FIRE FLAME EXPOSURE EFFECT ON SOME MECHANICAL PROPERTIES OF CONCRETE

A Thesis

Submitted to the College of Engineering of the University of Babylon in fulfillment of partial requirements for the degree of Master of Science in Civil Engineering

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جامعـــــة بابـــــــل
كليـــة الهندســـــة
قسم الهندسة المدنية

تأثير لهب النار على بعض الخواص الميكانيكية

رسالة مقدمة إلى

قسم الهندسة المدنية

في كلية الهندسة-جامعة بابل

من قبل

وهي جزء من متطلبات نيل شهادة ماجستير علوم في الهندسة المدنية

مرزة كريم عمران

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ABSTRACT

. A lot of information about the properties of concrete and steel after exposure to high temperatures are available. However, information about the effect of direct exposure to fire flames on properties of concrete is limited. In this study, some mechanical properties of concrete and load–deflection behavior of rectangular reinforced concrete beams under the effect of fire flame exposure is presented. Two concrete mixes were used with target compressive strength (٣٠ and ٤٠ MPa) and named series A and series B respectively. The properties investigated were density, compressive strength, splitting tensile strength, flexural strength, and modulus of elasticity. Two non-destructive tests, the ultrasonic pulse velocity and rebound hammer were used. The concrete specimens were subjected to fire flame temperatures ranging from (٢٥-٧٠٠ ˚C) at different ages of ٣٠, ٦٠, and ٩٠ days Three temperature levels of ٤٠٠˚C, ٥٠٠˚C and ٧٠٠ ˚ C were chosen with four different exposure duration of ٠.٥, ١.٠, ١.٥ and ٢.٠ hours. Twelve rectangular reinforced concrete beams (١٠٠x١٥٠x١٠٠٠mm) were cast from each batch of concrete and subjected to fire flame at temperature levels of ٥٠٠˚C and ٧٠٠ ˚C. Two ages of ٣٠ and ٩٠ days and ١.٠ hour period of exposure were used.

Based on the results of this work, the properties of concrete were very sensitive to fire flame and they deteriorated, when the fire flame intensity was increased, for all ages and periods of exposure. The reduction in density ranged between (١.٩-٧.٧٪) at ٤٠٠˚C, (٤-٨٪) at ٥٠٠˚C and (٨-١١.٢٪) at ٧٠٠˚C.The residual compressive strength ranged between (٧٠-٨٥٪) at ٤٠٠ ˚C, (٥٩-٧٨٪) at ٥٠٠ ˚C and (٤٣-٦٢٪) at ٧٠٠˚C.Cooling by water caused further reduction in the compressive strength, compared with specimens cooled in air, the percentage reduction in compressive strength of the specimens cooled in water was (٢-٨٪) more than the specimens cooled in air. The compressive strength test results of

this study together with results obtained by other investigators were compared with CEB strength reduction curve and that of CEN. It was noticed that the test results agreed with CEN design curve rather than with that of CEB.

For the splitting tensile strength, the residual tensile strength was (٦٧-٧٨٪) at ٤٠٠°C, (٤٠- ٦٧٪) at ٥٠٠°C and (٢٠-٤٥٪) at ٧٠٠°C. But the reduction in splitting tensile strength specimens cooled in water showed more reduction than specimens cooled in air by (٣-١٤٪). The residual flexural strength was in the range of (٧١-٧٩٪), (٤٢-٥٨٪) and (٢٢-٤١٪) and the residual modulus of elasticity ranged between (٥٥-٧٥٪), (٣٤-٥١٪) and (١٦-٣٤٪) at ٤٠٠°C, ٥٠٠°C, and ٧٠٠°C respectively. The ultrasonic pulse velocity and rebound hammer tests also showed more reduction in the test results after burning the if compared with control specimens. Mathematical models for the prediction of some properties before and after exposure to fire flame were developed in this study. These models used non-linear regression equations to evaluate good coefficient correlation with fewer variables introduced in them.

For the load-deflection behavior, the control beams deflection was measured and compared with the values obtained by using equations of different Codes (ACI-٣١٨-٩٥ and CP١١٠-١٩٧٢). When the beams were exposed to fire flame (٥٠٠-٧٠٠°C) for one hour, it was noticed that the deflection that corresponds to the yield and ultimate load is greater than that for the control beams. On the other hand, the yield and ultimate loads are less than that before fire flame exposure.

الخلاصة

تتوفر معلومات كثيرة حول تأثير درجات الحرارة على خواص الخرسانة والحديد ، ولكن المعلومات حول تأثير لهب النار المباشر على هذه الخواص قليلة جدا. لقد تناول هذا البحث تأثير لهب النار المباشر على بعض الخواص الميكانيكية للخرسانة ، إضافة إلى تأثيرة في سلوك العتبات الخرسانية المسلحة وفي العلاقة بين الحمل وإلا نحراف بعد تعرضها إلى لهب النار المباشر.خلطتان خرسانيتان بمقاومة انضغاط (٣٠،٤٠) ميكاباسكال درست خواصهما الميكانيكية وهي:-
الكثافة ، مقاومة الانضغاط، مقاومة الشد (الانشطار)، معاير التصدع ومعامل المرونة . تم استخدام فحصين غير اتلافين، هما فحص الأمواج فوق الصوتية ومطرقة شميد ت المرتدة.عرضت النماذج الخرسانية للنار بدرجات حرارة تراوحت بين (٢٥-٧٠٠) درجة مئوية وبأعمار مختلفة تراوحت بين (٣٠،٦٠،٩٠) يوما مع ثلاث مستويات للحرارة هي(٤٠٠،٥٠٠،٧٠٠) درجة مئوية وبأربعة مديات للتعرض تتراوح بين (٠.٥،١،١,٥،٢) ساعة.

تم تعريض العتبات الخرسانية (١٠٠x١٥٠x١٠٠٠) ملم إلى درجتين حراريتين هما (٥٠٠،٧٠٠) درجة مئوية ولفترة زمنية مقدارها ساعة واحدة، وكانت النماذج بعمر(٣٠،٩٠) يوما. اعتمادا على نتا ئج البحث فان الخواص الميكانيكية للخرسانة تدهورت بتعرضها إلى لهب النار وان تدهورها ا ازداد بارتفاع درجات حرارة أللهب لكل الأعمار ولكل فترات التعرض.لقد تمثل هذا التدهور في انخفاض في الكثافة تراوح بين (١.٩-٧,٧%) في درجة ٤٠٠ مئوية (٤-٨.٧%) قي درجة٥٠٠ درجة مئوية و(٨-١١%) في درجة٧٠٠ درجة مئوية .

إما مقاومة الانضغاظ المتبقية فكانت فقد تراوحت بين(٧٠-٨٥%) في درجة ٤٠٠ درجة مئوية و (٥٩-٧٨%) قي درجة ٥٠٠ درجة مئوية و (٤٣-٦٢%) في درجة ٧٠٠ درجة مئوية وحصل نقصان اكبر في مقاومة الانضغاظ نتيجة لتبريدها بالماء مقارنة مع النماذج الخرسانية المبردة بالهواء، وكانت النسبة في الانخفاض هي (٢-٨%).

قورنت النتائج العملية لمقاومة الانضغاظ المعرضة إلى لهب النار في الدراسة الحالية مع بعض نتائج الدراسات الأخرى مع منحيين مقترحين من قبل المدونات الأوربية لتقييم نقصان المقاومة بتأثير درجات الحرارة فوجد أن النتائج العملية متوافقة أقرب مع توصيات المدونة الأوربية (CEN) عن (CEB).

أما بالنسبة الى مقاومة الشد(الانشطار) فكانت المقاومة المتبقية هي (٦٧-٧٨%) في درجة ٤٠٠ درجة مئوية و (٤٠-٦٧%) في درجة ٥٠٠ درجة مئوية و (٢٠-٤٥%) في درجة ٧٠٠ درجة مئوية.إما النقصان الإضافي في مقاومة الشد نتيجة لتبريد النماذج بالماء فكانت نسبته تتراوح بين (٢-١٤%) مقارنة مع النماذج المبردة بالهواء.

إما معاير التصدع المتبقية فتراوحت بين (٧١-٧٩%) و (٤٢-٥٨%) و (٢٢-٤١) وكذلك معامل المرونة المتبقي تراوح بين (٥٥-٧٥%) و (٣٤-٥١%) و(١٦-٣٤%) في درجات الحرارة ٤٠٠،٥٠٠،٧٠٠ درجة مئوية على التوالي . شهدت ذبذبات السرعة فوق الصوتية ،بعد فحص النماذج المعرضة نقصانا كبيرا في القراءة مقارنة مع النماذج قبل التعرض.

تم استخدام موديلات رياضية جديدة لغرض استخراج بعض الخواص الميكانيكية للخرسانة بعد التعرض إلى لهب النار، هذه الموديلات تعتمد على معادلات غير خطية متعددة المتغيرات للحصول على قيم تخمينية مع أيجاد معامل ارتباط جيد بين هذه الخواص باستخدام بعض المعلومات عن الخواص الميكانيكية للخرسانة قبل تعرضها إلى لهب النار.

أما بالنسبة إلى العلاقة بين الحمل المسلط والانحراف في العتبات الخرسانية المسلحة فان الانحراف المقاس بالنسبة للعتبات غير المعرضة إلى لهب النار تمت مقارنتها مع المدونة الأمريكية (ACI-٣١٨-٩٥) والبريطانية (CP-١١٠-١٩٧٢). بعد تعرض العتبات الخرسانية المسلحة إلى لهب النار وبدرجات حرارة تراوحت بين (٥٠٠-٧٠٠) درجة مئوية ولمدة ساعة واحدة، لوحظ إن الانحراف عند الحمل

المسبب لإجهاد الخضوع في الحديد والحمل المسبب للفشل في الخرسانة اكبر مقارنة بالعتبات الغير معرضة إلى النار. كما إن الخضوع والحمل الأقصى أقل من قيمتها قبل التعرض إلى لهب النار.

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نَرْفَعُ دَرَجَاتٍ مَنْ نَشَاءُ وَفَوْقَ كُلِّ ذِي عِلْمٍ عَلِيمٌ

صدق الله العظيم

سورة يوسف ، الاية ٧٦

SUPERVISORS CERTIFICATE

We certify that the thesis titled " ***Fire Flame Exposure Effect on Some Mechanical Properties of Concrete*** **",** was prepared by "***Mirza Karim Umran***", under our supervision at Babylon University in fulfillment of partial requirements for the degree of ***Master of Science in Civil Engineering***.

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Mirza Karim Umran

LIST OF CONTENTS

SUBJECT	PAGE
Acknowledgement	I
Abstract	II
Contents	IV
Notation	VI
CHAPTER ONE : INTRUDUCTION	
١-١ General	١
١-٢ Research Significance	٢
١-٣ Research Layout	٣
CHAPTER TWO: LITERATURE REVIEW	
٢-١ Effect of Fire on Concrete	٤
٢-١-١ Introduction	٤
٢-١-٢ Fire Effect on the Mechanical Properties of Concrete	٤
٢-١-٣ Fire Flame Effect on Non-destructive Tests.	١٦
٢-١-٣-١ Ultrasonic Pulse Velocity	١٦
٢-١-٣-٢ Rebound Method	١٦
٢-١-٤ Fire Effect on Reinforced Concrete	٢٠
CHAPTER THREE: MATERIALS AND EXPERIMENTAL WORK	
٣-١ Introduction	٢٦
٣-٢ Materials	٢٦
٣-٢-١ Cement	٢٦
٣-٢-٢ Fine Aggregate (sand)	٢٧
٣-٢-٣ Coarse Aggregate (gravel)	٢٧
٣-٢-٤ Water	٢٧
٣-٢-٥ Reinforcing Steel	٢٧
٣-٣ Mix Preparations	٢٨
٣-٣-١ Mix Design	٢٨
٣-٣-٢ Mix proportions	٢٨
٣-٤ Fresh Concrete	٢٨
٣-٤-١ Mixing of Concrete	٢٨
٣-٤-٢ Compaction	٢٩
٣-٥ Testing of Hardened Concrete	٢٩
٣-٥-١ Destructive Tests	٢٩

SUBJECT		PAGE
٣-٥-١-١	Compressive Strength Test	٢٩
٣-٥-١-٢	Splitting Tensile Strength Test	٢٩
٣-٥-١-٣	Flexural Strength Test	٢٩
٣-٥-١-٤	Static Modulus of Elasticity Test	٣٠
٣-٥-٢	Non- Destructive Tests.	٣١
٣-٥-٢-١	Ultrasonic Pulse Velocity Test (U.P.V).	٣١
٣-٥-٢-٢	Rebound Hammer Test.	٣٢
٣-٦	Heating and Cooling.	٣٢
٣-٧	Testing Reinforced Concrete Beams.	٣٤
٣-٧-١	Beam Specimens Preparation.	٣٤
٣-٧-٢	Beam Specimens Details.	٣٥
CHAPTER FOUR : RESULTS AND DISCUSSION		
٤-١	Introduction	٣٦
٤-٢	The Density	٣٦
٤-٣	Compressive Strength	٤١
٤-٣-١	Air- Cooling	٤٣
٤-٣-٢	Water- Cooling	٤٤
٤-٣-٣	The effect of the age after exposure to fire	٤٧
	Comparison between residual compressive strength Results and recommended design curves (CEB &CEN).	٤٧
٤-٤	Splitting tensile strength	٤٩
٤-٤-١	Cooling by Air	٥٢
٤-٤-٢	Cooling by Water	٥٢
٤-٤-٣	Effect of the age at heating	٥٣
٤-٥	Modulus of Rupture	٥٦
٤-٦	Modulus of Elasticity	٦٠
٤-٧	Ultrasonic pulse velocity results (U.P.V)	٦٤
٤-٨	Surface hardness results	٦٨
	Mathematical models for prediction of Mechanical properties of concrete after exposure to fire flame.	٧٣
٤-٩-١	Models for prediction of Compressive strength.	٧٣
	Models for prediction of Splitting tensile strength.	٧٤
٤-٩-٣	Models for prediction of Flexural strength.	٧٤
	Models for prediction of static modulus of elasticity.	٧٥
٤-١٠-١	Load-deflection behavior. Introduction.	٧٧

SUBJECT		PAGE
٤-١٠-٢	Ultimate moment strength.	٧٨
٤-١٠-٣	Deflection.	٨١
٤-١٠-٣-١	Deflection before exposure to fire flame.	٨١
	The effect of fire flame on the reinforcing steel bar.	٨٢
٤-١٠-٣-٣	Deflection after burning by fire flame.	٨٣
CHAPTER FIVE : CONCLUSIONS AND RECOMMENDATIONS		
٥-١	Conclusions	٩٠
٥-٢	Recommendations and future works	٩٤
	References	٩٥
	Appendix	

CERTIFICATE

We certify that we have read the thesis entitled ***(Fire Flame Exposure Effect on Some Mechanical Properties of Concrete)*** and as an examining committee, examined the student ***(Mirza Karim Umran)*** in its content and in what is connected with it, and that in our opinion it meets the standard of thesis for the degree of ***Master of Science in Civil Engineering (Construction Materials).***

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Mirza Karim Umran

Notations

Most of commonly used symbols are listed below, these and others are defined where they appear in the research

Item	Description
A_s	Area of tension reinforcement.
$A_{s'}$	Area of compression reinforcement
Age	The age of the specimens at time of exposure to fire flame.
b	Width of the beam.
CEB	Euro-Code –١٩٩١.
CEN	Euro-Code-١٩٩٣-١٩٩٤.
d	Effective depth of the beam.
d_s	Distance from tension bar center to the tension face.
d'	Depth from compression face to centered of compression steel.
E_{ca}	Modulus of elasticity of the concrete after exposure to fire flame temperature.
E_{cb}	Modulus of elasticity of the concrete before exposure to fire flame temperature
E_s	Modulus of elasticity of the steel.
f_y	Yield stress of steel reinforcing bar.
f_u	Ultimate strength of steel reinforcing bar.
f_{cub}	Compressive strength of concrete before exposure to fire flame temperature.
f_{cua}	Compressive strength of concrete after exposure to fire flame temperature.
f_{sb}	Splitting tensile strength before exposure to fire flame temperature.
f_{sa}	Splitting tensile strength after exposure to fire flame temperature.
f_{rb}	Flexural strength before exposure to fire flame temperature.
f_{ra}	Flexural strength after exposure to fire flame temperature.
h	Depth of the beam.
HSC	High strength concrete
HPC	High performance concrete.
I_{cr}	Second moment of area of cracked transformed section.
$K_c\phi$	Strength-reduction factor.

Notations

Item	Description
K	Fracture coefficient dependent on load or reaction type and on the support condition.
L	Clear span
$M.O.R$	Modulus of rupture.
$M_{c\,r}$	Cracking moment.
M_n	Nominal ultimate moment.
M_u	Ultimate moment
M'	Reduced moment of resistance.
P_u	Ultimate load.
P_e	Period of exposure.
R_{Na}	Rebound number test after exposure to fire flame temperature.
R_{Na}	Rebound number test before exposure to fire flame temperature.
$(U.P.V)_a$	Ultrasonic pulse velocity test after exposure to fire flame temperature.
$(U.P.V)_b$	Ultrasonic pulse velocity test before exposure to fire flame temperature.
Δ_u	Ultimate load deflection.
Δ_y	Yield load deflection.
ϕ_c	Compressive strength reduction.
ϕ_s	Steel stress reduction.

Chapter One

1

INTRODUCTION

١-١ General

Concrete is widely used in building construction because all the contents or the basic material of it (except cement) are natural materials and they are available in enough amount at low cost, the process of bringing them to the location of the building is easy and simple. Concrete has good stiffness and durability when it compounds with steel reinforcement. The concrete building construction could be exposed to the effect of fire. Human safety is one of the considerations in the design of residential, public and industrial buildings. Unlike wood and plastics, concrete is incombustible and does not emit toxic fumes on exposure to fire. On the contrary of steel, when subjected to temperature between (٧٠٠-٨٠٠)˚C, concrete is able to retain an adequate strength for reasonably long periods, thus permitting rescue operations by reducing the risk of structural collapse [١,٢,٣,٤].

There are many instances of exposing concrete structural members to elevated temperatures. One of the most common instances of exposure is by accidental fire in buildings. Another instance of heat exposure may be found in some industrial equipments when concrete is used in places exposed to sustained elevated temperatures. Long exposure to elevated temperature is imposed on foundations for blast furnaces and coke batteries, furnace walls and dampers, industrial chimneys and flues, floors on which metal parts are heat-treated, floors below boilers and kilns and nuclear- reactor pressure vessels [٥]. When a reinforced concrete structure has been involved in a fire, it is often possible to remain standing because of the good fire resisting properties of the concrete. This means that in dealing with such a situation, a choice can be made between reconstruction and reinstatement. Reinstatement can be often quicker and cheaper alternative. However, before a decision can be made, it is necessary to establish whether or not the damaged structure is suitable for such treatment. To do this, its residual capacity for structural performance must be assessed [٦].

In the structural design of buildings, in addition to the normal gravity and lateral loads, it is in many cases necessary to design the structure to safely resist exposure to fire. However it is usually necessary to guard against structural collapse for a given period of time [٧].

١-٢ Research Significance

A lot of research on concrete subjected to high temperatures were carried out. The object of those investigations was to determine the strength and deformation properties of concrete at elevated temperatures and to find out the causes of the changes that the material suffers in consequence of heat. The researchers exposed the concrete and mortar specimens to high temperatures in special furnaces.

There are indeed little research about temperature gradient and exposure time of concrete in direct contact with fire flames.

In the present work, there is an attempt to investigate the effect of exposure of concrete to fire flame on some mechanical properties of concrete.

Two concrete mixes properties and twelve reinforced concrete beams were investigated at different ages, different temperature levels and different exposure periods. There are many variables considered in this investigation, these cover the following aspects: -

١. Studying the fire effect on the mechanical properties of concrete, such as compressive strength, splitting tensile strength when cooled in air or in water, density, modulus of rupture (M.O.R), modulus of elasticity.

٢. Studying the fire flame effect on the immediate deflection of the reinforced concrete beams and comparing the results with control beams.

٣. Studying the fire flame effect on the specimens by using non-destructive tests, such as ultrasonic pulse velocity and Schmidt rebound hammer to estimate the degree of damage.

١-٣ Research Layout

In this research there are five chapters: -

Chapter one provides a general introduction.

Chapter two presents a review of both early and recent studies, including the effect of fire endurance on the mechanical properties of concrete and reinforced concrete beams under immediate load.

Chapter three deals with materials and experimental work, which include the program of testing.

Chapter four includes analysis of test results and their discussion.

Chapter five contains conclusions obtained from the test results and some possible recommendations for further work.

Chapter Two

2

LITERATURE REVIEW

٢-١ Effect of Fire on Concrete

٢-١-١ Introduction

Several factors control the response of concrete to high temperatures. Ingredients of concrete are important because both the cement paste and the aggregate consist of components that decompose through heating. It has already been stated that the type of aggregate and moisture content play important roles in the manner that concrete is affected. However, size and shape of aggregate, type of cement, admixture and water-cement ratio also influence the results during heating [٨]. A number of research dealt with the effect of fire on concrete, steel and reinforced members. It is clearly stated that both concrete and steel are affected by exposure to fire.

Some of the investigations, which tended to establish the effect of exposure of concrete to high temperature, are discussed here.

٢-١-٢ Fire Effect on The Mechanical Properties of Concrete

Malhotra [٩] in (١٩٥٦) studied the effect of high temperature on compressive strength of concrete by using cylinders of (٥١mm) in diameter and (١٠٢mm) long. Three mixes of aggregate / cement ratios (٣, ٤.٥, ٦) and four water- cement ratios (٠.٤٠, ٠.٤٥, ٠.٥٠, ٠.٦٥) with ordinary Portland cement, river sand and gravel aggregate were used. There were three 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 tested under a constant stress, 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 on compressive strength of concrete is independent of the water-cement ratio within the range of (٠.٤-٠.٦٥). He observed that the reduction in the compressive strength for lean mixes is smaller than for rich mixes. He also found that concrete under a compressive stress of the order of its design value has a smaller proportional decrease in strength than if the stress was absent. Finally the residual strength of heated concrete shows still further reduction in strength on cooling, being approximately ٢٠٪ less than the corresponding hot strength in temperature range ٢٠٠ to ٤٥٠°C for ١:٤.٥ and ١:٦ concrete mix.

Abrams[١٠] in (١٩٧١) investigated the effect of high temperature on the compressive strength of concrete by using (٧.٥x١٥cm) cylindrical specimens heated for short duration to temperature of (٩٣-٨٧١°C). The included variables were aggregate types (carbonate, siliceous and lightweight). The test specimens were heated without load then tested hot, heated with load and tested hot, and tested cool after heating. The original strength of concrete was (٢٣-٤٥.٥ N/mm٢). He found that carbonate aggregate concrete and lightweight aggregate concrete retained about ٧٥٪ of their original strength at a temperature up to ٦٤٩°C when heated without load and tested hot, while the corresponding temperature for the siliceous aggregate concrete was about ٤٢٧°C.He also found that the test procedure has a significant effect, where the strength of the specimens stressed in compression during heating, was generally higher than the specimens that were not stressed during heating. Moreover, the unstressed residual strength (specimens heated, cooled and tested) were lower than the strength of the companion hot tested specimens. Fig [٢-١] shows the effect of the test procedure on the strength of siliceous aggregate. He also concluded that the original strength of concrete has a little effect on the percentage strength retained at test temperature.

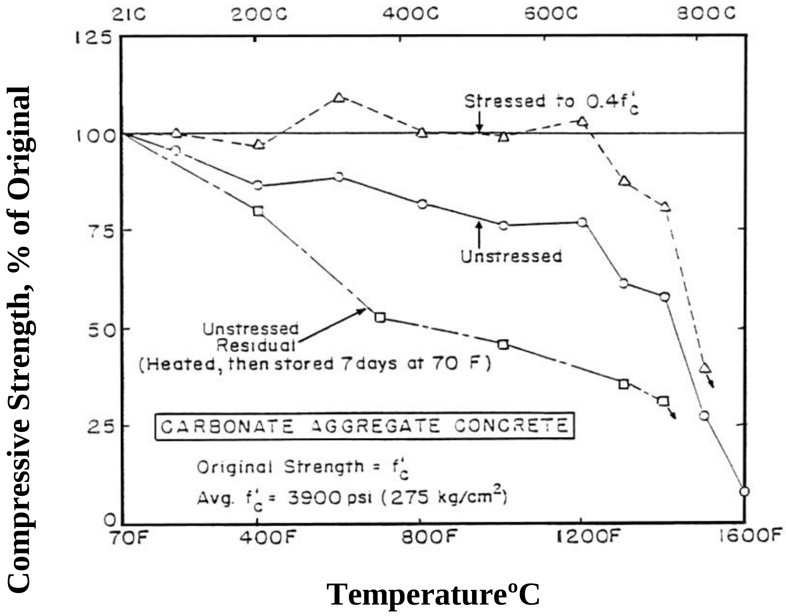


Fig.[٢-١-a]: *Effect of temperature on compressive strength of carbonate aggregate concrete, Ref:* (١٠)

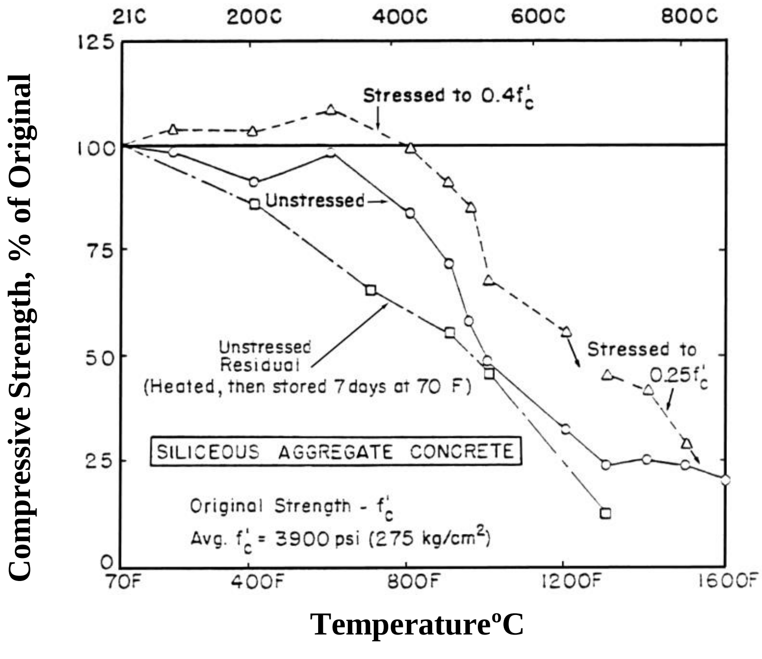


Fig.[٢-١-b]: *Effect of temperature on compressive strength of siliceous aggregate concrete, Ref:* (١٠)

Noriaki *et al.* [١١] in (١٩٧٢) studied the influence of elevated temperatures on the properties of concrete (compressive strength, modulus of elasticity and Poisson's ratio). Cylinders (١٥٠x٣٠٠mm) were used. After ٢٨ days of curing the specimens were stored in sealed containers, at (٤٠, ٧٠, ٩٠°C) for a period of [١-١٣week], the test results showed that the compressive strength for sealed specimens decreased with rising temperature. If the heating were continued for a long period of time (one year), the compressive strength would greatly be reduced. They found that the modulus of elasticity for sealed specimens at exposure to high temperatures depends on the compressive strength. Poisson's ratio was not affected under the storage temperature but stayed in the range of (٠.١٥-٠.٢٠).

Harada *et al.* [١٢] in (١٩٧٢) published a research about the thermal properties of concrete subjected to elevated temperatures. The investigated properties were, concrete compressive and tensile strengths. Concrete cylinders of (٥٠x١٠٠mm) were subjected to a slow rate of heating (١.٥°C/min) for one-hour exposure duration (short duration) and cooled gradually in the air and tested after the exposure to high temperature ranging between (١٠٠-٨٠٠°C). They found that the reduction in compressive strength was ٦٠٪ at ٤٠٠°C for all types of concrete. The same reduction took place in the tensile strength. Other concrete cylinders of (١٠٠x٢٠٠mm) were subjected to a long duration exposure. These specimens were heated at slow rate of heating (٠.٥°C/min) for ٧٢ hours inside the electrical furnace and gradually cooled for ٤٨ hours. They noticed a linear reduction in compressive strength with increase of temperature. They found that the percentage residual strength was ٨٠,٧٥,٦٠٪ at temperatures of ١٠٠, ٣٠٠, and ٤٥٠°C respectively.

Nasser and Marzauk [١٣] in (١٩٧٩) carried out an experimental study on the effect of high temperature on properties of mass concrete containing fly ash.

Tests were made on (٧.٥x٢٣.٥cm) concrete cylinders by using ordinary Portland cement. A ٢٥٪ replacement of cement by fly ash was used. After ٢٨ days of moist curing, the specimens were transferred to an electric oven. The specimens were heated for ٦ months at five different temperatures of (٧١, ١٢١, ١٤٩, ١٧٥ and ٢٣٢°C). At each temperature minimum of three specimens were tested after being exposed for (٣, ٧, ١٤, ٢٨, ٥٦, ٩١ and ١٨٠ days). All specimens were gradually cooled and tested at the room temperature. The test results showed that the strength and elasticity at (٧١°C) were almost equal to those at (٢١.٤°C) at all exposure times. They found that the increase in the strength and modulus of elasticity after ٦ months of exposure at (٢١.٤ and ٧١°C) was about ٢٠٪ of those at ٢٨ days. They also noticed that at temperature range of (١٢١ to ١٤٩°C), the compressive strength was greater than that at (٢١.٤°C). Furthermore, they observed that after exposure to temperatures of (١٧٧ and ٢٣٢°C) for ٦ months the strength reduced to about ٥٣ and ٢٧٪ respectively while the modulus of elasticity was reduced to ٤٣ and ٢٥٪ of the corresponding value.

Finally they found that the deterioration of the structural properties at (١٧٧ and ٢٣٢°C) was attributed to the transformation of most of tobermorite into crystalline alpha dicalcium silicates, which has poor binding qualities.

Carette *et al.* [١٤] in (١٩٨٢) published a research on the sustained high temperature effect on concrete made with normal Portland cement, normal Portland cement and slag, or normal Portland cement and fly ash. For each type of concrete, compressive and splitting tensile strengths were determined after ٢٨ days of moist curing, before and after periods of temperature exposure, also, changes in weight, pulse velocity and resonant frequency of the specimens were determined. For each condition of exposure, small samples of mortar obtained from specimens broken in compression were examined, the temperature range was from ٧٥ to ٦٠٠°C. Cylinders (١٠٢x٢٠٣mm) were cast. After ٢٨ days of curing, the specimens were stored in a normal room temperature for ١٦ weeks before heating. The exposure temperatures were ٧٥, ١٥٠, ٣٠٠, ٤٥٠ and ٦٠٠°C. The periods of exposure were ١, ٤, and ٨ months. The test results showed that the compressive and splitting tensile strength decreased with the increase of temperature up to ٧٥°C. Their decrease was ١٠٪ with respect to the reference concrete strengths. At ١٥٠, ٣٠٠,٤٥٠ and ٦٠٠°C the reduction in strengths was (١٠-٢٠٪), (٢٤-٣٩٪), (٣٨-٥٩٪) and (٥٠-٧٥٪) respectively. They found that the incorporation of fly ash and slag in the concrete did not improve the mechanical properties of concrete after exposure to sustained high temperatures. This was true regardless of the exposure temperature and water-cement ratio. They also observed that the significant changes in the mechanical properties of the concrete under long-term exposure occurred within the first month.

Nuri [١٥] in (١٩٨٣) studied the effect of high temperature on some properties of concrete. Cylindrical specimens (١٠٢ x ٢٠٣mm) were used for compressive and splitting tensile strengths. The temperature levels ranged between (٢٠- ٦٠٠°C) and the periods of exposure of heating were (٣٠, ٦٠ and ٩٠ minutes) at different ages of concrete (٣, ٧, ٢٨ and ٦٠ days). The test results showed that the compressive and splitting tensile strengths decrease with the increasing temperature. He found that the compressive strength increased for early age. He also noticed that when concrete is heated at age (٣, ٧, and ٢٨ days) from (١٥٠-٣٠٠°C), considerable recovery of concrete compressive strength occurred, whereas the splitting tensile strength suffered further losses. After ٣٠٠°C exposure, concrete retained from (٥٩-١٠٢٪) and (٤٤-١٠٠٪) from the original compressive and tensile strengths respectively. Finally he observed that after ٦٠٠°C, concrete retained (٢٨-٦٤٪) and (٢٠-٦٢٪) from original compressive and splitting tensile strengths respectively.

Al-Ausi and Faiyadh [١٦] in (١٩٨٥) carried out an investigation to study the effect of heating to high temperatures and cooling in the air and in the water on the compressive strength of concrete. Concrete cubes (١٥٠-x ١٥٠-x١٥٠ mm) were used. The specimens were exposed to different levels of heat up to ٧٠٠°C for different periods up to constant weight. The specimens were tested hot, others were tested after cooling in the air the third group was tested after cooling in the water for two hours. Figure [٢-٢] shows the compressive strength for different water-cement ratios after cooling in the air. They noticed that the compressive strength was not affected significantly after heating to ١٠٠°C. They found that there was an increase in the compressive strength of air-cooled specimens which were heated to ٢٠٠°C when compared with hot tested specimens They noticed further reduction in compressive strength when the specimens were heated beyond ٢٠٠°C , the reduction was ٨٨-٩١٪ at ٧٠٠°C.They also noticed that the effect of water-cement ratio on the compressive strength at a high temperature was very little.

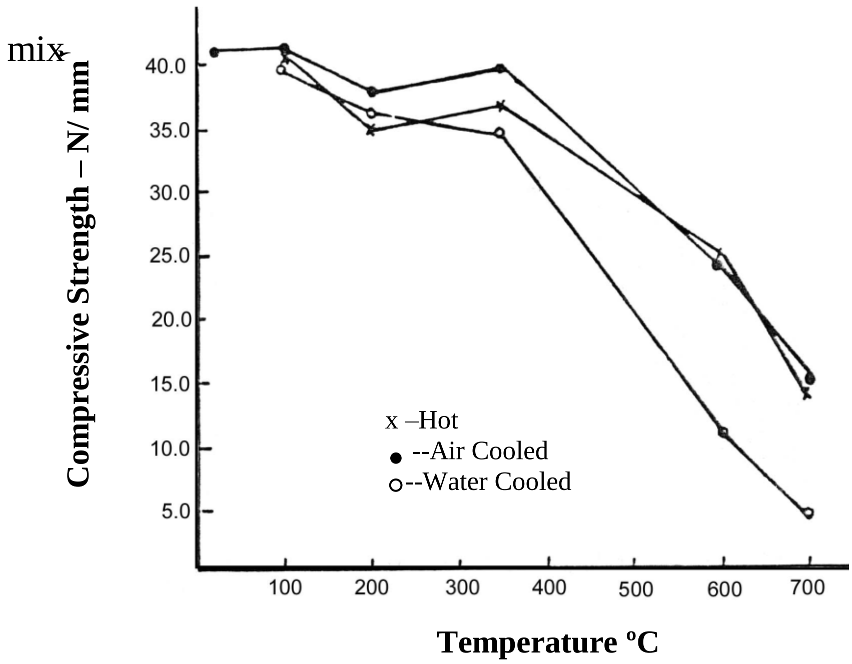


***Fig.[٢- ٢]:** Relation between temperature and compressive strength , heated for ٩٠ minutes , Ref :* [١٦]

Mohamedbhai [١٧] in (١٩٨٦) studied the effect of exposure time and rates of heating and cooling on residual strength of heated concrete. The included variables were exposure time at maximum temperature ١, ٢, ٣ and ٤ hours and four temperature levels ٢٠٠, ٤٠٠, ٦٠٠ and ٨٠٠°C. He used two methods for heating and cooling, slow and quick. The test results showed that the exposure time beyond one hour has a significant effect on the residual strength of concrete, but the effect diminishes on the level of exposure temperature increases. The bulk of strength loss occurs within the first two hours of temperature exposure. He found that the residual strength of concrete after one hour of exposure at ٢٠٠, ٤٠٠, ٦٠٠ and ٨٠٠°C approximately was ٨٠, ٧٠, ٦٠ and ٣٠٪, respectively of its unheated strength. As for the case of two or more hours of exposure, the residual strength was ٧٠, ٦٠, ٤٥, and ٢٥٪. He also found that the rate of heating and cooling had no effect on the residual strength of concrete heated to ٦٠٠°C and beyond. The pulse velocity measurements appear to give better indication of the level of temperature to which concrete has been exposed rather than the residual strength of that concrete.

Elizzi *et al.* [١٨] in (١٩٨٧) investigated the influence of different temperatures on the compressive strength and density of concrete. They used (١٠٠ x ١٠٠ x ١٠٠ mm) cubes heated for a short duration (one hour) to temperatures ranging from ٢٠-١٠٠٠°C and the ages of concrete at heating were (١٤, ٢٨, ٩٠ days). The test results showed that the compressive strength decreased ١٠٪ from the original strength up to ٤٠٠°C and at ٦٠٠°C the strength reduction was ٥٠٪ from the original. They noticed that there was a large strength reduction when heated to temperatures above ٤٠٠°C. They also mentioned that the small reduction in density up to ٣٠٠°C was a result to the loss of the free water from concrete specimens. At temperature above ٣٠٠°C, large reduction in density took place because of loss of the reaction water in concrete.

Kumar *et al.*[١٩]in (١٩٨٩) studied the effect of temperature on the properties of superplasticized concrete such as compressive strength, modulus of elasticity and Poisson's ratio. The test specimens were (١٥٠mm cubes) and

(١٠٠x١٠٠x٥٠٠mm) prisms. The specimens were exposed to temperatures of ١٥٠, ٣٠٠, ٤٥٠ and ٦٠٠°C for ٣ and ٦ hour in an electrical furnace. The test results showed that the residual strength of superplasticized concrete increased at ٣٠٠°C while conventional concrete decreased. At ٤٥٠°C the residual strength in superplasticized concrete decreased more than conventional concrete. They also noticed a slight increase in compressive strength for both concrete below ٣٠٠°C, while there was a rapid drop beyond ٣٠٠°C. They found that the modulus of rupture decreased as the temperature increased, but the reduction in superplasticized was more than in conventional Concrete. They also observed that the superplasticized concrete possessed better fire resistance than the conventional concrete.

Castillo and Durrani [٢٠] in (١٩٩٠) studied the effect of transient high temperature on high strength concrete. The concrete compressive strength varied between (٣١.١- ٨٩) MPa and the temperature exposure was in the range of (٢٣-٨٠٠°C). Cylindrical specimens (٥١x١٠٢mm) were used. The specimens were tested under both stressed and unstressed conditions. They concluded that high strength concrete showed (١٥-٢٠٪) loss of compressive strength at temperature range of ١٠٠-٣٠٠°C, and they found that after an initial loss of strength, high strength concrete recovered its strength between ٣٠٠ and ٤٠٠°C reaching the maximum value of (٨-١٣٪) above the room temperature strength. They noticed that the strength dropped about ٣٠٪ of the room temperature strength at ٨٠٠°C. They also observed that the modulus of elasticity of high strength concrete decreased by ٥ to ١٥ % when exposed to temperature in the range of ١٠٠ to ٣٠٠°C and at (٦٠٠-٨٠٠°C) the modulus of elasticity was only ٢٠ to ٢٥٪ of the value at the room temperature.

Valiasis and Papayianni [٢١] in (١٩٩٠) studied the effect of high temperature on the mechanical properties of concrete in which Portland cement Concrete cylinder specimens (١٥٠x٣٠٠ mm) were used. After ٢٨ days of curing and six months of drying, the specimens were exposed to four temperature levels, ٢٠٠, ٤٠٠, ٦٠٠ and ٨٠٠°C without any imposed load. Groups of three specimens each were crushed at ١ day, ٧days and ٣months after heating. The included variables were compressive strength, splitting tensile strength and modulus of elasticity. The test results showed that the concrete with Portland cement only had a reduction in strength about ٢٥٪. While the concrete with pozzolanic material showed a reduction from ٣٨٪ to ٥٠٪ at ٢٠٠°C. They observed that at a temperature over ٤٠٠°C all tested concrete suffered deterioration and lost ٧٠-٨٠٪ of their initial strength. Also, they found that the thermal behavior of concrete with lignite fly ash (high lime) is closer to those of concrete with Portland cement of Greek-type.

Phan and Carino [٢٢] in (١٩٩٨) provided a systematic comparison of results conducted by various researchers, about the effect of high temperature exposure on the mechanical properties of both normal strength concrete (NSC) and high strength concrete (HSC). Various types of specimens (prisms and cylinders) were used. The specimens, some were conventional with Portland cement, others included additives such as silica fume, fly ash and steel fiber. Two types of aggregate normal weight (NWA) and lightweight aggregate, were used. The compressive strengths of the specimens at testing ranged between (٢٠-٥٠ MPa). Three conditions of testing were used, which were; stressed, unstressed and unstressed residual strength. They compared the test data with the design recommendations, prescribed by the CEN Euro Codes (CEN ١٩٩٣ [٢٣], ١٩٩٤ [٢٤] and CEB ١٩٩١.[٢٥] Table [٢-١] showed the strength reduction factors according to CEN (١٩٩٣ and ١٩٩٤). They noticed that the material properties of HSC vary with temperature differently from those of NSC. The difference is more pronounced in the temperature range between ٢٥-٤٠٠°C. They also noticed that the recommended design curves of fire exposure of concrete are more relevant to NSC than HSC. The modulus of elasticity of all types of concrete suffers higher rate of decrease beyond ٣٠٠°C.

***Table[٢- ١]:**Strength- reduction factors $K_{C,\theta}$ for NSC according to CEN (١٩٩٣ and ١٩٩٤) ENV[٢٢] .*

Concrete Temperature °C θ	$K_{C,\theta}$= $f_{C,\theta}$ / $f_{C,٢٠}$ ˚C		
	NSC		LWC (٤)
	Siliceous (٢)	Calcareous (٣)	
٢٠	١	١	١
١٠٠	٠.٩٥	٠.٩٧	١
٢٠٠	٠.٩٠	٠.٩٤	١
٣٠٠	٠.٨٥	٠.٩١	١
٤٠٠	٠.٧٥	٠.٨٥	٠.٨٨
٥٠٠	٠.٦٠	٠.٧٤	٠.٧٦
٦٠٠	٠.٤٥	٠.٦٠	٠.٦٤
٧٠٠	٠.٣٠	٠.٤٣	٠.٥٢
٨٠٠	٠.١٥	٠.٢٧	٠.٤٠
٩٠٠	٠.٠٨	٠.١٥	٠.٢٨
١٠٠٠	٠.٠٤	٠.٠٦	٠.١٦
١١٠٠	٠.٠١	٠.٠٢	٠.٠٤
١٢٠٠	٠	٠	٠

Chan and Sun [٢٦] in (٢٠٠٠) carried out an experimental program to study the mechanical properties and pore structure of high performance concrete (HPC) and normal-strength concrete after exposure to high temperature. After the concrete specimens were subjected to a temperature of ٨٠٠°C, their residual compressive strength was measured. The porosity and pore size distribution of the concrete were investigated by using mercury intrusion porosimetry. The test result showed that (HPC) had higher residual strength, although the strength of (HPC) degenerated more sharply than the normal-strength concrete after exposure to high temperature. They found that the changes in pore structure could be used to indicate the degradation of mechanical property of (HPC) subjected to high temperature. Another paper published by the same authors [٢٧] in (٢٠٠٠) investigated the effect of heating and cooling regimes on residual strength and microstructure of normal strength and high performance concrete after they were exposed to high temperatures, ٨٠٠°C and ١١٠٠°C and two cooling regimes. The test results obtained showed that the residual strength of both (HPC) and (NSC) dropped sharply after exposure to high temperatures. They observed that water cooling which resulted in a significant thermal shock, caused a bit more severe deterioration in strength compared to furnace cooling. They found that the thermal shock was not necessarily the primary cause for spalling in (HPC). They used mercury intrusion porosimetery to measure variation in the pore structure of concrete. They also noticed the significant changes in the cumulative pore volume curves before and after high temperatures in both (NSC) and (HPC). Moreover, they found that the cumulative pore volume of (HPC) increased more remarkably than of (NSC).

Habeeb [٢٨] in (٢٠٠٠) investigated the effect of high temperatures on the mechanical properties of high strength concrete (HSC). The specimens were subjected to elevated temperature ranging between (١٠٠-٨٠٠°C). Five temperature levels of (١٠٠, ٣٠٠, ٥٠٠, ٦٠٠ and ٨٠٠°C) were chosen with three different exposure duration of ١, ٢ and ٤ hours without any imposed loads during heating. The specimens were heated and cooled under the same regime and tested either one day or one month after heating. Compressive strength of ١٠٠mm cubes and flexural strength (١٠٠x ١٠٠ x ٥٠٠mm) prisms were measured. Ultrasonic pulse velocity (U.P.V) and dynamic modulus of elasticity (Ed) were tested also. He observed that (HSC) was more sensitive to high temperature exposure than normal strength concrete (NSC). He found that the residual compressive strength ranged between (٩٠-١٠٠٪) at ١٠٠°C, (٧٢-١٠٣٪) at ٣٠٠°C, (٥٥-٨٧٪) at ٥٠٠°C and (٢٢-٦٦٪) between ٦٠٠-٨٠٠°C. The flexural strength was found to be more sensitive to high temperature exposure than the compressive strength. The residual flexural strength was in the range of (٩٢-٩٨٪), (٥٢-٩٨٪)

and (٢٩-٤٧٪) at ١٠٠°C, ٣٠٠°C and ٥٠٠°C respectively and (٢-٣٠٪) at ٦٠٠-٨٠٠°C. He also found that ultrasonic pulse velocity (U.P.V) and dynamic modulus of elasticity (Ed) were more sensitive to elevated temperatures than the compressive strength. He also noticed that exposure time after one hour has a significant effect on residual compressive strength of concrete.

٢-١-٣ Fire Flame Effect on Non-destructive Test.

٢- ١- ٣- ١ 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. [٢٩, ٣٠, ٣١, ٣٢]

The pulse velocity of concrete is affected by variety of factors; the composition and maturity of concrete, the geometry of section being tested, and conditions at test time all affect the measured pulse velocity of Portland cement concrete, [٣٣, ٣٤, ٣٥]

٢- ١- ٣- ٢ Rebound Method

The surface hardness of concrete members is tested by the "Schmidt Rebound hammer". This testing by hammer estimates the surface hardness by the rebound number which can be taken as a measure of the concrete strength and percentage of voids. [٣٦, ٣٧, ٣٨]

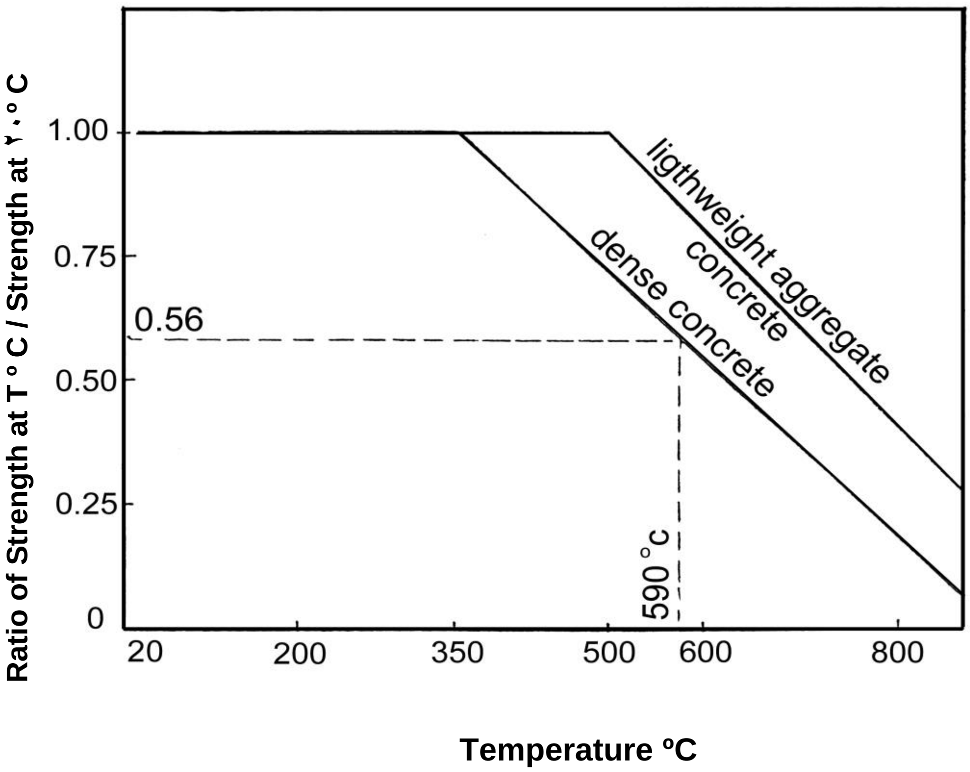
A considerable number of research work had dealt with the effect of fire on the non-destructive testing of the concrete.

Logothetis and Economou [٣٩] in (١٩٨١) investigated the influence of high temperatures on the properties of concrete by using non-destructive methods such as the rebound hammer and pulse velocity. Two series of concrete specimens (٢٠x ٢٠ x ٢٠cm) cubes were tested; the first series (A) consisted of ٣٢ specimens of different mixes and subjected to different curing conditions. These were submitted to various oven temperatures (١٠٠, ٢٠٠, ٣٠٠, ٤٠٠, ٥٥٠, and ٨٠٠°C) for three periods (١, ٢ and ٣ hours). The second series (B) consisted of ٣٠ specimens cast in ٣ groups of ١٠ specimens each., These series were exposed to four oven temperature (١٠٠,٢٠٠, ٥٠٠, ٨٠٠°C) for a period of ٢ hours. They noticed that the pulse velocity has a constant decrease with increasing temperature. They also found that mixing proportions and heating time proved to have small influence on the results. Moreover, they observed that the rebound measurements showed no significant changes in the range up to around ٦٠٠°C.

Chung and Law [٤٠] in (١٩٨٣) tested the fire damage of concrete by ultrasonic pulse velocity. The test specimens were (١٥٠ x ١٥٠ x ٧٥٠mm) prisms made from concrete of different composition by using granite aggregate. The specimens were heated to different temperatures not exceeding ٥٠٠°C at age ٢٨ or ٩٠ days. Two types of cooling, some cooled in the air, others quenched in the water. The test results showed that the reduction in pulse velocity was less than ٣٠ 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 is accompanied by ٢٠ percent reduction in strength. They noticed that the reduction in pulse velocity beyond ٣٠ percent is accompanied by reduction in strength at faster rate both under air-cooled and quenched conditions. Also, they observed that ultrasonic testing could be used to assess the fire damage of concrete. The assessment is approximate and the compressive strength of concrete varies mainly with the type of the aggregate.

The same authors published in (١٩٨٥) another paper [٤١], to assess the fire damage on concrete. The test specimens were concrete prisms ١٥٠x١٥٠x٧٥٠. The test 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 target 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 temperature of heating in the range between (٦٠٠-٨٠٠°C) with a maximum duration of ١٧٥ minutes.They found that the reduction in pulse velocity was between (٣٣-٥٥٪) from the initial pulse velocity.

Mahmoud [٤٢] in (١٩٩٥) published a research on the strength of prestressed concrete girders of a hall roof exposed to three hours fire. The roof consisted of a grid of prestressed secondary girders forming a dome like roof. This dome was supported on four peripheral main prestressed concrete large girders, he made visual inspection and concrete testing. The concrete tests were ultrasonic pulse velocity (U.P.V) and core test. From the colours of concrete surface, spalling at various locations and conditions of debris on the floor, he concluded that the temperature inside the hall reached about ٦٠٠°C. From ultrasonic pulse velocity and core test, he found that the concrete compressive strength was equivalent to ٢٧.٥ MPa. While the original strength from project document was about ٤٩.٦MPa. He also found that the ratio of concrete strength after and before fire was ٠.٥٦. He used a graph from BS-٨١١٠ part ٢, ١٩٨٥ [٤٣] which is titled " Design curves of concrete strength with temperature ", with reduction ratio ٠.٥٦, the corresponding temperature for dense concrete is ٥٩٠°C. Due to the reduction of load, the carrying capacity of four main girders were calculated. He suggested a new type of roof with new supports and the girders were lightly loaded after being repaired.



***Fig[٢-٣]:**Design curves for variation of concrete strength with temperature, Ref: (٤٣)*

Katwan and Abdul-Hammed (٤٤) in (٢٠٠٠) 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 semidestructive tests, such as concrete cores or non-destructive tests and such as ultrasonic pulse velocity and Schmidt hammer. It was found that the concrete compressive strength was equivalent to٢٣ MPa. They found that the actual compressive strength of concrete as obtained from the project document was more than ٣٥ MPa, it was estimated that the ratio between strength after and before fire was less than ٠.٦٦. They found the attained temperature from BS-٨١١٠ part ٢. The corresponding temperature was estimated at ٥٢٠ °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.

Essa [٤٥] in (٢٠٠١) studied the effect of burning by fire flame on the properties of concrete. The investigated properties were concrete compressive strength and density. Ultrasonic pulse velocity (U. P.V) and rebound hammer (R) were used also. The tested specimens were heated to two temperature levels ٥٠٠°C (achieved by subjecting the cubes to direct fire flame from petroleum gas burner) and ٨٠٠°C (achieved by using a special oven). The heating durations were ١ and ٢ hours for the specimens exposed to ٥٠٠°C, while it was ١ hour for the specimens exposed to ٨٠٠°C. The author concluded that the reduction in compressive strength ranged between (٢٣-٣١٪) and ٣٩٪ at ٥٠٠°C, when the periods of burning were ١ and ٢ hours respectively. At ٨٠٠°C the reduction at ١ hour was ٧٧٪ from the original strength. He also found that the reduction in density ranged between (١٠-١٤٪) and (٢٢- ٢٤٪) at ٥٠٠°C, when the periods were ١ and ٢ hours respectively. Moreover, the ultrasonic pulse velocity (U. P. V) and rebound number (R) reduced at ٥٠٠°C to (١٦-٣٢٪) and (٣٩-٥٠٪) and to about (١١-١٢٪) and (١٦-٢١٪) for exposure periods ١ and ٢ hours respectively. At exposure to ٨٠٠°C for ١ hour duration the reduction in (U. P. V) was ٥٠٪ while that in rebound number was about ١٨.٥٪.

٢-١-٤ Fire Effect on Reinforced Concrete.

The behavior of reinforced concrete structures exposed to fire depends on the thermal properties of steel and concrete, strength and stiffness properties of the concrete and steel at elevated temperatures, and on the ability of the structure to redistribute internal forces during the course of the fire. [٤٦]

Gustaffero *et al* [٤٧] in (١٩٧١) studied the structural behavior of prestressed and reinforced concrete beams exposed to standard fire (ASTM-E١١٩) [٤٨]. The test included ١٣ beams with ١٢.٢ m length loaded uniformly with design load. The beams were divided in two groups, the first group was pre-tensioned concrete beams, the second was post-tensioned concrete beams. Each group reinforced by prestressing steel or ordinary deformed steel bars of grade ٤٠ or ٦٠. 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 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.

Ellingwood and Shaver [٤٩] in (١٩٧٧) investigated the effect of some factors such as the fire temperature and yield stress of reinforcing steel and the average decrease in resistance of reinforcing steel and the concrete cover of reinforced concrete on the behavior of a simply supported beam exposed to standard fire.The analysis method includes finding the coefficients of the amount of suspect in the yield stress, available information accuracy about temperature and yield stress and the depth of the reinforcing bar and the reduction in yield stress to the original stress, which affect the amount of the maximum strength of the section . Relations of suspect coefficient of these variances were derived. By using these coefficients, the maximum resistance of the section at any temperature degree could be assessed. A relation was also derived to count the beam resistance period of fire. They have concluded the followings:

١- The beam resistance to fire increased when the thickness of the concrete cover of the reinforced concrete beam increased because the bar temperature keeps low.

٢- The coincidence between the theoretical calculations and practical results when compared was acceptable.

٣- The main factors, which affect determining the beam behavior, are the accuracy of assessing the bar temperature and the changes of its properties by heat.

Khan and Royles[٥٠] in (١٩٨٦) studied the behavior of reinforced concrete beams after subjecting them to elevated temperatures. They investigated the load-deflection relationship, cracks pattern and steel to concrete bond. Prismatic concrete beams (٩٦٠x ١٤٠x ٦٦ or ١٠٧mm) were used. ٨-mm plain bars and ١٦mm torbars were used to reinforce them. The specimens were heated to temperature ranges from ٢٠-٨٠٠°C at a slow rate of heating (٢°C/min) for one hour exposure duration. They found that the effect of temperature is insignificant at temperatures ranging from ١٠٠ to ٢٠٠°C, but the strength decreases significantly between ٣٥٠ to ٥٠٠°C. Compared with ambient condition, the flexural strength characteristics weakened by ٥٠٪ 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 the deflection cracks were more prominent in the shear zone of the smaller section beams, whereas for larger beams, flexural tensile cracking developed in the mid-span region. They also found that the deflection cracking initially followed some of the thermally induced cracks.

Asa'ad [٥١] in (١٩٨٧) studied the behavior of structural reinforced concrete specimens subjected to elevated temperature. Four types of reinforced concrete samples were used. Singly and doubly reinforced concrete beams having the dimension of (١٠٠x ١٠٠x ١١٠٠mm) 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 (١٠٠x ١٥٠x ١٣٠٠mm) and structural frame with outer dimensions of (٩٠٠x ٧٥٠mm) were used. The frame had a cross-section of (١٠٠x ١٥٠mm) for the beam and (١٠٠x١٠٠mm) for the column. The specimens were subjected to temperatures of ١٥٠, ٣٠٠, ٦٠٠,٧٥٠ and ٩٠٠°C at the age of ٣٠ or ٩٠ days and tested in flexure after cooling. The researcher found that both flexural strength and stiffness decreased with the temperature increase. He also noticed that the use of top reinforcement had limited this decrease. Moreover, he observed that the increase in temperature led an increase in the magnitude of moment redistribution in continuous beams.

Fahmi and Khalil [٥٢] in (١٩٩٢) studied the effect of high temperature exposure on the behavior of prestressed concrete beams. The specimens used were unbounded post-tensioned beams having the dimensions of ٨٠x١٢٠x٩٠٠mm. They were heated to a temperature ranging between ٢٥-٨٠٠°C and at age of ٢٨ and ٩٠ days to the specified temperatures for periods ١,٣ and ٥ hours. The beams were reinforced with ١٠mm diameter prestressed deformed bars and ٣ and ٤mm bars for longitudinal and stirrups respectively. Cylindrical concrete specimens with dimension (١٥٠x٣٠٠mm) were used for compressive and modulus of elasticity determination. Prisms of (١٠٠x ١٠٠x ٤٠٠mm) were used for flexural strength. The test results showed that there is a slight increase in compressive strength at temperature up to ٣٠٠°C at ٢٨ days age. At ٦٠٠°C and ٨٠٠°C the compressive strength reduction was ٥٢٪ and ٨٢٪ in comparison with original strength respectively. They also found that the flexural capacity of all prestressed concrete beams had decreased with the increase of the temperature. They noticed that the reduction in modulus of elasticity and flexural strength at ٨٠٠°C exposure were ٩٧٪ and ٩١٪ from the original values respectively.

Sanjayan and Stocks [٥٣] in (١٩٩٣) carried out an investigation to study spalling of concrete in fire. They observed that the high strength concrete might be more prone to spalling in the fire than the normal strength concrete. Moreover, they noticed that the reason for spalling could not be attributed entirely to the large cover because a similar cover at web did not cause any spalling. No spalling occurred in the web, possibly because in the web the concrete was exposed to three sides and therefore the distance for the moisture to escape was much shorter and also because of the existence of wider flexural cracks in the web.

Al-Owasiy [٥٤] in (٢٠٠١) studied the effect of temperatures on bond in reinforced concrete. Single concrete mix ١: ١.٥: ٣ with two water-cement ratios of (٠.٤٥ and ٠.٦٠). The concrete specimens were heated in electrical furnace for ١ hour period of exposure at four temperature levels , ١٥٠ , ٢٥٠, ٤٠٠ and ٥٠٠°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 revealed that the reduction in compressive and splitting tensile strength for concrete specimens of water-cement ratio of ٠.٦٠ and cooled in air were ١٢٪, ٢٢٪, ٣٤٪ and ١٤٪, ٢٨٪ and ٦٤٪ at ١٥٠, ٤٠٠ and ٥٠٠°C respectively. For concrete of water-cement ratio of ٠.٤٥, the reduction in compressive and splitting tensile strengths at ٤٠٠ and ٥٠٠°C were ٢٤٪, ٣٣٪ and ٣٨٪, ٦٣٪ respectively. He found that the reduction in bond strength for pull-out with ١٢mm and ٢٠mm reinforcing bar diameters at ١٥٠, ٢٥٠, ٤٠٠ and ٥٠٠°C was about ١٤٪, ١٠٪ , ٢١٪ , ٣٩٪ and about ١٨٪ , ١٦٪ , ٢٤٪ and ٤٢٪ respectively. The percentage reduction in compressive and bond strengths cooled in water compared to the air cooled specimens at ٢٥٠, ٤٠٠ and ٥٠٠°C was about ٨٪, ١٠٪, ١٢٪ and about ٦٪, ١١٪ and ١٣٪ respectively. He observed also that the splitting tensile strength reduction for water cooled specimens was about ١٩٪, ٣٠٪, ٤٤٪ and ٧٠٪ after exposure to ١٥٠, ٢٥٠, ٤٠٠ and ٥٠٠°C respectively.

Chapter Three

3

MATERIALS AND EXPERIMENTAL WORK

٣-١ Introduction

In this chapter, the detail of the experimental program of the present work is presented. It includes details of materials used, specimen preparation and tests procedure. Two mixes with design compressive strength of ٣٠ and ٤٠ MPa were used. Experimental variables were:

١. Density.
٢. Compressive strength.
٣. Splitting tensile strength.
٤. Modulus of rapture.
٥. Modulus of elasticity.
٦. Load-deflection.

The specimens were cast, moist cured for ٢٨ days, air-dried in the laboratory. They were tested for each mix at each of the ages ٣٠, ٦٠ and ٩٠ days before and after exposure to fire flame at three temperature levels, ٤٠٠, ٥٠٠ and ٧٠٠°C and for four exposure periods of ٠.٥, ١.٠, ١.٥ and ٢.٠ hours. Two types of tests, destructive and non-destructive tests were used.

٣ –٢ Materials

٣-٢-١ Cement

Ordinary Portland cement (O.P.C) manufactured by the New Cement Plant of Kufa was used throughout this investigation. This cement complied with the Iraqi specification No.٥/ ١٩٨٤.[٥٥] The cement was stored in a dry place to avoid the exposure to the atmosphere. The physical properties and chemical compositions are presented in Tables [A-١] and [A-٢].

٣-٢-٢ Fine Aggregate (Sand)

AL-Akhaidur well-graded natural silica sand was used. The results of physical and chemical properties of the sand are listed in Table [A-٣]. Its grading conformed to the Iraqi specification No.٤٥/١٩٨٤[٥٦], Zone (٣).

٣-٢-٣ Coarse Aggregate (gravel)

The coarse aggregate was AL-Nibaee gravel with a maximum size of ١٩mm. The coarse aggregate was washed, then stored in air to dry. The coarse aggregate used conforms to the Iraqi specification No.٤٥/١٩٨٤.Table [A-٤] shows the grading and properties of the coarse aggregate.

٣-٢-٤ Water

Tap water was used throughout this work for both making and curing the specimens.

٣-٢-٥ Reinforcing Steel.

Deformed bars of diameters ٨ and ١٠ mm were used for longitudinal reinforcement and plain bars of diameter ٦ mm were used for stirrups. Table [٣-١] gives the results of testing.

Table [٣-١]: *Properties of reinforcement.*

Approximate Diameter (mm)	Measured Diameter (mm)	Area ($mm^٢$)	Yield stress *Fy* (MPa)	Ultimate Strength *Fu* (MPa)	Modulus of Elasticity (GPa)
١٠	١٠	٧٨.٥٤	٤٧٠.٣	٦٩٠.٥	٢١٠
٨	٨.٠١	٥٠.٣٩	٤٦٠	٦٦٣	٢١٠
٦	٦	٢٨.٢٧	٣٥٠	٤٤٠	٢٠٠

٣-٣ Mix Preparations

٣-٣-١ Mix Design

Two target design strengths of ٣٠ and ٤٠ MPa were denoted as series A and B respectively. They were designed according to British mix design method BS ٥٣٢٨: part٢: ١٩٩١ (٥٧) specifications.

٣-٣-٢ Mix proportions

The proportions of the concrete mixes are summarized in Table [٣-٢]

***Table [٣-٢]:** Mixes proportions*

		Mix Proportions (kg/m٣)				
Series	W/c-ratio	Water	Cement	Sand	Gravel	Slump(mm)
A	٠.٥٢	١٩٥	٣٧٥	٥٩١	١١٩٩	٨٠
B	٠.٤٥	١٩٥	٤٣٥	٥١٥	١٢١٥	٦٠

٣-٤ Fresh Concrete

٣-٤-١ Mixing of Concrete

The concrete was mixed in a horizontal drum laboratory mixer, with a capacity of ٠.١m٣. The interior surface of the mixer was cleaned and moistened before placing the materials. The gravel, sand and cement were weighed and added in accordance with the provisions of ASTM C ١٩٢ (٥٨). They were dry mixed in the mixer for about one minute before the required water was added to the mixture. The constituents were mixed wet for about two minutes until a homogeneous concrete was obtained and the slump was measured according to ASTM C ١٤٣- ٨٩a: ١٩٨٩ (٥٩) immediately after mixing.

٣-٤-٢ Compaction

After mixing, the concrete was poured into the moulds with two layers, each layer was compacted by using a vibrating table for about ٣٠ second until no air bubbles emerged from the surface of the concrete. The moulds were leveled by hand trawling and covered with polyethylene sheet in the laboratory for about ٢٤ hours.

٣-٥ Testing of Hardened Concrete

٣-٥-١ Destructive Tests

٣-٥-١-١ Compressive Strength Test

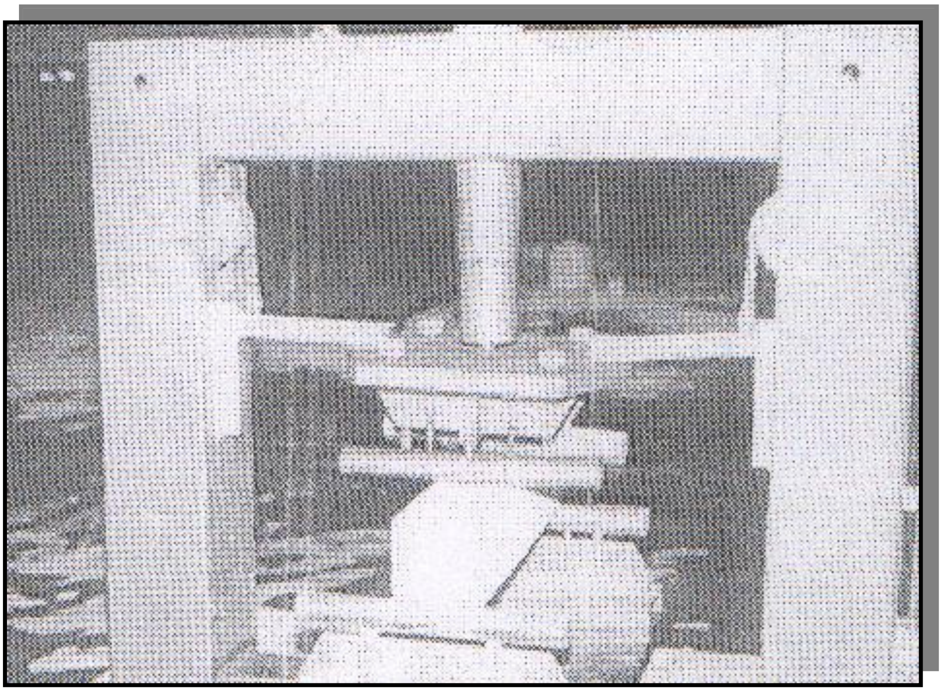
Compressive strength was carried out and tested according to BS ١٨٨١: part ١١٦: ١٩٨٣[٦٠] . A total number of ٤٥٠ standard cubes (١٥٠mm) were tested by using a digital universal testing machine of ٢٠٠٠ kN maximum capacity. The load was applied gradually and increased continuously at a constant rate. Each cube was weighed at a test before and after heating to determine its density. Each compressive strength value was the average of strength of three cubes.

٣-٥-١-٢ Splitting Tensile Strength Test

The splitting tensile strength was determined according to the procedures outlined in ASTM C-٤٩٦ [٦١]. A total number of ٤٥٠ cylinders (١٠٠x٢٠٠mm) were tested. Cylinders were cast, demolded and cured in a similar way as the cubes. Each splitting tensile strength value was the average of strength of three specimens.

٣-٥-١-٣ Flexural Strength Test

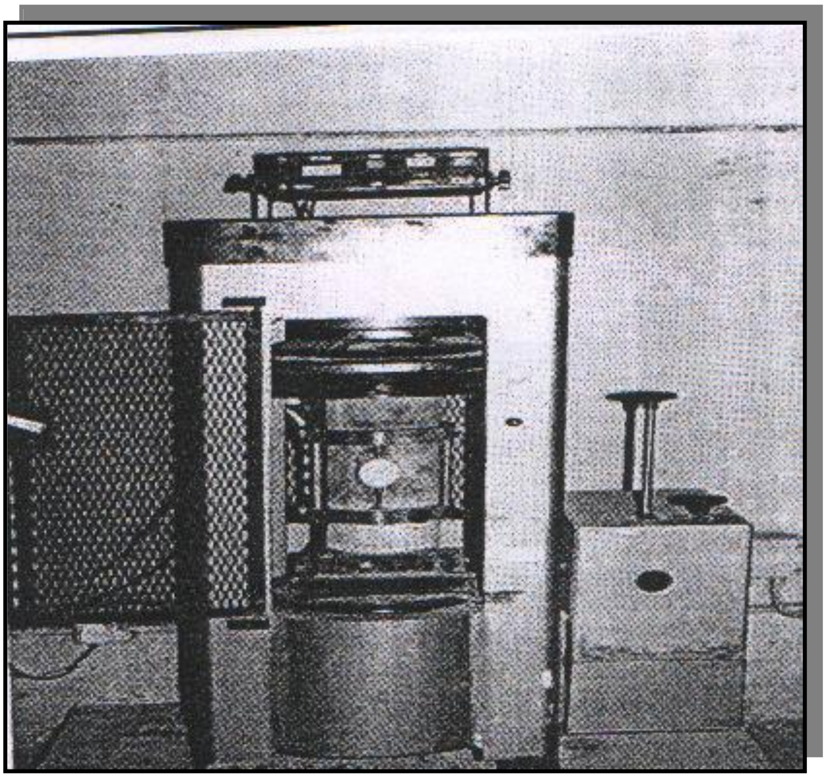
Concrete prisms of dimensions (١٠٠x ١٠٠x٤٠٠mm) were cast according to ASTM C٧٨-٨٤: ١٩٨٩ [٦٢] procedure. A total number of ٢٣٤ prisms were tested. The prisms were cast, demolded and cured in a similar manner as the cubes. Modulus of rupture tests were performed according to ASTM C٧٨-٨٤: ١٩٨٩ procedure using two-point load as shown in Plate [٣-١]. Each value of the modulus of rupture was the average of the test results of three prisms.



Plate[٣- ١]: *Set-up modulus of rupture test*

٣-٥-١-٤ Static Modulus of Elasticity Test

The static modulus of elasticity was determined according to ASTM C-٤٦٩(٦٣) specifications. A total number of ١٥٦ cylinders (١٥٠x ٣٠٠mm) were tested. They were cast, demolded and cured as for the cubes. 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 applied gradually and increased continuously at constant rate of ١.٢٧mm/min until ٤٠٪ of the ultimate load. The compressometer used has a gauge length of ١٥٠ mm and gauge with an accuracy of ٠.٠٠٢mm as shown in Plate [٣-٢]. The recorded results were the average readings of two cylinders.

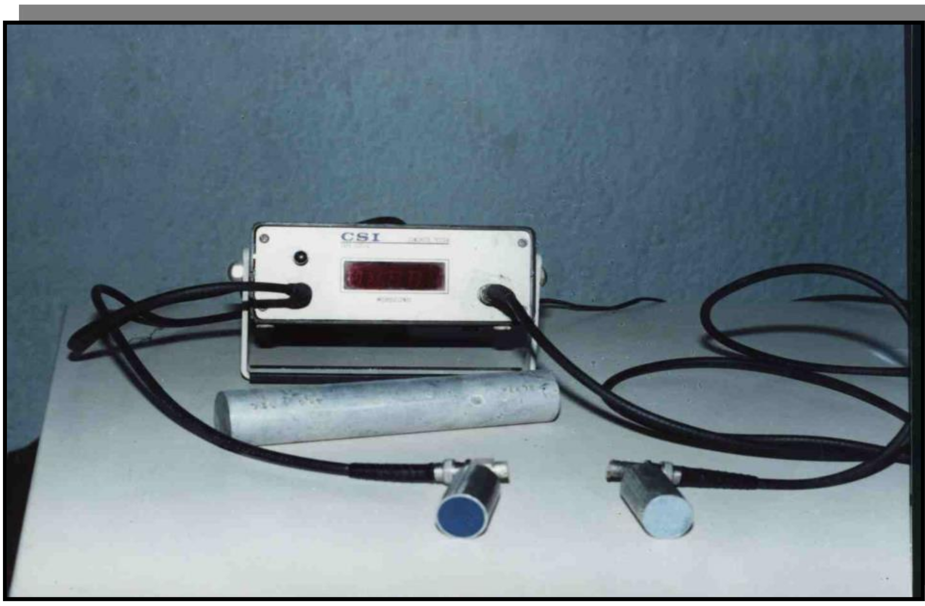


Plate[٣- ٢]: *Set-up modulus of elasticity test*

٣-٥-٢ - Non-Destructive Tests

٣-٥-٢-١ Ultrasonic Pulse Velocity Test (U.P.V)

Ultrasonic pulse velocity was used to monitor the variations in compressive strength, quality and the intensity of microcracking of concrete before and after heating. The ultrasonic pulse velocity was measured by an ultrasonic concrete tester (CS١), type CC-٤ as shown in Plate [٣-٣]. The test method is prescribed by BS- ١٨٨١: part ٢٠٣: ١٩٨٦[٦٤] specifications.



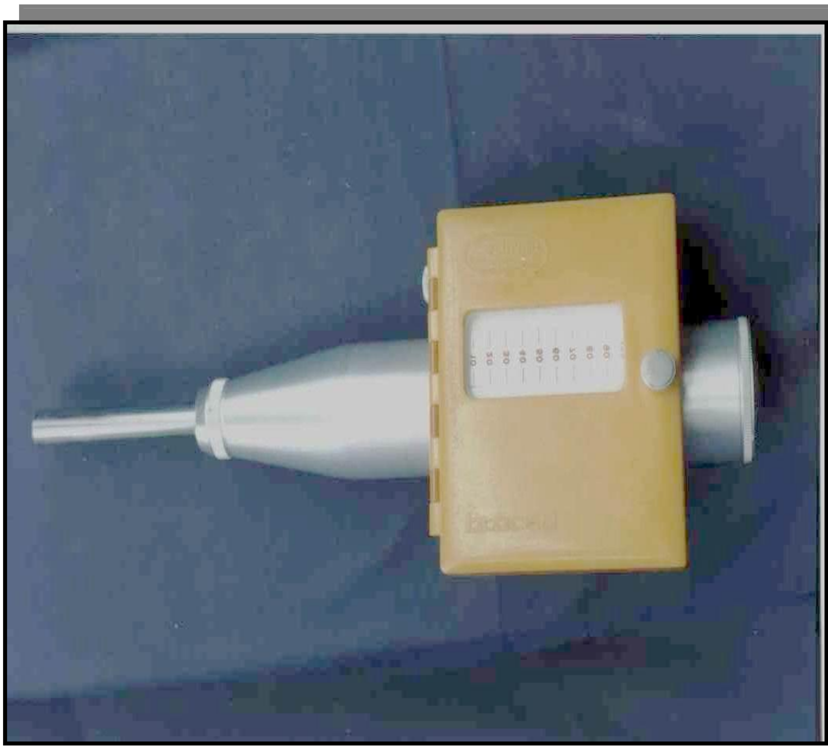
Plate[٣-٣]: *Ultrasonic pulse transit time*

٣-٥-٢-٢ Rebound Hammer Test.

Schmidt hammers were used to estimate the surface hardness of concrete specimens by recording the rebound number, which could be as a measure of the concrete strength and percentage of voids. Schmidt hammer type (Proceq) was used as shown in Plate [٣-٤]. The test method is prescribed by BS-٤٤٠٨: part ٤:١٩٧١[١٥] specifications.

٣-٦ Heating and Cooling

The concrete specimens were burnt with direct fire flame from a net of methane burners inside a brick stove with dimensions of (٧٥٠ x ٩٠٠-x١٢٠٠ mm) as shown in Plate [٣-٥]. The bare flame was intended to simulate the heating condition in an actual fire. The intensity of the flame was adjusted to raise the specimens to different temperatures. When the target temperature was reached, the temperatures were continuously recorded by two digital thermometers, one of them was positioned in flame contact with the bottom, while the other was at the top of the specimen as shown in Plate [٣-٦]. After burning, part of the concrete specimens was allowed to cool inside the stove for ٣ hours and stored in the laboratory environment about ٢٠ hours. The other specimens were quenched immediately in water for ٢ hours and then stored in laboratory environment about ٢٠ hours also before testing.



Plate[٣-٤]: Rebound hammer test

Plate[٣-٥]: Brick stove of net methane burners

Plate[٣- ٦]: *Digital thermometer*

٣-٧ Testing Reinforced Concrete Beams

٣-٧-١ Beam Specimens Preparation

The reinforcing bars were cut to the desired length, and ٩٠-degree hooks were formed at the ends of each bar dimensioned according to sections (٧-١) and (٧-٢) of ACI ٣١٨- ٩٥[٦٧] code, stirrups made from ٦mm diameter plain bars were provided to prevent the shear failure. The total number of beams cast from each mix was twelve. Four beams were retained as reference beams for ٣٠ and ٩٠ days of age, eight were exposed to fire flame with different ages, different periods of exposure and different temperature levels of exposure. A steel cage was fixed in the steel mould to prevent its movement during casting and vibration. The control specimens including cubes, cylinders and prisms were cast from each batch of concrete in addition to the beam specimens as mentioned before. The beams and control specimens were covered with polyethylene sheet in the laboratory for about ٢٤ hours, and then demolded for curing in water for ٢٨ days.

٣-٧-٢ Beam Specimens Details

The beams were simply supported. All beams were ١٠٠٠mm length, ١٥٠mm height and ١٠٠ mm width as shown in Fig [٣-١]

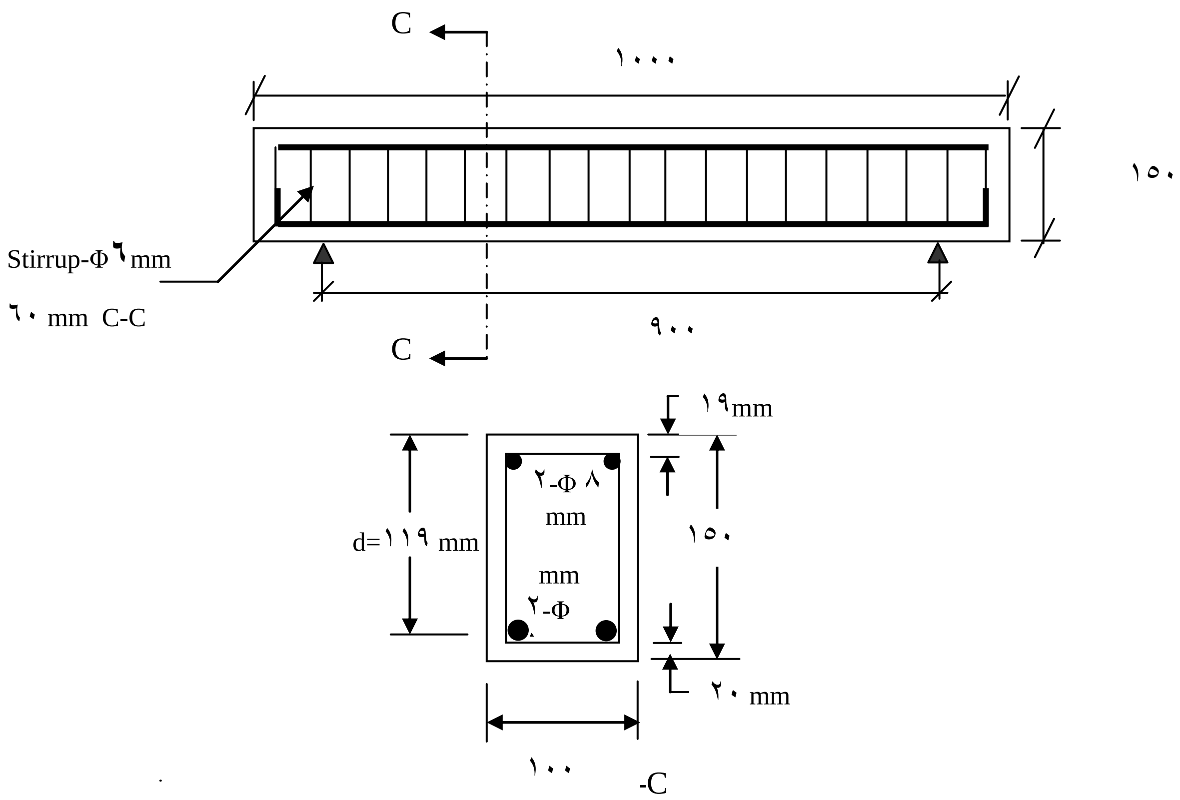


Fig. [٣- ١]: *Reinforced concrete beam*

Chapter Four

4

RESULTS AND DISCUSSIONS

٤-١ Introduction

Experimental results are presented in this chapter showing the effect of the fire flame on some mechanical properties of concrete, such as density, compressive strength, splitting tensile strength, flexural strength and modulus of elasticity. The test results of the load-deflection relationship of reinforced concrete beams are presented also.

٤-٢ The Density

Tables [٤-١] and [٤-٢] show the effect of the exposure to fire flame on the density of concrete, while Figures [٤-١] to [٤-٣] and [٤-٤] to [٤-٨] show the relations between fire flame temperature and the density for series A and B respectively. It can be seen from these Tables and Figures that the density behaved as follows:-

١- At ٤٠٠ºC fire flame temperature and for all ages and all periods of exposure, the reduction in density ranged between (٤.٨ – ٧.٧ %) and (١.٩ – ٤.٥ %) for series A and B respectively if compared with the initial density before exposure to fire.

٢- The reduction was (٥.٩ -٨٪) and (٤ – ٧.١ %) for series A and B respectively at ٥٠٠ºC fire flame temperature.

٣- More reduction in density took place when the fire flame temperatures increased.At٧٠٠ºC,the reduction in density was (٨–١١.٢٪), and (٨–٩.٦ %) for series A and B respectively. The loss in density of series A was ١.٦٪ more than that in density of series B.It can be observed that series B lose less weight than series A, which can be attributed to the less water- cement ratio and less evaporation of adsorbed water on interface of the gel crystals of hardened cement paste, also to less amount of pores and voids in series B.These results confirmed with that of Al-Elizzi[١٨] , Habeeb[٢٨] and Essa [٤٥]

Table [٤- ١]: *Test values of the density of concrete specimens of series-A- after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Density (kg/ m³) Temperature ºC				Ratios ρ_a / ρ_b		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/ ١ (٥)	٣/ ١ (٦)	٤/ ١ (٧)
٣٠	٠.٥	٢٣٢٥	٢٢٠٢	٢١٦٧	٢١٣٩	٠.٩٤٧	٠.٩٣٢	٠.٩٢٠
	١.٠		٢١٨٨	٢١٦٢	٢١٢٠	٠.٩٤١	٠.٩٣٠	٠.٩٠٨
	١.٥		٢١٦٣	٢١٥٣	٢٠٧٩	٠.٩٣٠	٠.٩٢٦	٠.٨٩٤
	٢.٠		٢١٤٨	٢١٣٩	٢٠٥١	٠.٩٢٣	٠.٩٢٠	٠.٨٨٢
٦٠	٠.٥	٢٣٢٣	٢٢٠٧	٢١٧٩	٢١٤٩	٠.٩٥٠	٠.٩٣٨	٠.٩٢٥
	١.٠		٢١٩٠	٢١٦٥	٢١٢١	٠.٩٤٣	٠.٩٢٤	٠.٩١٣
	١.٥		٢١٧٦	٢١٥٨	٢١١٢	٠.٩٣٧	٠.٩٢٤	٠.٩٠٩
	٢.٠		٢١٦٩	٢١٦٠	٢٠٩١	٠.٩٣٠	٠.٩٢٥	٠.٩٠٠
٩٠	٠.٥	٢٣٢٠	٢٢٠٩	٢١٨١	٢١٥٨	٠.٩٥٢	٠.٩٤٠	٠.٩٣٠
	١.٠		٢١٩٥	٢٢٥٤	٢١٣٧	٠.٩٤٦	٠.٩٤١	٠.٩٢٢
	١.٥		٢١٨١	٢١٦٧	٢١٣٤	٠.٩٤٠	٠.٩٣٤	٠.٩٢٠
	٢.٠		٢١٦٩	٢١٥٨	٢١١٦	٠.٩٣٥	٠.٩٣٠	٠.٩١٢

$\rho_a =$ The density of concrete after exposure to fire flame.

$\rho_b =$ The density of concrete before exposure to fire flame

Table [٤- ٦]: *Test values of the density of concrete specimens of series-B-after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Density (kg/ m٣) Temperature ºC				Ratios ρ_a / ρ_b		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٢٣٧٨	٢٣٠٧	٢٢٤٠	٢٢٠٢	٠.٩٧٠	٠.٩٤٣	٠.٩٢٦
	١.٠		٢٢٩٥	٢٢٢٨	٢١٨٥	٠.٩٦٥	٠.٩٣٧	٠.٩١٩
	١.٥		٢٢٨٢	٢٢٢٨	٢١٦٤	٠.٩٥٩	٠.٩٣٥	٠.٩١٠
	٢.٠		٢٢٧١	٢٢٠٩	٢١٥٠	٠.٩٥٥	٠.٩٢٩	٠.٩٠٤
٦٠	٠.٥	٢٣٧٦	٢٣١٦	٢٢٥٩	٢٢١٢	٠.٩٧٤	٠.٩٥٠	٠.٩٣١
	١.٠		٢٣٠٢	٢٢٥٠	٢١٨١	٠.٩٦٩	٠.٩٤٦	٠.٩١٨
	١.٥		٢٢٩٠	٢٢٣٩	٢١٧٤	٠.٩٦٣	٠.٩٤٢	٠.٩١٥
	٢.٠		٢٢٩٣	٢٢١٨	٢١٦٢	٠.٩٦٥	٠.٩٣٨	٠.٩١٠
٩٠	٠.٥	٢٣٧٥	٢٣٣٢	٢٢٨٠	٢٢٣٣	٠.٩٨١	٠.٩٦٠	٠.٩٤٠
	١.٠		٢٣٢٠	٢٢٧٥	٢٢٠٦	٠.٩٧٧	٠.٩٥٨	٠.٩٢٩
	١.٥		٢٣١٢	٢٢٦٧	٢١٩٥	٠.٩٧٣	٠.٩٥٤	٠.٩٢٤
	٢.٠		٢٣٠٠	٢٢٦٠	٢١٨٥	٠.٩٦٨	٠.٩٥١	٠.٩٢٠

ρ_a = The density of concrete after exposure to fire flame.

ρ_b = The density of concrete before exposure to fire flame.

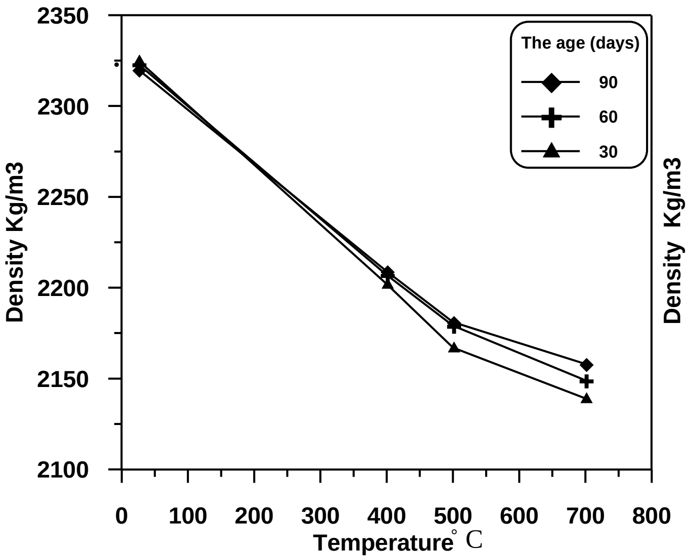


Fig.[4-1] The effect of fire flame on the density of series-A at 0.5 hour period of exposure.

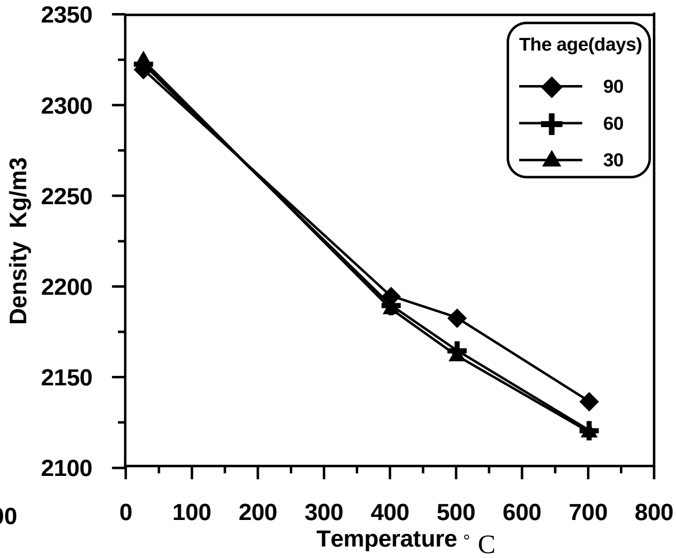


Fig.(4-2) The effect of fire flame on the density of series-A at 1.0 hour period of exposure.

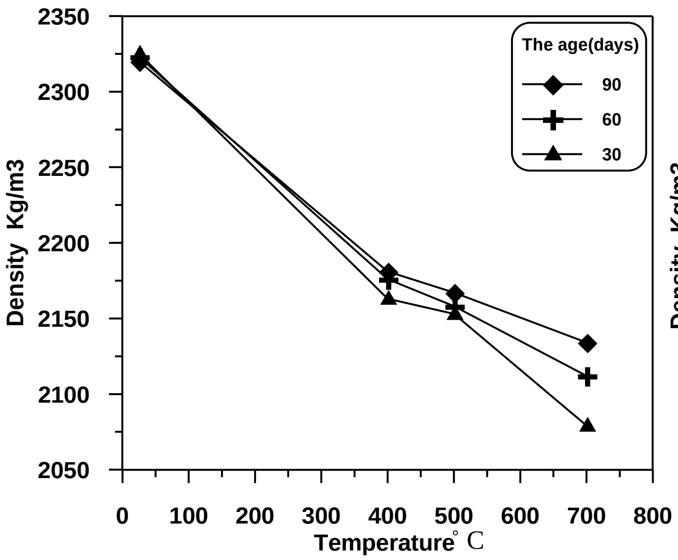


Fig.(4-3) The effect of fire flame on the density of series-A at 1.5 hours period of exposure.

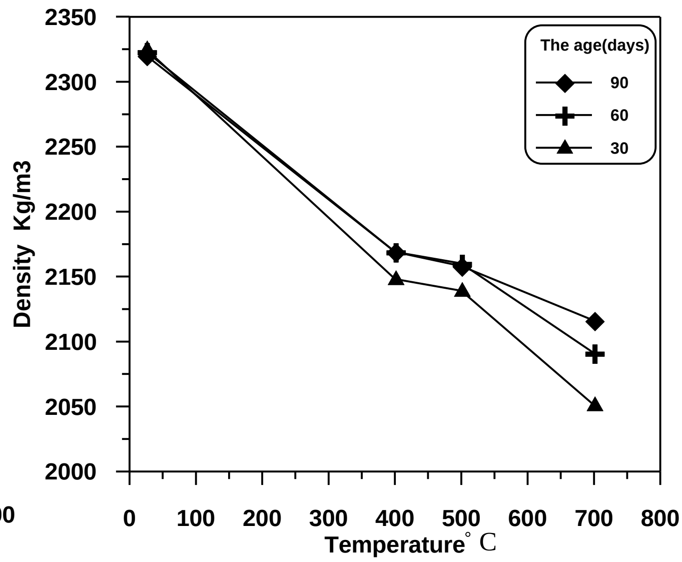


Fig.(4-4) The effect of fire flame on the density of series-A at 2.0 hours period of exposure.

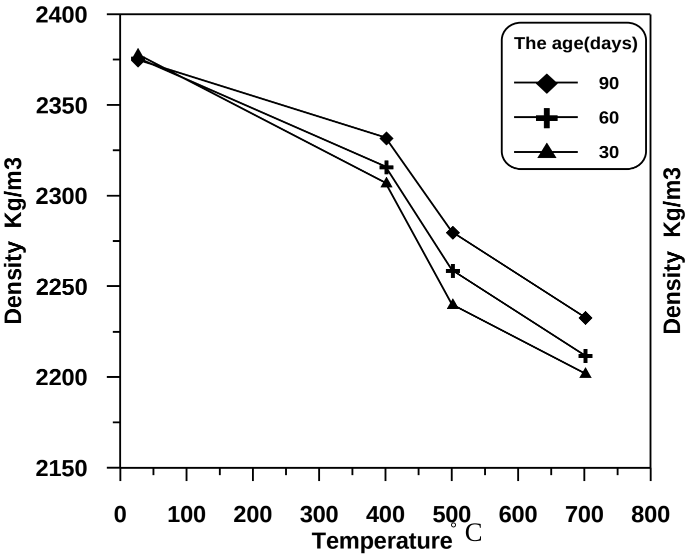


Fig.(4-5) The effect of fire flame on the density of series-B at 0.5 hour period of exposure.

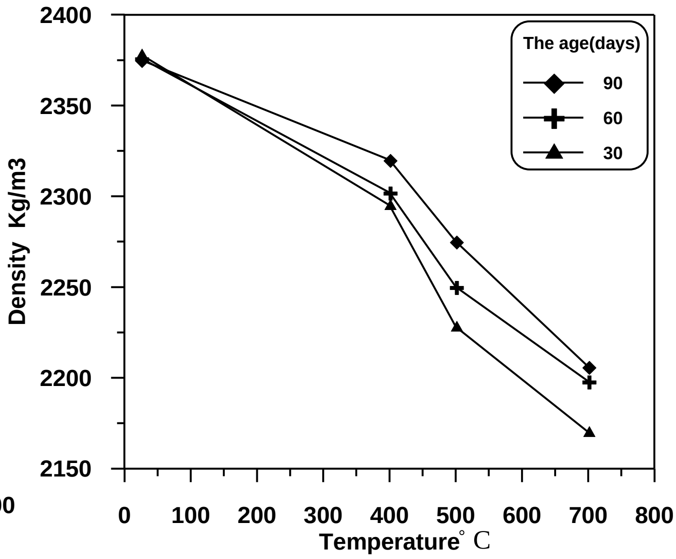


Fig.(4-6) The effect of fire flame on the density of series-B at 1.0 hour peroid of exposure.

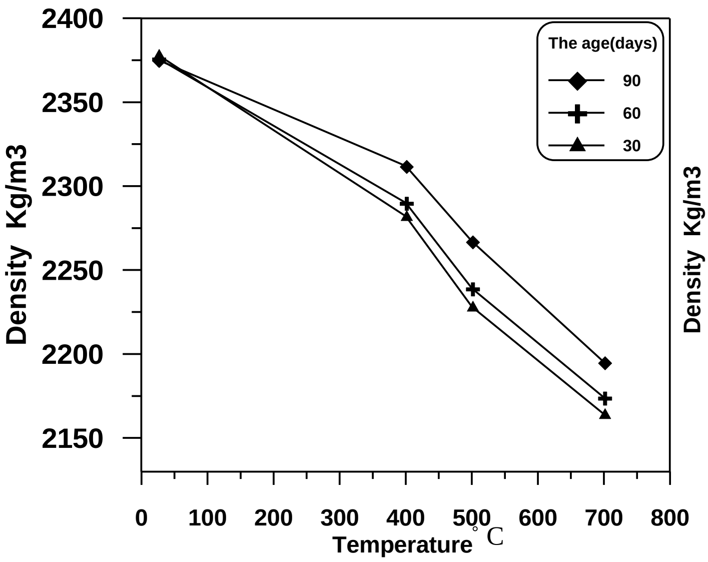


Fig.(4-7) The effect of fire flame on the density of series-B at 1.5 hours period of exposure.

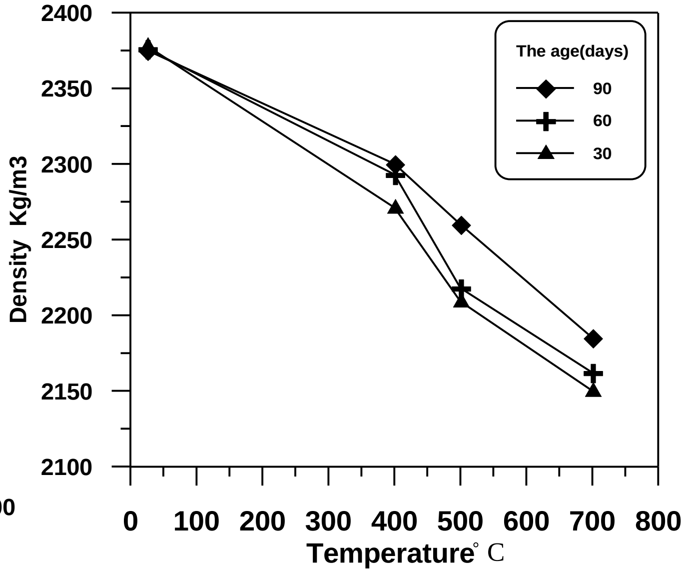


Fig.(4-8) The effect of the fireflame on the density of series -B-at 2.0 hours period of exposure.

٤-٣ Compressive Strength

The compressive strength results are summarized in Tables [٤-٣] and [٤-٤] for both series A and B respectively, while Figures [٤-٩] to [٤-١٦] show the relation between compressive strengths and fire flame temperatures for both series A and B respectively.

Table [٤-٣]: *Test values of compressive strength of concrete specimens of series -A- after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Compressive Strength (MPa) Temperature ºC				Ratios $f_{cu\,a}/f_{ub}$			Type of Cooling
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)	
٣٠	٠.٥	٣٠.٥٠	٢٢.٥٧	٢٠.٤٣	١٦.١٧	٠.٧٣	٠.٦٧	٠.٥٣	Air
			٢١.٣٥	١٨.٩١	١٤.٠٣	٠.٧٠	٠.٦٢	٠.٤٦	Water
	١.٠		٢٢.٥٧	١٩.٢٢	١٤.٣٤	٠.٧٤	٠.٦٣	٠.٤٧	Air
			٢١.٣٥	١٧.٦٩	١٢.٨١	٠.٧٠	٠.٥٨	٠.٤٢	Water
	١.٥		٢١.٦٦	١٨.٣٠	١٣.٧٣	٠.٧١	٠.٦٠	٠.٤٥	Air
			٢٠.١٣	١٦.٤٧	١٠.٩٨	٠.٦٦	٠.٥٤	٠.٣٦	Water
	٢.٠		٢٢.٨٨	١٨.٠٠	١٣.١٢	٠.٧٥	٠.٥٩	٠.٤٣	Air
			٢١.٠٥	١٥.٨٦	١٠.٠٧	٠.٦٩	٠.٥٢	٠.٣٣	Water
٦٠	٠.٥	٣٤.١٢	٢٦.٩٥	٢٤.٢٣	١٨.٧٧	٠.٧٩	٠.٧١	٠.٥٥	Air
			٢٥.٥٩	٢٢.٨٦	١٧.٠٦	٠.٧٥	٠.٦٧	٠.٥٠	Water
	١.٠		٢٥.٩٣	٢٢.٥٢	١٧.٤٠	٠.٧٦	٠.٦٦	٠.٥١	Air
			٢٤.٢٣	٢٠.٨١	١٥.٦٩	٠.٧١	٠.٦١	٠.٤٦	Water
	١.٥		٢٧.٣٠	٢١.١٥	١٦.٣٨	٠.٨٠	٠.٦٢	٠.٤٨	Air
			٢٥.٢٥	١٩.٧٩	١٤.٦٧	٠.٧٤	٠.٥٨	٠.٤٣	Water
	٢.٠		٢٦.٢٧	٢٠.٤٧	١٥.٣٥	٠.٧٧	٠.٦٠	٠.٤٥	Air
			٢٤.٢٣	١٨.٤٢	١٣.٦٥	٠.٧١	٠.٥٤	٠.٤٠	Water
٩٠	٠.٥	٣٧.٣١	٣٠.٩٧	٢٧.٩٨	٢٢.٣٩	٠.٨٣	٠.٧٥	٠.٦٠	Air
			٢٩.٨٥	٢٦.١٢	٢٠.٥٢	٠.٨٠	٠.٧٠	٠.٥٥	Water
	١.٠		٢٩.٨٥	٢٥.٧٤	٢٠.١٥	٠.٨٠	٠.٦٩	٠.٥٤	Air
			٢٨.٣٦	٢٣.٥١	١٧.٩١	٠.٧٦	٠.٦٣	٠.٤٨	Water
	١.٥		٢٩.١٠	٢٣.٨٨	١٨.٦٦	٠.٧٨	٠.٦٤	٠.٥٠	Air
			٢٧.٦١	٢١.٦٤	١٦.٧٩	٠.٧٤	٠.٥٨	٠.٤٥	Water
	٢.٠		٢٩.٤٧	٢٢.٧٦	١٧.٩٠	٠.٧٩	٠.٦١	٠.٤٨	Air
			٢٧.٢٤	٢٠.١٥	١٥.٦٧	٠.٧٣	٠.٥٤	٠.٤٢	Water

f_{cua} = Compressive strength (cube) after exposure to fire flame.
f_{cub} = Compressive strength (cube) before exposure to fire flame.

Table [٤-٤] :*Test values of compressive strength of concrete specimens of series- B- after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Compressive Strength (MPa)				Ratios $f_{cu\,a}/f_{cub}$			Type of Cooling
		Temperature °C							
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)	
٣٠	٠.٥	٤١.٣٠	٢٨.٩١	٢٨.٥٠	٢٣.٩٥	٠.٧٠	٠.٦٩	٠.٥٨	Air
			٢٧.٦٧	٢٦.٨٥	٢١.٤٨	٠.٦٧	٠.٦٥	٠.٥٢	Water
	١.٠		٣٢.٢١	٢٦.٨٥	٢٠.٦٥	٠.٧٨	٠.٦٥	٠.٥٠	Air
			٣٠.٩٨	٢٥.٦١	١٨.١٧	٠.٧٥	٠.٦٢	٠.٤٤	Water
	١.٥		٣٠.٩٨	٢٦.٠٢	١٩.٠٠	٠.٧٥	٠.٦٣	٠.٤٦	Air
			٣٠.٥٦	٢٤.٣٧	١٥.٦٩	٠.٧٤	٠.٥٩	٠.٣٨	Water
	٢.٠		٣١.٣٩	٢٥.١٩	١٧.٧٦	٠.٧٦	٠.٦١	٠.٤٣	Air
			٢٩.٣٢	٢٢.٧٢	١٤.٤٦	٠.٧١	٠.٥٥	٠.٣٥	Water
٦٠	٠.٥	٤٥.١٥	٣٦.٥٧	٣٣.٤١	٢٧.٠٤	٠.٨١	٠.٧٤	٠.٦٠	Air
			٣٤.٧٧	٣١.٦١	٢٤.٣٨	٠.٧٧	٠.٧٠	٠.٥٤	Water
	١.٠		٣٥.٢٢	٣١.٦١	٢٣.٩٣	٠.٧٨	٠.٧٠	٠.٥٣	Air
			٣٣.٤١	٢٩.٧٠	٢١.٦٧	٠.٧٤	٠.٦٦	٠.٤٨	Water
	١.٥		٣٤.٣١	٢٩.٣٥	٢٢.٥٨	٠.٧٦	٠.٦٥	٠.٥٠	Air
			٣٢.٩٦	٢٧.٠٩	٢٠.٣٢	٠.٧٣	٠.٦٠	٠.٤٥	Water
	٢.٠		٣٥.٦٧	٢٧.٩٩	١٩.٨٧	٠.٧٩	٠.٦٢	٠.٤٤	Air
			٣٤.٧٧	٢٦.١٩	١٧.٦١	٠.٧٧	٠.٥٨	٠.٣٩	Water
٩٠	٠.٥	٤٨.٢٠	٤١.٤٥	٣٧.٦٠	٢٩.٨٨	٠.٨٦	٠.٧٨	٠.٦٢	Air
			٣٩.٥٢	٣٦.١٥	٢٦.٩٩	٠.٨٢	٠.٧٥	٠.٥٦	Water
	١.٠		٣٩.٠٤	٣٦.١٥	٢٧.٤٧	٠.٨١	٠.٧٥	٠.٥٧	Air
			٣٧.٦٠	٢٤.٢٢	٢٥.٥٥	٠.٧٨	٠.٧١	٠.٥٣	Water
	١.٥		٣٧.٦٠	٣٣.٧٤	٢٥.٠٦	٠.٧٨	٠.٧٠	٠.٥٢	Air
			٣٥.٦٧	٣١.٣٣	٢٣.١٤	٠.٧٤	٠.٦٥	٠.٤٨	Water
	٢.٠		٣٦.٦٣	٣٠.٨٥	٢٣.١٤	٠.٧٦	٠.٦٤	٠.٤٨	Air
			٣٥.١٩	٢٨.٤٤	٢٠.٢٤	٠.٧٣	٠٥٩	٠.٤٢	Water

f_{cua} = Compressive strength (cube) after exposure to fire flame.
f_{cub} = Compressive strength (cube) before exposure to fire flame.

٤-٣-١ Air- Cooling

The residual compressive strength values were found for the two series exposed to ٤٠٠°C-fire flame temperature and for ages ٣٠, ٦٠,and ٩٠ days and for all periods of exposure. The percentage of the residual compressive strengths was (٧٠ – ٨٣ %) for series A and (٧٠ – ٨٥ %) for series B, for all periods of exposure. Surface cracks of about (١mm) width took place on the concrete specimens. These results agreed with that obtained by other investigators, Neville and Brooks [٣٨] result was ٨٥٪, Collet and Tavernier [٦٧] obtained ٧٥ %, while Abram's [١٠] result was ٨٦ %. Logothetis and Economou [٣٩] result was ٧٣ %. Al–Ausi and Faiyadh [١٦] result was (٧٩–٩٠٪), while Al–Owaisy [٥٤] obtained (٧٦–٧٨ %). The test specimens showed a further loss in compressive strength ranging from (٥٩ – ٧٥ %) and (٦١– ٧٨ %) for mix A and B respectively at ٥٠٠°C fire flame temperature for all ages and all periods of exposure. It is observed that the colour of the concrete specimens changed to pink and increased in intensity. This may be due to hydration conditions of iron oxide component and other mineral constituents of the fine and coarse aggregates [٥ , ٩, ٦٨]. The surface – cracks increased in number, length and depth due to temperature rise. These results of the residual compressive strength at ٥٠٠°C were similar to the results obtained by others [٥,١٠,١٦,٢٨, ٣٩, ٤٥, ٥٤.,]. At ٧٠٠°C of fire flame exposure and for all ages of specimens and periods of exposure, the percentage of the retaining compressive strengths was (٤٣ – ٦٠ %) for series A and (٤٣ – ٦٢ %) for series B, while Logothetis and Economou's [٣٩] result was ٤٣ % at ٧٠٠°C . The difference can be partly explained by the factor which affects the residual strength of heated concrete and this is the amount of moisture driven off during heating, the greater the amount of moisture lost, the lower strength [١٧,٢٨]. Moreover the decrease in compressive strength of concrete is attributed to the break-down of interfacial bond due to incompatible volume change between cement paste and aggregate during heating and cooling [٣ , ١١ , [٦٩] and the formation of relatively week hydration products (dehydration of the calcium – silica hydrate in cement paste).

٤-٣-٢ Water- Cooling

From previous tables and figures, it can be observed that the water – cooled concrete specimens suffer more reduction in strength than the air – cooled specimens. The residual compressive strength after exposure to fire flame and cooling in water was as follows:-

At ٤٠٠ºC the residual compressive strengths compared to the original strength before exposure to fire flame were (٦٦ – ٨٠ %) for series A and (٦٧ – ٨٢ %) for series B . These results conformed to Al – Ausi and Faiyadh result which was (٦٠ – ٧١ %) and Al – Owaisy result that was ٧١ %. Raising the temperature to ٥٠٠ºC caused concrete strength retaining (٥٢– ٧٠ %) from the original strength before burning for series A and (٥٥ – ٧٥ %) for series B. These results were similar to that obtained by Al- Ausi and Faiyadh and Al- Owaisy which were (٣٥ – ٥٣ %), and ٥٨٪ respectively. Further loss in compressive strength took place in specimens burnt to ٧٠٠ºC and then cooled in water. The residual compressive strength ranged between (٣٣ – ٥٥ %) for series A and (٣٥ – ٥٦ %) for series B. The concrete specimens cooled in water exhibited more deterioration than the air–cooled specimens, the further reduction in strength was ranging from (٣-٥٪), (٤-٧٪) and (٥-١٠٪) for series A and (٢-٥٪), (٣-٦ %) and (٥-٨٪) for series B compared with the strength of companion air-cooled specimens. This is because of the destructive thermal shock produced when quenching the hot specimens in water[٧٠] and the penetration of the water into the concrete pores and cracks .The penetration of water into dry concrete pores is known to result in the dilation of the cement gel thereby decreasing the cohesive forces between the cement particles and reducing the strength.[١٦] Also, decomposition of the calcium hydroxide occurs, so that the lime is left behind in consequence of drying. If , however , after cooling, water ingress into concrete takes place the rehydration of the lime can be disruptive [٥].

The test results show that large proportion of the decrease in compressive strength occurs at the first ٠.٥-hour period of exposure. It can be seen that the adverse effect of fire is pronounced on series A more than on series B, while the effect is equal when the period of fire exposure reaches two hours.

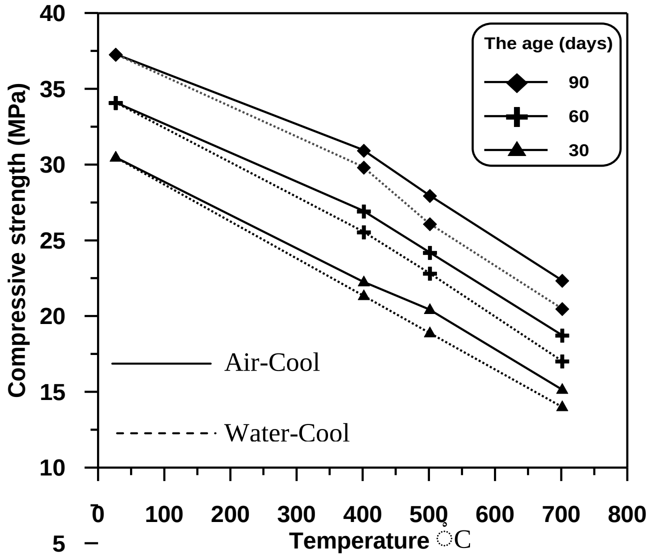


Fig (4-9) The effect of fire flame on the compressive strength of series A at 0.5 hour period of exposure.

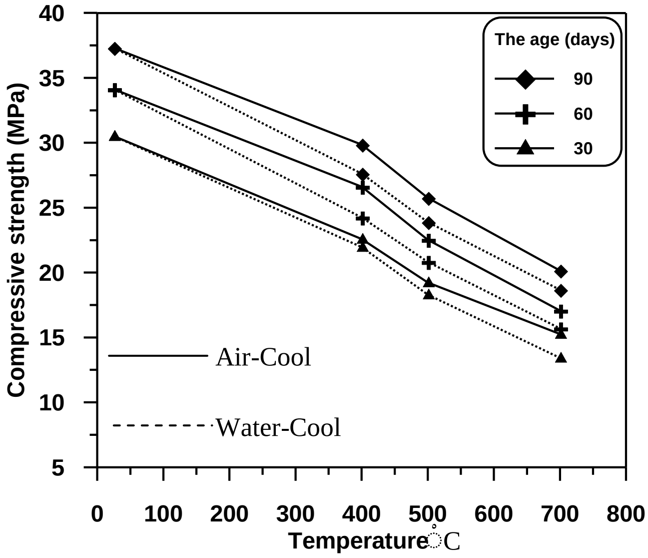


Fig (4-10) The effect of fire flame on the compressive strength of series-A at 1.0 hour period of exposure .

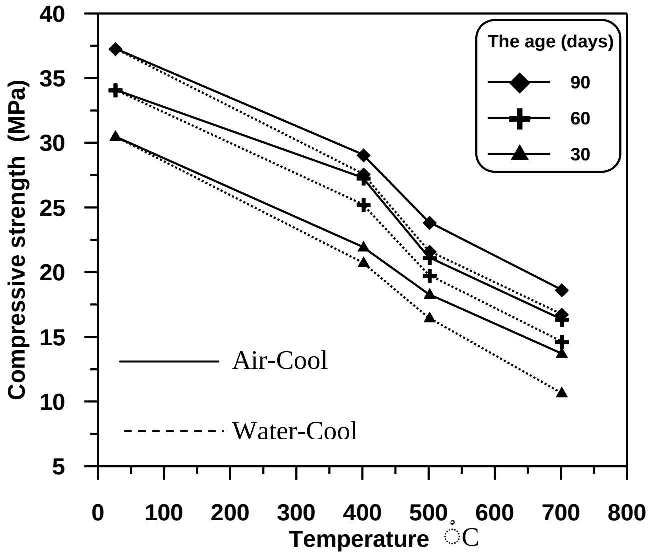


Fig.(4-11) The effect of fire flame on the compressive strength of series-A at 1.5 hours period of exposure.

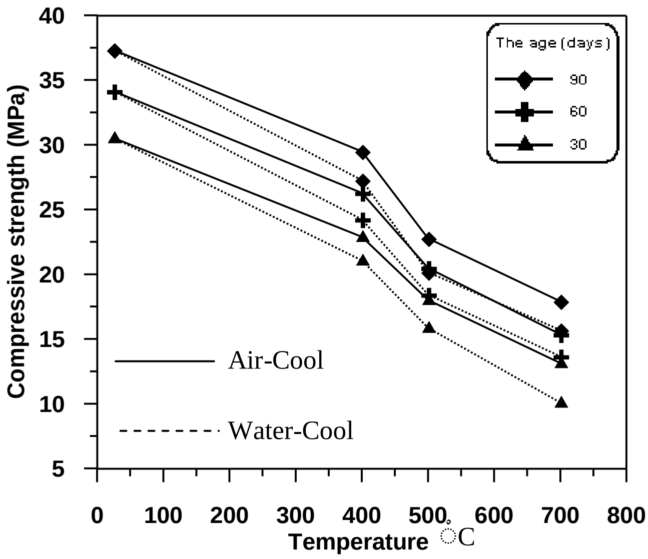


Fig.(3-12) The effect of fire flame on the compressive strength of series - A at 2.0 hours period of exposure.

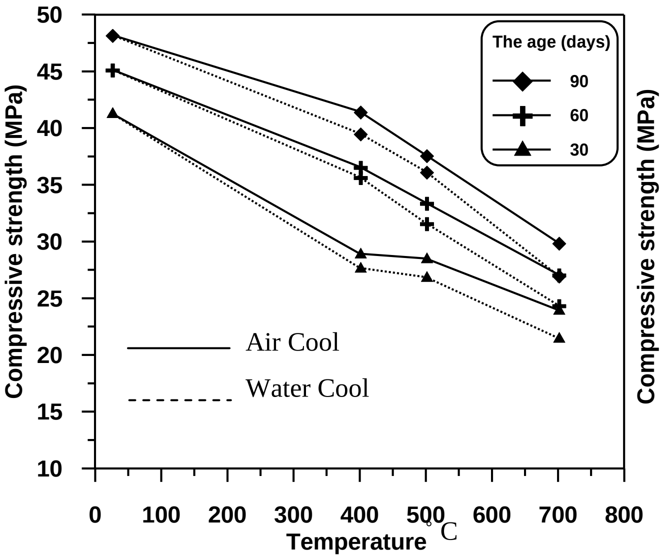


Fig.(4-13) The effect of fire flame on the compressive strength of series-B at 0.5 hour period of exposure.

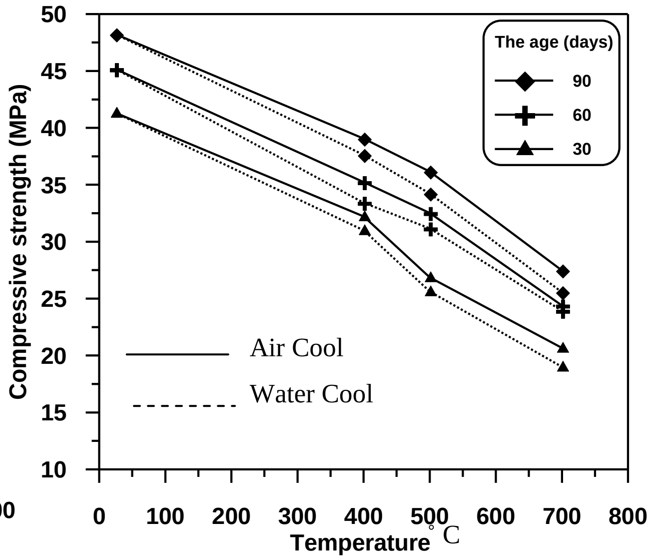


Fig.(4-14) The effect of fire flame on the compressive strength of series-B at 1.0 hour period of exposure.

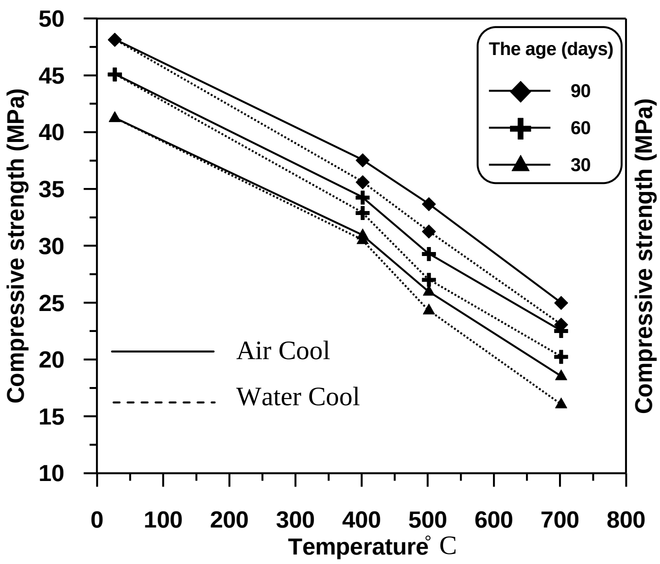


Fig.(4-15) The effect of fire flame on the compressive strength of series-B at 1.5 hours period of exposure.

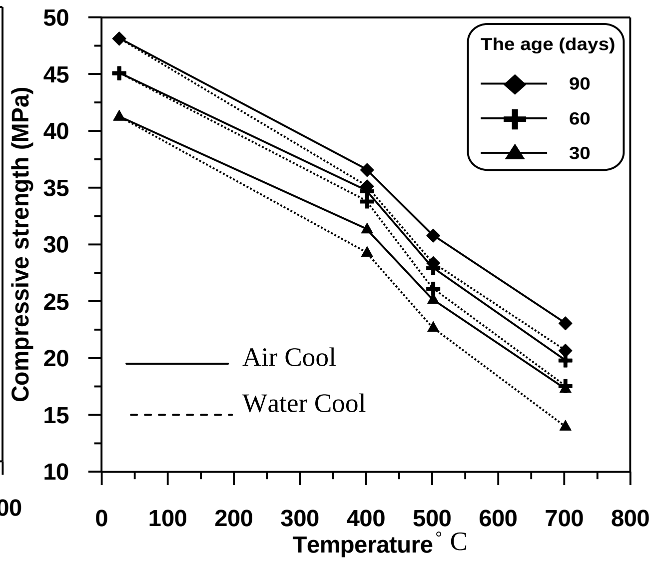


Fig.(4-16) The effect of fire flame on the compressive strength of series-B at 2.0 hours period of exposure.

٤-٣-٣ The effect of the age of concrete at exposure to fire

The test results show that the residual compressive strength after exposure to fire, the reduction at ٣٠ days age was more than the reduction at ٦٠ and ٩٠ days. This may be attributed to the fact that hydration of cement paste is more complete at later ages.

٤-٣-٤ Comparison between residual compressive strength results and recommended design curves (CEB & CEN)

Euro-Code Nos.٢ and ٤ [Committee European de Normalization] (CEN) (١٩٩٣,١٩٩٤), specified rules for strength and deformation properties of uniaxially stressed, normal weight aggregate (NWA) (siliceous and calcareous) concrete at temperatures up to ١٢٠٠ºC.The CEB Bulletin D' Information No.٢٠٨ (CEB) (١٩٩١),[٢٥]recommended design curves for compressive strength of siliceous (NWA) concrete based on the work of many investigators.The residual compressive strength results obtained from tests carried out by many researchers in addition to the present study are plotted with the CEN and CEB design curves as shown in Fig.[٤-١٧].It can be seen that the temperature range between (٢٠٠-٣٠٠˚C)was characterized by a maximum residual strength(Malhotra[٩] Abrams[١٠] ,Collet et al[٦٧],Al-Ausi and Faiyhad[١٦] and Al-Owasiy[٥٤]). Improvement in compressive strength was observed to a maximum value in a temperature ranged between (١٠٠-١٥٠˚C) (Fahmi and Asa'ad , Al-Ausi and Faiyhad and Habeeb). This behavior was attributed to the cement paste response due to causing weakening of bond between (٥٠-١٥٠˚C), thereafter, densification due to thermal drying contributed to the increase of strength.. Beyond ٣٠٠˚C there was a further reduction in compressive strength. In the present study some of test results of residual compressive strength were found to lie between CEB and CEN curves, while other results were far away from the CEB curve and near the CEN curve especially at ٤٠٠˚C and ٥٠٠˚C of fire exposure. At ٧٠٠˚C the test results were found to be near the CEN curve only. From the figure, it can be concluded that the test results of the current study have better agreement with CEN design curve than with CEB curve.

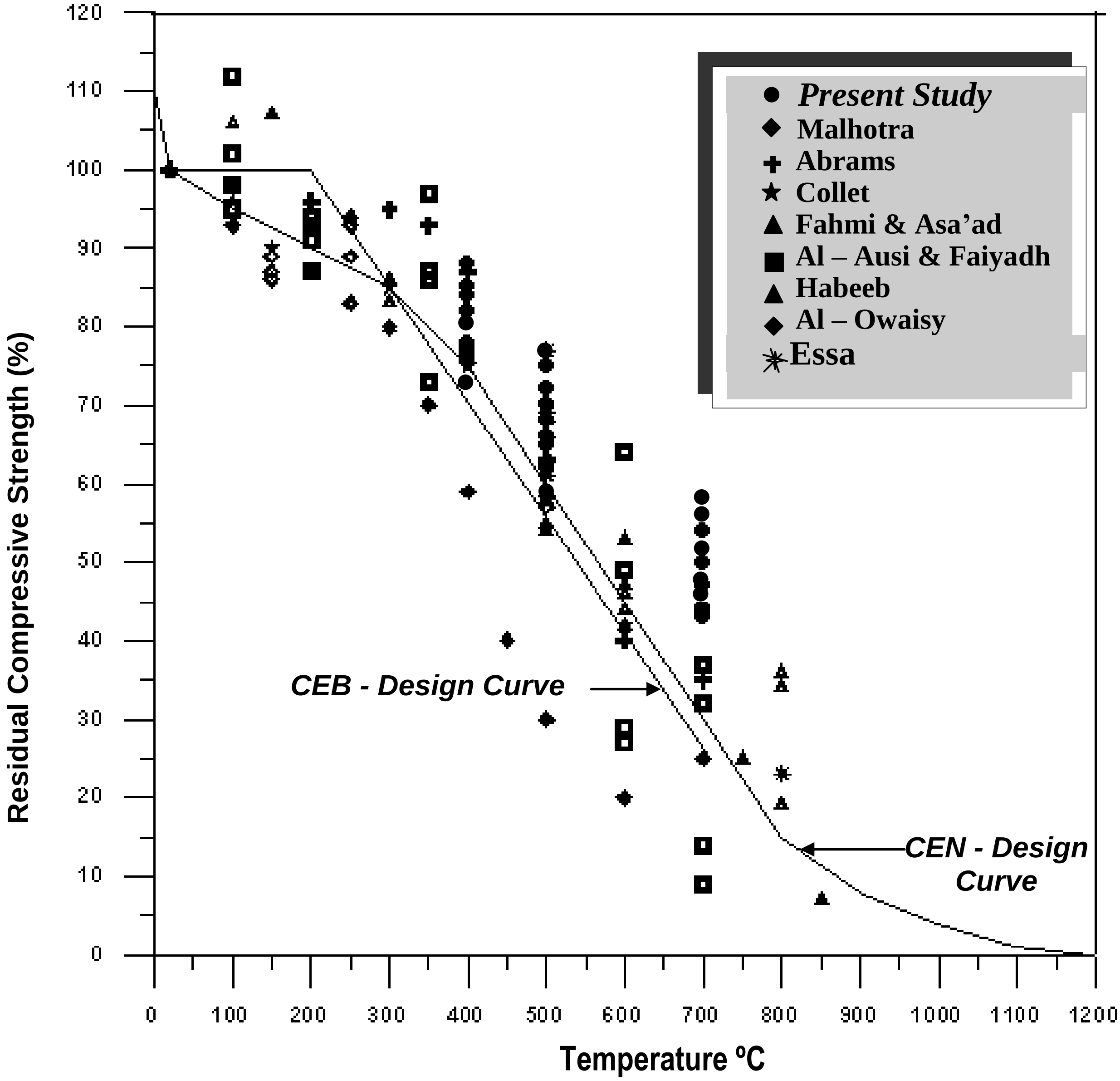


Fig.[٤-١٧]: *Comparison of residual compressive strength results and the recommended design curves of CEB and CEN.*

٤-٤ Splitting Tensile Strength.

The values of splitting tensile strength of the specimens considered in the present investigation are abstracted in Tables [٤-٥] and [٤-٦]. The relation between the splitting tensile strength and fire flame exposure temperature is shown in Figures [٤-١٨] to [٤-٢١] for series A and (٢-٢٢) to (٤-٢٥) for series B. It is clear from the test results that the splitting tensile strength is more sensitive to the exposure to high temperatures than the concrete compressive strength. It can be observed that concrete deteriorates at a faster rate when tested in tension rather than in compression. This observation confirms the results obtained by other investigators [٣,٩.١٦].

Table [٤- ٥]: *Test values of splitting tensile strengths of concrete specimens of series- A-after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Splitting Tensile Strength (MPa) Temperature °C				Ratios f_{sa}/f_{sb}			Type of Cooling
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)	
٣٠	٠.٥	٣.٤٣	٢.٤٧	١.٦٥	١.١٠	٠.٧٢	٠.٤٨	٠.٣٢	Air
			٢.٣٣	١.٢٠	٠.٧٢	٠.٦٨	٠.٣٥	٠.٢١	Water
	١.٠		٢.٤٠	١.٦٠	٠.٨٩	٠.٧٠	٠.٤٧	٠.٢٦	Air
			٢.٢٣	١.٣٧	٠.٦٢	٠.٦٥	٠.٤٠	٠.١٨	Water
	١.٥		٢.٤٧	١.٧٢	٠.٧٩	٠.٧٢	٠.٥٠	٠.٢٣	Air
			٢.٢٦	١.٤٤	٠.٣٨	٠.٦٦	٠.٤٢	٠.١١	Water
	٢.٠		٢.٣٠	١.٦١	٠.٦٢	٠.٦٧	٠.٤٧	٠.٢١	Air
			٢.٠٢	١.٢٧	٠.٣٤	٠.٥٩	٠.٣٧	٠.١٠	Water
٦٠	٠.٥	٣.٨٢	٢.٨٧	٢.٠٦	١.٣٠	٠.٧٥	٠.٥٤	٠.٣٤	Air
			٢.٦٧	١.٩٥	١.٠٧	٠.٧٠	٠.٥١	٠.٢٨	Water
	١.٠		٢.٧١	١.٩٩	١.١١	٠.٧١	٠.٥٢	٠.٢٩	Air
			٢.٥٢	١.٦٨	٠.٨٤	٠.٦٦	٠.٤٤	٠.٢٢	Water
	١.٥		٢.٨٣	٢.٠٦	١.١٥	٠.٧٤	٠.٥٤	٠.٣٠	Air
			٢.٦٤	١.٨٠	٠.٨٤	٠.٦٩	٠.٤٧	٠.٢٢	Water
	٢.٠		٢.٧٢	١.٨٣	٠.٩٩	٠.٧١	٠.٤٨	٠.٢٥	Air
			٢.٤١	١.٦٤	٠.٧٧	٠.٦٣	٠.٤٣	٠.٢٠	Water
٩٠	٠.٥	٤.٠٥	٣.٠٨	٢.٢٧	٢.٣٥	٠.٧٦	٠.٥٨	٠.٤١	Air
			٢.٩٢	١.٩٤	٢.١١	٠.٧٢	٠.٥٢	٠.٣٣	Water
	١.٠		٢.٩٦	٢.٣١	١.٥٨	٠.٧٣	٠.٥٧	٠.٣٩	Air
			٢.٧٥	٢.٠٠	١.٢٢	٠.٦٨	٠.٤٩	٠.٣٠	Water
	١.٥		٣.٠٤	٢.٤٣	١.٥٤	٠.٧٥	٠.٦٠	٠.٣٨	Air
			٢.٨٨	٢.١٥	١.١٧	٠.٧١	٠.٥٣	٠.٢٩	Water
	٢.٠		٢.٨٣	٢.١٩	١.٣٨	٠.٧٠	٠.٥٤	٠.٣٤	Air
			٢.٧٩	١.٨٦	١.٠٥	٠.٦٩	٠.٤٦	٠.٢٦	Water

f_{sa} = Splitting tensile strength after exposure to fire flame.
f_{sb} = Splitting tensile strength before exposure to fire flame.

Table[٤- ٦]: *Test values of splitting tensile strengths of concrete specimens of series- B-after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Splitting Tensile Strength (MPa) Temperature ºC				Ratios f_{sa}/f_{sb}			Type of Cooling
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)	
٣٠	٠.٥	٤.٢٤	٣.١٠	٢.٢٩	١.٤٠	٠.٧٣	٠.٥٤	٠.٣٣	Air
			٢.٩٣	٢.٠٤	٠.٩٨	٠.٦٩	٠.٤٨	٠.٢٣	Water
	١.٠		٣.٠٥	٢.٠٨	١.١٥	٠.٧٢	٠.٤٩	٠.٢٧	Air
			٢.٨٨	١.٧٨	٠.٧٦	٠.٦٨	٠.٤٢	٠.١٨	Water
	١.٥		٣.١٤	١.٨٢	٠.٩٣	٠.٧٤	٠.٤٣	٠.٢٢	Air
			٣.٠١	١.٥٧	٠٣٨	٠.٧١	٠.٣٧	٠.٠٩	Water
	٢.٠		٢.٩٣	١.٧٠	٠.٨٥	٠.٦٩	٠.٤٠	٠.٢٠	Air
			٢.٦٧	١.٤٠	٠.٤٢	٠.٦٣	٠.٣٣	٠.١٠	Water
٦٠	٠.٥	٤.٥٠	٣.٣٦	٢.٧٠	١.٦٢	٠.٧٥	٠.٦٠	٠.٣٦	Air
			٣.٢٠	٢.٣٤	١.٤٤	٠.٧١	٠.٥٢	٠.٣٢	Water
	١.٠		٣.٠٦	٢..٥٢	١.٥٣	٠.٦٨	٠.٥٦	٠.٣٤	Air
			٢.٧٩	٢.٢٥	١.١٧	٠.٦٢	٠.٥٠	٠.٢٦	Water
	١.٥		٣.٣٦	٢.٤٣	١.٣٥	٠.٧٥	٠.٥٤	٠.٣٠	Air
			٣.٢٠	٢.٢١	٠.٩٩	٠.٧١	٠.٤٩	٠.٢٢	Water
	٢.٠		٣.٢٤	٢.٢٥	١.٠٨	٠.٧٢	٠.٥٠	٠.٢٤	Air
			٣.٠٢	١.٩٨	٠.٨٥	٠.٦٧	٠.٤٤	٠.١٩	Water
٩٠	٠.٥	٤.٨٦	٣.٦٩	٣.٢٦	٢.١٩	٠.٧٦	٠.٦٧	٠.٤٥	Air
			٣.٥٠	٢.٧٣	١.٩٠	٠.٧٢	٠.٥٦	٠.٣٩	Water
	١.٠		٣.٧٩	٢.٧٢	١.٩٤	٠.٧٨	٠.٥٦	٠.٤٠	Air
			٣.٥٥	٢.٤٨	١.٥٤	٠.٧٣	٠.٥٢	٠.٣٣	Water
	١.٥		٣.٦٠	٢.٥٣	١.٨٠	٠.٧٤	٠.٥٢	٠.٣٧	Air
			٣.٣٥	٢.٢٨	١.٤١	٠.٦٩	٠.٤٧	٠.٢٩	Water
	٢.٠		٣.٥٥	٢.٣٨	١.٤٦	٠.٧٣	٠.٤٩	٠.٣٠	Air
			٣.٣٠	٢.٠٤	١.١٢	٠.٦٨	٠.٤٣	٠.٢٣	Water

f_{sa} = Splitting tensile strength after exposure to fire flame.
f_{sb} =Splitting tensile strength before exposure to fire flame.

٤-٤-١ Cooling by Air

١- At ٤٠٠ºC fire flame exposure temperature, for all ages and for all periods of exposure, the residual splitting tensile strengths were in order of (٦٧– ٧٦٪) for series A and (٦٩ – ٧٦ %) for series B. The test carried out by Al-Ausi and Faiyadh [٧١] showed that the residual splitting tensile strength was ٦٧ %, whereas Al –Owaisy [٥٤] investigation was ٦٢ % ,also Takeuchi's[٧٢] result was ٦٥٪. These results were obtained at the same temperature and the same type of cooling

٢- Further decrease in splitting tensile strength ranged between (٤٧ – ٥٨ %) for series A and (٤٠ – ٦٧ %) for series B at ٥٠٠ºC.These results agreed with Al-Ausi and Faiyadh, Carrete and Painter[١٤] and Hidayet's[٧٣] results obtained at the same temperature and the same cooling regime.

٣- At ٧٠٠ºC fire flame temperature the effect on splitting tensile strength was more severe than on the compressive strength. The residual tensile strengths were (٢٠ – ٤٥٪) for series A and (٢١- ٤٥ %) for series B .The reduction in the splitting tensile strength can be attributed to the formation of tensile stresses during the contraction of the hardened cement paste upon cooling, which, when superimposed onto the already existing tensile stresses formed during heating would cause an increase in the amount and rate of crack formation[٦٩].

٤-٤-٢ Cooling by Water

Cooling by water causes further reduction in the strength. The residual splitting tensile strengths after exposure to fire flame and quenched in water ranged between (٥٩ – ٧٢ %) for series A and (٦٣ – ٧٢ %) for series B at ٤٠٠ºC fire flame temperature.

At ٥٠٠ºC fire flame caused an extra reduction in the splitting tensile strengths ranging from (٣٧ – ٥٢ %) for series A and (٣٣ – ٥٦ %) for series B.At ٧٠٠ºC for all ages of specimens and periods of exposure, the residual splitting tensile strength was (١٠ – ٣٣ %) for series A and (٩– ٣٩ %) for mix B compared with the original tensile strength before exposure to fire. Cooling in water caused further reduction in splitting strength compared with companion specimens cooled in air and the difference ranged from (٤-٨٪), (٦-١٠٪) and (٦-١٢٪) for series A and (٣-٦ %), (٤-٧٪) and (٥-١٤٪) for series B at ٤٠٠ºC, ٥٠٠ºC and ٧٠٠ºC respectively. The reduction in the strength was mainly due to fast penetration of water into the pores and microcracks of concrete. The penetration of water into the dry concrete pores is known as a result of dilation of cement gel thereby decreasing the cohesive forces between the cement particles and reducing strength [٧٤, ٧٥].It is also possible that the different rate of cooling between the surface and the inner part of the specimens causes an increase in the amount and volume of cracks thus lowering the tensile strength [٧٠].

Similar trend was obtained for the splitting tensile strength effect by fire flame as in the compressive strength but a slight decrease in series B than series A was observed at exposure period of ٢.٠ hours.

٤-٤-٣ Effect of the age at heating

As in compressive strength the effect of the fire flame at ٣٠ days of age was more than at ٦٠ and ٩٠ days of ages. This is also due to further completion of hydration of the cement at later age.

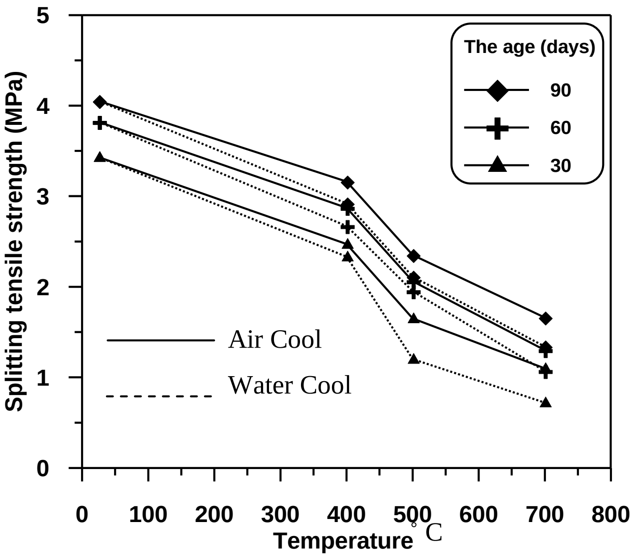


Fig.(4-18) The effect of fire flame on the splitting tensile strength of series-A at 0.5 hour period of exposure.

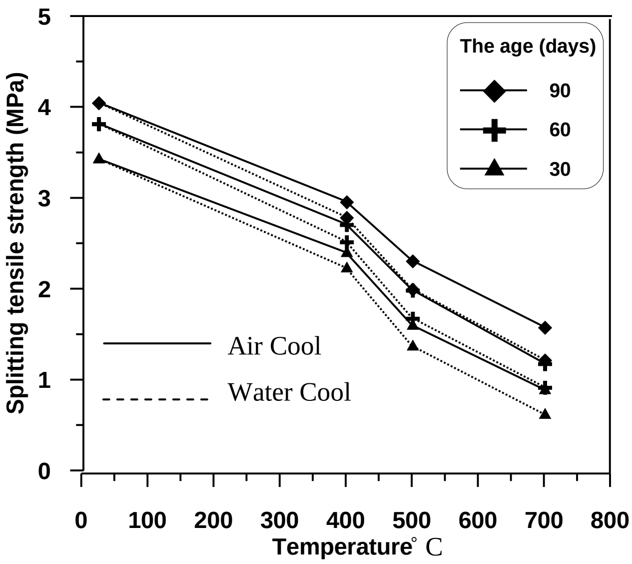


Fig.(4-19) The effect of fire flame on the splitting tensile strength of series-A at 1.0 hour period of exposure.

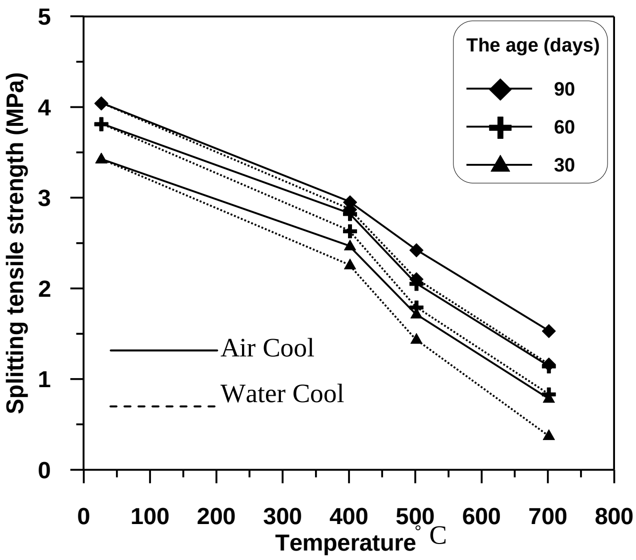


Fig.(4-20) The effect of fire flame on the splitting tensile strength of series A at 1.5 hours period of exposure.

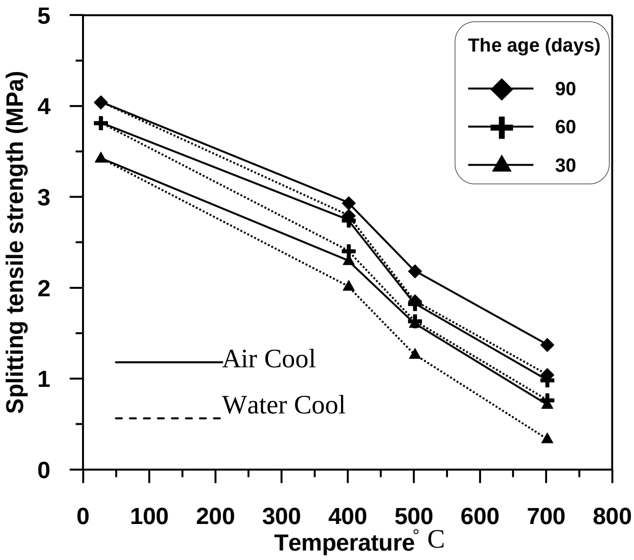


Fig.(4-21) The effect of fire flame on the splitting tensile strength of series A at 2.0 hours period of exposure.

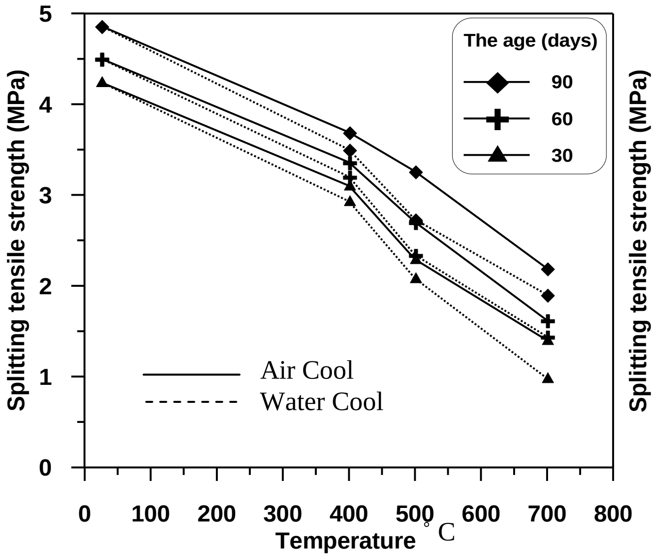


Fig (4-22) The effect of fire flame on the splitte tensile strength of series-B at 0.5 hour period of exposure.

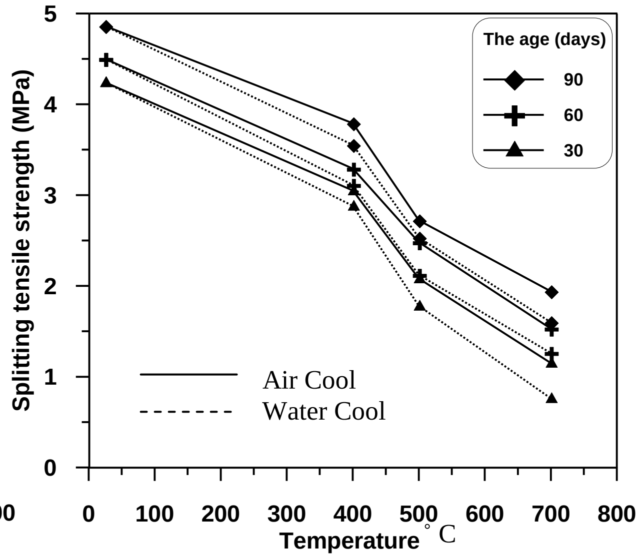
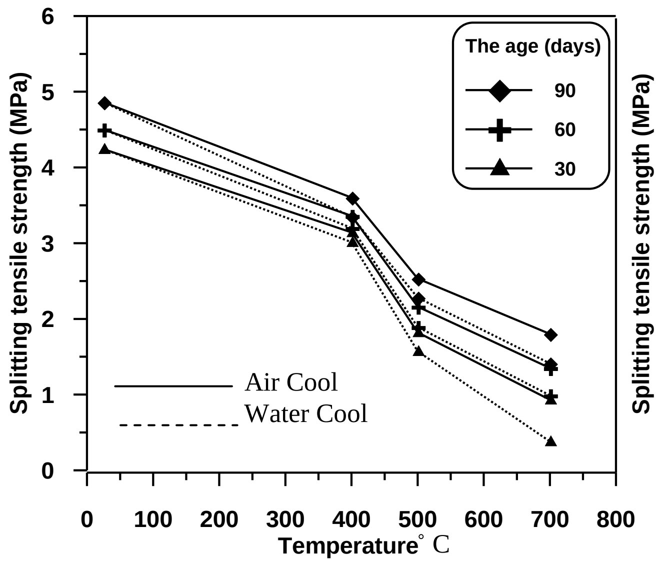


Fig.(4-23) The effect of fire flame on the splitting tensile strength of series-B at 1.0 hour period of exposure.



Fig(4-24) The effect of fire flame on the splitting tensile strength of series-B at 1.5 hours period of exposure.

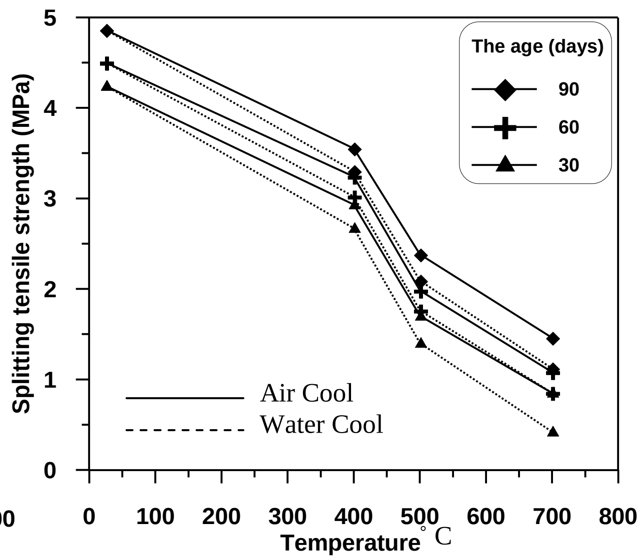


Fig.(4-25) The effect of fire flame on the splitting tensile strength of series-B at 2.0 hours period of exposure.

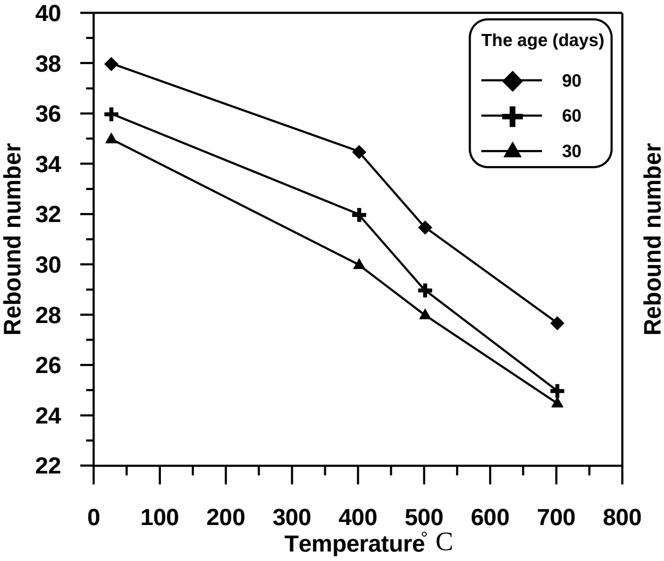


Fig.(4-54) The effect of fire flame on the rebound number of series-B- at 0.5 hour period of exposure.

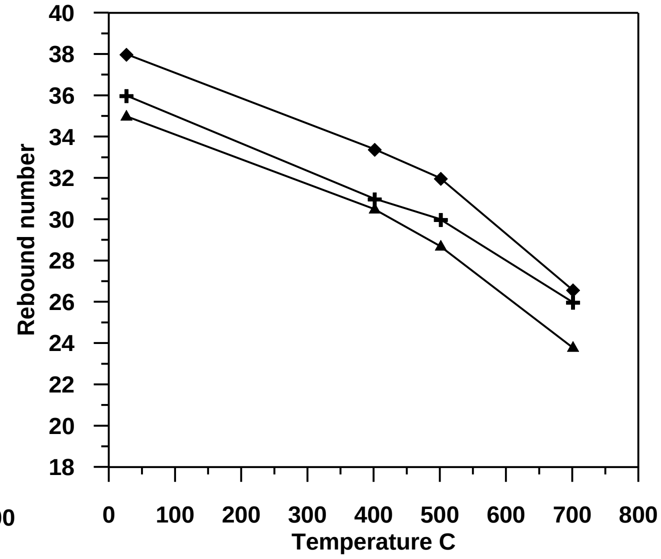


Fig.(4-55) The effect of fire flame on the rebound number of series-B- at 1.0 hour period of exposure

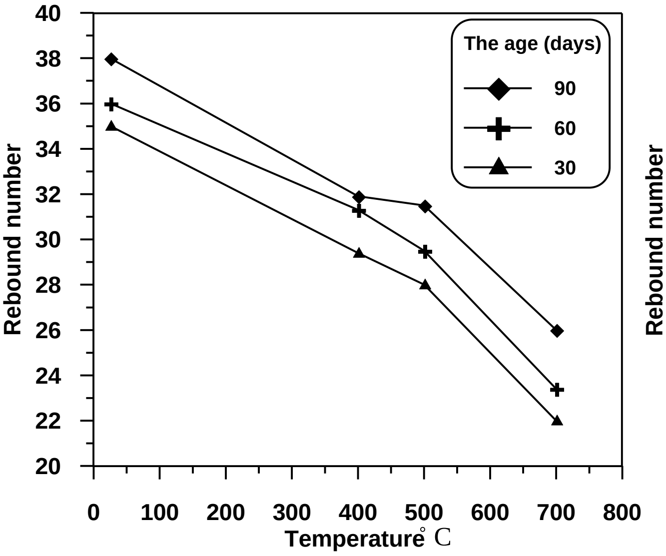


Fig.(4-56) The effect of fire flame on the rebound number of series -B- at1.5 hours period of exposure.

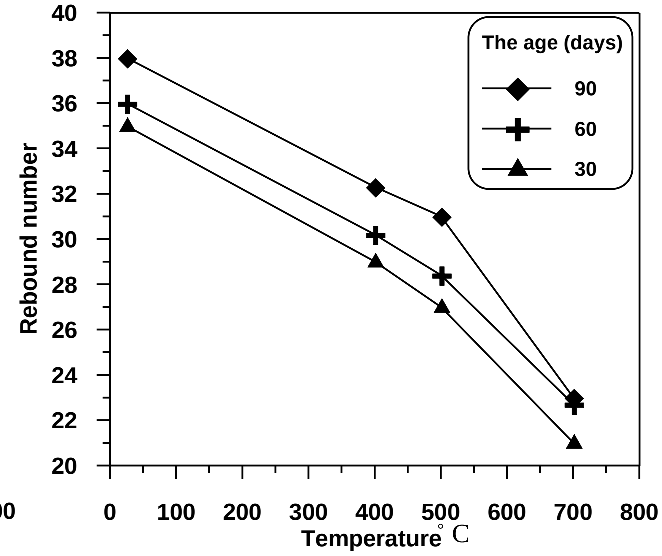


Fig.(4-57) The effect of fire flame on the rebound number of series -B- at 2.0 hours period of exposure.

٤-٩ Mathematical models for prediction of mechanical properties of concrete after exposure to fire flame

In order to obtain useful mathematical relationships, that yield good prediction accuracy, nonlinear regression is used for this purpose, due to its efficiency in derivation of exponential equations, which are extremely useful for fitting experimental data with more than one independent variable [٢٨,٧٩, ٨٠]. The exponential equation used was of the following general form:

$$Y = a_0 \cdot x_1^{a1} \cdot x_2^{a2} \cdot x_3^{a3} \ldots\ldots\ldots x_n^{n} \qquad \text{------------------ (٤-١)}$$

where:

Y = Dependent variable.

$x_1, x_2 \ldots\ldots\ldots x_n$ =Independent variables.

$a_0, a_1, a_2 \ldots\ldots a_n$ =Constants.

The mechanical properties of concrete are assumed to be known before exposure to fire flame, such as initial compressive strength , temperature of fire flame , flexural strength , ultrasonic pulse velocity , rebound number, ages, period of exposure and density . Independent variables were ranged in these equations according to their descending degree of significance.

٤-٩-١ Models for prediction of compressive strength (f_{cua})

To find the regression for prediction of (f_{cua}) .Equation (٤-١) can be written as follows :

$$f_{cua} = a_0 (f_{cub})^{a1} (T)^{a2} (U \cdot P \cdot V)^{a3} (R_N)^{a4} (\rho)^{a5} (Pe)^{a6} (Age)^{a7} \qquad \text{-------- (٤-٢)}$$

where

f_{cua} = Compressive strength of the specimens after exposure to fire flame temperature (MPa).

f_{cub} = Compressive strength of the specimens before exposure to fire flame temperature (MPa).

T = Temperature of fire flame (ºC).

U.P.V = Ultrasonic pulse velocity (km/sec)

R_N = Rebound number after exposure to fire flame.

ρ = Density of concrete after exposure to fire flame (kg/m٣) .

P_e = The period of exposure to fire flame (hour).

Age = The age of the specimens at the time of exposure (days)

٤-٩-٢ Models for prediction of splitting tensile strength f_{sa} .

The regression for prediction of f_{sa} can be written in form as follow:

$$f_{sa} = a_0 (f_{sb})^{a1} (T)^{a2} (Pe)^{a3} (Age)^{a4} (f_{cub})^{a5} \text{------(٤-٣)}$$

where :

fsa = Splitting tensile strength of specimens after exposure to fire flame (MPa) .

T = Temperature of fire flame (ºC).

P_e = Period of fire flame exposure (hours).

Age = The age of the specimens at the time of exposure (days)

f_{cub} = Compressive strength of the specimens before exposure to fire flames (MPa).

٤-٩-٣ Models for prediction of flexural strength (f_{ra}).

The regression for prediction of f_{ra} can be written in form as follows:

$$f_{ra} = a_0 (f_{rb})^{a1} (T)^{a2} (Pe)^{a3} (Age)^{a4} (f_{cub})^{a5} \text{--------(٤-٤)}$$

where:

f_{ra} = Modulus of rupture of specimens after exposure to fire flame (MPa).

f_{rb} = Modulus of rupture of specimens before exposure to fire flame(MPa).

T = Temperature of fire flame (ºC).

P_e = The period of exposure to fire flame (hour).

Age = The age of the specimen at exposure to fire flame (days).

f_{cub} = Compressive strength of specimens before exposure to fire flame (MPa) .

٤-٩-٤ Models for prediction of static modulus of elasticity (E_c).

The regression for prediction of (E_c) can be written as follows:

$$E_{ca} = a_0 (E_{cb})^{a1} (T)^{a2} (Pe)^{a3} (Age)^{a4} (f_{cub})^{a5} \text{-------.(٤-٥)}$$

where:

E_{ca} = Static modulus of elasticity after exposure to fire flame (GPa).

E_{cb} = Static modulus of elasticity before exposure to fire flame (GPa).

T = Temperature of fire flame (°C).

P_e = The period of exposure to fire flame (hours)

Age = The age of specimen at the time of exposure to fire flame .

f_{cub} = Compressive strength of concrete specimens before exposure to fire flame (MPa) .

Table (٤-١٥) gives the values of the constants ($a_{٠}$, $a_{١}$, $a_{٢}$... a_n) for the regressions for f_{cua} , f_{sa}, f_{ra} and E_{ca} . This table also gives the values of the coefficients of correlation. From the values of the coefficients of correlation obtained, it can be concluded that these regressions are statically significant, for the size of data investigated of ٧٢, each with ٤-٧ independent variables.

***Table [٤-١٥]** : Coefficient of exponential regressions for the prediction of:*

١-f_{cua}

Regr. No.	a٠	a١ (f_{cub})	a٢ (T)	a٣ (U.P.V)	a٤ (R_N)	a٥ (P_e)	a٦ (Age)	a٧ (ρ)	C.C*
١	٧.٢١٧١	٠.٨٣٢٧	-٠.٣٢٨٦	٠.١٦٧٧	٠.٥٢٢	-٠.٢٣٢	-٠.٠٥٧	٠.٠٤٤	٠.٩٦٧
٢	٦٠٢.٤٩	٠.٩٠١٨	-٠.٢٨٦٤	٠.١٩١٢	٠.٦٧٩	-٠.٩٢١	-٠.٠٥٧	-	٠.٩٦٤
٣	٠.١٠٩٥	٠.٦٣٣٢	-٠.٠٢٠٥	٠.٢٩٢٥	٠.٨٦٣	٠.٠١٩١	-	-	٠.٩٥٦
٤	٠.١٢٥٨	٠.٦٣٥٣	-٠.٠٢٠٨	٠.٢٩٣٢	٠.٨٦٣	-	-	-	٠.٩٥٠
٥	٢.٣٩٥٧	١.٠١٠٨	-٠.٢٧٥٠	٠.٤٠٣١	-	-	-	-	٠.٩٣٨
٦	٣٢.٠٤٢	١.١٨٨١	-٠.٧٣٦٠	-	-	-	-	-	٠.٩١٢
٧	٣٢.٢٦٧	١.١٠٠٣	-٠.٧٢٨٤	-	-	-	-٠.٠٩٠	٠.٦٨٥	٠.٩٥٨
٨	٣٠.٥٦٠	١.١٩٠٧	-٠.٧٢٨٨	-	-	-	-٠.٠٨٩	-	٠.٩٤٧
٩	٣٣.٧٩٧	١.٠٩٩٠	-٠.٧٣٥٧	-	-	-	-	٠.٠٦٧	٠.٩٢٣

٢-f_{sa}

Regr.	a٠	a١	a٢	a٣	a٤	a٥	C.C*

No.		(f_{sa})	(T)	(P_e)	(Age)	(f_{cub})	
١	٠.٠٠٩٤١	٠.٦٨٦٢	-١.٢٩٣٤	-٠.٠٣٤٣	٠.١٥٢٠	١.٥٤٣٣	٠.٩٣٧١
٢	١٨٧٤.٧٧	١.٠٧١٧	-١.٤١٠٨	-٠.٠٥١٦	٠.١١٧٠	-	٠.٩٣٥٦
٣	٢١٩٥.٨٣	١.٣٢٦٣	-١.٤١٨٣	-٠.٥٢١٠	-	-	٠.٩٢٠٠
٤	٢٣١٦.٧٨	١.٣١٩٣	-١.٤٢٦٩	-			٠.٩١٣٦
٥	١٩٧٤.٤٣	١.٠٦٣٧	-١.٤١٨٠	-	٠.١١٧٥	-	٠.٩٢٩٣

٣- f_{ra}

Regr. No.	a٠	a١ (f_{ra})	a٢ (T)	a٣ (P_e)	a٤ (Age)	a٥ (f_{cub})	C.C*
١	٤٦٠٣.٠٢	٠.٩٦٩٠	-١.٥٤٢٧	-٠.٠٨٢	٠.٠٥٢٨	٠.٩٤٦٥	٠.٩٦٣٢
٢	٥٤٣٢.٠٦	١.١٠٤٢	-١.٥٤٢٨	-٠.٠٨٣	٠.٠٤٧٩	-	٠.٩٦٣١
٣	٥٦٧٣.٦٨	١.٢٠٨٨	-١.٥٤٣٩	-٠.٠٨٣	-	-	٠.٩٦٠٨
٤	٥٦٧٣.٦٨	١.١٠٣٥	-١.٥٥٢١	-	٠.٠٤٧٩	-	٠.٩٤٧٠
٥	٥٩٨٠.٠٥	١.٢٠٧٩	-١.٥٥٣٥	-	-	-	٠.٩٤٥٠

٤- E_{ca}

Regr. No.	a٠	a١ (E_{cb})	a٢ (T)	a٣ (P_e)	a٤ (Age)	a٥ (f_{cub})	C.C*
١	٦٣٦.٩٤٩	٢.٩٠٨	-١.٨٢٢	-٠.١٤٠	٠.٠٤٧٥	٠.٧٤٥١	٠.٩٥٧٨
٢	٨٤١٧.٧٠	١.٣٦٠	-١.٨٢٤	-٠.١٤٠	٠.٠٥٤٢	-	٠.٩٥٧٩

٣	٦٣٢٣.٣٧	١.٥٠٩	-١.٨٢٥	-٠.١٤٠	-	-	٠.٩٦٤٩
٤	٩٢٩٧.٨١	١.٣٤٨	-١.٨٣٥	-	٠.٠٥٤٦	-	٠.٩١٩٧
٥	٦٩٧٥.٠٢	١.٤٩٧	-١.٨٣٦	-	-	-	٠.٩١٨٠

* C.C … Coefficient of Correlation.

From Table [٤-١٥], the following equations could be used to estimate the corresponding values with good coefficient correlation and fewer variables introduced in them.

$$f_{cua} = 2.394(f_{cub})^{1.011}(T)^{-0.275}(U.P.V)^{0.403} \qquad \text{---------.(٤-٦)}$$

$$f_{sa} = 1974.32(f_{sb})^{1.064}(T)^{-1.418}(Age)^{0.118} \qquad \text{---------(٤-٧)}$$

$$f_{ra} = 5980.05(f_{rb})^{1.208}(T)^{-1.554} \qquad \text{---------(٤-٨)}$$

$$E_{ca} = 6975.02(E_{cb})^{1.497}(T)^{-1.837} \qquad \text{---------(٤-٩)}$$

٤–١٠ Load – Deflection Behavior

٤–١٠–١ Introduction

The deflection which occurs immediately upon application of the load is called short-term deflection or instantaneous deflection. The principal factors that influence this type of deflection are the magnitude and distribution of the load, span and conditions of end restraint, section properties including steel percentage and material properties. (٨١)

To study the influence of fire flame on the immediate deflection, twelve rectangular beams of ١٠٠-mm width, ١٥٠-mm depth and ١٠٠٠-mm length were tested in flexure. Two series of beams were cast with same mix proportion of series A and B. Two control beams were tested for each series without exposure to fire, the others were subjected to fire flame at the age of ٣٠ and ٩٠ days with temperature level of ٥٠٠°C and ٧٠٠°C for ١ hour period of exposure. The properties of the reinforced concrete beams are shown in Table [٤-١٦].

٤-١٠-٢ Ultimate Moment Strength

One-point load on simply supported beams gives only one section of maximum stresses, this occurs under the point load as shown in Plate [٤-١].

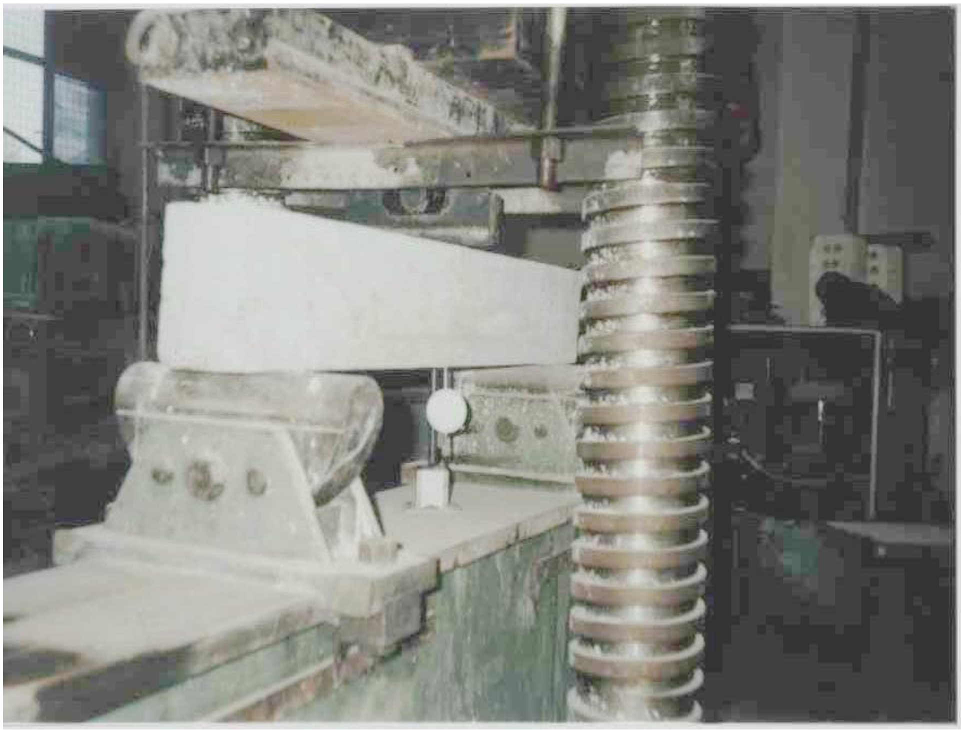


Plate [٤-١]: *Beam under testing*

The theoretical ultimate strengths of beams which were calculated according to ACI-Code [٦٦] by using Equation (٤-١١) and according to CP١١٠ [٨٢] by using Equation (٤-١٦) are shown in Table (٤-١٧).

The ACI- Code nominal moment equation is:-

$$M_n = \rho f_y b d^2 \left(1 - 0.59\rho \frac{f_y}{f_c}\right) \qquad \text{-----------------.(٤-١٠)}$$

The ultimate moment M_u (for design) is :

$M_u = \phi M_n$, $\phi = 0.90$

$$M_u = \phi \rho b d^2 \left(1 - 0.59\rho \frac{f_y}{f_c}\right) \qquad \text{-----------------(٤-١١)}$$

where $f_c = 0.85 f_{cu}$

The CP-١١٠ ultimate moment equations are: -

$$M_u = 0.87 f_y As Z \qquad \text{-----------------(٤-١٢)}$$

$$Z = \left[1 - \frac{1.1 f_y As}{fcu.bd}\right] d \qquad \text{----------------- (٤-١٣)}$$

By eliminating the safety factors,

$$Z = d - \frac{f_y As}{1.34 f_{cu} b} \qquad \text{------------------(٤-١٤)}$$

$$M_n = f_y As Z \qquad \text{-----------------(٤-١٥)}$$

$$M_u = 0.87 f_y As \left[d - \frac{f_y As}{1.34 f_{cu} b}\right] \qquad \text{-----------------.(٤-١٦)}$$

Table [٤-١٧]: *Measured and calculated ultimate moment strengths*

Series	Beam No.	Temp (˚ C)	Age (days)	Experimental M_u (kN.m)	Calculated M_u (kN.m)	Ratio (١)/(٢)	Ratio (١)/(٣)

				(١)	ACI (٢)	CP١١٠ (٣)		
A	A١	٢٥	٣٠	١١.٥٢	٦.٨٠	٦.٤٨	١.٦٩	١.٧٨
	A٤	٢٥	٩٠	١١.٩٩	٦.٩٣	٦.٧٠	١.٧٥	١.٨١
B	B١	٢٥	٣٠	١٢.٦٩	٧.٠٨	٦.٧٩	١.٧٥	١.٨٣
	B٤	٢٥	٩٠	١٣.٧٤	٧.٢١	٦.٩١	١.٩١	١.٩٩

It can be noticed that the ACI Code method predicts the values of ultimate moment strengths very close to CP١١٠ method. But both methods underestimate the actual ultimate moment strength of beam (for conservative design). The ratio of experimental to calculated ultimate moment of the beam ranged from (١.٦٩) to (١.٩١) with an average value of (١.٧٨) when the calculations were based on ACI- Code equations and ranged from (١.٧٨) to (١.٩٩) with an average value of (١.٨٥) when the calculations were based on CP١١٠ equations. The calculated theoretical values are lower than the experimental values due to the considerable residual capacity of the section beyond reaching concrete and steel to yielding,

٤-١٠-٣ Deflection

Single load was applied at midspan because of the limitation of the machine available, which did not permit the application of two-point load to get pure constant moment region. The deflection were recorded at each stage of loading, the load at the first visible crack and at failure were recorded.

٤-١٠-٣-١ Deflection before exposure to fire flame

The failure load of the reference beams (A١, A٤, B١ and B٤) was divided by (١.٦) to calculate the service load. A comparison of the measured and computed midspan deflection of the beams at service load according to ACI Code and BS code are shown in Table [٤-١٨]. An example of the calculation of deflection is given in Appendix B.

Table [٤-١٨]: *Measured and calculated service load deflection at mid-span of the beam.*

Series	Beam No.	Temp. (° C)	Service Load (kN)	Experimental deflection (mm) (١)	Calculated deflection (mm)		Ratio (١)/(٢)	Ratio (١)/(٣)
					ACI (٢)	CP١١٠ (٣)		
A	A١	٢٥	٣٢.٠٠	١.٩٠	١.٨٢	١.٧٠	١.٠٤	١.١٢
	A٤	٢٥	٣٣.٦٣	١.٨٢	١.٩١	١.٨٠	٠.٩٥	١.٠١
B	B١	٢٥	٣٤.٤٥	١.٧٩	١.٩٠	١.٧٩	٠.٩٠	١.٠٠
	B٤	٢٥	٣٨.١٩	١.٧٣	٢.٠٥	١.٩٤	٠.٨٤	٠.٨٩

The ratio of the measured deflection to the calculated service load deflection of the beams ranged from (١.٠٤) to (٠.٨٤) with an average value of (٠.٩٣) when the calculations were based on ACI- Code and ranged from (١.١٢) to (٠.٨٩) with an average value of (١.٠) when the calculations were based on CP١١٠–Code. It can be concluded that CP١١٠ procedure gives comparable results to the measured values for all beams. This behavior was due to the fact that as the load increases, the section of the beam become closer to the partially cracked section used in CP١١٠ calculations.

٤-١٠-٣-٢ The effect of fire flame on the reinforcing steel bars

The effect of fire flame on the properties of reinforcing steel bar was summarized in Table [٤-١٩]. At ٥٠٠˚C, both burning and subsequent cooling did not affect the mechanical properties of the reinforcing steel bars, but the effect was observed at ٧٠٠˚C. The residual in the yield stress was (٨٣٪) and in the ultimate stress (٦٤.٢٪) for the bar of ٨ mm diameter .But for the bar of ١٠ mm diameter the residual in yield stress was (٨٤.٥٪) and the ultimate stress was (٨٥.٨٪). The modulus of elasticity was not affected by burning and cooling at all range of temperatures. Similar behavior was observed by others [١٢, ١٥] .

Table [٤- ١٩]: *The effect of fire flame on properties of steel bars*

Bar diameter (mm)	Exposure Temp. (˚C)	Yield stress *N/mm ٢*	Residual Stress %	Ultimate Stress *N/mm ٢*	Residual Stress %	Modulus of elasticity E_s	Residual %
٨.٠١	٢٥	٤٦٠	١٠٠	٦٦٣	١٠٠	٢١٠	١٠٠
	٥٠٠	٤٦٠	١٠٠	٦٦٣	١٠٠	٢١٠	١٠٠
	٧٠٠	٣٨١.٨	٨٣	٥٥٨.٢	٨٤.٢	٢١٠	١٠٠
١٠	٢٥	٤٧٠.٣	١٠٠	٦٩٠.٥	١٠٠	٢١٠	١٠٠
	٥٠٠	٤٧٠.٣	١٠٠	٦٩٠.٥	١٠٠	٢١٠	١٠٠

	٧٠٠	٣٩٧.٤	٨٤.٥	٥٩٢.٤	٨٥.٨	٢١٠	١٠٠

٤-١٠-٣-٣ Deflection after burning by fire flame

The test results were summarized in Table [٤-٢٠] and the relation between the load and deflection were illustrated in figures from (٤-٥٨) to (٤-٦١). After the beams were subjected to fire flame, two types of cracks developed, the first was thermal cracks appearing in a honeycomb fashion all over the surface. They originated from top or bottoms edges and terminated near the mid-depth of the beam. The crack width was about (١-mm). The patterns of fine cracks were consistent with the release of moisture being greater in the outer layers than in the interior resulting in differential shrinkage. The second cracks were flexural tensile cracking due to loading developed in the mid-span region.

At ٥٠٠ °C, (a)- for ٣٠ days age and ١.٠ hour period of exposure, first crack and yield stress in steel were observed at load (١١.١٪) and (٨٦.o%) from the ultimate load respectively for beam A٢ and (١١.٦٪) and (٨٧.٦٪) from the ultimate load for beam B٢. (b)- for ٩٠ days age and ١.٠ hour period of exposure, first crack and yield stress in steel were observed at load (١٥.٥٪) and (٨٣.٤٪) from the ultimate load for beam A٥ and (١١.٠٪) and (٨٨.٤٪) for the beam B٥.

At ٧٠٠ °C, (a)- for ٣٠ days age and ١.٠ hour period of exposure, first crack and yield stress in steel were noticed at load (٨.٥٪) and (٨٧.١٪) from the ultimate load for beam A٣ and (٩.٥٢٪) and (٨٨.٣٪) from the ultimate load for beam B٣. (b)- for ٩٠ days age and ١.٠ hour period of exposure, first crack and yield stress in steel were observed at load (٩.٥٧٪) and (٨٧.٣٪) from the ultimate load for beam A٦ and (٨.٧٢٪) and (٨٩.٠٪) for the beam B٦.

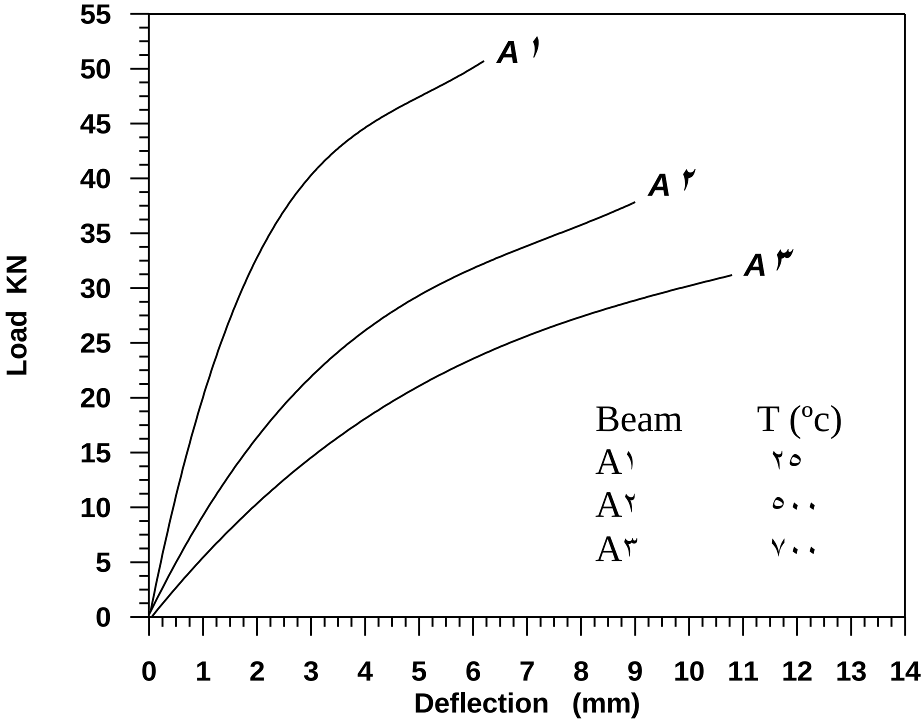


Fig [٤-٥٨] : *Load-deflection behavior for the beams of series A-٣٠ days after exposure to fire flame at ١.٠ hour*

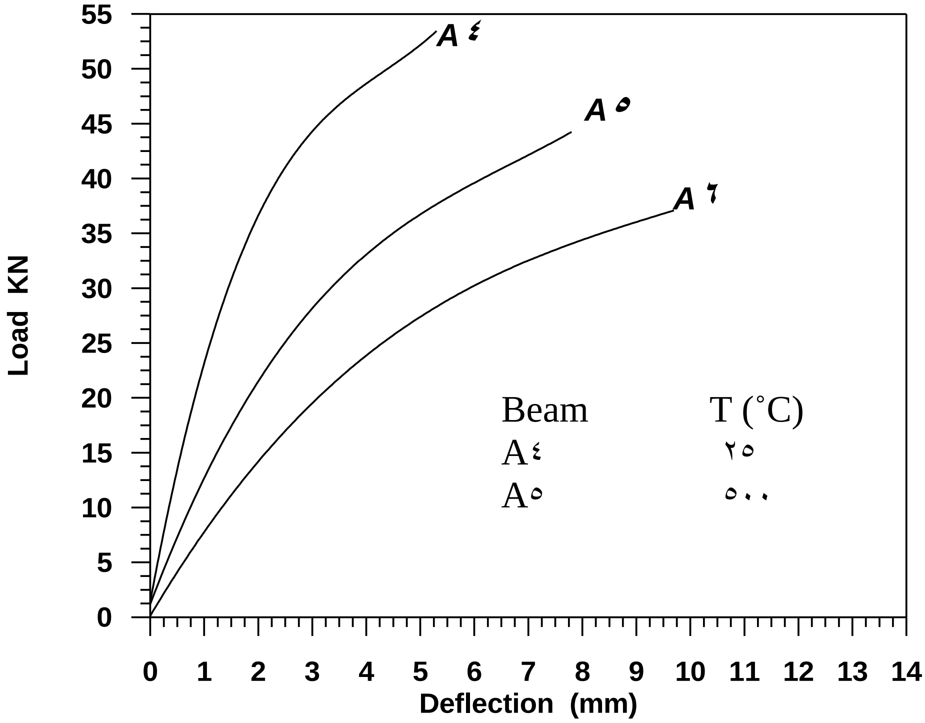


Fig [٤-٥٩]: *Load-deflection behavior for the beams of series A-٩٠ days after exposure to fire flame at ١.٠ hour*

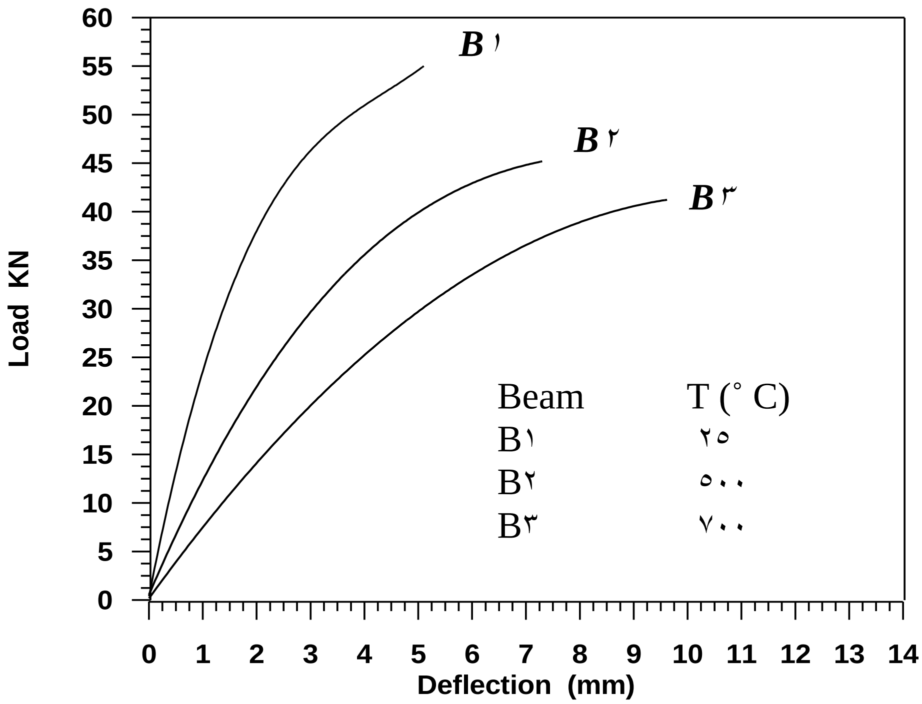


Fig [٤-٦٠] : *load-deflection behavior for the beams of series B-٣٠ days after exposure to fire flame at ١.٠* hour

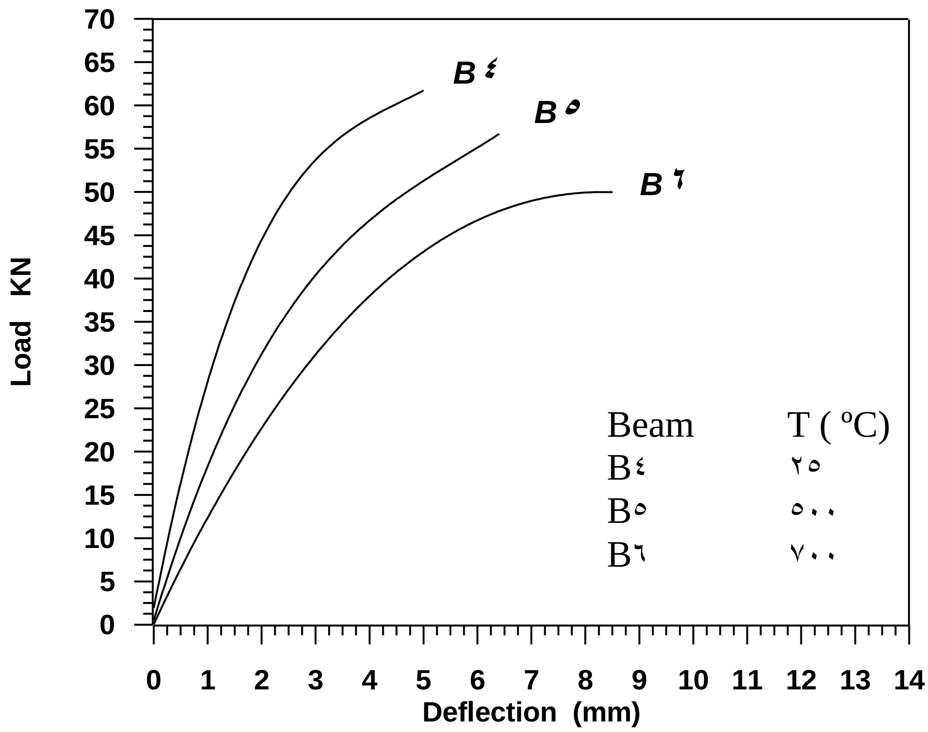


Fig [٤-٦١]: *Load-deflection behavior for the beams of series B-٩٠ days after exposure to fire flame at ١.٠ hour*

The ability to predict the residual strength of a member following a fire is essential in the process of establishing whether and to what extent a fire damaged structure could be reinstated . A procedure for this purpose was applied to the beams tested in this work and the results are given in Table [٤-٢١]. This procedure is similar to that proposed by Malhotra [٨٣] and used by Khan and Royles[٥٠] for fire resistance computations.

On the basis of a limiting deflection an experimental value for the reduced moment of resistance of a test beam after a particular fire exposure was determined from the load-deflection data, Figures (٤-٥٨) to (٤-٦١) as now explained.

١- The ultimate load, Pu and its associated deflection , Δ_u for the unexposed condition , T_f =٢٥ °C were found.

٢- Using the curve appropriate to the fire exposure, a load, P', corresponding to Δ_u was established.

٣- The reduced moment of resistance , M', was evaluated from,

A typical calculation is given in an example in Appendix C. The experimental and predicted values for the reduced moment resistance are compared in Table [٤-٢١] .The results of the unexposed beams to fire agreed with the results of Khan and Royles but differed when the beams exposed to high temperatures because they used different reduction factors

Table [٤-٢١]: *Comparison of experimental and theoretical reduced moment of resistance*(٥٠)

Series	*Beam* No.	*Age* (days)	Temp. (˚C)	Moment (kN. m)	
				Experimental	*Theoretical*
A	A ١	٣٠	٢٥	١١.٥٢	٩.٤٨
	A ٢	٣٠	٥٠٠	٧.٢٠	٨.٤٧
	A٣	٣٠	٧٠٠	٥.٤٠	٥.٥٤
	A٤	٩٠	٢٥	١١.٩٠	١٠.٩٣
	A٥	٩٠	٥٠٠	٨.٣٣	٨.٤٧
	A٦	٩٠	٧٠٠	٤.٢٨	٦.٨٤
B	B١	٣٠	٢٥	١٢.٣٩	١١.٦٨
	B٢	٣٠	٥٠٠	١٠.٢٤	٨.٧٩
	B٣	٣٠	٧٠٠	٩.٢٤	٦.٨٨
	B٤	٩٠	٢٥	١٣.٧٤	١٣.٣٣
	B٥	٩٠	٥٠٠	١٢.٦٩	١٠.٧٧
	B٦	٩٠	٧٠٠	١١.٢٥	٨.٣٠

٤-٥ Modulus of Rupture

The test results of flexural strength of the two series are gives in Tables [٤-٧] and [٤-٨]. Figures [٤-٢٦] to [٤-٢٩] and [٤-٣٠] to [٤-٣٣] show the relationship between residual flexural strength and the fire flame temperatures for series A and B respectively. The residual flexural strength (f_{ra}) after burning at different temperatures is expressed as a ratio f_{ra} / f_{rb} where f_{rb} is the flexural strength of concrete at the same age without burning.

Table [٤-٧]: *Test values of the modulus of rupture of concrete specimens of series-A- after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Modulus of Rupture (MPa) Temperature ºC				Ratios f_{ra}/f_{rb}		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٣.٧١	٢.٧٥	١.٨٢	١.٢٦	٠.٧٤	٠.٤٩	٠.٣٤
	١.٠		٢.٦٧	٢.٠٠	١.١٥	٠.٧٢	٠.٥٤	٠.٣١
	١.٥		٢.٧٨	١.٧١	٠.٩٦	٠.٧٥	٠.٤٦	٠.٢٦
	٢.٠		٢.٦٣	١.٥٦	٠.٨٥	٠.٧١	٠.٤٢	٠.٢٣
٦٠	٠.٥	٤.١٠	٣.١٦	٢.٢١	١.٥٢	٠.٧٦	٠.٥٤	٠.٣٧
	١.٠		٣.٠٣	٢.٠٩	١.٣٥	٠.٧٤	٠.٥١	٠.٣٣
	١.٥		٣.٠٨	٢.٠١	١.١٥	٠.٧٥	٠.٤٩	٠.٢٨
	٢.٠		٢.٩٩	١.٨٥	١.٠٧	٠.٧٣	٠.٤٥	٠.٢٦
٩٠	٠.٥	٤.٢٨	٣.٣٠	٢.٤٨	١.٦٣	٠.٧٧	٠.٥٨	٠.٣٨
	١.٠		٣.٢٥	٢.٤٠	١.٣٧	٠.٧٦	٠.٥٦	٠.٣٢
	١.٥		٣.١٧	١.٩٥	١.٣٣	٠.٧٤	٠.٤٨	٠.٣١
	٢.٠		٣.٠٨	٢.٠١	١.٤١	٠.٧٢	٠.٤٧	٠.٣٣

f_{ra} = Modulus of rupture after exposure to fire flame
f_{rb} = Modulus of rupture before exposure to fire flame

Table [٤- ٨]: *Test values of the modulus of rupture of concrete specimens of series-B- after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Modulus of Rupture (MPa) Temperature ºC				Ratios f_{ra} / f_{rb}		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٤.٤٢	٣.١٤	٢.٣٤	١.٥٧	٠.٧١	٠.٥٣	٠.٣٧
	١.٠		٣.١٨	٢.٢١	١.٥٠	٠.٧٢	٠.٥٠	٠.٣٤
	١.٥		٣.٣٦	٢.٠٨	١.٢٤	٠.٧٦	٠.٤٧	٠.٢٨
	٢.٠		٣.١٤	١.٩٠	٠.٩٧	٠.٧١	٠.٤٣	٠.٢٢
٦٠	٠.٥	٤.٩٥	٣.٦١	٢.٧٧	١.٩٨	٠.٧٣	٠.٥٦	٠.٤٠
	١.٠		٣.٥١	٢.٦٧	١.٦٣	٠.٧١	٠.٥٤	٠.٣٣
	١.٥		٣.٧٦	٢.٥٢	١.٤٩	٠.٧٦	٠.٥١	٠.٣٠
	٢.٠		٣.٦١	٢.٢٨	١.٢٤	٠.٧٣	٠.٤٦	٠.٢٥
٩٠	٠.٥	٥.٢٥	٤.١٠	٣.١٥	٢.١٥	٠.٧٨	٠.٦٠	٠.٤١
	١.٠		٤.١٥	٣.٠٥	١.٨٩	٠.٧٩	٠.٥٨	٠.٣٦
	١.٥		٣.٩٩	٢.٧٨	١.٥٢	٠.٧٦	٠.٥٣	٠.٢٩
	٢.٠		٣.٨٩	٢.٦٣	١.٦٨	٠.٧٤	٠.٥٠	٠.٣٢

f_{ra} = Modulus of Rupture after exposure to fire flame
f_{rb} = Modulus of Rupture before exposure to fire flame

At ٤٠٠ºC the residual flexural strengths were in the range of (٧١ – ٧٧ %) for series A, while series B showed a residual flexural strength of order (٧١–٧٩ %).

At ٥٠٠ºC the specimens exhibited a loss of flexural strength, the residual flexural strengths were (٤٢ – ٥٨ %) for series A and (٤٨- ٦٠ %) for series B. Hair-cracks and pink color were observed in the prisms. This trend is similar to that obtained by Habeeb. [٢٨] A closer result was found by Purkiss, [٤٦] that was ٤٠٪ from the original value.

At ٧٠٠ºCthe residual flexural strengths were (٢٣ – ٣٨ %) and (٢٢ – ٤١ %) for series A and series B respectively. The effect of the fire was seen to be in a range less than that obtained by Asa'ad [٥١] and Purkiss [٤٦] . These results may are attributed to less homogeneity of the heat in stoves than the heat in the furnace especially at temperature more than ٥٠٠°C.

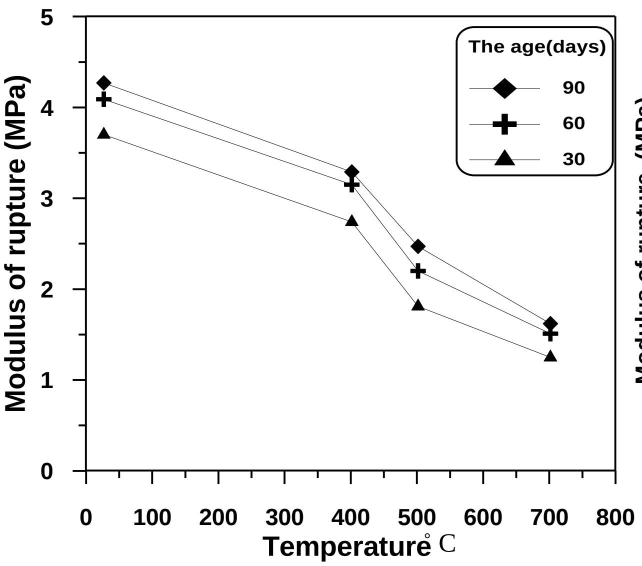


Fig.(4-26) The effect of fire flame on the modulus of rupture of series-A at 0.5 hour period of exposure.

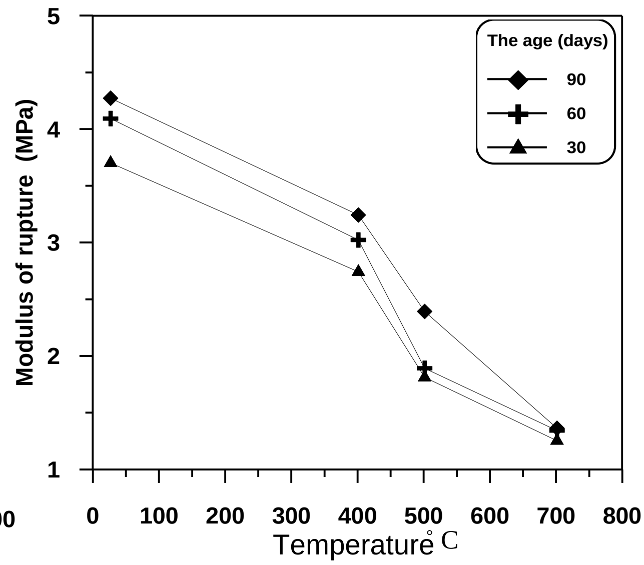


Fig.(4-27) The effect of fire flame on the modulusof rupture of series -A at 1.0 hour period of exposure

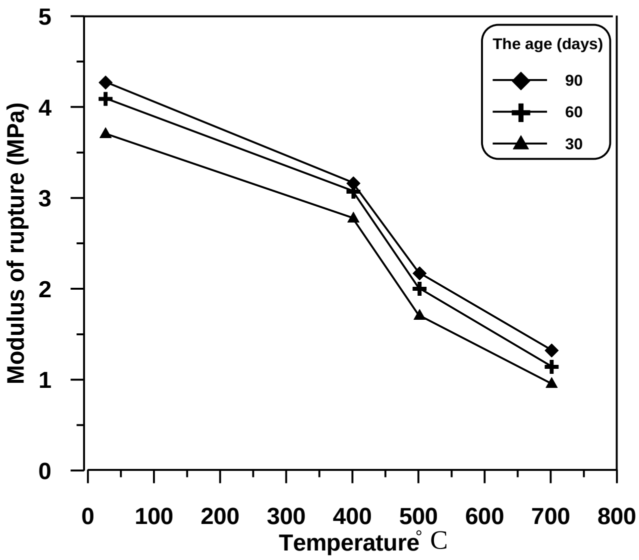


Fig (4-28) The effect of fire flame on the modulus of rupture of series -A at 1.5 hours period of exposure.

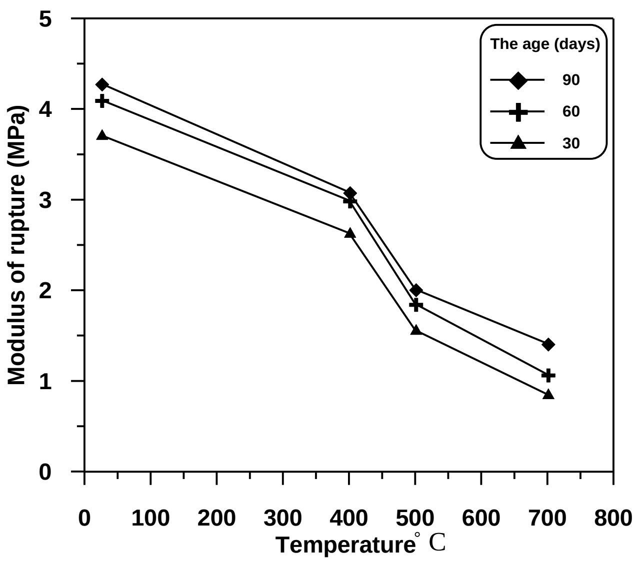


Fig.(4-29) The effect of fire flame on the modulus of rupture of series-A at 2.0 hours period of exposure.

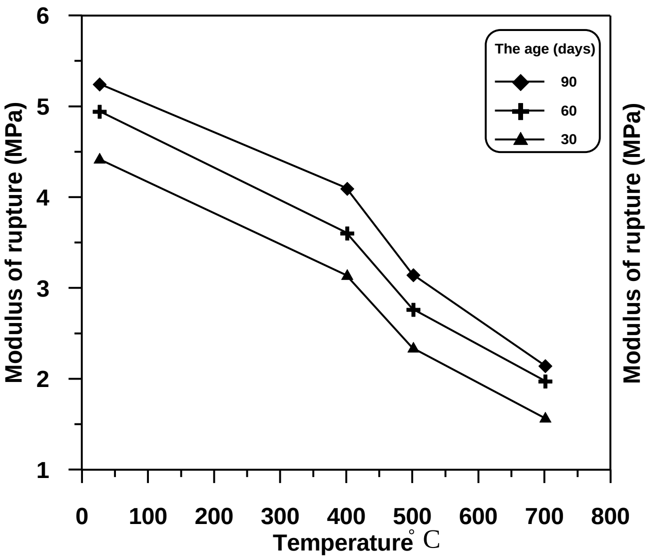


Fig.(4-30) The effect of fire flame on the modulus of rupture of series -B at 0.5 hour period of exposure.

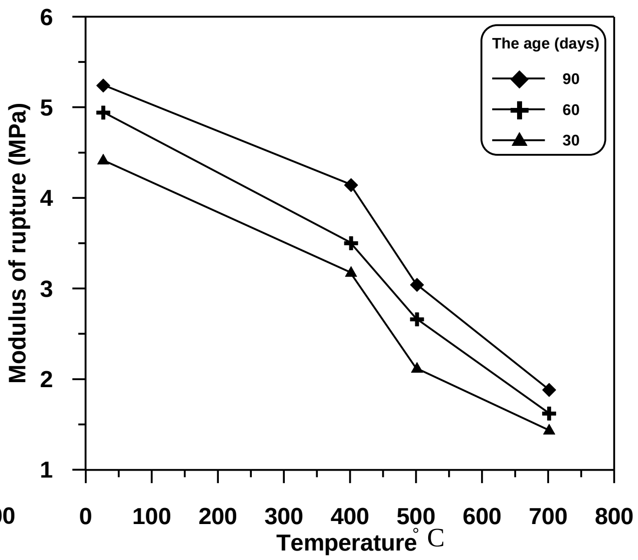


Fig.(4-31) The effect of fire flame on the modulus of rupture of series -B at 1.0 hour period of exposure.

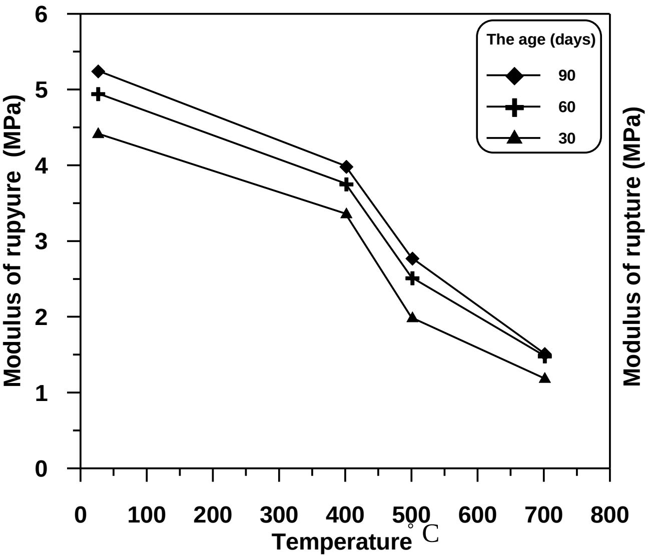


Fig.(4-32) The effect of fire flame on the modulus of rupture of series -B at1.5 hours period of exposure.

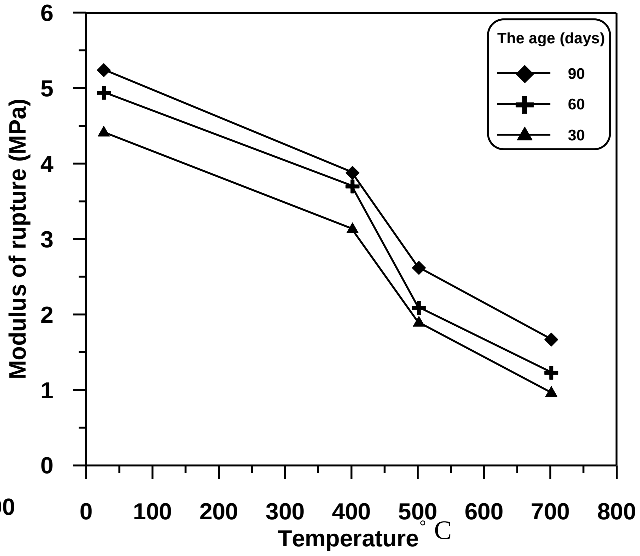


Fig (4-33) The effect of fire flame on the modulus of rupture of series -B at 2.0 hours period of exposure.

٤-٦ Modulus of Elasticity

Test results of the modulus of elasticity are summarized in Tables [٤-٩] and [٤-١٠] for mix A and mix B respectively. Figures [٤-٣٤] to [٤-٣٧] and [٤-٣٨] to [٤-٤١] illustrate the relationship between the residual modulus of elasticity and fire flame exposure temperatures for series A and series B respectively.

From the figures, it can be seen that the test results for *Ec* somewhat is similar to the pattern of compressive strength and flexural strength, but with reduction values which is more than the compressive and flexural strength at fire flame temperatures.

Table [٤-٩]: *Test values of the modulus of elasticity of concrete specimens of series-A- after exposure to fire flame temperatures.*

Age at exposure (days)	Period of Exposure (hours)	Modulus of elasticity(GPa) Temperature ºC				Ratios E_{ca}/E_{cb}		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٢٥.٤٠	١٦.٢٦	١٠.٤١	٠٦.١٠	٠.٦٤	٠.٤١	٠.٢٤
	١.٠		١٧.٢٧	١٠.٦٧	٠٥.٠٨	٠.٦٨	٠.٤٢	٠.٢٠
	١.٥		١٥.٢٤	٩.٦٥	٠٤.٥٧	٠.٦٠	٠.٣٨	٠.١٨
	٢.٠		١٣.٩٧	٨.٨٩	٠٤.٠٦	٠.٥٥	٠.٣٥	٠.١٦
٦٠	٠.٥	٢٧.٥١	١٨.٤٣	١١.٥٥	٠٦.٨٨	٠.٦٧	٠.٤٢	٠.٢٥
	١.٠		١٩.٢٦	١٢.٣٨	٠٦.٣٣	٠.٧٠	٠.٤٥	٠.٢٣
	١.٥		١٧.٣٣	١٠.٧٣	٠٥.٧٨	٠.٦٣	٠.٣٩	٠.٢١
	٢.٠		١٥.٩٦	٩.٣٥	٠٤.٩٥	٠.٥٨	٠.٣٤	٠.١٨
٩٠	٠.٥	٢٨.٧٥	٢٠.٤١	١٢.٣٦	٠٨.٦٣	٠.٧١	٠.٤٣	٠.٣٠
	١.٠		١٩.٨٤	١٤.٣٨	٠٧.٧٦	٠.٦٩	٠.٤٥	٠.٢٧
	١.٥		١٨.٤٠	١١.٥٠	٠٦.٣٣	٠.٦٤	٠.٤٠	٠.٢٢
	٢.٠		١٧.٥٤	١٠.٣٥	٠٥.٧٥	٠.٦١	٠.٣٦	٠.٢٠

E_{ca} = Modulus of elasticity after exposure to fire flame.
E_{cb} = Modulus of elasticity before exposure to fire flame.

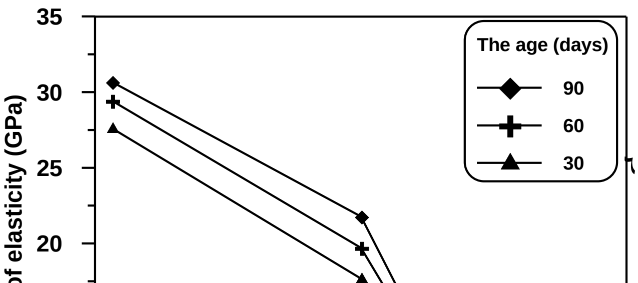
***Table [٤-١٠]:** Test values of the modulus of elasticity of concrete specimens of series-B- after exposure to fire flame.*

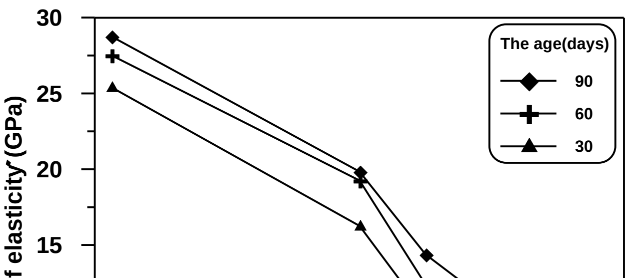
Age at exposure (days)	Period of exposure (hours)	Modulus of elasticity(GPa) Temperature ºC				Ratios E_{ca}/E_{cb}		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٣٢.١٥	٢٢.١٨	١٤.٤٧	٠٨.٣٦	٠.٦٩	٠.٤٥	٠.٢٦
	١.٠		٢١.٥٤	١٣.٥٠	٠٦.٧٥	٠.٦٧	٠.٤٢	٠.٢١
	١.٥		١٩.٤٦	١٣.١٨	٠٦.١١	٠.٦١	٠.٤١	٠.١٩
	٢.٠		١٨.٣٣	١١.٩٠	٠٥.١٤	٠.٥٧	٠.٣٧	٠.١٦
٦٠	٠.٥	٣٣.٤٦	٢٤.٠٩	١٦.٠٦	١٠.٠٣	٠.٧٢	٠.٤٨	٠.٣٠
	١.٠		٢٢.٧٥	١٥.٣٩	٠٩.٣٧	٠.٦٨	٠.٤٦	٠.٢٨
	١.٥		٢١.٠٨	١٤.٠٥	٠٧.٧٠	٠.٦٣	٠.٤٢	٠.٢٣
	٢.٠		١٩.٧٤	١٢.٧١	٠٦.٠٢	٠.٥٩	٠.٣٨	٠.١٨
٩٠	٠.٥	٣٤.٧٤	٢٥.٣٦	١٧.٧٢	١١.٨١	٠.٧٣	٠.٥١	٠.٣٤
	١.٠		٢٦.٠٦	١٦.٣٢	١٣.٥٩	٠.٧٥	٠.٤٧	٠.٣٩
	١.٥		٢٢.٥٨	١٤.٥٩	٠٨.٣٤	٠.٦٥	٠.٤٢	٠.٢٤
	٢.٠		٢٠.٨٤	١٣.٥٥	٠٧.٣٠	٠.٦٠	٠.٣٩	٠.٢١

E_{ca} = Modulus of elasticity after exposure to fire flame.
E_{cb} = Modulus of elasticity before exposure to fire flame.

At ٤٠٠ºC, there was a significant reduction in the modulus of elasticity due to the effect of fire flame. The residual modulus of elasticity was (٥٥ – ٧١ %) and (٥٧ – ٧٥ %) for series A and B respectively. Similar results were obtained by Schenider,[٧٦] Fahmi and Ibrahim[٧٧]. At ٥٠٠ºC, there was a sharp reduction in Ec values for both series and the residual of modulus of elasticity was (٣٤ – ٤٥ %) for series A and (٣٧ – ٥١٪) for series B. These results confirmed with Lankard et al [٧٨], Schneider [٧٦], Fahmi and Ibrahim [٧٧].

At ٧٠٠ºC, the residual modulus of elasticity was (١٦ – ٣٠ %) and (١٦ – ٣٤ %) for series A and series B respectively. Similar results were reported by Castello and Durani [٢٠] and Schneider [٧٦].





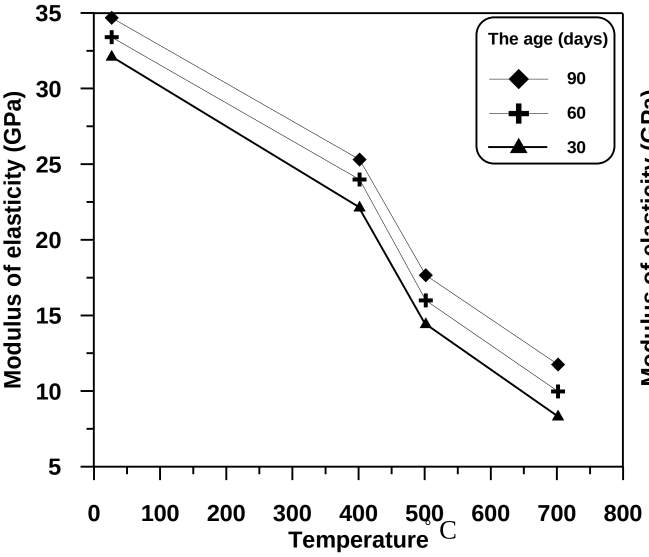


Fig.(4- 38) The effect of fire flame on the modulus of elasticity of Series- B at 0.5 hour priod of exposure.

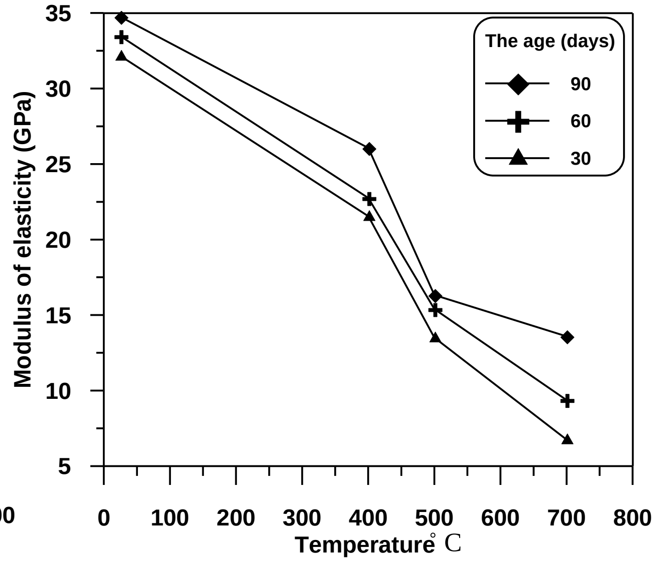


Fig.(4- 39) The effect of fire flame on the modulus of elasticity of Series- B at 1.0 hour period of exposure.

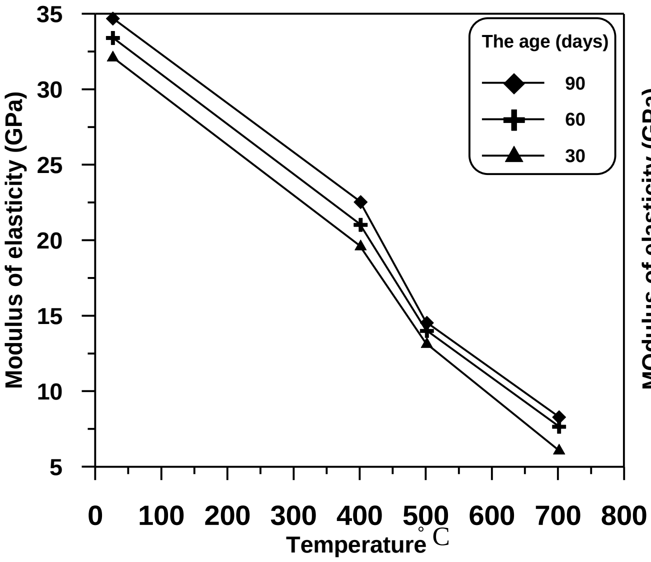


Fig.(4-40) The effect of fire flame on the modulus of elasticity of Series- B at 1.5 hours period of exposure.

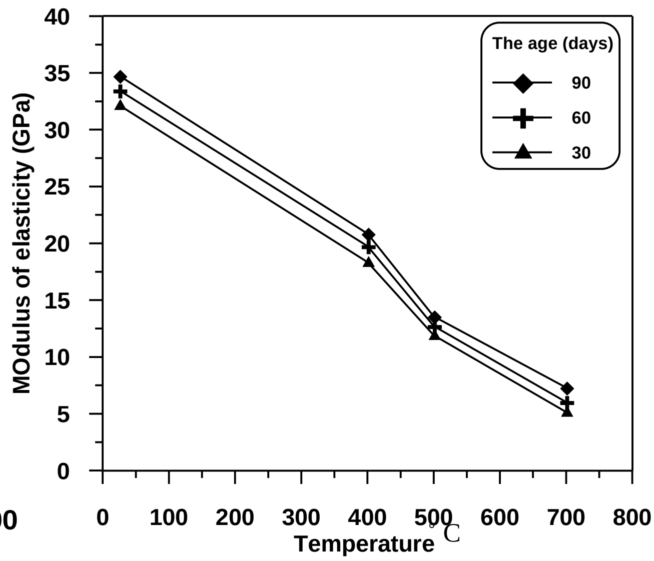


Fig.(4-41) The effect of fire flame on the modulus of elasticity of Series- B at 2.0 hours period of exposure.

٤-٧ Ultrasonic pulse velocity results (U.P.V).

The ultrasonic pulse velocity test results are presented in Tables [٤-١١] and [٤-١٢] for series A and series B respectively.

Table [٤–١١]: *Test values of the ultrasonic pulse velocity of concrete specimens of series -A- after exposure to fire flame .*

Age at Exposure (days)	Period of exposure (hours)	(U.P.V) km/sec Temperature ºC				Ratios (U.P.V)a / (U.P.V)b		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٤.٤٥	٣.٢٥	٢.٢٣	١.٨٣	٠.٧٣	٠.٥٠	٠.٤١
	١.٠		٣.٣٤	٢.٢٧	١.٧٤	٠.٧٥	٠.٥١	٠.٣٩
	١.٥		٣.١٦	٢.٠٠	١.٥٦	٠.٧١	٠.٤٥	٠.٣٥
	٢.٠		٣.٢٥	١.٨٣	١.٤٧	٠.٧٣	٠.٤١	٠.٣٣
٦٠	٠.٥	٤.٤٨	٣.٤٠	٢.٣٢	١.٩٢	٠.٧٦	٠.٥٢	٠.٤٣
	١.٠		٣.٣١	٢.٢٤	١.٧٠	٠.٧٤	٠.٥٠	٠.٣٨
	١.٥		٣.٢٧	٢.١٠	١.٥٢	٠.٧٣	٠.٤٧	٠.٣٤
	٢.٠		٣.١٨	١.٩٢	١.٥٧	٠.٧١	٠.٤٣	٠.٣٥
٩٠	٠.٥	٤.٤٩	٣.٤٩	٢.٤٣	٢.٠٧	٠.٧٧	٠.٥٤	٠.٤٦
	١.٠		٣.٤١	٢.٣٨	١.٨٤	٠.٧٦	٠.٥٣	٠.٤١
	١.٥		٣.٣٧	٢.٢٠	١.٧١	٠.٧٥	٠.٤٩	٠.٣٨
	٢.٠		٣.٢٨	٢.٠٧	١.٦٢	٠.٧٣	٠.٤٦	٠.٣٦

$(U.P.V)_a$ = Ultrasonic pulse velocity after exposure to fire flame .
$(U.P.V)_b$ = Ultrasonic pulse velocity before exposure to fire flame .

Table [٤-١٢]: *Test values of the ultrasonic pulse velocity of concrete specimens of series-B- after exposure to fire flame temperatures.*

Age at exposure (days)	Period of exposure (hours)	(U.P.V) km/sec Temperature ºC				Ratios $(U.P.V)_a / (U.P.V)_b$		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٤.٦٤	٣.٢٥	٢.٧٨	٢.٠٩	٠.٧٠	٠.٦٠	٠.٤٥
	١.٠		٣.٥٧	٢.٦٠	١.٩١	٠.٧٧	٠.٥٦	٠.٤١
	١.٥		٣.٣٤	٢.٣٧	١.٧٦	٠.٧٢	٠.٥١	٠.٣٨
	٢.٠		٣.٣٩	٢.٧٣	١.٤٨	٠.٧٣	٠.٤٩	٠.٣٢
٦٠	٠.٥	٤.٦٦	٣.٧٣	٢.٩٠	٢.٢٨	٠.٨٠	٠.٦٢	٠.٤٩
	١.٠		٣.٦٣	٢.٦٦	٢.١٠	٠.٧٨	٠.٥٧	٠.٤٥
	١.٥		٣.٣٥	٢.٥٢	١.٨٦	٠.٧٢	٠.٥٤	٠.٤٠
	٢.٠		٣.٤٩	٢.٦١	١.٥٨	٠.٧٥	٠.٥٦	٠.٣٤
٩٠	٠.٥	٤.٦٧	٣.٨٣	٢.٩٤	٢.٣٨	٠.٨٢	٠.٦٣	٠.٥١
	١.٠		٣.٦٤	٢.٨٠	٢.٠١	٠.٧٨	٠.٦٠	٠.٤٣
	١.٥		٣.٥٥	٢.٦٢	٢.١٥	٠.٧٦	٠.٥٦	٠.٤٦
	٢.٠		٣.٦٠	٢.٦٦	١.٦٤	٠.٧٧	٠.٥٧	٠.٣٥

$(U.P.V)_a$ = Ultrasonic pulse velocity after exposure to fire flame .
$(U.P.V)_b$ = Ultrasonic pulse velocity before exposure to fire flame .

Figures [٤-٤٢] to [٤-٤٥] and [٤-٤٦] to [٤-٤٩] show the relationship between the residual ultrasonic pulse velocity and the fire flame temperatures. It can be seen from figures that the reductions in the ultrasonic pulse velocity after exposure to fire flame were as follow: -

At ٤٠٠ºC, the reduction in (U.P.V) was (٢٣ – ٢٩ %) for series A and (١٨ – ٢٨ %) for series B. Similar results were observed by Logothetis and Economou [٣٩]

At ٥٠٠ºC the reduction was (٤٦ – ٥٩ %) for series A and (٣٧ – ٥١ %) for series B . These results agreed with the results found by Habeeb[٢٨], which were (٤٠ – ٤٥ %) . Essa [٤٥] result showed that the reduction in (U.P.V) at the same temperature was (٣٩ – ٥٦ %). At ٧٠٠ºC, the reduction in (U.P.V) was

(٥٤ – ٧٧ %) for series A and (٤٩ – ٦٨ %) for series B. Purkiss [٤٦] reported that the reduction in (U.P.V) at this temperature was ٧٠٪.

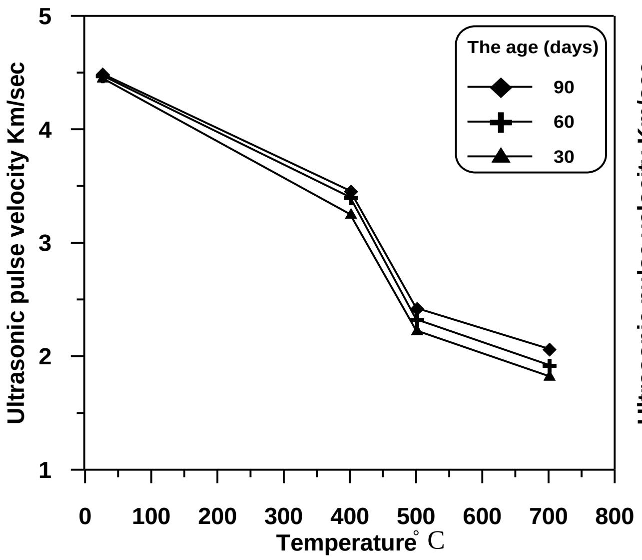


Fig.(4-42) The effect of fire flame on the ultrasonic pulse velosity of series A -at 0.5 hour period of exposure.

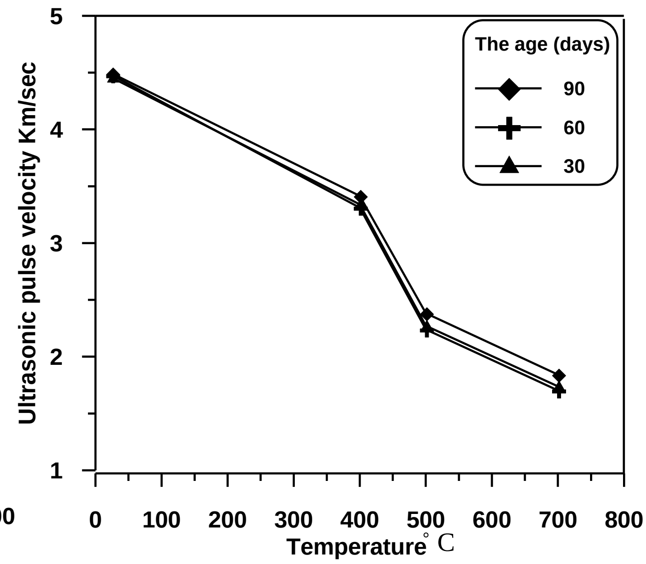


Fig.(4-43) The effect of fire flame on the ultrasonic pulse velocity of series A- at 1.0 hour period of exposure.

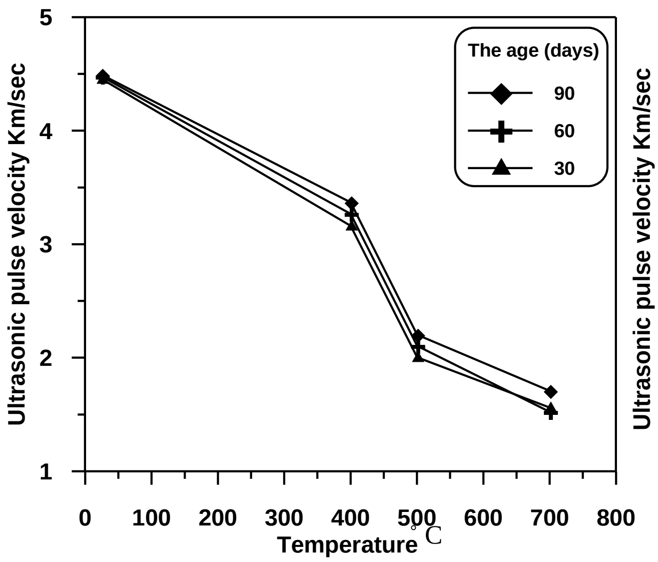


Fig.(4-44) The effect of fire flame on the ultrasonic pulse velocity of series A at 1.5 hours eriod of exposure.

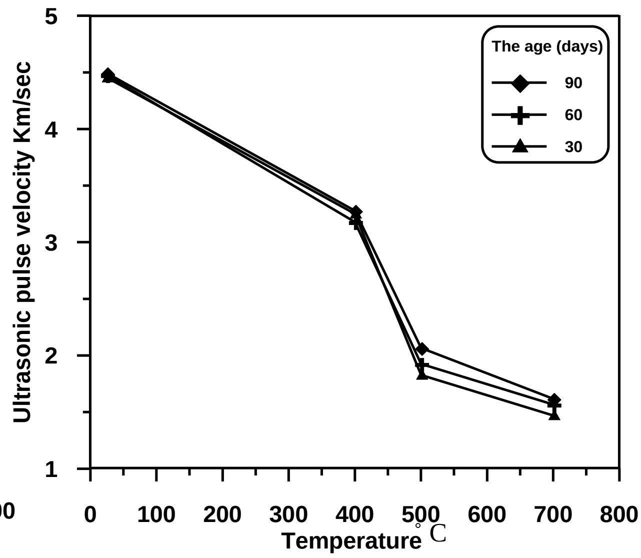


Fig.(4-45) The effect of fire flame on the ultrasonic pulse velocity of series A at 2.0 hours period of exposure.

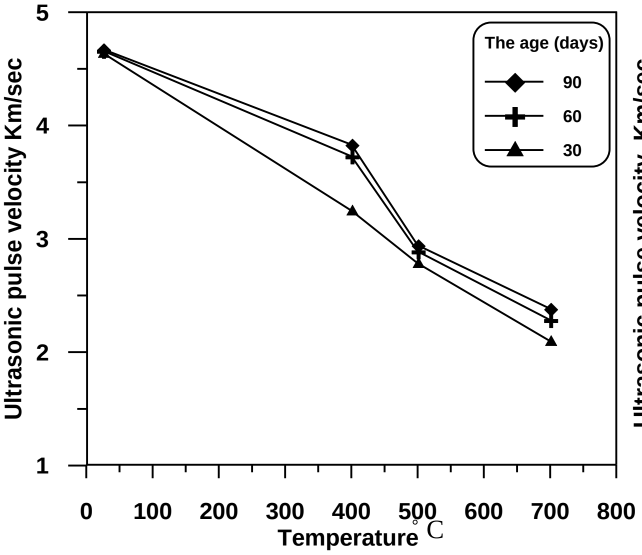


Fig.(4-46) The effect of fire flame on the ultrasonic pulse velocity of series B at 0.5 hour period of exposure.

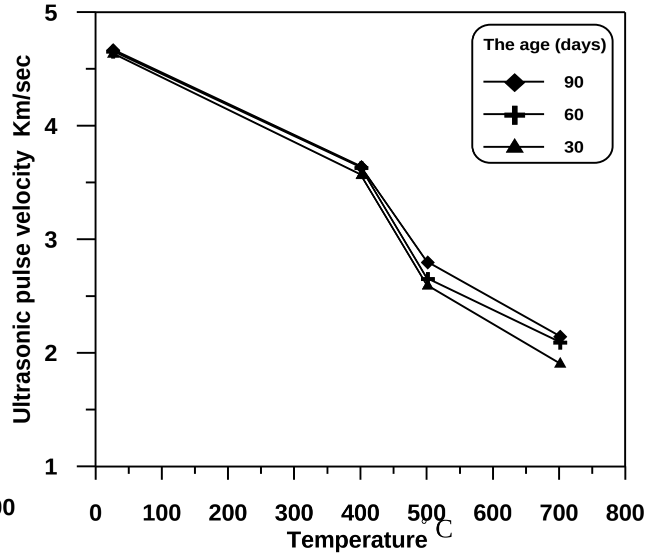


Fig.(4-47) The effect of fire flame on the ultrasonic pulse vlocity of series B at 1.0 hour period of exposure.

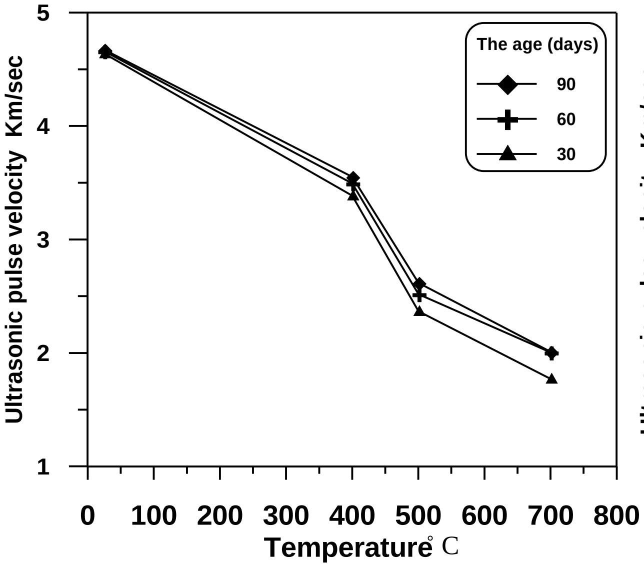


Fig.(4-48)The effect of fire flame on the ultrasonic pulse velocity of series B at 1.5 hours period of exposure.

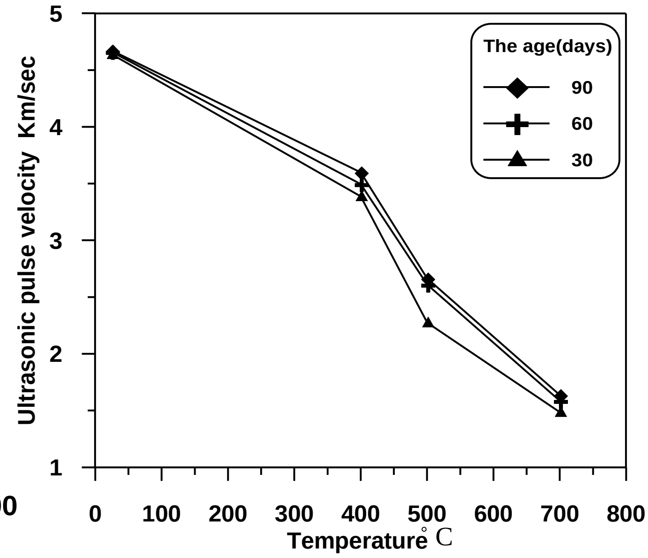


Fig.(4-49) The effect of fire flame on the ultrasonic pulse velocity of series B at 2.0 hours period of exposure.

٤-٨ Surface Hardness Results

Surface hardness of the concrete cubes was assessed by the " Schmidt rebound hammer ". Tables [٤-١٣] and [٤-١٤] show the results of the rebound number for the concrete specimens of the two series before and after exposure to fire flame.

Table [٤-١٣]: *Test values of the rebound number of concrete specimens of series-A-after exposure to fire flame temperatures.*

Age at exposure (days)	Period of exposure (hours)	Rebound number Temperature ºC				Ratios R_{Na} / R_{Nb}		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٣١	٢٦.٠	٢٥.٥	٢٠.٠	٠.٨٤	٠.٨٢	٠.٦٥
	١.٠		٢٧.٠	٢٤.٠	٢١.٤	٠.٨٧	٠.٧٨	٠.٦٩
	١.٥		٢٧.٠	٢٥.٤	١٩.٨	٠.٨٧	٠.٨٢	٠.٦٤
	٢.٠		٢٦.٠	٢٢.٦	١٩.٠	٠.٨٤	٠.٧٣	٠.٦٢
٦٠	٠.٥	٣٢	٢٨.٠	٢٧.٠	٢١.٤	٠.٨٧	٠.٨٤	٠.٦٧
	١.٠		٢٧.٨	٢٥.٠	٢٢.٧	٠.٨٧	٠.٧٨	٠.٧١
	١.٥		٢٨.٥	٢٥.٦	٢١.٤	٠.٨٩	٠.٨٠	٠.٦٧
	٢.٠		٢٦.٦	٢٤.٠	٢١.٠	٠.٨٣	٠.٧٥	٠.٦٥
٩٠	٠.٥	٣٤	٣٠.٠	٢٧.٠	٢٣.٨	٠.٨٨	٠.٨٠	٠.٧٠
	١.٠		٣٠.٦	٢٧.٥	٢٣.٠	٠.٨٣	٠.٨١	٠.٦٨
	١.٥		٣٠.٦	٢٨.٠	٢٣.٥	٠.٩٠	٠.٨٢	٠.٦٩
	٢.٠		٢٩.٠	٢٦.٠	٢٢.٥	٠.٨٥	٠.٧٦	٠.٦٦

$R_{N\,a}$ = Rebound number after exposure to fire flame.
$R_{N\,b}$ = Rebound number before exposure to fire flame

Table [٤ - ١٤]: *Test values of the rebound number of concrete specimens of series -B- after exposure to fire flame temperatures.*

Age at exposure (days)	Period of Exposure (hours)	Rebound number Temperature ºC				Ratios R_{Na} / R_{Nb}		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٣٥	٣٠.٠	٢٨.٠	٢٤.٥	٠.٨٦	٠.٨٠	٠.٧٠
	١.٠		٣٠.٥	٢٨.٧	٢٣.٨	٠.٨٧	٠.٨٢	٠.٦٨
	١.٥		٢٩.٤	٢٨.٠	٢٢.٠	٠.٨٤	٠.٨٠	٠.٦٣
	٢.٠		٢٩.٠	٢٧.٠	٢١.٠	٠.٨٣	٠.٧٧	٠.٦٠
٦٠	٠.٥	٣٦	٣٢.٠	٢٩.٠	٢٥.٠	٠.٨٩	٠.٨١	٠.٦٩
	١.٠		٣١.٠	٣٠.٠	٢٦.٠	٠.٨٦	٠.٨٣	٠.٦٩
	١.٥		٣١.٣	٢٩.٥	٢٣.٤	٠.٨٧	٠.٨٢	٠.٧٥
	٢.٠		٣٠.٢	٢٨.٤	٢٢.٧	٠.٨٤	٠.٧٩	٠.٦٨
٩٠	٠.٥	٣٨	٣٤.٥	٣١.٥	٢٧.٧	٠.٩١	٠.٨٣	٠.٧٣
	١.٠		٣٣.٤	٣٢.٠	٢٦.٦	٠.٨٨	٠.٨٤	٠.٧٠
	١.٥		٣٢.٠	٣١.٥	٢٦.٠	٠.٨٤	٠.٨٣	٠.٦٩
	٢.٠		٣٢.٣	٣١.٠	٢٣.٠	٠.٨٥	٠.٨٢	٠.٦١

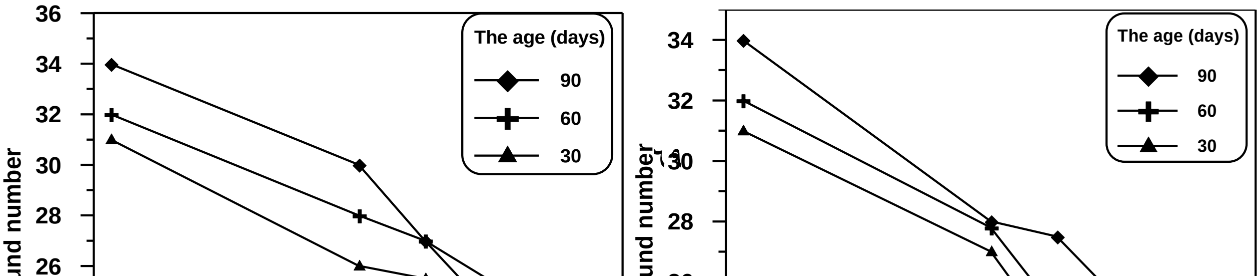
$R_{N\,a}$ = Rebound number after exposure to fire flame.
$R_{N\,b}$ = Rebound number before exposure to fire flame.

The effect of the burning by fire flame on rebound number is shown in Figures [٤-٥٠] to [٤-٥٣] and [٤-٥٤] to [٤-٥٧]. It can be seen that subjecting the concrete surface to fire causes to decrease the rebound number significantly as follows: -

At ٤٠٠ºC the reduction in the rebound number was (١٠ – ١١ %) and (٩ – ١١ %) for the series A and B respectively. The tests carried out by Logothetis and Economou[٣٩] show that the reduction in the rebound number was ١٢٪.

At ٥٠٠ºC the reduction in the rebound number was (١٦ – ٢٧ %) and (١٦ – ٢٣ %) for series A and B respectively. Essa [٤٥] showed that the reduction in rebound number was (١١ – ٢١٪) at this temperature, while Logothetis and Economou[٣٩] showed that the reduction in rebound number was ٢٠ %.

At ٧٠٠ºC the reduction in rebound number was (٢٩ – ٣٨ %) and (٢٥- ٤٠ %) for series A and B respectively, while Logothetis and Economou[٣٩]showed that the reduction was ٤٠٪ at the same temperature. The decrease in the rebound number with increase in temperature can be attributed to the fact that fire causes damage to the surface of concrete rather than to concrete in the core of the member.



٤-٧ Ultrasonic pulse velocity results (U.P.V).

The ultrasonic pulse velocity test results are presented in Tables [٤-١١] and [٤-١٢] for series A and series B respectively.

Table [٤– ١١]: *Test values of the ultrasonic pulse velocity of concrete specimens of series -A- after exposure to fire flame .*

Age at Exposure (days)	Period of exposure (hours)	(*U.P.V) km/sec*				Ratios $(U.P.V)_a / (U.P.V)_b$		
		Temperature ºC						
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٤.٤٥	٣.٢٥	٢.٢٣	١.٨٣	٠.٧٣	٠.٥٠	٠.٤١
	١.٠		٣.٣٤	٢.٢٧	١.٧٤	٠.٧٥	٠.٥١	٠.٣٩
	١.٥		٣.١٦	٢.٠٠	١.٥٦	٠.٧١	٠.٤٥	٠.٣٥
	٢.٠		٣.٢٥	١.٨٣	١.٤٧	٠.٧٣	٠.٤١	٠.٣٣
٦٠	٠.٥	٤.٤٨	٣.٤٠	٢.٣٢	١.٩٢	٠.٧٦	٠.٥٢	٠.٤٣
	١.٠		٣.٣١	٢.٢٤	١.٧٠	٠.٧٤	٠.٥٠	٠.٣٨
	١.٥		٣.٢٧	٢.١٠	١.٥٢	٠.٧٣	٠.٤٧	٠.٣٤
	٢.٠		٣.١٨	١.٩٢	١.٥٧	٠.٧١	٠.٤٣	٠.٣٥
٩٠	٠.٥	٤.٤٩	٣.٤٩	٢.٤٣	٢.٠٧	٠.٧٧	٠.٥٤	٠.٤٦
	١.٠		٣.٤١	٢.٣٨	١.٨٤	٠.٧٦	٠.٥٣	٠.٤١
	١.٥		٣.٣٧	٢.٢٠	١.٧١	٠.٧٥	٠.٤٩	٠.٣٨
	٢.٠		٣.٢٨	٢.٠٧	١.٦٢	٠.٧٣	٠.٤٦	٠.٣٦

$(U.P.V)_a$ = Ultrasonic pulse velocity after exposure to fire flame .
$(U.P.V)_b$ = Ultrasonic pulse velocity before exposure to fire flame .

Table [٤-١٢]: *Test values of the ultrasonic pulse velocity of concrete specimens of series-B- after exposure to fire flame temperatures.*

Age at exposure (days)	Period of exposure (hours)	(U.P.V) km/sec Temperature ºC				Ratios $(U.P.V)_a / (U.P.V)_b$		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٤.٦٤	٣.٢٥	٢.٧٨	٢.٠٩	٠.٧٠	٠.٦٠	٠.٤٥
	١.٠		٣.٥٧	٢.٦٠	١.٩١	٠.٧٧	٠.٥٦	٠.٤١
	١.٥		٣.٣٤	٢.٣٧	١.٧٦	٠.٧٢	٠.٥١	٠.٣٨
	٢.٠		٣.٣٩	٢.٧٣	١.٤٨	٠.٧٣	٠.٤٩	٠.٣٢
٦٠	٠.٥	٤.٦٦	٣.٧٣	٢.٩٠	٢.٢٨	٠.٨٠	٠.٦٢	٠.٤٩
	١.٠		٣.٦٣	٢.٦٦	٢.١٠	٠.٧٨	٠.٥٧	٠.٤٥
	١.٥		٣.٣٥	٢.٥٢	١.٨٦	٠.٧٢	٠.٥٤	٠.٤٠
	٢.٠		٣.٤٩	٢.٦١	١.٥٨	٠.٧٥	٠.٥٦	٠.٣٤
٩٠	٠.٥	٤.٦٧	٣.٨٣	٢.٩٤	٢.٣٨	٠.٨٢	٠.٦٣	٠.٥١
	١.٠		٣.٦٤	٢.٨٠	٢.٠١	٠.٧٨	٠.٦٠	٠.٤٣
	١.٥		٣.٥٥	٢.٦٢	٢.١٥	٠.٧٦	٠.٥٦	٠.٤٦
	٢.٠		٣.٦٠	٢.٦٦	١.٦٤	٠.٧٧	٠.٥٧	٠.٣٥

$(U.P.V)_a$ = Ultrasonic pulse velocity after exposure to fire flame .
$(U.P.V)_b$ = Ultrasonic pulse velocity before exposure to fire flame .

Figures [٤-٤٢] to [٤-٤٥] and [٤-٤٦] to [٤-٤٩] show the relationship between the residual ultrasonic pulse velocity and the fire flame temperatures. It can be seen from figures that the reductions in the ultrasonic pulse velocity after exposure to fire flame were as follow: -

At ٤٠٠ºC, the reduction in (U.P.V) was (٢٣ – ٢٩ %) for series A and (١٨ – ٢٨ %) for series B. Similar results were observed by Logothetis and Economou [٣٩]

At ٥٠٠ºC the reduction was (٤٦ – ٥٩ %) for series A and (٣٧ – ٥١ %) for series B . These results agreed with the results found by Habeeb[٢٨], which were (٤٠ – ٤٥ %) . Essa [٤٥] result showed that the reduction in (U.P.V) at the same temperature was (٣٩ – ٥٦ %). At ٧٠٠ºC, the reduction in (U.P.V) was

(٥٤ – ٧٧ %) for series A and (٤٩ – ٦٨ %) for series B. Purkiss [٤٦] reported that the reduction in (U.P.V) at this temperature was ٧٠٪.

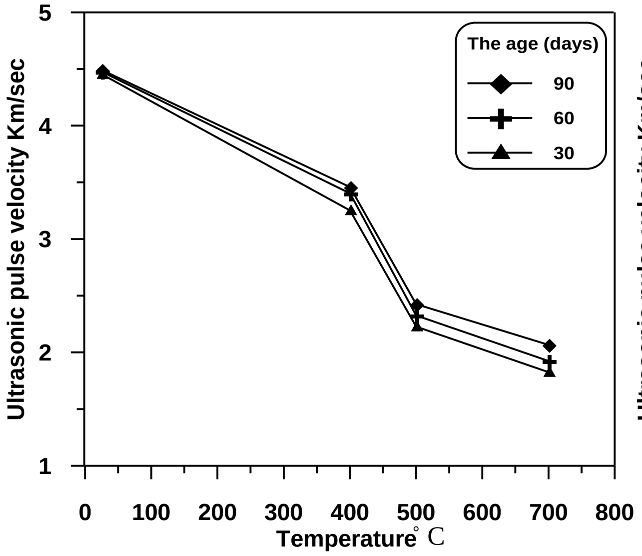


Fig.(4-42) The effect of fire flame on the ultrasonic pulse velosity of series A -at 0.5 hour period of exposure.

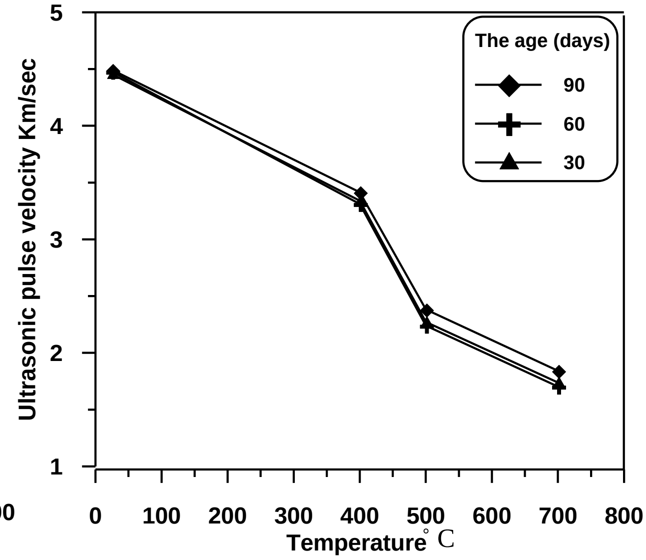


Fig.(4-43) The effect of fire flame on the ultrasonic pulse velocity of series A- at 1.0 hour period of exposure.

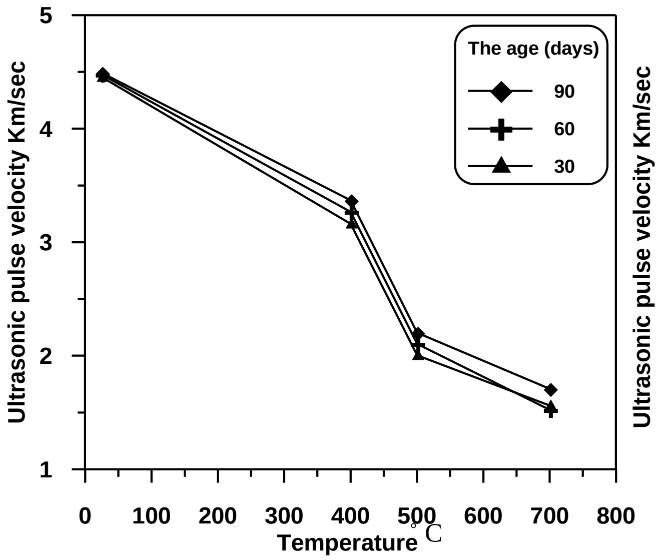


Fig.(4-44) The effect of fire flame on the ultrasonic pulse velocity of series A at 1.5 hours eriod of exposure.

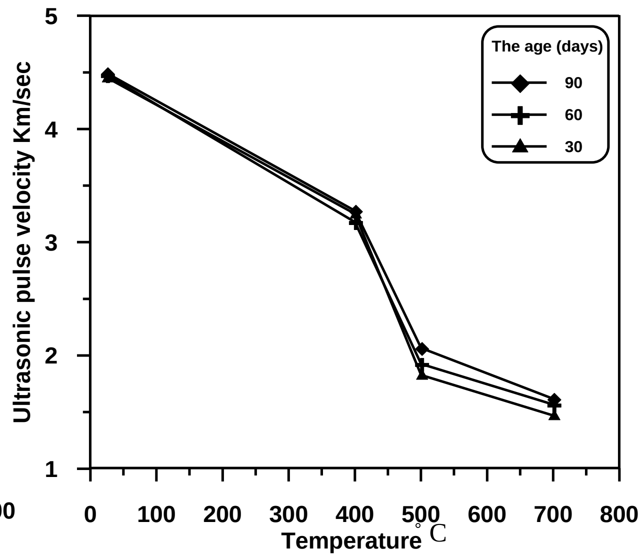


Fig.(4-45) The effect of fire flame on the ultrasonic pulse velocity of series A at 2.0 hours period of exposure.

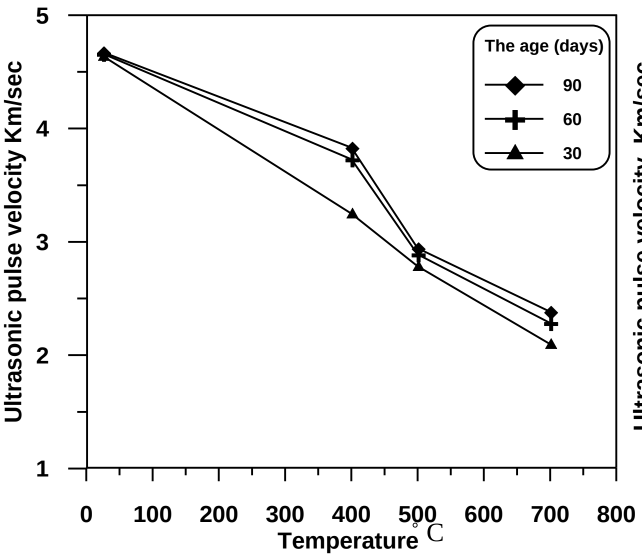


Fig.(4-46) The effect of fire flame on the ultrasonic pulse velocity of series B at 0.5 hour period of exposure.

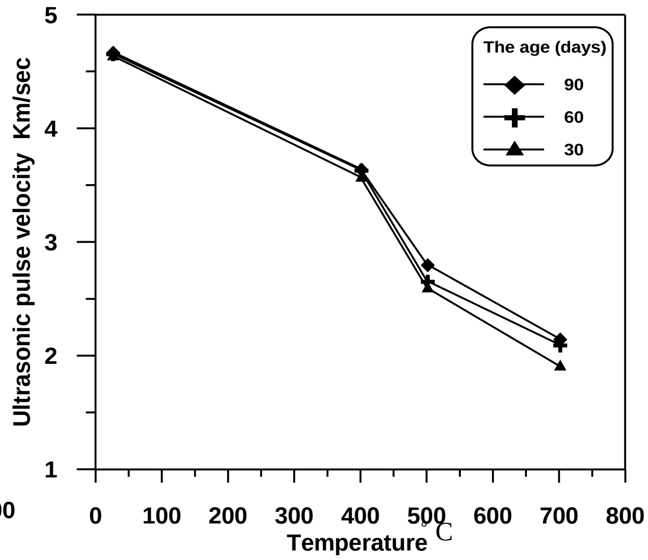


Fig.(4-47) The effect of fire flame on the ultrasonic pulse vlocity of series B at 1.0 hour period of exposure.

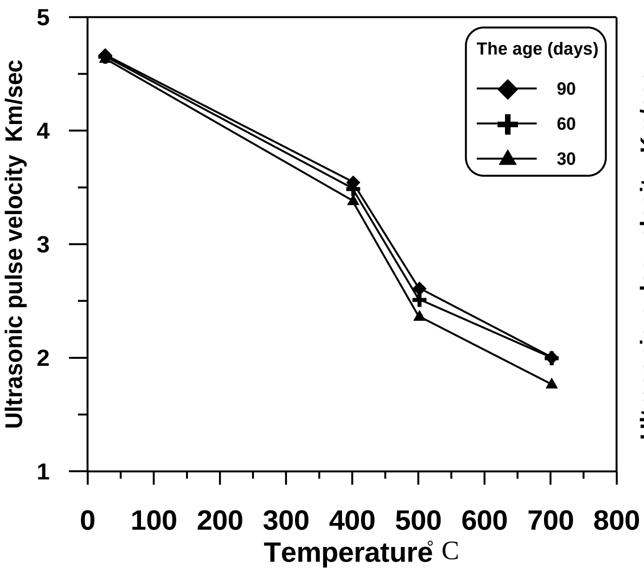


Fig.(4-48)The effect of fire flame on the ultrasonic pulse velocity of series B at 1.5 hours period of exposure.

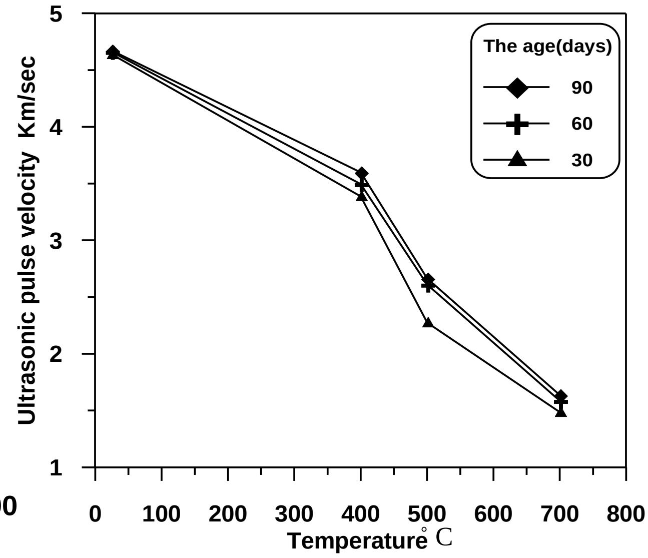


Fig.(4-49) The effect of fire flame on the ultrasonic pulse velocity of series B at 2.0 hours period of exposure.

٤-٨ Surface Hardness Results

Surface hardness of the concrete cubes was assessed by the " Schmidt rebound hammer ". Tables [٤-١٣] and [٤-١٤] show the results of the rebound number for the concrete specimens of the two series before and after exposure to fire flame.

Table [٤-١٣]: *Test values of the rebound number of concrete specimens of series-A-after exposure to fire flame temperatures.*

Age at exposure (days)	Period of exposure (hours)	Rebound number Temperature ºC				Ratios R_{Na}/R_{Nb}		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٣١	٢٦.٠	٢٥.٥	٢٠.٠	٠.٨٤	٠.٨٢	٠.٦٥
	١.٠		٢٧.٠	٢٤.٠	٢١.٤	٠.٨٧	٠.٧٨	٠.٦٩
	١.٥		٢٧.٠	٢٥.٤	١٩.٨	٠.٨٧	٠.٨٢	٠.٦٤
	٢.٠		٢٦.٠	٢٢.٦	١٩.٠	٠.٨٤	٠.٧٣	٠.٦٢
٦٠	٠.٥	٣٢	٢٨.٠	٢٧.٠	٢١.٤	٠.٨٧	٠.٨٤	٠.٦٧
	١.٠		٢٧.٨	٢٥.٠	٢٢.٧	٠.٨٧	٠.٧٨	٠.٧١
	١.٥		٢٨.٥	٢٥.٦	٢١.٤	٠.٨٩	٠.٨٠	٠.٦٧
	٢.٠		٢٦.٦	٢٤.٠	٢١.٠	٠.٨٣	٠.٧٥	٠.٦٥
٩٠	٠.٥	٣٤	٣٠.٠	٢٧.٠	٢٣.٨	٠.٨٨	٠.٨٠	٠.٧٠
	١.٠		٣٠.٦	٢٧.٥	٢٣.٠	٠.٨٣	٠.٨١	٠.٦٨
	١.٥		٣٠.٦	٢٨.٠	٢٣.٥	٠.٩٠	٠.٨٢	٠.٦٩
	٢.٠		٢٩.٠	٢٦.٠	٢٢.٥	٠.٨٥	٠.٧٦	٠.٦٦

$R_{N\,a}$ = Rebound number after exposure to fire flame.
$R_{N\,b}$ = Rebound number before exposure to fire flame

Table [٤ - ١٤]: *Test values of the rebound number of concrete specimens of series -B- after exposure to fire flame temperatures.*

Age at exposure (days)	Period of Exposure (hours)	Rebound number Temperature ºC				Ratios R_{Na} / R_{Nb}		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٣٥	٣٠.٠	٢٨.٠	٢٤.٥	٠.٨٦	٠.٨٠	٠.٧٠
	١.٠		٣٠.٥	٢٨.٧	٢٣.٨	٠.٨٧	٠.٨٢	٠.٦٨
	١.٥		٢٩.٤	٢٨.٠	٢٢.٠	٠.٨٤	٠.٨٠	٠.٦٣
	٢.٠		٢٩.٠	٢٧.٠	٢١.٠	٠.٨٣	٠.٧٧	٠.٦٠
٦٠	٠.٥	٣٦	٣٢.٠	٢٩.٠	٢٥.٠	٠.٨٩	٠.٨١	٠.٦٩
	١.٠		٣١.٠	٣٠.٠	٢٦.٠	٠.٨٦	٠.٨٣	٠.٦٩
	١.٥		٣١.٣	٢٩.٥	٢٣.٤	٠.٨٧	٠.٨٢	٠.٧٥
	٢.٠		٣٠.٢	٢٨.٤	٢٢.٧	٠.٨٤	٠.٧٩	٠.٦٨
٩٠	٠.٥	٣٨	٣٤.٥	٣١.٥	٢٧.٧	٠.٩١	٠.٨٣	٠.٧٣
	١.٠		٣٣.٤	٣٢.٠	٢٦.٦	٠.٨٨	٠.٨٤	٠.٧٠
	١.٥		٣٢.٠	٣١.٥	٢٦.٠	٠.٨٤	٠.٨٣	٠.٦٩
	٢.٠		٣٢.٣	٣١.٠	٢٣.٠	٠.٨٥	٠.٨٢	٠.٦١

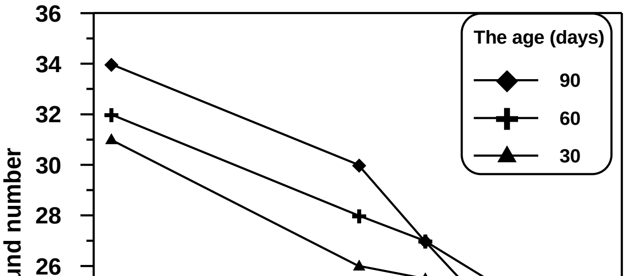
$R_{N a}$ = Rebound number after exposure to fire flame.
$R_{N b}$ = Rebound number before exposure to fire flame.

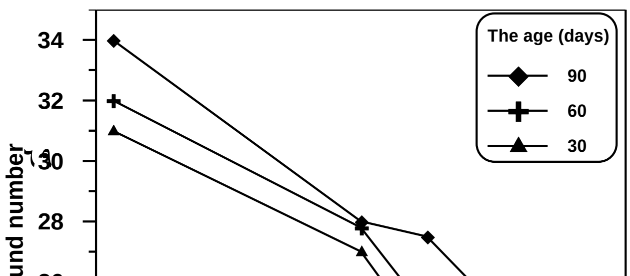
The effect of the burning by fire flame on rebound number is shown in Figures [٤-٥٠] to [٤-٥٣] and [٤-٥٤] to [٤-٥٧]. It can be seen that subjecting the concrete surface to fire causes to decrease the rebound number significantly as follows: -

At ٤٠٠ºC the reduction in the rebound number was (١٠ – ١١ %) and (٩ – ١١ %) for the series A and B respectively. The tests carried out by Logothetis and Economou[٣٩] show that the reduction in the rebound number was ١٢٪.

At ٥٠٠ºC the reduction in the rebound number was (١٦ – ٢٧ %) and (١٦ – ٢٣ %) for series A and B respectively. Essa [٤٥] showed that the reduction in rebound number was (١١ – ٢١٪) at this temperature, while Logothetis and Economou[٣٩] showed that the reduction in rebound number was ٢٠ %.

At ٧٠٠ºC the reduction in rebound number was (٢٩ – ٣٨ %) and (٢٥- ٤٠ %) for series A and B respectively, while Logothetis and Economou[٣٩]showed that the reduction was ٤٠٪ at the same temperature. The decrease in the rebound number with increase in temperature can be attributed to the fact that fire causes damage to the surface of concrete rather than to concrete in the core of the member.





Chapter Four

4

RESULTS AND DISCUSSIONS

٤-١ Introduction

Experimental results are presented in this chapter showing the effect of the fire flame on some mechanical properties of concrete, such as density, compressive strength, splitting tensile strength, flexural strength and modulus of elasticity. The test results of the load-deflection relationship of reinforced concrete beams are presented also.

٤-٢ The Density

Tables [٤-١] and [٤-٢] show the effect of the exposure to fire flame on the density of concrete, while Figures [٤-١] to [٤-٣] and [٤-٤] to [٤-٨] show the relations between fire flame temperature and the density for series A and B respectively. It can be seen from these Tables and Figures that the density behaved as follows:-

١- At ٤٠٠ºC fire flame temperature and for all ages and all periods of exposure, the reduction in density ranged between (٤.٨ – ٧.٧ %) and (١.٩ – ٤.٥ %) for series A and B respectively if compared with the initial density before exposure to fire.

٢- The reduction was (٥.٩ -٨٪) and (٤ – ٧.١ %) for series A and B respectively at ٥٠٠ºC fire flame temperature.

٣- More reduction in density took place when the fire flame temperatures increased.At٧٠٠ºC,the reduction in density was (٨–١١.٢٪), and (٨–٩.٦ %) for series A and B respectively. The loss in density of series A was ١.٦٪ more than that in density of series B.It can be observed that series B lose less weight than series A, which can be attributed to the less water- cement ratio and less evaporation of adsorbed water on interface of the gel crystals of hardened cement paste, also to less amount of pores and voids in series B.These results confirmed with that of Al-Elizzi[١٨] , Habeeb[٢٨] and Essa [٤٥]

Table [٤- ١]: *Test values of the density of concrete specimens of series-A- after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Density (kg/ m٣) Temperature ºC				Ratios ρ_a / ρ_b		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/ ١ (٥)	٣/ ١ (٦)	٤/ ١ (٧)
٣٠	٠.٥	٢٣٢٥	٢٢٠٢	٢١٦٧	٢١٣٩	٠.٩٤٧	٠.٩٣٢	٠.٩٢٠
	١.٠		٢١٨٨	٢١٦٢	٢١٢٠	٠.٩٤١	٠.٩٣٠	٠.٩٠٨
	١.٥		٢١٦٣	٢١٥٣	٢٠٧٩	٠.٩٣٠	٠.٩٢٦	٠.٨٩٤
	٢.٠		٢١٤٨	٢١٣٩	٢٠٥١	٠.٩٢٣	٠.٩٢٠	٠.٨٨٢
٦٠	٠.٥	٢٣٢٣	٢٢٠٧	٢١٧٩	٢١٤٩	٠.٩٥٠	٠.٩٣٨	٠.٩٢٥
	١.٠		٢١٩٠	٢١٦٥	٢١٢١	٠.٩٤٣	٠.٩٢٤	٠.٩١٣
	١.٥		٢١٧٦	٢١٥٨	٢١١٢	٠.٩٣٧	٠.٩٢٤	٠.٩٠٩
	٢.٠		٢١٦٩	٢١٦٠	٢٠٩١	٠.٩٣٠	٠.٩٢٥	٠.٩٠٠
٩٠	٠.٥	٢٣٢٠	٢٢٠٩	٢١٨١	٢١٥٨	٠.٩٥٢	٠.٩٤٠	٠.٩٣٠
	١.٠		٢١٩٥	٢٢٥٤	٢١٣٧	٠.٩٤٦	٠.٩٤١	٠.٩٢٢
	١.٥		٢١٨١	٢١٦٧	٢١٣٤	٠.٩٤٠	٠.٩٣٤	٠.٩٢٠
	٢.٠		٢١٦٩	٢١٥٨	٢١١٦	٠.٩٣٥	٠.٩٣٠	٠.٩١٢

$\rho_a =$ The density of concrete after exposure to fire flame.

$\rho_b =$ The density of concrete before exposure to fire flame

Table [٤- ٢]: *Test values of the density of concrete specimens of series-B-after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Density (kg/ m٣) Temperature ºC				Ratios ρ_a / ρ_b		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٢٣٧٨	٢٣٠٧	٢٢٤٠	٢٢٠٢	٠.٩٧٠	٠.٩٤٣	٠.٩٢٦
	١.٠		٢٢٩٥	٢٢٢٨	٢١٨٥	٠.٩٦٥	٠.٩٣٧	٠.٩١٩
	١.٥		٢٢٨٢	٢٢٢٨	٢١٦٤	٠.٩٥٩	٠.٩٣٥	٠.٩١٠
	٢.٠		٢٢٧١	٢٢٠٩	٢١٥٠	٠.٩٥٥	٠.٩٢٩	٠.٩٠٤
٦٠	٠.٥	٢٣٧٦	٢٣١٦	٢٢٥٩	٢٢١٢	٠.٩٧٤	٠.٩٥٠	٠.٩٣١
	١.٠		٢٣٠٢	٢٢٥٠	٢١٨١	٠.٩٦٩	٠.٩٤٦	٠.٩١٨
	١.٥		٢٢٩٠	٢٢٣٩	٢١٧٤	٠.٩٦٣	٠.٩٤٢	٠.٩١٥
	٢.٠		٢٢٩٣	٢٢١٨	٢١٦٢	٠.٩٦٥	٠.٩٣٨	٠.٩١٠
٩٠	٠.٥	٢٣٧٥	٢٣٣٢	٢٢٨٠	٢٢٣٣	٠.٩٨١	٠.٩٦٠	٠.٩٤٠
	١.٠		٢٣٢٠	٢٢٧٥	٢٢٠٦	٠.٩٧٧	٠.٩٥٨	٠.٩٢٩
	١.٥		٢٣١٢	٢٢٦٧	٢١٩٥	٠.٩٧٣	٠.٩٥٤	٠.٩٢٤
	٢.٠		٢٣٠٠	٢٢٦٠	٢١٨٥	٠.٩٦٨	٠.٩٥١	٠.٩٢٠

ρ_a = The density of concrete after exposure to fire flame.

ρ_b = The density of concrete before exposure to fire flame.

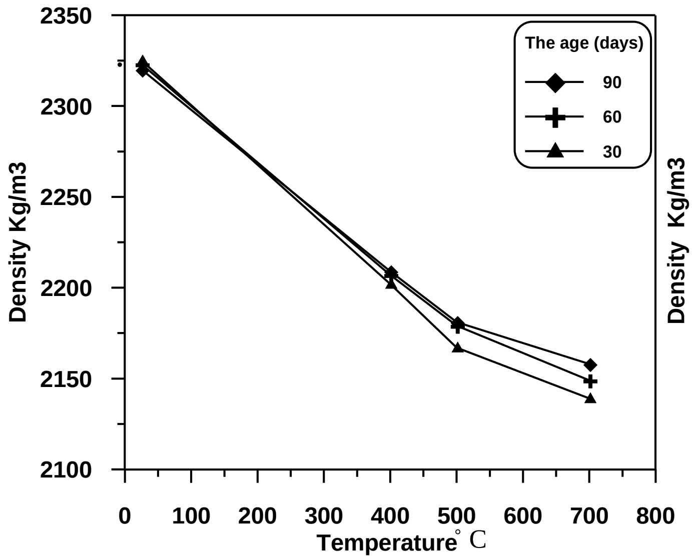


Fig.[4-1] The effect of fire flame on the density of series-A at 0.5 hour period of exposure.

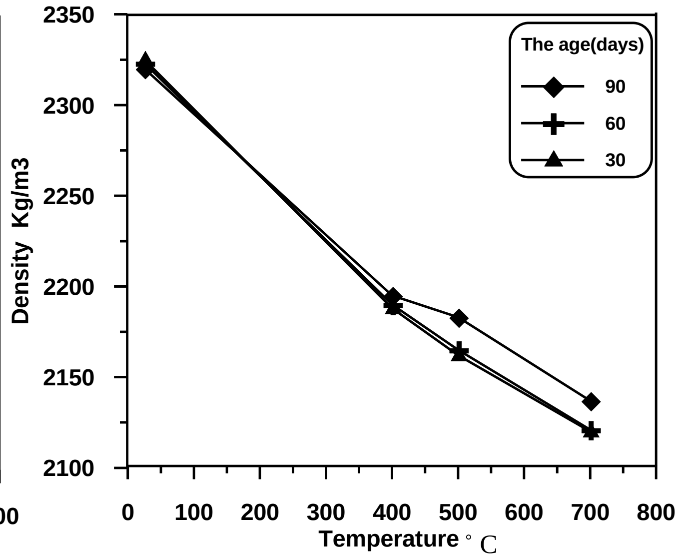


Fig.(4-2) The effect of fire flame on the density of series-A at 1.0 hour period of exposure.

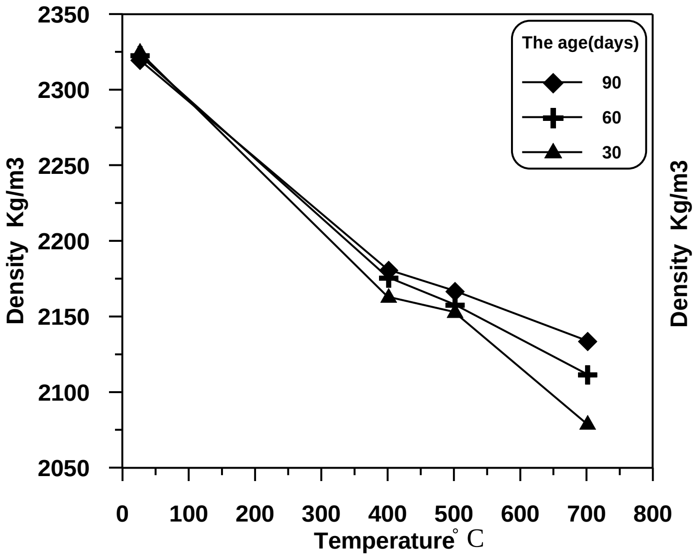


Fig.(4-3) The effect of fire flame on the density of series-A at 1.5 hours period of exposure.

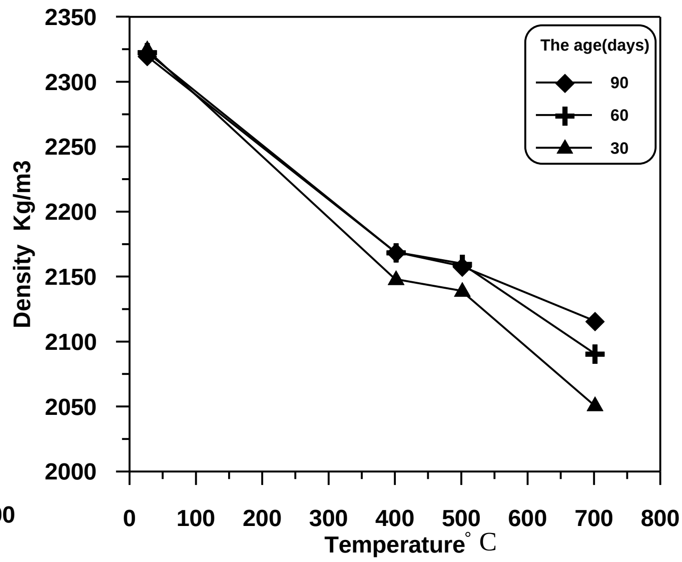


Fig.(4-4) The effect of fire flame on the density of series-A at 2.0 hours period of exposure.

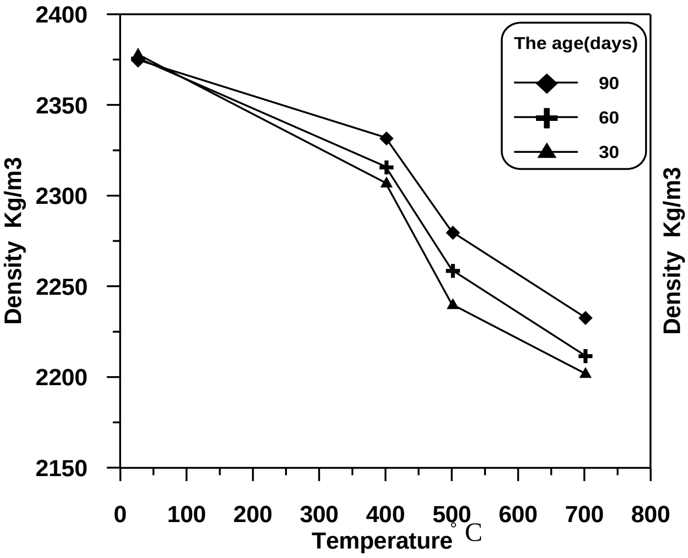


Fig.(4-5) The effect of fire flame on the density of series-B at 0.5 hour period of exposure.

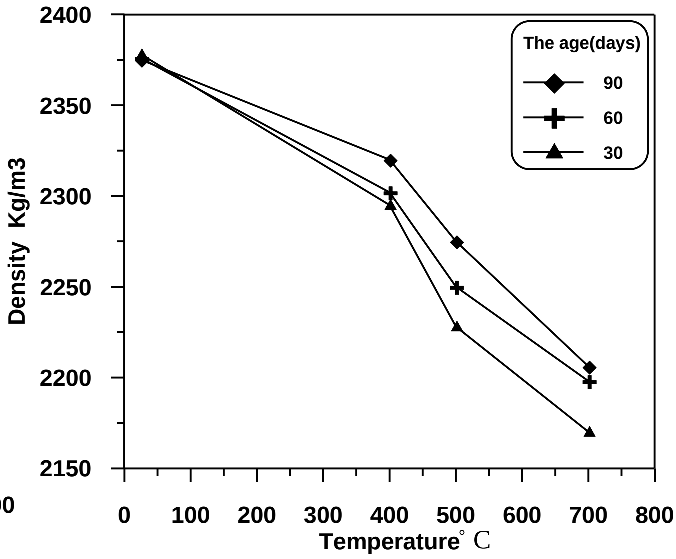


Fig.(4-6) The effect of fire flame on the density of series-B at 1.0 hour peroid of exposure.

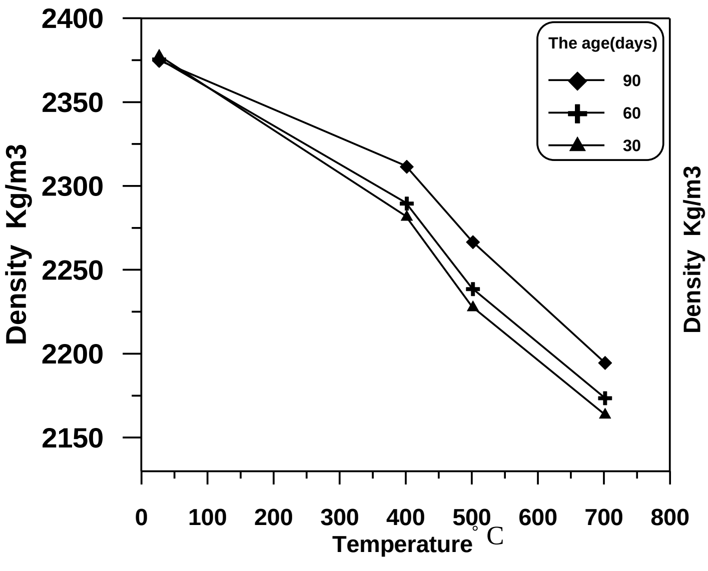


Fig.(4-7) The effect of fire flame on the density of series-B at 1.5 hours period of exposure.

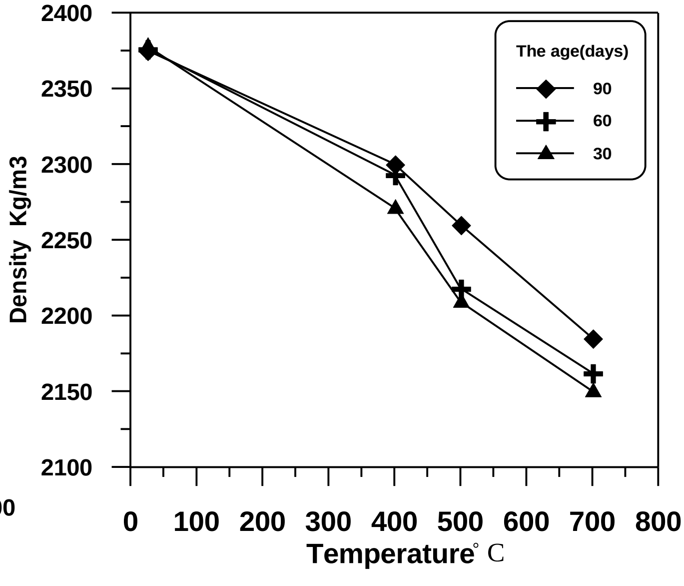


Fig.(4-8) The effect of the fireflame on the density of series -B-at 2.0 hours period of exposure.

٤-٣ Compressive Strength

The compressive strength results are summarized in Tables [٤-٣] and [٤-٤] for both series A and B respectively, while Figures [٤-٩] to [٤-١٦] show the relation between compressive strengths and fire flame temperatures for both series A and B respectively.

Table [٤-٣]: *Test values of compressive strength of concrete specimens of series -A- after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Compressive Strength (MPa)				Ratios $f_{cu\,a}/f_{ub}$			Type of Cooling
		Temperature ºC							
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)	
٣٠	٠.٥	٣٠.٥٠	٢٢.٥٧	٢٠.٤٣	١٦.١٧	٠.٧٣	٠.٦٧	٠.٥٣	Air
			٢١.٣٥	١٨.٩١	١٤.٠٣	٠.٧٠	٠.٦٢	٠.٤٦	Water
	١.٠		٢٢.٥٧	١٩.٢٢	١٤.٣٤	٠.٧٤	٠.٦٣	٠.٤٧	Air
			٢١.٣٥	١٧.٦٩	١٢.٨١	٠.٧٠	٠.٥٨	٠.٤٢	Water
	١.٥		٢١.٦٦	١٨.٣٠	١٣.٧٣	٠.٧١	٠.٦٠	٠.٤٥	Air
			٢٠.١٣	١٦.٤٧	١٠.٩٨	٠.٦٦	٠.٥٤	٠.٣٦	Water
	٢.٠		٢٢.٨٨	١٨.٠٠	١٣.١٢	٠.٧٥	٠.٥٩	٠.٤٣	Air
			٢١.٠٥	١٥.٨٦	١٠.٠٧	٠.٦٩	٠.٥٢	٠.٣٣	Water
٦٠	٠.٥	٣٤.١٢	٢٦.٩٥	٢٤.٢٣	١٨.٧٧	٠.٧٩	٠.٧١	٠.٥٥	Air
			٢٥.٥٩	٢٢.٨٦	١٧.٠٦	٠.٧٥	٠.٦٧	٠.٥٠	Water
	١.٠		٢٥.٩٣	٢٢.٥٢	١٧.٤٠	٠.٧٦	٠.٦٦	٠.٥١	Air
			٢٤.٢٣	٢٠.٨١	١٥.٦٩	٠.٧١	٠.٦١	٠.٤٦	Water
	١.٥		٢٧.٣٠	٢١.١٥	١٦.٣٨	٠.٨٠	٠.٦٢	٠.٤٨	Air
			٢٥.٢٥	١٩.٧٩	١٤.٦٧	٠.٧٤	٠.٥٨	٠.٤٣	Water
	٢.٠		٢٦.٢٧	٢٠.٤٧	١٥.٣٥	٠.٧٧	٠.٦٠	٠.٤٥	Air
			٢٤.٢٣	١٨.٤٢	١٣.٦٥	٠.٧١	٠.٥٤	٠.٤٠	Water
٩٠	٠.٥	٣٧.٣١	٣٠.٩٧	٢٧.٩٨	٢٢.٣٩	٠.٨٣	٠.٧٥	٠.٦٠	Air
			٢٩.٨٥	٢٦.١٢	٢٠.٥٢	٠.٨٠	٠.٧٠	٠.٥٥	Water
	١.٠		٢٩.٨٥	٢٥.٧٤	٢٠.١٥	٠.٨٠	٠.٦٩	٠.٥٤	Air
			٢٨.٣٦	٢٣.٥١	١٧.٩١	٠.٧٦	٠.٦٣	٠.٤٨	Water
	١.٥		٢٩.١٠	٢٣.٨٨	١٨.٦٦	٠.٧٨	٠.٦٤	٠.٥٠	Air
			٢٧.٦١	٢١.٦٤	١٦.٧٩	٠.٧٤	٠.٥٨	٠.٤٥	Water
	٢.٠		٢٩.٤٧	٢٢.٧٦	١٧.٩٠	٠.٧٩	٠.٦١	٠.٤٨	Air
			٢٧.٢٤	٢٠.١٥	١٥.٦٧	٠.٧٣	٠.٥٤	٠.٤٢	Water

f_{cua} = Compressive strength (cube) after exposure to fire flame.
f_{cub} = Compressive strength (cube) before exposure to fire flame.

Table [٤-٤] : *Test values of compressive strength of concrete specimens of series- B- after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Compressive Strength (MPa) Temperature °C ٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	Ratios $f_{cu\,a}/f_{cub}$ ٢/١ (٥)	٣/١ (٦)	٤/١ (٧)	Type of Cooling
٣٠	٠.٥	٤١.٣٠	٢٨.٩١	٢٨.٥٠	٢٣.٩٥	٠.٧٠	٠.٦٩	٠.٥٨	Air
			٢٧.٦٧	٢٦.٨٥	٢١.٤٨	٠.٦٧	٠.٦٥	٠.٥٢	Water
	١.٠		٣٢.٢١	٢٦.٨٥	٢٠.٦٥	٠.٧٨	٠.٦٥	٠.٥٠	Air
			٣٠.٩٨	٢٥.٦١	١٨.١٧	٠.٧٥	٠.٦٢	٠.٤٤	Water
	١.٥		٣٠.٩٨	٢٦.٠٢	١٩.٠٠	٠.٧٥	٠.٦٣	٠.٤٦	Air
			٣٠.٥٦	٢٤.٣٧	١٥.٦٩	٠.٧٤	٠.٥٩	٠.٣٨	Water
	٢.٠		٣١.٣٩	٢٥.١٩	١٧.٧٦	٠.٧٦	٠.٦١	٠.٤٣	Air
			٢٩.٣٢	٢٢.٧٢	١٤.٤٦	٠.٧١	٠.٥٥	٠.٣٥	Water
٦٠	٠.٥	٤٥.١٥	٣٦.٥٧	٣٣.٤١	٢٧.٠٤	٠.٨١	٠.٧٤	٠.٦٠	Air
			٣٤.٧٧	٣١.٦١	٢٤.٣٨	٠.٧٧	٠.٧٠	٠.٥٤	Water
	١.٠		٣٥.٢٢	٣١.٦١	٢٣.٩٣	٠.٧٨	٠.٧٠	٠.٥٣	Air
			٣٣.٤١	٢٩.٧٠	٢١.٦٧	٠.٧٤	٠.٦٦	٠.٤٨	Water
	١.٥		٣٤.٣١	٢٩.٣٥	٢٢.٥٨	٠.٧٦	٠.٦٥	٠.٥٠	Air
			٣٢.٩٦	٢٧.٠٩	٢٠.٣٢	٠.٧٣	٠.٦٠	٠.٤٥	Water
	٢.٠		٣٥.٦٧	٢٧.٩٩	١٩.٨٧	٠.٧٩	٠.٦٢	٠.٤٤	Air
			٣٤.٧٧	٢٦.١٩	١٧.٦١	٠.٧٧	٠.٥٨	٠.٣٩	Water
٩٠	٠.٥	٤٨.٢٠	٤١.٤٥	٣٧.٦٠	٢٩.٨٨	٠.٨٦	٠.٧٨	٠.٦٢	Air
			٣٩.٥٢	٣٦.١٥	٢٦.٩٩	٠.٨٢	٠.٧٥	٠.٥٦	Water
	١.٠		٣٩.٠٤	٣٦.١٥	٢٧.٤٧	٠.٨١	٠.٧٥	٠.٥٧	Air
			٣٧.٦٠	٢٤.٢٢	٢٥.٥٥	٠.٧٨	٠.٧١	٠.٥٣	Water
	١.٥		٣٧.٦٠	٣٣.٧٤	٢٥.٠٦	٠.٧٨	٠.٧٠	٠.٥٢	Air
			٣٥.٦٧	٣١.٣٣	٢٣.١٤	٠.٧٤	٠.٦٥	٠.٤٨	Water
	٢.٠		٣٦.٦٣	٣٠.٨٥	٢٣.١٤	٠.٧٦	٠.٦٤	٠.٤٨	Air
			٣٥.١٩	٢٨.٤٤	٢٠.٢٤	٠.٧٣	٠.٥٩	٠.٤٢	Water

f_{cua} = Compressive strength (cube) after exposure to fire flame.
f_{cub} = Compressive strength (cube) before exposure to fire flame.

٤-٣-١ Air- Cooling

The residual compressive strength values were found for the two series exposed to ٤٠٠ºC-fire flame temperature and for ages ٣٠, ٦٠,and ٩٠ days and for all periods of exposure. The percentage of the residual compressive strengths was (٧٠ – ٨٣ %) for series A and (٧٠ – ٨٥ %) for series B, for all periods of exposure. Surface cracks of about (١mm) width took place on the concrete specimens. These results agreed with that obtained by other investigators, Neville and Brooks [٣٨] result was ٨٥٪, Collet and Tavernier [٦٧] obtained ٧٥ %, while Abram's [١٠] result was ٨٦ %. Logothetis and Economou [٣٩] result was ٧٣ %. Al–Ausi and Faiyadh [١٦] result was (٧٩–٩٠٪), while Al–Owaisy [٥٤] obtained (٧٦–٧٨ %). The test specimens showed a further loss in compressive strength ranging from (٥٩ – ٧٥ %) and (٦١– ٧٨ %) for mix A and B respectively at ٥٠٠ºC fire flame temperature for all ages and all periods of exposure. It is observed that the colour of the concrete specimens changed to pink and increased in intensity. This may be due to hydration conditions of iron oxide component and other mineral constituents of the fine and coarse aggregates [٥ , ٩, ٦٨]. The surface – cracks increased in number, length and depth due to temperature rise. These results of the residual compressive strength at ٥٠٠ºC were similar to the results obtained by others [٥,١٠,١٦,٢٨, ٣٩, ٤٥, ٥٤.,]. At ٧٠٠ºC of fire flame exposure and for all ages of specimens and periods of exposure, the percentage of the retaining compressive strengths was (٤٣ – ٦٠ %) for series A and (٤٣ – ٦٢ %) for series B, while Logothetis and Economou's [٣٩] result was ٤٣ % at ٧٠٠ºC . The difference can be partly explained by the factor which affects the residual strength of heated concrete and this is the amount of moisture driven off during heating, the greater the amount of moisture lost, the lower strength [١٧,٢٨]. Moreover the decrease in compressive strength of concrete is attributed to the break-down of interfacial bond due to incompatible volume change between cement paste and aggregate during heating and cooling (٣ , ١١ , [٦٩] and the formation of relatively week hydration products (dehydration of the calcium – silica hydrate in cement paste).

٤-٣-٢ Water- Cooling

From previous tables and figures, it can be observed that the water – cooled concrete specimens suffer more reduction in strength than the air – cooled specimens. The residual compressive strength after exposure to fire flame and cooling in water was as follows:-

At ٤٠٠°C the residual compressive strengths compared to the original strength before exposure to fire flame were (٦٦ – ٨٠ %) for series A and (٦٧ – ٨٢ %) for series B . These results conformed to Al – Ausi and Faiyadh result which was (٦٠ – ٧١ %) and Al – Owaisy result that was ٧١ %. Raising the temperature to ٥٠٠°C caused concrete strength retaining (٥٢– ٧٠ %) from the original strength before burning for series A and (٥٥ – ٧٥ %) for series B. These results were similar to that obtained by Al- Ausi and Faiyadh and Al- Owaisy which were (٣٥ – ٥٣ %), and ٥٨٪ respectively. Further loss in compressive strength took place in specimens burnt to ٧٠٠°C and then cooled in water. The residual compressive strength ranged between (٣٣ – ٥٥ %) for series A and (٣٥ – ٥٦ %) for series B. The concrete specimens cooled in water exhibited more deterioration than the air–cooled specimens, the further reduction in strength was ranging from (٣-٥٪), (٤-٧٪) and (٥-١٠٪) for series A and (٢-٥٪), (٣-٦ %) and (٥-٨٪) for series B compared with the strength of companion air-cooled specimens. This is because of the destructive thermal shock produced when quenching the hot specimens in water[٧٠] and the penetration of the water into the concrete pores and cracks .The penetration of water into dry concrete pores is known to result in the dilation of the cement gel thereby decreasing the cohesive forces between the cement particles and reducing the strength.[١٦] Also, decomposition of the calcium hydroxide occurs, so that the lime is left behind in consequence of drying. If , however , after cooling, water ingress into concrete takes place the rehydration of the lime can be disruptive [٥].

The test results show that large proportion of the decrease in compressive strength occurs at the first ٠.٥-hour period of exposure. It can be seen that the adverse effect of fire is pronounced on series A more than on series B, while the effect is equal when the period of fire exposure reaches two hours.

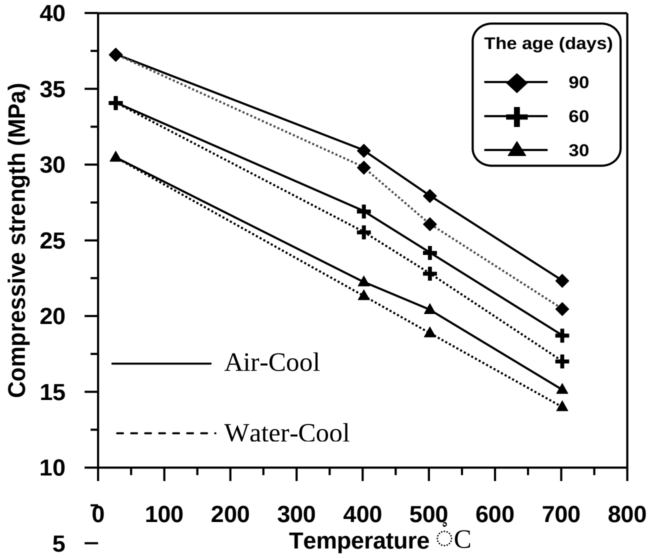


Fig (4-9) The effect of fire flame on the compressive strength of series A at 0.5 hour period of exposure.

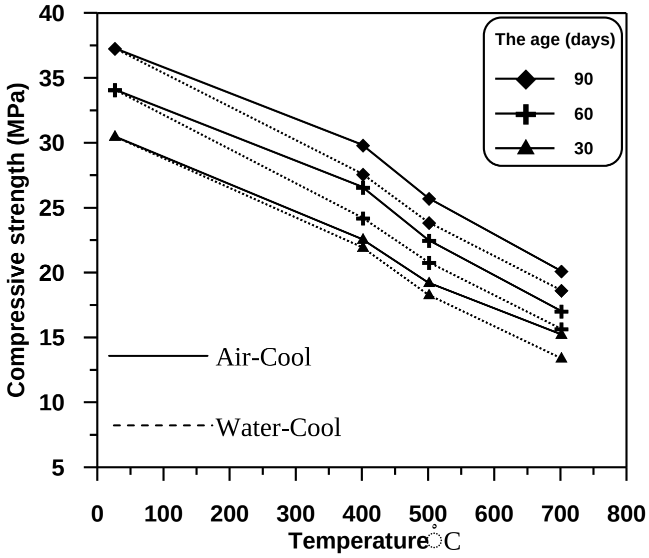


Fig (4-10) The effect of fire flame on the compressive strength of series-A at 1.0 hour period of exposure .

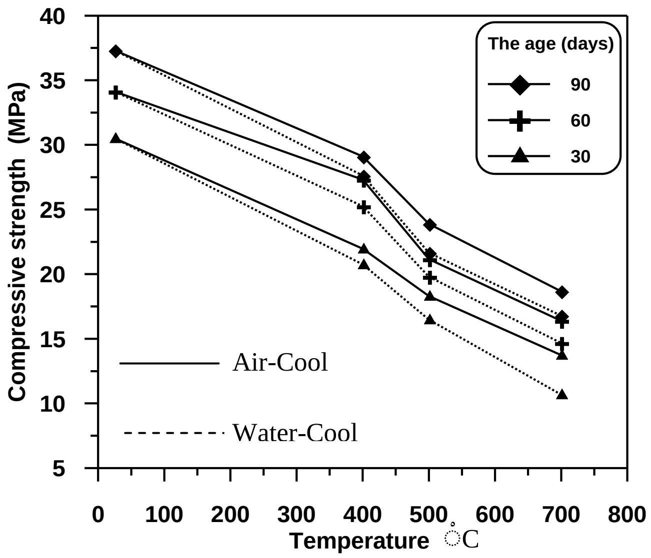


Fig.(4-11) The effect of fire flame on the compressive strength of series-A at 1.5 hours period of exposure.

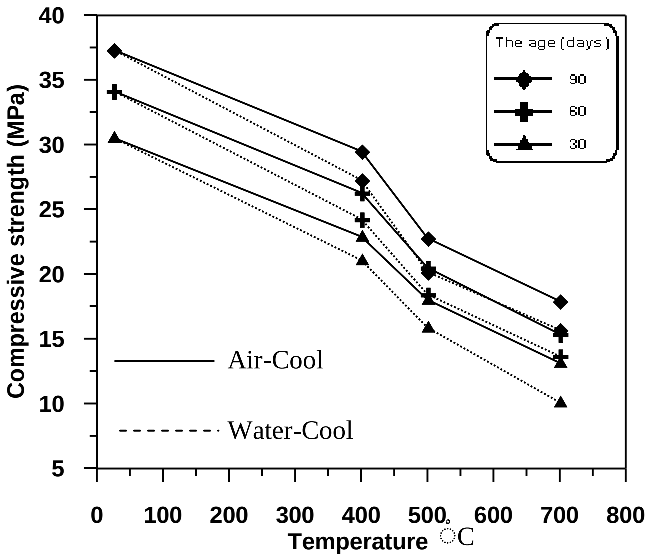


Fig.(3-12) The effect of fire flame on the compressive strength of series - A at 2.0 hours period of exposure.

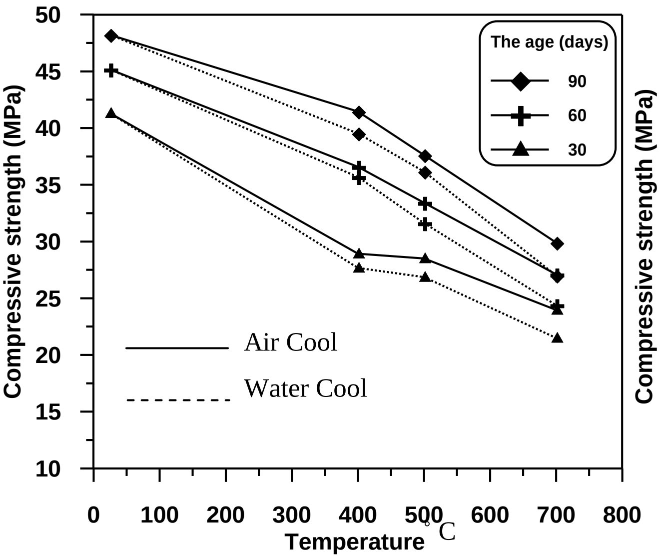


Fig.(4-13) The effect of fire flame on the compressive strength of series-B at 0.5 hour period of exposure.

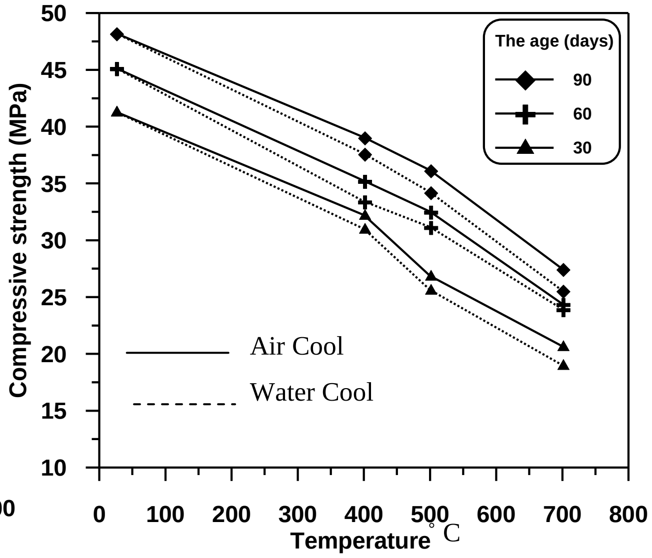


Fig.(4-14) The effect of fire flame on the compressive strength of series-B at 1.0 hour period of exposure.

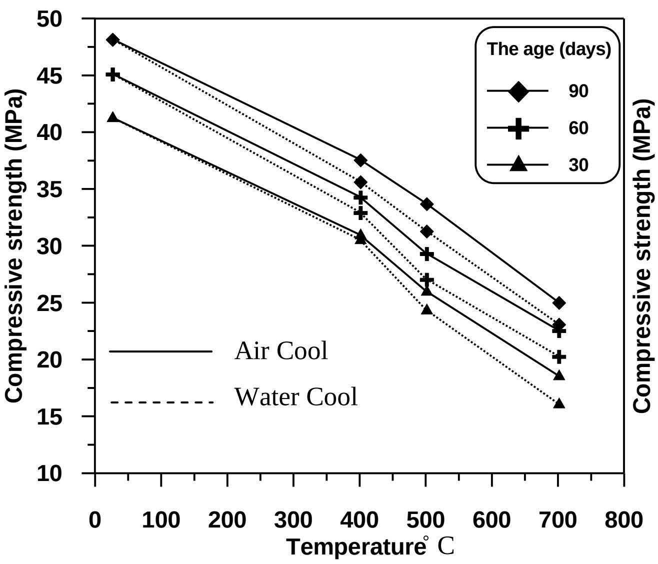


Fig.(4-15) The effect of fire flame on the compressive strength of series-B at 1.5 hours period of exposure.

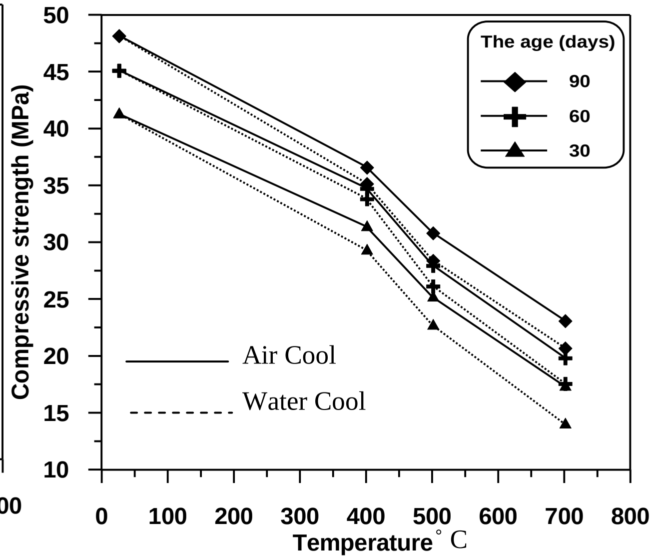


Fig.(4-16) The effect of fire flame on the compressive strength of series-B at 2.0 hours period of exposure.

٤-٣-٣ The effect of the age of concrete at exposure to fire

The test results show that the residual compressive strength after exposure to fire, the reduction at ٣٠ days age was more than the reduction at ٦٠ and ٩٠ days. This may be attributed to the fact that hydration of cement paste is more complete at later ages.

٤-٣-٤ Comparison between residual compressive strength results and recommended design curves (CEB & CEN)

Euro-Code Nos.٢ and ٤ [Committee European de Normalization] (CEN) (١٩٩٣,١٩٩٤), specified rules for strength and deformation properties of uniaxially stressed, normal weight aggregate (NWA) (siliceous and calcareous) concrete at temperatures up to ١٢٠٠ºC.The CEB Bulletin D' Information No.٢٠٨ (CEB) (١٩٩١),[٢٥]recommended design curves for compressive strength of siliceous (NWA) concrete based on the work of many investigators.The residual compressive strength results obtained from tests carried out by many researchers in addition to the present study are plotted with the CEN and CEB design curves as shown in Fig.[٤-١٧].It can be seen that the temperature range between (٢٠٠-٣٠٠˚C)was characterized by a maximum residual strength(Malhotra[٩] Abrams[١٠] ,Collet et al[٦٧],Al-Ausi and Faiyhad[١٦] and Al-Owasiy[٥٤]). Improvement in compressive strength was observed to a maximum value in a temperature ranged between (١٠٠-١٥٠˚C) (Fahmi and Asa'ad , Al-Ausi and Faiyhad and Habeeb). This behavior was attributed to the cement paste response due to causing weakening of bond between (٥٠-١٥٠˚C), thereafter, densification due to thermal drying contributed to the increase of strength.. Beyond ٣٠٠˚C there was a further reduction in compressive strength. In the present study some of test results of residual compressive strength were found to lie between CEB and CEN curves, while other results were far away from the CEB curve and near the CEN curve especially at ٤٠٠˚C and ٥٠٠˚C of fire exposure. At ٧٠٠˚C the test results were found to be near the CEN curve only. From the figure, it can be concluded that the test results of the current study have better agreement with CEN design curve than with CEB curve.

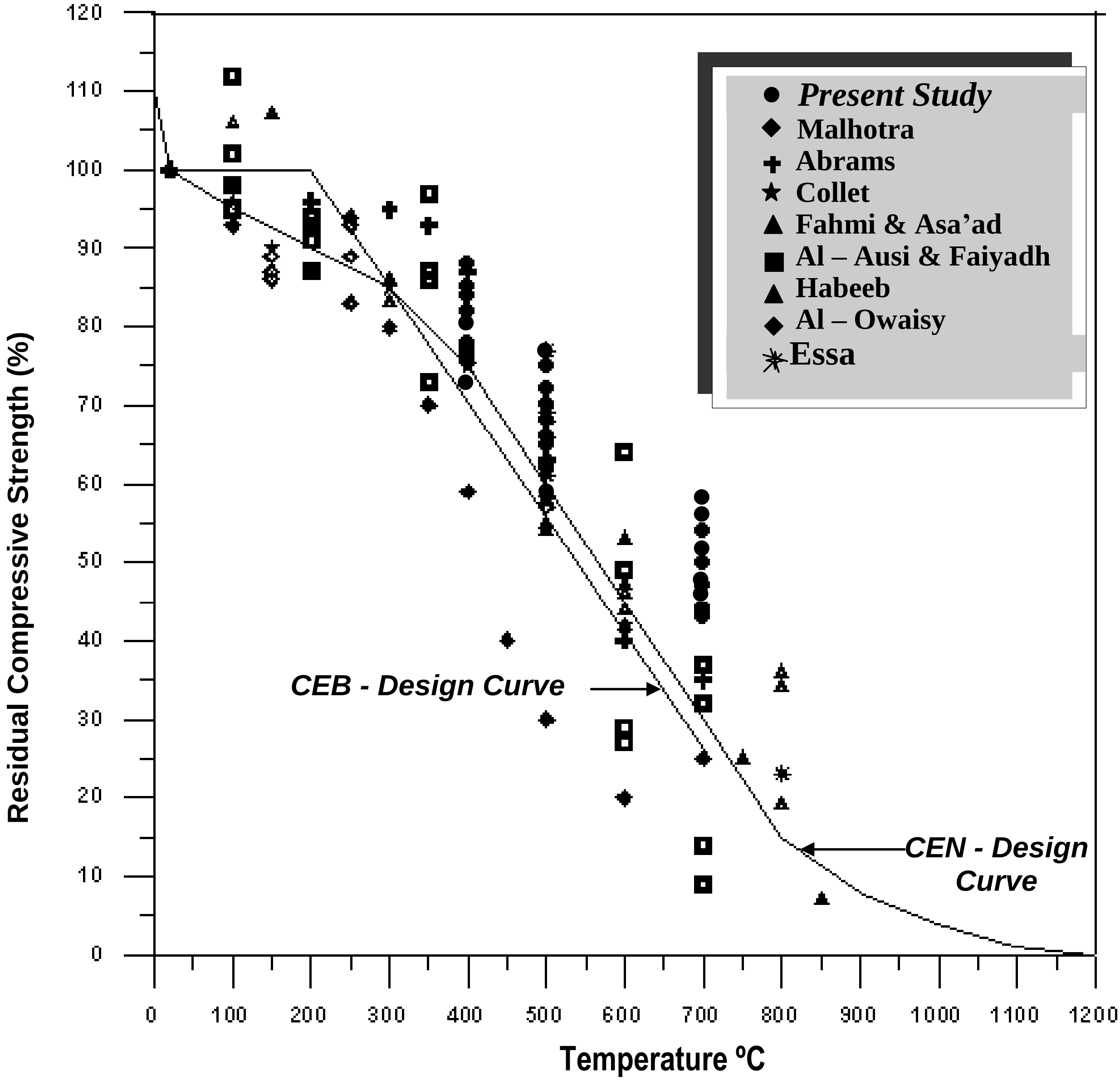


Fig.[٤-١٧]: Comparison of residual compressive strength results and the recommended design curves of CEB and CEN.

٤-٤ Splitting Tensile Strength.

The values of splitting tensile strength of the specimens considered in the present investigation are abstracted in Tables [٤-٥] and [٤-٦]. The relation between the splitting tensile strength and fire flame exposure temperature is shown in Figures [٤-١٨] to [٤-٢١] for series A and (٢-٢٢) to (٤-٢٥) for series B. It is clear from the test results that the splitting tensile strength is more sensitive to the exposure to high temperatures than the concrete compressive strength. It can be observed that concrete deteriorates at a faster rate when tested in tension rather than in compression. This observation confirms the results obtained by other investigators [٣,٩.١٦].

Table [٤- ٥]:*Test values of splitting tensile strengths of concrete specimens of series- A-after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Splitting Tensile Strength (MPa)				Ratios f_{sa}/f_{sb}			Type of Cooling
		Temperature °C							
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)	
٣٠	٠.٥	٣.٤٣	٢.٤٧	١.٦٥	١.١٠	٠.٧٢	٠.٤٨	٠.٣٢	Air
			٢.٣٣	١.٢٠	٠.٧٢	٠.٦٨	٠.٣٥	٠.٢١	Water
	١.٠		٢.٤٠	١.٦٠	٠.٨٩	٠.٧٠	٠.٤٧	٠.٢٦	Air
			٢.٢٣	١.٣٧	٠.٦٢	٠.٦٥	٠.٤٠	٠.١٨	Water
	١.٥		٢.٤٧	١.٧٢	٠.٧٩	٠.٧٢	٠.٥٠	٠.٢٣	Air
			٢.٢٦	١.٤٤	٠.٣٨	٠.٦٦	٠.٤٢	٠.١١	Water
	٢.٠		٢.٣٠	١.٦١	٠.٦٢	٠.٦٧	٠.٤٧	٠.٢١	Air
			٢.٠٢	١.٢٧	٠.٣٤	٠.٥٩	٠.٣٧	٠.١٠	Water
٦٠	٠.٥	٣.٨٢	٢.٨٧	٢.٠٦	١.٣٠	٠.٧٥	٠.٥٤	٠.٣٤	Air
			٢.٦٧	١.٩٥	١.٠٧	٠.٧٠	٠.٥١	٠.٢٨	Water
	١.٠		٢.٧١	١.٩٩	١.١١	٠.٧١	٠.٥٢	٠.٢٩	Air
			٢.٥٢	١.٦٨	٠.٨٤	٠.٦٦	٠.٤٤	٠.٢٢	Water
	١.٥		٢.٨٣	٢.٠٦	١.١٥	٠.٧٤	٠.٥٤	٠.٣٠	Air
			٢.٦٤	١.٨٠	٠.٨٤	٠.٦٩	٠.٤٧	٠.٢٢	Water
	٢.٠		٢.٧٢	١.٨٣	٠.٩٩	٠.٧١	٠.٤٨	٠.٢٥	Air
			٢.٤١	١.٦٤	٠.٧٧	٠.٦٣	٠.٤٣	٠.٢٠	Water
٩٠	٠.٥	٤.٠٥	٣.٠٨	٢.٢٧	٢.٣٥	٠.٧٦	٠.٥٨	٠.٤١	Air
			٢.٩٢	١.٩٤	٢.١١	٠.٧٢	٠.٥٢	٠.٣٣	Water
	١.٠		٢.٩٦	٢.٣١	١.٥٨	٠.٧٣	٠.٥٧	٠.٣٩	Air
			٢.٧٥	٢.٠٠	١.٢٢	٠.٦٨	٠.٤٩	٠.٣٠	Water
	١.٥		٣.٠٤	٢.٤٣	١.٥٤	٠.٧٥	٠.٦٠	٠.٣٨	Air
			٢.٨٨	٢.١٥	١.١٧	٠.٧١	٠.٥٣	٠.٢٩	Water
	٢.٠		٢.٨٣	٢.١٩	١.٣٨	٠.٧٠	٠.٥٤	٠.٣٤	Air
			٢.٧٩	١.٨٦	١.٠٥	٠.٦٩	٠.٤٦	٠.٢٦	Water

f_{sa} = Splitting tensile strength after exposure to fire flame**.**

f_{sb} = Splitting tensile strength before exposure to fire flame.

Table[٤- ٦]:*Test values of splitting tensile strengths of concrete specimens of series- B-after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Splitting Tensile Strength (MPa)				Ratios f_{sa}/f_{sb}			Type of Cooling
		Temperature °C							
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)	
٣٠	٠.٥	٤.٢٤	٣.١٠	٢.٢٩	١.٤٠	٠.٧٣	٠.٥٤	٠.٣٣	Air
			٢.٩٣	٢.٠٤	٠.٩٨	٠.٦٩	٠.٤٨	٠.٢٣	Water
	١.٠		٣.٠٥	٢.٠٨	١.١٥	٠.٧٢	٠.٤٩	٠.٢٧	Air
			٢.٨٨	١.٧٨	٠.٧٦	٠.٦٨	٠.٤٢	٠.١٨	Water
	١.٥		٣.١٤	١.٨٢	٠.٩٣	٠.٧٤	٠.٤٣	٠.٢٢	Air
			٣.٠١	١.٥٧	٠٣٨	٠.٧١	٠.٣٧	٠.٠٩	Water
	٢.٠		٢.٩٣	١.٧٠	٠.٨٥	٠.٦٩	٠.٤٠	٠.٢٠	Air
			٢.٦٧	١.٤٠	٠.٤٢	٠.٦٣	٠.٣٣	٠.١٠	Water
٦٠	٠.٥	٤.٥٠	٣.٣٦	٢.٧٠	١.٦٢	٠.٧٥	٠.٦٠	٠.٣٦	Air
			٣.٢٠	٢.٣٤	١.٤٤	٠.٧١	٠.٥٢	٠.٣٢	Water
	١.٠		٣.٠٦	٢..٥٢	١.٥٣	٠.٦٨	٠.٥٦	٠.٣٤	Air
			٢.٧٩	٢.٢٥	١.١٧	٠.٦٢	٠.٥٠	٠.٢٦	Water
	١.٥		٣.٣٦	٢.٤٣	١.٣٥	٠.٧٥	٠.٥٤	٠.٣٠	Air
			٣.٢٠	٢.٢١	٠.٩٩	٠.٧١	٠.٤٩	٠.٢٢	Water
	٢.٠		٣.٢٤	٢.٢٥	١.٠٨	٠.٧٢	٠.٥٠	٠.٢٤	Air
			٣.٠٢	١.٩٨	٠.٨٥	٠.٦٧	٠.٤٤	٠.١٩	Water
٩٠	٠.٥	٤.٨٦	٣.٦٩	٣.٢٦	٢.١٩	٠.٧٦	٠.٦٧	٠.٤٥	Air
			٣.٥٠	٢.٧٣	١.٩٠	٠.٧٢	٠.٥٦	٠.٣٩	Water
	١.٠		٣.٧٩	٢.٧٢	١.٩٤	٠.٧٨	٠.٥٦	٠.٤٠	Air
			٣.٥٥	٢.٤٨	١.٥٤	٠.٧٣	٠.٥٢	٠.٣٣	Water
	١.٥		٣.٦٠	٢.٥٣	١.٨٠	٠.٧٤	٠.٥٢	٠.٣٧	Air
			٣.٣٥	٢.٢٨	١.٤١	٠.٦٩	٠.٤٧	٠.٢٩	Water
	٢.٠		٣.٥٥	٢.٣٨	١.٤٦	٠.٧٣	٠.٤٩	٠.٣٠	Air
			٣.٣٠	٢.٠٤	١.١٢	٠.٦٨	٠.٤٣	٠.٢٣	Water

f_{sa} = Splitting tensile strength after exposure to fire flame.

f_{sb} =Splitting tensile strength before exposure to fire flame.

٤-٤-١ Cooling by Air

١- At ٤٠٠°C fire flame exposure temperature, for all ages and for all periods of exposure, the residual splitting tensile strengths were in order of (٦٧– ٧٦٪) for series A and (٦٩ – ٧٦ %) for series B. The test carried out by Al-Ausi and Faiyadh [٧١] showed that the residual splitting tensile strength was ٦٧ %, whereas Al –Owaisy [٥٤] investigation was ٦٢ % ,also Takeuchi's[٧٢] result was ٦٥٪. These results were obtained at the same temperature and the same type of cooling

٢- Further decrease in splitting tensile strength ranged between (٤٧ – ٥٨ %) for series A and (٤٠ – ٦٧ %) for series B at ٥٠٠°C.These results agreed with Al-Ausi and Faiyadh, Carrete and Painter[١٤] and Hidayet's[٧٣] results obtained at the same temperature and the same cooling regime.

٣- At ٧٠٠°C fire flame temperature the effect on splitting tensile strength was more severe than on the compressive strength. The residual tensile strengths were (٢٠ – ٤٥٪) for series A and (٢١- ٤٥ %) for series B .The reduction in the splitting tensile strength can be attributed to the formation of tensile stresses during the contraction of the hardened cement paste upon cooling, which, when superimposed onto the already existing tensile stresses formed during heating would cause an increase in the amount and rate of crack formation[٦٩].

٤-٤-٢ Cooling by Water

Cooling by water causes further reduction in the strength. The residual splitting tensile strengths after exposure to fire flame and quenched in water ranged between (٥٩ – ٧٢ %) for series A and (٦٣ – ٧٢ %) for series B at ٤٠٠°C fire flame temperature.

At ٥٠٠°C fire flame caused an extra reduction in the splitting tensile strengths ranging from (٣٧ – ٥٢ %) for series A and (٣٣ – ٥٦ %) for series B.At ٧٠٠°C for all ages of specimens and periods of exposure, the residual splitting tensile strength was (١٠ – ٣٣ %) for series A and (٩– ٣٩ %) for mix B compared with the original tensile strength before exposure to fire. Cooling in water caused further reduction in splitting strength compared with companion specimens cooled in air and the difference ranged from (٤-٨٪), (٦-١٠٪) and (٦-١٢٪) for series A and (٣-٦ %), (٤-٧٪) and (٥-١٤٪) for series B at ٤٠٠ºC, ٥٠٠ºC and ٧٠٠ºC respectively. The reduction in the strength was mainly due to fast penetration of water into the pores and microcracks of concrete. The penetration of water into the dry concrete pores is known as a result of dilation of cement gel thereby decreasing the cohesive forces between the cement particles and reducing strength [٧٤, ٧٥].It is also possible that the different rate of cooling between the surface and the inner part of the specimens causes an increase in the amount and volume of cracks thus lowering the tensile strength [٧٠].

Similar trend was obtained for the splitting tensile strength effect by fire flame as in the compressive strength but a slight decrease in series B than series A was observed at exposure period of ٢.٠ hours.

٤-٤-٣ Effect of the age at heating

As in compressive strength the effect of the fire flame at ٣٠ days of age was more than at ٦٠ and ٩٠ days of ages. This is also due to further completion of hydration of the cement at later age.

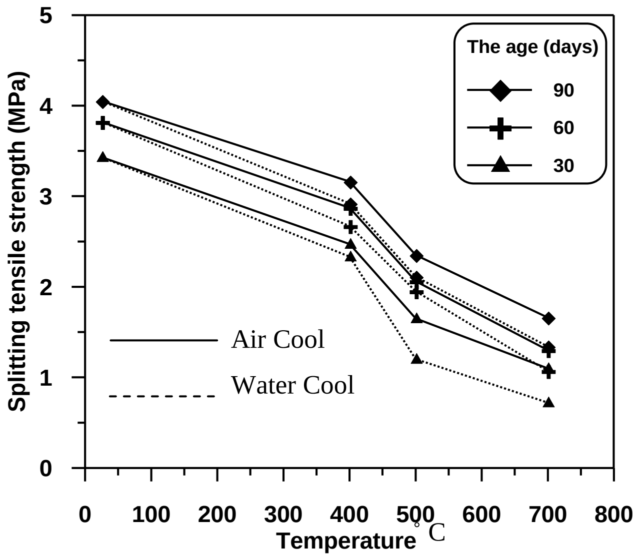


Fig.(4-18) The effect of fire flame on the splitting tensile strength of series-A at 0.5 hour period of exposure.

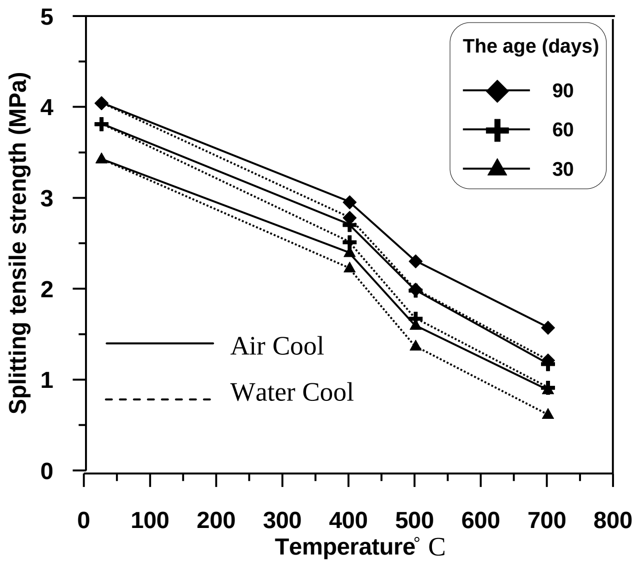


Fig.(4-19) The effect of fire flame on the splitting tensile strength of series-A at 1.0 hour period of exposure.

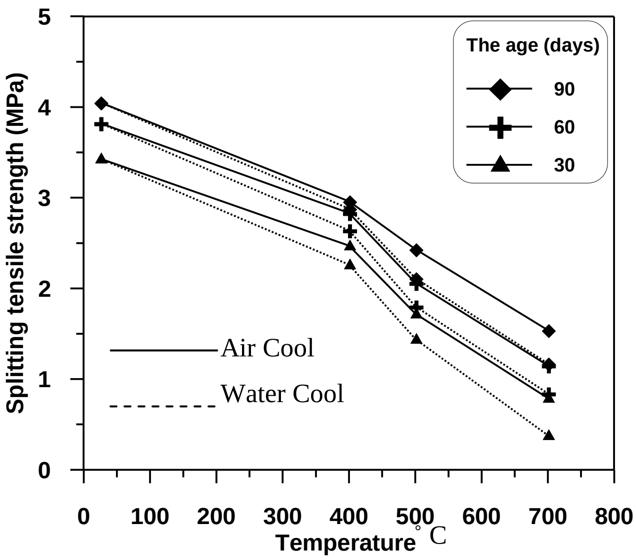


Fig.(4-20) The effect of fire flame on the splitting tensile strength of series A at 1.5 hours period of exposure.

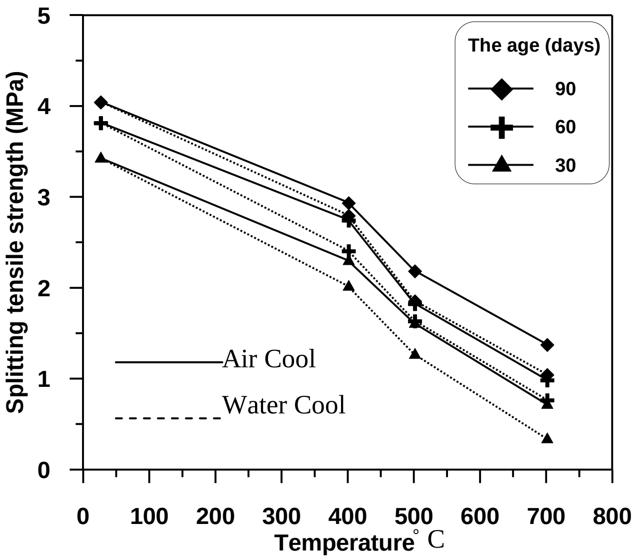


Fig.(4-21) The effect of fire flame on the splitting tensile strength of series A at 2.0 hours period of exposure.

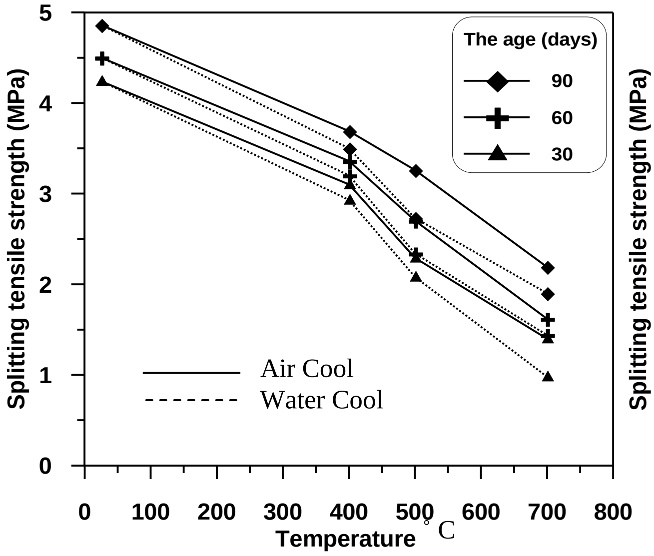


Fig (4-22) The effect of fire flame on the splitte tensile strength of series-B at 0.5 hour period of exposure.

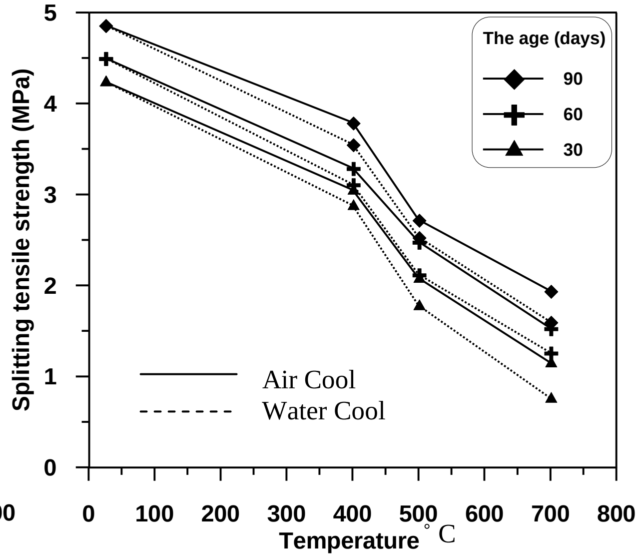
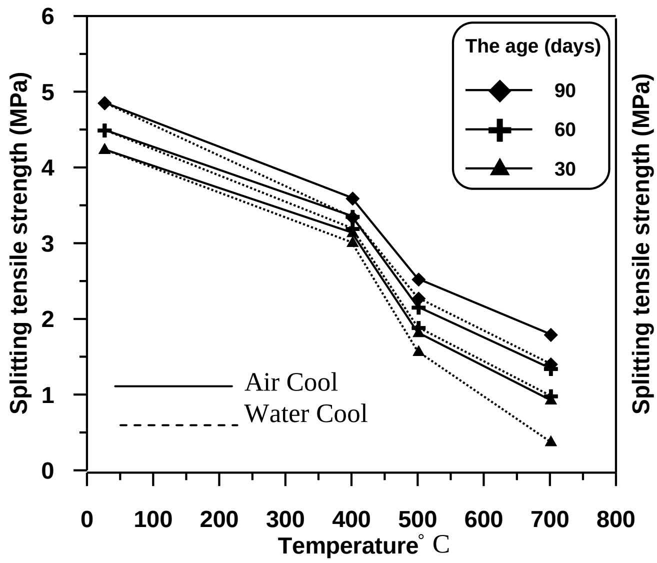


Fig.(4-23) The effect of fire flame on the splitting tensile strength of series-B at 1.0 hour period of exposure.



Fig(4-24) The effect of fire flame on the splitting tensile strength of series-B at 1.5 hours period of exposure.

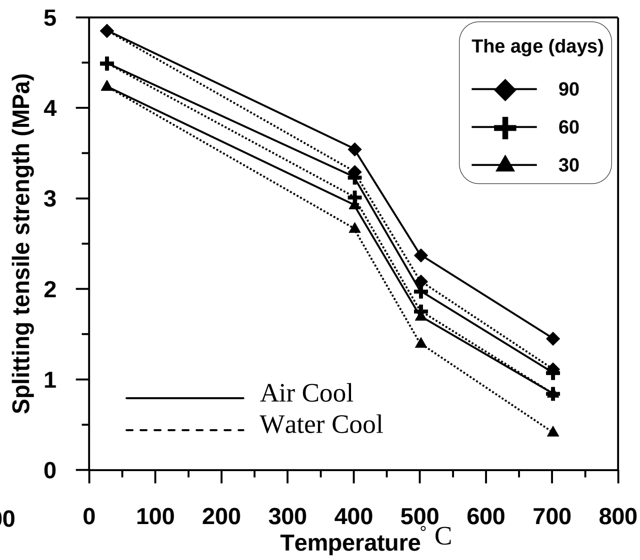


Fig.(4-25) The effect of fire flame on the splitting tensile strength of series-B at 2.0 hours period of exposure.

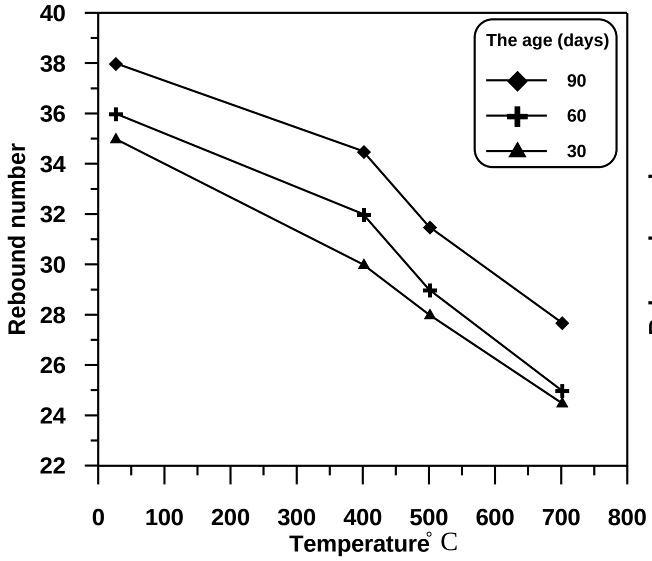


Fig.(4-54) The effect of fire flame on the rebound number of series-B- at 0.5 hour period of exposure.

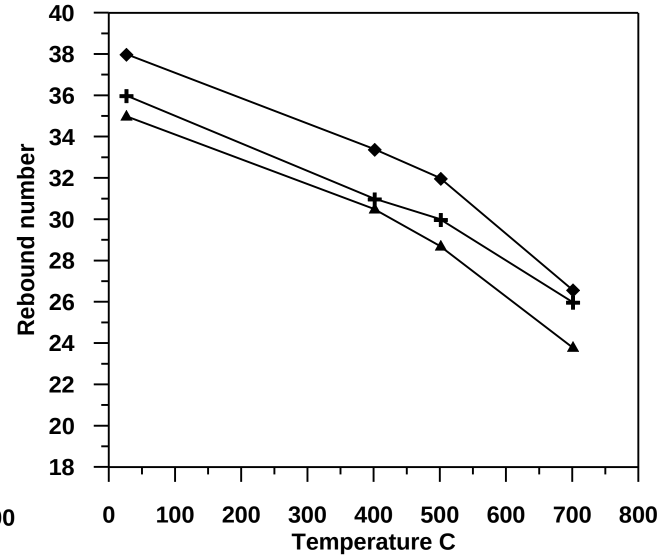


Fig.(4-55) The effect of fire flame on the rebound number of series-B- at 1.0 hour period of exposure

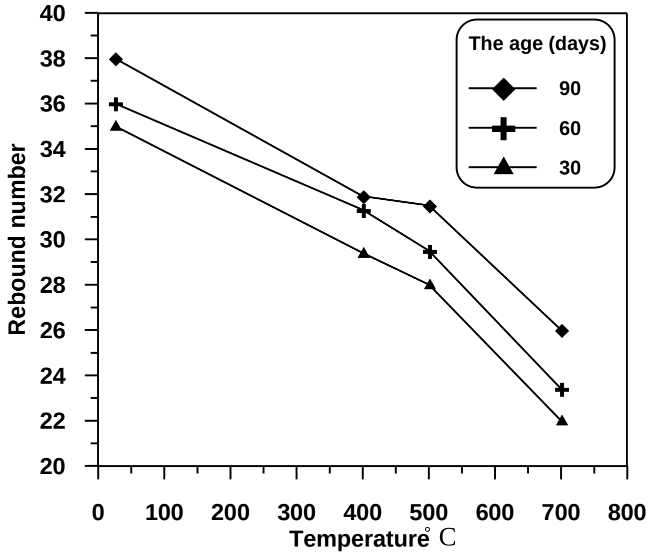


Fig.(4-56) The effect of fire flame on the rebound number of series -B- at1.5 hours period of exposure.

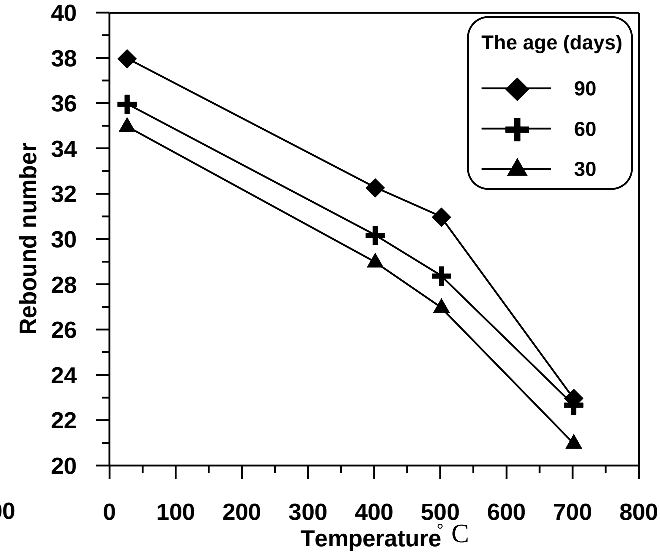


Fig.(4-57) The effect of fire flame on the rebound number of series -B- at 2.0 hours period of exposure.

٤-٩ Mathematical models for prediction of mechanical properties of concrete after exposure to fire flame

In order to obtain useful mathematical relationships, that yield good prediction accuracy, nonlinear regression is used for this purpose, due to its efficiency in derivation of exponential equations, which are extremely useful for fitting experimental data with more than one independent variable [٢٨,٧٩, ٨٠]. The exponential equation used was of the following general form:

$$Y = a_0 \cdot x_1^{a1} \cdot x_2^{a2} \cdot x_3^{a3} \ldots\ldots\ldots\ldots x_n^{n} \qquad \text{------------------ (٤-١)}$$

where:

Y = Dependent variable.

$x_1, x_2 \ldots\ldots\ldots\ldots x_n$ =Independent variables.

$a_0, a_1, a_2 \ldots\ldots\ldots a_n$ =Constants.

The mechanical properties of concrete are assumed to be known before exposure to fire flame, such as initial compressive strength , temperature of fire flame , flexural strength , ultrasonic pulse velocity , rebound number, ages, period of exposure and density . Independent variables were ranged in these equations according to their descending degree of significance.

٤-٩-١ Models for prediction of compressive strength (f_{cua})

To find the regression for prediction of (f_{cua}) .Equation (٤-١) can be written as follows :

$$f_{cua} = a_0 (f_{cub})^{a1} (T)^{a2} (U \cdot P \cdot V)^{a3} (R_N)^{a4} (\rho)^{a5} (Pe)^{a6} (Age)^{a7} \qquad \text{-------- (٤-٢)}$$

where

f_{cua} = Compressive strength of the specimens after exposure to fire flame temperature (MPa).

f_{cub} = Compressive strength of the specimens before exposure to fire flame temperature (MPa).

T = Temperature of fire flame (ºC).

U.P.V = Ultrasonic pulse velocity (km/sec)

R_N = Rebound number after exposure to fire flame.

ρ = Density of concrete after exposure to fire flame (kg/m٣) .

P_e = The period of exposure to fire flame (hour).

Age = The age of the specimens at the time of exposure (days)

٤-٩-٢ Models for prediction of splitting tensile strength f_{sa} .

The regression for prediction of f_{sa} can be written in form as follow:

$$f_{sa} = a_0 (f_{sb})^{a1} (T)^{a2} (Pe)^{a3} (Age)^{a4} (f_{cub})^{a5} \text{------(٤-٣)}$$

where :

fsa = Splitting tensile strength of specimens after exposure to fire flame (MPa) .

T = Temperature of fire flame (ºC).

P_e = Period of fire flame exposure (hours).

Age = The age of the specimens at the time of exposure (days)

f_{cub} = Compressive strength of the specimens before exposure to fire flames (MPa).

٤-٩-٣ Models for prediction of flexural strength (f_{ra}).

The regression for prediction of f_{ra} can be written in form as follows:

$$f_{ra} = a_0 (f_{rb})^{a1} (T)^{a2} (Pe)^{a3} (Age)^{a4} (f_{cub})^{a5} \text{ --------(٤-٤)}$$

where:

f_{ra} = Modulus of rupture of specimens after exposure to fire flame (MPa).

f_{rb} = Modulus of rupture of specimens before exposure to fire flame(MPa).

T = Temperature of fire flame (ºC).

P_e = The period of exposure to fire flame (hour).

Age = The age of the specimen at exposure to fire flame (days).

f_{cub} = Compressive strength of specimens before exposure to fire flame (MPa) .

٤-٩-٤ Models for prediction of static modulus of elasticity (E_c).

The regression for prediction of (E_c) can be written as follows:

$$E_{ca} = a_0 (E_{cb})^{a1} (T)^{a2} (Pe)^{a3} (Age)^{a4} (f_{cub})^{a5} \text{-------.(٤-٥)}$$

where:

E_{ca} = Static modulus of elasticity after exposure to fire flame (GPa).

E_{cb} = Static modulus of elasticity before exposure to fire flame (GPa).

T = Temperature of fire flame (°C).

P_e = The period of exposure to fire flame (hours)

Age = The age of specimen at the time of exposure to fire flame .

f_{cub} = Compressive strength of concrete specimens before exposure to fire flame (MPa) .

Table (٤-١٥) gives the values of the constants ($a_٠$, $a_١$, $a_٢$... a_n) for the regressions for f_{cua} , f_{sa}, f_{ra} and E_{ca} . This table also gives the values of the coefficients of correlation. From the values of the coefficients of correlation obtained, it can be concluded that these regressions are statically significant, for the size of data investigated of ٧٢, each with ٤-٧ independent variables.

***Table [٤-١٥]** : Coefficient of exponential regressions for the prediction of:*

١-f_{cua}

Regr. No.	a٠	a١ (f_{cub})	a٢ (T)	a٣ (U.P.V)	a٤ (R_N)	a٥ (P_e)	a٦ (Age)	a٧ (ρ)	C.C*
١	٧.٢١٧١	٠.٨٣٢٧	-٠.٣٢٨٦	٠.١٦٧٧	٠.٥٢٢	-٠.٢٣٢	-٠.٠٥٧	٠.٠٤٤	٠.٩٦٧
٢	٦٠٢.٤٩	٠.٩٠١٨	-٠.٢٨٦٤	٠.١٩١٢	٠.٦٧٩	-٠.٩٢١	-٠.٠٥٧	-	٠.٩٦٤
٣	٠.١٠٩٥	٠.٦٣٣٢	-٠.٠٢٠٥	٠.٢٩٢٥	٠.٨٦٣	٠.٠١٩١	-	-	٠.٩٥٦
٤	٠.١٢٥٨	٠.٦٣٥٣	-٠.٠٢٠٨	٠.٢٩٣٢	٠.٨٦٣	-	-	-	٠.٩٥٠
٥	٢.٣٩٥٧	١.٠١٠٨	-٠.٢٧٥٠	٠.٤٠٣١	-	-	-	-	٠.٩٣٨
٦	٣٢.٠٤٢	١.١٨٨١	-٠.٧٣٦٠	-	-	-	-	-	٠.٩١٢
٧	٣٢.٢٦٧	١.١٠٠٣	-٠.٧٢٨٤	-	-	-	-٠.٠٩٠	٠.٦٨٥	٠.٩٥٨
٨	٣٠.٥٦٠	١.١٩٠٧	-٠.٧٢٨٨	-	-	-	-٠.٠٨٩	-	٠.٩٤٧
٩	٣٣.٧٩٧	١.٠٩٩٠	-٠.٧٣٥٧	-	-	-	-	٠.٠٦٧	٠.٩٢٣

٢-f_{sa}

Regr.	a٠	a١	a٢	a٣	a٤	a٥	C.C*

No.		(f_{sa})	(T)	(P_e)	(Age)	(f_{cub})	
١	٠.٠٠٩٤١	٠.٦٨٦٢	-١.٢٩٣٤	-٠.٠٣٤٣	٠.١٥٢٠	١.٥٤٣٣	٠.٩٣٧١
٢	١٨٧٤.٧٧	١.٠٧١٧	-١.٤١٠٨	-٠.٠٥١٦	٠.١١٧٠	-	٠.٩٣٥٦
٣	٢١٩٥.٨٣	١.٣٢٦٣	-١.٤١٨٣	-٠.٥٢١٠	-	-	٠.٩٢٠٠
٤	٢٣١٦.٧٨	١.٣١٩٣	-١.٤٢٦٩	-			٠.٩١٣٦
٥	١٩٧٤.٤٣	١.٠٦٣٧	-١.٤١٨٠	-	٠.١١٧٥	-	٠.٩٢٩٣

٣- f_{ra}

Regr. No.	a٠	a١ (f_{ra})	a٢ (T)	a٣ (P_e)	a٤ (Age)	a٥ (f_{cub})	C.C*
١	٤٦٠٣.٠٢	٠.٩٦٩٠	-١.٥٤٢٧	-٠.٠٨٢	٠.٠٥٢٨	٠.٩٤٦٥	٠.٩٦٣٢
٢	٥٤٣٢.٠٦	١.١٠٤٢	-١.٥٤٢٨	-٠.٠٨٣	٠.٠٤٧٩	-	٠.٩٦٣١
٣	٥٦٧٣.٦٨	١.٢٠٨٨	-١.٥٤٣٩	-٠.٠٨٣	-	-	٠.٩٦٠٨
٤	٥٦٧٣.٦٨	١.١٠٣٥	-١.٥٥٢١	-	٠.٠٤٧٩	-	٠.٩٤٧٠
٥	٥٩٨٠.٠٥	١.٢٠٧٩	-١.٥٥٣٥	-	-	-	٠.٩٤٥٠

٤- E_{ca}

Regr. No.	a٠	a١ (E_{cb})	a٢ (T)	a٣ (P_e)	a٤ (Age)	a٥ (f_{cub})	C.C*
١	٦٣٦.٩٤٩	٢.٩٠٨	-١.٨٢٢	-٠.١٤٠	٠.٠٤٧٥	٠.٧٤٥١	٠.٩٥٧٨
٢	٨٤١٧.٧٠	١.٣٦٠	-١.٨٢٤	-٠.١٤٠	٠.٠٥٤٢	-	٠.٩٥٧٩

٣	٦٣٢٣.٣٧	١.٥٠٩	-١.٨٢٥	-٠.١٤٠	-	-	٠.٩٦٤٩
٤	٩٢٩٧.٨١	١.٣٤٨	-١.٨٣٥	-	٠.٠٥٤٦	-	٠.٩١٩٧
٥	٦٩٧٥.٠٢	١.٤٩٧	-١.٨٣٦	-	-	-	٠.٩١٨٠

* C.C … Coefficient of Correlation.

From Table [٤-١٥], the following equations could be used to estimate the corresponding values with good coefficient correlation and fewer variables introduced in them.

$$f_{cua} = 2.394(f_{cub})^{1.011}(T)^{-0.275}(U.P.V)^{0.403} \quad \text{---------.(٤-٦)}$$

$$f_{sa} = 1974.32(f_{sb})^{1.064}(T)^{-1.418}(Age)^{0.118} \quad \text{---------(٤-٧)}$$

$$f_{ra} = 5980.05(f_{rb})^{1.208}(T)^{-1.554} \quad \text{---------(٤-٨)}$$

$$E_{ca} = 6975.02(E_{cb})^{1.497}(T)^{-1.837} \quad \text{---------(٤-٩)}$$

٤–١٠ Load – Deflection Behavior

٤–١٠–١ Introduction

The deflection which occurs immediately upon application of the load is called short-term deflection or instantaneous deflection. The principal factors that influence this type of deflection are the magnitude and distribution of the load, span and conditions of end restraint, section properties including steel percentage and material properties. [٨١]

To study the influence of fire flame on the immediate deflection, twelve rectangular beams of ١٠٠-mm width, ١٥٠-mm depth and ١٠٠٠-mm length were tested in flexure. Two series of beams were cast with same mix proportion of series A and B. Two control beams were tested for each series without exposure to fire, the others were subjected to fire flame at the age of ٣٠ and ٩٠ days with temperature level of ٥٠٠°C and ٧٠٠°C for ١ hour period of exposure. The properties of the reinforced concrete beams are shown in Table [٤-١٦].

٤-١٠-٢ Ultimate Moment Strength

One-point load on simply supported beams gives only one section of maximum stresses, this occurs under the point load as shown in Plate [٤-١].

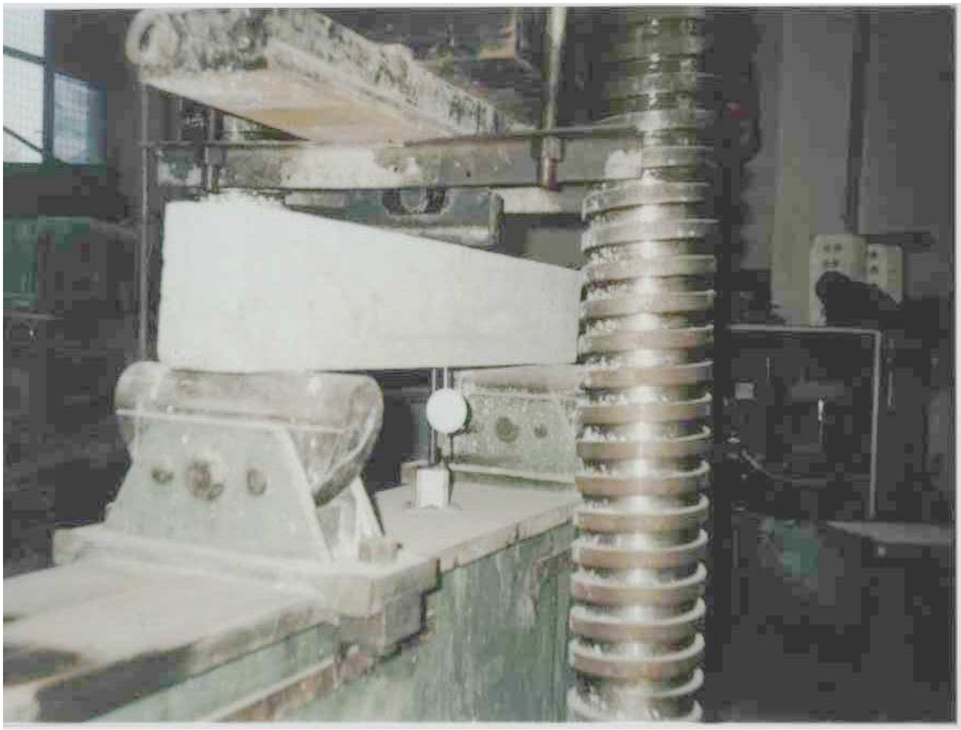


Plate [٤-١]: *Beam under testing*

The theoretical ultimate strengths of beams which were calculated according to ACI-Code [٦٦] by using Equation (٤-١١) and according to CP١١٠ [٨٢] by using Equation (٤-١٦) are shown in Table (٤-١٧).

The ACI- Code nominal moment equation is:-

$$M_n = \rho f_y b d^2 \left(1 - 0.59 \rho \frac{f_y}{f_c}\right) \quad \text{-----------------.(٤-١٠)}$$

The ultimate moment M_u (for design) is :

$$M_u = \phi M_n \quad , \quad \phi = 0.90$$

$$M_u = \phi \rho b d^2 \left(1 - 0.59 \rho \frac{f_y}{f_c}\right) \quad \text{-----------------(٤-١١)}$$

where $f_c = 0.85 f_{cu}$

The CP-١١٠ ultimate moment equations are: -

$$M_u = 0.87 f_y As Z \quad \text{-----------------(٤-١٢)}$$

$$Z = \left[1 - \frac{1.1 f_y As}{fcu.bd}\right] d \quad \text{----------------- (٤-١٣)}$$

By eliminating the safety factors,

$$Z = d - \frac{f_y As}{1.34 f_{cu} b} \quad \text{------------------(٤-١٤)}$$

$$M_n = f_y As Z \quad \text{-----------------(٤-١٥)}$$

$$M_u = 0.87 f_y As \left[d - \frac{f_y As}{1.34 f_{cu} b}\right] \quad \text{-----------------.(٤-١٦)}$$

Table [٤-١٧]: *Measured and calculated ultimate moment strengths*

Series	Beam No.	Temp (˚ C)	Age (days)	Experimental M_u (kN.m)	Calculated M_u (kN.m)	Ratio (١)/(٢)	Ratio (١)/(٣)

				(١)	ACI (٢)	CP١١٠ (٣)		
A	A١	٢٥	٣٠	١١.٥٢	٦.٨٠	٦.٤٨	١.٦٩	١.٧٨
	A٤	٢٥	٩٠	١١.٩٩	٦.٩٣	٦.٧٠	١.٧٥	١.٨١
B	B١	٢٥	٣٠	١٢.٦٩	٧.٠٨	٦.٧٩	١.٧٥	١.٨٣
	B٤	٢٥	٩٠	١٣.٧٤	٧.٢١	٦.٩١	١.٩١	١.٩٩

It can be noticed that the ACI Code method predicts the values of ultimate moment strengths very close to CP١١٠ method. But both methods underestimate the actual ultimate moment strength of beam (for conservative design). The ratio of experimental to calculated ultimate moment of the beam ranged from (١.٦٩) to (١.٩١) with an average value of (١.٧٨) when the calculations were based on ACI- Code equations and ranged from (١.٧٨) to (١.٩٩) with an average value of (١.٨٥) when the calculations were based on CP١١٠ equations. The calculated theoretical values are lower than the experimental values due to the considerable residual capacity of the section beyond reaching concrete and steel to yielding,

٤-١٠-٣ Deflection

Single load was applied at midspan because of the limitation of the machine available, which did not permit the application of two-point load to get pure constant moment region. The deflection were recorded at each stage of loading, the load at the first visible crack and at failure were recorded.

٤-١٠-٣-١ Deflection before exposure to fire flame

The failure load of the reference beams (A١, A٤, B١ and B٤) was divided by (١.٦) to calculate the service load. A comparison of the measured and computed midspan deflection of the beams at service load according to ACI Code and BS code are shown in Table [٤-١٨]. An example of the calculation of deflection is given in Appendix B.

Table [٤-١٨]: *Measured and calculated service load deflection at mid-span of the beam.*

Series	Beam No.	Temp. (° C)	Service Load (kN)	Experimental deflection (mm) (١)	Calculated deflection (mm)		Ratio (١)/(٢)	Ratio (١)/(٣)
					ACI (٢)	CP١١٠ (٣)		
A	A١	٢٥	٣٢.٠٠	١.٩٠	١.٨٢	١.٧٠	١.٠٤	١.١٢
	A٤	٢٥	٣٣.٦٣	١.٨٢	١.٩١	١.٨٠	٠.٩٥	١.٠١
B	B١	٢٥	٣٤.٤٥	١.٧٩	١.٩٠	١.٧٩	٠.٩٠	١.٠٠
	B٤	٢٥	٣٨.١٩	١.٧٣	٢.٠٥	١.٩٤	٠.٨٤	٠.٨٩

The ratio of the measured deflection to the calculated service load deflection of the beams ranged from (١.٠٤) to (٠.٨٤) with an average value of (٠.٩٣) when the calculations were based on ACI- Code and ranged from (١.١٢) to (٠.٨٩) with an average value of (١.٠) when the calculations were based on CP١١٠–Code. It can be concluded that CP١١٠ procedure gives comparable results to the measured values for all beams. This behavior was due to the fact that as the load increases, the section of the beam become closer to the partially cracked section used in CP١١٠ calculations.

٤-١٠-٣-٢ The effect of fire flame on the reinforcing steel bars

The effect of fire flame on the properties of reinforcing steel bar was summarized in Table [٤-١٩]. At ٥٠٠˚C, both burning and subsequent cooling did not affect the mechanical properties of the reinforcing steel bars, but the effect was observed at ٧٠٠˚C. The residual in the yield stress was (٨٣٪) and in the ultimate stress (٦٤.٢٪) for the bar of ٨ mm diameter .But for the bar of ١٠ mm diameter the residual in yield stress was (٨٤.٥٪) and the ultimate stress was (٨٥.٨٪). The modulus of elasticity was not affected by burning and cooling at all range of temperatures. Similar behavior was observed by others [١٢, ١٥] .

Table [٤- ١٩]: *The effect of fire flame on properties of steel bars*

Bar diameter (mm)	Exposure Temp. (˚C)	Yield stress *Nlmm* ٢	Residual Stress %	Ultimate Stress *N/mm* ٢	Residual Stress %	Modulus of elasticity E_s	Residual %
٨.٠١	٢٥	٤٦٠	١٠٠	٦٦٣	١٠٠	٢١٠	١٠٠
	٥٠٠	٤٦٠	١٠٠	٦٦٣	١٠٠	٢١٠	١٠٠
	٧٠٠	٣٨١.٨	٨٣	٥٥٨.٢	٨٤.٢	٢١٠	١٠٠
١٠	٢٥	٤٧٠.٣	١٠٠	٦٩٠.٥	١٠٠	٢١٠	١٠٠
	٥٠٠	٤٧٠.٣	١٠٠	٦٩٠.٥	١٠٠	٢١٠	١٠٠

	٧٠٠	٣٩٧.٤	٨٤.٥	٥٩٢.٤	٨٥.٨	٢١٠	١٠٠

٤-١٠-٣-٣ Deflection after burning by fire flame

The test results were summarized in Table [٤-٢٠] and the relation between the load and deflection were illustrated in figures from (٤-٥٨) to (٤-٦١). After the beams were subjected to fire flame, two types of cracks developed, the first was thermal cracks appearing in a honeycomb fashion all over the surface. They originated from top or bottoms edges and terminated near the mid-depth of the beam. The crack width was about (١-mm). The patterns of fine cracks were consistent with the release of moisture being greater in the outer layers than in the interior resulting in differential shrinkage. The second cracks were flexural tensile cracking due to loading developed in the mid-span region.

At ٥٠٠ °C, (a)- for ٣٠ days age and ١.٠ hour period of exposure, first crack and yield stress in steel were observed at load (١١.١٪) and (٨٦.o%) from the ultimate load respectively for beam A٢ and (١١.٦٪) and (٨٧.٦٪) from the ultimate load for beam B٢. (b)- for ٩٠ days age and ١.٠ hour period of exposure, first crack and yield stress in steel were observed at load (١٥.٥٪) and (٨٣.٤٪) from the ultimate load for beam A٥ and (١١.٠٪) and (٨٨.٤٪) for the beam B٥.

At ٧٠٠ °C, (a)- for ٣٠ days age and ١.٠ hour period of exposure, first crack and yield stress in steel were noticed at load (٨.٥٪) and (٨٧.١٪) from the ultimate load for beam A٣ and (٩.٥٢٪) and (٨٨.٣٪) from the ultimate load for beam B٣. (b)- for ٩٠ days age and ١.٠ hour period of exposure, first crack and yield stress in steel were observed at load (٩.٥٧٪) and (٨٧.٣٪) from the ultimate load for beam A٦ and (٨.٧٢٪) and (٨٩.٠٪) for the beam B٦.

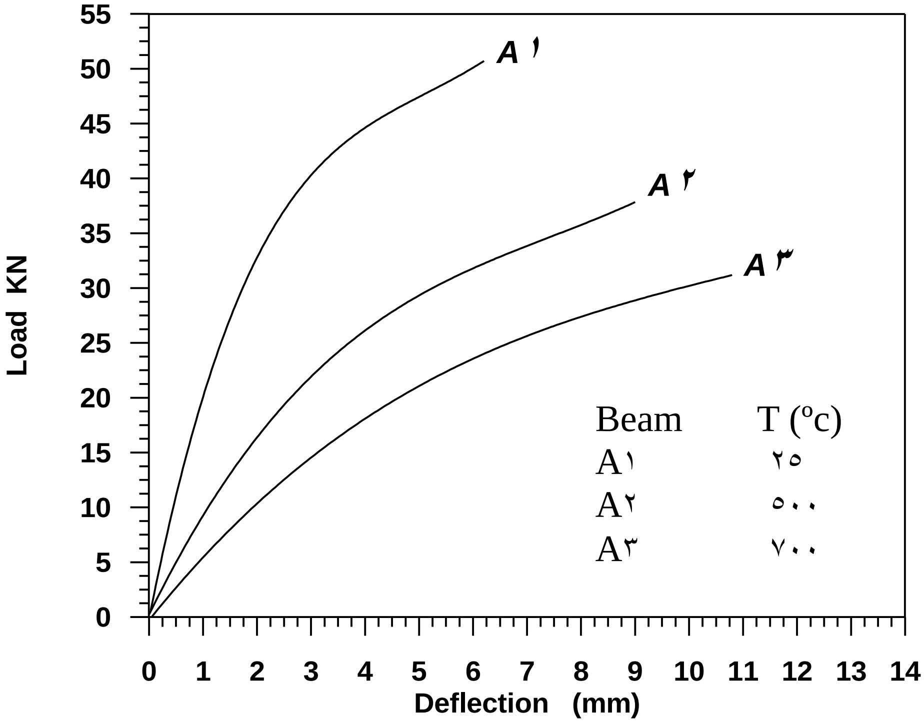


Fig [٤-٥٨] : *Load-deflection behavior for the beams of series A-٣٠ days after exposure to fire flame at ١.٠ hour*

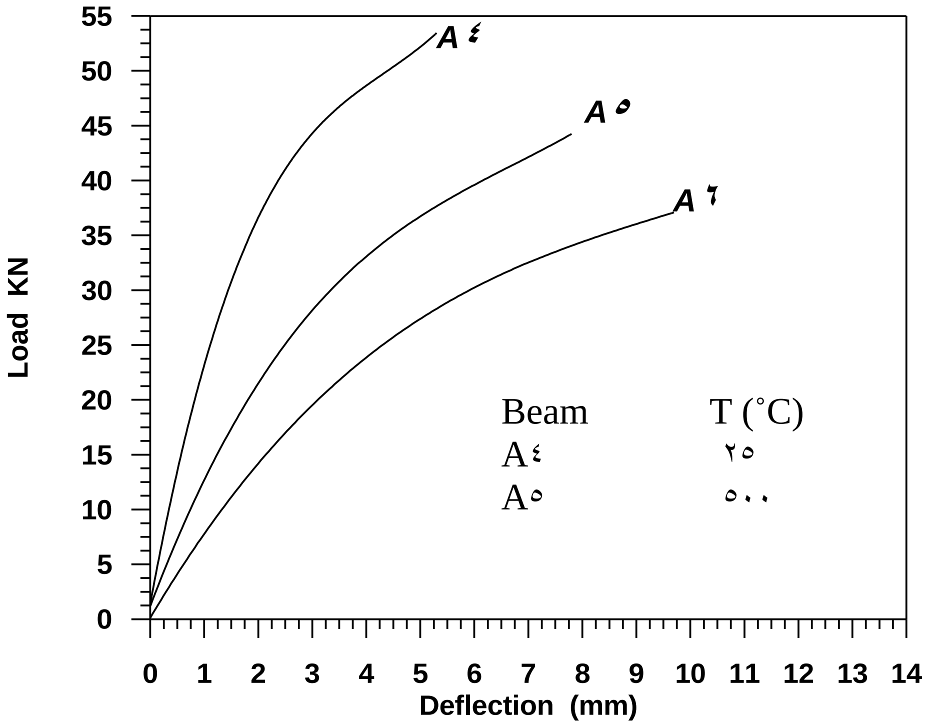


Fig [٤-٥٩]: *Load-deflection behavior for the beams of series A-٩٠ days after exposure to fire flame at ١.٠ hour*

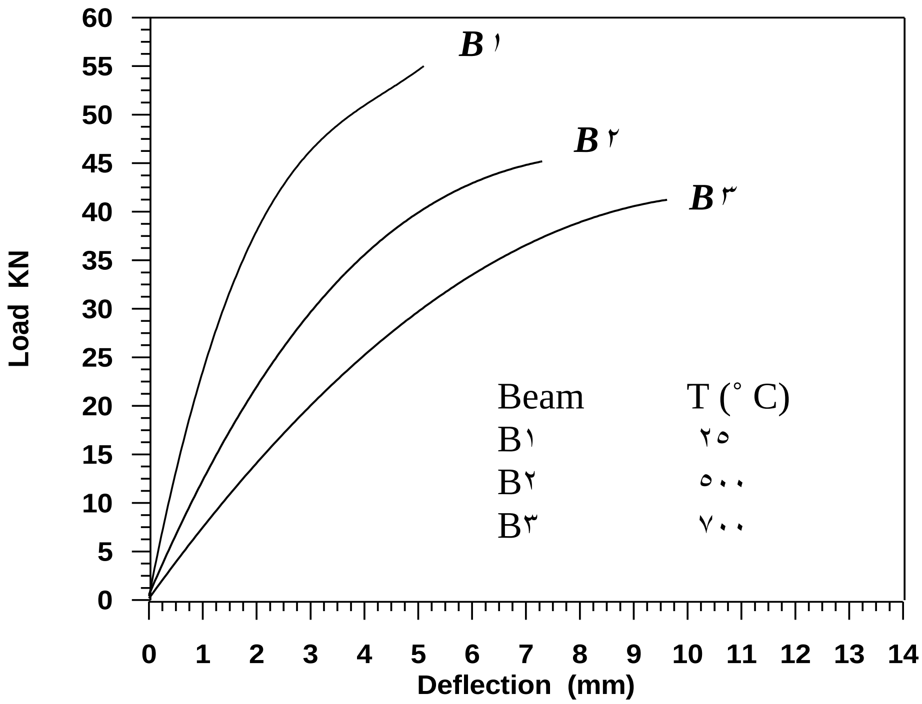


***Fig [٤-٦٠] :** load-deflection behavior for the beams of series B-٣٠ days after exposure to fire flame at ١.٠* hour

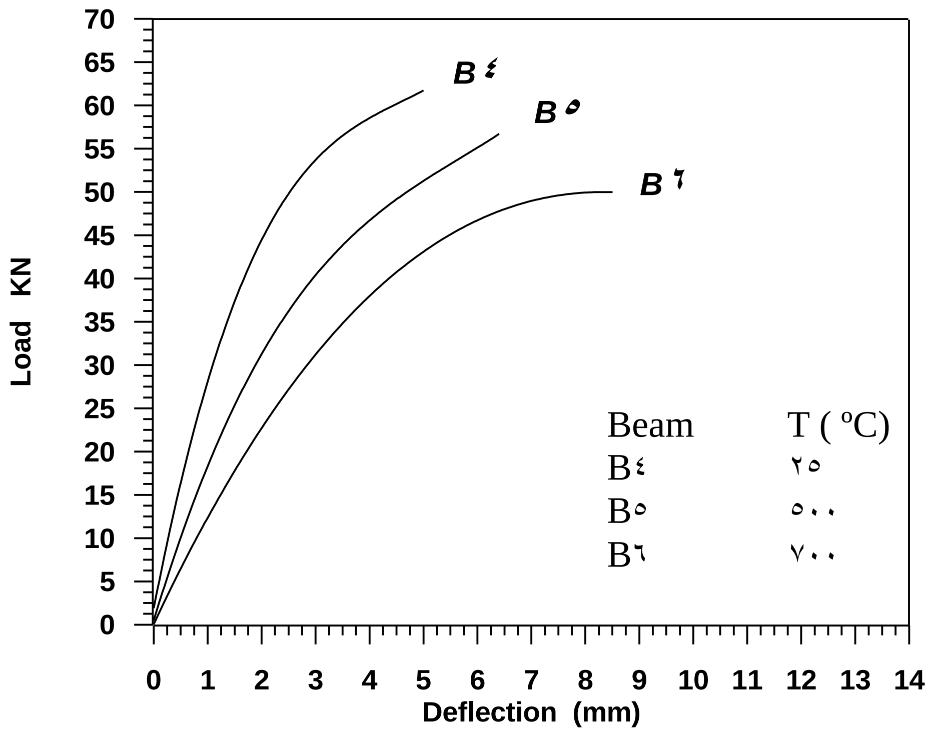


***Fig [٤-٦١]:** Load-deflection behavior for the beams of series B-٩٠ days after exposure to fire flame at ١.٠ hour*

The ability to predict the residual strength of a member following a fire is essential in the process of establishing whether and to what extent a fire damaged structure could be reinstated . A procedure for this purpose was applied to the beams tested in this work and the results are given in Table [٤-٢١]. This procedure is similar to that proposed by Malhotra (٨٣) and used by Khan and Royles(٥٠) for fire resistance computations.

On the basis of a limiting deflection an experimental value for the reduced moment of resistance of a test beam after a particular fire exposure was determined from the load-deflection data, Figures (٤-٥٨) to (٤-٦١) as now explained.

١- The ultimate load, *Pu* and its associated deflection , Δ_u for the unexposed condition , T_f =٢٥ °C were found.

٢- Using the curve appropriate to the fire exposure, a load, P', corresponding to Δ_u was established.

٣- The reduced moment of resistance , M', was evaluated from,

A typical calculation is given in an example in Appendix C. The experimental and predicted values for the reduced moment resistance are compared in Table [٤-٢١] .The results of the unexposed beams to fire agreed with the results of Khan and Royles but differed when the beams exposed to high temperatures because they used different reduction factors

Table [٤-٢١]: *Comparison of experimental and theoretical reduced moment of resistance*(٥٠)

Series	Beam No.	Age (days)	Temp. (˚C)	Moment (kN. m)	
				Experimental	Theoretical
A	A ١	٣٠	٢٥	١١.٥٢	٩.٤٨
	A ٢	٣٠	٥٠٠	٧.٢٠	٨.٤٧
	A٣	٣٠	٧٠٠	٥.٤٠	٥.٥٤
	A٤	٩٠	٢٥	١١.٩٠	١٠.٩٣
	A٥	٩٠	٥٠٠	٨.٣٣	٨.٤٧
	A٦	٩٠	٧٠٠	٤.٢٨	٦.٨٤
B	B١	٣٠	٢٥	١٢.٣٩	١١.٦٨
	B٢	٣٠	٥٠٠	١٠.٢٤	٨.٧٩
	B٣	٣٠	٧٠٠	٩.٢٤	٦.٨٨
	B٤	٩٠	٢٥	١٣.٧٤	١٣.٣٣
	B٥	٩٠	٥٠٠	١٢.٦٩	١٠.٧٧
	B٦	٩٠	٧٠٠	١١.٢٥	٨.٣٠

٤-٥ Modulus of Rupture

The test results of flexural strength of the two series are gives in Tables [٤-٧] and [٤-٨]. Figures [٤-٢٦] to [٤-٢٩] and [٤-٣٠] to [٤-٣٣] show the relationship between residual flexural strength and the fire flame temperatures for series A and B respectively. The residual flexural strength (f_{ra}) after burning at different temperatures is expressed as a ratio f_{ra} / f_{rb} where f_{rb} is the flexural strength of concrete at the same age without burning.

Table [٤-٧]: *Test values of the modulus of rupture of concrete specimens of series-A- after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Modulus of Rupture (MPa) Temperature ºC				Ratios f_{ra}/f_{rb}		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٣.٧١	٢.٧٥	١.٨٢	١.٢٦	٠.٧٤	٠.٤٩	٠.٣٤
	١.٠		٢.٦٧	٢.٠٠	١.١٥	٠.٧٢	٠.٥٤	٠.٣١
	١.٥		٢.٧٨	١.٧١	٠.٩٦	٠.٧٥	٠.٤٦	٠.٢٦
	٢.٠		٢.٦٣	١.٥٦	٠.٨٥	٠.٧١	٠.٤٢	٠.٢٣
٦٠	٠.٥	٤.١٠	٣.١٦	٢.٢١	١.٥٢	٠.٧٦	٠.٥٤	٠.٣٧
	١.٠		٣.٠٣	٢.٠٩	١.٣٥	٠.٧٤	٠.٥١	٠.٣٣
	١.٥		٣.٠٨	٢.٠١	١.١٥	٠.٧٥	٠.٤٩	٠.٢٨
	٢.٠		٢.٩٩	١.٨٥	١.٠٧	٠.٧٣	٠.٤٥	٠.٢٦
٩٠	٠.٥	٤.٢٨	٣.٣٠	٢.٤٨	١.٦٣	٠.٧٧	٠.٥٨	٠.٣٨
	١.٠		٣.٢٥	٢.٤٠	١.٣٧	٠.٧٦	٠.٥٦	٠.٣٢
	١.٥		٣.١٧	١.٩٥	١.٣٣	٠.٧٤	٠.٤٨	٠.٣١
	٢.٠		٣.٠٨	٢.٠١	١.٤١	٠.٧٢	٠.٤٧	٠.٣٣

f_{ra} = Modulus of rupture after exposure to fire flame
f_{rb} = Modulus of rupture before exposure to fire flame

***Table [٤- ٨]:** Test values of the modulus of rupture of concrete specimens of series-B- after exposure to fire flame temperatures.*

Age at Exposure (days)	Period of Exposure (hours)	Modulus of Rupture (MPa) Temperature ºC				Ratios f_{ra} / f_{rb}		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٤.٤٢	٣.١٤	٢.٣٤	١.٥٧	٠.٧١	٠.٥٣	٠.٣٧
	١.٠		٣.١٨	٢.٢١	١.٥٠	٠.٧٢	٠.٥٠	٠.٣٤
	١.٥		٣.٣٦	٢.٠٨	١.٢٤	٠.٧٦	٠.٤٧	٠.٢٨
	٢.٠		٣.١٤	١.٩٠	٠.٩٧	٠.٧١	٠.٤٣	٠.٢٢
٦٠	٠.٥	٤.٩٥	٣.٦١	٢.٧٧	١.٩٨	٠.٧٣	٠.٥٦	٠.٤٠
	١.٠		٣.٥١	٢.٦٧	١.٦٣	٠.٧١	٠.٥٤	٠.٣٣
	١.٥		٣.٧٦	٢.٥٢	١.٤٩	٠.٧٦	٠.٥١	٠.٣٠
	٢.٠		٣.٦١	٢.٢٨	١.٢٤	٠.٧٣	٠.٤٦	٠.٢٥
٩٠	٠.٥	٥.٢٥	٤.١٠	٣.١٥	٢.١٥	٠.٧٨	٠.٦٠	٠.٤١
	١.٠		٤.١٥	٣.٠٥	١.٨٩	٠.٧٩	٠.٥٨	٠.٣٦
	١.٥		٣.٩٩	٢.٧٨	١.٥٢	٠.٧٦	٠.٥٣	٠.٢٩
	٢.٠		٣.٨٩	٢.٦٣	١.٦٨	٠.٧٤	٠.٥٠	٠.٣٢

f_{ra} = Modulus of Rupture after exposure to fire flame
f_{rb} = Modulus of Rupture before exposure to fire flame

At ٤٠٠ºC the residual flexural strengths were in the range of (٧١ – ٧٧ %) for series A, while series B showed a residual flexural strength of order (٧١–٧٩ %).

At ٥٠٠ºC the specimens exhibited a loss of flexural strength, the residual flexural strengths were (٤٢ – ٥٨ %) for series A and (٤٨- ٦٠ %) for series B. Hair-cracks and pink color were observed in the prisms. This trend is similar to that obtained by Habeeb. [٢٨] A closer result was found by Purkiss, [٤٦] that was ٤٠٪ from the original value.

At ٧٠٠ºCthe residual flexural strengths were (٢٣ – ٣٨ %) and (٢٢ – ٤١ %) for series A and series B respectively. The effect of the fire was seen to be in a range less than that obtained by Asa’ad [٥١] and Purkiss [٤٦] . These results may are attributed to less homogeneity of the heat in stoves than the heat in the furnace especially at temperature more than ٥٠٠°C.

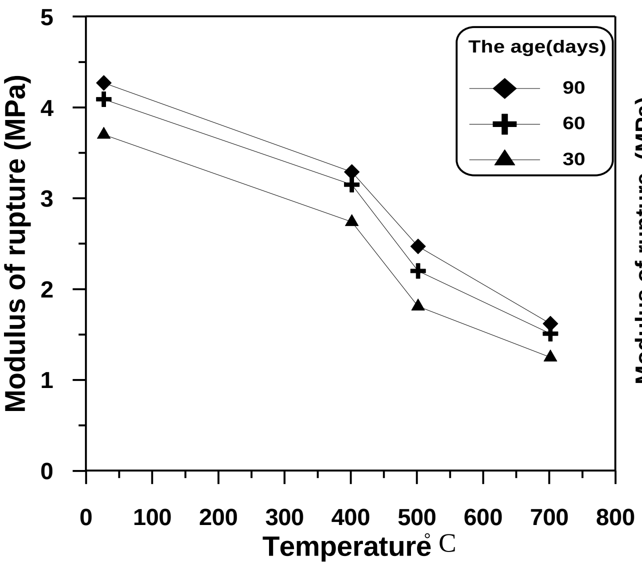


Fig.(4-26) The effect of fire flame on the modulus of rupture of series-A at 0.5 hour period of exposure.

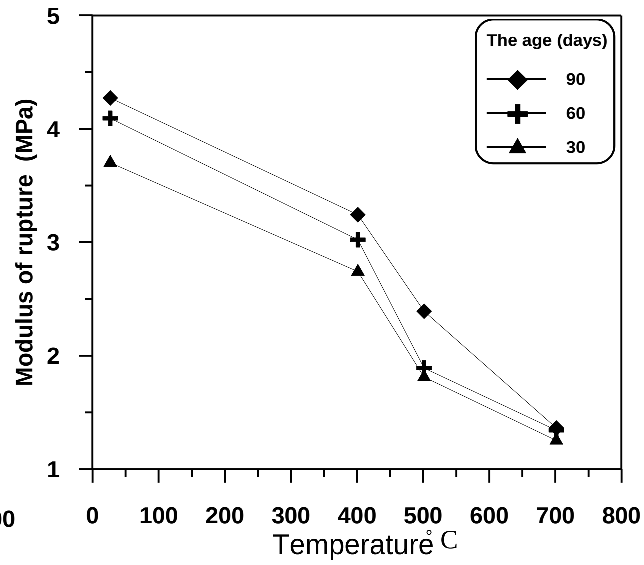


Fig.(4-27) The effect of fire flame on the modulusof rupture of series -A at 1.0 hour period of exposure

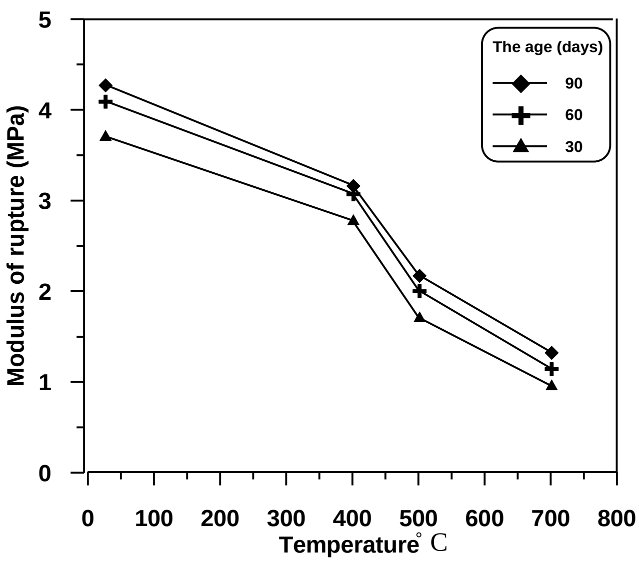


Fig (4-28) The effect of fire flame on the modulus of rupture of series -A at 1.5 hours period of exposure.

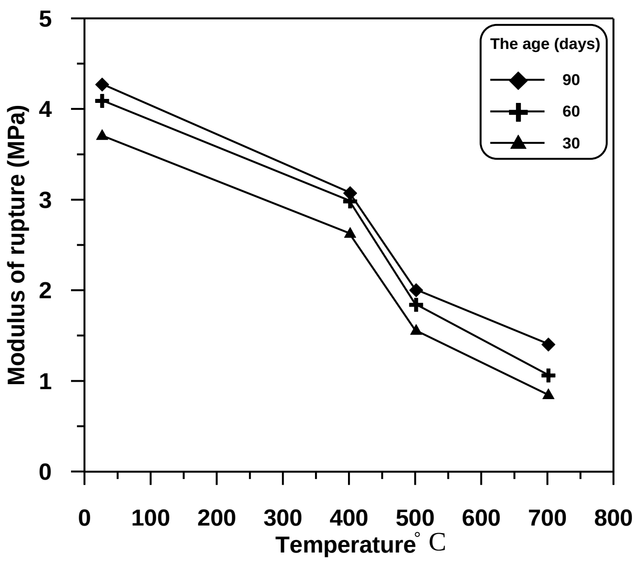


Fig.(4-29) The effect of fire flame on the modulus of rupture of series-A at 2.0 hours period of exposure.

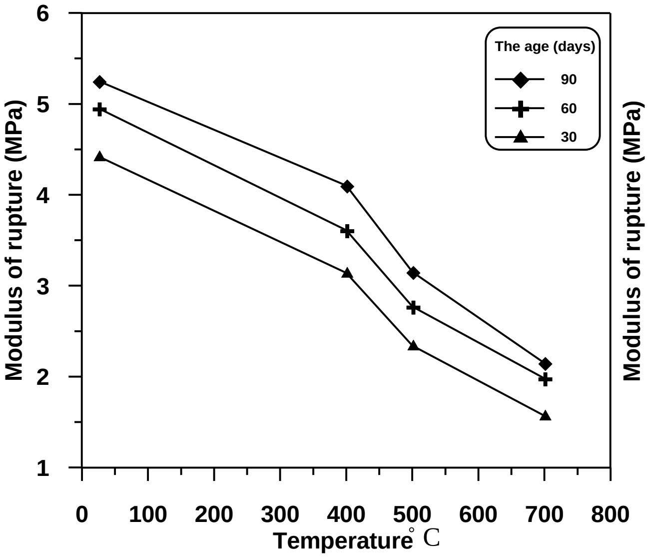


Fig.(4-30) The effect of fire flame on the modulus of rupture of series -B at 0.5 hour period of exposure.

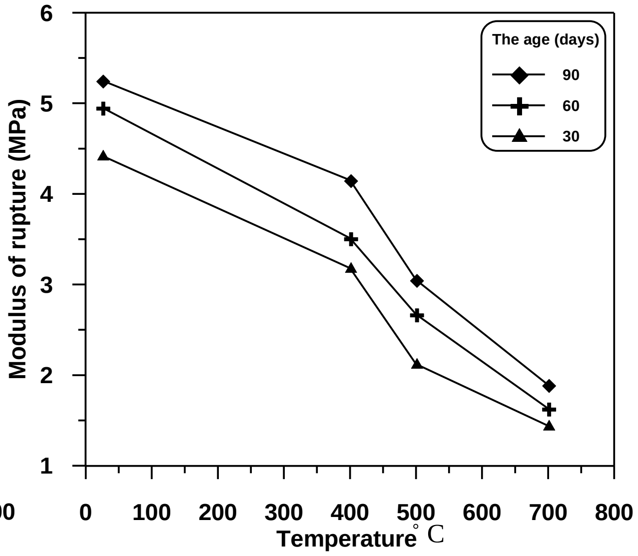


Fig.(4-31) The effect of fire flame on the modulus of rupture of series -B at 1.0 hour period of exposure.

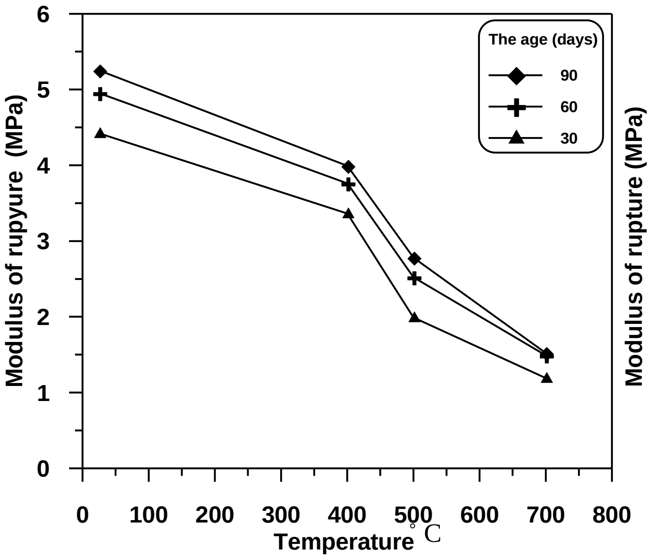


Fig.(4-32) The effect of fire flame on the modulus of rupture of series -B at1.5 hours period of exposure.

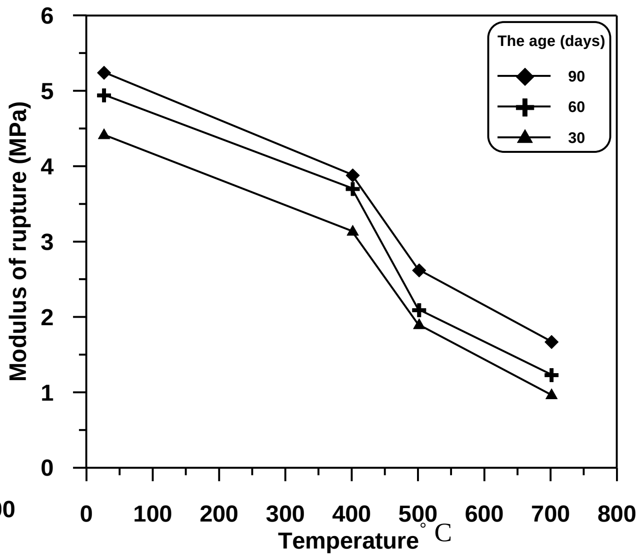


Fig (4-33) The effect of fire flame on the modulus of rupture of series -B at 2.0 hours period of exposure.

٤-٦ Modulus of Elasticity

Test results of the modulus of elasticity are summarized in Tables [٤-٩] and [٤-١٠] for mix A and mix B respectively. Figures [٤-٣٤] to [٤-٣٧] and [٤-٣٨] to [٤-٤١] illustrate the relationship between the residual modulus of elasticity and fire flame exposure temperatures for series A and series B respectively.

From the figures, it can be seen that the test results for *Ec* somewhat is similar to the pattern of compressive strength and flexural strength, but with reduction values which is more than the compressive and flexural strength at fire flame temperatures.

Table [٤-٩]: *Test values of the modulus of elasticity of concrete specimens of series-A- after exposure to fire flame temperatures.*

Age at exposure (days)	Period of Exposure (hours)	Modulus of elasticity(GPa) Temperature ºC				Ratios E_{ca}/E_{cb}		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٢٥.٤٠	١٦.٢٦	١٠.٤١	٠٦.١٠	٠.٦٤	٠.٤١	٠.٢٤
	١.٠		١٧.٢٧	١٠.٦٧	٠٥.٠٨	٠.٦٨	٠.٤٢	٠.٢٠
	١.٥		١٥.٢٤	٩.٦٥	٠٤.٥٧	٠.٦٠	٠.٣٨	٠.١٨
	٢.٠		١٣.٩٧	٨.٨٩	٠٤.٠٦	٠.٥٥	٠.٣٥	٠.١٦
٦٠	٠.٥	٢٧.٥١	١٨.٤٣	١١.٥٥	٠٦.٨٨	٠.٦٧	٠.٤٢	٠.٢٥
	١.٠		١٩.٢٦	١٢.٣٨	٠٦.٣٣	٠.٧٠	٠.٤٥	٠.٢٣
	١.٥		١٧.٣٣	١٠.٧٣	٠٥.٧٨	٠.٦٣	٠.٣٩	٠.٢١
	٢.٠		١٥.٩٦	٩.٣٥	٠٤.٩٥	٠.٥٨	٠.٣٤	٠.١٨
٩٠	٠.٥	٢٨.٧٥	٢٠.٤١	١٢.٣٦	٠٨.٦٣	٠.٧١	٠.٤٣	٠.٣٠
	١.٠		١٩.٨٤	١٤.٣٨	٠٧.٧٦	٠.٦٩	٠.٤٥	٠.٢٧
	١.٥		١٨.٤٠	١١.٥٠	٠٦.٣٣	٠.٦٤	٠.٤٠	٠.٢٢
	٢.٠		١٧.٥٤	١٠.٣٥	٠٥.٧٥	٠.٦١	٠.٣٦	٠.٢٠

E_{ca} = Modulus of elasticity after exposure to fire flame.
E_{cb} = Modulus of elasticity before exposure to fire flame.

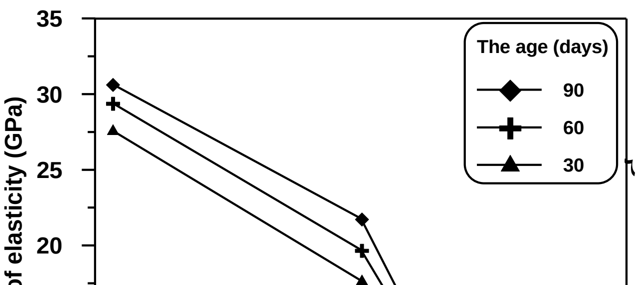
Table [٤-١٠]: *Test values of the modulus of elasticity of concrete specimens of series-B- after exposure to fire flame.*

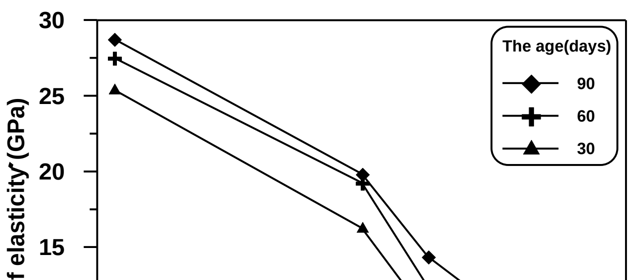
Age at exposure (days)	Period of exposure (hours)	Modulus of elasticity(GPa) Temperature ºC				Ratios E_{ca}/E_{cb}		
		٢٥ (١)	٤٠٠ (٢)	٥٠٠ (٣)	٧٠٠ (٤)	٢/١ (٥)	٣/١ (٦)	٤/١ (٧)
٣٠	٠.٥	٣٢.١٥	٢٢.١٨	١٤.٤٧	٠٨.٣٦	٠.٦٩	٠.٤٥	٠.٢٦
	١.٠		٢١.٥٤	١٣.٥٠	٠٦.٧٥	٠.٦٧	٠.٤٢	٠.٢١
	١.٥		١٩.٤٦	١٣.١٨	٠٦.١١	٠.٦١	٠.٤١	٠.١٩
	٢.٠		١٨.٣٣	١١.٩٠	٠٥.١٤	٠.٥٧	٠.٣٧	٠.١٦
٦٠	٠.٥	٣٣.٤٦	٢٤.٠٩	١٦.٠٦	١٠.٠٣	٠.٧٢	٠.٤٨	٠.٣٠
	١.٠		٢٢.٧٥	١٥.٣٩	٠٩.٣٧	٠.٦٨	٠.٤٦	٠.٢٨
	١.٥		٢١.٠٨	١٤.٠٥	٠٧.٧٠	٠.٦٣	٠.٤٢	٠.٢٣
	٢.٠		١٩.٧٤	١٢.٧١	٠٦.٠٢	٠.٥٩	٠.٣٨	٠.١٨
٩٠	٠.٥	٣٤.٧٤	٢٥.٣٦	١٧.٧٢	١١.٨١	٠.٧٣	٠.٥١	٠.٣٤
	١.٠		٢٦.٠٦	١٦.٣٢	١٣.٥٩	٠.٧٥	٠.٤٧	٠.٣٩
	١.٥		٢٢.٥٨	١٤.٥٩	٠٨.٣٤	٠.٦٥	٠.٤٢	٠.٢٤
	٢.٠		٢٠.٨٤	١٣.٥٥	٠٧.٣٠	٠.٦٠	٠.٣٩	٠.٢١

E_{ca} = Modulus of elasticity after exposure to fire flame.
E_{cb} = Modulus of elasticity before exposure to fire flame.

At ٤٠٠ºC, there was a significant reduction in the modulus of elasticity due to the effect of fire flame. The residual modulus of elasticity was (٥٥ – ٧١ %) and (٥٧ – ٧٥ %) for series A and B respectively. Similar results were obtained by Schenider,[٧٦] Fahmi and Ibrahim[٧٧]. At ٥٠٠ºC, there was a sharp reduction in Ec values for both series and the residual of modulus of elasticity was (٣٤ – ٤٥ %) for series A and (٣٧ – ٥١٪) for series B. These results confirmed with Lankard et al [٧٨], Schneider [٧٦], Fahmi and Ibrahim [٧٧].

At ٧٠٠ºC, the residual modulus of elasticity was (١٦ – ٣٠ %) and (١٦ – ٣٤ %) for series A and series B respectively. Similar results were reported by Castello and Durani [٢٠] and Schneider [٧٦].





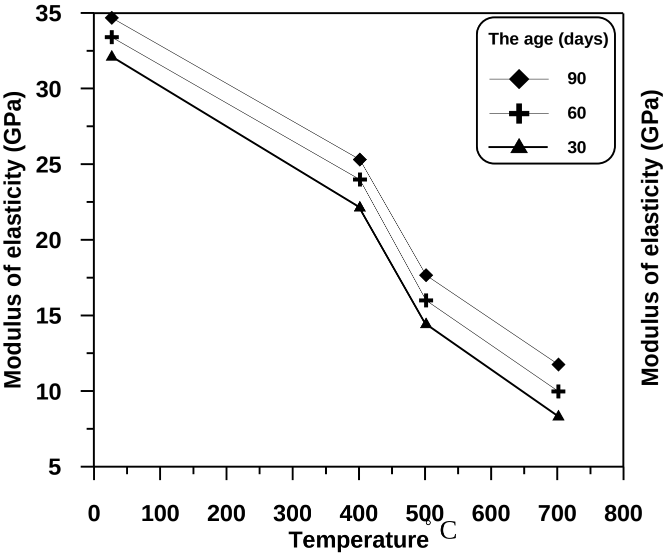


Fig.(4- 38) The effect of fire flame on the modulus of elasticity of Series- B at 0.5 hour priod of exposure.

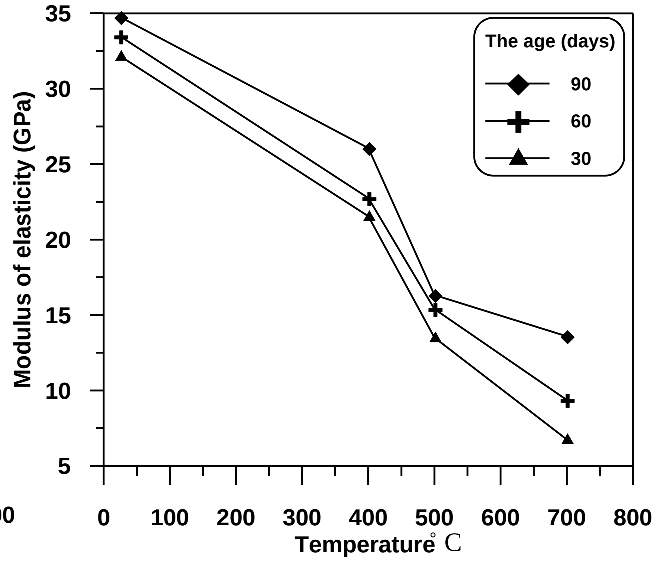


Fig.(4- 39) The effect of fire flame on the modulus of elasticity of Series- B at 1.0 hour period of exposure.

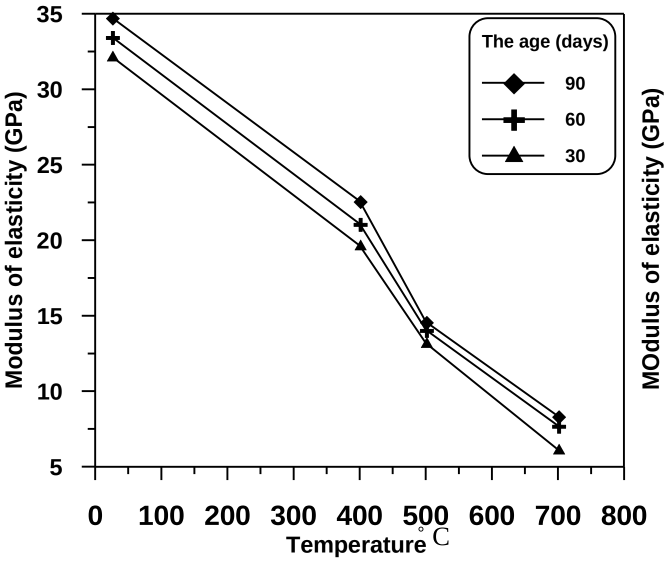


Fig.(4-40) The effect of fire flame on the modulus of elasticity of Series- B at 1.5 hours period of exposure.

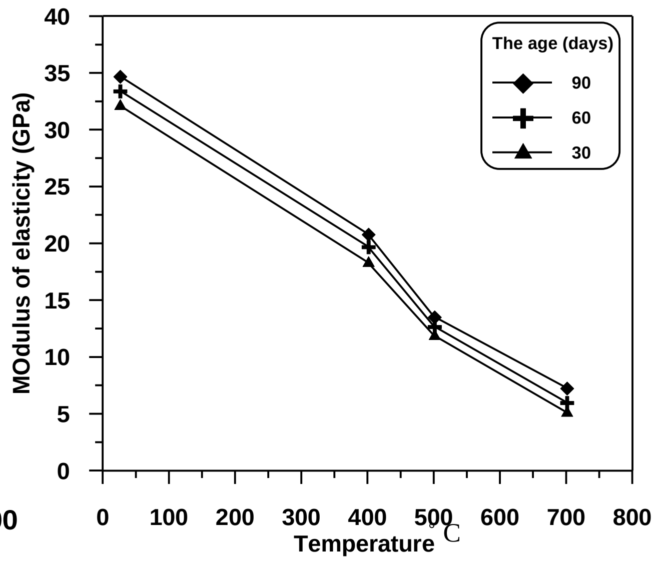


Fig.(4-41) The effect of fire flame on the modulus of elasticity of Series- B at 2.0 hours period of exposure.

Chapter Five

5

CONCLUSIONS AND FUTURE WORK

٥-١ Conclusions

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

١- The density of concrete decreases with increasing fire flame temperature. The reduction in density all ranged between (١.٩-٧.٧٪) at ٤٠٠ ˚C, (٤-٨٪) at ٥٠٠ ˚C and (٨-١١.٢٪) at ٧٠٠ ˚C.

٢- The residual compressive strength ranged between (٧٠-٨٥ %) at ٤٠٠ ˚C, (٥٩-٧٨ %) at ٥٠٠ ˚C and (٤٣-٦٢ %) at ٧٠٠ ˚C.

٣- Large proportion of drop in compressive strength occurs at the first ٠.٥ hour period of exposure and the adverse effect of fire is pronounced on series A more than on series B.

٤- Beyond ٥٠٠ ˚C the effect of the fire flame on the compressive strength was equal for both series when the period of fire exposure reached two hours.

٥- Cooling in water causes further reduction in compressive strength for both series, the residual compressive strengths were (٦٦-٨٢ %) at ٤٠٠ ˚C, (٥٢-٧٥ %) at٥٠٠ ˚C and (٣٣-٦٦ %) at ٧٠٠ ˚C of fire exposure temperature.

٦- The percentage reduction in compressive strength of water cooled specimens as compared with air cooled specimens at ٤٠٠ ˚C, ٥٠٠ ˚C and ٧٠٠ ˚C was (٣-٥ %), (٤-٧ %) and (٥-٨ %) for series A and (٢-٥ %), (٣-٦ %) and (٥-٨ %) for series B respectively.

٧- For the studied temperature range in this study, the compressive strength-reduction curve, recommended by the Euro codes CEN (١٩٩٣, ١٩٩٤) is in better agreement with the test results rather than CEB (١٩٩١) strength-reduction curve.

٨- The splitting tensile strength was more sensitive to fire flame than the compressive strength. The residual splitting tensile strength for both series ranged between (٦٧-٧٦ %) at ٤٠٠ ˚C, (٤٠-٦٧٪) at ٥٠٠ ˚C and (٢٠-٤٥ %) at ٧٠٠ ˚C.

٩- Extra reduction in splitting tensile strength took place for both series when the specimens were quenched in water, the reduction for both series were ranging from (٥٩-٧٣ %) at ٤٠٠ ˚C, (٣٣-٥٦ %) at ٥٠٠ ˚C and (٩-٣٩ %) at ٧٠٠ ˚C.

١٠- Specimens cooled by water showed more reduction in splitting tensile strength compared to that cooled in air, the percentage of reduction ranged between (٤-٨٪), (٦- ١٠ %) and (٦-١٢٪) for series A and (٣-٦ %) , (٤-٧ %) and (٥-١٤ %) for series B at ٤٠٠ ˚C, ٥٠٠ ˚C and ٧٠٠ ˚C respectively.

١١- The flexural strength was very sensitive to fire flame temperatures. The residual flexural strengths ranged between (٧١-٧٧٪). (٤٢-٥٨٪) and (٢٣-٣٨٪) for the series A and (٧١-٧٩٪) , (٤٣-٦٠٪) and (٢٢-٤١٪) for series B at ٤٠٠ ˚C, ٥٠٠ ˚C and ٧٠٠ ˚C respectively.

١٢- Modulus of elasticity is most affected by fire flame temperature than compressive strength and flexural strength. The physico-chemical transformation in concrete constituents during burning will yield strength loss. The residual modulus of elasticity ranged between (٥٥-٧١٪), (٣٤-٤٥٪) and (١٦-٣٠٪) for series A and (٥٧-٧٥٪), (٣٧-٥١٪) and (١٦-٣٤٪) for series B at ٤٠٠ ˚C, ٥٠٠ ˚C and ٧٠٠ ˚C respectively.

١٣- The ultrasonic pulse velocity test showed a response to the effect of fire flame, the reduction in *(U.P.V)* was (٢٣-٢٩٪), (٤٦-٥٩٪) and (٥٤-٧٧٪) for series A, while the reduction was (١٨-٢٨٪), (٣٧-٥١٪) and (٤٩-٦٨٪) for series B at ٤٠٠ ˚C, ٥٠٠ ˚C and ٧٠٠ ˚C respectively. It appears that this non-destructive method gives better-predicted values for the residual strength when it is used in the mathematical models if compared with other destructive tests.

١٤- The reduction in rebound number was (١٠-١١٪), (١٦-٢٧٪) and (٢٩-٣٨٪) for series A and (٩-١١٪), (١٦-٢٣٪) and (٢٥-٤٠٪) for series B at ٤٠٠ ˚C, ٥٠٠ ˚C and ٧٠٠ ˚C respectively. The decrease in the rebound number with increasing in fire temperature can be attributed to the fact that fire causes damage to the surface of concrete rather than to concrete in the core of the member.

١٥- Mathematical models based on the statistical regression analysis were developed in this study. These models obtained to obtain reliable predicted values for the mechanical properties of concrete. The maximum difference between predicted and measured values was ± ٦٪ for the data in this study.

١٦- For the reinforced concrete beams before burning, the ACI-٣١٨-٩٥-Code method predicts values of ultimate moment strength very close to CP١١٠-١٩٧٢ method, but both methods underestimate the actual ultimate strengths of the beams. The ratio of experimental to calculated ranged from (١.٦٩) to (١.٩١) with an average value of (١.٧٨) and ranged from (١.٧٨) to (١.٩٩) with an average value (١.٨٥) when calculations were based on ACI – ٣١٨ Code and CP١١٠ respectively.

١٧- The ratio of measured to calculated service load deflection of the beams ranged from (١.٠٤) to (٠.٨٤) with an average value of (٠.٩٣) when the calculations were based on ACI–Code and ranged from (١.١٢) to (٠.٨٩) with average value of (١.٠) when the calculations were based on CP١١٠-Code . It can be concluded that CP١١٠- Code procedure gives comparable results to the measured values for all beams.

١٨- After the beams were subjected to fire flame, two types of cracks developed. The first was thermal cracks, which appeared in honeycomb fashion all over the surface. The second cracks originated at mid-span region due to bending from the applied load and called flexural cracks.

١٩- At ٥٠٠°C, for ٣٠ days age and ١.٠ hour period of exposure, the first and yield stress in steel were observed at a load of (١١.١٪) and (٨٦٪) from the ultimate load respectively for beam A٢ and (١١.٦٪), (٨٧,٦ %) from the ultimate load for the beam B٢. For the same period and at ٩٠ days age, the first cracks in concrete and yield stress in steel were obtained at a load of (١٥.٥٪) and (٨٣.٤٪) from the ultimate load for A٥ and (١١.٠٪) ,(٨٨.٤٪)for the beam B٥.

٢٠- At ٧٠٠°C,for ٣٠ days of age and ١.٠ hour period of exposure, the first crack in concrete and yield stress in steel were noticed at a load of (٨.٥٪) and (٨٧.١٪) from the ultimate load respectively for the beam A٣ and (٩.٥٢٪), (٨٨.٣ %) from the ultimate load for the beam B٣. For the same period and at ٩٠ days of age, the first crack in concrete and yield stress in steel were obtained at a load of (٩.٥٧٪) and (٨٧.٣٪) from the ultimate load respectively for the beam A٦ and (٨.٧٢٪) and (٨٩.٠٪) for the beam B٦.

٢١- At ٥٠٠°C, both burning and subsequent cooling did not affect the mechanical properties of reinforced bars, but the effect was observed at ٧٠٠°C. The residual yield stress was (٨٣.٠٪) and ultimate stress was (٨٤.٢٪) for ٨-mm bar diameter . For the bars of ١٠-mm diameter, the residual yield stress was (٨٤.٥٪) and the ultimate stress was (٨٥.٨٪).

٢٢- The modulus of elasticity of the reinforcing steel bar was not affected by burning and subsequent cooling at all ranges of temperature.

٢٣- It was noticed that the load-deflection relations to specimens exposed to fire flames are flat, representing softer load- deflection behavior than that of

٥-٢ Recommendations and Future Works

the control beams. This can be attributed to the early cracking and lower modulus of elasticity.

The following recommendations are subjected to further researches.

١- Studying the effect of fire flame on high strength concrete properties.

٢- Studying the effect of fire flame on lightweight concrete properties.

٣- Studying the effect of fire flame on mechanical properties of concrete for long period of exposure more than two hours and at ages more than three months.

٤- Investigating the effect of high level of fire flame temperature on the behavior of reinforced concrete slabs.

٥- Studying the effect of the fire flame on carrying moment capacity and load –deflection behavior on reinforced concrete beams with different steel ratios.

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