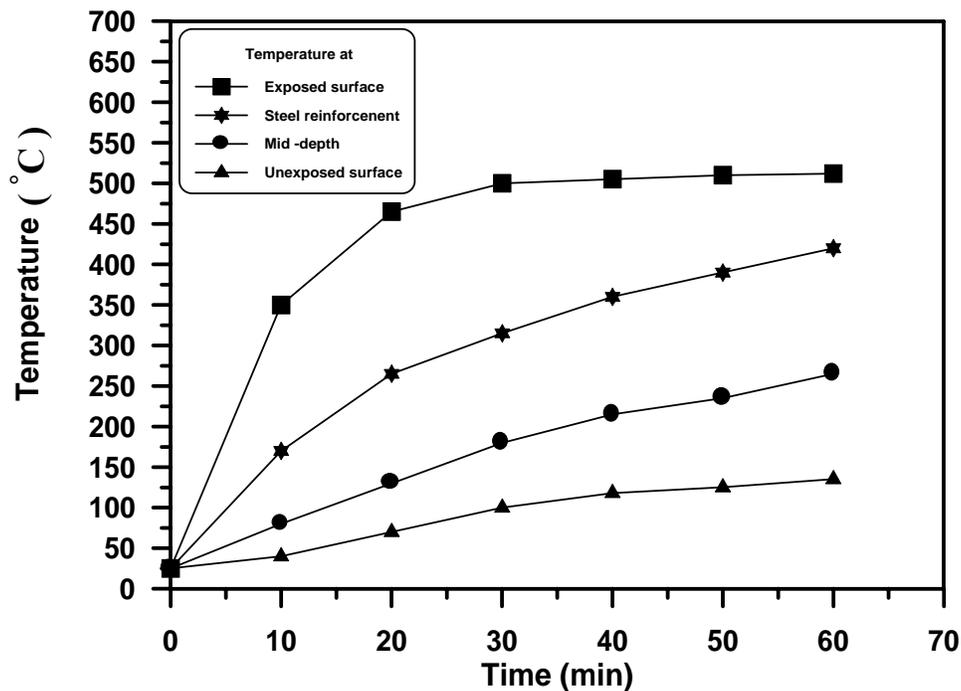

Figure (4- 17): Slab temperature as a function of time at various depths for slab No. B₁₃, at exposure temperature of 500 °C .
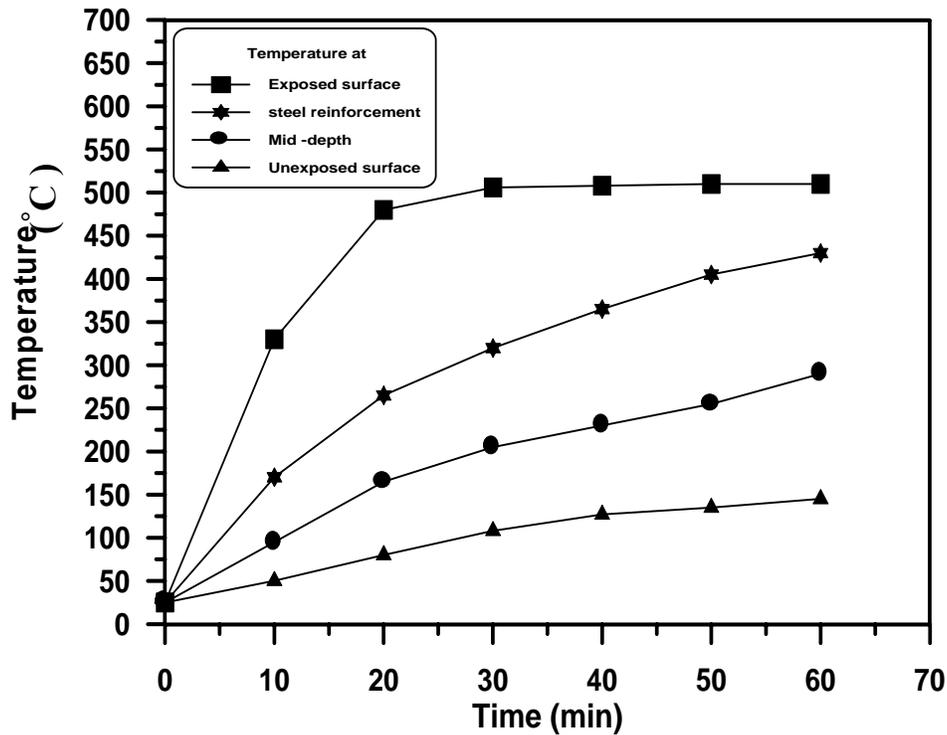
Figure (4-18) : Slab temperature as a function of time at various depths for slab No. B$_{23}$, at exposure temperature of 500 $^\circ$C.
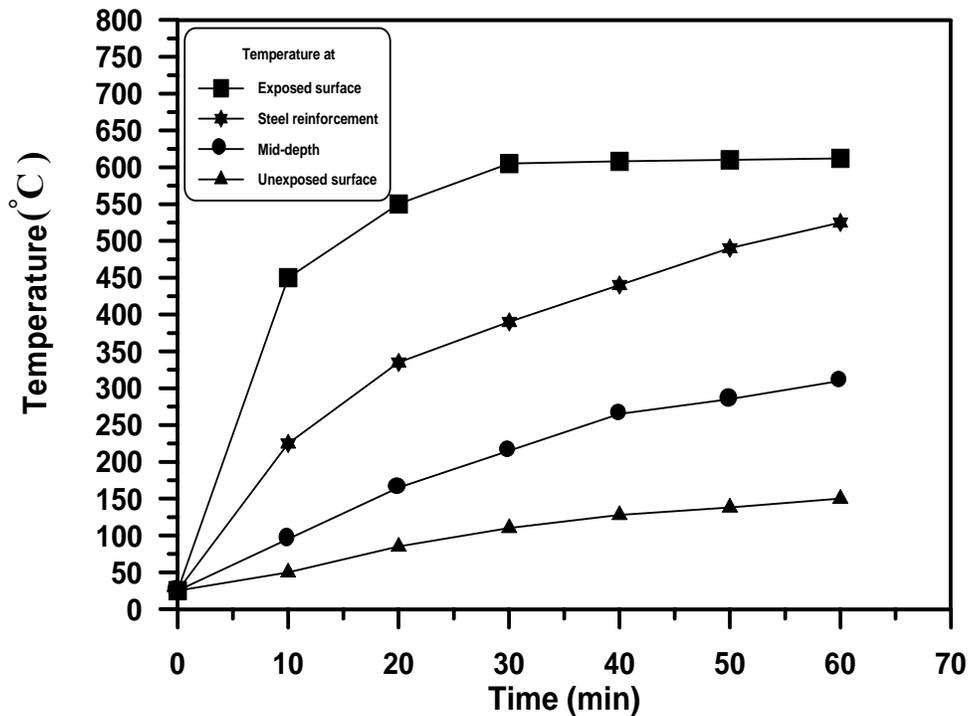
Figure (4-19) : Slab temperature as a function of time at various depths for slab No. B$_{14}$,at exposure temperature of 600 $^\circ$C .

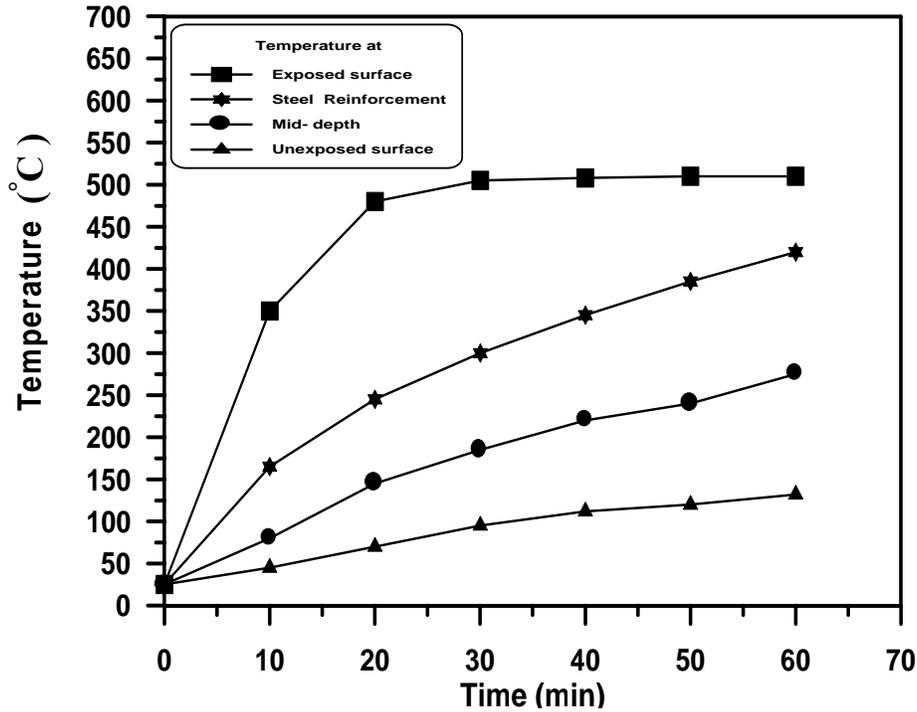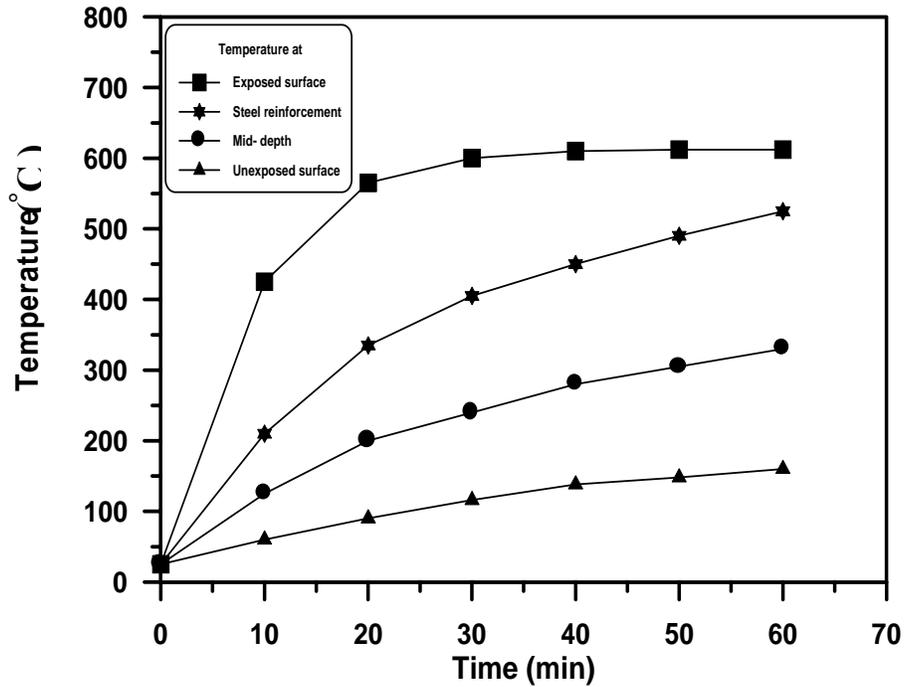**Figure (4- 20): Slab temperature as a function of time at various depths for slab No. B$_{24}$, at exposure temperature of 600 °C.**
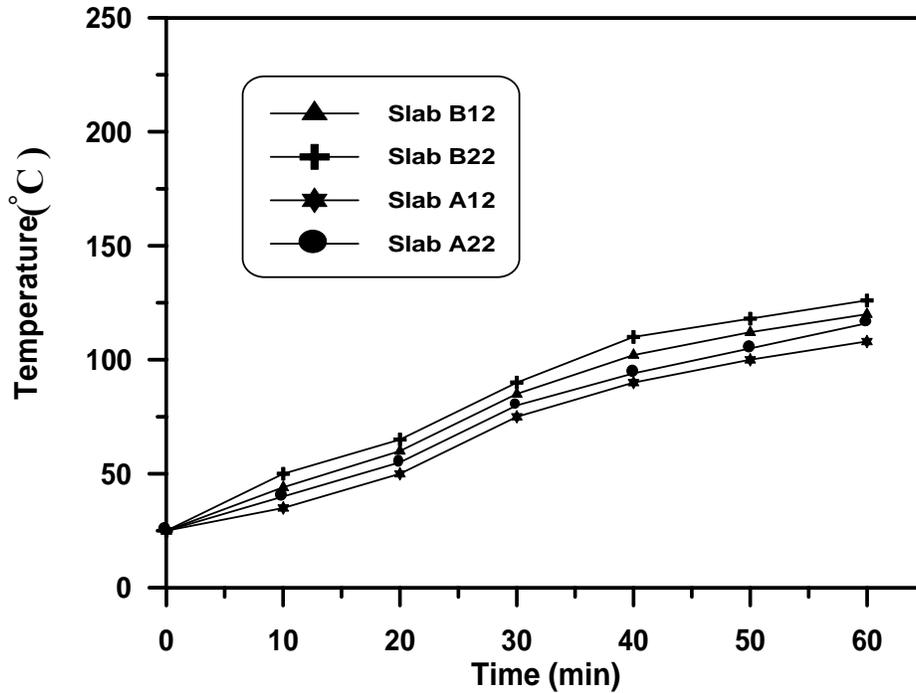
Figure (4- 21) : Average temperature rise of unexposed surface of reinforced concrete slab specimens ,at exposure temperature of 400 °C .
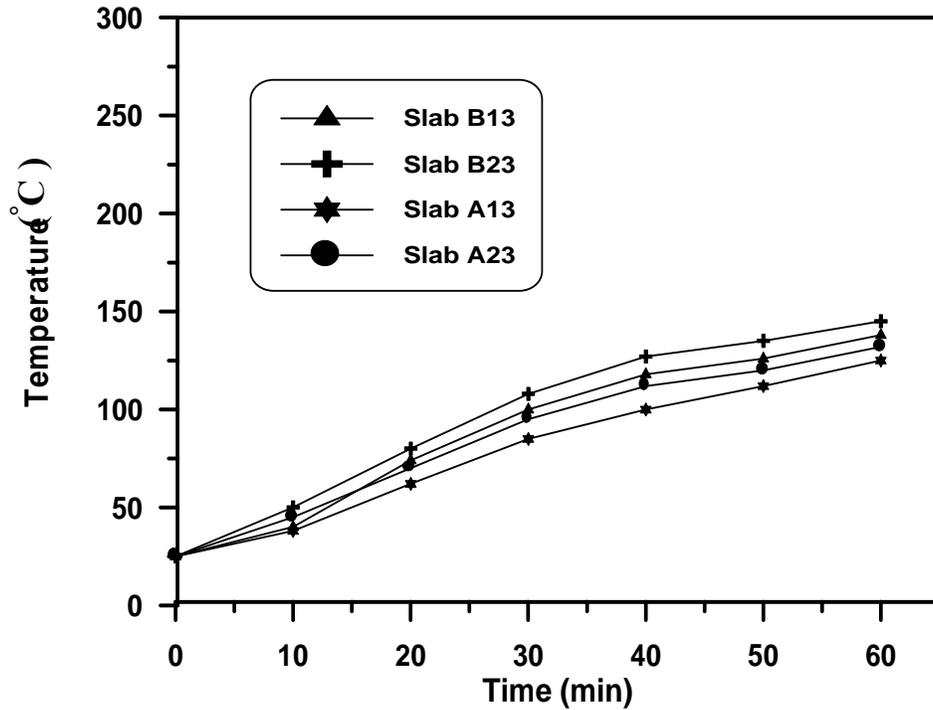
Figure (4- 22) : Average temperature rise of unexposed surface of reinforced concrete slab specimens ,at exposure temperature of 500 °C .
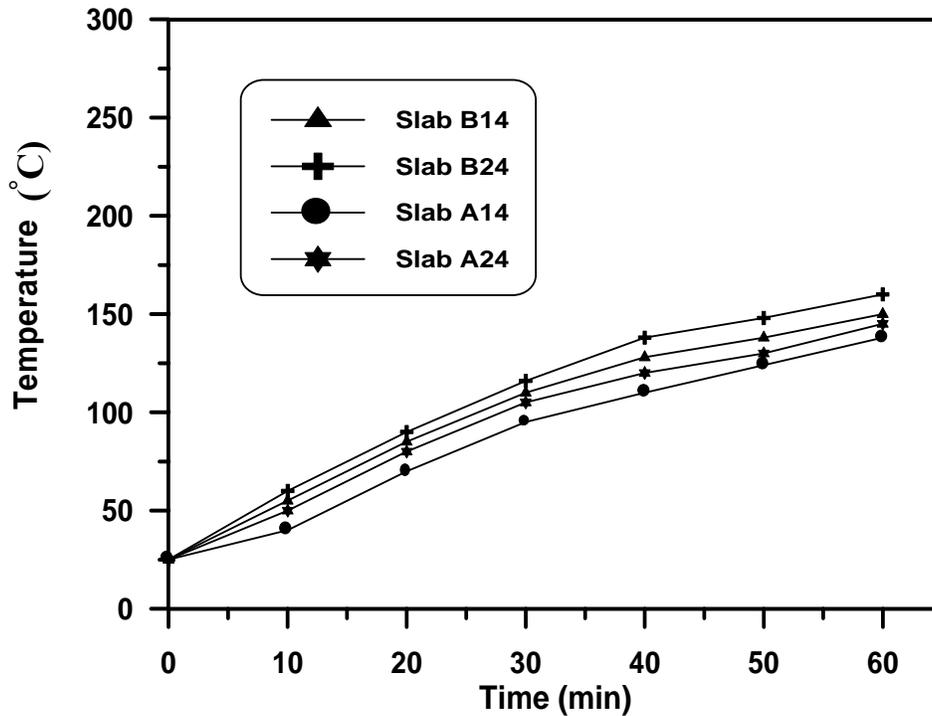
Figure (4- 23) : Average temperature rise of unexposed surface of reinforced concrete slab specimens ,at exposure temperature of 600 °C

Table (4- 5): Fire test results

Series	Slab No.	Exposure Temp. ($^{\circ}$C)	Total Time of Fire Test (min)	Fire Endurance Time (min)
A	A_{12}	400	60	–
	A_{13}	500	60	56
	A_{14}	600	60	47
	A_{22}	400	60	–
	A_{23}	500	60	52
	A_{24}	600	60	40
B	B_{12}	400	60	-
	B_{13}	500	60	44
	B_{14}	600	60	36
	B_{22}	400	60	54
	B_{23}	500	60	37
	B_{24}	600	60	31

4-7 Effect of Fire Flame on the Behavior of Reinforced Concrete Slab Specimens.

4-7-1 Ultimate Load Carring Capacity

The results of the ultimate load of the reference slab specimens and that exposed to fire flame are summarized in Table (4-6) for both series A and B. It is clear from the results that the values of ultimate load capacity, decreased for all specimens after exposing to fire flame.

It was observed that the slab specimens, which are related to series A and for steel ratio of (0.00492), had retained (95.8, 93.2 and 86.0 %) as a percentage from the control specimens after exposure to fire temperature of

(400,500 and 600°C) respectively. A similar trend of reduction was observed for the other series, as recorded in Table (4-6).

It is obvious from the results that the percentages of residual ultimate load at temperature of (400 and 500°C) were higher than that at 600°C for all specimens. This behavior can be attributed to fire response of steel reinforcement, where the yield stress of steel reinforcement was not affected by exposing to fire temperature of (400 and 500°C),while it decreased at temperature of 600 $^{\circ}$C by about (10.28 *%)*.Similar behavior was observed by other investigators[15,35,37]

The percentage residual ultimate load verses fire temperature for slab specimens reinforced with two steel ratios are shown in Figures(4-24)and (4-25). It was observed that as the burning took place, the percentage residual ultimate load for the specimens reinforced with the lower steel ratio was higher than that for those reinforced with higher steel ratio. This can be attributed to the higher internal restraint, which is caused by the steel bars of the higher steel ratio, which means that more micro cracks would occur due to the incompatible volume changes between the concrete and the reinforcing steel bars during burning and cooling. These results agreed with that obtained by Hidayat [37].

**Table (4-6): Test values of ultimate load and percentage residual
ultimate load for referenece slab specimens and
that exposed to fire flame .**

Series	Slab No.	Exposure Temp. (°C.)	Compressive Strength (MPa)	Steel Ratio (ρ)	Balance Ratio of steel (ρb)	Ultimate Load (kN)	Percentage Residual Ultimate Load %
A	A_{11}	25	30.8		0.041	54.8	100
	A_{12}	400	25.9		0.035	52.2	95.8
	A_{13}	500	22.8	0.00492	0.031	51.0	93.2
	A_{14}	600	19.1		0.030	47.2	86.0
	A_{21}	25	30.6		0.041	79.9	100
	A_{22}	400	26.1		0.035	74.6	93.4
	A_{23}	500	22.4	0.00875	0.031	72.7	91.0
	A_{24}	600	19.6		0.030	67.2	84.2
B	B_{11}	25	38.4		0.052	58.0	100
	B_{12}	400	31.9		0.043	56.6	97.5
	B_{13}	500	27.6	0.00492	0.037	54.5	94.0
	B_{14}	600	23.0		0.036	49.3	85.0
	B_{21}	25	38.6		0.05	83.8	100
	B_{22}	400	31.3		0.04	80.3	95.8
	B_{23}	500	26.5	0.00875	0.035	77.0	91.8
	B_{24}	600	22.4		0.033	70.2	83.7
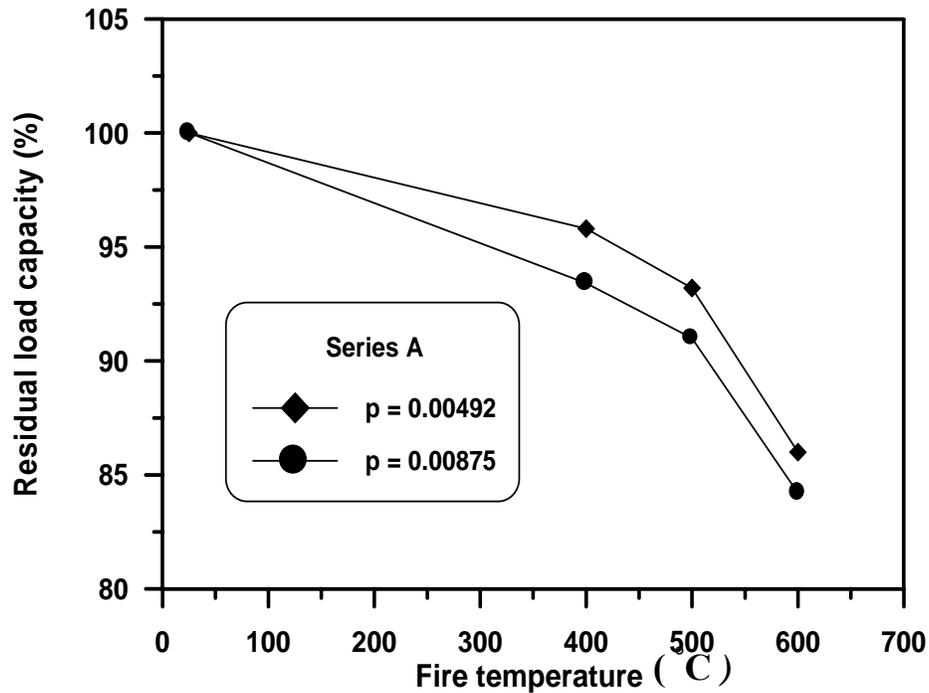
Figure (4- 24): Effect of fire temperature on the percent of residual load capacity of reinforced concrete slab specimens,(series A)
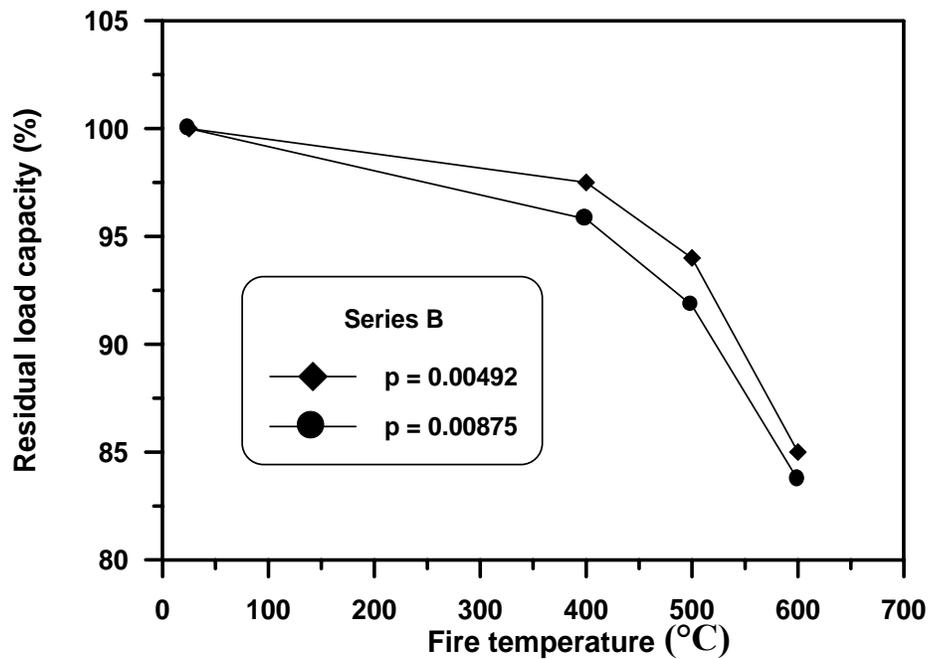
Figure (4-25): Effect of fire temperature on the percent of residual load capacity of reinforced concrete slab specimens,(series B)

4-7-2 Load –Deflection Relationship

Effect of fire temperature on the load- deflection relationship of slab specimens is presented in Figures (4-26) to (4-29). It can be seen that for the same steel reinforcement ratio and under a certain load the deflection increases with increasing the burning temperature.This can be attributed to the fact that fire causes a reduction in slab stiffness which is essentially due to the reduction in the modulus of elasticity of concrete and the effective moment of inertia of the specimens [15, 35, 37].

With respect to these curves, it can be seen that the load- deflection curve at $25^°$**C** exhibits three stages each with different slope. The first stage is the elastic or uncracked stage , the second stage is multiple cracking stage; and the third is the plastic stage which represents the yielding of reinforcement and widening of cracks.

At temperature of $400^°$**C** and $500^°$**C**, the three above stages were less pronounced .

At temperature of $600^°$**C,** the load-deflection relations of slab specimens are flater , representing softer load-deflection behavior than that of control slab specimens. This can be attributed to the reduction in modulus of elasticity of concrete and increase in the amount of cracks formation. These results were in line with other researchers' results [15, 35, 37]**.**

The load–deflection results obtained from tests carried out by Hidayat [37] in addition to the results of present study are plotted in Figures (4- 30) to (4- 35). It can be seen that the deflections of slab specimens of the current study, which were subjected to the fire flame from the lower surface only are slightly lower than the deflection results obtained by Hidayat[37] under the same load , and the ultimate loads of the slab specimens of present study were slightly

higher than of that of slab specimens investigated by Hidayat[37] . This behavior can be attributed to the fact that the deterioration of the mechanical properties of the slab specimens subjected to fire flame from the lower surface only is lower than that of the slab specimens which were heated in electrical furnace homogeneously.
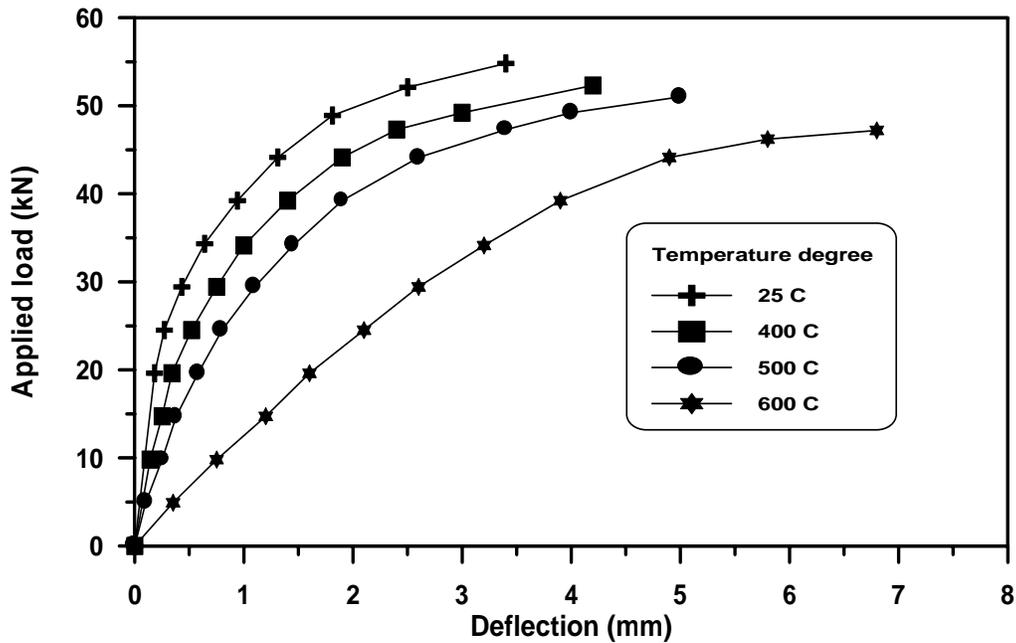
Figure (4- 26): Load – deflection relationship for the reinforced concrete slab specimens at different burning temperatures of series A (ρ = 0.00492).
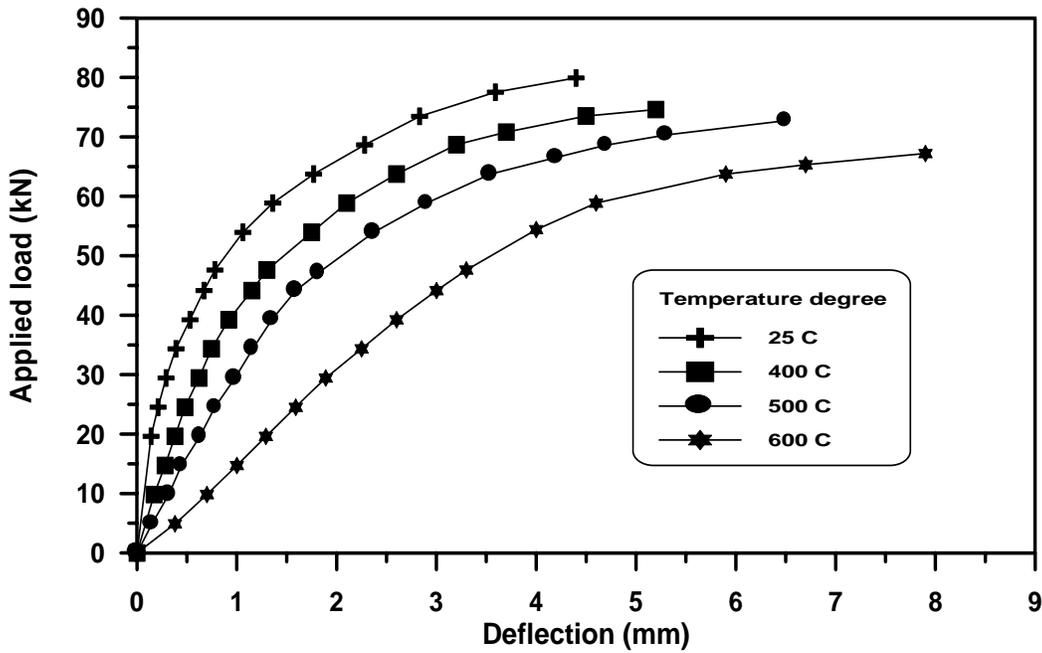
Figure (4- 27): Load – deflection relationship for the reinforced concrete slab specimens at different burning temperatures of series A (ρ = 0.00875).
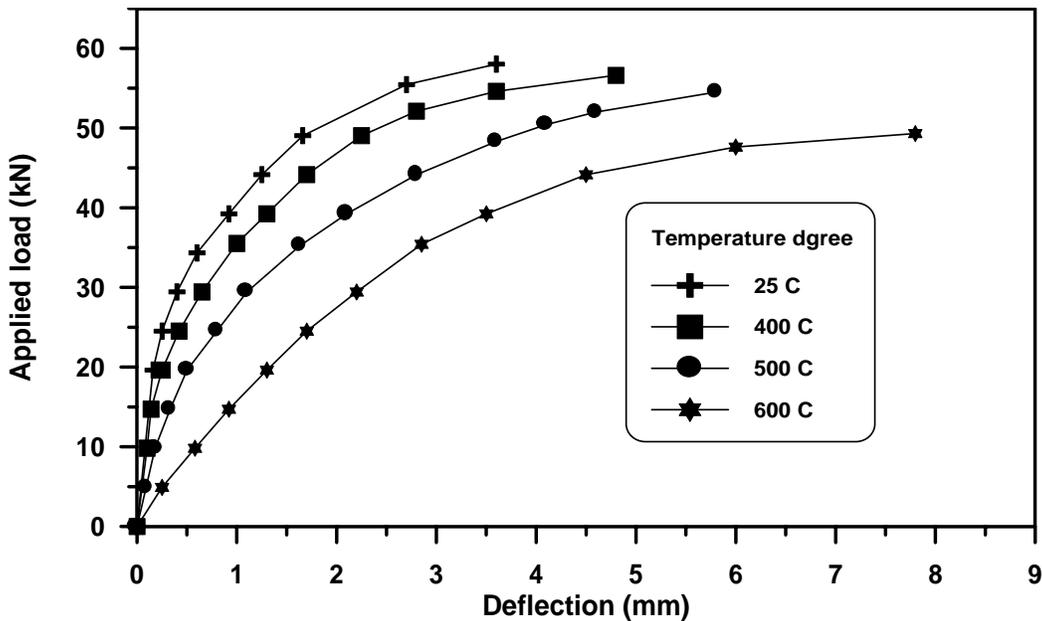
Figure (4- 28) : Load – deflection relationship for the reinforced concrete slab specimens at different burning temperatures of series B (ρ = 0.00492).
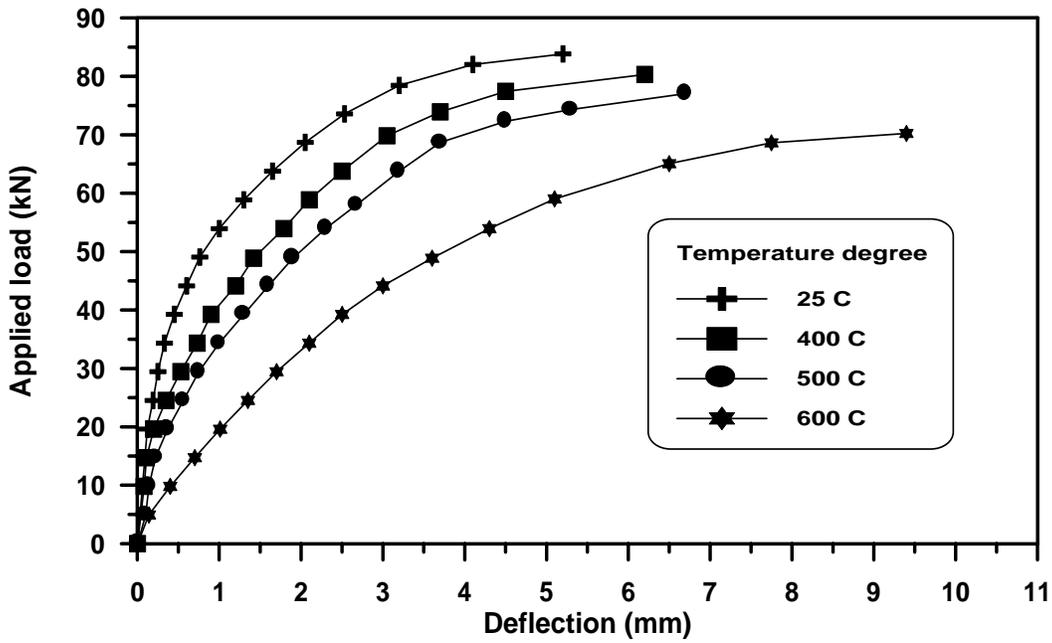
Figure (4- 29) : Load – deflection relationship for the reinforced concrete slab specimens at different burning temperatures of series B (ρ = 0.00875).
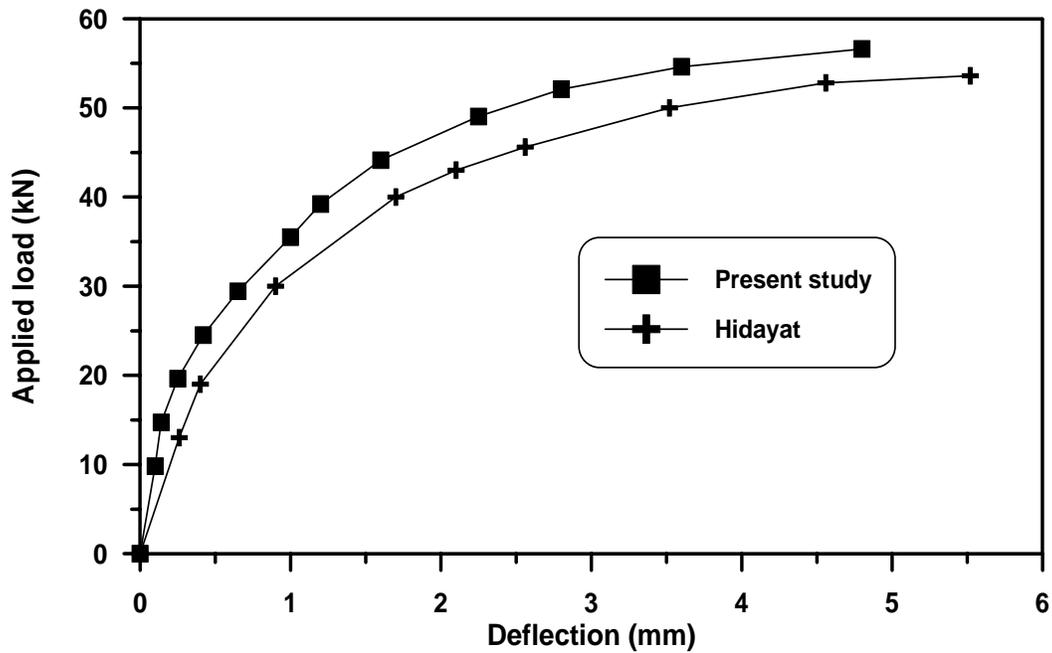
Figure (4-30): Load – deflection curves of present study and Hidayat[37] study, at temperature of 400 °C , (series B ,ρ =0.00492) .
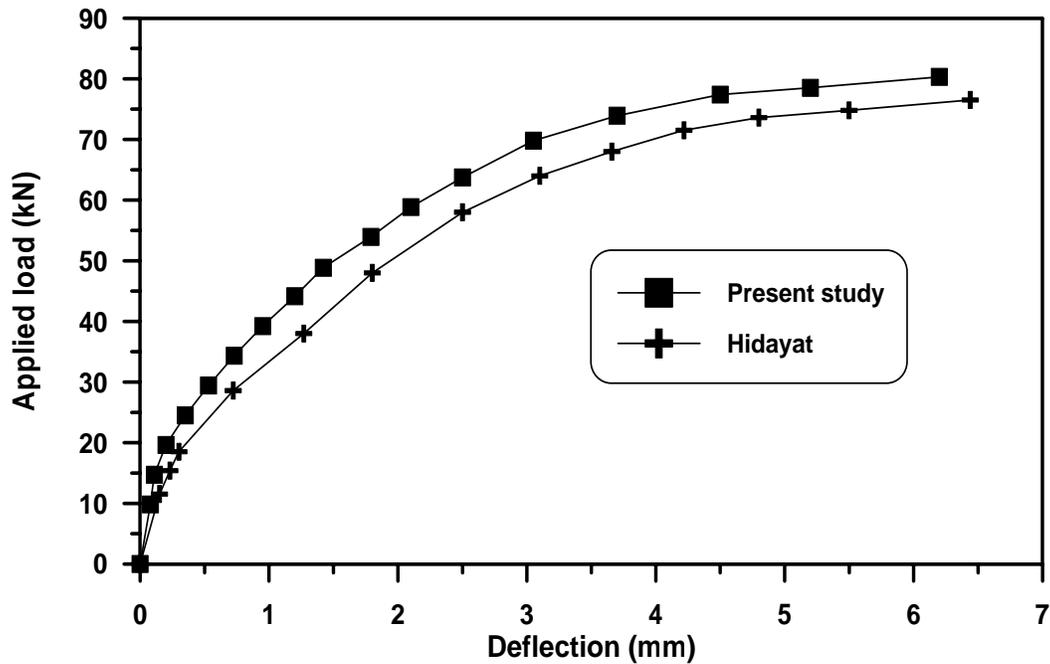
Figure (4-31): Load – deflection curves of present study and Hidayat[37]
study , at temperature of 400 °C , (series B ,ρ =0.00875) .
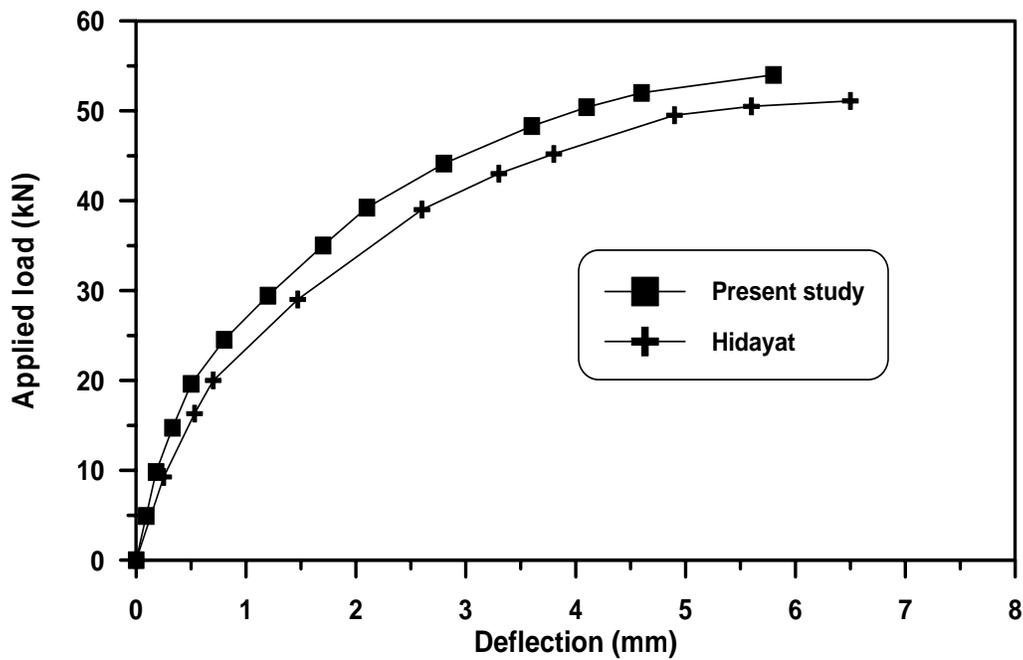
Figure (4-32): Load – deflection curves of present study and Hidayat[37]
study, at temperature of 500 °C , (series B , ρ =0.00492) .

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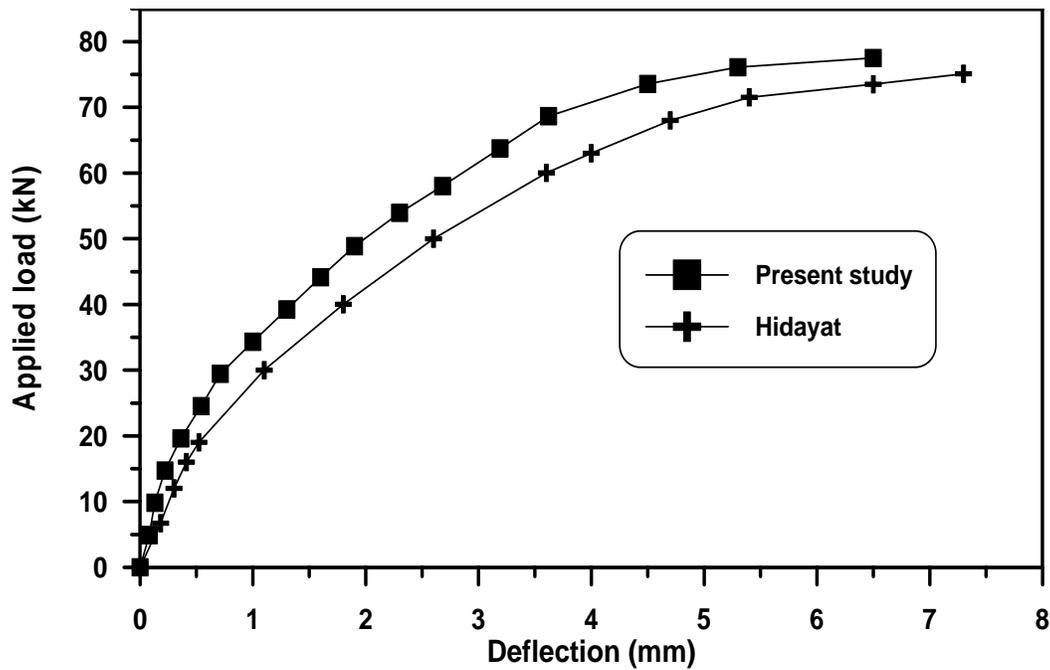
Figure (4-33): Load – deflection curves of present study and Hidayat[37] **study, at temperature of 500 °C , (series B ,ρ =0.00875) .**
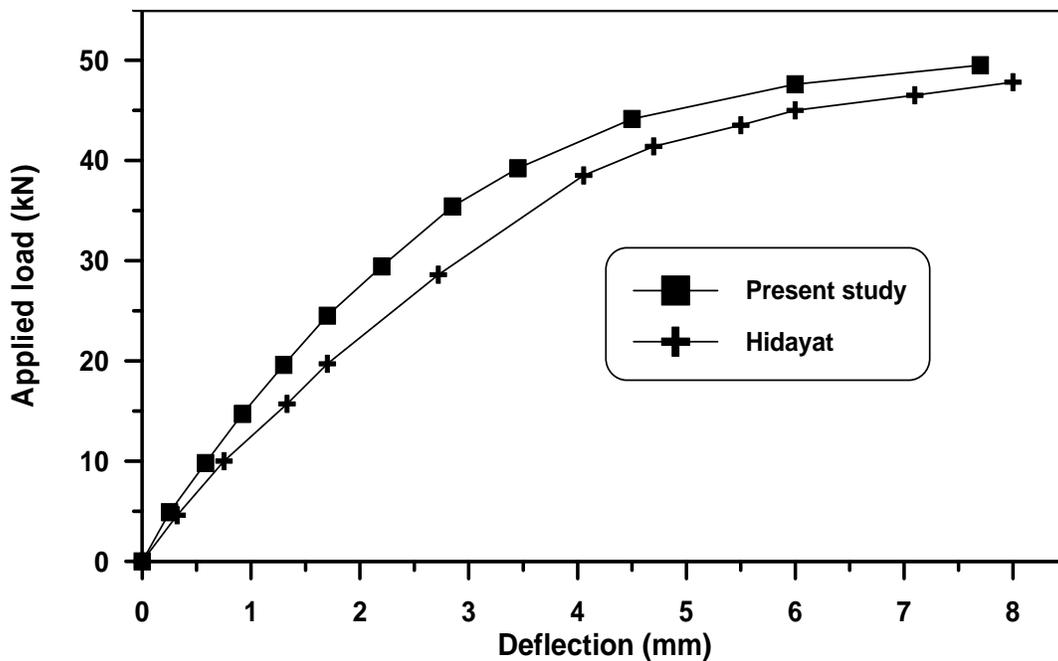
Figure (4-34) : Load – deflection curves of present study and Hidayat[37] **study, at temperature of 600 °C , (series B ,ρ =0.00492) .**
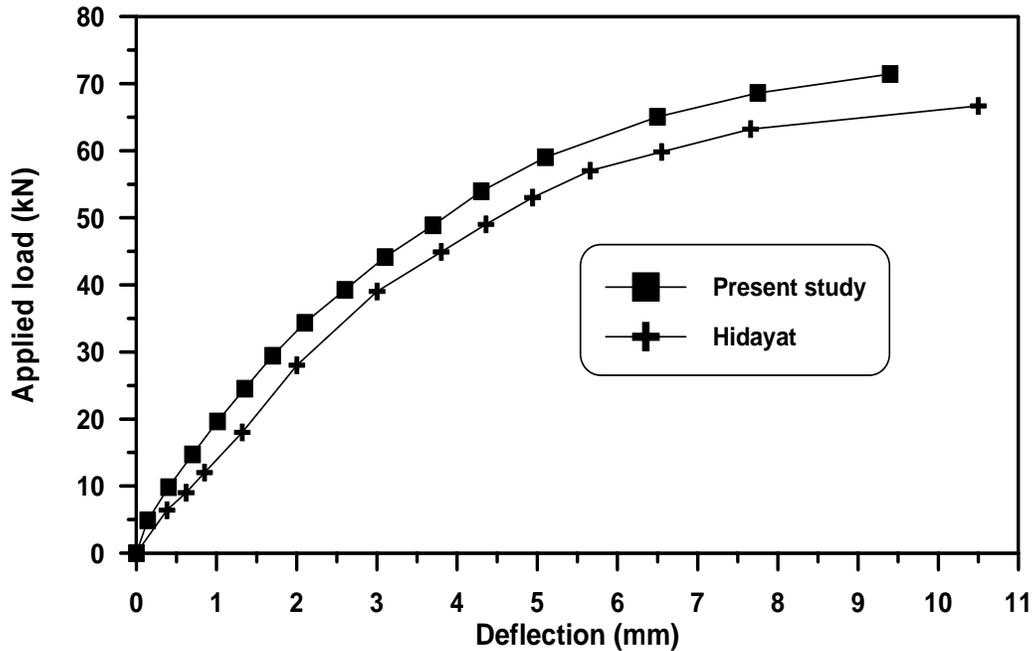
Figure (4- 35): Load – deflection curves of present study and Hidayat [37] study, at temperature of 600 $^{\circ}$C , (series B, ρ =0.00875) .

4-7-3 Cracks Appearance in Reinforced Concrete Slab Specimens

For the reinforced concrete slab specimens, the first crack appeared in the central region and then extended diagonally toward the corners rapidly. As loading increased , more cracks initiated in this region and extended towards the corners. While formation of new cracks, the earlier cracks became wider.

From Table (4-7), it can be seen that the percentages of the first crack load to the ultimate load at 25°C were (39.2%, 32.4%) for specimens of series A with steel ratio of (0.00492, 0.00875) respectively, and (42.2%, 33.4%) for specimens of series B with the same steel ratio respectively. The ratios of the first crack load to the ultimate load decreased with the increasing fire

temperatures , where at $600^{\circ}C$ they were about (32%, 26%) for series A with steel ratio of (0.00492, 0.00875)respectively and (32%, 28.5%) for series B with the same steel ratios respectively.

Table (4- 7): Test values of first crack load , ultimate load and the ratio between them of reference slab specimens and that exposed to fire flame.

Series	Slab No.	Exposure Temp. $^{\circ}$C.	First Visible Crack Load (kN)	Ultimate Load (kN)	*F.C.L/ *U.L ratio
A	A_{11}	25	21.5	54.8	0.392
	A_{12}	400	19.7	52.2	0.377
	A_{13}	500	17.9	51	0.350
	A_{14}	600	15.1	47.2	0.32
	A_{21}	25	25.9	79.9	0.324
	A_{22}	400	22.4	74.6	0.30
	A_{23}	500	20.5	72.7	0.282
	A_{24}	600	17.5	67.2	0.26
B	B_{11}	25	24.5	58.0	0.422
	B_{12}	400	23.4	56.6	0.413
	B_{13}	500	21.6	54.5	0.4
	B_{14}	600	16.1	49.3	0.32
	B_{21}	25	28.0	83.8	0.334
	B_{22}	400	26.0	80.3	0.323
	B_{23}	500	24.2	77.0	0.314
	B_{24}	600	20.0	70.2	0.285

*F.C.L :First crack load .
*U.L   : Ultimate load .

4-7-4 Deflection at Service Load

The load of reference slab specimens (A_{11}, A_{21}, B_{11}, B_{21}) which causes a maximum allowable deflection of (L/180) was divided by (1.6) to calculate the service load .

Branson [61], suggested that the maximum allowable deflection of plates should not exceed (L/180) which equals to (3.1mm) for the investigated specimens in this work.

The measured mid-span deflection of slab specimens at service load at different temperatures are presented in Table (4-8). A comparssion of the allowable and measured mid-span deflections at service load , it was found that they were within the allowable limit at all investigated temperatures except the slab specimens (A_{24}, B_{24}) gain deflections over the allowable limit.

Table (4-8): Effect of fire flame exposure on the deflection at service

Loads of reinforced concrete slab specimens

Series	Slab No.	Exposure Temp. (°C)	Ultimate Load (kN)	Service Load (kN)	Deflection at Sevice Load (mm)	*S.L.D.F
A	A_{11}	25	54.8	34.1	0.65	1.0
	A_{12}	400	52.2	34.1	1.0	1.54
	A_{13}	500	51.0	34.1	1.6	2.46
	A_{14}	600	47.2	34.1	2.8	4.30
	A_{21}	25	79.9	47.5	0.82	1.0
	A_{22}	400	74.6	47.5	1.3	1.6
	A_{23}	500	72.7	47.5	1.9	2.32
	A_{24}	600	67.2	47.5	3.3	4.0
B	B_{11}	25	58.0	35.5	0.6	1.0
	B_{12}	400	56.6	35.5	1.0	1.60
	B_{13}	500	54.5	35.5	1.63	2.72
	B_{14}	600	49.3	35.5	2.85	4.75
	B_{21}	25	83.8	48.8	0.77	1.0
	B_{22}	400	80.3	48.8	1.42	1.85
	B_{23}	500	77.0	48.8	1.9	2.47
	B_{24}	600	70.2	48.8	3.6	4.67

*S.L.D.F= Service load deflection factor.

$$S.L.D.F = \frac{\text{service load deflection of fired slab}}{\text{service load deflection of reference slab}} .$$

4-8 Deflection Behavior of Reinforced Concrete Slab

Specimens During Exposure to Fire Flame .

The mid – span deflection of the slab specimens which were both loaded and exposed to fire at the same time was measured during this process .Each slab specimen was loaded to two levels, the first level was 40% of the ultimate load and the other level was the calculated service load .

Deflection of these slab specimens, which occurred immediately when they were subjected to fire flame, this deflection is called immediate deflection or instantaneous deflection. Deflection measurement was taken continually during the test and the rate of increase in deflection was controlled to provide warning of impending collapse of the slab specimen.

Figures (4-36) to (4-39) illustrate the measured deflections of slab specimens as a function of fire exposure time. From these Figures, it can be seen that the rate of deflection increases rapidly during the early minutes of fire test ,then remains nearly constant for the later period of exposure to fire flame , then the slab specimens exhibit a sudden increase in deflection which was identified as failure.

This can be attributed to the weaker bond strength between the concrete and steel reinforcement .This de –bonding is resulted by the cracks that formed at tension surface, and they were at location close to the steel reinforcement [33]. As these cracks elongated and widend, the fire penetrated into the inner part of the slab specimens and the steel reinforcement through the "crack opening". This behavior agrees with the conclusions of researchers [33, 34,39] .

From the tests, it can be seen that the slab specimens which were loaded with 40% of ultimate load exhibited a fire resistance time longer than that of

specimens which were loaded the service load . In addition, it can be seen that the slab specimens of series A exhibited a fire resistance time longer than that of series B.

Table(4-9)lists the fire resistance rating based on the sudden increment in deflection of the slab specimens.
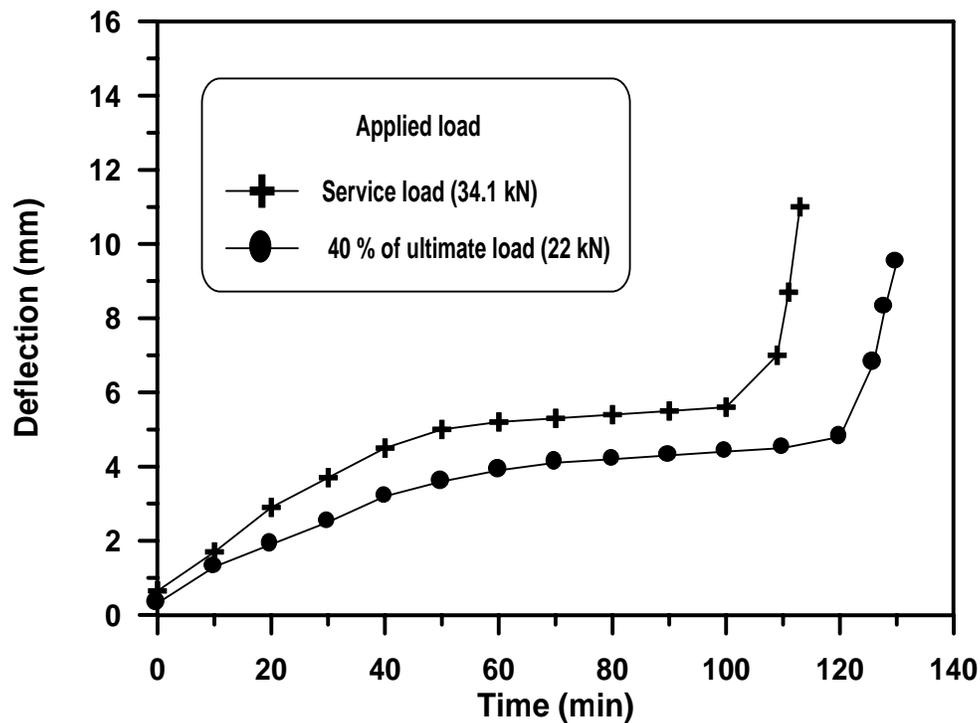
Figure (4- 36): Relation between mid- span deflection with time during exposure to fire flame of reinforced concrete slab specimens A$_{15}$, A$_{16.}$
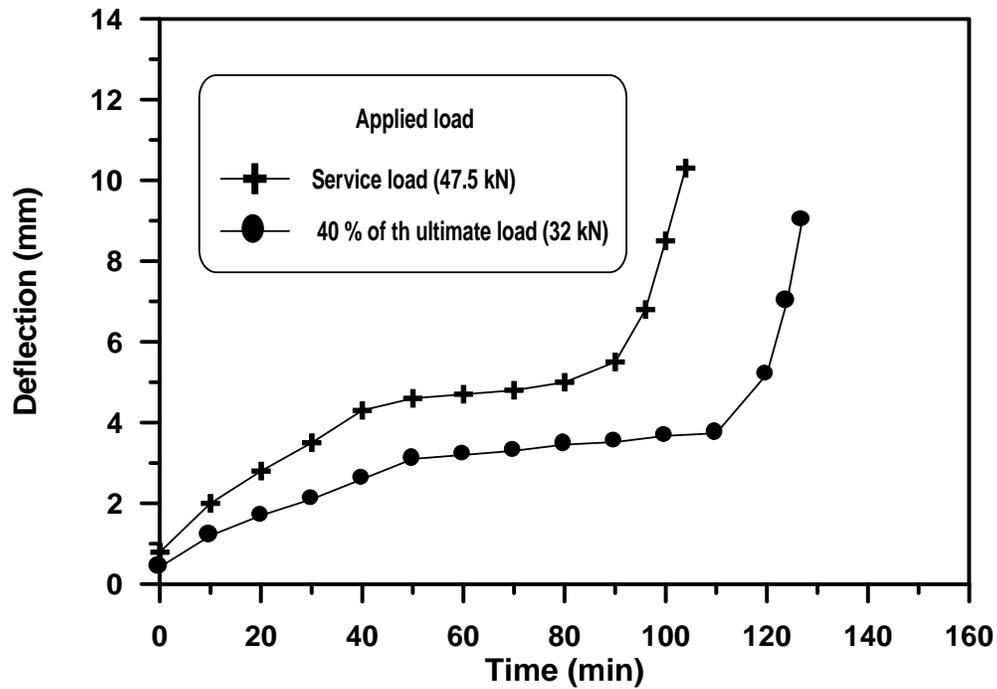
Figure (4- 37): Relation between mid- span deflection with time during exposure to fire flame of reinforced concrete slab specimens A₂₅ , A₂₆.
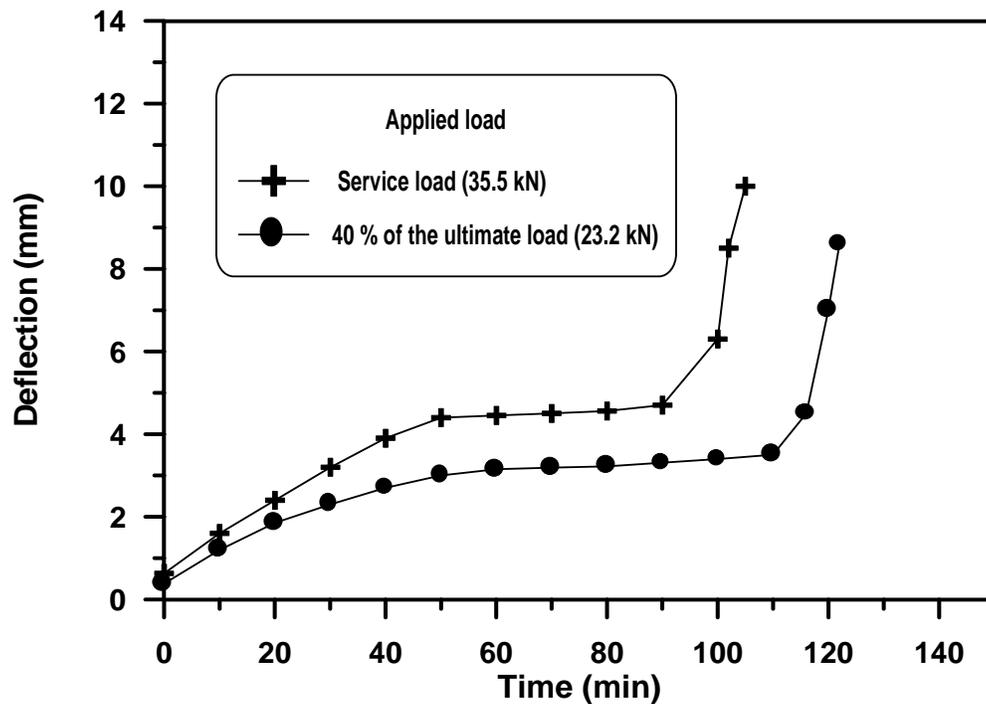
Figure (4- 38): Relation between mid- span deflection with time during exposure to fire flame of reinforced concrete slab specimens B₁₅ , B₁₆.

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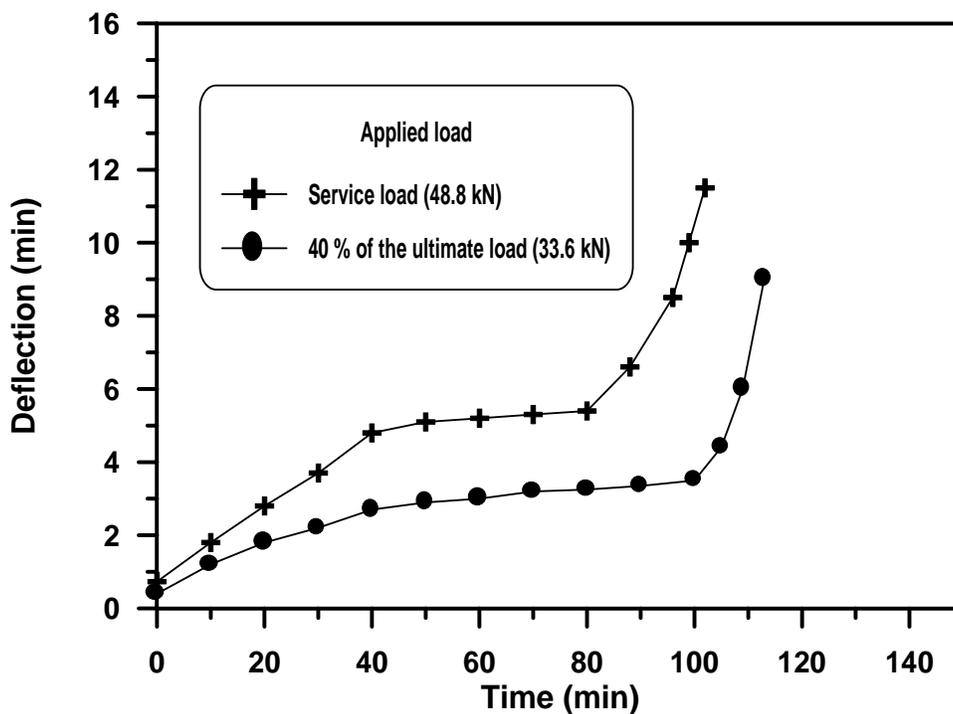
Figure (4- 39): Relation between mid- span deflection with time during exposure to fire flame of reinforced concrete slab specimens B$_{25}$, B$_{26}$.

Table (4-9): Results of loaded burning slab specimens

Series	Slab No.	Exposure Temp. ($^{\circ}$C)	Applied Load (KN)	Failure Time (min)	Deflection at Failure Time (mm)	Mid-Span Deflection at End of Test (mm)
A	A$_{15}$	600	34.1	110	7.0	11.0
	A$_{16}$	600	22.0	126	4.5	10.0
	A$_{25}$	600	47.5	96	7.0	10.5
	A$_{26}$	600	32	117	5.2	9.0
B	B$_{15}$	600	35.5	100	6.3	10.2
	B$_{16}$	600	23.2	117	4.5	9
	B$_{25}$	600	48.8	90	6.6	11.6
	B$_{26}$	600	33.6	105	4.5	9.0

4-9 Theoretical Ultimate Load Capacity of Reinforced Concrete Slab Specimens

The equivalent rectangular compressive stress block of concrete assumption and the yield line theory were used to calculate theoretical the moment capacity of reinforced concrete slab specimens after exposure to high temperatures as follows [62]:

$$M_{nt} = A\cdot_s f_y \cdot R_{yt} \cdot (d - \frac{a_t}{2})\cdots\cdots(4-1)$$

Where: -

a_t:- is the depth of equivalent rectangular compressive stress block at ultimate load (mm) .

$$a_t = \frac{A_s \cdot f_y \cdot R_{yt}}{0.85 f_{c'} \cdot R_{fc'} \cdot b}\cdots\cdots(4-2)$$

Mnt : the nominal moment capacity of the slab specimen exposed to

(T°c) temperature (N.mm) .

As: is the area of the reinforcing steel (mm²).

fy: is the yield tensile stress of the steel reinforcement (MPa) .

d: the effective depth of the slab specimen (it is the distance from the centroid of the reinforcing steel to the extreme compressive fiber), (mm.)

fc´: is the compressive strength of concrete , (MPa) .

b   :   is the width of the slab , (mm) .

R_{yt}: is the ratio of the yield tensile stress of steel reinforcement at temperature (T°C) to the yield tensile stress of steel reinforcement at 25°C.

$R_{fc'}$:- is the ratio of the compressive strength of concrete at temperature (T°C) to the compressive strength of concrete at 25°C.

The ultimate load, predicted from the yield line theory as follows: -

$$M_u = \frac{q_u \, L^2}{24} \cdots\cdots (4-3)$$

Where

qu: The theoretical ultimate load , (N/mm^2)

This method was adopted by other researchers [37, 52, 53].

4-10 Theoretical Load – Deflection Relationship.

The principal factors that affect the deflection of reinforced concrete slabs are the magnitude and distribution of the load, span length and conditions of the end restraint, section properties including steel ratio and materail properties [63].

To predict the deflection resulting from the applied uniform load, the differential equation is derived from plate theory as follows [64]:

$$\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{q(x, y)}{D} \cdots\cdots (4-4)$$

By using finite differences method and taking into account the simply supported condition of the slab specimen, the vertical deflection at mid – span of slab can be expressed as follows :

$$\Delta = \frac{0.00406 \, q \, L^4}{D} \cdots\cdots (4-5)$$

Where

$$D = \frac{E_c \, R_{Ec} h^3}{12 \, (1 - \upsilon^2)} \cdots\cdots (4\text{-}6)$$

q: Applied uniform load , (N/mm²).

L: length of the span, mm.

D: slab stiffness.

Ec: Modulus of elasticity of concrete at temperature of 25°C, (MPa).

R_{Ec}: Ratio of the modulus of elasticity of concrete at temperature (T°C) to

the modulus of elasticity of the concrete at 25°C .

υ : Poisson's ratio .

Substituting the value of (D) and neglecting the poisson's ratio due to its so small effect in the equation (4 – 6) , the equation (4 – 5) can be given as:

$$\Delta = \frac{0.00406 \ q \ L^5}{E_c \ I \ R_{Ec}} \cdots\cdots\cdots(4-7)$$

Where

I: The moment of inertia of the slab section, (mm4).

q: Applied uniform load , (N/mm²) .

The solution of differential equation (4 – 4) and using finite differential equation method are given in Appendix B.

The present study adopted the Resheidat method [65] and Beeby method [63, 66]

4-10-1 Resheidat Method [65].

The load – deflection relationship consists of two bilinear curves, the first one is until crack initiation, while the second is till reaching the service load .The first line is predicted using equation (4 – 7), but by using the gross moment of inertia of the slab (Iv) and the cracked load (qcr) which is obtained from the cracked moment of the slab.

The cracked moment of the slab is calculated as follows: -

$$M_{crt} = \frac{f_r \ L \ h^2 \ R_{fr}}{6} \cdots\cdots\cdots(4-8)$$

Where:

Mcrt: The cracked moment of the slab, N.mm.

f_r: Modulus of rupture of concrete at 25°C, MPa .

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R_{fr}: The ratio of the modulus of rupture of concrete at (T°C) temperature

to the modulus of rupture at 25°C.

h: Thickness of the slab ,mm .

L: Span length, mm.

$$I_v = C_1 \cdot I_g \ldots\ldots\ldots(4-9)$$

$$Ig = \frac{b.h^3}{12}$$

$$C_1 = 1 + 12\eta^2 + 12\rho n \frac{d}{h}(\frac{d}{h} - \frac{1}{2} - \eta)^2 \cdots\cdots(4-10)$$

Where :-

n: Modular ratio (the ratio of the modulus of elasticity of steel to the

modulus of elasticity of concrete at (T $^\circ$C))

ρ: steel ratio.

$$\eta = \frac{n\rho\frac{d}{h}(\frac{2d}{h}-1)}{2(1+n\rho\frac{d}{h})}\cdots\cdots(4-11)$$

The second line of the relationship can be obtained as follows :-

$$\Delta_s = \Delta_{cr} + \Delta_2\left[1 - \frac{M_{cr}}{M_s}\right]\cdots\cdots(4-12)$$

Where :-

M_s:-The service moment of the slab, N.mm

By using equation (4-7) and the moment of inertia of cracked section (Icr) and the service load, which is calculated from the service moment by using equation (4-3), the value of Δ_2 can be obtained.

The moment of inertia of the cracked section can be calculated as follows:-

$$I_{cr} = C_2 \cdot I_g \cdots\cdots(4-13)$$

Where

$$C_2 = (\frac{d}{h})^3 \left[4 K^3 + 12 \, n \, \rho \, (1 - K)^2 \right] \cdots\cdots\cdots (4-14)$$

$$K = \left[(n \, \rho)^2 + 2 \, n \, \rho \right]^{\frac{1}{2}} - n \, \rho \cdots\cdots\cdots (4-15)$$

An example of the calculation of theoretical load- deflection relationship is given in Appendix C-1

4-10-2 Beeby Method [63, 66].

The load – deflection relationship is consists of two bilinear curves also, the first line extends until the crack stage initiation, while the other extends till the failure stage. The first line is represented by equation (4 – 7), where the gross moment of inertia is used and the cracked load which is obtained from the cracked moment.

The cracked moment is calculated from equation (4 – 8). Thus, equation (4 – 7) would be written as follows :

$$\Delta_{cr} = \frac{0.00406 \, q_{cr} \, L^5}{E_c \, I_g \, R_{Ec}} \cdots\cdots\cdots (4-16)$$

Where: -

q_{cr} :cracked load , (N/mm²)

According to this method, for the second line, the modulus of elasticity is reduced by about 43% beyond the cracking stage, where: -

$$E_c' = 0.57 \, E_c \cdots\cdots\cdots (4-17)$$

The moment of inertia of the cracked section, which took into account the effect of high temperature due to fire, is calculated as follows:

$$I_{cr}' = [\frac{b \, cn^3}{3} + n \, \rho \, d \, b \, (d - cn)^2] \cdots\cdots\cdots (4-18)$$

Where:

Cn = the depth of the neutral axis from the compressive fiber of the slab, mm.

The load (q_1) is obtained from (M_1) where:

$M_1 = Ma - Mcr \ldots$ (4-19)

Ma: the ultimate applied moment, N.mm

Equation (4-7) can be written as follows:

$$\Delta_2 = \frac{0.00406 \; q_1 \; L^5}{E'_c \; I'_{cr} \; R_{Ec}} \; \cdots\cdots\cdots(4-20)$$

Then, the ultimate deflection can be obtained as follows:-

$$\Delta u = \Delta cr + \Delta_2 \ldots\ldots(4-21)$$

An example of the calculation of the theoretical load – deflection relationship according this method is given in Appedix C-2.

4-11 structural Analysis .

4-11-1 Ultimate Load .

Table (4-10) gives the experimental and theoretical results of ultimate load of the slab specimens. The values of theoretical ultimate load of slab specimens were calculated according to the method which was described in section (4-9).A comparison between the experimental and theoretical values of ultimate load, it can be seen that the theoretical values were lower than experimental values due to the considerable residual capacity of the section beyond reaching concrete and steel to yielding. The percentage of maximum value of the difference between the experimental and theoretical data was (21.4 %) for the slab specimen B_{11}. The percentage of difference decreased with increasing the temperature up to 500°C and then ,relatively increased at 600°C.The ratio of the experimental to theoretical ultimate load of slab specimens ranged from (1.03) to (1.27) .

**Table (4 -10) : The experimental and theoretical results of the
ultimate load of reinforced concrete slab specimens.**

Series	Slab No.	Exposure Temp. ($^{\circ}$c)	Ultimate Load (kN)		The % of Diffrence between (1) and (2)	Ratio (1) / (2)
			Experimental (1)	Theoretical (2)		
A	A_{11}	25	54.8	45.3	17.3	1.2
	A_{12}	400	52.2	44.9	14.0	1.17
	A_{13}	500	51.0	44.7	12.3	1.14
	A_{14}	600	47.2	40.0	15.3	1.18
	A_{21}	25	79.9	71.9	10.0	1.11
	A_{22}	400	74.6	71.2	4.5	1.05
	A_{23}	500	72.7	70.4	3.2	1.03
	A_{24}	600	67.2	63.0	6.3	1.07
B	B_{11}	25	58.0	45.6	21.4	1.27
	B_{12}	400	56.6	45.3	20	1.25
	B_{13}	500	54.6	45.0	17.6	1.2
	B_{14}	600	49.3	40.3	18.3	1.22
	B_{21}	25	83.8	72.8	13.1	1.16
	B_{22}	400	80.3	72.0	10.3	1.11
	B_{23}	500	77.0	71.3	7.4	1.08
	B_{24}	600	70.2	63.7	9.3	1.1

4-11- 2 Load –Deflection relationship

Figures (4-40) to (4-55) show the experimental and theoretical results of applied load – deflection relationship for slab specimens. The theoretical results are calculated according to Resheidat method [65] and Beeby method [63, 66].

From these Figures,it can be noticed that there was a good agreement between experimental and theoretical results of load – deflection relationship.

For the slab specimens of series A (A_{11},A_{12} and A_{13}) ,it can seen that the experimental results are in line with the theoretical results , while the experimental results of slab specimen A_{14} is closer to theoretical results using Beeby method[63, 66] than Resheidat method[65] .Similar behavior was obtained for slab specimens (A_{21}, A_{22}, A_{23} and A_{24}).

For slab specimens of series B (B_{11}, B_{12} and B_{13}),the experimental results are closer to Resheidat method[65] than Beeby method[63, 66], while theoretical results using Beeby method[63, 66] are closer to experimental results than Resheidat method[65] for the slab specimen B_{14} .Similar behavior was obtained for slab specimens (B_{21}, B_{22}, B_{23} and B_{24}). .It can be concluded that both Resheidat method[65] and Beeby method[63, 66] are easy to apply and they show a good agreement with experimental results of slab specimens.

Figure (4- 40): Experimental and theoretaical results of applied load and deflection for slab specimen (A$_{11}$),at 25 $^\circ$C .

Figure (4- 41): Experimental and theoretaical results of applied load and deflection for slab specimen (A$_{12}$),at 400 $^\circ$C .

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Figure (4-42): Experimental and theoretaical results of applied load and deflection for slab specimen (A$_{13}$),at 500 $^{\circ}$C .

Figure (4-43): Experimental and theoretical results of applied load and deflection for slab specimen (A$_{14}$),at 600 $^{\circ}$C .

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Figure (4- 44): Experimental and theoretaical results of applied load and deflection for slab specimen (A$_{21}$),at 25 $^{\circ}$C .

Figure (4-45): Experimental and theoretaical results of applied load and deflection for slab specimen (A$_{22}$),at 400 $^{\circ}$C .

Figure (4- 46): Experimental and theoretaical results of applied load and deflection for slab specimen (A₂₃),at 500 °C .

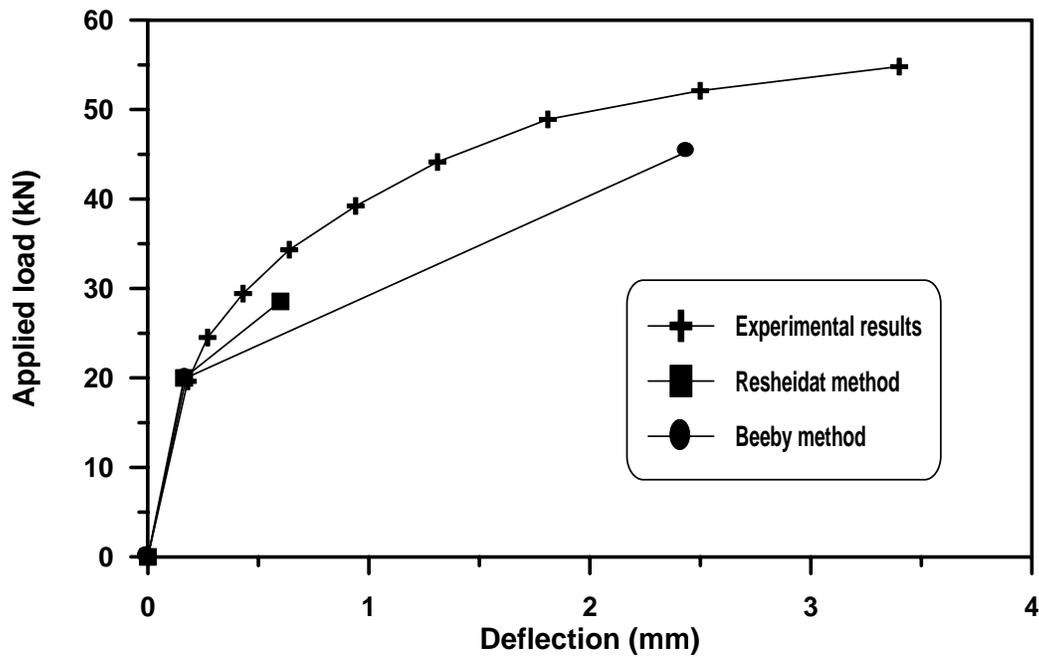**Figure (4- 47): Experimental and theoretaical results of applied load and deflection for slab specimen (A₂₄),at 600 °C .**
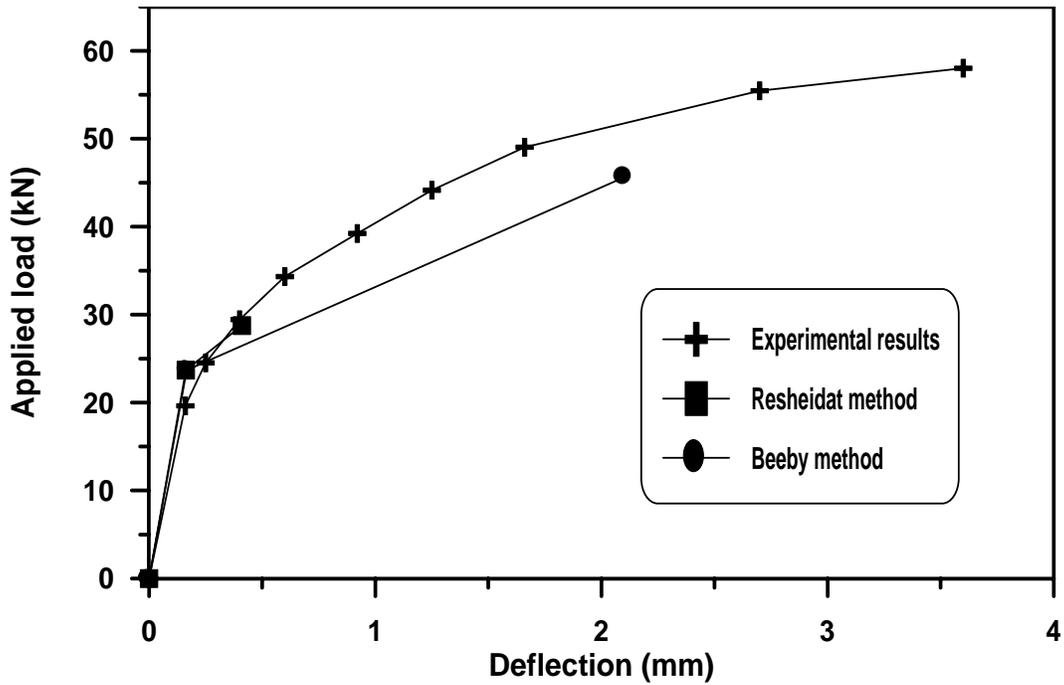
Figure (4-48): Experimental and theoretaical results of appleid load and deflection for slab specimen (B$_{11}$),at 25 $^\circ$C .
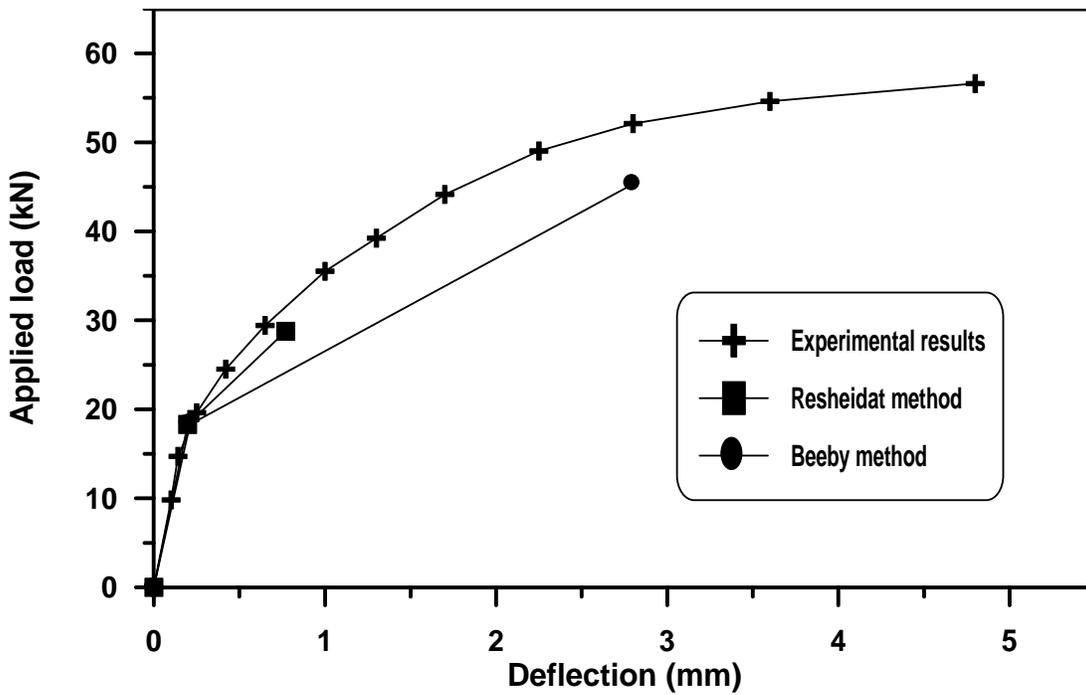
Figure (4-49): Experimental and theoretaical results of appleid load and deflection for slab specimen (B$_{12}$),at 400 $^\circ$C .

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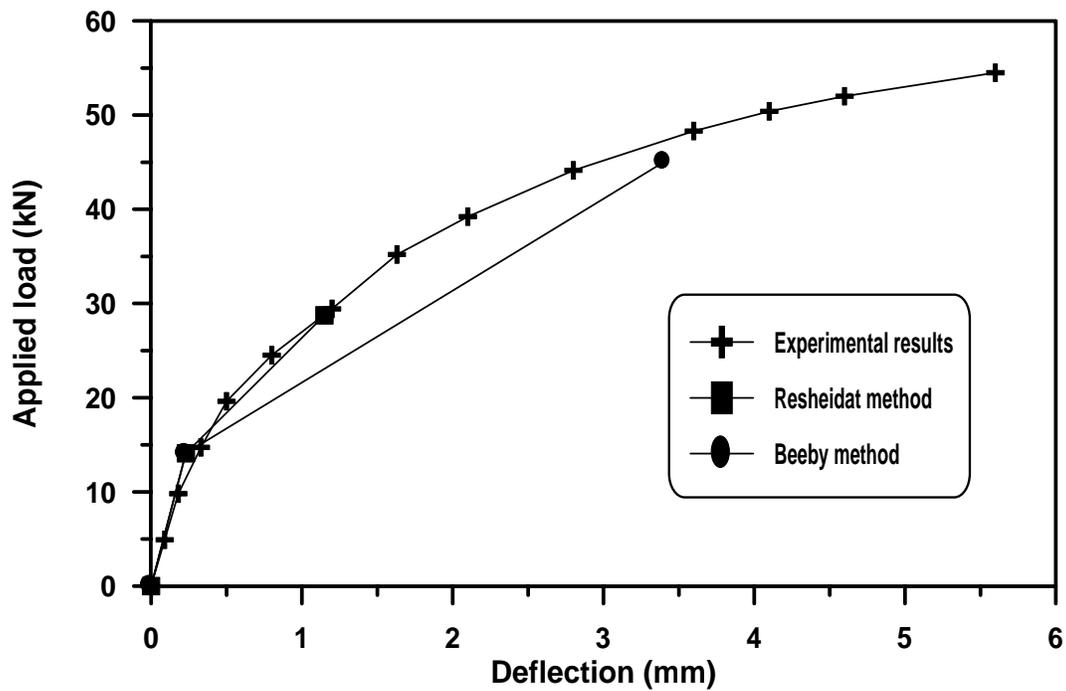
Figure (4-50): Experimental and theoretaical results of appleid load and deflection for slab specimen (B$_{13}$),at 500 $^{\circ}$C .
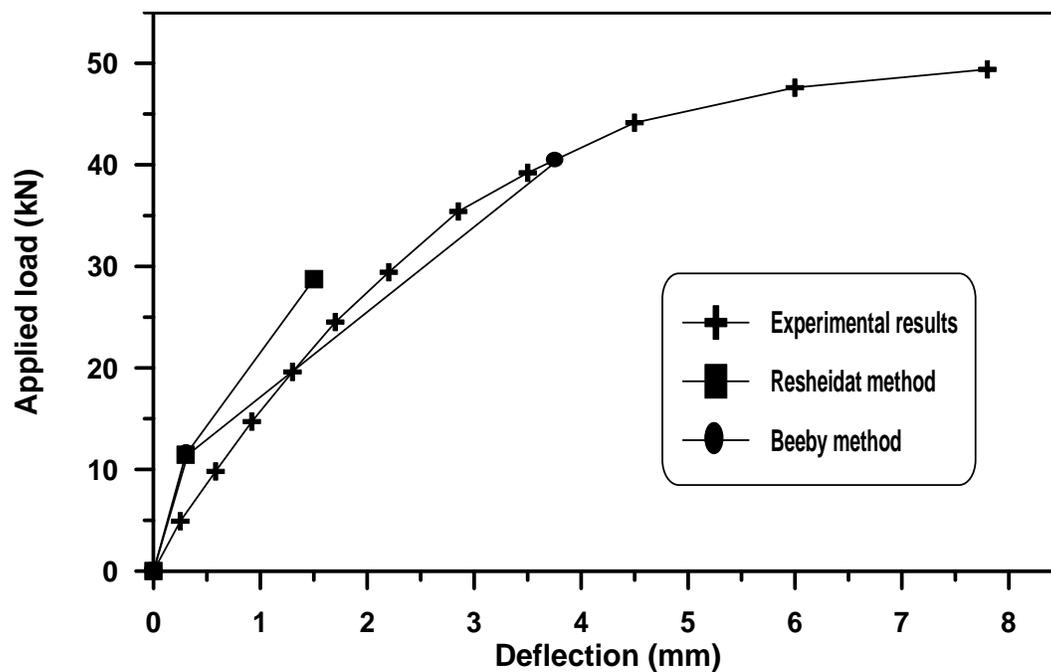
Figure (4-51): Experimental and theoretaical results of applied load and deflection for slab specimen(B$_{14}$),at 600 $^{\circ}$C .
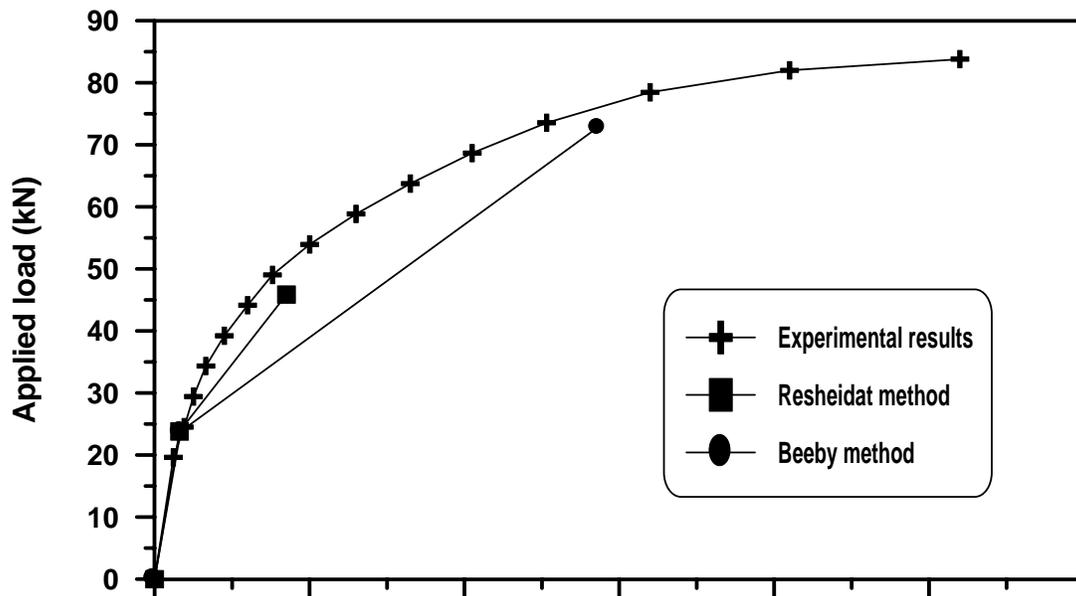
Figure (4-52): Experimental and theoretaical results of applied load and deflection for slab specimen(B$_{21}$),at 25 $^{\circ}$C .
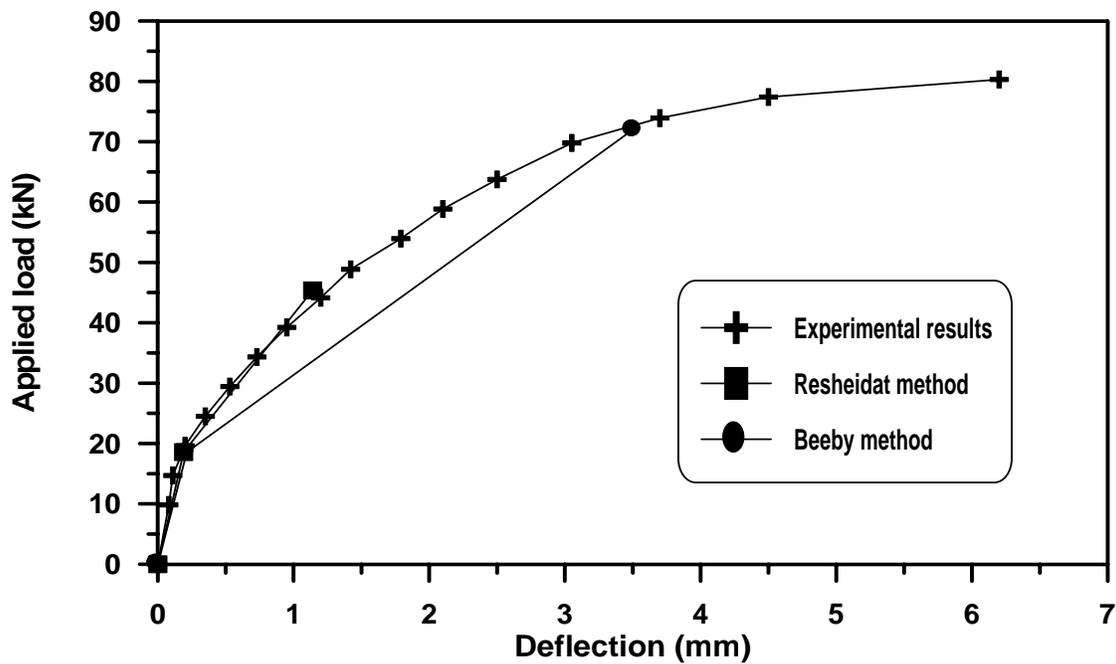
Figure (4-53): Experimental and theoretaical results of applied load and deflection for slab specimen(B$_{22}$),at 400 $^{\circ}$C .
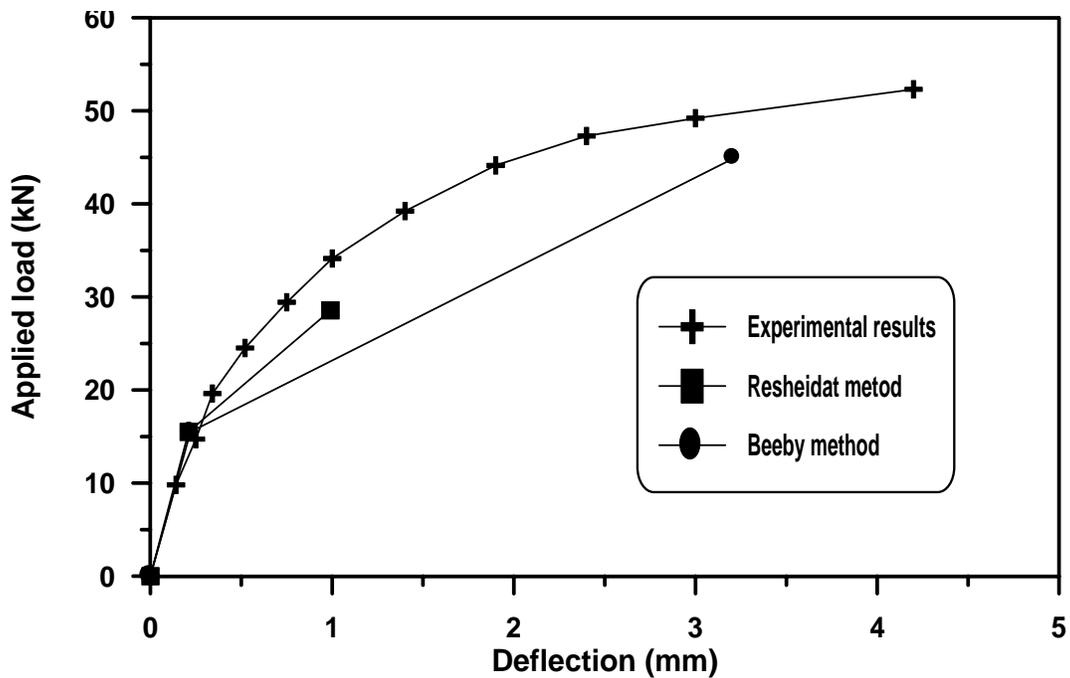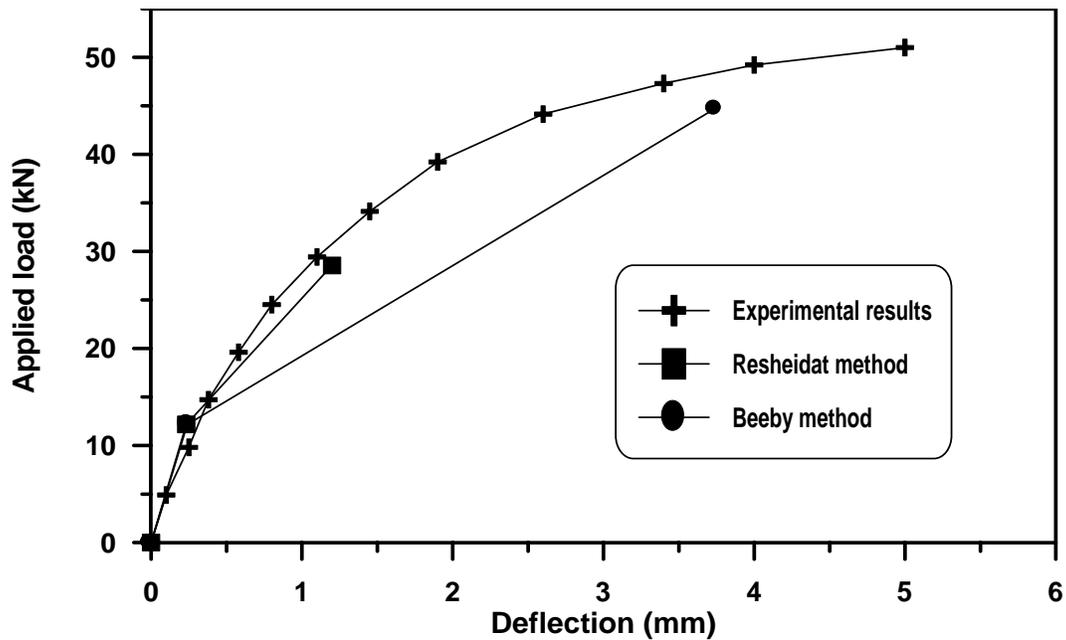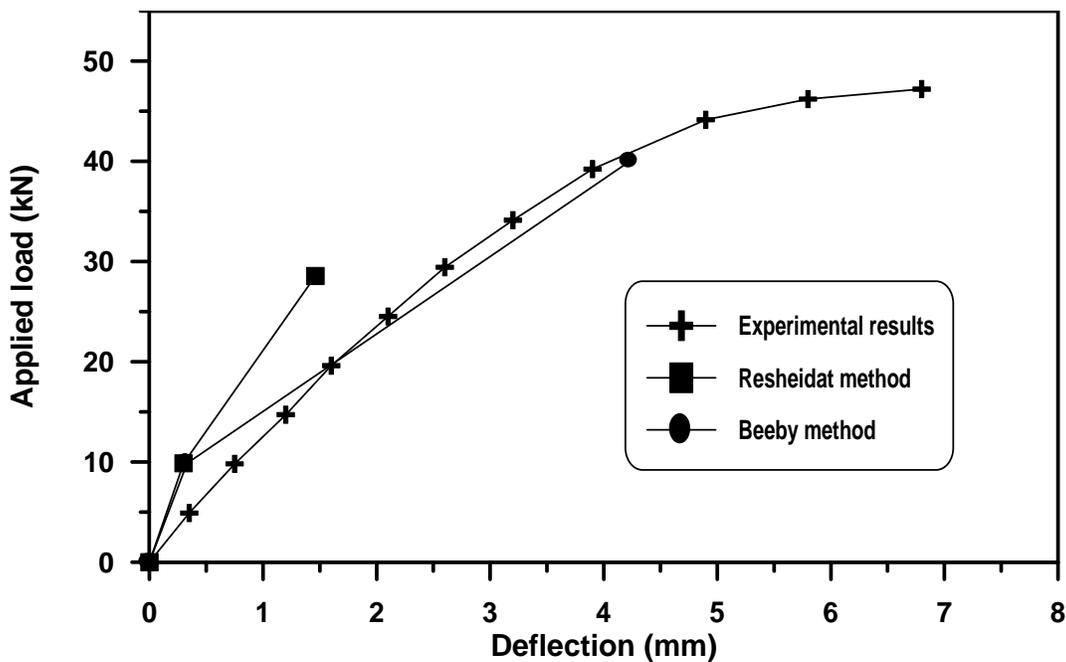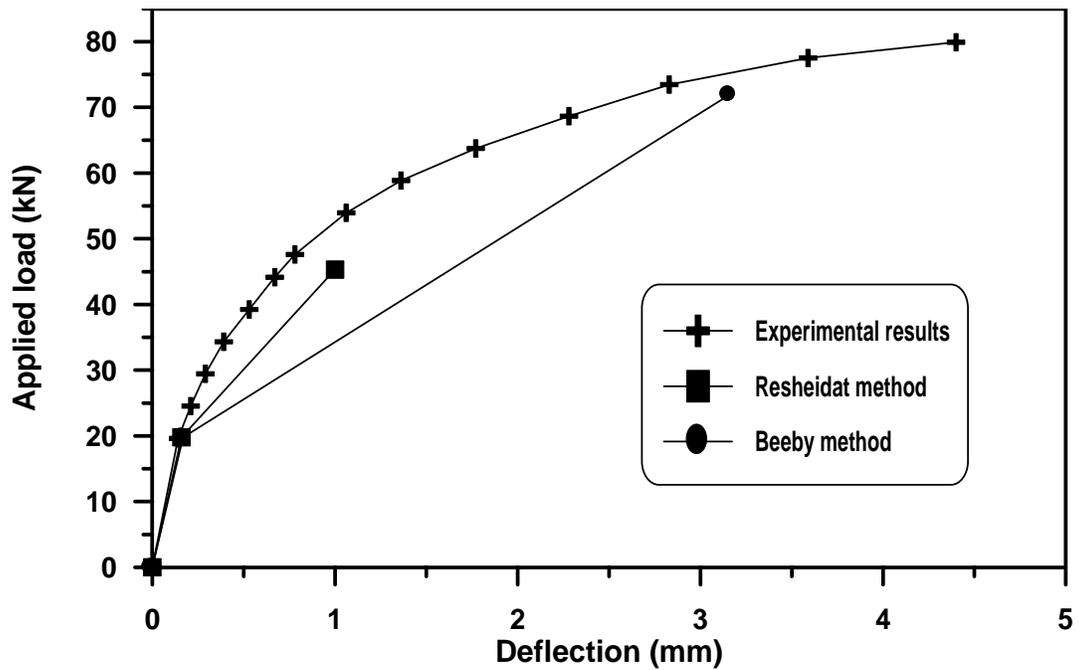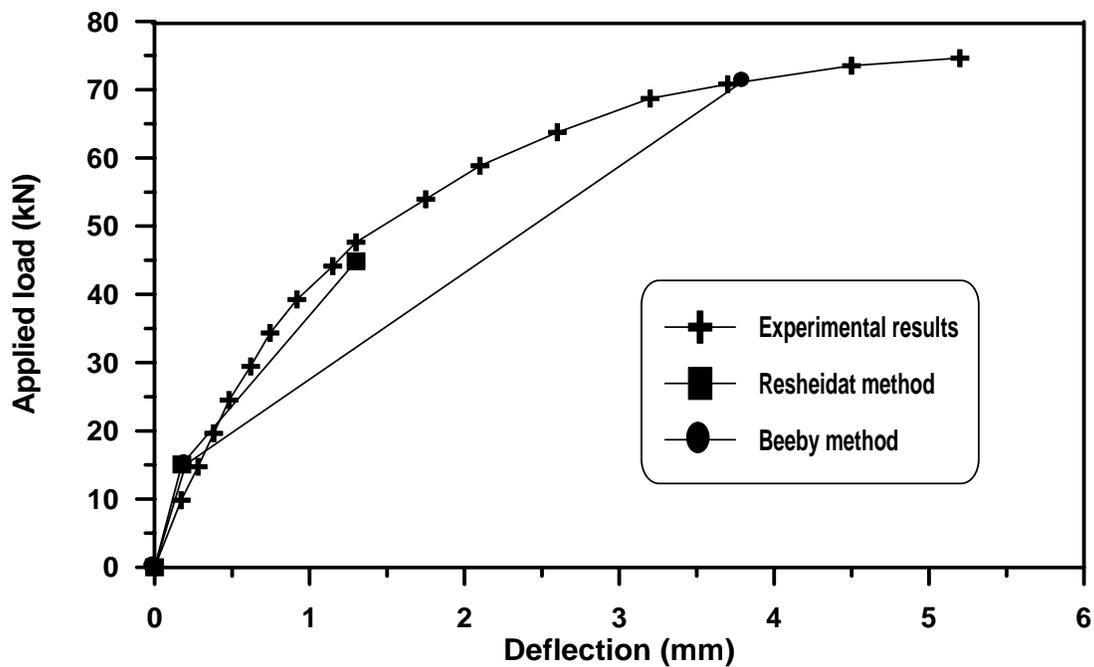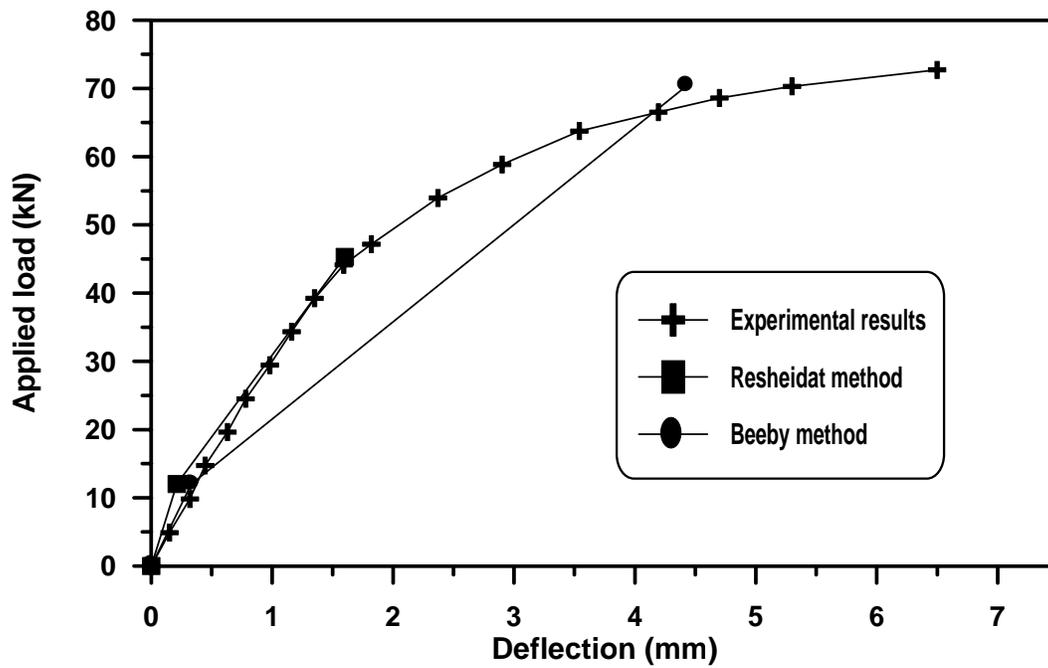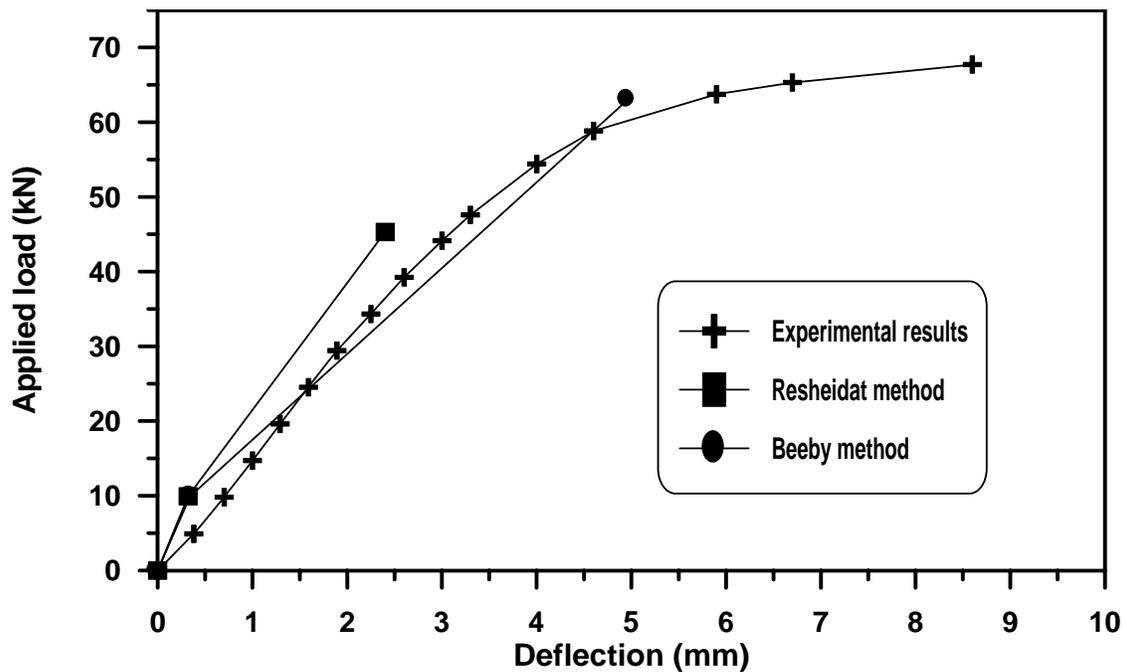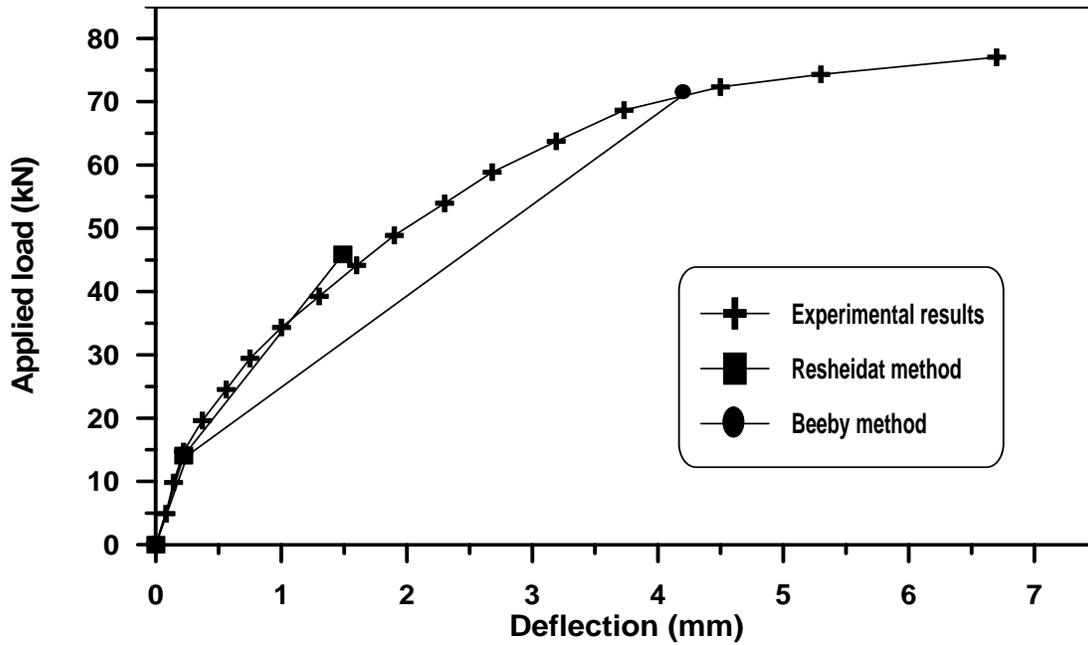
Figure (4-54): Experimental and theoretaical results of applied load and deflection for slab specimen(B_{23}),at 500 °C .
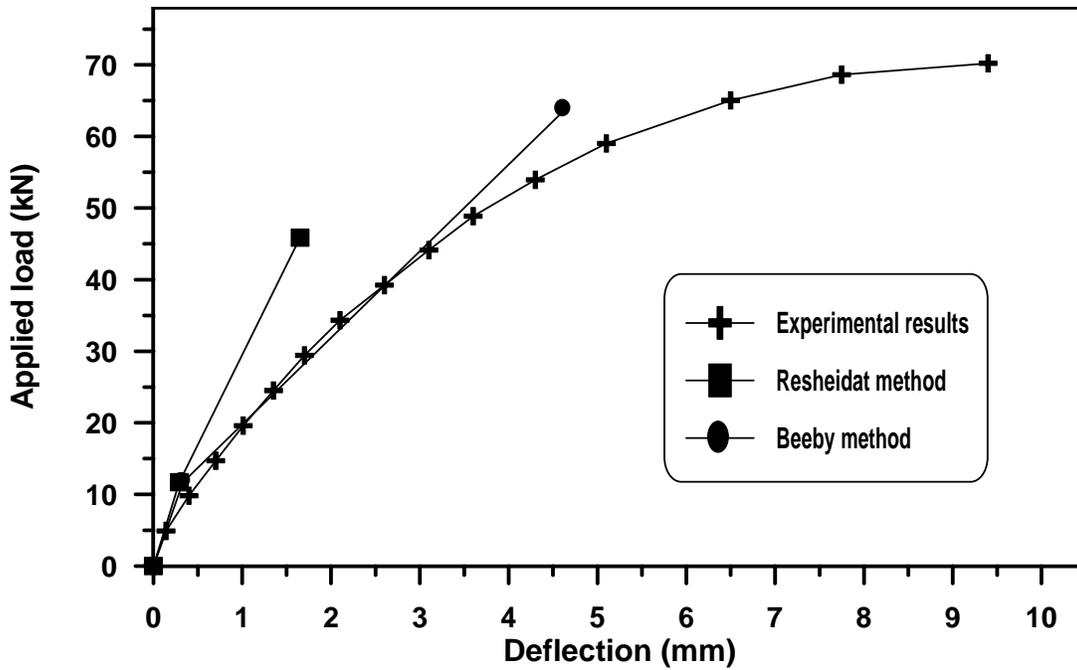
Figure (4-55): Experimental and theoretaical results of applied load and deflection for slab specimen(B_{24}),at 600 °C .

4-12 The Mathematical Regression Model

In order to obtain an accurate mathematical model that can predict the ultimate load of slab specimens after exposing to fire flame and the maximum deflection under a certain load during exposing to fire flame, nonlinear regression is used for this purpose . The analysis of the experimental data included the regression analysis and the choice of the best mathematical model which has the highest correlation coefficient.This has been carried out by the aid of a computer program package called " STATISTIC 5.0 " ,this program includes extensive statistical operations and regression analysis capabilities.

Several attempts had been made and combinations of groups of variables were used as alternative choices in order to obtain the best regression model that can be used with high accuracy for the ultimate load and maximum deflection.

4-12 -1 Choosing of The Mathematical Regression Model

In order to fit a function that minimizes the sum of the squares of residuals between the data and the function,the regression methods had been used ,such methods are termed least – squares regressions.

When the dependent and independent variables are related to each other in a linear fashion, the linear or multiple linear least – squares regression is utilized [67].

For situations where the multiple dependency is non – linear,the power equation is used.

The general form of such equation is :

$$y = a_o + a_1 x_1^{b_1} + a_2 x_2^{b2} + a_3 x_3^{b_3} + \cdots\cdots + a_m x_m^{b_m} \cdots (4\text{-}22)$$

This equation is also called multivariable power equation, and in engineering variables are of ten dependent on several independent varibles; this functional dependency is best characterized by the equation mentioned earlier and is said to give more realistic results too [68,69].

In this study ,the multivarable non-linear equation was found to be very suitable for predicting the ultimate load of the slab specimen after exposing to fire flame and the maximum deflection under a certain load ,as the relationship between the dependent and independent variables is a non-linear one.

Combinations of variables were used in such alternative choices between these combinations, in the power equation to determine which of the variables used (or group of them) could give a significant improvement to the fitness of the equation.

These combinations are :-

1-   sand / gravel ratio,w/c ratio.

2-   Es / E_c

3-   splitting strength.

When analyzing the results using the model that comprises these variables,it was found that there is no significant improvement in the correlation coefficient of the proposed model. Furthermore ,the existing variables in the model (without the new variables) yielded good and reasonable results. Also ,it is not preferable to load the prediction model with large number of variables,because it is preferable to use a model with lesser number of variables with higher possible accuracy to assure the rapid and easy use of the model.

The final forms of the proposed models with the used variables were :-

$$UL = a_o + [(\rho)^{a1} (f'_c)^{a2} (\frac{fy}{340})^{a3} (E)^{a4} (f_r)^{a5} (wt+1)^{a6} (U.P.V)^{a7} (RN)^{a8}](Ln\ T)^{a9} \ldots\ldots(4-23)$$

The above equation is proposed to predict the ultimate load for slab specimen subjected to elevated temperature of (T° C)and cooled to the room temperature before loading.

The following equation is proposed to predict the maximum deflection of the slab specimens subjected to fire flame of (600 ° C) during which they were under loading.

$$D_f = a_o FD + [q^{(a1-a2*FD)} \rho^{(a3-a4*FD)} f_c^{(a5-a6*FD)} f_r^{(a7-a8*FD)}](LnFD)^{a9} \cdots\cdots(4-24)$$

The following equation is proposed to predict the maximum deflection of the slab specimen loaded at room temperature:

$$D_f = a_o + (q)^{a1} (\rho)^{a3} (f_c)^{a5} (f_r)^{a7} \cdots\cdots(4-25)$$

Table (4-11) gives the regression coeficients for the prediction models, the value of correlation cofficients (R) and the range of the difference (df) between the actual and predicted value.

In this Table,several notes have to be out ,to predict the ultimate load:-

1 - U.P.V and R.N were taken togther.

2 – fc , fy , E , fr and wt were used all togther.

To predict the maximum deflection :- ρ , f c , fr and the load (q) were taken as one grop.

From Tables (4-11) and (4-12) also ,it can be seen that the correlation coefficient increases slightly with increasing the number of variables used in the proposed model.

The third model is the combination of the first and second model it takes into account the effect of fc ,fy , E , fr , wt , U.P.V   and R.N   as well as the effect of steel ratio (ρ) and burning temperature .Also, fc , fr , ρ , q and the fire duration (FD) were used in the fourth model to predict the maximum deflection of the slab specimen subjected to fire during which it is under loading.The variables   (fc , fr , ρ and q) were used in the fifth model to predict the maximum deflection of the slab specimen at room temperature.

Table (4-11): Regression coefficients for the ultimate load prediction models .

Variable	Coefficient	Model 1	Model 2	Model 3
	a_0	35.569	38.169	29.182
ρ	a_1	1.445	1.747	1.225
f_c	a_2		3.455	-0.046
fy/340	a_3		0.798	1.399
E	a_4		0.844	0.214
fr	a_5		-4.537	-0.803
wt	a_6		-1.022	-0.236
UPV	a_7	-0.301		-0.403
RN	a_8	3.017		2.784
T	a_9	0.4394	2.479	0.875
R		**0.88**	**0.91**	**0.94**
df		± 4.37 (KN)	± 2.21 (KN)	± 0.75 (KN)

Table (4-12): Regression coefficients for maximum deflection prediction models .

Variable	Coefficient	Model 1	Model 2
	a_0	0.0514	0.2116
q	a_1	0.398	4.29
	a_2	-0.087	
ρ	a_3	-0.813	-2.09
	a_4	0.0301	
F_c	a_5	-0.8146	-13.407
	a_6	0.2544	
Fr	a_7	-1.456	16.432
	a_8	-0.357	
FD	a_9	-0.72	
R		**0.89**	**0.95**
df		**± 0.67 (MM)**	**± 0.29 (MM)**

Table (4-13) gives the difference ranges for the proposed models for different confidence intervals.

The range of the difference (df) between the actual and predicted value was calculated for each model within confidence interval of (0.95) .This means that there is a probability of 95% of the difference between the actual and the predicted value falls within arange of ±df ,thus,

Actual ultimate load = predicted ultimate load± df

and

Actual deflection = predicted deflection ± df

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Table (4-13): Confidence intervals and difference ranges for the proposed models.

Confidence interval %	Difference interval $*10^{-6}$				
	Model 1	**Model 2**	**Model 3**	**Model 4**	**Model 5**
50	-1.52 - +1.52	-0.81 - +0.81	-0.26 - +0.26	-0.23 - +0.23	-0.10 - +0.10
75	-2.56 - +2.56	-1.42 - +1.42	-0.44 - +0.44	-0.39 - +0.39	-0.17 - +0.17
90	-3.67 - +3.67	-1.90 - +1.90	-0.63 - +0.63	-0.56 - +0.56	-0.24 - +0.24
95	-4.37 - +4.37	-2.21 - +2.21	-0.75 - +0.75	-0.67 - +0.67	-0.29 - +0.29

Chapter Five

Conclusions and Further Work

5-1 Conclusions

Based on the results and theoretical analysis of the present study, the following conclusions can be drawn:

[1] - The compressive strength of concrete decreases with fire exposure fire at all ranges of temperature by an amount depending on the exposing temperature .The residual values of compressive strength at temperature around 600 $^\circ$C were (58% and 62%) for series B and A respectively .

[2] – The flexural strength is more sensitive to fire flame temperatures. The residual flexural strengths at temperatures around 600 $^\circ$C were (48%, 50%) for series B and A respectively .

[3] – Modulus of elasticity of concrete is the most affected by fire flame temperature rather than compressive strength and flexural strength. The residual modulus of elasticity at temperature around 600 $^\circ$C were (22% , 26%) for series B and A respectively .

[4] - The splitting tensile strength was more sensitive than the compressive strength. The residual splitting tensile strength at temperature around 600 $^\circ$C were (35%, 39%) for series B and A respectively .

[5]- The effect of the fire flame on the mechanical properties of concrete is relatively equal for both series A and series B.

[6]- At temperatures of (400 $^\circ$C and 500 $^\circ$C) , both burning and subsequent cooling did not affect the mechanical properties of steel reinforcement; the effect was observed at 600 $^\circ$C . The residual yield tensile stress was (89.7%) and residual ultimate stress (91.8%) .

[7]- The weight loss of reinforced concrete slab specimens increases with increasing fire temperature. The reduction in weight loss ranged between (3%-4.8%) at 400 $^\circ$C ,(4.7%-6%) at 500 $^\circ$C and (5.4%-6.5%)at 600 $^\circ$C for both series B and A respectively .

[8]- Ultrasonic pulse velocity test showed a response to the fire flame. The reductions in (U.P.V) were (23- 25 %) ,(42- 43%) and (47 – 48%) for series A ,while the reductions were (24– 26%),(41 -44%) and (49-51%)for series B at temperature of (400 , 500 and 600 $^\circ$C)respectively .

[9] - The reductions in rebound number were (12-13%) , (17 -20%) and (24 -26%) for series A ,while the reduction were (12-14%) ,(19-21%) and (27 - 29%) for series B at temperature of (400, 500, 600 $^\circ$C) respectively .

[10] - Fire endurance of the slab specimens which were reinforced with a lower steel ratio is slightly higher than that of the slab specimens which were reinforced with a higher steel ratio .

[11] -The percentages of the residual values of ultimate load at temperature around of 600 $^\circ$C were (84.2% , 86.0%) for series A for steel ratio(0.00875 , 0.00492) respectively, and (83.7, 85.0%) for series B with the same steel ratios.

[12] - It was noticed that the load- deflection relations of slab specimens exposed to fire flame temperatures around 600 $^\circ$C are flat , representing softer load-deflection behavior than that of the control slab specimens. This behavior can be attributed to the continual decrease in specimens stiffness with the increase fire flame temperatures .

[13] –Measured deflections of the fired slab specimens at service   loads were   higher than that of   the control slab   specimens .Thus , these slab specimens were found to be capable of resisting the service loads but with less factor of safety .

[14] – The predicted theoretical values of ultimate   load capacity using equivalent rectangular compressive stress block of concrete and yield line theory were underestimated the experimental ultimate strength of slab specimen. The ratio of the experimental   to theoretical ultimate load capacity ranged from (1.03) to (1.27) .

[15] – Theoretical results of the applied load – deflection relationship for slab   specimens , which were predicted according   to Resheidat method and   Beeby   method , showed a good   agreement   with   the present experimental results of load- deflection relationship.

 [16] – Fire resistance ratings   (load bearing duration) of slab specimens which were fired and loaded with service loads were (110 , 96 , 100 and 90 minutes)for specimens (A_{15} ,A_{25} ,B_{15} ,B_{25})respectively, and that loaded with 40% of   the ultimate load were (126,117,117 and 105 minutes) for specimens (A_{16}, A_{26} ,B_{16} ,B_{26})respectively .

 [17] –It is clearly that the loading intensity affects the fire resistance rating , i.e. as increasing the loading intensity ,   the fire resistance duration is decreased .

[18] - Mathematical models were developed in this study to obtain reliable prediction of the ultimate load of the slab specimens after exposure to fire, and maximum deflection under a certain load during exposure to fire. The maximum correlation coefficient was (R = 0.95) for the third model .

5-2 Recommendations for Future Work

There are further problems still need to be studied , some of which include :-

[1] – Further work is required to investigate the effect of using lightweight aggregate in reinforced concrete members exposed to fire flame .

 [2] – Studying the effect of support conditions on the behavior of reinforced concrete slabs or beams after exposure to fire flame .

[3] – Studying the effect of thickness of the structural members (such as beams ,slabs ,…etc) on the fire resistance of these members .

[4] – Studying the effect of covering the reinforced concrete members by a protection materials on the fire resistance of these members .

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The differential equation of plate [64] ,

$$\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4} = \frac{q(x,y)}{D}$$

where

$$D = \frac{Eh^3}{12(1-\upsilon^2)}$$

This equation can be solved as follows :

The load can expressed as

$$q(x,y) = \sum_{m=1}^{\infty}\sum_{n=1}^{\infty} q_{mn} \sin\frac{m\pi x}{a} \sin\frac{n\pi y}{a} \ .$$

Boundary Conditions." Simple Support "

1- For x = 0,a   w= 0   , Mx =o

as it is known

$$M_x = -D\left(\frac{\partial^2 w}{\partial x^2} + \upsilon \ \frac{\partial^2 w}{\partial y^2}\right) = 0$$

for any value of (y)

$$w = 0 \ , \frac{\partial w}{\partial y} = 0 \ , \frac{\partial^2 w}{\partial y^2} = 0$$

$$\therefore \ \frac{\partial^2 w}{\partial x^2} = 0$$

2- For y = o ,a

as above   ,

$$w = 0 , \ \ \frac{\partial^2 w}{\partial y^2} = 0$$

An expression for w (x,y) satisfies boundary conditions can be written as follows :

$$w(x,y) = \sum_{m=1}^{\infty}\sum_{n=1}^{\infty} W_{mn} \sin\frac{m\pi x}{a} \sin\frac{n\pi y}{a}$$

For specific values of (m) and (n)

$$\frac{\partial^4 w}{\partial x^4} = W_{mn} \ (\frac{m\,\pi}{a})^4 \sin \frac{m\pi\,x}{a} \sin \frac{n\,\pi\,y}{a}\ .$$

$$\frac{\partial^4 w}{\partial y^4} = W_{mn} \ (\frac{n\pi}{a})^4 \sin \frac{m\,\pi\,x}{a} \sin \frac{n\,\pi\,y}{a}\ .$$

$$\frac{\partial^4 w}{\partial x^2\,\partial y^2} = W_{mn} \ (\frac{m\,\pi}{a})^2 \ (\frac{n\,\pi}{a})^2 \sin \frac{m\,\pi\,x}{a} \sin \frac{n\pi\,y}{a}\ .$$

   Substituting these values in the differential equation leads to :-

$$\left[(\frac{m\,\pi}{a})^4 + 2(\frac{m\,\pi}{a})^2 (\frac{n\,\pi}{a})^2 + (\frac{n\,\pi}{a})^4 \right] W_{mn} \sin \frac{m\pi\,x}{a} \sin \frac{n\,\pi\,y}{a}$$

$$= \frac{q_{mn}}{D} \sin \frac{m\,\pi\,x}{a} \sin \frac{n\,\pi\,y}{a}$$

$$W_{mn} = \frac{q_{mn}}{\pi^4 \ D(\ (\frac{m}{a})^2 + (\frac{n}{a})^2)^2}\ .$$

Where

$$q_{mn} = \frac{4}{a^2} \int_{x_1}^{x_2} \int_{y_1}^{y_2} q_{mn} \sin \frac{m\pi\,x}{a} \sin \frac{n\,\pi\,y}{a} \ dx\ dy$$

   where x_1 , x_2 , y_1 , y_2 are the coordinates of the loaded area within the plate , and after applying these limits in the expression :

$$q_{mn} = \frac{4\,q}{a^2} \ \frac{a^2}{\pi^2 \ mn} \left[(\cos m\pi - 1)\,(\cos n\pi - 1) \right] \Big|_0^a \Big|_0^a$$

   For uniformly loaded plate , m , n are odd

$$q_{mn} = \frac{16\ q}{\pi^2 \ mn}\ .$$

$$W_{mn} = \frac{16\ q / \pi^2}{\pi^4 \ D\ (\frac{m^2}{a^2} + \frac{n^2}{a^2})^2}$$

$$W_{mn} = \frac{4\,q\,a^4}{D\pi^6}$$

$$w(x, y) = \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} \frac{q_{mn}}{\pi^4 D \left((\frac{m}{a})^2 + (\frac{n}{a})^2 \right)^2} \sin\frac{m\pi x}{a} \sin\frac{n\pi y}{a}$$

$$w = \frac{4 q a^4}{D \pi^6} \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} \sin\frac{m\pi x}{a} \sin\frac{n\pi y}{a}$$

For the value of m=n =1

At center ,

$$x = \frac{a}{2} , \quad y = \frac{a}{2}$$

$$w = \frac{0.00406 \ q \ a^4}{D}$$

This equation can be expressed in anther form :-

$$\Delta = \frac{0.00406 \ q \ L^4}{D}$$

Where

$$D = \frac{E_C \ R_{E_C} \ h^3}{12(1 - \upsilon^2)}$$

Substituting the value of (D) and neglecting the possion's ratio due to it is so small effect in above equation ,

$$\Delta = \frac{0.00406 \ q \ L^5}{Ec \ I \ R_{Ec}}$$

Where

I :- the moment of inertia of the slab specimen ,(mm^4)

q :- applied uniform load , (N/mm^2) .

To show the procedure for predicting the load- deflection relationship according to both Resheidat and Beeby methods , slab specimen B_{13} was chosen . The compressive strength of concrete was 27.6MPa , modulus of elasticity of concrete was 13.16 GPa ,and that of steel was 205 GPa ,the modulus of rupture is 2.2 MPa ,(Note : all values were at 500 $^{\circ}$C) . The values of these properties at temperature 25 $^{\circ}$C were (38.4MPa , 31.7GPa , 205 GPa and 3.7 MPa) , compressive strength modulus of elasticity of concrete , modulus of elasticity of steel and modulus of rupture respectivly .

BA- Resheidat Method

1- Find the first line of the relationship .

The cracking moment of the section , is :-

$$M_{cr} = \frac{f_r \ L \ h^2 \ R_{f_r}}{6}$$

Where :-

f_r :- the modulus of rupture of concrete at 25 $^{\circ}$C , MPa

L :- length of the span , mm

h :- depth of the slab specimen , mm

R_{f_r} :-the ratio of the modulus of rupture of concrete at (T $^{\circ}$C) to the
        modulus of rupture of concrete at 25 $^{\circ}$C .

$$M_{cr} = \frac{3.7 \times 560 \times (40)^2 \times \frac{2.2}{3.7}}{6}$$

$$M_{cr} = 328533.34 \, N.mm$$

$$M_{cr} = \frac{q_{cr} \, L^2}{24}$$

$$q_{cr} = 14.08 \, kN = 0.04489 \, \frac{N}{mm^2}$$

$$I_v = C_1 \, I_g$$

$$I_g = \frac{bh^3}{12}$$

$$Ig = \frac{600 \times (40)^3}{12} = 3200000 \, mm^4$$

$$C_1 = 1 + 12 \, \eta^2 + 12 \, \rho \, n \, \frac{d}{h} \left(\frac{d}{h} - \frac{1}{2} - \eta \right)^2$$

Where : -

$$n = \frac{Es}{Ec}$$

$$n = \frac{205}{13.16} = 15.57$$

$$\eta = \frac{15.57 (0.00492) \left(\frac{32}{40} \right) \left(\frac{2 \times 32}{40} - 1 \right)}{2 \left(1 + (15.57) (0.00492) \left(\frac{32}{40} \right) \right)}$$

$$\eta = 0.01732$$

$$C_1 = 1 + 12 \, (0.01732)^2 + 12 \, (0.00492) \, (15.57) \, (0.8) \, (0.8 - 0.5 - 0.0173)^2$$

$$C_1 = 1.0623$$

$$Iv = (1.0623) \frac{600 \, (40)^3}{12} = 3399565.75 \, mm^4$$

$$\Delta c \, r = \frac{0.00406 \, q_{cr} \, L^5}{Ec \quad Iv \quad R_{Ec}}$$

$$\Delta c \, r = \frac{0.00406 \times 0.04489 \times (560)^5}{31700 \times 3399565.75 \times \left(\frac{13160}{31700} \right)} = 0.23 \, mm$$

2- Find the second line of the relationship as :-

$$\Delta s = \Delta c \, r + \Delta_2 \left[1 - \frac{M_{cr}}{M_s} \right]$$

M_s : the service moment of the slab .

$$M_s = \frac{q_s \, L^2}{24}$$

$$q_s = \frac{ultimate load}{1.6}$$

$$q_s = \frac{45}{1.6} = 28.35 \, kN = 0.093 \, \frac{N}{mm^2}$$

$$M_s = 680610 \, N.mm$$

Find the moment of inertia of the cracked section as :-

$$I_{cr} = C_2 \, I_g$$

Where : -

$$C_2 = (\frac{d}{h})^3 \left[4 \, K^3 + 12 \, n \, \rho \, (1-K)^2 \right]$$

$$K = \left[(n \, \rho)^2 + (2 \, n \, \rho) \right]^{\frac{1}{2}} - n \, \rho$$

$$K = \left[(15.57 \times 0.00492)^2 + (2 \times 15.57 \times 0.00492) \right]^{\frac{1}{2}} - 15.57 \times 0.00492$$

$$K = 0.322$$

$$C_2 = (\frac{32}{40})^3 \left[4 \, (0.322)^3 + 12 \, (15.57)(0.00492)(1 - 0.322)^2 \right]$$

$$C_2 = 0.2847$$

$$Icr = 0.2847 \, (\frac{600 \times (40)^3}{12}) = 911131.47 \, mm^4$$

$$\Delta_2 = \frac{0.00406 \times 0.093 \times (560)^5}{31700 \times 911131.47 \times (\frac{13160}{31700})}$$

$$\Delta_2 = 1.7 \, mm$$

Finally the mid – span deflection at service load is :-

$$\Delta_s = \Delta_{cr} + \Delta_2 \left[1 - \frac{Mcr}{Ms} \right]$$

$$\Delta_s = 0.23 + 1.7 \left[1 - \frac{328533.34}{680610} \right] = 1.07 \, mm$$

BB- Beeby Method

1- The cracking moment of the section , is :

$$M_{cr} = \frac{f_r\, L\, h^2 R_{fr}}{6}$$

$$M_{cr} = \frac{3.7 \times 560 \times (40)^2 \times (\frac{2.2}{3.7})}{6} = 328533.34\, N.mm$$

$$q_{cr} = 14.08\, kN = 0.00449\, \frac{N}{mm^2}$$

$$\Delta_{cr} = \frac{0.00406\, q_{cr}\, L^5}{Ec\, Ig\, R_{Ec}}$$

$$Ig = \frac{600 \times (40)^3}{12} = 3200000\, mm^4$$

$$\Delta cr = \frac{0.00406 \times 0.0449 \times (560)^5}{31700 \times 3200000 \times (\frac{13160}{31700})} = 0.24\, mm$$

$$Ec' = 0.57\, Ec$$

$$Ec' = 0.57\,(31700) = 18069\, MPa$$

2 – Find the moment of inertia of the cracked section as :-

$$I'cr = \left[\frac{b\, cn^3}{3} + n\, \rho\, b\, d\, (d - cn)^2 \right]$$

where :-

d = 40-8 =32 mm

  cn : The depth of the neutral axis from the compressive fiber of the slab ,mm .

$$\frac{b\,(cn)^2}{2} = n\, As\,(d - cn)$$

$$\frac{600 \times (cn)^2}{2} = 15.57 \times 100.53\,(32 - cn)$$

$$cn^2 + 5.21\, cn - 167 = 0$$

$$cn = 10.6\, mm$$

$$I_{cr} = \left[\frac{600 \times (10.6)^3}{3} + 15.57 \times 0.00492 \times 600 \times 32 \times (32-10.6)^2\right]$$

$I'cr = 911772.82 \text{ mm}^4$

$M_1 = Ma - Mcr$

$M_1 = 1050000 - 328533.34 = 721466.67 \text{ N.mm}$

$q_1 = 30.92 \text{ kN} = 0.0986 \text{ N/mm}^2$

$$\Delta_2 = \frac{0.00406 \, q_1 \, L^5}{E'c \, I'cr \, R_{Ec}}$$

$$\Delta_2 = \frac{0.00406 \times 0.0986 \times (560)^2}{18069 \times 911772.82 \times (\frac{13160}{31700})} = 3.22 \text{mm}$$

Finally the mid – span deflection at ultimate load is :

$\Delta u = \Delta cr + \Delta_2$

$\Delta u = 0.24 + 3.22 = 3.46 \text{ mm}$

الخلاصة

عند تعرض المنشآت الخرسانية المسلحة إلى درجات الحرارة العالية كما في حالة الحرائق، يجب تقييم سعتها التحملية في حالة عدم انهيار هذه المنشآت.

تهدف الدراسة لتقييم تأثير لهب النار على سلوك نماذج سقوف خرسانية مسلحة بعد وأثناء تعرضها الى لهب النار المباشر. يتضمن الجزء العملي اعداد وفحص 24 نموذج لسقوف خرسانية مسلحة بأبعاد (600×600×40 ملم)، تم تصميم خلطتان خرسانيتان بمقاومة انضغاط (30 و 38 ميكا باسكال) فئة أ ، فئة ب و كل فئة تحتوي على نسبتين تسليح(0.00492و0.00875).

تم تعريض ستة عشر من نماذج السقوف الى لهب النار المباشر من السطح الأسفل لثلاث مستويات للحرارة (400 ، 500 ، 600 م°) لمدة ساعة واحدة، بعد تبريد النماذج تدريجياً الى درجة الحرارة الاعتيادية، تم فحصها لحد الفشل بتسليط حمل موزع بانتظام على سطحها. تم اعداد وفحص نماذج للسيطرة من الخرسانة (مكعبات، اسطوانات، مواشير) وقضبان التسليح بعد تعرضها الى نفس الظروف التي عرضت إليها نماذج السقوف الخرسانية المسلحة لغرض ايجاد التغير في خواص الخرسانة (مقاومة الانضغاط، مقاومة الانثناء، معامل المرونة و مقاومة شد الانشطار) والتغير في خواص حديد التسليح لها مع درجات الحرارة العالية. كذلك تم اجراء فحصين لا اتلافيين، هما فحص الامواج فوق الصوتية ومطرقة شميدت المرتدة، و اجراء فحص الفقدان بالوزن لنماذج السقوف هذه قبل وبعد التعرض الى لهب النار المباشر.

اعتمادا على نتائج هذا البحث يمكن الاستنتاج بان قيم الحمل الأقصى انخفضت لنماذج السقوف بزيادة درجات حرارة النار. حصل اكبر انخفاض في نماذج السقوف ذات نسبة التسليح العالية وبدرجة حرارة 600 م°. الحمل الاقصى المتبقي عند 600 م° يتراوح بين (84.2 -86%) لنماذج السقوف الخرسانية فئة(أ) المسلحة بنسب تسليح (0.00875و0.00492) على التوالي و(83.7-85%) لنماذج السقوف فئة(ب) ولنفس نسب التسليح. وقد حافظت كافة النماذج التي تم تعريضها الى لهب النار على قدرتها لمقاومة حمل التشغيل ولكن بمعاملات أمان اقل0

شهدت نتائج الفحوص اللا اتلافية بعد فحص نماذج السقوف المعرضة للحرارة العالية نقصانا في القراءة مقارنة مع النماذج قبل التعرض .

كـذلك أظهـرت نتـائج الطـرق المبـسطة المـستخدمة للتحليـل الانـشائي(طريقتـي Resheidat، Beeby) لنماذج السقوف المعرضة الى النار توافقا جيدا مع النتائج العملية مما يؤكد صـلاحيتهم للاغراض العملية والتصميمية .

تم تحميل ثمانية من نمـاذج السقوف بمستوين تحميل وتعريضها في نفس الوقت الى لهب النـار، مـستوى التحميـل الاول 40% مـن الحمـل الاقـصى لنمـاذج السقوف (22، 32، 23.2، 33.6 كيلونيوتن)،والمستوى الثاني هو الحمل الخدمي (34.1، 47.5، 35.5، 48.8 كيلو نيوتن) . تم قياس الانحراف لنمـاذج السقوف باستمرار اثنـاء التعرض الى لهب النـار. بالاعتمـاد على الزيادة المفاجئـة في الانحراف تم حساب فترات مقاومة النارلهذه النمـاذج وكالاتي (126، 117، 117، 105 دقائق) لنماذج السقوف التي تم تحميلها 40% من الحمل الاقصى و(110، 96، 100، 90 دقائق) لنماذج السقوف التي تم تحميلها .

تم استخدام موديلات رياضية جديدة لغرض التنبؤ للحمل الاقصى بعد التعرض الى النار، كذلك التنبؤ بالانحراف اثنـاء التعرض الى لهب النـار. تعتمد هذه المـوديلات على معـادلات غير خطيـة متعددة المتغيرات للحصول على قيم معامل ارتباط جيد (0.95) مع تقليل الفرق بين القيم التخمينبـة و العملية.

جمهورية العراق
وزارة التعليم العالي والبحث العلمي
جامعة بابل
كلية الهندسة
قسم الهندسة المدنية

تحري سلوك و خواص البلاطات الخرسانية المسلحة المتعرضة الى لهب النار

رسالة مقدمة الى
كلية الهندسة في جامعة بابل
كجزء من متطلبات نيل درجة الماجستير
في علوم الهندسة المدنية

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اشراف

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